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CHAPTER 8: WATER TREATMENT**8.1 Introduction**

The aim of water treatment is to produce and maintain water that is hygienically safe, aesthetically pleasing, and palatable in an economical manner. Though the treatment of water would achieve the desired quality, the evaluation of its quality should not be confined to the end of the treatment facilities but should be extended to the point of consumer use.

Methods of Treatment and Flow Sheets

The method of treatment to be employed depends on the nature of raw water constituents and the desired standards of water quality as per IS: 10500 (2012). The traditional surface water sources are streams, rivers, tributaries, manmade and natural lakes, and canals. Traditional ground water sources are dug well and bore wells. The seawater desalination and reuse of treated sewage (indirect potable use) are considered as prospective sources. The impurities in raw water can be classified as suspended solids, colloidal solids, and dissolved solids which may be harmful to humans. Impurities also can be classified as inorganic, organic, and microbial contaminants. For drinking water treatment to remove suspended or precipitated solids and contaminants, unit processes viz. coagulation, flocculation, clarification, filtration, and disinfection are a must. Recently, biological filtration (after conventional filtration) is also included to remove organics and Disinfection by Products (DBP). The degree of treatment shall be directly related to the quality of raw water including emergent contaminants and DBPs. Chapter 7 of the Manual discusses various types of impurities that could be present in the Raw Water.

Sometimes a highly polluted stretches of water due to discharge of wastewater (organic matter) are encountered, which require much higher chlorine doses, however, adding excessive chlorine is dangerous and leads to formations of Disinfectant by-Products (DBPs). In such situation a different treatment technology has to be resorted to, such as enhanced coagulation (Alum + Polyelectrolyte) followed by pressure membrane (details are given in Part A chapter 10 Specific water treatment system).

Desirable Raw Water Quality for Conventional Treatment: As mentioned earlier, Coagulation, Flocculation, Clarification/ precipitation, Filtration and Disinfection are the main unit operations and processes classified as Conventional Treatment. Normally these processes and units are employed to treat surface waters and ground water influenced by surface water. The surface water sources in India have predominantly inorganic suspended solids. However in the lean flow season due to pollution by sewage, industrial wastewater, farm discharges organic suspended and colloidal impurities also occur in the raw water. Coloured water (due to Natural decay of vegetation) also contribute to organic load. Microbial contaminants viz., algae is also present in many polluted water in lean flow season. Jar tests with different coagulants and coagulant aid give a representative analysis of settleable floc formation. Maximum organic loading in terms of BOD and COD should be restricted to 5 mg/L and 10 mg/L respectively.

Non Conventional Treatment Technologies for highly polluted water

Due to excessive river pollution as a result of constituents mentioned in the earlier para, the organic and microbial contaminants increase beyond the desirable limits. Conventional technologies become ineffective. For BOD and COD values consistently more than 10 mg/L and 30 mg/L throughout the year, the option of adopting Advanced technologies need to be analysed. The membrane technologies (submersible or pressure) can eliminate difficult to coagulate particles with lower molecular weights. The MF, UF and RO are increasingly being used worldwide for the above described contaminants including THMs, PFAS etc. However, the feed water to the membranes needs to be prepared with conventional pre-treatment. Dissolved Air Floatation (DAF) has increasing applications for highly algal laden waters. In the extreme case, if the waters are highly polluted with sewage, biological treatment units like SBR or MBBR need to be incorporated to prepare the feed water to membranes. Though membranes-based solutions require less foot print, their Capex and Opex are relatively higher.

Ground water with high TDS

In many costal zones in India the ground water has TDS in the range of 1000 to 5000 mg/L (Brackish to Saline). In such cases, low pressure RO membranes need to be included in the treatment process.

The unit operations in water treatment include aeration, flocculation (rapid and slow mixing) and clarification, filtration, disinfection, softening, de-ferrization, de-fluoridation, water conditioning, and many different combinations of these are selected to suit these requirements. The treatment technologies for removal of emergent contaminants and DBPs have been included in the Chapter 10 of this manual.

The choice of any particular sequence of treatment units will depend not only on the qualities of the raw water available and treated water desired but also on the comparative economics of alternative treatment steps applicable. Sketches of flow sheets are presented in Figure 8.1.

In the case of groundwater and surface water with storage that is well protected, where the water has turbidity below 10 NTU, and they are free from odour and colour, plain disinfection by chlorination is adopted before supply, as shown in Figure 8.1 (a) and (b).

Where groundwater contains excessive iron, dissolved carbon dioxide, and odorous gases, aeration followed by flocculation (rapid and slow mixing), sedimentation, rapid gravity or pressure filtration, and disinfection may be necessary, as in Figure 8.1 (c).

In case it contains only carbon dioxide or odorous gases, aeration followed by disinfection may be sufficient.

In surface waters with turbidities not exceeding 50 NTU and where a sufficient area is available, plain sedimentation followed by slow sand filtration and disinfection is practiced as in Figure 8.1 (d).

Better option will be blending with imported low TDS waters as Silica is not a parameter in drinking water standards.

Conventional treatments including pre-chlorination, aeration, flocculation (rapid and slow mixing) and sedimentation, rapid gravity filtration, and post-chlorination are adopted for highly polluted surface waters laden with algae or other microorganisms as in Figure 8.1 (e).

Sometimes, unconventional flow sheets may be adopted for waters of low turbidity (below 10 to 15 NTU) and containing a low concentration of suspended matter (less than 50 mg/L), as in Figure 8.1 (f). Such raw waters are applied to the rapid sand filters with alum addition which may or may not be accompanied by slow mixing for a short period (10 min).

Slow sand filters can also be used to polish the filtrate from a rapid sand filtration plant. Water with excessive hardness needs softening, as in Figure 8.1 (g); for removal of dissolved solids, demineralization by ion exchange may form a part of the domestic or industrial water treatment units, as in Figure 8.1 (h).

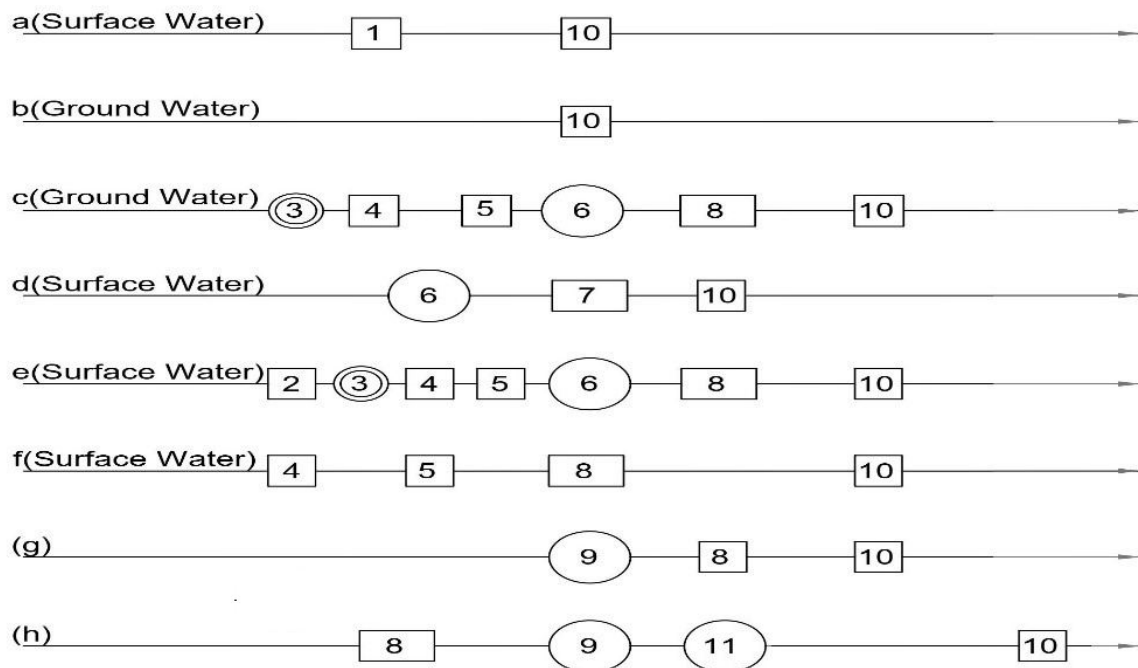


Figure 8.1: Sketches for treatment process flow sheets

1. Storage
2. Chlorination (Pre)
3. Aeration
4. Rapid Mixing
5. Flocculation – Slow Mixing

6. Sedimentation
7. Slow Sand Filtration
8. Rapid Sand Filtration
9. Softening
10. Chlorination (Post)
11. Demineralization

Plant Capacity and Hydraulic Overloading:

Hydraulic loading is defined as the volume of raw water or the water (design flow for next 15 years from the base year) applied to the unit operation per time period.

The treatment plant need to be designed for 20% hydraulic overloading. However, the interconnecting pipings and channels are designed to carry 20% excess over the design flow. It also means establishing the hydraulic gradient of the treatment plant so that the desired freeboard are maintained in the open channels and process units.

8.2 Pre-Sedimentation and Storage

The turbidity of raw water from rivers and streams may exhibit wide fluctuations, and values exceeding a few thousand NTU are not uncommon during high flow season. The river's sediment load during floods chiefly derives from soil erosion and consists predominantly of coarse suspended solids. Pre-sedimentation and storage can accomplish the removal of large-sized and rapidly settleable silt and other materials before the raw water reaches the treatment plant.

When removal of coarse and rapidly settling silt is aimed at in Pre-sedimentation, detention periods of 2 hrs. to 4 hrs. and surface loading as per Table 8.5 has been recommended. Detritus tanks with 10 to 15 min detention time can be provided if the raw water carries sand particles more than 0.5 mm in size. These plain sedimentation tanks can be constructed with conventional construction material or with dug out of the earth with sloping sides. Minimum two nos. of the tank are provided if the settled solids ~~sludge~~ removal is manual.

Unlike pre-settling basins, the storage basins or reservoirs are designed for very large detention periods ranging from about one week to a few months. The storage basins or reservoirs are proposed when the source is canal water (to take care of the canal closure period). They are also provided in the coastal region, where the tides affect the salinity of rivers. The storage basins also help reduce raw water's turbidity and suspended solids.

8.3 Aeration

Aeration is the process by which air is circulated through, mixed with, or dissolved in a liquid or substance. Aeration is generally the first process for water treatment.

Aeration is to promote the exchange of gases between the water and the atmosphere. In water treatment, aeration is practiced for four purposes:

- a) To enhance the aesthetic purpose or value of the water treatment complex as a whole.
- b) To add oxygen to water for imparting freshness, e.g., water from underground sources devoid of or deficient in oxygen. This may also happen if the length of the raw water main is substantially high.
- c) Expulsion of carbon dioxide, hydrogen sulphide, and other volatile substances, including Volatile Organic Compounds (VOC), causing taste and odour.
- d) Removal of iron, Manganese, etc., from ground water (water from deeper layers of rocks formation when contain iron, manganese etc.)

8.3.1 Limitations of Aeration

The unit operation of aeration requires a significant head of water. The water is rendered more corrosive after aeration when the dissolved oxygen content is increased, though, in certain circumstances, it may be otherwise due to the removal of aggressive carbon dioxide. The designer should carefully consider the merits of other alternatives because of the additional cost of lifting, which may be involved in aeration. Aeration is not highly effective for taste and odour removal but can be used in combination with chlorine or activated carbon to reduce their doses.

8.3.2 Aeration Process

Gases are dissolved in or liberated from water until the concentration of the gas in the water has reached its saturation value. The concentration of gases in a liquid generally obeys Henry's law which states that the concentration of each gas in water is directly proportional to the partial pressure (product of the volume percent of the gas and the total pressure of the atmosphere) or concentration of gas in the atmosphere in contact with water. The saturation concentration of a gas decreases with temperature and dissolved salts in water, and aeration tends to accelerate the gas exchange.

The rate of exchange of gas is governed by the area of interface between the gas and the liquid, the thickness of the interlayers, time of contact, the partial pressure of the gas in the overlaying atmosphere, and the degree of under-saturation or over-saturation of the gas in the liquid.

To ensure proper aeration, it is necessary:

- a) To increase the area of water in contact with the air, i.e., if the water is sprayed, the smaller the droplets produced, the greater will be the area available. Similarly, if the water is being made to fall as a film over packing material in a tower, the smaller the size of the packing material, the greater will be the area available.
- b) To keep the surface of the liquid constantly agitated so as to reduce the thickness of the liquid film, which would govern the resistance offered to the rate of exchange of the gas; and

- c) To increase the time of contact of water droplets with air or to increase the time of flow, which can be achieved by increasing the height of the jet in the spray aerator and increasing the height of the bed in the case of packed media.

Where oxygen is to be dissolved in water, the concentration or partial pressure of the oxygen may be increased by increasing the total pressure of the gases in contact with water. For this reason, air injected into a main under pressure is a reasonably efficient method of increasing the amount of dissolved oxygen.

The exchange of gases from water to air or from air to water which takes place at the air-water interface, can be described by the following formulae:

$$C_t = C_s - (C_s - C_o) \exp \left(- \left(k \frac{A}{V} t \right) \right) \quad (8.1)$$

(Gas absorption)

And

$$C_t = C_s + (C_s - C_o) \exp \left(- \left(k \frac{A}{V} t \right) \right) \quad (8.2)$$

(Gas release)

Where;

C_t = Actual concentration of the gas in the water after a given period;

A/V = Ratio of the exposed area to the volume of water;

C_s = Gas saturation concentration;

K = Gas transfer coefficient (having a dimension of velocity);

C_o = Concentration of gas initially present in the water; and

t = Aeration period

The gas saturation values of H_2S and CO_2 are generally 0 and 0.5 mg/L when exposed to a normal atmosphere having partial pressures of the gases of 0% and 0.03%, respectively. Because of the low saturation values, removal of H_2S and CO_2 by aeration is practicable.

If the initial concentration of the gas to be removed from water is much above the saturation limit, a sizeable reduction in the concentration of the gas by aeration is possible.

8.3.3 Types of Aerators

There are two main types of aerators depending upon the mechanics of aeration:

- a) Drop Aerators: Those forming drops or thin sheets of water exposed to the atmosphere, i.e., water is exposed to come in contact with the ambient air. These include Spray, water-fall or multiple trays, cascade and mechanical aerators
- b) Bubble/Diffused Aerator: Those forming small bubbles of air that rise in the water, i.e., the air is brought in contact with the water. These include diffusion aerators.

8.3.3.1 Spray Aerators

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

Nozzles usually have diameters varying from 10 mm to 40 mm spaced in the pipe at intervals of 0.5 m to 1 m or more. Special (patented) types of corrosion-resistant nozzles and sometimes plain openings in pipes, serving as orifices, are used. The pressure required at the nozzle head is usually 7 m of water, but practice varies from 2 to 9 m, and the discharge ratings per nozzle vary from 18 to 36 m³/hr. Usually aerator area of 0.03 to 0.09 m²/m³/hr. of design flow is provided.

The time of exposure of the droplets, the head required, and the Flow from each nozzle can be calculated from the following formulae:

$$v = C_v \sqrt{2gh} \quad (8.3)$$

$$q = C_d \sqrt{2gh} \quad (8.4)$$

$$t = 2C_v \frac{\sqrt{2h}}{g} \sin x \quad (8.5)$$

Where;

h = Total head of water at the nozzle;

g = Acceleration due to gravity;

v = Initial velocity of drop emerging from the nozzle;

C_v = Coefficient of velocity,

C_d = Coefficient of discharge;

q = Discharge rate from each nozzle;

a = Area of cross-section of nozzle opening; and

t = Time of travel or exposure, and x is the angle of inclination of the spray from the horizontal.

The vertical jet gives the longest exposure time for a given value of h (2 seconds for a head of 6 m), while the inclined jets can have less interference between falling drops. Wind can influence the path of the trajectory of each drop, and allowance must be made for its effect. The dimensions of the tray must take into account the velocity and direction of the wind to ensure that no water is lost by carry-away. As already explained, the size, number, and spacing of nozzles, aeration time, and interference between adjacent sprays are also factors governing aeration efficiency. Spray aerators are usually quite efficient with respect to the gas transfer and can be expected to remove 70 % to 90% of CO₂ and 90 % to 99% H₂S and add to the appearance of a water treatment plant. They require a large area and are consequently difficult to be housed readily. They pose operating problems due to corrosion, iron deposition after precipitation, and choking of the nozzles, particularly during freezing weather.

The diameters of the pipe grid and orifices should be so designed as to ensure a uniform discharge (with a maximum variation of 5 %) through all the nozzles in the grid. The loss of head in the pipe is kept low compared to the loss of head in the nozzle. Theoretically, numerous small nozzles capable of producing atomized water could be used. Practically, however, extremely small nozzles are to be avoided because of clogging and consequent excessive maintenance needed. Common friction formulae are used in the estimation of loss of head, excepting that the pipe with nozzles has to be considered to be carrying uniformly decreasing flow.

8.3.3.2 Waterfall or Multiple Tray Aerators

Water is discharged through a riser pipe and distributed onto a series of trays or steps from which the water falls either through small openings to the bottom or over the edges of the trays. Water is caused to fall into a collection basin at the base. In most aerators, coarse media such as coke, stone, or ceramic balls, ranging from 50 to 150 mm in diameter, are placed in the trays to increase efficiency. For iron removal (see 9.4.3), this may be beneficial. The trays, about 4 to 9 in number (with a spacing of 300 mm to 750 mm), are arranged in a structure 1 m to 3 m high. With the media, good turbulence is created, and large water surface is exposed to the atmosphere. By the addition of more trays, the time of contact can be increased. The space requirements vary from 0.013 to 0.042 m² per m³/hr. of flow. Natural ventilation or forced draft is provided. Removal efficiencies varying from 65 % to 90 % for CO₂ and 60 % to 70 % for H₂S have been reported.

8.3.3.3 Cascade Aerators

In cascade aerators, water is allowed to flow downwards after spreading over an inclined surface in thin sheets, and the turbulence is secured by allowing the water to pass through a series of steps or baffles. The Central Shaft (Inlet) of the Aerator is circular in shape. It can be constructed in RCC or can be a pipe. The velocity in the shaft is limited to 0.60 m/sec to reduce the exit turbulence. The top opening of the shaft is provided with a metallic grille from a safety point of view. The number of steps is usually 4 to 6. Exposure time can be increased by increasing the number of steps, and the area-to-volume ratio can be improved by adding baffles to produce turbulence. Head requirements vary from 0.5 m to 3.0 m (optimum is 0.50 m to 1.20 m) and the space requirements vary from 0.015 to 0.045 m²/m³/hr. Generally, the “rise” of the cascade is limited to 0.15 m to 0.25 m. The “tread” to “rise” ratio of cascades needs to be more than 2 (two) to avoid the tendency of the water to “jump over” instead of forming a thin film. In cold climates, these aerators must be housed with adequate provision for ventilation. Corrosion and slime problems may be encountered. The gas transfer efficiency is less compared to the spray type. Removal of gas varies from 20 % to 45 % for CO₂ and up to 35 % for H₂S. Well-designed circular cascades aerators enhance the aesthetics of the treatment plant. This is the most commonly employed aerator for surface water treatment plants in municipal and government sector drinking water schemes.



Figure 8.2: Small plants with three cascades

8.3.3.4 Diffused Aerators

This is an obverse of waterfall type aerator. This type of aerator consists of a basin in which perforated pipes, porous tubes, or plates are used to release fine bubbles of compressed air, which then rise through the water being aerated. As the rising bubbles of the air have a lower average velocity than the falling drops, a diffused air type provides a longer aeration time than the waterfall type for the same power consumed. These have higher initial costs and require greater recurring expenditure. Tanks are commonly 3 m to 4.5 m deep and 3 m to 9 m wide. Compressed air is injected through the system to produce fine bubbles, which on rising through the water, produce turbulence resulting in a continual change of exposed surface. Ratios of width to depth should not exceed 2:1 for effective mixing and the desired detention period varies from 10 min to 30 min. The amount of air required ranges from 0.06 m³ to 1 m³ of air per m³ of water treated. The air diffusers are located on one side of the tank. The power requirements of the blower vary from 3 to 13 W/m³/hr.

The air should be filtered before passing through porous diffusers, and an oil trap is also provided before diffusers. Diffused aerators require less space than spray aerators but more than tray aerators, and cold weather operating problems are not encountered. The aerators can also be used for the mixing of coagulants.

Compressor power requirements may be estimated from the airflow, discharge and inlet pressures, and air temperatures using the following equation, which is based upon the assumption of adiabatic conditions:

$$P = \frac{wRT_1}{(8.4)e} \left[\left(\frac{P_2}{P_1} \right)^{0.283} - 1 \right] \quad (8.6)$$

Where;

- P = Power required in kW;
- P₁ = Absolute inlet pressure in atm. (normally 1 atm);
- P₂ = Absolute outlet pressure in atm;
- R = Gas constant (8.314 J/mole, °K);
- W = Air mass flow in kg/s;

E = Efficiency of the machine, (usually 0.7 to 0.8); and

T_1 = Inlet temperature in degrees °K

8.3.3.5 Mechanical Aerators

These are not normally used in water treatment because of the availability of more economical alternatives but find application in wastewater treatment.

8.4 Measurement of Flow

The measurement of flow in open channel is very important and crucial in operating the various process in water treatment plant to monitor and control the process. The various methods generally used are explained in the following sections.

8.4.1 Triangular Notches or V-Notch

There are generally 3 types of triangular notches used for flow measurement i.e., 30°, 60° and 90°. 90° triangular notches are used for measuring small quantities of flows upto about 1.25 m³/s.

i. Installation Requirements

The approach channel should be reasonably smooth, free from disturbances and straight for a length equal to at least 10 times the width. The structures in which the notch is fixed shall be rigid and water-tight and the upstream face vertical. The downstream level should be always at least 5 cm below the bottom-most portion of the notch (inverted apex) ensuring free flow.

ii. Specification for Materials

The plate should be smooth and made of rust-proof and corrosion-resistant material. The thickness should not exceed 2 mm, with the downstream edge chamfered at an angle of not less than 45° with the crest surface.

iii. Measurement of Head Causing the Water Flow

The head causing flow over the notch shall be measured by standard hook gauge upstream at a distance of 3 to 4 times the maximum depth of flow over the notch.

iv. Discharge Equation

The discharge Q (in m³/sec) for V-Notch is given by the expression:

$$Q = \frac{8}{15} C_e \sqrt{2g} \tan \frac{\theta}{2} h^{2.5} \quad (8.7)$$

Where;

C_e = effective discharge coefficient

G = acceleration due to gravity (9.806 m/s²)

θ = angle of the notch at the centre

h = measured head causing flow in m,

For 90° V-Notch which is generally used, the discharge is given by the expression

$$Q = 2.362 C_e h^{2.5}$$

C_e values vary from 0.603 to 0.686 for values of head varying from 0.060 to 0.377m

v. Limitations

The triangular notches should be used only when the head is more than 60 mm.

vi. Accuracy

The values obtained by the equation for triangular notches would vary from 97% to 103% of the true discharge for discharges from 0.008 m³/s to 1.25 m³/s.

8.4.2 Rectangular Notches

The installation requirements, specifications, head measurements, head limits and accuracy will be the same as for triangular notches. The width of notch should be at least 150 mm.

There are two types of rectangular notches viz., (i) with end contractions and (ii) without end contractions.

i. With End Contractions

The contraction from either side of the channel to the side of the notch should be greater than 0.1 m.

The discharge (m³/s) through a rectangular notch with end contractions is given by the equation:

$$Q = \frac{2}{3} E C_e \sqrt{2g} b_e H^{1.5} \quad (8.8)$$

Where;

b_e = effective width = actual width of the notch + k (value of k being 2.5 mm, 3 mm and 4 mm for b/B ranges of upto 0.4, 0.4 to 0.6 and 0.6 to 0.8 respectively);

b/B = ratio of the width of the notch to the width of the channel;

$H^{1.5}$ = effective head = actual head measured (h) + 1 mm;

g = acceleration due to gravity (9.806 m/s²); and

C_e = varies from 0.58 to 0.70 for values of b/B from 0 to 0.8.

ii. Without End Contractions

The discharge (m³/s) through a rectangular notch without end contractions is given by the following expression:

$$Q = \frac{2}{3} C_e \sqrt{2g} b H^{1.5} \quad (8.9)$$

Where;

b = width of the notch (m)

H = effective head = actual /measured head (h) + 1.2 mm

$C_e = 0.602 + 0.075 h/p$

: p = height of the bottom of the notch from the bed of the channel

8.4.3 Parshall Flume

Parshall Flume is a type of standing wave flume widely used. However, its use requires application of different equations, based on the throat size, if accuracy in results similar to other types of flumes is expected.

The approximate equation applicable for the entire range of its usage, namely, discharges varying from 0.001 m³/s to 100 m³/s (i.e., throat widths varying from 75 mm to 15,000 mm) is given by:

$$Q = 2.42 W h^{2.58} \quad (8.10)$$

Where;

Q = discharge in m³/s

W = throat width in m and

h = upstream gauged depth in m,

The numerical factors 2.42 and 2.58 are subject to 4% variation in extreme cases (less in case of smaller widths).

The minimum head and accuracy will be the same as for standing wave flumes.

IS: 14371/ ISO: 9826 prescribes various methods to be adopted for measurement of flow of water in open channels through Parshall Flume in water treatment plant.

Simplified formulae of measurement of flow in open channels

90° V notch : $Q = 1.38 H_w^{5/2}$

Rectangular Weir/notch : $Q = 1.84 B H_w^{3/2}$

Parshall Flume : $Q = 2.27 W H_a^{3/2}$

(Source: Shulz and Okun: Surface Water Treatment for communities in Developing Countries)

Where;

Q = Discharge (m³/sec)

H_w = Head on weir (m)

H_a = Depth at entrance to the flume at specified measuring point (m)

B = Length of the weir (m)

W = width of throat (m)

8.4.4 Instruments – Flow Indicators and Recorders

8.4.4.1 Simple Calibrated Scale

Simple Calibrated Scale is a device which indicates the depth of flow which can be used further to calculate the discharge with the help of some calibrated equations.

8.4.4.2 Float and Dial Type Indicator

Float Flow Meter is also called rotor flow meter. It is mainly used for small and medium caliber flow measurement. It can measure liquid, gas, steam, etc. based upon the Bernoulli's theorem. It consists of a uniformly tapered flow tube, a float and a measurement scale. It consists of two parts. One of the rotameter is a tapered tube that gradually expands from bottom to top. The other is a rotor that is placed in the tapered tube and can move freely up and down along the centre-line of the tube.

When measuring the flow of fluid, the measured fluid flows in from the lower end of the tapered tube. As the fluid is introduced into a uniformly tapered flow tube a float rises, its weight supported by the fluid flowing underneath, until the entire volume of fluid can flow past the float. The position of the float corresponds to a point on the tube's measurement scale and provides an indication of the fluid's flow rate. The flow of fluid impacts the rotor and produces a force on it. The magnitude of this force varies with the flow rate. Rotameters are simple, reliable, inexpensive, easy to install flow measuring instruments, however, they are generally not suitable for low-flow applications and precautions need to be taken to keep the rotameters (floats) vertically to maintain the accuracy in flow measurements.

Float and Dial Type Level Indicator:

In water supply system, water tanks are used at different stages requiring level measurements in the tanks. Float and dial type level indicator is used for measuring the level in fluid tanks such as water tanks and sewage tanks. It consists of a float, tied to a SS wire rope other side of which is wound on a drum carrying constant torque spring to maintain the rope under continuous tension. Due to change in fluid level the float rises or falls and rotate the drum. This motion is transmitted through gear mechanism to a pointer moving over a calibrated dial to display level in meters. Digital/analogue output can also be provided. Float type level indicators are simple construction and are easy to install. They are very sturdy and require very less maintenance.

8.4.4.3 Mechanical Integrator

The ball-and-disk integrator is a key component of many advanced mechanical integrators. A float is attached to the input carriage so the bearing moves up and down with the level of the water. As the water level rises, the bearing is pushed farther from the center of the input disk, increasing the output's rotation rate. By counting the total number of turns of the output shaft (for example, with an odometer-type device), and multiplying by the cross-sectional area, the total amount of water flowing past the meter can be determined. It is easy to maintain and operate.

8.4.4.4 Ultrasonic Flowmeter

The flow is measured based on the ultrasonic pulse which is emitted and received by transducers. The transit times of the pulses depends on the velocity of the fluid through

which it passes. These transit times are measured, and their difference is proportional to the fluid flow rate. The disadvantage of this flow meter, the flow rate depends on a cross-sectional velocity profile.

8.4.4.5 Electro Magnetic Probe Method

When an electro-magnetic probe is immersed in flowing water, a voltage is created around the probe. This voltage, sensed by electrodes imbedded in the probe is transmitted through the cable to the meter box. The voltage created by water flowing through the magnetic field is proportional to the velocity of flow of water. These small voltages are electronically processed and displayed on the panel meter.

Details of various meters can be referred from Chapter 13 in Part A of this manual.



Figure 8.3: Ultrasonic Flow Indicator over Parshall Flume



Figure 8.4: Float and Dial Type Mechanical Indicator

8.5 Coagulation And Flocculation

'Coagulation' describes the effect produced by the addition of a coagulant and coagulant aids to a colloidal dispersion, resulting in particle destabilization and charge neutralization. Operationally, this is achieved by the addition of appropriate coagulant and coagulant aids and rapid intense mixing (the coagulants are mixed in a flash and therefore, this unit is also called as a *flash mixer*) for obtaining uniform dispersion of the coagulant.

'Flocculation' is the second stage of the formation of settleable particles (or flocs) from destabilized colloidal sized particles and is achieved by gentle and prolonged mixing which ensures continuous multiple re-contacts of solids.

In modern terminology, agitation (slow mixing) that produces aggregation of particles is designated by the single term "flocculation". It is a common practice to provide an initial rapid or flash mixing for dispersal of the coagulant or other chemicals into the water followed by slow mixing where growth of floc takes place.

8.5.1 Rapid Mixing

Rapid mixing is an operation by which the coagulant is rapidly and uniformly dispersed throughout the volume of water, to create a more or less homogeneous single or

multiphase system. This helps in the formation of micro-flocs and results in proper utilization of coagulant preventing localization of concentration and premature formation of hydroxides which lead to less effective utilization of the coagulant. The coagulant is normally introduced at some point of high turbulence in the water. The sources of power for rapid mixing to create the desired intense turbulence are gravitational, mechanical and pneumatic.

The intensity of mixing is dependent upon the temporal mean velocity gradient, 'G'. This is defined as the rate of change of velocity per unit distance normal to a section (or relative velocity of two flow lines divided by the perpendicular distance between them) and has the dimensions of and generally expressed as s^{-1}). The turbulence and resultant intensity of mixing is based on the rate of power input to the water and G can be measured or calculated in terms of power input by the following expression:

$$G = \sqrt{\frac{P}{\mu \times Vol}} \quad (8.11)$$

Where;

- G = Temporal mean Velocity gradient, s^{-1} ;
- P = Total input of power in water, watts
- μ = Absolute viscosity of water, $N.s/m^2$., and
- Vol = Volume of water to which power is applied, m^3

Where head loss through the plant is to be conserved as much as possible and where the flow exceeds $300 m^3/hr.$, mechanical mixing also known as flash mixing, is desirable. Multiple units may be provided for large plants. Normally a detention time of 30 s to 60 s is adopted in the flash mixer. Head loss of 0.2 m to 0.6 m of water, which is approximately equivalent to 1 Watts/ m^3 to 3 Watts/ m^3 of flow per hour is usually required for efficient flash mixing. Gravitational or hydraulic devices are simple but not flexible, while mechanical or pneumatic devices are flexible, but require external power and maintenance of rotary parts.

The IS 7090 (1985, Reaffirmed 2001): Guidelines for rapid mixing devices may be referred to for details, which lays down design considerations, guidelines for materials and methods of construction of the different types of rapid mixing devices:

- a) Mechanical - i) Vane type mixer, ii) Propeller type mixer, and iii) Jet type mixer; and
- b) Hydraulic - i) Hydraulic jump, and ii) Baffled channel

8.5.1.1 Gravitational or Hydraulic Devices

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

(a) Hydraulic Jump Mixing

This is achieved by a combination of a chute followed by a channel with or without a sill. The chute creates super critical flow (velocity 3 m/s to 4 m/s), the sill defining the location of the hydraulic jump and the gently sloping channel induces the jump.

Standing wave flumes specially constructed for measurement of flow can also be used in which the hydraulic jump takes place at the throat of the flume. In the hydraulic jump mixing, loss of head is appreciable (0.3 m or more) and the detention time is brief. This device though relatively inflexible, is simple and free from moving pans. This can be used as a standby in large plants to the mechanical mixers while for small plants, this can serve directly as the main unit. Typical residence time of 2 s to 5 s and G value of 800 s^{-1} to 1000 s^{-1} have been reported. Overflow weirs have also been used for rapid mixing. A head loss of 0.3 to 0.6 m across the weir has been reported.

Weir Mixing

Sudden drop in hydraulic level of water over a weir can cause turbulence and coagulants can be added at this "plunge" point with the aid of diffusers. This is a rectangular weir or a notch. A chamber of 1 min detention time is provided on the upstream side of the weir. A free fall of 0.5 m to 0.60 m on the downstream side creates sufficient turbulence (G value 800 to 1000 sec^{-1}) for instantaneous mixing of raw water with coagulants. A small chamber of detention time 5 s to 10 s is provided on the downstream side of the thorough mixing area.

Perforated Coagulant feeder pipe/pipes are provided about 0.30 m to 0.50 m above the "nappe" of the weir to ensure the penetration of coagulants in the falling water. The coagulant feeder pips generally dose primary coagulants like Aluminium or Ferric salts. If the alkali (viz., lime) is required to raise the alkalinity, then that pipe is provided prior to coagulant pipe. Polyelectrolytes if provided are dosed after 10 s (detention time) in the conveying channel (to flocculation tanks) in the diminishing zone of turbulence, so as not to break the heavy molecular weight polymer chains. The conveying channel with detention time of 15 - 20 s is provided with circular pipe staggered baffles to complete the coagulation process. Correct location of coagulation feeder pipe, as described above is absolutely crucial to ensure the effective mixing and optimum consumption. This is the most simple and effective maintenance free Rapid Mix Unit. The power is required in terms of the head for free-fall which is normally embedded in the raw water supply head (viz pumps). It is highly recommended to use this unit for small as well as large capacity plants. Since most of the plants are operated at a uniform flow rate, issue of the lack of flexibility in operation, does not come in to picture. The weir mixers are recommended to be adopted for small, medium and large plants.

G (m/sec/m) value can be found out by this expression

$$G = \sqrt{\frac{(\rho \times g \times h \times Q)}{(k \times v)}} \quad (8.12)$$

Where;

Density of liquid (ρ) = 1000 kg/m^3

Head-loss over weir (h)	= m
Rate of flow (Q)	= m ³ /sec
Dynamic viscosity (k)	= 0.00101 Ns/m ² (20° C)
Volume of chamber (v)	= m ³

(Ref Renato Pinheiro, Proceedings of seminar from AWWA Annual Conference 1974)



Figure 8.5: Rapid Mix Weir - Small Plant

(b) Baffled Channel Mixing

In this method, the channel section (neglecting the baffle) is normally designed for a velocity of 0.6 m/s. The angle subtended by the baffle in the channel is between 40° to 90° with the channel wall. This angle should ensure a minimum velocity of 1.5 m/s while negotiating the baffle.

The main walls of the channels are constructed of brick masonry, stone masonry or reinforced cement concrete finished smooth to avoid growing of weed etc. The baffles are made of concrete or brick, finished in the same manner as the channel. A minimum free board of 150 mm is normally provided.

(c) In-Line Mixers

In pressure conduits, the coagulants can be added at the throat of a Venturi or just upstream of orifice located within the pipe- In this system, no effective control is possible even though mixing takes place. Rapid mixing can also be obtained by injection of coagulants preferably, in the suction end or delivery end of low lift pumps where the turbulence is maximum. In this system also, the detention time is brief while the cost is low. Many manufacturers have developed artificial in-line mixers (Static Mixers) by using different profile shapes. However, they are not very

much practiced in India. However, in future they may find a place in the coagulation practices when the enough data is available on design parameters.

(d) Other type of Hydraulic Mixing

Sudden drop in hydraulic level of water over a weir can cause turbulence and coagulants can be added at this "plunge" point with the aid of diffusers. Similarly in pressure conduits, the coagulants can be added at the throat of a Venturi or just upstream of orifice located within the pipe. In this system, no effective control is possible even though mixing takes place. Rapid mixing can also be obtained by injection of coagulants preferably, in the suction end or delivery end of low lift pumps where the turbulence is maximum. In this system also, the detention time is brief while the cost is low.

8.5.1.2 Mechanical Devices

Propeller type impellers / turbines are commonly employed in flash mixers, with high revolving speeds ranging from 100 to 1400 rpm or more. The blades are mounted on vertical or inclined shaft and generate strong axial currents. Turbine types and paddle types are also used. In the design of a mechanical flash mixer unit, a detention time of 30 s to 60 s. is provided. The relatively high-powered mixing devices should be capable of creating velocity gradients of 300 s^{-1} (m/sec/m) or more. Power requirements are ordinarily 1 to 3 watts per m^3/hr . of flow. Usually, the flash mixers are deep, circular or square tanks. The usual ratio of impeller diameter to tank diameter is 0.2 to 0.4 and the shaft speed of propeller greater than 96 rpm imparting a tangential velocity greater than 3 m/s (optimum 3 m/s to 4 m/s) at the tip of the blade. The ratio of tank height to diameter of 1: 1 to 3: 1 is preferred for proper dispersal when one set of impeller/turbine is located on the vertical shaft. Vertical paddle type impeller creates radial flow loops one above the impeller and one below the impeller. The advance turbines like pitch blade or hydrofoils are energy efficient and predominantly create axial flows. They can be applied when design data is available from the manufacturers.

In case of circular tanks, vertical strips or baffles, projecting $1/10$ to $1/12$ tank diameter, at minimum of four places, along the walls of the tank should be provided to reduce vortex formation or rotational movement of water about the impeller shaft. This problem can be resolved by using tanks square in the plan. The mixing chamber can be placed below the coagulant feed floor ensuring short coagulant feed lines. The usual mechanical agitator drive is an electric motor with continuous duty, operating through a reduction gear. Good results are achieved by adding the coagulant at the inlet or just above the top of the blade or the propeller in the tank. Coagulant aid (polymer) if employed need to be dosed at the outlet in diminishing velocity gradient zone. Mechanical type consumes very little head of water and permits flexibility of operation. When there is possibility of short circuiting in the tank, one more compartment may be provided. This requires more external power input and needs constant attention and maintenance. The location of inlet and outlet need to be diametrically or diagonally opposite (in plan as well in section) to avoid short-circuiting of

Part A: Engineering Design

flow. The mechanical agitators consume high amount of electrical power and need regular maintenance of rotary parts. If process demands then flow path through the mixer could be altered by placing a baffle at the inlet or outlet and is a good engineering practice.

Formulae governing the design for flat blade turbines:

Net energy input is

$$P = kG^2 V$$

Where;

P = Net (or water) Power, Watts

G = Velocity gradient, m/sec/m

k = dynamic viscosity, Ns/m²

V = volume of the tank, m³.

And

$$P = \frac{1}{2} C_D \times d \times A_p \times V_D^3 \quad (8.13)$$

Where;

P = Net (or water) Powers, Watts

C_D = drag coefficient, dimensionless number, 1.8 for flat blades

ρ = density of water, kg/ m³

A_p = area of paddles, m²

V_D = velocity differential, 0.75 x V_p, where V_p is paddle tip velocity

Table 8.1: Recommended Detention Time and Net Power Required

Note: Power calculations are based on water temperature of 30°C (μ= 0.8x10⁻³ N.S./m²)

Detention Time	Velocity gradient	Net Power input per unit volume	Net Power input per unit discharge
s	s ⁻¹	Watts/m ³ of volume	Watts /m ³ of flow/hr.
60	300	72	1.2
50	360	104	1.4
40	450	162	1.8
30	600	288	2.4
25	720	415	2.9
20	900	648	3.6

A lot of new development has taken place in the impeller design. It is recommended to use pitch blade turbine or other turbine with axial flow as well as radial flow characteristics. The axial flow characteristics could be utilised to have a recirculation action to ensure very effective mixing. Normally the direction of recirculation (pumping up/down) is kept against the general flow direction in the tank. It is recommended to have a pumping capacity of 1Q

to $2Q$ for the impeller where Q is the flow through rate. The manufacturers of these turbines furnish the data of head and flow characteristics from which the turbine selection can be done.



Figure 8.6: Flat blade Turbine flow mixers

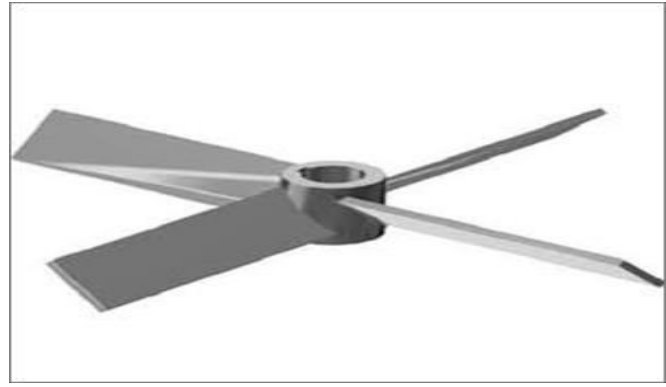


Figure 8.7: Pitch blade turbine or radial (Axial – Radial flow characteristics)

8.5.1.3 Pneumatic Devices

When air is injected or diffused into water after suitable compression, it normally expands isothermally and the resultant work done by the air can be used for necessary agitation. They are not common in water works practice. The typical range of velocity gradients and contact times are in the range of 3000 s^{-1} to 5000 s^{-1} and 0.5 s to 0.4 s. respectively.

Taking into account the various types of rapid mixing devices velocity gradients and the detention times, the following equation is proposed:

$$G = 2790 t^{-0.35} \quad (8.14)$$

Where;

G = Velocity gradient, s^{-1}

t = Detention time, s (in Sec.)

In the field, it has been observed that the detention time reduces much faster with increase in the value of G . Hence the $G.t$ value instead of remaining constant reduces with increase in G value. Equation 8.8 is based on this field experience. Variation in the value of G could be from 300 s^{-1} to 5000 s^{-1} .

Location of coagulant dosing points

The primary coagulant is dosed just before the zone of maximum velocity gradient (turbulence) in the tank. For Weir Mixer it is located over the “nappe” of free-falling weir. In the mechanical Rapid Agitator tank, it is near to the inlet of the tank. In the Weir Mixer, the coagulant aid is dosed about 8 s to 10 s in the downstream mixing channel. In the Mechanical Agitator, coagulant aid is dosed near the outlet of the tank (or below the

agitator) in the diminishing zone of velocity gradient. Lime if dosed to supplement the alkalinity deficiency in the raw water, is introduced prior or along with the primary coagulant.



Figure 8.8: Dosing point locations in the WTP with Flash Mixer

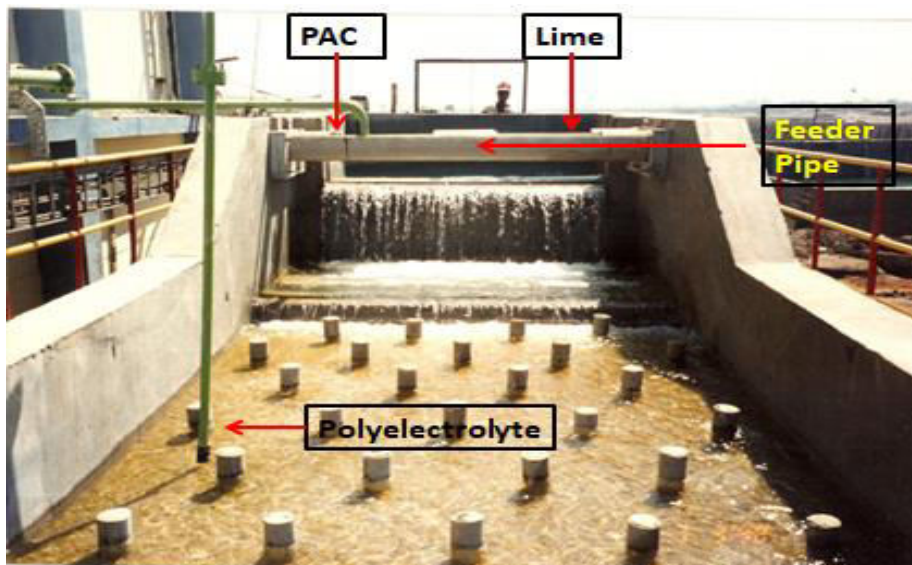


Figure 8.9: Dosing point locations in the WTP with Weir Mixer

Undesirable dosing practices

At a few places in India, it is observed that the operator dump blocks directly in to the Elevated Channel or Mixing Channel. This is totally undesirable as the alum mixing and dose becomes non uniform resulting in deteriorated treated water quality. The primary reason for this is the non-functional or defunct alum agitators or the clogged piping and valves. Agitators need to be robust with well-designed and reliable drives consisting of gearboxes and motors. The paddles and shaft need to be fabricated in non-corrosive material like SS 316. The piping should be either Rigid PVC or HDPE with size large enough so as to not to get clogged. Impurities in Alum are also responsible for the clogging. This is one

more reason for switching over to PAC (Poly Aluminium Chloride) as a primary coagulant. It is generally observed that powdered PAC doses required are one third of the alum doses in mg/L.

General observations on coagulant doses and raw water turbidity

Jar test studies and plant performance confirm the fact that for raw waters of low turbidity relatively high coagulant (Alum) doses are required. To trap or enmesh fewer colloidal particles (TSS), large volume of Aluminium Hydroxide floc is required (sweep flocculation). On the contrary for higher turbidity (higher colloidal particles) relatively smaller amount of alum doses are required. In this process the colloidal particles get attached through bonding by aluminium hydroxide molecules through the inter-spaces (Charge Neutralization). Optionally, in case of low turbidity water there are chances of aluminium getting carried over in the clarifiers, which is not desirable from human health point of view. Hence it is advisable to operate the plant in “direct filtration” mode when the turbidity is less than 10 NTU. The direct filtration mode (bypassing clarifiers) require small amount of alum dose to form micro-flocs. Coagulant and coagulant aids Feeding

The coagulants are introduced into the water for the purposes of ~~pH correction~~, coagulation and flocculation. In general, coagulants are added as solutions or dilute suspensions. As the treatment is a continuous process, the flow of coagulant is regulated and measured continuously, which can be either the solution feed type or the dry feed type. The installation of coagulant feeders obviously promotes the uniform distribution of coagulants and eliminates wastage. Every coagulant feeder should be arranged and positioned in such a way that checking of the dosing rate can be made at regular intervals to verify the discharge rate. The reaction of alum with water molecule needs sufficient **alkalinity to react and form the flocs. If needed alkalinity chemicals also to be added.**

8.5.2 Solution Feed

Preparation of the solution of the coagulant in the water of desired strength is the first step and is done in the solution tanks. This solution is fed to the raw water through controlled feeders which are of gravity or pressure type. This selection of the proper type of feeders and the point of application are important. For example, when mixing is done in a channel, it should be at a number of points in the cross-section of maximum turbulence. Also, as different coagulants are to be fed at different points, the location at which the coagulants are fed is important to derive maximum efficiency.

8.5.2.1 Solution Tanks

There should be at least two tanks for each coagulant feed. The capacity of each tank should generally be such as to hold 12 hours requirement at the maximum demand of coagulant at the design flow. If three tanks are provided, each should have a capacity of 8 hours requirement. A minimum freeboard of 0.3 m is necessary. Dissolving trays or boxes and also adequate facilities for draining the solution tanks should be provided.

The solution tank may be constructed in reinforced cement concrete. Coating with bituminous paint may be adequate for alum tanks, while for tanks for handling other corrosive coagulants, a suitable lining of rubber, PVC, or Epoxy resin may be necessary to resist corrosion. Small tanks can be of plastic (Polyethylene or Polypropylene) material.

The coagulant solution tanks should be located in or as near the coagulant storage area as possible to avoid unnecessary lifting and handling of coagulants. These tanks should preferably be located at a suitable elevation to facilitate gravity feed of the coagulant solution.

A lifting tackle for lifting the coagulants to the elevated tanks should be provided. Each tank should have a platform that should be at least 1 m wide to allow the workers sufficient space for handling the coagulants and preparing the solution; wherever necessary, the platforms should have railings upto a minimum height of 1.0 m. The platforms/walkways should be located at an elevation to have clear headroom of 2.5 m from the ceiling. The top of the solution tank should not be higher than 1.0 m from the floor of the platforms.

A Dissolving Trays or Boxes

After being carefully weighed, the coagulants are placed into the dissolving trays, which vary in size to suit the capacity of the treatment plant. The trays or boxes may be constructed of wood, cast iron, cement, or cement concrete with slots or perforations at the sides and bottom. These may be placed either inside or just above the solution tanks.

For small tanks, a pipe perforated with small holes to provide a spray of water to help dissolve the coagulants may be placed above these trays. For plants of medium and large size, dissolving boxes should preferably be constructed of concrete with a pipe manifold having holes either at the bottom or at the sides for dissolving coagulants.

B Preparation of Solutions

It is essential to ensure that all the coagulants are dissolved before the solution is put into operation and the homogeneity of the prepared coagulant solution is maintained. This can be achieved by proper mixing either by compressed air, recirculating the solution, or mechanical agitation. Manual mixing may be adopted for very small plants with capacities not exceeding 3.0 MLD, ensuring proper mixing. Common practice is to provide electrically driven mechanical agitators. The drive mechanism shall be located on the top of the tank. The agitator shaft and paddles shall be of SS 304/316 material.

Knowledge of the solubility characteristics of the coagulant as well as the solution strength that is used in normal practice will facilitate the choice of feed equipment. The solution strength of alum which is the most widely used coagulant, shall not be more than 5% for manual operations and 10% for other operations with efficient mixing. It may be desirable to dilute down to 1% prior to addition. For other Coagulants, reference may be made to Appendix 8.10, which gives the strengths to be used with mechanical mixing. With the manual operation, lower strengths are recommended.

The coagulant solution is conveyed from the solution tanks to the point of application by means of coagulant feed lines. These should be as short and straight as possible, and the coagulant feed pipes shall be of Rigid PVC or HDPE of suitable strength.

Liquid Alum

Liquid alum contains 5.8 % to 8.5% water soluble alumina as against 17 % for crystalline alum but is lower priced. Since its use also avoids the construction of solution tanks, it may be economical in large plants especially if the waterworks are within a reasonable trucking distance of alum producing works. Acid proof equipment such as rubber-lined or stainless-steel tanks and piping is necessary for transport, handling, and storage.



Figure 8.10: Solution Preparation tanks, Dissolving basket, and Agitator drive

C Solution Feed Devices

Solution feed devices are used to regulate the doses of coagulants fed into water. The rate of flow of the coagulant solution of known strength prepared in the solution tank is measured by means of either an orifice Rota meter, positive displacement pump, or weirs. The solution feed equipment should be simple in operation and corrosion resistant.

The constant head orifice is the most common device used for measuring the rate of flow of solution. It is usually contained in a unit consisting of corrosion resistant, a constant-level box with a float valve and an orifice. The orifice can be of either variable size or constant size, the adjustment in the latter being made by using the required size to give the desired rate of flow. The unit should also be capable of adjustment to allow setting for various depths of solution in the box.

In large systems, automatic control of coagulant feed could be practiced which assures that the quantity of coagulant measured is not prone to human errors. The principle

must be based upon the measurement of some attributes of the water such as the rate of flow, pH, colour, conductivity, chlorine residual. Since the flow of water can fluctuate, it is necessary to maintain the flow of coagulant in a fixed proportion to the flow of water for which a proportional feed device is necessary. The IS 9222-1 (1990, reaffirmed 2001): Recommendations for handling and dosing devices for chemicals for water treatment, Part I: Coagulants which covers recommendations for handling, storage, weighing, method of preparing solution and dosing devices of coagulants such as alum and iron salts used in water treatment plant may be referred to.

Measurement of the water can be done in a number of ways, the simplest possibly being the tipping bucket or a pump with positive meter which provides a positive method of measurement but is applicable to the smaller installations only. The more common measuring device is a weir, Venturi tube or orifice plate. In this regard, the below listed IS codes of practice explicitly describes various methods of flow measurement which may be referred to,

- a) IS 9922 (2010): Measurement of Liquid Flow in Open Channels- General Guidelines for Selection of Method which provides general guidelines for the selection of a suitable method for measurements of liquid flow in open channels.
- b) IS 1192 (1981, reaffirmed 2001): Velocity Area Methods for Measurement of Flow of water in Open Channels.
- c) IS 6330 (2012): Liquid Flow Measurement in Open Channels by Weirs and Flumes - End Depth Method for Estimation of Flow in Rectangular Channels with a Free Overfall (Approximate Method), which specifies a method for the estimation of sub-critical flow of clear water in smooth, straight, rectangular prismatic open channels with a vertical drop and discharging freely. Using the measured depth at the end, the flow in rectangular channels (horizontal or sloping) with confined nappe and unconfined nappe may be estimated.
- d) IS 9108 (1979, reaffirmed 2003): Liquid flow measurement in open channels using thin plate weirs, which specifies methods for measurement of water flow in open channels using rectangular and triangular-notch (V-notch) thin- plate weirs.
- e) IS 13083 (1991, reaffirmed 2003): Liquid flow measurement in open channels flat-V weirs which deals with the measurement of flow in rivers and artificial channels using flat V weirs under steady or slowly varying flow conditions.
- f) IS 14574 (1998): Measurement of Liquid Flow in Open Channels by Weirs and Flumes - End Depth Method for Estimation of Flow in Non-rectangular Channels with a Free Overfall (Approximate Method), which specifies a method for the estimation of subcritical flow of clear water in smooth, essentially horizontal, straight open channels with a vertical drop and discharging freely. Gentle positive slopes not greater than 1 in 2000 are admissible. This International Standard covers channels with the following types of cross-section, the nappe being unconfined: a) trapezoidal; b) triangular; c) parabolic; d) circular. Using the measured depth at the end, the flow can be estimated.

- g) IS 14673 (1999, reaffirmed 2004): Liquid flow measurement in open channels by weirs and flumes - Triangular profile weirs which specifies methods for the measurement of the flow of water in open channels under steady flow conditions using triangular profile weirs. The flow conditions considered are steady flows which are uniquely dependent on the upstream head and drowned flows which depend on downstream as well as upstream levels.
- h) IS 14869 (2000): Liquid Flow Measurement in Open Channels Rectangular, Trapezoidal and U-Shaped Flumes which deals with the measurement of flow in rivers and artificial channels under steady or slowly varying flow conditions, using certain types of standing-wave (or critical depth) flumes.
- i) IS 14974 (2001): Liquid flow Measurement in Open Channels by Weirs and Flumes - Rectangular Broad-crested Weirs, which lays down requirements for the use of rectangular broad-crested weirs for the measurement of flow of clear water in open channels under free flow conditions.
- j) IS 14975 (2001): Measurement of Liquid flow in Open Channels - Streamlined Triangular Profile Weirs which specifies methods for the measurement of the flow of water in open channels under steady flow conditions using streamlined triangular profile weirs.
- k) IS 15123 (2002): Hydrometric Determinations - Flow Measurement in Open Channels Using Structures - Trapezoidal Broad-Crested Weirs which specifies a method of steady-flow measurement in open channels using a trapezoidal broad-crested weir under modular and non-modular conditions.
- l) IS 15353 (2003): Liquid Flow Measurement in Open Channels by Weirs and Flumes - V-Shaped Broad-Crested Weirs which specifies a method for the measurement of sub critical flow in small rivers and artificial channels using V-shaped broad-crested weirs.

Another method is based on the actuation of a flow regulator directly or through a relay from the primary measuring unit. This usually involves the empirical calibration of some link in the system and care must always be taken to see that such arrangements are properly adjusted for they do not depend on a state of equilibrium.

The most satisfactory method of control is one that depends upon the matching of two factors, one of which is associated with the primary measuring unit (control) and the other with the flow of coagulant. For example, a Venturi tube will produce a differential pressure bearing known relationship to the flow of water through it. If it is desired to control the flow of a coagulant solution, then some similar measurement associated with the flow of a coagulant solution must be compared with the differential pressure and means provided for adjusting the flow of coagulant so that the two factors so compared are mutually in equilibrium. Such a system is basically stable.

D Solution Feeders

There are several types of solution feeders, some of which are discussed below:

(a) Pot Type Coagulant Feeders

The pot type coagulant feeder is a simple type of equipment for feeding alum or alkali into water. The coagulant, in large crystal or lump form, is charged into the feeding pot. A special orifice fitting, placed in the raw water line, contains an orifice plate which creates a pressure differential in pipes which connect the coagulant pot into the orifice fitting.

This pressure differential causes a small stream of water to flow from the high-pressure side of the orifice plate through a pipe and a regulating valve, into the bottom of the coagulant feeding pot and this forms an equivalent stream of the coagulant solution, formed in the pot, to flow out of the top of the pot into the raw water line on the low-pressure side of the orifice plate.

Since the same pressure differential acts across the regulating valve as across the orifice, the flow through the regulating valve, at any setting, is a definite fraction of the flow through the orifice. Consequently, the rates of flow of the small stream of coagulant fed to the raw water are directly proportional to the rates of flow of the raw water. These find use in small plants because they do not permit a uniform feed rate and the feed rate cannot be also checked. Sediment tanks are usually employed with these feeding lines.

(b) Pressure Solution coagulant Feeders

Pressure solution coagulant feeders are much more accurate than the pot type coagulant feeders. In these a coagulant solution of a definite strength is made by dissolving a weighed amount of coagulant in a specified volume of water in the coagulant solution tank. This batch of coagulant solution, when required, is charged into the displacement tank through the bottom. As the specific gravity of the coagulant solution is higher than that of water, the water in the displacement tank is displaced upwardly to waste through a valve.

A sight glass at the side of the feed tank has in it a glass float, which is so constructed that it floats in the heavy coagulant solution but sinks in water. This float indicates, at all times, the level of the coagulant solution thus notifying the operator when recharging is necessary.

A special orifice fitting, placed in the raw water line, contains an orifice plate which creates a pressure differential in the pipes connecting the displacement feed tank to the orifice fitting. This pressure differential causes a small stream of water to flow from one side of the orifice plate. The greater part of this stream flows through a secondary orifice and the smaller through an adjustable needle valve into the top of the displacement feed tank, where it displaces downwardly an equivalent stream of the heavier coagulant solution.

This small stream of coagulant solution is diluted when it discharges on the other side of the secondary orifice into the water flowing through this orifice and this diluted coagulant solution is fed into the raw water line on the other side of

primary orifice. This dilution serves to make the density of the effluent column approach the density of the influent column thus assuring a greater degree of accuracy, at varying flow rates, than is possible with a single orifice control.

Since the same pressure differential acts across the primary orifice as across the needle valve, the flow through the needle valve at each setting, is a constant fraction of the flow through the primary orifice. As the rates of flow of the coagulant solution are directly proportional to the rates of flow of the raw water, this type of feed is applicable to water supplies of varying flow rates and pressure. Sediment tanks are usually employed with pressure solution coagulant feeders to keep sediment out of the feeding line. In cases, where corrosive coagulants are handled, special pressure solution coagulant feeders are employed.

(c) Electro-coagulant Feeders

The water flows through an integrating raw water meter causing an electrical circuit to start the feed control unit through a time switch. The feed control unit is a mechanism designed to lower the swing draw off pipe at a rate which is proportional to the rate of flow of raw water. It consists of a motor, a speed reducing mechanism, two drums on which separate tapes are wound, a manual rewinding mechanism, a switch for operating an alarm for stopping the feed at low level in the solution tank and a dial for indicating directly the depth of solution removed from the tank.

(d) Gravity Orifice coagulant Feeders

The gravity orifice coagulant feeder is limited in application to those cases where the flow rate of the water being treated is constant. The solution from the coagulant solution tank flows by gravity, through a strainer and through a float valve, into the orifice box.

The float valve keeps the coagulant solution in the orifice box always at the same level so that the adjustable orifice operates under constant head. By gravity, the coagulant solution flows from the orifice box through the adjustable orifice to the point of application. Instead of orifice a calibrated 15° V notch with graduation is also provided in dosing tanks.

To stop and start the coagulant and water simultaneously, a float switch may be used in the settling basin to operate a solenoid operated valve on the orifice box discharge and an electrically controlled valve on the raw water line. Thus, the flows of raw water and coagulant solution are stopped whenever the level of the water in the basin has reached a certain height. When the level has fallen a certain distance, the float switch closes an electric circuit thus starting simultaneously the flows of raw water and coagulant solution.

Instead of being connected to an electrically controlled valve in the raw water line, the float switch may be connected so as to start or stop a raw water pump simultaneously with the starting or stopping of the coagulant feeder. The amount of coagulant

solution fed to the raw water may be varied over a wide range by means of the adjustable orifice located in the orifice box.

Instead of the coagulant solution flowing by gravity to the point of application, it may be discharged in to a pump suction box from which it is pumped to the point of application.



Figure 8.11: Gravity feed dosing box with V notch

(e) Reciprocating / Positive Displacement Pump Coagulant Feeders

This method of feeding coagulant employs a motor-driven reciprocating coagulant pump. The pump withdraws a coagulant solution, or suspension of suitable strength, from a tank and discharges the solution or suspension to the point of application under any desired pressure. The feeding pump may be designed to treat either a variable or a constant flow of water.

The coagulants to be fed are prepared in solution tanks. If the coagulant to be fed is relatively insoluble, a high-speed motor-driven agitator maintains uniform suspension throughout the full depth of the tank. If the coagulant forms a clear solution, a dissolving basket is furnished and the mechanical agitator is omitted.

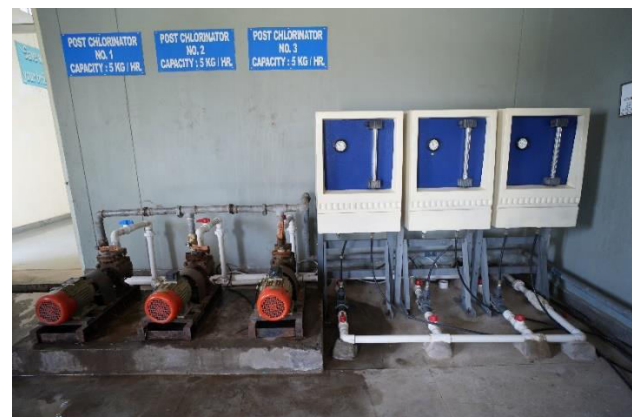


Figure 8.12: Reciprocating / Positive Displacement Pump Coagulant Feeders**(f) Variable rate proportional Feeders**

If the rate of flow of water being treated varies, proportional feeding of coagulants is necessary. This is carried out by accurately measuring the amount of coagulant fed by the pump. This pump is a proportioning and metering device which delivers a definite volume of coagulant with each stroke. A water meter with an electrical contactor is placed in the raw water line. The contactor closes a circuit every time a given volume of water flows through the meter. The closing of the circuit energizes the motor of the reciprocating pump, which then operates to deliver a given volume of coagulant until an electric time switch breaks the circuit, thereby stopping the pump. The cycle repeats itself approximately every thirty seconds, at maximum flow, with the pump operating for approximately twenty seconds after each contact. The amount of coagulant fed is thus accurately proportioned to the flow of water regardless of variations in the rate of flow, because both the volume of water treated between meter contacts and the volume of coagulant added to treat the water are accurately measured. However, this suffers from the disadvantage that, particularly when used with alum solutions, the water is subject to an overdose and no-dose sequence. It is better to have the coagulant pump run continuously and to modulate the stroke of the pump either manually or with a mechanical device.

For a number of coagulants fed simultaneously, one meter control serves to operate any number of pumps.

(g) Constant rate feeding for uniform flow

If the flow of water being treated is constant, the coagulant pump operates continuously at the set dosage. When the flow of water ceases, the coagulant pump is stopped automatically so as to shut off the flow of coagulants. When the flow of water begins again, the coagulant feeding is automatically resumed.

(h) Adjustment of feeding rates

Two methods are available for adjusting the rate of coagulant feeding. Firstly, the length of the pump stroke can be changed to vary the rate of feeding of a given strength of solution over a wide range. Secondly, the strength of the coagulant solution or suspension in the coagulant tank can be changed when a new coagulant charge is made up so as to provide a different coagulant dosage for the same setting of the coagulant pump.

The method of adjustment of the coagulant feeding rate varies with the type of proportioning pump used. The single feed pump varies the feeding rate by a simple screw adjustment, which changes the length of the plunger stroke. The duplex pump varies its feeding rate by screwing the adjusting coupling toward the liquid end of the pump to increase the capacity or away from the end to decrease the capacity.

The reciprocating coagulant pumps can be provided with ball check valves on both suction and discharge, thus assuring maximum efficiency of displacement, non-clogging and self-cleaning features, elimination of air binding and the minimizing of wire drawing of valve seats. The check valves are readily opened, to inspect the ball checks and seats, without disconnecting either suction or discharge piping.

8.5.2.2 Dry Feed

Dry coagulant feeders incorporate a feed hopper which sometimes serves as a storage hopper also mounted above the feeding device. This device may consist of a rotating table and scraper, a vibrating trough or an oscillating displacer or some equivalent method of moving the coagulant from the point where it leaves the feed hopper to the point of discharge. The rate of movement of the coagulant determines the quantity to be discharged on a volumetric basis. Gravimetric feeders are also available in which the quantity discharged in a unit of time is continuously weighed and the speed of operation automatically controlled to maintain a constant weight. The feeder may be designed for constant rate operation or for feeding coagulants in proportion to the rate of flow of water. The dry feeders with a completely enclosed feeding mechanism have many advantages over the solution feeder like accuracy of feeding, reproducibility of feeding rate for any feeder setting with a step-less adjustment of dosage in a wide feeding range. A single feeder serves as a spare for a group of feeders handling different materials and the height of coagulant in standard or extension hopper has no effect on feeding rate. When small rates of coagulant feeding are desired, one hopper full of coagulant will allow the feeder to operate for several days unattended.

Coagulants stored in a steep-sided hopper feed downward to a discharge opening at the bottom of the hopper. Coagulants which have a tendency to arch or stick, such as lime and soda ash, are made free-flowing by a vibrator mounted on the side of the hopper at a point where it produces the most effective vibration. Exact volumes of coagulants are sliced off and displaced from the bottom of the discharge opening by an endless belt with integral lugs in the form of equally spaced partitions.

Machined guides on both sides and above the lugs insure that each pocket is filled with an exact volume of coagulant. As the belt moves forward it passes over a pulley where each pocket is stretched open as the belt goes over and then under this pulley so that all the coagulant is dropped into a mixing or dissolving chamber. A jet of water admitted tangentially or power-driven paddles in the mixing chamber provides the agitation needed for mixing or dissolving the coagulant. This coagulant solution, or suspension then overflows or is pumped to service.

Where the quantity to be handled is large a storage hopper is usually constructed above the relatively small feed hopper. The capacity of the storage hopper is usually arranged for recharging once a day or once a shift. Because of the height of such hoppers, it is almost inevitable that storage of coagulants has to be at an elevated place to obviate the need of lifting of the coagulants every time.

8.5.2.3 Coagulants

A Coagulants used and their properties

Appendix 8.10 gives the list of coagulants commonly used in water treatment and their properties.

B Coagulant Storage

The coagulant store should be of damp proof construction, properly drained. Special precautions against flooding should also be taken.

For coagulants purchased in bags, storage by piling on the floor of the store room may be arranged. A height of stack not exceeding 2 m is recommended. Hygroscopic coagulants should be obtained in moisture-proof bags and stored in air-tight containers.

All plants, particularly small ones, should keep on hand at all times, a supply of coagulants sufficient to provide a safety factor. A storage of 3 months is advisable. Minimum storage of 1 month of monsoon requirement should be provided. But this again depends upon the location of the plant as well as the source of supply, transport facilities and the arrangement made with the suppliers for the supply of coagulants.

In cases where the major storage is provided at a place away from the feed equipment, a week's storage space should be provided near the plant.

Dampness may cause severe caking even in coagulants such as aluminum sulphate which usually are free from such troubles. Quick lime gradually expands on prolonged storage and may even burst the containers if kept too long.

Coagulants such as powdered activated carbon which are likely to cause dust problems should preferably be stored in separate rooms.

Storage of acid materials near alkalis is undesirable as their contact generates considerable heat resulting in combustion. This is also true of oxidizing coagulants such as chloride of lime mixed with activated carbon. Hence, they should be isolated. It is advisable to store chlorine cylinders separately as gaseous chlorine in contact with activated carbon leads to severe fire hazards.

8.5.3 Influencing Factors

Both these stages in flocculation are greatly influenced by physical and chemical forces such as electrical charges on particles, exchange capacity, particle size and concentration, pH water temperature, electrolyte concentrations and mixing.

8.5.3.1 Coagulant Dosage

Although there is some relation between turbidity of the raw water and the proper coagulant dosage, the exact quantity can be determined only by trial conducted with Jar Tests. Even thus determined, the amount will vary with other factors such as time of mixing and water temperature. The use of the minimum quantity of coagulant determined to be

effective in producing good flocculation in any given water, will usually require a fairly long stirring periods varying from 15 min to 30 min in summer and 30 min to 60 min in the colder months, as water temperatures approach the freezing point. Addition of coagulants in excess of the determined minimum quantity may increase bactericidal efficiency. It is however, usually more economical to use the minimum quantity of coagulant and to depend on disinfectant for bacterial safety.

Very finely divided suspended matter is more difficult to coagulate than coarse particles, necessitating a larger quantity of coagulant for a given turbidity. The cation-exchange capacity of the particles of turbidity bears a significant relationship to the success of flocculation. Advance techniques for determining the dose of coagulant is by measuring “Zeta Potential” of the particles. The surface charge, or more importantly zeta potential (ζ), is determined by measuring the particle velocity induced when a potential difference is applied across a capillary cell containing the sample (Zetasizer, Malvern Panalytical). Zeta potential affects the size and density of flocs formed. Increases in density cause more rapid flocculation. Low zeta potentials reduce the electrostatic interactions between particles allowing the particles to approach closely and hence produce more compact flocs. At zeta potentials more negative than -22 mV, the effluent turbidity rises sharply as the suspended particles become effectively stabilized in the water due to mutual repulsion. The characteristics of water especially pH have considerable influence on the satisfactory formation of flocs. Some natural waters need certain adjustments in acidity or alkalinity of water.

8.5.3.2 Jar Test

Jar Test is a laboratory procedure that simulates coagulation/flocculation with differing chemical doses. The purpose of the procedure is to estimate the minimum coagulant dose required to achieve certain water quality goals. Samples of water to be treated are placed in 6 jars. Various amounts of chemicals are added to each jar and stirred, and the settling of solids is observed. The lowest dose of chemicals that provides satisfactory settling is the dose used to treat the water.

To do extensive jar testing, it is best to use a gang stirrer. A gang stirrer is an instrument that can stir multiple beakers of water at a time, so multiple different chemicals or dosages can be tested on a sample at once. The instrument also ensures that the mixing is uniform throughout the samples.

Once the stirring system is setup, Use the coagulant, flocculant, or both. Sometimes an acid or base is also needed due to certain water requiring a pH adjustment to be treated. Other equipment required for effective jar testing includes syringes, pH meters, magnets, a notebook, and a pen.

(A) Stock solution preparation

Stock solutions of coagulant, coagulant aids and other chemicals (for pH adjustment, etc.) shall be prepared at concentrations such that quantities suitable for use in jar test

can be measured accurately and with ease. The Table 1 Stock Solutions for Coagulation Test” of the IS 3025-50(2001) may be referred to for this.

(B) Full List of Jar Testing Equipment Required:

- Mixing device (Gang stirrer)
- Glass Beakers, bottles
- Syringes
- pH meter or pH paper
- Chemicals
- Timer
- Graduated Cylinder
- Magnets (for mixing plates)
- Notebook

The below listed IS code may be referred to which prescribes the sampling method and method of determining optimum dosage of coagulant (single/mixed) coagulant aids and the optimum pH of coagulation for removal of turbidity and colour caused by colloidal and non-settleable particles followed by sedimentation under quiescent (undisturbed/tranquil) condition:

- a) The BIS Code IS 3025-50 (2001): Methods of Sampling and Tests (Physical and Chemical) for Water and Waste Water, Part 50: Jar Test (Coagulation Test), and
- b) “IS 3025-1 (1987, Reaffirmed 2003): Methods of sampling and test (physical and chemical) for water and wastewater Part 1 – Sampling”.

8.5.3.3 Optimum pH Zone

There is at least one pH zone for any given water in which good flocculation occurs in the shortest time with a given dose of coagulant, or in a given time with the required minimum dose of coagulant. Coagulation should be carried out within this optimum zone using alkalis and acids for correction of pH wherever necessary. For many waters, usually those which are low in colours and well buffered and having pH in the optimum zone, no adjustment of pH is necessary. However, in waters of low mineral content, or in the presence of interfering organic matter, constant attention is needed for pH adjustment. Failure to operate within the optimum zone, may be a waste of coagulants and may be reflected in the lowered quality of the plant effluent. As a result of studies of the effect of pH on coagulation, it has been found that "the more dilute the water in total dissolved solid and the less the alum added, the narrower becomes the pH zone".

In the case of coagulation with alum, the control over the alkalinity is very important. Not only should the water contain sufficient alkalinity to completely react with the aluminium sulphate but there should be a sufficient residual to ensure that the treated water is not corrosive. A consideration of the reaction involved shows that one molecule of "filter alum" (molecular weight of $\text{Al}_2 (\text{SO}_4)_3 \cdot 18 \text{H}_2\text{O} = 666$ requires three molecules of calcium bicarbonate $[\text{Ca} (\text{HCO}_3)_2] \times 3 = 486$ for complete reaction).

If the alkalinity is expressed in terms of calcium carbonate, the theoretical requirement of 666 parts of "filter alum" works out to 300 parts of alkalinity, i.e., approximately in the ratio of 2:1. This reduction of alkalinity should be taken into consideration and sufficient alkalinity should be added to the water, if necessary. For this purpose, hydrated lime $\text{Ca}(\text{OH})_2$ is usually added, or "soda ash" (Na_2CO_3) may be used when the increase of hardness is to be avoided.

When ferrous sulphate is used as a coagulant, the pH should be maintained above 9.5 to ensure complete precipitation of the iron. This is done by the addition of hydrated lime. For this reason, the process is sometimes known as "iron and lime process".

8.5.3.4 Coagulant Aids

Coagulant aid is a coagulant, which when used along with main coagulant, improves or accelerates the process of coagulation and flocculation by producing quick-forming, dense and rapid-settling flocs.

Finely divided clay, fuller's earth, bentonites and activated carbon are the most commonly used materials as nuclei to floc formation. The particles may become negatively charged making them subject to attraction by the positively charged aluminium ion.

Activated silica, i.e., sodium silicate activated with aluminum sulphate, sulphuric acid, carbon dioxide or chlorine, when applied to water, produces a stable solution having a high negative charge which unites with the positively charged alum or other floc to make it denser and tougher. It is especially useful for clear waters that do not coagulate well with the usual processes. It has a wider range of use in water softening.

Polyelectrolytes which are polymers containing ionizable units have been used successfully as both coagulant aids and coagulants but care should be taken to guard against their toxicity. They are soluble in water, conduct electricity and are affected by the electrostatic forces between their charges. Cationic, anionic and ampholytic polyelectrolytes have been used the cationic being able to serve as both a coagulant and coagulant aid while the other two as coagulant aids primarily. Polyelectrolytes create extraordinarily slippery surfaces when spilled on floor and are difficult to clean up.

Toxicity of any polyelectrolyte has to be checked before it can be used as coagulant or coagulant aid.

8.5.3.5 Choice of Coagulant

In selecting the best coagulant for any specific treatment problem, a choice has to be made from among various coagulants, each of which may offer specified advantages under different conditions. The common coagulants used in water works practice are salts of aluminium viz., filter alum, PAC (poly-aluminum chloride), sodium aluminate and liquid alum and iron salts like ferrous sulphate (Copperas), ferric sulphate, ferric chloride and chlorinated copperas which is an equi-molecular mixture of ferrous sulphate and ferric

chloride being obtained by chlorinating ferrous sulphate. Some coagulants derived from natural products such as 'Nirmali seeds' (*Strychnos Potatorium*) have also been used.

Selection of aluminum or iron coagulants is largely decided by the suitability of type and its easy availability. Both filter alum and ferric sulphate have certain specific advantages. Alum does not cause the unsightly reddish-brown staining of floors, walls and equipment which may result when iron salts are used; nor is its solution as corrosive as the ferric form of iron salts. The dissolving of ferric sulphate also offers difficulties not encountered with alum. The trivalent aluminum ion is not reduced to a more soluble bivalent ion, as may be the case when ferric salts are used with waters high in organic matter. On the other hand, ferric floc is denser than alum floc and is more completely precipitated over a wider range. However, good flocculation with any coagulant is possible with detailed analysis of water quality and laboratory tests.

The choice of the coagulant to be used for any particular water should preferably be based upon a series of jar tests, so planned that it will permit accurate comparison of the materials being studied under identical experimental conditions. The coagulant dose in the field should be judiciously controlled in the light of the jar test values. It is seen that in India Aluminium based salts are used for treatment of surface waters. Poly Aluminium Chloride (PAC) is seen to be replacing Alum as it is efficient over a wide range of pH. Ferric salts are preferred in pretreatment of RO feed water (Desalination Process) as the membranes are not tolerant to Aluminium.

A few of the many substances used in coagulation of water are listed in Appendix 8.10. Quality of coagulants and coagulants shall be as per relevant IS code. The Table 1 of BIS code IS 9222 (Part 1) (1990, reaffirmed 2001) mentions few common coagulants used in water treatment.

8.5.4 Slow Mixing or Flocculation

Slow mixing is the hydrodynamic process which results in the formation of large and readily settleable flocs (orthokinetic flocculation) by bringing the finely divided matter into contact with the micro-flocs formed during rapid mixing. Alum is a trivalent coagulant (carrying +++ charges on its surface) and it nullifies 3 colloidal particles (each one carrying –ve charge) and therefore, Van der Waals forces of attraction increases which forms the micro-flocs and later on these micro-flocs again collide together to form a settleable floc. The flocs can be subsequently removed in settling tanks and filters.

8.5.4.1 Design Parameters

The rate at which flocculation proceeds depends on physical and chemical parameters such as charges on particles, exchange capacity, particle size and concentration, pH, water temperature, electrolyte concentration, time of flocculation, size of mixing basin and nature of mixing device. Influence of these and other unknown factors which vary widely for different waters, is not yet fully understood. Information on the behavior of the water to

be treated can be had by examination of nearby plants treating similar water and by laboratory testing using Jar Test. as described in sub section 8.5.4.2.

The physical forces of slow mixing of the coagulant led water and adhesion, controlled by coagulant and electrical forces are responsible to a large extent in influencing the flocculation processes.

Slow mixing is meant to bring the particles to collide and then agglomerate. The rate of collision among the particles is dependent upon the number and size of particles in suspension and the intensity of mixing in the mixing chamber.

Since flocculation is a time-rate process, the time provided for flocculation to occur is also significant factor in addition to the intensity of agitation and the total number of particles. The number of collisions is proportional to $G.t$, where G is a velocity gradient & t is the detention time of the flocculation basin. The product $G.t$ is non-dimensional and is a useful parameter for the design and operation of flocculation.

The desirable values of G in a flocculator vary from 20 s^{-1} to 75 s^{-1} and $G.t$ from 2 to 6×10^4 for aluminium coagulants and 1 to 1.5×10^5 for ferric coagulants. The usual detention time provided, varies from 10 min to 30 min. Very high G values tend to shear flocs and prevent them from building to size that will settle rapidly. Too low G values may not be able to provide sufficient agitation to ensure complete flocculation.

Another useful parameter is the product of $G.t$ and the floc volume concentration ' C ' (Volume of floc per unit volume of water). This parameter $G.C.t$ reflects to a certain extent the contact opportunity of the particles but the usefulness of this parameter is not yet fully established. The values are of the order of 100.

To ensure maximum economy in the input of power and to reduce possible shearing of particles floc formation, tapered flocculation is sometimes practiced. The value of G in a tank is made to vary from 100 in the first stage to 50 or 60 in the second stage and then brought down to 20 s^{-1} in the third stage in the direction of flow.

Tapering of velocity gradient in the direction of flow ensured effective Flocculation. For this, two or three tanks are provided in series. Highest G value is provided in the first tank and lowest in the last tank.

Table 8.2: Correction factors for Detention Time with respect to water temperature

Temp (° C)	Dynamic Viscosity (k) Ns/m ²	Detention Time factor
0	0.001792	1.35
5	0.00152	1.25
10	0.00131	1.15
20	0.001009	1.07
25	0.000895	0.95
30	0.000800	0.90
40	0.0006531	
50	0.0005471	

8.5.4.2 Types of Slow Mixers

Similar to rapid mixing units, these can be categorized under gravitational or hydraulic, mechanical and pneumatic. The hydraulic type uses the kinetic energy of water flowing through the plant created usually by means of baffles, while mechanical type uses the external energy which produces agitation of water.

(1) Gravitational or Hydraulic Type Flocculator

Several types of gravitational or hydraulic flocculators are used in practice which are discussed as follows. All Hydraulic Flocculators require relatively less maintenance than Mechanical flocculators.

(a) Horizontal Flow Baffled Flocculator

Figure 8.13 shows the plan of a typical horizontal flow baffled flocculator. This flocculator consists of several around-the-end baffles with in between spacing of not less than 0.45 m to permit cleaning. Clear distance between the end of each baffle and the wall is about 1.5 times the distance between the baffles, but never less than 0.6 m. Water depth is not less than 1.0 m and the water velocity is in the range of 0.10 m/s to 0.30 m/s. The detention time is between 15 min and 20 min. The flocculator is well suited for very small treatment plants. It is easier to drain and clean. The head loss can be changed as per requirement by altering the number of baffles. The velocity gradient can be achieved in the range $10\text{--}100\text{ s}^{-1}$.

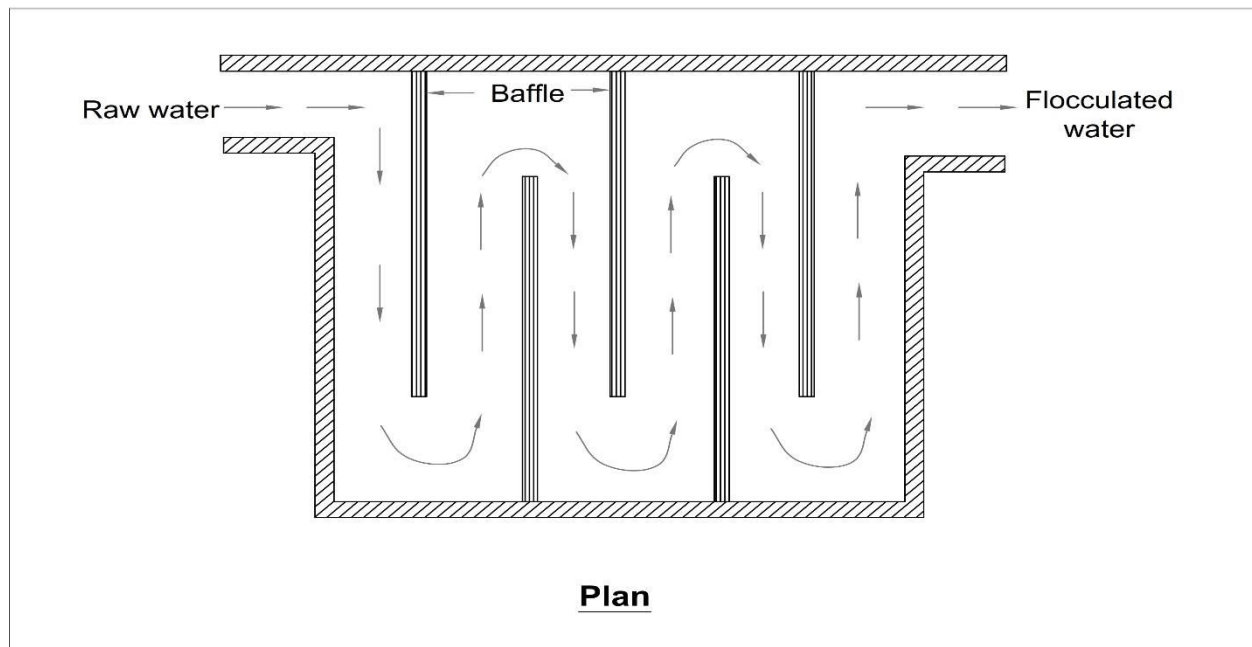


Figure 8.13: Plan of a Typical Horizontal Flow Baffled Flocculator

(b) Vertical Flow Baffled Flocculator

Figure 8.14 shows the cross section of typical vertical flow baffled flocculator. The distance between the baffles is not less than 0.45 m. Clear space between the upper edge of the baffles and the water surface or the lower edge of the baffles and the basin bottom is about 1.5 times the distance between the baffles. Water depth varies between 1.5 to 3 times the distance between the baffles and the water velocity is in the range 0.1-0.2 m/s. The detention time is between 10-20 min. This flocculator is mostly used for medium and large size treatment plants.

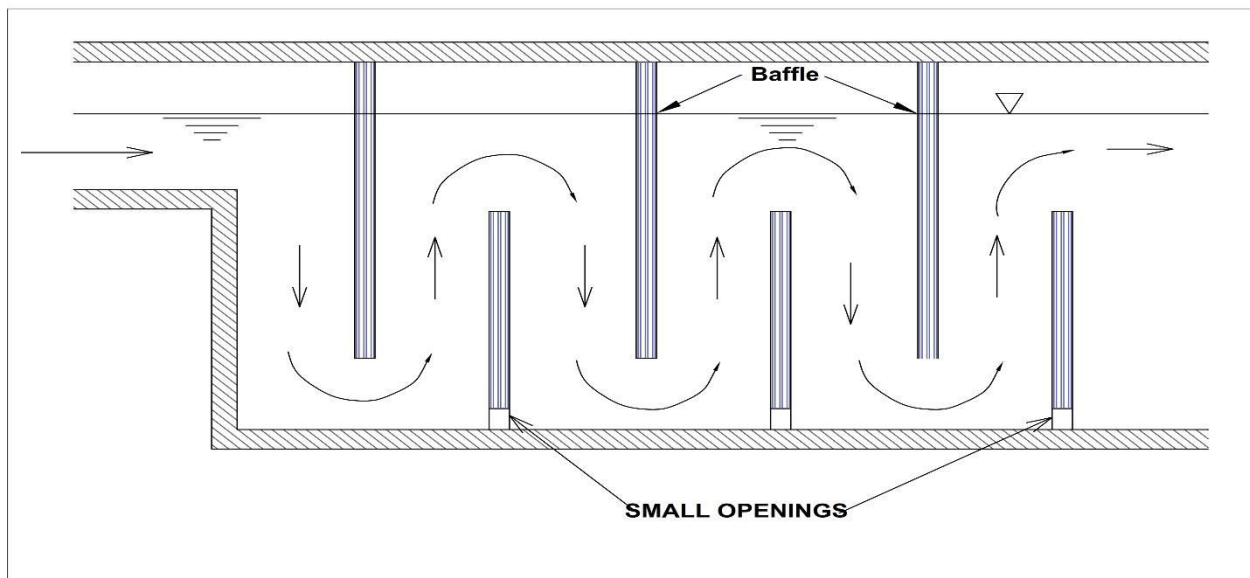


Figure 8.14: Vertical Flow Baffled Flocculator

(c) Jet Flocculator (with modified ports)

This is a less known type of hydraulic flocculator and is suitable for small treatment plants. Entry of coagulated water to the unit is from the top, taking tapping from the launders. A tee is fitted instead of bend to make it vertical. Upper end of tee is open to air. At the bottom of Vertical pipe a small drum is fitted to accommodate four jets at 90° . At the end of each jet pipe a 90° bend is fitted with end reducing suitably.

When flow starts, jets operate as outlets and splash to the walls of container. Flow automatically gets circular direction. The velocity at bottom is very high and as it moves upwards it faces larger area and the velocity is reduced. Hence, initial some portion acts as a flash mixer and thereafter at higher levels it slowly turns into flocculation velocity. This arrangement ensures taper velocity gradient for effective flocculation. Velocity at exit to jets is 2.5 m/s to 3.0 m/s, the detention time is 15 min to 30 min., The height of the tank including that of hopper is 5.0 m. The tapered velocity range is from 150 m/s/m to 10 m/s/m.



Figure 8.15: Floc formation at the top of the tank

(2) Mechanical Paddle Type flocculator

Paddle flocculators are widely used in practice. Figure 8.16 & Figure 8.17 shows two types of mechanical type flocculator with paddles. The design criteria are: depth of tank = 3 to 4.5 m; detention time, $t = 10$ to 40 min. normally 30 min; velocity of flow = 0.2-0.8 m/s normally 0.4 m/s; total area of paddles = 10 to 25% of the cross-sectional area (Length or Width x Depth) of the tank; range of peripheral velocity of blades = 0.2-0.7 m/s; (0.3-0.5 m/s is recommended); range of velocity gradient, $G = 10$ to 75 s^{-1} range of dimensionless factor $Gt = 10^4$ - 10^5 and power consumption; 10.0 to 36.0 kW/MLD, outlet velocity to settling tank where water has to flow through pipe or channel = 0.15 to 0.25 m/s to prevent settling or breaking of flocs. For paddle flocculator, the velocity gradient is given by

$$G = \left[\frac{1}{2} \frac{C_D A_P \rho (V_P - V_W)^3}{\mu (\text{Vol})} \right]^{\frac{1}{2}} \quad (8.15)$$

In which C_D = coefficient of drag (0.8 to 1.9), A_p = area of paddle (m^2), Vol = volume of water in the flocculator (m^3) V_p = velocity of the tip of paddle (m/s), V_W = Velocity of the water adjacent to the tip of paddle (m/s).

For waters with varying turbidity, physico-chemical characteristics, polluted waters, algae laden waters it is essential to provide variable speed drive (VSD) to ensure flexibility in applying desired G value. The speed of the agitator can also be made variable by incorporating a stepped pulley in the mechanism.

The optimum value of G ; can be calculated

$$G_{\text{opt}}^{2.8} t \cdot c = 44 \times 10^5 \quad (8.16)$$

In which G = optimum velocity gradient, s^{-1} , t = time of flocculation, min.; and c = alum concentration (mg/L).

In large plants, it is desirable to provide more than one compartment in series to lessen the effect of short circuiting. While translating laboratory jar test data to plant scale, it

must be borne in mind that the good mixing conditions available in the laboratory cannot be simulated in the plant.

The paddles can be driven by electric motors or by turbines rotated by water fall when sufficient head is available. The direction of flow is usually horizontal moving parallel or at right angles to the paddle shafts. The shape of the container also affects the process of flocculation. For the same volume and height of water in the containers of several shapes such as circular, triangular, square, pentagonal and hexagonal, it was observed that the pentagonal shape gave the best performance.

Introduction of stators in the flocculator helps to improve the performance of flocculation.

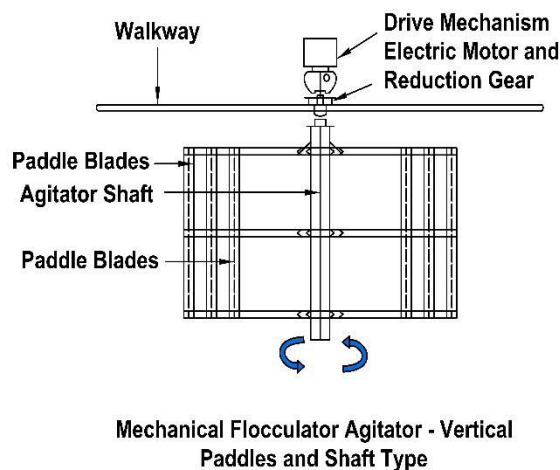


Figure 8.16: Mechanical Type Vertical Flow Flocculator with Paddles

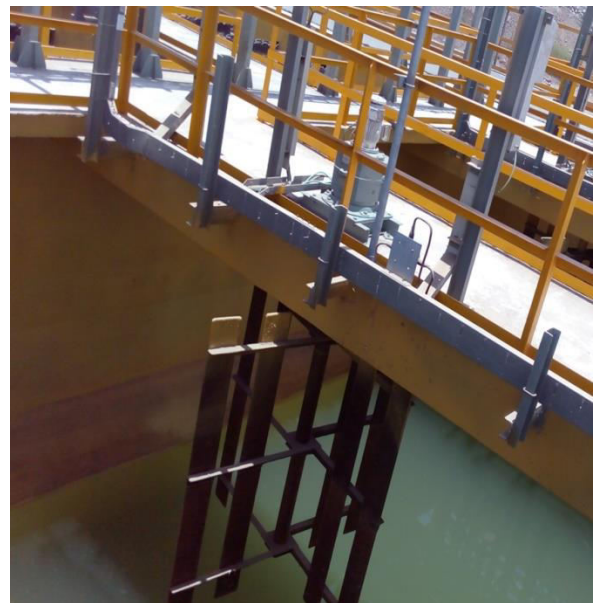


Figure 8.17: Site Photo of Mechanical Type Vertical Flow Flocculator with Paddles

(3) Pebble Bed Flocculator

The pebble bed flocculator contains pebbles of size ranging from 20 mm to 50 mm. Smaller the size of the pebbles, better is the efficiency, but faster is the buildup of the head loss and vice-versa. The depth of the flocculator is between 2.0 m to 3.0 m.

The velocity gradient is given by

$$G = \left[\frac{\rho g Q h_f}{\alpha \mu A} \right]^{\frac{1}{2}} \quad (8.17)$$

Where;

h_f = Head loss across the bed (m);
 α = Porosity of bed;

A = Area of flocculator (m^2); and
 L = Length of the bed (m)

The main advantage of the pebble bed flocculator is that it requires no mechanical devices and electrical power. It is suitable for raw water with low turbidity and very seldomly used. The operation and maintenance cost is also low. The drawback of this flocculator is that there is gradual buildup of the head loss across the pebble bed and therefore needs periodical cleaning by simultaneous draining and hosing.

(4) Surface Contact Flocculator

The surface contact flocculator was studied experimentally in India to overcome the inherent problem of choking, which increases the head loss over a period of time in pebble bed flocculators. However, this type of flocculator is seldom used.

The surface contact flocculators consist of studded plates, placed in a zigzag form along the direction of flow. An experimental flocculator, shown in Figure 8.18, comprised of 55 mild steel plates, 140 mm x 60 mm in size, arranged in 11 rows of 5 plates each. These plates were fixed at 45° to a base plate in zigzag fashion. The flocculator was tested in a continuous down flow system with velocity of flow ranging from 5 m/h to 25 m/h and turbidity ranging from 50 to 1600 NTU.

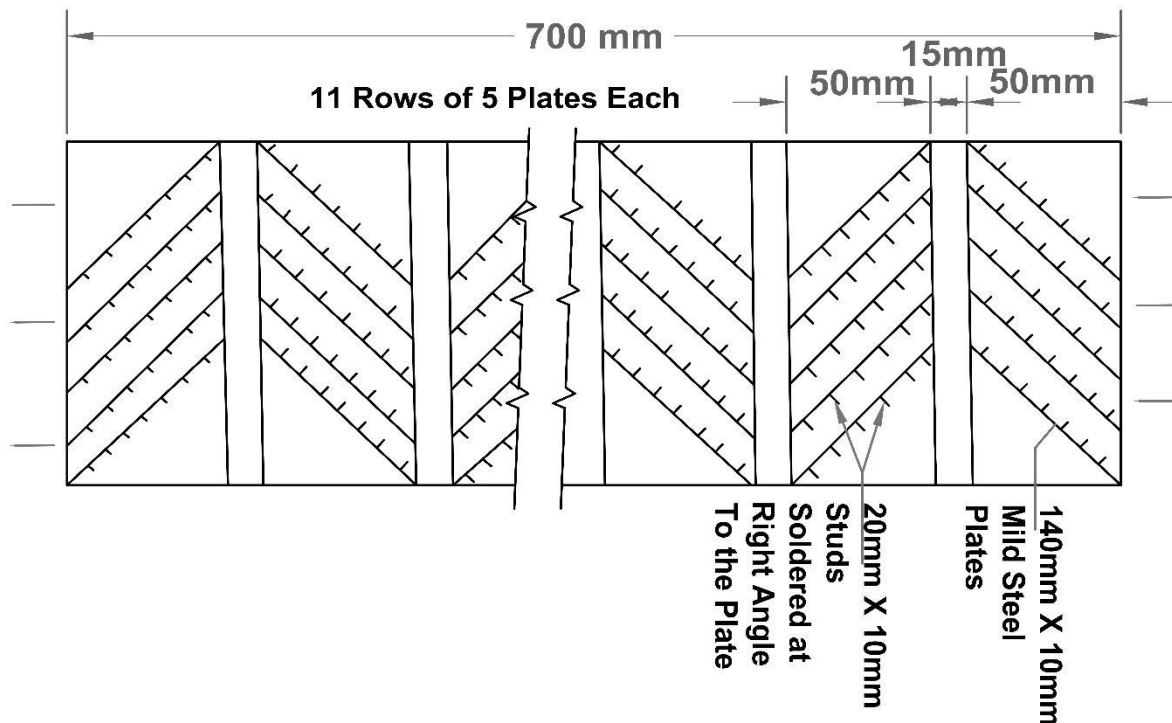


Figure 8.18: Surface Contact Flocculator

(5) Pulsating Sludge Blanket Clarifier

In these type of clarifier, flocculation and clarification zones are superimposed. In flat bottom tanks the flocculation takes place in bottom part. In already formed sludge

blanket, coagulated water is injected in high flow rate for a very short period followed by the low flow rate for a long period. As the “pulsed” water passes uniformly through the sludge blanket the blanket goes through the alternative cycles of expansion and contraction, thus dense floc is formed which has high settling velocity. The clarified water is collected at the top. The excessive sludge generated is drained off periodically.

The advantages are that it is simple in construction and its ability to adapt to all shapes of tanks. As flocculation and clarification zones are superimposed it produces far more compact plants. It has operational flexibility. It is reliable especially due to the fact that it involves no mechanical appliances. Placing of plates in the clarification zone, further increases its efficiency. These types of clarifiers work well when raw water turbidity is low or moderate.

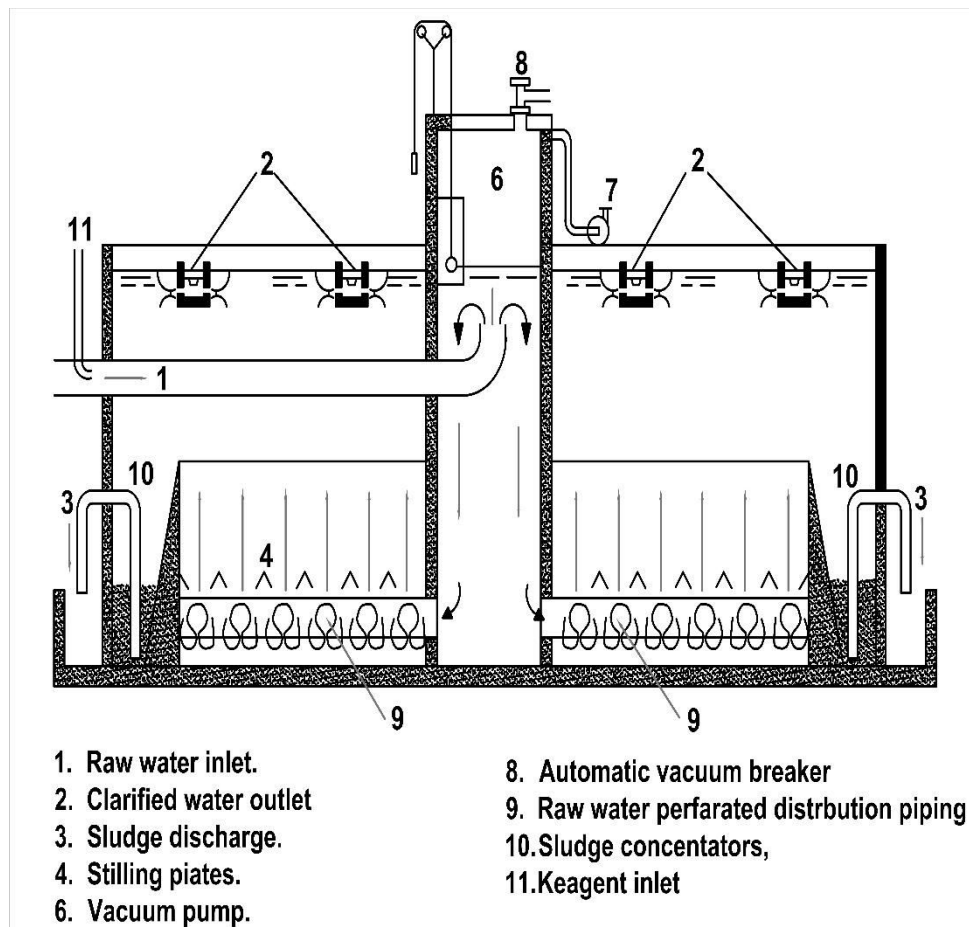


Figure 8.19: Pulsator Clarifier

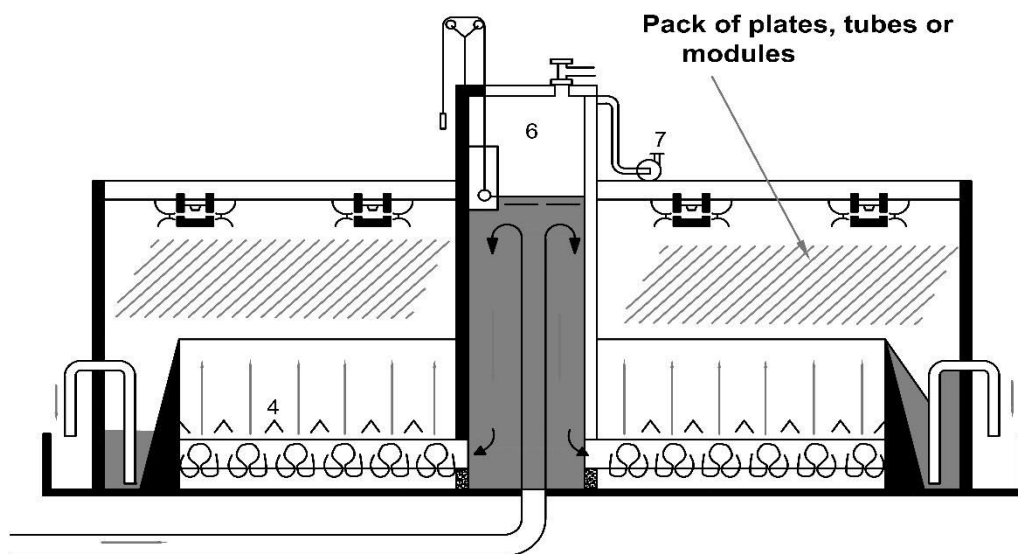


Figure 8.20: Superpulsator Clarifier

8.6 Sedimentation (Clarification)

Sedimentation is the separation of suspended particles from water by gravitational settling that are heavier than water. It is one of the most commonly used unit operation in the flow sheet of conventional water treatment. Sedimentation (settling or clarification) is used to remove readily settling sediments such as sand and silt, coagulated impurities such as colour and turbidity and precipitated impurities such as hardness and iron. When suspended solids are separated from the water by the action of natural forces alone i.e. by gravitation with or without natural aggregation, the operation is called plain sedimentation. Plain sedimentation is usually employed as a preliminary process to reduce heavy sediment loads from highly turbid raw waters prior to subsequent treatment processes such as coagulation/filtration. Finely divided solids and colloidal particles, which cannot be removed by plain sedimentation within commonly used detention periods of few hours, are converted into settleable flocs by coagulation and flocculation and subsequently settled in sedimentation tanks.

The factors that influence sedimentation are:

- a. Size, shape, density and nature (discrete or flocculent) of the particles
- b. Viscosity, density and temperature of water
- c. Surface over flow rate;
- d. Velocity of flow;
- e. Inlet and outlet arrangements;
- f. Detention period; and
- g. Effective depth of settling zone.

8.6.1 Types of Suspended Solids

In water treatment practice, three main types of suspended particles are to be separated from water. The first type of suspended particles are finely divided silt, silica and clay having specific gravities ranging from 2.65 for sand and 1.03 for flocculated mud particles containing 95 % water. The grain size may be 0.012 mm or more. Alum and iron flocs constitute the second type of suspended solids. These absorb and entrain water and specific gravities for alum and iron flocs may range from 1.18 and 1.34 respectively to as little as 1.002. Floc particles range in size from submicroscopic to 1 mm or more. The third type is the precipitated crystals of calcium carbonate obtained from lime-soda softening operations. Their specific gravity is 2.7 with particle size of 15 μm to 20 μm . However, due to absorption of water upto 75 %, the specific gravity reduce to 1.2 and formation of cluster of crystals increases the size to a typical value of 0.1 mm.

Suspended particles may settle either as discrete or flocculent particles. Discrete particles do not change their size, shape or weight during settling. The settling velocity of discrete particles can be computed by well-defined mathematical relationships as described in section 8.8.2. On the contrary; flocculent particles tend to agglomerate forming clusters of

different size, shape and weight. Though the density of these floc clusters decreases due to entrapment and absorption of water, they settle faster due to increased size.

8.6.2 Settling Velocity of Discrete Particles

The settling velocity (also referred to as the “sedimentation velocity”) is defined as the terminal velocity of a particle in still fluid. The following equations may be used in arriving at settling velocity of discrete spherical particles;

Table 8.3: Equations for settling velocity of discrete spherical particles

Law	Equation	Applicable for range of	
		Reynolds Number. N_R	Particle size in mm for specific gravity of 2.65 and temp. of 20° C
Stock's (Laminar)	$V_s = \frac{g}{1g} \left(\frac{\rho_s - \rho}{\mu} \right) d^2$	1	Up to 0.1
Hazen's (Transition)	$V_s = \left[\frac{4gd}{3C_D} \left(\frac{\rho_s - \rho}{\rho} \right) \right]^{0.5}$	1-1000	0.1-1.0
Newton's (Turbulent)	$V_s = \left[3.3g \left(\frac{\rho_s - \rho}{\rho} \right) d \right]^{0.5}$	10^3 - 10^4	Greater than 1

Where;

V_s = Settling velocity of particle, (L/T)

ρ_s = Mass density of the particle, (M/L³)

ρ = Mass density of water, (M/L³)

g = Acceleration due to gravity, (L/T²)

d = Diameter of the particle, (L)

C_D = Dimensionless drag coefficient defined by

$$C_D = \frac{24}{N_R} + \frac{3}{\sqrt{N_R}} + 0.34 \quad (8.18)$$

N_R = Reynolds number = $\frac{V_s d \rho}{\mu}$, Dimensionless

μ = Absolute or dynamic viscosity of water $\left(\frac{M}{LT} \right)$

Table 8.4: Settling Velocities of particles in Water

Diameter of particle, mm	Settling Velocity, m/hr.				
	Sand, Sp. gr. 2.65		Alum floc, Sp. gr. 1.05		Calcium softening, Sp. gr. 1.2
	Temp 10°C	Temp 20°C	Temp 10°C	Temp 20°C	
1	400	450	2.50	3.60	5.0-8.5
0.1	30	38			
0.05	5.5	8.2			
0.01	0.36	0.50			

For actual designing, multiplying factor of safety of 0.5 to 0.75 should be adopted

8.6.3 Removal Efficiencies of Discrete and Flocculent Suspensions

The removal efficiency of a uni-size discrete suspension settling in an ideal settling tank is given by the ratio of settling velocity of the particles, v_s , and the surface overflow rate (SOR) which is numerically equal to flow divided by the plan area of the basin. For an ideal sedimentation tank, SOR represents the velocity of settling of those particles which covers the depth of the basin in time equal to the theoretical detention period or the settling velocity of the slowest settling particles which are 100 % removed.

When water contains discrete particles of different sizes and densities, the overall removal, R , of suspended particles is given by

$$R = (1 - P_0) + \frac{1}{V_0} \int_0^{P_0} V_s dp \quad (8.19)$$

P_0 is portion of particles with a settling velocity $\leq V_0$, the surface overflow rate.

Flocculent particles coalesce during settling increasing the mass of particles which settle faster. The degree of flocculation depends on the contact opportunities which in turn are affected by the surface overflow rate, the depth of the basin, the concentration of the particles, the range of particle sizes and the velocity gradients in the system. To determine the removal efficiency of a flocculent suspension, no adequate mathematical equation exists and settling column analyses are to be performed.

Settling analyses of flocculent suspensions are performed in column at least 300 mm in diameter and having depth equal to that of proposed basin. The column usually has ports at 0.6 m interval for withdrawal of samples. The flocculent suspension for which the settling characteristics are to be determined is introduced into the column in such a way that a uniform distribution of particle size occurs from top to bottom. The settling is allowed to occur under quiescent conditions and at constant temperature to eliminate convection currents. Samples are withdrawn at various selected time intervals from different depths and analyzed to determine the suspended solids concentrations. The percentage removals of suspended solids are computed at different times and depths and the percentage removal is plotted as a number against time and depth. The iso-percent removal curves are drawn in a similar manner as contours are drawn from spot levels.

A generalized plot is given in Figure 8.21. The percentage removal for a given time, t , can be computed from the relationship:

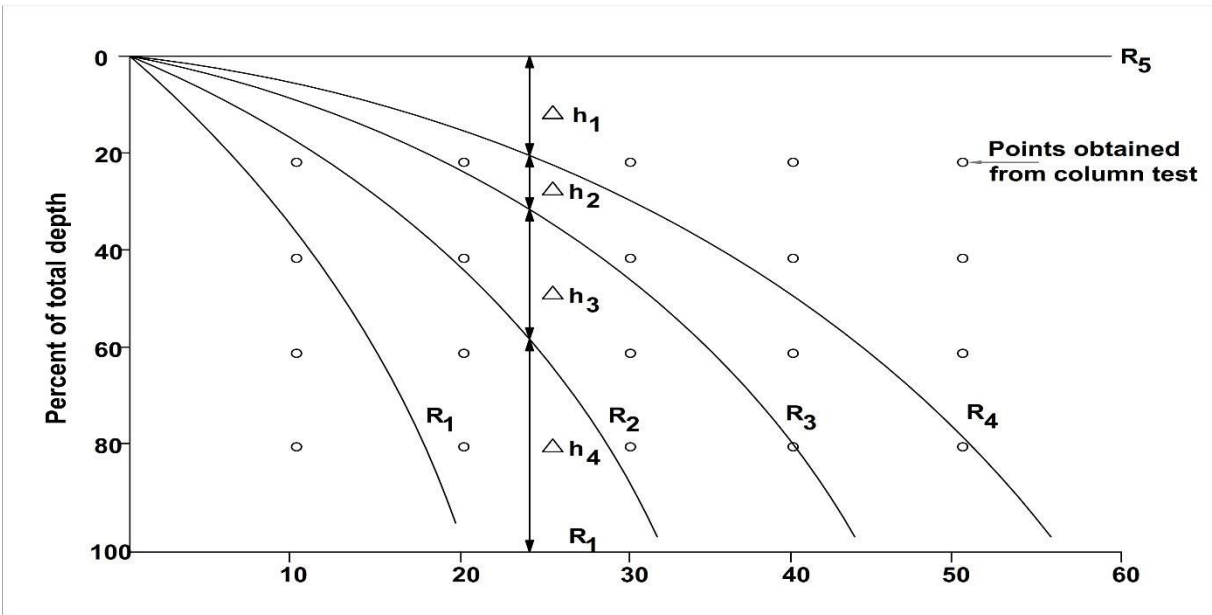


Figure 8.21: Settling of Flocculant Suspension

$$\text{Percentage removal} = \frac{(R_1 + R_2)}{2} \cdot \frac{\Delta h_4}{h} + \frac{(R_2 + R_3)}{2} \cdot \frac{\Delta h_3}{h} + \frac{(R_3 + R_4)}{2} \cdot \frac{\Delta h_2}{h} + \frac{(R_4 + R_5)}{2} \cdot \frac{\Delta h_1}{h} \quad (8.20)$$

Where, R_1 , R_2 , R_3 , R_4 and R_5 are percent removals and R_1 is the percent removal at time t and at 100% depth.

The curves can also be used to determine the detention period, depth and surface overflow rate required to obtain a given percentage removal of flocculent particles.

8.6.4 Types of Tanks

The tanks may be categorized into horizontal flow tanks or vertical flow tanks on the basis of direction of flow of water in the tank. The tanks may be rectangular, square or circular in plan.

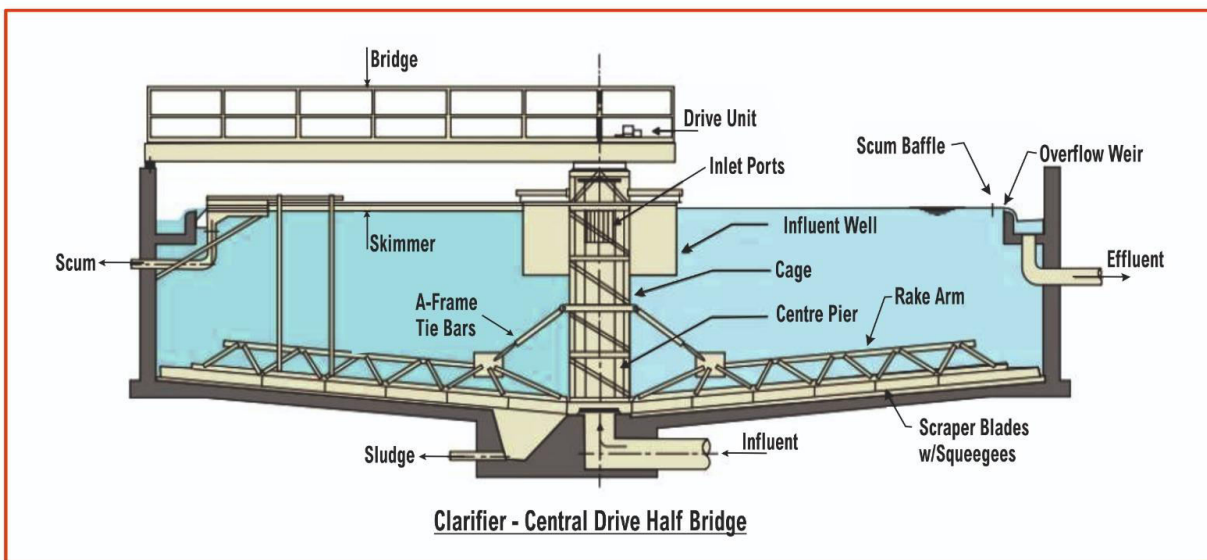
8.6.4.1 Horizontal Flow Tanks

In the design of a horizontal flow tank, the aim is to achieve as nearly as possible the ideal conditions of equal velocity at all points lying on each vertical line in the settling zone. The direction of flow in the tanks is substantially horizontal. Among the representative designs of the horizontal flow settling tanks, the following may be mentioned:

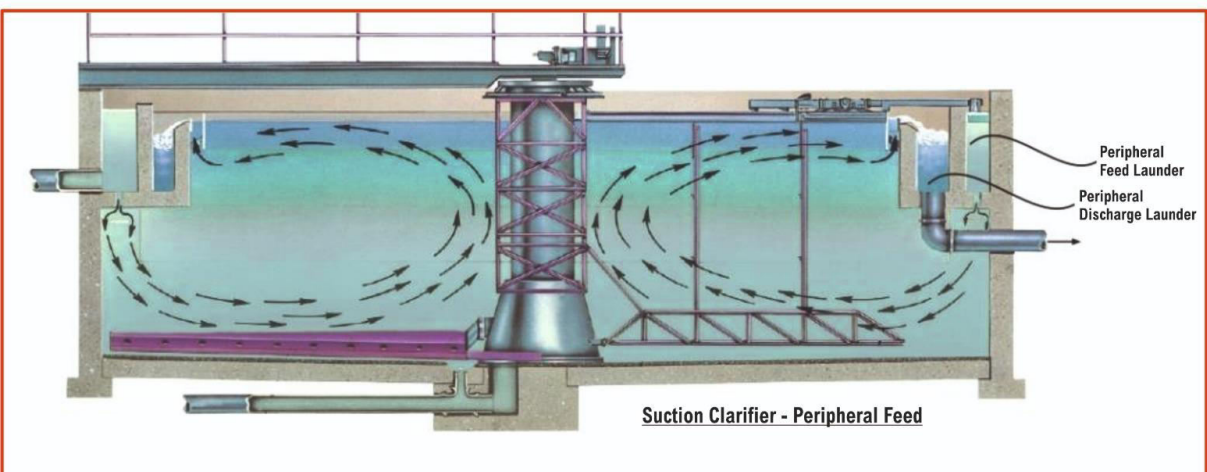
- Radial flow circular tank with central feed:** The water enters at the center of the tank and emanates from multiple ports of circular well in the center of tank to flow radially outwards in all directions equally. The aim is to achieve uniform radial flow with decreasing horizontal velocity as the water flows towards the periphery and is withdrawn from the tank through effluent structure. The sludge is plowed to central sump mechanically and continuously and is withdrawn during operation. The sludge

removal mechanism consists of scraper blades mounted on two or four arms revolving slowly.

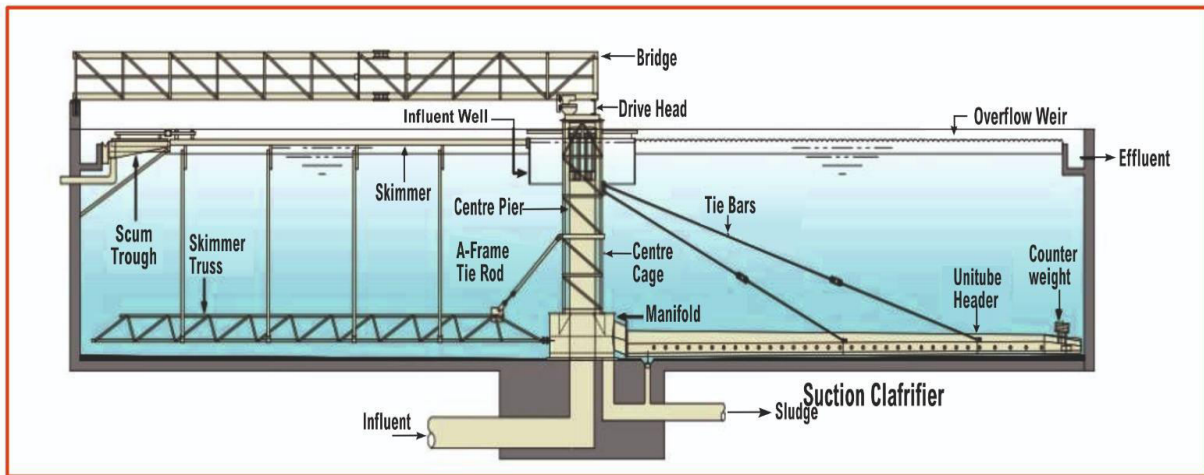
- b. **Radial flow circular tanks with peripheral feed:** These tanks differ from the central feed circular tanks in that the water enters the tank from the periphery or the rim. It has been demonstrated that the average detention time is greater in peripheral feed basins leading to better performance.
- c. Rectangular tanks with longitudinal flow where the tanks are cut out of operation for cleaning. The solids are flushed to sump for removal from the dewatered tank.
- d. Rectangular tanks with longitudinal flow where sludge is mechanically scraped to the sludge pit located usually towards the influent end and removed continuously or periodically without disrupting the operation of the tanks.



(a)



(b)



(c)

Figure 8.22: (a) Clarifier with Centre feed and peripheral drive, (b) Peripheral Feed Circular Clarifier with Effluent and Influent Channels separated by a skirt & (c) Peripherally driven suction clarifier

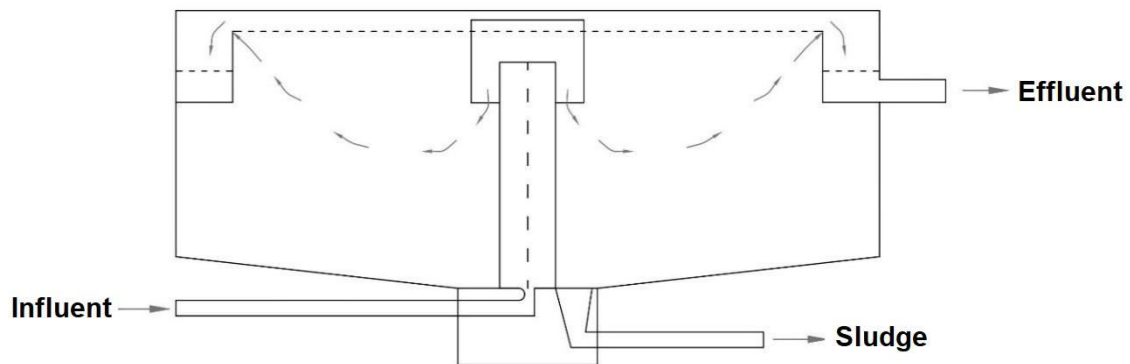


Figure 8.23: Circular Clarifier with Centre Feed

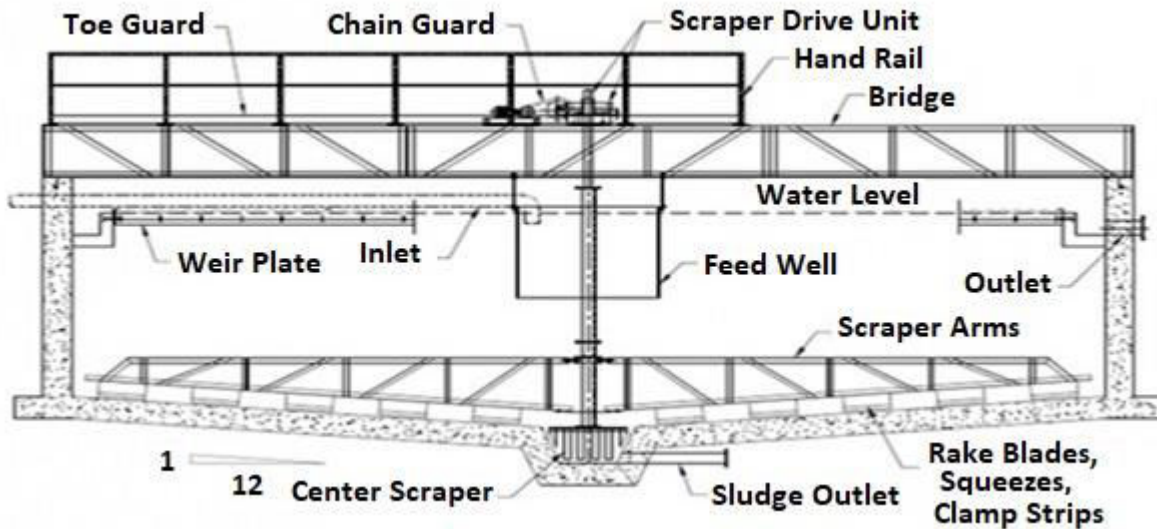


Figure 8.24: Centrally driven center feed clarifier

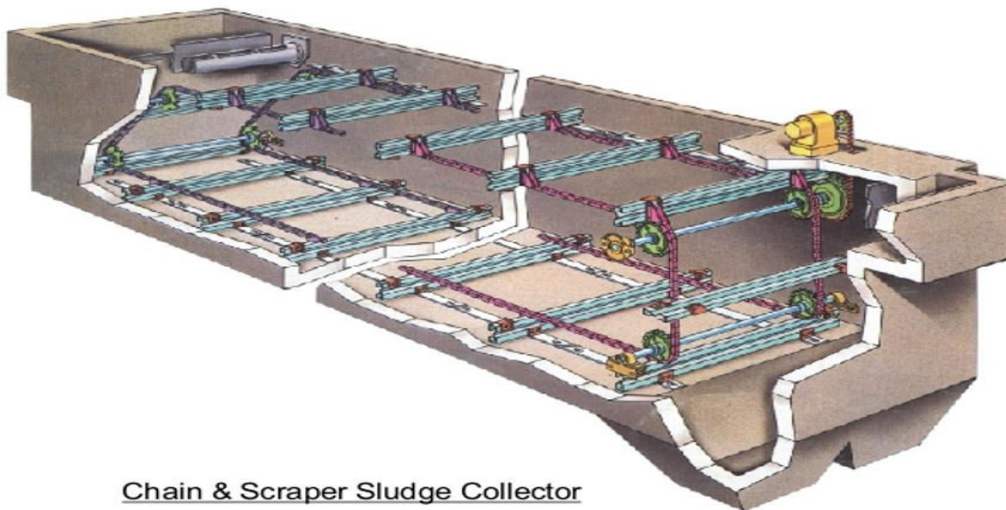


Figure 8.25: Chain and flight scraper type rectangular clarifier

The BIS code IS 10313 (1982, Reaffirmed 2010): Requirements for Settling Tank (Clarifier Equipment) for Water Treatment Plant, which covers requirements for settling tank (clarifier equipment) for water treatment plant may be also referred to.

8.6.4.2 Vertical Flow Tanks

Vertical flow tanks normally combine sedimentation with flocculation. These tanks are square or circular in plan and may have hopper bottoms. The influent enters at the bottom of the unit where flocculation takes place as particles co-join into aggregates. The up-flow velocity decreases with increased cross-sectional area of the tank. There is a formation of blanket of floc through which the rising floc must pass. Because of this phenomenon, these

tanks are also called as up-flow sludge blanket clarifier. The clarified water is withdrawn through circumferential or central weir.

These tanks have no moving parts and except for a few valves, require no mechanical equipment. They are compact units requiring less land area.

8.6.5 Clariflocculator

Clariflocculators are widely used in India in water and wastewater treatment. The flocculation and sedimentation processes are effectively incorporated in a single unit in the clariflocculator. The unit has concentric circular flocculation zone at the center and annular or peripheral clarification zone.

All these units consist of 2 or 4 flocculating paddles (slow agitators) placed equidistantly. These paddles rotate on their vertical axis. Their drive mechanisms are located on the walkway platform of rotating bridge. For plants less than 15 mld capacity two agitators are sufficient, for higher capacities four are provided. The flocculating paddles may be of rotor-stator type rotating in opposite direction around this vertical axis. The clarification unit outside the flocculation compartment is served by inwardly raking rotating blades. The water mixed with coagulants is fed in the flocculation compartment fitted with paddles rotating at slow speeds.

The coagulated water enters into flocculation zone through the central shaft at the top. The velocity in the central shaft and that through outlet ports is restricted to 0.60 m/s. The flocculator wall is supported on the equidistance columns. The flocculated water passes out from the bottom of the flocculation tank to the clarifying zone through the wide openings in between the supporting columns. the area of the opening being large enough to maintain a very low velocity (not more than 0.3 m per minute). Under quiescent conditions in the annular settling zone the floc embedding the suspended particles settle to the bottom and the clear effluent overflows into the peripheral launder. The sludge which settles down to the bottom is continuously swept towards the central sludge pocket by the scraper arms of rotating bridge. The slope of the tank bottom is in the range of 1:12 to 1:10 towards the center. The option of providing collection launder inside or outside may be taken. If constructed inside the projected area below launder should not be taken in to design (capacity) consideration. If located outside, walkway should be provided with RCC slab having manhole openings for maintenance. The collection weir of the launder is provided in concrete (Ogee) with a sharp collection edge (like rectangular weir). Sometimes V notch weir plate is also provided as a collection weir. Generally, the maximum overall tank diameter is limited to 40 m – 50 m. The rotating bridge rests at one end on the bearings provided over the central shaft. At the other end the end carriage drive moves on the outer wall. Normally the wheel moves on the rail fixed on the wall. Sometimes rubber tyres are provided for the wheel. The tangential speed of scraper rake tip is 2.5 m/min to 3 m/min. The side water depth of the clariflocculator is in the range of 3.5 m to 4.0 m. As was the practice of the past, it is not necessary to make provision for extra depth for storage of sludge in the clarification zone. For capacity calculations the

volume of the conical bottom need not be taken in to consideration. For tank diameter up to 20 m, sludge drain pipe diameter needs to be minimum 200 mm, for larger tanks it needs to be minimum 300 mm.

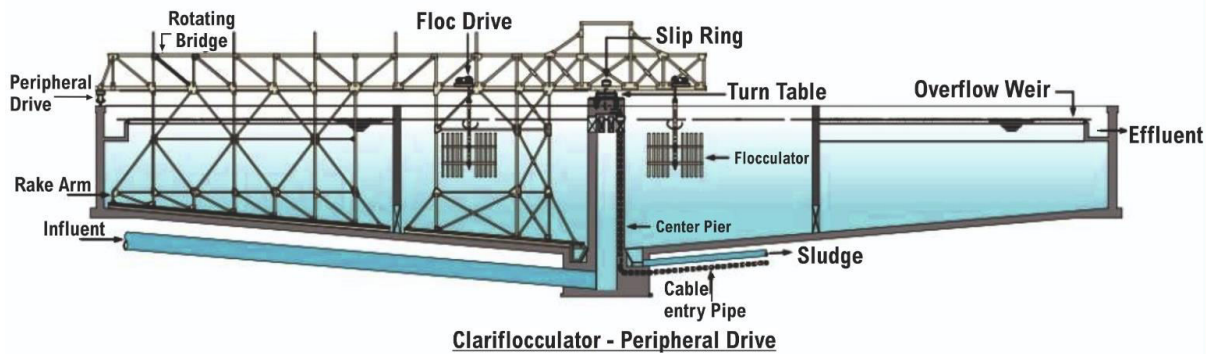


Figure 8.26: Cross section of Clariflocculator



Figure 8.27: Flocculation zone with slow agitators & Clarification zone with scraper rotating bridge

8.6.6 Tank Dimensions

The settling basins may be long and narrow rectangular tanks, square or almost square tanks and circular tanks. The rectangular tanks have lengths commonly up to 30 m but larger lengths up to 100 m have also been adopted. The length to width ratio of rectangular tanks should preferably be from about 3: 1 to 5: 1. The narrower the tank, the less chance there is for setting up of cross currents and eddies due to wind action, temperature changes and other factors involved. In very large size tanks where the depth is necessarily great, it may be advisable to provide longitudinal baffles to confine the flow to definite straight channels. These walls could be of thin sections since the pressure on both sides will be the same.

The diameter of the circular tank is governed by the structural requirement of the trusses that carry the scraping mechanism. Circular tanks up to 60 m in diameter are in use but are generally up to 30 m to reduce wind effects. Square tanks are generally smaller usually

with sides up to 20 m. Square tanks with hopper bottoms having vertical flow have sides generally less than 10 m to avoid large depths.

The depth of the settling basin depends on the character of sludge handled, storage capacity required and cost. In warm climates and where the sludge is likely to contain considerable organic matter, it is not advisable to store sludge for long periods; otherwise, the decomposition of the sludge adversely affects the settling process. Depths commonly used in practice vary from 2.5 m to 5 m with 3.0 m being a preferred value. Bottom slopes are kept 1 % or less in rectangular tanks and 1 in 12 or steeper for circular tanks. The slopes of sludge hoppers range from 1.2:1 to 2:1 (vertical: horizontal).

8.6.7 Common Surface Loadings and Detention Periods

The removal of particles of varying hydraulic subsidence values is solely a function of surface overflow rate also called "surface loading" and is independent of the depth of the basin for discrete particles and unhindered settling. However, contact opportunities among particles leading to aggregation increase with increasing depths for flocculent particles having tendency to agglomerate while settling, such as alum and iron flocs. The range of surface loadings and detention periods for average design flow for different types of sedimentation tanks are as follows:

Table 8.5: The range of surface loadings and detention periods for average design flow for different types of sedimentation tanks

Tank type	Surface loading $\text{m}^3/\text{m}^2/\text{d}^*$		Detention period, hr.*		Particles normally removed
	Range	Typical value for design	Range	Typical value for design	
Plain Sedimentation	Up to 60	15-30	1-15	3-4	Sand, silt and clay
Horizontal flow, Circular/ Clariflocculators	25-75	30-40	2-8	2-2.5	Alum and iron floc
Vertical flow (Up-flow) Clarifiers	-	40-50	-	1-1.5	Flocculent

*at average design flow

8.6.8 Inlets and Outlets

Inlet structures must:

- (i) Uniformly distribute flow and suspended particles over the cross section at right angle to flow within individual tanks and into various tanks in parallel
- (ii) Minimize large-scale turbulence and
- (iii) Initiate longitudinal or radial flow, if high removal efficiency is to be achieved.

Water leaving the flocculator units should flow into the sedimentation basin through slots or effluent ports.

For uniform distribution of flow, the flow being divided must encounter equal head loss or the head loss between inlets on inlet openings must be small in comparison to the head available at the inlets. If h_1 and q_1 are the head and discharge at the first inlet from the point of supply in a settling tank and h_n , and q_n being the head and discharge at the n^{th} inlet opening, farthest from the point of supply, the following relationship holds

$$h_n = kq_n^2 = k(mq_1)^2 = m^2h_1 \quad (8.21)$$

If the discharge in n^{th} inlet is held to mq_1 where $m < 1$, the head at the first inlet can also be expressed in terms of head lost between the first and n^{th} inlet, h_n

$$h_n = h_1 - h_f = m^2h_1, \text{ and}$$

$$h_f = (1-m^2)h_1$$

For $m = 0.99$, $h_f = 0.02 h_1$, and $h_n = 0.98 h_1$

Freely discharging weirs, anywhere between Flocculator and settling tank, have a tendency to break fragile floc, hence, free-fall is not recommended. Where water carrying floc has to pass through channels or pipes before reaching sedimentation basins, the velocity in such channels or pipes should be held between 0.15 to 0.25 m/s to prevent settling or breaking up of the floc.

Inlet or influent structures may have different arrangements as shown in Figure 8.28. Each inlet opening must face a baffle so that most of the kinetic energy of incoming water will be destroyed and a more uniform lateral and vertical distribution of flow can occur. One of the satisfactory method of attaining uniform velocity of flow is to pass the water through a training or dispersion wall perforated by holes or slots. The velocity of flow through such slots should be about 0.2 to 0.3 m/s and head loss is estimated as 1.7 times the velocity head. The diameter of the hole should not be larger than the thickness of the diffuser wall.

Outlet or effluent structure comprises of weir, notches or orifices; effluent trough or launder and outlet pipe. V-notches attached to one or both sides of single and multiple troughs are normally preferred as they provide uniform distribution at low flows. To ensure uniform collection all along the periphery mild steel weir plate with 90° V-notches along with suitable weir clamps shall be provided. The V-notches are generally placed 150-300 mm center to center. A baffle is provided in front of the weir to stop the floating matter from escaping into effluent.

Effluent troughs act as lateral spillway and can be designed on similar lines to those of wash water troughs in rapid gravity filters. The widely used equation for the design of effluent trough is

$$Y_1 = \sqrt{Y_2^2 + \frac{2(qLN)^2}{gb^2Y_2}} \quad (8.22)$$

Part A: Engineering Design

Which was originally developed for flumes with level inverts and parallel sides; channel friction is neglected and the drawdown curve is assumed parabolic.

Y_1 = water depth at upstream end of launder, m

Y_2 = water depth in trough at a distance 'l', from upstream end, m

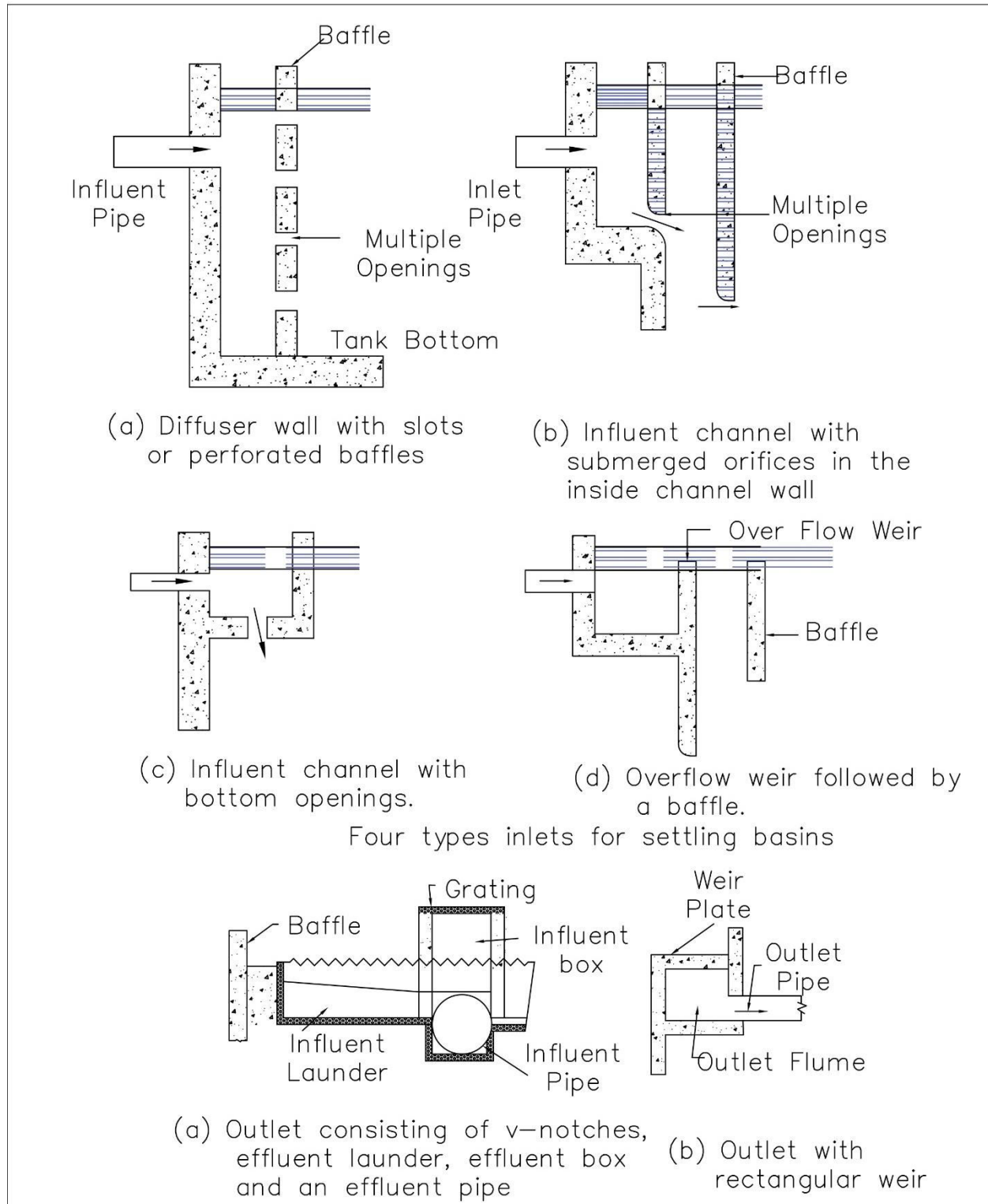
q = discharge per unit length of the weir, m^3/m^2

b = width of launder, m

N = number of sides the weir receives the flow (one or two)

In the absence of any control device, it is reasonable and customary to assume critical flow at the lower end of the channel and hence Y_2 at lower end of channel of length L. is,

$$Y_2 = \left[\frac{(qL)^2}{b^2g} \right]^{\frac{1}{3}} \quad (8.23)$$

**Figure 8.28: Typical Outlet for Settling Tanks**

There is a growing trend towards the use of effluent launders or troughs covering a good part of the surface of the settling basins. These are spaced at a distance of one tank depth between the troughs. The use of maximum feasible weir length in the tank from the outlet

towards the inlets assists greatly in controlling density currents. Weirs, however, suffer from the difficulty in levelling which is not faced with perforated pipe launder. Perforated launders, with ports commonly submerged 30 to 600 mm below the surface are useful in varying the water level in the basin during operation and prevent floating matter passing to the filters.

8.6.9 Weir Loading

Weir length relative to surface area determines the strength of the outlet current. Normal weir loadings are up to $300 \text{ m}^3/\text{d}/\text{m}$. But when settling tanks are properly designed, well clarified waters can be obtained at weir loadings of even up to $1500 \text{ m}^3/\text{d}/\text{m}$.



Figure 8.29: Finger weirs for Large Diameter Clarifiers/Clariflocculators in case they exceed weir loading

8.6.10 Sludge Removal

Sludge is normally removed under hydrostatic pressure through pipes. The size of the pipe will depend upon the flow and the quantity of suspended matter. It is advisable to provide telescopic sludge discharge arrangement for easy operation and for minimizing the wastage of water. For non-mechanized units, pipe diameters of 200 mm or more are recommended. Pipe diameters of 100 to 200 mm are preferred for mechanized units with continuous removal of sludge with hydrostatic head. In circular tanks, where mechanical scrapers are provided, the floor slopes should not be flatter than 1 in 12, to ensure continuous and proper collection of sludge. For manual cleaning, the slope should be about 1 in 10.

The power required for driving the scraping mechanism in a circular tank depends upon the area to be scraped and the design of the scraper. The scraping mechanism is rotated slowly to complete one revolution in about 30 min to 40 min or preferably the tip velocity of

the scraper should be around 0.3 m/min or below. Power requirements are about 0.75 W/m² of tank area.

Sludge and wash water should be properly disposed of without causing any problems of pollution if discharged into water courses.

For sludge blanket type vertical flow settling tanks, the slope of the hoppers should not be less than 55° to horizontal to ensure smooth sliding and removal of sludge. In such tanks special slurry weirs are provided with their crests in level with the top of sludge blanket for continuous bleeding of the excess sludge.

Special types of consolidation tanks with a capacity of 30 min are sometimes provided to consolidate the sludge and recover water from it.

In non-mechanized horizontal flow rectangular settling tanks, the basin floors should slope about 10% from the sides towards the longitudinal central line adopting a longitudinal slope of at least 5% from the shallow outlet end towards the deeper inlet area where the drain is normally located. Manual cleaning of basins is normally done hydraulically, using high pressure hoses. Admitting settled water through the basin outlet helps this function. If sludge is to be withdrawn continuously or nearly continuously from the bottom of the basin by gravity without mechanical equipment, hopper bottoms have to be used with slope of not less than 55° to the horizontal.

Reclamation of water from the sludge removed from the settling basin should be encouraged. The various methods include disposal of sludge on land or on sludge drying beds.

8.6.11 Settling Tank Efficiency

The efficiency of basins is reduced by currents induced by inertia of the incoming water, wind, turbulent flow, density and temperature gradients. Such currents short circuit the flow. The efficiency of real basin affected by current induced short circuiting may be mathematically expressed as

$$\frac{Y}{Y_0} = 1 - \left[1 + n \frac{V_0}{Q/A} \right]^{-\frac{1}{n}} \quad (8.24)$$

Where;

$\frac{Y}{Y_0}$ = Efficiency of removal of suspended particles.

n = Coefficient that identifies basin performance

V_0 = Surface over flow rate for ideal settling basin

Q/A = Required surface overflow rate for real basin to achieve an efficiency of $\frac{Y}{Y_0}$ for

given basin performance.

The values of n are assumed 0 for best possible performance, $1/8$ for very good performance, $1/4$ for good performance, $1/2$ for average performance and 1 for very poor performance. Mathematical analysis of longitudinal mixing in settling tanks indicates that the value of n can be approximated by the ratio of the differences between the mean and modal flow-through period to the mean flow-through period.

The short-circuiting characteristics of tanks are usually measured by addition of a slug of dye, electrolyte or tracer and observing the emergence of this tracer substance with passage of time. A frequency distribution plot of the concentration with respect to time is plotted. Modal, median and mean flow-through periods identify the central tendency of the time-concentration distribution and percentiles reflect its variance. The ratio of the median time to the mean time or the ratio of the difference between the mean and the modal (or mean and median) to the mean indicate the stability or efficiency of the basin. The lower the first value is from unity or the higher the second value, the lesser the efficiency and the more the short-circuiting. A well-designed tank should be capable of having a volumetric efficiency of at least 70%.

To achieve better clarification, the flow regime in settling basin should be as close as possible to ideal plug flow. A narrow and long rectangular tank approximates plug flow conditions better than wide shallow rectangular tank, peripheral feed circular tank and center feed radial flow tank.

Settling tanks should be capable of giving settled water having turbidity not exceeding 20 NTU and preferably less than 10 NTU.

8.6.12 Tube Settlers

In the year 1905, Hazen established the theory of shallow depth sedimentation, that the efficiency of a settling basin (particle removal) is solely a function of settling velocity of the particles, surface area of the basin and the rate of flow. It is independent of the detention time (depth) of the basin. It resulted in the fact that doubling the surface area by inserting one horizontal tray would roughly double the capacity of the basin.

Attempts were made to use this concept to achieve better efficiency and economy in space as well as cost. Wide shallow trays inserted within conventional basins with a view to increase the surface area have not met with success due to operational limitations. However, small diameter tubes having a large wetted perimeter relative to wetted area providing laminar flow conditions did show good promise. The cluster of tubes or plates in parallel are generally termed as Lamella.

Such tube settling devices provide excellent clarification with detention times of equal to or less than 10 min. Tube configurations can be horizontal or steeply inclined. In inclined tubes (about 60°) based on the “counter current” flow principal, continuous gravity drainage of the settled sludge can be achieved.

With horizontal tubes (normally inclined at 5°) auxiliary scouring of settled solids is necessary. However, their application has remained limited to only small proprietary plants.

While tube settlers have been used for improving the performance of existing basins, they have also been successfully used in a number of installations as a sole or independent settling unit. It has been found that if one-fifth of the outlet end of a basin is covered with tube or plate settlers, the effective surface loading on the tank is nearly halved or the flow through the basin can be nearly doubled without impairment of effluent quality.

The tubes may be square, hexagonal, diamond shaped, triangular, and rectangular or chevron shaped. The material of tubes is normally rigid PVC (other forms of plastics are also been used). Tubes can be extruded over the machine or alternatively fabricated out of thin plastic corrugated sheets (1.0 mm). Normally black color is preferred to prevent the growth of algae. In India commonly tubes with 50mm x 50 mm (ID) section are practiced. The raw material of tubes is virgin Rigid PVC granules having specific gravity more than that of water.

8.6.12.1 Analysis of Tube Settlers

The performance of the tube settlers is normally evaluated by a parameter, S , defined as

$$S = \frac{V_s}{V_0} (\sin \theta + L \cos \theta) \quad (8.25)$$

Where;

V_s = Settling velocity of the particle in a vertically downward direction (L/T)

V_0 = Velocity of flow along the tube settler

θ = Angle of inclination of tube settlers, degrees.

L = Relative length of settler = l/d , dimensionless

l, d = Length and diameter (width) of the tube settlers, (L)

If the value S equals or exceeds a critical value, S_c for any particle, it is completely removed in the tube settlers under ideal conditions. For laminar flow regime in tube settlers, the value of S_c has been determined as 4/3, 11/8 and 1 for circular, square and parallel plates type of tube settlers assuming uniform flow.

It is found that the performance of tube settlers is improved significantly with L values of up to 20 and insignificantly beyond 20. Therefore, it is desirable to design tube settlers with L value 20 or less. Increasing the angle of inclination of tube settlers beyond 40°, results in deterioration in their performance. Essentially horizontal tube settlers perform better than steeply inclined tube settlers. It is opined that from relative economics point of view, the order of preference for tube settlers is parallel plates followed by circular tubes and square conduits.

Part A: Engineering Design

It is recommended to increase the dimensionless length L of tube settlers by an additional amount L' to account for transition zone near inlet to change to fully developed laminar flow.

$$L' = 0.058 \frac{V_0 d}{\nu} \quad (8.26)$$

Where ν is the kinematic viscosity of water.

Following Table 8.6 shows the surface loading rates to be adopted for different length of tubes (ready reckoner), assuming settling velocity of settling solids (floc) is 35 m/day (1.25 m/hr.)

Table 8.6: The Surface Loading Rates to be adopted for Different Length of Tubes

Tube Size (mm x mm)	Slant Length (mm)	Surface loading rate (on plan area)	
		$\text{m}^3/\text{m}^2/\text{d}$	$\text{l}/\text{m}^2/\text{hr.}$
50 x 50	600	123	5125
50 x 50	750	150	6250
50 x 50	1000	195	8125
	Above 1000 mm lengths are not recommended		

In India, in municipal and government sectors generally tubes with 600 mm length are popularly used, as they achieve the overall cost efficiency of the basin. With length of tube 600mm and size 50*50 mm, relative length works out to 12.

Importance of water depth in basin:

Essentially the particle removal mechanism in a Tube is the same as that in a rectangular basin. However, the depth of water below and above the Tube zone plays a crucial part in achieving overall clarifier efficiency. The depth of a typical Tube Settling tank can be typically divided in four zones:

- a. Buffer Zone (Depth of water below the Tube Modules, generally 1.5 m to 2.0 m)
- b. Tube Zone (modules)
- c. Collection zone (Depth of water above the tube modules, up to collection troughs, 0.60 m to 0.90 m)
- d. Sludge concentration and removal zone

Inlet and Outlet Considerations:

Since the detention time affects the efficiency of tube settling tanks, inlet and outlet needs to be designed very carefully. The inlet velocity at the entry of the flocculated water needs to be less than the floc break velocity (0.1 m/s to 0.3 m/s). The entry ports need to be at least 1.0 m to 1.50 m below the bottom of the tube modules. The flocculated water is distributed uniformly with entry from one side. The outlet is in the form of uniformly spaced collection troughs with spacing varying from 1.0 m to 2.50 m. The troughs are located 0.60

m to 0.90 m above the top of the modules. The troughs ensure that uniform surface loading is imparted on the tubes.

Design Parameters:

For the above-described size and configuration of tubes, for steeply inclined tube settlers (60°) the surface loading rates (overflow rates) based on the plan area of the basin are recommended to be 5000 l/h/m^2 to 6500 l/h/m^2 . These are three to four times higher than the conventional basins. The overall detention time of the tank should be 30 min to 45 min which is 25% to 40% of the conventional. The weir loading should be less than $300 \text{ m}^3/\text{m/d}$. These loading rates are applicable for average water temperatures of 15°C to 30°C . It is possible to achieve settled or clarified water turbidity of 5-10 NTU or even less for raw water turbidity range of 100 - 1000 NTU. The tube settling plants can be designed in square, rectangular or circular configurations.

Sludge Removal:

Tube Settlers with multiple hopper bottom are recommended to be adopted for small and medium capacity plants up to 25-30 MLD for effective sludge removal. Hopper plan area is restricted to $4\text{m} \times 4\text{m}$ at the top. The hopper walls are inclined at 45° . The drain pipe from each hopper (minimum 100 mm diameter) is laid horizontally from the hopper pit. The drain valve provided on pipe ensures periodical removal of sludge. For Large plants, centrally driven scraper rake or rotating bridge scrapers are employed in India. The tube settling systems are also compatible with chain and flights scraper mechanism.



Figure 8.30: Tube Settling and flocculation under operation Modules & Troughs Capacity Plants, WTP at Pandharpur, Maharashtra



Figure 8.31: Performance photo at high turbidity (Tulsi, Mumbai, MCGM, Maharashtra)



Figure 8.32: Performance photo at low turbidity (River Tapti, Nimb, Jalgaon, Maharashtra)

8.6.13 Combination of technologies

(a) Tube – Clariflocculator (Clarissettler)

Recent advances over last 10 to 15 years have resulted in development of different combinations of “Flocculator-Tube Settlers” configurations. Essentially these lead to saving of space (land) and are especially suited to urban area where the land availability is a prime factor.

One such configuration is “Tube-Clariflocculator” or “Clarissettler”. This unit has similar configuration (flow path) that of a conventional Clariflocculator, however the clarification area is provided (packed) with tube settlers. The concentric central flocculation is provided with paddle type slow agitators. Tube modules are placed on radial trusses fixed on the outer wall. Since the surface loading rate of Tube Settlers is four times that of conventional clarification, the overall diameter of the Clarissettler unit becomes one half that of the Clariflocculator. Clarissettler can be employed with centrally driven rake with fixed bridge or peripherally driven rake bridge. The design criteria for flocculation tank and tube clarification zone are as described earlier. These units are recommended to be adopted for plant capacities above 25 MLD.



Figure 8.33: Clarisettler with rotating bridge scraper, WTP, Sangli, Maharashtra, Cap 80 mld

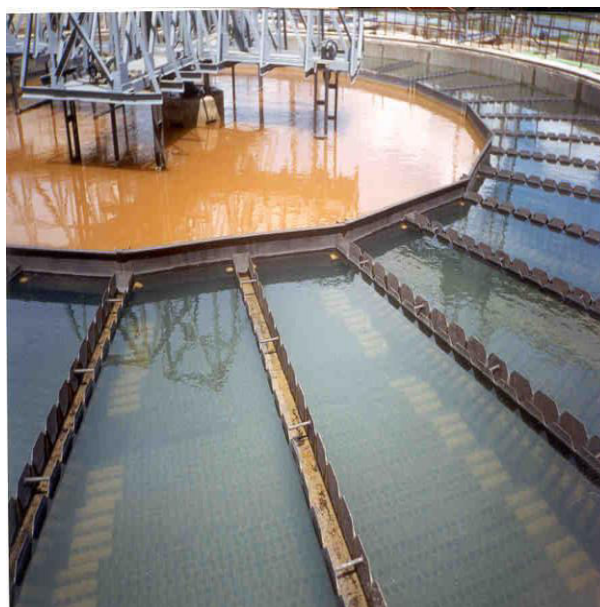


Figure 8.34: Clarisettler with central rake and fixed bridge

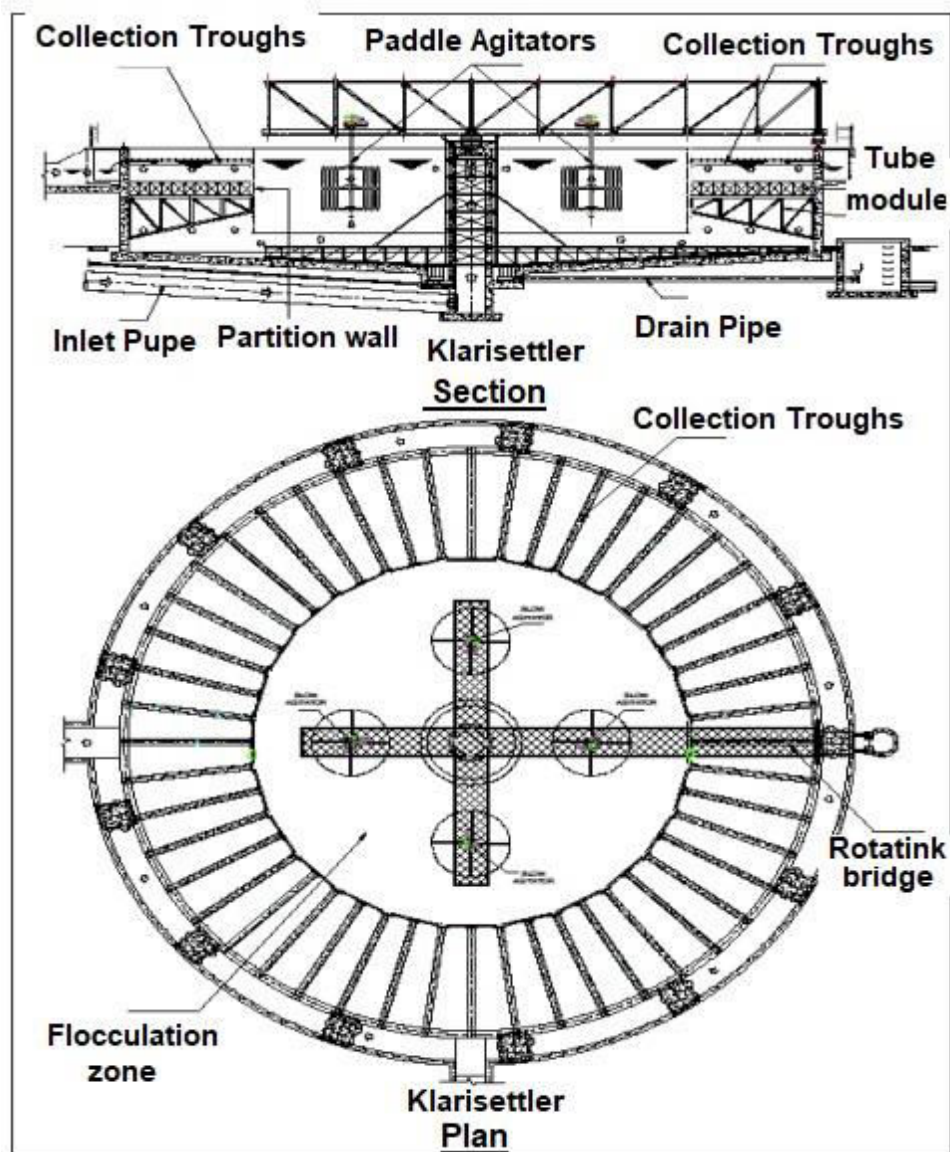


Figure 8.35: Typical Details of Clarisettlers

(b) Tube Clarifier with central rake

This is another configuration of Flocculator - Tube Settlers. In this case the paddle flocculators and Tube Settlers are located in separate units with a common wall. The Tube settlers are configured in to a square tank. The sludge is removed by the central scraper rake. In case of multiple units, common wall construction makes these units very compact. This is alternative to hopper bottom tanks for large capacity plants. The design criteria for flocculation tank and tube clarification zone are as described earlier. These units are recommended to be adopted for plant capacities above 25- 50 MLD.



Figure 8.36: WTP at Ambikapur, Madhya Pradesh, Capacity 18 mld with External flocculators and Tube Settling Tank with Central Rake

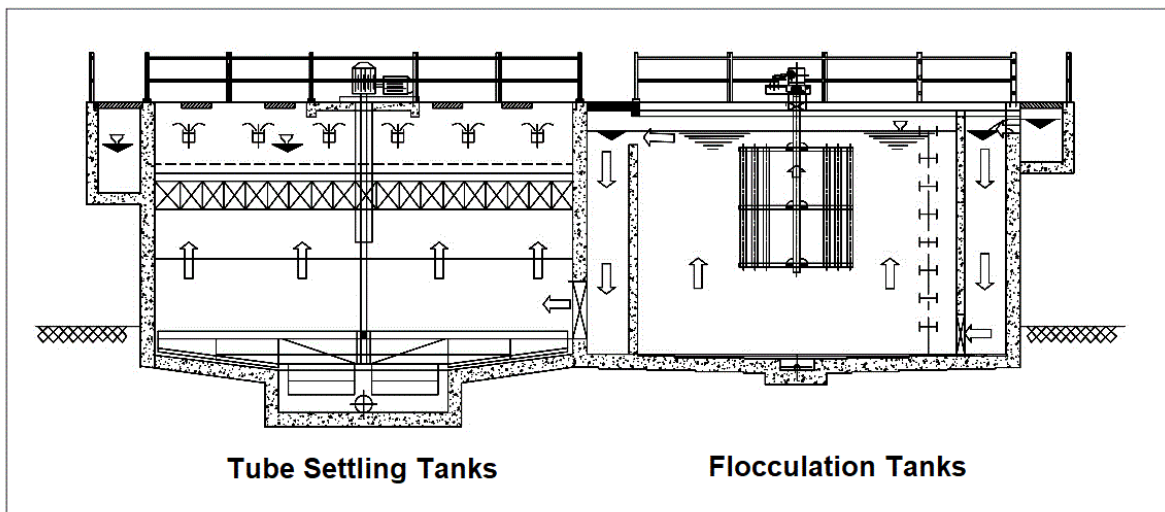


Figure 8.37: Schematic Flow Path of Ambikapur Plant

8.6.14 Plate Clarifiers

A plate clarifier or inclined plate settler (IPS) is a type of settler designed to remove particulates from liquids. Like tube settlers they function on the principle of shallow depth sedimentation. The Plate Clarifier consists of a series of closely spaced flat plates inclined at an angle. Raw water with entrained solids along with flocculating agents enters the plate pack and flows between the plates.

Part A: Engineering Design

The path length, plate spacing, and angle of the plate are the usual engineering variables. As the water flows between pairs of plates, the heavy solids with a specific gravity higher than the surrounding water will settle onto the top surface of the lower plate, and slide down the inclined surface to be collected in the sludge hopper. Clear, near solids-free water then exits the top of the plate area and flows over an adjustable weir.

Benefits of the plate separators:

- Compact design: saves more than 85% of space compared to conventional sedimentation tanks
- As a result of the 60° slant of the funnel, a compact sludge is created which can be transported precisely e.g., via pumps. That is why no manual sludge handling is required.
- No moving parts. Only by the usage of a scraper there is a drive with a shaft.
- Almost no maintenance
- Low-energy system

The inlet water flow is stilled upon entry into the clarifier. Solid particles begin to settle on the plates and begin to accumulate in collection hoppers at the bottom of the clarifier unit. The sludge is drawn off at the bottom of the hoppers and the clarified liquid exits the unit at the top over a weir.

They require an efficient flocculation system which is critical for successful operation. The flocculated water enters at the base of clarifier plates and travels upward between the plates counter-current to settled moving down sludge. In some designs the uneven distribution to the inlet of the clarifiers is corrected by introducing the flocculated water flow individually into each of the space between the plates via slotted openings in the side walls of channels running on both sides of the plate pack along the length of the container. Each space between the plates therefore acts as an independent settling module. The plates extend the full depth of the container and rise about 125 mm above the top water level. Clarified water is collected in decanting launders running along each side of the plate pack by submerged orifices or V-notches one between each pair of plates. About 1.5 m is allowed in the bottom of the container for the collection of sludge which is removed by a circular scraper or a scraper of the chain and flight or reciprocating type to a central hopper or a series of small hoppers at one end of the container. For small containers, sludge could be collected in hoppers placed underneath the plate pack. The plates are inclined at 55°–60° to the horizontal. The turbidity of the clarified water is about 5–10 NTU.

With this arrangement, the settling area available is equal to the sum of the projections of plates in a horizontal plane. Thus, the settling area is very large on account of the overlapping of plates but occupies a relatively small plan area.

The total settling area = $(n - 1) LW \cos \theta$

Where;

n = the number of plates,

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

W = the plate width (m) and

θ = the angle of inclination of the plates to the horizontal.

The value of n should be determined taking plate thickness and spacing between plates into consideration. Plate width is about 1.25–1.5 m and plate length is about 2.5–3.25 m including the length of 125 mm above the normal water surface; plate thickness is usually 0.7 mm for stainless steel. The horizontal spacing between plates is varied according to the application and is normally in the range 50–80 mm. Plate clarifiers are designed on the basis of settling rates related to the projected area of the plates. Depending on the settling velocity of the flocculated particles, the settling rate (also known as the Hazen velocity) could be in the range of 0.6–1.5 m/hr., with the higher rates (>1.0 m/hr.) being used for waters containing high levels of turbidity. Surface loading rates equate to about 8.5–25 m³/m²/hr. (based on plan area of the clarifier) and can therefore give a much-reduced plan area (up to 95%) compared with more conventional horizontal flow clarifiers. This also means that the retention time within the clarifier is low, sometimes 20 min or less, so that control of coagulant treatment becomes more exacting. The tendency for algae to grow on the plates is a problem with plate clarifiers. Such clarifiers should therefore be enclosed in a building. The use of chlorine for algae control could corrode the length of the stainless steel plates above water (due to air above the water containing moist chlorine) and the plates should therefore be protected by a plastic laminate down to about 300 mm below the water surface. Range of applications Incline plate clarifiers can be used in a range of industries including mining and metal finishing, as well as used to treat groundwater, industrial process water and backwash from sand filters.

Limitation of Plate Clarifier

The plates should be flat and not corrugated and they are usually made of stainless steel but sometimes of plastic. This adds to the overall cost of the plants. Plastic/FRP plates if used have found to have a sagging tendency resulting in clogging due to sludge and algae. The plate settlers are difficult to operate in raw waters of low turbidity. Normally polyelectrolyte is required to supplement the primary coagulant.



**Figure 8.38: Plate Settlers at an Industrial plant
(at Jamnagar in India)**



**Figure 8.39: Small Capacity Plate
Settlers unit (at Bhopal in India)**

8.6.15 Ballasted flocculation and Settling

This technology is relatively new to India, however, in future it is likely to be adopted to reduce the footprint. Ballasted flocculation and settling is a clarification process that includes the use of micro-sand as ballast to increase the specific gravity of the floc particles to improve their rate of settling. The micro-sand provides a surface to which the floc particles can attach in the presence of a high molecular weight polymer, and serves as a weight to accelerate in settling. The sand-ballasted floc settles rapidly, which makes it possible to design clarifiers with high overflow rates and short retention times. Such designs make it possible to build systems that have footprints 5% to 20% of the area occupied by conventional clarification systems of similar capacity. This can result in significant cost savings, particularly if expensive excavation is required or the site is space limited. The ballast flocculation followed by tube settler with design surface loading rates of $25\text{--}50 \text{ m}^3/\text{m}^2/\text{hr.}$ will substantially reduce settling area compared to conventional sedimentation basins.

Process Description:

Figure 8.40 & Figure 8.41 shows Jar test with coagulants and micro-sand flocculation.



Figure 8.40: Mixing time 2-5 min**Figure 8.41: Settling time 10 sec**

While there are several variations, a coagulant is typically injected ahead of the first mixing chamber (flash mixer No 1, DT of 1.4-2 min). Polymer and micro-sand (both recycled and make-up sand for that lost with the sludge) are added in the second mixing chamber (Flash mixer No.2, DT of 2-3 min), also referred to as an injection chamber. The size of the micro-sand is normally 45-100 microns. Pilot testing is often used to determine final micro-sand sizing. The sand is quartz or silica with specific gravity 2.4-2.65 and having other properties similar to that of filter sand. The micro-sand feed rate is established to provide consistent sand/solids concentration, typically around 2 – 4 g/l, in the maturation zone which provides a multitude of sites for the influent solids to attach on.

Flocculation is provided in the third maturation chamber (DT of 4-8 min) to enhance particle aggregation. Small particles attach to heavier micro-sand which allow for rapid settling in the sedimentation basin (DT of 8-15 min). The basin is equipped with lamella tubes to provide quiescent conditions and increase the particle removal efficiency. Polymer is a critical component of the treatment process. Its optimized use results in low settled water turbidity and long filter run time. Like other sedimentation processes, a higher polymer dosage may result in lower settled water turbidity but shorter filter runs. Carryover of polymer can occur if too much polymer is applied; therefore, the operational staff must switch from a clarification process optimization philosophy centered on settled water turbidity to optimization of ballasted flocculation for filterability (filter effluent turbidity and run time).

Settled solids (sludge), including the micro-sand, are removed from the sedimentation basin either by a mechanical scrapper or a hopper. The sludge is then directed to a hydro-cyclone via sand recycle pumps. The hydro-cyclone offers a high zone of turbulence to detach lighter particle attached to micro-sand. Sand is re-circulated back to the second mixing chamber. The sludge is treated separately. As the hydro-cyclone operates at a constant flow, the solids concentration in the sludge is lower than what would be from a conventional sedimentation basin, typically less than 1 % solids.

Advantages of a ballasted flocculation system include:

1. Excellent settled water quality: Clarified turbidity around 0.5 NTU and TOC removal between 40% and 75%.
2. Compact System: While the equipment cost is higher than conventional sedimentation basin construction, the reduced size and concrete requirements can result in lower civil work costs and lower total installed costs.
3. Stable Process: Capable of handling wide temperature extremes, high and low turbidity and color levels (option of automating system).
4. Short Start-up Time: The response to dosage changes is very rapid (10 to 15 min). This allows for start-up of the process and steady-state conditions very quickly (approximately 30 min).

Potential disadvantages of this system can be:

1. Energy consumption by the sand pumps which operate at a constant flow.
2. Dilute nature of sludge stream.
3. Make-up micro-sand is required during the operation (1.5 mg/L to 2 mg/L).
4. Reliance on polymer feed
5. Specially trained and efficient O&M staff is required.

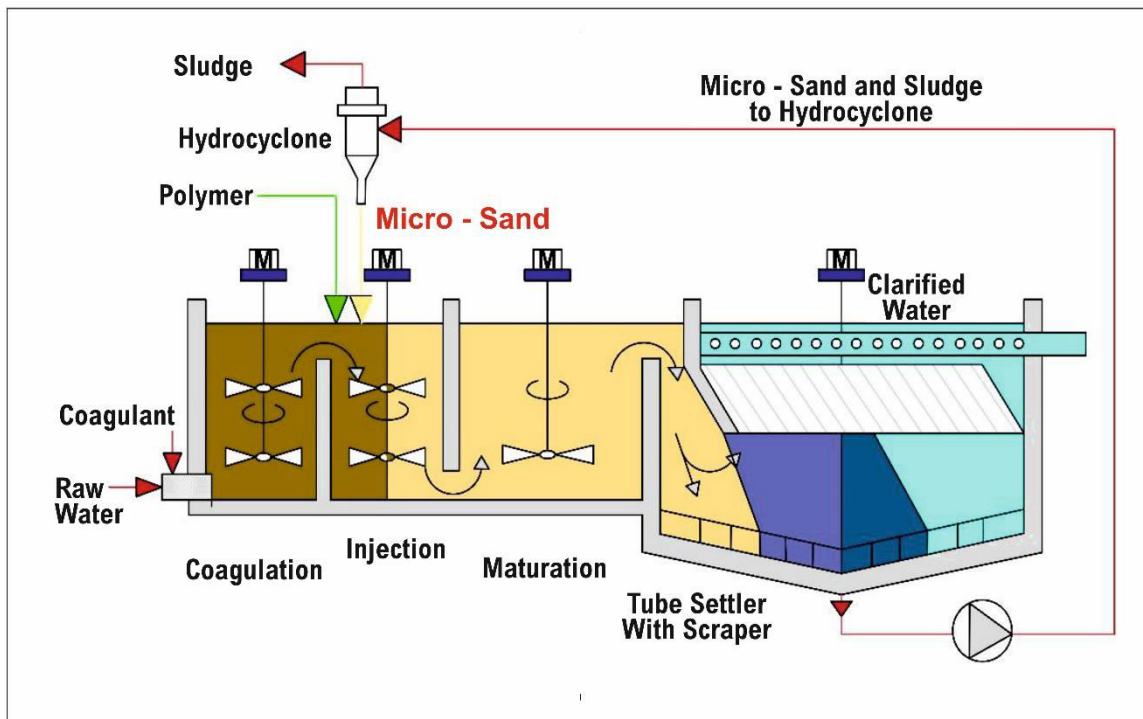


Figure 8.42: Ballast flocculation System with tube settler

8.6.16 Dissolved Air Flotation (DAF)

In flotation, the effects of gravity settling are offset by the buoyant forces of small air bubbles. These air bubbles are introduced to the flocculated water, where they attach to the floc particles and then float to the surface. In gravity settling the flocculation ensures large flocs to settle at the bottom of the basin. In DAF, flocculation is designed to create a large number of smaller flocs.

For efficient flotation, flocculated particles must come in contact with large number of air bubbles. The attachment process is by adhesion of air bubbles to the surface of flocs, entrapment of bubbles under the floc and absorption of bubble in to floc mass. At the surface the bubble floc forms the sludge layer having consistency of 3% to 5% W/V. Floating solids are normally skimmed off by chain and flight skimmer.

The micro-bubbles of size 10 to 100 micron are introduced at the bottom by releasing air super saturated recycle water through proprietary nozzles or orifices. The recycled water is pumped at the rate of 5 % to 10% and is injected by high pressure air from the saturation

tanks. Air is supplied by the air compressor. The tanks can be of rectangular or circular shape.

Design Parameters:

The DAF basin surface loading rates range from $10 \text{ m}^3/\text{m}^2/\text{hr.}$ to $12 \text{ m}^3/\text{m}^2/\text{hr.}$ (10 to 12 m/hr.). Recycle flow rate are 5% to 10% of plant flow rates. Dissolved air pressure is in the range of 4 kg/cm^2 to 6 kg/cm^2 .

The efficiency of DAF depends on the proprietary nozzle to a great extent. These units are compact because of the high solids loading rate. However, they required pretreatment by coagulation and flocculation. This process is particularly effective for raw water with algae, color and having low turbidity.

Advantages and disadvantages:

Application of DAF in Municipal plants for clarification are rare in India. They are advantageous when the TSS is of light nature (viz algae). They have been used in RO feed-water pretreatment for seawater to control the algal blooms.

However, they require high amount of electrical energy for the air blowers (energy intensive). The plant requires skilled manpower for operation and maintenance. Under the Indian techno-economic scenario they exhibit no special advantage over conventional tube/plate settling units.

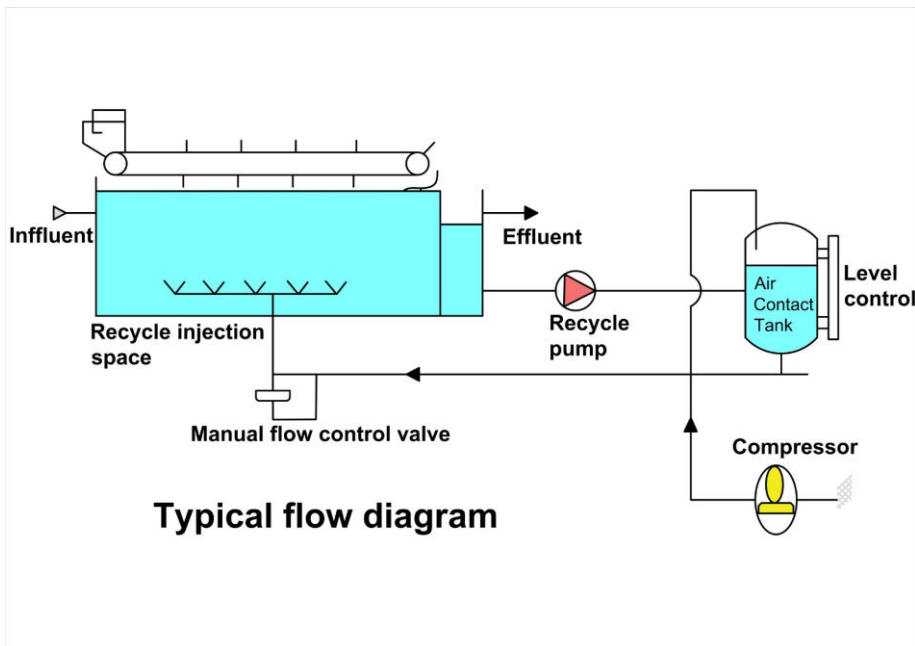


Figure 8.43: Typical Flow Diagram of DAF

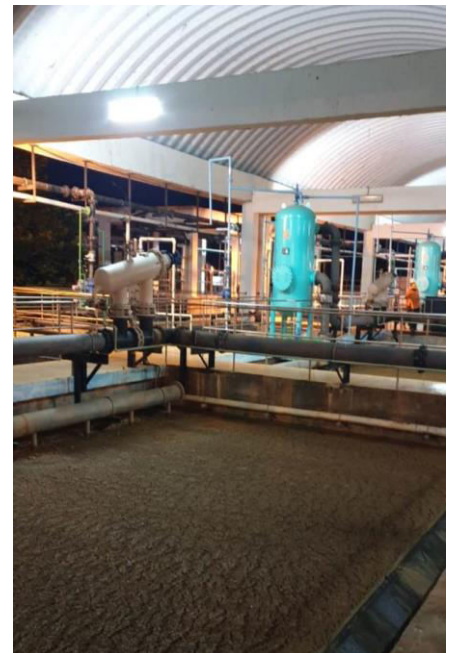


Figure 8.44: WTP at T K Halli, Bengaluru

8.7 Filtration

8.7.1 General

Filtration is a process for separating suspended and colloidal impurities from water by passage through a porous medium or porous media. Filtration, with or without pretreatment, has been employed for treatment of water to effectively remove turbidity (e.g., silt and clay), colour, microorganisms, precipitated hardness from softened waters and precipitated iron and manganese from aerated waters. Removal of turbidity is essential not only from the requirement of aesthetic acceptability but also for efficient disinfection which is difficult in the presence of suspended and colloidal impurities that serve as hideouts for the microorganisms.

Filters can be classified according to (1) the direction of flow (2) types of filter media and beds (3) the driving force (4) the method of flow rate control and (5) the filtration rate. Depending upon the direction of flow through filters, these are designated as down flow, up-flow, bi-flow, radial flow and horizontal flow filters. Based on filter media and beds, filters have been categorized into (a) granular medium filters and (b) fabric and mat filters and micro-strainers. The granular medium filters include single-medium, dual-media and multi-media (usually tri-media) filters. Sand, coal, crushed coconut shell, diatomaceous earth and powdered or granular activated carbon have been used as filter media but sand filters have been most widely used as sand is widely available, cheap and effective in removing impurities. The driving force to overcome the frictional resistance encountered by the flowing water can be either the force of gravity or applied pressure force. The filters are accordingly referred to as gravity filters and pressure filters. In the fourth category (flow rate control) are constant rate and declining or variable rate filters. Lastly dependent upon the flow rates, the filters are classified as slow or rapid sand filters.

Filtration of municipal water supplies normally is accomplished using

- (a) slow sand filters, and
- (b) rapid sand filters

Both of these types of filters are down flow, granular- medium (Single-medium) gravity filters. The rapid sand filters have been conventionally operated at constant rate of filtration.

8.7.2 Slow Sand Filters

8.7.2.1 General

Slow sand filters can provide a single step treatment for polluted surface waters of low turbidity (< 20 NTU) when land, labour and filter sand are readily available at low cost, coagulants and equipment are difficult to procure and skilled personnel to operate and maintain are not available locally.

When raw water turbidity is high, pre-treatment such as storage, sedimentation or primary filtration will be necessary to reduce it to within desirable limits. Coagulation and flocculation are not desirable as the gelatinous floc rapidly clog the sand media.

8.7.2.2 Description

A slow sand filter consists of an open box about 3.0 m deep rectangular or circular in shape and made of concrete or masonry (Figure 8.45). The box contains a supernatant water layer, a bed of filter medium, an under-drainage system and a set of control valves and appurtenances.

The supernatant provides the driving force for the water to flow through the sand bed and to overcome frictional resistance in other parts of the system. It can also provide a storage of several hours to the incoming water before it reaches the sand surface.

The filter bed consists of natural sand with an effective size (E.S.) of 0.25 mm to 0.35 mm and uniformity coefficient (U.C) of 3 to 5. For best efficiency, the thickness of filter bed should be not less than 0.4 - 0.5 m. As a layer of 10-20 mm sand will be removed every time the filter is cleaned, a new filter should be provided with an initial sand depth of about 1.0 m. Re-sanding will then become necessary only once in 2-3 years.

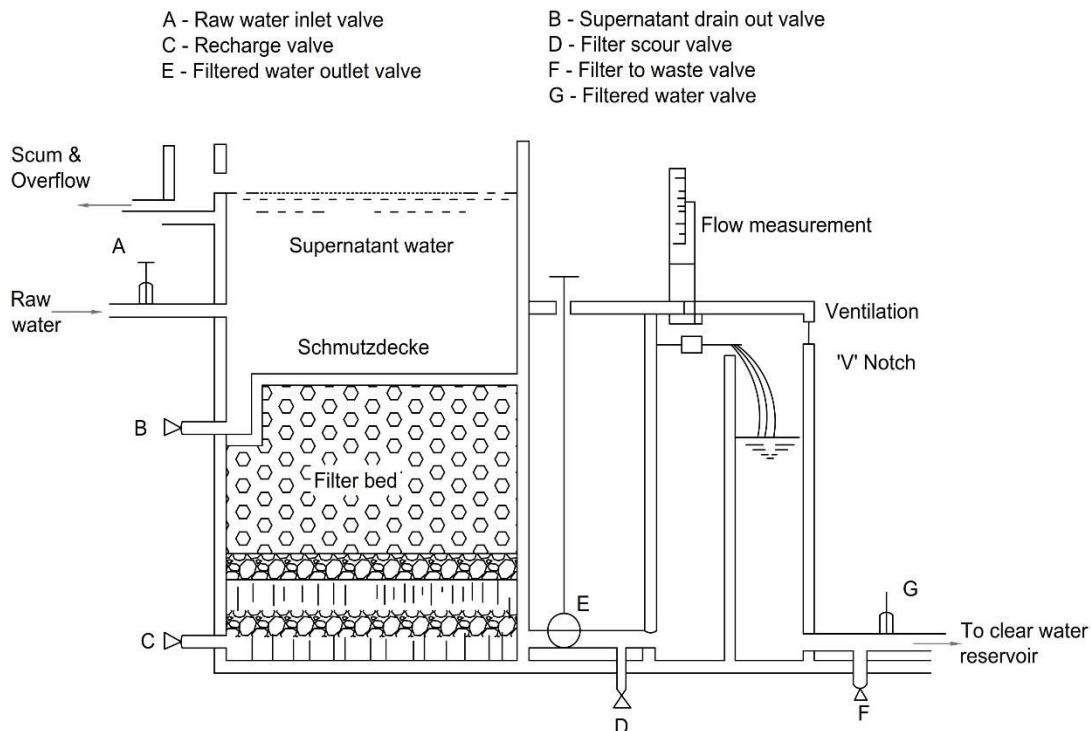


Figure 8.45: Basic Element of Slow Sand Filter (Schematic)

The under-drainage system supports the sand bed and provides unobstructed passage for filtered water to leave the underside of the filter. The under-drains may be made of unjointed bricks laid to form channels, perforated pipes or porous tiles laid over drains. Graded gravel to a depth of 0.2 - 0.3 m is placed on the under-drains to prevent the sand

from entering the under-drains and ensure uniform abstraction of filtered water from the entire filter bed.

A system of control valves facilitates regulation of filter rate and adjustment of water level in the filter at the time of cleaning and backfilling when the filter is put back into operation after cleaning.

A Purification in a Slow Sand Filter

In a slow sand filter, water is subject to various purifying influences as it percolates through the sand bed. Impurities are removed by a combination of straining, sedimentation, bio-chemical and biological processes. Shortly after the start of filtration, a thin slimy layer called the 'Schmutzdecke (Dirty layer)' is formed on the surface of sand bed. It consists of a great variety of biological organisms which feed on the organic matter and convert it into simple, harmless substances. Considerable portion of inert suspended particles is mechanically strained out in this layer. During its passage through 0.4 - 0.6 m of sand bed, the water becomes virtually free from suspended solids, colloids, and pathogens. The result is a simultaneous improvement in the physical, chemical and bacteriological quality of water. Starting with an average quality of raw water, a properly designed and operated filter can produce a filtrate satisfying normal drinking water standards. Nevertheless, the filtrate should be disinfected to render it safe.

8.7.2.3 Design Considerations

(a) Filter shape and layout

Rectangular filters offer the advantage of common wall construction and may be preferred except for very small installations where circular shape may be economical. Arranging filters in a row maximizes the number of common walls and facilitates construction, operation and maintenance. Filters can also be arranged symmetrically on either side of a central pipe gallery. The layout will be determined by local topography and the placement of pump houses, storage and other facilities.

(b) Depth of Filter Box

The elements that determine the depth of the filter box and their suggested depths are free board (0.2 m), supernatant water reservoir (1.0 m), filter sand (1.0 m), supporting gravel (0.3 m), and under-drainage system (0.2 m) with a total depth of 2.70 m. The use of proper depths for these elements can reduce the cost of the filter box considerably without adversely affecting efficiency.

(c) Filter Sand and Gravel

Undue care in the selection and grading of sand for slow sand filters is neither desirable nor necessary. Use of builder grade or locally available sand can keep the cost low. Similarly, rounded gravel, which is often quite expensive and difficult to obtain, can be replaced by hard, broken stones to reduce cost.

8.7.2.4 Construction Aspects

(a) General

The construction of slow sand fillets should be based on sound engineering principles. Some of the important considerations that need attention are (i) the type of soil and its bearing capacity; (ii) the ground water table and its fluctuation and (iii) the availability and cost of construction materials and labour. Water tight construction of the filter box should be guaranteed, especially when the ground water table is high. This will prevent loss of water through leakage and contamination of filtered water. The top of the filter should be at least 0.5 m above the ground level in order to keep away dust, animals and children. The danger of short circuiting of raw water may be prevented by rough finishing of the inside of the walls up to maximum sand level. The drainage system should be carefully laid as it cannot be inspected, cleaned or repaired without the complete removal of the filter bed material.

(b) Inlet

The inlet structure is an important component of a slow sand filter and should be so designed and constructed as to cause minimum disturbance to the filter bed, while admitting raw water and to facilitate routine operation and maintenance. A filter needs to be cleaned periodically and this is done by lowering the water level a few centimeters below the sand bed and scraping the top layer of 10-20 mm of sand. It is found in practice that draining the water through the filter bottom takes several hours, at times 1-2 days. In order to obviate this difficulty, a supernatant drain out chamber with its top just above the sand level, has to be provided. By a proper design, the filter inlet and the supernatant drain out could be suitably combined in a single chamber (Figure 8.46 & Figure 8.47).

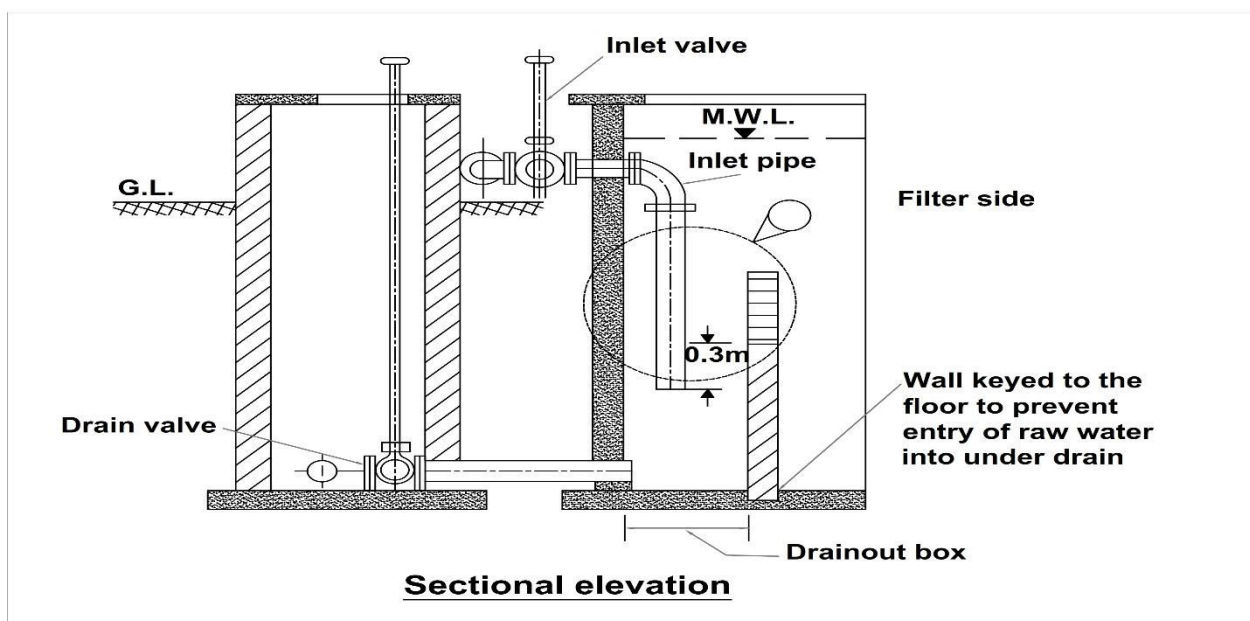


Figure 8.46: Sectional Elevation of Inlet Section

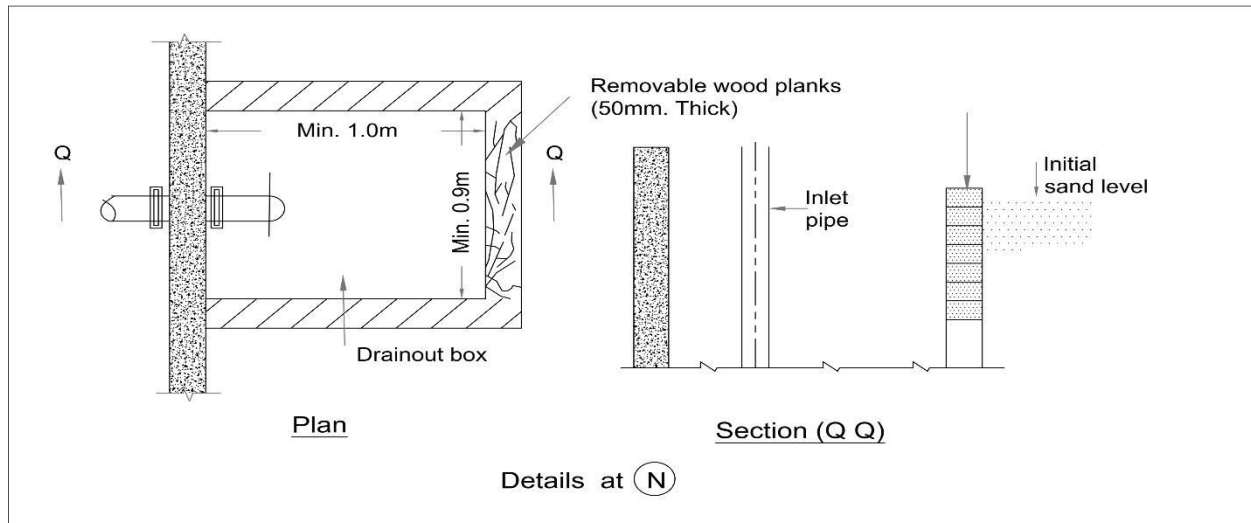


Figure 8.47: Inlet cum Supernatant Drain-out Box

(c) Outlet

The outlet structure incorporates means for measuring the filter flow and backfilling with clean water after sand scraping and re-commissioning of the filter.

In small filters, the outlet chamber is usually constructed in two parts separated by a wall with a weir. The sill of the weir is fixed above the highest sand level in the filter bed. This makes filter operation independent of fluctuations in the clear water storage level and prevents occurrence of negative head in the filter. It also aerates the filtered water thus raising its oxygen content. To facilitate aeration, a ventilation opening properly screened is provided in the chamber (Figure 8.48).

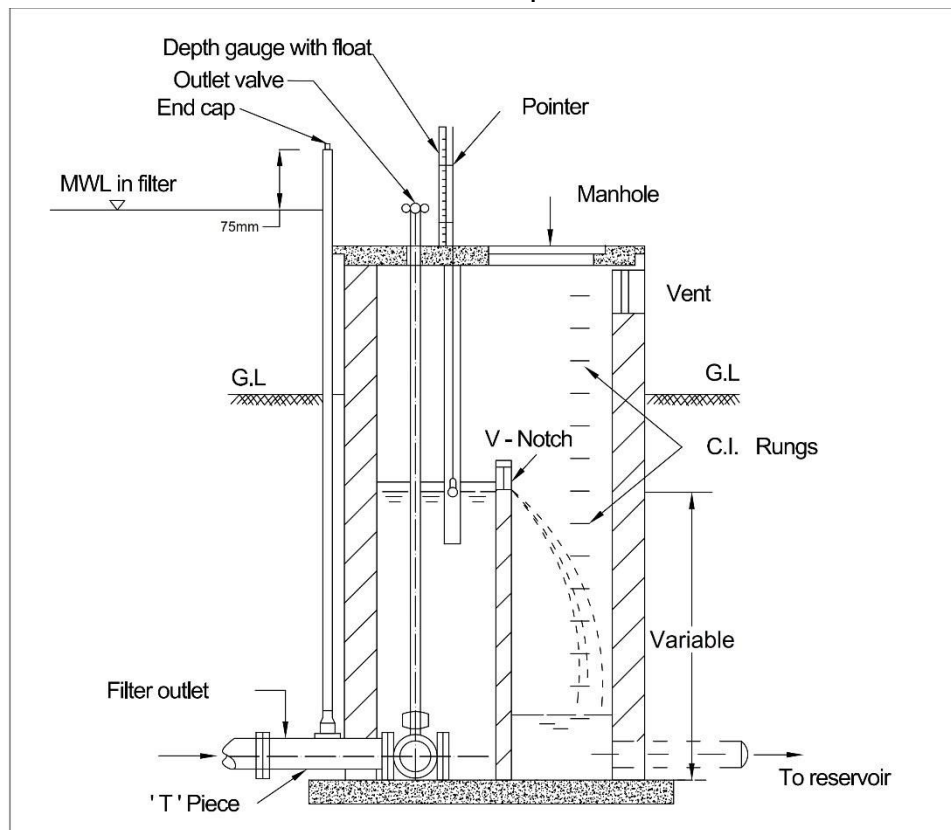


Figure 8.48: Outlet Chamber

(d) Scum and overflow outlet

To facilitate drainage of surplus water entering the filter and scum that may accumulate on the supernatant water, an overflow pipe/weir should be provided in the filter.

Note: The Slow sand filters have served their purpose in the past well but have been gradually phased out from Indian drinking water sector in last two decades. They are recommended to be used only for remote and small communities with relatively clean raw water sources. The cost of these plants is also an issue of concern. primary reason being the rate of filtration being very slow (50 to 100 times less than that of rapid sand gravity filters), they occupy a huge area hence land cost and construction cost is high. These filters cannot operate with moderate turbidity (more than 20 NTU). If provided with pretreatment (without coagulation) then it adds to the overall cost. Manual cleaning by scrapping the sand requires manpower which too has become costly.

8.7.3 Rapid Sand Filters

8.7.3.1 Filtration Process

The rapid sand filter consists of a bed of sand serving as a single medium granular matrix supported on gravel overlying an under-drainage system. The distinctive features of rapid sand filtration as compared to slow sand filtration include careful pretreatment of raw water to effectively flocculate the colloidal particles, use of higher filtration rates and coarser but more uniform filter media to utilize greater depths of filter media to trap influent solids without excessive head loss and backwashing of filter bed by reversing the flow direction to clean the entire depth of filter. Pretreatment of filter influents should be adequate to achieve efficient removal of colloidal and suspended solids despite fluctuations in raw water quality. A typical sketch for granular medium gravity flow filter is shown in Figure 8.49.

When water containing suspended matter is applied to the top of filter bed, suspended and colloidal solids are trapped in the granular medium matrix. Accumulation of suspended particles in the pores and on the surface of filter medium leads to build up of head loss as pore volume is reduced. The media offers greater resistance to the flow of water, simultaneously with the buildup of head loss. At a predetermined terminal value, the suspended solids removal efficiency of successive layers of filter medium is reduced as solids accumulate in the pore space and reach an ultimate value of solids concentration as defined by operating conditions. This results eventually in breakthrough of suspended solids and the filtrate quality deteriorates. Ideally, a filter run should be terminated when the head loss reaches a predetermined value simultaneously with the suspended solids in filtrate attaining the preselected level of acceptable quality.

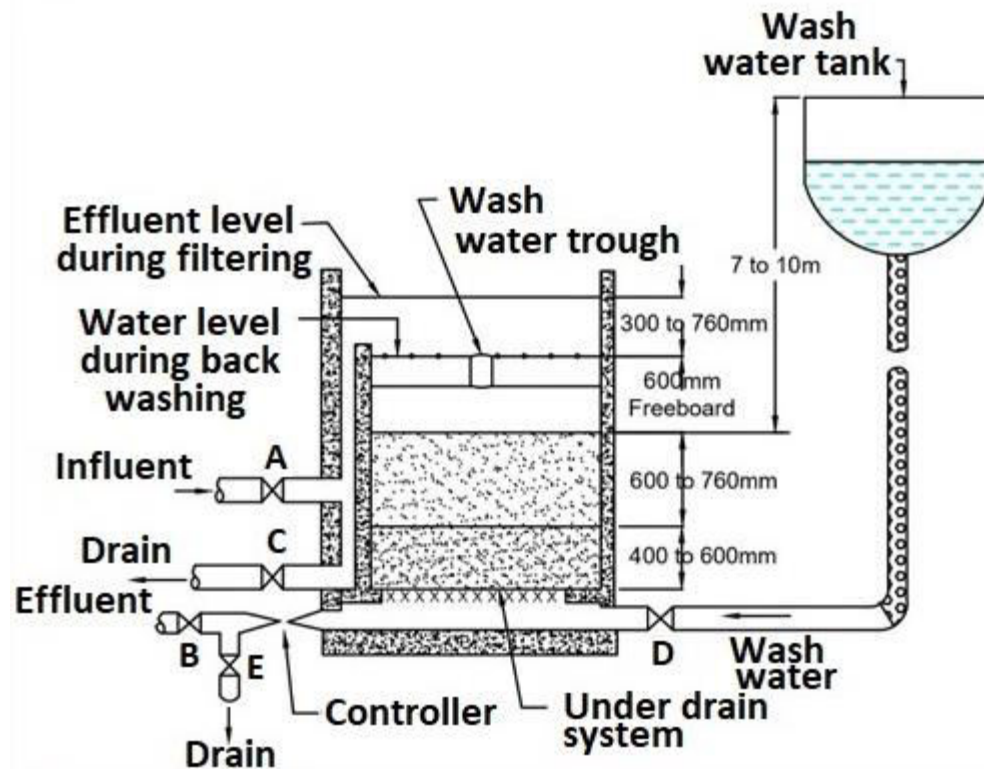


Figure 8.49: Granular Medium Gravity Filter

8.7.3.2 Principal Mechanisms of Particle Removal

The removal of particles within a deep granular-medium filter, such as rapid sand filter, occurs primarily within the filter bed and is referred to as depth filtration. Several mechanisms either single or in combination, act to achieve overall removal of suspended and colloidal matter in depth filtration. Conceptually the removal of particles takes place in two distinct steps, a transport and an attachment step. In the first step, the impurity particle must be brought from the bulk of the liquid within the pores close to the surfaces of the medium or the previously deposited solids on the medium. Once the particles come closer to the surface, an attachment step is required to retain it on the surface instead of letting it flow down the filter.

The transport step may be accomplished by straining, gravity settling, impaction, interception, hydrodynamics and diffusion and it may be aided by flocculation in the interstices of the filter. The particle transport is a physical process principally affected by those parameters which govern mass transfer. These physical variables include size of filter medium, d_m ; filtration rate, v ; density, ρ_s ; size of the suspended particle, d_p ; and water temperature.

The particle attachment step is a physicochemical process involving electrostatic interactions, Van der Waal's forces of molecular attraction, chemical bridging or specific adsorption. Attachment is affected by chemical characteristics of the water and filter medium. Pretreatment of filter influents by coagulants and pH of water affect the efficiency

of attachment step and consequently of solid removal in a filter. The need, therefore, of adequate pretreatment before filtration to achieve efficient removal of suspended solids is evident.

Dimensionless parameters have been defined for various transport and attachment mechanisms and mathematical equations are proposed to predict the removal efficiency of particles based on physical variables such as d_p , d_m , ρ_s , v , ρ and μ the liquid density and absolute viscosity, porosity of filter medium and concentration of suspended solids and chemical characteristics of water and filtering medium. An analysis of these analytical expressions indicates that filter efficiency may improve by decreasing the size of filter media, reducing the rate of filtration and at higher temperatures. The suspended particles falling into the categories of significantly more than and less than $1\ \mu\text{m}$ diameter are efficiently removed. A particle with a size of $1\ \mu\text{m}$ has the lowest efficiency of removal. Further, ample attachment opportunities exist in conventional deep granular filters. If such filters do not remove solids efficiently, pretreatment should be changed to improve attachment of suspended particles to filter media grains.

8.7.3.3 Rate of Filtration

The standard rate of filtration through a conventional rapid sand filter is usually 80 to 100 $\text{l/m}^2/\text{min}$ ($4.8\text{--}6\ \text{m}^3/\text{m}^2/\text{hr.}$). Practice is tending towards higher rates by adopting Dual Media Gravity Filters or Mono Media Deep Bed Filters (Described later). A prudent arrangement would be to design the filters on the basis of average consumption at an average rate of 5.5 m/hr. or less and then ensure that under overloading condition, when one bed is out of service, maximum rate of filtration is less than $6\ \text{m}^3/\text{m}^2/\text{hr.}$ (The overloading check may applied only if no of beds are more than 4, four). The inlet and the outlet control arrangements are designed to permit overload for emergent occasions or when one filter is under backwash operation. (This overloading factor should be 100% when two filter beds are provided, 50% in case of three beds, 33.33% in case of four beds and 20% when more than four beds are provided).

8.7.3.4 Capacity of a Filter Unit

The capacity of the rapid sand filters should be such that the number of units can take care of the total quantity of water to be filtered and is optimum to keep the filters working without undue overloading at any time. The Table 8.7 shows the number of rapid sand filters for different plant capacities. The smaller the number of units, the fewer the appurtenances but the larger the wash-water equipment that will be required. Thus, while designing large size filters, one must consider the rate at which wash-water must be supplied and the hydraulic problems for securing uniform distribution of wash-water due to the large area. A maximum area of $100\ \text{m}^2$ for a single unit is recommended for plants of greater than 100 MLD consisting of two halves each of $50\ \text{m}^2$ area. Also, for flexibility of operation a minimum of four units should be provided which could be reduced to two for smaller plants.

Table 8.7: Numbers of Rapid Sand Filters for Given Plant Capacity

Capacity in MLD (24 hr basis)	Number of beds*
1	1W (with one section)
2	2W (with one section)
3-8	2W
9- 15	3W
15- 30	4W
31- 50	6W
51-80	8W
80-100	9W +1S

Note:

1. Above 80 MLD capacity it is recommended to provide High-Rate Filtration viz Dual Media or Mono Media Deep Bed Gravity Filters, described later in the Manual
2. *Filter beds with two sections separated by central gutter or gullet

8.7.3.5 Dimensions of Filter Unit

Layout of the plant, economy and convenience determine the relationship between the length and the breadth of the units. Where filters are located on both sides of a pipe gallery, the ratio of length to width of a filter-box shall be preferably between 1.25 to 1.33. The bed shall have an overall depth from 2.6 m to 3.75 m depending on type of filter excluding a free board of 0.5 m.

The filter shell (boxes) shall be in reinforced concrete to ensure a water tight structure. Except in locations where seasonal extremes of temperature are prevalent (extremely heavy rains or significant snowfall areas), it is not necessary to provide a roofing over the filters. The filters shall be preferably covered with roof in a building. The filters are recommended to be covered if they lie in the vicinity of thermal power plants or carbon product manufacturing units having chimneys to avoid water getting covered by the soot.

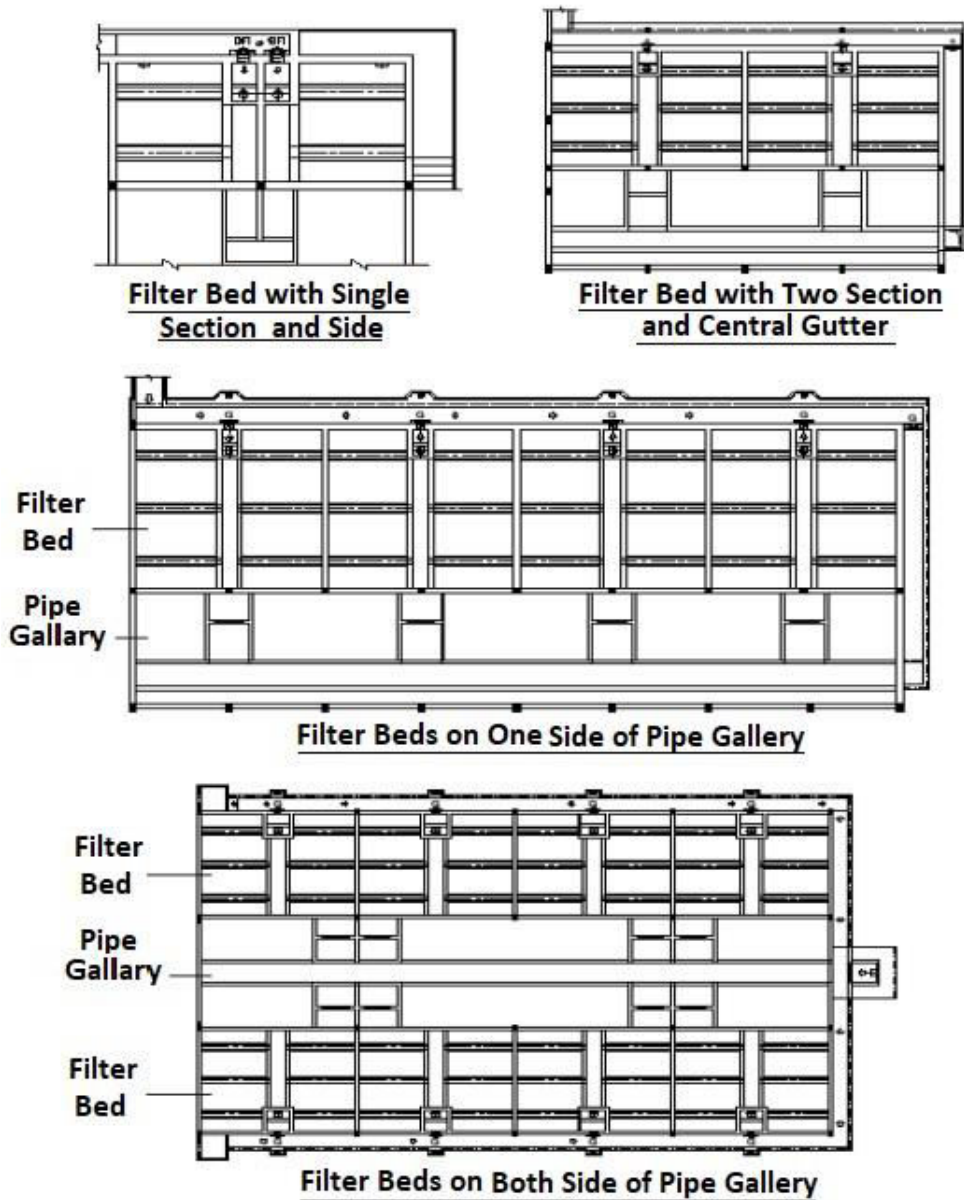


Figure 8.50: Filter Beds

8.7.3.6 Filter Sand

Filter sand is defined in terms of *effective size* and uniformity coefficient. Effective size is the sieve size in millimeters that permits 10% (D_{10}) by weight to pass. Uniformity in size is specified by the uniformity coefficient which is the ratio between the sieve size that will pass 60% (D_{60}) by weight and the effective size.

Shape, size and quality of filter sand shall satisfy the following norms:

- (a) Sand shall be of hard and resistant quartz or quartzite and free of clay, fine particles, soft grains and dirt of every description.

- (b) Effective size shall be in average between 0.45 to 0.70 mm. The effective size of sand can range from 0.8 to 1.5 mm for deep sand filters (like mono media deep sand filter).
- (c) Uniformity coefficient shall not be more than 1.7 nor less than 1.3
- (d) Ignition loss should not exceed 0.7% by weight.
- (e) Soluble fraction in hydrochloric acid shall not exceed 5.0% by weight.
- (f) Silica content should be not less than 90%.
- (g) Specific gravity shall be in the range between 2.55 to 2.65.
- (h) Wearing loss shall not exceed 3%

A Depth of Sand

Usually, the sand layer has a depth of 0.60 to 0.75 m, but for higher rate filtration when the coarse medium is used, deeper sand beds are suggested. The standing depth of water over filter varies between 1 and 2 m. The free board above the water level should be at least 0.5 m so that when air binding problems are encountered, it will facilitate the additional levels of 0.15 to 0.30 m of water being provided to overcome the trouble.

B Preparation of Filter Sand

The sand to be used in the filter is specified in terms of effective size and uniformity coefficient. From a sieve analysis of the stock sand, the coarse and fine portion of stock sand that must be removed in order to meet the size specifications, can be computed in terms of p_1 the percentage of stock sand that is smaller than the desired effective size d_1 , which is also equal to 10% of the usable sand and p_2 the percentage of the stock sand that is smaller than the desired 60 percentile size d_2 .

The percentage of suitable stock sand p_3 , is then $= 2(p_2 - p_1)$ because the sand lying between the d_1 and d_2 sizes will constitute half of the specified sand.

To meet the specified composition, this sand can contain $0.1p_3$, of a sand below d_1 size. Hence the percentage p_4 , below which the stock sand is too fine to use is

$$P_4 = P_1 - 0.1P_3 = P_1 - 0.2(P_2 - P_1) = 1.2P_1 - 0.2P_2$$

Likewise, the percentage P_5 above which the stock sand is too coarse for use is

$$\begin{aligned} P_5 &= P_2 + 40\% \text{ of usable sand} \\ &= P_2 + 0.4 \times 2 (P_2 - P_1) \\ &= P_2 + 0.8(P_2 - P_1) \\ &= 1.8 P_2 - 0.8P_1 \end{aligned}$$

From the size-cumulative frequency curve, the grain sizes of stock sand corresponding to p_4 , and p_5 are determined (d_4 and d_5). The sizes below d_4 and above d_5 will have to be separated out from the stock sand to bring it to the desired specification. This may be done by sieving. The finer portion can also be removed in a sand washer designed to float out the particles of size smaller than d_4 by maintaining velocity in the upward

flow washers slightly less than the hydraulic subsidence value corresponding to d_4 size, such that all particles less than d_4 size are floated out with the flowing water.

C Procurement of Filter Sand

The natural filter sand is procured from Banda in Uttar Pradesh, Poanta Sahab in Himachal Pradesh, Jaipur in Rajasthan, Godra & Vejalpur in Gujarat, Bhandara in Maharashtra, Belagavi in Karnataka and other sites.

Further the IS:8419 (Part 1) (1977, Reaffirmed 2010) entitled “requirement for filtration equipment- Filtration Media Sand and Gravel” may be referred to for details.

The water utility shall identify and request the district administration to reserve the stretch of river from where required quality filter sand media and gravel conforming to the IS code can be obtained/procured.

8.7.3.7 Filter Bottoms and Strainer Systems

The IS:8419 (Part 2) (1984, Reaffirmed 1996) entitled “requirement of rapid sand gravity filtration equipment -under-draianage system” maybe referred to for details.

The under-drainage system of the filter is intended to collect the filtered water and to distribute the wash water in such a fashion that all portions of the bed may perform nearly the same amount of work and when washed, receive nearly the same amount of cleaning. Since the rate of wash is several times higher than the rate of filtration, the former is the governing factor in the hydraulic design of filters which are cleaned by backwashing. The design of under-drain is most critical for efficient backwash operation and functioning of the filters. The material needs to be sturdy. Once buried under the gravel and sand, the under-drain has no access.

The most common type of under-drain is a central manifold with laterals either perforated on the bottom or having umbrella type strainers on top. Other types such as wheeler bottom, block under-drains, false bottom with nozzles.

Manifold and Laterals

In the case of central manifold with lateral system, the manifolds are cast iron or mild steel pipes buried in the pit. Rectangular or square concrete conduits located below the filter bottom are commonly used. The laterals are of Rigid PVC (10 kg/cm^2), HDPE, cast iron, and galvanized iron or SS316 material. The velocity of jets issuing from perforations or orifices is destroyed by directing the openings downwards against the filter bottom and into the coarse gravel surrounding the pipes. The lost head, therefore, will be equal to the driving head during the wash. In practice this controlling head loss is set between 1 to 4.5 m. A nonferrous under-drain system is preferable where the water has a low pH and is corrosive and when the correction for pH has to follow the filtration process. However, A.C. pipes should not be used. The best material is plastics.

The following values, may be used in design of an under-drain system consisting of central manifold and laterals.

The perforations vary from 5 to 12 mm in diameter and should be staggered at a slight angle from the vertical axes of the pipe. Spacing of perforations along the laterals may vary from 80 mm for perforations of 5 mm to 200 mm for perforations of 12 mm.

Ratio of total area of perforations to the under-drain system to total cross sectional area of lateral should not exceed 0.5 for perforations of 12 mm and should decrease to 0.25 for perforations of 5 mm.

Ratio of total area of perforations to the entire filter area may be about 0.3% to 0.35%. The ratio of length to diameter of the lateral should not exceed 60. The spacing of laterals closely approximates the spacing of orifices and shall be 300 mm.

The cross-sectional area of the manifold should be preferably 1.5 to 2 times the total area of the laterals to minimize frictional losses and to give the best distribution. It is useful to check the design for uniformity of distribution of wash water in laterals of the under-drains.

The central manifold with literal type of under-drainage system is shown in Figure 8.51.

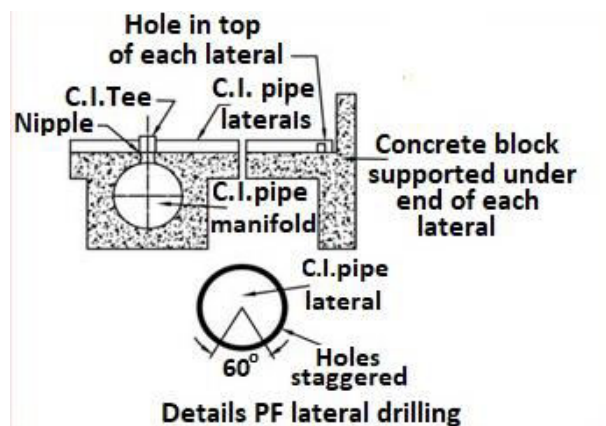


Figure 8.51: Perforated Pipe Under-drain



Figure 8.52: Manifold-Laterals Assembly connector block to Manifold pit, Laterals Manifold pit, Laterals and air distribution header

Figure 8.53: “Concrete Tee” as a with air feeder pipes

Nozzle Systems on Header Laterals

Nozzle geometries are typically either cylindrical, conical or umbrella top with long hollow stem. Injection molded nozzles utilize V-shaped, vertical slots that prevent fouling. Some believe that the older-style conical nozzles have an angular backwashing benefit; however, cylindrical nozzles are manufactured with reinforcing screen ribs to ensure added longevity.

The size of a nozzle is directly proportional to flow rate. Hence, nozzle density in the under-drain system factors into the flow rate per nozzle. For example, for the same lpm per square metre backwash rate, designing nozzles on 200 mm centers each way requires 78% more flow per nozzle than designing nozzles on 150 mm centers each way. It is recommended that nozzles to be located as close as practicable to ensure effective media cleaning through more points of energy input and to potentially eliminate gravel and other types of problematic packing layers.

The filter floor should consist of a series of lateral pipes each connecting to a central channel or manifold and fitted with nozzles. The pipes should have holes on the top to receive ‘nozzle plates’ in which the nozzles are screwed. The portions of the pipes bridging the channel, formed in the floor of the filter tank should have slots on the undersides. All the pipes should be grouted to the floor and embedded in concrete with suitable reinforcement, after which the hole should be screwed to a smooth surface flush with the upper faces of the nozzle plates. The nozzles should then be screwed into the position.

Mounting methods of non-lateral nozzles depend on the under-drain type. For monolithic, reinforced concrete under-drains, nozzles thread into concrete sleeves that are cast into the floor. Non-tapered threads require an ‘O-ring’ to lock the replacement nozzle into place.

Nozzles for pipe header-laterals may be mounted directly using a pipe saddle and expanding ring. For header-lateral systems embedded in concrete, pipe sleeves with temporary caps may be glued onto PVC or HDPE pipe. After the concrete has been poured and cured, the caps may be removed and nozzles installed. Alternatively, a bottom-threaded concrete sleeve with temporary cap may be threaded into an expanding ring with pipe saddle.

Materials used for nozzle products are of polypropylene, glass-reinforced polypropylene, un-plasticized PVC, HDPE, cast iron, stainless steel, brass and combinations of materials such as a stainless-steel screen section with polypropylene stem etc.

Polypropylene is appropriate for most water treatment applications. However, for example, if water temperatures exceed 60°C (or 140°F) or if specific chemical resistance is necessary, other materials of construction should be selected.

Nozzle screen slot widths vary from 0.1 mm for very small ion exchange resin up to 5.0 mm for under-drains with a deep gravel packing layer. Most filtration applications tend to result in screen slot widths between 0.2 mm and 0.5 mm. Nozzle cages with color-coding found helpful for the ease of identification of the slot width.

These dimensions are typically chosen to be between 50 % to 70% of the effective size of the smallest media in order to retain the granules as well as to not foul with fines or biology. For example, the effective size of typical sand filter media of 0.5 mm; a 0.3 mm-slotted screen should be chosen. An adsorber may have 1.2 mm effective size media; thus, a 0.8 mm-slotted screen would be appropriate.

Appropriate amounts of pressure drop or head loss through the nozzle at operating flow rate ranges is important to ensure proper distribution during backwash. Enough pressure drop through the under-drain is required to overcome the head loss of the dirty media bed. Otherwise, uneven distribution occurs, allowing unwanted channeling and poor media cleaning, which reduces subsequent filter run times and creates maintenance issues due to the left-over solids. For typical sand/anthracite/Granulated Activated Carbon applications, at least 500 to 600 mm of Water Column is recommended at the design high backwash rate.

There are two main design elements that determine pressure drop through a nozzle at a given flow rate. First is screen construction. A choice may be required between more than one appropriate width. For the same number of slots, smaller width slots impart more pressure drop than larger. Utilizing the same width slot, fewer slots in a screen will create more pressure drop at the same flow rate.

The second design element is the bore size of the thread section and the tailpipe, if used. In increasing pressure drop for a given flow rate, bore diameters of 21 mm, 16 mm, 16 mm with a 13 mm restriction in the thread section, and also 13mm may be utilized. To add even more pressure drop, a closed stem with holes may be used.

Proper nozzle characteristics ensure media retention, negate fouling, and produce excellent distribution, which will deliver optimal filter performance.

A typical nozzle for under-drainage system is shown in Figure 8.54.

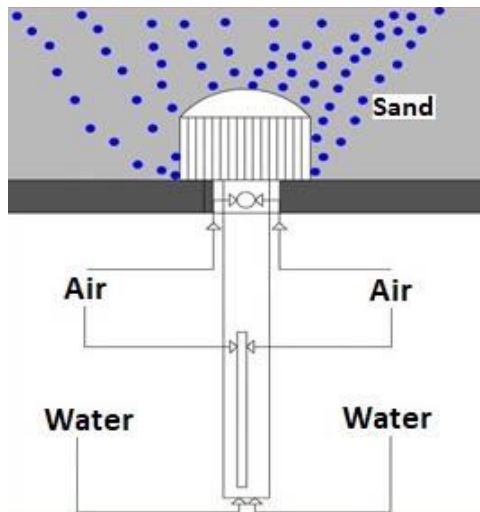


Figure 8.54: Nozzles on Laterals

Note: From past experience, it is recommended that 30-40 nozzles per m^2 have to be provided for uniform backwash. Smaller quality of nozzles per m^2 gives rise to pyramidal dead zones which in turn leads to bacteria culture. The typical flow rates of nozzle are approximately 1.5 to 2 $m^3/hr.$ at 1 kg/cm^2 of pressure difference.

Strainer Nozzles with False Bottom and Plenum Chamber

The nozzles with stems are embedded in to a false bottom concrete slab. The concrete slab is normally 100 to 125 mm thick. The slab is supported with concrete stub columns. The portion below the slab is called as “Plenum Chamber “. For maintenance purpose the height of the plenum chamber needs to be minimum 900 mm. A manhole is provided on the side wall of the filter to enter into plenum chamber.

During casting of the false bottom concrete slab, the nozzle sleeves are embedded in it. The spacing of the nozzles in both direction needs to be uniform. Later the nozzle with stem is screwed in to sleeves. There are various types of nozzles with different area of opening. Normally the nozzle with 0.2 mm slit width is adopted. The manufacturer has to furnish the data on area of opening and head-loss that occurs during the filtration cycle and backwash cycle. The system is suitable for air, air/water and water wash. However, it is cumbersome to construct. The false bottom slab and plenum chambers increases the civil cost. The false bottom slab needs to be designed for the tension on both sides.

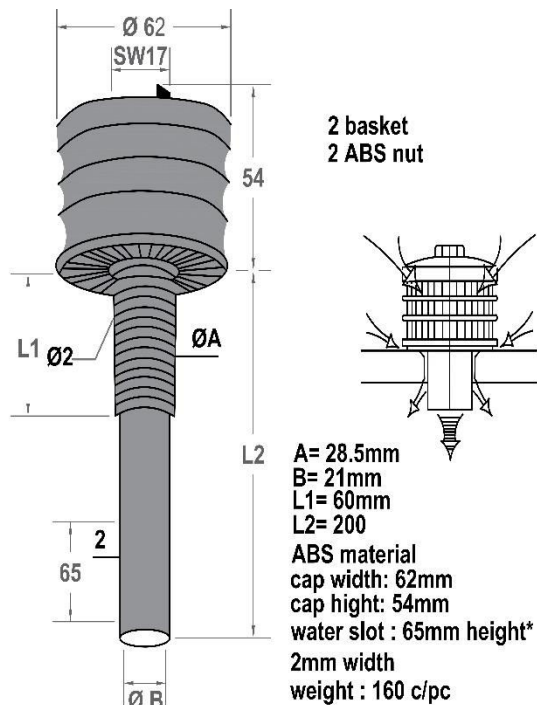


Figure 8.55: Typical cylindrical nozzle with stem & sleeve



Figure 8.56: Finished Filter floor after erection of nozzles

New Upcoming Technology - Dual-parallel under-drain blocks (Flat bottom flumes)

This system provides an improved distribution of backwash water to the filter media through the use of dual compensating laterals. Backwash water enters the primary laterals and then passes through the control orifices into the secondary laterals. Backwash air is properly distributed by careful design of upper control orifices between the primary and secondary laterals, providing an even air distribution. Media retention plates maximize the available filter tank depth. The media retention plates prohibit media pass through and provide proper flow characteristics.

The blocks size ranges from 200 mm x 200 mm to 300 mm x 300 mm typically. They are made of Rigid PVC or other sturdy plastics. The block under-drain system can permit separate air scour, water backwash, simultaneous air and water backwash modes. The inherent design of under-drain eliminates uneven distribution of air and water even in large filter beds. Generally, the blocks are designed for filtration rates $25 \text{ m}^3/\text{m}^2/\text{hr}$. The water back wash upflow range is $20\text{-}90 \text{ m}^3/\text{m}^2/\text{hr}$. For simultaneous air-water wash the rate of flows can be designed as $70 \text{ m}^3/\text{m}^2/\text{hr}$. and $15 \text{ m}^3/\text{m}^2/\text{hr}$. respectively.

The filter under-drain system, when installed is designed for a net internal loading during backwash of either $2.93 \text{ m H}_2\text{O}$ or 200% of the maximum pressure at maximum backwash rates, whichever is higher. The filter under-drain system is designed to withstand a net downward loading of not less than $6.84 \text{ m H}_2\text{O}$.

The block under-drain system is extremely appropriate for their ability to operate the backwash in various modes, the media retainer plate eliminates the requirement for

supporting gravel for filter. Like nozzle with plenum chamber provision of the additional filter depth/height is avoided. Hence additional depth is available for various media configurations. The under-drain blocks are suitable to be adopted with high rate or very high-rate filtration viz., Mono media deep bed filters described later.

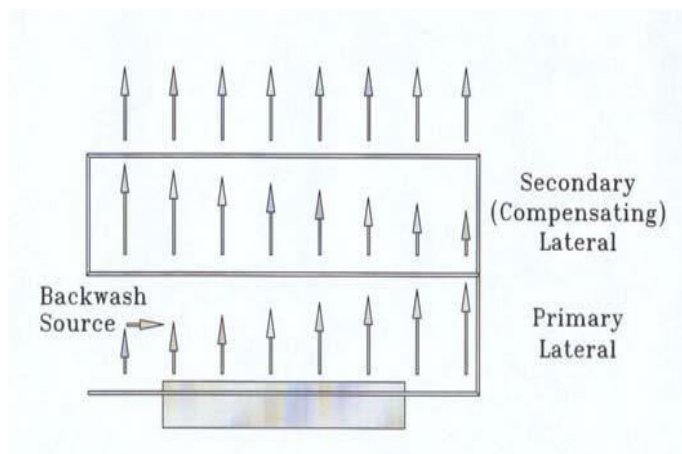


Figure 8.57: Flow distribution through dual-parallel blocks



Figure 8.58: Under-drain blocks

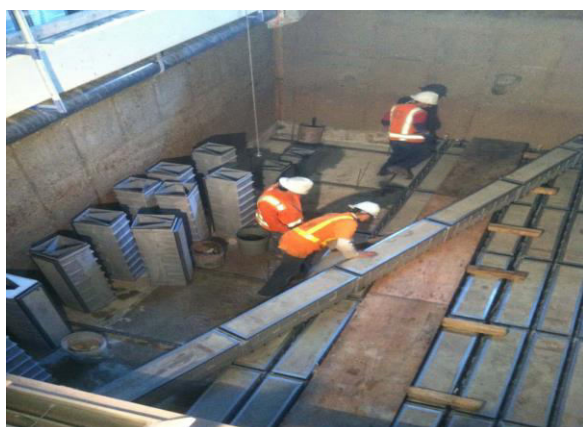


Figure 8.59: Installation of under-drain blocks



Figure 8.60: Filter floor with under-drain

8.7.3.8 Filter Gravel

Gravel is placed between the sand and the under-drain system to prevent sand from entering the under-drains and to aid uniform distribution of wash water. The gravel should accomplish both purposes without being displaced by the rising wash water. Sizes of gravel vary from 50 mm at the bottom to 2 mm at the top preferably with a 0.45 m – 0.60 m depth. The faster the rate of application of water, the larger the gravel size required. Reference may be made to IS: 8419 Part (1)-1977 (Reaffirmed 2001) for filter gravel.

The depth shall vary according to the type of filter bottom and strainer system used, except in the case of porous bottom where no gravel is required. Wheeler bottoms and other false

bottoms may be substituted for part of coarser layer of gravel. The filter gravel shall be as spherical as possible, hard, clean and uniform in quality and also shall not contain such impurities as dirt and clay. Size of gravel and depth of gravel layer shall be determined in accordance with the following rules:

- (a) For strainer or wheeler type under-drain system, gravel shall be of 2 mm minimum size, 50 mm maximum size and 0.30 m to 0.50 m deep; and
- (b) For perforated pipe under-drain system, gravel shall be 2 mm minimum size, 25 mm maximum size and 0.50 m in depth.

The filler gravel shall be classified by sieves into four or more size grades, sieves being placed with the coarsest on top and the finest at the bottom.

Suggested gravel configuration for Manifold-Lateral under-drain system

Table 8.8: Conventional Rapid Sand Gravity Filters

Gravel Size, mm	Gravel Depth, mm
2 – 5 (3 – 6)	10
5 – 10 (6 -12)	10
10 - 20	20
20 -50	20

8.7.3.9 Wash Water Gutters

Materials used for wash water gutters include concrete, plastic, steel or FRP. The gutters made with plastic and FRP should be properly anchored. The horizontal travel of backwashed water over the surface of the filter is kept between 0.6 m to 1.0 m before reaching the gutter. There are units without the troughs too. They have only main gutters where the up-flowing dirty water is pushed towards the gutters by horizontal flow side wash gutters. It is uneconomical to place wash water gutter against the side walls of the filter.

The upper edge of the wash water gutter (cross – troughs) should be placed at optimum distance above the surface of the sand. Too large the distance, then a large quantity of dirty water is left in the filter after the completion of washing. Too small a distance then there are chances of carryover of media during the back wash. Usually, the top of the cross troughs is 0.60 m to 0.70 m above top of the media. The bottom of the troughs (soffit) should clear the top of the expanded sand by 50 mm or more.

During air-scour the depth of water in the filters should be lowered 0.20 m to 0.25 m below the crest of the troughs. Where air and water are applied simultaneously precaution should be taken with designing the cross-trough heights.

The cross-troughs should be large enough to carry all the water delivered to it with at least 50 mm between the surface of the water flowing in the gutter and the upper edge of the gutter (free-fall). Any submergence of the gutter will reduce the efficacy of the wash. The

cross-troughs shall be made with the same cross-section throughout its length and without any longitudinal bottom slope towards the outlet end. The troughs are designed as free-falling weirs or spillways. For free falling rectangular troughs with level invert, the discharge capacity Q in m^3/s may be computed from the formula

$$Q = 1.376 bh^{3/2} \quad (8.27)$$

Where b is the width of the trough in m and h is the water depth in m and Q is in m^3/sec .

The cross-troughs discharge the dirty water in to the central gutter. The crest of cross-troughs shall be 50 mm to 75 mm below the top of the central gutter. The central gutter shall have parallel walls extending right from the bottom of the filter beds. In no case the central gutter shall have overhang above or in to the filter media. Generally, the height of the central gutter varies from 1.80 m to 2.1 m depending on the filter media configuration.



Figure 8.61: Filter bed under air-scour operation



Figure 8.62: Filter bed during water (hard) wash operation

(The cross-troughs and central gutter function is visible, Courtesy –Nagpur Municipal Corporation)

8.7.3.10 High-Rate Backwash

Back wash should be arranged at such a pressure that sand should expand to about 130% - 150% of its undisturbed volume. The pressure at which the wash water is applied is about 5 m head of water as measured at under-drains.

Normally, the rate at which wash water is applied, where no other agitation is provided, is 36 m (600 l/m/m²) for a period of 10 min. The tendency in design is towards higher rates of washing, primarily because of the larger sizes of sand being used, which require a faster application of water for equal expansion unless surface agitation by auxiliary means is provided. The maximum friction that particles which are free to move and expand can offer is their submerged weight in water. Increasing the flow, any further beyond this point, may lead to the carryover of the grains along with the wash water. For high-rate wash, the pressure in the under-drainage system should be 6 to 8 m with the wash-water requirement being 40-50 m/hr. for a duration of 6 to 10 min.

The supply of wash water can be made through an overhead storage tank or by direct pumping or by tapping the rising main of the treated water if the clear water motors are not overloaded. The capacity of the storage tank must be sufficient to supply wash water to two filter units, one at a time, where the units are more than 6. The tank filling pumps shall have flow rate to fill up the tank in two hours. Where the number of filters is less than six, the capacity of the storage tank shall be sufficient to supply wash water to one bed. However, in this case the tank filling pump flow rate to fill the tank in one hour.

The bottom of the wash water tank shall have minimum effective head of 8.0 m above filter under-drain. To find the actual head, losses in valves piping shall be added.

8.7.3.11 Surface Wash

The upper layer of the filter bed become the dirtiest and any inadequate washing will lead to the formation of mud balls, cracks and clogged spots in the filters. These troubles are overcome by adequate surface wash which can be accomplished by stirring the expanded filter bed mechanically with rakes, hydraulically with jets of water directed into the suspended sand or pneumatically with air, either during or more commonly before expansion. The latter two methods being common, are discussed.

(a) Hydraulic System

1. The fixed type surface wash system shall consist of pipes not less than 25 mm in diameter arranged vertically at intervals of 0.6 to 0.9 m. The lower ends of the pipes are to be situated to 0.1 m approximately over the sand surface and nozzles shall be located on the lower ends of pipes. An alternate fixed type may consist of piping horizontally arranged at intervals of 0.6 m approximately at a height of 0.05 to 0.1 m over the sand surface. The horizontal pipes shall be perforated at intervals of 0.3 m approximately and provided with non-clogging orifice units to prevent entry of the filter media,
2. The rotary type shall consist of rotating units suspended at a height of 50 to 75 mm at adequate intervals over the bed to provide complete coverage. Jet nozzles shall be located on side and bottom of arms and jet action of water causes the arms to rotate at a rate of 7 to 10 rpm.

(b) Air Wash System

In the air wash system, compressed air is used to secure effective scrubbing action with a smaller volume of wash water. The air may be forced through the under-drains before the wash water is introduced or through a separate piping system placed between the gravel and the sand layer. Though the former results in better washing, the gravel is likely to be disturbed. With the former procedure, free air of about 36 to 45 m/hr. (600 to 900 l/m/m² of the filter area) at 0.35 kg/cm² is forced through the under-drains until the sand is thoroughly agitated, for a duration of about 5 min following which, wash water is introduced through the same under-drains at a rate of 24 to 36 m/hr. (400 to 600 l/m/m² of area). On the other hand,

with the latter procedure, while water is forced through the under-drains, about the same volume of air is forced simultaneously through a separate piping. In the practice of backwashing employing conjunctive air and water wash, air is usually applied at a rate of 45-50 m/hr. and water at 12-15 m/hr.

Ensuring uniformity of air distribution is one of the most important issues when air and water are forced through the same set of laterals (air followed by water) in Conventional Rapid Sand Filter plants. The main air header entering into filter bed (above the top of water level) shall have equi-spaced vertical branches. The branches shall be connected to bottom header, connecting all the laterals through nozzles (through the concrete TEE or otherwise). The recommended velocities of air in the main header, vertical branches and nozzles feeding the laterals are 20 to 25 m/sec, 40 to 45 m/sec and 60 to 80 m/sec respectively.

8.7.3.12 Operation of Filters

Before starting a filter, it is backwashed at increasing rates until the sand bed has been stratified vertically by the wash water which carries various sizes of sand to different levels. The filter run of rapid gravity filters depends on several attendant factors. It is necessary to calculate the total loss of head in backwashing to arrive at the pump capacity and staging height of backwash water reservoir. The total loss of head includes loss due to expansion of sand, loss in orifices or under-drainage system, loss in incoming pipe and height of wash water gutter with respect to under-drainage system.

The loss of head immediately after washing should not exceed 0.2 to 0.3 m. The finer the sand or the greater the rate of filtration, the greater the initial head loss. The head loss builds as the filter grows dirty during a run. It is usual to allow a filter head loss of 1.8 to 2 m before cleaning such filters (also known as terminal head loss). Under no circumstances, a build-up of negative head within the filter media be allowed. The simplest arrangement to ensure this is to provide crest of filter outlet weir 0.15 m to 0.20 m above the top of the sand (media).

The duration of the washing process varies for different conditions of cleaning of the filter, sizes and character of filter media, rate of wash and the desired quality of filtrate. It should not normally exceed 10 min. In a properly operated plant, the quantity of wash water used should not exceed 2% - 3% of the total amount of water filtered. Lower amounts are possible where large quantities of wash water are involved and water is scarce. Adequate arrangement for 100% reuse or recycle of the wash water is recommended. Following the washing process, it is usually advisable to waste the first few minutes of flow through the filter, unless the quality of the filter effluent immediately following the wash may make this unnecessary. A turbidity of 1.0 NTU or less, measured by an accurate instrument, is the best criterion of the stability of effluent of a freshly washed filter. The test can be used to advantage in most plants.

The water standing on the bed at the close of wash should be clear with a turbidity not exceeding 10 NTU. In a well-designed and operated filter, there should be no air binding either during filtration or during washing; there should be no carry away of sand with the wash-water and the sand bed should settle down fairly uniformly without undulations. Formation of mud balls and their retention in the bed even after washing indicates poor performance. At the commencement of the filter run after a wash, the initial loss of head should not exceed 0.3 m.

8.7.3.13 Hydraulics of Filtration

The detailed hydraulics of filtration can be referred in Appendix 4.11.

8.7.3.14 Appurtenances

Filter appurtenances include manually, hydraulically or electrically operated sluice valves on the influent, effluent, drain and wash water lines; measuring devices such as Venturi meters; rate controllers activated by measuring device; loss of head and rate of flow gauges; sand expansion indicators; wash water controllers and indicators; operating tables and water sampling devices; and ejectors and sand washers; wash water tanks and pumps.

(a) Rate of Flow Controllers

The primary purpose of rate of flow controllers is to regulate the flows of liquids in the lines and specifically, in filter plant, to maintain at all times a uniform rate of filtration through each filter unit. Without these control features in the filter effluent lines, raw water will pass through the sand bed at different velocities, higher when the sand bed is clean and lower when coagulated deposit has accumulated on its surface.

Sudden changes of rate of flow also must be avoided if the filter medium is to be maintained in an unbroken and efficient condition. Any changes in rate must be gradual and predetermined maximum must not be exceeded. Such unfavorable operating conditions may be eliminated by the use of rate of flow controllers. These are normally employed to regulate the flow through filter beds, where filters are designed to be operated on “Constant head, Constant Rate” principle. The Rate setter is adjusted to the rated flow which flow over the outlet weir. The double beat float system, One in filter bed and other in rate control chamber is connected by a fulcrum and a lever. As the head-loss progressively increases, the rate of flow decreases. Both the floats get self adjusted to a constant level to ensure rated flow throw the control valve aperture.

The flow can also be controlled by means of a V-notch or a rectangular weir or a Venturi tube.

Mechanism of Flow controller:

Rate of flow controller may be either of double beat type or Venturi type. The latter type consists of a Venturi section, diaphragm chamber arrangement, valve

mechanism and casing, counter-weighted scale beam group and recovery outlet section. By virtue of the arrangement of the parts, straight line flow through the unit is simulated.

Water flowing through the Venturi section produces different pressures at the main and throat, due to the difference of velocities at these points. Since connections from the main and throat lead to the upper and lower halves, respectively, of the diaphragm chamber, these differential pressures are reflected directly on the piston, moving it a certain distance, dependent on the difference between the pressures being exerted. Since downward pressure on the top of the piston is greater than upward pressure from below, a downward pull that is balanced by the counter-weight on the long arm of the beam is transmitted to the scale beam. This balance of counter-weight and piston load regulates the valve opening and limits the maximum rate of discharge through the controller.

In filter operation, the controller, by virtue of its throttling action, uses up all the head due to the difference in raw and filtered water which is not required to overcome friction due to sand, piping, velocity head, etc., and as the loss of head through the sand increases, the head consumed by the controller diminishes by a corresponding amount. During the entire operation, therefore, the rate of filtration remains practically constant.

However, it must be emphasized that rate of flow controllers requires proper operation and maintenance to ensure that filtration is done at constant rate. These devices are getting progressively omitted and getting replaced with declining rate of filtration or constant rate filtration with influent splitting weirs systems.

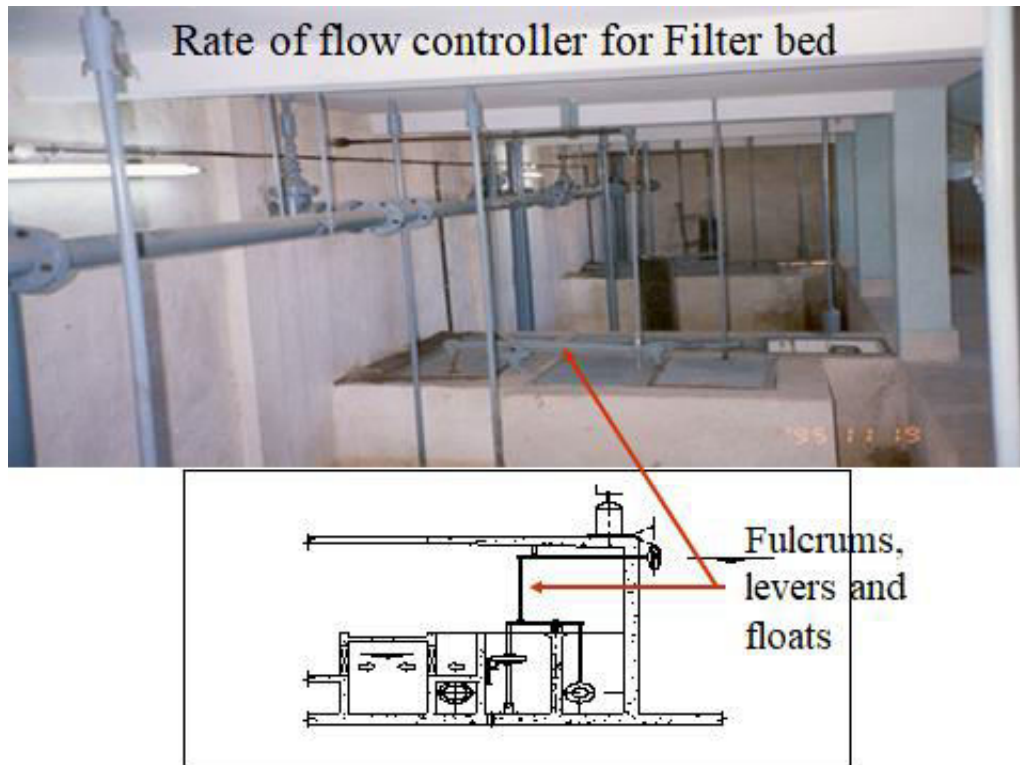


Figure 8.63: Rate of flow controller for filter bed

(b) Filter Gauges

Filter gauges are essential to the operation of the modern filter plant in order to measure accurately the rate of flow through each filter box and to determine the loss of head occurring at any given time during the filter run. Gauges are available in various combinations of rate of flow and loss of head, both indicating and recording or as single recording or indicating units.

These gauges use the float and mercury principle for the conversion of differential pressure into measurement of loss of head or rate of flow. The primary pressure differential producing device required for the rate gauge usually is the Venturi section of the effluent rate controller, connections to the high- and low-pressure sides of the gauge cylinder being made to the main and throat sections of the controller. The differential pressure for the gauge is the difference between the water level in the filter box and the pressure head in the effluent pipe, pressure connections being led from these sources to the high- and low-pressure gauge cylinder taps.

Piezometers (manometers) shown in Figure 8.64. The head-loss can also be measured using mechanical equipment as shown in Figure 8.65. These can also be used for the purpose, though they suffer from the disadvantage that they have to be cleaned from time to time. They are simpler, more positive and much less expensive than the conventional types of instruments.



Figure 8.64: Headloss Indicator (Manometer)



Figure 8.65: Rate of flow and head-loss indicator (Mechanical)

(c) Sand Expansion Gauges

Properly designed sand expansion gauges accurately determine the correct percent of sand expansion and the proper washing cycle for each filter bed. This form of gauge operates by means of a conical metal float.

The conical float, being counter-weighted to suit the specific gravity of the filter media moves upward and continually rests upon the surface of the media as it is expanded. The float is so designed that it will faithfully follow the surface of the media as it rises or falls in the filter during the washing cycle.

In some plants, the expansion of sand is not given emphasis and the fluidization is checked by means of probing with a vertical rod during the backwash cycle to see whether the probe will easily go down to the gravel.

8.7.3.15 Pipe Gallery

The influent, effluent, wash and waste water pipes together with rate controllers, appurtenances and pure water outlet chamber are placed in the pipe gallery. Galleries should be well designed to provide adequate space, ventilation drainage and easy accessibility to all pipe-work and other fittings. When filter beds are arranged in a single row, the pipe gallery is on one side of beds. When the beds are arranged in two rows, the pipe gallery is located in between. Pure or filtered water conduit or Channel is located in the pipe gallery or by the side of pipe gallery. The top slab of pure water channel is used as a lower-level walkway.

Upper-level walkway is provided at the top of filter box. The width of the walkway is normally same as that of pipe gallery. Minimum distance between upper-level walkway and lower-level walkway shall be 2.25 m to 2.50 m. In a manually operated plant, Upper-level walkway houses valve operating headstocks, wheels, and gear boxes. In automated plants, it houses Filter Operating Consoles. Pipe gallery and pure water channel is housed

Part A: Engineering Design

in building with headroom over upper-level walkway as 3.5 m to 4.0 m. Adequate lighting and ventilation shall be provided to the pipe gallery.

Normally filter annex building housing air blowers, back wash tank filling pumps etc. is adjacent to the Filter House Structure. In that case interconnecting staircases or walkways need to be provided to facilitate operator movement.

Table 8.9: Filter Port Velocities

Filter Port	Velocity (m/s)	Adopted in Maharashtra
Filter Inlet	0.9 – 1.8	1.2
Filter Outlet	0.9 to 1.8	1.2
Wash Water Inlet	2.4 to 3.65	3.0
Wash Water Outlet	2 to 2.4	2.5
Air Scour Isolation	20 to 25	25

Source: Water Supply & Waste Water Disposal by Fair & Geyer

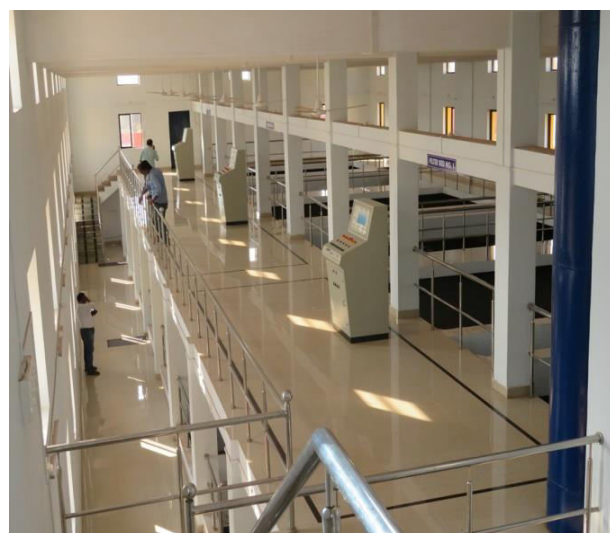
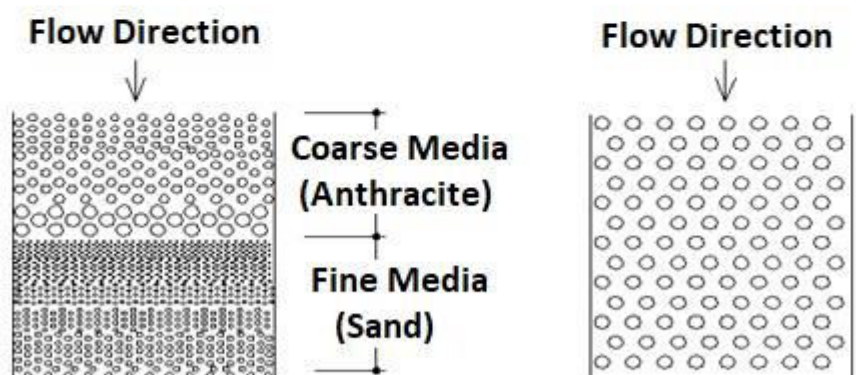


Figure 8.66: Pictures of Pipe Gallery and Upper-level walkway**8.7.3.16 Limitations of Rapid Sand Filters**

The inherent drawback of the rapid sand filtration system is the surface clogging tendency due to unfavorable stratification of sand medium. A rapid sand filter consists of a sand bed which becomes stratified after back washing. The size gradation is from fine to coarse with finest sand particles being at the top of bed. Since majority of the impurities are removed and stored in the limited pore space available in top sand layers, it leads to surface elapsing with relatively quicker buildup of head loss at higher velocities of filtration leading to the under-utilization of sand bed. Consequently, the rapid sand filters have been operated at lower filtration rates (around 5 m/hr.) with filter runs of the order of 24 hours. Another drawback of fine to coarse size gradation of filter medium is the possibility of poor filtrate quality resulting from the non-removal of finer floc particles which escape the top sand layers also break through the lower layers containing larger-size sand medium. This is also known as “Surface Filtration”.

Various approaches have been recommended to overcome the above limitations of the rapid sand filters. These include up-flow filtration, horizontal-flow filtration and dual-media, multimedia filtration and Mono media deep bed filters. Central to the development of these concepts is the principle of contacting the impurity-laden water first with the layers of filter medium having maximum pore size and pore space to accommodate the arrested impurities. As water travels deeper into the filter bed, it comes in contact with filter bed layers containing smaller pore sizes resulting in removal of even very fine floc particles. This leads to better-quality filtrate and greater utilization of lower layers to remove impurities. The dual media, multi-media and mono media filters which are being increasingly used can be operated at higher rates of filtration with production of higher quantities of filtered water of good quality per filter run compared to rapid sand filters. This is also known as “In depth filtration”.

**Fig - 8a Coarse to Fine Filtration****Fig - 8b Coarse to Fine Filtration
(Ideal Configuration)****Fig - 8c Coarse to Fine Filtration
(Dual or Multimedia Filtration)****Fig - 8d Mono - Grade
Deep Bed Filtration****Figure 8.67: Rapid Sand Filtration****8.7.3.17 Performance Capabilities of Rapid Sand Filters**

For rapid sand filters performance standards may be based on the following criteria:

- The filtrate should be clear with the turbidity of 1 NTU or less.
- The filtrate should be free from colour (with 3 or less on the cobalt scale).
- The filter runs should normally be not less than 24 hours with a loss of head not exceeding 2 m.
- For an efficient filter, the wash water consumption should not exceed 2 % of the quantity filtered in between washing (Calculated over one year of operation period).

Frequently Observed Problems in Filter Beds

Problems in filter operation and performance can arise from either poor design or poor operation. However, in the last two decades a lot of advances in the engineering design of

filters, filter controls and appurtenances have made water filtration an inherently stable, efficient and reliable unit treatment process.

Some potential filter problems are as follows.

- Surface clogging and cracking
- Short runs due to rapid increases in head-loss
- Short runs due to floc breakthrough and high effluent turbidity
- Variations in effluent quality with changes in applied water flow rate or quality
- Gravel displacement or mounding
- Mud balls
- Growth of filter grains, bed shrinkage and media pulling away from side walls
- Sand leakage
- Negative head and air binding
- Air leakage in to system
- Non uniform air scour operation
- Defunct Rate of flow Controllers

The solution to these problems is discussed in Part B of Manual: Operation and Maintenance

8.7.4 Rapid Gravity Dual Media Filters

The rapid gravity dual media filters are filters containing two media, normally coal and sand, and water is applied in downward direction under gravity.

A Constructional Features

The enclosure tank containing filter media is usually a rectangular box, made of concrete. The plan area of these filters may range between 40 m² and 200 m² with depths between 3.0 m and 3.75 m. The filter media is supported on gravel laid over top of the under-drainage system. In addition to the under-drainage system, used for collecting filtered water and distributing the backwash water, the tanks have troughs spanning across the length or width of filter for distribution of water to be filtered and for collection of wash water. The troughs remain submerged during filtration and their top edge is normally kept 600 mm above the filter medium to prevent loss of medium during backwash and to minimize the amount of dirty water left above the filter bed at the end of the wash.

The filters are commonly arranged in rows on one or both sides of a pipe gallery. The gallery houses the influent, effluent, wash water supply, wash water drainage piping, valves and other appurtenances including rate of flow controller. The pressure gauges to indicate head loss and Venturi meter or rate of flow recorder are also located above and/or below the gallery floor.

B Filtration Media

With a view to maintain coarse to fine gradation of pore sizes and pore volume with increasing depth of filter bed, two media of different density and sizes are chosen. However, dual media filtration & Multimedia filtration is seldom used now a days for public water supply. The top layer consists of a Lower density material like coal having larger particle size over a layer of higher density material like silica sand having smaller diameter particles. Since in India anthracite coal is not easily available, the coarse medium may consist of high-grade bituminous coal or crushed coconut shell which have been recommended for use after laboratory and field trials. (The bituminous coal variety available in India is of softer quality. The experience has shown that the coal particles get disintegrated in to powder form and cannot withstand the vigorous backwashing of media). The effective size (E.S.) of coal (specific gravity 1.4) is usually 1 mm (0.85-1.6 mm range) with uniformity coefficient (U. C.) of 1.3 to 1.5. Depths of 0.3 to 0.4 m have been reported to be satisfactory without excessive head loss build up and these depths can flocculate particles besides removing large flocculated impurities. The finer media-layer usually consists of 0.3 - 0.4 m thick silica sand (specific gravity 2.65) with effective size of around 0.5 mm (0.45 to 0.6 mm range) and uniformity coefficient of 1.3 to 1.5.

The basic principle in designing the dual media bed is to have coal as coarse as is consistent with solids removal to prevent surface blinding but to have the sand as fine as possible to provide maximum solids removal subject to the constraint that the finer sand should not be present in the upper layers after backwashing in appreciable quantity.

In addition to high grade bituminous coal, crushed coconut shell has been effectively used as coarse media in dual media filters. The size ranges from 1.0 mm to 2.0 mm with depths of 0.3 m - 0.4 m. The uniformity coefficient is below 1.5 and specific gravity 1.4. The sand used in conjunction with crushed coconut shell has effective size varying between 0.44 mm to 0.55 mm with uniformity coefficient below 1.5. The sand depths may vary between 0.3 m and 0.4 m. Water treatment plants with capacities ranging between 1 to 26 MLD have been constructed employing dual media filters using crushed coconut shell and sand. At some places imported Anthracite is used instead of bituminous coal.

Anthracite coal has been extensively used in dual media filters. It is recommended that 0.4 m - 0.75 m of anthracite coal of effective size of 1.0 mm to 1.6 mm (specific gravity 1.45 -1.55) be used above a sand layer of 0.15 m - 0.30 m. The effective size of sand may vary between 0.45 mm to 0.8 mm with 0.45 mm being preferred. The sand is silica sand with specific gravity of 2.65.

C Design of Media Depth and Media Sizes

The detailed design of media depth and media sizes can be referred in Appendix 8.12.

D Filtration Rates and Filtrate Quality

Dual media and multimedia filters have been successfully operated at rates of filtration ranging from 10 to 20 $\text{m}^3/\text{m}^2/\text{hr.}$ with acceptable filtrate quality. Filtrate turbidities are generally less than 1 NTU and coliform removal is around 95%.

It may be recommended to operate dual media filters at higher rates 10 to 12 $\text{m}^3/\text{m}^2/\text{hr.}$ to achieve filter run of 24 hr. or more.

The backwash rates of 42 to 54 $\text{m}^3/\text{m}^2/\text{hr.}$ (700-900 l/m/m^2) have been recommended to clean the filters. The higher back wash rates are essential as the media need to be expanded up to 30%. At the closure of back wash the hydraulic grading of the particles takes place. Coarse and lighter media stays on top. Fine and dense media settles to the bottom. It is extremely important to ensure expansion of media during back washing to ensure the media configuration. Like Rapid sand gravity filters, air scour is also employed prior to water wash.



Figure 8.68: Pilot plant for Dual Media Filters



Figure 8.69: Checking media expansion during backwash using sand expansion stick



Figure 8.70: Pictures of a Dual Media Filter Bed during internal erection and media loading

8.7.4.2 Multimedia Filters

The multimedia filters normally contain three media such as anthracite coal, silica sand and garnet sand with specific gravities being around 1.4, 2.65 and 4.2. The size of media may vary from 2 mm at the top to 0.15 mm at the bottom. A typical tri-medium filter may contain 0.45 m of coal with an effective size of 1.4 mm, followed by 0.23 m of silica sand of effective size of 0.5 mm and 0.08 m of garnet sand having an effective size of 0.3 mm. Media of polystyrene, anthracite, crushed flint sand, garnet and magnetite whose specific gravities are 1.04, 1.40, 2.65, 3.83 and 4.90 respectively are being tried.

8.7.4.3 Mono Media Deep Bed Gravity Filters

Mono media deep bed gravity filters (MMDB Filters) or Uniform Media Filters are shown in Figure 8.71 & Figure 8.72.



Figure 8.71: MMDB filters at Pench PH-II, Nagpur



Figure 8.72: "Coarse" and uniform OCW, media is used in MMDB filters

In these filters, the medium is normally sand. The recommended rate of filtration is same as adopted for Dual Media filters. The grain size of sand is larger and is usually 0.80 mm to 1.5 mm (E.S.) or more. To compensate for the greater porosity (larger pore sizes), the depth of the media is required to be increased up to 1.0 m to 2.0 m. The uniformity coefficient (U.C.) has to be low and is 1.25 to 1.30. An attempt is made to provide as uniform a medium as possible.

Fluidisation of these types of beds is difficult during backwashing as they require very high flow rate. Therefore, these beds are backwashed with combined (concurrent) air-water scour, and low-rate water wash. The sequence of washing is that first high-rate air scour is employed for 5 min (36 m/hr. to 54 m/hr), followed by combined air-water wash for 10 min (air-scour rate 36 to 54 m/hr and wash water rate of 15-25 m/hr.) and then by high-rate

water wash for 5 min (36 m/hr.). These filters require under-drain system of strainer nozzles with plenum chamber or Under-drain blocks as described earlier. The configuration of filter is same as Conventional rapid sand filter with central gutter and cross-troughs.

To carry out the dirt effectively to central or side gullet, some times during backwashing, horizontal flow of water is introduced at the top of the medium (cross flow). This is normally done using clarified water. The filtration rates and the filtrate quality are comparable with dual or multi-media filters. The rate of filtration is comparable with “Coarse to fine” media filters. These techniques were essentially developed in Europe.

8.7.4.4 Pressure Filters

A General

In pressure filters the removal mechanism is same as that of the rapid sand filtration. The filtration rate is high (6000-15000 l/m²/hr) and water is passed through the filter under pressure (3-7 kg/cm²) through a cylindrical tank, usually made of steel or cast iron, wherein the under-drain, gravel and sand are placed. They are compact and can be prefabricated and moved to site. Economy is possible in certain cases by avoiding double pumping. Pretreatment is essential. The tank axis may be either vertical or horizontal.

Pressure filters can be used for small capacity plants for removal of iron, manganese, etc. They can also be used for swimming pool water.

In vertical pressure filters the sand media is confined at the bottom and the pressurized water is applied uniformly at top surface of the media, so that the churning of the media is avoided.

B Disadvantages

Pressure filters suffer from the following disadvantages:

- a. The treatment of water under pressure seriously complicates effective feeding, mixing and flocculation of water to be filtered.
- b. In case of direct supply from pressure filters, it is not possible to provide adequate contact time for chlorine.
- c. The water under filtration and the sand bed are out of sight and it is not possible to observe the effectiveness of the back wash or the degree of agitation during washing process.
- d. Because of the inherent shape of the pressure filters it is difficult to provide wash water gutters effectively designed so that the material washed from the sand is discharged to waste and not flushed back to other portions of the sand bed.
- e. It is difficult to inspect, clean and replace the sand, gravel and under-drains of pressure filters.

- f. Because the water is under pressure at the delivery end, on occasions when the pressure on the discharge main is released suddenly, the entire sand bed might be disturbed violently with disastrous results to the filter effluent.

In view of these disadvantages, pressure filters are not recommended for community water supplies, particularly for large ones. They may be used for industrial needs and swimming pools.

8.7.4.5 Additional Modifications of Conventional Rapid Gravity Filters (Rate of Flow Control System without Mechanical Rate Controllers)

A Constant and Declining Rate Filtration

(a) Constant Rate Filtration by Influent Flow Splitting

In conventional rapid sand filters, constant rate of flow is maintained by installing a rate of flow controllers on the effluent line. These mechanical rate of flow controller devices are quite complex and hence, are difficult to maintain. They are high in initial and maintenance costs. Alternative systems have been proposed which are relatively simple to build, operate and maintain. These devices are progressively being discarded in practice. These are being replaced with constant rate hydraulic flow control arrangement.

One of the simplest methods is rate control by influent flow splitting which is depicted in Figure 8.73. The filter influent is divided equally among all the operating filters in parallel by means of a weir at each filter inlet. The size of the filter influent conduit/ channel is kept relatively large so that the head loss is not significant and the water level does not vary significantly along the length of the conduit/channel. This helps in maintaining nearly same head on each of the weir and filter influent is equally split among all the operating filters. The filtration rate is controlled jointly for all the filter units by the inflow feeding rate. At the beginning of filter run when a backwashed filter is put into service, the level of water in that filter is minimum. As the filtration proceeds and head loss builds up, the water level rises in the filter till it reaches the maximum permissible level above the filter bed, which may be, for example, equal to the level of influent weir. The filter is then taken out of service for backwashing

The advantages of this system include elimination of rate controllers and slow and smooth changes in rates due to gradual rise and fall of water level above filter bed with less harmful effects on filtrate quality in comparison to filters having rate of flow controllers. To completely eliminate the possibility of negative head in the filter and to avoid accidental dewatering on media, the effluent control weir must be located above filter media as depicted in the Figure 8.86. In such case, the crest of filter outlet weir is provided 0.15 m to 0.20 m above the top of media.

The only disadvantage of the influent flow splitting system is the additional depth of the filter box which is 0.5 m to 1 m more than in conventional filters. The total depth of water is from 3.50 m to 3.75 m.

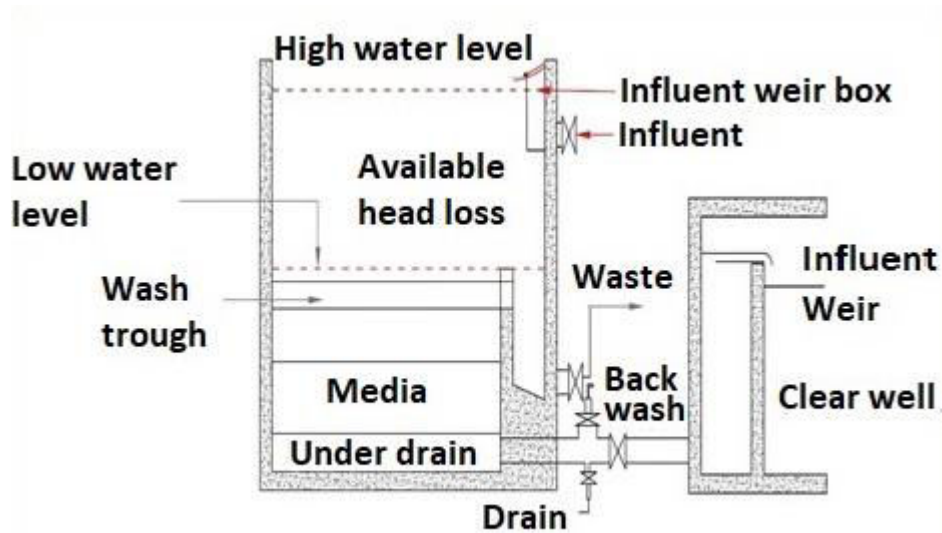


Figure 8.73: Gravity Filter Arrangements for Rate Control by Influent Flow Splitting

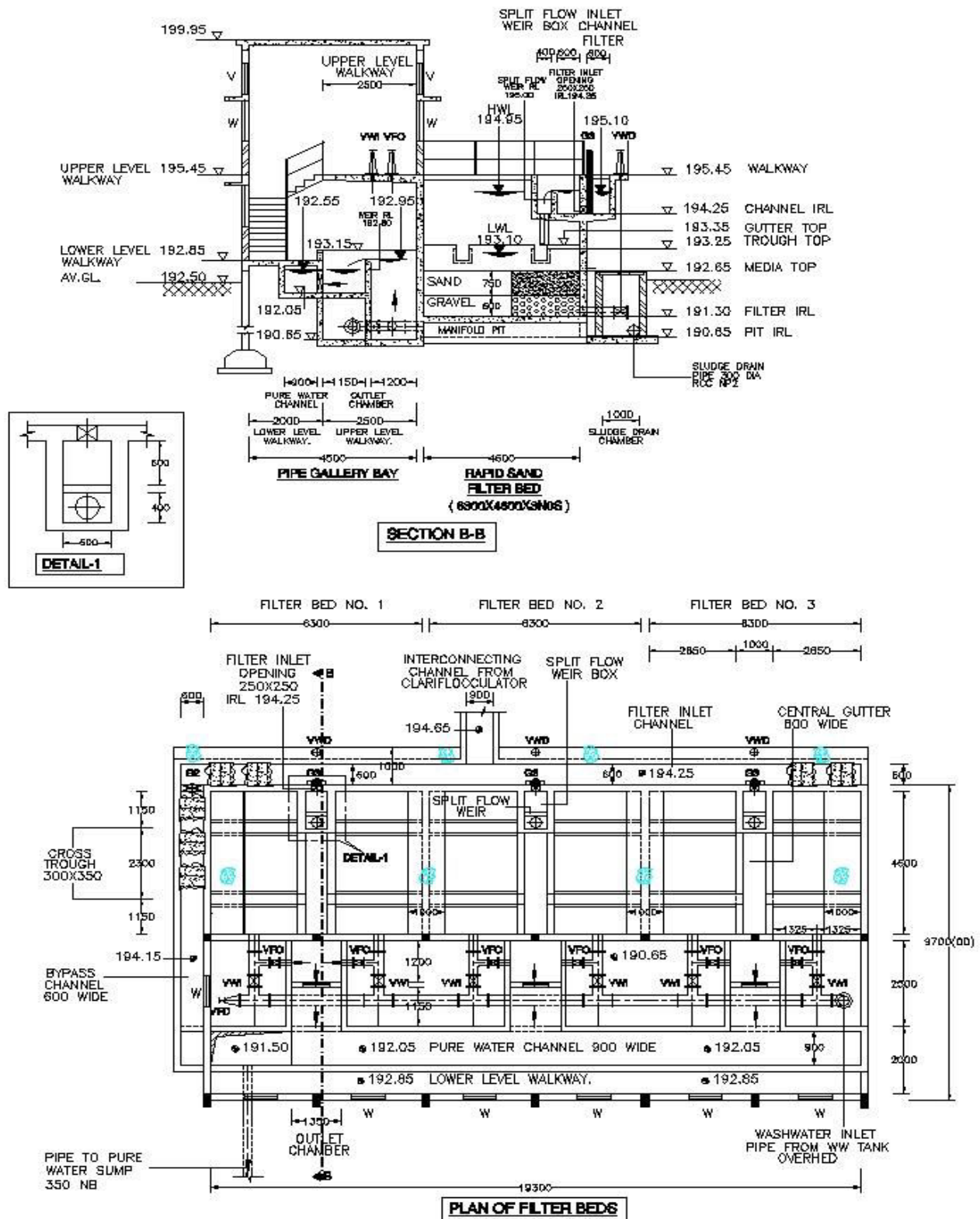


Figure 8.74: Constant Rate Filtration with Influent Spitting Weirs



Filter Inlet Channel and Inlet Weir
with chambers



Close-up of splitter weir and Chamber



Filter Outlet Chamber with weir, Weir
crest located above top of the media

Figure 8.75: Filter Chamber

(b) Declining Rate Filtration

This is also referred to as variable declining rate filtration. In this system, the filter influent enters below the low water level of the filters and not above as in the case of influent flow splitting system described in sub section 8.6.8. A relatively large influent header (pipe or channel) serves all the filters and a relatively large influent valve is used for each individual filter. This results in relatively small head losses in the influent header and influent valve and water level is essentially the same in all operating filters at all times. The essential features for variable declining rate filtration system are shown in Figure 8.76. No rate of flow controllers is used in this system also.

During the course of filtration by a series of filters being served by a common header, as the filters get clogged, the flow through the dirtiest filters decreases most rapidly. This causes redistribution of load among all of the filters increasing the water level providing the additional head needed by the cleaner filters for handling additional flow. Therefore, the capacity lost by the dirtier filters is picked up by the cleaner filters.

The advantages of this system include significantly better filtrate quality than obtained with constant-rate filtration, and less available head needed than that required for constant-rate operation. However recently a new concern has emerged for these types of filters. After the backwash of a bed, that bed tends to discharge more water. This is associated with higher initial turbidity of the filtrate. It is found that *Cryptosporidium* and *Giardia* cysts and oocysts escape in the filtered water (initial filtrate). Therefore, many advanced countries including the United States have discouraged its use in public water systems.

Another type of declining rate filtration is called "controlled-head" operation. In this type of filters, the filter effluent lines are connected to a common header. A fixed orifice is built into the effluent piping for each filter so that no filter, after washing, will take an undue share of the flow. The filtered water header pressure may be regulated by a throttle valve which discharges to a filtered water reservoir. Costly rate controllers are replaced with fixed orifices and, therefore, would make the units economical particularly in large water works involving batteries of filters. The quality of water produced by the declining rate filters and filters controlled by conventional rate controllers are reported to be almost the same. For equal durations of filter runs the total output per day from a declining rate filter is higher than that in the conventional one. In a group of filters operating at an average rate of $6 \text{ m}^3/\text{m}^2/\text{hr.}$, fixed orifice will be so designed that a recently cleaned filter will begin operation at $9 \text{ m}^3/\text{m}^2/\text{hr.}$, while the filter next in line for backwashing will have slowed down to about $3 \text{ m}^3/\text{m}^2/\text{hr.}$ Usually, the depths of filter boxes for declining rate filters are more than those for the conventional ones. These would permit longer filter runs and consequent reduced wash water requirements. The possibility of "break through" resulting in increased concentration of suspended solids in the effluent in filters with rate controllers is avoided in this system.

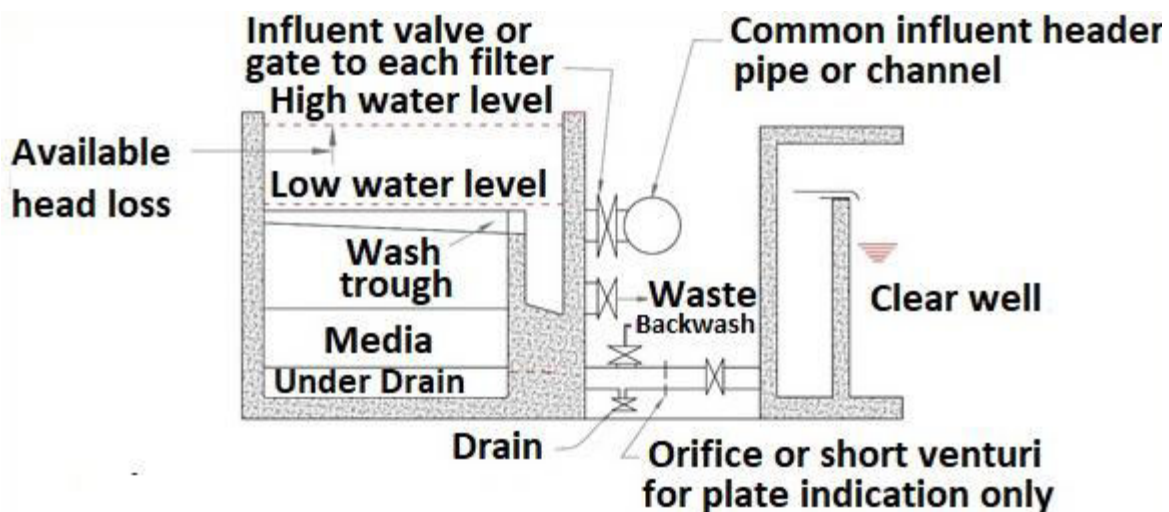


Figure 8.76: Gravity Filter Arranged for Variable Declining Rate of Filtration

8.7.5 Up-Flow Filters

Up-Flow Filters are not practiced now a days in India. In up-flow filtration, the water is passed under pressure in an upward direction through the coarser medium followed by finer medium. Thus, larger size suspended solid particles are first retained in the larger interstices of the lower part of bed and as the water percolates upwards, it receives a progressive polishing until it emerges in a fully filtered condition at top of the filter bed. Thus, the entire depth of media is made effective in removal of suspended solids and as a result low head loss and longer filter runs could be expected. Besides, many other advantages are claimed for up-flow filtration such as elimination of the rate controller and absence of negative head. Unfiltered water can be used for washing filter since the first few minutes of flow through the filter after washing has to be necessarily run to waste. Filter depths as low as 0.6 m and as high as 1.5 m have been successfully used. Although wash water rate and consumption are greater per wash cycle than the conventional filter, wash water used as a percentage of finished water is much less because of low loss of head and long filter runs. But initially compressed air scouring is desirable to dislodge the impurities collected in the lower portions of the bed. The only disadvantage is fluidization of the top fine layers of the sand bed which results in the deterioration of the filtrate quality. Complete bed fluidization occurs when the head loss equals the depth of bed. Control of head loss is much more significant than the upward velocity through the filter. It is desirable that the hydraulic gradient through the up-flow sand bed is restricted to 0.6.

8.7.6 Automatic Valveless Gravity Filters

This type of filters are used in typical industrial installations. These filters operate without butterfly valves, pilot mechanisms, rate controllers, gauges and air compressors. They have two compartments, the filtering section and wash water storage compartment. As the incoming water is admitted to the filter, a head gets built upon the top of the sand and causes the water level to rise in the backwash pipe. When the water level reaches the top of the loop, usually designed with a 2 m differential, syphon action is started and back-washing begins at the required rate of 30 to 42 m³/m²/ hr. Wash water flows from the storage tank up through the sand bed and is discharged through the backwash pipe. A syphon breaker ends the wash cycle. The filter washes itself automatically, at the proper time at a given loss of head, without any mechanical instrument or operating tables. There is no maintenance from a mechanical standpoint of view. These filters are useful for low turbidity waters and for small installations.

8.8 Disposal and Recycling of Filter Back Wash Water

The NRW values in water supply system as mentioned in chapter 2 in Part A of this manual is 15% out of which 3% NRW is considered as water lost in WTP. In order to minimize this loss the recycling of backwash water is adopted.

Earlier this backwash water was disposed off into natural water bodies. Recently due to acute water shortages, scarcity, and as per CPCB norms, recycling of backwash waste

water has been adopted by many plants/utilities. Reuse of filter backwash water is recommended for Water Treatment Plants above 5 MLD capacity for economy and operation and maintenance. (Appendix 8.13)

Filter backwash water should be recycled to inlet chamber or upstream of flow measuring flume or weir and to be done as uniformly as possible. It should not increase more than 2 to 6% of plant flow rate. It requires a wash water recirculation tank of adequate volume (Equivalent to the storage capacity of Elevated Wash Water Tank) to receive and store the filter backwash water. For plants less than 30 MLD capacity one tank is sufficient. For larger plants one tank with two compartments shall be provided. If the air grid or agitator is provided for the tank, then it eliminates the need for manual de-silting.

The recycle rate needs to be worked out carefully taking into consideration the required redundancy. Normally recycle pumping hours are 16 to 20. To reduce the recirculation shock loading (to main stream) it is better to have more number of pumps of smaller capacity. Vertical or horizontal centrifugal pumps can be provided for recycle. Normally the location of wash-water recirculation tank is at the lowest available contour at site. Because of the plant hydraulics it is experienced that abnormally high freeboards are required to be provided to receive the waste water from filters by gravity.

There is no need for clarification or settling of recycle back wash water if the quantity is limited to 2% to 6% of total flow. Excessive recycle rates may lead to change the characteristics of raw water. In some countries there is a practice of disinfecting the recycle water by chlorine or ozone to eliminate Giardia Lamblia cysts, Cryptosporidium oocysts and to inactivate viruses.

The sludge from clariflocculator is not mixed with backwash water from filters. Therefore, the entire backwash water should be recycled. This will not only minimize the losses through treatment plant, but also enhance the removal efficiency. Though, theoretically outlet to this backwash recycle tank is not needed, however, in exigencies (if both the pumps, working as well as standby fail to operate) the outlet for this tank is needed to prevent them to overflow/ flood the pump floor.



Figure 8.77: Wash Water Recycle Tanks

8.9 Direct Filtration by disc filters and low pressure membranes

Surface water bodies may contain high level of contaminants due to discharge of untreated sewage and conventional water treatment methods may have limitations. In such situation direct filtration by disc filters, screen filters and low pressure membranes can be used. Such plants have been installed at BHEL, Tiruchirapalli where the old Lime-Alum clariflocculation-RSF plant has been replaced by direct filtration by this methods as shown in Figure 8.78. The 14 MLD Direct filtration WTP at BHEL, Tiruchirapalli for raw water drawn from river Cauvery through infiltration wells and in footprint of 14 m * 12 m

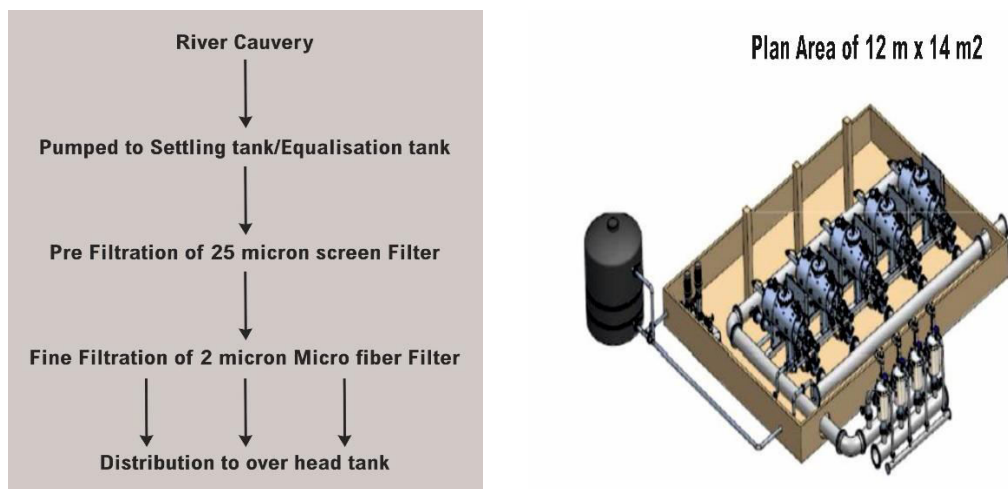


Figure 8.78: The 14 MLD Direct filtration WTP at BHEL, Tiruchirapalli

Another similar installation is at the North Bay WTP of 79 MLD (Figure 8.79) using low pressure membranes as under which avoids the problems of coagulant handling and sludge issues

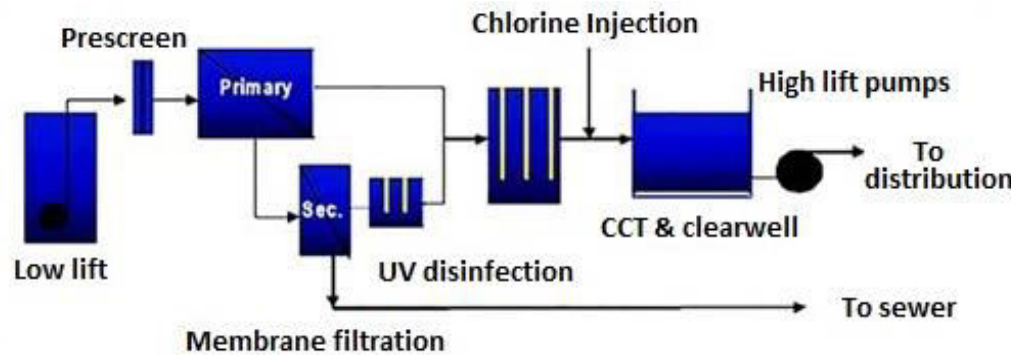


Figure 8.79: North Bay WTP of 79 MLD

8.10 Disposal of Wastes and Sludge from Water Treatment Processes

Disposal of wastes from the water treatment plants has become increasingly important with the availability of technology and the need for protection of the environment. The present CPCB norms do not allow more than 100 mg/L of Total Suspended Solids (TSS) to be disposed into water bodies. Treatment of waste solids adds to the cost of construction and operation of treatment plants. Sludge from water treatment plants comprise of:

- Sludge from sedimentation of particulate matter in raw water, flocculated and precipitated material resulting from coagulation, or residuals of excess coagulant dosage, plankton etc.
- Sludge generated from reactions of coagulants. However, these quantities are minor.
- Sludge generated from lime-soda softening processes and silica removal.
- Wastes from regeneration processes of ion exchange softening treatment plant containing cations of calcium, magnesium and unused sodium and anions of chlorides and sulphates originally present in the regenerate.
- The primary focus of the discussion in this chapter is on sludge generated by plants with surface water sources (a). The plants based on lime-soda softening plants (c) and ion-exchangers (d) are not very common in public water supply systems or they are limited to very small plants in India.

The reuse of alum/PAC sludge is possible, however, techno-economic feasibility of recovery of alum/PAC/Alumina is only feasible for exceptionally large plants.

8.10.1 Disposal Methods

As a result of presence of suspended, precipitated solids, and reactions with many coagulants results in generation of sludge. The quantity of sludge generated is directly proportionate to Total Suspended Solids (TSS) in raw water. Most of the sludge (90% to 98%) is effectively removed from water in different clarification units such as simple sedimentation tanks, conventional clarifiers, Clariflocculators, Tube settlers, Lamella clarifiers and the like. The separated sludge contains almost 92 – 95% water and just 5 – 8% insoluble / inert solids and precipitates.

Clarifiers or Settling tanks shall be a single point source to take out the sludge from the system for treatment and disposal. The dirty backwash water shall have a separate stream for recycle as described above sections. It shall not be mixed with clarifier sludge for further thickening.

Various types of clarifiers and settling tanks produce sludge of different consistency (also called as an underflow) depending on TSS in raw water. The sludge consistency from clarifiers/settling tank varies from 2% to 6% W/V. For designing the thickening system, it is recommended to adopt value of 2%. The principle of sludge treatment is progressive thickening of the sludge from dilute turbid liquid to moist solids cake with consistency 20% to 25% W/V.

Equation for sludge/solids mass balance for clarifier (As well as for Thickener) is as follows.

Total suspended solids (TSS) rate at Inlet to system (mg/hr.) =
Total suspended solids (TSS) rate at outlet of the systems (mg/hr.) + Total suspended solids (TSS) rate in sludge flow (mg/hr.)

Simplified as follows to find Sludge flow rate in cum/hr.

$[\text{Inlet flow (m}^3/\text{hr.)} \times \text{Inlet TSS (mg/L)}] =$
 $[\text{Inlet flow (m}^3/\text{hr.)} - \text{Sludge flow (m}^3/\text{hr.)}] \times \text{Outlet TSS (mg/L)}] + [\text{Sludge flow (m}^3/\text{hr.)} \times \text{Sludge TSS (mg/L)}]$

8.10.2 Gravity Sludge Thickener

This sludge from clarifiers or settling tanks shall be further thickened by using Gravity thickener. In some cases, it can be directly led to Centrifuges or Sand drying beds. For thickener, the design criteria are 60-80 kg/m²/d (Max up to 100 kg/m²/d) based on the solids loading (dry basis) and is followed by sludge dewatering devices. A thickener is similar to central feed, central rake drive clarifier having depth of 3.5 m to 4.0 m. The rake tip speed is 1.0 m/min to 2.0 m/min. For inlet sludge (underflow) consistency of 2% (W/V), the thickener underflow consistency is in the range of 3% to 6% W/V. For design purpose 5% W/V value is recommended. The lime sludge have shown better dewatering characteristics. The P.E. dosing is practiced at the inlet to thickener to improve up on the quality of underflow. The thickener rake has normally pickets mounted on it (Vertical angles or supported flats). Theses pickets help in releasing the water interlocked between the solid's particles in the hindered settling (bottom sludge) zone. The scraping mechanism has to be coupled with high-torque speed reduction mechanism.

The underflow (thickened sludge) coming out of thickener shall be further de-watered by processing it through, Sand Drying Beds, centrifuges or presses. The consistency of dry cake coming out of filter presses can be up to 20 to 25%, while that from centrifuges is up to 15-20%. (Freezing and thawing method) improves the sludge consistency. However, it is

not very relevant to our Public Water Sector systems. The supernatant from the sludge thickener is recycled to the backwash water recycle tank/ sump.

A typical illustration of Gravity Sludge Thickener is shown in Figure 8.80 and an actual photo is shown in Figure 8.81.

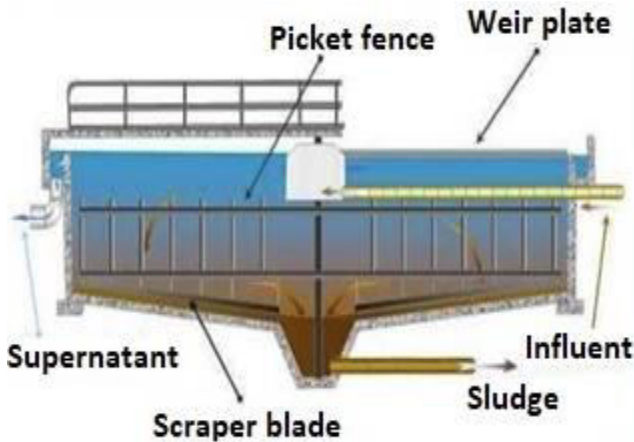


Figure 8.80: Typical illustration of Gravity Sludge Thickener



Figure 8.81: Picture of a Sludge Thickener (Source – Sangli, Maharashtra)

8.10.3 Sludge dewatering devices

A variety of dewatering devices/systems are employed, including but not limited to:

1. Sludge drying beds (SDBs)
2. Continuous Decanter centrifuges
3. Batch type filter presses
4. Continuous filter presses
5. Batch type lagoons

The IS 10037-2 (1983, reaffirmed 1996): Requirements for sludge de-watering equipment, Part 2: Vacuum filtration equipment which lays down requirements for vacuum filtration equipment used for sludge de-watering may be referred to.

8.10.3.1 Sludge Drying Beds (Sand Beds)

The sand (sludge) drying beds for dewatering occupy a huge area. Their performance is also affected by climatic conditions. They have their use in dry, arid climate, where rainfall is less or moderate. The area requirement for these beds is based on solids loading criteria of 150 to 400 kg/m²/d. The drying cycle is designed for 6 to 7 days. However, both these criteria are dependent on the weather and climate of the region. The depth of sludge application is restricted to 200 to 400 mm. The sand depth is 300mm to 400mm with E.S. varying from 0.3 mm to 0.75 mm and U.C. in between 3 to 4. The sand is supported over gravel/pebble of depth 200 mm to 250 mm. Perforated laterals of cement pipes are preferred to convey the filtrate up to central manifold. The bottom slope of the beds is

1:100 towards manifold pit. With bright sunshine for at least 8 – 10 hours a day, and with proper sand gradation, it is possible to achieve solids concentration of 40 - 60% in the dried cake. Sludge Drying bed are recommended for plants less than 5 MLD capacity.

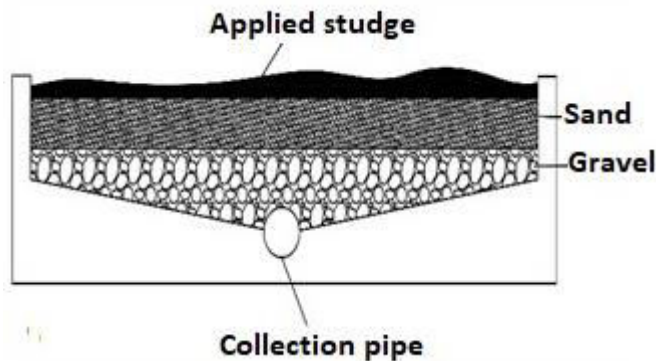


Figure 8.82: Schematic section of Drying Bed



Figure 8.83: A Typical sludge Drying Bed
(source – Lucknow, U.P.)

8.10.3.2 Continuous Decanter Centrifuges (For Large Plants)

A decanter centrifuge operates on the principle of solid-liquid separation based on enhanced difference in specific gravity, when a centrifugal force is applied on the mixture. A decanter centrifuge separates solids from liquid phase in a continuous process. The denser solid particles are pressed outwards against the rotating bowl wall, while the less dense liquid phase forms a concentric inner layer.

Different 'dam plates' are used to vary the depth of the liquid – the so-called pond – as required. The centrifugal force compacts the solids and expels the surplus liquid. The dried solids then discharge from the bowl. The clarified liquid phase or phases overflow the dam plates situated at the opposite end of the bowl.

The Continuous Decanter Centrifuge are shown in Figure 8.84 and actual photos are shown in Figure 8.85 and Figure 8.86

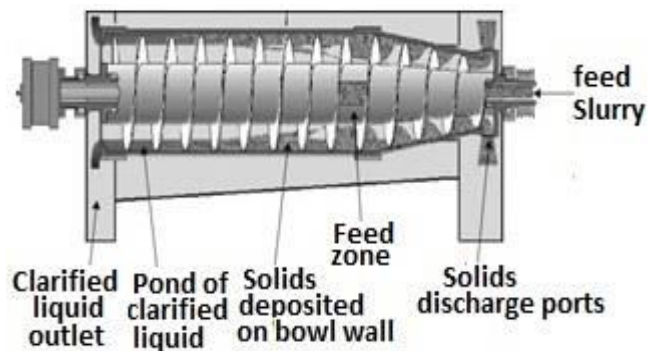


Figure 8.84: Continuous Decanter Centrifuge



Figure 8.85: Centrifuge Building with “chute” (source- Sangli, Maharashtra)



Figure 8.86: Centrifuges located on the first floor (source- Sangli, Maharashtra)

8.10.3.3 Batch type filter presses

Filter presses are also used on a large scale for dewatering of sludge. These machines are also referred to as ‘plate and frame’ filter presses. A filter press is a batch operation, fixed volume machine that separates liquids and solids using pressure filtration. A slurry is pumped into the filter press and dewatered under pressure. It is used for water and wastewater treatment in a variety of different applications ranging from industrial to municipal. Hence this press operates in ‘cycles’. Cycle time typically consists of filling, filtration against pump pressure, further draining through air pressure, plates opening, sludge removal (may be manual or auto) and re-assembly. At times, even water wash is also provided for further extraction of soluble substances, if required by the application. Typical cycle time can vary from about 2 hours and may extend upto even 8 hours or beyond, depending on drainability of sludge, applied pressure, type of filter cloth selected etc.

Though most of the dewatering takes place under feed pressure, for further drying of sludge (cake), many times, pressurized air is passed through the plates, to forcefully expel the moisture. In such cases, a higher dryness cake can be obtained. Designer must remember that this is a batch operation, hence once the filter press is loaded, there must be enough storage volume available for sludge storage, or a second working press is to be provided.



Figure 8.87: Batch Type Filter Press

8.10.3.4 Continuous Filter Press

A continuous filter press is a unit in which sludge is squeezed between two continuous travelling porous belts (sometimes referred to as 'felts'). Multiple rollers are provided which progressively press the felts, thereby draining moisture from the sludge. Dewatered sludge is then discharged from the final roller or scraper. Usually, to keep the felts clean, and to remove any sludge particles stuck to felts, pressure water is sprayed from both sides of both the felts, so that porosity of filter media is maintained.

The advantage of such belt filter press over the plate and frame press is the continuous operation. However, such continuous filter presses demand higher maintenance, higher fresh water consumption (for felt washing). Continuous filter presses can be more compact as compared to batch type filter presses, and usually there is no standby required, except few essential spares. Typical 3D and sketch illustration of Continuous Filter Press are shown in Figure 8.88.

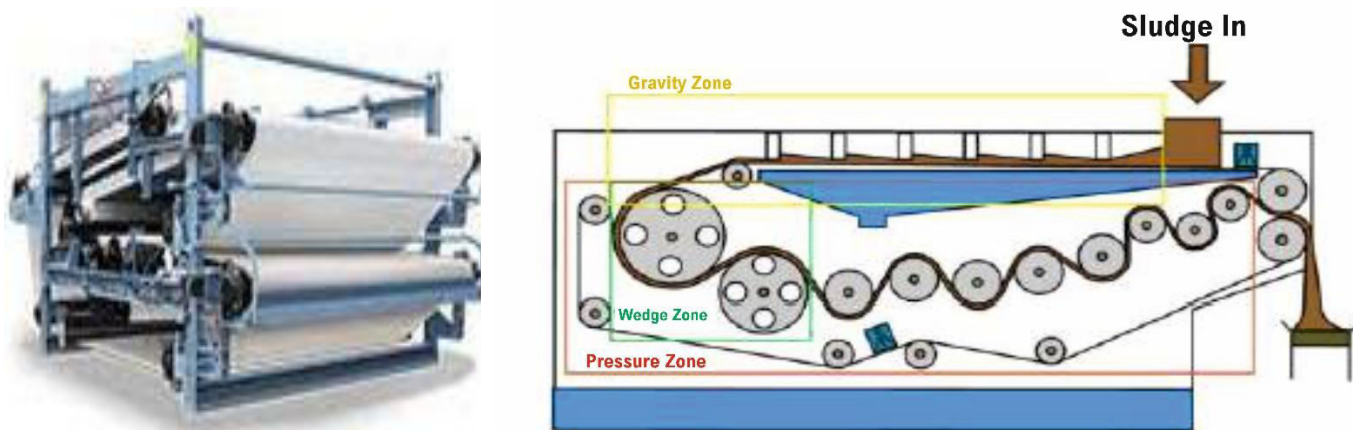


Figure 8.88: Typical 3D (Left) and Sketch (Right) illustration of Continuous Filter Press

8.11 Treatment Plant Hydraulics

As far the Public Water Treatment Plants are concerned the flow through the main process units shall be based on gravity. Generally, it is observed the hydraulic losses in the treatment plant vary from 4.50 m to 6.50 m, bigger the plant more the head-loss. In the steeply sloping or hilly terrain the head-loss is likely to increase due to lack of space to suit the designed hydraulic gradient. The reason for restricting the total losses through the plant is conservation and saving of overall energy.

As far as safe practices are concerned provision of following minimum losses are recommended.

Rapid Mix Unit (Flash Mixer) to Clarifier	= 0.40 m
Clarifier to Filter Bed	= 0.40 m

Filter Pure Water Channel to Pure Water Sump = 0.40 m.

The treatment plant hydraulics comprises of Open channel and piping (closed conduits) losses, Valves, Gates, pipe specials, free-falls and terminal head-loss of filter beds. For Open channels and drainage systems it is recommended to use the Mannings Formula. For Piping and Closed conduits, it is recommended to use the Hazen-Williams Formula. Valves, Gates and Pipe Specials are governed by $h = K (V^2 / 2 \times g)$, where h = losses in m, V is the velocity in m/s, g is the gravitational constant and K is the coefficient or the multiplying factor. The value of K may vary slightly in different reference books; however, it is important that when one value is selected, it shall be uniformly applied to the entire system.

As a good engineering practice, it is recommended to design open channels, piping and closed conduits for 20% overloading factor over the rated design capacity. (Please note that process units are not to be sized for overloading) It is recommended that open channels shall be designed for maximum velocities in the range of 0.6 to 1.0 m/s under rated flow condition. Generally, the FSL (Full Supply Level) of Pure Water Sump or CWR (Clear Water Sump) shall not be below the FGL (Finished Ground Level). However, it may not be possible in a few exceptional cases. The PW Sump and CWR shall be provided with an overflow arrangement, either a bell-mouth pipe or sharp crested weir.

Whenever there are two units provided in parallel the flow shall be divided equally by providing split flow weir arrangement on the upstream in order to ensure equal distribution. This i.e., especially true for the Rapid Mix Unit (Flash Mixers) and Clarifiers. Free-falls in the plant shall be provided judiciously to avoid the flooding of upstream units.

Layout of Water Treatment Plants

Normally Water Treatment Plant Complex apart from the process units include ancillary structures like Filter Annex Building, Chemical (Coagulant, etc) House and Store, MCC / Switch Room, Chlorination Room, Tonner Yard, Generator Room, Transformer Yard, Office, Staff Quarters and security cabin etc. Many a times it includes elevated Service Reservoir or MBR also. The entire plant complex should be planned in a such a way that process and hydraulic structures are located in sequential and logical way to reduce piping and interconnecting channels. Due consideration should be given to the convenience of operator movement. All the structures should be interconnected by walkway at the operating level. The roads and pathways should be provided on the ground, interconnecting all the units. It is advisable to club together main treatment units structures on one side and waste recycle and treatment structures on the other side. All ancillary structures should be located to complement the process and hydraulic structures with respect to their functions. The overall layout should be aesthetically pleasing by providing garden in the vacant spaces and with trees on the plot boundary.

Ideally a site with mild slope is most suitable to ensure minimum excavation. The ideal Soil Bearing Capacity is 10 to 15 MT/m². As far as possible sites with exposed hard rock, Black Cotton soil and water-logged Sites should be avoided. The plant should be located near

the community and not in the isolated places for better supervision and control. However, in practice such ideal conditions are rarely available especially in the hilly terrain. In many cases adequate area is not available due to cost constraints. There also could be obstructions at site like existing structures, buried pipelines and conduits, Overhead HT Cables, road, rivulets etc. Hence it becomes a challenging job for the designer engineer. The design engineer needs to study the contours (1m interval) of the land and Hydraulic Flow Diagram to locate various units in an economical manner to prepare the Layout Plan. A very general rule is to place the structures with higher elevations on higher contours and structures with lower elevations on lower contours so that their bottom slabs are on a sound foundation. Wherever there is a double pumping scheme (raw as well as pure water) the engineer has some freedom to select the levels of the structures. But the constraints start building up when gravity flows (to and from the plant) are necessitated by the overall scheme hydraulics. The site selection becomes extremely critical if it is a double gravity scheme (raw as well as pure water).

The Layout of the plant complex should take in to account the future expansion of the plant. Micro planning involves routing the buried pipelines, plant drainage, site drainage, routing of piping, cable trays, planning interconnections, headroom clearances, operator safety etc. Adequate architectural features should be incorporated in the plant buildings to impart the prominence and importance of the plant to the community.



Figure 8.89: Augmentation of Gangtok W/S Scheme, Selep, Sikkim (2014)

Figure 8.90: Layout of a Small Capacity Plant of 10 MLD capacity on a fairly leveled site (Double pumping scheme)

Figure 8.91: Layout of a Medium capacity Plant of 25 MLD on a steeply sloping hill (Raw water by pumping, P.W. Supply by gravity)

Figure 8.92: Layout of a Medium Capacity Plant of 50 MLD capacity with hard rock as strata (Double pumping scheme)

Figure 8.93: Layout of a Large Capacity Plant of 168 MLD with severe space constraints

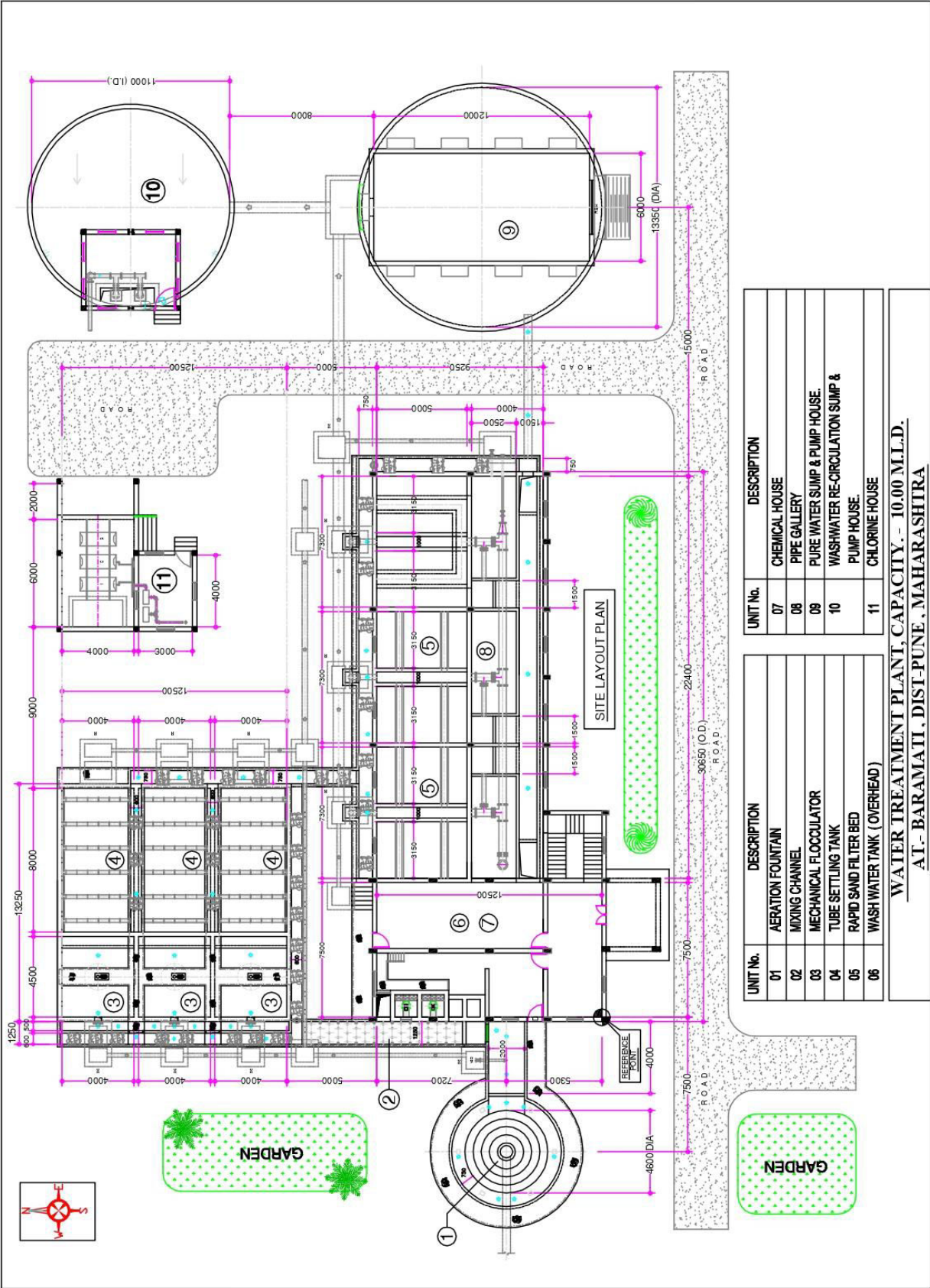


Figure 8.90: Layout of WTP Capacity 10 MLD at Baramati, Dist Pune, Maharashtra

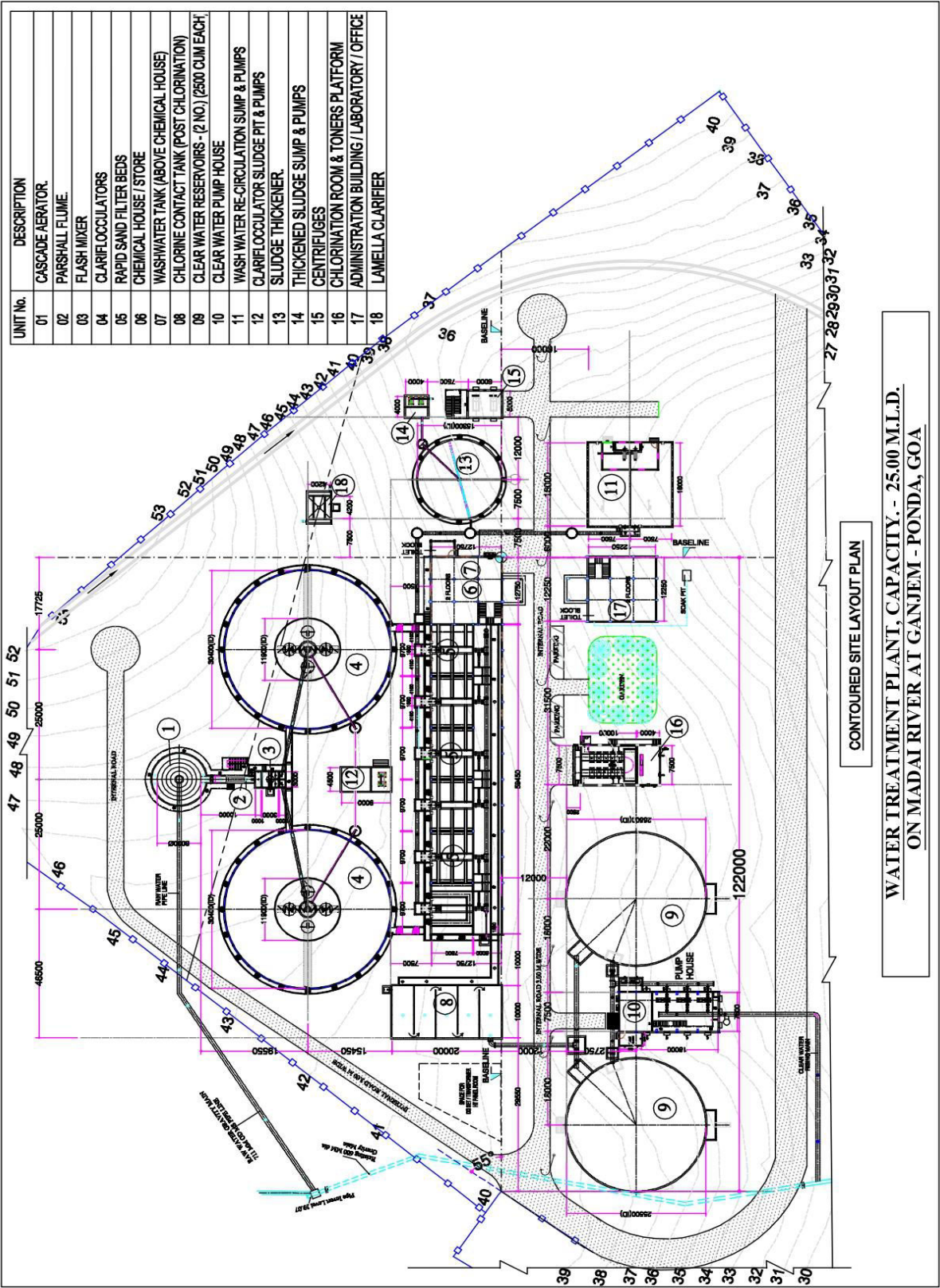


Figure 8.91: Layout of WTP Capacity 25 MLD at Ganjem, Ponda, Goa

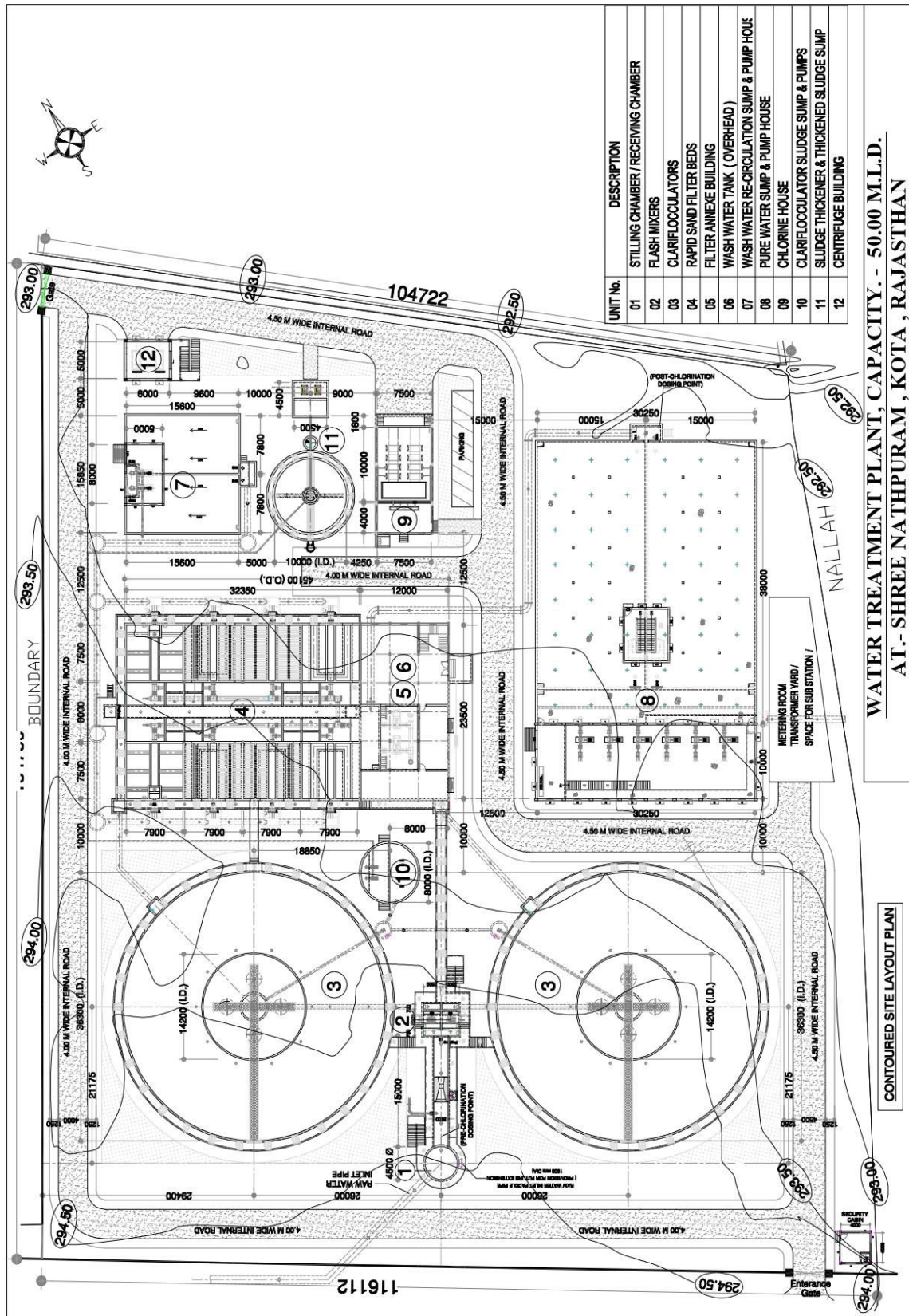


Figure 8.92: Layout of WTP Capacity 50 MLD at Shreenathpuram, Kota, Rajasthan

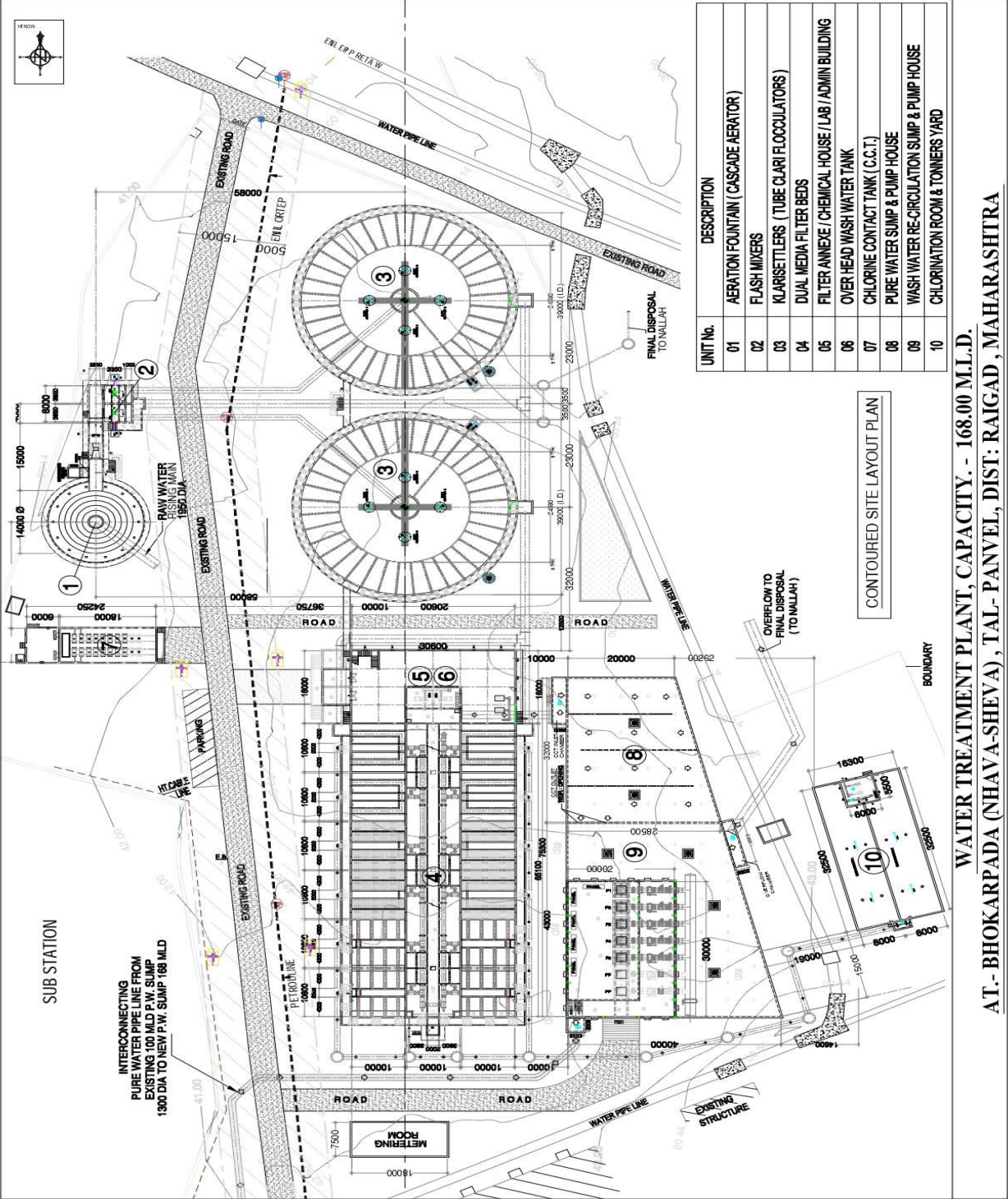


Figure 8.93: Layout of WTP Capacity 168 MLD at Nhava-Sheva, Dist. Raigad, Maharashtra

Approximate land area requirement for Conventional Water Treatment Plants consisting of Cascade Aerator, Parshall Flume, Flash Mixers, Clariflocculators, Rapid Sand Gravity Filter Beds, Filter Annex Building, Store Room, Wash Water Recycling Tank, Sludge Thickener, Centrifuges, MCC Room, Generator Room, Transformer Yard etc.

Table 8.10: Approximate land area requirement for Conventional Water Treatment Plants

S. No.	Plant Capacity (24 hr basis) (MLD)	Area in sqm	Area in acre (average)
1.	1.0	1000-1200	0.30
2.	5.0	2500-3000	0.70
3.	10.0	4500-5000	1.15
3.	20.0	8,000 -10,000	1.75
4.	50.0	20,000-22,000	3.00
5.	100.0	25,000-27000	6.50
6.	200.0	60,000-70,000	16.00

8.12 Augmentation or upgradation of Existing Water Treatment Plants

Augmentation of the existing plant can be considered if the life of the WTP has not approached to its design life (30 years) but there is disproportionate increase in demand and there is constraint of space to construct the new structures.

Augmentation can be achieved either by addition of new plant and/ or by retrofitting all the units in the existing plant with the appropriate technologies such as installation of Plate/ tube settlers, high rate gravity filters, disc filters, low pressure submersible membranes, etc.

Upgradation of Existing Treatment plant can be considered and redesigned by retrofitting the existing units if there is change in raw water characteristics and the existing plant is not able to produce treated water of desired quality. Such as in case of increase in nitrate due to discharge of wastewater upstream.

The existing water treatment plants can be augmented (capacity wise) or upgraded (quality wise) by retrofitting the conventional process units with high rate and advanced unit processes (described earlier). Simultaneously hydraulics of the plants is also required to be managed for the enhanced capacity. The cost of the augmented plant (say to double the capacity) is 60% to 70% that of a new plant. In addition, there is saving in the precious land cost too. The augmented plant can be operated with the existing manpower. There is only marginal increase in electrical power. During retrofitting the machinery of the existing plants gets overhauled, increasing the longevity of the plant. Generally, it has been experienced that the existing conventional plants consisting of Flash Mixers, Clariflocculators and Rapid Sand gravity filters can be augmented to twice their original capacity or more.

Conceptual Design considerations:

(a) Flash Mixers and Flocculators: Velocity gradient (G) and Detention time (t) are the governing factors for the design. By enhancing the flow through the unit, the detention time reduces. To supplement this, velocity gradient needs to be increased to main the " Gt " value as per the norms.

(b) Clarifiers or Clariflocculators: The surface loading rates of Tube Settlers four to five times that of conventional clarifier. Hence these units can be retrofitted with tube

settlers to enhance the capacity. Usually, part of the outer annular zone of clarifier is partly covered with the module.

- (c) **Rapid Sand Gravity Filter Beds:** The rate of filtration of Dual Media Gravity Filters and Mono Media Deep Bed Filters is two to three times that of Rapid sand Filters. These filter beds can be retrofitted with high-rate filter media configuration.
- (d) **Hydraulics:** The interconnecting piping, channels, inlet & outlet ports if required need to be redesigned to maintain the head-loss between any two units same as per the existing plant. Inherent redundancy in the sizing of the existing piping and channels need to be utilized judiciously.



Figure 8.94: Augmentation of clariflocculator by installation of Tube Modules installed (Navi Mumbai, Maharashtra)



Figure 8.95: Retrofitting of a rectangular tank with tube settler (Selep, Gangtok, Sikkim)



Figure 8.96: Upgrading of Existing conventional filter beds with mono media deep bed filters (Pench II, OCW, Nagpur)



Figure 8.97: Upgraded 75 MLD WTP to 123 MLD at Madurai by inserting tube settlers in clarifier rims and changeover of RSFs to Dual Media Filters

8.13 Prefabricated Packaged water Treatment Plants

These are installed mostly in small urban town and village water supply schemes which have less than 5 MLD capacity. These are constructed in an environment where skilled manpower is not available. These schemes are mostly executed by the local contractors and they require guidance on all aspects of construction. In-situ concrete or civil plants take long time to construct and execution is error prone. For such small plants, pre-fabricated packaged plants are recommended, where the quality of construction can be assured in the controlled environment. The emphasis is on to provide time tested unit processes with least mechanical or rotary parts. These shall be fabricated in mild steel plates of 6mm thickness. The inside of the plants (surface coming in contact with water) shall be provided with protective lining of FRP with 5mm thickness. From outside the surface shall be painted with primer and two coats of zinc-based paint. These plants shall be designed to be operated manually. Size of the units shall be such that these can be transported by standard trucks easily. Only Pure Water Sump, Pump House and Control Room shall be constructed in RCC in-situ construction. Based on experience, life of such plants can be considered as 15-20 years.

As per the practices of the State of Maharashtra (various W/S Organizations), one of which is shown in Figure 8.98, following unit processes and design criteria are recommended.

- (a) Rapid Mix weir or 90° V-notch with flow measurement, mixing channel
- (b) Pebble (Gravel) bed, Plastic Tetrapod or conventional paddle type flocculator with detention period of 15-20 min
- (c) Tube/Plate Settling tank with hopper, with surface loading rate less than $6.5 \text{ m}^3/\text{m}^2/\text{hr}$.
- (d) Rapid Sand Gravity Filters with rate of filtration $5.5 \text{ m}^3/\text{m}^2/\text{hr}$.
- (e) Disinfection by Calcium or Sodium hypochlorite (bleaching powder)



Pre-Fab Packaged Water Treatment Plants in Maharashtra (Cap 0.50 to 3.0 mld)

Figure 8.98: Pre-Fabricated Treatment Plants

8.14 Computer-Aided Optimal Design of Water Treatment System

The advent of computerized design systems/ methodology has drastically changed the design process of water supply system including water treatment plant design. Many companies have developed in-house design programs for optimizing every step of water treatment plant design.

The water treatment process involves hydraulics, Electro-mechanical as well as instrumentation and are interlinked. The optimal design aided by iterative process of computer programming, helps in adhering to all the design/ code specifications and come out with a best possible size of various components and machinery.

It is recommended to use Computer-Aided Optimal Design to construct the plant which ultimately will be useful in effective operation and maintenance including automation and SCADA. Details of SCADA can be referred from Chapter 14 in Part A of this manual.

Computer aided design of water treatment plants only generate sizing and losses calculations. However, in practice the designer must submit the Site Layout, Hydraulic Flow Diagram and General Arrangement Drawing of all the units showing all the detailing not generated by the design program.

Appendix 8.1: Design of Spray Type Aerator (Removal of Iron & Manganese)

1. Problem Statement

Design a spray aerator given the following data:

- (a) Design flow = 250 m³/hr.
 Pipe used = 70 mm diameter 'B' Class C.I. Pipe with a C value of 100
 S = 3.60 m/1000 m
 V = 1.35 m/s
- (b) Iron present in raw water = 1.8 mg/L
- (c) Saturation concentration of O₂ at 28° C = 8.92 mg/L.
- (d) Aeration constant (to the common base) at 28 °C = 70 cm/hr.

2. Design Criteria

- (a) Nozzle dia. usually 10 to 40 mm - spaced in the pipe at intervals of 0.5 to 1.0 m
- (b) Nozzles are usually tilted 3° to 5° to the vertical to avoid interference due to falling water.
- (c) Nozzle discharges should be uniform as far as possible. Variation in no case should be greater than 5% i.e., the discharge ratio between the first and the last nozzle, should not be less than 0.95 (a variation 2 to 5% may be allowed).
- (d) Velocity of water in the aerator pipe should be between 1 and 1.5 m/s.
- (e) Pressure required at the nozzle varies from 2 to 9 meter of water (usually 7m).
- (f) Discharge ratings per nozzle vary from 300 to 600 l/m,
- (g) Aerator area should be 1.25x 10⁻³ to 3.75 x10⁻³ m² per m³/day of design flow.

3. Solution

- (a) Design flow = 6000 m³/day.
- (b) Assuming 25 mm diameter nozzle with an inclination of 3° to the vertical, diameter of one drop is 25 mm.
- (c) Iron present in raw water = 1.8 mg/l.
- (d) Permissible limit of iron in treated water = 0.1 mg/L.
- (e) Iron to be removed = (1.8 - 0.1) mg/L = 1.7 mg/L.
- (f) $\text{Fe}^{+4} + 3\text{O}_2 = 2\text{Fe}_2\text{O}_3$
- (g) 1 mg/L of Fe requires $1.7 \times 96/224 = 0.7286$ mg/1 of O₂.
- (h) By applying 'Gas absorption equation" (Sub-Section 8.3.2) in the form

$$\text{Log}_{10} [(C_a - C_o) / (C_a - C_t)] = K.At/V$$

Where;

$C_a = 8.92$ mg/1 at 28°C, $C_o = 0.0$ mg/1.

$C_t = 0.73$ mg/1, $K = 70$ cm/hr.

Part A: Engineering Design

$$\frac{A}{V} = \frac{6}{d} = \frac{6}{2.5} \left(\frac{1}{\text{cm}} \right)$$

Therefore,

$$\log_{10} \frac{8.92}{8.19} = \frac{70}{60 \times 60} \times \frac{6}{2.5} \times t = \frac{7}{150} t$$

$$t = \frac{150}{7} \times \log_{10} \frac{8.92}{8.19}$$

$$t = \frac{150}{7} \times 0.042$$

$$t = 0.9 \text{ seconds}$$

Say $t = 1$ second & small case

$$t_r = \text{time of rise} = t/2 = 0.5 \text{ seconds}$$

V = nozzle velocity and α = inclination to horizontal.

$$V \sin \alpha = g \cdot t_r$$

Therefore,

$$\begin{aligned} V &= (g \cdot t_r / \sin \alpha) \\ &= (980 \times 0.5) / \sin (90 - 3)^\circ \\ &= (980 \times 0.5) / \sin 87^\circ \\ &= 4.91 \text{ cm/s} \end{aligned}$$

4. Number of nozzles

Assuming:

N = No. of nozzles required

$$q = \text{Discharge through each nozzle} = C_d \times V \times a$$

Where;

C_d = Coefficient of discharge = 0.9 (assuming)

V = nozzle velocity = 4.91 mps

a = nozzle area = $(3.14/4) d^2$

d = diameter of nozzle = 25 mm

Therefore,

$$\begin{aligned} \text{Discharge through "N" number nozzles} &= N \times C_d \times V \times a \\ &= N \times 0.9 \times 4.91 \times (3.14/4) \times [25 \times 10^{-3}]^2 \text{ m}^3/\text{sec.} \end{aligned}$$

But design flow i.e., discharge through N nozzles = $6000 \text{ m}^3/\text{day}$

$$N \times 0.9 \times 4.91 \times (3.14/4) \times [25 \times 10^{-3}]^2 \times 60 \times 60 \times 24 = 6000 \text{ m}^3/\text{day}$$

$$\therefore N = 32$$

$\therefore$ Nozzles required = 32 Nos. of 25 mm dia. each.

5. Spacing of Aerator Pipes

Radius of spray = $V \cos \alpha \times 2t_r = 4.91 \cos 87^\circ \times 2 \times 0.5 = 0.257 \text{ m}$

Assuming wind velocity = 8 km/hr.

Wind Drag = $C_d \times V_w \times t$ (assuming $C_d = 0.6$)

$$= 0.6 \times [(8 \times 10^3) / (60 \times 60)] \times 1 = 1.33 \text{ m}$$

∴ Minimum spacing required = Radius of Spray + Wind drag

$$= 0.257 + 1.33$$

$$= 1.587 \text{ m} \quad \text{say} = 2 \text{ m apart}$$

6. Arrangement of nozzles

Nozzles are fixed on 4 rows of pipes as shown below;

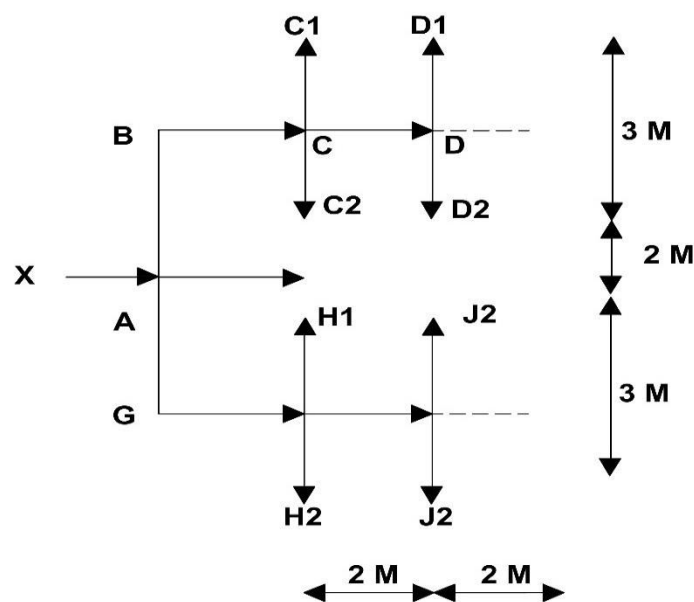


Figure 1

No. of nozzles in each pipe = $32 / 4 = 8$

Providing a spacing of 0.3 m c/c of nozzles and spacing in two adjacent rows and in staggered position.

Provide 4 pipes each of length 3m at a spacing of 2 m.

Allowing 2m space on all the sides, the size of the aerator tray will be 8m x 6m,

Checking

Aerator pipes enclose an area of $2 \times (3 \times 2) = 12 \text{ m}^2$;

Area provided per m^3/day of design flow = $12 / (250 \times 24) = 2.0 \times 10^{-3} \text{ m}^2/\text{m}^3/\text{day}$ of design flow.

(O.K. since it is between 1.25×10^{-3} to $3.75 \times 10^{-3} \text{ m}^2/\text{day}$ of design flow)

7. Uniformity in distribution

The uniformity in distribution of water is maintained by arrangement of aerator pipes as in figure 1 above.

$$\text{Discharge through each pipe} = (250 \times 24)/4 = 1500 \text{ m}^3/\text{d}$$

Assuming h = head loss at each nozzle

$$V = C_v \sqrt{2gh} = 4.91 \text{ m/s}$$

$$h = (4.91)^2 / [(0.9)^2 \times 2 \times 9.81] \text{ (Assuming } C_v = 0.9)$$

Assuming variation of head = 2%

$$m_1 = \frac{\text{discharge through last nozzle in the pipe}}{\text{discharge through first nozzle in the pipe}} = 0.98$$

$$H = h (1 - m_1^2) = 1.52 (1 - 0.98^2) = 0.06 \text{ m}$$

Head loss in the pipe for gradually diminishing flow = $H = 0.06 \text{ m}$;

Corresponding head loss for uniform flow = $H_u = 3H$

$$= 3 \times 0.06$$

$$= 0.18 \text{ m (per aerator pipe length);}$$

$$\text{Head loss /1000 m} = (0.18 / 1.5) \times 1000 = 120$$

8. Design of pipes and head losses:

The arrangement of pipe is shown in Figure 1. The aerator pipes are so chosen that the velocity remains within 1 to 1.5 m/s and corresponding head losses for pipes (C.I.) are calculated and are shown in the following table 1;

Table 1

Pipe Section	Design Flow (m ³ /d)	Length (m)	Diameter (m)	Velocity (m/s)	Head Loss (m)	Total Head Loss (m)
1	2	3.0	4	5	6	7
AB	3000	2.5	200	1.1	0.01	0.025
BC	3000	2.0	200	1.1	0.01	0.020
C ₁ C ₂	1500	3.0	125	1.42	0.03	0.090
CD	1500	2.0	125	1.42	0.03	0.060
D ₁ D ₂	1500	3.0	125	1.42	0.03	0.090
Total						0.285

$$\begin{aligned} \text{Total Head loss} &= 0.285 + 10\% \text{ for valves and specials} \\ &= 0.314 \text{ m} \end{aligned}$$

$$\text{Say} = 0.32 \text{ m}$$

$$\begin{aligned} \text{Head at 'A'} &= \text{Terminal head} + \text{Total Head loss} \\ &= 1.52 + 0.32 = 1.84 \text{ m} \end{aligned}$$

Appendix 8.2: Design of Mechanical Rapid Mix Unit

A. Problem Statement

Design a mechanical rapid mix unit using following data:

- | | |
|---|---|
| (a) Design flow to be treated | = 250 m ³ / hr. |
| (b) Detention time | = 30 s (20 - 60 s) |
| (c) Ratio of tank height to diameter | = 1.5:1 (1 - 3:1) |
| (d) Ratio of impeller diameter to tank diameter | = 0.4:1 (0.2 - 0.4:1) |
| (e) Rotational speed of impeller | = 120 rpm (>100 rpm) |
| (f) Velocity gradient | = 600 s ⁻¹ (>300 s ⁻¹) |
| (g) Assume temperature of 20° C. | |

B. Solution**(a) Determine dimensions of tank**

$$\begin{aligned}\text{Volume} &= \text{Flow} \times \text{detention time} \\ &= 250 \times (30/3600) = 2.083 \text{ m}^3\end{aligned}$$

Diameter of the tank, D, is calculated from:

$$(\pi/4) D^2 (1.5D) = 2.083$$

Therefore, diameter of tank = 1.20 m

And height of tank = 1.80 m

Total height of the tank = 2 m which will provide a free board of 0.2 m

(b) Compute power requirements

$$\begin{aligned}\text{Power spent, } P &= \mu G^2 \cdot (\text{Volume of tank}) \\ &= 1.0087 \times 10^{-3} \times (600)^2 \times 2.083 = 756 \text{ watts}\end{aligned}$$

$$\begin{aligned}\text{Power per unit volume} &= 756/2.083 = 362.94 \\ &= \text{say } 363 \text{ watts /m}^3\end{aligned}$$

$$\text{Power per unit flow of water} = 756/250 = 3.02 \text{ watts/ m}^3/\text{hr. of flow}$$

Determine dimensions of flat blades and impeller

$$\begin{aligned}\text{Diameter of impeller} &= 0.4 \times \text{tank diameter} \\ &= 0.4 \times 1.2 = 0.48 \text{ m}\end{aligned}$$

$$\begin{aligned}\text{Velocity of the tip of impeller} &= (2\pi \cdot r \cdot n/60) \text{ m/s} \\ &= [(2\pi \times 0.48)/2] \times (120/60) \text{ m/s} \\ &= 3.02 \text{ m/s (O.K.)}\end{aligned}$$

Determine the area of blades A_p of impeller by the equation.

$$\text{Power spent} = (1/2) \cdot C_D \cdot \rho \cdot A_p \cdot V_r^3$$

Part A: Engineering Design

Assuming Newtons coefficient of drag; $C_D = 1.8$ for flat blades and relative velocity of paddle,

V_r , as three fourths of the tangential velocity of the tip of the blade,

$$756 = (1/2) \times (1.8 \times 1000 \times A_p) \times [0.75 \times 3.03]^3$$

$$A_p = 0.072 \text{ m}^2$$

Provide 6 blades of size 0.1 x 0.12 m

- (c) Provide 4 Nos. of baffles of length 1.9 m and projecting 0.10 m from the wall of the tank to reduce vortex formation.
- (d) Provide inlet and outlet pipes of 200 mm diameter.

Appendix 8.3: Design of Clariflocculator

A. Problem Statement

Design clariflocculator using following data and design criteria:

- (a) Desired average outflow from clariflocculator = $250 \text{ m}^3/\text{hr}$
- (b) Water lost in de-sludging = 2% (flow coming from clariflocculator)
- (c) Design average flow = $(250 \times 100) / (100-2)$
= $255.1 \text{ m}^3/\text{hr}$.
- (d) Detention period = 30 minutes
- (e) Average value of velocity gradient, G = 40 s^{-1}

B. Components to be Designed

A circular clariflocculator is to be designed having vertical paddles. The water enters through a central influent pipe and is fed into the flocculation zone through parts. The effluent from flocculation zone passes below the partition wall dividing the flocculator portion and the clarifier portion. The clarified effluent is collected by a peripheral effluent launder. The components of clariflocculator to be designed include the influent pipe, the flocculator, the clarifier and the effluent launder.

C. Design of Influent Pipe (Central Shaft in Concrete)

Assuming a velocity of 0.60 m/s

$$\text{Influent pipe diameter} = \sqrt{[(255.1 \times 4) / (3600 \times 0.6 \times \pi)]} = 0.39 \text{ m}$$

Provide central shaft diameter = 0.60 m I.D. (This is minimum diameter to be provided for central shaft)

The O.D. of the shaft will be $0.60 \text{ m} + 0.20 \text{ m} + 0.20 \text{ m} = 1.0 \text{ m}$ (Considering shaft wall thickness as 0.20 m)

D. Design of Flocculator

$$\text{Dimensionless Parameter, } G.t = 40 \times (30 \times 60) = 4.8 \times 10^4$$

This is acceptable as $G.t = 10^4 \text{ to } 10^5$ for alum (coagulant)

$$\text{Volume of flocculator} = (255.1 \times 30) / 60 \text{ m}^3 = 128.55 \text{ m}^3$$

Provide a water depth of 3.5 m , same as SWD of clarifier

$$\text{Plan area of flocculator} = (128.55 / 3.5) \text{ m}^2 = 36.44 \text{ m}^2$$

Let D be the diameter of flocculator and D_p the diameter of the inlet pipe. Then,

$$(\pi/4) \cdot (D^2 - D_p^2) = 36.44$$

$$(\pi/4) \cdot (D^2 - 1.0^2) = 36.44$$

$$D = 6.88 \text{ m, say } 6.90 \text{ m}$$

Provide a tank diameter of 6.90 m. (I.D.)

Part A: Engineering Design

The O.D. of the flocculation tank will be 6.90 m + 0.15 m + 0.15 m = 8.2 m (Considering wall thickness as 0.15 m)

E. Dimensions of Paddles

$$\begin{aligned} \text{Total power input to flocculator, } P &= G^2 \mu (\text{vol.}) \\ &= (40)^2 \times [0.89 \times 10^{-3}] \times [\pi \times (6.9)^2 \times 3.5/4] = 186.37 \text{ watts} \end{aligned}$$

$$\text{Power input} = \frac{C_D \times \rho \times A_p \times (V-v)^3}{2}$$

(V – v) is velocity differential and is 0.75 (V)

Where;

$$\begin{aligned} C_D &= \text{Newtons coefficient of drag, } 1.8 \\ \rho &= \text{Density of water at } 25^\circ \text{ C, } 997 \text{ kg/ m}^3 \\ V &= \text{Velocity of the tip of blades} \\ &= 0.6 \text{ m/s (recommended range 0.3 - 0.6 m/s)} \\ v &= \text{Velocity of water at tip of blade} \\ &= 0.25 \times (V) = 0.25 \times 0.60 = 0.15 \text{ m/s} \\ (V - v) &= (0.60 - 0.15) \\ &= 0.45 \text{ m/s.} \end{aligned}$$

$$\begin{aligned} 186.37 &= \frac{1.8 \times 997 \times A_p \times 0.45^3}{2} \\ A_p &= 2.28 \text{ m}^2 \end{aligned}$$

Ratio of area of paddles to cross-sectional area of flocculator

$$\begin{aligned} &= \frac{A_p}{(D - D_p) \times h} \\ &= \frac{2.28}{(6.9 - 1.0) \times 3.5} = 0.11 \text{ or } 11\% \end{aligned}$$

This is acceptable as it is within the limits of 10 to 25%

Provide 8 Nos. of paddles of height 1.9 m and width of 0.15 m

Two shafts will support eight paddles, each shaft supporting 4 paddles.

The shaft will be at a distance of $(6.9 - 1.0)/4 = 1.48$ m from the outer diameter of central shaft.

The paddles will rotate at a rpm of 5.

Distance of paddle edge, r, from the centre line of vertical shaft is given by the equation.

$$\begin{aligned} V &= (2. \pi . r . n) / 60 \\ 0.6 &= (2. \pi . r \times 5) / 60 \\ \therefore r &= 1.14 \text{ m} \end{aligned}$$

Let the velocity of water below the partition wall between the flocculator end clarifier be 0.35 m/min (0.3m/min to 0.4m/min). Therefore, area of opening required for a velocity 0.35 m/min below the partition wall will be

$$\text{Area} = 255.1 / (0.35 \times 60) = 12.14 \text{ m}^2$$

Depth of opening from bottom slab to bottom of partition wall. The partition wall will rest on the peripherally located stub columns.

$$= 12.14 / (\pi \times 6.9) = 0.56 \text{ m,}$$

Add 15% more area for column projections = 0.084m

Total depth/height of the opening = 0.56 + 0.084 = 0.64 m, say 0.75 m

Minimum depth of opening shall not be less than 0.75m any case, please note that this depth of opening is part of overall flocculator depth (swd) of 3.5m.

Provide 1 in 12 slope for the bottom slab of clarifier towards central drain pit

F. Design of Clarifier

Assume a surface overflow rate of $30 \text{ m}^3/\text{m}^2/\text{day}$

$$\text{Surface area of clarifier} = 255.1 \times 24 / 30 = 204.08 \text{ m}^2$$

Diameter of the clariflocculator, D_{cf} is given by

$$(\pi/4) [D_{cf}^2 - (8.2)^2] = 204.08$$

$$D_{cf} = 18.65 \text{ m}$$

$$\text{Length of weir} = \pi D_{cf} = \pi \times 18.65 = 58.9 \text{ m}$$

$$\text{Weir loading} = (255.1 \times 24) / (58.9) \text{ m}^3/\text{day}/\text{m} = 103.5 \text{ m}^3/\text{day}/\text{m} (< 300 \text{ m}^3/\text{day}/\text{m}) \text{ O.K.}$$

Check for detention time of clarification zone:

Clarifier Side Water Depth (swd) = 3.50 m.

$$\{(\pi/4) [(18.65)^2 - (8.2)^2] \times 3.50\} / 255.1 = 2.79 \text{ hr.}, (\text{more than } 2.5 \text{ hr.}), \text{ hence, O.K.}$$

Appendix 8.4: Design of Rectangular Plain Sedimentation Tank**A. Problem Statement**

Design rectangular sedimentation tank with following data.

- | | |
|--|--|
| a) Desired Average Outflow from sedimentation tank | = 250 m ³ /hr. |
| b) Water lost in desludging | = 2% |
| c) Design Average flow | = (250 x 100) / (100 – 2) |
| | = 255.1 m ³ /hr. |
| d) Minimum size of the particle to be removed | = 0.02 mm |
| e) Expected removal efficiency of min. size particle | = 75% |
| f) Nature of particles | = discrete and non-flocculating |
| g) Specific gravity of particles | = 2.65 |
| h) Assumed performance of the settling tank | = good (n = ¼) |
| i) Kinematic viscosity of water at 20° C | = 1.01 x 10 ⁶ m ² /s |

B. Design Procedure

For the given diameter and specific gravity of minimum size particles to be removed in settling tank, vertical settling velocity of the particle is calculated initially using Stoke's law. The computed settling velocity is used to determine Reynolds number to check whether Stoke's law is applicable. If Reynolds number exceeds 1, Hazen's formula is used to determine the settling velocity of particle. The settling velocity thus calculated is employed for computation of surface over flow rate for expected removal efficiency of minimum size particles and assumed performance of the settling basin. Alternatively, the surface over flow rate for average design flow may be assumed on the basis of data presented in Table in section 8.6.8. The plan area is determined next, followed by tank dimensions. The depth of tank may be determined using detention period. Sizing of components of inlets and outlets is done using relevant design criteria & assumptions.

C. Design Steps

Compute vertical settling velocity of minimum size particles.

$$\begin{aligned}
 V_s &= \frac{g \times (S_g - 1) \times d^2}{(18 \times \nu)} \\
 &= \frac{9.81 \times (2.65 - 1) \times (0.02 \times 10^{-3})^2}{(18 \times 1.01 \times 10^{-6})} \\
 &= 3.56 \times 10^{-4} \text{ m/s}
 \end{aligned}$$

$$\begin{aligned}
 \text{Reynolds number} &= (V_s \cdot d) / \nu \\
 &= 3.56 \times 10^{-4} \times (0.02 \times 10^{-3}) / (1.01 \times 10^{-6}) \\
 &= 704 \times 10^{-3} < 1
 \end{aligned}$$

Hence Stoke's law is applicable and computed settling velocity is correct.

D. Determine Surface Overflow Rate

Part A: Engineering Design

For Ideal settling basin and complete removal of minimum size particles, equate settling velocity to theoretical surface overflow rate for 100% removal,

$$V_s = V_o$$

$$V_o = 3.56 \times 10^{-4} \text{ m/s}$$

$$= 3.56 \times 10^{-4} \times 3600 \times 24 = 30.76 \text{ m/d}$$

However due to short circuiting, there is reduction in efficiency and decrease in surface overflow rate. To obtain design surface overflow rate, which would give expected removal efficiency of minimum size particles in real basin, use following relationship.

$$y/y_o = 1 - [1 + n \cdot (V_o / (Q/A))]^{-1/n}$$

For $y/y_o = 0.75$, $n = 1/4$ (good performance of tank)

$$\begin{aligned} V_o/(Q/A) &= 1/n \{ [1 - y/y_o]^{-n} - 1 \} \\ &= 4 \times [(1 - 0.75)^{-1/4} - 1] \\ &= 1.66 \end{aligned}$$

Hence Design Surface overflow rate at average design flow, Q/A

$$Q/A = (V_o / 1.66) = 30.76 / 1.66 = 18.53 \text{ m/d}$$

Typical values for design surface overflow rate range between 15 and 30 $\text{m}^3/\text{m}^2/\text{d}$ for plain sedimentation tanks.

E. Calculate Dimensions of Tank

$$\begin{aligned} \text{Surface area of tank, } A &= (Q / (Q/A)) \\ &= 255.1 [\text{m}^3/\text{hr.}] \times 24 / 18.53 \\ &= 330.4 \text{ m}^2 \end{aligned}$$

Assume length to width ratio as 4

Length x width = surface area

$$\text{Width, } B = \sqrt{(330.4/4)} = 9.09$$

$$\text{Length of tank, } L = 36.36 \text{ m}$$

Assume detention period, t , as 4 hrs.

$$\begin{aligned} \text{Water depth of settling zone at average flow} &= Q \times t/A \\ &= 255.1 \times 4 / (36.36 \times 9.09) = 3.09 \text{ m} \end{aligned}$$

F. Check Against Resuspension of Deposited Particles

Flow velocity that can initiate re-suspension of deposited particles in the sludge zone, V , is given by

$$V = \sqrt{\frac{8k \times g \times (S_g - 1) \times d}{f}}$$

For uni-granular particles $k = 0.04$ and Weisbach – Darcy friction factor, $f = 0.03$

Part A: Engineering Design

$$V = \sqrt{\frac{8 \times 0.04 \times 9.81 \times (2.65 - 1) \times (0.02 \times 10^{-3})}{0.03}}$$

$$= 5.88 \times 10^{-2} \text{ m/s}$$

To avoid re-suspension, this critical displacement velocity should not be exceeded and horizontal velocity of flow in basin should be less than critical displacement velocity. Horizontal velocity of flow in settling basin at average flow, V_h

$$V_h = Q / (B \times D)$$

$$= 255.1 \text{ [m}^3\text{/hr.]} / (3600 \times 9.09 \times 3.09) = 2.52 \times 10^{-3} \text{ m/s} < 5.88 \times 10^{-2} \text{ m/s, hence, O.K.}$$

G. Influent Structure

The influent structure is designed to minimize turbulence, to distribute the water and suspended solids uniformly across the width and throughout the depth of settling basin and to avoid deposition of suspended solids in influent structure. It may consist of an influent channel, submerged orifices and baffles in front of orifices.

Provide 0.6 m wide and 0.6-m deep influent channel that runs across the width of the tank. Provide 4 submerged orifices 0.20 m x 0.20 m in the inside wall of influent channel to distribute the flow uniformly into the basin. A baffle 1 m deep is provided at a distance of 1 m away from orifices to reduce turbulence.

$$\text{Velocity of flow in channel} = \frac{255.1}{3600 \times 0.6 \times 0.4}$$

at average design flow

(Assuming a depth of flow of 0.4 m) = 0.3 m/s.

$$\text{Head loss through orifices} = \left[\frac{255.1}{3600 \times 4 \times 0.6 \times 0.2^2 \times \sqrt{2 \times 9.81}} \right]^2$$

$$= 0.03 \text{ m}$$

H. Effluent Structure

The components of effluent structure are effluent weir, effluent launder, outlet box and an outlet pipe.

- (i) Compute weir length & number of V-notches

Outflow from sedimentation tank = 250 m³ /hr.

Assuming a weir loading of 200 m³ / d per m length of weir,

Weir length = (250 x 24) / 200 = 30 m

No. of 90° V-notches assuming centre to centre spacing of 200 mm = 30 x 1000 / 200 = 150

- (ii) Provide 30-m length of effluent launder with V-notches fixed only on one side of the launder. For a 0.30-m wide effluent launder, the critical depth at the end of effluent launder can be computed from

$$y_2 = [(q'L)^2 / (b^2 \times g)]^{1/3}$$

$$= [(250 / (2 \times 3600))^2 / ((0.3)^2 \times 9.81)]^{1/3}$$

Depth of water at upper end of the trough, y_2 is

$$= [y_2^2 + 2(q'LN)^2 / (gb^2)]^{1/2}$$

$$= [0.11^2 + 12 \times \{250 / (2 \times 3600) \times 1\}^2 / \{9.81 \times (0.3)^2 \times 0.11\}]^{1/2}$$

$$= 0.19 \text{ m}$$

Accounting for head loss due to frictional resistance in the launder channel and the free board, a depth of launder of 0.4 m may be provided.

Appendix 8.5: Design for Radial Circular Settling Tank**A. Problem Statement**

Design a secondary circular sedimentation tank to remove alum floc with following data.

(a) Average output from settling tank	= 250 m ³ /hr.
(b) Amount of water lost in desludging	= 2%
(c) Average design flow	= 255.1 m ³ /hr.
(d) Minimum size of alum floc to be removed	= 0.8 mm
(e) Specific gravity of alum floc	= 1.002
(f) Expected removal efficiency of alum floc	= 80%
(g) Assumed performance of settling tank	= Very good (a = 1/8)
(h) Kinematic viscosity of water at 20°C	= 1.01 x 10 ⁻⁶ m ² /s

B. Design Solution Steps

Calculate the settling velocity of particles

$$V_s = \frac{g \times (S_g - 1) \times d^2}{(18 \times \nu)}$$

$$= \frac{9.81 \times (1.002 - 1) \times (0.8 \times 10^{-3})^2}{(18 \times 1.01 \times 10^{-6})}$$

$$= 6.91 \times 10^{-4} \text{ m/s}$$

$$\text{Reynolds number} = N_R = (V_s \cdot d) / \nu = (6.91 \times 10^{-4} \times 0.8 \times 10^{-3}) / 1.01 \times 10^{-6} = 0.55 < 1$$

Hence Stoke's law is applicable.

C. Compute Surface Overflow Rate, S or

For ideal basin and complete removal of wanted particles

$$V_s = V_o$$

$$V_o = 6.91 \times 10^{-4} \text{ m/s} = 59.7 \text{ m/d.}$$

However due to short circuiting etc., basin efficiency is reduced and to achieve desired removal efficiency, the surface overflow rate has to be decreased.

$$y/y_o = 1 - [1 + nV_o/(Q/A)]^{1/n}$$

For given values of $y/y_o = 0.8$, $n = 1/8$

$$V_o/(Q/A) = 1.78$$

$$Q/A = 59.9/1.7 = 33.49 \text{ m/d O.K.}$$

This is acceptable as it is within the typical design range of 30-40 m³/m²/d.

D. Determine the Dimension of Tank

Part A: Engineering Design

$$\begin{aligned}\text{Surface area of tank, } A &= Q / (Q/A) \\ &= 255.1 \times 24 / 33.49 = 182.8 \text{ m}^2\end{aligned}$$

$$\text{Hence diameter of tank} = 15.26 \text{ m}$$

Assume detention period, t , of 2.5 hours as given in Table 8.5.

$$\begin{aligned}\text{Depth of tank} &= Q \times t / A \\ &= 255.1 \times 2.5 / 182.8 \\ &= 3.49 \text{ m, say } 3.5 \text{ m}\end{aligned}$$

E. Check For Weir Loading

$$\begin{aligned}\text{Weir length} = \text{periphery of the tank} &= \pi D \\ &= \pi \times 15.25 \\ &= 48.94 \text{ m}\end{aligned}$$

$$\begin{aligned}\text{Weir loading} &= 255.1 \times 24 / 48.94 \\ &= 128.7 \text{ m}^3/\text{d}/\text{m} < 300 \text{ m}^3/\text{d}/\text{m}\end{aligned}$$

Hence, O.K.

Appendix 8.6: Design for Tube Settlers**A. Problem Statement**

Design tube settler module of square cross section with following data

- | | |
|--|-----------------------------|
| 1. Average output required from tube settler | = 250 m ³ /hr. |
| 2. Loss of water in desludging | = 2% of output required |
| 3. Average design flow = (250 x 100) / (100 - 2) | = 255.1 m ³ /hr. |
| 4. Cross section of square tubes | = 50 mm x 50 mm |
| 5. Length of tubes | = 0.60 m (600 mm) |
| 6. Angle of inclination of tubes | = 60° |

B. Design Steps

Compute relative length of settler

$$L_R = 600/50 = 12$$

Effective relative length of tube, L

$$\begin{aligned} L &= L_R - 0.058 N_R \\ &= L_R - 0.058 \times V_o \times d / v \\ &= 12 - (0.058 \times V_o \times 0.05) / (1.01 \times 10^{-6} \times 86400) \\ &= 12 - 0.033 V_o \end{aligned}$$

Where, V_o is flow through velocity for tube settler in m/d

C. Determine Flow Velocity Through Tubes

$$\begin{aligned} S_c &= (V_s / V_o) \times (\sin \theta + L \cos \theta) \\ 11/8 &= (35/V_o) \times (\sin 60 + (12 - 0.03 V_o) \cos 60) \\ V_o &= 123 \text{ m/d (m}^3\text{/m}^2\text{/day)} = 5.12 \text{ m/hr.} \\ V_s &= \text{Velocity of settling solids 35 m/day (1.45 m/hr.)} \end{aligned}$$

Alternatively refer the Table 8.6 for length of tube vs surface loading rate given in the description

D. Compute Total Plan Area for Tube Settlers

$$\text{Tube entrance area} = Q / V_o = 255.1/5.12 = 49.82 \text{ m}^2$$

Add 10% (5% to 10%) to the plan area to accommodate wall thickness of tubes

$$\text{Hence total area} = 49.82 \times 1.1 = 54.81 \text{ m}^2$$

Provide 4 (four no of hoppers), Area of each hopper $54.81/4 = 13.70 \text{ m}^2$, Plan size at top = 3.70 m x 3.70 m each, Size of each stream 3.7 m x 8.40 m, Hopper slope 45° to 50°

E. Distribution of Flocculated Water Below Tube Modules

Provide 4 (four) no. of Rigid PVC perforated pipes for each stream over hopper top

Part A: Engineering Design

Rate of flow per pipe = $[(255.1)/2]/4 = 31.88 \text{ m}^3/\text{hr.}$,

Select pipe diameter 250mm (0.25m), Velocity in pipe 0.18 m/sec, hence OK.

Distribution holes or perforations:

Velocity 0.10 m/s, Area required 0.088 m^2 , Diameter of holes: 50 mm (0.050m), No of holes per pipe: 45,

Spacing of holes: (Length of pipe/no of holes) x 2 rows = $(8.4/45) \times 2 = 0.32 \text{ m c/c}$ in two rows.

Provide holes in two rows 120° apart (staggered) at the bottom at 300 mm c/c.

ALTERNATIVELY, SOME PROPRIETARY PRACTICES INCLUDE DISTRIBUTION OF FLOCCULATED WATER THROUGH A HORIZONTAL SLIT OPENING PROVIDED BELOW THE MODULES THROUGH THE SIDE WALL. HOWEVER, IN SUCH CASE A BAFFLE WALL IS REQUIRED AT THE OUTLET SIDE OF FLOCCULATION TANK TO DAMPEN THE TURBULENCE CAUSED DUE TO PADDLE ROTATION.

F. Uniform Collection of Settled Water

Provide Settled Water Collection troughs 0.90m above top of the modules (0.75m to 1.0m)

Provide spacing at 1.25m c/c (1.0m to 1.50m), No of troughs: $8.40/1.25 = 5.92$, say 6 no.

Rate of flow per trough: $[(255.1/2)]/6 = 21.25 \text{ m}^3/\text{hr.}$

The depth of water in troughs is found by the expression: $Q = 1.375 B h^{3/2}$.

For width of troughs (B) 0.15 m, the depth (h) will be 0.09 m.

Provide uniformly spaced 90° V-notches on both sides of the troughs at top as collection edges.

The total height of the troughs = 0.09m (water depth) + 0.05m (free-fall) + 0.05m (V-notch depth)

= 0.19m, say 0.20m.

The depth of water over V notch can be found by the expression $Q = 1.38 h^{3/2}$.

The spacing of the V-notches can be 200 mm – 350 mm c/c to ensure the weir loading less than $300 \text{ m}^3/\text{m}/\text{day}$.

Appendix 8.7: Design for Rapid Gravity Filter

A. Problem Statement

Design rapid gravity filter for producing a net filtered water flow of 250 m³ /hr. The relevant data is

- | | |
|------------------------------------|---|
| i. Quantity of backwash water used | = 3% of filter output |
| ii. Time lost during back washing | = 30 min |
| iii. Design rate of filtration | = 5 m ³ /m ² /hr. |
| iv. Length to width ratio | = 1.25 - 1.33: 1 |
| v. Under drainage system | = Central manifold with laterals |
| vi. Size of perforations | = 9 mm |

B. Solution**(a) Filter Dimensions**

Required flow of filtered water = 250 m³/hr.
 Design flow for filter after accounting for backwash water = 250 x (1 + 0.03) x 24 / 23.5

= 263 m³/hr.
 Plan area of filter required = 263/5 = 52.6 m²

Provide two filter units, two being minimum no. to be provided

Length x width = 26.3

Assume length to width ratio as 1.3: 1

Width of the filter = (26.3/1.3)^{0.5} = 4.50 m

Length of the filter = 5.85 m

Provide two filter units, each with a dimension of 5.85 x 4.50 m

(b) Estimation of Sand Depth

Assume a depth of sand as 60 cm and effective size of sand as 0.5 m.

The depth can be checked against breakthrough of floc through sand bed by calculating minimum depth required by Hudson formula

In F.P.S unit,

$$\frac{Q \times d^3 \times h}{l} = B \times 29323$$

Where;

Q = the rate of filtration in gpm/sft,

d = the sand size in cm,

h = the terminal loss of head in ft.,

l = the depth of bed in inches and

b = breakthrough index whose value ranges between 4×10^{-4} to 6×10^{-3} depending on response to coagulation and degree of pretreatment of filter influent,

In metric units,

$$\frac{Q \times d^3 \times h}{l} = B \times 29323$$

Where Q is in $\text{m}^3/\text{m}^2/\text{h}$, d in mm and h & l are in m.

Assume $B = 4 \times 10^{-4}$ for poor response to filtration and average degree of pre-treatment, terminal head loss of 2.5 m, rate of filtration = $5 \times 2 = 10 \text{ m}^3/\text{m}^2/\text{hr}$. (Assuming 100 % overloading of filter under emergencies), and assuming $d = 0.6$ mm as mean diameter,

$$\frac{10 \times 0.6^3 \times 2.5}{l} = 4 \times 10^{-4} \times 29323$$

Minimum depth of sand required to avoid breakthrough = 46 cm. Hence the assume depth of 60 cm to be adequate to avoid breakthrough of floc.

(c) Estimation of Gravel and Size Gradation

Assume a size gradation of 2 mm at top to 50 mm at the bottom. The requisite depth ' l ' in inches of a component gravel layer of size ' d ' in inches can be computed from empirical formula

$$l = k (\log d + 1.40)$$

Where k varies from 10 to 14.

The equivalent formula in metric units where l is in cm and d is in mm is

$$l = 2.54 k (\log d)$$

For $k = 12$, the depth of various layers of gravel are

Size, mm	2	5	10	20	40
Depth, cm	9.2	21.3	30.5	40	49
Increment, cm	9.2	12.1	9.2	9.5	9

Provide a gravel depth of 50 cm

(d) Design Of Under Drainage System

$$\text{Plan area of each filter} = 5.85 \times 4.50 = 26.33 \text{ m}^2$$

$$\begin{aligned} \text{Total area of perforations} &= 3 \times 10^{-3} \times \text{Area of filter} \\ &= 0.0789 \text{ m}^2 \\ &= 790 \text{ cm}^2 \end{aligned}$$

$$\text{Total number of perforations of 9 mm dia.} = 790 / ((\pi/4)(0.90)^2) = 1241.8, \text{ say } 1242$$

$$\text{Total cross-sectional area of laterals} = 3 \times \text{Area of perforations}$$

$$= 3 \times 790 = 2370 \text{ cm}^2$$

$$\begin{aligned} \text{Area of central manifold} &= 2 \times \text{Area of laterals} \\ &= 2 \times 2370 \text{ cm}^2 \\ &= 4740 \text{ cm}^2 \end{aligned}$$

$$\begin{aligned} \text{Diameter of central manifold} &= \sqrt{[(4740 \times 4) / \pi]} \\ &= 78.7 \text{ cm} \end{aligned}$$

Provide a commercially available diameter of 800 mm

Assuming a spacing of 15 cm for laterals,

$$\text{The number of laterals} = (2 \times 5.85 \times 100) / 15 = 78$$

$$\text{Cross sectional area of each lateral} = 2370 / 78 \text{ cm}^2 = 30.39 \text{ cm}^2$$

$$\text{Diameter of lateral} = \sqrt{[(30.39 \times 4) / \pi]} = 6.22 \text{ cm}$$

Provide laterals of diameter of 80 mm

$$\text{Number of perforation per lateral} = 1242 / 78, \text{ say } 16$$

$$\begin{aligned} \text{Length of lateral} &= 1/2 (\text{width of filter} - \text{diameter of manifold}) \\ &= 1/2 (4.5 - 0.8) = 1.85 \text{ m} \end{aligned}$$

$$\text{Spacing of perforations} = 1.85 \times 100 / 16 = 11.56 \text{ cm}$$

Provide 16 perforations of 9 mm dia. at centre to centre spacing of 115 mm.

(e) Compute Dimension of Wash Water Trough

Assume a wash water rate of $36 \text{ m}^3/\text{m}^2/\text{hr}$.

$$\begin{aligned} \text{Wash water discharge for 1 filter} &= 36 \times 26.33 \text{ m}^3/\text{hr} \\ &= 948.88 \text{ m}^3/\text{hr} \\ &= 0.2633 \text{ m}^3/\text{s} \end{aligned}$$

Assuming a spacing of 1.6 m for wash water trough which will run parallel to the longer dimension of the filter unit.

$$\text{No. of troughs} = 4.50 / 1.6 = 3$$

$$\text{Discharge per unit trough} = 0.2633 / 3 = 0.0878 \text{ m}^3 / \text{sec}$$

For a width of 0.4m, the water depth at upper end is given by

$$\begin{aligned} Q &= 1.376 b h^{3/2} \\ 0.0878 &= 1.376 \times 0.4 \times h^{3/2} \\ h &= 0.294, \text{ say } 0.3 \text{ m} \end{aligned}$$

Assume a free board of 0.1 m, provide a depth of 0.4 m

Provide three trough of 0.4 m wide x 0.4 m deep in each filter

(f) Computation of Total Depth of Filter Box

Depth of filter box = sum of depths for (i) under-drains, (ii) gravel, (iii) sand, (iv) water depth, (v) free board

$$= 0.8 + 0.45 + 0.6 + 1.2 + 0.3 = 3.35 \text{ m}$$

(g) Determine Initial Head Loss

The sieve analysis of filter sand is as follows:

Sand size, mm	0.3	0.4	0.5	0.6	0.7	0.8	1.0	1.45
(% of sand smaller than stated size)	0.0	2.0	10.0	28.0	50.0	70.0	90.0	100.0
Porosity of sand bed	= 0.4							
Sphericity of sand	= 1.0							

Head loss for a clean filter can be determined using Kozeny's equation for stratified beds

$$\frac{h}{l} = \frac{kv}{g} v \frac{(1-f)^2}{f^3} \left(\frac{6}{\phi}\right)^2 \sum_{i=0}^n \frac{p_i}{d_i^2}$$

Where 'h' is the head loss, 'l' the depth of sand bed, 'k' Carman Kozeny constant having a value of 5, 'v' velocity of filtration, 'v' kinematic viscosity, 'f' porosity of clean bed, 'φ' grain sphericity, 'p' fraction of sand and 'd' geometric mean diameter of sand.

Computation of $\sum_{i=0}^n \frac{p_i}{d_i^2}$

Size of sand in mm	% Of sand larger than stated size	Sand fraction within adjacent sieve size $p_i \times 100$	d_i cm x 100	p_i/d_i^2
0.3	0.0	2	3.5	16
0.4	2.0	8	4.5	40
0.5	10.0	17	5.5	56
0.6	28.0	23	6.5	52
0.7	50.0	20	8.5	36
0.8	70.0	20	8.9	25
1.0	90.0	10	11.8	8
1.4	100.0			
100				233

$$\frac{h}{l} = \frac{5 \times 500}{981 \times 3600} \times 1.01 \times 10^{-2} \times \frac{(1-0.4)^2}{0.4^3} \left(\frac{6}{1.0}\right)^2 \times 233 = 0.337$$

$$h = 0.337 \times 0.6 = 0.20 \text{ m}$$

Head loss for clean filter bed for given sand is 0.20 m,

Appendix 8.8: Preparation of Filter Sand from Stock Sand**1. Problem Statement**

Prepare a filter sand of effective size 0.5 mm and uniformity coefficient 1.5 from the stock sand, the sieve analysis for stock sand being given as follows:

Sand size, mm	0.21	0.30	0.42	0.84	1.12	1.68	2.38
Cumulative weight, %	3.5	11	22	42	64	83	90

2. Solution

The given size distribution of stock sand is plotted on log - normal probability paper and from the plot determine the percentages of sand having size less than the effective size of 0.5 mm (p_1) and having size less than the 60-percentile size (p_2)

$$p_1 = 24\%$$

$$p_2 = 43\%$$

$$\begin{aligned} \text{i. Hence percentage of usable sand} &= 2 \times (p_2 - p_1) \\ &= 2 \times (43 - 24) = 38\% \end{aligned}$$

ii. Percentage of stock sand below which stock sand is too fine,

$$p_4 = 1.2 p_1 - 0.2 p_2$$

$$p_4 = 1.2 \times 24 - 0.2 \times 43$$

$$p_4 = 20.2\%$$

Determine the size of this sand, d_4 from graph

$$\text{For } p_4 = 20.2\%, d_4 = 0.41 \text{ mm}$$

iii. Percentage of stock sand above which stock sand is too coarse

$$p_5 = 1.8 p_2 - 0.8 p_1$$

$$p_5 = 1.8 \times 43 - 0.8 \times 24$$

$$p_5 = 58.2\%$$

Determine the size of this sand, d_5 from the graph,

$$\text{For } p_5 = 58.2\%, d_5 = 1.0 \text{ mm}$$

It follows that all-stock sand finer than 0.41 mm size and coarser than 1.0 size should be removed to obtain the filter sand of effective size 0.5 mm and uniformity coefficient of 1.5.

Appendix 8.9: Information to be included in the Tender Specifications for Water Treatment Plant

GENERAL

The principal requirement must be a spacious and convenient layout. The structures should represent a pleasing appearance with aesthetic features forming a balance between function and form. The interiors of the structures shall be eye appealing and in keeping with the objectives of the plant viz., production of pure and wholesome water.

While the mode of design and construction could be a matter of individual choice, it should be ensured that all materials, design, construction and fabrication details for different units including doors and windows conform to the relevant IS Specifications and codes of practice wherever available and in their absence, to the established standards.

Adequate provision shall be made in the Civil Engineering works for laboratory, office buildings, administration area, sanitary facilities and water supply etc. The area requirement of these ancillary requirements shall be stipulated. Roadways with adequate lighting shall be provided. Adequate ladders or steps and handrails where required shall be provided for easy access to each unit of the treatment plant and wherever necessary, walkways should be provided. Interconnecting facilities shall be provided to enable the operator to move freely for maintenance and operation of the plant.

All water retaining structures shall be designed in conformity with IS:3370 (Part 1) (2009); IS:3370 (Part2) (2009); IS:3370 (Part 3) (1967, reaffirmed 2008); IS 3370 (Part 4) (1967, reaffirmed 2008) while the other structures shall be designed according to IS 456 (2000, reaffirmed 2005).

The tender specifications should include *inter alia*, process requirements and specifications for equipment.

A. Process Requirements

1. The following data shall be furnished to the renderers:

- a) Raw water analysis comprising of monthly average figures preferably for a full year period covering various seasonal variations in respect of, at least the following. If the full year data is not available, the worst seasonal values may be given:
 - i. pH
 - ii. Turbidity
 - iii. Total Alkalinity
 - iv. Total hardness
 - v. Chlorides
 - vi. Coliform organism (MPN)
- b) Any other additional data, if the water is known to contain constituents or contaminants which are required to be removed:

- i. Phenols
- ii. Taste and odour
- iii. Colour
- iv. Carbon dioxide
- v. Algal content
- vi. Iron
- vii. Manganese
- viii. Hardness (Carbonate and noncarbonate along with magnesium content of water)
- ix. Fluoride content, and
- x. Chlorine demand and any other pollutants arising from industrial effluents and agricultural runoff

c) Hydraulic data such as the relevant raw water inlet and filtrate outlet levels.

d) The following requirements shall be furnished:

- i. The flow requirements of the plant in terms of the net output expected of the plant for a given period of time, say 23.5 hours a day (allowing for washing of the filters, etc. and also overload capacity.)
- ii. The quality of the treated water in terms of pH, turbidity, coliform organisms (MPN) and *E. coli*: and where needed Iron, Manganese, Hardness (carbonate and non-carbonate along with Magnesium content of water), fluoride content and colour.
- iii. Design parameters for various treatment units such as coagulants and coagulation aids dosing, rapid mixing, slow mixing, sedimentation, filtration and chlorination as well as special processes like aeration, micro-straining, iron and manganese removal, fluoride removal, taste and odour control as per specific local requirements and in accordance with the details furnished in the Manual.
- iv. A suggested layout of a Water Treatment Plant including following details, to the extent possible:
 - 1. Unit sizes and location of plant structures;
 - 2. Schematic flow diagram showing flow through various units;
 - 3. Piping arrangement including bypasses showing the material and size of pipes as well as direction of flow;
 - 4. Hydraulic profile of the units showing the flow of water.
 - 5. Contour map of the area including provision for future expansion.
 - 6. Approach roads and water supply facilities for construction purposes.
 - 7. Other information about site such as proneness to flooding and earthquakes, ground water table fluctuations, type and nature of soils met up to maximum anticipated depths, soil characteristics like bearing capacity and corrosivity, intensity and duration of rainfall and total annual rainfall, locations of areas for disposal of excavated spoils and of borrow pits if required for filling purposes.

Part A: Engineering Design

The contract should establish where guarantees apply and clearly define their requirements performance guarantees must be demonstrated by a test run of specified length or over an agreed period of operation.

B. Mechanical Equipment

- a) The following data may be given while inviting tenders for pumping plant
- b) Number of units required to work in parallel.
- c) Nature of liquid to be pumped:
 - i. Fresh or salt water
 - ii. Temperature of liquid
 - iii. Specific gravity
 - iv. Amount of suspended matter present
- d) Required capacity as well as minimum and maximum amount of liquid the pump must deliver.
- e) Suction conditions:
 - i. Suction lift or suction head.
 - ii. Constant or variable suction condition.
- f) Discharge conditions.
 - i. Maximum/Minimum discharge pressures against which pump has to deliver liquid.
 - ii. Static head description: constant or variable.
 - iii. Friction head description and how estimated.
- g) Type of service: continuous or intermittent.
- h) Pump installation: horizontal or vertical position. (If vertical type of pit, wet and dry).
- i) Power available to drive the pump.
- j) Space, weight or transportation limitations.
- k) Location of installation.
- l) Special requirements with respect to pump design: construction or performance. The following requirements may be indicated.
 - i. The pump equipment as well as the component shall conform to the relevant I.S standards and in their absence, to any other accepted international or national standard.
 - ii. Any special duty conditions such as temperature, humidity, corrosive atmosphere should be specified.
 - iii. Submerged structure parts except hot rolled sections shall not be less than 6 mm thick under normal atmosphere and 8 mm in aggressive atmospheres.
 - iv. Prime movers and allied components such as electrical motor, starter switches reduction gear, drive mechanism, bearings, Plummer blocks etc. shall be of approved make.

- v. All rotating machinery particularly gears shall be designed with adequate safety margins and service factors.
 - a) An item wise price list of spare parts shall be furnished by the tenderer. At least two years requirement of fast-moving spares should be supplied along with the equipment.
 - b) The supplier of special equipments like softeners, recording gauges, rate controllers, chlorinators, proportioning coagulant feeders, meters, etc. shall furnish the services of a competent representative for a specified number of days during a specified period to instruct the plant operating personnel in the maintenance and care of the equipment and to conduct tests and make recommendations for producing most efficient results.
 - c) Equipment selection with respect to specification, spare units, spare parts and servicing can affect maintenance, operating and investment costs. It is the purchaser's responsibility to incorporate into the contract all requirements and limitation which affect cost. Equipment performance is usually guaranteed by the manufacturer.

The contractor shall furnish bonds covering items of work like mechanical equipments, piping etc. for specified period as a guarantee of satisfactory operation and correction and correction of any defect in the work, material or equipments furnished by them.

On special equipment extended guarantees, maintenance over a period of time and supervision of a complete installation may be provided by the manufacturer. On most large equipments the manufacturer provides field service with respect to installation.

- d) All water submerged parts, rotating mechanical parts, and steel pipes under water shall be adequately protected after surface preparation. Oil, grease, dirt, soil and all surface contaminants from structural and fabricated steel parts are removed by cleaning with solvent vapour, alkali emulsion, or steam. Loose rust or paint, weld spatter, etc. are removed by hand chipping, scraping, sanding, wire brushing and grinding. The bare finished shafting, finished flanges and other mechanical surfaces are protected by grease line or rust protection measures. Structural mechanism support and super structure, Walkway, hand rails, fabricated shafts, etc., shall be protected with at least one coat of primer and two coats of paint.
- e) The below listed IS codes are recommended to be followed for resilient building structures for the water treatment plant and water retaining process units structures along with other annexing units, etc:
 - i. SP 16 (1980) refers to Design aids for reinforced concrete to IS 456,
 - ii. IS:456 (2000, reaffirmed 2005) refers to plain and reinforced concrete,
 - iii. SP 20 (1991): Handbook on Masonry Design and Construction
 - iv. SP 23 (1982): Handbook on Concrete Mixes

- v. SP 24 (1983): Explanatory Handbook on Indian Standard Code of Practice for Plain and Reinforced Concrete (IS 456:1978)
 - vi. IS: 800 (2007) gives the code of practice for use of structural steel in general building construction;
 - vii. IS:875 (Part 1) {1987, (incorporating IS 1911(1967), reaffirmed 2003}-refers to dead loads other than earth quake;
 - viii. IS:875 (Part 2) (1987, reaffirmed 2003)-refers to imposed loads;
 - ix. IS:875 (Part 3) (2015)-refers to wind loads;
 - x. IS:875 (Part 4) (1987, reaffirmed 2003)- refers to snow loads;
 - xi. IS 4326 (1993, reaffirmed 2008) Earthquake Resistant Design and Construction of Buildings--Code of Practice
 - xii. IS:1893 (Part 1) (2002)- Criteria for earthquake resistant Design of structures Part1 general provisions and buildings
 - xiii. SP 22 (1982) Explanatory handbook on codes for earthquake engineering
- f) In addition, below listed IS codes are recommended to be followed for reinforcement detailing and guidance:
- i. SP:34 (S&T) -1987: Handbook on concrete reinforcement detailing.
 - ii. IS:432- 1982: Mild steel & medium tensile steel bars and hard drawn steel wires for concrete reinforcement : Part-II -Hard drawn steel wire.
 - iii. IS:1786-2008:Specification for High strength deformed steel bars and wires for concrete reinforcement.
 - iv. IS:2502-1963: Code of practice for bending & fixing of bars for concrete reinforcement.
 - v. IS:2751 -1998: Recommended practice for welding of mild steel plain & deformed bars for reinforced construction.
 - vi. IS:5525 -1969: Recommendation for detailing of reinforcement in reinforced concrete works.
 - vii. IS:9077 -1979: Code of practice for corrosion protection of steel reinforcement in RB & RCC construction.

Appendix 8.10: Common Coagulants Used in Water Treatment

S. No.	Coagulant	Common Name	Formula	Use	Available Forms	Commercial Strength	Appearance and Properties	Usual Solution or Suspension Strength	Method of Feeding	Material used for Handling Solution	Remarks
1	2	3	4	5	6	7	8	9	10	11	12
1	Activated Carbon		C	Taste and Odour control, de-chlorination	Granular	Not less than 80% C	Black granules 1-3 mm	Water passed through granular beds	Dry		
2	Activated Carbon		C	Do	Black powder	Do	200 to 400 μ m black powder insoluble		Dry or in slurry form careful mixing required to maintain proper slurry	Iron or Steel Tank	
3	Activated Silica	Silica Sol.	SiO ₂	Coagulant aid	Produced at site as needed from sodium silicate and activating agents		Clear, often opalescent	0.6	Wet batch made up by pH adjustment and aged	Mild steel or stainless steel or rubber containers	Appliances likely to be clogged with improper pH adjustment or feed

**Chapter-8:
Water Treatment**

Part A: Engineering Design

S. No.	Coagulant	Common Name	Formula	Use	Available Forms	Commercial Strength	Appearance and Properties	Usual Solution or Suspension Strength	Method of Feeding	Material used for Handling Solution	Remarks
1	2	3	4	5	6	7	8	9	10	11	12
4	Aluminium Sulphate	Alum; filter alum; sulphate of alumina	$\text{Al}_2(\text{SO}_4)_3 \cdot 4\text{H}_2\text{O}$	Coagulant	Blocks; Lumps; Powder	At least 16% Al_2O_3	Light tan to grey; crystalline; acidic; corrosive; slightly hygroscopic	8 - 10%	Wet or Dry	Acid proof brick tanks, bitumen coated concrete or rubber lined tanks	
5	Aluminium Sulphate (ferric)	Alum; filter alum; sulphate of alumina; aluminoferric	$\text{Al}_2(\text{SO}_4)_3 \cdot 4\text{H}_2\text{O}$ (approx.)	Do	Do	15% (approx.) Al_2O_3	Brown to dark brown; crystalline; acidic; corrosive; hygroscopic	8 - 10%	Do	Do	High concentration of Iron
6	Filter Alum	Filter Alum	$\text{Al}_2(\text{SO}_4)_3 \cdot 18\text{H}_2\text{O}$	Coagulant	Solution Sp. G. 1.1	8% Al_2O_3	Brown solution; acidic; corrosive	Direct or in 1% solution	Wet, orifice box rotameter and proportioning pumps	Acid proof brick tanks, bitumen coated concrete or rubber lined tanks	Costs less than dry alum if close enough to source of manufacture
7	Ammonium Sulphate	Sulphate of ammonia	$(\text{NH}_4)_2\text{SO}_4$	Chloramine treatment in disinfection	Crystal	20-25% NH_3	White sugar sized crystals	0.1 to 0.5%	Wet proportioning pumps	Stainless or plastic containers	
8	Anhydrous ammonia	Ammonia	NH_3	Do	Liquified Gas	98-99% NH_3	Colourless gas; pungent irritating offensive odour		Wet ammoniaator	Iron, Steel or glass	Dangerous coagulant

S. No.	Coagulant	Common Name	Formula	Use	Available Forms	Commercial Strength	Appearance and Properties	Usual Solution or Suspension Strength	Method of Feeding	Material used for Handling Solution	Remarks
1	2	3	4	5	6	7	8	9	10	11	12
9	Bentonite	Colloidal clay	H_2O (Al_2O_3 , Fe_2O_3 , MgO) $4SiO_3 \cdot nH_2O$	Coagulant aid, floc weighting agent	Powder pellets		Yellow Brown		Wet suspension	Iron or steel	
10	Calcium Hydroxide	Hydrated lime slaked	$Ca(OH)_2$	pH adjustment and softening	Powder	80-90% $Ca(OH)_2$	White powder; caustic	1.50%	Dry or wet. Can be fed in suspension	Iron steel or concrete tanks	Not very soluble, lead tank cannot be used
11	Calcium Hypochlorite	HTH, per chlorone	$Ca(OCl)_2 \cdot 4H_2O$	Disinfectant taste or odour control	Granular powder	70% available chlorine	White	2 to 4%	Wet	Stoneware, plastic, rubber tank	Dangerous coagulant
12	Calcium Oxide	Quick lime, burnt unslaked lime	CaO	pH adjustment and softening	Pebble crushed lumps to powder	40-90% CaO	White or light grey, caustic	1.50%	Dry or wet. Can be fed in suspension. Dry feeders generally discharge to slake before application	Iron, steel or concrete tanks	Lead tank cannot be used
13	Chlorine	Chlorine gas, Liquid chlorine	Cl_2	Disinfectant taste and odour control, general oxidant	Liquified Gas under pressure	99-99.8% Cl_2	green yellowish gas, pungent, corrosive, heavier than air, health hazard		Wet chlorinator	Dry iron copper steel solution, silver glass, hard rubber, lead special alloys	Dangerous coagulant very careful handling required. Should use gas mask

S. No.	Coagulant	Common Name	Formula	Use	Available Forms	Commercial Strength	Appearance and Properties	Usual Solution or Suspension Strength	Method of Feeding	Material used for Handling Solution	Remarks
1	2	3	4	5	6	7	8	9	10	11	12
14	Chlorinated ferrous sulphate	Chlorinated copperas	$\text{Fe}_2(\text{SO}_4)_3 \cdot \text{FeCl}_2$	Coagulant	Yellow solution	Produced at site by reaction of chlorine and ferrous sulphate		3-5%	Wet	Rubber lined or stainless-steel containers, plastic containers	
15	Chlorinated lime	Bleaching powder, chloride of lime	$\text{CaO}, 2\text{CaOCl}_2 \cdot \text{H}_2\text{O}$	Disinfection	Powder	25-33% available chlorine	White, hygroscopic, unstable pungent powder	1-2%	Wet	Plastic, stoneware or rubber tanks	
16	Chlorine dioxide		ClO_2	Taste and odour control disinfection	Gas	26.3% available chlorine	Generated at site	0.10%	Wet	Plastic, soft rubber	
17	Copper sulphate	Blue vitriol	$\text{CuSO}_4 \cdot 5\text{H}_2\text{O}$	Algicide	Crystal lumps, powder	90-95%	Clear blue crystals	1-2%	Dry put in bags, dragged behind boat, dusted on surface with special equipments	Stainless steel, plastics	

S. No.	Coagulant	Common Name	Formula	Use	Available Forms	Commercial Strength	Appearance and Properties	Usual Solution or Suspension Strength	Method of Feeding	Materials Used for Handling Solution	Remarks
1	2	3	4	5	6	7	8	9	10	11	12
18	Ferric Chloride	Chloride of Iron	$\text{FeCl}_2 \cdot 6\text{H}_2\text{O}$	Coagulant	Lump sticks crystal	60% FeCl_3	Yellow Brown highly hygroscopic, very corrosive	3-5%	Wet Proportionating pump	Rubber lined tank or stoneware containers plastic	-
19	Ferric Chloride (Solution)	Do	FeCl_2	do	Solution	30-40% FeCl_3	Brown Solution, Very Corrosive	3-5%	Wet Proportionating pump	do	-
20	Ferric Sulphate	Iron Sulphate, Ferrifloc	$\text{Fe}_2(\text{SO}_4)_3 \cdot \text{H}_2\text{O}$	Coagulant	Granules crystal lumps	18.5-0.1% Fe	Red brown or grey crystals mildly hygroscopic	3-5%	wet	Dry; iron stainless steel and concrete wet: lead or stainless-steel plastic	Solution is Corrosive
21	Ferrous Sulphate	Coappears green vitriol; sugar Sulphate	$\text{FeSO}_4 \cdot 7\text{H}_2\text{O}$	do	do	20% Fe	Green to Brownish yellow crystals hygroscopic	+8%	wet	do	Cakes in storage above 20° lime addition necessary
22	Hydrazine hydrate	-	$\text{H}_2\text{N}_2\text{H}_2\text{O}$	Deoxygenation	Solid Powder	64%	White Powder	-	Wet Proportionating pump	Stainless steel Pump	plastics
23	Hydrochloric Acid	Muriatic acid	HCL	in Cation exchange	Regenerant	Liquid	30% HCl liquid	5-10%	Wet pump	Glass rubber lined	Dangerous Coagulant

S. No.	Coagulant	Common Name	Formula	Use	Available Forms	Commercial Strength	Appearance and Properties	Usual Solution or Suspension Strength	Method of Feeding	Materials Used for Handling Solution	Remarks
1	2	3	4	5	6	7	8	9	10	11	12
24	Potassium Aluminium Sulphate	Potash Alum	$K_2SO_4 \cdot Al_2(SO_4)_3 \cdot 24H_2O$	Coagulant	Lump powder, blocks	8-11% Al_2O_3	White crystalline, acidic corrosive, hygroscopic	1%	Wet or dry		Generally used in business filters
25	Sodium Aluminate	Soda Alum	$Na_2Al_2O_3$	Coagulant	Crystalline flakes, solution	43-45% Al_2O_3	White to grey crystal, liquid caustic, alkaline	5%	Wet or dry	Cast iron mild steel concrete	Generally used along with filter alum
26	Sodium Carbonate	Soda Ash	Na_2CO_3	pH adjustment softening	Dense crystal, Powder	98-99% Na_2CO_3	White powder, Caustic	1-10%	Dry or wet Proportionating pump		Dangerous Coagulant
27	Sodium Chloride	Common Salt	$NaCl$	Softening regenerant	Crystals	90-95% $NaCl$	Colorless crystals	8-10%	Wet, fed through injection nozzles	Bitumen or epoxy coated M.S. tanks	
28	Sodium Chromate	Chromate	$NaCrO_4$	Corrosion preventive anodic inhibition	Crystals	80-90% Na_2CrO_4	Yellow / Brown Crystals	-	Wet Proportionating pump	Stainless steel, plastic	Dangerous Coagulant, harmful to eyes. Used in combination with sodium silicate

S. No.	Coagulant	Common Name	Formula	Use	Available Forms	Commercial Strength	Appearance and Properties	Usual Solution or Suspension Strength	Method of Feeding	Materials Used for Handling Solution	Remarks
1	2	3	4	5	6	7	8	9	10	11	12
29	Sodium Hexa-meta phosphate	Calgan, glassy phosphate	(NaPO ₂) ₄	Scale and corrosion prevention	Powder flakes	60-63% P ₃ O ₂	Like broken glass	0.25	Wet Proportionating pump	Stainless steel, plastic, hard rubber	Holds up ppm of Fe, Mn, Ca and Mg, for M.S. protective coating
30	Sodium Hydroxide	Caustic Soda	NaOH	pH adjustment, softening and filter cleaning	Flakes, lumps, pellets, powder	96-99%	Alkaline, corrosive, hygroscopic	1-10%	Wet Proportionating pumps, orifices box, rotameter	Cast iron, mild steel, rubber lined	Dangerous Coagulant
31	Sodium Hydroxide (Solution)	Caustic Soda, lye	NaOH	Do	Solution	10-40% NaOH	Syrupy solution	Do	Do	Do	Do
32	Sodium Sulphate		Na ₂ SO ₄	Deoxygenation	Powder, lumps	90-99% Na ₄ SO ₂	White powder	1%	Wet Proportionating pump	Stainless steel, plastic	8 mg/L sodium sulphate required to remove 1 mg/L O ₂
33	Sulphur dioxide		SO ₂	De-chlorination or filter cleaning	Gas	100%	Colorless, pungent	-	Dry	Steel	

S. No.	Coagulant	Common Name	Formula	Use	Available Forms	Commercial Strength	Appearance and Properties	Usual Solution or Suspension Strength	Method of Feeding	Materials Used for Handling Solution	Remarks
1	2	3	4	5	6	7	8	9	10	11	12
34	Sulphuric Acid	Vitriol	H ₂ SO ₄	pH adjustment, lowering of alkalinity	Liquid	60-90% (commercial H ₂ SO ₄)	Syrupy, corrosive, hygroscopic	1-2%	Wet, dilute solution, orifice box, rotameter	Lead, porcelain or rubber	Always add acid to water, dangerous coagulant
35	Sulphuric Acid	Do	Do	regenerant Cation exchange operation	Do	98% H ₂ SO ₄	Do	Do	Wet pumping	Lead, glass or rubber lined tank	Do

Appendix 8.11: Hydraulics of Filtration

The head loss, h , through a clean filter bed of depth l can be computed using Kozeny's equation

$$\frac{h}{l} = \frac{k}{g} \cdot \frac{\mu}{\rho} \cdot v \cdot \frac{(1-f)^2}{f^3} \left(\frac{A}{V} \right)^2$$

Where;

K = residual dimensionless coefficient, about 5 under most condition of water filtration,

μ = Absolute viscosity of water, (N.s)/m²

ρ = Density of water, (Kg/m³)

v = Macroscopic velocity of filtration, (m/s)

f = Porosity of clean sand bed, dimensionless

A = Surface area of the grains (m²)

V = Volume of the grains (m³)

For uni-size special medium particles of diameter d

$$\frac{A}{V} = \frac{6}{d}$$

For non-spherical grains, sphericity is defined as the surface area of the equivalent volume sphere to actual surface of non-spherical particles. The sphericity, Ψ assumes values of 1.0 for spherical grains, 0.98 for rounded grains, 0.94 for worn grains, 0.81 for sharp grains, 0.78 for angular grains and 0.70 for crushed grains for sand medium.

For stratified beds, as obtainable in rapid sand filters after back washing, the head loss in a clean bed is the sum of the head losses in successive sand layers. If p_i is the fraction of medium of sieved size d_i , the head loss is given by

$$\frac{h}{l} = \frac{k}{g} \cdot \frac{\mu}{\rho} \cdot v \cdot \frac{(1-f)^2}{f^3} \left(\frac{6}{\Psi} \right)^2 \sum_{i=1}^n \frac{p_i}{d_i^2}$$

For unstratified beds e.g., slow sand filter, the head loss becomes

$$\frac{h}{l} = \frac{k}{g} \cdot \frac{\mu}{\rho} \cdot v \cdot \frac{(1-f)^2}{f^3} \left(\frac{6}{\Psi} \sum_{i=1}^n \frac{p_i}{d_i} \right)^2$$

Appendix 8.12: Design of Media Depth and Media Sizes**(a) Design of Media Depth**

The efficiency of removal of suspended particles is a function of the surface area of the media grains.

For a filter of depth 'l' comprising of 'N' particles of average size 'd' and sphericity 'Ψ'

$$A = \frac{l(1-f)}{\frac{\pi(\psi d)^3}{6}} \cdot \pi(\psi d)^2$$

$$A = \frac{6(1-f)}{\psi} \left(\frac{l}{d} \right)$$

The equation can be employed to design the depths of filter. For example, for typical high-rate filters $f = 0.45$, $\psi = 0.8$ and $(l/d) = 680$, $A \cong 2800 \text{ m}^2$. Since the effective size of sand is normally specified, $(l/d) = 680$ corresponds to $(l/d_{10\%}) = 950$, where $d_{10\%}$ is the effective size. Figures can be developed for predetermined value of A, based on pilot data, between the effective size of medium and filter medium depth for different values of V. These figures can be used to estimate depths of various combinations of dual media.

(b) Design of Media Sizes

The dual media filters, consisting of coarse but lighter medium particles on top of finer but heavier particles, must retain their stratified character during backwashing and resettling. Equal expansion during backwashing for dual media comprising of coal and sand indicate equal fluidization velocity for both media. It can be shown that

$$\frac{d_u}{d_l} = \left(\frac{\psi_l}{\psi_u} \right) \left[\frac{\rho_l - \rho}{\rho_u - \rho} \right]^{\frac{1}{2}}$$

When subscripts u and l respectively denote the largest grain within the upper layer (coal) and the smallest grains within the lower layer (sand). It follows that mixing during settling as well as during expansion determines the maximum allowable ratio of the grain sizes in the two layers.

For sharp interface and no intermixing, the ratio of maximum diameter of coal to the minimum diameter of sand that will ensure both equal expansion and equal settling

can be computed using above mentioned equation for a density of coal of 1.5 and its sphericity 0.70, and, sphericity and density of sand of 0.85 and 2.65, this ratio is

$$\frac{d_u}{d_l} = \frac{0.85}{0.70} = \left[\frac{2.65-1}{1.5-1} \right]^{0.5} = 2.26 \cong 2.3$$

If partial intermixing is to be achieved the size of the coarsest coal must be more than 2.3 times the minimum diameter of sand for characteristic of coal and sand given.

Appendix 8.13: Design for Wash Water Recycling System

A. Problem Statement

Design rapid gravity filter for producing a net filtered water flow of $250 \text{ m}^3/\text{hr}$. The relevant data is

- | | |
|------------------------------------|--|
| i. Quantity of backwash water used | = 3% of filter output |
| ii. Time lost during back washing | = 30 minutes |
| iii. Design rate of filtration | = $5 \text{ m}^3/\text{m}^2/\text{hr}$. |
| iv. Length to width ratio | = 1.25 - 1.33: 1 |
| v. Under drainage system | = Central manifold with laterals |
| vi. Size of perforations | = 9 mm |

B. Solution

(a) Filter Dimensions

Required flow of filtered water = $250 \text{ m}^3/\text{hr}$.

Design flow for filter after accounting
for backwash water and time = $250 \times (1 + 0.03) \times 24 / 23.5 \text{ m}^3$
/hr.

lost in backwashing = $263 \text{ m}^3/\text{hr}$.

Plan area of filter required = $263/5 = 52.6 \text{ m}^2$

Provide two filter units, two being minimum no. to be provided

Length x width = 26.3

Assume length to width ratio as 1.3: 1

Width of the filter = $(26.3/1.3)0.5 = 4.50 \text{ m}$

Length of the filter = 5.85 m

Provide two filter units, each with a dimension of 5.85 x 4.50 m

(b) Wash water generated per wash of 10 min

Rate of backwash = 36 m/hr.

Wash water rate per bed: $5.85\text{m} \times 4.50\text{m} \times 36 \text{ m/hr} = 946.08 \text{ m}^3/\text{hr}$.

Quantity required for one bed (10 min wash) = $946.08 \times (10/60) = 158.68 \text{ m}^3$

Total backwash water generated per day (Assuming both filters are backwashed)
= $158.68 \times 2 = 315.36 \text{ m}^3$

(c) Capacity of wash water recycle tank = Equivalent to storage of one wash quantity
= 158.68 m^3 , say 160 m^3

(d) Wash water Recycle Rate

No of hours = 20 (Twenty)

Rate of recycle = $315.36 / 20 = 15.76 \text{ m}^3/\text{hr.}$, say $16 \text{ m}^3/\text{hr.}$

Percentage of Recycle water = $(16/263) \times 100 = 6\%$, hence OK

Appendix 8.14: Summary of Recommended Design Criteria for conventional and commonly employed unit processes (Typical values)

S. No.	Description	Range of values
Unit Process : Cascade Aerator		
1.	Area loading rate	0.030 sqm/cum/hr
2.	No of steps	4 -6
3.	Rise of steps	0.15m to 0.25m
4.	Tread of steps	Minimum , twice the rise
5.	Total drop over cascades	0.60m to 1.50m (Maximum 2.0m)
6.	Velocity in central shaft	Less than 0.60m/sec
7.	Shape of the unit	Circular
Unit Process : Coagulation		
A. Mechanical Flash Mixer		
1.	Detention time	60 sec
2.	Velocity Gradient	300 m/sec/m
3.	Paddle tip velocity	3 to 4 m/sec
4.	Ration of Depth to Diameter	1:1 to 3:1
5.	Shape	Circular or Square
B. Hydraulic Weir Mixer		
1.	Drop over weir (free-fall)	0.4 to 0.60m
2.	Velocity gradient	800-1000 m/sec/m
3.	Detention time in downstream chamber	2 – 5 sec
4.	Detention time in upstream chamber/channel	Minimum 1.0min
Unit Process: Flocculation		
Mechanical Flocculator		
1.	Detention time (t)	20-30 min
2.	Velocity Gradient (G)	40-70 m/sec/m (Variable speed)
3.	Product “G t”	2 to 6 x 10 ⁴
4.	Paddle tip velocity	0.40 to 0.70 m/sec
5.	Area of paddles	10-25% of cross sectional area of the tank
6.	Shape	Circular, Square or Rectangular
Unit Process: Clarification or sedimentation		
A. Clariflocculators		
1	Surface loading rate	30-40 cum/sqm/day
2.	Side Water Depth	Minimum 3.50m

3.	Detention time	2 – 3 hr
4.	Weir loading rate	200 – 300 cum/RM/day
5.	Inlet velocity (flocculated water)	Maximum 0.005 m/sec
6.	Sludge storage depth	Volume of Conical bottom below SWD
7.	Velocity of water in central shaft	Max 0.60 m/sec
8.	Velocity of water at inlet ports	Max 0.40 m/sec

B. Tube Settling Tanks

Tube size 50mm x 50mm or equivalent shape, Tube length 600 mm, Angle of inclination 60 degree

1.	Surface loading rate	5 – 6 cum/sqm/hr (5000-6000 lph/sqm)
2.	SWD	3 – 4 m (above hoppers or scrapper)
3.	Detention time	30-45 min
4.	Weir loading rate	300-400 cum/RM/day

Unit Process: Granular Media Gravity Filtration**Rapid Sand Gravity Filter beds**

1.	Rate of filtration	5 – 6 cum/sqm/hr (5000 to 6000 lph/sqm)
2.	Ratio of Length to width of Filter box (Two sections with central gullet)	1 to 1.6
3.	Depth of water over media	2 to 2.3 m
4.	Depth of sand	0.60 to 0.80m (ES 0.55 to 0.6 mm), UC (1.5 to 1.7)
5.	Depth of supporting gravel	0.45 to 0.60m, Graded
6.	Rate of backwash (water wash)	35-45 cum/sqm/min
7.	Head of backwash water tank	Effective head 8.50m above filter bottom + losses in piping
8.	Duration of backwash	8 to 10 min
9.	Rate of air scour	35 – 45 cum/sqm/min
10.	Pressure rating	0.35 to 0.40 kg/sq cm
11.	Filter under-drain	Manifold block and lateral pipes
12.	Lateral travel of wash water	0.60 to 1.0m
13.	Distance between top of the media and crest of the trough	0.65m to 0.75m
14.	Filter Control System	Constant Rate Filtration with Influent splitting weirs
15.	Height of the filter outlet weir above media top	0.10 to 0.15m

16.	Filter Port Velocities	
	Inlet	Up to 1.0 m/sec
	Outlet	Up to 1.0 m/sec
	Wash Inlet	Up to 3.0 m/sec
	Wash Disposal	Up to 2.5 m/sec
	Air scour piping	Up to 25 m/sec
Unit Process: Disinfection		
Chlorine Contact Tank		
1.	Shape	Square or Circular
2.	Detention time	30 min
3.	No of baffles	4 to 6
Note: For Advanced or high rate unit processes please refer to their specific discussion in the relevant section/sub-section of the Manual		

Appendix 8.15: Process Design and Sizing of units of Conventional Water Treatment Plant

Design Basis

Flow rates :

1. Overall proposed water demand : 20 mld (22 hr basis)
(Including 3% losses in sludge and backwash water)
2. Design capacity of Water Treatment Plant based
On 24 hr basis : $(20 \text{ mld} / 22 \text{ hr}) \times 24 = 21.81 \text{ mld}$
Say 22.00 mld
3. Corresponding flow rate : 916.67 cum / hr
Say 920 cum/hr

1. Cascade Aerator -

- 1.a No of units : 1, one
- 1.b Flow rate : 920.00 cum/hr
- 1.c Area loading : 0.030 sqm / cum/hr
- 1.d Area required : $920 \times 0.030 = 27.62 \text{ sqm}$
- 1.e Diam of central shaft (RCC) : 0.90 m ID, (Velo. 0.41 m/sec)
: 1.20m OD (0.90 + 0.15 + 0.15)

(ID –Internal Diam, OD –Outer Diam.)

- 1.f Area of central shaft : $(3.14 / 4) \times 1.20^2 = 1.13 \text{ sqm}$
- 1.g Total area required : $27.62 + 1.13 = 28.75 \text{ sqm}$
- 1.h No of cascades : 4, Four
- 1.i Let tread of cascade be : 0.65m
- 1.j Diam of lowest cascade : $(0.65 \times 8) + 1.20\text{m} = 6.40\text{m}$
- 1.k Actual area provided : $(3.14/4) \times 6.40^2 = 32.15 \text{ sqm}$
: (hence OK)
- 1.l Rise of cascade : 0.20m
- 1.m Total drop over cascade : $(4 + 1) \times 0.20 = 1.00\text{m}$
- 1.n **Collection launder** : Two semi-circular channels
- 1.n.1 Flow per channel : $920 / 2$
: 460 cum / hr
- 1.n.2 Width : 0.90 m

- 1.n.3 Velocity : 0.60 m / sec
 1.n.4 SWD (Side Water Depth) : 460 / (3600 x 0.90 x 0.60)
 : 0.24 m, say 0.35m
 1.n.5 Freeboard : 0.30m

2. Elevated channel with Parshall flume-

- 2.a No of units : 1, one
 2.b Flow rate : 920 cum/hr (0.26 cum/sec)
 2.c Width of channel : 1.25 m
 2.d Width of Parshall flume : 0.30 m, (1.0 foot)

Design of Parshall Flume (Ref IS14371:1996, Parshall Flume)

a. Head U/S of Flume (Table 3, Page 6)

For throat width of 0.3m, $Q = 0.679 h_a^{1.521}$

$Q = 0.25$ cum/sec

$H_a = 0.52$ m

Depth of water U/S ($H_a + N$) = $0.52 + 0.238 = 0.76$ m, say 0.8m

Head range, $H_a = 0.03$ m to 0.75m, hence OK

Discharge range, cum/sec x 10^3 : 250, (3.5 to 400) hence OK

b. Approach Channel , Froud No. (5.2.2., Page No.2)

$Fr = Q / [(A \times \sqrt{g \times h})]$

$Q = 0.25$ cum/sec,

$A =$ c/s area U/S of flume: $1.25\text{m} \times 0.80\text{m swd} = 1.0$ sqm

$g = 9.81\text{m/sec}^2$

$h =$ depth of water upstream of flume = 0.80m

hence $Fr = 0.25 / [(1.0 \times \sqrt{9.81 \times 0.80})]$

= 0.09 (less than 0.5)

Discharge range, cum/sec x 10^3 : 200 (3.5 to 400) hence OK

c. Velocity in channel (upstream of flume) : $V = Q / A$

$V = 920 / (3600 \times 1.25 \times 0.80)$

: 0. 26 m/sec

2.h Conditions on d/s of flume : Free falling free discharge

2.j Free board : 0.30m on upstream side of flume

3. Flash mixer

- 3.a No of units : 1, One

- 3.b Flow rate : 920 cum/hr
 3.c Let Size be : 2.25 m x 2.25 m x 3.50 m swd
 3.d Detention time: $[(2.25 \times 2.25 \times 3.5) / 920] \times 60 = 1.15 \text{ min}$
 (More than 1 min, hence OK)
 3.e Rapid agitator : Radial turbine type mixer
 3.f Velocity gradient (G) : 300 m / sec / m
 3.g Inlet chamber : 2.25 m x 1.25 m
 3.i Outlet chamber : 2.25m x 1.25m
 3.k Diameter of inlet pipe to clarifier : 0.60 m (600 mm)
 3.k Velocity in pipe : 0.90 m / sec (hence OK)

4. Clariflocculator

- 4.a No of units proposed : 1, one
 4.b Capacity / Flow per unit : 920 cum/hr
 4.c Diameter of central shaft : 0.90 m (ID), 1.30 m (OD)
 (ID –Internal Diam, OD –Outer Diam.) (1.30 = 0.90 +0.20+0.20)
 4.d Velocity through shaft : 0.40 m / sec
 4.e Size of outlet openings : 0.60 m x 0.30 m x 4 no
 4.f Velocity through openings : 0.35 m / sec
 4.g Let Diameter of flocculator be : 13.00 m(ID), 13.40 m
 (OD =13.00 +0.2 +0.2)
 4.h Side water depth : 3.50 m
 4.i Volumetric capacity : $3.50 \times 3.14/4 \times (13.00^2 - 1.30^2)$
 : 461.3 cum
 4.j Detention time : $(461.3/920.) \times 60 = 30.05 \text{ minute}$
 4.k Let Overall diam of clariflocculator be: 33.00 m (Internal Diam.)
 4.l Side water depth : 3.50 m, F.B. 0.50 m
 4.m Volumetric capacity : $3.5 \times 3.14/4 (33.0^2 - 13.40^2)$
 : 2500 Cum
 4.n Detention time : $(2500/920) = 2.71 \text{ Hr}$
 4.o Surface loading : $920 / [3.14/4 (33.0^2 - 13.40^2)]$
 : 1.29 cum / sqm / hr (31 cum / sqm / day)
 Hence OK.
 4.s Actual weir length : $(3.14 \times 33.0) = 103.7 \text{ m}$
 4.t Weir loading : $(920 / 103.7) \times 24$

: 213 cum / rm / day

Hence OK

4.u Size of Outlet openings from flocculator to clarification zone: (At bottom)

4.u.1 Velocity through opening : 0.005 m/sec

4.u.2 Area of opening : $(920/3600) / 0.005$
= 51.11 sqm

4.u.2 Perimeter of flocculation wall : $14.40 \times 3.14 = 45.21\text{m}$

4.u.3 Height of opening : $51.11 / 45.21 = 1.13 \text{ m}$, Say 1.15m

(Flocculator wall is supported on columns, if 8 columns are provided, size of each opening approximately is $45.21 / 8 = 5.61\text{m} \times 1.15\text{m}$) :

4.u **Collection launder** : Two semi-circular channels

4.u.1 Flow per channel : $920 / 2$
: 460 cum / hr

4.u.2 Width : 0.75 m

4.u.4 Velocity : 0.60 m/sec

4.u.3 SWD (Side Water Depth) : $460 / (3600 \times 0.75 \times 0.60)$
: 0.28m, say 0.40m

5. Rapid sand gravity filter beds (4 beds)

5.a No of beds proposed : 4, four

5.b Capacity / Flow rate : 920 cum / hr

5.c Flow rate per bed : $920 / 4 = 230 \text{ cum / hr}$

5.d Configuration of bed : Two sections with central gutter (gullet)

5.e Let Net area of filtration/bed : $6.0 \text{ m} \times 4.2 \text{ m}$ each section x 2
: $25.20 \times 2 = 50.4 \text{ sq. m.}$

5.f Rate of filtration : $230 / 50.4 = 4.56 \text{ cum / sqm. / hr}$
(less than 5 cum/sqm/hr, hence OK)

5.g.1 Width of central gutter : $0.60\text{m I.D.} + 0.20\text{m} + 0.20\text{m} = 1.00\text{m}$
(Wall thickness 0.20m)

5.g.2 Width of filter unit : $4.2\text{m} + 1.00\text{m} + 4.2\text{m} = 9.40\text{m}$

5.g Overall size of bed : $9.40\text{m} \times 6.0 \text{ m} \times 3.65 \text{ m SWD}$

Ratio of length to width : $9.40 / 6.0 = 1.56$, hence OK

5.h Depth of filter bed

5.h.1 Depth of supporting gravel : 0.60 m (size 2 mm to 50 mm, graded)

Part A: Engineering Design

5.h.2	Depth of filter silica sand	: 0.75 m (E.S.. 0.55 mm, U.C. 1.5 to 1.7)
5.h.2	Depth of water column over media	: 2.30 m

	Total depth	: 3.65 m
5.h.3	Free board	: 0.50 m

5.i	Backwash flow rate	: Separate wash of both sections
5.i.1	Backwash flow rate	: 36 cum / sq m / hr
5.i.2	Flow rate	: 25.20 sq m x 36 = 907.2 cum / hr
5.i.3	Duration of Backwash	: 10 min/bed
5.i.4	Requirement per wash	: 907.2 / 60 x 10 : 151.2 cum.
5.i.5	Provide capacity	: 151.2 x 2 = 302.4, Say 315 cum
5.i.6	Wash-water tank filling pumps (To fill the tank in two hours)	: 160 cum/hr (1W +1S) @ 15 MWCX
5.i.7	Backwash header to Filter Beds	: Diam 0.40 m (velocity = 2.00 m / sec)
5.i.8	WWT Head	: Effective head 8.50m + pipe losses
	Frictional head-loss in pipe	: $Q = 907.2 \text{ cum/hr}$, Diam 0.35m $H_f = 14.2\text{m}/1000 \text{ m}$
5.i.9	Length of pipe	: 60m
5.i.10	Total losses	: $(14.2 / 1000) \times 60 = 0.85\text{m}$
5.i.11	Head of tank	: $8.50 + 0.85 = 9.35\text{m}$, say 9.50m

5.j Air scour : Separate scour of both sections

5.j.1	Rate of air scour	: 36 cum / sq m / hr
5.j.2	Capacity of blower	: 25.2 sqm x 36 = 907.2 cum / hr : provide 910 cum / hr
5.j.3	No of blowers	: 1 W + 1 S
5.j.4	Pressure	: 0.40 kg / sq cm, (4.0 MWC)
5.j.5	Air scour header	: diam 150 mm (velocity = 14.26 m / sec)

5.k. Wash-water troughs

5.k.1	No of troughs / bed	: 6, (six), three per section
5.k.2	Flow rate / trough	: $907.2 / 3 = 302.4 \text{ cum / hr}$

- 5.k.3 Size of trough : 0.084 cum / sec
 $Q = 0.084 \text{ cum/sec}$: $Q = 1.376 b h^{3/2}$
 $b = \text{width } 0.35 \text{ m}$: $0.084 = 1.376 \times 0.35 \times h^{3/2}$
: $h = 0.31 \text{ m}$
- 5.k.4 Provide height of trough : 0.35 m
- 5.k.5. Size of trough provided : 0.35 m (width), 0.35 m (height)
- 5.k.6 Lateral travel of water : $\{6.0 - (0.5 \times 3)\} / 6 = 0.75 \text{ m}$
(Length of filter bed = 6.0, Width of cross-trough = 0.50m including wall thickness)

5.l Central wash disposal gutter

- 5.l.1 Flow rate : 907.2 cum / hr
- 5.l.2 Size of gutter : $Q = 1.376 b h^{3/2}$
 $Q = 0.25 \text{ cum/sec}$: $0.25 = 1.376 \times 0.60 \times h^{3/2}$
 $B = \text{width } 0.60 \text{ m}$: $h = 0.45 \text{ m}$
- 5.l.3 Diam of disposal pipe : 0.40m (Velocity : 2.00 m/sec)
- 5.l.4 Velocity head ($h = V^2 / 2g$) : $(2.00)^2 / (2 \times 9.81) = 0.20 \text{ m}$
- 5.l.5 Total water depth in gutter : 5.l.2 + 5.l.3 + 5.l.4
(0.45 + 0.40 + 0.20 = 1.05m)
- 5.l.6 Size of gutter provided : width 0.60 m (ID), 1.00 m (OD)
: height 2.10 m

5.m. Filter ports:

- 5.m.1 Filter inlet gate : One per bed, Size 0.35m x 0.35m
($Q = 230 \text{ cum / hr}$) : (velocity = 0.52 m / sec)
- 5.m.2 Filter outlet, B/F valve : Two per bed, diam – 0.20m (200mm)
($Q = 115 \text{ cum / hr}$) : (velocity = 1.02 m / sec)
- 5.m.3 Wash-water inlet, B/F valve : Two per bed, diam – 0.40m (400mm)
($Q = 907.2 \text{ cum / hr}$) : (velocity = 2.00 m / sec)
- 5.m.4 Wash-water disposal, B/F valve : One per bed, diam – 0.40m (400mm)
($Q = 907.2 \text{ cum / hr}$) : (velocity = 2.00 m / sec)
- 5.m.5 Air scour inlet, B/F valve : Two per bed, diam – 0.15m (150mm)
($Q = 907.2 \text{ cum / hr}$) : (velocity = 14.55 m / sec)

5.n Filter Under-drain system : for each section

- 5.n.1. Area of perforations : 0.30 % of bed (section) area

- 5.n.2 Diam of perforations : (25.2) x 0.30 / 100 = 0.076 sqm
- 5.n.3 No of perforations : 962
- 5.n.4 Spacing of laterals : 300 mm c/c on both sides
- 5.n.5 No of laterals : $6.0 / 0.30 \times 2 = 40$
- 5.n.6 No of perforations / lateral : $962 / 40 = 24$
- 5.n.7 Length of lateral : 1.95 m each
- 5.n.8 Spacing of perforations : $1.95 / 24 \times 2 = 0.162$ m c/c in two rows
Say 0.15m c/c
- 5.n.9 Diam of lateral : 90 mm OD Rigid PVC
: 10 kg / sq cm, (85 mm ID)
- 5.n.10 Total area of laterals : $3.14 / 4 \times (0.085^2) \times 40 = 0.23$ sqm
- 5.n.11 Ratio-area of laterals to area of perforations : $0.23 / 0.076 = 2.98$
- 5.n.12 area of manifold pit : 2 x area of laterals : $2 \times 0.23 = 0.46$ sq m
- 5.n.13 Provide Area of manifold pit : $0.70 \text{ m} \times 0.70 \text{ m} = 0.49$ sq m
- 5.n.14 Ratio of area of manifold pit to area of laterals : $0.49 / 0.23 = 2.13$

5.o Filter inlet channel

- 5.o.1 No of channels : 1, One
- 5.o.2 Flow rate / channel : 920 cum / hr
- 5.o.3 Width : 0.75 m
- 5.o.4 Velocity : 0.60 m / sec
- 5.o.4 SWD : 0.57 m, provide 0.65m

5.p Filter Inlet and Outlet weir (No of chambers = 4)

- 5.p.1 Flow rate : $920 / 4 = 230$ cum / hr
- 5.p.2 Head over weir : $Q = 1.84 b h^{3/2}$
 $Q = 0.064$ cum/sec : $0.064 = 1.84 \times 1.00 \times h^{3/2}$
 B = length of weir 1.00 m : $h = 0.10$ m

5.q Pure (Filtered)water channel

- 5.q.1 Flow rate : 920cum / hr
- 5.q.2 Width : 1.25 m
- 5.q.3. Velocity : 0.60 m /sec
- 5.q.4 SWD : 0.34m, say 0.45 m

5.s Filtered water pipe to Pure Water Sump

- 5.s.1 No of pipes : 1, One, Diam 0.60m (600 mm)
 5.s.2 Flow rate / pipe : 920 cum / hr
 5.s.3 Velocity : 0.92 m / sec

5.t Pure Water Sump/ MBR

- 5.t.2 Capacity of sump : 920 cum (1 hr)
 5.t.5 Side Water Depth : 3.50 m
 5.t.5 Diameter : 18.3m, say 18.50 m,
 FB 0.50m

6. Chemical / coagulant dosing system**6.a Alum dosing system**

- 6.a.1 Dose considered : 50 mg / lit
 6.a.2 Flow : 920 cum / hr
 6.a.3 Alum consumption : 46 kg / hr
 6.a.4 Strength of solution : 10 %
 6.a.5 Rating of tanks : 8 hr (3 no tanks)
 6.a.6 Volume of alum solution per shift : 46 kg/hr x 8 x (100 / 10)
 : 3680 lit , say 4000 lit
 6.a.7 Dosing pumps : 4000/ 8 = 500 lit , say 0-750 lit/hr
 (1W +1S) @ 15 MWC

6.b Lime dosing system

- 6.b.1 Dose considered : 25 mg / lit
 6.b.2 Flow : 920 cum / hr
 6.b.3 Lime consumption : 23 kg / hr
 6.b.4 Strength of solution : 10 %
 6.b.5 Rating of tanks : 12 hr (2 no tanks)
 6.b.6 Volume of lime solution per shift : 23 kg/hr x 12 x (100 / 10)
 : 2760 lit , say 2800 lit
 6.a.7 Dosing pumps : 2800/ 12 = 233 lit , say 0-400 lit/hr
 (1W +1S) @ 15 MWC

6.c TCL Powder dosing system (Emergency Disinfection)

6.c.1 Dose considered	: 3 mg / lit of free chlorine
6.c.2 Flow	: 920 cum / hr
6.c.3 TCL consumption	: 2.76 kg / hr
6.c.3.a Available chlorine	: 25%
6.c.3.b Powder requirement	: 11 kg/hr
6.c.4 Strength of solution	: 10 %
6.c.5 Rating of tanks	: 8 hr (3 no tanks)
6.c.6 Volume of lime solution per shift	: 11 kg/hr x 8 x (100 / 10)
	: 880 lit , say 900 lit

7. Pre and Post Chlorination

7.b.1 Pre - Dose considered	: 5 mg / lit of chlorine
7.b.2 Flow	: 920 cum / hr
7.b.3 Chlorine requirement	: 4.6 kg / hr, say 5 kg/hr
7.b.4 No of chlorinators	: 2 , Two, 1W +1S
7.b.5 Post - Dose considered	: 3 mg / lit of chlorine
7.b.6 Flow	: 920 cum / hr
7.b.7 Chlorine requirement	: 2.76 kg / hr, say 3 kg/hr
7.b.8 No of chlorinators	: 2, Two (1W + 1S)

8. Wash-water re-circulation sump

8.a Capacity required	: 315 cum (For one filter backwash)
	Provide capacity : 325 cum
8.b SWD	: 2.50 m
8.d Diam	: 13.0m , FB : As per site condition

9.e. Wash-water re-circulation pumps

9.e.1 Quantity of one filter backwash (10 min)	: 325 cum
9.e.2 No of filters backwashed per day	: 325 x 4 beds in one day = 1300 cum
(Worst case assumed for design)	
9.e.3 Re-circulation operation period	: 16 hr
9.e.4 Re-circulation flow rate	: 1300 / 16 = 81.25 cum/hr
9.e.5 Capacity of pumps provided	: 85 cum/hr
9.e.6 No of pumps	: 2, Two (1W + 1S) @ 20 MWC

10. Sludge Treatment & Disposal**Sludge Mass Balance: Design Capacity :**

- A) TSS in Raw water (Average) : 100 mg/L (Silt and Turbidity).....Assumption
- TSS due to Alum dose (50mg/L) : 13 mg/L (50 x 0.26)
- TSS due to lime dose (25 mg/L) : 25 mg/L
-
- Total TSS : 138 mg/L, say 150 mg/L
- Rate plant capacity : 920 cum/hr
- Sludge consistency from clarifiers : 2% W/V (20,000 mg/L)
- Mass balance :
- Total Input solids = Total output solids + Total solids in the sludge
- $[(920) 150 = (920 - s) \times 10] + 20000 s$
- Where s : sludge flow from clarifiers, cum/hr
- Hence, s = 7.0 cum/hr
- B) Thickener underflow consistency : 5% W/V (50,000 mg/L)
- Mass balance :
- $(20000 \times 7.0) = (7.0 - s) 100 + 50000 s$
- Where s : sludge flow from Thickeners, cum/hr
- Sludge flow to Thickener = 3.0 cum/hr
- Hence, s (Feed to centrifuge) = 3.0 cum/hr
- No of Centrifuges = 1W +1S
- Overflow (supernatant) from Thickener : $7.0 - 3.0 = 4.0$ cum/hr
- C) Centrifuge treated sludge consistency = 25%
- Centrate from Centrifuge: $3.0 \times 0.75 = 2.25$ cum/hr

12. Sludge Sump for clarifiers:

- 12.a Flow Rate (sludge) : 7.0 cum/hr
- 12.b Detention time : 2 hr
- 12.c Capacity : 14.0 cum
(Size: Suitable)
- 12.d Sludge pumps : 7.0 cum/hr @ 20 MWC ,
(1W +1S)

13. Sludge Thickeners

No of units	: 1, One
13.a Flow Rate (Sludge from clarifiers):	7.0 cum/hr
13.b Solids loading	: $(7.0 \times 20000 \times 24) / 1000$ = 3360 kg/day
13.c Solids loading rate	: 80 kg/sqm/day
13.d Area required	: $3360 / 80$ = 42.0 sqm
13.g Diam of unit	: 7.31 m , say 7.50m, F.B. 0.50M, each
13.h SWD	: 4.0m

14. Thickened sludge sump

14.a Flow Rate (sludge)	: 3 cum/hr
14.b Detention time	: 2 hr
14.c Capacity	: 6.00 cum (Size: Suitable)
14.d Sludge pumps	: 3.0 cum/hr @ 20 MWC (16 hr), (1W +1S)

15. Centrifuge:

Capacity	: 3.0 cum/hr (1W+1S)
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16. Polyelectrolyte for Centrifuge:

16.a PE Dose	: 1 kg/ ton of dry solids
16.b Dry solids per day	: 3.36 ton
16.c PE requirement /day	: 3.36 kg
16.d. Strength of solution	: 0.5%
16.e Requirement of solution	: $(3.36 \times 100) / 0.5$: 672 lit, say 700 lit
16.f No of tanks	: 1, one
16.g Capacity of pumps	: 0-50 lit/hr (1W + 1S)

Abbreviations:

Cum/hr – cubic meter per hour

SWD – side water depth

FB – freeboard

Cuft/sec – cubic feet per sec

RPM – rotations per min

Sqm – square meter

Rm – running meter

Diam - diameter

M.W.C. –Meter Water Column

F.S.L. – Full Supply Level

T.W.L. – Total water level

W	–	Working,	S	–Standby
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Appendix 8.16: Tube-Clariflocculator or (Clari-Tube Settler) – Circular Configuration
(Concentric flocculation zone with Peripheral annular tube settling zone)

- a. No of units proposed : 1, one
- b. Capacity / Flow per unit : 920 cum/hr
- c. Diameter of central shaft : 0.90 m (ID), 1.30 m (OD)
 (ID –Internal Diam, OD –Outer Diam.) (1.30 = 0.90 +0.20+0.20)
- d. Velocity through shaft : 0.40 m / sec
- e. Size of outlet openings : 0.60 m x 0.30 m x 4 no
- f. Velocity through openings : 0.35 m / sec
- g. Let Diameter of flocculator be : 13.00 m(ID),
- h. Side water depth : 3.50 m
- i. Volumetric capacity : $3.50 \times 3.14/4 \times (13.00^2 - 1.30^2)$
 : 461.3 cum
- j. Detention time : $(461.3/920.) \times 60 = 30.05$ minute
- k. Surface area of flocculation zone (A1) : $461.3 / 3.5 = 131.8$ sqm
- l. Surface loading rate on Tube Settling clarification zone : Peripheral Annular zone
 : 5.0 cum/sqm/hr
 (Tube size 50mm x 50mm, Length 600mm, Inclination 60 degree)
- i. Surface area of clarification (A2) : $920 / 5.0$
 = 184 sqm
- ii. Total area of clariflocculator (A1 + A2) : $131.8 + 184$
 = 315.80 sqm
- iii. Diameter of clariflocculator (I.D.) : $\sqrt{\frac{315.8}{3.14/4}} = 20.04\text{m}$
 Say 20.10 m
- m. Detention time in clarifier zone : swd 3.75m
 Actual volume in clarification zone : $(20.10^2 - 13.0^2) \times (3.14/4) \times 3.75$
 = 691.85 cum
 Detention time : $691.85 / 920$
 = 0.75 hr (45 min)
- n. No of radial collection troughs = 20
- o. Length of trough = $(20.10 - 13.0) / 2 = 3.55\text{m}$

- p. 1. Collection weir length/trough = 3.55m
(Equi-spaced 90 degree V notches (50 mm depth) as collection edges)
2. Total weir length : $3.55 \times 20 = 71 \text{ m}$
- q. Weir Loading Rate: = $(920 \times 24) / 71 = 300 \text{ cum/m/day}$
- i. No of slow mixers / flocculators 2, two per unit
- ii. G value of flocculator agitator : 45-50 m/sec/m
- r. Size of Outlet openings from flocculator to clarification zone: (At bottom)
- i. Velocity through opening : 0.005 m/sec
- ii. Area of opening : $(920/3600) / 0.005$
= 51.11 sqm
- iii. Perimeter of flocculation wall : $14.40 \times 3.14 = 45.21 \text{ m}$
- iv. Height of opening : $51.11 / 45.21 = 1.13 \text{ m}$, Say 1.15m

(Flocculator partition wall fabricated out of 5 mm FRP sheets, supported by MS frame. The partition wall is supported by horizontal cantilevered trusses fixed to the outer RCC wall, below the modules.)

Configuration of Tube Settling zone:

1. Depth of water from collection troughs to top of Tube Modules: 0.90m
2. Height of Tube Modules : 0.52m
3. Depth of water below modules [$3.75 \text{ m} - (0.90 \text{ m} + 0.52 \text{ m})$] = 2.33m
4. Bottom slope 1:12 towards center
5. Sludge scrapper bridge : Either centrally driven or peripherally driven

s. **Collection launder (Outside)** : Two semi-circular channels

- i. Flow per channel : $920 / 2$
: 460 cum / hr
- ii. Width : 0.75 m
- iii. Velocity : 0.60 m/sec :
- iv. SWD (Side Water Depth) : $460.4 / (3600 \times 0.75 \times 0.60)$
: 0.28m, say 0.40m

Note: Flocculation tanks and Tube settling tanks can be also configured in rectangular shape with a common partition wall. In such case multiple hoppers are provided at the bottom of tube settlers with slope not less than 50 degrees for sludge removal.