The diameter of the circular tank is governed by the structural requirement of the trusses that carry the scraping mechanism. Circular tanks upto 60 m in diameter are in use but are generally upto 30 m to reduce wind effects. Square tanks are generally smaller usually with sides upto 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 very from 2.5 to 5 m with 3.0 m being a preferred value. Bottom slopes may range from 1 % in rectangular tanks to about 8% in circular tanks. The slopes of sludge hoppers range from 1.2:1 to 2:1 (vertical: horizontal).

7.5.6 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 particle and unhindered settling. However, contact opportunities among particles leading to aggregation increase with increasing depths for flocculent particles having tendency to agglomerate while settling, such as alum and iron flocs. The range of surface loadings and detention periods for average design flow for different types of sedimentation tanks are as follows:

Tank type	Surface loading $m^3/m^2/d$*		Detention period, hr*		Particles normally removed
	Range	Typical value for design	Range	Typical value for design	
Plain Sedimentation	upto 6000	15-30	0.01-15	3-4	Sand, silt and clay
Horizontal flow, Circular	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

7.5.7 Inlets and Outlets

Inlet structures must (i) uniformly distribute flow and suspended particles over the cross section at right angle to flow within individual tanks and into various tanks in parallel (ii) minimize large-scale turbulence and (iii) initiate longitudinal or radial flow, if high romoval efficiency is to be achieved. For uniform distribution of flow, the flow being divided must encounter equal head loss or the head loss between inlets on inlet openings must be small in

comparison to the head available at the inlets. If hi and q, are the head and discharge at the first inlet from the point of supply in a settling tank and h_n and q_n. being the head and discharge at the nth inlet opening, farthest from the point of supply, the following relationship holds

$$h_n = k\, q_n^2 = k\,(mq_1)^2 = m^2\, h_1 \qquad (7.18)$$

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

$h_n = h_1 - h_f = m^2 h_1$, and

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

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

Inlet or influent structures may have different arrangements as shown in Fig. 7.13. Each inlet opening must face a baffle so that most of the kinetic energy of incoming water will be destroyed and a more uniform lateral and vertical distribution of flow can occur. One of the satisfactory method of attaining uniform velocity of flow is to pass the water through a training or dispersion wall perforated by holes or slots. The velocity of flow through such slots should be about 0.2 to 0.3 m/s and head loss is estimated as 1.7 times the velocity head. The diameter of the hole should not be larger than the thickness of the diffuser wall.

Outlet or effluent structure comprises of weir, notches or orifices; effluent trough or launder and outlet pipe. V-notches attached to one or both sides of single or multiple troughs are normally preferred as they provide uniform distribution at low flows. 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.

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

$$Y_1 = \sqrt{Y_2^2 + \frac{2(qL.N)^2}{gb^2 Y_2}} \qquad (7.19)$$

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

Y_1 = water depth at upstream end of launder, m

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

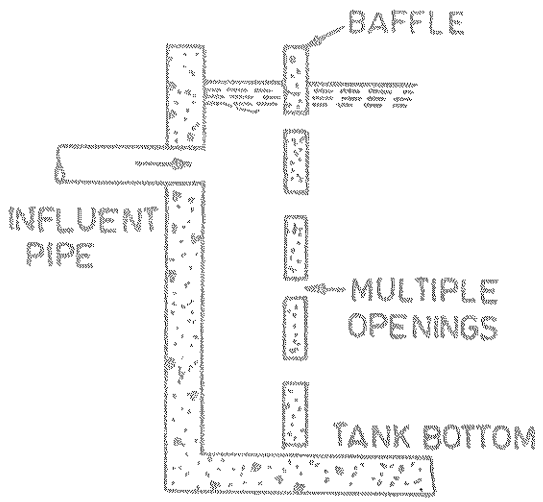
q = discharge per unit length of the weir, $m^3/s.m$

b = width of launder, m

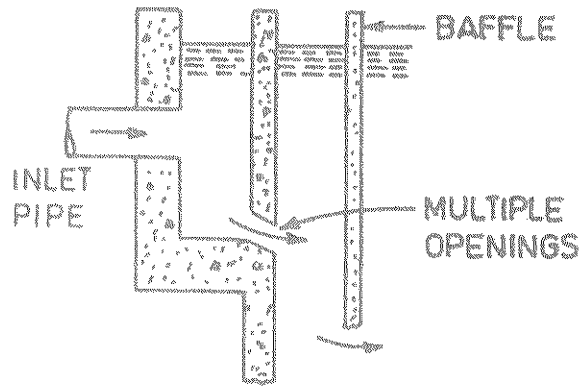
N = number of sides the weir receives the flow (one or two)

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

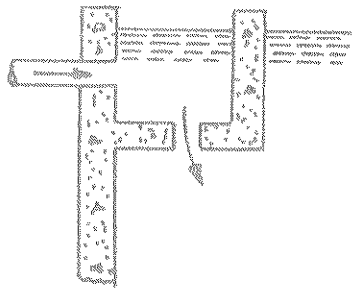
$$Y_2 = \left[\frac{(qL)^2}{b^2 g}\right]^{1/3} \qquad (7.20)$$



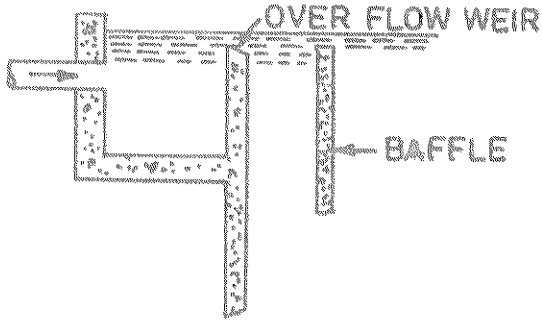
(a) DIFFUSER WALL WITH SLOTS OR PERFORATED BAFFLES



(b) INFLUENT CHANNEL WITH SUBMERGED ORIFICES IN THE INSIDE CHANNEL WALL

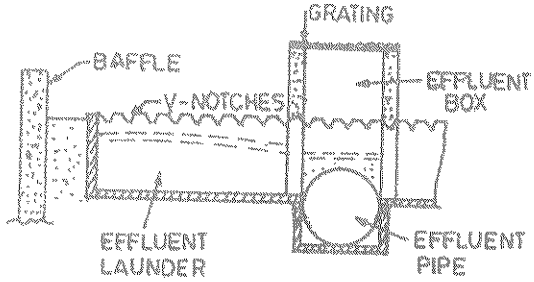


(c) INFLUENT CHANNEL WITH BOTTOM OPENINGS.

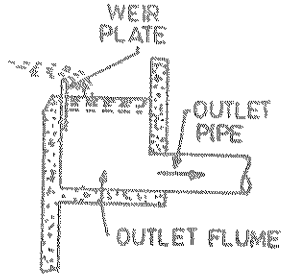


(d) OVERFLOW WEIR FOLLOWED BY A BAFFLE.

FIG. 7.13(A) FOUR TYPES OF INLETS FOR SETTLING BASINS



(a) OUTLET CONSISTING OF V-NOTCHES, EFFLUENT LAUNDER, EFFLUENT BOX AND AN EFFLUENT PIPE



(b) OUTLET WITH RECTANGULAR WEIR

FIGURE 7.13(B) : TYPICAL OUTLETS FOR SETTLING TANKS

There is a growing trend towards the use of effluent launders or troughs covering a good part of the surface of the settling basins. These are spaced at a distance of one tank depth between the troughs. The use of maximum feasible weir length in the tank from the outlet towards the inlets assists greatly in controlling density currents. Weirs, however, suffer from the difficulty in levelling which is not faced with perforated pipe launder. Perforated launders, with ports commonly submerged 30 to 600 mm below the surface are useful in varying the water level in the basin during operation and prevent floating matter passing to the filters.

7.5.8 Weir Loading

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

7.5.9 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 to 40 minutes or preferably the tip velocity of the scraper should be around 0.3 m/min or below. Power requirements are about 0.75 w/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 slopes of the hoppers should not be less than 55° to horizontal to ensure smooth sliding and removal of sludge. In such tanks special slurry weirs are provided with their crests in level with the top of sludge blanket for continuous bleeding of the excess sludge.

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

In non-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.

7.5.10 Settling Tank Efficiency

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

$$\frac{Y}{Y_0} = 1 - \left[1 + n\frac{V_0}{Q/A}\right]^{-\frac{1}{n}} \qquad (7.21)$$

where,

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

n = Coefficient that identifies basin performance

V_0 = Surface over flow rate for ideal settling basin

Q/A = Required surface overflow rate for real basin to achieve an efficiency of Y/Y_o for given basin performance.

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

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

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

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

7.5.11 Presedimentation and Storage

The turbidity of raw water from rivers and streams may exhibit wide flucutations and values exceeding a few thousand NTU are not uncommon during high flow season. The sediment load of the river during floods chiefly derives from soil erosion and consists predominantly of coarse suspended solids. Removal of large- sized and rapidly settleable silt and other materials can be accomplished by presedimentation and storage before the raw water reaches the treatment plant. Presedimentation and storage have been used for both highly turbid waters and waters of relatively low turbidity.

When removal of coarse and rapidly settling silt is aimed at in presedimentation, lower detention periods of 0.5 to 3 hours and higher surface loading of 20 to 80 $m^3/m^2/d$ have been recommended. These plain sedimentation tanks can be constructed with wooden sheet piles or dug out of the earth with sloping sides besides being or made of conventional materials like masonry or concrete. The storage basins or reservoirs, unlike presettling basins, are designed for very large detention periods ranging from about one week to a few months. While storage is best considered for waters of extremely high turbidity, big storage basins have also been constructed for waters of low initial turbidity.

7.5.12 Tube Settlers

Settling efficiency of a basin is primarily dependent upon surface area and is independent of depth. Attempts have been made to use this concept to achieve better efficiency and economy in space as well as cost. Wide shallow trays inserted within conventional basins with a view to increase the surface area have not met with success. However, very small diameter tubes having a large wetted perimeter relative to wetted area providing laminar flow conditions and low surface loading rate have shown good promise. Such tube settling devices provide excellent clarification with detention times of equal to or less than 10 minutes. Tube configurations can be horizontal or steeply inclined. In inclined tubes (about 60°) continuous gravity drainage of the settleable material can be achieved. At angles greater than 40°, the units lose efficiency rapidly whereas with angles less than 60°, sludge will not slide down the floors. Under such situations, hosing down the sediments may have to be resorted to. With horizontal tubes (normally inclined at 5°) auxiliary scouring of settled solids is necessary. 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 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 impairement of effluent quality.

The tubes may be square, circular, hexagonal, diamond shaped, triangular, rectangular or chevron shaped. A widely used material for their construction is thin plastic sheet (1.5 mm) black in colour, though plastic and asbestos cement pipes have also been used. There are number of proprietory devices such as Lemella clarifier.

7.5.12.1 Analysis Of Tube Settlers

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

$$S = \frac{V_s}{V_0}(\sin\theta + L\cos\theta) \qquad (7.22)$$

Where,

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

V_o = Velocity of flow along the tube settler

θ = Angle of inclination of tube settlers, degrees.

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

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

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

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

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

$$L' = 0.058\frac{v_0 d}{\nu} \qquad (7.23)$$

Where ν is the kinematic viscocity of water.

7.6 FILTRATION

7.6.1. General

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

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

Filtration of municipal water supplies normally is accomplished using

(a) slow sand filters, or

(b) rapid sand filters

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

7.6.2 Slow Sand Filters

7.6.2.1 General

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

When raw water turbidity is high, simple pre-treatment such as storage, sedimentation or primary filtration will be necessary to reduce it to within desirable limits. Chemical coagulation and flocculation have also been successfully tried to effectively pretreat turbid waters without adverse effect on filtrate quality by slow sand filtration.

7.6.2.2 Description

A slow sand filter consists of an open box about 3.0 m deep rectangular or circular in shape and made of concrete or masonry (Fig. 7.14). The box contains a supernatant water layer, a bed of filter medium, an underdrainage system and a set of control valves and appurtenances.

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

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

A — RAW WATER INLET VALVE
B — SUPERNATANT DRAINOUT VALVE
C — RECHARGE VALVE
D — FILTER SCOUR VALVE
E — FILTERED WATER OUTLET VALVE
F — FILTER TO WASTE VALVE
G — FILTERED WATER VALVE

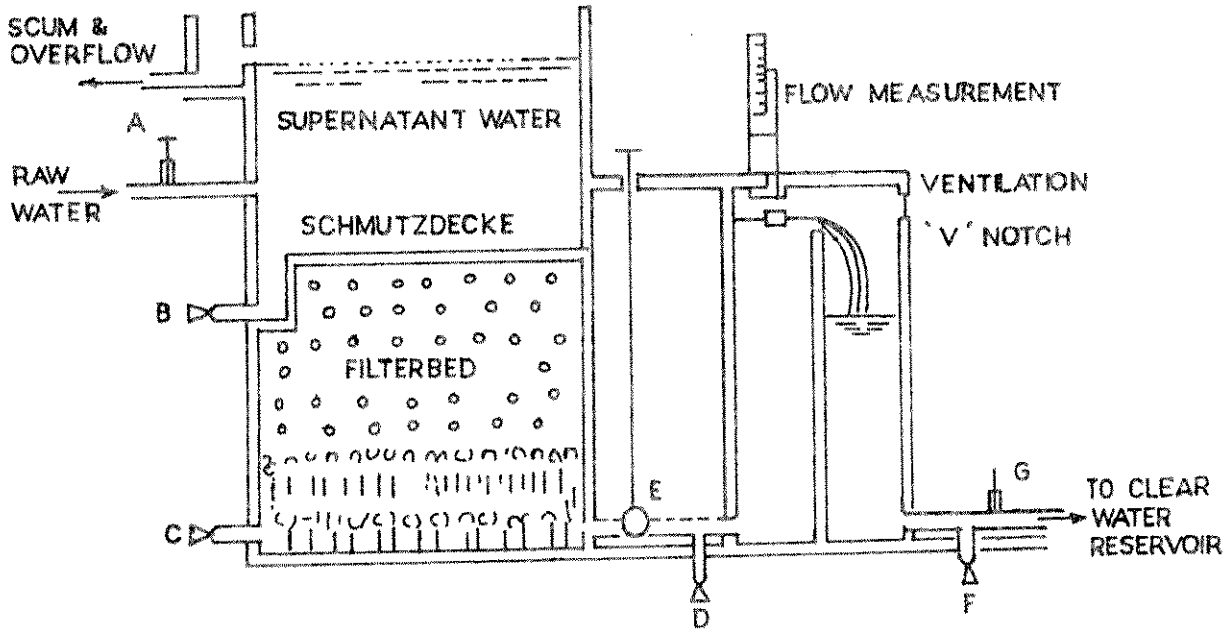


FIGURE 7.14 : BASIC ELEMENTS OF SLOW SAND FILTER (SCHEMATIC)

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

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

7.6.2.3 Purification in a Slow Sand Filter

In a slow sand filter, water is subject to various purifying influences as it percolates through the sand bed. Impurities are removed by a combination of straining, sedimentation, bio-chemical and biological processes. Shortly after the start of filtration, a thin slimy layer called the 'schmutzdecke' is formed on the surface of sand bed. It consists of a great variety of biological organisms which feed on the organic matter and convert it into simple,

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harmless substances. Considerable portion of inert suspended particles is mechanically strained out in this layer. During its passage through 0.4 - 0.6 m of sand bed, the water becomes virtually free from suspended solids, colloids, pathogens and complex salts in solution. The result is a simultaneous improvement in the physical, chemical and bacteriological quality of water. Starting with an average quality of raw water, a properly designed and operated filter can produce a filtrate satisfying normal drinking water standards. Nevertheless, the filtrate should be disinfected to render it safe.

7.6.2.4 Design Considerations

(a) Design Period

In slow sand filter construction, there is no economic advantage in building large plants to serve long years into future. Therefore, the design period should be short, say 10 years. This will help to optimise the long-term investment in water supply and will allow the available money to finance more new projects immediately.

(b) Plant Capacity

The per capita water supply multiplied by the projected future population gives the design demand. It would be convenient to convert the daily required volume to a design flow 'Q', the quantity of water to be treated per hour rather than per day, This is because the daily requirement of water may be treated over a period of 24 hours just in a few hours as is done in small plants. Thus for a given daily output, the size of the plant depends on duration of filter operation.

(c) Filtration rate and number of filters

To provide for changes in raw water quality and uncertainties in operation and maintainance, it is desirable to design filters for a normal filtration rate of 0.1 m/hr. A minimum of two filter units should be provided. This will restrict the overload rate to 0.2 m/hr when one unit is taken out for cleaning and would ensure uninterrupted production. There is no need to provide for any standby unit. The number of filter units for a given area can be increased to gain higher flexibility and reliability. For a given area, the optimum number and size of filters which will be only 10 percent more expensive than the minimum 2 bed unit are given in Table 7.2

TABLE 7.2

RECOMMENDED NUMBERS OF SLOW SAND FILTERS FOR GIVEN PLAN AREAS

Area in m^2	Number of beds
Upto 20	2
20- 249	3
250-649	4
650- 1200	5
1201 - 2000	6

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(d) Filter shape and layout

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

(e) Depth of Filter Box

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

(f) Filter Sand and Gravel

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

7.6.2.5 Construction Aspects

(a) General

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

(b) Inlet

The inlet structure is an important component of a slow sand filter and should be so designed and constructed as to cause minimum disturbance to the filter bed, while admitting raw water and to facilitate routine operation and maintenance. A filter needs to be cleaned periodically and this is done by lowering the water level a few centimeters below the sand bed and scraping the top layer of 10-20 mm of sand. It is found in practice that draining the water through the filter bottom takes several hours, at times 1-2 days. In order to obviate this difficulty, a supernatant drain out chamber with its top just above the sand level, has to be provided. By a proper design, the filter inlet and the supernatant drain out could be suitably combined in a single chamber (Fig. 7.15).

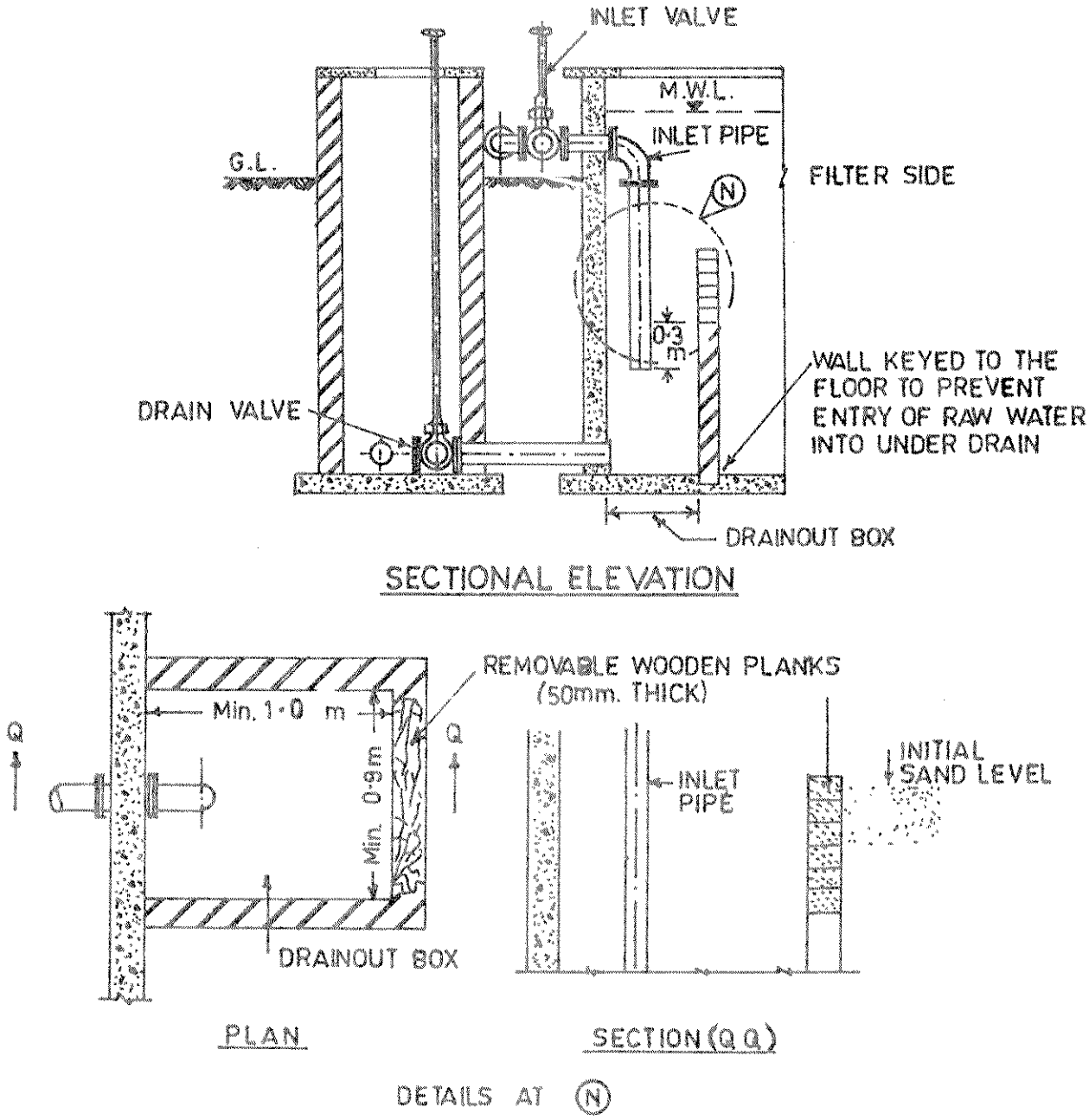

FIGURE 7.15 : INLET CUM SUPERNATANT DRAINOUT BOX

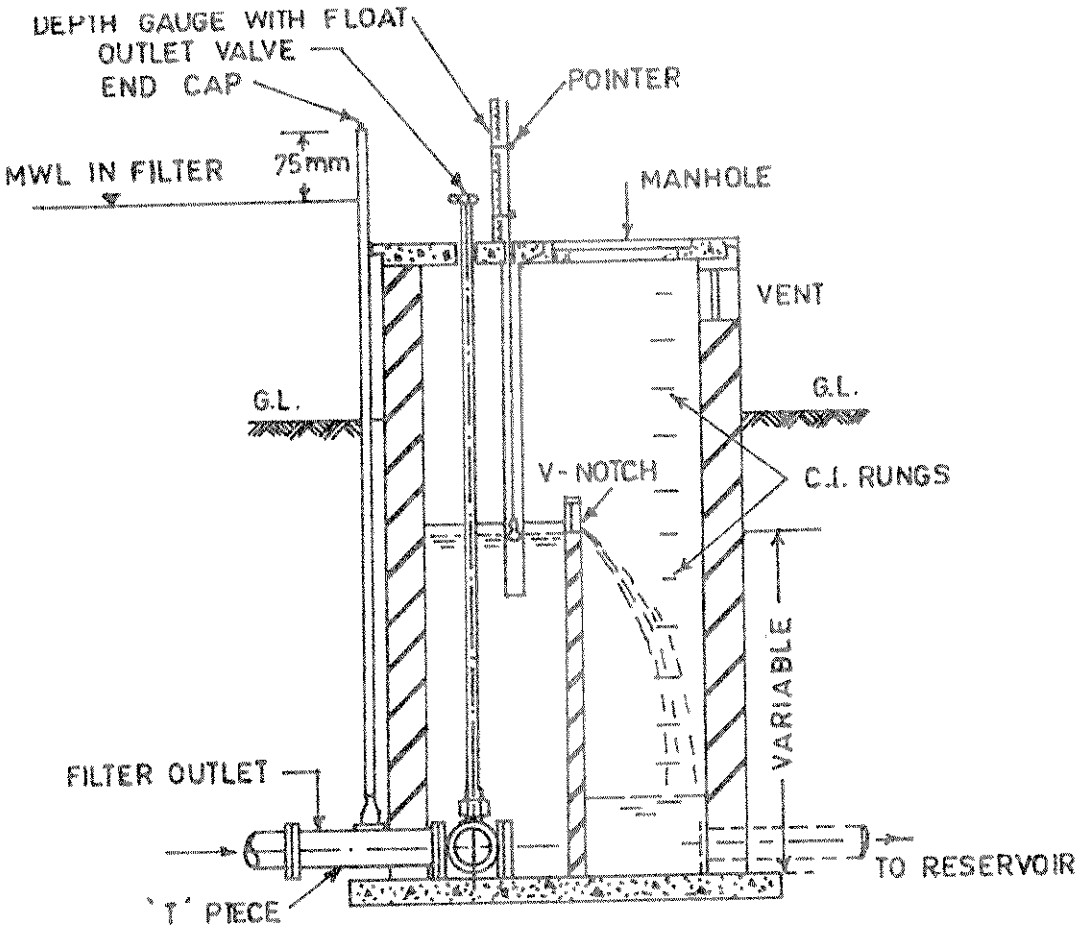

FIGURE 7.16 : OUTLET CHAMBER

(c) Outlet

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

In small filters, the outlet chamber is usually constructed in two parts separated by a wall with a weir. The sill of the weir is fixed above the highest sand level in the filter bed. This makes filter operation independent of fluctuations in the clear water storage level and prevents occurrence of negative head in the filter. It also aerates the filtered water thus raising its oxygen content. To facilitate aeration, a ventilation opening properly screened is provided in the chamber (Fig. 7.16).

(d) Scum and overflow outlet

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

7.6.2.6 Operation And Maintenance

(a) Initial Commissioning

While commissioning, a newly constructed filter is charged with water from bottom through the underdrain until it rises 0.1-0.15 m above the sand bed. This ensures expulsion of entrapped air in the filter bed and the underdrainage system. The inlet valve is then gradually opened and water is admitted to the filter from top. The water is allowed to filter at approximately the normal filtration rate and the effluent is run to waste till the formation and 'ripening' of the filter skin is complete. The ripening period is complete when the bacteriological analysis indicates that the effluent quality is good and can be put into the distribution system.

(b) Flow Control

The rate of filtration can be controlled either at the inlet or at the outlet of the filters. Both methods have advantages.

In an inlet-controlled filter, the rate of filtration is set by the inlet valve. Once the desired rate is reached, no further manipulation of the valve is required. At first the water level over the filter will be low but gradually it will rise to compensate for the increasing resistance of the filter skin. Once the level has reached the overflow outlet, the filter has to be taken out for cleaning.

Inlet control reduces the amount of work which has to be done on the filter to just clean it. The rate of filtration will always be constant with this method and the build-up of resistance in the filter skin is directly visible. On the other hand, the water is not retained for very long at the beginning of the filter run, which may reduce the efficiency of treatment.

In an outlet-controlled filter, which is more common, the rate of filtration is set with the outlet valve. Daily or every two days this valve has to be opened a bit to compensate for the increase in resistance in the filter skin. The disadvantage of this method is that the outlet valve also has to be manipulated on a regular basis, causing a slight variation in the rate of filtration. Thus, the operator is forced to visit the plant atleast every day, otherwise the output will fall. The water is retained for five to ten times as long as in the inlet-controlled filter at the beginning of the filter run, which may make purification more efficient. Removal of scum will also be much simpler than with inlet-controlled filtration.

In many situations electricity and diesel fuel are not available all the time, so existing slow sand filter plants sometimes function only for part of the day. Research has shown that this leads to a serious deterioration in the quality of the out-flowing water and must be avoided. If electricity for working pumps is likely to be intermittent, either a raw-water storage reservoir which can feed water to the filters under gravity supply can be built, or 'declining rate filtration' should be used. That is, when the raw water is stopped, all valves remain in the same position and filtration continues at a declining rate as the water level in the plant falls. When the raw water supply is resorted, the water standing over the sand bed will rise to its earlier level. Where declining rate filtration is used, a larger filter is needed.

(c) Filter Cleaning

When the filter has attained the maximum permissible headloss, it is taken out of service for cleaning. The inlet is closed and the supernatant is drained out or allowed to filter though so as to expose the sand bed. Experience has shown that filtering through takes a long time, occasionally even one or two days. Hence, lowering the water level by opening the supernatant drain out valve should be preferred. When the supernatant is drained out, the water level is lowered 10- 15 cm below the top of the sand bed by opening the scour valve. Without allowing the bed to dry up, the filter is cleaned manually by removing the top layer of 2-3 cm of sand along with the filter skin. The filter is returned to service by admitting through bottom filtered water from the adjacent filter to a level of a few centimeters above the sand bed before allowing raw water from top. The removed sand is washed, dried and stored for future use.

(d) Resanding

Due to periodic cleaning, when the sand depth is reduced to a minimum of 0.4 m, it is necessary to make up the sand depth to the original level. This is done by replenishing with a fresh lot of sand, taking care to see that the remaining old sand is placed on top of the new sand. This avoids accumulation of dirt in the deeper layers of filter bed and helps in quick ripening after resanding.

7.6.2.7 Cost Aspects

(a) Minimum filter cost

The cost of a filter excluding pipes and valves is made up of two components: the total cost for floor, underdrains, sand and gravel; and the cost of walls of the filter box.

This cost in general is

$$C = K_A A + K_P P$$

Where, A is the total filter bed area in m^2 ; P the total wall length in m, K_A, the cost per unit area of filter bed, and K_P, the cost per unit length of wall. For rectangular filters with common walls, the problem is to minimize C subject to:

$$A = nlb \text{ and } P = 2nb + l(n+1) \tag{7.25}$$

Where 'n' is the number of filters, b is breadth, and 'l' is the filter length.

The term $K_A A$ is constant for any value of 'n' and any filter shape. Hence, the minimum cost solution is the solution that minimises P, which is

$$l^2 = \frac{2A}{n+1} \tag{7.26}$$

and

$$b = \frac{(n+1)l}{2n} \tag{7.27}$$

The equation for b, when re-arranged, shows that 2 nb = (n + 1)l, or the condition for minimum filter cost is to have the sum of the length equal to the sum of the breadths. It can be shown that this is true whether filter units are arranged in a single row or as blocks on each side of a central gallery. The general expression for the minimum cost is found by substituting Eqn. 7.26 and 7.27 for Eqn. 7.24.

$$C = K_A A + 2K_P\left(\sqrt{2A(n+1)}\right) \tag{7.28}$$

The values of K_A and K_p, can be worked out for any place based on prevailing prices for construction materials. For Nagpur, India (1983),

$$C = 500A + 1660\left(\sqrt{2A(n+1)}\right) \tag{7.29}$$

(b) Economy of Scale

A general cost model for the filter beds be written as:

$$C = K(A)^a \tag{7.30}$$

Where 'A' is the total area of the filter beds, K is the cost per unit area of filter bed construction including walls, and 'a' is the exponent that represents the economy of scale factor.

The cost data obtained from Eq. 7.29 for various values of A can be used to determine the parameters K and 'a' of the function by the method of least squares. The resulting equation for Nagpur (1983) is given by:

$$C = 1617\ A^{0.869} \tag{7.31}$$

Large economics of scale are associated with small values of the exponent. Until the exponent decreases to about 0.6 or 0.7, there is no economic incentive to overdesign. Thus, very little saving is accomplished by increasing the size of the project in order to provide service over a long time into the future.

(c) Cost of Slow Versus Rapid Sand Filters

There is a general misconception that slow sand filters, because of their relatively larger area, are expensive. However, this is not always true. Comparative cost analysis for slow and rapid filters has shown that that slow sand filters are cost effective, especially for rural and small community water supplies. The economic capacities have to be determined for specific situations using local cost data before deciding on the choice between the two types of filters.

TABLE 7.3

SUMMARY GUIDELINES FOR DESIGN OF SLOW SAND FILTERS

Description	Recommended Design value	Description	Recommended Design value
Design period	10 years	Depth of Supernatant water	1.0m
Filteration rate		Free Board	0.2m
Normal Operation	0.1m/hr	Depth of filter sand Initial	1.0m
Max. overload rate	0.2m/hr	Final (Minimum)	0.4m
Number of filter beds Minimum	2	Size of sand Effective size	0.2-0.3mm
Areas upto 20 m^2	2	Uniformity coefficient (U.C)	5
Areas upto 20-249 m^2	3	Gravel (3-4 layers) depth	0.3m
Areas upto 250-649 m^2	4	Underdrains (made of bricks or perforated pipes)	0.2m
Areas upto 650-1200 m^2	5	Depth of filter box	2.7m
Areas upto 1201-2000 m^2	6	Effluent weir level above sand bed	20-30mm

7.6.3 Rapid Sand Filters

7.6.3.1 Filtration Process

The rapid sand filter comprises of a bed of sand serving as a single medium granular matrix supported on gravel overlying an underdrainage system. The distinctive features of rapid sand filtration as compared to slow sand filtration include careful pretreatment of raw water to effectively flocculate the colloidal particles, use of higher filtration rates and coarser but more uniform filter media to utilise greater depths of filter media to trap influent solids

without excessive head loss and backwashing of filter bed by reversing the flow direction to clean the entire depth of filter. Pretreatment of filter influents should be adequate to achieve efficient removal of colloidal and suspended solids despite fluctuations in raw water quality. A typical sketch for granular medium gravity flow filter is shown in figure 7.17.

When water containing suspended matter is applied to the top of filter bed, suspended and colloidal solids are left behind in the granular medium matrix. Accumulation of suspended particles in the pores and on the surface of filter medium leads to build up of head loss as pore volume is reduced and greater resistance is offered to the flow of water simultaneously with the build up of head loss to 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 break through 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.

7.6.3.2 Principal Mechanisms of Partiicle Removal

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

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

The particle attachment step is a physicochemical process involving electrostatic interactions, Van der Waal's forces of molecular attraction, chemical bridging or specific adsorption. Attachment is affected by chemical characteristics of the water and filter medium. Pretreatment of filter influents by coagulants and pH of water affect the efficiency of attachment step and consequently of solid removal in a filter. The need, therefore, of adequate pretreatment before filtration to achieve efficient removal of suspended solids is evident.

Dimensionless parameters have been defined for various transport and attachment mechanisms and mathematical equations are proposed to predict the removal efficiency of particles based on physical variables such as dp, dm, ρ_s, v, ρ and μ the liquid density and

absolute viscosity, porosity of filter medium and concentration of suspended solids and chemical characteristics of water and filtering medium. An analysis of these analytical expressions indicates that filter efficiency may improve by decreasing the size of filter media, reducing the rate of filtration and at higher temperatures. The suspended particles falling into the categories of significantly more than and less than 1μm dia are efficiently removed. A particle with a size of 1 μm has the lowest efficiency of removal. Further, ample attachment opportunities exist in conventional deep granular filters. If such filters do not remove solids efficiently, pretreatment should be changed to improve attachment of suspended particles to filter media grains.

7.6.3.3 Rate Of Filtration

The standard rate of filtration through a rapid sand filter is usually 80 to 100 lpm/m^2 (4.8-6m/hr). Practice is tending towards higher rates (upto 10 m/hr) in conjuction with greater care in conditioning the water before filtration and with the use of coarser sand (effective size upto 1 mm). A prudent arrangement would be to design the filters on the basis of average consumption at a normal rate of 4.8 m/hr but with the inlet and the outlet control arrangements designed to permit a 100% overload for emergent occasions.

7.6.3.4 Capacity Of A Filter Unit

The capacity of the rapid sand filters should be such that the number of units can take care of the total quantity of water to be filtered and is optimum to keep the filters working without undue overloading at any time. The 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^2 for a single unit is recommended for plants of greater than 100 mld consisting of two halves each of 50 m^2 area. Also for flexibility of operation a minimum of four units should be provided which could be reduced to two for smaller plants.

7.6.3.5 Dimensions Of Filter Unit

Layout of the plant, economy and convenience determine the relationship between the length and the breadth of the units. Where filters are located on both sides of a pipe gallery, the ratio of length to width of a filter-box has been found to lie, in a number of installations, between 1.11 and 1.66 averaging about 1.25 to 1.33. A minimum overall depth of 2.6m including a free board of 0.5 m is adopted.

The filter shell may be in masonry or concrete to ensure a water tight structure. Except in locations where seasonal extremes of temperature are prevalent, it is not necessary to provide a roofing over the filters, the operating gallery alone being roofed over.

7.6.3.6 Filter Sand

Filter sand is defined in terms of effective size and uniformity coefficient. Effective size is the sieve size in millimeters that permits 10% 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% by weight and the effective size.

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

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

(b) Effective size shall be 0.45 to 0.70 mm.

(c) Uniformity coefficient shall not be more than 1.7 nor less than 1.3.

(d) Ignition loss should not exceed 0.7 per cent by weight.

(e) Soluble fraction in hydrochloric acid shall not exceed 5.0% by weight.

(f) Silica content should be not less than 90%.

(g) Specific gravity shall be in the range between 2.55 to 2.65.

(h) Wearing loss shall not exceed 3%.

IS:8419 (Part I) 1977 entitled Filtration Media sand and Gravel may be referred to for details.

7.6.3.7 Depth Of Sand

Usually the sand layer has a depth of 0.60 to 0.75 m, but for higher rate filtration when the coarse medium is used, deeper sand beds are suggested. The standing depth of water over filter varies between 1 and 2m. 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.

7.6.3.8 Preparation Of Filter Sand

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

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

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

$$p_4 = p_1 - 0.1\, p_3 = p_1 - 0.2\,(p_2 - p_1) = 1.2\, p_1 - 0.2\, p_2$$

Likewise, the percentage p_5 above which the stock sand is too coarse for use is

$$p_5 = p_2 + .40\% \text{ of usable sand}$$

$$= p_2 + 0.4 \times 2(p_2 - p_1) = p_2 + 0.8(p_2 - p_1) = 1.8\, p_2 - 0.8\, p_1$$