### 11.1.2 Medium Screens

Medium bar screens have clear openings of 20 to 50mm. Bars are usually 10mm thick on the upstream side and taper slightly to the downstream side. These mechanically raked units are used before all pumps or treatment units such as the stabilization ponds. The bars used for the screens are rectangular in cross-section usually about 10mm x 50mm and are placed with the larger dimension parallel to the flow. A weir on the side of the screen may be used as an overflow bypass.

### 11.1.3 Fine Screens

Fine screens are mechanically cleaned devices using perforated plates, woven wire cloth or very closely spaced bars with clear openings of less than 20mm. Fine screens are used for pretreatment of industrial wastes to remove materials which tend to produce excessive scum or foam on the top of digestion tank contents. Fine screens are not normally suitable for sewage because of the clogging possibilities.

Fine screens may be of the drum or disc type, mechanically cleaned and continuously operated. Fine screens have generally a net submerged open area of not less than 0.05 $m^2$ for every 1000$m^3$ of average daily flow of sewage from a separate system, the corresponding figure being 0.075 $m^2$ for combined systems. They are also used for beach protection where sewage without any further treatment is discharged into sea for disposal by dilution.

### 11.1.4 Comminuting Devices

A comminuting device is a mechanically cleaned screen which incorporates a cutting mechanism that cuts the retained material enabling it to pass along with the sewage. The solids from the comminutor may, however, lead to the production of more scum in the digester.

### 11.1.5 Location of Screens

Screening devices are usually located where they are readily accessible because the nature of materials handled requires frequent inspection and maintenance of the installation. Where screens are placed in deep pits or channels, it is necessary to provide sufficiently wide approaches from the top and ample working space for easy access and maintenance. Provision should be made for the location of penstocks and bypass arrangements for the screens.

### 11.1.6 Housing of Screens

The need for a structure to house the screening equipment depends on two factors viz., the design of the equipment and the climatic conditions. If climatic conditions are not severe and could be withstood by the equipment, the screen house can be omitted. Mechanically cleaned screens generally need suitable housing to protect the equipment, prevent accidents to operating personnel and improve the appearance of the treatment facility. Ventilation of the housing is necessary to prevent accumulation of moisture and removal of corrosive atmosphere.

### 11.1.7 Hydraulics

A screen by its very nature and function collects material which will impede flow. If the screen is cleaned continuously by mechanical arrangement, this interference will be kept to a minimum. Screens with periodic cleaning arrangements are likely to produce considerable damming effect leading to surges of relatively high flow soon after cleaning. The usually accepted design is to place the base of the screen several centimetres below the invert of the approach channel and steepen the grade of the influent conduit immediately preceding the screen.

### 11.1.8 Velocity

The velocity of flow ahead of and through a screen varies materially and affects its operation. The lower the velocity through the screen, the greater is the amount of screenings that would be removed from sewage. However, the lower the velocity, the greater would be the amount of solids deposited in the channel. Hence, the design velocity should be such as to permit 100% removal of material of certain size without undue depositions. Velocities of 0.6 to 1.2 mps through the open area for the peak flows have been used satisfactorily. When considerable amounts of storm water are to be handled, approach velocities of about 0.8 mps are desirable, to avoid grit deposition at the bottom of the screen, which might otherwise become inoperative when most needed during storm though lower value of 0.6 mps is used in current practice. Further, the velocity at low flows in the approach channel should not be less than 0.3 mps to avoid deposition of solids. A straight channel ahead of the screen insures good velocity distribution across the screen and maximum effectiveness of the device.

### 11.1.9 Head Loss

Head loss varies with the quantity and nature of screenings allowed to accumulate between cleanings. The head loss created by a clean screen may be calculated by considering the flow and the effective areas of the screen openings, the latter being the sum of the vertical projections of the openings. The head loss through clean flat bar screens is calculated from the following formula:

$$h = 0.0729\,(V^2 - v^2) \qquad (11.1)$$

in which

$h$ = head loss in m

$V$ = velocity through the screen in mps

$v$ = velocity before the screen in mps

Usually accepted practice is to provide loss of head of 0.15m but the maximum loss with clogged hand cleaned screen should not exceed 0.3 m. For the mechanically cleaned screen, the head loss is specified by the manufacturers.

Another formula often used to determine the head loss through a bar rack is Kirschmer's equation:

$$h = \beta\,(W / b)^{4/3}\; h_v \sin\Theta \qquad (11.2)$$

where $h$ = head loss, in m

$\beta$ = bar shape factor which is assigned value of 2.42 for sharp edged rectangular bar, 1.83 for rectangular bar with semicircle upstream, 1.79 for circular and 1.67 for rectangular bar with both u/s and d/s faces as semicircular.

$W$ = maximum width of bar u/s of flow, m

$b$ = minimum clear spacing between bars, m

$h_v$ = velocity head of flow approaching rack, m

= $(v^2 / 2g)$

$\Theta$ = angle of inclination of rack with the horizontal

The head loss through fine screens is given by the formula

$$h = (1 / 2g)\ (Q / CA)^2 \qquad (11.3)$$

in which

| | | |
|---|---|---|
| h | = | head loss, m |
| Q | = | discharge, $m^3/s$ |
| C | = | Coefficient of discharge (typical value 0.6) |
| A | = | effective submerged open area, $m^2$ |

### 11.1.10 Quantity of Screenings

The quantity of screenings varies with the size of screen used and on the nature of sewage. Generally it has been found that the screenings from sanitary sewage vary from $0.0015 m^3/ml$ with screen sizes of 10cm to $0.015\ m^3/ml$ in case of 2.5cm size.

### 11.1.11 Disposal of Screenings

The methods of disposal of screenings could be burial or composting. The screenings should not be left in the open or transported in uncovered conveyors as it would create nuisance due to flies and insects. If conveyors are used, they should be kept as short as possible for sanitary reasons. Burial in trenches usually 7.5cm to 10cm deep is practised particularly in small installations. At large works, where sufficient land for burial is not available within a reasonable distance from the plant, screenings are incinerated either by utilising the sludge gas obtained from the digestion tank or by using oil fuel. Where possible, the screenings are transported and mixed with town refuse for production of compost.

## 11.2 GRIT REMOVAL

Grit removal is necessary to protect the moving mechanical equipment and pump elements from abrasion and accompanying abnormal wear and tear. Removal of grit also reduces the frequency of cleaning of digesters and settling tanks. It is desirable to provide screens or comminuting device ahead of grit chambers to reduce the effect of rags and other large floating materials on the mechanical equipment, in case of mechanised grit chambers. But, where sewers are laid at such depths as to make the location of grit chambers ahead of pumping units undesirable or uneconomical, only a bar screen is provided ahead of pumps, with grit chambers and other units following the pumps.

### 11.2.1 Composition of Grit

Grit in sewage consists of coarse particles of sand, ash and clinkers, egg shells, bone chips and many inert materials inorganic in nature. Both quality and quantity of grit varies depending upon (a) types of street surfaces encountered (b) relative areas served (c) climatic conditions (d) types of inlets and catch basins (e) amount of storm water diverted from combined sewers at overflow points (f) sewer grades (g) construction and condition of sewer system (h) ground and ground water characteristics (j) industrial wastes (k) relative use of dumping chutes or pail depots where night soil and other solid wastes are admitted to sewers and (1) social habits. The specific gravity of the grit is usually in the range of 2.4 to 2.65. Grit is nonputrescrible and possesses a higher hydraulic subsidence value than organic solids. Hence it is possible to separate the gritty material from organic solids by differential sedimentation in a grit chamber.

### 11.2.2 Types

Grit chambers are of two types mechanically cleaned and manually cleaned. The choice depends on several factors such as the quantity and quality of grit to be handled, headloss requirements, space requirements, topography and economic considerations with respect to both capital and operating costs. In very small plants mechanisation may be uneconomical. But for all sewage treatment plants receiving flows over 10 mld mechanised grit removal units are preferred.

#### 11.2.2.1 MECHANICALLY CLEANED GRIT CHAMBERS

These grit chambers are provided with mechanical equipment for collection, elevation and washing of grit which are operated either on a continuous or intermittent basis. Scraper blades or ploughs rotated by a motor drive, collect the grit settled on the floor of the grit chamber. The grit so collected is elevated to the ground level by several mechanisms such as bucket elevators, jet pump, screws and air lift. The grit washing mechanisms are also of several designs most of which are basically agitation devices using either water or air to produce washing action. In intermittently (normally once or twice a day) operated type, sufficient storage capacity to hold the grit between intervals of grit elevation should be provided.

#### 11.2.2.2 MANUALLY CLEANED GRIT CHAMBERS

These should provide for adequate capacity for storage of grit between intervals of cleaning. These tanks should be cleaned at least once a week. The simplest method of removal is by means of shovel and wheel-barrows.

### 11.2.3 Aerated Grit Chambers

An aerated grit chamber is a special form of grit chamber consisting of a standard spiral flow aeration tank provided with air-diffusion tubes placed on one side of the tank, 0.6 to 1 m from the bottom. The grit particles tend to settle down to the bottom of the tank at rates dependent upon the particle size and the bottom velocity of roll of the spiral flow, which in turn is controlled by the rate of air diffusion through the diffuser tubes and the shape of the tank. The heavier grit particles with their higher settling velocities drop down to the floor whereas the lighter organic particles are carried with roll of the spiral motion and eventually out of the tank. The velocity of roll, however, should not exceed the critical velocity of scour of grit particles. Normally a transverse velocity of flow, not exceeding 0.4 to 0.6 mps at the top of the tank should satisfy this requirement for differential scour. No separate grit washing mechanism or control device for horizontal velocity is necessary in aerated grit chambers.

### 11.2.4 Design Data

The basic data essential for a rational approach to the design of grit chambers are hourly variations of sewage flow and typical values for minimum, average and peak flows. Since the grit chamber is designed for peak flows and the flow through velocity is maintained constant within the range of flow, successful design and operation of grit chamber calls for a fairly accurate estimation of the flows.

The quantity and quality of grit varies from sewage to sewage. Data relating to these two factors is very useful in proper design of grit collecting, elevating and washing mechanisms. In the absence of specific data, grit content may be taken as 0.05 to 0.15 $m^3$/mL for domestic sewage and 0.06 to 0.12 $m^3$/mL for combined sewage. The quantity may increase three to four fold during peak flow hours which may last for 1 to 2 hours.

### 11.2.5 Design of Grit Chambers

11.2.5.1 SETTLING VELOCITY

Grit chamber may be designed on a rational basis by considering it as a sedimentation basin. The grit particles are treated as discrete particles settling with their own settling velocities. The settling velocity is governed by the size and specific gravity of the grit particles to be separated and the viscosity of the sewage. The minimum size of the grit to be removed is 0.20mm although 0.15mm is preferred for conditions where considerable amount of ash is likely to be carried in the sewage. The specific gravity of the grit may be as low as 2.4 but for design purposes a value of 2.65 is used. The settling velocity of discrete particles can be determined using the appropriate equation depending upon the Reynolds number.

**a)** ***Stoke's Law***

$$v_s = \frac{g}{18}\frac{(\rho_s - \rho)}{\rho}\frac{d^2}{\nu} \qquad (11.4)$$

or

$$= \frac{g}{18}(S_s - 1)\frac{d^2}{\nu}$$

*where* $v_s$ = *settling velocity, m/s*

$g$ = *acceleration due to gravity,* $m/s^2$

$\rho_s$ = *mass density of grit particle,* $kg/m^3$

$\rho$ = *mass density of liquid,* $kg/m^3$

$S_s$ = *specific gravity of grit particle, dimensionless*

$d$ = *size of the particle, m*

$\nu$ = *kinematic viscosity of sewage,* $m^2/s$

Stoke's law holds good for Reynolds number R, below 1.0.

$$(R = v_s d / \nu)$$

For grit particles of specific gravity of 2.65 and liquid temperature at 10 degree

$(\nu = 1.01 \times 10^{-6}\ m^2/s)$

this corresponds to particles of size less than 0.1 mm. The flow conditions are laminar where viscous forces dominate over inertial forces.

**b)** ***Transition Law***

The design of grit chamber is based on removal of grit particles with minimum size of 0.2mm or 0.15mm and therefore Stoke's Law is not applicable to determine the settling velocity of the grit particles for design purposes.

The settling velocity of a discrete particle is given by the general equation

$$v_s = \sqrt{\frac{4}{3}\frac{g}{C_D}\frac{(\rho_s - \rho)}{\rho} d} \qquad (11.5)$$

Where $C_D$ is the Newton coefficient of Drag which is a function of Reynolds number. The transition flow conditions hold when Reynolds number is between 1 and 1000. In this range, $C_D$ can be approximated by

$$C_D = \frac{18.5}{R^{0.6}} = \frac{18.5}{\left(\frac{v_s d}{\nu}\right)^{0.6}} \qquad (11.6)$$

Substituting the value of $C_D$ in equation (11.5) and simplifying

$$V_s = [0.707\,(S_s - 1)\,d^{1.6}\,\nu^{-0.6}]^{0.714} \qquad (11.7)$$

The settling velocity of grit particles in the transition zone is also calculated by the Hazen's modified formula

$$v_s = 60.6(S_s - 1)d\frac{3T + 70}{100} \qquad (11.8)$$

where d in equation (11.8) is in cm and T is the temperature in degree. C and

$v_s$ in cm/s.

The settling velocity of grit particles in the range of 0.1mm and 1mm can be determined using equation (11.7) and this equation or its approximate empirical form of Eq. (11.8) should be used in design of grit chambers which are designed to remove particles of size 0.15 mm or 0.2 mm.

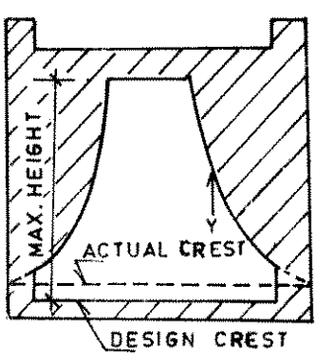


FIG.11.1(a): PROPORTIONAL FLOW WEIR

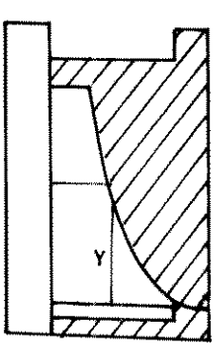


FIG. 11.1 (b): SUTRO WEIR

c) ***Newton's Law***

When the particle size increases beyond 1 mm and Reynolds number beyond 1000, the Newton coefficient drag $C_D$ assumes a constant value of 0.4 and the following equation can be used to determine the settling velocity of grit particles.

$$v_s = [\ 3.3\ g\ (Ss-1).d\ ]^{0.5} \qquad (11.9)$$

11.2.5.2 SURFACE OVER FLOW RATE

Efficiency of an ideal settling basin is expressed as the ratio of the settling velocity of the particles to be removed ($v_s$) to the surface overflow rate ($v_o$).

$$\eta = \frac{V_s}{V_o}$$

Where $v_o$ is defined as the ratio of flow of sewage to be treated in an ideal settling tank to the plan area of the tank i.e. Q/A. It is equal to the settling velocity of those particles which will be 100% removed in ideal settling tank.

In an ideal settling basin, all particles having settling velocity, $v_s \geq v_o$ are completely removed. However, particles having settling velocity, $v_s < v_o$ are removed in proportion to the ratio of $v_s$ to $v_o$.

Table 11.1 gives settling velocition of different size particles of specific gravity 2.65 (inorganic grit particles) and 1.20 (organic matter) and corresponding surface overflow rates for 100% removal of these particles based on Eq.(11.7).

**TABLE 11.1**
**SETTLING VELOCITIES AND SURFACE OVERFLOW RATES FOR IDEAL GRIT CHAMBER AT 10 DEGREE C.**

| Diameter of Particles, mm | Settling velocity m/s. | | Surface Overflow rate $m^3$ / d $m^2$ | |
|---|---|---|---|---|
| | $S_s$ = 2.65 | $S_s$ = 1.20 | $S_s$ = 2.65 | $S_s$ = 1.20 |
| 0.20 | 0.025 | 0.0054 | 2160 | 467 |
| 0.15 | 0.018 | 0.0039 | 1555 | 337 |

However, the behaviour of a real grit chamber departs significantly from that of the ideal settling basin due to turbulence and short-circuiting resulting from eddy, wind and density currents. Hence, the surface overflow rates (SOR) should be diminished to account for the basin performance. Following equation could be used to determine the SOR for a real basin for a given efficiency of grit removal and basin performance.

$$\eta = 1 - [1 + n.v_s/(Q/A)]^{-\frac{1}{n}} \qquad (11.11)$$

*where* $\eta$ = *desired efficiency of removal of grit particle*

$v_s$ = *settling velocity of minimum size of grit particle to be removed*

$Q/A$ = *design surface over flow rate applicable for grit chamber to be designed*

$n$ = *an index which is a measure of the basin performance*

The values of n are 1/8, 1/4, 1/2 and 1 for very good, good, poor and very poor performance. It can be seen that the design surface overflow rate will be 66.67%, 58.8%, 50% and 33.3% of the settling velocity of the grit particles to be removed to achieve 75% removal efficiency in grit chamber with very good, good and very poor tank performance respectively. In practice, values of two thirds to one half are used in design depending upon the type of the grit chamber. These values are much higher than those needed for organic solids of specific gravity 1.2.

### 11.2.5.3 DETENTION PERIOD

A detention period of 60 sec is usually adopted.

### 11.2.5.4 BOTTOM SCOUR AND FLOW THROUGH VELOCITY

Bottom scour is an important factor affecting grit chamber efficiency. The scouring process itself determines the optimum velocity of flow through the unit. This may be explained by the fact that there is a critical' velocity of flow '$V_c$' beyond which particles of a certain size and density once settled, may be again placed in motion and reintroduced into the stream of flow. The critical velocity for scour may be calculated from modified Schield's formula:

$$V_c = K_c\sqrt{g(S_s - 1)d} \qquad (11.12)$$

Where $K_c$ = 3 to 4.5. A value of 4.0 is usually adopted for grit particles.

For a grit particle size of 0.2mm, the formula gives critical velocity values of 17.1 to 25.6 cm/sec. In actual practice, a horizontal velocity of flow of 15 to 30 cm/sec. is used at peak flows. The horizontal velocity of flow should be maintained constant at other flow rates also to ensure that only organic solids and not the grit are scoured from the bottom. Bottom scour is an important factor particularly affecting the grit chamber efficiency.

### 11.2.5.5 VELOCITY CONTROL DEVICES

Numerous devices have been designed in an attempt to maintain a constant horizontal velocity of flow through grit chambers in the recommended range of 15 to 30 cm/sec. Since none of the control devices designed so far have been able to maintain the velocity at a constant level at all flows, a limit of variance in the velocity of 5-10% above and below the desired velocity of flow is recommended. Multiple channels with the total capacity to carry the maximum flow and velocity control either manually or by means of side-flow weirs in a distribution box or a diversion channel is sometimes adopted but is not economical. A satisfactory method of controlling velocity of flow through the grit channels is by

using a control section which placed at the end of the channel, varies the cross sectional areas of flow in the section in direct proportion to the flow.

As for example, for a flow of 5 cumecs, the cross-sectional area of flow should be 5 $m^2$ and when flow decreases to 3 cumecs the cross-sectional area of flow should be reduced to 3 $m^2$ to maintain the velocity of flow constant at 1 mps. Such control sections include Proportional flow weirs, Sutro weirs, Parshall flumes, Palmer flumes etc., of which the former three are commonly used.

**a)** ***Proportional Flow Weir***

The proportional flow weir is a combination of a weir and an orifice. It maintains a nearly constant velocity in the grit channels by varying the cross-sectional area of flow through the weir so that the depth is proportional to flow (Fig.11.1(a)).

The general equation for determining the flow through weir, Q, is

$$Q = cb\sqrt{2ag}\left(H - \frac{a}{3}\right) \qquad (11.13)$$

where c is a coefficient which is assumed 0.61 for symmetrical sharp-edge weirs.

a = dimension of weir usually assumed between 25mm and 50mm

b = base width of the weir

H = depth of flow

To determine the shape of the curve forming the outer edges of the cut portion, the following equation of curve forming the edge of the weir may be used.

$$x = \frac{b}{2}\left(1 - \frac{2}{\pi}\tan^{-1}\sqrt{\frac{Y}{a} - 1}\right) \qquad (11.14)$$

The weir shall be set 100mm to 300mm above the bottom of grit chamber to provide grit storage or for operation of mechanical grit chamber. The weir should also be set at such an elevation as to provide a free fall into the outlet channel as it cannot function under submerged conditions. Each grit chamber should be provided with a separate control weir.

**b)** ***Sutro Weir***

The Sutro-weir is a half proportional flow weir cut symmetrically and centrally along the vertical axis as illustrated in Fig.11.1(b). The orifice has a straight horizontal bottom forming the weir.

**c)** ***Parshall Flume***

A Parshall flume is an open constricted channel which can be used both as a measuring device and also as a velocity control device, more commonly used for the latter

purpose in grit chambers. The flume has a distinct advantage over the proportional flow weir, as it involves negligible head loss and can work under submerged conditions upto certain limits. The limits of submergence are 50% in case of 150mm throat width and 70% for wider throat widths upto 1 m. Another advantage is that one control section can be installed for 2 to 3 grit

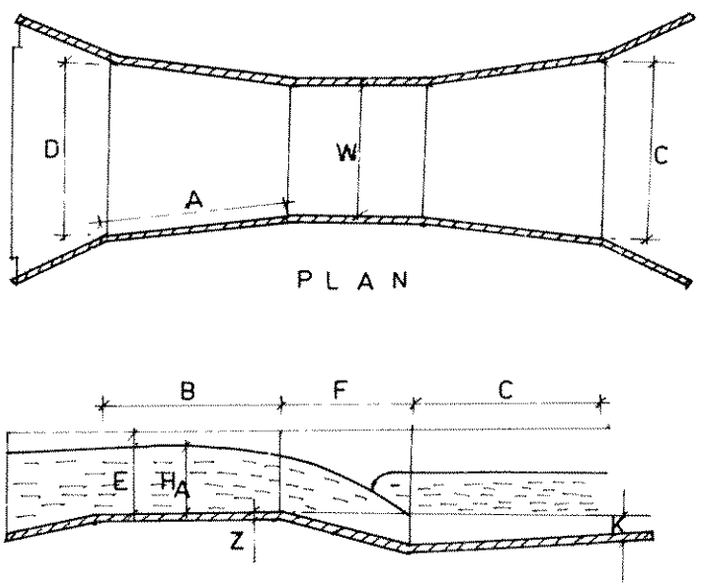


FIG. 11.2: DIMENSIONS FOR PARSHALL FLUME

TABLE 11.2

DIMENSIONS OF PARSHALL FLUME (mm)

| +FLOW RANGE Qmax (mLd) | W | A | ++ B | C | +++ D | F | G | K | Z |
|---|---|---|---|---|---|---|---|---|---|
| UP TO 5 | 75 | 460 | 450 | 175 | 255 | 150 | 300 | 25 | 56 |
| 5 - 30 | 150 | 610 | 600 | 315 | 391 | 300 | 600 | 75 | 113 |
| 30 - 45 | 225 | 865 | 850 | 375 | 566 | 300 | 750 | 75 | 113 |
| 45 - 170 | 300 | 1350 | 1322 | 600 | 831 | 600 | 900 | 75 | 225 |
| 170 - 250 | 450 | 1425 | 1397 | 750 | 1010 | 600 | 900 | 75 | 225 |
| 250 - 350 | 600 | 1500 | 1472 | 900 | 1188 | 600 | 900 | 75 | 225 |
| 350 - 500 | 900 | 1650 | 1619 | 1200 | 1547 | 600 | 900 | 75 | 225 |
| 500 - 700 | 1200 | 1800 | 1766 | 1500 | 1906 | 600 | 900 | 75 | 225 |
| 700 - 850 | 1500 | 2100 | 2060 | 2100 | 2625 | 600 | 900 | 75 | 225 |
| 850 - 1400 | 2400 | 2400 | 2353 | 2700 | 3344 | 600 | 900 | 75 | 225 |

+ FOR AVERAGE FLOW AND PEAK FACTORS, SEE 3.2.5

++ VALUE SHOULD BE EQUAL TO $1.5 \times (Q\ max)^{1/3}$ BUT NOT LESS THAN THOSE SHOWN IN THE TABLE

+++ FOR HIGHER VALUES OF B (THAN SHOWN IN THE TABLE), THE VALUES OF D ALSO TO BE INCREASED TO KEEP D/B RATIO SAME AS IN TABLE.

chambers. The flume is also self cleansing and there is no problem of clogging. As the parshall flume is a rectangular control section, the grit chamber above it must be designed to approach a parabolic cross section. However, a rectangular section with a trapezoidal bottom may be used with a parshall flume in which case the variations in velocity at maximum and minimum flow conditions from the designed velocity of flow should be within permissible limits as given by the following equations.

$$Q = 2264W(H_A)^{\frac{3}{2}} \tag{11.15}$$

$$D + Z = 1.1H_A \tag{11.16}$$

and

$$\frac{D\min}{D\max} = \frac{1.1\left(\frac{Q\min}{2264W}\right)^{2/3} - Z}{1.1\left(\frac{Q\max}{2264W}\right)^{2/3} - Z} \tag{11.17}$$

$$D = 1.1\left(\frac{Q}{2264W}\right)^{2/3} - Z \tag{11.18}$$

$$b = \frac{Q\max}{1000D\max.V\max} \tag{11.19}$$

or

$$b = \frac{Q\min}{1000D\min.V\min} \tag{11.20}$$

$$V = \frac{Q}{1000b.D} \tag{11.21}$$

where

| | | |
|---|---|---|
| Q | = | rate of flow in lps |
| Q min | = | minimum rate of flow in lps |

| | | |
|---|---|---|
| Q max | = | maximum rate of flow in lps |
| W | = | throat width in m |
| $H_A$ | = | depth of flow in upstream leg of the flume at one-third point in m |
| Z | = | a constant in m |
| D | = | depth of flow in the grit chamber in m |
| b | = | width of grit chamber in m and |
| V | = | velocity of flow in mps at a particular depth of flow |

Recommended throat widths for different ranges of flow along with the dimensions of the various elements of the flume (Fig.11.2) for the different throat widths are given in Table 11.2 which should be strictly adhered to.

A typical example is shown in Appendix 11.2.

11.2.5.6 NUMBER OF UNITS

In case of manually cleaned grit chambers at least two units should be provided. All mechanically cleaned units should be provided with a manually cleaned unit to act as a bypass.

11.2.5.7 DIMENSIONS OF EACH UNIT

The surface areas required for each unit is worked out on the basis of the overflow rate chosen as discussed in 11.2.5.2. The breadth of tank is fixed with reference to the control device adopted. The length is then worked out on the basis of the selected overflow rate. In case of mechanised grit chambers, the horizontal dimension may be readjusted to suit the standard sizes of the mechanical equipment after ensuring that the flow-through velocity is within the prescribed limits. The depth of flow is determined by the horizontal velocity and the peak flow. Additional depth for storage of grit between intervals of cleaning should be provided in case of intermittent cleaning. A free board of 150 to 300 mm should be provided. Bottom slopes are based on the type of scraper mechanism used.

### 11.2.6 Loss of Head

Loss of head in a grit chamber varies from 0.06m to 0.6 m depending on the device adopted for velocity control.

### 11.2.7 Disposal of Grit

Clean grit is characterised by the lack of odours. Washed grit may resemble particles of sand and gravel, interspersed with particles of egg shell, and other similar relatively inert materials from the households. Grit washing mechanism has to be included whenever the detention time is more and flow through velocity is less. Unless washed, it may contain considerable amount of organic matter. This becomes an attraction to rodents and insects and is also unsightly and odorous. The grit may be disposed of by dumping or burying or by sanitary land fill. The ultimate method used however depends upon the quantity and characteristics of the grit, availability of land for dumping, filling, or burial. In general, unless grit is washed, provision for burial should be made.

CHAPTER 12

# SEDIMENTATION

## 12.1 GENERAL

The purpose of sedimentation of sewage is to separate the settleable solids so that the settled wastewater, if discharged into water courses, does not form sludge banks and when used for land disposal does not lead to clogging of soil pores and excessive organic loading. Primary sedimentation of sewage also reduces the organics load on secondary treatment units. Sedimentation is used in wastewater treatment to remove (i) inorganic suspended solids or grit in grit chamber described in chapter 11 (ii) Organic and residual inorganic solids, free oil and grease and other floating material in plain sedimentation or primary sedimentation tanks which are located after screens and grit chambers (iii) Bio-flocculated solids or bioflocs from effluents of secondary biological treatment units such as trickling filter and activated sludge in secondary settling tanks or secondary clarifiers and (iv) chemical flocs produced during chemical coagulation and flocculation in secondary settling tanks. In addition to primary and secondary settling tanks, intermediate settling tanks are also used to remove the bioflocculated solids in multistage biological treatment units.

Septic tanks, Imhoff tanks and claridigesters are combination units where sedimentation and digestion of settleable solids are combined in single units. Settling also occurs in waste stabilisation ponds and facultative aerated lagoons alongwith organic matter stabilization.

## 12.2 CHARACTERISTICS OF SETTLEABLE SOLIDS

The settleable solids to be removed from wastewater in primary or secondary settling tanks after grit removal are mainly organic and flocculent in nature, either dispersed or flocculated. The specific gravity of organic suspended solids, may vary from 1.01 to 1.20. The bulk of the finely divided organic solids reaching primary sedimentation tanks are low specific gravity solids which are incompletely flocculated but are susceptible to flocculation. Flocculation occurs within primary settling tanks due to eddying motion of the fluid and aggregation of dispersed flocculent solids becomes more complete as the wastewater is detained for longer periods (hydraulic residence time) in these tanks. Since the particles are subject to flocculation, such settling tanks cannot be designed on the basis of surface overflow rate alone but will have to take into consideration hydraulic residence time or detention period also. However, rate of flocculation rapidly decreases as detention period is increased beyond certain values.

## 12.3 TYPES OF SETTLING

Basically, four categories of settling occur depending on the tendency of particles to interact and the concentration of solids. These settling types are (i) Discrete settling (ii) Flocculent settling (iii) Hindered or zone settling and (iv) Compression.

### 12.3.1 Discrete Settling

Discrete particles do not change their size, shape or mass during settling. Grit in wastewater behave like discrete particles. The settling velocity of discrete particles is determinable using Stokes or Transition law. Organic solids in raw wastewater and bioflocs in biologically treated wastewaters cannot be considered as discrete particles and hence Stoke's law is not applicable for these particles.

### 12.3.2 Flocculent Settling

Flocculent particles coalesce during settling increasing the mass of particles which settle faster. Flocculent settling refers to settling of flocculent particles of low concentration usually less than 1000 mg/l. The degree of flocculation depends on the contact opportunities which in turn are affected by the surface overflow rate, the depth of the basin, the concentration of the particles, the range of particle sizes and the velocity gradients in the system. No adequate mathematical equation exists to describe flocculent settling and therefore overflow rates to achieve a given removal efficiency are determined using data obtained from settling column studies.

The removal of raw sewage organic suspended solids in primary settling tanks, settling of chemical flocs in settling tanks and of bioflocs in the upper portion of secondary sedimentation tanks are examples of flocculent settling.

### 12.3.3 Hindered or Zone Settling

When concentration of flocculent particles is in intermediate range, they are close enough together so that their velocity fields overlap causing hindered settling. The settling of particles results in significant upward displacement of water. The particles maintain their relative positions with respect to each other and the whole mass of particles settles as a unit or zone. This type of settling is applicable to concentrated suspensions such as are found in secondary settling basins following activated sludge units. In the hindered settling zone, the concentration of particles increases from top to bottom leading to thickening of sludge. Such secondary clarifiers where zone settling occurs are designed on the basis of solid flux or solids loading and checked for surface overflow rate, both of which can be determined by conducting settling column analysis.

### 12.3.4 Compression

In compression zone, the concentration of particles becomes so high that particles are in physical contact with each other, the lower layers supporting the weight of upper layers. Consequently any further settling' results due to compression of the whole structure of particles and accompanied by squeezing out of water from the pores between the solid particles. This settling phenomenon occurs at the bottom of deep sludge mass, such as in the bottom of secondary settling tanks following secondary biological treatment by trickling filters and activated sludge process and in tanks used for thickening of sludge.

## 12.4 DESIGN CONSIDERATIONS

### 12.4.1 Factors Influencing Design

Several factors such as flow variations, density currents, solids concentration, solids loading, area, detention time and overflow rate influence the design and performance of sedimentation tanks. In the design of some plants, only a few of these factors may have significant effect on performance while in others, all of them may play an important role. Sedimentation tanks are designed for average flow conditions. Hence during peak flow periods, the detention period gets reduced with increase in the overflow rate and consequent overloading for a short period. If hourly flow variations are wide as in the case of some industrial waste flows, it may be necessary to build an equalization tank, ahead of the treatment units so that uniform loading is made possible in all treatment units.

### 12.4.2 Design Criteria

For primary sedimentation tanks, both, surface overflow rate and detention period (Hydraulic Residence Time) are important design criteria as the solids to be settled are flocculent in nature and undergo flocculation. The major design parameters for secondary settling tanks designed to remove bioflocculated solids are solids loading rate or solid flux as well as surface over flow rate. The plan surface area of secondary settling tanks is determined using both criteria and the greater of the two is

adopted for design. In addition, other design parameters are depth, displacement velocity (horizontal velocity of flow) and weir loading rate. The design of sludge removal arrangements and inlet and outlet should be done in accordance with the criteria prescribed in subsequent sections for satisfactory performance of any settling tank. Typical values for design parameters of settling tanks are given in Table 12.1.

### 12.4.2.1 OVERFLOW RATE OR SURFACE LOADING RATE

The overflow rate represents the hydraulic loading per unit surface area of tank in unit time expressed as $m^3/d/m^2$. Overflow rates must be checked both at average plant flows and peak flow. The overflow rates to be adopted for the different settling tanks are given in Table 12.1. The smaller values in the ranges given are applicable to small plants of capacities less than 5 MLD.

**TABLE 12.1**
**DESIGN PARAMETERS FOR SETTLING TANKS**

| Type of Settling | Overflow rate, $m^3/m^2.d$ | | Solid loading, $kg/m^2.d$ | | Depth | Detention Time |
|---|---|---|---|---|---|---|
| | Average | Peak | Average | Peak | m | hr |
| A. PRIMARY SETTLING | | | | | | |
| 1) Primary Settling only | 25 - 30 | 50 - 60 | | | 2.5 - 3.5 | 2.0 - 2.5 |
| 2)Primary settling followed by secondary treatment | 35 - 50 | 80 - 120 | | | 2.5 - 3.5 | " |
| 3) Primary settling with activated sludge return | 25 - 35 | 50 - 60 | | | 3.5 - 4.5 | " |
| B SECONDARY SETTLING | | | | | | |
| 4) Secondary settling for trickling filter | 15 - 25 | 40 - 50 | 70 - 120 | 190 | 2.5 - 3.5 | 1.5 - 2.0 |
| 5) Secondary settling for activated sludge (excluding extended aeration) | 15 - 35 | 40 - 50 | 70 - 140 | 210 | 3.5 - 4.5 | " |
| 6) Secondary settling for extended aeration | 8 - 15 | 25 - 35 | 25 - 120 | 170 | 3.5 - 4.5 | " |

### 12.4.2.2 DETENTION PERIOD

The rate of removal of BOD and SS is maximum during the first 2 to 2½ hours of settling and thereafter decreases appreciably. Hence, increase in the detention time beyond 2 to 2½ hours will not increase the percentage removal of BOD or SS proportionately. Longer detention period may affect the tank performance adversely due to setting in of septic conditions, particularly in tropical climates. Experience has shown that a detention period of 2 to 2½ hours for primary settling tanks and 1½ to 2 hours for secondary settling tanks will produce the optimum results.

Longer detention periods in secondary settling tanks may result in denitrification which adversely affects the settling efficiency.

### 12.4.2.3 SOLIDS LOADING RATE

The solids loading rate or solid flux is an important decision variable for the design of secondary sedimentation tank receiving bioflocculated solids. The solid flux represents the solids loading per unit surface area of tank per unit time and is expressed as kg $SS/m^2.d$. Design solid loadings at average and peak flows are presented in Table 12.1

12.4.2.4 WEIR LOADING

Weir loading influences the removal of solids in sedimentation tank, particularly in secondary settling tanks where flocculated solids are settled. There is no positive evidence that weir loading has any significant effect on removal of solids in primary settling tanks. However, certain loading rates based on practice are recommended both for primary as well as secondary tanks. For all primary, intermediate and secondary settling tanks, except in the case of secondary tanks for activated sludge process, weir loading of the order of 125 $m^3$/d.m. for average flows is recommended. For secondary settling tanks in activated sludge or its modifications, the weir loading is around 185 $m^3$/d.m. The loading should however ensure uniform withdrawal over the entire periphery of the tank to avoid short circuiting or dead pockets. Performance of existing sedimentation tanks can be improved by merely increasing their weir length.

This restriction in weir overflow rate requires special outlet weir design including a total weir length several times the tank width, for rectangular tanks and often two weirs with an outlet channel between them for circular tanks. Very long weirs cannot be maintained truly level over their full length, except perhaps at considerable expense; and satisfactory distribution of flow is more readily obtained by forming indentations at regular intervals such as shallow-V-notches say 50mm deep spaced 0.15 to 0.3 m apart. In addition to the head above the V-notches, a reasonable free fall of 0.05 to 0.15m should be allowed for maximum flows depending in part, on the total head available.

12.4.2.5 DEPTH

The depth sets the detention time in the settling tank and also influences sludge thickening in secondary settling tanks of activated sludge plants. The depth recommended for horizontal flow tanks are given in Table 12.1. In vertical flow tanks, depth may be 2.0m excluding hoppers.

12.4.2.6 SLUDGE REMOVAL

Sludge can be removed manually, hydrostatically or mechanically from the sedimentation tanks. Mechanical cleaning of sludge should be preferred to manual cleaning even in small plants, where power is available for running the plant machinery. Even where power is not available or inadequate or exorbitantly costly, hydrostatic removal should be adopted to avoid manual handling of sludge to prevent exposure of workers to health hazards.

Manual removal requires the tank to be put out of commission for dewatering. Sludge is then flushed by a jet of water into sumps. Workers may also have to enter tank and push the sludge into sumps by means of brushes and squeegees. The sludge collected in the sumps is withdrawn from the tank by gravity, hydrostatic pressure or pumping. The slope of the tank floors should be gentle, not more than 1 to 2 percent, towards the sump for men to walk on the floor. Tank capacity should also provide for storage of sludge between intervals of cleaning not exceeding 7 days in tropical climates. Manual cleaning has been largely given up in favour of mechanical cleaning in modern practice.

Tanks are provided with hopper bottoms for hydrostatic sludge removal. Generally horizontal flow tanks are provided with rectangular hoppers and vertical tanks with circular or square types. Side slopes of the hoppers should be of the order of 1.2:1 to 2:1 preferably with values greater than 1.7:1 and 1.5:1 for pyramidal and conical hoppers respectively. The floor of the hoppers should not be wider than 0.6 m.

Mechanical sludge scraping is best suited for circular or square tanks and occasionally adopted in rectangular tanks. The scrapers or ploughs push the sludge along the tank bottom to sludge collecting channel or pocket from where it is either pumped directly or gravitated to a sludge sump for further disposal.

In rectangular tanks, sludge hoppers are generally placed at the inlet end. But they may be placed at mid-length in long tanks or at the outlet end in case of secondary settling tank of activated

sludge plant. The sludge scraping mechanism may be of a moving bridge type of flight scrapers mounted on endless chain conveyors. The linear conveyor speed should not exceed 0.010 to 0.015 mps. In case of flight scrapers, the maximum width of tanks is greater than twice the depth. Multiple flight scrapers are placed side by side, in which case the width of tank could be increased upto a maximum 30 m. When multiple flight scrapers are used, the receiving sludge hoppers is designed as a trough with transverse collectors to convey the sludge to a single outlet pocket. A bottom slope of 1% is recommended for mechanical scraping of sludge.

The most common type of sludge scraping in circular or square tanks consists of a revolving mechanism with radial arms having ploughs or blades set at an angle just above floor level. The ploughs push the sludge to a central hopper as the arms are rotated. Sludge from the central hopper is removed to a sludge sump by the side of the tank from where it is pumped. For small dia. upto 9m the revolving bridge is spanned across the tank dia while for larger sizes it is supported on the tank wall on one side and on a hollow pillar at the centre of the tank on the other side which also serves as an inlet. Drive motors can be either stationary or movable in the case of traction drive and are placed above the tank.

The rotating mechanism of the sludge scraper for square tanks is similar to that of circular tanks except for additional pivoted corner blades for removing sludges from the corners. All rotary mechanisms are operated at a low speed of 1 to 2 rph.

The interval between sludge removal should be preferably less than 4 hours and never exceed 12 hours. Light flocculent sludges such as the activated sludge or mixture of activated sludge and primary sludge are scraped and removed continuously from the tank to avoid septicity. The peripheral speed of the scraper should be between 2.5 to 4 cm/sec.

Where sludge is removed intermittently with intervals longer than 4 hrs, provision for sludge storage in the hoppers should be made. Sludge conveyor pipes should not be less than 200 mm in dia. Hopper volumes should be excluded from the effective sedimentation volume of the tank.

### 12.4.2.7 INLETS AND OUTLETS

Performance of sedimentation tanks is very much influenced by inlet devices which are intended to distribute and draw the flow evenly across the basin. All inlets must be designed to keep down the entrance velocity to prevent formation of eddy or inertial currents in the tank to avoid short circuiting. Design should ensure least interference with the settling zone to promote ideal settling conditions. Choice of inlet and outlet design depends on the geometry of sedimentation tank and the mode of entry and exit from the tank.

In horizontal flow rectangular tanks, inlets and outlets are placed opposite each other separated by the length of tank with the inlet perpendicular to the direction of flow.

In the design of inlets to rectangular tanks the following methods are used to distribute the flow uniformly across the tank:

(a) Multiple pipe inlets with baffle boards of depth 0.45 to 0.6m in front of the inlets, 0.6 to 0.9 m away from it, and with the top of baffle being 25mm below water surface for the scum to pass over

(b) channel inlet with perforated baffle side wall between the tank and the channels, or

(c) inlet channel with submerged weirs discharging into tank followed by a baffle board inside the tank.

A stilling chamber is necessary ahead of inlets if the sewage is received under pressure from pumping mains.

Outlet is generally an overflow weir located near the effluent end, preferably adjustable for maintaining the weir at a constant level. V-notches are provided on the weir to provide for uniform distribution of flow at low heads of discharge over the weir. Weir lengths could be increased by placing outlet channel inside the tank with weirs on both sides. Scum baffles are provided ahead of outlet devices to prevent the escape of scum with the effluent.

In radial flow circular tanks the usual practice is to provide a central inlet and a peripheral outlet. The central inlet pipe may be either a submerged horizontal pipe from wall to centre or an inverted siphon laid beneath the tank floor. An inlet baffle is placed concentric to the pipe mouth generally with a diameter of 10-20% of the tank diameter and extending 1 to 2 m below water surface. Where the inlet pipe discharges into a central hollow pillar, the top of the pillar is flared to provide adequate number of inlet diffusion ports through which sewage enters the tank with an entry velocity of 0.10 to 0.25 mps through the ports. The entry ports are submerged 0.3 to 0.6 m below water surface.

The outlet is generally a peripheral weir discharging freely into a peripheral channel. The crest of the weir is provided with V-notches for uniform draw off at low flows. In all primary settling tanks a peripheral scum baffle extending 0.20 to 0.30 m below water surface is provided ahead of effluent weir. If the length of the peripheral weir is not adequate, a weir trough mounted on wall brackets near the periphery with adjustable overflow weir on both sides is provided to increase the length of weir.

### 12.4.2.8 Scum Removal

One distinct feature of primary settling tank is the skimming device which, though desirable is not normally provided in intermediate or secondary settling tanks. The skimming device could be operated by the same scraper mechanism used for sludge scraping at the bottom of the tank. It generally consists of a skimmer arm to which a scraper blade is attached and moved, partly submerged and partly projecting above the water surface, from the outlet end towards the inlet end in case of rectangular tanks or in a circular path in the case of circular tanks. The floating scum is thus collected at the forward end of the scraper blade and moved till it is tripped manually or automatically into a scum trough which discharges the scum to a sump outside the tank from where it is removed for burial, burning or feeding to the digester. A scum baffle at least 0.15 m above and extending to at least 0.30m below water level is provided along the periphery, ahead of outlet device, to prevent the escape of scum with effluent.

### 12.4.2.9 Types and Shapes

Circular tanks are more common than rectangular or square tanks. Upflow tanks have been used for sewage sedimentation but horizontal flow types are more popular. Rectangular tanks need less space than circular tanks and could be more economically designed where multiple units are to be constructed in a large plant. They can form a more compact layout with the rectangular secondary treatment units such as aeration tanks in the activated sludge system.

For rectangular tanks, maximum length and widths of 90 and 30m respectively with length to width ratios of 1.5 to 7.5 and length to depth ratios of 5 to 25 are recommended. A minimum depth of 2.5 m in case of primary settling tanks and 3.5 m in case of secondary settling tanks for activated sludge should be provided. Bottom slopes of 1% are normally adopted. Peak velocities greater than 1.5 mph should be avoided.

Diameters of circular tanks vary widely from 3 to 60 m although the most common range is 12 to 30m. Diameters and depths could be chosen at the discretion of the designer in conformity with the manufactured sizes of scraper mechanisms in the country. The water depth varies from 2m for primary to 3.5m for secondary settling tanks. Floors are sloped from periphery to centre at a rate of 7.5 to 10%. The inlet to the tank is generally at the centre and outlet is a peripheral weir, the flow being radial and horizontal from centre to the periphery of the tank. Multiple units are arranged in pairs with feed from a central control chamber.

## 12.5 PERFORMANCE

Primary sedimentation of domestic sewage may be expected to accomplish 30 to 45% removal of BOD and 45 to 60% removal of SS depending on concentration and characteristics of solids in suspension. Secondary settling tanks, if considered independently, remove a very high percentage of flocculated solids, even more than 99%, particularly following an activated sludge unit where a high mixed liquor suspended solids concentration is maintained in the aeration chamber. However, the efficiency of the biological treatment process is always defined in terms of the combined efficiency of the biological treatment units and its secondary settling tank with reference to the characteristics of the incoming sewage.

## 12.6 CHEMICAL-AIDED SEDIMENTATION

Chemical-aided sedimentation of sewage or industrial wastewater is analogous to chemical coagulation, flocculation and sedimentation in water treatment. The colloidal and finely dispersed solids which cannot be removed by plain primary sedimentation alone as they possess extremely low settling velocities are aggregated into settleable particles by addition of chemicals in chemical-aided sedimentation. Commonly used chemicals are trivalent or divalent metallic salts such as aluminium sulphate, ferric and ferrous salts and lime. Polyelectrolytes and polymers; cationic, anionic and nonionic, have also been used both as primary coagulants as well as coagulant-aids. The colloidal particles in domestic wastewaters are usually negatively charged and therefore do not agglomerate naturally into settleable mass. Addition of chemical coagulants results in destabilisation, aggregation and binding together by any one or more than one mechanisms of (i) Ionic Layer compression (ii) Adsorption and Charge Neutralisation (iii) Enmeshment in Precipitate and (iv) Interparticle Bridging. When Aluminium and Iron Salts are added to wastewaters in quantities sufficient to exceed the solubility limits of metal hydroxide, polymers of hydroxometal [ e.g. $Al_{13}(OH)^{5+}_{34}$ ] complexes are formed which are adsorbed on colloidal particles and neutralise their charge or form bridging between colloidal particles if the polymers are of the same charge as that on colloidal particles. Further, colloidal particles can get entrapped within the metal hydroxide precipitates which are heavy and settleable. Chemically-aided sedimentation produces intermediate results between plain sedimentation and secondary biological treatment. With proper dosages of chemicals, this treatment process may be expected to remove 60 to 80% of suspended solids and 45 to 65% of BOD when it is not preceeded by any plain sedimentation.

Chemical-aided sedimentation involves the unit processes and operations of chemical coagulation, flocculation and sedimentation. Therefore, it will not remove dissolved solids. On the contrary, addition of chemicals which are soluble may add to the total dissolved solids concentration of wastewater.

As compared to secondary biological treatment methods such as trickling filter or activated sludge, chemical treatment methods will be less efficient and will work out to be uneconomical and are therefore recommended only when

i) Plants are operated seasonally or variations in strength and volume of sewage are high

ii) intermediate treatment between plain sedimentation and secondary biological treatment is adequate

iii) sludge conditioning for dewatering is needed.

### 12.6.1 Chemicals Used

The most commonly used chemicals are ferrous sulphate; ferric chloride, ferric sulphate, chlorinated copperas, alum, aluminium chloride, lime and sodium carbonate. Choice of chemical and its dosage depend on cost of chemical, degree of treatment required and the characteristics of the