Basrah University

College of Engineering

Department of Civil Engineering

Lectures in Sanitary Sewage Engineering

4th Class Course

3hours/week

15 weeks course

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Ch.1 Quantity of Sewage

1.1 Sources of Sewage

Sewage is defined as a combination of;

(a) the wastewater discharged from residences, business buildings and institutions (sanitary or domestic sewage),

(b) the wastewater discharged from industrial establishments (industrial wastewater),

(c) storm water, and

(d) infiltration.

1.2 Quantity of Sanitary Sewage

1.2.1 Average sewage flow rate

a. Residential Areas:

Average sewage flow rate produced in residential areas is estimated to be 70 to 130% of average rate of water consumption. A percentage of 100% is frequently assumed including a moderate allowance for infiltration.

b. Non-residential Development:

Average daily flows for non-residential developments are given in Table (1.1).

Table (1.1) Average sewage flows for non-residential developments

Type of establishment	Unit	Average flow (litre/day/unit)	
		Range	Typical
Shopping centre	m^2	2.5-5.0	
Airports	Passenger	11-19	15
Theatre	Seat	8-15	10
Office	Person	26-60	50
School	Student	40-80	60
Hotel	Guest	150-230	190
Hospital	Bed	660-1500	1000
	Employee	20-60	40

1.2.2 Maximum sewage flow rate

Maximum sewage flow rate = M× average sewage flow rate

$$M = 1 + \frac{14}{4 + \sqrt{P}} \qquad \ldots(1.1)$$

where; p= served population in thousands

1.2.3 Peak sewage flow rate

$$Peak\ sewage\ flow\ rate = Peak\ factor \times average\ sewage\ flow\ rate \qquad \ldots(1.2)$$

The value of peak factor is dependent on served population in thousand and can be obtained using Fig. (1.1).

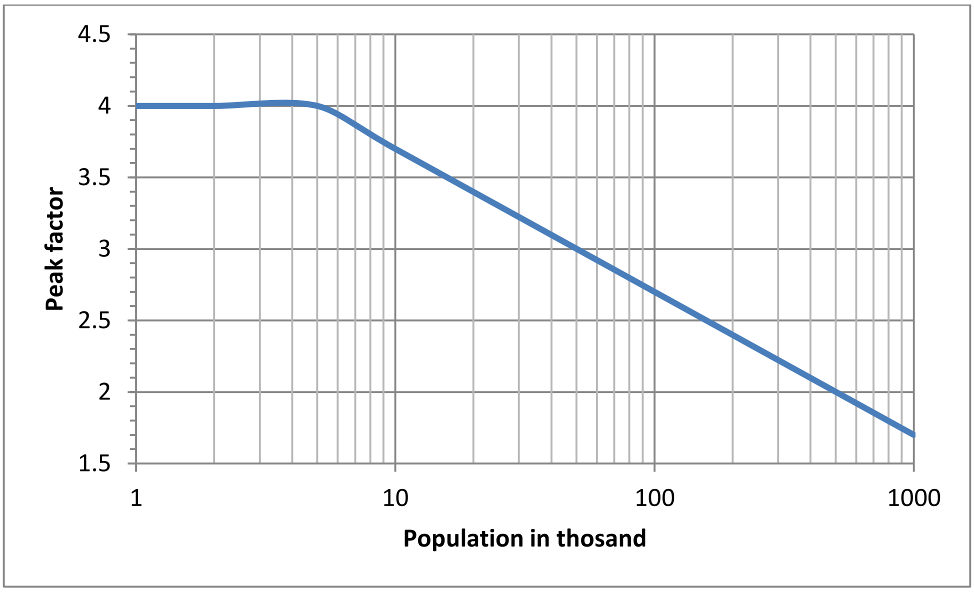


Fig.(1.1) Peak factor verses population in thousands

1.2.4 Minimum sewage flow rate

Minimum sewage flow rate is estimated to be 50% of average sewage flow rate.

1.2.5 Infiltration

Infiltration is the groundwater enters the sewers through poor joints, cracked pipes and manhole walls. Infiltration is estimated to be 45 litre per km of sewer length per day per mm of sewer diameter. Infiltration can also be estimated based on the area served by the sewer system and may range from 0.2 to 28 m^3 per hectare of land area per day.

Example 1.1

A city has a population of 30000. In this city, there are;

- 3 schools, each of 250 student capacity.
- One hotel of 120 guest capacity.
- One hospital of 200 bed capacity in which the number of employees is 150.

Estimate the average, maximum, peak and minimum sanitary sewage flowrates of the city.

Solution

Average sewage flow rate of the city

Average sewage flow rate of the city is the sum of average sewage flowrates from residential area, from the schools, from the hotel and from the hospital.

Average sewage flowrate from residential area;

Assume average water demand= 300 liter/capita/day.

$$\therefore average\ water\ demand = \frac{300}{1000} \times 30000 = 9000 m^3/day$$

$$assume\ average\ sewage\ flowrate = average\ water\ demand$$

Average sewage flowrate from residential area= 9000m^3/day.

Average sewage flowrate from the schools;

Let avg. sewage flowrate= 60 liter/student/day

$$\therefore average\ sewage\ flowrate\ from\ the\ schools = 3 \times 250 \times \frac{60}{1000}$$
$$= 45 m^3/day$$

Average sewage flowrate from the hotel;

Let avg. sewage flowrate= 190 liter/guest/day

$$\therefore average\ sewage\ flowrate\ from\ the\ hotel = 120 \times \frac{190}{1000}$$
$$= 22.8m^3/day$$

Average sewage flowrate from the hospital;

Let avg. sewage flowrate = 1000 liter/bed/day and 40 liter/employee/day

$$\therefore average\ sewage\ flowrate\ from\ the\ hospital$$
$$= 200 \times \frac{1000}{1000} + 150 \times \frac{40}{1000} = 206m^3/day$$

$$\therefore average\ sewage\ flowrate\ of\ the\ city = 9000 + 45 + +22.8 + 206$$
$$= 9273.8\ m^3/day$$

Maximum sewage flowrate;

Maximum sewage flow rate = M× average sewage flow rate

$$M = 1 + \frac{14}{4 + \sqrt{P}}$$

P=30000/1000=30

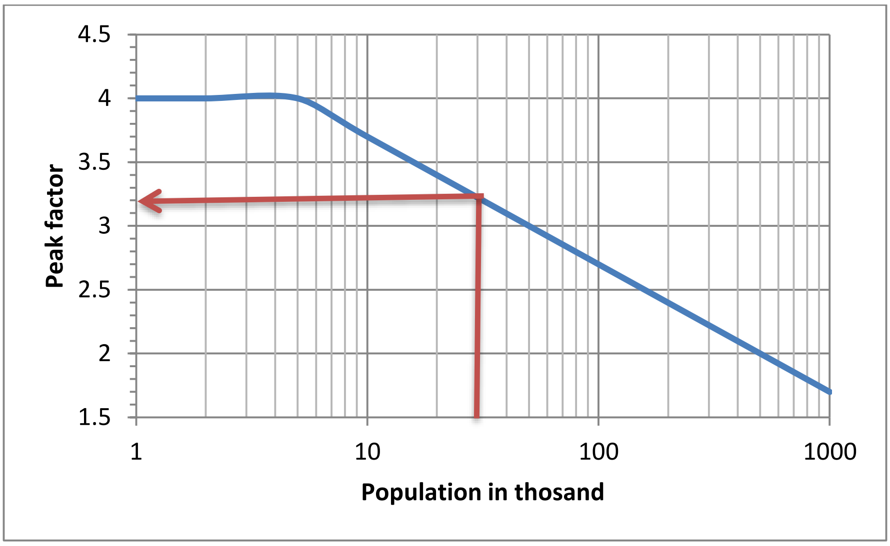
$$M = 1 + \frac{14}{4 + \sqrt{30}} = \mathbf{2.48}$$

$$\therefore\ maximum\ sewage\ flowrate = 2.48 \times 9273.8 = 22999.02m^3/day$$

Peak sewage flowrate;

$$Peak\ sewage\ flow\ rate\ =\ Peak\ factor \times\ average\ sewage\ flow\ rate$$

For population of 30000, the peak factor is obtained from Fig.(1.1) to be 3.2.



$$Peak\ sewage\ flow\ rate\ =\ 3.2 \times\ 9273.8 = 29676.2 m^3/day$$

Minimum sewage flowrate;

$$Minimum\ sewage\ flowrate = 0.5 \times avg.sewage\ flowrate$$

$$Minimum\ sewage\ flowrate = 0.5 \times 9273.8 = 4636.9\ m^3/day$$

Example 1.2

Estimate the infiltration rate flows into a sewer line has a length of 375m and a diameter of 300mm.

Solution

Infiltration is estimated to be 45 liter per km of sewer length per day per mm of sewer diameter.

$$Infiltration\ rate = 45 \times \frac{375}{1000(\frac{m}{km})} \times 300 = 5062.5\ liter/\mathrm{day}$$

Example 1.3

Estimate the infiltration rate flows into a sewer system serves an area of 8 km^2.

Solution

Based on served area, infiltration rate = 0.2 to 28 m^3 per hectare of land area per day

Let infiltration rate= 10 m^3/ha.day

Served area= 8 km^2= 800 ha (1 km^2=100 ha)

$Infiltration\ rate = 10 \times 800 = 8000\ m^3/\text{day}$

1.3 Quantity of Storm Water

The first step in designing storm sewers is the estimation of storm water quantity. The quantity of storm water is calculated using the rational formula;

$$Q = C\,.\,i\,.\,A \qquad \ldots(1.3)$$

where;

Q = runoff flow rate, m^3/hr.

C = runoff coefficient which is dependent on the characteristics of the drainge area surface.

i = rainfall intensity, m/hr.

A = drainage area, m^2.

1.3.1 Runoff coefficient

Runoff coefficient is a ratio of the amount of rainwater which will be carried by the storm sewer to the total rainfall. Average values of C commonly used for various surfaces are presented in Table (1-1). For composite drainage area, average value of runoff coefficient is calculated as;

$$C_{avg} = \frac{\sum_{i=1}^{i=n} C_i A_i}{A_T} \qquad \ldots(1.4)$$

where; C_i is runoff coefficient for subarea A_i and A_T is the total drainage area.

Table (1-1) Runoff coefficients for various surfaces

<table>
<tr><th colspan="2">Type of surface</th><th>C</th></tr>
<tr><td colspan="2">Watertight roofs</td><td>0.7-0.95</td></tr>
<tr><td colspan="2">Asphaltic streets</td><td>0.85-0.9</td></tr>
<tr><td colspan="2">Paved driveways and walks</td><td>0.75-0.85</td></tr>
<tr><td colspan="2">Gravel driveways and walks</td><td>0.15-0.3</td></tr>
<tr><td rowspan="3">Lawns, sandy soil</td><td><2% slope</td><td>0.05-0.1</td></tr>
<tr><td>2-7%</td><td>0.1-0.15</td></tr>
<tr><td>>7%</td><td>0.15-0.2</td></tr>
<tr><td rowspan="3">Lawns, heavy soil</td><td><2% slope</td><td>0.13-0.17</td></tr>
<tr><td>2-7%</td><td>0.18-0.22</td></tr>
<tr><td>>7%</td><td>0.25-0.35</td></tr>
</table>

According to land use, the values of runoff coefficient (C) are given in Table (1-2).

Table (1-2) Runoff coefficient for different areas

<table>
<tr><th colspan="2">Description of area</th><th>C</th></tr>
<tr><td rowspan="2">Commercial area</td><td>Downtown</td><td>0.7-0.95</td></tr>
<tr><td>Neighbourhood</td><td>0.5-0.7</td></tr>
<tr><td rowspan="5">Residential area</td><td>Single family</td><td>0.3-0.5</td></tr>
<tr><td>Multi-units detached</td><td>0.4-0.6</td></tr>
<tr><td>Multi-units attached</td><td>0.6-0.75</td></tr>
<tr><td>suburban</td><td>0.25-0.4</td></tr>
<tr><td>Apartments</td><td>0.5-0.7</td></tr>
<tr><td colspan="2">Parks</td><td>0.1-0.25</td></tr>
<tr><td colspan="2">Unimproved areas</td><td>0.1-0.3</td></tr>
</table>

Example 1.4

Determine the runoff coefficient for an area of 0.3km^2. 5000 m^2 is covered by buildings. 6000 m^2 covered by paved driveways and walks and 3000m^2 by asphaltic streets. The remaining area is flat, clay soil governed by grass lawn.

Solution

$$C_{avg} = \frac{\sum_{i=1}^{i=n} C_i A_i}{A_T} = \frac{\sum_{i=1}^{i=4} C_i A_i}{A_T} = \frac{C_1A_1 + C_2A_2 + C_3A_3 + C_4A_4}{A_1 + A_2 + A_3 + A_4}$$

For building's roof→ C=0.7-0.95

For paved driveways → C=0.75-0.85

For asphaltic streets → C=0.85-0.9

For grass lawn, clay soil → C=0.13-0.17

If the upper limit of C value for each area is used;

C_1=0.95 ; C_2=0.85 ; C_3= 0.9 ; C_4=0.17

A_1=5000 m^2 ; A_2=6000 m^2 ; A_3= 3000 m^2 ; A_4=0.3×10^6-(5000+6000+3000)= 286000 m^2

$$C_{avg} = \frac{0.95 \times 5000 + 0.85 \times 6000 + 0.9 \times 3000 + 0.17 \times 286000}{0.3 \times 10^6} = 0.204$$

1.3.2 Rainfall intensity

Rainfall intensity can be obtained using the following formula;

$$i = \frac{1000}{t + 30} \qquad \text{... (1.5)}$$

where;

i= rainfall intensity (mm/hr)

t= rainfall duration (min)

It is important to mention that the constants in Eq. (1.5) are dependent on drainage area and return period. See Table (13-3), p.327, which gives different formula in USA.

1.3.3 Time of Concentration

The maximum rate of runoff for a given rainfall intensity will occur when the rainfall has continued for a period sufficient to permit flow to reach the inlet from the most remote point of the drainage area. The rainfall duration which will correspond with the maximum rate of runoff to develop is known as the time of concentration (t_c).

Fig.2 illustrates a rectangular drainage area discharging into an inlet I. it is assumed that it takes 5min for water to run from the boundary of one zone to the next, or to the inlet in zone A. Only zone A will contribute flow after 5 min, only A and B after 10min, and all the three zones after 15min or more. If the rain is lasted only 10min, the water arriving at I from zone C during the period 10 to 15min after beginning would offset by decreasing runoff from zone A.

The same issue can be applied to the area shown in Fig.3. The water from area A enters the sewer at I_1 and that from area B at I_2. The time of concentration at I_2 is either the time of concentration for area B or the inlet time plus the time of flow from I_1 to I_2, whichever is greater.

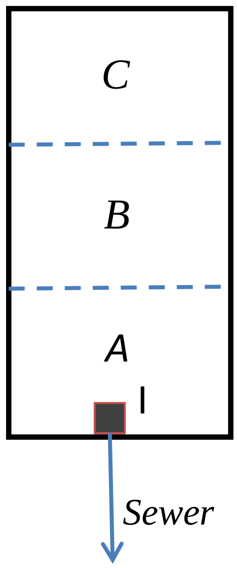


Fig.2 Inlet time

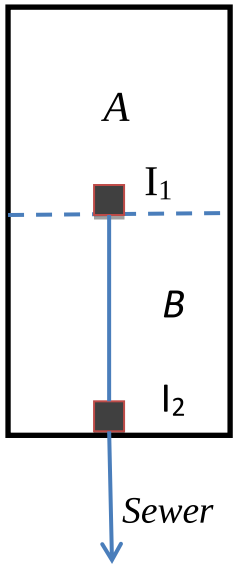


Fig.3 Inlet time and flow time

Generally, the time of concentration (t_c) is calculated as;

$$t_c = t_o + t_f \qquad ...(1.6)$$

where t_o is overland flow time of concentration and t_f is pipe flow time. The value of t_o (in min) is calculated as;

$$t_o = K\left(\frac{NL}{\sqrt{S_O}}\right)^{0.6} \qquad ...(1.7)$$

where L is the length of longest flow path (m), S_O is the slope of ground surface, K is a constant and N is an overland texture factor. The values of K and N are given in Tables (1-3) and (1-4), respectively.

Table (1-3) values of K

Rainfall intensity (mm/hr)	**K**
< 20 (light rain)	3
20-30 (moderate rain)	2.2
>30 (heavy rain)	1.4

Table (1-4) Overland texture factor N

Overland surface	**Low**	**Medium**	**High**
Smooth asphalt pavement	0.01	0.012	0.015
Smooth impervious surface	0.011	0.013	0.015
Tar and sand pavement	0.012	0.014	0.016
Land use			
Dense residential	0.025	0.04	0.06
Suburban residential	0.03	0.055	0.08
Parks and lawns	0.04	0.075	0.12

Assuming full flow condition, the time of flow (t_f) to the downstream end of the j-th pipe is expressed as:

$$t_{fj} = \sum_{j=1}^{N} \frac{L_j}{V_{fj}} \qquad \text{... (1.8)}$$

Where;

$\boldsymbol{L_j}$ = length of i-th pipe

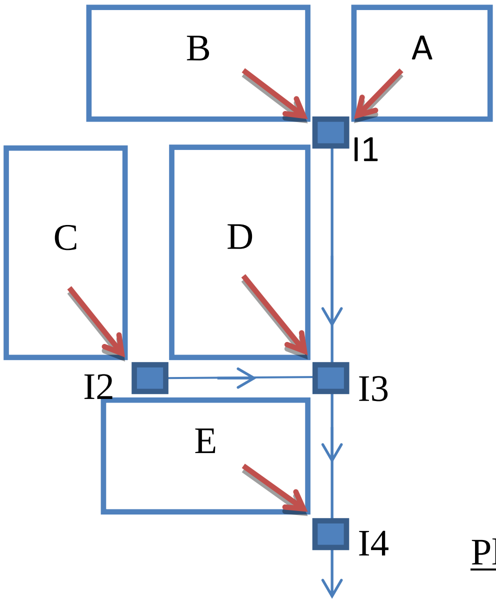
$\boldsymbol{V_{fj}}$ = full flow velocity of j-th pipe

N = number of pipes defining the longest flow path from any part of the catchment to the point being considered. Note that t_{fi} includes the time of flow down the current pipe.

Over land flow time (t_o) or inlet time may simply be assumed, frequently being taken as 5 to 10min.

Example 1.5

The storm sewer shown below receives storm water from five subareas (A, B, C, D, and E). For each subarea; the drainage area, the longest flow path length, the slope of ground surface and runoff coefficient are as given in Table I. Estimate storm water flowrate received by each of the four inlets. Assume the area is dense residential area. The flow time in each sewer is given in Table II.



Plan layout

Table I

zone	Area (m^2)	L (m)	Slope of ground surface	Runoff coefficient
A	8000	75.8	0.01	0.8
B	12000	127.3	0.0081	0.7
C	12000	121.2	0.012	0.4
D	20000	194	0.01	0.6
E	20000	200	0.01	0.6

Table II

Sewer line		Flow time (min)
From	To	
I1	I3	3.2
I2	I3	2.5
I3	I4	2.0

Solution

$\boldsymbol{Q = C \,.\, i \,.\, A}$

$$i = \frac{1000}{t+30}$$

t= time of concentration (t_c);

$t_c = t_o + t_f$

Inlet I1

I1 receives storm water from subareas A and B. Then, t_c for I1 equals t_c for subarea A or t_c for subarea B whichever the greater.

Subarea A;

$t_c = t_o$ (inlet time)

$$t_o = K\left(\frac{NL}{\sqrt{S_O}}\right)^{0.6}$$

Assume moderate rain→ K=2.2

For denes residential area→ N=0.04

$$t_o = 2.2\left(\frac{0.04 \times L}{\sqrt{S_O}}\right)^{0.6}$$

$$t_o = 2.2\left(\frac{0.04 \times 75.8}{\sqrt{0.01}}\right)^{0.6} = 17.04min$$

$\therefore t_c \ for \ subarea \ A = 17.04min$

Subarea B;

$t_c = t_o$ (inlet time)

$$t_o = 2.2\left(\frac{0.04 \times 127.3}{\sqrt{0.0081}}\right)^{0.6} = 24.8min$$

$\therefore t_c \ for \ subarea \ B = 24.8min$

$\therefore t_c \ for \ I1 = 24.8min$

$i = \frac{1000}{t+30} = \frac{1000}{24.8+30} =$ 18.25 mm/hr.

A= area of A+ area of B

A=8000+12000=20000m^2

$$C_{avg} = \frac{\sum_{i=1}^{i=n} C_i A_i}{A_T} = \frac{\sum_{i=1}^{i=2} C_i A_i}{A_T} = \frac{C_1 A_1 + C_2 A_2}{A_T}$$

$$C_{avg} = \frac{0.8 \times 8000 + 0.7 \times 12000}{20000} = 0.74$$

$$Q_{I1} = 0.74 \times \frac{18.25}{1000} \times 20000 = 270.1 m^3/hr.$$

<u>Inlet I2</u>

I2 receives storm water from subareas C only. Then, t_c for I2 equals t_c for subarea C.

Subarea C;

$t_c = t_o$ (inlet time)

$$t_o = 2.2 \left(\frac{0.04 \times L}{\sqrt{S_O}}\right)^{0.6}$$

$$t_o = 2.2 \left(\frac{0.04 \times 121.2}{\sqrt{0.012}}\right)^{0.6} = 21.38 min$$

$$\therefore t_c \ for \ subarea \ C = 21.38 min$$

$i = \frac{1000}{t+30} = \frac{1000}{21.38+30} =$ 19.5 mm/hr.

A= area of C= 12000m^2

C=0.4

$$Q_{I2} = 0.4 \times \frac{19.5}{1000} \times 12000 = 93.6 m^3/hr.$$

<u>Inlet I3</u>

I3 receives storm water from subareas D, sewer I1-I3, and sewer I2-I3. Then, t_c for I3 equals t_c for subarea D, t_c for sewer I1-I3, or for sewer I2-I3 whichever the greatest.

Subarea D;

$t_c = t_o$ (inlet time)

$$t_o = 2.2\left(\frac{0.04 \times 194}{\sqrt{0.01}}\right)^{0.6} = 29.95min$$

$\therefore t_c \; for \; subarea \; D = 29.95min$

Sewer I1-I3;

$t_c = t_o + t_f$

$t_O = 24.8min$

$t_f = 3.2min$

$t_c = 24.8 + 3.2 = 28 \; min$

Sewer I2-I3;

$t_c = t_o + t_f$

$t_O = 21.38min$

$t_f = 2.5 \; min$

$t_c = 21.38 + 2.5 = 23.88 \; min$

$\therefore t_c \; for \; I3 = 29.95min$

$i = \frac{1000}{t+30} = \frac{1000}{29.95+30} = 16.68$ mm/hr.

A_T= area of A+ area of B +area of C+ area of D

A_T=8000+12000+12000+20000=52000m^2

$$C_{avg} = \frac{\sum_{i=1}^{i=4} C_i A_i}{A_T} = \frac{C_1A_1 + C_2A_2 + C_3A_3 + C_4A_4}{A_T}$$

$$C_{avg} = \frac{0.8 \times 8000 + 0.7 \times 12000 + 0.4 \times 12000 + 0.6 \times 20000}{52000} = 0.608$$

$$Q_{I3} = 0.608 \times \frac{16.68}{1000} \times 52000 = 527.4m^3/hr.$$

<u>Inlet I4</u>

I4 receives storm water from subareas E and sewer I3-I4, then, t_c for I4 equals t_c for subarea E or t_c for sewer I3-I4 whichever the greater.

Subarea E;

$t_c = t_o$ (inlet time)

$$t_o = 2.2\left(\frac{0.04 \times 200}{\sqrt{0.01}}\right)^{0.6} = 30.5min$$

$$\therefore t_c \ for\ subarea\ E = 30.5min$$

Sewer I3-I4;

$$t_c = t_c for\ I3 + t_f$$

$$t_c for\ I3 = 29.95min$$

$$t_f = 2.0\ min$$

$$t_c = 29.95 + 2.0 = 31.95\ min$$

$i = \frac{1000}{t+30} = \frac{1000}{31.95+30} = 16.14$ mm/hr.

A_T= area of A+ area of B +area of C+ area of D+ area of E

A_T=8000+12000+12000+20000+20000=72000m^2

$$C_{avg} = \frac{\sum_{i=1}^{i=4} C_i A_i}{A_T} = \frac{C_1A_1 + C_2A_2 + C_3A_3 + C_4A_4 + C_5A_5}{A_T}$$

$$C_{avg} = \frac{0.8 \times 8000 + 0.7 \times 12000 + 0.4 \times 12000 + 0.6 \times 20000 + 0.6 \times 20000}{72000} = 0.606$$

$$Q_{I4} = 0.606 \times \frac{16.14}{1000} \times 72000 = 704.22\ m^3/hr.$$

Ch.2 Flow in Sewers

2.1 Collection of Sewage

Sewage is collected from the different sources using sewer systems. Sewer systems are classified into;

1- Separate sewer system: Herein, the city is provided by two sewer systems;

   a. Sanitary sewer system for collecting sanitary sewage, industrial wastewater in addition to infiltration and inflow (the surface water enters the sewer through perforated manhole covers and roof drains). This system transports the collected sewage to a sewage treatment plant.

   b. Strom sewer system for collecting the storm water. This system discharges the collected water to a nearest stream.

2- Combined sewer system: Herein, the city is provided by one sewer system for collecting all types of sewage. This system transports the collected sewage to a sewage treatment plant, Fig. (2-1).

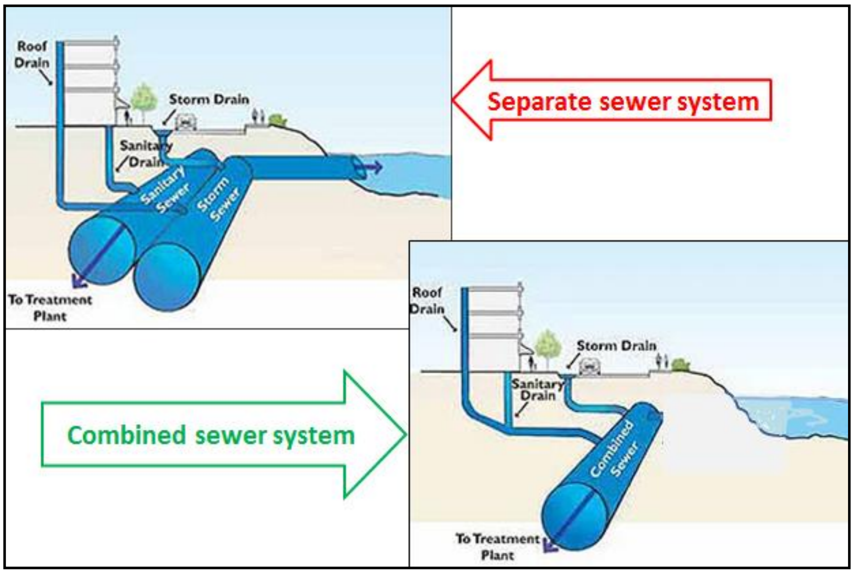


Fig.(2-1) Separate and combined sewer system

2.2 Components of Sewer Systems

Sanitary and storm sewer systems are composed of the following main components:

1. Sewers

Sewers are available in a variety of materials. They can be made of ductile iron, PVC (polyvinyl chloride), concrete, HDPE (high density polyethylene), and GRP (glass reinforced plastic).

<u>Vertical alignment</u>

Fig.(2.2) illustrates how the vertical position of a sewer is defined by its invert level (IL). The invert of a pipe refers to the lowest point on the outside of the pipe. The invert level is the vertical distance of the invert above some fixed level or datum. Soffit level is the highest point on the inside of the pipe and the crown level is the highest point on the outside of the pipe.

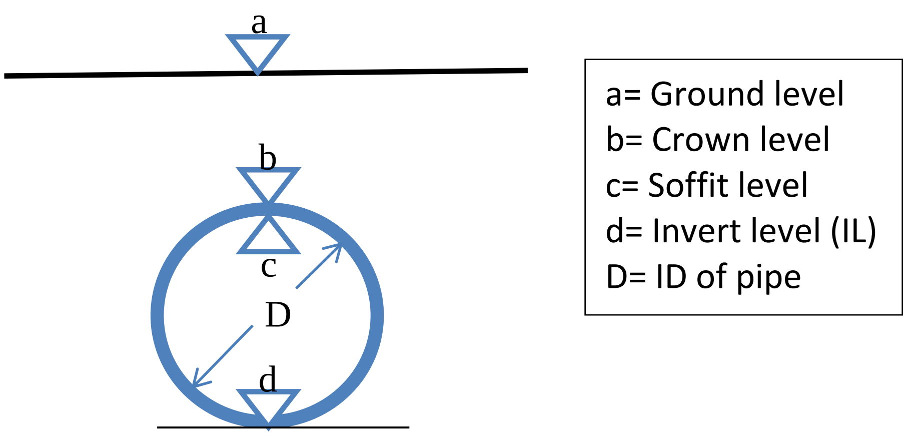


Fig. (2-2) Level definitions associated with sewers

2. Manholes

Manholes are structures designed to provide access to a sewer. Access is required for testing, visual inspection of sewers, and placement and maintenance of flow or water quality monitoring instruments. There are different types of manholes. According to the Iraqi code for the construction of sewer systems, the available manhole types are presented in Table (2-1). A typical vertical profile in a precast concrete manhole is shown in Fig. (2-3).

Table (2-1) Types of manholes

<table>
<tr><th>Manhole type</th><th>Manhole depth (m)</th><th>Dia. of outgoing sewer (mm)</th><th>Maximum number of incoming sewers</th><th>Manhole shape</th><th>Interior dimensions of manhole, m</th><th>Maximum spacing between manholes</th></tr>
<tr><td>AS</td><td>0.75-1.69</td><td>200-400</td><td>3</td><td>Rectangular</td><td>0.6×0.9</td><td>50</td></tr>
<tr><td>BS</td><td>1.7-2.99</td><td>200-400</td><td>2</td><td>Circular</td><td>Φ 1.1</td><td>50</td></tr>
<tr><td>BD</td><td>≥3</td><td>200-400</td><td>2</td><td>Circular</td><td>Φ 1.1</td><td>60</td></tr>
<tr><td rowspan="2">CS</td><td rowspan="2">1.7-3.24</td><td>200-400</td><td>3</td><td>Circular</td><td>Φ 1.5</td><td>60</td></tr>
<tr><td>450-700</td><td>Not limited</td><td>Circular</td><td>Φ 1.5</td><td>100</td></tr>
<tr><td rowspan="2">CD</td><td rowspan="2">≥3.25</td><td>200-400</td><td>3</td><td>Circular</td><td>Φ 1.5</td><td>60</td></tr>
<tr><td>450-700</td><td rowspan="10">Not limited</td><td>Circular</td><td>Φ 1.5</td><td>100</td></tr>
<tr><td>DS</td><td>2.2-3.24</td><td>800-1000</td><td>Circular</td><td>Φ 2.0</td><td>120</td></tr>
<tr><td>DD</td><td>≥3.25</td><td>800-1000</td><td>Circular</td><td>Φ 2.0</td><td>120</td></tr>
<tr><td>ES</td><td>2.4-3.49</td><td>1200-1500</td><td>Rectangular</td><td>1.5×2.5</td><td>150</td></tr>
<tr><td>ED</td><td>≥3. 5</td><td>1200-1500</td><td>Rectangular</td><td>1.5×2.5</td><td>150</td></tr>
<tr><td>FS</td><td>2.8-3.49</td><td>1600-2000</td><td>Rectangular</td><td>2.0×3.0</td><td>200</td></tr>
<tr><td>FD</td><td>≥3. 5</td><td>1600-2000</td><td>Rectangular</td><td>2.0×3.0</td><td>200</td></tr>
<tr><td>GD</td><td>≥4. 5</td><td>2200-2500</td><td>Rectangular</td><td>2.5×3.5</td><td>300</td></tr>
<tr><td>HD</td><td>≥4. 5</td><td>2600-3000</td><td>Rectangular</td><td>3.0×4.0</td><td>300</td></tr>
</table>

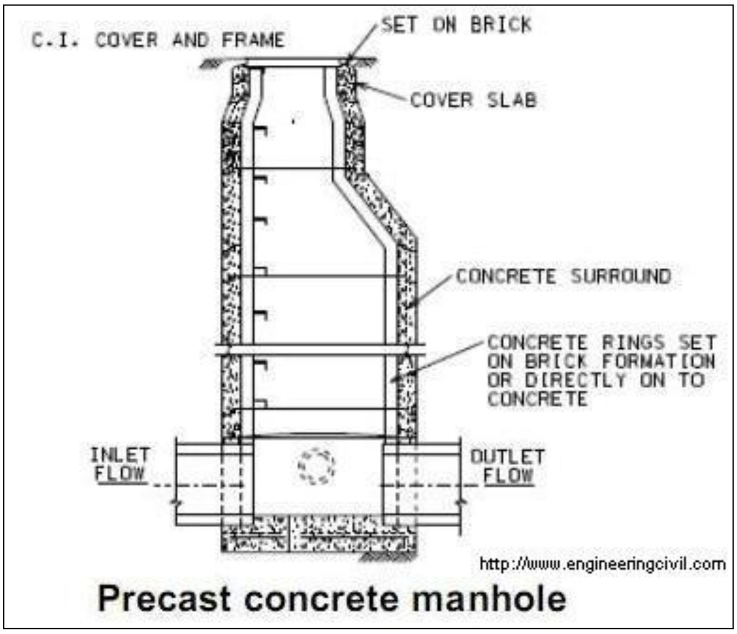


Fig. (2-3) Vertical profile in a manhole

3. Gully Inlets

Gully inlets (or catch basins) are inlets where surface water from roads and paved areas are entering the sewer system. Gullies consist of a grating and usually an underlying sump to collect heavy material in the flow. A water seal is incorporated to act as an odor trap for those gullies connected to combined sewers. Gullies are connected to the sewer by lateral pipes, see Fig.(2-4).

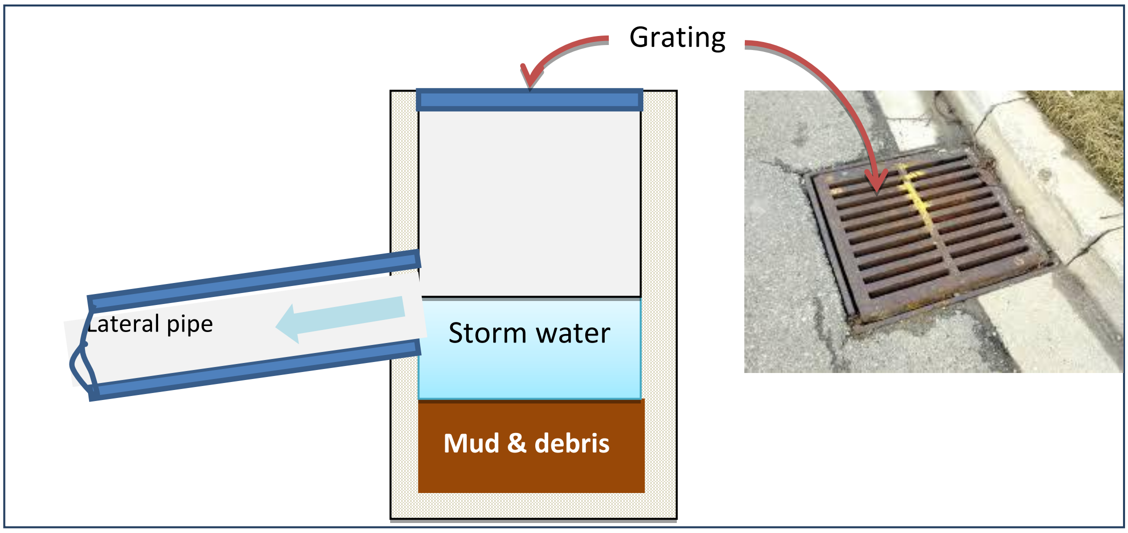


Fig. (2-4) Vertical profile in a catch basin

4. **Ventilation**

It is important to have adequate air ventilation in all urban drainage systems, but particularly in combined sewers. It is needed to ensure that aerobic conditions are maintained within the pipe, and to avoid the possibility of build-up of toxic or explosive gases (H_2S and CH_4). Ventilation columns are placed at spacing of 150 to 300m. They are also provided at the upper end of every branch sewer and at every point where sewer diameter changes. Sewer systems can be ventilated using perforated manhole covers, through which the sewer gets exposed to the atmosphere. This will help in achieving some ventilation.

2.3 Flow in sewers

In spite of sewer system type, most sewers are designed to be under partial flow condition as open channel (flow by gravity) and not under pressure, even though they may flow full at some times. There are exceptions, such as inverted siphon and discharge lines from sewage pumping stations which are always under pressure. Occasionally the capacity of storm sewers will be overloaded, inlets will be full and overflowing, and water will rise in manholes. Sewers in such condition are said to be surcharged. Sanitary sewers may also be surcharged by excessive inflow during storms, by stoppages in the sewers or by flows greater than those designed for.

2.4 Flow Formula

Sanitary and storm gravity sewers are designed to provide design flow capacity, without surcharging, using Manning's formula;

$$V = \frac{1}{n} R^{2/3} S^{1/2} \qquad \textbf{...(2-1)}$$

Where;

V= flow velocity, m/sec.

n= Manning roughness coefficient.

R= hydraulic radius, m=A/P

A= cross sectional area of flow, m^2.

P= wetted perimeter, m

S= slope of sewer which is the slope or gradient of its invert.

For full flow condition, R=D/4, then Eq.(2.1) is written as;

$$V_{full} = \frac{1}{n} (\frac{D}{4})^{2/3} S^{1/2} \qquad \textbf{...(2-2)}$$

and the flowrate at full flow condition is;

$$\boldsymbol{Q_{full} = V_{full} \times \frac{\pi D^2}{4}} \qquad \textbf{...(2-3)}$$

The values of Manning roughness coefficient (n) is depending on pipe material. They vary on the range 0.009 (for plastic pipes) to 0.017 (for tuberculated ductile iron pipe). However, the most applied n value is 0.013.

Example 2.1

A sanitary sewer is used to carry 0.08m^3/sec when flowing full and conditions such as minimum flow velocity must be applied. Determine the commercial pipe size that must be used and the grade (slope).

Solution:

$$\boldsymbol{Q_{full} = V_{full} \times \frac{\pi D^2}{4}}$$

Minimum flow velocity for sanitary sewer= 0.6m/sec

$0.08 = 0.6 \times \frac{\pi D^2}{4} \rightarrow \quad D = 0.412m = 412mm$

Use commercial pipe diameter of 450mm

$$V_{full} = \frac{1}{n}\left(\frac{D}{4}\right)^{\frac{2}{3}} S^{\frac{1}{2}}$$

Put v= 0.6m/sec and find S;

$$0.6 = \frac{1}{0.013}\left(\frac{0.45}{4}\right)^{\frac{2}{3}} S^{\frac{1}{2}} \rightarrow S = 1.12 \times 10^{-3}$$

Hint: Commercial pipe diameters in mm:

100, 150, 200, 250, 300, 350, 400, 450, 500, 600, 700, 800, 900, 1000, 1200, 1400, 1600, 1800, 2000, ….See pipes manufacturer catalogues for other available sizes.

2.5 Required Flow Velocity

An important issue in sewers design is the flow velocity in sewers. Experience showed that a minimum flow velocity of 0.6m/sec for full flow condition is required in sanitary sewer to prevent the settlement of sewage solids. This minimum flow velocity value is called self-cleansing velocity. In storm sewers greater velocities are required than in sanitary sewers because of the heavy sand which washed into them. The minimum allowable velocity is 0.75m/sec and 0.9m/sec is desirable.

Because of the abrasive character of the solids, excessively high velocity should be avoided, 2.4 m/sec being considered as the maximum allowable flow velocity in storm and sanitary sewers.

2.7 Slopes of Sewers

The minimum allowable slopes of sewers are those which give self-cleansing

flow velocities which are 0.6m/sec and 0.75m/sec for sanitary and storm sewers, respectively when sewer is flowing full. Greater slopes should be used if they compatible with existing topography. While the maximum slopes are those which give the maximum flow velocity of 2.4m/sec. design slopes are selected to be between the minimum and maximum slopes.

For different commercial pipe sizes, the minimum and maximum sewer slope values for sanitary sewers are given in Table (2-2). The slopes are those produce minimum and maximum full flow velocities of 0.6 m/sec and 2.4m/sec respectively, based on Manning's formula using n of 0.013 and the pipe flowing full.

H.W.: Develop a table similar to Table (2-1), but for storm sewers.

Table (2-2) Minimum and maximum slopes of sanitary sewers

Sewer Size (mm)	Minimum Slope (%)	Maximum Slope (%)
300	0.192	3.078
350	0.157	2.506
400	0.131	2.097
450	0.112	1.792
500	0.097	1.558
600	0.076	1.221
700	0.062	0.994
800	0.052	0.832
900	0.044	0.711
1000	0.039	0.618
1100	0.034	0.544
1200	0.030	0.485
1300	0.027	0.436
1400	0.025	0.395

2.8 Partial Flow Diagram

To determine the velocity and depth of sewage in a pipe which is flowing only partial full, the diagram shown in Fig. (2-5) is used. In this figure, the x-axis represents the ratios of actual to full flow velocities (v/V_{full}) and actual to full flow rates (q/Q_{full}) and the y-axis represents the ratio of sewage depth to sewer diameter (d/D). In using this diagram to find the partial flow velocity and flow depth it is necessary to find the full flow velocity and full flow rate using Manning formula.

To find the flow depth and actual flow velocity, specify the actual flow rate to full flow rate ratio on x-axis and follow it upward to the discharge curve and from the point of intersection read the ordinate on the y-axis which gives the ratio of flow depth to diameter. From this ratio the flow depth can be determined for a given diameter. Then from the specified d/D ratio draw a line to the right until it intersects the velocity curve and read the abscissa to find v/V_{full}, then find v.

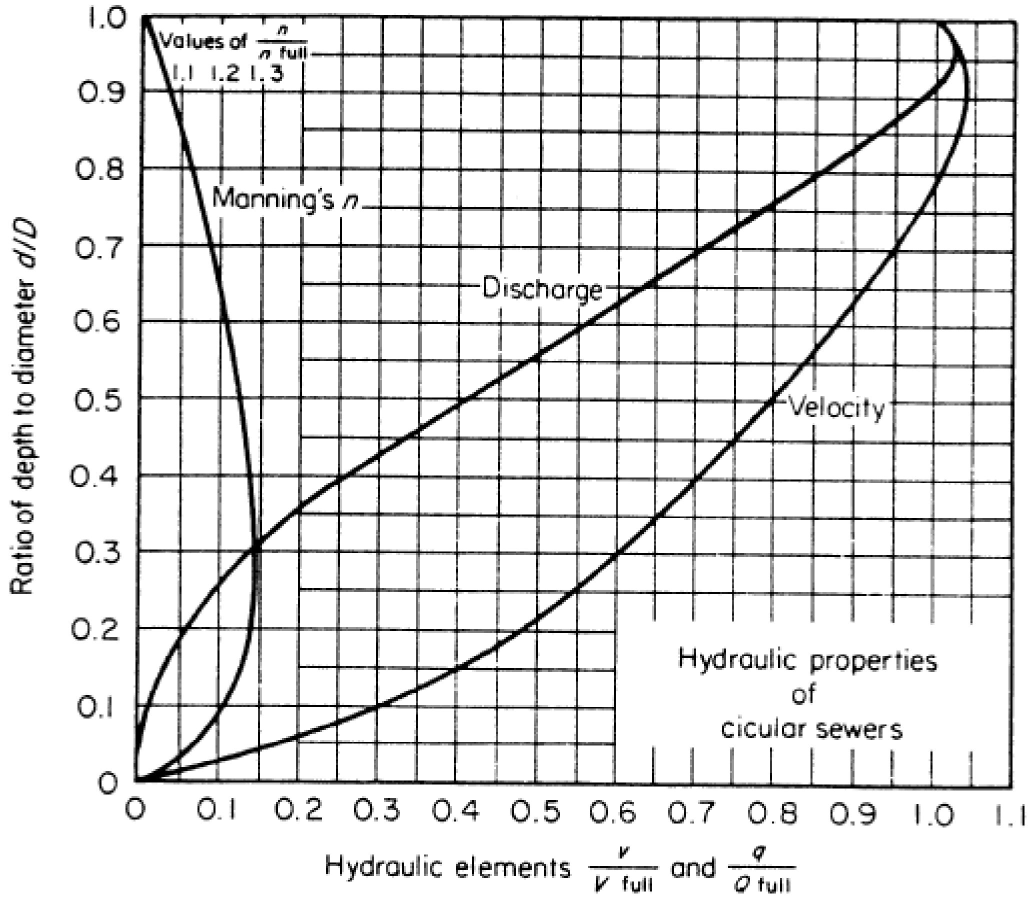


Fig. (2-5) Hydraulic elements of circular sewers

2.9 Maximum Flow Depth

Maximum allowable depth of flow in sanitary and storm sewers at peak flows is as follows:

Sewer diameter	Flow depth/pipe diameter
250 or smaller	0.50
300 and larger	0.75

Example 2.2

A sewer of 500mm diameter is laid on a slope of 0.002. What will be the actual flow velocity and depth if the sewer is carrying sewage at a flow rate of 0.08m^3/sec.

Solution:

$$V_{full} = \frac{1}{n}\left(\frac{D}{4}\right)^{2/3} S^{1/2}$$

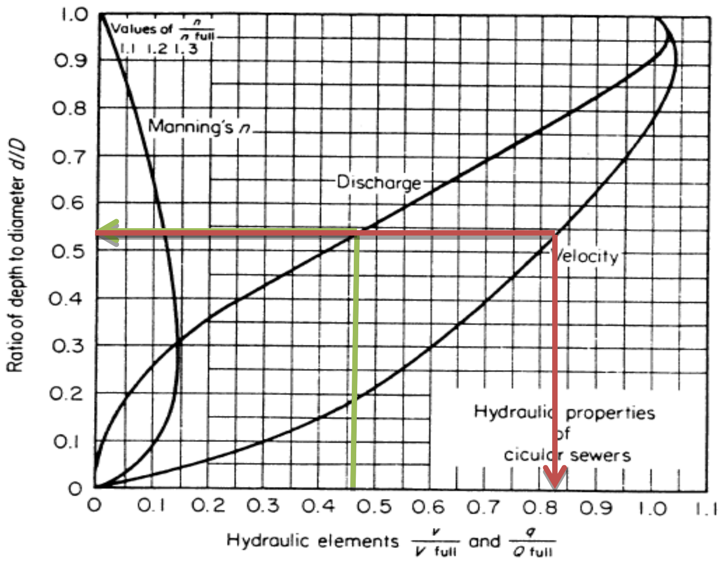
Assume n=0.013

$$V_{full} = \frac{1}{0.013} \times \left(\frac{0.5}{4}\right)^{2/3} \times 0.002^{1/2} = 0.86\, m/sec$$

$$\boldsymbol{Q_{full} = V_{full}} \times \frac{\boldsymbol{\pi D^2}}{\boldsymbol{4}} = 0.86 \times \frac{\boldsymbol{\pi \times 0.5^2}}{\boldsymbol{4}} = 0.17 m^3/sec$$

$$\frac{q}{Q_{full}} = \frac{0.08}{0.17} = 0.47$$

From Fig. (2-5) → $\frac{d}{D}$=0.54



$$\therefore d = 0.54 \times 0.5 = 0.27m$$

From Fig. (2.5), for d/D=0.54→v/V_{full}=0.83

$$v = 0.83 \times 0.86 = 0.71 m/sec$$

Example 2.3

A storm sewer carries storm water at a flow rate of 0.12m^3/sec. Check whether a sewer size of 500mm placed at a grade of 0.002 is suitable.

Solution

Minimum flow velocity in storm sewer at full flow condition= 0.75m/sec.

Check flow velocity at full flow condition;

$$V_{full} = \frac{1}{n}\left(\frac{D}{4}\right)^{\frac{2}{3}} S^{\frac{1}{2}}$$

$V_{full} = \frac{1}{0.013}\left(\frac{0.5}{4}\right)^{\frac{2}{3}} 0.002^{0.5} \rightarrow V_{full} = 0.86m/sec > 0.75m/sec.$ O.K.

Check d/D ratio;

$$\boldsymbol{Q_{full} = V_{full} \times \frac{\pi D^2}{4}}$$

$$\boldsymbol{Q_{full} = 0.86 \times \frac{\pi \times 0.5^2}{4}} = 0.1689m^3/sec$$

q/Q_{full}=0.12/0.1689=0.71

From Fig.(2-5), d/D= 0.7 <0.75…..O.K.

Then, the sewer is suitable to carry storm water at a flow rate of 0.12m/sec.

Ch.3

Design of Sewer Systems

3.1 Design of Sanitary Sewer System

Sanitary sewer system consists of pipes and manholes intended to carry either domestic or industrial wastewater or both. The following sections govern the design of sanitary sewer system.

3.1.1 Layout of Sewer System

Sanitary sewer mains are located in the street along the centerline whenever possible and must not be located underneath the sidewalk or along the curb and gutter. In order to align sanitary sewers, the followings are required:

- Master plan of the project area shows the streets with names and dimensions and land use areas labeled as single family residential, multi-family residential, commercial, industrial, schools, parks, open space, streams, etc...
- Topographic map of the project area showing the distribution of ground surface elevations.

3.1.2 Estimation of Sewer Design Flowrate

The design flowrate of each sewer line is estimated based on:

- Actual number of houses and public buildings connected to that sewer.
- Number of individuals: (a) 10 /house ; (b) 500/school
- Design sewage flow= Peak sewage flow+ Infiltration

For example, if a sewer line receives sanitary sewage from 12 houses, then, the design flowrate is;

$$Q_{design} = 12 \times 10 \times Per\ capita\ average\ sewage\ flow\ \times peak\ factor + infiltration$$

3.1.3 Minimum Sewer Size

The size of a sewer pipe is defined as the inside diameter of the pipe. Many American standards specified the minimum size of sanitary sewer to be 200mm in residential areas and 250mm in commercial and industrial areas. However, Basrah Sewerage Directorate specified the minimum size of sanitary sewer to be 250mm regardless of area type.

3.1.4 Depth of Cover

Depth of cover refers to how deep the pipeline will be installed, or the depth of backfill (soil) above the top of the pipe, Fig.(3-1). It is dependent on pipe material and can be obtained from pipe manufacturer catalogues. For PVC pipes, as an example, the minimum depth of cover is dependent on class of backfill and varies on the range (0.6-1.2m).

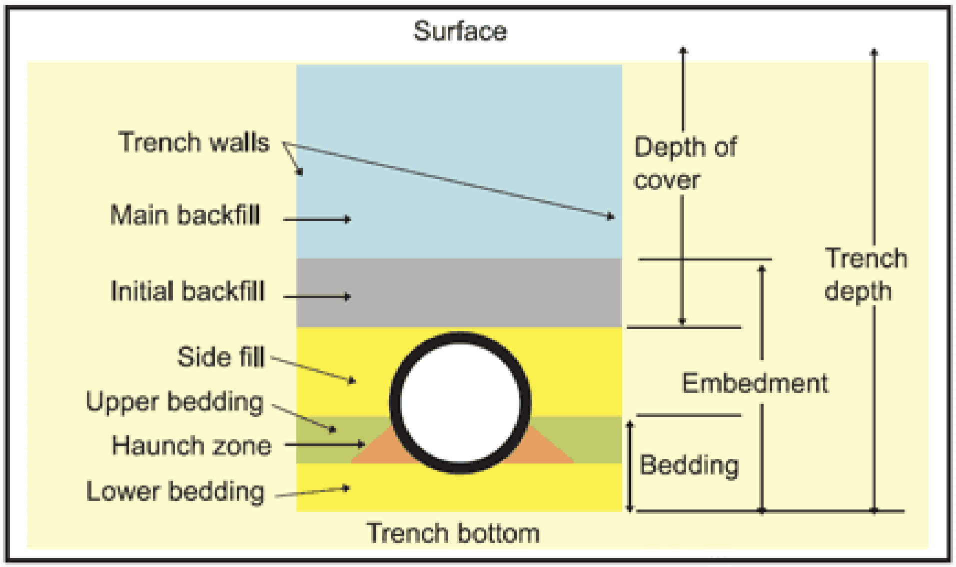


Fig.(3-1) Trench details

3.1.5 Design Slopes

Sewer slope is the difference in elevation at each end of the pipe divided by the horizontal length of the pipe, and it is a constant value between two manholes. The values of minimum and maximum slopes for sanitary sewers are as given in Table (2.2). For any sewer size, the design slope is selected to be greater than the minimum slope and lower than the maximum slope. For example, the design slope of sanitary sewer has a diameter of 500 mm shall be;

$$0.097\% < Design\ slope < 1.558\%$$

3.1.6 Manholes

A- Location

Manholes are provided at the following locations:

1. Termination of existing and future lines.
2. Changes in sewer direction, horizontal or vertical.
3. Changes in sewer size.
4. Junctions with other sewers.
5. At service connections of 200 or larger.
6. At maximum spacing (Access spacing) not exceeding the values given in Table (2-1).

B- **Minimum Drop across Manholes**

Drop across a manhole is the difference in invert levels between the sewer incoming to the manhole and the sewer outgoing from the manhole.

1. For the same size pipes with a change in alignment of 45° or less, no drop is required, see Fig.(3-2a);

$$IL\ of\ outgoing\ sewer = IL\ of\ incoming\ sewer$$

For the same size pipe with a change in alignment of greater than 45, a 6cm drop is required, see Fig.(3-2b);

$$IL\ of\ outgoing\ sewer = \ IL\ of\ incoming\ sewer - 6cm$$

2. For junction of two incoming sewers of the same size, a 6cm drop is required, see Fig.(3-2c);

$$IL\ of\ outgoing\ sewer = \ min.IL\ of\ incoming\ seweres - 6cm$$

For junction of three or more incoming sewers of the same size, a 9cm drop is required, see Fig.(3-2d);

$$IL\ of\ outgoing\ sewer = min\ IL\ of\ incoming\ sewers - 9cm$$

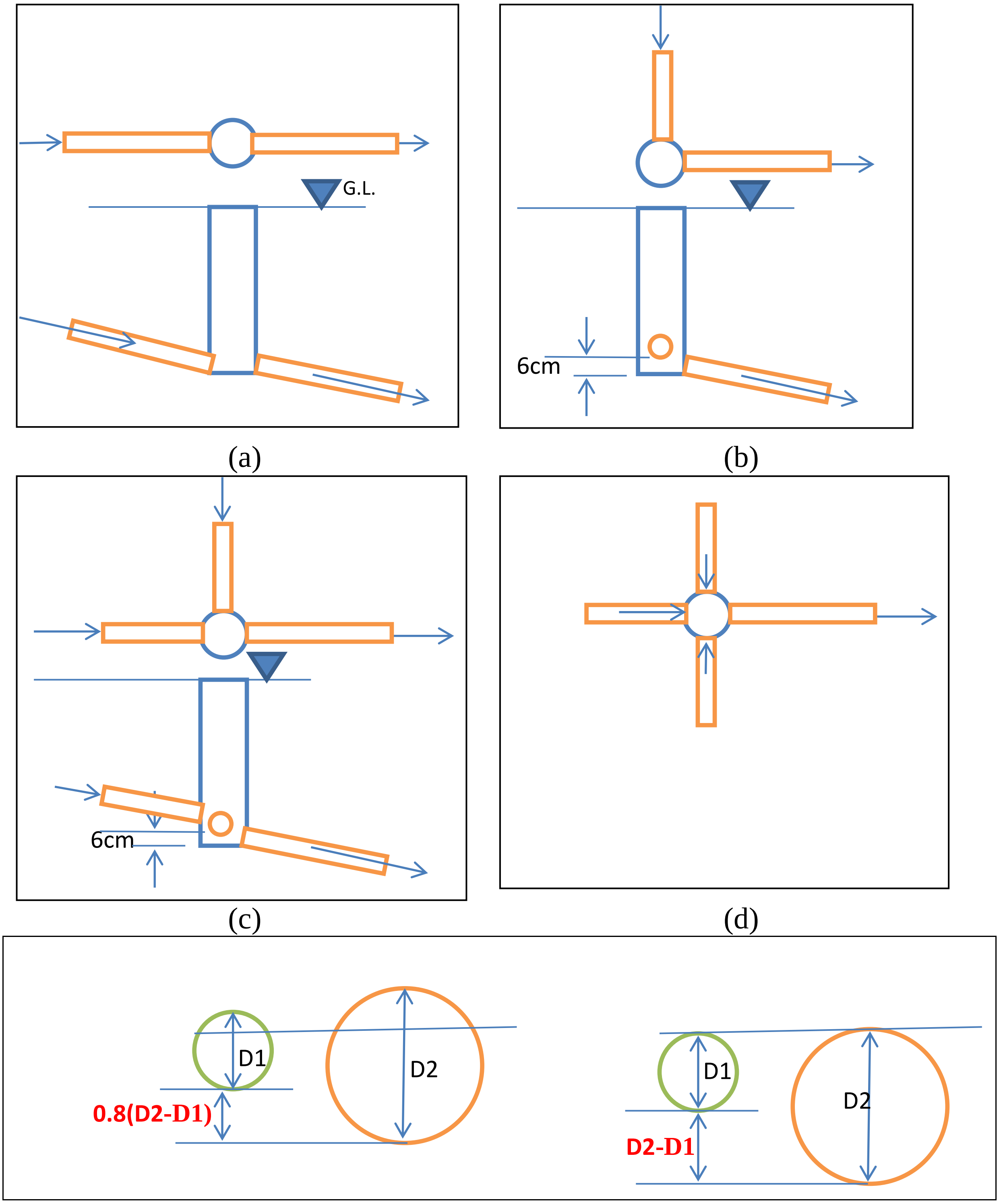
When a smaller sewer joins a larger one, the invert of the larger sewer shall be lowered sufficiently to match the 0.8 depth point of both sewers at the same elevation. Or the sewers are connected crown to crown, i.e., the drop across the manhole equals to the difference in sewers diameters, see Fig.(3-2e).

If the incoming sewer has a diameter of D1 and the outgoing sewer has a diameter of D2, then;

$$IL\ of\ outgoing\ sewer = IL\ of\ incoming\ sewers - 0.8(D2 - D1)$$

Or;

$$IL\ of\ outgoing\ sewer = \ IL\ of\ incoming\ sewers - (D2 - D1)$$

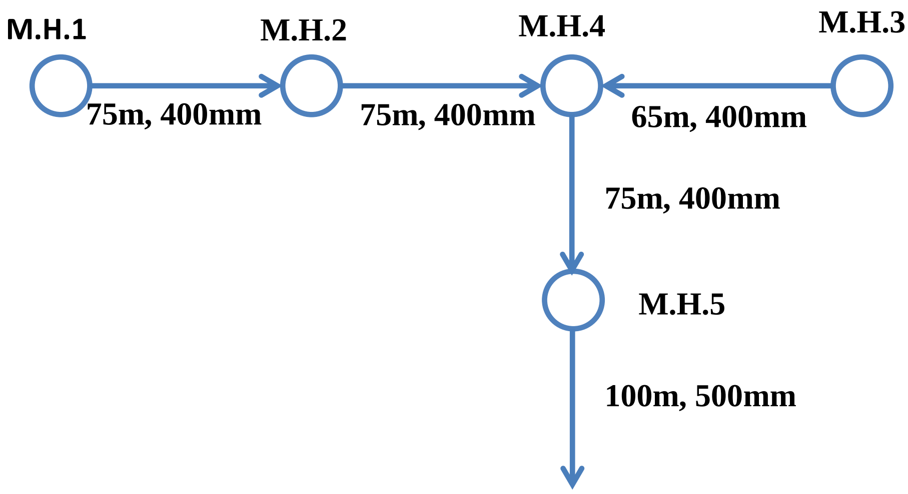


(e)

Fig. (3-2) Drop across manhole

Example 3-1

For the following sewer system, find the invert level (IL) of the sewer outgoing from Manhole No.5 and the depth of M.H.5. Assume minimum soil cover of 1m, the design slope for a sewer of 400mm diameter is 0.0018 and the ground level is 2.5m.



Solution:

$$IL\ of\ outgoing\ sewer\ from\ M.H.1$$
$$= ground\ level - soil\ cover - sewer\ diameter$$
$$IL\ of\ outgoing\ sewer\ from\ M.H.1 = 2.5 - 1 - 0.4 = 1.1m$$
$$IL\ of\ incoming\ sewer\ to\ M.H.2$$
$$= IL\ of\ outgoing\ sewer\ from\ M.H.1 - sewer\ length \times sewer\ slope$$
$$IL\ of\ incoming\ sewer\ to\ M.H.2 = 1.1 - 75 \times 0.0018 = 0.965m$$
$$IL\ of\ outgoing\ sewer\ from\ M.H.2 = IL\ of\ incoming\ sewer\ to\ M.H.2 = 0.965m\ (no\ drop\ is\ required)$$

$$IL\ of\ incoming\ sewer\ to\ M.H.4 =$$
$$IL\ of\ outgoing\ sewer\ from\ M.H.2 - sewer\ length \times sewer\ slope$$
$$IL\ of\ incoming\ sewer\ to\ M.H.4 = 0.965 - 75 \times 0.0018 = 0.83m$$
$$IL\ of\ outgoing\ sewer\ from\ M.H.3 = ground\ level - soil\ cover - sewer\ diameter$$
$$IL\ of\ outgoing\ sewer\ from\ M.H.3 = 2.5 - 1 - 0.4 = 1.1m$$
$$IL\ of\ incoming\ sewer\ to\ M.H.4 = IL\ of\ outgoing\ sewer\ from\ M.H.3 - sewer\ length \times sewer\ slope$$
$$IL\ of\ incoming\ sewer\ to\ M.H.4 = 1.1 - 65 \times 0.0018 = 0.983m$$
$$IL\ of\ outgoing\ sewer\ from\ M.H.4 = min\ IL\ of\ incomimg\ sewers - 0.06$$
$$IL\ of\ outgoing\ sewer\ from\ M.H.4 = 0.83 - 0.06 = 0.77m$$
$$IL\ of\ incoming\ sewer\ to\ M.H.5 = 0.77 - 75 \times 0.0018 = 0.635m$$

At M.H.5, we have change in sewers diameter. If crown to crown connection of sewers is made;

$$IL\ of\ outgoing\ sewer\ from\ M.H.5 = IL\ of\ incomimg\ sewers\ to\ M.H.5 - (D2 - D1)$$
$$IL\ of\ outgoing\ sewer\ from\ M.H.5 = 0.635 - (0.5 - 0.4) = 0.535m$$

Manhole depth=ground level- IL of outgoing sewer=2.5-0.535=1.965m

C- Manholes Size

1. For sewers of 450mm diameter or less, manholes shall have a 1.2m inside diameter.
2. For sewers of 400mm and larger, manholes shall have a 1.5m inside diameter.

D- Drop Manhole

Drop manhole (Fig.3-3) shall be provided for manholes with any pipe having a difference in invert elevation more than 0.6m above the invert of the sewers leaving such manholes.

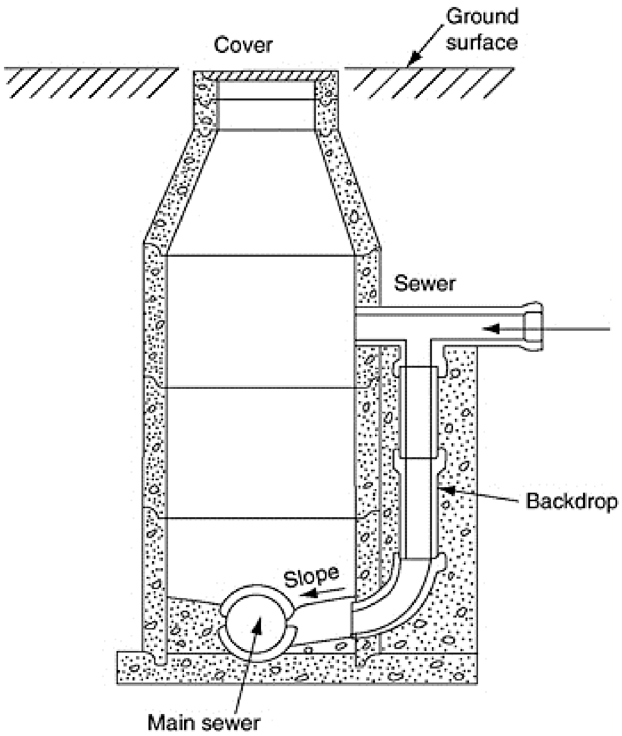


Fig.(3-3) Drop manhole

3.1.7 Service Sewers

Sanitary sewer services shall be a minimum of 150mm diameter and connected to the sewer main with a manufactured wye at a minimum angle of 30 degrees and a maximum angle of 45 degrees. Sanitary sewer services shall be extended to the property line or building at a minimum gradient of 1%. Sanitary sewer service connections to sewer mains greater than 2m in depth shall be constructed with 150mm tee and riser.

3.1.8 Number of Individual House Connections

Each service pipe will be connected to one house.

3.1.9 Design Steps

The steps followed in designing sanitary sewer system are:

1. Layout of sewer system : This step includes;
    a. Selecting the location of lift station (pumping station).
    b. Lay of sewers.
    c. Specifying the locations of manholes.
    d. Numbering of manholes (if the total number of manholes is 250, then, manhole No.1 shall be the most remote manhole from the lift station and manhole No.250 shall be the last manhole before the lift station. The sewer outgoing from manhole No.250 shall be the incoming sewer to the lift station).
2. From the plan of sewer system layout, the followings are obtained;
    a. The length of each sewer line (the line extended between two successive manholes).
    b. The number of service sewers of houses and public buildings connected to each sewer line.

    These data are arranged as in the following table.

Sewer line		No. of service sewers		L (m)
From	To	House	Public building	

3. Calculation of direct sewage inflow into each sewer line, see section (3.1.2). Direct sewage inflow is the flowrate of all service sewers connected to the considered sewer line.
4. Calculation of total sewage inflow into each sewer line (q).

$$Q_{Total} of\ sewer\ line\ extended\ from\ M.H.i\ to\ M.H.j$$
$$= Q_{Direct}\ of\ sewer\ line\ extended\ from\ M.H.i\ to\ M.H.j$$
$$+ Q_{Tributraies}$$

Where;

$$Q_{Tributraies} = \sum Q_{Total}\ of\ all\ incoming\ sewers\ connected\ to\ M.H.i$$

Assume the diameter of each sewer line (starting with D=250mm)

Selection of design slope for each sewer line according to the assumed sewer diameter.

Calculation of flow velocity at full flow condition (V_{full}) for each sewer line using Manning formula.

Calculation of sewage flowrate at full flow condition (Q_{full}) for each sewer line.

Calculation of q to Q_{full} ratio (q/Q_{full}) for each sewer line.

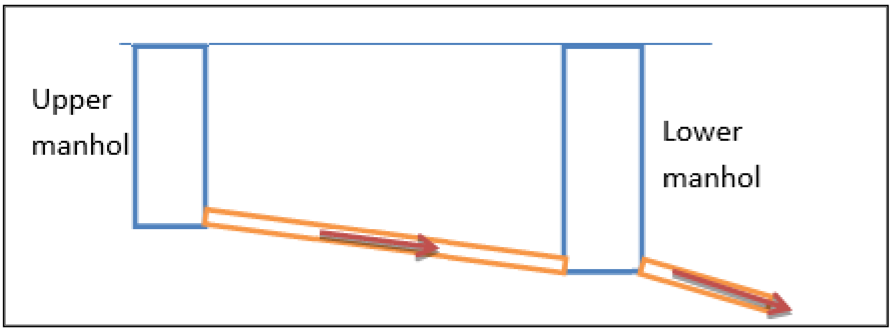
Find flow depth to diameter ratio (d/D) for each sewer line using partial flow diagram.

Check d/D for each sewer line. If d/D >0.5 for D=250mm or d/D>0.75 for D≥300mm, increase the diameter and repeat steps 6 to 11 until the satisfaction of d/D ratio.

Due to repeated calculations in steps 3 to 11 for all the sewer lines, the design of sanitary sewer system can be facilitated using the following table for hydraulic calculations;

Sewer line		Q_{direct} (m^3/day)	$Q_{trib.}$ (m^3/day)	Q_{total} (m^3/day)	D (mm)	Slope (‰)	V_{full} (m/sec)	Q_{full} (m^3/sec)	q/Q_{full}	d/D
From	To									

5. Find the invert levels of each sewer line at upper and lower manholes.



6. Arrange the invert levels data as given in the following table;

Sewer Line		Length (m)	D (mm)	Slope (‰)	Ground Level (m)		Invert Level (m)	
From	To				Upper	Lower	Upper	Lower

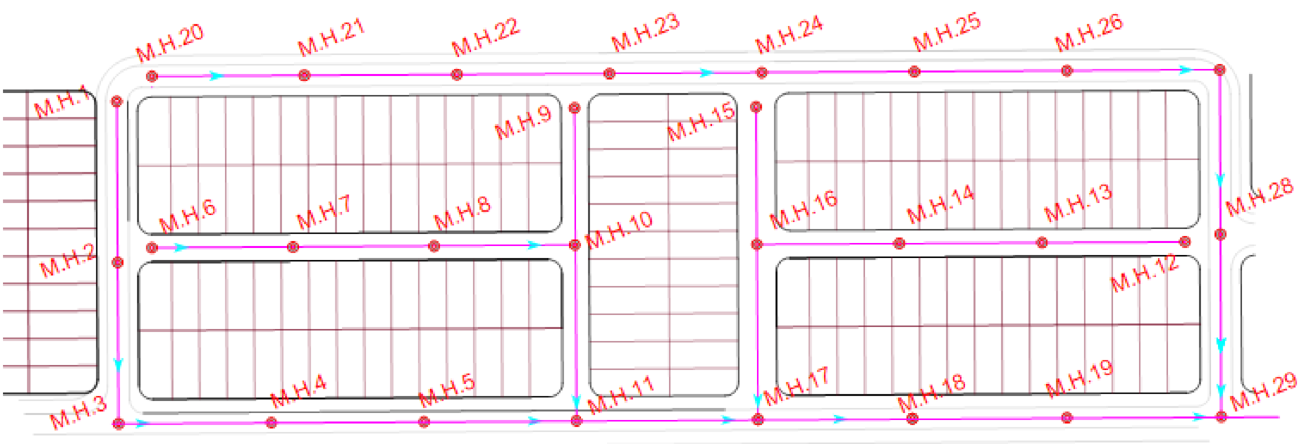
7. Find the depth of each manhole

$$depth\ of\ M.H.i = \ ground\ level\ at\ location\ of\ M.H.i -$$

$$invert\ level\ of\ outgoing\ sewer\ from\ M.H.i$$

Example 3-2

Find the diameters of sewer lines extended from M.H.1 to M.H.11. Assume the number of individuals in each house is 10. Neglect the infiltration.



Solution

Find the number of service sewers connected to each sewer line;

Sewer line		No. of service sewers		L (m)
From	To	House	Public building	
1	2	6	0	
2	3	5	0	
3	4	5	0	
4	5	5	0	
5	11	5	0	
6	7	10	0	
7	8	10	0	
8	10	10	0	
9	10	5	0	
10	11	6	0	

Sewer line extended from M.H.1 to M.H.2;

Assume average water demand for domestic use=300 liter/capita/day

Assume average sewage flow= average water demand=300 liter/capita/day

Total number of served population= 6×10=60

Average sewage flowrate=60×300/1000=18m^3/day

From Fig.(1.1);

Peak factor=4

$$Peak\ sewage\ flow\ rate\ =\ Peak\ factor \times\ average\ sewage\ flow\ rate$$

$$Peak\ sewage\ flow\ rate\ =\ 4\times\ 18 = 72m^3/day$$

$$Q_{Direct} = 72\ m^3/day$$

$$Q_{Trib.} = 0$$

$$Q_{Total} = Q_{Direct} + Q_{Trib.} = 72 + 0 = 72\ \ m^3/day$$

q=72 m^3/day=8.33×10^{-4} m^3/sec

Assume D=250mm

For D=250mm, S_{min}=0.0025 and S_{max}=0.00392

Assume S=0.0035

$$V_{full} = \frac{1}{n}(\frac{D}{4})^{2/3}S^{1/2}$$

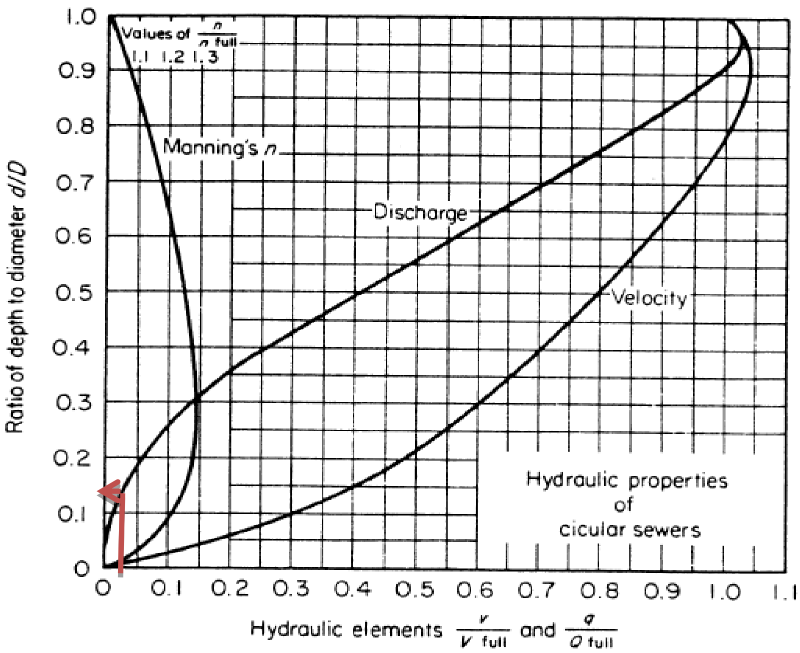
$$V_{full} = \frac{1}{0.013}(\frac{0.25}{4})^{2/3}(0.0035)^{1/2} = 0.72m/sec$$

$$Q_{full} = V_{full} \times \frac{\pi D^2}{4}$$

$$Q_{full} = 0.72 \times \frac{\pi 0.25^2}{4} = 0.0353m^3/sec$$

q/Q_{full}= $8.33\times10^{-4} / 0.0353 = 0.024$

From partial flow diagram;



d/D= 0.14<0.5…..d/D is satisfied (for D=250mm, max d/D=0.5)

∴Sewer diameter=250mm

<u>Sewer line extended from M.H.2 to M.H.3;</u>

Total number of served population= 5×10=50

Average sewage flowrate=50×300/1000=15m^3/day

From Fig.(1.1);

Peak factor=4

$$Peak\ sewage\ flow\ rate\ =\ Peak\ factor \times\ average\ sewage\ flow\ rate$$

$$Peak\ sewage\ flow\ rate\ =\ 4 \times\ 15 = 60m^3/day$$

$$Q_{Direct} = 60\ m^3/day$$

$$Q_{Trib.} = 72\ m^3/day$$

$$Q_{Total} = Q_{Direct} + Q_{Trib.} = 60 + 72 = 132\ \ m^3/day$$

q=132 m^3/day=1.53×10^{-3} m^3/sec

Assume D=250mm

For D=250mm, S_{min}=0.0025 and S_{max}=0.00392

Assume S=0.0035

$$V_{full}\ = 0.72m/sec$$

$$Q_{full} = 0.0353m^3/sec$$

q/Q_{full}= 1.53×10^{-3} / $0.0353 = 0.043$

From partial flow diagram;

d/D=0.18

d/D= 0.18<0.5…..d/D is satisfied (for D=250mm, max d/D=0.5)

∴Sewer diameter=250mm

<u>Sewer line extended from M.H.3 to M.H.4;</u>

Total number of served population= 5×10=50

Average sewage flowrate=50×300/1000=15m^3/day

From Fig.(1.1);

Peak factor=4

$$Peak\ sewage\ flow\ rate\ =\ Peak\ factor \times\ average\ sewage\ flow\ rate$$

$Peak\ sewage\ flow\ rate\ =\ 4 \times\ 15 = 60m^3/day$

$Q_{Direct} = 60\ m^3/day$

$Q_{Trib.} = 132\ m^3/day$

$Q_{Total} = Q_{Direct} + Q_{Trib.} = 60 + 132 = 192\ \ m^3/day$

q=192 m^3/day=2.22×10^{-3} m^3/sec

Assume D=250mm

Assume S=0.0035

$V_{full}\ = 0.72m/sec$

$Q_{full} = 0.0353m^3/sec$

q/Q$_{full}$= 2.22×10^{-3} / $0.0353 = 0.063$

From partial flow diagram;

d/D=0.25

d/D= 0.2<0.5…..d/D is satisfied (for D=250mm, max d/D=0.5)

∴Sewer diameter=250mm

<u>Sewer line extended from M.H.4 to M.H.5;</u>

Total number of served population= 5×10=50

Average sewage flowrate=50×300/1000=15m^3/day

From Fig.(1.1);

Peak factor=4

$Peak\ sewage\ flow\ rate\ =\ Peak\ factor \times\ average\ sewage\ flow\ rate$

$Peak\ sewage\ flow\ rate\ =\ 4 \times\ 15 = 60m^3/day$

$Q_{Direct} = 60\ m^3/day$

$Q_{Trib.} = 192\ m^3/day$

$Q_{Total} = Q_{Direct} + Q_{Trib.} = 60 + 192 = 252\ \ m^3/day$

q=252 m^3/day=2.92×10^{-3} m^3/sec

Assume D=250mm

Assume S=0.0035

$V_{full} = 0.72 m/sec$

$Q_{full} = 0.0353 m^3/sec$

q/Q_{full}= 2.92×10^{-3} / $0.0353 = 0.083$

From partial flow diagram;

d/D=0.25

d/D= 0.25<0.5…..d/D is satisfied (for D=250mm, max d/D=0.5)

∴Sewer diameter=250mm

<u>Sewer line extended from M.H.5 to M.H.11;</u>

Total number of served population= 5×10=50

Average sewage flowrate=50×300/1000=15m^3/day

From Fig.(1.1);

Peak factor=4

$Peak\ sewage\ flow\ rate\ =\ Peak\ factor \times\ average\ sewage\ flow\ rate$

$Peak\ sewage\ flow\ rate\ =\ 4 \times\ 15 = 60 m^3/day$

$Q_{Direct} = 60\ m^3/day$

$Q_{Trib.} = 252\ m^3/day$

$Q_{Total} = Q_{Direct} + Q_{Trib.} = 60 + 252 = 312\ \ m^3/day$

q=312 m^3/day=3.61×10^{-3} m^3/sec

Assume D=250mm

Assume S=0.0035

$V_{full} = 0.72 m/sec$

$Q_{full} = 0.0353m^3/sec$

q/Q$_{full}$= 3.61×10^{-3} / $0.0353 = 0.1$

From partial flow diagram;

d/D=0.26

d/D= 0.26<0.5…..d/D is satisfied (for D=250mm, max d/D=0.5)

∴Sewer diameter=250mm

<u>Sewer line extended from M.H.6 to M.H.7;</u>

Total number of served population= 10×10=100

Average sewage flowrate=100×300/1000=30m^3/day

From Fig.(1.1);

Peak factor=4

$Peak\ sewage\ flow\ rate\ =\ Peak\ factor \times\ average\ sewage\ flow\ rate$

$Peak\ sewage\ flow\ rate\ =\ 4\times\ 30 = 120m^3/day$

$Q_{Direct} = 120\ m^3/day$

$Q_{Trib.} = 0$

$Q_{Total} = Q_{Direct} + Q_{Trib.} = 120 + 0 = 120\ \ m^3/day$

q=$120\ \ m^3/day$=1.39×10^{-3} m^3/sec

Assume D=250mm

Assume S=0.0035

$V_{full}\ = 0.72m/sec$

$Q_{full} = 0.0353m^3/sec$

q/Q$_{full}$= 1.39×10^{-3} / $0.0353 = 0.04$

From partial flow diagram;

d/D=0.18

d/D= 0.18<0.5…..d/D is satisfied (for D=250mm, max d/D=0.5)

∴Sewer diameter=250mm

Sewer line extended from M.H.7 to M.H.8;

Total number of served population= 10×10=100

Average sewage flowrate=100×300/1000=30m^3/day

From Fig.(1.1);

Peak factor=4

$$Peak\ sewage\ flow\ rate\ =\ Peak\ factor \times\ average\ sewage\ flow\ rate$$

$$Peak\ sewage\ flow\ rate\ =\ 4 \times\ 30 = 120 m^3/day$$

$$Q_{Direct} = 120\ m^3/day$$

$$Q_{Trib.} = 120\ m^3/day$$

$$Q_{Total} = Q_{Direct} + Q_{Trib.} = 120 + 120 = 240\ \ m^3/day$$

q=240 m^3/day=2.78×10^{-3} m^3/sec

Assume D=250mm

Assume S=0.0035

$$V_{full}\ = 0.72 m/sec$$

$$Q_{full} = 0.0353 m^3/sec$$

q/Q_{full}= 2.78×10^{-3} / $0.0353 = 0.08$

From partial flow diagram;

d/D=0.25

d/D= 0.25<0.5…..d/D is satisfied (for D=250mm, max d/D=0.5)

∴Sewer diameter=250mm

Sewer line extended from M.H.8 to M.H.10;

Total number of served population= 10×10=100

Average sewage flowrate=100×300/1000=30m^3/day

From Fig.(1.1);

Peak factor=4

$Peak\ sewage\ flow\ rate\ =\ Peak\ factor \times\ average\ sewage\ flow\ rate$

$Peak\ sewage\ flow\ rate\ =\ 4 \times\ 30 = 120 m^3/day$

$Q_{Direct} = 120\ m^3/day$

$Q_{Trib.} = 120\ \ m^3/day$

$Q_{Total} = Q_{Direct} + Q_{Trib.} = 120 + 240 = 360\ \ m^3/day$

q=360 m^3/day=4.2×10^{-3} m^3/sec

Assume D=250mm

Assume S=0.0035

$V_{full}\ = 0.72 m/sec$

$Q_{full} = 0.0353 m^3/sec$

q/Q_{full}= 4.2×10^{-3} / $0.0353 = 0.12$

From partial flow diagram;

d/D=0.28

d/D= 0.28<0.5…..d/D is satisfied (for D=250mm, max d/D=0.5)

∴Sewer diameter=250mm

<u>Sewer line extended from M.H.9 to M.H.10;</u>

Total number of served population= 5×10=50

Average sewage flowrate=50×300/1000=15m^3/day

From Fig.(1.1);

Peak factor=4

$Peak\ sewage\ flow\ rate\ =\ Peak\ factor \times\ average\ sewage\ flow\ rate$

$Peak\ sewage\ flow\ rate\ =\ 4\times\ 15 = 60m^3/day$

$Q_{Direct} = 60\ m^3/day$

$Q_{Trib.} = 0$

$Q_{Total} = Q_{Direct} + Q_{Trib.} = 60 + 0 = 60\ \ m^3/day$

q=60 m^3/day=6.94×10^{-4} m^3/sec

Assume D=250mm

Assume S=0.0035

$V_{full}\ = 0.72m/sec$

$Q_{full} = 0.0353m^3/sec$

q/Q_{full}= 6.94×10^{-4} / $0.0353 = 0.02$

From partial flow diagram;

d/D=0.13

d/D= 0.13<0.5…..d/D is satisfied (for D=250mm, max d/D=0.5)

∴Sewer diameter=250mm

<u>Sewer line extended from M.H.10 to M.H.11;</u>

Total number of served population= 6×10=60

Average sewage flowrate=60×300/1000=18m^3/day

From Fig.(1.1);

Peak factor=4

$Peak\ sewage\ flow\ rate\ =\ Peak\ factor \times\ average\ sewage\ flow\ rate$

$Peak\ sewage\ flow\ rate\ =\ 4\times\ 18 = 72m^3/day$

$Q_{Direct} = 72\ m^3/day$

$Q_{Trib.} = 60 + 360 = 420\ m^3/day$

$Q_{Total} = Q_{Direct} + Q_{Trib.} = 72 + 420 = 492\ \ m^3/day$

q=492 m^3/day=5.69×10^{-3} m^3/sec

Assume D=250mm

Assume S=0.0035

$V_{full} = 0.72m/sec$

$Q_{full} = 0.0353m^3/sec$

q/Q$_{full}$= 5.69×10^{-3} / $0.0353 = 0.16$

From partial flow diagram;

d/D=0.32

d/D= 0.32<0.5…..d/D is satisfied (for D=250mm, max d/D=0.5)

∴Sewer diameter =250mm

The table below shows the hydraulic calculations of the considered part of sanitary sewer system.

Sewer line		Q_{direct} (m^3/day)	$Q_{trib.}$ (m^3/day)	Q_{total} (m^3/day)	D (mm)	Slope (‰)	V_{full} (m/sec)	Q_{full} (m^3/sec)	q/Q_{full}	d/D
From	To									
1	2	72	0	72	250	3.5	0.72	0.0353	0.024	0.14
2	3	60	72	132	250	3.5	0.72	0.0353	0.043	0.18
3	4	60	132	192	250	3.5	0.72	0.0353	0.063	0.2
4	5	60	192	252	250	3.5	0.72	0.0353	0.083	0.25
5	11	60	252	312	250	3.5	0.72	0.0353	0.1	0.26
6	7	120	0	120	250	3.5	0.72	0.0353	0.04	0.18
7	8	120	120	240	250	3.5	0.72	0.0353	0.08	0.25
8	10	120	240	360	250	3.5	0.72	0.0353	0.12	0.28
9	10	60	0	60	250	3.5	0.72	0.0353	0.02	0.13
10	11	72	420	492	250	3.5	0.72	0.0353	0.16	0.32

3.2 Design of Storm Sewer System

Storm sewer system is designed to carry storm water. It consists of pipes, gully inlets and manholes. The followings govern the design of storm sewer system.

3.2.1 Layout of Sewer System

As in sanitary sewer system, storm sewers are located in the streets. The minimum horizontal separation between storm sewer and sanitary sewer is 1.5m. In order to align storm sewers, master plan and topographic map of the project area are required.

3.2.2 Determination of drainage area

After completing the layout of storm sewer system (Fig.3-4), the drainage districts tributary to each sewer line (pipeline extended between two successive manholes) are sketched, see Fig.(3-5). The areas of the drainage districts are computed from the map and written on the table of hydraulic calculations.

3.2.3 Estimation of Storm Sewer Design Flowrate

The design flowrate of each sewer line is obtained using the rational formula as described in Ch.1;

$$Q_{Total} of\ sewer\ line\ extended\ from\ M.H.i\ to\ M.H.j = CiA + infiltration$$

Where; A is the total surface area of drainage districts from which the storm water goes to storm sewer line extended from M.H.i to M.H.j. A is obtained as;

$$A = A_{direct} + A_{indirect}$$

$$A_{direct} = drainage\ \ area\ from\ which\ the\ storm\ water\ goes\ to\ gully\ inlets\ connected\ to\ M.H.i$$

$$A_{indirect} = summation\ of\ drainage\ areas\ from\ which\ the\ storm\ water\ goes\ to\ all\ incoming\ sewers\ connected\ to\ M.H.i$$

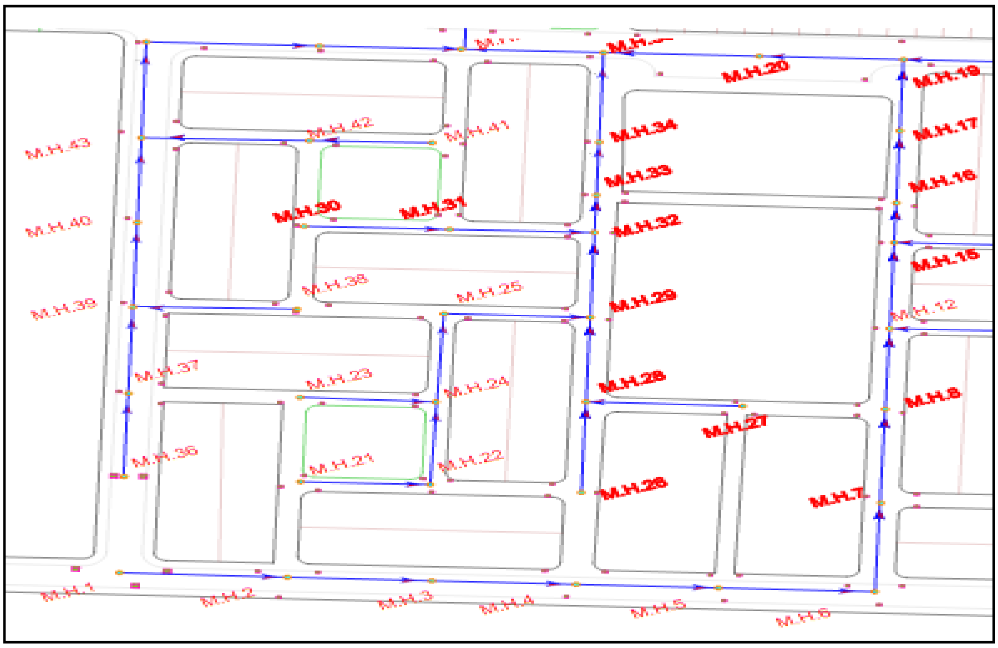


Fig.(3-4) Layout of storm sewer system

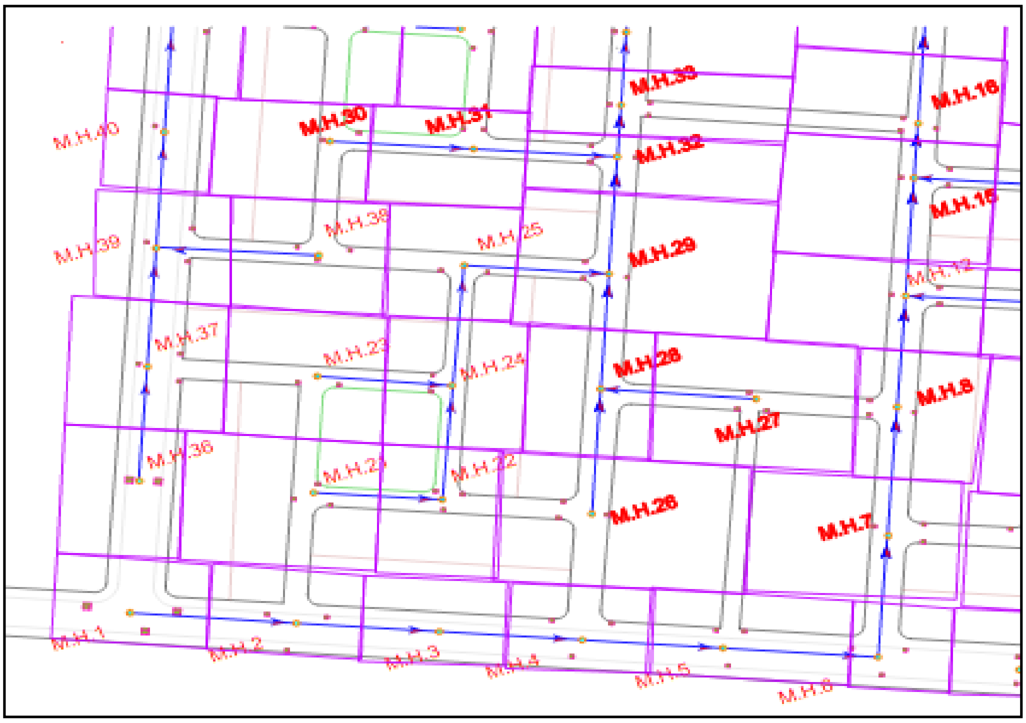


Fig. (3-5) Sketching of drainage districts

3.2.4 Minimum Sewer Size

The minimum size of storm sewer is 300mm.

3.2.5 Depth of Cover

As in sanitary sewer system.

3.2.6 Design Slopes

Sewer slope is the difference in elevation at each end of the pipe divided by the horizontal length of the pipe, and it is a constant value between two manholes. The values of minimum and maximum slopes for storm sewers are those give flow velocities of 0.75m/sec and 2.4m/sec, respectively. They are given in Table (3.1). For any sewer size, the design slope is selected to be greater than the minimum slope and lower than the maximum slope.

Table (3-1) Minimum and maximum slopes for storm sewers

Sewer Size (mm)	Minimum Slope (%)	Maximum Slope (%)
300	0.301	3.078
350	0.245	2.506
400	0.205	2.097
450	0.175	1.792
500	0.152	1.558
600	0.119	1.221
700	0.097	0.994
800	0.081	0.832
900	0.069	0.711
1000	0.060	0.618
1100	0.053	0.544
1200	0.047	0.485
1300	0.043	0.436
1400	0.039	0.395

3.2.7 Manholes

Manholes spacing and drop across manholes and when one must use drop manhole are as described in sanitary sewer system.

3.2.8 Gully Inlet Spacing

Gully inlets are placed at the gutters, usually at street corners and at spacing not exceeding 50m, or one gully for every 200 m^2 of impervious areas. In general, gullies spacing can be obtained as;

$$D = \frac{280\sqrt{S}}{W}$$

Where; D is the gully spacing (m), S is the gradient (crossfall of roads) percent, and W is the width of paved area (m).

The average values for crossfall for roads, Fig.(3-6) vary from 1 in 35 to 1 in 50, depending on the type of surfacing.

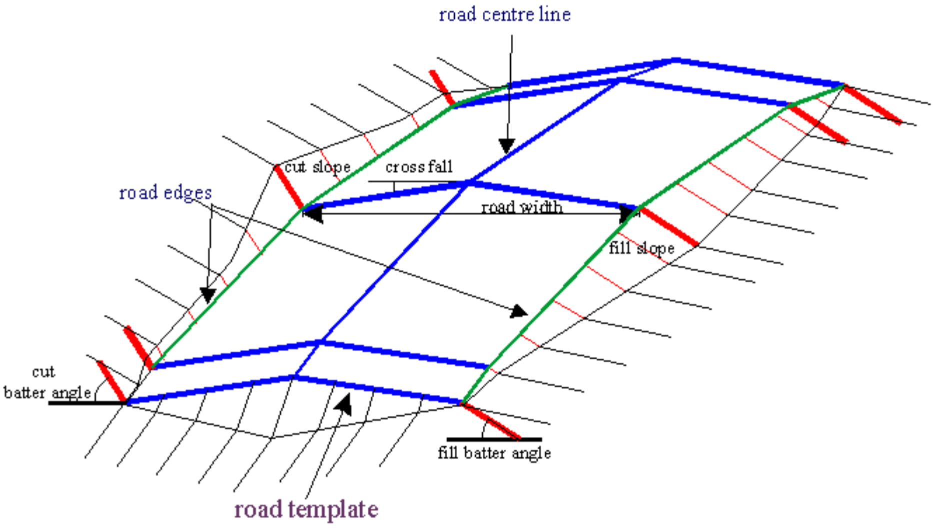


Fig.(3-6) Crossfall of a road

The short branches needed to connect the inlets to the main sewer may all enter at a manhole (Fig.3-7) or may enter through Wye connection at the nearest points.

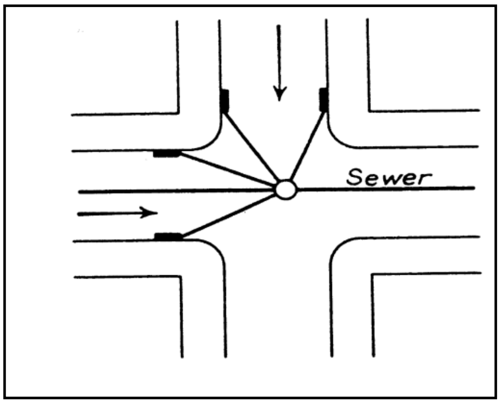


Fig (3-7) Gully inlets at street corner and the short branches enters a manhole.

3.1.8 Design Steps

The steps followed in designing storm sewer system are:

1. Layout of sewer system: This step includes;
    a. Selecting the location of lift station (pumping station).
    b. Lay of sewers.
    c. Specifying the locations of gully inlets.
    d. Specifying the locations of manholes.
    e. Numbering of manholes.
2. From the plan of sewer system layout, the followings are obtained;
    a. The length of each sewer line (the line extended between two successive manholes).
    b. The area of drainage districts from which the storm water goes to the gully inlets connected to the upper manhole of the considered sewer (A_{direct}).
3. Calculation of design flowrate of each sewer line (q).
4. Assume the diameter of each sewer line (starting with D=300mm)
5. Selection of design slope for each sewer line according to the assumed sewer diameter.
6. Calculation of flow velocity at full flow condition (V_{full}) for each sewer line using Manning formula.
7. Calculation of sewage flowrate at full flow condition (Q_{full}) for each sewer line.
8. Calculation of q to Q_{full} ratio (q/Q_{full}) for each sewer line.
9. Find flow depth to diameter ratio (d/D) for each sewer line using partial flow diagram.
10. Check d/D for each sewer line. If $d/D>0.75$ for $D\geq300$mm, increase the diameter and repeat steps 6 to 11 until the satisfaction of d/D ratio.

11.Find the invert levels of each sewer line at upper and lower manholes. Arrange the invert levels data as given in the following table;

Sewer Line		**Length (m)**	**D (mm)**	**Slope (‰)**	**Ground Level (m)**		**Invert Level (m)**	
From	**To**				**Upper**	**Lower**	**Upper**	**Lower**

12.Find the depth of each manhole

$$depth\ of\ M.H.i = \ ground\ level\ at\ location\ of\ M.H.i -$$

$$invert\ level\ of\ outgoing\ sewer\ from\ M.H.i$$

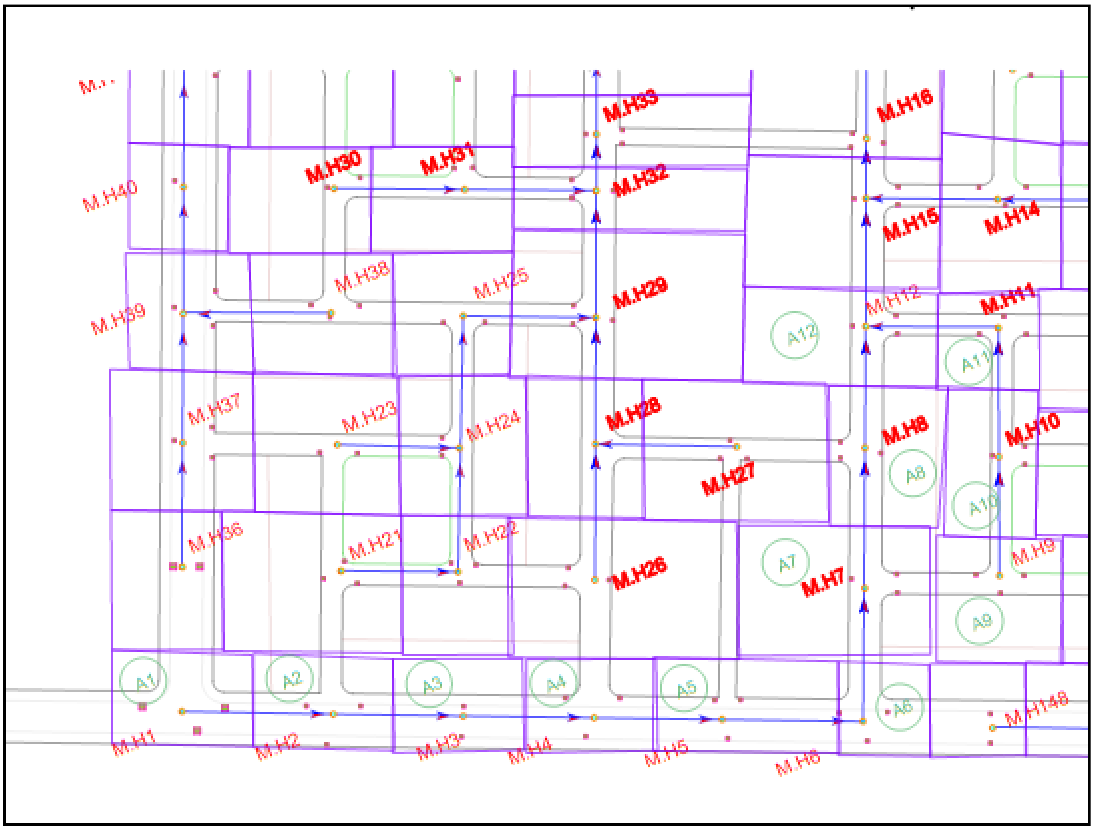
Due to repeated calculations in steps 4 to 11 for all the sewer lines, the design of storm sewer system can be facilitated using the following table.

Sewer line		**L**	**Area (m^2)**			**D**	**S**	**V_{full} (m/sec)**	**Q_{full} (m^3/sec)**	**t_f (min.)**	**t_o (min.)**	**t_c (min.)**	**i (mm/hr)**	**q (m^3/sec)**	**q/Q_{full}**	**d/D**
From	**To**	**(m)**	**Direct**	**Indirect**	**Total**	**(mm)**	**(‰)**									

Example 3-3

For the part of storm sewer system shown below, find the diameter of sewer line extended from M.H.12 to M.H.15. The values of direct drainage area for each sewer line are given in the table below. Assume all sewer lines have a length of 50m, over land flow time for all gully inlets is 10min and runoff coefficient for the residential area is 0.75. Hint: $i = \frac{1000}{t+30}$ and neglect the infiltration.

Drainage district	A1	A2	A3	A4	A5	A6	A7	A8	A9	A10	A11	A12
Area (m^2)	2728	2841	2770	2595	3583	1849	5557	3428	3466	2912	2233	4065



Solution:

To find the diameter of sewer line extended from M.H.12 to M.H.15, we must determine the capacity (Q_{total}) of this sewer line. The storm water flows in this sewer line is collected starting from M.H.1.

For sewer line extending from M.H.1 to M.H.2;

$A = A_{direct} + A_{indirect}$

$A_{direct} = 2728m^2 \quad and \quad A_{indirect} = 0$

$A = 2728 + 0 = 2728m^2$

M.H.1 receives storm water from subarea A1 only;

$t_c = t_o + t_f$; t_o=10min and t_f=0

t_o=10min and t_f=0

$t_c = 10 + 0 = 10min$

$i = \frac{1000}{t+30} = \frac{1000}{10+30}$= 25 mm/hr=0.025 m/hr

$Q_{Total} = CiA$

$Q_{Total} = 0.75 \times 0.025 \times 2728 = 51.15m^3/hr$→q=0.0142$m^3/sec$

Assume D=300mm

For D=300mm, S_{min}=0.003 and S_{max}=0.03078

Assume S=0.004

$V_{full} = \frac{1}{n}(\frac{D}{4})^{2/3}S^{1/2}$

$V_{full} = \frac{1}{0.013}(\frac{0.3}{4})^{2/3}(0.004)^{1/2} = 0.865m/sec$

$Q_{full} = V_{full} \times \frac{\pi D^2}{4}$

$$Q_{full} = 0.865 \times \frac{\pi \times 0.3^2}{4} = 0.0615m^3/sec$$

q/Q_{full}= 0.0142/ 0.0615 = 0.23

From partial flow diagram;

d/D=0.38

d/D= 0.38<0.75…..d/D is satisfied (for D=300mm, max d/D=0.75)

∴Sewer diameter=300mm and V_{full}=0.865m/sec

For sewer line extending from M.H.2 to M.H.3;

$A_{direct} = 2841m^2 \quad and \quad A_{indirect} = 2728m^2$

$A = 2841 + 2728 = 5569m^2$

M.H.2 receives storm water from subarea A2 (direct) and the sewer line incoming to M.H.2, then, t_c for M.H.2 equals t_c for subarea A2 or t_c for the sewer line incoming to M.H.2 whichever the greater.

For subaera A2, t_c=t_o=10min

For the sewer line;

$t_c = t_o + t_f$

t_o=10min

$\boldsymbol{t_f = \sum\frac{L}{V_f} = \frac{50}{0.865}}$ **=57.8sec=0.963min**

$t_c = 10 + 0.963 = 10.963min$

$\therefore \boldsymbol{t_c\, for\, M.H.2 = 10.963min}$

$i = \frac{1000}{t+30} = \frac{1000}{10.963+30}$= 24.41 mm/hr=0.02441 m/hr

$Q_{Total} = 0.75 \times 0.02441 \times 5569 = 101.95\, m^3/hr$ →q=0.0283 m^3/sec

Assume D=300mm and S=0.004

$V_{full} = \frac{1}{0.013}(\frac{0.3}{4})^{2/3}(0.004)^{1/2} = 0.865m/sec$

$Q_{full} = 0.865 \times \frac{\pi \times 0.3^2}{4} = 0.0615m^3/sec$

q/Q_{full}= 0.0283/ 0.0615 = 0.46

From partial flow diagram;

d/D=0.56

d/D= 0.56<0.75…..d/D is satisfied (for D=300mm, max d/D=0.75)

∴Sewer diameter=300 mm and V_{full}=0.865m/sec

For sewer line extending from M.H.3 to M.H.4;

$A_{direct} = 2770m^2 \quad and \quad A_{indirect} = 5569m^2$

$A = 2770 + 5569 = 8339m^2$

M.H.3 receives storm water from subarea A3 (direct) and the sewer line incoming to M.H.3, then, t_c for M.H.3 equals t_c for subarea A3 or t_c for the sewer line incoming to M.H.3 whichever the greater.

For subaera A3, t_c=t_o=10min

For the sewer line;

$t_c = t_o + t_f$

t_o=10min

$t_f = \sum \frac{L}{V_f} = \frac{50}{0.865} + \frac{50}{0.865}$ =115.6sec=1.93min

$t_c = 10 + 1.93 = 11.93min$

$\therefore t_c \; for \; M.H.3 = 11.93min$

$i = \frac{1000}{t+30} = \frac{1000}{11.93+30}$= 23.85 mm/hr=0.02385 m/hr

$Q_{Total} = 0.75 \times 0.02385 \times 8339 = 149.16 \; m^3/hr$ →q=0.0414 m^3/sec

Assume D=300mm and S=0.004

$V_{full} = \frac{1}{0.013}(\frac{0.3}{4})^{2/3}(0.004)^{1/2} = 0.865m/sec$

$Q_{full} = 0.865 \times \frac{\pi \times 0.3^2}{4} = 0.0615m^3/sec$

q/Q$_{full}$= 0.0414/ 0.0615 = 0.67

From partial flow diagram;

d/D=0.69

d/D= 0.69<0.75…..d/D is satisfied (for D=300mm, max d/D=0.75)

∴Sewer diameter=300 mm and V_{full}=0.865m/sec

For sewer line extending from M.H.4 to M.H.5;

$A_{direct} = 2595m^2 \quad and \quad A_{indirect} = 8339m^2$

$A = 2595 + 8339 = 10934m^2$

M.H.4 receives storm water from subarea A4 (direct) and the sewer line incoming to M.H.4, then, t_c for M.H.4 equals t_c for subarea A4 or t_c for the sewer line incoming to M.H.4 whichever the greater.

For subaera A4, t_c=t_o=10min

For the sewer line;

$t_c = t_o + t_f$

t_o=10min

$\boldsymbol{t_f = \sum \frac{L}{V_f} = \frac{50}{0.865} + \frac{50}{0.865} + \frac{50}{0.865}}$ **=173.41sec=2.89min**

$t_c = 10 + 2.89 = 12.89min$

$\boldsymbol{\therefore t_c \; for \; M.H.4 = 12.89min}$

$i = \frac{1000}{t+30} = \frac{1000}{12.89+30}$= 23.32 mm/hr=0.0233 m/hr

$Q_{Total} = 0.75 \times 0.0233 \times 10934 = 191.07 \; m^3/hr$ →q=0.0531 m^3/sec

Assume D=300mm and S=0.004

$V_{full} = \frac{1}{0.013} (\frac{0.3}{4})^{2/3} (0.004)^{1/2} = 0.865m/sec$

$Q_{full} = 0.865 \times \frac{\pi \times 0.3^2}{4} = 0.0615m^3/sec$

q/Q_{full}= 0.0531/ 0.0615 = 0.86

From partial flow diagram;

d/D=0.8

d/D= 0.8>0.75…..d/D is not satisfied (for D=300mm, max d/D=0.75)

∴ increase sewer diameter

*l*et D=350mm and S=0.003

$$V_{full} = \frac{1}{0.013}(\frac{0.35}{4})^{2/3}(0.003)^{1/2} = 0.83m/sec$$

$$Q_{full} = 0.83 \times \frac{\pi \times 0.35^2}{4} = 0.0798m^3/sec$$

q/Q_{full}= 0.0531/ 0.0798 = 0.67

From partial flow diagram;

d/D=0.68

d/D= 0.68<0.75…..d/D is satisfied (for D=350mm, max d/D=0.75)

∴Sewer diameter=350 mm and V_{full}=0.83m/sec

The results of complete calculations are shown in the table below.

The diameter for sewer line extending from M.H.12 and M.H.15 is 600mm

Hydraulic calculations table for storm sewer system design

Sewer line		L (m)	Area (m^2)			D (mm)	S (‰)	V_{full}	Q_{full}	t_f	t_o	t_c	i	q	q/Q_{full}	d/D
From	To		Direct	Indirect	Total			(m/sec)	(m^3/sec)	(min.)	(min.)	(min.)	(mm/hr)	(m^3/sec)		
1	2	50	2728	0	2728	300	4	0.865	0.0611	0.000	10	10.00	25.000	0.0142	0.23	0.38
2	3	50	2841	2728	5569	300	4	0.865	0.0611	0.963	0	10.96	24.412	0.0283	0.46	0.54
3	4	50	2770	5569	8339	300	4	0.865	0.0611	0.963	0	11.93	23.851	0.0414	0.68	0.69
4	5	50	2595	8339	10934	350	3	0.830	0.0799	1.004	0	12.93	23.294	0.0531	0.66	0.68
5	6	50	3583	10934	14517	400	2.7	0.861	0.1082	0.968	0	13.90	22.780	0.0689	0.64	0.66
6	7	50	1849	14517	16366	400	2.7	0.861	0.1082	0.968	0	14.87	22.289	0.0760	0.70	0.70
7	8	50	5557	16366	21923	500	2	0.860	0.1688	0.969	0	15.83	21.818	0.0996	0.59	0.62
8	12	50	3428	21923	25351	500	2	0.860	0.1688	0.969	0	16.80	21.366	0.1128	0.67	0.68
9	10	50	3466	0	3466	300	4	0.865	0.0611	0.000	10	10.00	25.000	0.0181	0.30	0.42
10	11	50	2912	3466	6378	300	4	0.865	0.0611	0.963	0	10.96	24.412	0.0324	0.53	0.58
11	12	50	2233	6378	8611	300	4	0.865	0.0611	0.963	0	11.93	23.851	0.0428	0.70	0.70
12	15	50	4065	33962	38027	600	1.4	0.813	0.2296	1.026	0	17.83	20.908	0.1656	0.72	0.72

Ch.4 Treatment of Sanitary Sewage

4-1 Important Contaminants in Sanitary Sewage

Sanitary sewage must be treated before its final disposal to the environment because it contains many contaminants (pollutants). The important contaminants in sanitary sewage are;

- Suspended solids:

  Suspended solids can increase water turbidity and the development of sludge deposits when untreated sewage is discharged in the aquatic environment.
- Biodegradable organics:

  They composed of proteins, carbohydrates, and fats. Biodegradable organics are measured in terms of BOD (biochemical oxygen demand) and COD (chemical oxygen demand). If they are discharged to the environment without treatment, their biological oxidation can lead to the depletion of natural oxygen and development of septic conditions.
- Pathogens:

  Communicable diseases can be transmitted by the pathogenic organisms in sewage.
- Nutrients:

  They include nitrogen and phosphorous compounds. These compounds are essential for plants growth. When they are discharged to the aquatic environment, they can cause eutrophication phenomenon (excessive growth of algae).
- Refractory organics:

  Refractory organics like surfactants and pesticides can reduce the efficiency of biological treatment processes.
- Dissolved inorganic solids:

4-2 Treatment of Sanitary Sewage

The intent of sewage treatment is to produce an effluent that can be discharged to the surrounding environment without any detrimental impact. The best known and widely applied effluent standard is the so called 20/30 (20mg/l biochemical oxygen demand and 30 mg/l suspended solids). Biochemical oxygen demand (BOD) is the amount of oxygen consumed by bacteria to break down organic matter present in wastewater. Fig.(4-1) shows a flow sheet of sanitary sewage treatment plant.

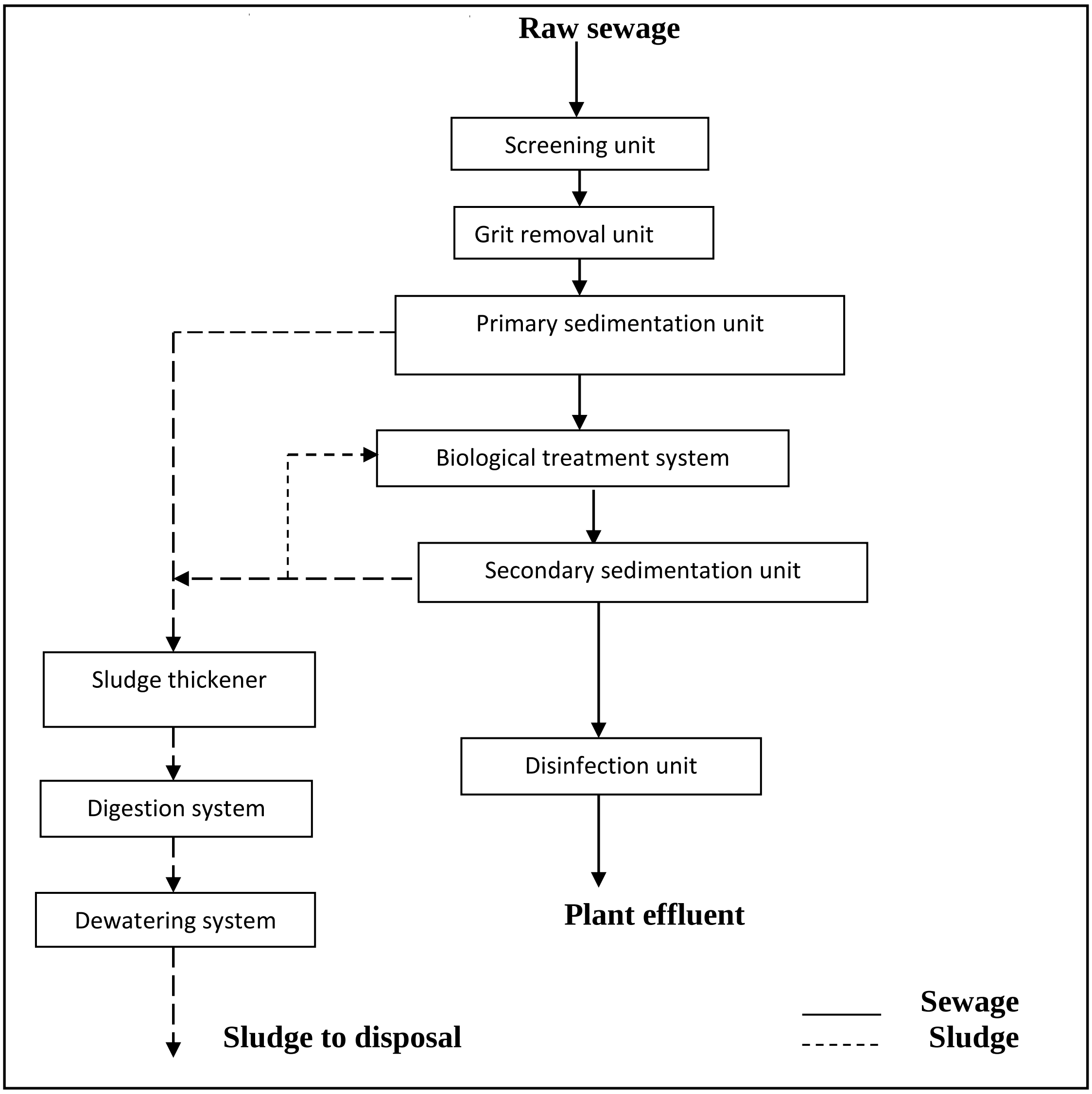
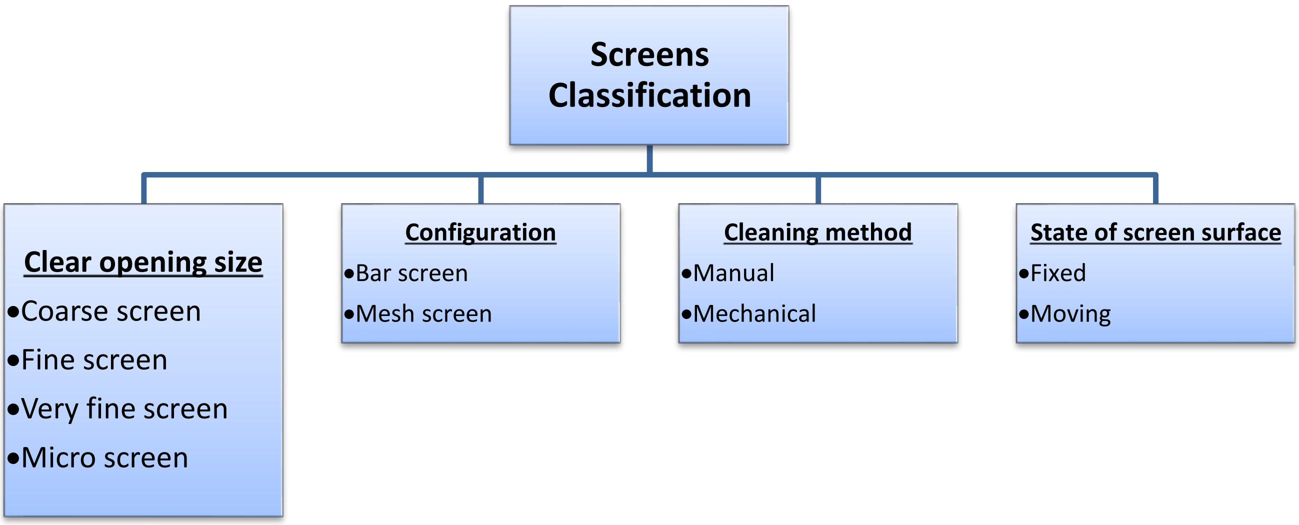


Fig.(4-1) Flow sheet of sanitary sewage treatment plant

4-3 Screening Unit (preliminary treatment)

Screens are used to remove coarse and floating matter which may damage or interfere with the operation of pumps and treatment plant equipment. Screens can be classified based on four bases as given below:



4-3-1 Design of Screening Unit

Screens are installed into an open channel, Fig.(4-2). Bar screens are made of rectangular, circular or teardrop bars. If circular or teardrop bars are used, the wider width dimension is on the upstream side.

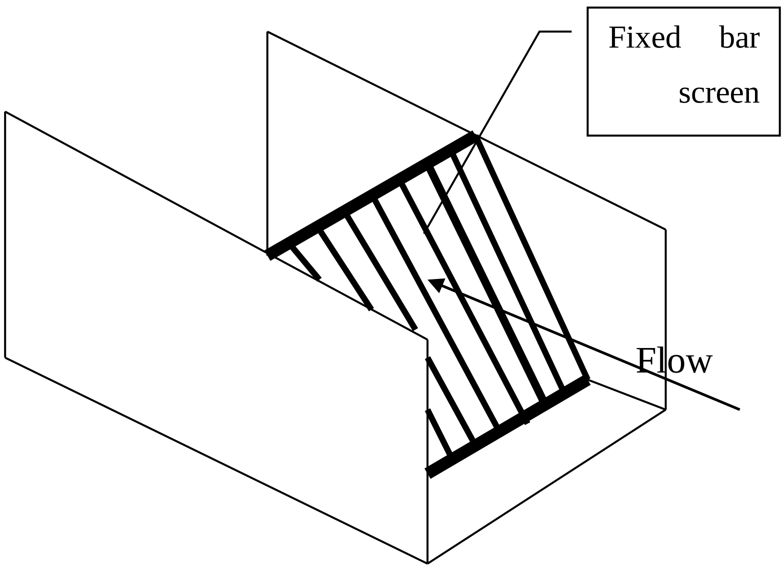


Fig.(4-2) Fixed bar screen

The design criteria of the screen and the channel are:

⇒ Flow velocity in the approach velocity;
0.3 to 0.6 m/sec for manual screens.

0.3 to 1.0 m/sec for mechanical screens.

⇒ Flow velocity through the screen;
It should be less than 0.9 m/sec during peak flow.

⇒ Clear opening size (s);
For coarse screens, s= 6 to 75mm.

For fine screens, s= 1.5 to 6mm.

For very fine screens, s= 0.25 to 1.5mm.

For micro screens, s= 1 μm to 0.3mm.

⇒ The angle of screen surface with the horizontal direction (θ) is 45° to 60°

⇒ Head loss through the screen;

150mm for manual screens

150 – 600 mm for mechanical screens

<u>Design of approach channel</u>

1. Assume flow velocity in the approach channel during peak flow condition.
2. Assume depth of flow (D) during peak flow as 2/3 of channel width (W).
3. Find width and depth using the following equation;

$$V_a = \frac{Q}{W \times D}$$

V_a= flow velocity in the approach channel (m/sec)

W= channel width (m) and D= flow depth (m)

4. Calculate the slope of the approach channel using Manning formula with the application of peak flow condition;

$$v = \frac{1}{n} R^{2/3} S^{1/2}$$

Where;

n= Manning roughness coefficient (n=0.015 for concrete channel and n=0.013 for PVC channel)

R= hydraulic radius (m) = $\frac{A}{P} = \frac{W \times D}{W + 2D}$

S= slope of hydraulic grade line= bottom slope

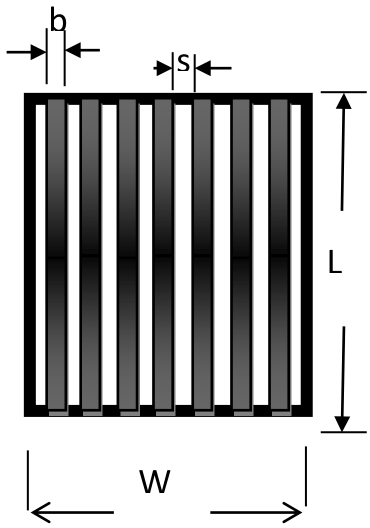
5. Find flow velocity in the approach channel during average flow and check whether it is > 0.3m/sec.
6. Provide a free board of not less than 30cm.

Design of fixed bar screen

1. Assume flow velocity through the screen (V_{screen}) at peak flow condition.
2. Assume spacing of bars (s).
3. Find bar thickness (b);

$$A_{screen} = \frac{Q}{V_{screen}}$$

$$A_{screen} = \frac{s}{b+s} W.D$$



4. Find the number of bars;

$$number\ of\ bars = \frac{W}{b+s}$$

Where;

A_{screen} = clear area of screen.

W =width of approach channel.

D= flow depth in the approach channel.

b= bar thickness.

s= bars clear spacing.

5. Decide the inclination angle (ϑ) and find the bars length (L);

$$L = D/\sin(\theta)$$

<u>Head loss across the screen</u>

The minimum head loss (in case of clean screen) can be calculated as;

$$h_L = \beta\left(\frac{b}{s}\right)^{4/3} h_v \sin\theta$$

where

s = the minimum opening between the bars (m)

h_L= the head loss through the bar rack (m)

h_v = the velocity head of the approaching flow (m)

b= the maximum width of the bars facing the flow (m)

β= a dimensionless shape factor for the bars, Table (4-1)

θ= the angle between the facial plane of the bar rack and the horizontal direction.

Table (4-1) β values for different bar shapes

Bar Cross Section	Shape Factor (Dimensionless)
Sharp-edged, rectangular	2.42
Rectangular with semicircular upstream face	1.83
Circular	1.79
Rectangular with semicircular faces upstream and downstream	1.67
Teardrop with wide face upstream	0.76

Example 4-1

Design screening unit for a sewage treatment plant has average flowrate of 16128m^3/day and peak sewage flowrate of 42787.6 m^3/day.

<u>Solution</u>

Let flow velocity in the approach channel 0.6m/sec

$$V_a = \frac{Q}{W \times D}$$

$$Q = \frac{42787.6}{24 \times 3600} = 0.495 m^3/sec$$

$0.6 = \frac{0.495}{W \times \frac{2}{3} W} \rightarrow W = 1.113m\ use\ W = 1.12m$

$D = \frac{2}{3} \times W = \frac{2}{3} \times 1.12 = 0.75m$ (water depth at peak flow condition)

Use freeboard of 40cm

Total depth of approach channel=0.75+0,4=1.05m

Find bottom slope of approach channel;

$$v = \frac{1}{n} R^{2/3} S^{1/2}$$

$$\frac{0.495}{1.12 \times 0.75} = \frac{1}{0.015} \left(\frac{1.12 \times 0.75}{1.12 + 2 \times 0.75} \right)^{2/3} \times S^{0.5}$$

$S=3.56 \times 10^{-4}$

Check flow velocity at average flow condition;

At average flow condition;

Q= 16128 m^3/day=0.187 m^3/sec

$$v = \frac{1}{n} R^{2/3} S^{1/2}$$

$$\frac{0.187}{1.12 \times D} = \frac{1}{0.015} \left(\frac{1.12 \times D}{1.12 + 2D} \right)^{2/3} \left(3.56 \times 10^{-4} \right)^{1/2}$$

The above equation is solved by trial and error;

D	0.5	0.7	0.3	0.35	0.36	0.362
LHS	0.334	0.2386	0.5567	0.477	0.464	0.4613
RHS	0.518	0.5775	0.423	0.451	0.457	0.458

Use D=0.362m (water depth at average flow condition)

V=$\frac{0.187}{1.12 \times 0.362} = 0.46m/sec$ which is >0.3m/sec…..O.K.

Let s=15mm and V_{screen} =0.8m/sec

$$A_{screen} = \frac{Q}{V_{screen}} = \frac{0.495}{0.8} = 0.619m^2$$

$$A_{screen} = \frac{s}{b+s} W.D$$

$$o.619 = \frac{0.015}{0.015+b} \times 1.12 \times 0.75$$

b=0.00535m=6mm

$$number\ of\ bars = \frac{W}{b+s} = \frac{1.12}{0.006+0.015} = 53.33 \rightarrow use\ 54\ bars$$

Find bars length;

Let θ=60°

L=D/ sin θ=0.75/sin60=0.87m

Calculate the head loss through the screen;

$$h_L = \beta \left(\frac{b}{s} \right)^{4/3} h_v \sin\theta$$

Use rectangular bars; $\beta = 2.42$

$$h_L = 2.42 \times \left(\frac{6}{15} \right)^{4/3} \times \frac{0.59^2}{2 \times 9.81} \times \sin 60$$

h_L=0.011m (head loss of clean screen)

Use h_L of 15cm

4-4 Grit Chamber (Preliminary treatment)

Grit chambers are provided to remove inorganic grit. Removing the grit that washes off streets or land during storms is very important, especially in cities with combined sewer systems. Large amounts of grit and sand entering a treatment plant can cause serious operating problems, such as excessive wear of pumps and other equipment, clogging of aeration devices, or set hard in sludge hoppers, transmission pipes, and in the bottom of digesters.

Grit removal is usually achieved by differential settlement, in which the flow velocity is controlled so that heavier inorganic grit particles are removed, while organic solids are retained in suspension. Grit chambers are commonly designed to remove inorganic particles down to an equivalent diameter of 0.2mm.

On the basis of the method used in controlling flow velocity, three types of grit chambers may be distinguished:

1- Constant velocity grit chambers; in which longitudinal flow velocity is controlled hydraulically.
2- Aerated grit chambers; in which helical rolling motion is induced by controlled introduction of air along one side of the chamber. In the rolling flow pattern, all solids are carried to the bottom of the basin with the flow resuspended the organic matter while the inorganic remain behind.
3- Mechanically-stirred tanks; in which rotary motion is produced by mechanical mixer in a cylindrical chamber.

4-4-1 Constant Velocity Grit Chamber

Constant velocity grit chamber consists of two essential components;

- The grit chamber itself, and
- The velocity control device.

The required constant velocity can be achieved either by;

- Parabolic grit chamber matched to a rectangular control flume (Fig.4-3), or
- Rectangular grit chamber matched to especially shaped control weir (Sutro weir).

4-4-2 Design Criteria of Constant Velocity Grit Chambers

5.6 cm/sec<Horizontal flow velocity (scour velocity)<23 cm/sec

The lower limit represents the scour velocity of organic matter, while the upper limit represents the scour velocity of inorganic matter. The desirable value of horizontal flow velocity is 21 cm/sec.

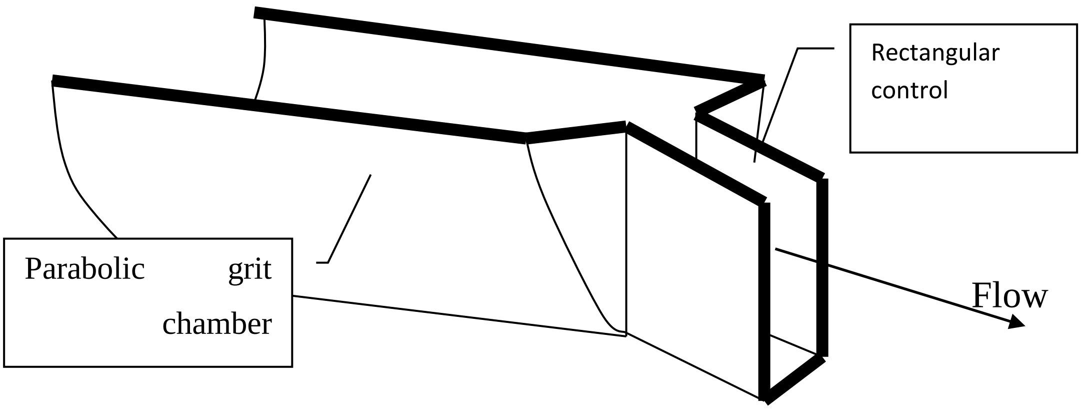


Fig.(4-3) Parabolic grit chamber with rectangular control flume

4-4-3 Design of Parabolic Channel with Rectangular Control Section

The required data includes; Q_{min}, Q_{max}, $Q_{avg.}$, and Q_{peak}. In addition the horizontal flow velocity (V_h) and the settling velocity of minimum grit particle to be removed are selected. The horizontal flow velocity is selected to be greater than the horizontal flow velocity required to scour organic particle and less than the horizontal flow velocity required to scour minimum inorganic particle to be removed.

All particles are assumed to settle according to Newton's law;

$$V_s = \left[\frac{4gd(S-1)}{3C_D}\right]^{0.5}$$

and to be scoured at a horizontal velocity of;

$$V_{Sc} = \left[\frac{8\beta(S-1)gd}{f}\right]^{0.5}$$

Where;

S= sp. gr. of solid particles=1.1 and 2.65 for organic solid particles and inorganic grit, respectively.

C_D= drag coefficient=10

d= particle diameter, m

β= constant= 0.06

f= constant= 0.03

g= gravity acceleration=9.81m/sec^2

Design Steps

1- Assume channel width at max. sewage flowrate.

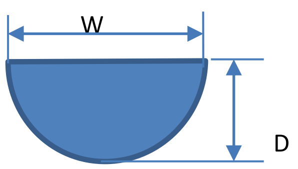
2- Determine the cross sectional area of flow (A) at max. flowrate;

$$A = \frac{Q_{max}}{V_h}$$

3- Find the water depth (D) at max. sewage flowrate;

$$D = \frac{3A}{2W}$$

Where for parabolic channel A=2WD/3 and W= width of water surface.



Find the flow velocity in the control section (V_c). The flow at the control section is critical, then;

$$D = d_C + \frac{{V_c}^2}{2g} + 0.1\frac{{V_c}^2}{2g}$$

where the head loss is assumed to be 10% of velocity head and d_c is the water depth in the control section and it is obtained as;

$$d_c = \frac{{V_c}^2}{g}$$

Thus;

$$D = 3.1\frac{{V_c}^2}{2g}$$

And then;

$$V_C = \left[\frac{2gD}{3.1}\right]^{0.5}$$

4- Find the flow cross sectional area in the control section;

$$A_c = \frac{Q}{V_c}$$

5- Find the width of the control section (b);

$$b = \frac{A_c}{d_c}$$

6- Then based on the following formula find d_c, V_c, D and the width of the channel at the water surface for each flow condition. Arrange these data into a table as shown below;

$$d_c = \sqrt[3]{\frac{Q^2}{gb^2}}$$

Q	d_c	V_c	D	w

7- Find the tank length;

$$\frac{V_h}{V_s} = \frac{L}{D}$$

Example 4-2

Design grit removal unit consists of grit chamber has parabolic section for a sewage treatment plant has minimum, average, maximum and peak sewage flowrates of 8064, 16128, 37255.7 and 42787.6 m^3/day, respectively.

<u>Solution</u>:

Assume channel width at maximum flow condition= 2.2m.

$$A = \frac{Q_{max}}{V_h}$$

$$Q_{max} = \frac{37255.7}{24 \times 3600} = 0.4312\ m^3/sec$$

Let V_h=21 cm/sec=0.21m/sec

$$\therefore A = \frac{0.4312}{0.21} = 2.053\ m^2$$

$$A = \frac{2WD}{3} \rightarrow D = \frac{3A}{2W} = \frac{3 \times 2.053}{2 \times 2.2} = 1.4m$$

$$D = 3.1\frac{{V_c}^2}{2g} \rightarrow \quad 1.4 = 3.1\frac{{V_c}^2}{2 \times 9.81} \rightarrow\ V_c = 2.98\ m/sec$$

$$A_c = \frac{Q}{V_c}\ \rightarrow A_c = \frac{0.4312}{2.98} = 0.145m^2$$

$$A_c = b \times d_c$$

$$d_c = \frac{{V_c}^2}{g} = \frac{2.98^2}{9.81} = 0.905m$$

$b = \frac{A_c}{d_c} = \frac{0.145}{0.905}$ =0.16m

For other flow conditions;

$$d_c = \sqrt[3]{\frac{Q^2}{gb^2}}$$

For Q_{min}= 8064m^3/day=0.0933 m^3/sec

$$d_c = \sqrt[3]{\frac{0.0933^2}{9.81 \times 0.16^2}} = 0.326\text{m}$$

$$V_c = \sqrt{gd_c} = \sqrt{9.81 \times 0.326} = 1.788\text{m/sec}$$

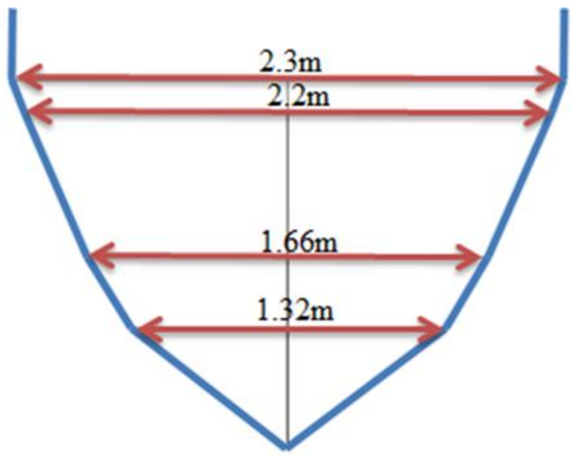
$$D = 3.1\frac{{V_c}^2}{2g} = 3.1\frac{1.788^2}{2 \times 9.81} = 0.505m$$

$$A = \frac{Q}{V_h} = \frac{0.0933}{0.21} = 0.444m^2$$

$$W = \frac{3A}{2D} = \frac{3 \times 0.444}{2 \times 0.505} = 1.32m$$

Similarly, for Q_{avg} and Q_{peak}, the results are given in the table below.

Q (m^3/day)	d_c (m)	V_c (m/sec)	D (m)	W (m)
8064	0.326	1.789	0.506	1.32
16128	0.518	2.254	0.802	1.66
37255.7	0.905	2.979	1.402	2.20
42787.6	0.992	3.120	1.538	2.30



Cross section of parabolic grit chamber

$$\frac{V_h}{V_s} = \frac{L}{D}$$

$$V_s = \left[\frac{4gd(S-1)}{3C_D}\right]^{0.5}$$

d= 0.2mm= 0.0002m; S=2.65; C_D=10

$$V_s = \left[\frac{4 \times 9.81 \times 0.002 \times (2.65-1)}{3 \times 10}\right]^{0.5} = 0.021m/sec$$

$$\frac{0.21}{0.021} = \frac{L}{1.538} \rightarrow L = 15.38m$$

4-4-4 Design of Aerated Grit Chamber

An aerated grit chamber consists of a standard spiral flow aeration tank provided with air diffusion tubes placed on one side of the tank, see Fig.(4-4). The grit particles tend to settle down to the bottom of the tank at rates depend upon the particle size and the bottom velocity of roll of the spiral flow, which in turn depends on the rate of air diffusion through diffuser tubes and shape of aeration tank. The heavier particles settle down whereas the lighter organic particles are carried with roll of the spiral motion.

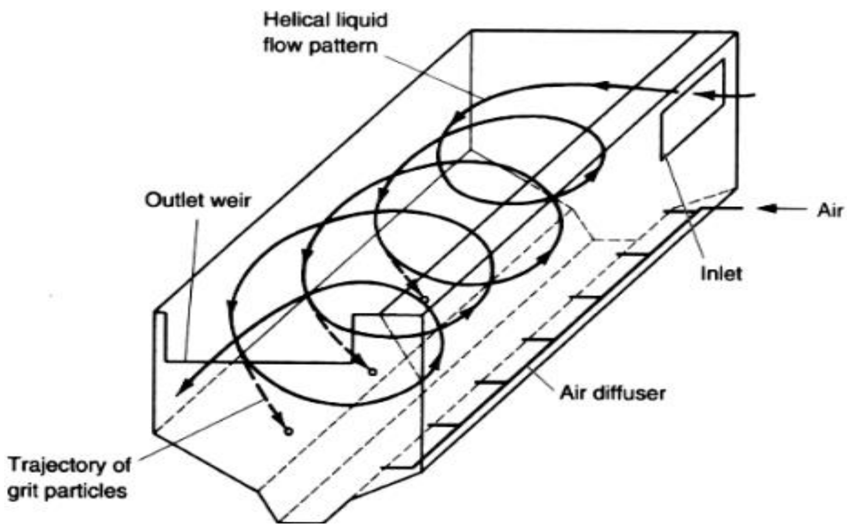


Fig.(4-4) Aerated grit chamber

Design criteria;

- Detention time=3min.
- Air supply= $0.04 m^3/min/m$ run of length.
- Depth to width ratio=1.2
- Depth= 2 to 5m
- Grit quantity=$50\times10^{-6}\ m^3/m^3$ of wastewater.
- The length is increased by 15% for inlet and exit details.

Example 4-3

Design grit removal unit consists of aerated grit chamber for a sewage treatment plant has peak sewage flowrate of $0.463\ m^3/sec$.

Solution:

Assume detention time=3min.

$Q=V/t \rightarrow\ V=Q\times t=0.463\times60\times3=83.34\ m^3$

$V=L\times D\times W$

Let D=3m

Let $D/W=1.2\ \rightarrow W=3/1.2=2.5m$

$\therefore 83.34 = L \times 3 \times 2.5$

L=11.112m

Increased the length by 15%;

Tank length=$1.15\times11.112=12.78m$

Air supply= $0.04 m^3/min/m$ run of length.
Air supply= $0.04\times11.112=0.4445\ m^3/min$

4-5 Primary Sedimentation Unit (Primary treatment system)

Generally, sedimentation is an operation in which solid-liquid separation takes place in a suspension with the formation of a clarified overflow and a concentrated underflow. Primary settling tanks may have circular, rectangular, or square shape. The function of primary sedimentation unit is to;

1. remove settleable solids matter, and
2. remove free oil and grease and other floating matter.

4-5-1 Circular Sedimentation Tank

In circular sedimentation tanks, the flow in the radial direction. The main components of circular primary sedimentation tank are:

1-<u>Stilling well</u>

This part represents the inlet zone of settling tank in which the influent pipe discharges raw sewage. It is extended 1 to 2 m below the water surface.

2-<u>Effluent launder</u>

It is an open channel located along the periphery of the tank in which settled sewage is discharged over an outlet weir. In this channel the effluent pipe is fitted. Effluent launder represents the outlet zone of settling tank.

3-<u>Sludge sump</u>

It is a conical shape pit (hopper) in the central part of tank bottom in which sludge accumulates. At the wall of this sump, sludge draw of pipe is fitted.

4-<u>Scum baffle</u>

It is a plate installed a head of outlet weir and extends 150 to 300mm below the water surface. This baffle prevents the passage of scum with the effluent.

5-<u>Scraper</u>

It is a group of blades fixed at a rotated arm. Scraper is used to scrap the settled sludge at tank bottom and directed it toward the sludge sump.

6-<u>Skimmer</u>

It is a rotated arm moves at the top edge of scum baffle and it is used to collect scum (floating matter) and directed it toward an outlet pipe fixed to a plate at a specific location of scum baffle.

Fig.(4-5) shows a plan view and vertical section of primary settling tank of circular shape, Fig.(4-6) shows a photo for this tank and Fig.(4-7) shows zigzag outlet weir, scum baffle and skimmer.

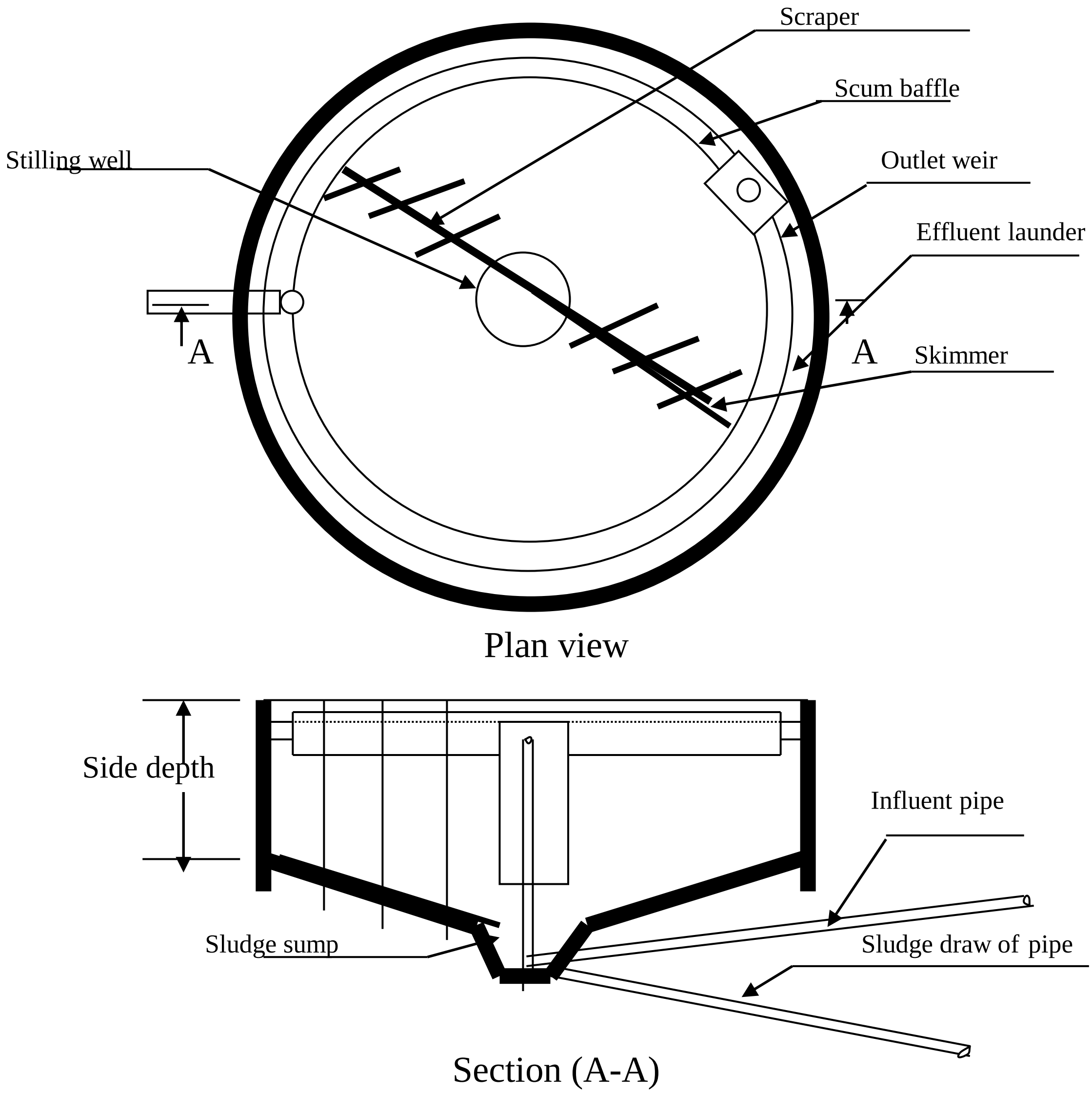


Fig.(4-5) Details of circular primary settling tank

Fig.(4-6a) Photo of primary sedimentation tank

Fig.(4-6b) Outlet weir, scum baffle and skimmer

4-5-2 Rectangular Sedimentation tank

Rectangular sedimentation tanks have the same components of the circular sedimentation tanks as shown in Fig (4-7b). however, the flow in these tanks is along the long axis.

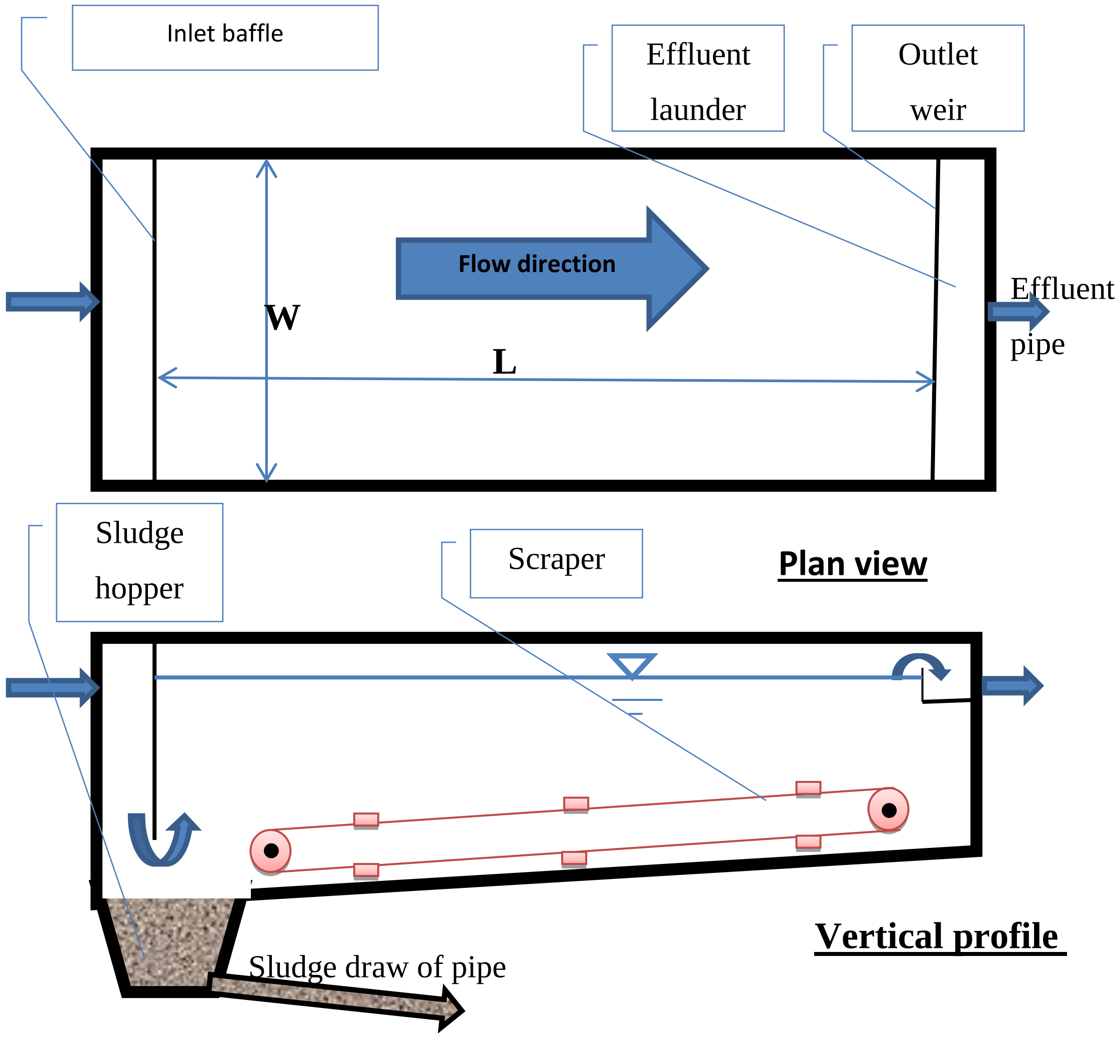


Fig. (4-7) Plan view and profile of a rectangular sedimentation tank

4-5-3 Design Criteria

In designing primary settling tanks, the following criteria are adopted:

- Detention time (t) =1 to 2 hours at peak flow
- Surface overflow rate (SOR) = 1 to 2.5 m^3/m^2 per hour
- Weir loading rate = 120 to 370 m^3/m per day

For circular tanks, other necessary design data include;

- Maximum diameter= 40m.
- Stilling well diameter = 0.1 to 0.2 of tank diameter and extends 1 to 2 m below the water surface.

- If scraper is used, bottom slope=1:24 to 1:12 (V:H).
- If scraper is not use (in case of small sedimentation tanks), bottom slope of 1:1 is usually adopted.

For rectangular tanks, other design data include:

- Length to width ratio (L/W)=3 to 6
- Tanks dimensions are selected to match the requirements of the chosen sludge collection equipment (scraper). Generally;
- Maximum tank length=100m
- Maximum tank width= 13.5m
- Bottom slope as that of circular tanks.

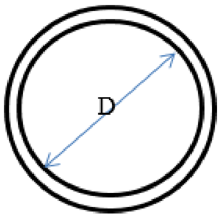
The weir loading rate is equal to;

$$weir\ loading\ rate = \frac{Q_{one}}{weir\ length}$$

Where;

Q_{one}= water flowrate received by one tank, m^3/day

For circular tanks; weir length= πD (D= tank diamete)



For rectangular tanks;

If one effluent launder is used; <u>weir length=W</u>

If n effluent launders are used; <u>weir length= (2n-1)W</u>

<u>Design steps</u>

1. Assume SOR

2. Find the total surface area (A) as;

$$A = Q/SOR$$

3. Assume the number of tanks=2

4. Find the surface area of one tank (A_{one})

5. Find the dimensions of tank;

If circular tanks are used;

$$A_{one} = \frac{\pi}{4}(D^2 - D_S^2)$$

Where; D_S= diameter of stilling well

if rectangular tanks are used;

$$A_{one} = W.L$$

6. If D>40m, increase the number of tanks (circular tanks)

7. If L >100m or W>13.5m, increase the number of tanks (retangular tanks)

8. Find Q_{one}

9. Check weir loading rate.

If weir loading rate is not checked and the tanks are circular, then;

Use v-notches or zig-zag weir, as shown below

V-noches outlet weir

If weir loading rate is not checked and the tanks are rectangular, then;

Increase the number of effluent launders to n and find n by putting weir loading rate = 370 m^3/m.day.

Important note:

The minimum number of sedimentation tanks is two, because the tanks must periodically be taken out of service for maintenance

10. Assume detention time (t).

11. Find the water volume in one tank as;

$$V_{one} = Q_{one} \times t$$

12. Find the side water depth (SWD);

$$SWD = \frac{V_{one}}{A_{one}}$$

Find the total tank depth by adding a free board of 10% water depth.

Design of effluent launder

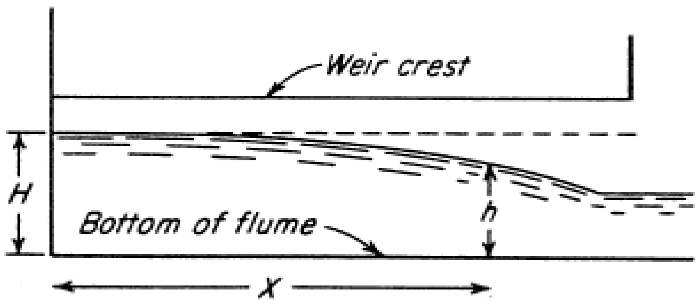
a- Find the minimum water depth in the effluent launder (h);

$$\text{h} = \sqrt[3]{\frac{Q_{one}^2}{gb^2}}$$

where; b= width of effluent launder, m, which assumed to be 300 to 600mm.

b- find the maximum water depth in the effluent launder as;

$$H = \left[h^2 + \frac{2q^2x^2}{gb^2h}\right]^{0.5}$$



Where;

q= water flowrate per meter length of weir, m^3/m.sec

x= length of water path.

For circular tanks;

x= $\frac{\pi D}{2}$

For rectangular tanks;

x=W/2 (if the effluent pipe fixed at the center of effluent launder)

and;

x=W (if the effluent pipe fixed at the end of effluent launder)

c- Find the total depth of effluent launder by adding a free board of 15cm.

Example 4-4

Design primary sedimentation unit for a sewage treatment plant has a design capacity of 60000 m^3/day using;

1. Circular tanks.
2. Rectangular tanks.

Solution:

1. Circular tanks

Assume SOR=1.5 m/hr

$$SOR = \frac{Q}{A} \rightarrow A = \frac{Q}{SOR} = \frac{60000/24}{1.5} = 1666.67\, m^2$$

Assume the number of tanks=2

$$A_{one} = \frac{1666.67}{2} = 833.33 m^2$$

$$A_{one} = \frac{\pi}{4}(D^2 - D_S^2)$$

Let $D_s = 0.15D \rightarrow \quad 833.33 = \frac{\pi}{4}(D^2 - (0.15D)^2)$

D= 32.95 m < 40 m → O.K.

D_s= 0.15×32.95= 4.94 m

$Check\ weir\ loading\ rate$;

$$weir\ loading\ rate = \frac{Q_{one}}{weir\ length}$$

$$Q_{one} = \frac{Q_{total}}{number\ of\ tanks} = \frac{60000}{2} = 30000\, m^3/day$$

Weir length= $\pi D = \pi \times 32.95 = 103.52\, m$

$$weir\ loading\ rate = \frac{30000}{103.52} = \frac{289.8 m^3}{m.day} < 370 \frac{m^3}{m.day} \rightarrow O.K.$$

Assume detention time (t)= 1.5 hours

Find the water volume in one tank as;

$V_{one} = Q_{one} \times t = \frac{30000}{24} \times 1.5 = 1875\, m^3$

Find the side water depth (SWD);

$SWD = \frac{V_{one}}{A_{one}} = \frac{1875}{833.33} = 2.25\, m$

Find the total tank depth;

Free board=0.1×2.25=0.225 m<0.3 m…..Use a free board= 0.3m

Total tank depth= 2.25+0.3= 2.55 m

Design of effluent launder;

Find the minimum water depth in the effluent launder (h);

Assume b=400mm=0.4m

$h = \sqrt[3]{\frac{Q_{one}^2}{gb^2}} = \sqrt[3]{\frac{\left(\frac{30000/2}{24\times3600}\right)^2}{9.81\times0.4^2}}$ =0.27m

Find the maximum water depth in the effluent launder (H);

$$H = \left[h^2 + \frac{2q^2x^2}{gb^2h}\right]^{0.5}$$

$q = \frac{289.8}{24\times3600}$ =0.00335m^3/m/sec

$$X = \frac{\pi D}{2} = \pi \times \frac{32.95}{2} = 51.76\, m$$

$$H = \left[0.27^2 + \frac{2\times0.00335^2\times51.76^2}{9.81\times0.4^2\times0.27}\right]^{0.5} = 0.46\, m$$

Add free board of 15cm

Total depth of effluent launder= 0.46+0.15=0.61 m

2. Rectangular tanks

Assume SOR=1.5 m/hr

$$SOR = \frac{Q}{A} \rightarrow A = \frac{Q}{SOR} = \frac{60000/24}{1.5} = 1666.67\, m^2$$

Assume the number of tanks=2

$$A_{one} = \frac{1666.67}{2} = 833.33 m^2$$

$A_{one} = W \times L$

Let L/W= 4 → L= 4W

$833.33 = W \times 4W \rightarrow$ W= 14.43 m (>13m Not. O.K.) →L=4×14.43=57.72 m→ increase L/W ratio

Let L/W=5 → L= 5W

$833.33 = 5W^2 \rightarrow W$ =12.91 m< 13.5m ……O.K.

L=12.91$\times 5 = 64.55 < 100\, m \ldots O.K$

Assume detention time (t)=1.5 hours

Find the water volume in one tank as;

$V_{one} = Q_{one} \times t = \frac{30000}{24} \times 1.5 = 1875\, m^3$

Find the side water depth (SWD);

$SWD = \frac{V_{one}}{A_{one}} = \frac{1875}{12.91 \times 64.55} = 2.25\, m$

Find the total tank depth;

Total tank depth=2.25+0.3= 2.55 m

Check weir loading rate;

$$weir\ loading\ rate = \frac{Q_{one}}{weir\ length}$$

$$Q_{one} = \frac{Q_{total}}{number\ of\ tanks} = \frac{60000}{2} = 30000\, m^3/day$$

If one effluent launder is used →Weir length= $W = 12.91\, m$

$weir\ loading\ rate = \frac{30000}{12.91} = \frac{2323.8\, m^3}{m.day} > 370 \frac{m^3}{m.day} \rightarrow Not\ O.K.$

Let the number of effluent launders=n

Weir length=(2n-1)W

$370 = \frac{30000}{(2n-1) \times 12.91}$ →n=3.64 use 4 effluent launders

Weir loading rate=$\frac{30000}{(2 \times 4 - 1) \times 12.91}$=331.97 $\frac{m^3}{m.day}$

<u>Design of effluent launder;</u>

Find the minimum water depth in the effluent launder (h);

Assume b=400mm=0.4m

Assume the effluent pipe is fixed at the end of effluent launder;

$$h = \sqrt[3]{\frac{Q_{one}^2}{gb^2}} = \sqrt[3]{\frac{\left(\frac{30000/4}{24\times3600}\right)^2}{9.81\times0.4^2}} = 0.17\text{m}$$

Find the maximum water depth in the effluent launder (H);

$$H = \left[h^2 + \frac{2q^2x^2}{gb^2h}\right]^{0.5}$$

q= $\frac{331,97}{24\times3600}$ =0.00384m^3/m/sec

$X = W = 12.91\ m$

$$H = \left[0.17^2 + \frac{2\times0.00384^2\times12.91^2}{9.81\times0.4^2\times0.17}\right]^{0.5} = 0.22\ m$$

Add free board of 15cm

Total depth of effluent launder= 0.22+0.15=0.37 m

4-5-3 Efficiency of Primary Setting tanks

The percent of BOD and suspended solids removal y primary settling tanks is related to SOR as shown in Fig.(4-8).

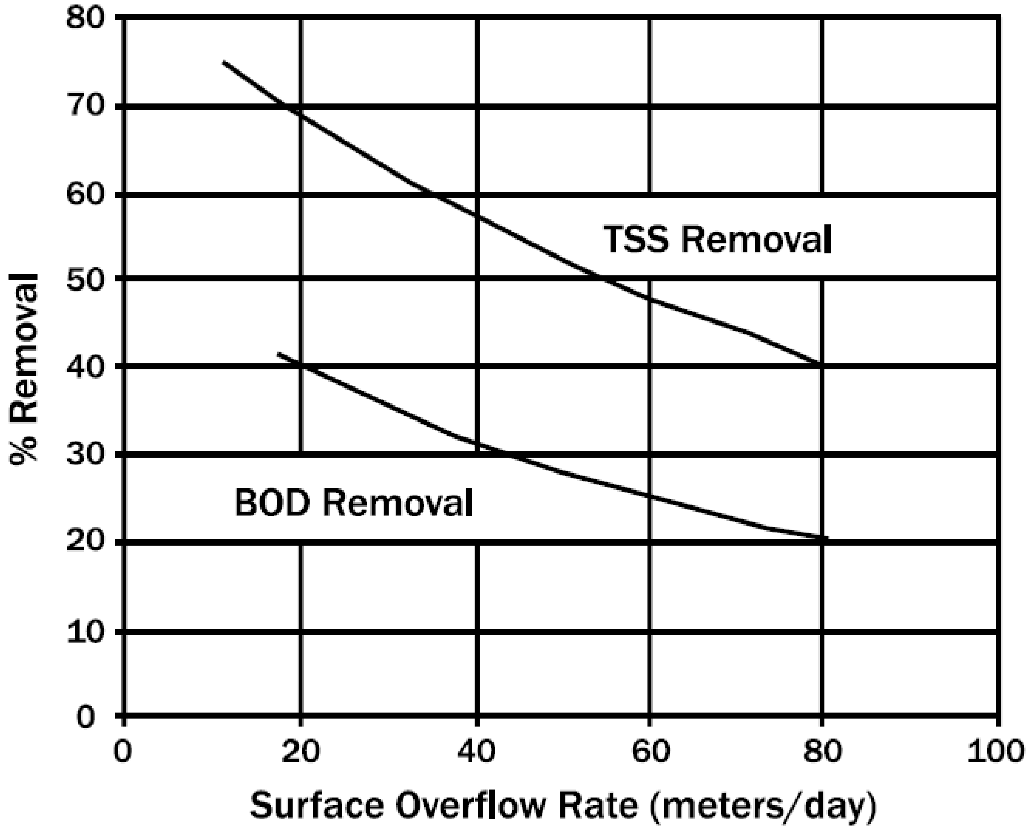


Fig.(4-8) Relationships between SOR and percentages of BOD and SS removals in primary sedimentation tanks.

4-6 Biological Treatment Unit

Biological treatment consists of application of a controlled natural process in which microorganisms remove soluble and colloidal organic matter from sewage by biological oxidation process. Biological treatment systems can be classified into suspended growth systems and attached growth systems.

Suspended growth systems maintain an adequate biological mass in suspension within the reactor by employing either natural or mechanical mixing. These systems include activated sludge and its various modifications and oxidation ponds.

Attached growth systems utilize a solid medium upon which biological solids are accumulated in order to maintain a high population. Examples of these systems include trickling filters and rotating biological contactors.

4-6-1 Activated Sludge Process

Activated sludge process involves the production of an activated mass or microorganisms capable of aerobically stabilizing waste. In the activated sludge process, the settled sewage is mixed with return (or recycled) activated sludge. The mixture enters an aeration tank, Fig.(4-9), where the organisms and sewage are mixed together with a large quantity of air. Under these conditions the organisms oxidize a portion of the waste organic matter to carbon dioxide (CO_2) and water and synthesize the other portion into new microbile cells.

The mixture then enters a settling tank (secondary settling tank) where the flocculant microorganisms settle, and are removed from the effluent stream. The settled microorganisms or activated sludge is then recycled to the head end of the aeration tank to be mixed again with sewage. New activated sludge is continuously being produced in this process, and the excess sludge produced each day must be disposed of together with the sludge from the primary sedimentation tank.

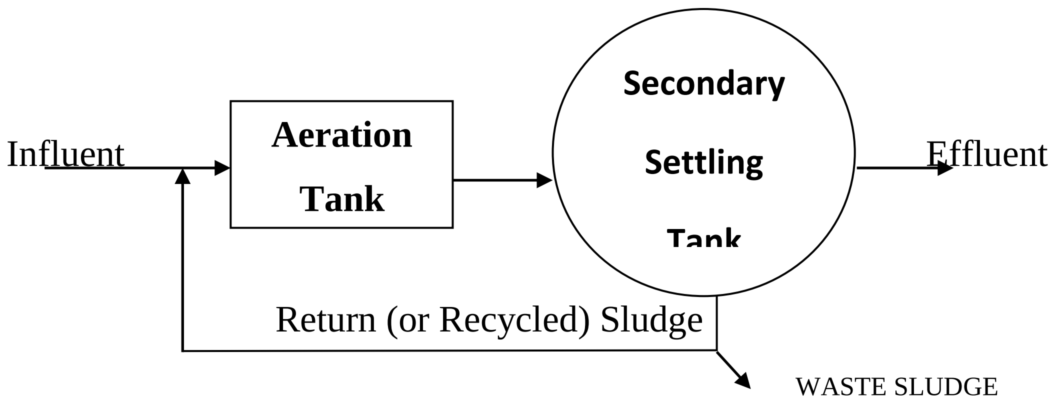


Fig.(4-9) Conventional activated sludge system

The oxygen necessary for the biological process can be supplied by using air or pure oxygen. There are two basic methods of introducing air to aeration tank, air diffuser or pure oxygen injection (Fig.4-10) and mechanical surface aerators which are rotating devices are used to mix the contents of the aeration basin and to introduce oxygen into the liquid by dispersing fine water droplets in the air so that oxygen can be observed, Fig.(4-11).

Fig.(4-10) Diffused aeration system

Fig.(4-11) Mechanical surface aerator

4-6-1-1 Design of Conventional Activated Sludge System

Conventional activated sludge system is used for domestic sewage treatment. The aeration basin is a long tank with a uniform distribution of air diffusers or surface aerators. Settled sewage and return activated sludge enters the tank head and flow down its length. A schematic diagram of a conventional activated sludge system is shown in Fig.(4-12). The design of aeration tank is done in accordance to the following procedure:

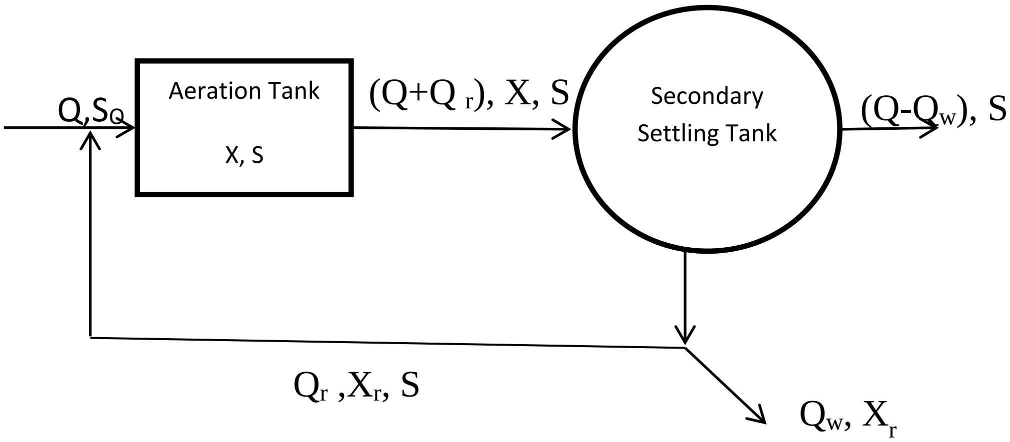


Fig.(4-12) Schematic diagram of conventional activated sludge system

1. Determination of aeration tank volume

The volume of aeration tank is determined using the following equation:

$$V = \frac{\theta_c QY(S_o - S)}{X(1 + K_d\theta_c)}$$

Where;

θ_c= sludge age (day)

Q= influent average flowrate (m^3/day)

Y= Biomass yield (=0.4 to 0.8)

S_o= influent BOD_5 (mg/l)

S= soluble effluent BOD_5 (mg/l)

X= mixed liquor volatile suspended solids (MLVSS) (mg/l)

K_d= microorganisms decay coefficient (=0.04 to 0.075 /day)

The soluble effluent BOD_5 (S) is determined as;

S= effluent BOD_5 - 0.63× effluent SS

2- Determination of waste sludge quantity

a- Determine the observed biomass yield (Y_{obs}) using the following equation;

$$Y_{obs} = \frac{Y}{1 + K_d\theta_c}$$

b- Determine the mass of volatile waste activated sludge (P_x);

$$P_x = Y_{obs}Q(S_o - S)$$

c- Determine the total mass of waste sludge based on total suspended solids;

$$P_{x_{ss}} = \frac{P_x}{MLVSS/MLSS}$$

Note: the ratio $MLVSS/MLSS$ is usually taken to be 0.8.

3- Compute the sludge wasting rate;

$$Q_{wr} = \frac{VX}{\theta_c X_r}$$

Where X_r = Return VSS concentration= 10000 to 15000 mg/l

4- Determination of recirculation ratio (r) by adopting mass balance around the inlet of the reactor;

$$X(Q + Q_r) = X_r Q_r$$

Then; find r=Q_r/Q

5- Check the hydraulic detention time (θ);

$$\theta = V/Q$$

6- Compute the oxygen requirement based on ultimate BOD (BOD_L);

a- Compute the mass of ultimate BOD of the incoming sewage that is utilized in the process;

$$BOD_{L\,utilized} = \frac{Q(S_o - S)}{0.68}$$

Where $BOD_L = BOD_5/0.68$

b- Compute the theoretical oxygen requirement;

$$kg\frac{O_2}{day} = total\ mass\ of\ BOD_L\ utilized, \frac{kg}{day} - 1.42 \times mass\ of\ organic\ wasted$$

7- Check the Food/ Microorganisms ratio (F/M);

$$\frac{F}{M} = \frac{S_o}{\theta X}$$

8- Check the volumetric loading;

$$Volumetric\ loading = S_o Q/V$$

9- Compute the required air volume assuming air contains 23.2% oxygen and has a density of 1.201 kg/m^3;

a- Compute the theoretical air requirement.

b- Compute the actual air requirement;

Actual air= theoretical air/0.08

c- Compute the design air requirement;

Design air requirement=actual air requirement×2

10- Check the air volume using the actual value by finding air requirement per kg BOD removed

Design Criteria

The criteria govern the design of conventional activated sludge system are given in Table (4-4).

Table (4-4) Typical design criteria for conventional activated sludge system

Normal loading		ϑ (days)	ϑ_c (days)	r	MLSS (mg/l)	Air supplied m^3/kg BOD
Kg BOD_5/m^3	Kg BOD_5/kg MLVSS					
0.56	0.2-0.5	0.25-0.3	4-14	0.15-0.3	1000 to 3000	45-90

Example 4-4

A sewage treatment plant treats sewage at average flowrate of 10000m^3/day.

The plant applies activated sludge process for biological treatment. Find;

1. Volume of aeration tank.
2. Design air requirement.
3. Flowrate of waste sludge.

Use the following data;

- BOD of raw sewage=250mg/l.
- Percent of BOD removal in primary sedimentation tank =32%.
- MLVSS=2400 mg/l.
- MLVSS in return sludge=10000mg/l.
- Y=0.6 and k_d=0.06/day.
- Effluent BOD=20mg/l.
- Effluent SS=30mg/l.

1. **Determination of aeration tank volume**

$$V = \frac{\theta_c Q Y (S_o - S)}{X(1 + K_d \theta_c)}$$

Let θ_c= 8 days

S_o= 0.68×250=170mg/l

S= effluent BOD_5 - 0.63× effluent SS=20-0.63×30=1.1 mg/l

$$V = \frac{8 \times 10000 \times 0.6 \times (170 - 1.1)}{2400(1 + 0.06 \times 8)} = 2282.43 m^3$$

Check the hydraulic detention time (θ);

$$\theta = V/Q = \frac{2282.43}{10000} = 0.228\ day < 0.25\ day\ not\ O.K.$$

Use $\theta = 0.25\ day$

$$V = \theta \times Q = 0.25 \times 10000 = 2500 m^3$$

Check the Food/ Microorganisms ratio (F/M);

$$\frac{F}{M} = \frac{S_o}{\theta X} = \frac{170}{0.25 \times 2400} = 0.283\ /day\ (0.2 - 0.5) \ldots\ldots O.K.$$

Check the volumetric loading;

$$Volumetric\ loading = \frac{S_o Q}{V} = \frac{170 \times 10000/1000}{2500} = 0.68 \frac{kgBOD}{m^3.day} > 0.56 \frac{kgBOD}{m^3.day}$$

Not O.K;

Let $\theta = 0.3\ day \rightarrow\ V = \theta \times Q = 0.3 \times 10000 = 3000m^3$

$$Volumetric\ loading = \frac{S_o Q}{V} = \frac{170 \times 10000/1000}{3000} = 0.56 \frac{kgBOD}{m^3.day} \text{.. O.K}$$

$$\therefore aeration\ tank\ volume = 3000m^3$$

Compute the theoretical oxygen requirement;

$$kg\frac{O_2}{day} = total\ mass\ of\ BOD_L\ utilized, \frac{kg}{day} - 1.42 \times mass\ of\ organic\ wasted$$

$$BOD_{L\,utilized} = \frac{Q(S_o - S)}{0.68} = \frac{10000(170 - 1.1)/1000}{0.68} = 2483.82kg/day$$

$$P_x = Y_{obs} Q(S_o - S)$$

$$Y_{obs} = \frac{Y}{1 + K_d \theta_c}$$

$Find$ θ_c;

$$V = \frac{\theta_c QY(S_o - S)}{X(1 + K_d \theta_c)}$$

$$3000 = \frac{\theta_c \times 10000 \times 0.6(170 - 1.1)}{2400(1 + 0.06 \times \theta_c)}$$

$$\theta_c = 12.38\ day$$

$$Y_{obs} = \frac{0.6}{1 + 0.06 \times 12.38} = 0.344$$

$$P_x = Y_{obs} Q(S_o - S) = 0.344 \times \frac{10000(170 - 1.1)}{1000} = 581.02 \frac{kg}{day}$$

$$kg\frac{O_2}{day} = total\,mass\,of\,BOD_L\,utilized,\frac{kg}{day} - 1.42 \times mass\,of\,organic\,wasted$$

$$theoretical\,air\,requirement = 2483.82 - 1.42 \times 581.02 = 1658.8\,kg/day$$

Theoretical air requirement in m^3/day=$\frac{1658.8}{1.201\times0.232} = \frac{5953.4m^3}{day}$

Actual air requirement=$\frac{theritical\,air\,requirement}{0.08}$=$\frac{5953.4}{0.08} = 74417.2m^3/day$

Design air requirement=2×actual air requirement

=2×74417.2=148834.3 m^3/day

Flowrate of waste sludge

$$Q_{wr} = \frac{VX}{\theta_c X_r}$$

$$Q_{wr} = \frac{3000 \times 2400}{12.38 \times 10000} = 58.16m^3/day$$

4-6-2 Attached Growth Biological Treatment System

Attached Growth is a biological treatment process in which microorganisms responsible for conversion of organic matter or other constituents in wastewater are attached to some inert material such as: rocks, sand or especially ceramic or plastic materials.

4-6-2-1 Trickling Filters

The most important considerations of trickling filter (TF) are:

♦ Wastewater applied to a TF must be passed through a primary clarifier where the majority of settleable and floatable solids are removed.

♦ Wastewater is distributed over the top of the medium and slowly trickles through it. The biological growth is attached to the media.

♦ TF effluent always passes through a secondary sedimentation unit to capture the biological solids generated as a result of biological treatment.

Components of Trickling Filters

Trickling filters are circular units with a rotary distributor. They are composed of three basic components, see Figs. (4-13) and (4-14); (1) distribution system, (2) filter media, and (3) underdrain system.

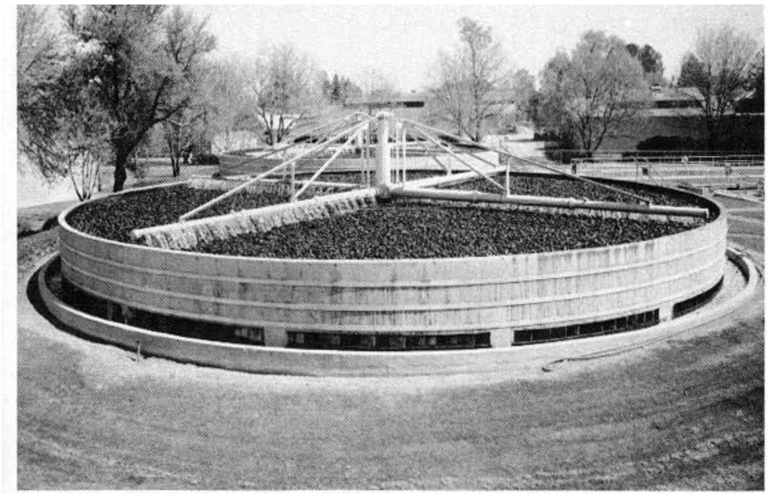


Fig.(4-13) Trickling filter overview

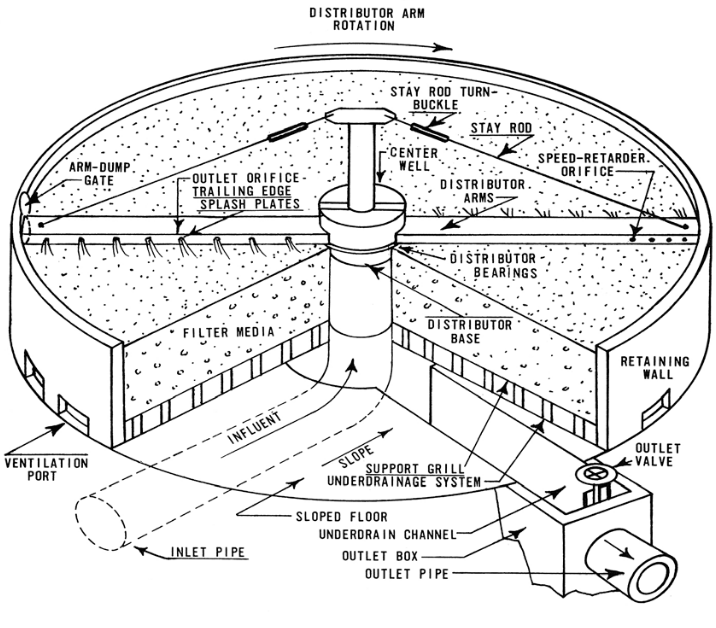


Fig.(4-14) Diagram of circular TF

1-Distribution system

The distribution system distributes wastewater over the media surface. Circular trickling filters use rotary arms to distribute wastewater. Rotary arms consist of two or more horizontal pipes suspended above the filter media.

Wastewater is distributed over the media through orifices located along one side of the pipes. They are typically set in motion by the force of the wastewater flowing out of nozzles on one end of the arm. They can also be motor-driven to control the rotational speed.

2- Filter medium

The filter medium provides a surface for the biological slime layer to attach and grow. The filter media needs to be durable and resistant to chemicals. There are generally three types of filter media: rock, redwood and synthetic materials.

Synthetic Material

Synthetic materials used as filter media are lightweight materials, typically plastics. Synthetic materials provide approximately 95% void space between the media. The advantages to synthetic material are:

♦ More surface area for microbial growth

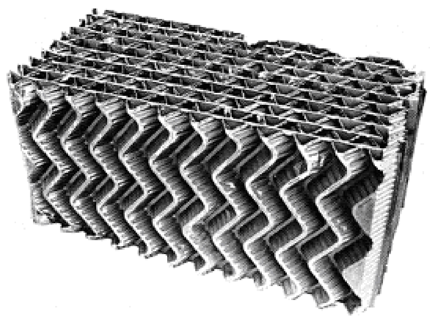
♦ More void space to allow air flow

♦ Uniform media allows even loading distribution

♦ The lightweight design allows ease of installation and handling of material and construction of deeper beds

The two general classifications of synthetic media are: Cross flow and random dump, Fig.(4-15).

Cross flow



Random dump

Fig.(4-15) Plastic media

3- Underdrain system

The underdrain system collects treated wastewater and solids discharged from the filter media and conveys them to the secondary sedimentation tank. The system is located below the filter media and operates by gravity flow. It has a sloped bottom which directs flow to a center channel. The underdrain system provides support for the filter media and allows air circulation through the media. It is typically composed of; (a) fiberglass grating over collection troughs or (b) vitrified clay blocks, Fig. (4-16).

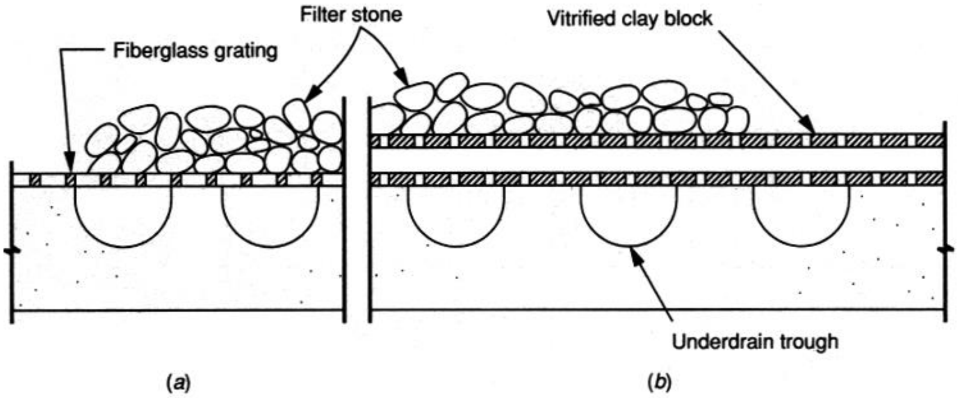


Fig.(4-16) Underdrain system

Classification of Trickling Filters

Trickling filters can be classified into four types according to their design hydraulic and organic loading rates. These types are; (1) low or standard rate filters, (2) intermediate rate filters, (3) high rate filters, and (4) roughing filters. The characteristics of these types are given in Table (4-5).

Design characteristics	Standard rate	Intermediate rate	High rate		Roughing filters
Filter medium	rock	rock	rock	plastic	Rock or plastic
Hydraulic loading rate (m^3/m^2.day)	1-4	4-10	10-40	10-75	40-200
Organic loading rate (kg BOD/m^3.day)	0.07-0.22	0.24-0.48	0.4-2.4	0.6-3.2	>1.5
Recirculation ratio	0	0-1	1-2	1-2	0-2
Filter depth (m)	1.8-2.4	1.8-2.4	1.8-2.4	3.0-12.2	0.9-6

In Table (4-5); the recirculation ratio (r) is defined as;

$$r = \frac{Q_r}{Q}$$

the hydraulic loading rate is defined as;

$$hydraulic\ \ loading\ \text{ rate (m}^3\text{/m}^2\text{.day)=}\frac{Q(1+r)}{A}$$

and the organic loading rate is obtained as;

$$organic\ \ loading\ \ rate\ \text{ (kg BOD/m}^3\text{.}day\text{)} = \frac{Q \times S\ /1000}{V}$$

Where;

Q_r = flow rate of recirculated flow, m^3/day.

Q = average flow rate of influent flow, m^3/day.

A= surface area of filter, m^2.

S= influent BOD concentration, mg/l.

V= filter volume (=A×h), m^3.

h= filter depth, m.

Recirculation in Trickling filters

Recirculation of the filter effluent or final effluent (secondary settling tank effluent) has the following advantages:

- It permits higher organic loading rates.
- It provides higher hydraulic loading on the filter to improve the liquid distribution and better control of slime layer thickness.
- It provides more oxygen in the influent wastewater flow.
- It returns viable organisms.
- It reduces the nuisance from odors and flies.

Single- and two-stage Trickling Filters

Intermediate and high rate trickling may be designed as single- or two-stage processes. Flow diagrams for some examples of trickling filter configurations are shown in Fig.(4-17).

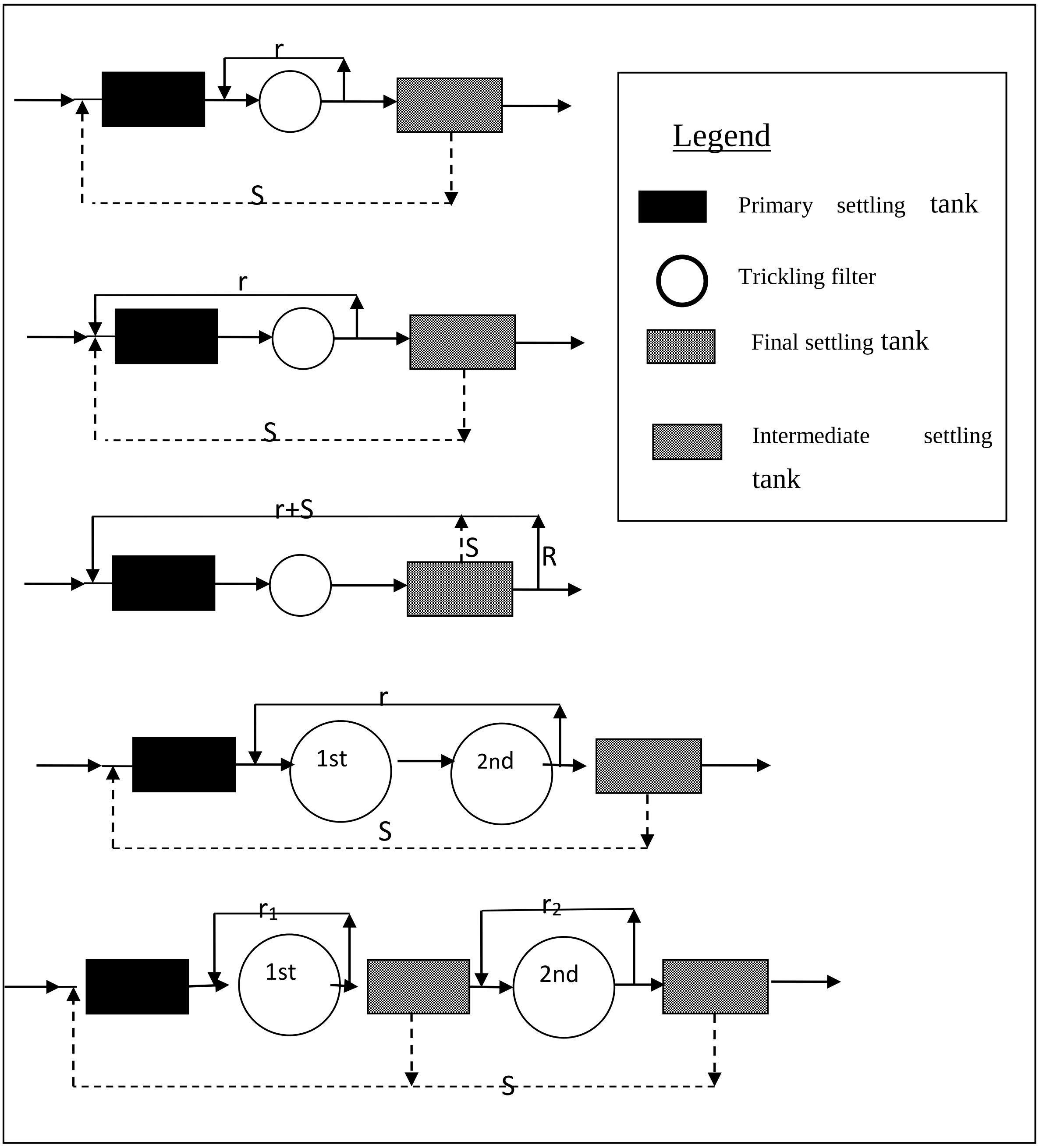


Fig.(4-17) Examples of trickling filter configurations

Design of Trickling Filters

The following equation is used for a single-stage system and the first stage of a two-stage system:

$$\frac{S_i - S_{e1}}{S_i} = \frac{1}{1 + 0.532\sqrt{\frac{QS_i}{V_1 F_1}}}$$

Where;
S_i =influent BOD, mg/l.
S_{e1}= effluent BOD of the first stage, mg/l.

Q= average sewage flowrate, m^3/min.

V_1= filter volume of the first stage, m^3.

F_1= recirculating factor of the first stage which is equal to;

$$F_1 = \frac{1 + r_1}{(1 + 0.1r_1)^2}$$

The following equation is used for the second stage;

$$\frac{S_{e1} - S_{e2}}{S_{e1}} = \frac{1}{1 + \frac{0.532}{1 - [(S_i - S_{e1})/S_i]}\sqrt{\frac{QS_{e1}}{V_2 F_2}}}$$

Where;

S_{e2}= effluent BOD of the second stage, mg/l.

V_2= filter volume of the second stage, m^3.

F_2= recirculating factor of the second stage which is equal to;

$$F_2 = \frac{1 + r_2}{(1 + 0.1r_2)^2}$$

<u>For a single stage trickling filter packed with plastic media, the following equation is used;</u>

$$S_{e1} = \frac{S_i}{(r + 1)\exp\left(\frac{0.21D\vartheta^{T-20}}{q(r + 1)^{0.5}}\right) - r}$$

Where;

S_i= influent BOD (mg/l)

S_{e1}= effluent BOD (mg/l)

D= depth of packing (m)

θ= temperature correction coefficient=1.035

r= recirculation ratio

q= hydraulic loading rate (liter/m^2.sec)

T= temperature (˚C)

Example 4-5: Determine the concentration of effluent BOD_5 for the two-stage trickling filter descried below;

- Design flowrate=0.0905 m^3/sec.
- Influent BOD_3 (after primary treatment)= 260 mg/l.
- Diameter of each filter=24m.
- Depth of each filter=1.83m.
- Recirculation flowrate for each filter=0.0594 m^3/sec

Solution

for the first stage;

$$Q = 0.0509 \times 60 = 3.054\, m^3/min$$

$$A_1 = A_2 = \frac{\pi D^2}{4} = \frac{\pi \times 24^2}{4} = 452.2m^2$$

$$V_1 = V_2 = A \times h = 452.2 \times 1.83 = 827.53m^3$$

$r = Q_r$/Q

$r_1 = r_2$ =0.0594/0.0509= 1.167

$$F_1 = \frac{1+r_1}{\left(1+0.1r_1\right)^2} = \frac{1+1.167}{(1+0.1\times 1.167)^2} = 1.74$$

$$\frac{260-S_{e1}}{260} = \frac{1}{1+0.532\sqrt{\frac{3.054\times 260}{827.53\times 1.74}}}$$

S_{e1}=73.63 mg/l

For the second stage; F_2= F_1 =1.74

$$\frac{S_{e1}-S_{e2}}{S_{e1}}=\frac{1}{1+\frac{0.532}{1-[(S_i-S_{e1})/S_i]}\sqrt{\frac{QS_{e1}}{V_1F_1}}}$$

$$\frac{73.63-S_{e2}}{73.63}=\frac{1}{1+\frac{0.532}{1-[(260-73.63)/260]}\sqrt{\frac{3.054\times 73.63}{827.53\times 1.74}}}$$

S_{e2}=31.37 mg/l

Example 4-6

Determine the diameter of a single stage rock media an applied filter to an applied BOD of 125mg/l to 25mg/l. use a flowrate of 0.14m^3/sec, recirculation ratio of 2 and filter depth of 1.83m.

Solution:

$$Q=0.14\times 60=8.4m^3/min$$

$$F_1=\frac{1+r_1}{(1+0.1r_1)^2}=\frac{1+2}{(1+0.1\times 2)^2}=2.083$$

$$\frac{S_i-S_{e1}}{S_i}=\frac{1}{1+0.532\sqrt{\frac{QS_i}{V_1F_1}}}$$

$$\frac{125-25}{125}-\frac{1}{1+0.532\sqrt{\frac{8.4\times 125}{V_1\times 2.083}}}$$

V_1=2282.45m^3

$$V=A\times h$$

A=V/h=2282.45/1.83=1247.24m^2=$\frac{\pi D^2}{4}$

D=39.85m

Example 4-7:

A single stage trickling filter is designed to treat sewage at a flow rate of 7200m^3/day and produce an effluent has BOD_5 of 20mg/l. The BOD_5 of raw sewage is 200mg/l and the percent of BOD_5 removal in primary settling tank is 30%. The Hydraulic loading rate is 18 m^3/m^2.day and the filter depth is 3m. Determine; (a) the filter diameter, (2) recirculation factor, (3) recirculation florwrate, and (4) organic loading rate.

Solution:

$$Q = \frac{7200}{24 \times 60} = 5m^3/min$$

Influent BOD_5 =200-0.3×200=140mg/l

Effluent BOD_5 =20 mg/l

$$\frac{S_i - S_{e1}}{S_i} = \frac{1}{1 + 0.532\sqrt{\frac{QS_i}{V_1 F_1}}} \rightarrow \frac{140-20}{140} = \frac{1}{1+0.532\sqrt{\frac{5\times 140}{V_1 F_1}}}$$

$$\sqrt{V_1 F_1} = 84.35$$

$V_1 = A \times h \rightarrow \sqrt{A_1 \times 3 \times F_1}$=84.35

$$A_1 F_1 = 2371.86$$

$A_1 \frac{(1+r_1)}{(1+0.1r_1)^2}$=2371.86………..(1)

$hydraulic\ load = \frac{Q(1+r)}{A}$=18

$$\frac{7200(1+r)}{A} = 18$$

$A = 400(1+r)$…………..(2)

Sub. Eq.2 into Eq.1;

$\frac{400(1+r_1)(1+r_1)}{(1+0.1r_1)^2}$=2371.86

r_1 =1.9 (recirculation ratio)

Sub. r_1 value in Eq.2 gives A=400(1+1.9)=1160 m^2

A=$\frac{\pi D^2}{4}$=1160→D=38.43m (filter diameter)

$$Q_r = r \times Q = 1.9 \times 7200 = \frac{13680m^3}{day}$$

$$organic\ loading = \frac{QS_i}{V} = \frac{7200 \times 140/1000}{3 \times 1160} = 0.29\ kg\frac{BOD}{m^3}.day$$

4-6-3 Secondary Settling Tank

The design of secondary clarifier is very important to the operation of biological treatment system. Like primary settling tank, secondary settling tanks may have circular or rectangular shape and compose of the same elements. The difference between them is in the design criteria.

4-6-3-1 Design Criteria

For secondary settling tank after activated sludge system;

- Surface overflow rate= 16-28 m^3/m^2.day at average flow.
- Surface overflow rate= 40-64 m^3/m^2.day at peak flow.
- Solid loading = 4-6 kg/m^2 per hour at average flow.
- Solid loading = 8 kg/m^2 per hour at average flow.
- Water depth=3.5-6m
- Weir loading rate is similar to that of primary settling tank.

Where;

$$Surface\ overflow\ rate\ (SOR)\ = Q/A$$

$$Solid\ loading = \frac{(Q + Q_r) \times MLSS}{A}$$

A = surface area of tank, m^2

For secondary settling tank after trickling filters, the adopted design criteria are;

- SOR= 25 to 33m/day at average flow and should not exceed 50m/day at peak flow.
- Weir loading rate and detention time are similar to those of primary settling tanks.

- The recirculated flow is included when sizing the settling tank if it actually passes through the tank.
- Sludge return is generally neglected since it is relatively small in trickling filters.

Example 4-8:

Design secondary settling unit following aeration tank in an activated sludge system. The activated sludge system treats sewage at average flowrate of 10000m^3/day and peak flowrate of 25000m^3/day. It was designed to have recirculated flowrate of 2500m^3/day and mixed liquor volatile suspended solids of 2000mg/l. Assume MLVSS/MLSS=0.7.

Solution

Assume SOR at average sewage flow=25m/day (16-28 m^3/m^2.day)

SOR=Q/A →A=Q/SOR=10000/25=400m^2

Check SOR at peak sewage flow;

SOR at peak sewage flow=25000/400=62.5 m/day …O.K.,

since Surface overflow rate= 40-64 m^3/m^2.day at peak flow.

Check solid loadings;

- Solid loading = 4-6 kg/m^2 per hour at average flow.
- Solid loading = 8 kg/m^2 per hour at peak flow.

$$Solid\ loading = \frac{(Q + Q_r) \times MLSS}{A}$$

MLSS=2000/0.7= 2857.14 mg/l

At average flowrate;

$$Solid\ loading = \frac{(10000+2500)\times 2857.14/1000}{400}$$ =89.3 kg/m^2.day

Solid loading =3.72 kg/m^2.hr …O.K.

At peak flowrate;

$Solid\ loading = \frac{(25000+2500)\times 2857.14/1000}{400}$ =196.42 kg/m^2.day

Solid loading =8.18 kg/m^2.hr>8 kg/ m^2.hr…Not O.K.

Let solid at peak flow=8 kg/ m^2.hr=192 kg/m^2.hr

$$Solid\ loading = \frac{(Q + Q_r) \times MLSS}{A}$$

$$192 = \frac{(25000 + 2500) \times 2857.14/1000}{A}$$

A= 409.22 m^2

Assume number of tanks=2

Area of one tank=409.22/2=204.61 m^2

Use circular tanks;

$$A = \frac{\pi}{4}\left[D^2 - {D_S}^2\right]$$

Let stilling well diameter (D_S)=0.1D

$$204.61 = \frac{\pi}{4}[D^2 - (0.1D)^2]$$

D=16.22m

Ds=0.1×16.22=1.62m

Check weir loading rate;

Weir loading rate= $\frac{Q_{one}}{\pi D} = \frac{25000/2}{\pi \times 16.22} = 245.4$ m^3/m.day <370 m^3/m.day…O.K.

Assume side water depth=4m

Total side depth of tank =1.1×4=4.4m

4-7 Wastewater Sludge

Wastewater sludge can be classified generally as primary and secondary (also called biological), and chemical. Sludge contains settleable solids such as fecal material, fibers, silt, food wastes, biological flocs, organic chemical compounds, and inorganics, including heavy metals and trace minerals. The sludge is raw sludge when it is not treated biologically or chemically for volatile solids or pathogen reduction. When the sludge is treated, the resulting biosolids can be classified by the treatment, such as thickened, aerobically digested, anaerobically digested, dewatered, and thermally dried. The treated sludge can be only primary or secondary, or a mixture of the two sludge types.

4.7.1 Primary Sludge

Most wastewater treatment plants use primary settling to remove settleable solids from raw wastewater. In a typical plant with primary settling and a conventional activated sludge secondary treatment process, the dry weight of the primary sludge solids is about 50% of that for the total sludge solids. The total solids concentration in raw primary sludge can vary between 2 and 7%. Compared to biological sludge, primary sludge can be dewatered rapidly because it is comprised of discrete particles and debris and will produce a drier cake. However, primary sludge generates an unpleasant odor if it is stored without treatment.

4.7.2 Secondary Sludge

Secondary sludge, also known as *biological sludge*, is produced by biological treatment processes. Plants with primary settling normally produce a fairly pure biological sludge as a result of the bacteria consuming the soluble and insoluble organics in biological treatment system. The sludge will also contain those solids that were not readily removed by primary clarification. Activated sludge and trickling filter sludge generally contain solids concentrations of 0.4 to 1.5%

and 1 to 4% respectively, in dry solids weight. Secondary sludge is more difficult to dewater than primary sludge because it is composed mainly of light biological flocs.

4.8 Sludge Quantity

Determining the quantity of sludge produced in the treatment of wastewater is required for the sizing of sludge processing units and equipment such as sludge pumps, storage tanks, thickeners, digesters, and dewatering systems. Generally, solids production rates range between 0.2 and 0.3 kg/m^3 of wastewater treated.

4.8.1 Primary Sludge

Primary sludge solids production can vary typically from 0.1 to 0.3 kg/m^3 of wastewater. The most common approach in estimating primary sludge production is by computing the quantity of suspended solids entering the treatment plant and assuming a removal efficiency of suspended solids. The solids removal efficiency can be correlated to either the hydraulic detention time or the surface overflow rate of the primary clarifier. The following equation is adopted to find the removal percentage of suspended solids based on detention time:

$$Suspended\ solids\ removal\ percentage\ (\%) = \frac{t}{0.406 + 0.0152t}$$

where *t* is the detention time (min.)

Solids production in primary settling tank is obtained as;

$$Solid\ production\ \left(\frac{kg}{day}\right) = Q \times SS_{in} \times SS\ removal\ percentage/1000$$

where Q is wastewater flowrate (m^3/day) and SS_{in} is influent suspended solids (mg/l). The relation of the solids removal percentage to the surface overflow rate is presented in Fig. (4-8).

4.8.2 Secondary Sludge

- For activated sludge system, solid production=$P_{x_{ss}}$ (see section 4-6-1-1)

- For trickling filter, solid production is estimated to be 0.2 to 0.5 kgVSS per kg BOD_5 removed.

4-9 Sludge Treatment Units

4-9-1 Sludge Thickening System

Thickening is a procedure used to increase the solid content of sludge by removing a portion of the liquid fraction. Thickening is generally accomplished by physical means, including gravity settling, flotation, and centrifugation.

4-9-1-1 Gravity thickening

Gravity thickening is accomplished in a tank similar in design to a conventional sedimentation tank, see Fig. (4-18). Normally a circular tank is used. Dilute sludge is fed to a centre feed well. The feed sludge is allowed to settle and compact, and the thickened sludge is withdrawn from the bottom of the tank. Conventional sludge –collecting mechanisms with deep trusses (Picket fence) stir the sludge gently, thereby opening up channels for water to escape and promoting densification. The continuous supernatant flow that results is returned to the primary settling tank. The thickened sludge is pumped to the digesters.

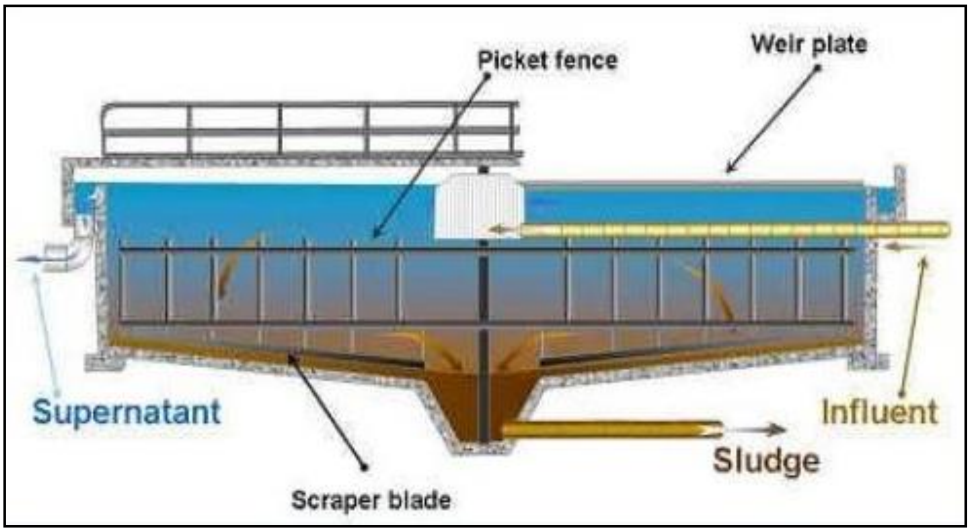


Fig.(4-18): Vertical profile in a gravity thickener

Design Criteria

Gravity thickeners are designed on the basis of hydraulic surface loading and solid loading. For primary plus activated sludge thickening, the following design criteria are recommended:

- Hydraulic surface loading (Q/A) =16 to 36 m^3/m^2.day
- Solid loading = 40 to 80 kg VSS /m^2.day

Where; Q is the flowrate of sludge (primary plus activated sludge) and A is the surface area of tank.

4-9-2 Sludge Digestion

Digestion is a form of stabilization where the volatile material in the wastewater solids can be decomposed naturally and the potential for odor production is reduced. Sludge stabilization can be accomplished using anaerobic or aerobic digestion processes.

4-9-2-1 Anaerobic sludge digestion

The anaerobic digestion process has been widely used for the stabilization of domestic waste sludge. During digestion the volatile material is reduced and the colloidal water binding structure of the sludge is destroyed. The effluent solid may dewater easier and possibly at less cost than if not digested. Anaerobic digestion has the added benefit of producing methane gas which can be recovered and used as a source of energy. The kinetics of sludge digestion is governed by two sequent groups of bacteria (Fig.4-19): Facultative anaerobic acid producers that convert carbohydrates, proteins, and fats into organic acids and alcohols; and, anaerobic methane fermenters that convert the acids and alcohols into methane and carbon dioxide.

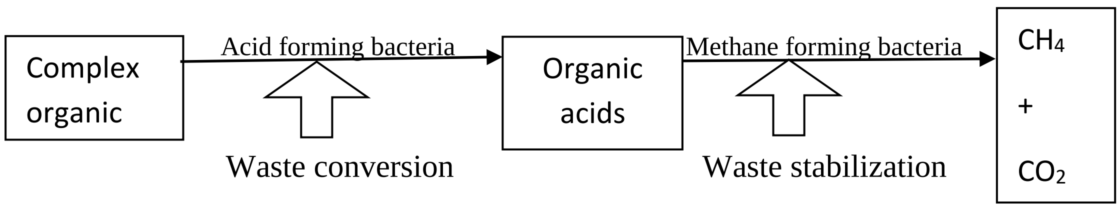


Fig.(4-19): Degradation of organic materials

Types of Anaerobic Digesters

Two types of anaerobic digesters are used, standard-rate (conventional) and high rate. In the conventional digestion process (Fig.4-20a), the contents of digester are usually unheated and unmixed. Detention times for this process vary from 30 to 60 days. In a high-rate digestion process (Fig.4-20b), the contents of the digester are heated and completely mixed. The required detention time is 15days or less. A combination of these two basic processes is known as the two-stage processes (Fig.4-20c). The primary function of the second stage is to separate the digested solids from the supernatant liquor.

Digestion Tank Elements

A number of elements should be considered in designing digesters. These include; tanks, heat exchangers, mixers, piping, recirculating pumps, cover design, and gas safety equipment.

The most common shape of digestion tank is circular with diameters range from 6 to 35m. A slope (80mm/m) is required in the tank bottom for removal of discrete solids.

Two types of covers are used for digestion tanks-fixed and floating. If it is desired to have a floating cover, the preference is to have it on the secondary digester, while the primary digester would have a fixed cover. It is advisable to provide at least two manholes in tank cover of 0.6 to 0.7m diameter or one central manhole of 1.5m diameter.

The piping system in digester includes the gas piping, sludge pipes (sludge feed inlet pipe and sludge withdrawal pipe), heating pipe, and recirculation pipe. In small tanks, one inlet pipe is necessary for discharge near tank centre. In large tanks, two or more inlet pipes are required tro provide better distribution of feed to the tank.

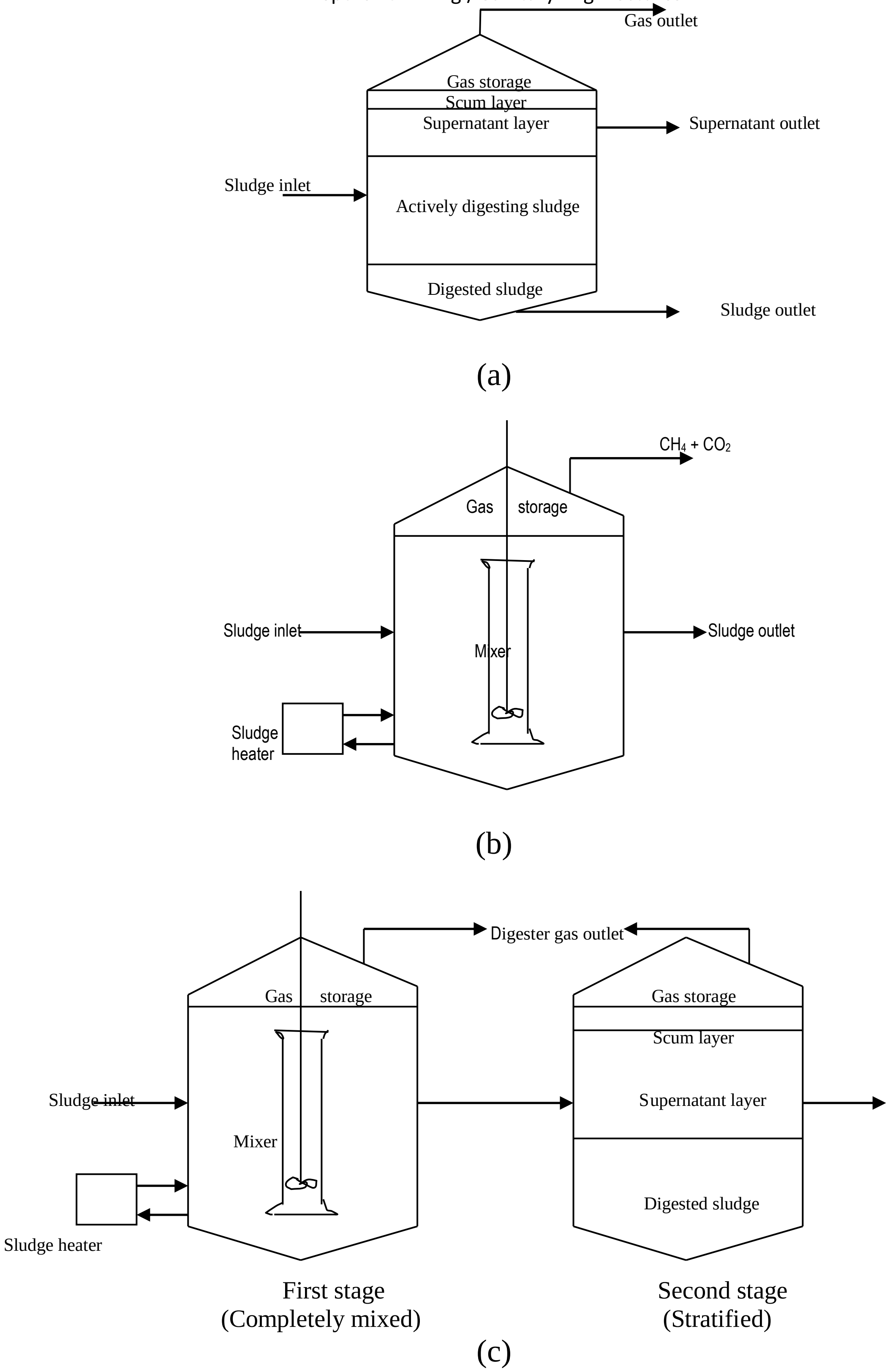


Fig.(4-20): Typical anaerobic digesters. (a) Conventional single-stage process. (b) High-rate single-stage process. (c) Two-stage process.

Design Criteria

Typical design criteria for high rate anaerobic digesters are as given below;

- Solid retention time (SRT) =10 to 20 days
- Solid loading = 1.6 to 6.4 kg VSS/day/m^3 of bulk tank volume
- Volume criteria = 0.075 to 0.112 m^3/capita (for primary plus activated sludge)
- Digester under flow concentration = 4 to 6%

The dependence of SRT on temperature is illustrated in Table (4-6).

Table (4-6) The effect of temp. on SRT.

Operation temperature (˚C)	SRT, days
10	55
18	40
24	30
30	25
35	20
38	20
44-60	15-10

Criteria of digesters performance

Whether or not sludge digestion tanks are operating properly can be checked in a number of different ways. Some of these are listed below:

1. The amount of gas collected should not fall below 1.2kg per kg volatile solids destroyed.
2. The pH of the digested sludge should not fall below 6.8.
3. Excessive amounts of volatile acids in digesting sludge are evidence that the tank is being overloaded with organic matter. A value of 2000mg/l as acetic acid is critical.
4. The odor of the sludge drawn should be tarry.
5. The sludge should release its water rapidly and leave clear filtrate when it is dewatered on sand beds.

4-9-2-2 Aerobic sludge digestion

Aerobic digestion of sludge is a natural biological degradation and purification process in which bacteria that thrive in oxygen-rich environments break down and digest the waste. During this oxidation process, pollutants are broken down into carbon dioxide, water, nitrates, sulfates and biomass (micro-organisms). By optimizing the oxygen supply with so called aerators the process can be significantly accelerated.

4-9-3 Dewatering Systems

Dewatering is a physical unit operation used to reduce the moisture content of sludge. Dewatering devices use a number of techniques for removing moisture. Some rely on natural evaporation and percolation to dewater the solids (drying beds). Mechanical dewatering devices use mechanically assisted physical means to dewater the sludge more quickly (filtration, vacuum withdrawal, centrifugal settling, and compaction). Drying beds system is the most economical technique for sludge dewatering in hot climate countries.

4-9-3-1 Sludge drying beds

Sludge-drying beds are used to dewater digested sludge. Sludge is placed on the beds in 200 to 300mm layer and allowed to dry. Dried sludge has a coarse, cracked surface and is black or dark brown. The moisture content is approximately 60% after 10 to 15days under favourable conditions. After drying the sludge is removed and then used as a fertilizer. Sludge removal is accomplished by manual shovelling into trucks.

The drying area is partitioned into individual beds, approximately 6m wide by 6 to 30m long. The interior partitions are of 380 to 460mm height. A typical cross section of a drying bed is shown in Fig.(4-21). In drying beds, the thickness of sand layer varies on the range (10-25) cm and the thickness of gravel layer varies on the range (20-45)cm. Fig.(4-22) shows photos of drying beds.

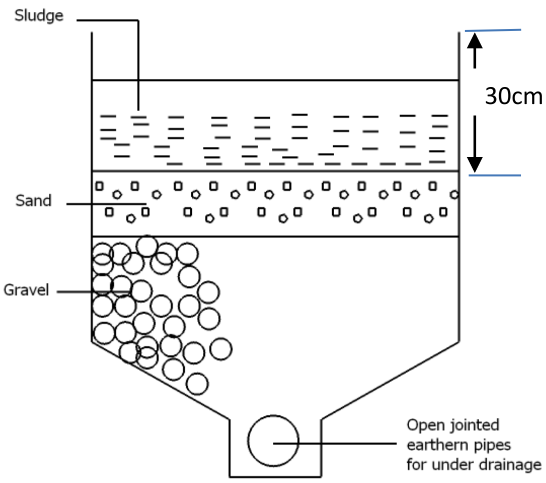


Fig.(4-21): Typical section in a drying bed

Fig.(4-22) Photos of drying beds

<u>Design Criteria</u>

For open sludge drying beds the following criteria are adopted:

Area requirement = 0.16 to 0.275m^2/capita

Sludge loading = 60 to 100kg dry solid/m^2.year

<u>Example 4-10</u>

Determine the quantity of solids in primary sludge of a sewage treatment plant treats sewage at a peak sewage flowrate of 36000m^3/day. Assume the primary sedimentation unit is composed of three tanks, each tank has a volume of 1000m^3 and the total suspended solids of raw sewage is 300mg/l.

<u>Solution</u>

$$Solid\ production\ \left(\frac{kg}{day}\right) = Q \times SS_{in} \times SS\ removal\ percentage/1000$$

$$Suspended\ solids\ removal\ percentage\ (\%) = \frac{t}{0.406 + 0.0152t}$$

To find the detention time (t);

$$Q = \frac{V}{t} \rightarrow t = \frac{V}{Q}$$

For one tank;

$$Q_{one} = \frac{36000}{3} = 12000m^3/day$$

$$\therefore t = \frac{1000}{12000} = 0.0833\ day = 120min$$

$$\therefore Suspended\ solids\ removal\ percentage\ (\%) = \frac{120}{0.406 + 0.0152 \times 120} = 53.8\%$$

$$\therefore Solid\ production\ \left(\frac{kg}{day}\right) = 36000 \times 300 \times \frac{0.538}{1000} = 5810.4kg/day$$

Example 4-11

Find the quantity of the solids produced in a secondary sludge of an activated sludge system using the following data:

- Biomass yield=0.5
- Microorganisms decay coefficient=0.06/day.
- Sludge age= 10days.
- Average flowrate of settled sewage= 10000m^3/day
- SOR of primary sedimentation unit= 40m/day.
- Soluble BOD if effluent of aeration tank=1.1mg/l
- MLVSS/MLSS=0.8

Solution

Solids quantity=$P_{x_{ss}}$

$$Y_{obs} = \frac{Y}{1 + K_d\theta_c} = \frac{0.5}{1 + 0.06 \times 10} = 0.3125$$

$$P_x = Y_{obs}Q(S_o - S)$$

To find So;

For SOR=40m/day, % of BOD removal in primary sedimentation unit=31% (from Fig.4.8)

$$\therefore S_o = (1 - 0.31) \times 200 = 138 mg/l$$

$$P_x = 0.3125 \times 10000 \times (138 - 1.1) = 427.8\ kg/day$$

$$P_{x_{ss}} = \frac{P_x}{MLVSS/MLSS} = \frac{427.8}{0.8} = 534.8\ kg/day$$

Example 4-12

A trickling filter treats sewage at a flowrate of 7000m^3/day. The influent and effluent BOD are 250 and 40mg/l, respectively. estimate the solids quantity in secondary sludge.

Solution

For trickling filter, solid production is estimated to be 0.2 to 0.5 kgVSS per kg BOD_5 removed.

Let solid production = 0.3 kgVSS per kg BOD_5 removed.

kg BOD_5 removed=$(250 - 40) \times \frac{7000}{1000} = 1470 kg/day$

$$\therefore solids\ production = 0.3 \times 1470 = 441\ kg/day$$

Example 4-13

Determine the total flowrate of primary sludge and waste activated sludge using the following data;

- Sewage flowrate= 5000m^3/day
- % of SS removal in primary sedimentation unit= 60%
- Moisture content of primary sludge=93%
- Volume of aeration tank=1500m^3
- MLVSS=2400mg/l
- Return VSS concentration= 12000mg/l
- Sludge age=10 days.

Solution

$$Mass\ of\ settled\ solids\ in\ primary\ sedimentation\ unit$$

$$= 0.6 \times 300 \times \frac{5000}{1000} = 900\ kg/day$$

$$Moisture\ content = \frac{M_W}{M_W + M_S} \times 100$$

$$93 = \frac{M_W}{M_W + 900} \times 100 \rightarrow M_W = 11957kg/day$$

Let $\rho_s = 1250kg/m^3$ and $\rho_w = 1000kg/m^3$

$$Daily\ Volume\ of\ primary\ sludge = \frac{M_S}{\rho_s} + \frac{M_W}{\rho_W} = \frac{900}{1250} + \frac{11957}{1000}$$

$$= 12.67m^3/day$$

Waste activated sludge flowrate=Q_{wr}

$$Q_{wr} = \frac{VX}{\theta_c X_r} = \frac{1500 \times 2400}{10 \times 12000} = 30\ m^3/day$$

$$\therefore Total\ sludge\ flowrate\ (primary\ sludge\ plus\ waste\ activated\ sludge)$$

$$= 12.67 + 30 = 42.67m^3/day$$

Example 4-14

A gravity thickener receives sludge at a flow rate of 42.67 m^3/day (see example 4-13). The total mass of volatile suspended solids of the sludge is 400 kg/day. Find the surface area of the thickener.

Solution

Let hydraulic surface loading =25 m^3/m^2.day

$$hydraulic\ surface\ loading = \frac{Q}{A} \rightarrow A = \frac{42.67}{25} = 1.707\ m^2$$

- Check solid loading;
- $Solid\ loading = \frac{400}{1.707} = 234.3\ kg\ VSS\ /m^2.day > 80$ kg VSS / m^2. day

Not O.K

Let solid loading= 80 kg VSS /m^2.day

$$A = \frac{400}{80} = 5m^2$$

Example 4-15

A sewage treatment plant serves a population of 100000. It is composed of two anaerobic digesters and 20 drying beds. Find the volume of each digester and the surface area of each drying bed.

Solution

For digesters;

Volume criteria = 0.075 to 0.112 m^3/capita (for primary plus activated sludge)

Let digester volume= 0.1m^3/capita

$$\therefore Total\ volume\ of\ digesters = 0.1 \times 100000 = 10000m^3$$

$$Volume\ of\ one\ digester = \frac{10000}{2} = 5000\ m^3$$

For drying beds;

Area requirement = 0.16 to 0.275 m^2/capita

Let drying beds area= 0.2 m^2/capita

$$\therefore Total\ area\ of\ drying\ beds = 0.2 \times 100000 = 20000\ m^2$$

$$Area\ of\ one\ drying\ bed = \frac{20000}{20} = 1000\ m^2$$

Example 4-16

A wastewater plant produces 1000 kg of dry solids per day at moisture content of 95%. The solids are 70% volatile with a specific gravity of 1.05 and 30% nonvolatile with specific gravity of 2.5. Determine the sludge volume

a. As produced.
b. After digestion, the digester reduces the volatile solids content by 50% and decreases the moisture content by 90%.
c. After dewatering to 75% moisture.
d. After drying to 10% moisture.

e. After incineration (only the nonvolatile solids remain).

Solution

(a) Produced sludge volume

$$Moisture\ content = \frac{M_W}{M_W+M_S} \times 100$$

$$95 = \frac{M_w}{M_w + 1000} \times 100 \rightarrow M_w = 19000kg$$

The mass of total solids is 1000 kg of which 700 kg (70%) volatile and at sp. gr. of 1.05 (ρ=1.05×1000=1050 kg/m^3), thus;

$$Volume\ of\ volatile\ solids\ = \frac{700}{1050} = 0.667\ m^3$$

The mass of nonvolatile solids=1000-700= 300 kg at sp. gr. of 2.5 (ρ=2.5×1000=2500 kg/m^3), thus;

$$Volume\ of\ nonvolatile\ solids\ = \frac{300}{2500} = 0.12\ m^3$$

Mass of water=19000 kg

$$Volume\ of\ water\ = \frac{19000}{1000} = 19\ m^3$$

$$\therefore Total\ volume\ of\ sludge$$
$$= Water\ volume + volume\ of\ volatile\ solids$$
$$+ volume\ of\ nonvolatile\ solids$$

$$Total\ volume\ of\ produced\ sludge = 19 + 0.667 + 0.12 = 19.787\ m^3$$

(b) After digestion;

$$Volatile\ solids\ mass = 0.5 \times 700 = 350\ kg$$

$$\therefore Total\ solids\ content$$
$$= mass\ of\ nonvolatile\ solids + mass\ of\ volatile\ solids$$

$$Total\ solids\ content = 300 + 350 = 650\ kg$$

$$Moisture\ content = \frac{M_W}{M_W + M_S} \times 100$$

$$90 = \frac{M_w}{M_w + 650} \times 100 \rightarrow M_w = 5850kg$$

$$Volume\ of\ volatile\ solids\ = \frac{350}{1050} = 0.333\ m^3$$

$$Volume\ of\ nonvolatile\ solids\ = \frac{300}{2500} = 0.12\ m^3$$

$$Volume\ of\ water\ = \frac{5850}{1000} = 5.85\ m^3$$

$$Total\ volume\ of\ \ sludge = 5.85 + 0.333 + 0.12 = 6.303\ m^3$$

(c) After dewatering to 75% moisture;

$$75 = \frac{M_w}{M_w + 650} \times 100 \rightarrow M_w = 1950kg$$

$$Volume\ of\ volatile\ solids\ = \frac{350}{1050} = 0.333\ m^3$$

$$Volume\ of\ nonvolatile\ solids\ = \frac{300}{2500} = 0.12\ m^3$$

$$Volume\ of\ water\ = \frac{1950}{1000} = 1.95\ m^3$$

$$Total\ volume\ of\ \ sludge = 1.95 + 0.333 + 0.12 = 2.403\ m^3$$

(d) after drying to 10% moisture;

$$10 = \frac{M_w}{M_w + 650} \times 100 \rightarrow M_w = 72.2kg$$

$$Volume\ of\ volatile\ solids\ = \frac{350}{1050} = 0.333\ m^3$$

$$Volume\ of\ nonvolatile\ solids\ = \frac{300}{2500} = 0.12\ m^3$$

$$Volume\ of\ water\ = \frac{72.2}{1000} = 0.072\ m^3$$

$$Total\ volume\ of\ \ sludge = 0.072 + 0.333 + 0.12 = 0.525\ m^3$$

(e) After incineration (only the nonvolatile solids remain).

$$Total\ volume\ of\ \ sludge = Volume\ of\ nonvolatile\ solids = 0.12m^3$$