9. TREATMENT OF SEWAGE

SETTLING VELOCITY

(i) $V_S = \frac{g}{18}(G-1)\frac{d^2}{\nu}$ for $d < 0.1$ mm

where, V_S = Velocity of settlement of particle or settling velocity in m/sec.

d = Diameter of the particle in meter.

G = Specific gravity of the particle.

ν = Kinematic viscosity of water in m^2/sec.

$\nu = \frac{Y}{\delta}$

Y = Dynamic viscosity

δ = Density

(ii) $V_S = \sqrt{\frac{\frac{4}{3} g \cdot (G-1)d}{C_D}}$ where, C_D = Coefficient of drag $= \frac{24}{R_e}$

→ For laminar flow

R_e = Reynolds number $= \frac{\delta v_s d}{Y} = \frac{V_S d}{\nu}$

$C_D = \frac{24}{R_e} + \frac{3}{(R_e)^{0.5}} + 0.34$ $C_D = \frac{18.5}{R_e^{0.6}}$ ⟶ for transition flow.

$C_D = 0.34 + 0.4$ ⟶ for turbulent flow.

(iii) $V_S = \left[\frac{g(G-1)}{13.88}\frac{d^{1.6}}{\nu^{0.6}}\right]^{0.714}$ 0.1 mm < d < 1.0 mm.

(iv) Newtons Equation for Turbulent Settling

$V_S = 1.8\sqrt{gd(G-1)}$ for d > 1 mm

(v) Modified Hazen's Equation for Transition Zone

(a) $\boxed{V_S = 60 \cdot 6d(G-1) \cdot \left(\frac{3T+70}{100}\right)}$ For $0.1 < d < 1$ mm

Where T = Temperature in °C.

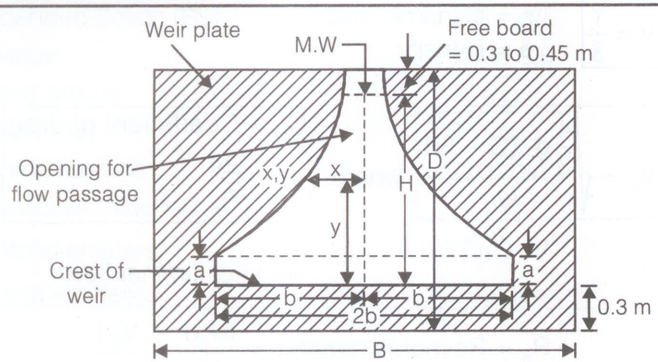
(b) Putting G = 2.65 for Inorganic Solids $\boxed{V_S = d(3T+70)}$

(c) Putting G = 1.2 for Organic Solids $\boxed{V_S = 0.12d(3T+70)}$

CRITICAL SCOUR VELOCITY IN CONSTANT VELOCITY HORIZONTAL FLOW GRIT CHAMBER (V_H)

$$\boxed{V_H = 3 \text{ to } 4.5\sqrt{gd(G-1)}}$$

PROPORTIONAL FLOW WEIR



$\boxed{x = \frac{2BV_h}{C_d \cdot \sqrt{2g} \cdot \pi \sqrt{y}}}$ where, B = Width of channel.

V_h = Horizontal flow velocity.

C_d = Coefficient of discharge.

x and y are coordinates on weir profile.

$\boxed{b = 1.467\,B\,V_h}$

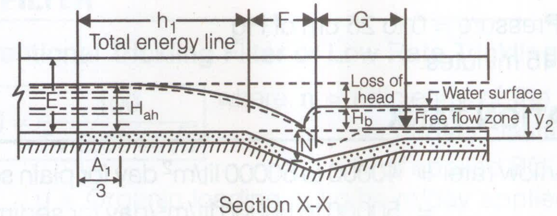
PARABOLICALLY OR V-SHAPED GRIT CHAMBER PROVIDED WITH A PARSHALL FLUME

(i) **Parshall Flume** $\boxed{Q = 2.264W(H_a)^{3/2}}$

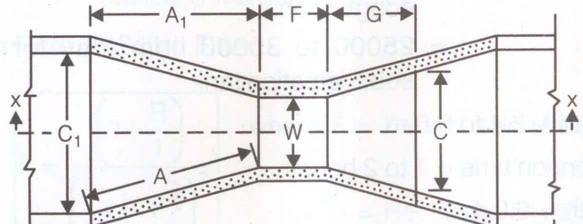
where, W = Width of the throat in meter.

Q = Flow in (m^3/sec) through Parshall flume.

H_a = Depth of flow in upstream leg of flume of one third portion in meter.



Section X-X



The Parshall flume

(ii) **Parabolic Grit Channel**

$\boxed{Q = C_1 \cdot \int_{y=0}^{y=y} x\,dy}$ where $\boxed{V_h = \frac{Q}{\int_0^y x\,dy} = C_1}$

$\boxed{Q = C_2 y^n}$ where, n = Discharge coefficients of the control section.

$\boxed{Y^{n-1} = \frac{C_1}{C_2} \cdot x}$ = 1.5 for parshall flume.

= 1 for proportional flow weir.

- **Aerated Grit Channels** $\boxed{\text{Detention period} = 3 \text{ minutes}}$
- **Detritus Tank** $\boxed{\text{Detention period} = 3 \text{ to } 4 \text{ minutes}}$

SKIMMING TANK

(i) Detentions Period = 3 to 5 minutes.

(ii) Amount of compressed air required = 300 to 6000 m^3 per million litre of sewage.

(iii) Surface Area,

$\boxed{A = 0.00622 \frac{q}{V_r}}$ where, q = Rate of flow of sewage in m^3/day.

V_r = Min. rising velocity of greasy material to be removed in m/min

= 0.25 m/min mostly.

- **Vacuators**

  Vacuum Pressure = 0 to 25 cm of Hg

  For 10 to 15 minutes.

SEDIMENTATION TANK

(i) Overflow rate = 40000 to 50000 lit/m^2 day for plain sedimentation.

= 50000 to 60000 lit/m^2/day for sedimentation with coagulation.

= 25000 to 35000 lit/m^2/day for secondary sedimentation tank

(ii) Depth ~ 2.4 to 3.6 m.

(iii) Detention time = 1 to 2 hour.

(iv) Width = 6.0 m

(v) Length = 4 to 5 times width

(vi) Velocity of flow V_f = 0.3 m/min.

(vii) $$V = \frac{Q}{BH}$$

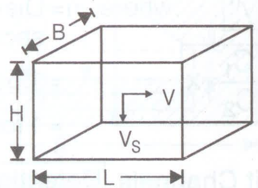
where, V = Flow velocity

B = Width of the Basin

H = Depth of sewage in the tank.

(viii) $$\frac{V}{V_S} = \frac{L}{H}$$

(ix) $$V_S = \frac{Q}{BL}$$



- **Detention Time**

(a) $$t = \frac{BLH}{Q}$$ For rectangular Tank

(b) $$t = \frac{d^2(0.011d + 0.785H)}{Q}$$ for circular tank

where d = Dia of the tank

H = Vertical depth of wall or side water depth

- **Displacement Efficiency (η)**

$$\eta = \frac{\text{Flowing through period}}{\text{Detention period}}$$

TRICKLING FILTER

(a) Conventional Trickling Filter or Low Rate Trickling Filter

$$\eta(\%) = \frac{100}{1 + 0.0044\sqrt{u}}$$

where, η = Efficiency of the filter and its secondary clarifier, in terms of % of applied BOD

u = Organic loading in kg/ha-m/day applied to the filter (called unit organic loading)

(b) High Rate Trickling Filter

(i) $$F = \frac{\left(1 + \frac{R}{I}\right)}{\left(1 + 0.1\frac{R}{I}\right)^2}$$

where, F = Recirculation factor

$\frac{R}{I}$ = Recirculation ratio

(ii) $$\eta(\%) = \frac{100}{1 + 0.0044\sqrt{\frac{Y}{VF}}}$$

where, Y = Total organic loading in kg/day applied to the filter i.e. the total BOD in kg.

$\frac{Y}{VF}$ = Unit organic loading in kg/Ha-m/day

V = Filter volume in Ha-m.

η% = % efficiency of single stage high rate trickling filter.

(iii) $$\eta'(\%) = \frac{100}{1 + \frac{0.0044}{1-\eta}\sqrt{\frac{Y'}{V'F'}}}$$

where,

η′ = Final efficiency in the two stage filter.

Y′ = Total BOD in effluent from first stage in kg/day.

F′ = Recirculation factor for second stage filter

V′ = Volume in second stage filter in ha-m.

Characteristics (2)	Conventional or Standrad rate filters (3)	High rate filters (4)
Depth of filter media	Varies between 1.6 to 2.4 m.	Varies between 1.2 to 1.8 m.
Size of filter media	25 to 75 mm.	25 to 60 mm.
Land required	More land area is required, as the filter loading is less.	Less land area is required as the filter loading is more.

Cost of operation	It is more for treating equal quantity of sewage.	It is less for treating equal quantity of sewage.
Method of operation	Continuous application, less flexible requiring less skilled supervision.	Continuous application, more flexible, and more skillful operation is required.
Type of effluent produced	The effulent is highly nitrified and stabilized, with BOD in effluent ≤ 20 ppm or so.	The effluent is nitrified up to nitrite stage only and is thus less stable, and hence it is slightly inferior quality. BOD in effluent ≥ 30 ppm. or so.
Doing interval	It generally varies between 3 to 10 minutes. The sewage is generally not applied continuously but is applied at intervals.	It is not more than 15 seconds, and the sewage is thus applied continuously.
Filter loading values (i) Hydraulic loading	Varies between 20 to 44 M.L. per day	Varies between 11 to 330 M.L. per hectare day.
(ii) Organic loading	Varies between 900 to 2200 kg of BOD_5 per hectare-metre of filter media per day.	Varies between 6000 to 18,000 kg of BOD_5 per hectare metre of filter media per day.
Recirculation system	Not provided generally.	Always provided for increasing hydraulic loading.
Quality of secondary sludge produced	Black, highly oxidised with slight particles.	Brown, not fully oxidised with fine particles.

Dunbar Filter

Surface loading = 2500 Ml/m²/day.

BOD removed = 85%

Sludge and its Moisture Content

$$V = V_1\left[\frac{100 - P_1}{100 - P}\right]$$

V_1 = Volume of sludge at moisture content P_1%

V = Volume of sludge at moisture content P%

SLUDGE DIGESTION TANK

(i) When the change during digestion is linear.

(a) $$V = \left(\frac{V_1 + V_2}{2}\right)t$$

where, V = Volume of digestion in m³.

V_1 = Raw sludge added per day (m³/day)

V_2 = Equivalent digested sludge produced per day on completion of digestion, m³/day.

t = Digestion period in day.

(b) $$V = \left(\frac{V_1 + V_2}{2}\right)t + V_2T$$ with monsoon storage

where, T = Number of days for which digested sludge (V_2) is stored (monsoon storage)

(ii) When the change during digestion is parabolic

(a) $$V = \left[V_1 - \frac{2}{3}(V_1 - V_2)\right]t$$ without monsoon storage

(b) $$V = \left[V_1 - \frac{2}{3}(V_1 - V_2)\right]t + V_2T$$ without monsoon storage

Type of raw sludge to be digested	kg. of volatile solids present per cu. m. of sludge per month	Per capita capacity in cum/capita
Primary sludge	8	0.021
Mixture of primary sludge and secondary sludge from trickling filters (humus tanks)	7.36	0.036
Mixture of primary sludge and secondary activated sludge	5.76	0.061
Chemically coagulated sludge	—	

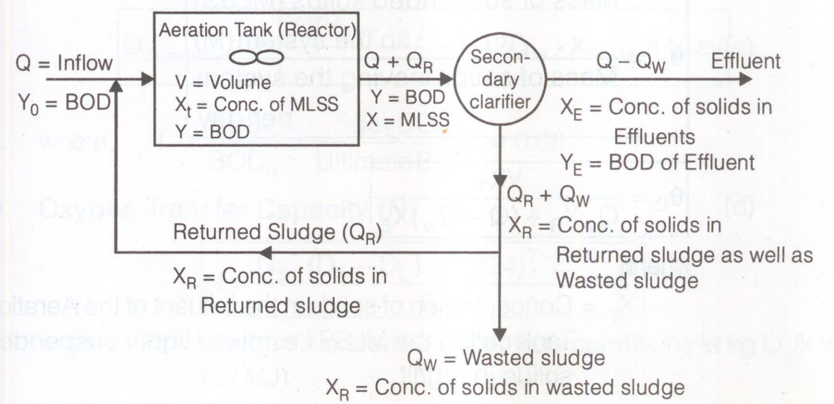
DESTRUCTION AND REMOVAL EFFICIENCY (DRE)

$$DRE = \frac{W_{in} - W_{out}}{W_{in}} \times 100$$

where, W_{in} = Mass fill rate of one POHC (Principal organic Hazardous constituent) in the waste stream.

W_{out} = Mass emissionrate of the same POHC present in the exhaust emission prior to release to the atmosphere.

AERATION TANK



(i) **Detention period,** $t = \frac{V}{Q}$

where V = Volume of the tank in m^3.

Q = Quantity of waste water flow into the aeration tank excluding the quantity of recycled sludge (m^3/day)

(ii) **Volumetric BOD Loading or Organic Loading, (U)** $u = \frac{QY_O}{V}$

where, QY_O = Mass of BOD applied per day to the aeration tank through influent sewage in gm.

V = Volume of aeration tank in m^3.

Q = Sewage flow into the aeration tank in m^3.

Y_O = BOD_5 in mg/lit (or gm/m^3) of the influent sewage.

(iii) $\frac{F}{M} = \frac{QY_O}{VX_t}$ where, $\frac{F}{M}$ = Food (F) to Microorganism (M) ratio

QY_O = Daily BOD applied to the aeration system in gm.

Y_O = 5 day BOD of the influent sewage in mg/lit.

Q = Flow of influent sewage in m^3/day.

X_t = MLSS (Mixed liquor suspended solids) in mg/lit.

V = Volume of the Aeration Tank (lit).

$M = X_tV$ = Total microbial mass in the system in gm.

(iv) **Sludge Age (θ_C)**

(a) $\theta_C = \frac{\text{Mass of suspended solids (MLSS) in the system (M)}}{\text{Mass of solids leaving the system per day}}$

(b) $\theta_C = \frac{VX_T}{Q_wX_R + (Q - Q_w)X_E}$

where,

X_T = Concentration of solids in the influent of the Aeration Tank called the MLSS i.e. mixed liquor suspended solids in mg/lit.

V = Volume of Aerator

Q_W = Volume of wasted sludge per day

X_R = Concentration of solids in the returned sludge or in the wasted sludge (both being equal) in mg/lit.

Q = Sewage inflow per day.

X_E = Concentration of solids in the effluent in mg/lit.

(v) **Sludge Volume Index (S.V.I)**

$SVI = \frac{V_{ob}}{X_{ob}} \times 1000$ where, X_{ob} = Concentration of suspended solids in the mixed liquor in mg/lit.

V_{ob} = Settled sludge volume in ml/lit.

S.V.I. = Sludge volume index in ml/gm.

(vi) **Sludge Recycle and Rate of Return Sludge**

$Q_R.X_R = (Q + Q_R) \times t$

$\frac{Q_R}{Q} = \frac{X_t}{X_R - X_t}$ where, Q_R = Sludge recirculation rate in m^3/day.

X_t = MLSS in the aeration tank in mg/lit.

X_R = MLSS in the returned or wasted sludge in mg/lit.

$X_R = \frac{10^6}{S.V.I}$ S.V.I = Sludge volume index in ml/gm.

- **Specific substrate utilization rate**

$U = \frac{Q(Y_0 - Y_E)}{V.X_t}$ $\frac{1}{\theta_C} = \alpha_y U - k_e$

α_y = 1 for MLSS and 0.6 for MLVSS, k_e = 0.06

- **Oxygen Requirement of the Aeration Tank**

$$O_{2(required)} = \left[\frac{Q(Y_O - Y_E)}{f} - 1.42Q_W \cdot X_R\right] gm/day$$

where, $f = \frac{BOD_5}{BOD_u} \simeq \frac{5\,day\,BOD}{Ultimate\,BOD} = 0.68$

- **Oxygen Transfer Capacity (N)**

$$N = \frac{N_S \cdot (D_S - D_L) \cdot (1.024)^{T-20°C} \cdot \alpha}{9.17}$$

where, N = Oxygen transferred under field conditions in kg O_2/k.wh (Or MJ)

N_S = Oxygen transfer capacity under standard conditions in kg O_2/kwh (or MJ)

D_S = Dissolved oxygen-saturation value for sewage at operating temperature.

D_L = Operation D·O level in Areation tank usually 1 to 2 mg/lit.

T = Temperature in °C

α = Correction factor for oxygen transfer for sewage usually 0.8 to 0.85.

OXIDATION PONDS

- Depth → 1.0 to 1.8 m.
- Detention period → 2 to 6 weeks.
- Organic loading → 150 to 300 kg/ha/day.

  Under hot condition → 60 to 90 kg/ha/day.

  Under cold conditions.
- Length to width ratio = 2
- Sludge Accumulation = 2 to 5 cm/year
- Minimum depth to be kept = 0.3 m.

For Inlet Pipe Design

Assume v = 0.9 m/s

Assume flow for 8 hrs.

For Outlet Pipe Design

Dia of outlet = 1.5 dia of inlet pipe

SEPTIC TANK

- Detention time = 12 to 36 hr.
- Sludge accumulation rate = 30 lit/cap/year.
- Sewage flow = 90 to 150 lit/capita/day.
- Cleaning period = 6 to 12 months
- Length to width ratio = 2 to 3 m.
- Depth = 1.2 to 1.8 m
- Width $\not<$ 0.9 m.
- Free board = 0.3 m.
- $$\boxed{\text{Volume of septic tank} = (\text{Sewage flow} \times \text{Detention time}) + (\text{Sludge accumulation rate}) \times \text{Cleaning rate}}$$