

## **CHAPTER 8**

### **8. Conventional Water Treatment**

## **CHAPTER 8: CONVENTIONAL WATER TREATMENT**

### **8.1 Introduction**

The objective of water treatment is to treat source water, in a cost-effective manner that meets drinking water standards of BIS code IS 10500 (2012) and its amendments.

#### **8.1.1 Methods of Treatment**

The method of treatment to be employed depends on the nature of raw water constituents. The traditional surface water sources are streams, rivers, tributaries, manmade and natural lakes, and canals. Traditional groundwater sources are dug wells and bore wells. The seawater, brackish water, and reuse of treated sewage (indirect potable use) are considered as alternative 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 or their combination, viz., coagulation, flocculation, clarification, filtration, membrane, and disinfection is a must. The degree of treatment processes required is directly related to the quality of raw water. Recently, biological filtration (after conventional filtration) is also included to remove organics and Disinfection By-Products (DBPs) to some extent. Biological Filtration means conventional high rate filters backwashed without chlorinated water. The degree of treatment shall be directly related to the quality of raw water including traditional and emergent contaminants. Chapter 7 of the Part A 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 DBPs. In such situation, a different treatment technology such as enhanced coagulation, pressure or submersible membranes has to be resorted to (details are given in Part A chapter 10 Specific water treatment system).

Typical sizing of units of conventional WTP is enclosed in **Annexure 8.1**.

#### **8.1.2 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 groundwater 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 contributes to organic load. Microbial contaminants, viz., algae are also present in many polluted waters in lean flow season. Jar tests with different coagulants and coagulant aid give a representative analysis of settleable floc formation. BOD should not be ideally more than 3 mg/L as per Standards notified by CPCB in 2007 as mentioned in the table 7.3 on Chapter 7 of Part A of this manual. However, if the raw water source is having BOD of 5 mg/L or less, it is recommended to adopt conventional treatment with post chlorination.

#### **8.1.3 Suggested Line of Treatment Options for Contaminated Surface Water Sources**

Due to excessive river pollution as a result of constituents mentioned in the Chapter 7, the organic and microbial contaminants increase beyond the desirable limits. Conventional technologies become

ineffective for BOD value consistently exceeding 5 mg/L throughout the year, therefore, the options of alternative/advanced technologies are needed to be adopted.

**a. Status of Polluted River Stretches as per CPCB Report, 2022**

As per the Report “Polluted River Stretches for Restoration of Water Quality” – 2022, Central Pollution Control Board (CPCB) assessed the water quality of 603 rivers of which 311 stretches of 279 rivers in 30 States and Union Territories are polluted (BOD more than 3 mg/L the Surface Water Quality Criteria of CPCB for drinking water sources). Following is the status of polluted river stretches in terms of BOD given in Table 8.1:

**Table 8.1: Status of Polluted River Stretches in terms of BOD**

Priority	Number of polluted stretches	No. of States/UTs	BOD Level mg/L
Priority 1	46	18	More than 30
Priority 2	16	10	20 - 30
Priority 3	39	16	10 - 20
Priority 4	65	21	6 - 10
Priority 5	145	26	3 - 6
<b>Total</b>	<b>311</b>	-	-

Progressively over the past, the wastewater (partially treated or untreated) has been discharged into the water bodies (traditional water sources). As the wastewater effluent and the pesticides leached from the agricultural fields discharged into the water bodies contain various contaminants including micro-pollutants, Endocrine Disrupting Chemicals etc., it is important to adopt high degree of treatment to protect the public health from the water related *life threatening* diseases. Conventionally, when organic content of source water is high, the pre-chlorination process in use may form Disinfection By-Products (DBPs) including Trihalomethanes (THMs), which are suspected carcinogenic. Therefore, the conventional water treatment processes are ineffective-to treat such variety of contaminants. The BOD is an indicator of contamination of the raw water source as most of the river stretches in India have high BOD as stated above.

It has been observed that ULBs are making huge investments for treating wastewater where the cost of conventional wastewater treatment per MLD is 3 to 4 times than that of the conventional water treatment plant. In order to maintain drinking water quality as per BIS and supply safe and protected water with a view to safeguard public health, the degree of treatment for raw water sources may be required to be enhanced with a judicious combination of appropriate treatment technologies by making additional investments.

**b. Nature of contaminants during lean flow season:**

1. The contaminants during the lean flow season are a mixture of suspended and dissolved inorganic and organic solids.
2. These are difficult to treat in conventional treatment processes comprising aeration, coagulation, flocculation, clarification, filtration, and disinfection
3. Inorganic solids have a fast settling velocity, as against the smaller velocity of biological solids. It is a mix, depending on the raw water source.

**c. Constitution of biological solids (Soluble or non-soluble)**

1. NOM (Natural Organic Material – Humic and Fulvic acids) or SOM (Synthetic Organic Material) contribute to BOD, COD, and TOC in the raw water.
2. Algae is another pollutant that grows due to the presence of Nitrogen & Phosphorus contributed by untreated or partially treated sewage in the raw water.

While the following multi-prong strategy can be adopted by the State Government/ULBs to improve surface water source quality, treatment of low-strength remains inevitable at several locations:

1. Improving compliance with Hon'ble National Green Tribunal wastewater discharge standards by augmenting (wastewater) treatment plants
2. Catchment protection
3. Maintaining minimum environmental (ecological) flow in rivers for sufficient dilution

ULBs should appropriately identify drinking water sources by avoiding polluted stretches. The river stretches having the value of BOD<sub>5</sub> equal to or less than 5 mg/L can be directly used as a source for water supply.

Further, attempts by the State authorities/ULBs should be made to,

- (i) improve source water quality by addition of a complementary treatment unit in the existing water treatment plant to treat polluted (alternatively low-strength wastewater) water in such river stretches, and
- (ii) manage the ecological quality of rivers by maintaining a minimum flow (Rivers must not dry-up or have their physical regimes significantly altered in order to conserve the hydrological and ecological functions of their drainage networks) since, Hon'ble National Green Tribunal (NGT) in OA 498 of 2015 has directed all States to maintain a minimum environmental flow of 15-20% of the average lean season flow in their rivers.
- (iii) The river authorities should consider the above flows (15-20%) to be discharged in the river to maintain the computation of water balance of the river sub basin.

**d. Recommended Treatment Technologies for Source Raw Water with BOD**

The BOD criteria for a raw surface water source to be used for drinking purpose as per CPCB is 3mg/L or less with conventional treatment. The BOD criteria may be relaxed up to 5mg/L with conventional treatment systems. However, the BOD level is varying from 3mg/L to more than 30mg/L in many Indian River water stretches. Thus, the following treatment technological Options/Units, are recommended in Table 1 for different range of BOD levels:

**Table 8.2: Recommended Treatment Technological Options/Units for Raw Water with different BOD levels**

Sr. No.	BOD <sub>5</sub> in mg/L	Recommended Technology/Units
1	5 or less	Conventional treatment with post chlorination to maintain the residual chlorine upto 0.2 mg/L at the fag end of the distribution system. If algae is found in raw water the necessary pre-treatment shall be given as suggested in Chapter 10.
2	> 5 - 10	Conventional treatment with Enhanced Coagulation - UV – post chlorination
3	> 10 - 20	Cascade aeration – conventional treatment with enhanced coagulation – Tertiary treatment such as Granular / <sup>*</sup> Biological Activated Carbon (GAV/BAC) – UV and chlorination

Sr. No.	BOD <sub>5</sub> in mg/L	Recommended Technology/Units
4	>20 - 30	<b>Option1:</b> Cascade aeration – conventional treatment with enhanced coagulation - tertiary treatment – Ozone, Granular /Biological Activated Carbon (GAV/BAC) – UV (optional as it reduces the chlorine dose) and minimal dose of chlorination to disinfect the residual micro-organisms in the distribution pipelines <b>Option 2:</b> Cascade aeration – Sedimentation ( if turbidity > 50 NTU) - Granular /Biological AC with Empty Bed Contact Time (EBCT) 10 minutes, ultrafiltration– UV and chlorination to disinfect the residual micro-organisms in the distribution pipelines
5	> 30 - 50	Attached Growth Biological Treatment - ultrafiltration, Ultra Violet (UV) and minimal dose of chlorination to disinfect the residual micro-organisms in the distribution pipelines
6	<b>Raw water having BOD<sub>5</sub> &gt; 50</b>	Attached Growth Biological Treatment - Granular Activated Carbon, Ultrafiltration, Ultra Violet (UV). This treated water is added to water bodies like lakes, ponds as an environmental buffer and then drawn into conventional water treatment for further treatment as desired with chlorination to disinfect the residual micro-organisms in the distribution pipelines. In case the TDS levels are significantly higher (more than 2000 mg/L) reverse osmosis (RO) can be used after ultrafiltration. This shall be considered in the absence of any alternative source.

## Note:

- i. It is advisable to carry out water quality analysis of the raw water sources and conduct pilot scale of 0.1 MLD studies/demonstration of the above stated options (S. Nos. 3, 4, 5 & 6) for raw water having BOD<sub>5</sub> more than 10 mg/L to ascertain design and operational parameter before embarking on large scale treatment plants. Public Awareness Programs / Campaigns shall be conducted before the project implementation.
- ii. When the water quality changes w.r.t. time in the river or any water source, design the treatment for the worst scenario considering the water quality of the raw water sources during the last 5 years in consultation with the state board. Based on the quality required, treatment units (some of the units may require by-pass) may be operated.
- iii. Various Water Treatment Systems/Processes/Technologies are explained in details of following chapters of Part A of the Manual as follows:
  - Conventional Water Treatment processes: Chapter 8
  - Disinfection Options: Chapter 9
  - Specific Water Treatment Processes/Technologies: Chapter 10
- iv. Additional units may be added (in case of high turbidity, colour etc.) suiting to the raw water quality as per the local conditions.
- v. \*Biological Activated Carbon (BAC) is Granular Activated Carbon (GAC) only. BAC is an operational mode. When Granular Activated Carbon is operated in Biological active mode, it is termed as BAC. In BAC mode, one does not back wash the filter with chlorinated water like normal filters. This is done to avoid loss of bacterial growth in the GAC bed. So in BAC mode the filter is backwashed with non-chlorinated water. In BAC mode, it is more effective in removing low BOD or Natural Organic Material (NOM)/ TOC through bacterial metabolism.
- vi. Treatment options as suggested at S. Nos. 5 and 6 should only be considered if no alternate and economically sustainable water source is available.

#### **8.1.4 Groundwater with High TDS**

In many coastal zones in India, the groundwater has TDS in the range of 1,000 to 5,000 mg/L (brackish to saline). In such cases, low-pressure RO membranes need to be included in the treatment process.

#### **8.1.5 Conventional Water Treatment Options**

The unit operations in water treatment include aeration, flocculation (rapid and slow mixing), clarification, filtration, disinfection, water conditioning, and many different combinations of these to suit these requirements. The treatment technologies for removal of emergent contaminants and DBPs have been included in Part A – 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.

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.

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.

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

In surface waters with turbidity not exceeding 50 NTU and where a sufficient area is available, plain sedimentation followed by slow sand filtration and disinfection is practised mostly in rural areas.

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

Sometimes, an unconventional treatment 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). Such raw waters are applied to the rapid sand filters with alum addition accompanied by slow mixing for a short period (10 min.).

However, due to lack of reliable classification of raw water sources (quality), full-fledged treatment is adopted at most of the places.

Water with excessive hardness needs softening or demineralisation by ion exchange.

#### **8.1.6 Plant Capacity and Hydraulic Overloading**

Hydraulic loading is defined as the volume of raw water or the water (design flow for next 30 years from the base year) applied to the unit operation per time period.

The treatment plant needs to be designed for 20% hydraulic overloading (not the process units). The interconnecting piping, channels and conduits 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 is maintained in the open channels and process units. However, if the raw water source is subjected to high turbidity during flash floods, coagulant/coagulant aids dose may be increased proportionately for such flash period, limiting hydraulic overloading to 20% and diverting excess hydraulic overloading to maintain economy as the interconnections and channels are designed for 20% hydraulic overloading only.

Generally, the plant may be categorised based on the capacity as given in Table 8.3.

**Table 8.3: Size of the Plant**

S. No.	Size of WTP	Capacity
1.	Small	Less than 5 MLD
2.	Medium	5-25 MLD
3.	Large	More than 25 MLD

### 8.2 Pre-Sedimentation and Storage

The turbidity of raw water from rivers and streams may exhibit wide fluctuations, and values exceeding 500–1,000 NTU are not uncommon during high flow season (monsoon). The river's sediment load during floods chiefly derives from soil erosion and consists predominantly of coarse suspended solids. In North India, snowmelt in summer can cause high silt loads. 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.

For pre-sedimentation, detention periods of two hrs. to four hrs. have been recommended. These plain sedimentation tanks can be constructed with conventional construction material or dug out of the earth with sloping sides. At least two tanks are provided if the settled solids 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

In water treatment, aeration is practised for four purposes:

- a) To enhance the aesthetic purpose or value of the water treatment complex as a whole for surface water as well groundwater. Aeration techniques are commonly used in pond, lake, and reservoir management to address low oxygen levels or algal blooms.
- b) To add oxygen to water to impart freshness, e.g., water from underground sources devoid of or deficient in oxygen so as to maintain DO level of 6-8.5 mg/L.
- c) Expulsion of carbon dioxide, hydrogen sulphide, and other volatile substances, including volatile organic compounds (VOC), causing taste and odour.
- d) Oxidation of iron, manganese, etc., from groundwater (water from deeper layers of rocks formation when contain iron, manganese, etc.). Iron and manganese removal improves with higher pH; therefore, aeration itself, which often raises pH due to stripping of CO<sub>2</sub>, improves iron and manganese removal. Between iron and manganese, iron oxidises better at lower pH.

#### 8.3.1 Types of Aerators

Following is the list of various types of aerators that the design engineer should consider and apply an appropriate type of aerator based on the raw water quality and site-specific conditions while understanding the pros and cons of each of the types of the aerators.

##### 8.3.1.1 Spray Aerators

Water is sprayed, through nozzles, upward into the atmosphere and broken up into either a mist or

droplets. Water is either directed vertically or at a slight inclination to the vertical. The installation consists of 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 (2 to 9 m), and the discharge ratings per nozzle vary from 18 to 36 m<sup>3</sup>/hr. Usually, an aerator area of 0.03 to 0.09 m<sup>2</sup>/m<sup>3</sup>/hr. of design flow is provided.

The diameters of the pipe grid are very important 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 atomised 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.

### **8.3.1.2 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 a large water surface is exposed to the atmosphere. By adding of more trays, the time of contact can be increased. The space requirements vary from 0.013 to 0.042 m<sup>2</sup> per m<sup>3</sup>/hr. of flow. Natural ventilation or forced draft is provided. Removal efficiencies varying from 65% to 90% for CO<sub>2</sub> and 60% to 70% for H<sub>2</sub>S have been reported.

### **8.3.1.3 Cascade Aerators**

This is the most commonly used aerator for surface water treatment plants (Figure 8.1). 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 m<sup>2</sup>/m<sup>3</sup>/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 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 CO<sub>2</sub> and up to 35% for H<sub>2</sub>S. Well-designed circular cascades aerators enhance the aesthetics of the treatment plant.



**Figure 8.1: Cascade Aerators with Four to Six Steps**

#### **8.3.1.4 Diffused Aerators**

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 m<sup>3</sup> to 1 m<sup>3</sup> of air per m<sup>3</sup> 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/m<sup>3</sup>/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. The designer is required to appropriately size the air compressor.

### **8.4 Measurement of Flow**

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

#### **8.4.1 Triangular Notches or V-Notch**

There are generally three 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 up to about 1.25 m<sup>3</sup>/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 watertight 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 three to four times the maximum depth of flow over the notch.

**iv. Limitations**

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

**v. 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 m<sup>3</sup>/s to 1.25 m<sup>3</sup>/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 (m<sup>3</sup>/s) through a rectangular notch with end contractions is given by the equation:

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

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 up to 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/s<sup>2</sup>); 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 (m<sup>3</sup>/s) through a rectangular notch without end contractions is given by the following expression:

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

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, for accuracy similar to other flume types, its use requires application of different equations based on the throat size. The important thing to be observed is that the length of approach channel should be 10 times the 'throat width'. The length of downstream channel should be seven times the 'throat width' of the flume.

IS: 14371-1996 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 (m <sup>3</sup> /sec)
H <sub>w</sub>	= Head on weir (m)
H <sub>a</sub>	= Depth at entrance to the flume at specified measuring point (m)
B	= Length of the weir (m)
W	= width of throat (m)

The below listed IS codes of practice describes various methods of flow measurement which may be referred to,

- a) IS 16698: 2019 Hydrometry — Selection, Establishment and Operation of a Gauging Station
- b) IS 1192: 2013 (Reaffirmed Year: 2018) Hydrometry - Measurement of Liquid Flow in Open Channels Using Current-Meters or Floats.
- c) IS 6330 (2012, Reaffirmed Year: 2022): 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: 2020 Hydrometry – Open Channel Flow Measurement Using Thin-Plate Weirs (Second Revision)
- e) IS 13083: 2017 (Reaffirmed Year: 2022) Hydrometric Determinations – Flow Measurement in Open Channels Using Structures – Flat -V Weirs (First Revision)
- f) IS 14673: 2022 Hydrometry-Open Channel Flow Measurement Using Triangular Profile Weirs
- g) IS 14869: 2016 (Reaffirmed Year: 2021) Flow Measurement Structures - Rectangular, Trapezoidal and U-Shaped Flumes (First Revision)
- h) IS 14974: 2018 Hydrometry – Open Channel Flow Measurement Using Rectangular Broad-Crested Weirs (First Revision)
- i) IS 14975 (2001, Reaffirmed Year: 2022): 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.
- j) IS 15123 (2002, Reaffirmed Year: 2022): 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.

- k) IS 15353 (2003, Reaffirmed Year: 2018): 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.

#### **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**

In a 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 rotates 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. In water treatment plants, a small chamber is constructed on the upstream side of weir/notch or a flume to place the float in that. The upstream level is transferred to the float chamber by means of a small pipe at the bottom.

##### **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 centre 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 metre 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 depend 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 flowmeter is that the flow rate depends on a cross-sectional velocity profile.

##### **8.4.4.5 Electromagnetic Probe Method**

When an electromagnetic 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 metre 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 metre.

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

## 8.5 Coagulation and Flocculation

'Coagulation' describes the effect produced by the addition of a coagulant and coagulant aids to raw water, resulting in particle destabilisation and charge neutralisation. This is achieved by rapid intense mixing in a unit called as a 'Flash Mixer' or 'Rapid Mixer' for obtaining uniform dispersion of the coagulant.

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

### 8.5.1 Rapid Mixing (Options for Coagulation)

The coagulant, coagulant aids and/or pH adjustment chemicals are 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, static, and pneumatic mixers.

Where head loss through the plant is to be conserved as much as possible and where the flow exceeds 300 m<sup>3</sup>/hr., mechanical mixing also known as flash mixing, is desirable. For larger plants, multiple units must be provided. Normally a detention time of 30 to 60 seconds 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<sup>3</sup> to 3 watts/m<sup>3</sup> of flow per hour, is usually required for efficient flash mixing. Gravitational or hydraulic devices are simple but not flexible, while mechanical mixers or pneumatic devices are flexible, but require external power and maintenance of rotary parts except for static mixers.

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

- a) Gravitational or Hydraulic:
  - i) Hydraulic jump/Weir mixers,
  - ii) Baffled channel, and
  - iii) In-line mixers Gravitational or Hydraulic Devices
- b) Mechanical:
  - i) Impeller/Propeller type mixer, and
  - ii) Pneumatic type mixer

#### a) Gravitational or Hydraulic:

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.

##### (i) Hydraulic Jump Mixing

This is achieved by a combination of a chute followed by a channel with or without a sill. The chute creates a 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.

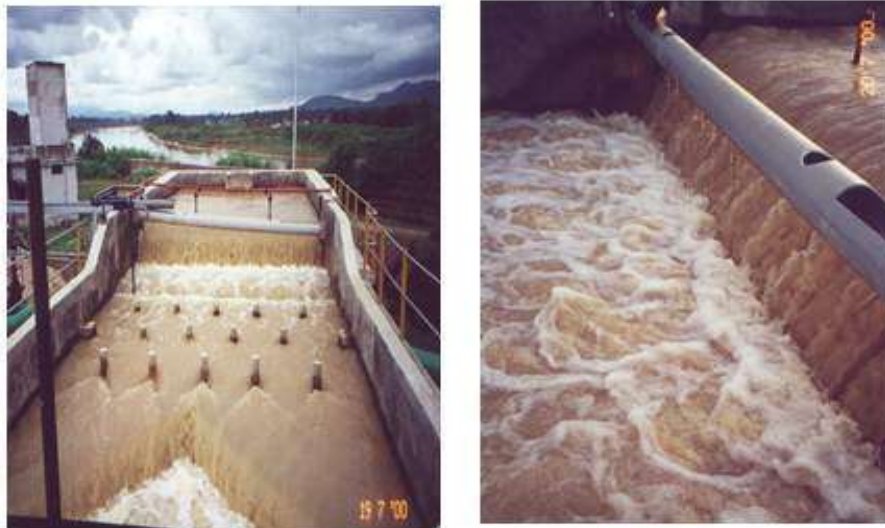
##### (ii) Weir Mixer

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 (Figure 8.2 & 8.3). This is a rectangular weir or a notch. A chamber of 1 min. detention time is provided on the upstream site of the weir. A freefall of 0.5 m to 0.60 m on the downstream side creates sufficient turbulence (G

value 800 to 1000  $\text{sec}^{-1}$ ) for instantaneous mixing of raw water with coagulants. A small chamber of detention time of 5 to 10 seconds is provided on the downstream side of the thorough mixing area.



**Figure 8.2: Weir Mixer (Rapid Mix Unit)**



**Figure 8.3: Rapid Mix Weir showing Rectangular Weir, Mixing Basin, Coagulant Feeder Pipe**

**(iii) 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  $40^\circ$  to  $90^\circ$  with the channel wall. This angle should ensure a minimum velocity of 1.5 m/s while negotiating the baffle.

**(iv) 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.

### Mechanical Devices

#### (i) Propeller Type Impellers/Turbines

These 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  $300 \text{ s}^{-1}$  (m/sec/m) or more. Power requirements are ordinarily 1 to 3 watts per  $\text{m}^3/\text{hr}$ . of flow.

#### (ii) 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  $3,000 \text{ s}^{-1}$  to  $5,000 \text{ s}^{-1}$  and 0. 5 s to 0. 4 s. respectively.

#### 8.5.1.1 Location of Coagulant Dosing Points

The primary coagulant is dosed just before the zone of maximum velocity gradient (turbulence) in the tank. For weir mixers, it is located over the ‘nappe’ of a 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 to 10 seconds in the downstream mixing channel. In the mechanical agitator, the 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. Typical Dosing point locations in the WTP with Flash Mixer are shown in Figure 8.4.



Figure 8.4: Dosing point locations in the WTP with Flash Mixer

#### 8.5.1.2 Undesirable Dosing Practices

Dumping of alum blocks directly into the elevated channel or mixing channel must be totally avoided and is undesirable. The alum mixing and dosing becomes non-uniform resulting in deteriorated treated water quality as shown in Figure 8.5



**Figure 8.5: Direct dumping Alum block (Not Recommended)**

## 8.5.2 Chemical 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. Also, as different coagulants are to be fed at different points, the location at which the coagulants are fed is important to derive maximum mixing 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 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. However, pressure feed is recommended for proper dosing and mixing.

### 8.5.2.2 Preparation of Solutions

It is essential to ensure that all the solid coagulants are dissolved before the solution is put into raw water and the homogeneity of the prepared coagulant solution is maintained. This can be achieved by proper mixing by mechanical agitation. Manual mixing may be adopted for very small plants with capacities not exceeding 3.0 MLD. The drive mechanism shall be located on the top of the tank. The agitator shaft and paddles shall be of SS 304/316 material.

### 8.5.2.3 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 is prepared in the solution tank and is measured by means of either an orifice rota metre, positive displacement pump, or weirs. The solution feed equipment should be simple in operation and corrosion resistant.

#### 8.5.2.4 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 chemicals into water. The coagulant, in large crystal or lump form, is deposited 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.

##### (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.

##### (c) Electro-Coagulant Feeders

The water flows through an integrating raw water metre 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. Gravity Orifice & Taper valve dosing Box and Gravity feed dosing box with V notch are shown in Figure 8.6 and 8.7 respectively.

##### (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. This is the most desired way of feeding coagulants, coagulant aids, and/or pH adjustment chemicals.



**Figure 8.6: Gravity Orifice & Taper valve Dosing Box**



**Figure 8.7: Gravity Feed Dosing Box with V-Notch**

**(f) Variable Rate Proportional Feeders**

If the rate of flow of water being treated varies, proportional feeding of coagulants is necessary and is a preferred method. 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 metre with an electrical contractor is placed in the raw water line. The contactor closes a circuit every time a given volume of water flows through the metre. The closing of the circuit energises 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 30 seconds, at maximum flow, with the pump operating for approximately 20 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 metre 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 with a mechanical device. Reciprocating / Positive Displacement Pump Chemical Feeders are shown in Figure 8.8.



**Figure 8.8: Reciprocating / Positive Displacement Pump Chemical Feeders**

**8.5.2.5 Dry Feed**

Dry coagulant feeders incorporate a feed hopper 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.

### 8.5.2.6 Coagulants

#### (i) Coagulants/chemicals used and their properties

**Annexure 8.4** gives the list of coagulants and/or chemicals commonly used in water treatment and their properties. The engineer must carefully consider which is the most effective coagulant and/or chemical that should be used for the treating water based on site-specific conditions.

#### (ii) 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 at least 15 centimetres above the floor of the storeroom 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 airtight containers.

All plants, particularly small ones (less than 5 MLD), should keep on hand at all times, a supply of coagulants sufficient to provide a safety factor. A storage of three months is advisable. Minimum storage of one month of monsoon requirement should be provided for larger plants.

Coagulants such as powdered activated carbon which are likely to cause dust problems should 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 oxidising 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 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 microflocs formed during rapid mixing. Alum is a trivalent coagulant (carrying +++ charges on its surface) and it nullifies three colloidal particles (each one carrying -ve charge) and therefore, Van der Waal forces of attraction increases, which forms the microflocs and later on, these microflocs again collide together to form a settleable floc. These flocs can be subsequently removed in settling tanks and filters.

#### 8.5.3.1 Design Parameters

The desirable values of  $G$  in a flocculator varies from  $20 \text{ s}^{-1}$  to  $75 \text{ s}^{-1}$  and  $G.t$  from 2 to  $6 \times 10^4$  for aluminium coagulants and 1 to  $1.5 \times 10^5$  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 efficiency in power input and to reduce possible shearing of particles during floc formation, tapered flocculation is sometimes practised. 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<sup>-1</sup> in the third stage in the direction of flow. Correction factors for Detention Time with respect to water temperature are given in Table 8.4.

**Table 8.4: Correction Factors for Detention Time with respect to Water Temperature**

Temp (° C)	Dynamic Viscosity (k) Ns/m <sup>2</sup>	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	

### 8.5.3.2 Types of Slow Mixers

Similar to rapid mixing units, these are categorised 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. Technologies that include a combination of rapid mix, flocculation, and clarification are very effective treatment process for raw water turbidity of 200 NTU or less and available land is limited. These technologies are known as upflow clarifiers, ContaClarifier, etc.

#### (i) Gravitational or Hydraulic Type Flocculators

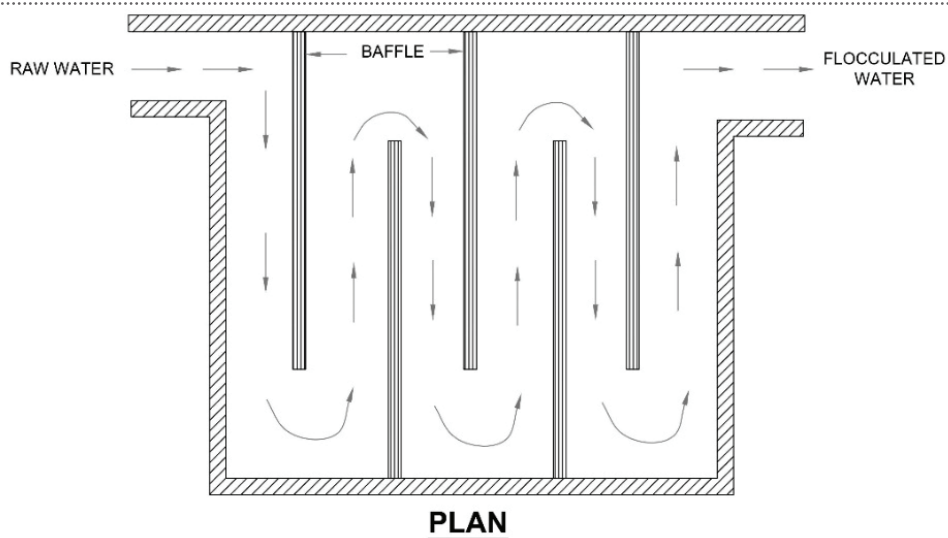
Several types of gravitational or hydraulic flocculators are used in practice which are discussed as follows. All hydraulic flocculators require relatively less maintenance than mechanical flocculators.

##### (a) Horizontal Flow Baffled Flocculator

These flocculators consist of several around-the-end baffles with in-between spacing of not less than 0.45 m to permit cleaning (Figure 8.9 and 8.10). 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 10-100 s<sup>-1</sup>.



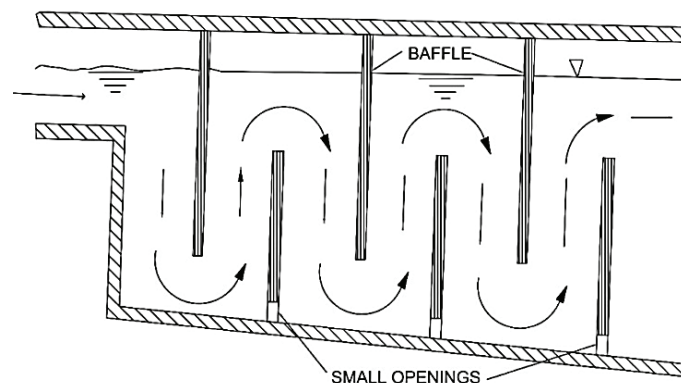
**Figure 8.9: Horizontal Flow Baffled Flocculator**



**Figure 8.10: Plan of a Typical Horizontal Flow Baffled Flocculator**

**(b) Vertical Flow Baffled Flocculator**

In these flocculators, the distance between the baffles is not less than 0.45 m (Figure 8.11). 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-sized treatment plants.



**Figure 8.11: Vertical Flow Baffled Flocculator**

**(ii) Mechanical Paddle Type flocculator**

Paddle flocculators are widely used in practice (Figure 8.12). The design criteria are: depth of tank is = 3 to 4.5 m; detention time is  $t = 10$  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  $\times$  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  $\text{s}^{-1}$  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.

It is desirable to provide more than one compartment in series to lessen the effect of short circuiting. It is desirable to provide a variable frequency drive (VFD) for paddle agitator to have flexible/adjustable  $G$  value as per the jar test.

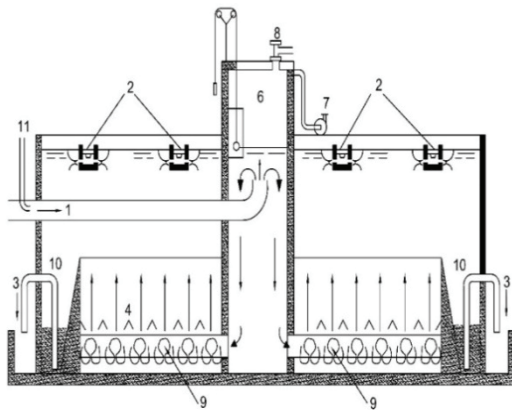


**Figure 8.12: Mechanical Type Vertical Flow Flocculator with Paddles**

**(iii) Pulsating Sludge Blanket Clarifier**

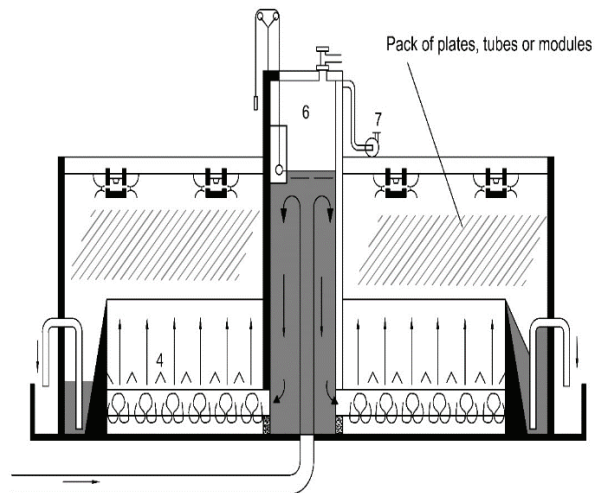
In this type of clarifier, flocculation and clarification zones are superimposed. In flat bottom tanks, the flocculation takes place in the bottom part by using static mixers (Figure 8.13 and 8.14). In an already formed sludge blanket, coagulated water is injected in high flow rate for a very short period. 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 a 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 adaptability to all shapes of tanks. As flocculation and clarification zones are superimposed, it produces far more compact plants. It has operational flexibility and 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.



1. Raw water inlet.  
2. Clarified water outlet  
3. Sludge discharge.  
4. Sillings plates.  
6. Vacuum pump.  
8. Automatic vacuum breaker  
9. Raw water perforated distribution piping  
10. Sludge concentrators.  
11. Keagent inlet

**Figure 8.13: Pulsator Clarifier**



**Figure 8.14: Superpulsator Clarifier**

## 8.6 Sedimentation (Clarification)

Sedimentation is the separation of suspended particles from water by gravitational settling. It is one of the most commonly used unit operation of conventional water treatment. Sedimentation (settling or clarification) is used to remove coagulated impurities.

The factors that influence sedimentation are:

- (i) size, shape, density, and nature (discrete or flocculent) of the particles;
- (ii) viscosity, density, and temperature of water;
- (iii) surface overflow rate;
- (iv) velocity of flow;
- (v) inlet and outlet arrangements;
- (vi) detention period; and
- (vii) effective depth of settling zone.

### 8.6.1 Types of Tanks

The tanks may be categorised 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.

#### 8.6.1.1 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:

#### 8.6.1.2 Radial Flow Circular Tank with Central Feed

The water enters at the centre of the tank and emanates from multiple ports of circular well in the centre 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 ploughed to the 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.

- a. **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.

- b. **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.
- c. **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.

The BIS code 'IS 10313 (1982, Reaffirmed Year: 2021): Requirements for Settling Tank (Clarifier Equipment) for Water Treatment Plant', may be also referred to.

### 8.6.1.3 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 upflow 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 upflow sludge blanket clarifier (Figure 8.15). 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.

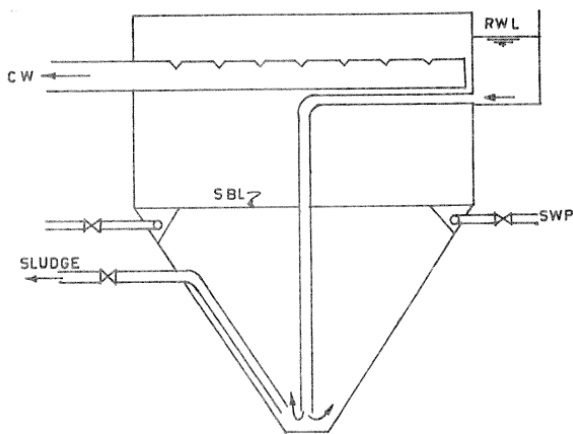


Figure 8.15: Sludge Blanket Clarifier

### 8.6.2 Clariflocculators and ContaClarifiers

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 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.

#### 8.6.2.1 ContaClarifiers or Upflow Contact Clarifiers

Upflow Contact Clarifiers can serve as a replacement for traditional coagulation, flocculation, and sedimentation in water treatment plants, saving on space and reducing maintenance on moving parts. The Upflow Contact Clarifier consists of an application specially sized and designed granular, non-buoyant media bed in three distinct sized zones, which is stable under all service flow conditions, an inlet distributor, air scour system, media support deck and screen, upper screen

system, and effluent launder. Chemically dosed water is introduced to the bottom of the Upflow Contact Clarifier, as it flows through the multiple sections of media, interparticle collisions result in forced flocculation and floc formation. Once floc is formed, it then flows into the removal zone and turbidity and solids are greatly reduced through contact with the non-buoyant media and previously retained solids. Once terminal head loss is reached, a rinse cycle takes place, utilising raw water pushed at a higher velocity along with an air scour, excess retained particles are cleaned and removed from the Upflow Contact Clarifier. Typical loading rates for Upflow Contact Clarifiers range from 320–400 LPM/m<sup>2</sup>, greatly increasing space efficiency over traditional sedimentation area.

### 8.6.2.2 Clariflocculators

Clariflocculators are widely used in water treatment in India (Figure 8.16, 8.17 and 8.18). The flocculation and sedimentation processes are effectively incorporated in a single unit in the clariflocculator. The unit has concentric circular flocculation zone at the centre 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 chemicals 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/sec. 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 0.3m 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 scrapper arms of rotating bridge. The slope of the tank bottom is in the range of 1:12 to 1:10 towards the center. It is advisable to locate the collection launder on the periphery outside the tank. If located inside, the projected area below the launder is not taken in to capacity calculations. 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 – 50m. 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 scrapper rake tip is 2.5 to 3 m per minute. The side water depth of the clariflocculator is in the range of 3.5m to 4.0m. 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 20m, sludge drain pipe diameter needs to be minimum 200mm and for larger tanks it needs to be minimum 300mm.

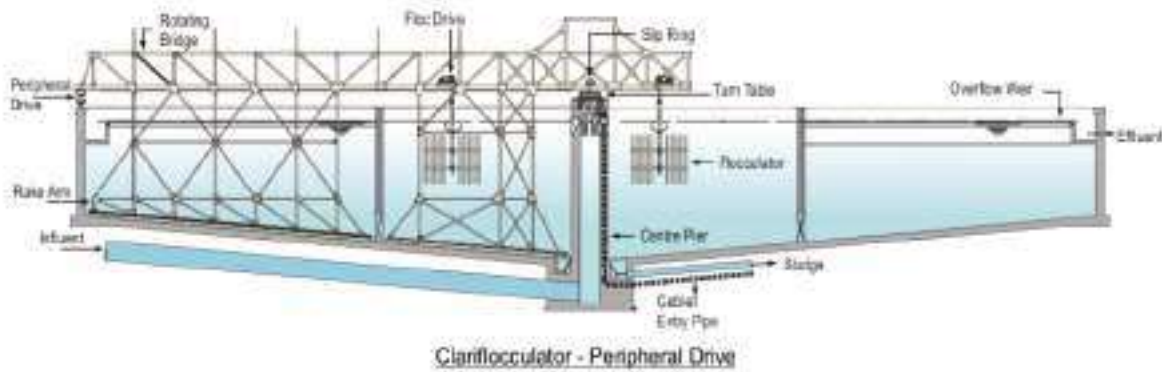


Figure 8.16: Cross section of Clariflocculator



Figure 8.17: Clarification zone with scrapper rotating bridge



Figure 8.18: Flocculation zone with slow agitators

### 8.6.3 Sedimentation Tank Dimensions

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-sized 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).

**8.6.4 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 given in Table 8.5:

**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 m <sup>3</sup> /m <sup>2</sup> /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/Clarifloculators	25-75	30-40	2-8	2-2.5	Alum and iron floc
Vertical flow (Upflow) Clarifiers	-	40-50	-	1-1.5	Flocculent

\*At average design flow

**8.6.5 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) minimise 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.

Freely discharging weirs, anywhere between flocculator and settling tank, have a tendency to break fragile floc, hence, freefall 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.

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.

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 centre to centre. A baffle is provided in front of the weir to stop the floating matter from escaping into effluent.

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 a 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.

### 8.6.6 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  $1,500 \text{ m}^3/\text{d}/\text{m}$ . Finger weirs are also employed for Large Diameter Clarifiers/ Clariflocculators in cases where the weir loading rate exceeds the above range (Figure 8.19).



**Figure 8.19: Finger weirs for Large Diameter Clarifiers/ Clariflocculators in case they exceed weir loading**

### 8.6.7 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 minimising the wastage of water. For non-mechanised units, pipe diameters of 200 mm or more are recommended. Pipe diameters of 100 to 200 mm are preferred for mechanised 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  $0.75 \text{ W}/\text{m}^2$  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  $55^\circ$  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-mechanised 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.

**8.6.8 Tube Settlers and Plate Settlers**

Tube and plate settling devices provide excellent clarification with detention times of equal to or less than 10 min (Figure 8.20, 8.21, 8.22 & 8.23). Tube configurations are steeply inclined. In inclined tubes (about 60°) based on the ‘counter current’ flow principal, continuous gravity drainage of the settled sludge can be achieved.

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 have also been used). Tubes can be extruded over the machine or alternatively fabricated out of thin plastic corrugated sheets (1.0 mm). Normally, black colour is preferred to prevent the growth of algae. In India, commonly, tubes with 50 mm × 50 mm (ID) section are practised. The raw material of tubes is virgin rigid PVC granules having specific gravity more than that if water.

Table 8.6 below 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 × mm)	Slant Length (mm)	Surface loading rate (on plan area)	
		m <sup>3</sup> /m <sup>2</sup> /d	l/m <sup>2</sup> /hr.
50 × 50	600	123	5125
50 × 50	750	150	6250
50 × 50	1000	195	8125
Above 1,000 mm lengths are not recommended			

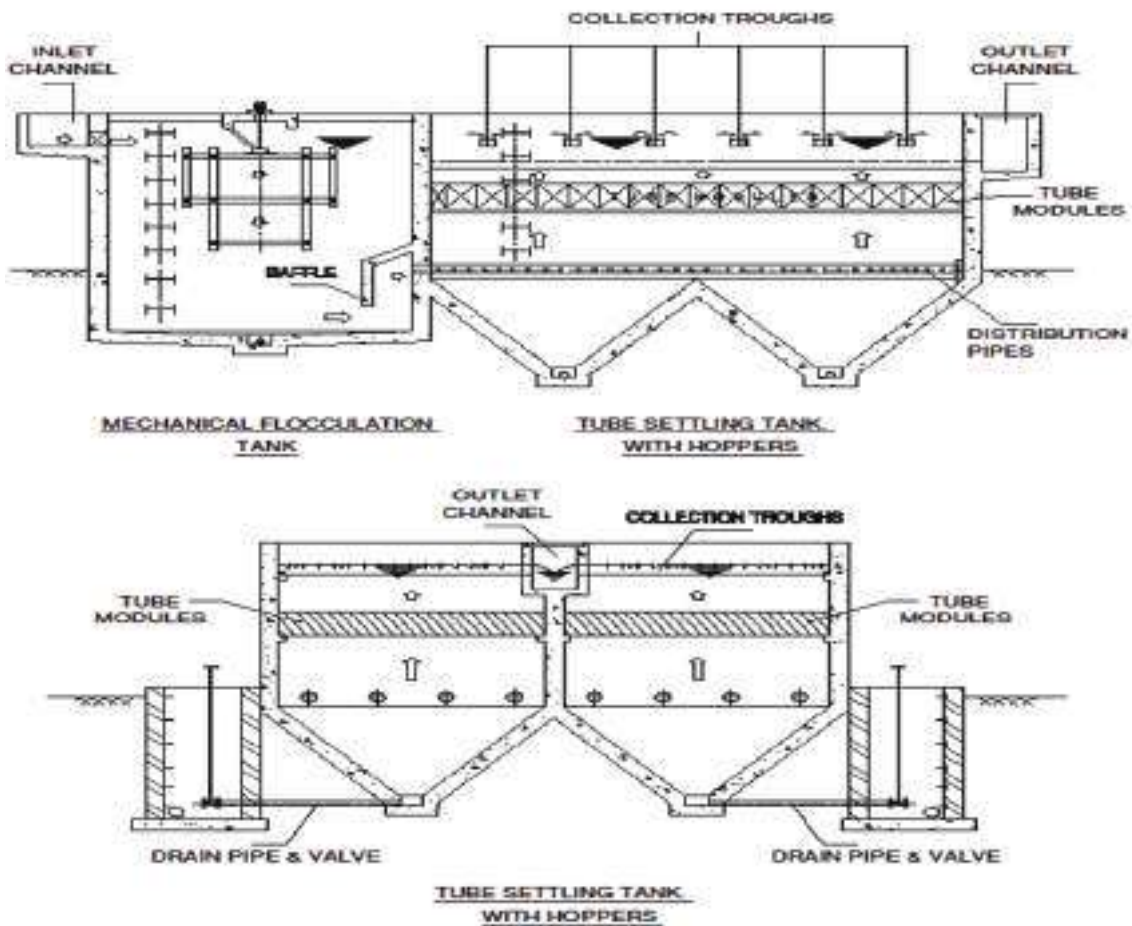


Figure 8.20: Typical Details of External Flocculation Tanks and Tube Settling Tanks with hopper bottom



Figure 8.21: Tube Settling and Flocculation under operation Modules & Troughs

#### 8.6.8.1 Inlet and Outlet Considerations

Since the detention time affects the efficiency of tube settling tanks, both 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.



**Figure 8.22: Performance photo at High Turbidity**



**Figure 8.23: Performance photo at Low Turbidity**

#### 8.6.8.2 Design Parameters

For the above-described size and configuration of tubes, for steeply inclined tube settlers ( $60^\circ$ ) the surface loading rates (overflow rates) based on the plan area of the basin are recommended to be  $5,000 \text{ l/h/m}^2$  to  $6,500 \text{ l/h/m}^2$ . 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 time. The weir loading should be less than  $300 \text{ m}^3/\text{m/d}$ . These loading rates are applicable for average water temperatures of  $15^\circ\text{C}$  to  $30^\circ\text{C}$ . 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.

#### 8.6.8.3 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  $4.5 \text{ m} \times 4.5 \text{ m}$  at the top. The hopper walls are inclined at  $45^\circ$ . The drainpipe 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. The Spyder type sludge removal systems are very effective. The tube settling systems are also compatible with a chain and flights scrapper mechanism.

#### 8.6.9 Combination of Technologies

##### (a) Tube – Clariflocculator (Clarissettler)

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

One such configuration is 'Tube-Clariflocculator' or 'Clarissettler' (Figure 8.24 & 8.25). 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 clarissetler unit becomes one-half that of the clariflocculator. Clarissetler 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. The sizing calculations for tube-clariflocculator or (clari-tube settler) are given in **Annexure 8.2**.



**Figure 8.24: Clarissetler with Rotating Scrapper Bridge**



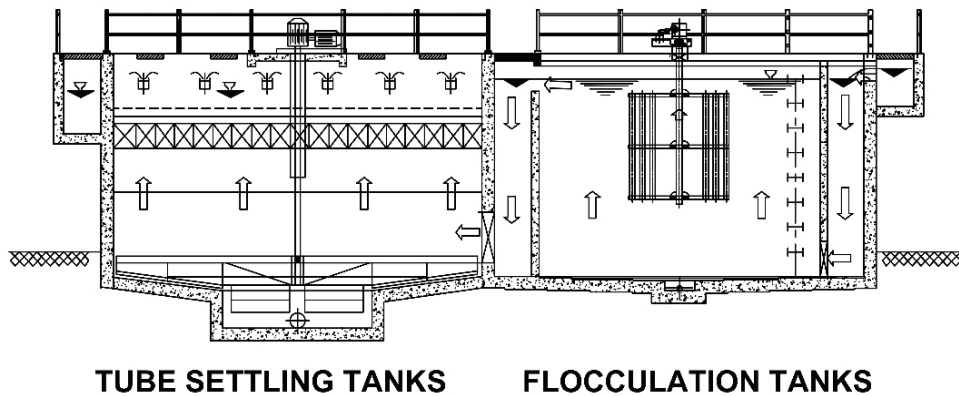
**Figure 8.25: Clarissetler with Central Rake**

#### (b) Tube Clarifier with Central Rake

This is another configuration of flocculator – tube settlers (Figure 8.26 and 8.27). In this case the paddle flocculators and tube settlers are located in separate units with a common wall. The tube settlers are configured into a square tank. The sludge is removed by the central scraper rake. In case of multiple units, a 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. Normally, the diameter of a tube clarifier is 50%–60% that of a clariflocculator of same capacity.



**Figure 8.26: External Flocculators and Tube Settling Tank with Central Rake**



**Figure 8.27: Schematic Flow Path**

**8.6.10 Ballasted Flocculation and Settling**

This technology is relatively new to India, however, in the future, it is likely to be adopted to reduce the footprint (Figure 8.28). 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 25–50 m<sup>3</sup>/m<sup>2</sup>/hr. will substantially reduce settling area compared to conventional sedimentation basins.

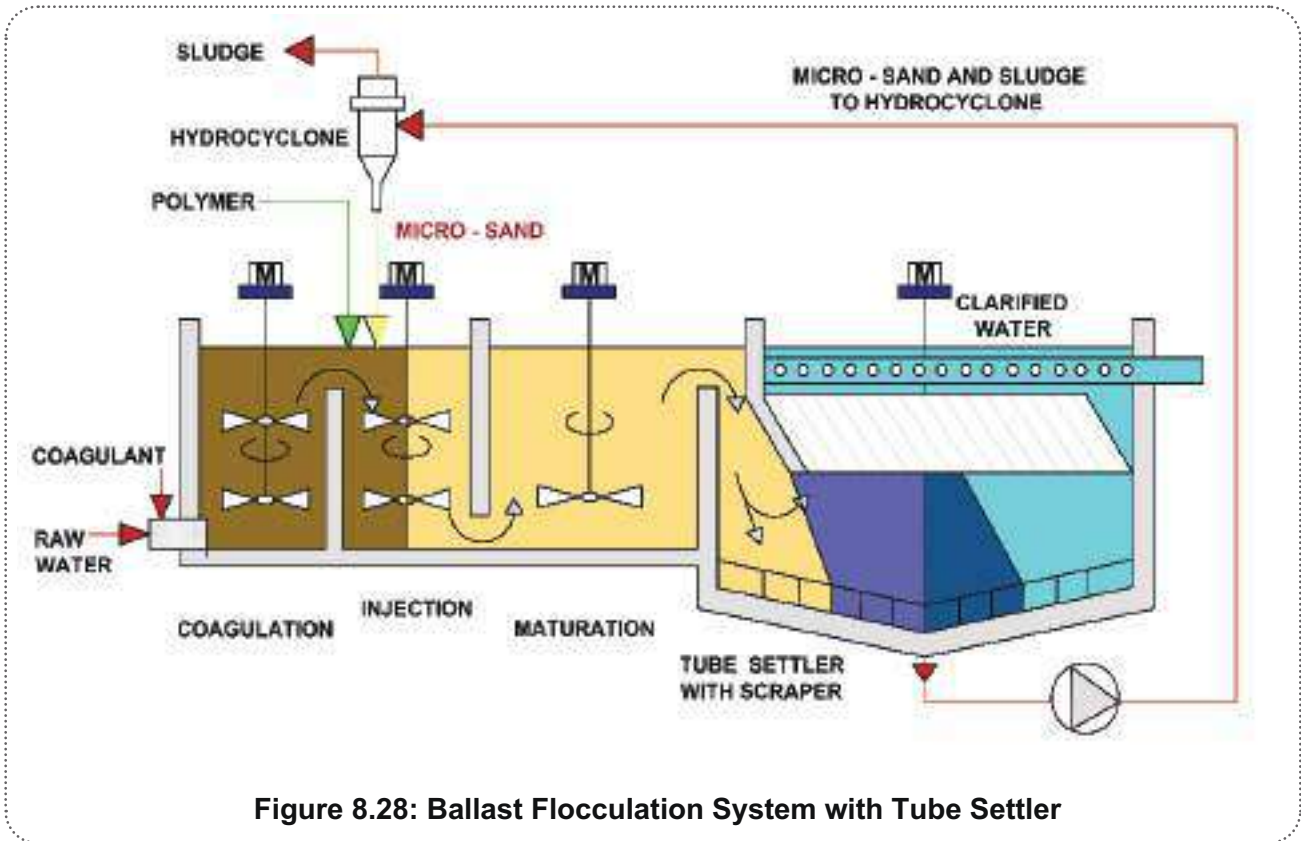


Figure 8.28: Ballast Flocculation System with Tube Settler

### 8.6.11 Dissolved Air Flootation (DAF)

This is a very rarely used technology in India. In DAF, 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 DAF, flocculation is designed to create a large number of smaller flocs (Figure 8.29 and 8.30).

For efficient floatation, 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 into 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 microbubbles 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.

#### 8.6.11.1 Design Parameters

The DAF basin surface loading rates range from 10 m<sup>3</sup>/m<sup>2</sup>/hr. to 12 m<sup>3</sup>/m<sup>2</sup>/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<sup>2</sup> to 6 kg/cm<sup>2</sup>.

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, colour and having low turbidity.

They have been used in RO feed water pretreatment for seawater to control the algal blooms.

However, DAF require a high amount of electrical energy for the air compressors (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.

Typical flow diagram

**Figure 8.29: Typical Flow Diagram of DAF**

**Figure 8.30: WTP at T K Halli, Bengaluru**

**8.6.12 Unconventional Water Treatment Plants up to 5 MLD Capacity**

In semi-urban areas where water treatment plant is required normally sufficient space is available. However, the skilled O&M staff is not available. In such cases WTP requiring minimum or no O&M will be preferred without compromising quality aspects.

In any WTP, sedimentation process is very important. Before inlet to filters, if the turbidity levels are maintained less than 15 units, then function of filtration will be more efficient. Hence, in the WTP construction inlet/outlet conditions are suitably modified to give better and assured results.

The sedimentation process in the plant is designed for plain sedimentation, with an overflow rate of 18 cum/sq. m/day. It is in two compartments and each is split into four settling units. All are hopper bottom units.

First hopper unit is redressed internally to conical shape for smooth flow of raw water splashed from jets, from bottom to top.

The first unit is jet flocculator in each compartment. The jet gets energy from the inlet pipe going about 5 m deep in the hopper. Water from jets is directed to move in circular motion due to the conical surface of the container and, at the same time, it rises in the unit passing through incrementally increasing area (Figure 8.31). At FSL, its rotational movement is just equivalent to that we normally in the flocculator. This unit is not expected to take out any turbidity, but it still separates out the inert solids.

**Figure 8.31: Water Jets at the Bottom of the Hopper**

Second hopper unit is having modified inlet. Water from Weir enters the launder which has several vertical pipes to direct the water to go down in hopper and is further directed to rise through similar vertical pipes in the end launder. Here the path of flow is purposely extended so that turbidity particles do not rise to the FSL and prefer settling in the unit. Direct flow between the launders is not possible.

Third hopper unit is also designed with down-take pipes through launders. However, the outgoing flow has options to move to the fourth hopper through slant pipes in the separator walls of the hoppers. Still the turbidity particles are required to reach FSL in the fourth hopper unit where the outlet launders are provided with weir flow rate up to 80 to 100 cum/m/day, ensuring quiescent flow conditions. In such a cases where the third and fourth hopper units become efficient to remove finer turbidity particles and it is observed to give outlet turbidity within the range of 10 to 15 NTU. For higher incoming turbidity in raw water, first two hopper units work very efficiently.

Efficiency of the first unit with jet flocculator will be increased with alum dosing and/or pre-chlorination. Sufficient cleaning time for compartments is available as the operation of one compartment can give similar efficient results. With chemical coagulation, in the first hopper of jet flocculator, capacity of WTP can be increased. It will increase design life of the WTP. As there are no moving parts in WTP, O&M is practically nil. Rapid sand gravity filters receive assured flow of raw settled water with optimum turbidity load; it results in the higher filter runs and higher filtration efficiency. In this type of plant, no mechanical controls are provided in the filters. As the filter run goes filtration resistance increases and water level reaches the maximum pre-decided level in the filter box. It indicates washing of filters. Outlets are provided with master pieces (Pipe in pipe) to operate at a controlled rate

**8.6.12.1 Design of Jet Flocculator**

Entry of water 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 the vertical pipe, a small drum is fitted to accommodate four jets at 90 degrees to adjoining jets. At the end of each jet pipe, a 90-degree bend is fitted with end reducing suitably. When flow starts, jets operate as outlets and splash to the walls of the container. Flow automatically gets circular direction. The velocity at the bottom is very high and as it moves upwards, it faces larger area, and the velocity is reduced. Hence, the initial portion acts as a flash mixer and thereafter, at higher levels, it slowly turns into flocculation velocity. The actual Gt values are tabulated after basic equation is arrived at.

**8.6.12.2 Velocity Gradient Variation**

The velocity gradient at the bottom of the flocculator is  $189 \text{ s}^{-1}$ , while at the top is  $8.74 \text{ s}^{-1}$ . For the present, the pattern of variation of G from bottom to top has been determined on the assumption that energy available at a point varies inversely with  $D^4$ , since the velocity varies inversely as square of the diameter of the flocculator, and energy is proportional to the square of the velocity. The values of GT are shown in Table 8.7

**Table 8.7: Table showing values of GT**

Height from the bottom apex	Height of part	Radius	Volume	Energy available	Local G
m	m	m	cum	m	$\text{g}^{-1}$
3.125	0.375	1.06	1.324	0.38913	189.1
3.5	0.5	1.2	2.262	0.24729	113.5
4	0.5	1.36	2.905	0.14496	77.6
4.5	0.5	1.52	3.629	0.0905	55.4

Height from the bottom apex	Height of part	Radius	Volume	Energy available	Local G
5	0.5	1.68	4.433	0.05938	40.9
5.5	0.5	1.84	5.318	0.04155	31.1
6	0.5	2	6.283	0.02863	24.2
6.5	0.5	2.16	7.329	0.02079	19.2
7	0.5	2.32	8.455	0.01546	15.5
7.5	0.5	2.48	9.661	0.01173	12.6
8	0.5	2.64	10.948	0.00906	10.5
8.5	0.5	2.8	12.315	0.00711	8.8
9	0.375	2.94	10.183	0.00566	8.7

## 8.7 Filtration

### 8.7.1 General

Filtration is the process of separating suspended and colloidal impurities from water by passing it through a porous medium. Filtration, without pretreatment will not 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.

The granular medium filters include single-medium, dual-media and multi-media (usually tri-media) filters. Sand, anthracite coal, crushed coconut shell, or granular activated carbon have been used as filter media. 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. The flow rate control is 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;
- (b) rapid sand filters;
- (c) high-rate filters.

### 8.7.2 Slow Sand Filters

These filters are not widely used since early 1990. Slow sand filters can provide a single step treatment for surface waters of low turbidity (<5 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, pretreatment 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.

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. The box contains a supernatant water layer, a bed of filter medium, an underdrainage system and a set of control valves and appurtenances.

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 and a new filter should be provided with an initial sand depth of about 1.0 m. Re-sanding will then become necessary once in two to three years.

### 8.7.3 Rapid Sand Filters

#### 8.7.3.1 Filtration Process

The rapid sand filter consists of a bed of a single or dual filter media supported on gravel overlying an underdrainage system (Figure 8.32). For rapid sand filtration to work efficiently, careful pretreatment of raw water is required to remove the suspended solids. Although not discussed in this section, high-rate filtration, deep bed filtration, multi-media bed filtration, use of activated carbon or anthracite coal as media are widely practised throughout the developed countries and should be considered based on site-specific conditions.

When water containing suspended matter is applied to the top of filter bed, suspended and colloidal solids are trapped in the granular medium matrix. The accumulation of suspended particles in the pores and on the surface of filter medium leads to the build-up of head loss as pore volume is reduced. The media offers greater resistance to the flow of water, simultaneously with the build-up of head loss across the filter. 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. The clogged filter bed is cleaned with air scour and water wash (hard wash). The direction of air and water wash is in the reverse direction that of filtration.

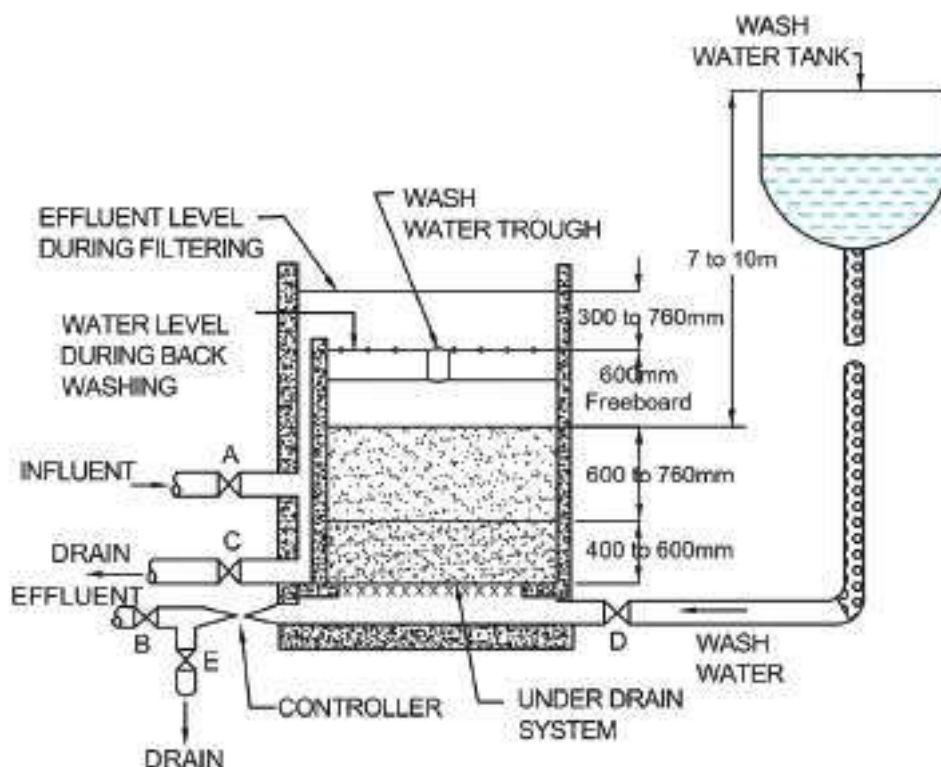


Figure 8.32: Granular Medium Gravity Filter

#### 8.7.3.2 Rate of Filtration

The standard rate of filtration through a conventional rapid sand filter is usually 80 to 100 l/m<sup>2</sup>/min.

(4.8–6 m<sup>3</sup>/m<sup>2</sup>/hr.). Current practice is leaning towards higher rates by adopting dual media gravity filters or mono media deep bed filters. An average rate of filtration shall be 5.5 m/hr. or less. It shall be ensured that under overloading conditions, when one bed is out of service, the maximum rate of filtration shall be less than 6 m<sup>3</sup>/m<sup>2</sup>/hr. (The overloading check applies only if the number of beds exceeds four. A plant having four filter beds shall be designed to be operated for 22 hrs.). 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). Again, much higher filtration rates are practised in developed countries and should be considered based on site-specific conditions.

**8.7.3.3 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. Table 8.8 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 m<sup>2</sup> for a single unit is recommended for plants of greater than 100 MLD consisting of two halves each of 50 m<sup>2</sup> area. Also, for flexibility of operation a minimum of four units should be provided which could be reduced to two for smaller plants.

**Table 8.8: Minimum 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. For above 80 MLD capacity, it is recommended to provide a 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.

**8.7.3.4 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 (Two sections with central gutter) shall be preferably between 1 to 1.66. The bed shall have an overall depth from 2.6 m to 3.75 m depending on the type of filter excluding a free board of 0.5 m.

The filter shell (boxes) shall be in reinforced concrete to ensure a watertight 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.

#### **8.7.3.5 Filter Sand**

Filter sand is defined in terms of effective size and uniformity coefficient. Effective size is the sieve size in millimetres 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 ( $D_{60}/D_{10}$ )

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

- (i) Sand shall be of hard and resistant quartz or quartzite and free of clay, fine particles, soft grains, and dirt of every description.
- (ii) 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).
- (iii) Uniformity coefficient shall not be more than 1.7 nor less than 1.3.
- (iv) Ignition loss should not exceed 0.7% by weight.
- (v) Soluble fraction in hydrochloric acid shall not exceed 5.0% by weight.
- (vi) Silica content should be not less than 90%.
- (vii) Specific gravity shall be in the range between 2.55 to 2.65.
- (viii) Wearing loss shall not exceed 3%.

#### **8.7.3.6 Depth of Sand**

Usually, the sand layer has a depth of 0.60 to 0.75 m. The standing depth of water over filter media top varies between 1 and 2.5 m depending on the filter control arrangement. 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.

##### **(i) Procurement of Filter Sand**

The natural filter sand is procured from various sources like Banda in Uttar Pradesh, Paonta Sahib in Himachal Pradesh, Jaipur in Rajasthan, Godra and Vejalpur in Gujarat, Bhandara in Maharashtra, Belgaum 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.

##### **(ii) Filter Bottoms**

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

The underdrainage 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 underdrain 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 underdrain has no access.

The most common type of underdrain is a central manifold with laterals either perforated on the bottom or having umbrella type strainers on top (Figure 8.33 & 8.34). Other types such as wheeler bottom, block underdrains, false bottom with nozzles, dual-parallel blocks, etc.

**Manifold and Laterals**

The following values may be used in design of an underdrain 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.

The ratio of the total area of perforations to the underdrain system to the 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 minimise 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 underdrains.



**Figure 8.33: Manifold-Laterals Assembly**



**Figure 8.34: "Concrete Tee" as a connector block to Manifold Pit**

**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 underdrain 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 underdrain type. For monolithic, reinforced concrete underdrains, 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, unplasticized 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 underdrains 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 underdrain 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/antracite/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 21mm, 16mm, 16mm with a 13mm 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 all will deliver optimal filter performance.

### **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 headloss 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.

**Dual-parallel underdrain blocks (Flat bottom flumes)**

This system is 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 underdrain system can permit separate air scour, water backwash, simultaneous air and water backwash modes. The inherent design of underdrain eliminates uneven distribution of air and water even in large filter beds. Generally, the blocks are designed for filtration rates 25 m<sup>3</sup>/m<sup>2</sup>/hr. The water back wash upflow range is 20-90 m<sup>3</sup>/m<sup>2</sup>/hr. For simultaneous air-water wash the rate of flows can be designed as 70 m<sup>3</sup>/m<sup>2</sup>/hr. and 15 m<sup>3</sup>/m<sup>2</sup>/hr. respectively.

The filter underdrain system, when installed is designed for a net internal loading during backwash of either 2.93 m H<sub>2</sub>O or 200% of the maximum pressure at maximum backwash rates, whichever is higher. The filter underdrain system is designed to withstand a net downward loading of not less than 6.84 m H<sub>2</sub>O.

The block underdrain 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 underdrain blocks are suitable to be adopted with high rate or very high-rate filtration viz. Mono media deep bed filters described later.

**8.7.3.7 Filter Gravel**

Gravel is placed between the sand and the underdrain system to prevent sand from entering the underdrains 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 dual-parallel block underdrain does not require the gravel support.

Suggested gravel configuration for manifold-lateral underdrain system are given in Table 8.9.

**Table 8.9: Conventional Rapid Sand Gravity Filters**

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

### 8.7.3.8 Wash Water Collection Troughs/ Gutters

Wash water collection troughs collect the dirty backwash water during the backwash and discharge in to a central/side gutter for further disposal out of filter bed. 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. 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. The cross-troughs should be large enough to carry all the water delivered to it with at least 50 mm of freefall between the surface of the water flowing in the gutter and the upper edge of the gutter (freefall). The discharge capacity  $Q$  in  $m^3/s$  may be computed from the formula:

$$Q = 1.376 bh^{3/2}$$

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 into the central gutter. The crest of cross-troughs shall be 50 mm to 75 mm below the top of the central gutter. In no case the central gutter shall have overhang above or into 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.

### 8.7.3.9 Air Scour and High-Rate Backwash

Normally, for effective cleaning of filter beds, air scour followed by water (hard) wash is adopted. The piping for the same can be separate or combined. The air scour is recommended at the rate of 36 to 45 m/hr. (of filter area) at 0.35 kg/sq. cm for duration of 5 min. The water wash is recommended at the rate of 24 to 40 m/hr. (of bed area). The effective head for wash water tank (overhead) is 8.50 m above the filter underdrain system. The duration of water wash is 10 min. In a filter bed with two sections with a central gutter (Figure 8.35), it is the designer's choice whether to back wash both sections separately or simultaneously.



**Figure 8.35: Cross-Troughs and Central gutter**

### 8.7.3.10 Mechanism of Flow Controller

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 counterweight on the long arm of the beam is transmitted to the scale beam. This balance of counterweight 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 emphasised that the rate of flow controllers requires proper operation and maintenance to ensure that filtration is done at a 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.

**(a) 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 8.36. The head loss can also be measured using mechanical equipment as shown in Figure 8.37. 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 8.36: Head Loss Indicator (Manometer)**



**Figure 8.37: Rate of Flow and Head Loss Indicator (Mechanical)**

### **(b) 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 to 2.50m. 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 in building with headroom over upper-level walkway as 3.5m to 4.0m. Adequate lighting and ventilation shall be provided to the pipe gallery.

Normally, the 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.

### **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.

#### **8.7.4.1 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 m<sup>2</sup> and 200 m<sup>2</sup> with depths between 3.0 m and 3.75 m. The filter media is supported on gravel laid over top of the underdrainage system. In addition to the underdrainage 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 minimise 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 metre or rate of flow recorder are also located above and/or below the gallery floor.

#### **8.7.4.2 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. 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.

### 8.7.4.3 Filtration Rates and Filtrate Quality

Dual media and multi-media filters have been successfully operated at rates of filtration ranging from 10 to 20 m<sup>3</sup>/m<sup>2</sup>/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 m<sup>3</sup>/m<sup>2</sup>/hr. to achieve filter run of 24 hr or more.

The backwash rates of 42 to 54 m<sup>3</sup>/m<sup>2</sup>/hr. (700–900 l/m/m<sup>2</sup>) 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 backwashing to ensure the media configuration. Like rapid sand gravity filters, air scour is also employed prior to water wash (Figure 8.38, 8.39 & 8.40).



Figure 8.38: Pilot Plant for Dual Media Filters



Figure 8.39: Checking Media Expansion During Backwash using Sand Expansion Stick



**Figure 8.40: Dual Media Filter Bed during Internal Erection and Media Loading**

### 8.7.5 Multi-Media Filters

The multi-media 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. However, it should be noted that multi-media filters are used infrequently as the fine media presents a problem in units without gravel, and also that of inter-mixing.

### 8.7.6 Mono Media Deep Bed Gravity Filters

Mono media deep bed gravity filters (MMDB filters) or uniform media filters are shown in Figure 8.41 and Figure 8.42.



**Figure 8.41: MMDB Filters**



**Figure 8.42: 'Coarse' and Uniform OCV, 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 underdrain system of strainer nozzles with plenum chamber or underdrain blocks as described earlier. The configuration of the 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, sometimes 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.

### 8.7.7 Pressure Filters

In pressure filters, the removal mechanism is same as that of the rapid sand filtration. The filtration rate is high (6,000–15,000 l/m<sup>2</sup>/hr.) and water is passed through the filter under pressure (3–7 kg/cm<sup>2</sup>) through a cylindrical tank, usually made of steel or cast iron, wherein the underdrain, 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 pressurised water is applied uniformly at top surface of the media, so that the churning of the media is avoided.

Pressure filters have the following disadvantages:

- a. 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.
- b. It is difficult to inspect, clean and replace the sand, gravel, and underdrains of pressure filters.
- c. 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.

### 8.7.8 Additional Modifications of Conventional Rapid Gravity Filters

#### 8.7.8.1 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 rates 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 8.43. 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 8.44, 8.45, 8.46 & 8.47. 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.

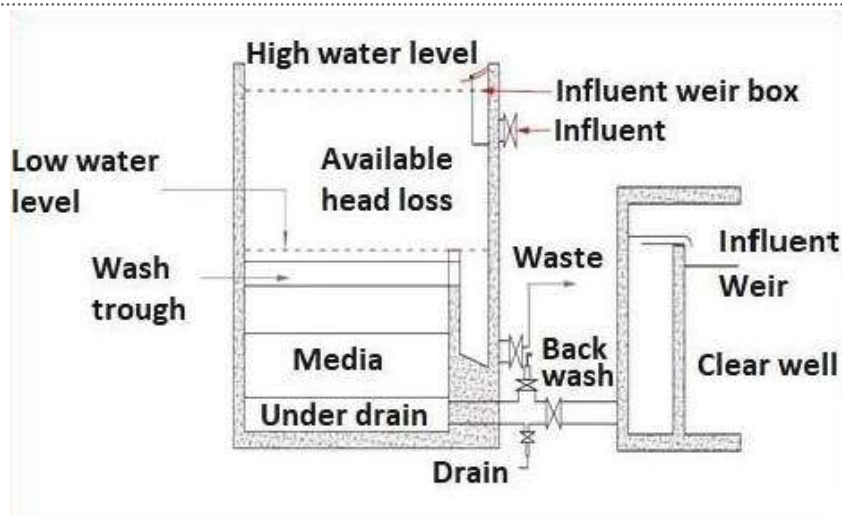


Figure 8.43: Gravity Filter Arrangements for Rate Control





**Figure 8.45: Filter Inlet Channel and Inlet Weir with Chambers**



**Figure 8.46: Close-up of Splitter Weir and Chamber**



**Figure 8.47: Filter outlet chamber with weir, Weir Crest Located above Top of the Media**

### 8.7.8.2 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 8.7.8. 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 8.48. 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 through an increase in filtration rate and may result in deterioration of the filtered water quality.

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.

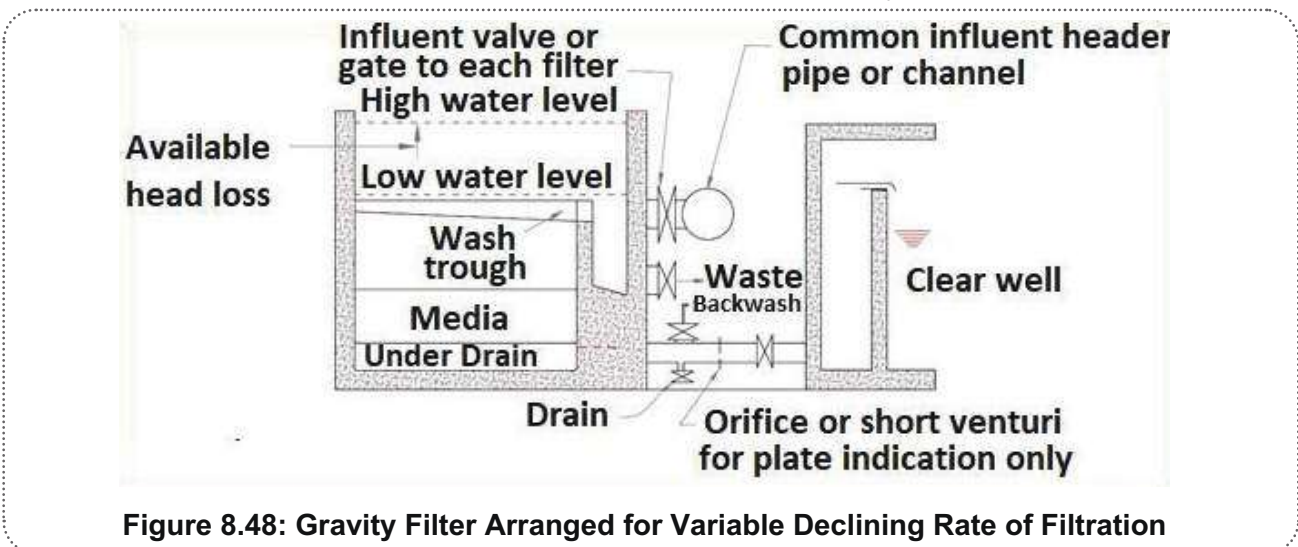


Figure 8.48: Gravity Filter Arranged for Variable Declining Rate of Filtration

### 8.7.8.3 Upflow Filters

Upflow filters are not practised nowadays in India. In upflow 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 upflow 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 fluidisation of the top fine layers of the sand bed which results in the deterioration of the filtrate quality. Complete bed fluidisation 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 upflow sand bed is restricted to 0.6.

#### **8.7.8.4 Automatic Valve-less Gravity Filters**

This type of filter is 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 backwashing begins at the required rate of 30 to 42 m<sup>3</sup>/m<sup>2</sup>/ 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 mentioned in Chapter 2, Part A of this manual is 15%, of which 3% NRW is considered as water lost in WTP. In order to minimise this loss, the recycling of backwash water is adopted (Figure 8.49).

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

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 re-circulation 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 re-circulation shock loading (to mainstream), 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 re-circulation 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 wastewater 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* cysts and to inactivate viruses. However, this may lead to formation of DBPs and this practice should be avoided.

The sludge from clariflocculator is not mixed with backwash water from filters. Therefore, the entire backwash water should be recycled. This will not only minimise the losses through treatment plant, but also enhance the removal efficiency. Though, theoretically outlet to this backwash recycle tank is not needed, however, in emergency (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.

In developed countries, the backwash recycle water is treated through the tube/plate settlers and chlorinated to eliminate suspended solids, *Giardia*, *Cryptosporidium* and *Schistosoma* before it is sent to the inlet chamber of the plant.



**Figure 8.49: Wash Water Recycle Tanks (Single Compartment and Two Compartments)**

### 8.9 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.

### 8.9.1 Disposal Methods

As a result of the presence of suspended and precipitated solids, as well as reactions with many coagulants, the process generates 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 in the 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.

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

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

#### 8.9.1.1 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/m<sup>2</sup>/d (max up to 100 kg/m<sup>2</sup>/d) based on the solids loading (dry basis) and is followed by sludge dewatering devices. A thickener is similar to a central feed, central rake drive clarifier having a 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. Lime sludge has shown better dewatering characteristics. PE dosing is practised at the inlet of the thickener to improve the quality of the underflow. The thickener rake has normally pickets mounted on it (Vertical angles or supported flats). These 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.

#### 8.9.1.2 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 dewatering equipment, Part 2: Vacuum filtration equipment, which lays down requirements for vacuum filtration equipment used for sludge dewatering may be referred to.

### 8.9.1.3 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/m<sup>2</sup>/d. The drying cycle is designed for six to seven 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 300 mm to 400 mm 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.

### 8.9.1.4 Sludge Drying Beds (Tile Beds)

Tile bed systems have been designed to effectively dewater both industrial and municipal sludges. Basins and bed sizes can be adjusted to fit any application, including upgrading an existing sand bed or converting lagoons. The system is flexible enough to handle industrial spikes or heavy surges from digesters and holding tanks. Tile systems are designed to meet the ever-changing demands of the water and wastewater industries by providing an environmentally responsible dewatering system. The operation is simple: flood the bed with thickened sludge, wait for the sludge to dry, and remove the dry sludge cake with a front-end loader. Following sludge removal, wash down the bed and repeat the cycle.

### 8.9.1.5 Continuous Decanter Centrifuges

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 (Figure 8.50 & 8.51).



**Figure 8.50: Centrifuge Building with "Chute"**



**Figure 8.51: Centrifuges located on the First Floor**

#### 8.9.1.6 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 two hours and may extend up to even eight hours or beyond, depending on the drainability of the 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, pressurised 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.

#### 8.9.1.7 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 freshwater 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.

### 8.10 Treatment Plant Hydraulics

To the extent possible, 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, the bigger the plant, the 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 the 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 Manning 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 low velocities to avoid breaking or fragmenting the floc particles. For piping the velocities are recommended in the range of 0.8 m/sec to 1.2 m/sec. Generally, the Full Supply Level (FSL) of pure water sump or clear water Reservoir/sump (CWR) shall not be below the finished ground level (FGL). 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. The maximum values of filter port velocities are shown in Table 8.10.

**Table 8.10: Maximum values of Filter Port Velocities**

Filter Port	Velocity (m/s)	Adopted in Maharashtra
Filter Inlet	0.9 to 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*

**8.11 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. Often it includes elevated service reservoir or MBR also. The entire plant complex should be planned in such a way that process and hydraulic structures are located in a 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 unit 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/m<sup>2</sup>. 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 (1 m 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. However, 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 into 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.

#### **Typical Layouts of WTPs for various capacities**

Figure 8.52: Layout of a small capacity plant of 10 MLD capacity on a fairly levelled site (double pumping scheme)

Figure 8.53: Layout of a medium capacity plant of 25 MLD on a steeply sloping hill (raw water by pumping, PW supply by gravity)

Figure 8.54: Layout of a medium capacity plant of 50 MLD capacity with hard rock as strata (double pumping scheme)

Figure 8.55: Layout of a large capacity plant of 168 MLD with severe space constraints

Approximate land area requirement (Table 8.11) for conventional water treatment plants consisting of cascade aerator, Parshall flume, flash mixers, clariflocculators, rapid sand gravity filter beds, filter annex building, storeroom, wash water recycling tank, sludge thickener, centrifuges, MCC room, generator room, transformer yard, etc.

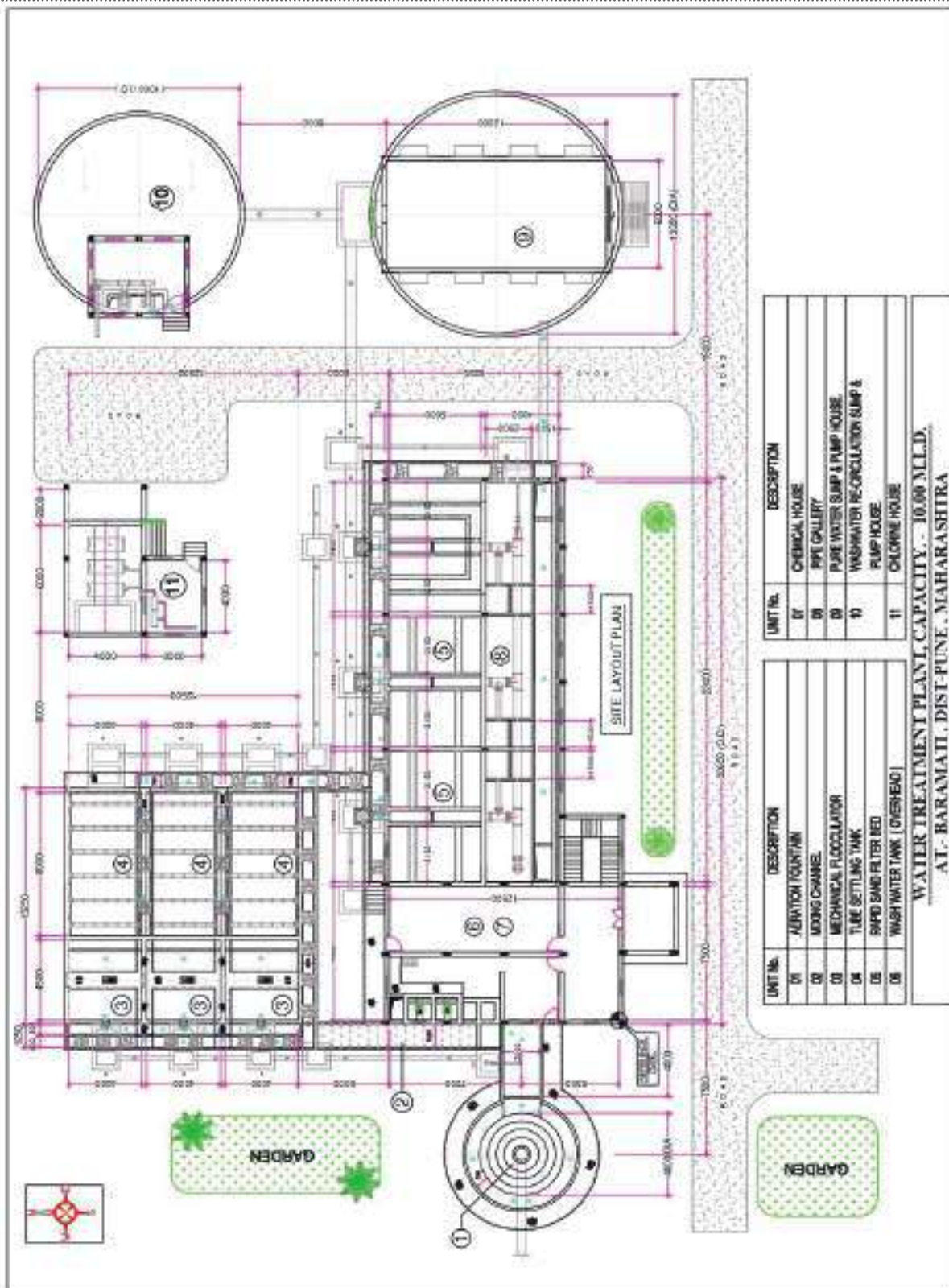


Figure 8.52: Layout of WTP Capacity 10 MLD at Baramati, Dist. Pune, Maharashtra

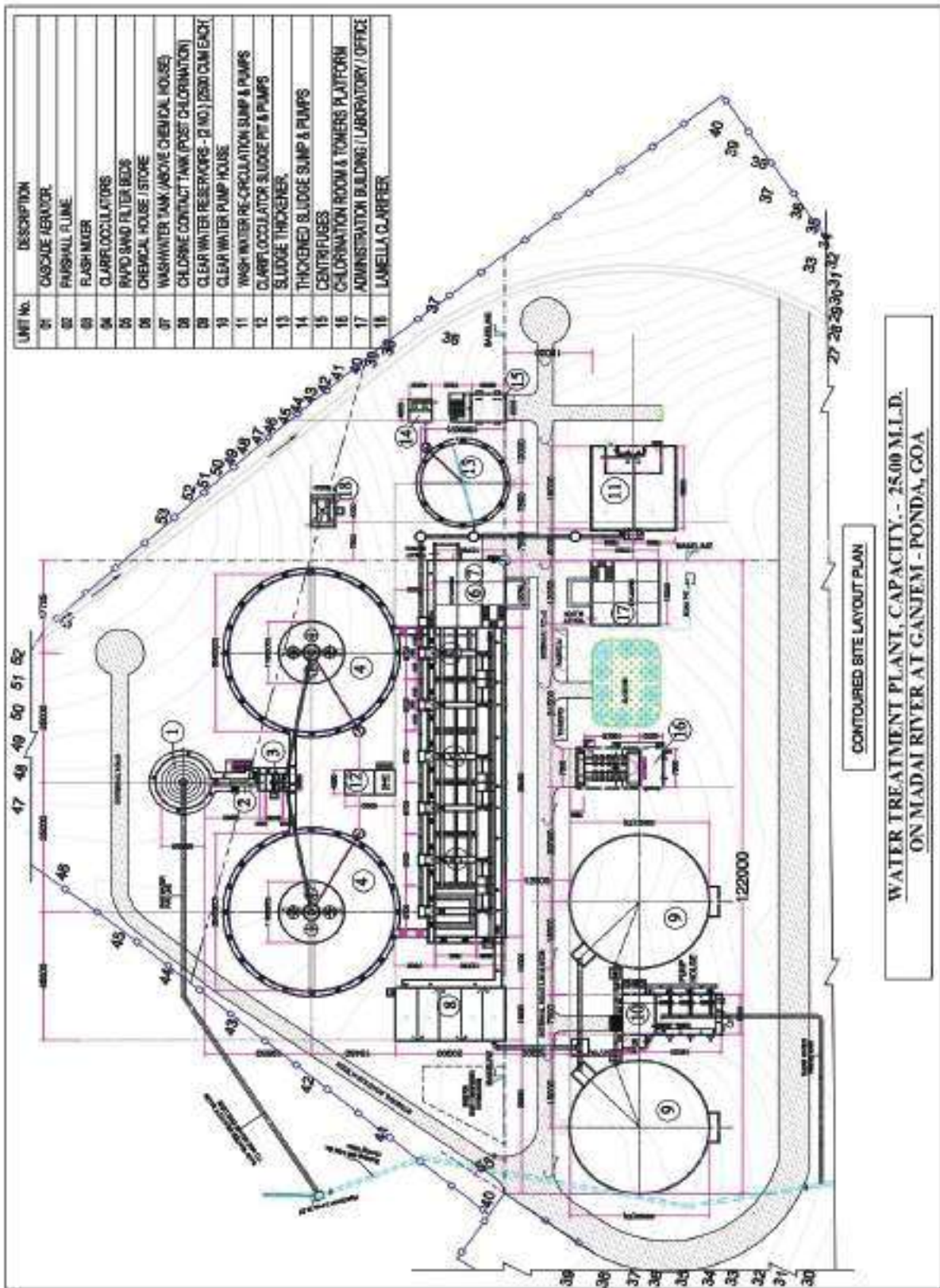


Figure 8.53: Layout of WTP Capacity 25 MLD at Ganjem, Ponda, Goa

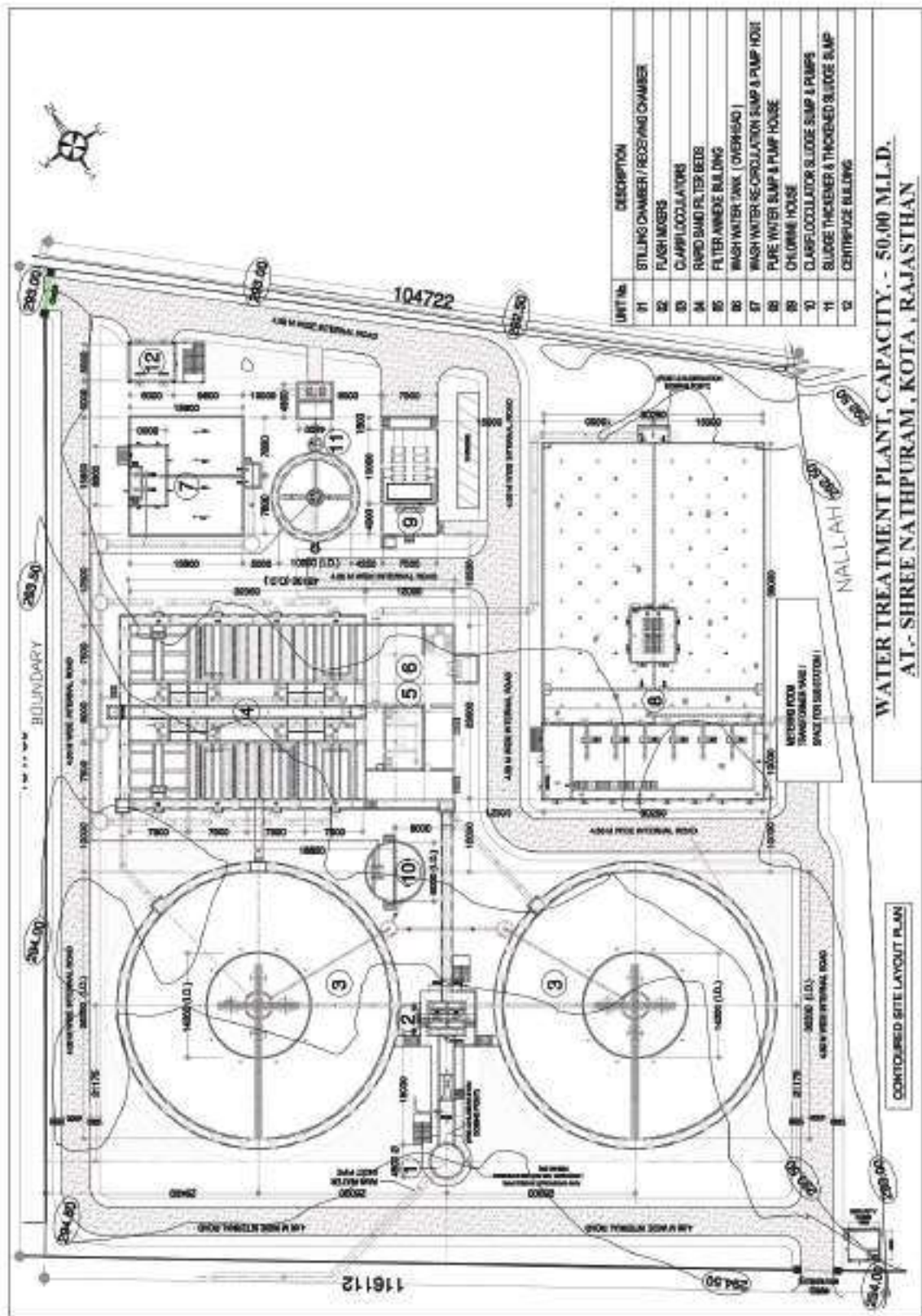


Figure 8.54: Layout of WTP Capacity 50 MLD at Shreenathpuram, Kota, Rajasthan

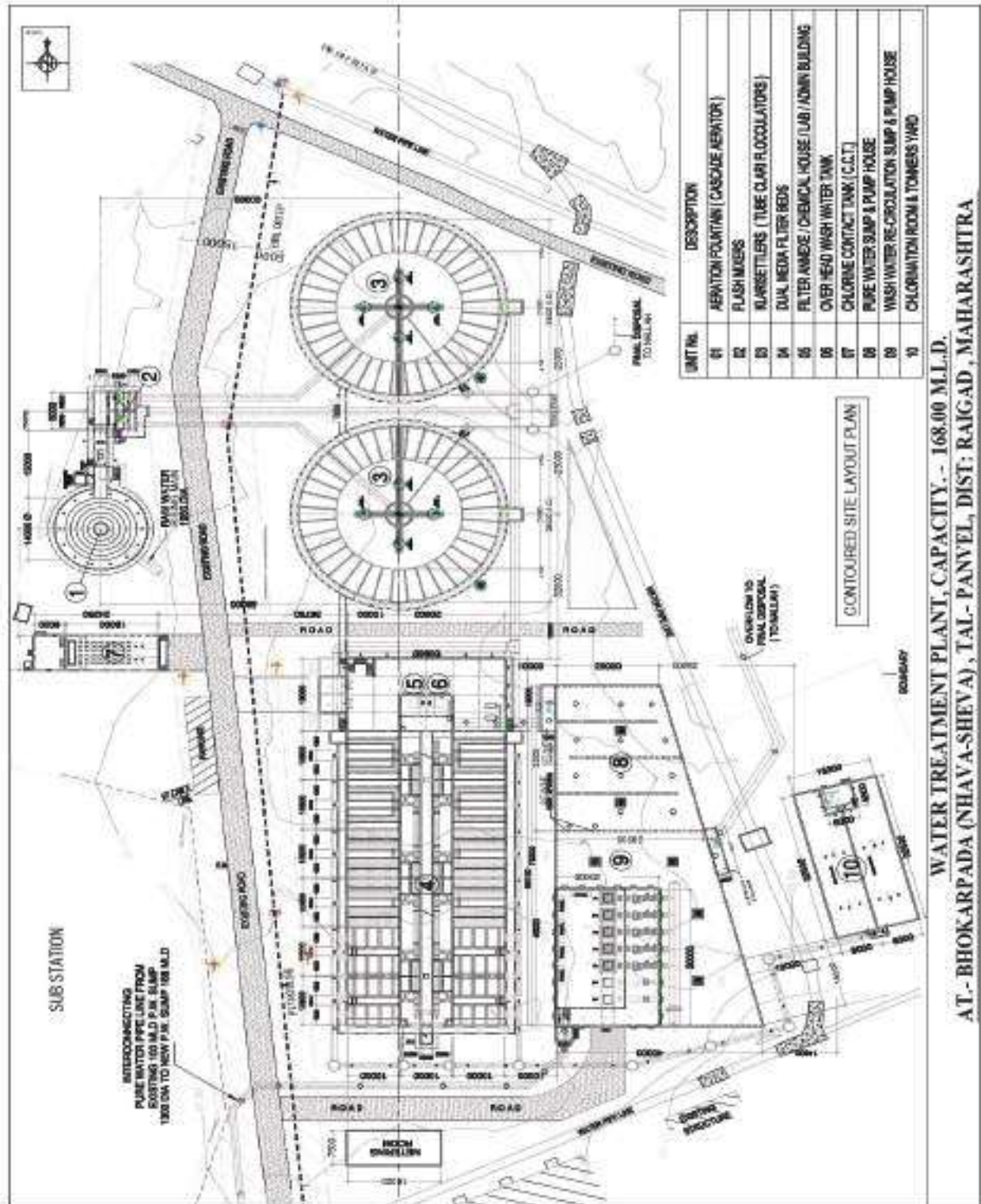


Figure 8.55: Layout of WTP Capacity 168 MLD at Nhava-Sheva, Dist. Raigad, Maharashtra

Table 8.11: 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	1,000–1,200	0.30
2.	5.0	2,500–3,000	0.70
3.	10.0	4,500–5,000	1.15

S. No.	Plant Capacity (24 hr. basis) (MLD)	Area in sqm	Area in acre (average)
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 a disproportionate increase in demand and there is a constraint of space to construct the new structures.

Augmentation can be achieved either by addition of a new plant 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, and low-pressure submersible membranes.

**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 the case of an 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 are 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 are savings 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 (Figure 8.56 & 8.57).
- (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 (Figure 8.58).
- (d) **Hydraulics:** The interconnecting piping, channels, inlet and 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 utilised judiciously.



**Figure 8.56: Tube Modules installed in Clariflocculator**



**Figure 8.57: Retrofitting of a rectangular tank with tube settle**



**Figure 8.58: Upgrading of Existing conventional filter beds with mono media deep bed 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 (Figure 8.59). They 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 a long time to construct, and execution is error-prone. For such small plants, prefabricated packaged plants are recommended, where the quality of construction can be assured in a controlled environment. The emphasis is on providing time-tested unit processes with the least mechanical or rotary parts. These shall be fabricated in mild steel plates of 6 mm thickness. The inside of the plants (surface coming in contact with water) shall be provided with protective lining of FRP with 5 mm 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. The 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, the life of such plants can be considered to be 15–20 years.

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).



**Figure 8.59: Prefabricated Packaged Water Treatment Plants**

#### **8.14 Computer-Aided Optimal Design of Water Treatment System**

- (i) The advent of computerised 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 programmes for optimising every step of water treatment plant design.

The water treatment process involves sizing, hydraulics, electro-mechanical aspects, as well as instrumentation, and they 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 a computer-aided optimal design to construct a plant which ultimately will be useful in effective operation and maintenance including automation and SCADA.

Computer-aided design of water treatment plants only generates 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 programme. Though the computer-aided design can reduce the time to some extent, it cannot substitute the experience of the designer.

- (ii) **Prequalifications for Designer of Water Treatment Plants (Hydraulics and Process)**

The designer must hold a master's degree with 10 years of experience, or a bachelor's degree with 20 years of experience in civil, environmental or chemical engineering. The designer shall have a minimum experience of designing one plant of the same (rated) capacity, or two plants of half capacity, or three plants of one-third capacity. The designer shall provide a certificate from competent authority of satisfactory completion and approval of documents of his past jobs. Information to be included in tender specification is enclosed

in **Annexure 8.3**. Summary of recommended design criteria for WTP is enclosed in **Annexure 8.5**.

The same prequalification criteria are suggested for the approving authority.

**(iii) Pilot Plant Studies**

When selecting high-rate technologies or units, to achieve the optimum performance it is prudent to run pilot plant trials. Many equipment manufacturers have scaled down models which they can transport to the site. Pilot plant testing can optimise the design values of various parameters to suit the specific raw water source. This is especially true for high-rate filtration where media configuration, media depth and grain size can be optimised to extract the best performance.