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

7.6.3.9 Filter Bottoms And Strainer Systems

The under-drainage system of the filter is intended to collect the filtered water and to distribute the wash water in such a fashion that all portions of the bed may perform nearly the same amount of work and when washed, receive nearly the same amount of cleaning. Since the rate of wash is several times higher than the rate of filtration, the former is the governing factor in the hydraulic design of filters which are cleaned by backwashing.

The most common type of under-drain is a central manifold with laterals either perforated on the bottom or having umbrella type strainers on top. Other types such as wheeler bottom-a false bottom with strainers spaced for the entire area at regular intervals or a porous plate floor supported on concrete pillars are all satisfactory when properly designed and constructed. Porous plates, however, are likely to be clogged by minute quantities of alumina which can penetrate through the filter bed which might lead to rupture.

In the case of central manifold with lateral system, the manifolds, headers and laterals are of cast iron, plastic, asbestos cement, concrete or other material. The velocity of jets issuing from perforations or orifices is destroyed by directing the openings downwards against the filter bottom and into the coarse gravel surrounding the pipes. The lost head, therefore, will be equal to the driving head during the wash. In practice this controlling head loss is set between 1 to 4.5m. A nonferrous under-drain system is preferable where the water has a low pH and is corrosive and when the correction for pH has to follow the filtration process. However, A.C. Pipes have a tendency to dissolve away in the presence of low pH alum treated waters.

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 axis of the pipe.Spacing of perforations along the laterals may vary from 80 mm for perforations of 5 mm to 200 mm for perforations of 12 mm.

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

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

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

The central manifold with lateral type of underdrainage system is shown in Fig. 7.18.

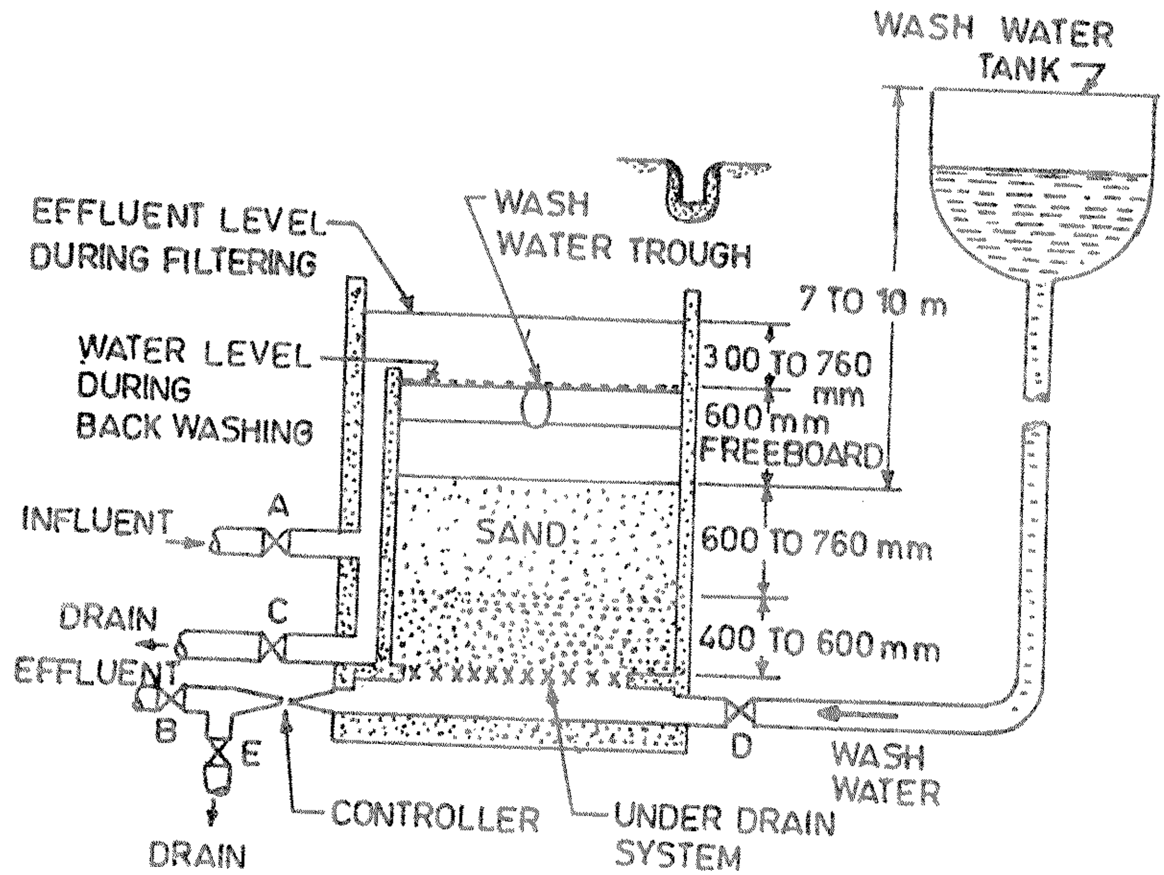

FIG. 7.17 DEFINITION SKETCH FOR OPERATION OF DOWNFLOW, GRANULAR–MEDIUM GRAVITY–FLOW FILTER

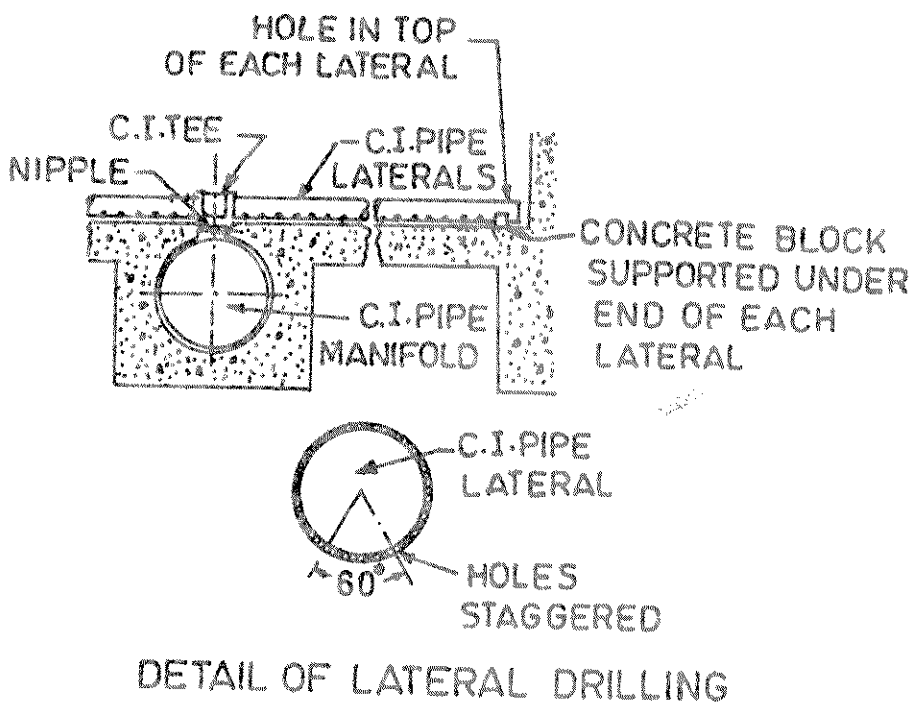

FIGURE 7.18 : A PERFORATED PIPE UNDERDRAIN

7.6.3.10 Filter Gravel

Gravel is placed between the sand and the underdrainage 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 to 5 mm at the top with a 0.45 m depth. The faster the rate of application of water, the larger the gravel size required. Reference may be made to IS: 8419 Part (I)-1977 for filter gravel.

The depth will vary according to the type of filter bottom and strainer system used, except in the case of porous bottom where no gravel is required. Wheeler bottoms and other false bottoms may be subsituted for part of coarser layer of gravel. The filter gravel shall be as spherical as possible, hard, clean and uniform in quality and also shall not contain such impurities as dirt and clay. Size of gravel and depth of gravel layer shall be determined in accordance with the following rules:

(a) for strainer or Wheeler type underdrain system, gravel shall be of 2 mm minimum size, 50 mm maximum size and 0.30 to 0.50 m deep; and

(b) for perforated pipe under-drain system, gravel shall be 2 mm minimum size, 25 mm maximum size and 0.50 m, in depth.

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

7.6.3.11 Wash Water Gutters

Materials used for wash water gutters include concrete, asbestos cement, plastic, cast iron and steel. While the horizontal travel of dirty water over the surface of the filter is kept between 0.6 to 1.0 m before reaching the gutter, there are successful units with troughs eliminated and having only main gutters where the dirty water travel has been as high as 3 m It is uneconomical to place wash water gutters against the side walls of the filter. The upper edge of the washwater gutter should be placed sufficiently near to the surface of the sand so that a large quantity of dirty water is not left in the filter after the completion of washing. At the same time, the top of the wash water gutter should be placed sufficiently high above the surface of the sand so that sand will not be washed into the gutter. The edge of the trough should be slightly above the highest elevation of the sand as expanded in washing. Where this height cannot be determined by test, a convenient rule is to place the edge of the gutter as far above the undisturbed sand surface as the washwater rises in one minute. Air and water should not be applied simultaneously with such gutter heights. The gutter should be large enough to carry all the water delivered to it with at least 50 mm between the surface of the water flowing in the gutter and the upper edge of the gutter. Any submergence of the gutter will reduce the efficacy of the wash. The gutter may be made with the same cross-section throughout its length or it might be constructed with varying crosssection increasing in size towards the outlet end. The bottom of the gutter should clear the top of the expanded sand by 50 mm or more.

The troughs are designed as free falling weirs or spillways, For free falling rectangular troughs with level invert, the discharge capacity Q in m^3/s may be computed from the formula

$$Q = 1.376\, bh^{3/2} \qquad (7.32)$$

Where b is the width of the trough in m and h is the water depth in m.

7.6.3.12 High Rate Backwash

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

Normally, the rate at which wash water is applied, where no other agitation is provided, is 36 m (600 lpm/m^2) for a period of 10 minutes. The tendency in design is towards higher rates of washing, primarily because of the larger sizes of sand being used, which require a faster application of water for equal expansion unless surface agitation by auxiliary means is provided. The maximum friction that particles which are free to move and expand can offer is their submerged weight in water. Increasing the flow, any further beyond this point, may lead to the carryover of the grains along with the wash water. For high rate wash, the pressure in the underdrainage system should be 6 to 8 m with the wash-water requirement being 40-50 m/hr for a duration of 6 to 10 minutes.

The supply of wash water can be made through an overhead storage tank or by direct pumping or by tapping the rising main of the treated water if the clear water motors are not overloaded. The capacity of the storage tank must be sufficient to supply wash water to two filter units, at a time, where the units are four or more.

7.6.3.13 Surface Wash

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

(a) Hydraulic System

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

shall be located on side and bottom of arms and jet action of water causes the arms to rotate at a rate of 7 to 10 rpm.

(b) Air Wash System

In the air wash system, compressed air is used to secure effective scrubbing action with a smaller volume of wash water. The air may be forced through the under-drains before the wash-water is introduced or through a separate piping system placed between the gravel and the sand layer. Though the former results in better washing, the gravel is likely to be disturbed. With the former procedure, free air of about 36 to 45 m/hr (600 to 900 lpm/m^2 of the filter area) at 0.35 kg/cm^2 is forced through the underdrains until the sand is thoroughly agitated, for a duration of about 5 minutes following which, wash water is introduced through the same under-drains at a rate of 24 to 36 m/hr (400 to 600 lpm/m^2 of area). On the other hand, with the latter procedure, while water is forced through the under-drains, about the same volume of air is forced simultaneously through a separate piping. In the practice of backwashing employing conjunctive air and water wash, air is usually applied at a rate of 45-50 m/ hr and water at 12-15 m/hr.

7.6.3.14 Operation Of Filters

Before starting a filter it is backwashed at increasing rates until the sand bed has been stratrified vertically by the wash water which carries various sizes of sand to different levels. The filter run of rapid gravity filters depends on several attendant factors. It is necessary to calculate the total loss of head in backwashing to arrive at the pump capacity and staging height of backwash water reservoir. The total loss of head includes loss due to expansion of sand, loss in orifices or underdrainage system, loss in incoming pipe and height of wash water gutter with respect to underdrainage system. The loss of head immediately after washing should not exceed 0.3 m. The finer the sand or the greater the rate of filteration, the greater the initial head loss. The head loss builds as the filter grows dirty during a run. It is usual to allow a filter head loss of 1.8 to 2 m before cleaning such filters. Under no circumstances, a build-up of negative head within the filter media be allowed.

The duration of the washing process varies for different conditions of cleaning of the filter, sizes and character of filter media, rate of wash and the desired quality of filtrate. It should not normally exceed 10 minutes.

In a properly operated plant, the quantity of wash water used should not exceed 2% of the total amount of water filtered. Lower amounts are possible where large quantities of wash water are involved and water is scarce, feasibility of the reuse or recycling of wash water after requisite treatment has also to be explored. Following the washing process, it is usually advisable to waste the first few minutes of flow through the filter, unless the quality of the filter effluent immediately following the wash may make this unnecessary. A turbidity of 1.0 NTU or less, measured by an accurate instrument, is the best criterion of the stability of effluent of a freshly washed filter. The test can be used to advantage in most plants.

The water standing on the bed at the close of of wash should be clear with a turbidity not exceeding 10 NTU. In a well designed and operated filter, there should be no air binding either during filtration or during washing; there should be no carry away of sand with the washed water and the sand bed should settle down fairly uniformly without undulations.

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Formation of mud balls and their retention in the bed even after washing indicates poor performance. At the commencement of the filter run after a wash, the initial loss of head should not exceed 0.3 m.

7.6.3.15 Hydraulics Of Filtration

The head loss, h, through a clean filter bed of depth l can be computed using Kozeny's equation

$$\frac{h}{l} = \frac{k}{g} \cdot \frac{\mu}{\rho} \cdot v \cdot \frac{(l-f)^2}{f^3} \left(\frac{A}{V}\right)^2 \qquad (7.33)$$

Where,

k = residual dimensionless coefficient, about 5 under most condition of water filtration,

μ = absolute viscosity of water, $\frac{(N.s)}{m^2}$

ρ = density of water, (Kg/m^3)

v = macroscopic velocity of filtration, (m/s)

f = porosity of clean sand bed, dimensionless

A = surface area of the grains (m^2)

V = volume of the grains (m^3)

For unisize sperical medium particles of diametere d

$$\frac{A}{V} = \frac{6}{d}$$

For non-sperical grains, sphericity is defined as the surface area of the equivalent volume sphere to actual surface of non-spherical particles. The sphericity, ψ assumes values of 1.0 for spherical grains, 0.98 for rounded grains, 0.94 for worn grains, 0.81 for sharp grains, 0.78 for angular grains and 0.70 for crushed grains for sand medium

For stratified beds, as obtainable in rapid sand filters after back washing, the head loss in a clean bed is the sum of the head losses in successive sand layers. If p_i, is the fraction of medium of sieved size di, the head loss is given by

$$\frac{h}{l} = \frac{k}{g} \cdot \frac{\mu}{\rho} \cdot v \cdot \frac{(1-f)^2}{f^3} \left(\frac{6}{\psi}\right)^2 \sum_{i=1}^{n} \frac{p_i}{di^2} \qquad (7.34)$$

For unstratified beds e.g. slow sand filter, the head loss becomes

$$\frac{h}{l} = \frac{k}{g} \cdot \frac{\mu}{\rho} \cdot v \cdot \frac{(1-f)^2}{f^3} \cdot \left(\frac{6}{\psi} \sum_{i=1}^{n} \frac{p_i}{di}\right)^2 \qquad (7.35)$$

7.6.3.16 Hydraulics Of Backwashing

High rate granular filters are backwashed to remove the impurities lodged in the medium matrix. The hydraulics of backwashing concerns with the determination of head loss across the filter bed during backwashing and to estimate backwash velocity at any required level of bed expansion and con-commitant porosity of expanded bed.

As the water is applied in upflow mode to a granular medium or media, frictional resistance is offered by the filter grains due to skin friction and form drag. The initial effect at low velocities of flow is to result in reorientation of the particles to minimize frictional resistance. At low backwash velocities, the filter bed does not expand and its porosity does not change. The head loss or pressure drop is a linear fuction of upward flow velocity at low velocities. As the water velocity is increased, the frictional resistance also increases till it reaches a value equal to the gravitational force acting upon the filter grains. Any further increase in the velocity of water fluidizes the filter bed resulting in bed expansion and increasing porosity of filter bed.

(a) Head loss across filter bed

The maximum frictional resistance that can be offered by the filter grains in fluidized state is their submerged weight. The head loss across the filter bed in fluidized condition is given by the equation:

$$\frac{h_b}{l_e} = \frac{(\rho_m - \rho)}{\rho}(1 - f_e) \qquad (7.36)$$

Where,

h_b = headloss across filter bed during backwashing, (m)

l_e = height of the expanded bed, (m)

ρ_m = mass density of the filter grains (Kg/m^2)

ρ = mass density of water, (Kg/ml)

f_e = porosity of expanded bed, dimensionless

Since the grain volume does not change before and during backwashing,

$(1\text{-}f)l = (1\text{-}f_e)l_e$

Eq. (7.36) can be rewritten using eq. (7-37)

$$h_b = \frac{(\rho_m - \rho)}{\rho}(1 - f)l \qquad (7.38)$$

(b) *Estimation of Backwash Velocity*

Several approaches are available for computation of backwash velocity to achieve a desired degree of bed expansion and attendant expanded bed porosity or to estimate bed expansion and expanded bed porosity at a given backwash velocity.

According to one of the approaches, first minimum fluidization velocity (v_{mf}) which is the superficial fluid velocity required to initiate fluidization of the bed is computed from the empirical nonhomogenous equation:

$$V_{mf}\,(gpm\ /\ ft^2) = \frac{0.00381\,(d_{60})^{1.82}\left[W_s(W_M - W_S)\right]^{0.94}}{\mu^{0.88}} \qquad (7.39\text{ (a)})$$

where,

v_{mf} = minimum fluidization velocity , U.S. gallons/minute/ft^2

d_{60} = 60% finer size of sand, mm

W_m, W_s = Specific weights of filter medium and water, lbs/ft^3

μ = absolute viscosity, centipoise

or in SI units

$$V_{mf}\,(m\ /\ h) = \frac{1.185 \times 10^{-7}(d_{60})^{1.82}\left[\rho(\rho_m - \rho)\right]^{0.94}}{\mu^{0.88}} \qquad (7\text{-}39\text{ (b)})$$

Where,

d_{60} = 60% finer size of sand (mm)

ρ = mass density of water, (kg/m^3)

ρ_m = mass density of filter medium, (kg/m^3)

μ = dynamic viscosity of water, (kg/m.s)

The minimum fluidization velocity is used to compute Reynolds number, Re_f.

$$Re_f = \frac{\rho V_{mf} d_{60}}{\mu} \qquad (7.40)$$

If Re_f is greater than 10, a multiplying correction factor K_R must be applied to V_{mf}:

$$K_R = 1.775\ Re_f^{-0.272}$$

The unhindered settling velocity, V_S, of the hypothetical average particle is then calculated using equation:

$$V_S = 8.45\ V_{mf} \qquad (7.41)$$

The Reynolds number for this particle, based on V_S, is determined

$$Re_0 = \frac{\left(\rho\ V_s\ d_{60}\right)}{\mu} = 8.45\ Re_f \qquad (7.42)$$

Using the value of Re_0 the expansion coefficient, n is computed using the equation:

$$n = 4.45\ Re_0^{-0.1} \text{ for } 1<Re_0<500 \qquad (7.43)$$

This value of expansion coefficient is finally used to compute the required backwash velocity, V_b, for a given bed expansion or expanded bed porosity.

$$V_b = V_{mf}\left(\frac{f_e}{f}\right)^n \qquad (7.44)$$

The expanded bed porosity, f_e, can be determined from the eq. (7.37) for a given bed expansion.

The expansion of graded filter sands at different temperatures has been predicted to an excellent degree by the above calculations. However, the application of this procedure to graded coal beds has yielded rather poor expansion prediction.

In another approach given in literature the expanded bed porosity of a fluidized bed is expressed by the equation:

$$f_e = \left(\frac{V_b}{V_s}\right)^{0.2} \tag{7.45}$$

Where, V_s is the unhindered settling velocity of the filter grain particle, determined using stokes law.

7.6.3.17 Optimum Backwashing

It is opined that collisional interactions between media grains do not occur in the fluidized state during backwash and the principal mode of cleaning is by hydrodynamic shear. Theoretically, supported with experimental evidence, it has been demonstrated that maximum hydrodynamic shear occured at expanded bed porosities of about 0.7.

According to Camp and Stein's equation, applied to a backwashed filter,

$$\frac{dv'}{dl} = \left[\frac{gv'}{\nu}\frac{dh}{dl}\right]^{\frac{1}{2}} \tag{7.46}$$

Where $\frac{dv'}{dl}$ = velocity gradient within pores

v' = velocity within pores

$\frac{dh}{dl}$ = head loss per unit length

Hydrodynamic shear 'γ' is given by

$$\gamma = \mu\left(\frac{dv'}{dl}\right) \tag{7.47}$$

combining equations (7.47) and (7.46)

$$\gamma = \left[\mu\left(\frac{gv'}{\nu}\cdot\frac{dh}{dl}\right)\right]^{\frac{1}{2}} \tag{7.48}$$

The velocity within pores can be expressed by

$$v' = \frac{v_e}{f_e} = K_e(fe)^{n-1} \tag{7.49}$$

$$and = \frac{dh}{dl} = \frac{(\rho_s - \rho)}{\rho}(1 - fe) \tag{7.50}$$

$$\gamma = K.[f_e^{n-1} - f_e^{n}]^{1/2} \tag{7.51}$$

where $K = [\mu g \, Ke(\rho_s - \rho)^{0.5}]$

Differentiating Eq. 7.51 and equating to zero:

$$\frac{d\gamma}{df_e} = K.\frac{1}{2}\left[f_e^{n-1} - f_e^{n}\right]^{3/2}\left[(n-1)f_e^{n-2} - nf_e^{n-1}\right] = 0$$

Optimization of the equation for shear intensity, by differentiating the equation and equating to zero yields the following expression for the porosity of maximum hydrodynamic shear:

$$f_e = \frac{n-1}{n} \tag{7.52}$$

According to this equation, the maximum hydrodynamic shear occurs in a fluidized bed at porosities of 0.68 to 0.71 for typically sized filter sands which corresponds to an expansion of 80 to 100%. However, the curve of the hydrodynamic shear versus porosity is quite flat, indicating that washing at porosities different from the theoretical optimum does not result in a major decrease in the efficiency of cleaning process. Optimal cleaning has been observed in some cases at expansion of 16-18% only.

It has been found that there is lack of abrasion during water backwash and therefore a backwashing with water alone is inherently a weak cleaning process. For effective cleaning, abrasion resulting from collision, between grains is achieved by auxiliary process like surface wash or air scour (Section 7.6.3.13).

7.6.3.18 Appurtenances

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

(a) Rate of Flow Controllers

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

Sudden changes of rate of flow also must be avoided if the filter medium is to be maintained in an unbroken and efficient condition. Any changes in rate must be gradual and predetermined maximum must not be exceeded. Such unfavourable operating conditions may be eliminated by the use of rate of flow controllers.

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

Rate of flow controller may be either of double beat type or venturi type. The later type consists of a venturi section, diaphragm chamber arrangement, valve mechanism and casing, counter-weighted scale beam group and recovery out-let section. By virtue of the arrangement of the parts, straight line flow through the unit is simulated.

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

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

However, it must be emphasized that rate of flow controllers require proper operation and maintenance to ensure that filtration is done at constant rate. These devices are omitted where declining rate of filtration is adopted.

(b) Filter Gauges

Filter gauges are essential to the operation of the modern filter plant in order to measure accurately the rate of flow through each filter box and to determine the loss of head occuring 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 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.

(c) Sand Expansion Gauges

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

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

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

7.6.3.19 Pipe Gallery

The influent, effluent, wash and waste water pipes together with rate controllers and appurtenances 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.

7.6.3.20 Limitations Of Rapid Sand Filters

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

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

7.6.4 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.

7.6.4.1 Constructional Features

The enclosure tank containing filter media is usually a rectangular box, made of concrete or masonry. The plan area of these filters may range between 40 and 200 m^2 with depths between 2.5 and 3.5 m. The filter media is supported on gravel laid over top of the under-drainage system. In addition to the under-drainage system, used for collecting filtered water and distributing the backwash water, the tanks have troughs spanning across the length or width of filter for distribution of water to be filtered and for collection of wash water. The troughs remain submerged during filtration and their top edge is normally kept 600 mm above the filter medium to prevent loss of medium during backwash and to 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 venturimeter or rate of flow recorder are also located above and/or below the gallery floor.

7.6.4.2 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 effective size (E.S.) of coal (specific gravity 1.4) is usually 1mm (0.85-1.6 mm range) with uniformity coefficient (U.C.) of 1.3 to 1.5. Depths of 0.3 to 0.4 m have been reported to be satisfactory without excessive head loss build up and these depths can flocculate particles besides removing large flocculated impurities. The finer media-layer usually consists of 0.3 – 0.4 m thick silica sand (specific gravity 2.65) with effective size of around 0.5 mm (0.45 to 0.6 mm range) and uniformity coefficient of 1.3 to 1.5.

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

In addition to high grade bituminous coal, crushed coconut shell has been effectively used as coarse media in dual media filters. The size ranges from 1.0 to 2.0 mm with depths of 03 - 0.4 m. The uniformity coefficient is below 1.5 and specific gravity 1.4. The sand used in conjunction with crushed coconut shell has effective size varying between 0.44 to 0.55 mm with uniformity coefficient below 1.5. The sand depths may very between 0.30 and 0.4 m. Water treatment plants with capacities ranging between 1 to 26 mld have been constructed employing dual media filters using crushed coconut shell and sand.

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

7.6.4.3 Design Of Media Depth And Media Sizes

a) Design of Media Depth

The efficiency of removal of suspended particles is a function of the surface area of the media grains. For a filter of depth l comprising of N particles of average size d and sphericity ψ

$$A = \frac{1(1-f)}{\frac{\pi(\psi d)^3}{6}} \cdot \pi(\psi d)^2 \tag{7.53}$$

$$A = \frac{6(1-f)}{\psi}\left(\frac{1}{d}\right) \tag{7.54}$$

The equation can be employed to design the depths of filter. For example, for typical high rate filters $f=0.45, \psi = 0.8$ and $\left(\frac{1}{d}\right) = 680$, $A \cong 2800\ m^2$. Since the effective size of sand is normally specified, $\left(\frac{1}{d}\right) = 680$ corresponds to $\left(\frac{1}{d_{10\%}}\right) = 950$ where $d_{10}\%$ is the effective size. Figures can be developed for predetermined value of A, based on pilot data, between the effective size of medium and filter medium depth for different values of V. These figures can be used to estimate depths of various combinations of dual media.

(b) Design of Media Sizes

The dual media filters, consisting of coarse but lighter medium particles on top of finer but heavier particles, must retain their stratified character during backwashing and resettling. Equal expansion during backwashing for dual media comprising of coal and sand indicate equal fluidization velocity for both media. It can be shown that

$$\frac{d_u}{d_l} = \left(\frac{\psi_l}{\psi_u}\right)\left[\frac{\rho_l - \rho}{\rho u - \rho}\right]^{\frac{1}{2}} \tag{7.55}$$

Where subscripts u and l respectively denote the largest grain within the upper layer (coal) and the smallest grains within the lower layer (sand). It follows that mixing during settling as well as during expansion determines the maximum allowable ratio of the grain sizes in the two layers.

For sharp interface and no intermixing, the ratio of maximum diameter of coal to the minimum diameter of sand that will ensure both equal expansion and equal settling can be computed using above mentioned equation. For a density of coal of 1.5 and its sphericity 0.70 and sphericity and density of sand of 0.85 and 2.65, this ratio is

$$\frac{d_u}{d_l} = \frac{0.85}{0.70} = \left[\frac{(2.65-1)}{(1.5-1)}\right]^{0.5} = 2.26 \cong 2.3$$

If partial intermixing is to be achieved the size of the coarsest coal must be more than 2.3 times the minimum diameter of sand for characteristic of coal and sand given.

7.6.4.4 Filtration Rates And Filtrate Quality

Dual -media and multimedia filters have been successfully operated at rates of filtration ranging from 15 to 20 $m^3/m^2/hr$ with acceptable filtrate quality. Field trails in India using high grade bituminous coal indicate that even with inadequate pretreatment of filter influents as obtainable in Indian conditions, filtration rates of 16 $m^3/m^2/hr$ could be recommended with filter run of at least 12 hours, though much higher rates of filtration upto 24 $m^3/m^2/hr$ could be employed if proper pretreatment is available. Filtrate turbidities are generally less than 1 NTU and coliform removal is around 95.

In general, it may be recommended to operate dual media filters at higher rates of 7.5 to 12 $m^3/m^2/hr$. The backwash rates of 42 to 54 $m^3/m^2/hr$ (700-900 lpm/m^2) have been employed to clean the filters.

7.6.5 MULTIMEDIA FILTERS

The multimedia filters normally contain three media such as anthracite coal, silica sand and garnet sand with specific gravities being around 1.4, 2.65 and 4.2. The size of media may vary from 2 mm at the top to 0. 15 mm at the bottom. A typical trimedium 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.

7.6.6 PRESSURE FILTERS

7.6.6.1 General

Based on the same principle as gravity type rapid sand filters, water is passed through the filter under pressure 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.

7.6.6.2 Disadvantages

Pressure filters suffer from the following disadvantages:

(a) The treatment of water under pressure seriously complicates effective feeding, mixing and flocculation of water to be filtered.

(b) In case of direct supply from pressure filters, it is not possible to provide adequate contact time for chlorine.

(c) The water under filtration and the sand bed are out of sight and it is not possible to observe the effectiveness of the back wash or the degree of agitation during washing process.

(d) Because of the inherent shape of the pressure filters it is difficult to provide wash water gutters effectively designed so that the material washed from the sand is discharged to waste and not flushed back to other portions of the sand bed.

(d) It is difficult to inspect, clean and replace the sand, gravel and underdrains of pressure

filters.

(e) 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.

7.6.7 DIATOMACEOUS EARTH FILTERS

Diatomaceous fiters, are not advocated for publiuc water supplies. Their utility is restricted to temporary and emergency water supplies of a limited nature where other arrangements are not easyor feasible.

The medium consists of diatomaceous earths which are skeletons of diatoms mined from deposits laid down in seas.

The filtering medium is a layer of diatomaceous earth built up on a porous septum by recirculating a slurry of diatomaceous earth until a firm layer is formed on the septum. The precoat thus formed is used for straining the turbidity in water. For this, diatomaceous earth is applied at 0.5 to 2.5 kg/m^2 of septum. Some times, when the turbidity is very high, the diatomaceous earth will have to be added to the incoming water as body feed. Body feed is added at three times the solids when organic slimes are present. Filtration rates range from 7.2 to 18 $m^3/m^2/hr$.

7.6.8 Additional Modifications of Conventional Rapid Gravity Filters

7.6.8.1 Constant And Declining Rate Filtration

(a) Constant Rate Filtration by Influent Flow Splitting

In conventional rapid sand filters, constant rate of flow is maintained by installing a rate of flow controller on the effluent line. These rate of flow controller can be quite complex and high in initial and maintenance Costs. Alternative systems have been proposed which are relatively simple to build, operate and maintain.

One of the simplest methods is rate control by influent flow splitting which is depicted in Fig. 7.19. The filter influent is divided equally among all the operating filters in series by means of a weir at each filter inlet. The size of the filter influent conduit 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. 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

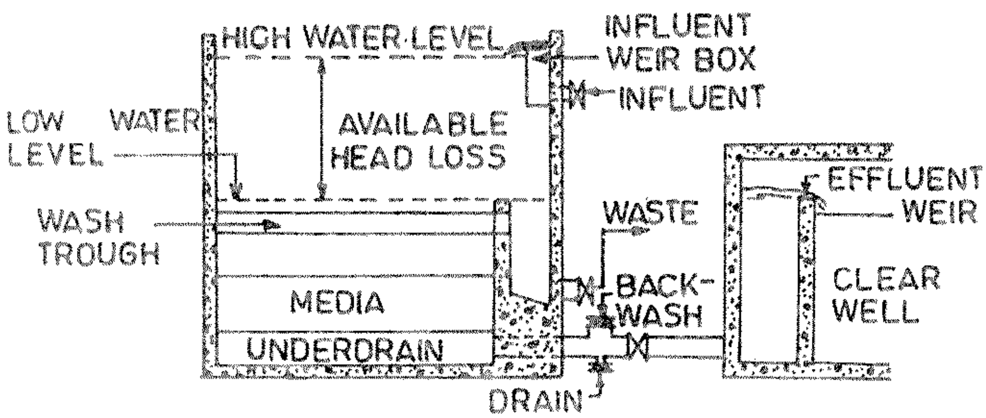

FIGURE 7.19 : GRAVITY FILTER ARRANGEMENTS FOR RATE CONTROL BY INFLUENT FLOW SPLITTING

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, the effluent control weir must be located above filter media as depicted in the Figure.

The only disadvantage of the influent flow splitting system is the additional depth of the filter box which is 1.5 to 2 m more than in conventional filters.

(b) Declining Rate Filtralion

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 section 7.6.8.1 (a). 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 Fig. 7.20 No rate of flow controllers are used in this system also.

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

The advantage claimed for 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.

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

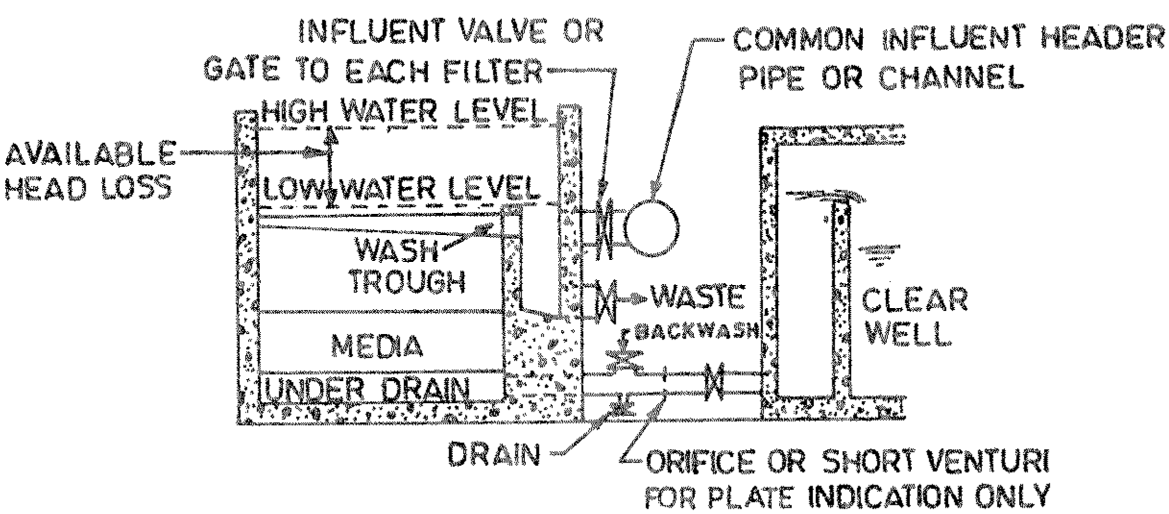

FIGURE 7.20 : GRAVITY FILTER ARRANGED FOR VARIABLE DECLINING RATE OF FILTERATION

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 $m^3/m^2/hr$, fixed orifice will be so designed that a recently cleaned filter will begin operation at 9 $m^3/m^2/hr$ while the filter next in line for backwashing will have slowed down to about 3 $m^3/m^2/hr$. 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.

7.6.9 UP-FLOW FILTERS

In up-flow filtration, the water is passed under pressure in an upward direction through the coarser medium followed by finer medium. Thus larger size suspended solid particles are first retained in the larger interstices of the lower part of bed and as the water percolates upwards, it receives a progressive polishing until it emerges in a fully filtered condition at top of the filter bed. Thus the entire depth of media is made effective in removal of suspended solids and as a result low head loss and longer filter runs could be expected. Besides, many other advantages are claimed for up-flow filtration such as elimination of the rate controller and absence of negative head. Unfiltratered water can be used for washing filter since the first few minutes of flow through the filter after washing has to be necessarily run to waste. Filter depths as low as 0.6 m and as high as 1.5 m have been successfully used. Although wash water rate and consumption are greater per wash cycle than the conventional filter, wash water used as a percentage of finished water is much less because of low loss of head and long filter runs. But initially compressed air scouring is desirable to dislodge the impurities collected in the lower portions of the bed. The only disadvantage is fluidization of the top fine layers of the sand bed which results in the deterioration of the filtrate quality. Complete bed fluidization occurs when the headloss equals the depth of bed. Control of headloss 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.

7.6.10 GRID OR IMMEDIUM TYPE FILTERS

The problem of bed fluidization in an upflow filter is eliminated in this type by providing a 'grid'. The grid is a system of parallel vertical plates placed within the bed a few centimeters below the top of the medium. This grid provides sufficient resistance to prevent expansion of the bed and breakthrough or channelling at relatively higher rates of filtration. The exact mechanism of how this plate grid restrains sand expansion has not been proved but it is believed that the upward flow of water causes formation of inverted arches of sand which bridges the gap between the adjacent vertical plates. Being in compression, these arches are strong enough to resist the upward force of liquid being filtered ending to break through the bed, thus minimising fluidization of sand. Generally the grid spacing is 100-150 times the size of fine sand at the top of the bed. They operate at the rate of 9-12 $m^3/m^2/$ hr and deliver four times the filtered water per run of down flow filter of the same size. Recent researches indicate that higher rates of 15-30 $m^3/m^2/hr$ could be employed. The operating rate, however, depends on quality of water to be filtered and the effluent quality desired.

7.6.11 Bi-Flow Filters

The possibility of fluidization of the finest sand layers in up-flow filtration is solved in this type by placing the effluent collecting pipe in the upper layers of sand bed and filtering simultaneously from below through the bulk of the media and from above through top layers of sand. With the same hydraulic head applied on top and bottom of the filter, the headloss across both the upper and lower beds to the centres of effluent pipe is the same and hence the effluent pipe is hydraulically balanced. The downward hydraulic gradient on the top portion prevents fluidization. The inflow filter's use sand of depth 1. 5 to 1.65 m, the filtrate collection system being placed 0.45 to 0.60 m below the top sand. Total filtration rate is 18 $m^3/m^2/$ hr , downward rate being half to two-thirds of the upward rate. Backwash rate adopted is 54 to 66 $m^3/m^2/hr$ for a period of 6 minutes. Initial cost is estimated to be 15-30% less than conventional filters even though piping, valves and controls are more complicated.

7.6.12 Submerged Filters

These filters have no rate controllers. This method employs direct pumping to permit automatic adjustment of treatment plant input and high level pumping to meet the varying and continous demands of a large city. These filters operate under varying rates as demand varies and through the use of a slow moving butterfly valve, the filtration rate can be changed without deterioration of water quality. The butterfly valve closes slowly over a period of time usually 5 to 10 minutes. These filters have proved satisfactory on a plant scale.

7. 6.13 RADIAL FLOW FILTERS

In these filters, flow comes in radially and washing is done continuously. The filter medium is sand, contained in the annular space ofprise of a closed cylindrical shell. Chemically treated water enters into a central hollow column and permeates radially through sand and is collected through peripheral ducts and flows out of the shell. The dirty sand is continously drawn from the bottom and airlifted to a compartment at the top of the filter where it is washed, the sand sinking to the cylinder while the washed water carrying dirt escapes out through an overflow device.

7.6.14 Automatic Valveless Gravity Filters

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 meters differential, syphon action is started and backwashing begins at the required rate of 30 to 42 $m^3/m^2/$ hr . Wash water flows from the storage tank up through the sand bed and is discharged through the back-