Sanitary and Environmental Engineering

PART 1: WATER SUPPLY ENGINEERING

Sanitary and Environmental Engineering (Syllabus)

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Text Books:

1) Water Supply and Sewerage by: E. W. Steel and T. J. McGhee

2) Water Supply and Wastewater Engineering (Part 1 and 2) by: D.Lal and A. K. Upadhyay

FIRST SEMESTER

Part One: Water Supply Engineering		
Lecture 1 Chapter One [(1) (2)]	**Introduction**	3-9
Lecture 2 Chapter Two [(1) (2)]	**Quantity of water:** Consumption for various purposes, the per capita demand, variation in rate of consumption, factors affecting consumption, fire demand, forecasting population	10-22
Lecture 3 Chapter Eight [(1)] Chapter Four [(2)]	**Quality of water supplies:** Impurities of water: Potable water, polluted water, water borne diseases, lead poisoning, fluoride, water bacteria, pathogen and coliform, soluble mineral, standards for drinking water	23-36
Lecture 4 Chapter Six [(1) (2)]	**Water distribution systems:** The pipe system, design, of water distribution systems, flow in pipes, equivalent pipe method, hardy cross method	37-55
Lecture 5 Chapter Six[(1)] Chapter Three[(2)]	**Collection and distribution of water:** Intakes, intakes of impounding reservoirs, river intakes	56-64
Lecture 6 Chapter Seven[(1)]	**Pumps and Pumping Stations:** Classification of Pumps, Work and efficiency of pumps, Pump capacity, head, efficiency, and power, Effect of varying speed, Characteristics curves of centrifugal pumps, Suction lift, Cavitation, pump station.	65-73
Lecture 7 Chapter Nine(1) Chapter Five[(2)]	**Treatment of water–Clarification- Plain Sedimentation:** Screens, principles of sedimentation, discrete particles, sedimentation tank details, scour.	74-83
Lecture 8 Chapter Nine(1) Chapter Five[(2)]	**Treatment of water–Clarification- Sedimentation with chemicals (Coagulation and Flocculation):** purposes and action of coagulation, simplified coagulation reactions, mixing, flocculation, suspended solids contact clarifiers, design criteria	84-98
Lecture 9 Chapter Ten[(1)] Chapter Five[(2)]	**Treatment of water-Filtration:** Filter types, rapid filters, theory of filtration through coarse media, filter media, gravel, filtration rates, under drain system, washing process, operating problems, pressure filters	99-112
Lecture 10 Chapter Eleven[(1)] Chapter Five[(2)]	**Miscellaneous water treatment methods-Disinfection:** Chlorine in water, chlorination, chloramines, use of chlorine gas, hypochlorination, other disinfection techniques, water softening, cation exchange method	113-119
Lecture 11 Chapter Eleven[(1)]	**Special Treatments: Hardness Removal or Water Softening:** water hardness, Softening Processes, Ion Exchange method, Membrane filtration.	120-128

PART 1: WATER SUPPLY ENGINEERING

Lecture 1: Introduction

Sanitary Engineering: The branch of civil engineering associated with the supply of water, disposal of sewage, other public health services and the management of water and sewage in civil engineering.

Sanitary Engineer:

1. An expert or specialist in the branch of civil engineering associated with the supply of water, collect and disposal of sewage, and other public health services.
2. An engineer whose training or occupation is in sanitary engineering.
3. An engineer specializing in the maintenance of urban environmental conditions conductive to the preservation of public health.

Work of sanitary engineering:

The development of sanitary engineering has paralleled and contributed to the growth of cities. Without an adequate supply of safe water, the great city could not exist, and life in it would be both unpleasant and dangerous unless and other wastes were promptly removed. The concentration of population in relatively small areas has made the task of the sanitary engineer more complex.

Groundwater supplies are frequently inadequate to the huge demand and surface waters, polluted by cities, towns, and villages on watersheds, must be treated more and more elaborated as the population density increases. Industry also demands more and better water from all available sources. The river receives ever-increasing amounts of sewage and industrial wastes, thus requiring more attention to sewage treatment, stream pollution, and the complicated phenomena of self-purification.

The public looks to the sanitary engineer for assistance in such matters as design, construction, and operation of water and sewage works are treated, the control of malaria by mosquito control, the eradication of other dangerous insects, rodent control collection and disposal of municipal refuse, industrial hygiene, and sanitation of housing and swimming pools. The activities just given, which are likely to be controlled by local or state health departments, are sometimes known as public health or environmental engineering, terms which while descriptive are not accepted by all engineering. The terms, however, are indicative of the important place the engineer holds in the field of public health and in the prevention of diseases.

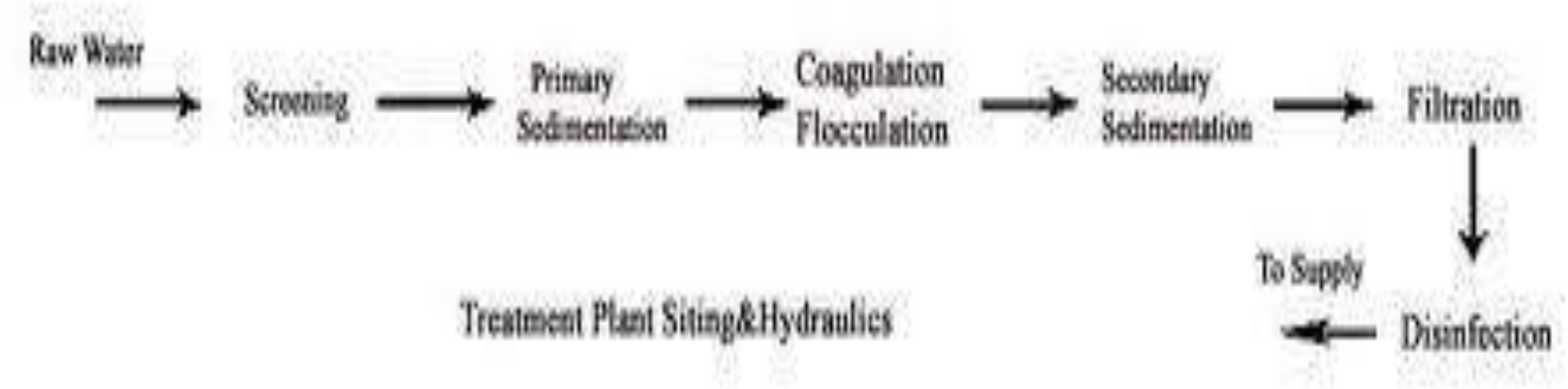
EPA: Environmental Protection Agency

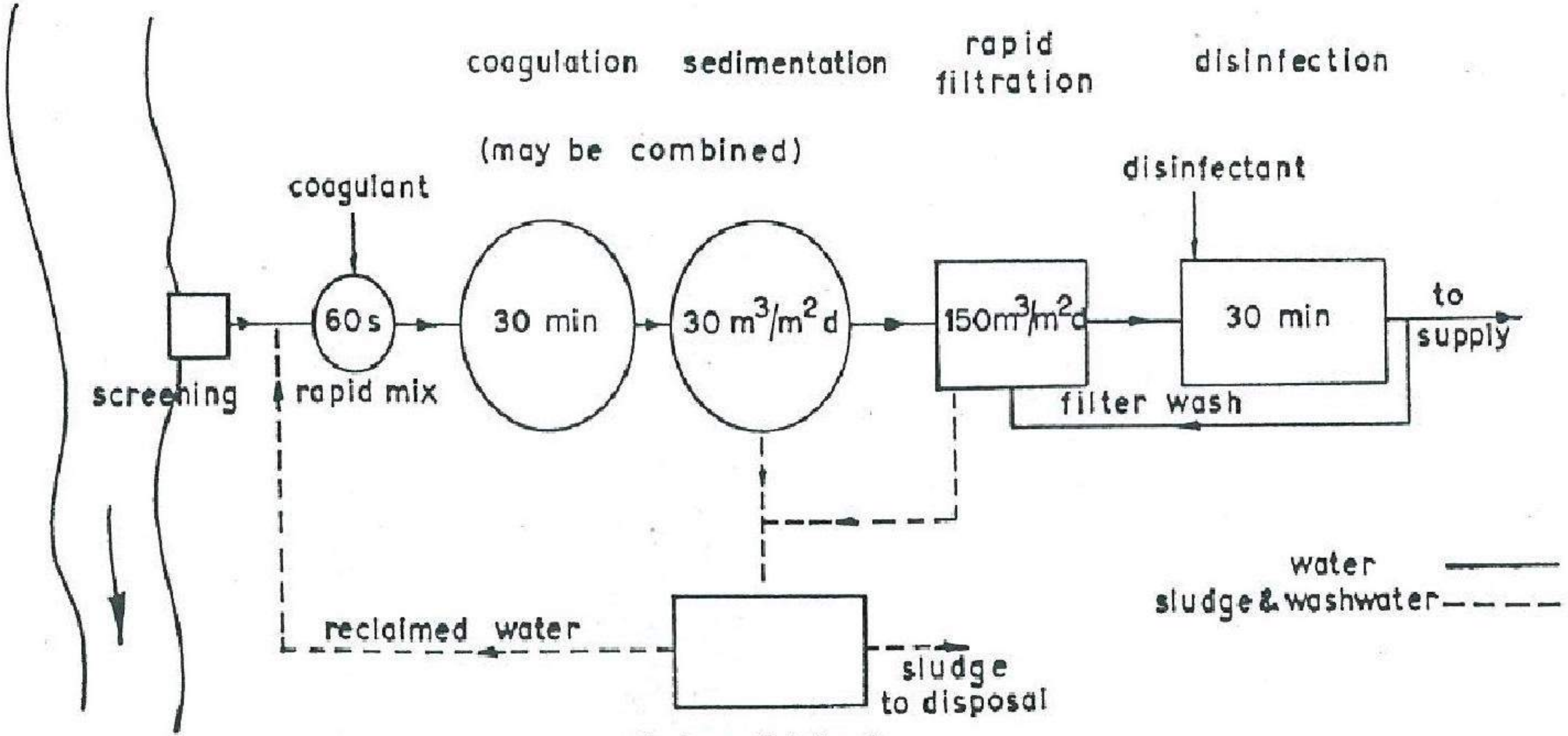
NPSES: National Pollutional Discharge Elimination System

FWPCA: Federal Water Pollution Control Administration

USPHS: U.S. Public Health Service

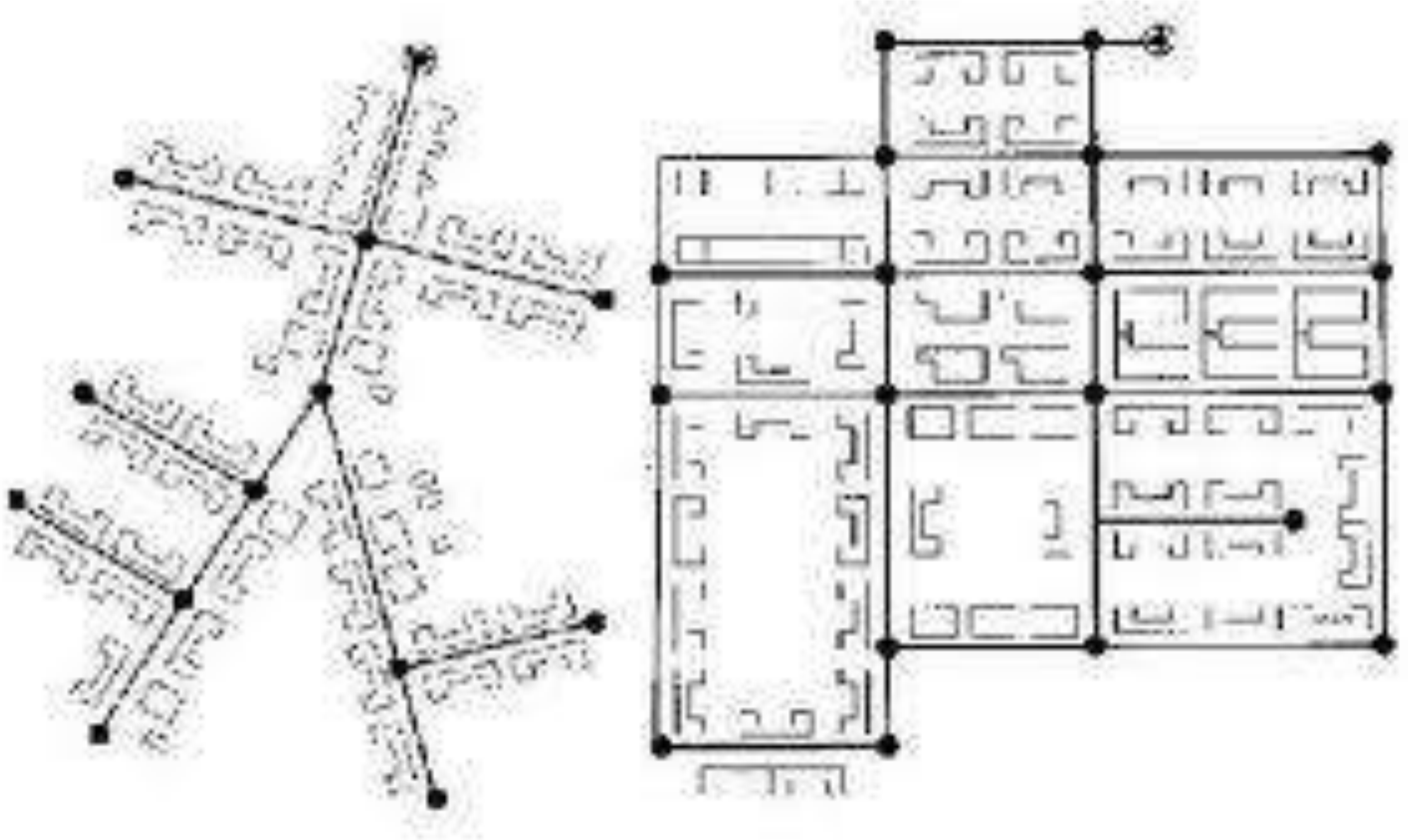
Water treatment plant





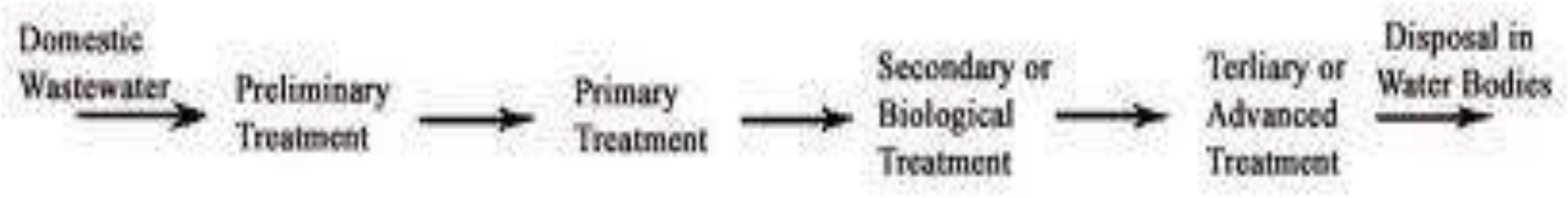
Conventional water treatment plant.

Water supply distribution networks

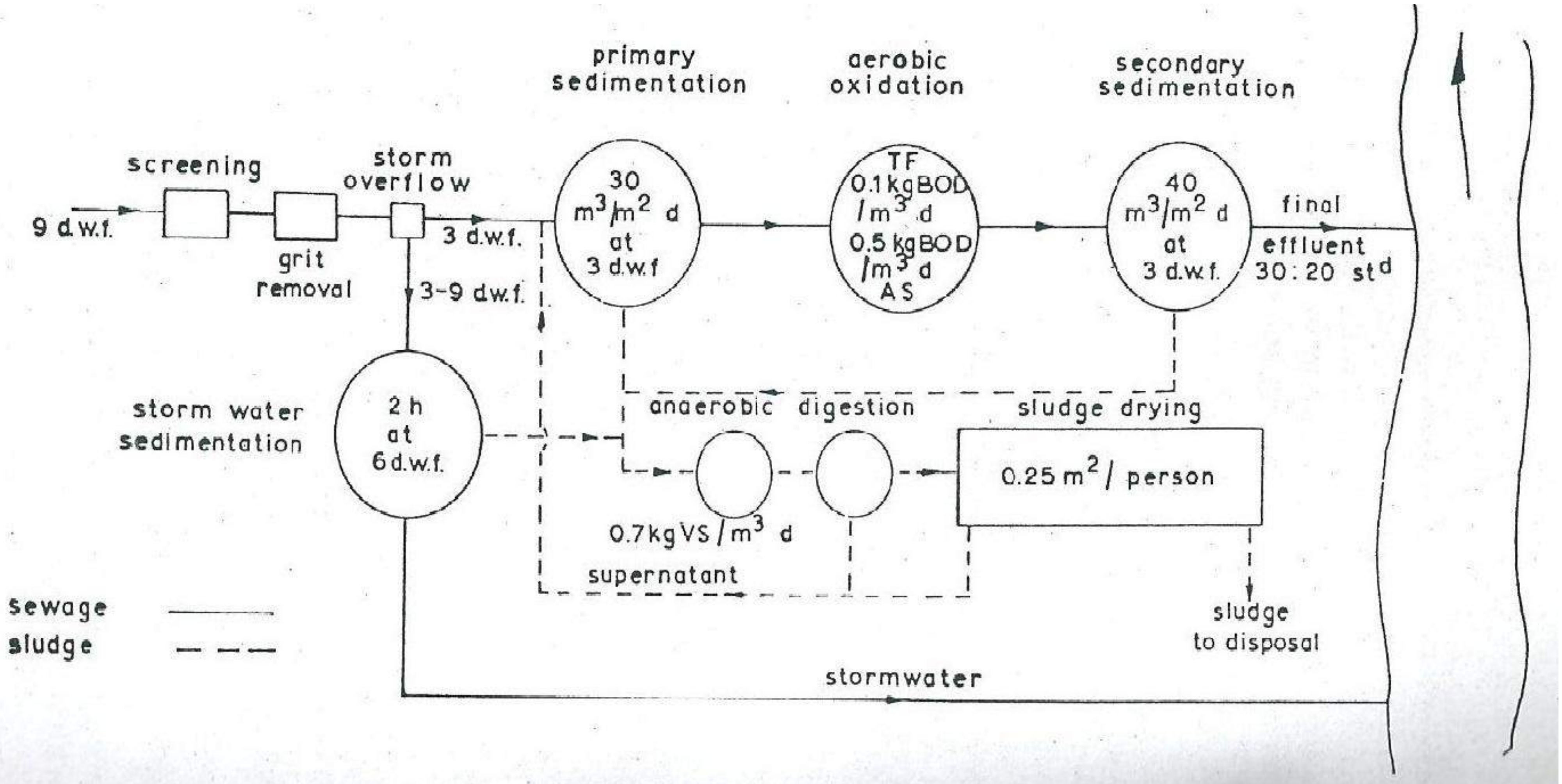


SEWAGE COLLECTION NETWORK

WASTEWATER TREATMENT PLANT



Siting and Hydraulics of Wastewater Treatment Plant



Conventional sewage treatment plant.

Sanitary and Environmental Engineering

PART 1: WATER SUPPLY ENGINEERING

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Lecture 2: Quantity of Water

Water Consumption

In the design of any waterworks project it is necessary to estimate the amount of water that is required. This involves determining the number of people who will be served and their per capita water consumption, together with an analysis of factors that may operate to affect consumption.

It is usual to express water consumption in liters or gallons per capita per day, obtaining this figure by dividing the total number of people in the city into the total daily water consumption. For many purposes the average daily consumption is convenient. It is obtained by dividing the population into the total daily consumption averaged over one year.

$$\text{Water consumption }\left(\frac{\text{L}}{(\text{Cap. day})}\right)=\frac{\text{Total daily water consumption }\left(\frac{\text{L}}{\text{day}}\right)}{\text{Total number of people in the city (Capita)}}$$

$$\text{Average daily consumption }\left(\frac{\text{L}}{(\text{Cap. day})}\right)=\frac{\text{Total daily consumption averaged over one year }\left(\frac{\text{L}}{\text{day}}\right)}{\text{Total population in the city (Capita)}}$$

$$\text{Average daily per capita demand }\left(\frac{\text{L}}{(\text{Cap. day})}\right)=\frac{\text{Quantity Required in 12 Months }\left(\frac{\text{L}}{\text{day}}\right)}{(360\text{ x Population (Capita)}).}$$

If this average demand is supplied at all the times, it will not be sufficient to meet the fluctuations.

Consumption for various purposes (Water Demand):

1. Domestic or Residential use 40 – 60% of the total water demand.
2. Industrial use 25- 30% a)domestic, b)industrial process
3. Commercial use 10 - 15%.
4. Public use 5 -10%.
5. Fire demand

For calculating the total water demand 10 -15% is added for losses and wastage.

The table shows same typical commercial and public water demand flows.

Type	Rate	Type	Rate
Hospital	950 L/bed/ d	Shopping center	6 L/m^2 floor area/d
School	76 L/student/ d	Barber shop	210 L/ chair/d
Rest home	380 L/ bed/d	Laundry	3000 L/machine/d
Restaurant	30 L/customer/d	Airport	10 L/passenger/d
Public parks	1.5 L/m^2/d	Car wash	209 L/ car/d
Store	40 L/person/d	Factory	100 L/employee/d

Domestic or Residential demand:

Domestic demand (volume/time) = rate of consumption (volume/time/capita) X population (capita)

Industrial demand:

For water used in industrial processing, **Symons formula** maybe used water demand =12.2 m^3/ 10^3 m^2 floor area per day. The table shows typical industrial water demands:

Type of industry	Quantity m^3/metric ton
Dairy	2 -3
Chemicals	8 - 10
Meat packing	15 – 25
Canning	30 -60
Paper	200 – 800
Steel	260 - 300
Textile	250 - 350
Petroleum	80 gallon/barrel

Consumption for various purposes (Summary):

Type of Consumption	Use	Purposes	Depend upon	Average Water Demand (L/c.d)	Percentage of Total
Domestic	Houses, hotels, etc	Sanitary, culinary, drinking, washing, bathing, air conditions of residences and irrigation or sprinkling of privately owned gardens or lawns	Living conditions of consumers	190-340	50
Commercial and industrial	Industrial and Commercial plants	Water process according to floor area per day ($12.2m^3/1000m^2$)	Local conditions	200	15-30
Public	Public building and public service	- City halls, jails, and school. - flushing streets and fire protection)	Local conditions	50-75	10
Loss and waste	Uncounted	Network, equipment	Execution degree	50-75	10

Factors affecting water consumption (per capita demand):

1. Size of the city: Per capita demand for big cities is generally large as compared to that for smaller towns as big cities have sewered houses.
2. Presence of industries and commerce.
3. Quality of water: If water is aesthetically and medically safe, the consumption will increase as people will not depend to private wells, etc.
4. Cost of water.
5. Pressure in the water distribution system.
6. Climatic conditions.
7. Characteristics of population: Habits of people and their economic status.
8. Policy of metering and charging method: Water tax is charged in two different ways: on the basis of meter reading and on the basis of certain fixed monthly rate.
9. Efficiency of waterworks administration: Leaks in water mains and services; and unauthorized use of water can be kept to a minimum by surveys.

Fluctuations in Rate of Demand

1. **Annual or yearly variation:**
2. **Seasonal variation:** The demand peaks during summer. Fire break outs are generally more in summer, increasing demand. So, there is seasonal variation.
3. **Monthly variation:**
4. **Weekly variation:**
5. **Daily variation**: Depends on the activity. People draw out more water on holidays and Festival days, thus increasing demand on these days.
6. **Hourly variations** are very important as they have a wide range. During active household working hours i.e. from six to ten in the morning and four to eight in the evening, the bulk of the daily requirement is taken. During other hours the requirement is negligible. Moreover, if a fire breaks out, a huge quantity of water is required to be supplied during short duration, necessitating the need for a maximum rate of hourly supply.

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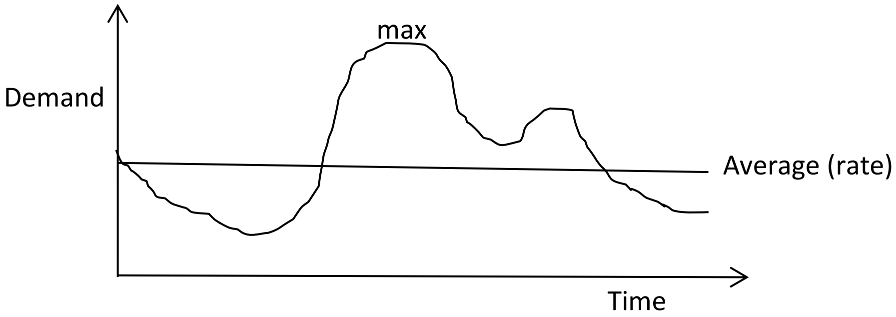


So, an adequate quantity of water must be available to meet the peak demand. To meet all the fluctuations, the supply pipes, service reservoirs and distribution pipes must be properly proportioned. The water is supplied by pumping directly and the pumps and distribution system must be designed to meet the peak demand. The effect of monthly variation influences the design of storage reservoirs and the hourly variations influences the design of pumps and service reservoirs. As the population decreases, the fluctuation rate increases. Figure above shows variation of water consumption with time.

Goodrich formula to estimate the percentage of annual average consumption ($p = 180\, t^{-0.1}$)

Where: p = percentage of the annual average consumption for time t

t = time in day (2hr/24 – 360). Hence; day (t=1), weekly (t=7), monthly (30) and yearly (360).

The formula gives the percentage of the maximum daily as 180 percent, the weekly consumption as 148 percent, and the monthly as 128 percent of the average daily demand. The maximum hourly consumption is likely to be about 150 percent of the maximum for that day.

Maximum hourly demand of maximum day i.e. Peak demand in certain area of a city will affect design of the distribution system while minimum rate of consumption is of less important than maximum flow but is required in connection with design of pump plants, usually it will vary from (25-50) percent of the daily demand.

Average daily demand = Average water consumption (L/Cap.day) x NO. of population

Maximum daily demand = p x average daily demand=1.8 x average daily demand

Maximum weekly demand = p x average daily demand=1.48 x average daily demand

Maximum monthly demand = p x average daily demand=1.28 x average daily demand

Maximum hourly demand = p x maximum daily demand =1.5 x maximum daily demand=1.5 x 1.8 x average daily demand=2.7 x average daily demand

Minimum hourly demand = p x maximum daily demand = 0.5 x maximum daily demand=0.5 x 1.8 x average daily demand=0.9 x average daily demand

The table below shows a proposal of the average domestic water demand. In Iraq it is assumed to be 400 L/c/d.

Single family	Rate L/c/d
Low income	270
Medium income	310
High income	380

Peaks of water consumption in certain areas of a city will affect design of the distribution system.

Fire demand

Firefighting systems: a) with water, b) without water

a) Firefighting systems using water

1-Hydrants: an outdoor system
2-Hose reel: an indoor system
3- Sprinkler: an indoor system

b) Firefighting systems without water

Fire extinguishers are used with engineering criteria for location and number of cylinders. It should not be used for: 1) electrical risk, 2) flammable liquids.

Fire demand formulas: The per capita fire demand is very less on an average basis but the rate at which the water is required is very large. The rate of fire demand is sometimes treated as a function of population and is worked out from following empirical formulas:

a)- Insurance Service Office Formula (ISO):

$$F_{gpm} = 18\,C\,\left(A_{ft^2}\right)^{0.5} \quad F_{L/min} = 223.18\,C\,(A_{m^2})^{0.5} \quad F_{L/sec} = 3.724\,C\,(A_{m^2})^{0.5}$$

F = Fire demand
C = Coefficient related to the type of construction material (0.6 – 1.5), 1.5 wood frame, 1 ordinary frame as brick & 0.6 resistance material.
A = Total floor area (ft^2) (m^2 = 10.76 ft^2) excluding the basement of the building.

Limitations:
1- One fire per day
2- Fire duration 4-10 hr
3- F = 500-8000 gpm

b)- National Board of Fire Underwriters (NBFU):

$$G_{gpm} = 1020\sqrt{P} \times \left(1 - 0.01\sqrt{P}\right) \;\&\; G_{m^3/min} = 3.86\sqrt{P} \times \left(1 - 0.01\sqrt{P}\right)$$

G= fire demand
P = population in thousand (10^3)

Limitations:

1- One fire per day
2- Fire duration 4 -10 hr
3- $P \leq 200 \times 10^3$ capita

	Authority	Formulae (P in thousands)	Q for 1 lakh Population)
1	American Insurance Association	Q (L/min)=4637 $P^{0.5}$ (1-0.01 $P^{0.5}$)	41760
2	Kuchling's Formula	Q (L/min)=3182 $P^{0.5}$	31800
3	Freeman's Formula	Q (L/min)= 1136.5(P/5+10)	35050
4	Ministry of Urban Development Manual Formula	Q (kilo liters/d)=100 $P^{0.5}$ for P>50000	31623

General limitation for Fire flow requirement

Details	Fire flow requirement (gpm)	
	gpm	L/min
In general case	8000	30240
For one story construction	6000	22680
The minimum fire flow	500	1890
The total for all purposes for single fire	12000	45360

To calculate the amount of water required for fire-fighting to a group of houses single and dual:

Distance between adjacent units		Required fire flow (gpm)	
ft	m	ft	m
> 100	> 30	500	1890
31-100	9.5-30.5	750-1000	2835-3780
11-30	3.4-9.2	1000-1500	3780-5670
$\leq$10	$\leq$3	1500-2000	5670-7560

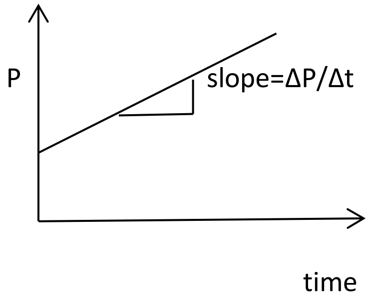
The amount of water required for firefighting must suffice as a minimum for four hours, most of the residential campus designed on the basis of the need for a period of 10 hours., So the times the fire depends on the amount of flow:

Required fire flow		Duration (hr)
gpm	L/min	
< 1000	< 3780	4
1000-1250	3780-4725	5
1250-1500	4725-5670	6
1500-1750	5670-6615	7
1750-2000	6615-7560	8
2000-2250	7560-8505	9
> 2250	> 8505	10

Forecasting Population

To design any waterworks element, the total water demand should be calculated for adopt period of design. It is necessary to estimate the maximum population expected to be served by the designed facility for any time in the future. To estimate the growing number of population the following methods are used:

1- Arithmetic method: Assumes a constant growth rate (ΔP/Δt = constant)



Pf = Po + K Δt

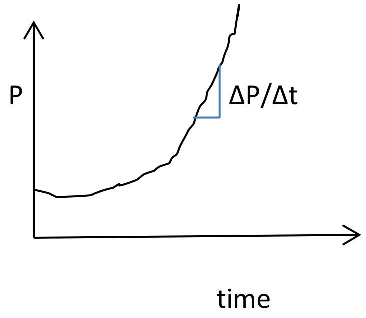
Pf = Population in the future (capita)

Po = Base population (capita)

K = Growth rate =ΔP/Δt (capita/ time)

Δt = Time increment between Pf and Po

2- Geometric method: The growth rate is proportional to population (ΔP/Δt ≈ P)



ln Pf = ln Po + $\bar{k}$ Δt

Pf = Population in the future (capita)

Po = Base population (capita)

$\bar{k}$ = [ln P_1- ln P_2]/Δt (ln capita/ time)

Δt = Time increment between Pf and Po

3- $P_f = P_0 \times (1 + i)^n$

Density of population

Population density, considering a whole city, rarely exceeds an average of (7500-10000 capita per km^2). More important to the designer engineer, solving water and sewerage problems, are the densities in particular areas:

Area	Density (capita per km^2)
The sparsely built-up residential sections	
Closely built-up single-family residential areas with small lots	8,800-10,000
Apartment and tenement districts	2,500-250,000
Commercial districts	High variable according to development

Periods of design and water consumption data required

Detail	Design period (year)	Design criteria
Development of source		The design capacity of the source should be adequate to provide the maximum daily demand anticipated during the design period.
For Groundwater For Surface Water	5 50	
Pipelines from source	> 25	The design capacity of the pipe lines should be based upon average consumption at the end of the design period.
Water treatment plant	10-15	Most WTP will be designed for average daily flow at the end of the design period. Hydraulic design should be based upon maximum anticipated flow.
Pumping plant	10	Pump selection requires knowledge of maximum flow including fire demand, average flow, and minimum flow during the design period.
Amount of storage	Minimum cost	Design requires knowledge of average flow, fire demand, maximum hour, maximum week, and maximum month, as well as the capacity of the source and pipe lines from the source.
Distribution system	Indefinite	Maximum hourly flow including fire demand is the basis for design.

Solved problems

Problem 2.1: Find the maximum daily domestic demand for a population of 1000 capita with a rate of 300 L/c/d?

Solution:

Average domestic daily demand=1000 x300=300 x10^3 L/day=300 m^3/day

Maximum daily demand =1.8 x average daily demand=1.8 x300=540 m^3/day

Problem 2.2: For the above Problem find the maximum demand if p = 250%?

Average domestic daily demand=1000 x300=300 x10^3 L/day=300 m^3/day

Maximum daily demand =2.5 x average daily demand=2.5 x300=750 m^3/day

Problem 2.3: A 6 story building was constructed for an engineering company. The building is an ordinary type structure with 10^3 m^2 for each floor. Determine: the maximum rate, and the total storage, for both domestic and fire demand in L/c/d. Knowing that the building can serve 22 X 10^3 capita having an average water consumption rate of 200 L/c/d?

Solution:

Average domestic demand = 22000 X 200 = 4.4 X 10^6 L/day

Maximum daily demand = Average domestic demand X1.8 = 4.4 X10^6X1.8=7.92 X10^6 L/day

$$F_{(gpm)} = 18CA_{ft^2}^{0.5} = 18\times1\times(1000\times10.76\times6)^{0.5} = 4574\,gpm = 17288\,L/\min = 24.89\times10^6\,L/day$$

Maximum rate = 7.92 X10^6 + 24.89 X10^6 = 32.81 X10^6 L/day for 10 hours = 1491 L per capita/day

The total flow required during this day would be:

7.92 X10^6 + 24.89X10^6 $\times\frac{10}{24}$= 18.29 X10^6 liter = $\frac{18.29\times10^6}{22000}\approx 83$ L per capita/day

Problem 2.4: A 4 story wooden building, each floor is 509 m^2. This building is adjacent to a 5-story ordinary type building, each floor is 900 m^2. Determine the fire flow in m^3/hr for each building, and both buildings if they are connected?

Solution:

For wooden building:

$$F_{(gpm)} = 18CA_{ft^2}^{0.5} = 18\times1.5\times(509\times10.76\times4)^{0.5} = 3996.306\,gpm = 15126.02\,L/\min = 21.79\times10^6\,L/day$$

For ordinary building:

$$F_{(gpm)} = 18CA_{ft^2}^{0.5} = 18 \times 1.0 \times (900 \times 10.76 \times 5)^{0.5} = 3960.81\, gpm = 14991.696\, L/\min = 21.59 \times 10^6\, L/day$$

By using the fractional area:

$$Total\ area = 4 \times 509 + 5 \times 900 = 6536\, m^2$$

$$Fraction\ for\ wooden\ building = \frac{4 \times 509}{6536} = 0.311$$

$$Fraction\ for\ ordinary\ building = \frac{5 \times 900}{6536} = 0.689$$

$$Total\ fire\ flow = 18 \times \left[1.5 \times 0.311 \times \sqrt{4 \times 509 \times 10.76} + 1 \times 0.689 \times \sqrt{5 \times 900 \times 10.76} = 3971.844\, gpm = 902 \frac{m^3}{hr} = 21648.138 \frac{m^3}{day}\right]$$

Which equivalent to 16500 population

Problem 2.5: For the data given find the population in year 2035.

Year	1965	1975	1985	1995	2005
Population	24700	29000	33500	38100	32000

Solution:

Information	year	population	ΔP	Δt	Arithmetic method			Geometric method			
					K	$K_{Average}$	Pt	k	$k_{Average}$	LN (Pt)	Pt
	1965	24700									
	1975	29000	4300	10	430			0.016049			
	1985	33500	4500	10	450			0.014425			
	1995	38100	4600	10	460			0.012867			
Base year	2005	32000	6100	10	610	487.5		-0.01745	0.006473		
Period design (t)	30										
First stage	2020						39312.5			10.47059	35263.05
Second stage	2035						46625			10.56769	38858.83

Problem 2.6: For the data given find the population in year 2035 by using arithmetic and geometric method, and average daily demand, maximum daily demand, weekly daily demand, and monthly daily demand as well as the maximum and minimum hourly demand. Also determine the fire water demand and total water demand if the water consumption of 500 liters per capita per day?

Year	1965	1975	1985	1995	2005
Population	25000	32000	42500	55000	69000

Solution:

Information	year	population	ΔP	Δt	Arithmetic method			Geometric method			
					K	$K_{Average}$	Pt	k	$k_{Average}$	LN (Pt)	Pt
	1965	25000									
	1975	32000	7000	10	700			0.024686			
	1985	42500	10500	10	1050			0.028377			
	1995	55000	12500	10	1250			0.025783			
Base year	2005	69000	14000	10	1400	1100		0.022677	0.025381		
Period design (t)	30										
First stage	2020						85500			11.52257	100969.5
Second stage	2035						102000			11.90328	147751.2

If adopting the arithmetic method:

Average daily demand = 500 x 102000=51000000 L/day=51000 m^3/day

Maximum daily demand =1.8 x average daily demand=1.8 x51000=91800 m^3/day

Maximum weekly demand =1.48 x average daily demand=1.48 x51000=75480 m^3/day

Maximum monthly demand =1.28 x average daily demand=1.28 x51000=65280 m^3/day

Maximum hourly demand =1. 5 x Maximum daily demand =1.5 x91800=137700 m^3/day

Minimum hourly demand =0. 5 x Maximum daily demand =0.5 x91800=45900m^3/day

Fire demand for 10 hr duration:

$$F = 1020 \times \sqrt{P} \times \left(1 - 0.01 \times \sqrt{P}\right) = 1020 \times \sqrt{102} \times \left(1 - 0.01 \times \sqrt{102}\right) \approx 9261 gpm \approx 50476\ m^3/day$$

Problem 2.7: A community has an estimated population of 40 X10^3 capita in a period of 25 years ahead. The present population is 30 X10^3 capita with a daily water consumption of 20 X 10^3 m^3/d. The existing water treatment plant (WTP) has a maximum design capacity of 45.6 X 10^3 m^3/day, assuming an arithmetic growth rate; determine for how many years this plant will reach its design capacity.

Solution:

$$Rate\ of\ consumption = \frac{20 \times 10^3}{30 \times 10^3} = 0.667\ m^3/(Capita.day)$$

$$k = \frac{\Delta P}{\Delta t} = \frac{40 \times 10^3 - 30 \times 10^3}{25} = 400\ capita/year$$

$$\text{Maximum daily demand} = \text{Average domestic demand} \times 1.8$$

$$45.6 \times 10^3 = 1.8 \times (30 \times 10^3 + 400 \times \Delta t) \times 0.667 \ \Rightarrow \Delta t \approx 20\ year$$

Problem 2.8: A community has an estimated population 20 year hence which is equal to 35000. The present population is 28000, and present average water consumption is $16 \times 10^6 \text{L/da}$. The existing water treatment plant has a capacity of 5 mgd. Assuming an arithmetic rate of population growth determine in what year the existing plant will reach its design capacity?

Solution:

$$\text{Average daily consumption} = \frac{16 \times 10^6\ l/day}{28000\ capita} = 571.42\ liter\ \text{per capita/d}$$

$$\text{Water treatment capacity} = 5 \times 10^6 \times 3.785 = 18.925 \times 10^6 l/day$$

$$\text{Number of capita at the end of design period WTP} = \frac{18.925 \times 10^6\ l/day}{571.42\ l/c.d} \approx 3312$$

$$k = \frac{P_O - P}{\Delta t} = \frac{35000 - 28000}{20} = 35$$

$$P_f = P_O + kt \Rightarrow 33120 = 28000 + 350t \Rightarrow t = 14.628 \approx 15\ yea$$

Problem 2.9: Determine the fire flow required for a residential area consisting of homes of ordinary construction, 2500 ft^2 in area, 10 ft apart. What total volume of water must be provided to satisfy the fire demand of this area?

Solution:

From (Table 2-3 page 18 STEEL), the residential fire flow = 1500 gpm

$$F_{(gpm)} = 18 C A_{ft^2}^{0.5} = 18\times 1\times (2500)^{0.5} = 900\, gpm$$

Water demand = 900 + 1500 = 2400 gpm = 9084 L/min = 545040 l/hr

$$V = 545040\, l/hr \times \frac{10}{24} \times \frac{1 m^3}{1000 l} = 227 m^3$$

Problem 2.10: A residential area of a city has a population density of 15000 capita per km^2 and an area of 120000 m^2. If the average water flow is 300 L/capita.day. Estimate the maximum rate to expected in m^3/sec?

Solution:

$A = \frac{120000}{1000000} = 0.12\ km^2$ & P = 15000*0.12 = 1800 capita

$$Average\ daily\ demand = 300 \frac{L}{cap.day} \times 1800 = 540000 \frac{L}{day} = 540\ m^3/day$$

$$Maximum\ daily\ demand = 1.8 \times 540 \frac{m^3}{day} = 972 \frac{m^3}{day} = 40.5 \frac{m^3}{hr} = 0.011\ m^3/sec$$

$$G = 1020 \times \sqrt{1.8} \times \left(1 - 0.01 \times \sqrt{1.8}\right) = 1350.113\ gpm = 5110.177\ Lpm = 5.11 \frac{m^3}{min} = 0.085 \frac{m^3}{sec}$$
$$= 7358.655\ m^3/day$$

$$Maximum\ daily\ demand = 972 + 7358.659 = 8330.659 \frac{m^3}{day} = 0.096\ m^3/sec$$

$$Total\ volume\ daily\ demand = 972 + 7358.655 \times \frac{10}{24} = 4038.105 \frac{m^3}{day} = 0.0467\ m^3/sec$$

Sanitary and Environmental Engineering

PART 1: WATER SUPPLY ENGINEERING

PART 1: WATER SUPPLY ENGINEERING

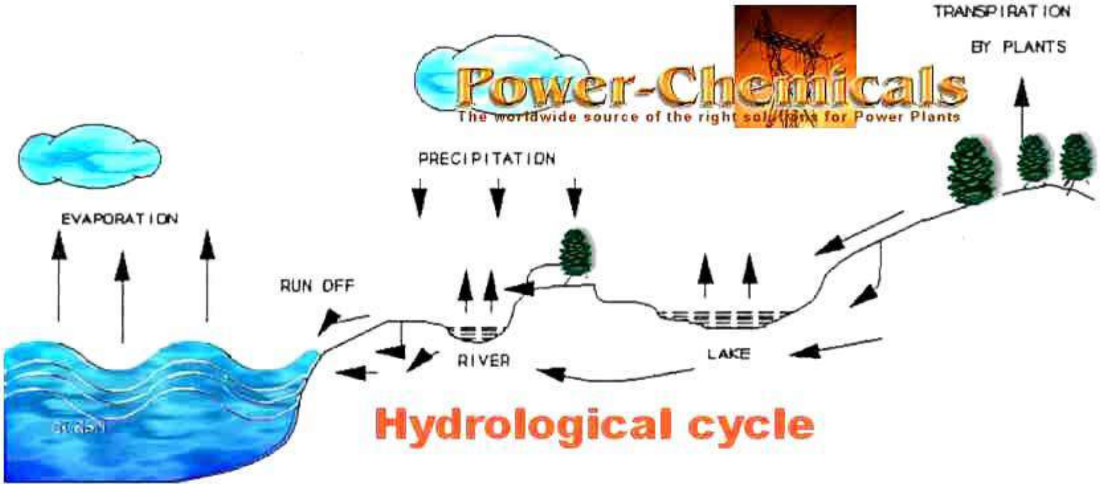
Lecture 3: Common Impurities in Water

Raw Water Source

The various sources of water can be classified into two categories:

1. *Surface sources*, such as (Ponds and lakes; Streams and rivers; Storage reservoirs; and Oceans, generally not used for water supplies, at present).
2. *Sub-surface sources or underground sources*, such as (springs; Infiltration wells; and Wells and Tube-wells).

Water is in continuous recirculation between the earth's surface and the atmosphere through various processes which are collectively bound under the heading 'The Hydrological Cycle' as shown in the diagram below explains the hydrological cycle which maintains the balance of the world's water.



Water absolutely pure is not found in nature. Water picks up different types of materials as it passes through the hydrological cycle. These particles make the water not pure and are called impurities which are suspended and dissolved in water.

Impurities in Water (sizes)

mm	1	10^{-1}	10^{-2}	10^{-3}	10^{-4}	10^{-5}	10^{-6}	10^{-7}
µm	10^3	10^2	10	1	10^{-1}	10^{-2}	10^{-3}	10^{-4}
	Suspended						Dissolved	
	Sand		Silt	Clay				
					Colloidal			
			Bacteria			Viruses		
	Protozoa, Algae, Fungi							

The size of the water molecular is 0.9 X 10^{-4} µm.

Impurities in Water (types)						
Suspended			Dissolved			
Soil	Microorganisms		Salts		Gases	Organic Matter
			Cation	Anion		
Sand	Bacteria		Ca	HCO_3	Oxygen	
Silt	Algae		Mg	CO_3	Carbon dioxide	
Clay	Fungi		Mn	SO_4	Hydrogen sulfide	
colloidal	Protozo		Fe	Cl	Methane	
	Viruses		Na, K	F	Nitrogen	
			Pb, Hg	NO_3, NO_2	Ammonia	

Impurities in Water (Causes):

Suspended Impurities	Causes
Bacteria	Disease
Algae	Odor, Color, Turbidity
Protozoan	Disease
Viruses	Disease
Silt	Murkiness and Turbidity
Clay	Murkiness and Turbidity
Colloids	Color, Murkiness and Turbidity

Dissolved Impurities	
Salts -Cations	Causes
Calcium	Hardness
Magnesium	Hardness
Iron	Hardness, Color
Manganese	Hardness, Color
Others	Dissolved Solids
Salts -Anions	Causes
Bicarbonate	Alkalinity
Carbonate	Alkalinity
Sulfate	Laxative
Chloride	Taste
Fluoride	Tooth Mottling
Organics	Color, Taste, Odor, Toxicity
Gases	Causes
Oxygen	Corrosive, Oxidizing Agent
Carbon Dioxide	Acid
Hydrogen Sulfide	Acid, Reducing Agent
Nitrogen	None
Ammonia	Caustic

Impurities of water may be divided into two classes:

1. **Suspended Impurities:**
    a) *Microorganisms:* They may get into water from the air with dust, etc., as rain falls, or commonly when soil polluted with human and animal wastes is washed into the water source. The latter type of impurity in water is the most dangerous one because a good number of microorganisms are pathogenic and cause disease.
    b) *Suspended solids:* Minute particles of soil, clay, silt, soot particles, dead leaves and other insoluble material get into water because of erosion from higher ground, drainage from swamps, ponds, top soil, etc. Toxic chemicals such as insecticides and pesticides are also included in this category. They are introduced to streams either as industrial wastes or drained in after rain from land treated with these chemicals. Generally, suspended solids cause taste, color or turbidity.
    c) *Algae:* Algae are minute plants that grow in still or stagnant water. Some algae are green, brown or red, and their presence in water causes taste, color and turbidity.

Some species of algae could be poisonous both for aquatic animals and humans.

There are different types of algae found in water:

i. Asterionell – Gives water an unpleasant odor.
ii. Spirogyra – Is a green scum found in small ponds and polluted water. It grows in thread like groups. It is slippery and non-toxic.
iii. Anabaena – Is blue- green and occurs in fishponds, pools, reservoir, and clogs filters.

2. **Dissolved Impurities:**
    a) *Gases:* Oxygen (O_2), carbon dioxide (CO_2), hydrogen sulphide (H_2S), etc, find their way into water as it falls as rain or, in the case of the latter two, from the soil as water percolates through the ground. All-natural water contains dissolved oxygen, and in certain circumstances carbon dioxide. The presence of CO_2 and H_2S (but not O_2) causes acidity in water. In addition, H_2S imparts a bad odor to the water.
    b) *Minerals:* Minerals get into water as it percolates downward though the earth layers. The type of minerals dissolved will depend on the nature of the specific rock formation of an area. Most common dissolved minerals in water are salts of calcium, magnesium, sodium, potassium, etc. Salts of the first two elements cause hardness in water, while salts of the latter two elements cause alkalinity. Salts of toxic elements, such as lead, arsenic, chromium, etc, get into water mainly as industrial wastes dumped into streams.
    c) *Plant dyes:* These originate from plants, which grow in or around water and cause acidity and color.

IMPURITY	EFFECT	PROBLEMS
Calcium (Ca^{++}) Magnesium (Mg^{++})	Precipitate soaps, anionic surfactants, anionic emulsifiers, corrosion inhibitors, and microbicides from the mix.	Product imbalance Product instability Rancidity Poor filtration Unsatisfactory production Residue in parts and/or machines Short product life
Sodium (Na^{+}) in the presence of sulfate	Chemically aggressive and tends to reduce pH value.	Mix instability Poor ferrous corrosion control
Carbonate ($CO_3^{=}$) Bicarbonate (HCO_3^{-})	Decrease chemical stability and increase total alkalinity	Residue on parts and/or machines Dermatitis Product imbalance Unsatisfactory production Short product life
Chloride (Cl^{-}) over 50 ppm	Promotes Corrosion	Poor corrosion control Product imbalance Short product life
Sulfate ($SO_4^{=}$) over 100 ppm	Similar to chloride in effect. Also promotes growth of sulfate-reducing bacteria.	Same as chloride, plus rancidity
Phosphates ($PO_4^{\equiv}$ and others)	Contribute to total alkalinity and stimulate microbial growth.	Dermatitis Rancidity Mold growth

Water Quality

Sources of water supply

Different of types of water sources are available in nature: rivers, lakes, reservoirs, groundwater and springs. Each of these sources has different raw water quality depending on the environmental conditions they flow through. Water quality deals with the type and quantity (amount) of impurities existing. This quality is considered according to certain regulations related to the usage of water, such as drinking, industrial, agriculture … etc.

Potable water: *Water safe to drink, pleasant in taste and usable for various purposes*. This quality is considered according to the WHO, EPA, or Iraqi Standards, Limitations, Specifications for drinking water. These limitations give the water properties as:

$$Concentration(mg/L) = \frac{Weight\ of\ substance\ or\ impurity}{Volume\ of\ water}$$

The unit used mg/L or ppm (part per million), in water ppm= mg/L/ Gs (specific gravity) of water. As Gs =1 in water then ppm = mg/L.

$$\rho_w = 1000\frac{kg}{m^3} = 1\frac{gm}{cm^3}, Volume\ of\ 1\ cm^3\ of\ water = weight\ of\ 1\ gm,$$

$$1cm^3 = 1mL = 1gm, 1L = 10^3\ mL\ \rightarrow So, 1mL = 1gm$$

$$\frac{mg}{L} = \frac{mg}{10^3\ mL} = \frac{mg}{10^3\ \times 10^3 mg} = \frac{1}{10^6}$$

Contaminated water is one that contains microorganisms, chemicals, industrial or other wastes, or sewage so that it is unfit for its intended use.

Polluted water is synonymous with contaminated.

The specification represents water properties in the forms:

Physical properties: Color, Turbidity, Odor, Temperature, Density, Electrical Conductivity.

Chemical properties: Total Dissolved Solids (Salts, Metals), pH.

Biological properties: Different type of microorganisms present in water.

Laboratory tests are very important to evaluate water quality; these tests are performed according to the standard tests for the examination of water and wastewater.

Water Quality

The raw or treated water is analyzed by testing their physical, chemical and biological characteristics:

Physical Characteristics:

Turbidity:

If a large amount of suspended solids is present in water, it will appear turbid in appearance. The turbidity depends upon fineness and concentration of particles present in water. Originally turbidity was determined by measuring the depth of column of liquid required to cause the image of a candle flame at the bottom to diffuse into a uniform glow. This was measured by Jackson candle turbidity meter. The calibration was done based on suspensions of silica from Fuller's earth. The depth of sample in the tube was read against the part per million (ppm) silica scales with one ppm of suspended silica called one Jackson Turbidity unit (JTU). Because standards were prepared from materials found in nature such as Fuller's earth, consistency in standard formulation was difficult to achieve.

These days' turbidity is measured by applying Nephelometry, a technique to measure level of light scattered by the particles at right angles to the incident light beam. The scattered light level is proportional to the particle concentration in the sample. *The unit of expression is Nephelometric Turbidity Unit (NTU). The IS values for drinking water is 5 NTU.*

Color:

Dissolved organic matter from decaying vegetation or some inorganic materials may impart color to the water. It can be measured by comparing the color of water sample with other standard glass tubes containing solutions of different standard color intensities. The standard unit of color is that which is produced by one milligram of platinum cobalt dissolved in one liter of distilled water. *The IS value for treated water is 5 to 25 cobalt units.*

Taste and Odor

Odor depends on the contact of a stimulating substance with the appropriate human receptor cell. Most organic and some inorganic chemicals, originating from municipal or industrial wastes, contribute taste and odor to the water. Taste and odor can be expressed in terms of odor intensity or threshold values.

A new method to estimate taste of water sample has been developed based on flavor known as 'Flavor Profile Analysis' (FPA). The character and intensity of taste and odor discloses the nature of pollution or the presence of microorganisms.

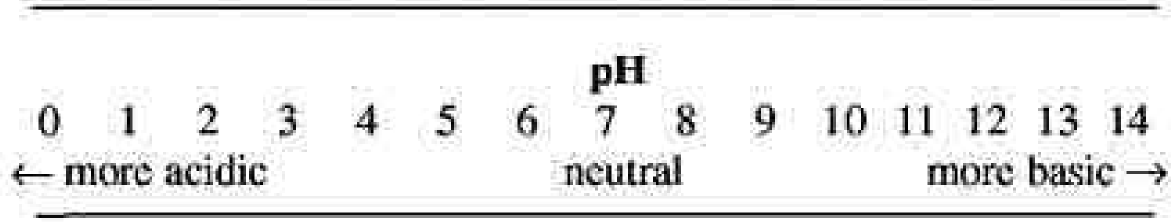
Temperature

The increase in temperature decreases palatability, because at elevated temperatures carbon dioxide and some other volatile gases are expelled. *The ideal temperature of water for drinking purposes is 5 to 12 °C* - above 25 °C, water is not recommended for drinking.

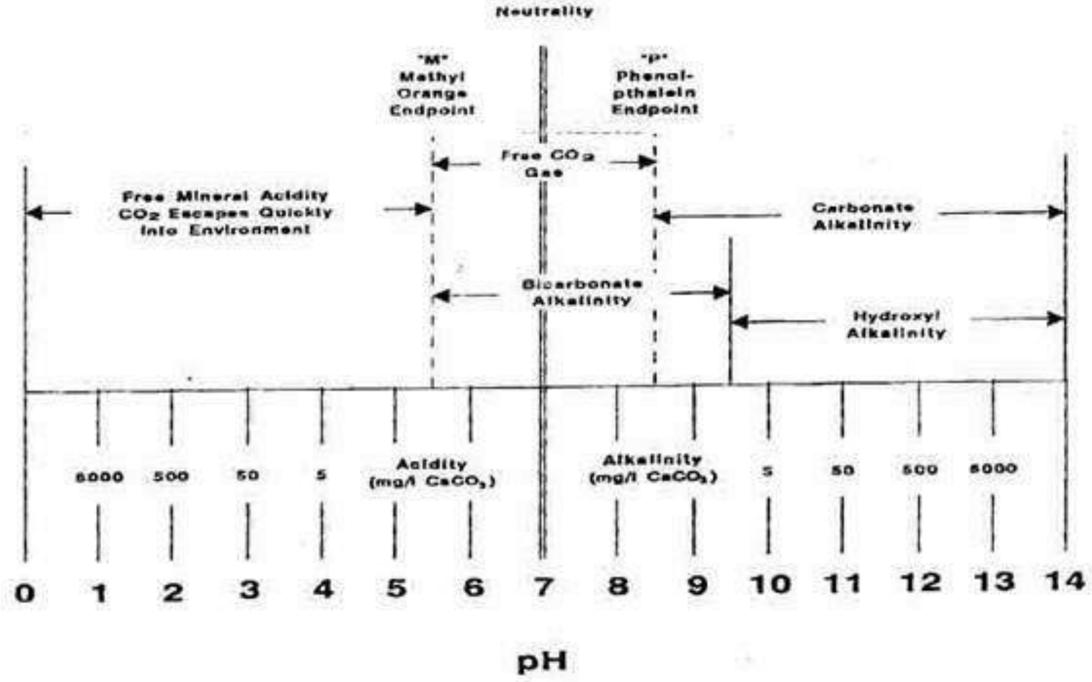
Chemical Characteristics:

pH

pH value denotes the acidic or alkaline condition of water. It is expressed on a scale ranging from 0 to 14, which is the common logarithm of the reciprocal of the hydrogen ion concentration. *The recommended pH range for treated drinking waters is 6.5 to 8.5.*



ACIDITY & VARIOUS TYPES OF ALKALINITY & THEIR pH RANGES



source US Army TECHNICAL BULLETIN 420-49-05

Acidity

The acidity of water is a measure of its capacity to neutralize bases. Acidity of water may be caused by the presence of uncombined carbon dioxide, mineral acids and salts of strong acids and weak bases. It is expressed as mg/L in terms of calcium carbonate. Acidity is nothing but representation of carbon dioxide or carbonic acids. Carbon dioxide causes corrosion in public water supply systems.

Alkalinity

The alkalinity of water is a measure of its capacity to neutralize acids. It is expressed as mg/L in terms of calcium carbonate. The various forms of alkalinity are (a) hydroxide alkalinity, (b) carbonate alkalinity, (c) hydroxide plus carbonate alkalinity, (d) carbonate plus bicarbonate alkalinity, and (e) bicarbonate alkalinity, which is useful mainly in water softening and boiler feed water processes. Alkalinity is an important parameter in evaluating the optimum coagulant dosage.

Hardness

If water consumes excessive soap to produce lather, it is said to be hard. Hardness is caused by divalent metallic cations. The principal hardness causing cations are calcium, magnesium, strontium, ferrous and manganese ions. The major anions associated with these cations are sulphates, carbonates, bicarbonates, chlorides and nitrates. The total hardness of water is defined as the sum of calcium and magnesium concentrations, both expressed as calcium carbonate, in mg/L. Hardness are of two types, temporary or carbonate hardness and permanent or non-carbonate hardness. Temporary hardness is one in which bicarbonate and carbonate ion can be precipitated by prolonged boiling. Non-carbonate ions cannot be precipitated or removed by boiling, hence the term permanent hardness. *IS value for drinking water is 300 mg/Las* $CaCO_3$.

Chlorides

Chloride ion may be present in combination with one or more of the cations of calcium, magnesium, iron and sodium. Chlorides of these minerals are present in water because of their high solubility in water. Each human being consumes about six to eight grams of sodium chloride per day, a part of which is discharged through urine and night soil. Thus, excessive presence of chloride in water indicates sewage pollution. *IS value for drinking water is 250 to 1000 mg/L.*

Sulphates

Sulphates occur in water due to leaching from sulphate mineral and oxidation of sulphide. Sulphates are associated generally with calcium, magnesium and sodium ions. Sulphate in drinking water causes a laxative effect and leads to scale formation in boilers. It also causes odor and corrosion problems under aerobic conditions. *Sulphate should be less than 50 mg/L, for some industries. Desirable limit for drinking water is 150 mg/L*. May be extended up to 400 mg/L.

Iron

Iron is found on earth mainly as insoluble ferric oxide. When it comes in contact with water, it dissolves to form ferrous bicarbonate under favorable conditions. This ferrous bicarbonate is oxidized into ferric hydroxide, which is a precipitate. Under anaerobic conditions, ferric ion is reduced to soluble ferrous ion. Iron can impart bad taste to the water, causes discoloration in clothes and incrustations in water mains. *IS value for drinking water is 0.3 to 1.0 mg/L.*

Solids

The sum total of foreign matter present in water is termed as 'total solids'. Total solids are the matter that remains as residue after evaporation of the sample and its subsequent drying at a defined temperature (103 to 105 °C). Total solids consist of volatile (organic) and non-volatile (inorganic or fixed) solids. Further, solids are divided into suspended and dissolved solids. Solids that can settle by gravity are settleable solids. The others are non-settleable solids. IS acceptable limit for total solids is 500 mg/L and tolerable limit is 3000 mg/Lof dissolved limits.

Nitrates

Nitrates in surface waters occur by the leaching of fertilizers from soil during surface run-off and also nitrification of organic matter. Presence of high concentration of nitrates is an indication of pollution. Concentration of nitrates above 45 mg/L causes a disease methemoglobin. IS value is 45 mg/L.

Bacteriological Characteristics:

Bacterial examination of water is very important, since it indicates the degree of pollution. Water polluted by sewage contains one or more species of disease producing pathogenic bacteria. Pathogenic organisms cause water borne diseases, and many non-pathogenic bacteria such as E.Coli, a member of coliform group, also live in the intestinal tract of human beings. Coliform itself is not a harmful group but it has more resistance to adverse condition than any other group. So, if it is ensured to minimize the number of coliforms, the harmful species will be very less. So, coliform group serves as indicator of contamination of water with sewage and presence of pathogens.

The methods to estimate the bacterial quality of water are:

Standard Plate Count Test

In this test, the bacteria are made to grow as colonies, by inoculating a known volume of sample into a solidifiable nutrient medium (Nutrient Agar), which is poured in a petridish. After incubating (35°C) for a specified period (24 hours), the colonies of bacteria (as spots) are counted. The bacterial density is expressed as number of colonies per 100 mL of sample.

Most Probable Number

Most probable number is a number which represents the bacterial density which is most likely to be present. E. coli is used as indicator of pollution. E.Coli ferment lactose with gas formation with 48 hours incubation at 35°C. Based on this E.Coli density in a sample is estimated by multiple tube fermentation procedure, which consists of identification of E.Coli in different dilution combination. MPN value is calculated as follows:

Five 10 mL (five dilution combination) tubes of a sample is tested for E.Coli. If out of five only one gives positive test for E.Coli and all others negative. From the tables, MPN value for one positive and four negative results is read which is 2.2 in present case. The MPN value is expressed as 2.2 per 100 mL. These numbers are given by Maccardy based on the laws of statistics.

Membrane Filter Technique

In this test a known volume of water sample is filtered through a membrane with opening less than 0.5 microns. The bacteria present in the sample will be retained upon the filter paper. The filter paper is put in contact of a suitable nutrient medium and kept in an incubator for 24 hours at 35°C. The bacteria will grow upon the nutrient medium and visible colonies are counted. Each colony represents one bacterium of the original sample. The bacterial count is expressed as number of colonies per 100 mL of sample.

Water Treatment

To use water for drinking from the available raw water in nature, it should be within drinking specifications; if not then water treatment is required.

Water treatment, a number of processes that are required for the removal of:

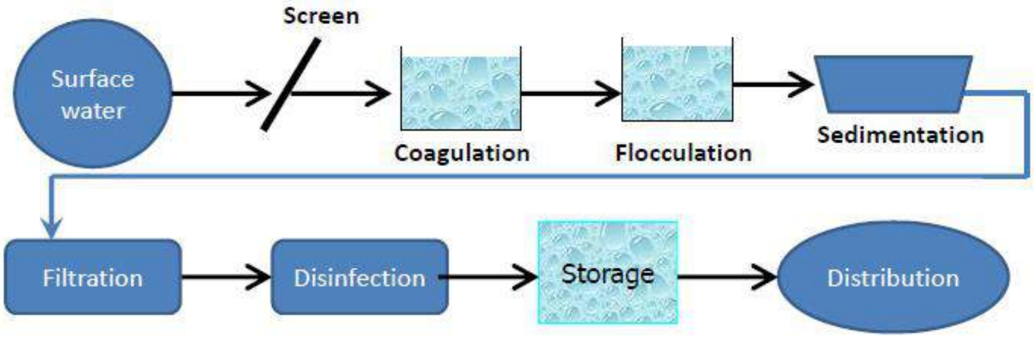
1. Pathogenic organisms;
2. Unpleasant taste and odor
3. Color and turbidity
4. Dissolved minerals
5. Harmful chemicals

The degree of treatment depends on:

1. Degree of contamination
2. Type of impurity
3. Size of impurity

Water treatment plants (WTP) consists of many units which are designed to remove the impurities of different types and sizes. The most common type of these plants is the conventional (typical or routine) treatment plant, which consists of:

1. Intake, (low pump station)
2. Screen (Coarse and fine),
3. Coagulation and flocculation, (Chemical Process)
4. Sedimentation, (Physical Process)
5. Filtration, (Clean Water)
6. Disinfection, (clean and safe water)
7. Storage water with high pump station (Enough Flow and Pressure).

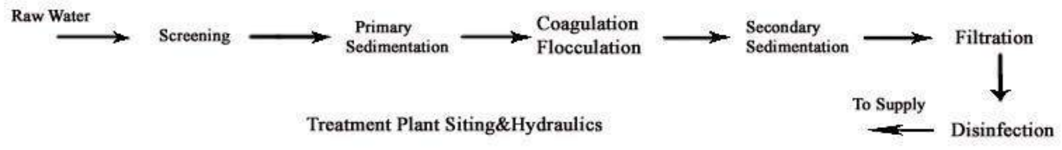


Water treatment plant schedule

Water Treatment Philosophy

The available raw waters must be treated and purified before they can be supplied to the public for their domestic, industrial or any other uses. The extent of treatment required to be given to the particular water depends upon the characteristics and quality of the available water, and also upon the quality requirements for the intended use.

The layout of conventional water treatment plant is as follows:



Depending upon the magnitude of treatment required, proper unit operations are selected and arranged in the proper sequential order for the purpose of modifying the quality of raw water to meet the desired standards. Indian Standards for drinking water are given in the table below.

Indian Standards for drinking water

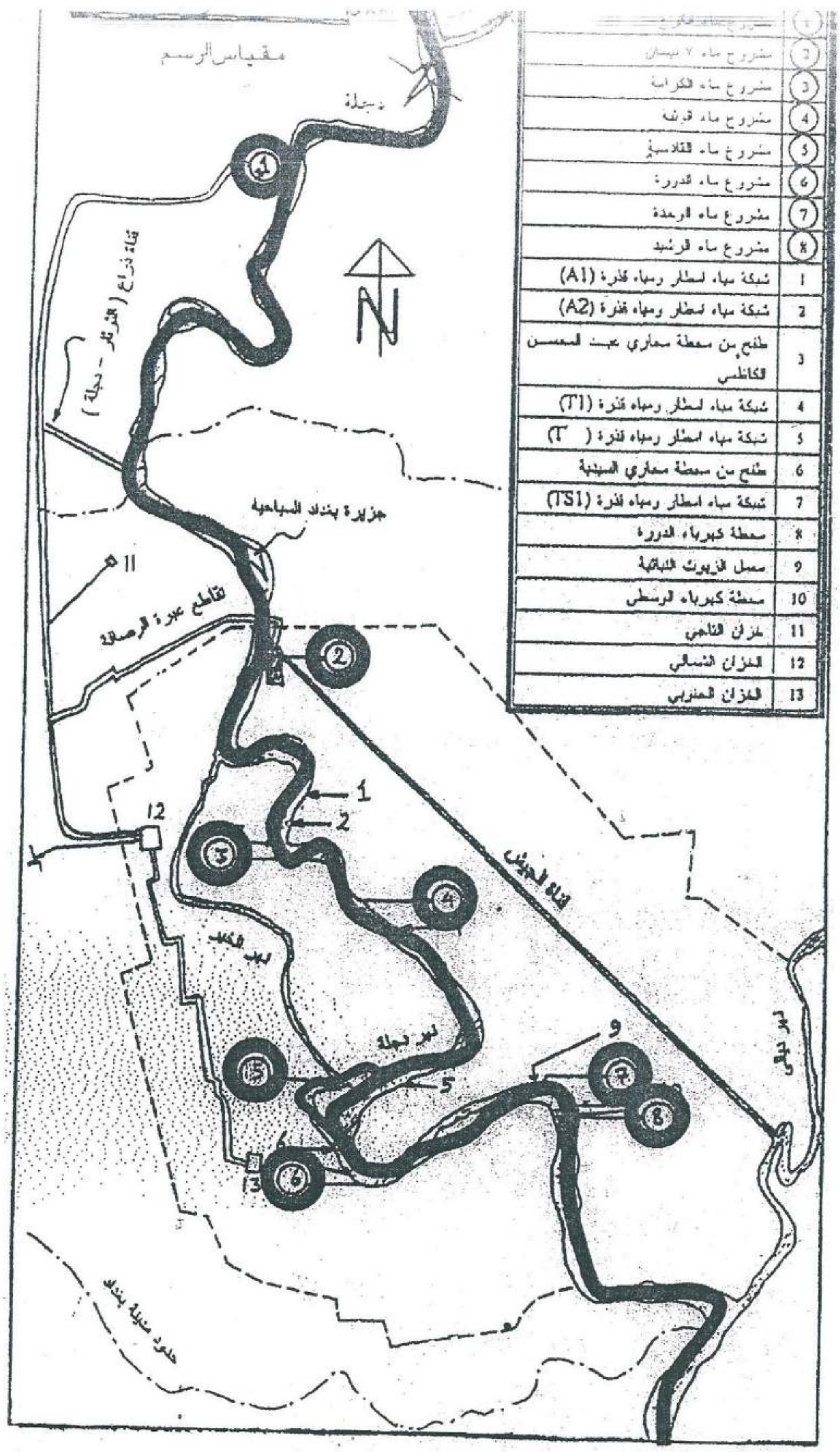
Parameter	**Desirable-Tolerable**	***If no alternative source available, limit extended upto***
Physical		
Turbidity (NTU unit)	< 10	25
Colour (Hazen scale)	< 10	50
Taste and Odour	Un-objectionable	Un-objectionable
Chemical		
pH	7.0-8.5	6.5-9.2
Total Dissolved Solids mg/L	500-1500	3000
Total Hardness mg/L(as $CaCO_3$)	200-300	600
Chlorides mg/L(as Cl)	200-250	1000
Sulphates mg/L(as SO_4)	150-200	400
Fluorides mg/L(as F)	0.6-1.2	1.5
Nitrates mg/L(as NO_3)	45	45
Calcium mg/L(as Ca)	75	200
Iron mg/L(as Fe)	0.1-0.3	1.0

The typical functions of each unit operations are given in the following table:
Functions of Water Treatment Units

Unit treatment	Function (removal)
Aeration, chemicals use	Colour, Odor, Taste
Screening	Floating matter
Chemical methods	Iron, Manganese, etc.
Softening	Hardness
Sedimentation	Suspended matter
Coagulation	Suspended matter, a part of colloidal matter and bacteria
Filtration	Remaining colloidal dissolved matter, bacteria
Disinfection	Pathogenic bacteria, Organic matter and Reducing substances

The types of treatment required for different sources are given in the following table:

Source	Treatment required
1. Ground water and spring water fairly free from contamination	No treatment or Chlorination
2. Ground water with chemicals, minerals and gases	Aeration, coagulation (if necessary), filtration and disinfection
3. Lakes, surface water reservoirs with less amount of pollution	Disinfection
4. Other surface waters such as rivers, canals and impounded reservoirs with a considerable amount of pollution	Complete treatment



الشكل (3-1). مواقع مشاريع تصفية الماء العاملة حالياً واهم مواقع تصريف الميــــاه

محطات المعالجة في بغداد

المشروع	مراحل التوسعات	الطاقة التصميمية م٣ يوم/	نسبة الطاقة المتحققة %	نسبة المساهمة لتجهيز مدينة بغداد %
الكرخ	١٩٨٥ ١٩٨٧ ١٩٨٨	١٣٦٥٠٠٠	٨٢	٥٥.٨
٧ نيسان	١٩٨٧ ١٩٨٥	٥٤٠٠٠٠	٧٤	٢٠
الكرامة	١٩٥٦ ١٩٦١ ١٩٨٠	٢٢٧٠٠٠	٩٤	٧.٥
الوثبة	١٩٣٨ ١٩٧٨	٧٠٠٠٠	٩٠	٣.٥
القادسية	١٩٦٦ ١٩٧٣	١٣٦٨٠٠	٨٤	٤.٩
الدورة	١٩٨٠	١١٥٠٠٠	٧٦	٣.٨
الوحدة	١٩٥٢	٦٨٠٠٠	٨٣	٢.٨
الرشيد	١٩٦٩	٥٠٤٠٠	٧٧	١.٧
المجموع		٢٦٠٠٠٠		

Sanitary and Environmental Engineering

PART 1: WATER SUPPLY ENGINEERING

PART 1: WATER SUPPLY ENGINEERING

Lecture 4: Water Distribution Systems

The primary purpose of this system is to transport treated water from the treatment facilities (plants) to the consumers. This system should have sufficient capacity to meet the water supply needs (demand) of the consumer, under all demand conditions in quantity, quality and pressure. In some communities this system may provide water for fire demand also. Distribution system is used to describe collectively the facilities used to supply water from its source to the point of usage.

Design stages of water distribution systems:

1. Preparation of a master plan.
2. Topography survey of the study area
3. Hydraulic study of the existing system.
4. The improvement programs.

Requirements of Good Distribution System:

5. Water quality should not get deteriorated in the distribution pipes.
6. It should be capable of supplying water at all the intended places with sufficient pressure head.
7. It should be capable of supplying the requisite amount of water during firefighting.
8. The layout should be such that no consumer would be without water supply, during the repair of any section of the system.
9. All the distribution pipes should be preferably laid one meter away or above the sewer lines.
10. It should be fairly water-tight as to keep losses due to leakage to the minimum.

Methods of distribution

1. By gravity
2. By means of pumps and storage
3. By pumps only

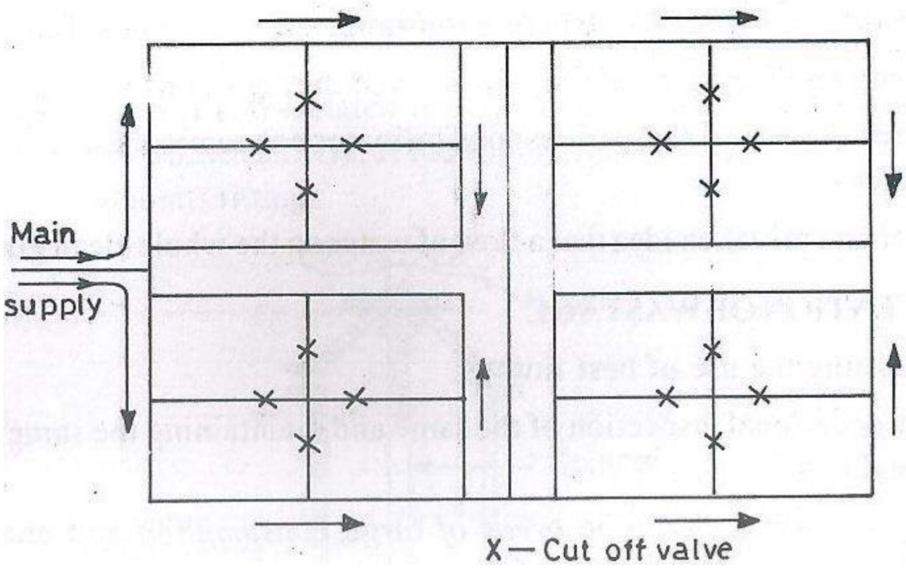
A typical distribution system consists of: pipes, nodes and loops, by which a network will be formed. The main components in this system are the pipes, which are in the following form according to their size:

1. Primary feeders.
2. Secondary feeders.
3. Small distribution mains.
4. Service pipes.

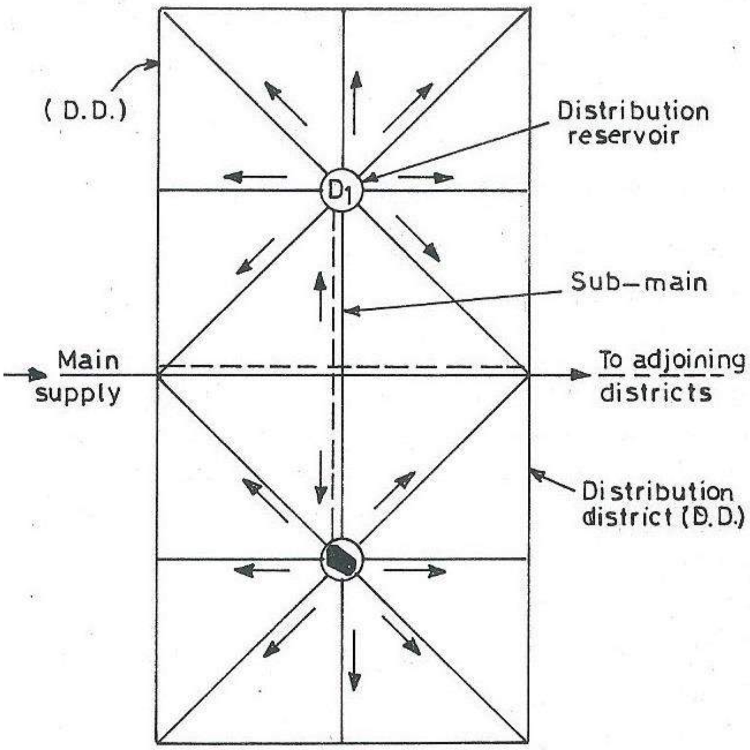
Types of networks, according to the method of connecting different types of pipes:

The distribution pipes are generally laid below the road pavements, and as such their layouts generally follow the layouts of roads. There are, in general, four different types of pipe networks; any one of which either singly or in combinations, can be used for a particular place. They are:

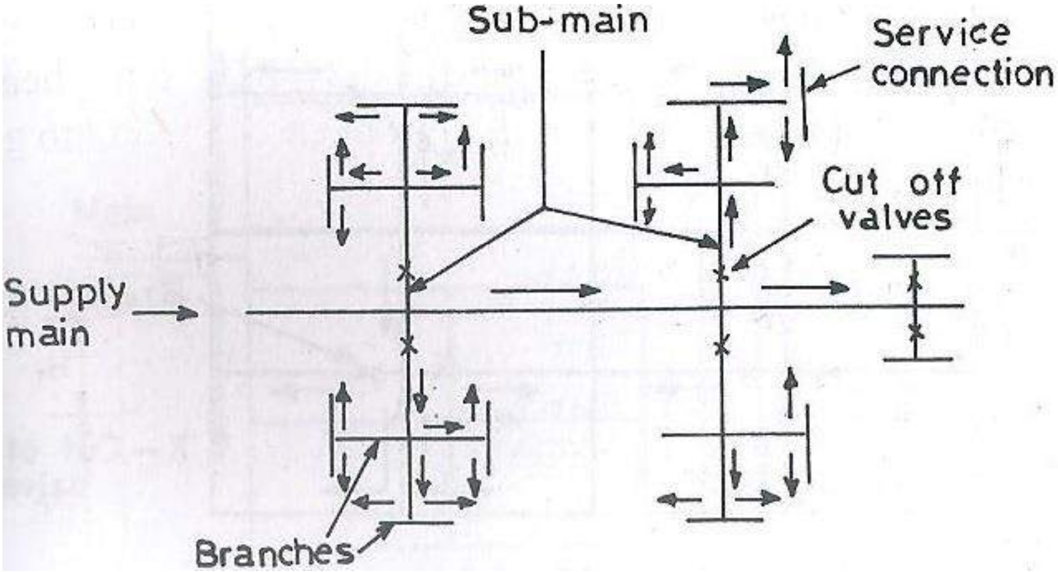
1. ring system
2. radial system
3. Dead end system
4. grid system



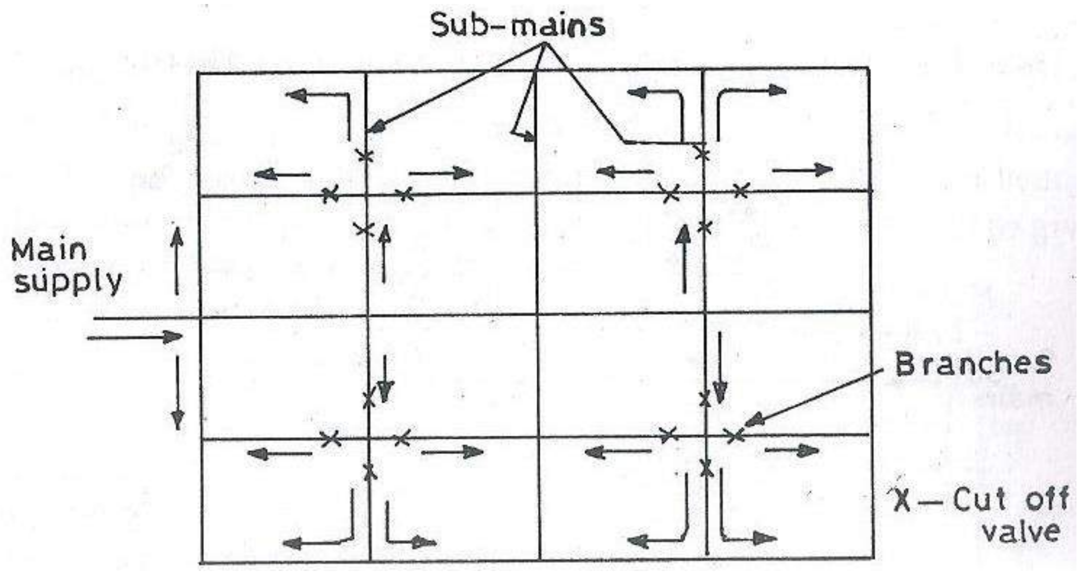
Ring system



Radial system



Dead end system



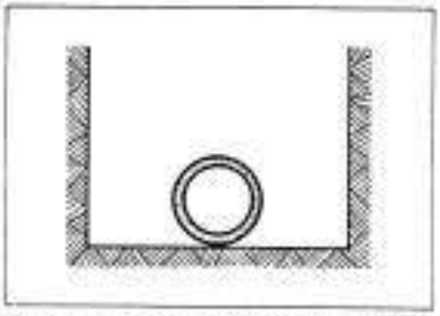
Grid iron system

Characteristics of pipes used in distributing water:

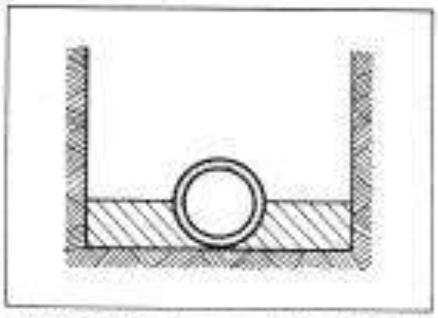
1. Adequate strength to with stand external and internal pressure.
2. Ability to resist impact loads.
3. Smooth and non-corrosive interior surface for minimum friction losses.
4. Can be provided with tight joints.

Types of pipes according to the materials used in their manufacture:

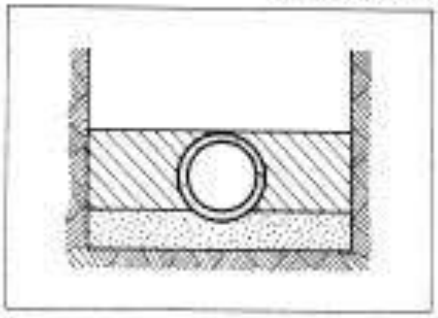
1. Cast iron – iron + silicon + carbon
2. Ductile – cast iron + Mg
3. Steel – iron + Cr + Ni
4. Galvanized steel – steel + Zn
5. Concrete
6. PVC- polyvinyl chloride pipes.
7. Asbestos cement-port land cement + silicon + asbestos fibers
8. Lead.
9. Copper.
10. Glass Fiber Reinforced Plastic (GRP).
11. High density polyethylene pipe (HDPE).



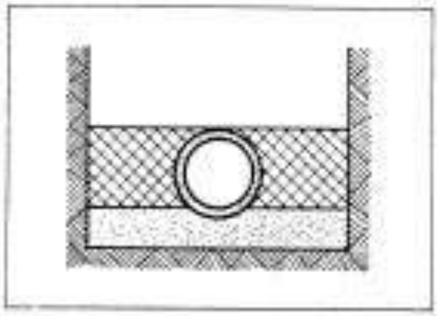
Type 1 Flat-bottom trench. Loose backfill



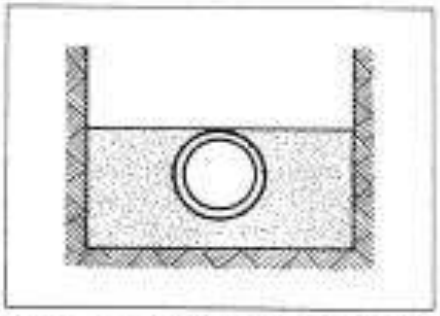
Type 2 Flat-bottom trench. Backfill lightly consolidated to centerline of pipe.



Type 3 Pipe bedded in 100-mm minimum loose soil. Backfill lightly consolidated to top of pipe.



Type 4 Pipe bedded in sand, gravel, or crushed stone to depth of $\frac{1}{8}$ pipe diameter, 100-mm minimum. Backfill compacted to top of pipe. (Approx. 80 percent Standard Proctor, AASHTO T-99)



Type 5 Pipe bedded to its centerline in compacted granular material, 100-mm minimum under pipe. Compacted granular or select material to top of pipe. (Approx. 90 percent Standard Proctor, AASHTO T-99).

Standard pipe laying conditions.

Distribution Reservoirs

Distribution reservoirs, also called service reservoirs, are the storage reservoirs, which store the treated water for supplying water during emergencies (such as during fires, repairs, etc.) and also to help in absorbing the hourly fluctuations in the normal water demand.

Functions of Distribution Reservoirs:

- To absorb the hourly variations in demand.
- To maintain constant pressure in the distribution mains.
- Water stored can be supplied during emergencies.

Location and Height of Distribution Reservoirs:

- Should be located as close as possible to the center of demand.
- Water level in the reservoir must be at a sufficient elevation to permit gravity flow at an adequate pressure.

Types of Reservoirs

- Underground reservoirs.
- Small ground level reservoirs.
- Large ground level reservoirs.
- Overhead tanks.

Storage Capacity of Distribution Reservoirs

The total storage capacity of a distribution reservoir is the summation of:

1. ***Balancing Storage:*** The quantity of water required being stored in the reservoir for equalizing or balancing fluctuating demand against constant supply is known as the balancing storage (or equalizing or operating storage). The balance storage can be worked out by *mass curve method*.
2. ***Breakdown Storage:*** The breakdown storage or often called emergency storage is the storage preserved in order to tide over the emergencies posed by the failure of pumps, electricity, or any other mechanism driving the pumps. A value of about 25% of the total storage capacity of reservoirs, or 1.5 to 2 times of the average hourly supply, may be considered as enough provision for accounting this storage.
3. ***Fire Storage:*** The third component of the total reservoir storage is the fire storage. This provision takes care of the requirements of water for extinguishing fires. A provision of 1 to 4 liter per person per day is sufficient to meet the requirement.

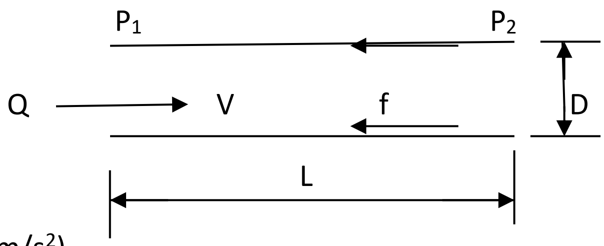
The total reservoir storage can finally be worked out by adding all the three storages.

Water flow in pipes

Since the flow is turbulent in pipes for water supply, the friction (f) will depend on:

1) Roughness of the pipe.
2) Reynolds no. which depends on the velocity of flow in the pipe and its diameter.

$$h_l = f \times \frac{L}{D} \times \frac{V^2}{2g}$$



Where: h_l = Head loss = $P_1 - P_2$

V = Velocity of flow (m/s)

g = Acceleration of gravity (m/s^2)

D = Diameter of the pipe (m)

P1, P2 = Pressure heads (m)

To predict the head loss (h_l), which is a function of the velocity of flow, Hazen – William formula is used:

$$V = k \times C \times R^{0.63} \times S^{0.54} = 0.85 \times C \times R^{0.63} \times S^{0.54}$$

Where: k = Experimental coefficient and a unit conversion factor= 0.85 (m/s, m)

C = Hazen – William coefficient depending on the roughness of the pipe according to its material and age.

R = Hydraulic radius = $R = \frac{D}{4}$

S = Hydraulic gradient = $S = \frac{h_l}{L}$ (L = length of the pipe)

As the discharge (Q) flowing in the pipe is filling, it is equal to the velocity (V) x cross section area of the pipe ($\pi D^2 /4$). By applying the velocity according to the Hazen – William formula, then the discharge is written as

$$Q = 0.85 \times C \times \left(\frac{D}{4}\right)^{0.63} \times S^{0.54} \times \frac{\pi D^2}{4} = 0.278 \times C \times D^{2.63} \times S^{0.54}$$

Hazen – William coefficient can be taken from the table below

Material of the pipe	**Value of C**
Cast Iron - new	130
5 years	120
10 years	110
20 years	90 -100
30 years	75 - 90
Concrete	120 – 140
PVC	150
Asbestos cement	120 - 140

According to the design criteria, V should be taken in the range 0.6 to 2.6 m/s for designing the distribution system.

Material	C	Material	C
ABS- Acrylonite Butadiene Styrene	130	Fiber	140
Aluminum	130 - 150	Fiber Glass Pipe - FRP	150
Asbestos Cement	140	Galvanized iron	120
Asphalt Lining	130 - 140	Glass	130
Brass	130 - 140	Lead	130 - 140
Brick sewer	90 - 100	Metal Pipes - Very to extremely smooth	130 - 140
Cast-Iron - new unlined (CIP)	130	Plastic	130 - 150
Cast-Iron 10 years old	107 - 113	Polyethylene, PE, PEH	140
Cast-Iron 20 years old	89 - 100	Polyvinyl chloride, PVC, CPVC	150
Cast-Iron 30 years old	75 - 90	Smooth Pipes	140
Cast-Iron 40 years old	64-83	Steel new unlined	140 - 150
Cast-Iron, asphalt coated	100	Steel, corrugated	60
Cast-Iron, cement lined	140	Steel, welded and seamless	100
Cast-Iron, bituminous lined	140	Steel, interior riveted, no projecting rivets	110
Cast-Iron, sea-coated	120	Steel, projecting girth and horizontal rivets	100
Cast-Iron, wrought plain	100	Steel, vitrified, spiral-riveted	90 - 110
Cement lining	130 - 140	Steel, welded and seamless	100
Concrete	100 - 140	Tin	130
Concrete lined, steel forms	140	Vitrified Clay	110
Concrete lined, wooden forms	120	Wrought iron, plain	100
Concrete, old	100 - 110	Wooden or Masonry Pipe - Smooth	120
Copper	130 - 140	Wood Stave	110 - 120
Corrugated Metal	60		
Ductile Iron Pipe (DIP)	140		
Ductile Iron, cement lined	120		

Hydraulic Analysis

This analysis is important to determine the flow (Q) and head loss (h_L) in each pipe, also the resulting pressure at critical points in the distribution system under different water supply demands and working conditions. Some recommended pressures are shown in the table below.

Case	Pressure (bar)
Houses up to 4 stories	1.8 – 2.8
Business areas	4.2
Fire fighting	5.3

Methods of analysis

1) Equivalent pipe method
2) Hardy Cross method
3) Circle method
4) Electrical method
5) Software such as **PIPE** $^{++}$ and **WATERCAD**.

Pipe Network Analysis

Analysis of water distribution system includes determining quantities of flow and head losses in the various pipe lines, and resulting residual pressures. In any pipe network, the following two conditions must be satisfied:

1. The algebraic sum of pressure drops around a closed loop must be zero, i.e. there can be no discontinuity in pressure.
2. The flow entering a junction must be equal to the flow leaving that junction; i.e. the law of continuity must be satisfied.

Based on these two basic principles, the pipe networks are generally solved by the methods of successive approximation. The widely used method of pipe network analysis is the Hardy-Cross method.

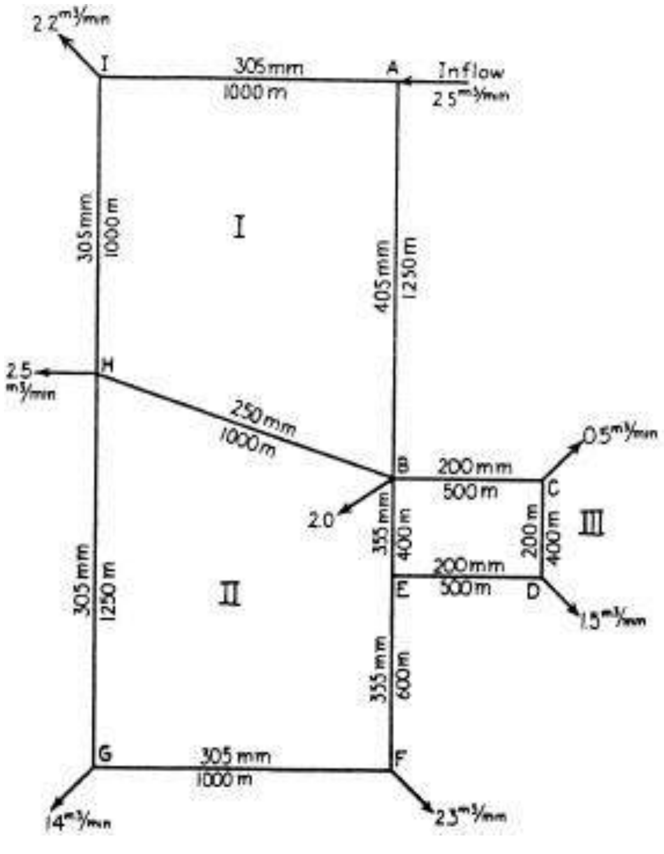
Hardy-Cross Method

This method consists of assuming a distribution of flow in the network in such a way that the principle of continuity is satisfied at each junction. A correction to these assumed flows is then computed successively for each pipe loop in the network, until the correction is reduced to an acceptable correction flow magnitude (**0.20 m³/min**).

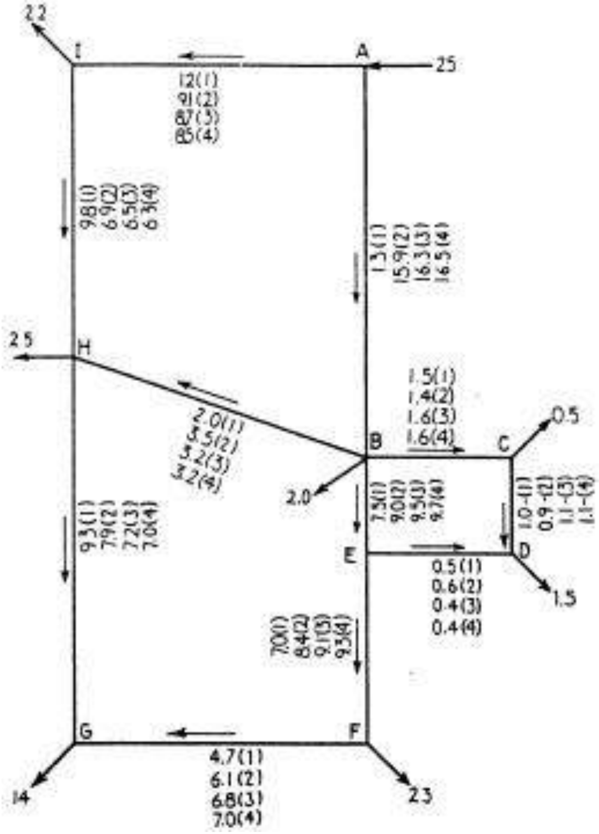
1. Assuming a distribution of flow in the network and balancing the resulting head losses (**Q $_{in}$ =Q $_{out}$**).
2. For a pipe circuit the sum of the head losses must be zero ($\Sigma h_f = 0$)
3. Clockwise flows are positive and produce positive head loss.

$$\Delta Q = \frac{\Sigma k Q_1^x}{\Sigma x k Q_1^{x-1}} = -\frac{\Sigma h}{x\Sigma\left(\frac{h}{Q}\right)} = -\frac{\Sigma h_f}{1.85\Sigma\left(\frac{h_f}{Q}\right)} \leq 0.2 m^3/\text{min}$$

Problem 11.1: The following example is solved by the Hardy cross method. Determine the residual pressure at G if the static pressure at A is 650 kPa and is it adequate for fire supply of 170 kPa. Assume (1 bar = 100 kPa = 14.503 psi, and 1 kPa = 0.102 m of water head).



Solution:



Steps taken in investigation of distribution system.

FIRST CORRECTION:

Loop I

Line	Flow, m³/min	Dia, m	Length, m	s	h, m	h/Q, m/m³/min
AB	13	0.40	1250	0.0110	13.75	1.058
BH	2	0.25	1100	0.0033	3.63	1.815
HI	−9.8	0.30	1000	−0.0260	−26.00	2.653
IA	−12	0.30	1000	−0.0380	−37.80	3.150
					−46.42	8.676

$$\Delta_I = -\frac{-46.42}{1.85(8.676)} = 2.9$$

Loop II

Line	Flow, m³/min	Dia, m	Length, m	s	h, m	h/Q, m/m³/min
BE	7.5	0.35	400	0.0075	3.00	0.400
EF	7.0	0.35	600	0.0066	3.96	0.566
FG	4.7	0.30	1000	0.0067	6.68	1.423
GH	−9.3	0.30	1250	−0.0236	−29.54	3.177
HB	−2.0	0.25	1100	−0.0033	−3.63	1.815
					−19.53	7.381

$$\Delta_{II} = -\frac{-19.53}{1.85(7.381)} = 1.4$$

Loop III

Line	Flow, m³/min	Dia, m	Length, m	s	h, m	h/Q, m/m³/min
BC	1.5	0.20	500	0.0058	2.91	1.937
CD	1.0	0.20	400	0.0028	1.10	1.110
DE	−0.5	0.20	500	−0.0008	−0.38	0.762
EB	−7.5	0.35	400	−0.0075	−3.00	0.400
					0.63	4.209

$$\Delta_{III} = -\frac{0.63}{1.85(4.209)} = -0.1$$

SECOND CORRECTION:

Loop I

Line	Flow, m^3/min	Dia, m	Length, m	s	h, m	h/Q, m/m^3/min
AB	15.9	0.40	1250	0.0157	19.65	1.236
BH	3.5	0.25	1100	0.0094	10.34	2.954
HI	−6.9	0.30	1000	−0.0136	−13.60	1.971
IA	−9.1	0.30	1000	−0.0227	−22.70	2.495
					−6.31	8.656

$$\Delta_I = 0.4$$

Loop II

Line	Flow, m^3/min	Dia, m	Length, m	s	h, m	h/Q, m/m^3/min
BE	9.0	0.35	400	0.0105	4.20	0.467
EF	8.4	0.35	600	0.0093	5.58	0.664
FG	6.1	0.30	1000	0.0108	10.80	1.770
GH	−7.9	0.30	1250	−0.0175	−21.88	2.769
HB	−3.5	0.25	1100	−0.0094	−10.34	2.954
					−11.64	8.624

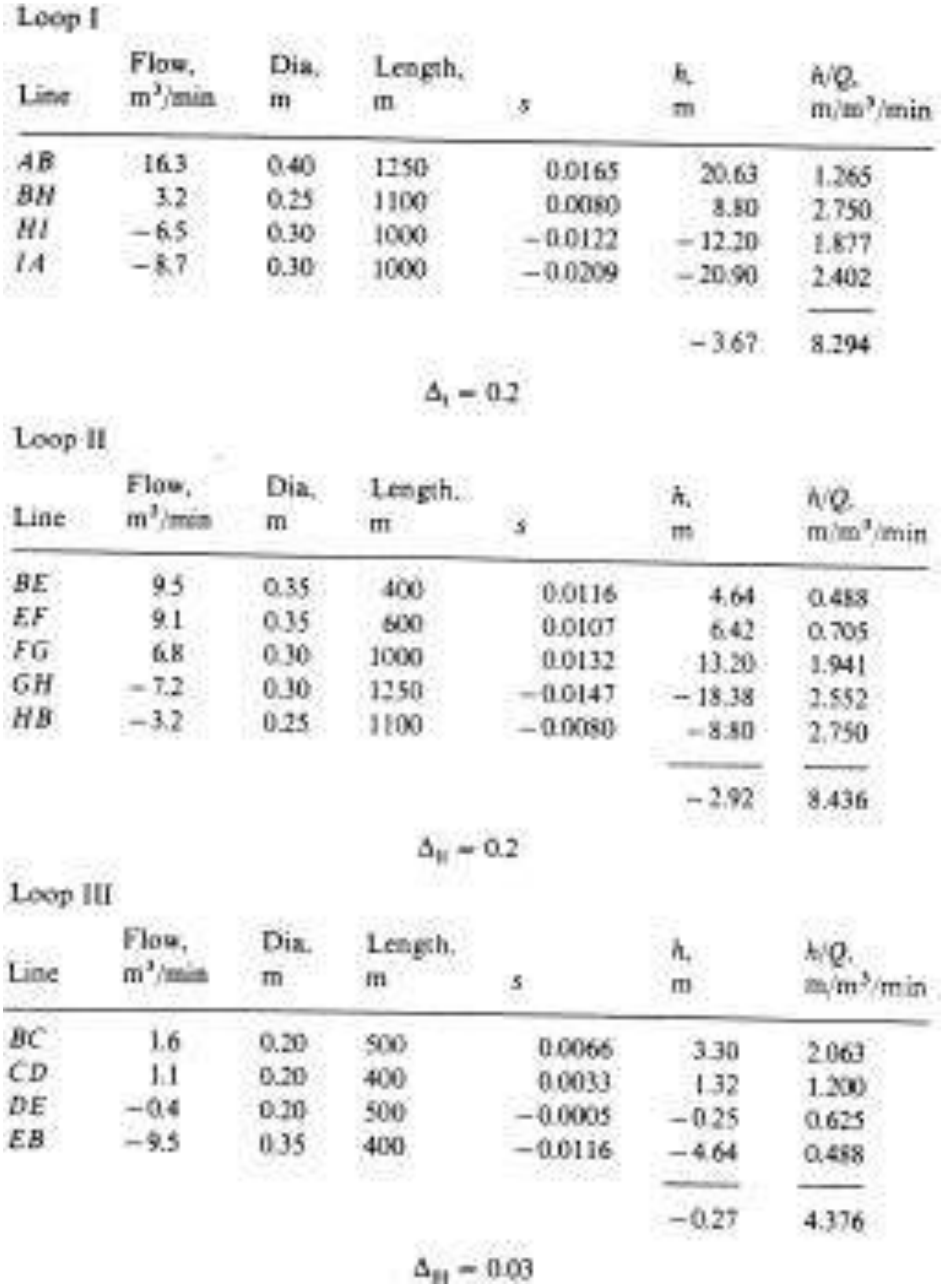
$$\Delta_{II} = 0.7$$

Loop III

Line	Flow, m^3/min	Dia, m	Length, m	s	h, m	h/Q, m/m^3/min
BC	1.4	0.20	500	0.0051	2.55	1.821
CD	0.9	0.20	400	0.0023	0.92	1.022
DE	−0.6	0.20	500	−0.0011	−0.55	0.917
EB	−9.0	0.35	400	−0.0105	−4.20	0.467
					−1.28	4.227

$$\Delta_{III} = 0.2$$

THIRD CORRECTION:



Loop I

Line	Flow, m^3/min	Dia, m	Length, m	s	h, m	h/Q, m/m^3/min
AB	16.3	0.40	1250	0.0165	20.63	1.265
BH	3.2	0.25	1100	0.0080	8.80	2.750
HI	−6.5	0.30	1000	−0.0122	−12.20	1.877
IA	−8.7	0.30	1000	−0.0209	−20.90	2.402
					−3.67	8.294

$$\Delta_I = 0.2$$

Loop II

Line	Flow, m^3/min	Dia, m	Length, m	s	h, m	h/Q, m/m^3/min
BE	9.5	0.35	400	0.0116	4.64	0.488
EF	9.1	0.35	600	0.0107	6.42	0.705
FG	6.8	0.30	1000	0.0132	13.20	1.941
GH	−7.2	0.30	1250	−0.0147	−18.38	2.552
HB	−3.2	0.25	1100	−0.0080	−8.80	2.750
					−2.92	8.436

$$\Delta_{II} = 0.2$$

Loop III

Line	Flow, m^3/min	Dia, m	Length, m	s	h, m	h/Q, m/m^3/min
BC	1.6	0.20	500	0.0066	3.30	2.063
CD	1.1	0.20	400	0.0033	1.32	1.200
DE	−0.4	0.20	500	−0.0005	−0.25	0.625
EB	−9.5	0.35	400	−0.0116	−4.64	0.488
					−0.27	4.376

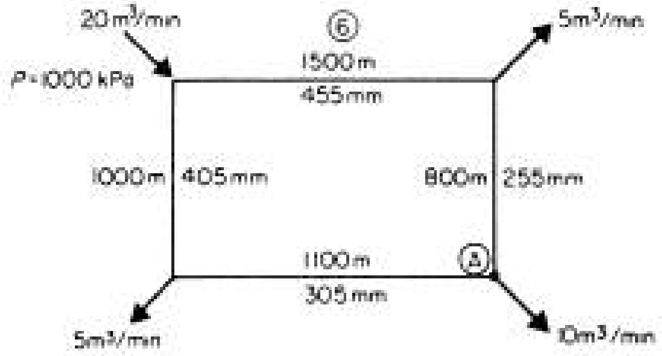
$$\Delta_{III} = 0.03$$

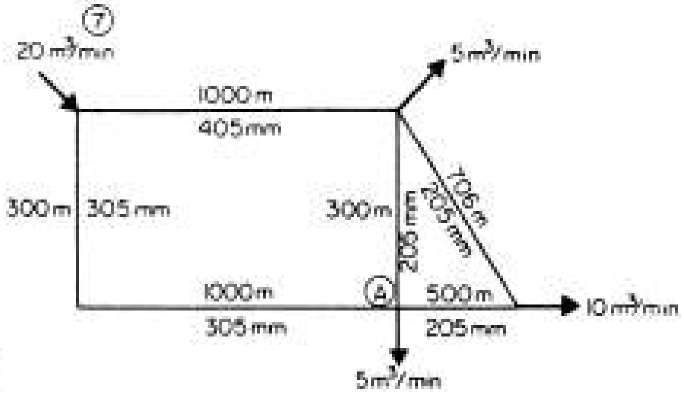
The maximum head loss:

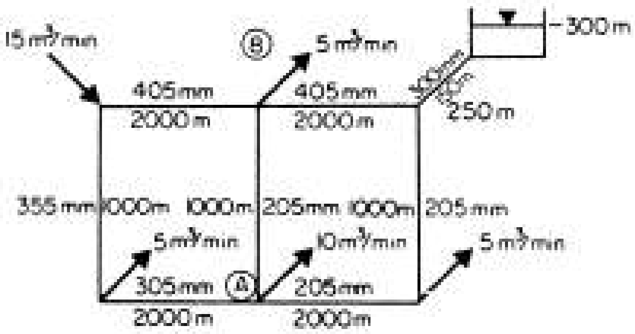
Path 1	Head loss	Path 2	Head loss	Path 3	Head loss
AI	20.90	AB	20.63	AB	20.63
IH	12.20	BH	8.80	BE	4.64
HG	18.38	HG	18.38	EF	6.42
				FG	13.20
T.H.L (m)	51.5		46.81		44.89
T.H.L (kPa)	505		468.72		440
Residual pressure at G	**<u>145</u>**		181.28		210

The residual pressure at G = 650 – 505 =145 kPa

Homework: In the pipe systems shown, find the distribution of flow and the pressure at A. assume all pipes not otherwise labeled are at identical elevations?



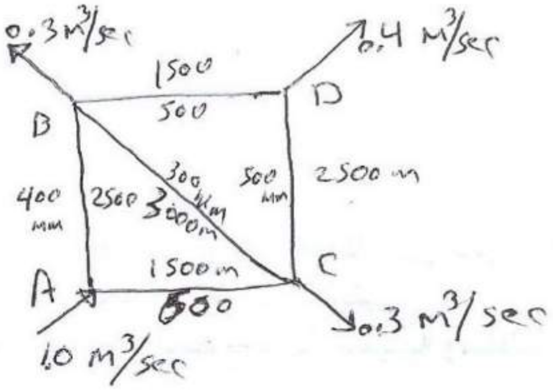




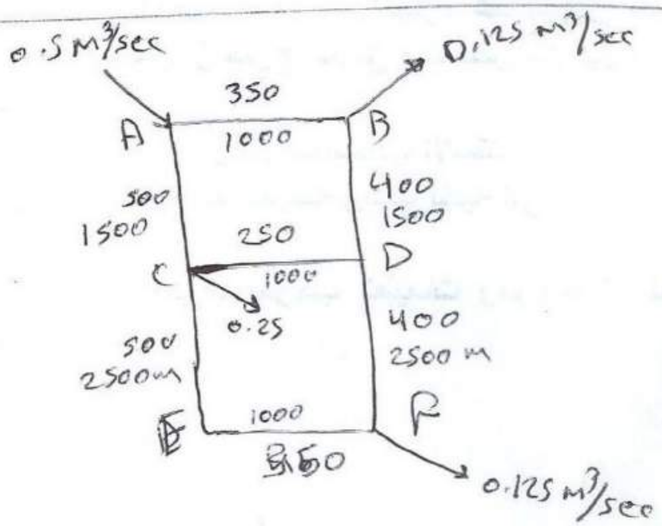
Homework: Define fire hydrant and what are the requirements of fire hydrant as manufacturing and location specification? Chapter 6.24 page 140

Homework: Problems 6.3, 6.4, & 6.5

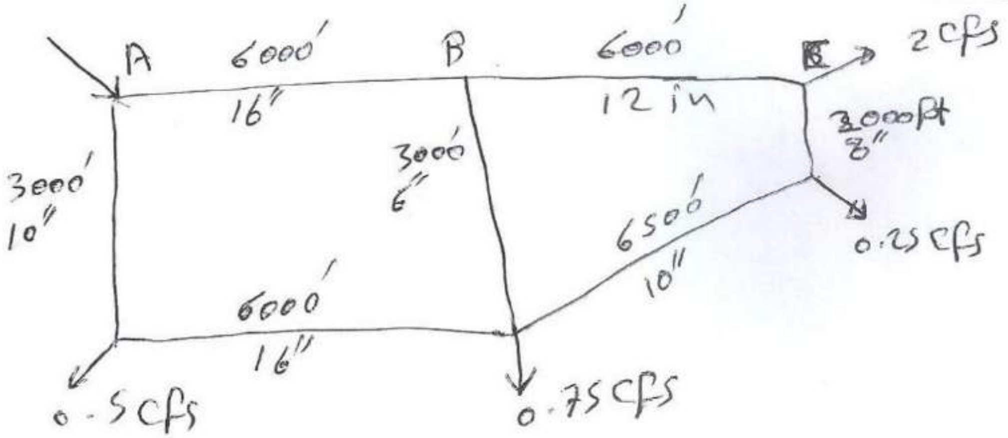
Homework: Design the water supply networks as shown below:



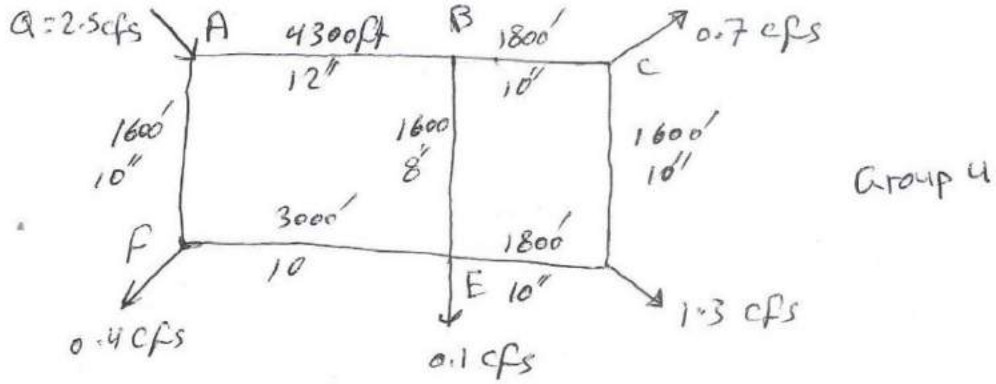
Group 1

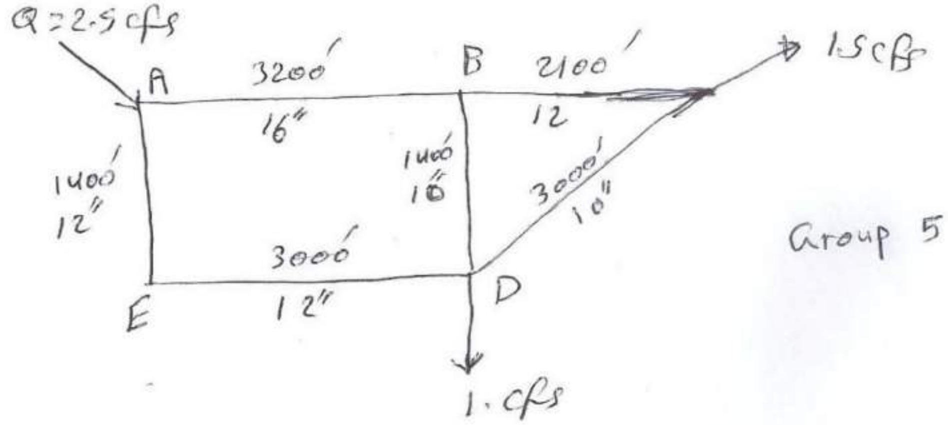


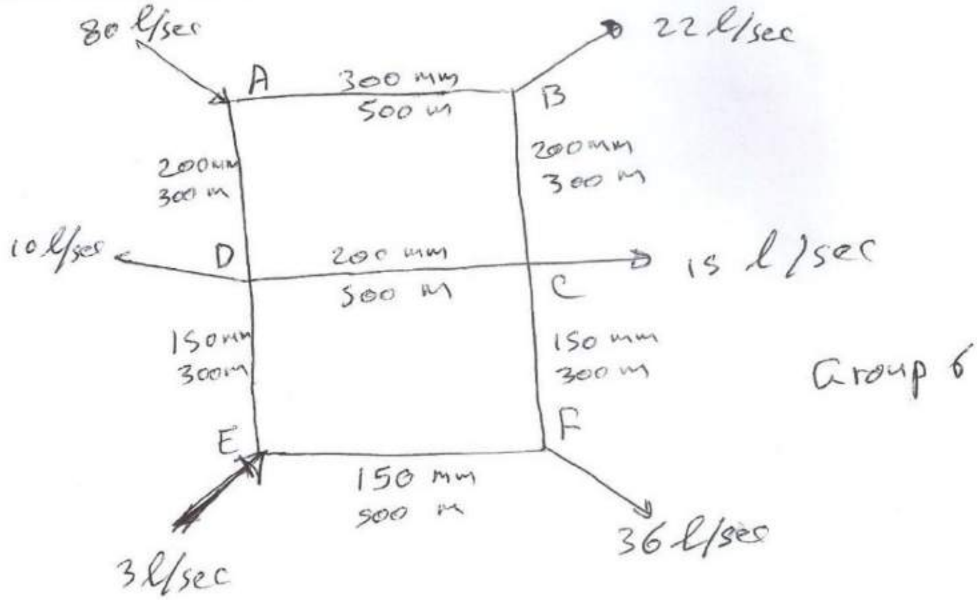
Group 2



Group 3


Q=2.5cfs
A
4300ft
12"
B
1800'
10"
0.7 cfs
C
1600'
10"
1600
8"
1600'
10"
Group 4
F
3000'
10
1800'
E 10"
1.3 cfs
0.4 cfs
0.1 cfs


Q=2.5 cfs
A
3200'
16"
B
2100'
12
1.5cfs
1400'
12"
1400'
10"
3000'
10"
Group 5
3000'
12"
E
D
1. cfs


80 l/sec
A
300 mm
500 m
22 l/sec
B
200mm
300 m
200mm
300 m
10 l/sec
D
200 mm
500 m
C
15 l/sec
150mm
300m
150 mm
300 m
Group 6
E
150 mm
500 m
F
3 l/sec
36 l/sec

Sanitary and Environmental Engineering

PART 1: WATER SUPPLY ENGINEERING

PART 1: WATER SUPPLY ENGINEERING

Lecture 5: Intakes and Screens

Intakes:

Definition: The intake is a structure made of several parts, mainly constructed to collect raw water from the source to water treatment plant. Intakes consist of the opening, strainer, or grating through which the water enters and conduit conveying the water, usually by gravity, to a well or sump. From the well the water is pumped to the mains or treatment plant.

Intakes should be so located and designed that possibility of interference with the supply is minimized and where uncertainty of continuous serviceability exists, intakes should be duplicated.

The following must be considered in designing and locating intakes:

a) The sources of water supply, whether impounding reservoir, lakes, or river (including the possibility of wide fluctuation in water level).
b) The character of the intake surroundings, depth of water, character of bottom, navigation requirements, the effects of currents, floods, and storms upon the structure and in scouring the bottom.
c) The location with respect to sources of pollution; and
d) The prevalence of the floating material such as ice, legs, and vegetation.

General requirements for the location of an intake:

1) Near to the water treatment plant (WTP).
2) Upstream waste disposal sites.
3) In pure water to avoid additional loading and/or complicated treatment.
4) Far away from navigation area.
5) It should be in deep water to provide sufficient quantity of water in dry weather conditions and if expansion is required in the water treatment plant.
6) Far away from the effects of currents, erosion and deposition.
7) In meandering rivers, the best location is on the concave side.

Intake criteria design:

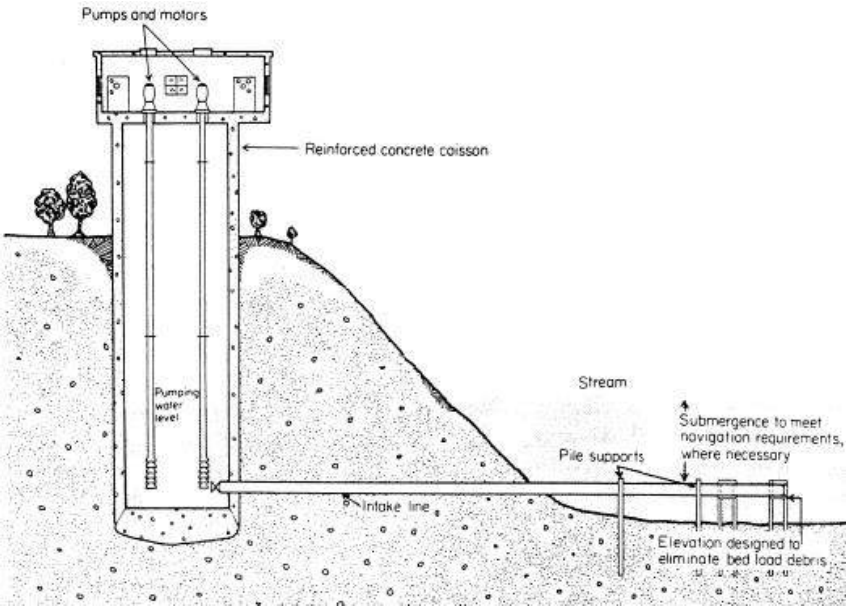
1) Water velocity through foot valve = (0.15-0.3) m/sec
2) Detention time in suction well = 20 min
3) Water velocity through suction pipe = (1-1.5) m/sec
4) Water velocity through back wash pipe = (3) m/sec
5) Water discharge through backwash pipe = $\frac{1}{3} suctic$ pipe
6) $Q=\frac{V}{T}$ & $Q=vA$

Types of intakes

1) Tower intake.
2) River intakes (pipe intake).

River intake:

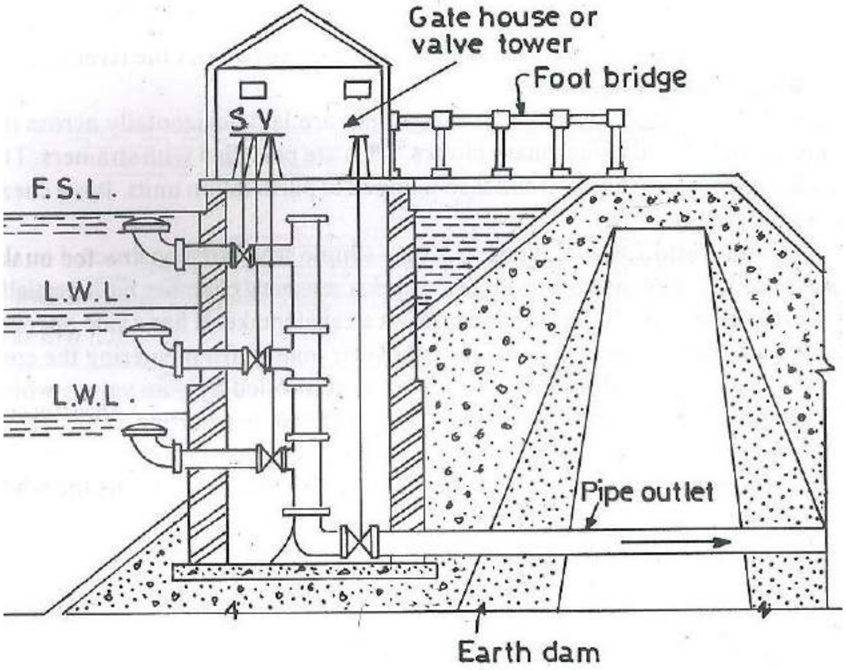
They must be sufficient stable, and the water deep enough to allow a submergence of at least 1 m at all times with a clear opening beneath the pipe so that any tendency to form a bar is overcome. River intakes are especially likely to need screens to exclude large floating matter which might injure pumps.



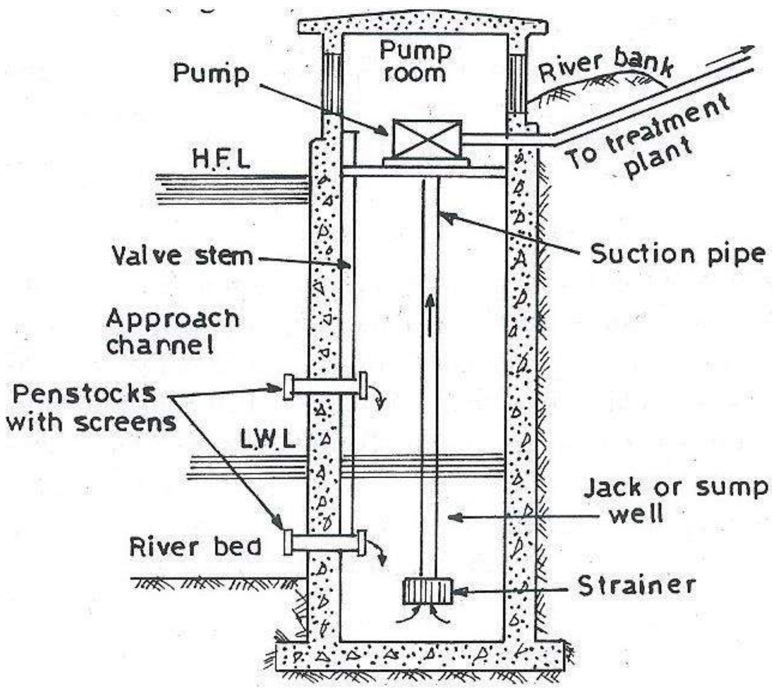
Screened pipe intake.

Intakes from impounding reservoirs:

1) The water of impounding reservoirs is likely to vary in quality at different levels, making it usually desirable to take water from about a meter below the surface.
2) The intake is usually a concrete tower located in deep water near the upstream toe of the dam.



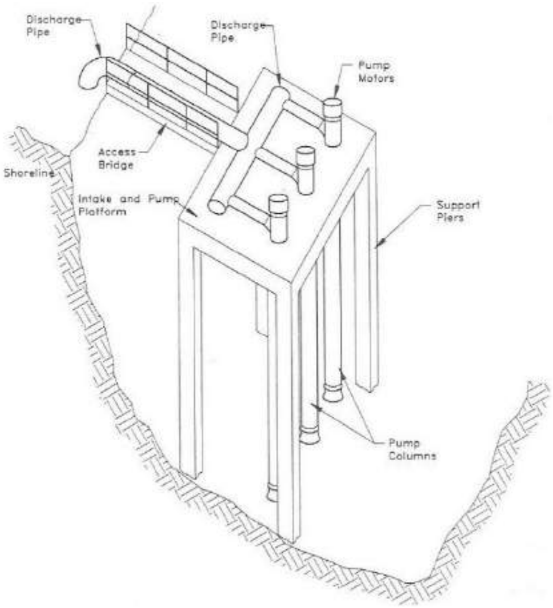
Reservoir intake.



River intake.

Lake intakes:

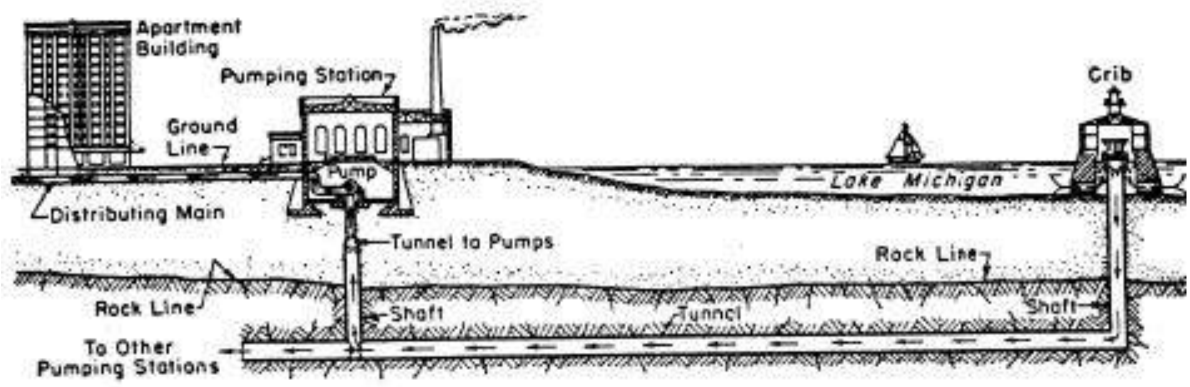
1) The intake should be so located that danger of the pollution will be minimized. This may require study of currents and effects of winds with particular attention to movement of sewage or industrial wastes, if these are discharged into lake.
2) Entering velocities less than (0.15 m/sec) have been used successfully.
3) To avoid offshore winds, intakes must be located at distance not less than (600-900 m) from shore.
4) A depth of (6-9 m) is required to avoid trouble caused by ice jams which may solidly fill the water at shallow depths.



Typical pier intake structure

The intake conduit:

1) A shore intake may also be the supply well for the suction pipe leading to the pumps.
2) Intakes located long distances from the pumps usually deliver their water to the pump well at the shore end by gravity.
3) The velocities in pipes will be low but not low enough to allow sedimentation.



Intake, tunnel and pump station.

Screens

Definition: Screening is a unit operation that removes floating and large suspended matter from water. Screens may be classified as coarse, fine, microstrainer, depending on the size of material removed. Screens may be located at the intake structure, raw water pump station, or water treatment plant.

The screen is a device with openings uniform in size. It is placed across the flow to retain floating particles. It serves as a protective unit.

Types of screens:

1) According to its shape 1- rack 2- mesh

2) According to its size:

a) Course, openings > 25 mm:

   Intake ports should be equipped with a coarse screen or bar rack to prevent large objects from entering the conveyance system. These screens consist of vertical flat bars or, in some cases a round pipes spaced with (5-8) cm of clear opening. The velocity through the coarse screen is generally less than 8 cm/s. Screens should be installed outside (on the water side) of any sluice gate or stop log slot, to prevent debris from interfering with their operation.

b) Medium 6 mm < openings < 25 mm

c) Fine, openings < 6 mm:

   Fine screens are used to remove smaller objects that may damage pumps or other equipment. They may be located either at the intake structure or at the raw water pump station. These screens consist of heavy wire mesh with 0.5 cm square openings or circular passive screens with similar opening widths. The screen area efficiency factor (0.5-0.6) and the typical velocity through the effective area is in the range of (0.4-0.8) m/s.

d) Microstrainer are used for the removal of plankton and algae from impounded waters. Therefore, microstrainer installed before chemical coagulation will often improve the performance of clarifiers.

3) According to its workability, manual and mechanical.

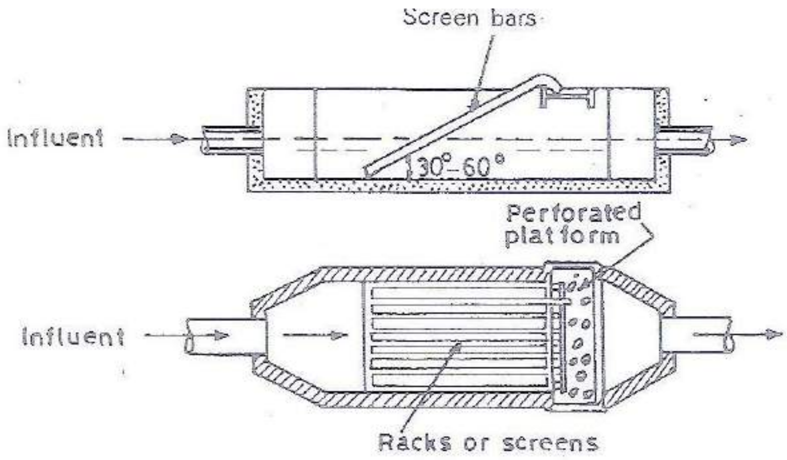


Fig. 7.2. Screening Chamber

Design criteria

1) Flowing velocity through the openings = 0.3 – 1.0 m/sec
2) Head loss (h_L) = 150 -300 mm

$$h_L = \frac{(v_s^2 - v_c^2)}{2g} \times \frac{1}{0.7}$$

Where:
v_s = Velocity through the screen (m/s)
v_c = Velocity in the canal (m/s)
g = Acceleration gravity (m/s^2)

SOLVED PROBLEMS

Problem 5.1: Design a coarse screen (rack) for a flow of 0.15 m^3/sec. The screen is placed in a canal 0.6 m in width and 0.4 m in height. Use rectangular bars 10 X 30 mm in cross section, assume size of the openings = 25 mm?

Solution:

Assume n = number of bars

$$\mathrm{Width\ of\ the\ canel} = \mathrm{n} \times \mathrm{Size\ of\ the\ bar} + (\mathrm{n}+1) \times \mathrm{Size\ of\ the\ openning}$$

$$600\ \mathrm{mm} = \mathrm{n} \times 10\ \mathrm{mm} + (\mathrm{n}+1) \times 25 \Rrightarrow 575 = 35\mathrm{n} \Rrightarrow \mathrm{n} = 16.4 \approx 16$$

Number of spacing = n+1= 16 +1 = 17

Check the velocity:

$$\mathrm{A_{Spacing}} = Area\ of\ openings = Height \times Width\ of\ openings$$

$$\mathrm{Width} = 600 - 16 \times 10 = 440\ \mathrm{mm} = 0.44\mathrm{m}$$

$$\mathrm{Velocity\ through\ the\ rack} = \frac{0.15 \frac{\mathrm{m}^3}{sec}}{0.4 \times 0.44} = 0.852 \frac{\mathrm{m}}{\mathrm{sec}}\ \mathrm{with\ acceptable\ velocity} \quad \mathrm{O.K}$$

$$\mathrm{Assume\ the\ velocity\ through\ the\ canal\ v_c} = \frac{\mathrm{Q}}{\mathrm{A_{Canal}}} = \frac{0.15}{0.6 \times 0.4} = 0.625\ \mathrm{m/sec}$$

$$\mathrm{Head\ loss\ through\ the\ openings\ h_L} = \frac{(\mathrm{v_s^2} - \mathrm{v_c^2})}{2\mathrm{g}} \times \frac{1}{0.7} = \frac{(0.852^2 - 0.625^2)}{2 \times 9.81} \times \frac{1}{0.7} = 0.0244\mathrm{m}$$

Or using 10 mm bars 35 mm c/c

$$\mathrm{Number\ of\ spaces} = \frac{\mathrm{Width}}{\mathrm{Spaces}\frac{\mathrm{c}}{\mathrm{c}}} = \frac{0.6\ \mathrm{m}}{0.035} \approx 17$$

$$\mathrm{Number\ of\ bars} = \mathrm{Number\ of\ spaces} - 1 = 17 - 1 = 16$$

$Size\ of\ openings = 35 - (5 + 5) = 25\ mm$, also, check vs

Or using velocity through openings 0.85 m/sec

Check the velocity: $\mathrm{v_s} = \frac{\mathrm{Q}}{\mathrm{A_{Spacing}}} \Rrightarrow A_s = \frac{0.15}{0.85} = 0.176\ m^2$

$$A_s = 0.6 \times 0.4 - 10 \times 10^{-3} \times n \times 0.4 \Rrightarrow n = 16\ \&\ n + 1 = 17$$

Also, check: $Size\ of\ spacing = 0.176 = 0.4 \times 17 \times s \Rrightarrow s = 0.026\ m = 26\ mm$

Problem 5.2: Design a coarse screen (rack) for a flow of 0.8 m^3/sec. The screen is placed in a channel 1.5 m in depth and 2 m width. Use square bars 13 X 13 mm in cross section, assume size of the openings c/c = 80 mm?

Solution:

$$\text{Width of the openning} = 8 - \frac{13}{10} = 6.7\ \text{cm}$$

Assume n= number of bars

$$\text{Width of the channel} = \text{n} \times \text{Size of the bar} + (\text{n} + 1) \times \text{Size of the openning}$$

$$2\ \text{m} = \text{n} \times 0.013\ \text{m} + (\text{n} + 1) \times 0.067\ \text{m} \Rightarrow 1.933 = 0.08\text{n} \Rightarrow \text{n} = 24.16 \approx 24$$

Number of openings = 24+1 = 25

Or, $\text{Number of spaces} = \frac{\text{Width}}{\text{Spaces c/c}} = \frac{2\ \text{m}}{0.08} = 25$

$$\text{Number of bars} = \text{Number of spaces} - 1 = 25 - 1 = 24$$

Assume freeboard depth for maintenance = 20 cm

$$\text{Depth of flow} = \text{channel depth} - \text{freeboard depth} = 1.5 - 0.2 = 1.3\ \text{m}$$

$$\text{Area of openings A} = \text{No. of opening} \times \text{Clear distance between bars} \times \text{Depth of flow}$$

$$\text{Area of openings A} = 25 \times 0.067 \times 1.3 = 2.178\ \text{m}^2$$

$$\text{Velocity through the rack} = \frac{0.8 \frac{\text{m}^3}{\text{sec}}}{2.178} = 0.367 \frac{\text{m}}{\text{sec}}\ \text{with acceptable velocity}$$

$$\text{assume the velocity through the canal} = 0$$

$$\text{Head loss through the openings h}_\text{L} = \frac{(\text{v}_\text{s}^2 - \text{v}_\text{c}^2)}{2\text{g}} \times \frac{1}{0.7} = \frac{(0.367^2 - 0)}{2 \times 9.81} \times \frac{1}{0.7} = 0.0098\ \text{m}$$

$$\text{Head loss through the openings, h}_\text{L} = \frac{(\text{v}_\text{s}^2 - \text{v}_\text{c}^2)}{2\text{g}} \times \frac{1}{0.7} = \frac{(0.364^2 - 0)}{2 \times 9.81} \times \frac{1}{0.7} = 0.00964\ \text{m}$$

Problem 5.3: Design a raw water intake system as a headwork to water treatment plant at design capacity 10000 m^3/hr including (10 suction pipes, 10 discharge pipes, 2 header or distributor, backwash pipe and wet well)? Assume water velocity in suction, discharge, header and backwash pipes 1, 2.25, 2, and 3 m/sec respectively and detention time in wet well 20 min.

Solution:

Suction Pipe:

$$Q_{SP} = v \times A = \frac{10000}{10} = 1000\frac{m^3}{hr} = 0.278\frac{m^3}{sec} = 1\frac{m}{sec} \times \frac{\pi}{4} \times D_{SP}^2 \rightarrow D_{SP} = 0.595 \approx 0.6\,m$$

Discharge Pipe:

$$Q_{DP} = v \times A = \frac{10000}{10} = 1000\frac{m^3}{hr} = 0.278\frac{m^3}{sec} = 2.25\frac{m}{sec} \times \frac{\pi}{4} \times D_{DP}^2 \rightarrow D_{DP} = 0.396 \approx 0.4\,m$$

Header Pipe:

$$Q_{HP} = v \times A = \frac{10000}{2} = 5000\frac{m^3}{hr} = 1.388\,\frac{m^3}{sec} = 2\frac{m}{sec} \times \frac{\pi}{4} \times D_{HP}^2 \rightarrow D_{HP} = 0.94 \approx 1\,m$$

Backwash Pipe:

$$Q_{BP} = v \times A = \frac{1000}{3} = 333.34\,\frac{m^3}{hr} = 0.0926\,\frac{m^3}{sec} = 3\frac{m}{sec} \times \frac{\pi}{4} \times D_{BP}^2 \rightarrow D_{BP} = 0.198 \approx 0.2\,m$$

Wet well:

Assume one wet well with two department

$$Q_{WW} = \frac{V}{t} = 5 \times 1000 = 5000\,\frac{m^3}{hr} = \frac{V}{\frac{20}{60}} \rightarrow V = 1666.67\,m^3$$

Assume effective water depth = 3 m

Assume rectangular cross-section and L = 4W

$$V = Depth \times A \rightarrow 1666.67 = 3 \times A \rightarrow A = 555.556\,m^2 = 4W^2 \rightarrow W = 11.785 \approx 12\,m\ and\ L = 48\,m$$

$$Total\ depth = 3 + Freeboard(0.5) = 3.5\,m$$

So, use two wet well each at dimension (48 x 12 x 3.5) m

Sanitary and Environmental Engineering

PART 1: WATER SUPPLY ENGINEERING

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Lecture 6: Pumps and Pumping Stations

A pump is a device which converts mechanical energy into hydraulic energy. It lifts water from a lower to a higher level and delivers it at high pressure. Pumps are employed in water supply projects at various stages for following purposes:

1. To lift raw water from wells.
2. To deliver treated water to the consumer at desired pressure.
3. To supply pressured water for fire hydrants.
4. To boost up pressure in water mains.
5. To fill elevated overhead water tanks.
6. To back-wash filters.
7. To pump chemical solutions, needed for water treatment.

Classification of Pumps

Based on principle of operation, pumps may be classified as follows:

1. Displacement pumps (reciprocating, rotary)
2. Velocity pumps (centrifugal, turbine and jet pumps)
3. Buoyancy pumps (air lift pumps)
4. Impulse pumps (hydraulic rams)

Work and efficiency of pumps

The work done by a pump is equal to the product of the mass flow and the total head against which the flow is moved.

Head, hydraulic energy, kinetic or potential energy is defining as equivalent to the potential energy of a column of water of specified height.

Pressure may be expressed in terms of either absolute or gauge reading

$One\ foot\ of\ water = 0.433\ psi$ or $One\ meter\ of\ water = 9.81\ kPa$

Total dynamic head (TDH) of a pump is the sum of the static suction head (SSH), the static discharge head (SDH), the total friction head (T.H.L), and velocity head (DH)

$$TSH = SSH + SDH$$

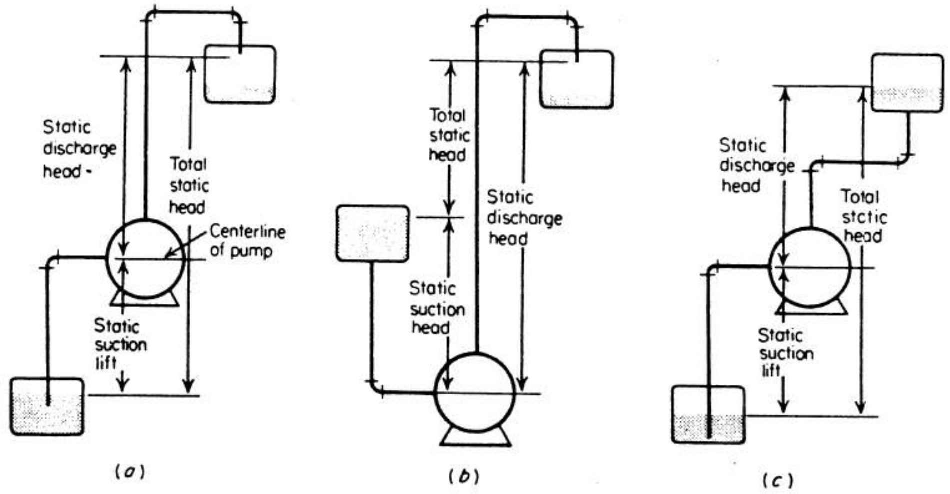


Fig. 1 Head terms used in pumps

Pump capacity (Q) is a term used to describe the maximum flow rate through a pump at its designed conditions. It is a measurement usually given in gallons per minute (gpm) or cubic meters per hour (m^3/h).

Head (H) is expressed in units of height such as meters or feet. The static head of a pump is the maximum height (pressure) it can deliver. The capability of the pump at a certain RPM can be read from its Q-H curve (flow vs. height).

$$\begin{aligned} Total\ Head\ (TH) & \\ &= Static\ head (SH) + Dynamic\ head\ (DH) + Total\ head\ loss\ (THL) \\ &+ Pressure\ required\ (P_r) \end{aligned}$$

Pump efficiency (η) is defined as the ratio of water horsepower output from the pump to the shaft horsepower input for the pump. The efficiency of a particular pumps estimated by determining two values. These values are pump flow rate and total head.

$$\eta = \frac{P_M}{P_W}\ \ \&\ \eta = \eta_{Pump} \times \eta_{Motor}$$

Pump water power (Pw), In the hydraulic field, the load of a pump is expressed in theory in height of water. It is the hydraulic power communicated to the liquid of its passage through the pump (Watt or kW).

$$P_W = \gamma \times Q \times H$$

Where:

Pw: water power (Kw), : water weight density (9.81 kN/m^3), Q: pump capacity (m^3/sec), and H: total head (m).

Pump Power (P) is a metric in fluid dynamics that quantifies the energy-efficiency of pump systems. It is a measure of the electric power that is needed to operate a pump (or collection of pumps), relative to the volume flow rate. ... It is commonly used when measuring the energy efficiency of buildings.

$$P_{E\ OR\ M} = \frac{P_w}{\eta} = \frac{\gamma \times Q \times H}{\eta}$$

Effect of varying speed

$$\frac{Q_1}{Q_2} = \frac{N_1}{N_2}, \frac{TDH_1}{TDH_2} = \frac{N_1^2}{N_2^2} \ \& \ \frac{P_1}{P_2} = \frac{N_1^3}{N_2^3}$$

Characteristics curves of centrifugal pumps:

Pump Characteristic Curves. The performance of a centrifugal pump can be shown graphically on a characteristic curve. A typical characteristic curve shows the total dynamic head, brake horsepower, efficiency, and net positive Suction head all plotted over the capacity range of the pump.

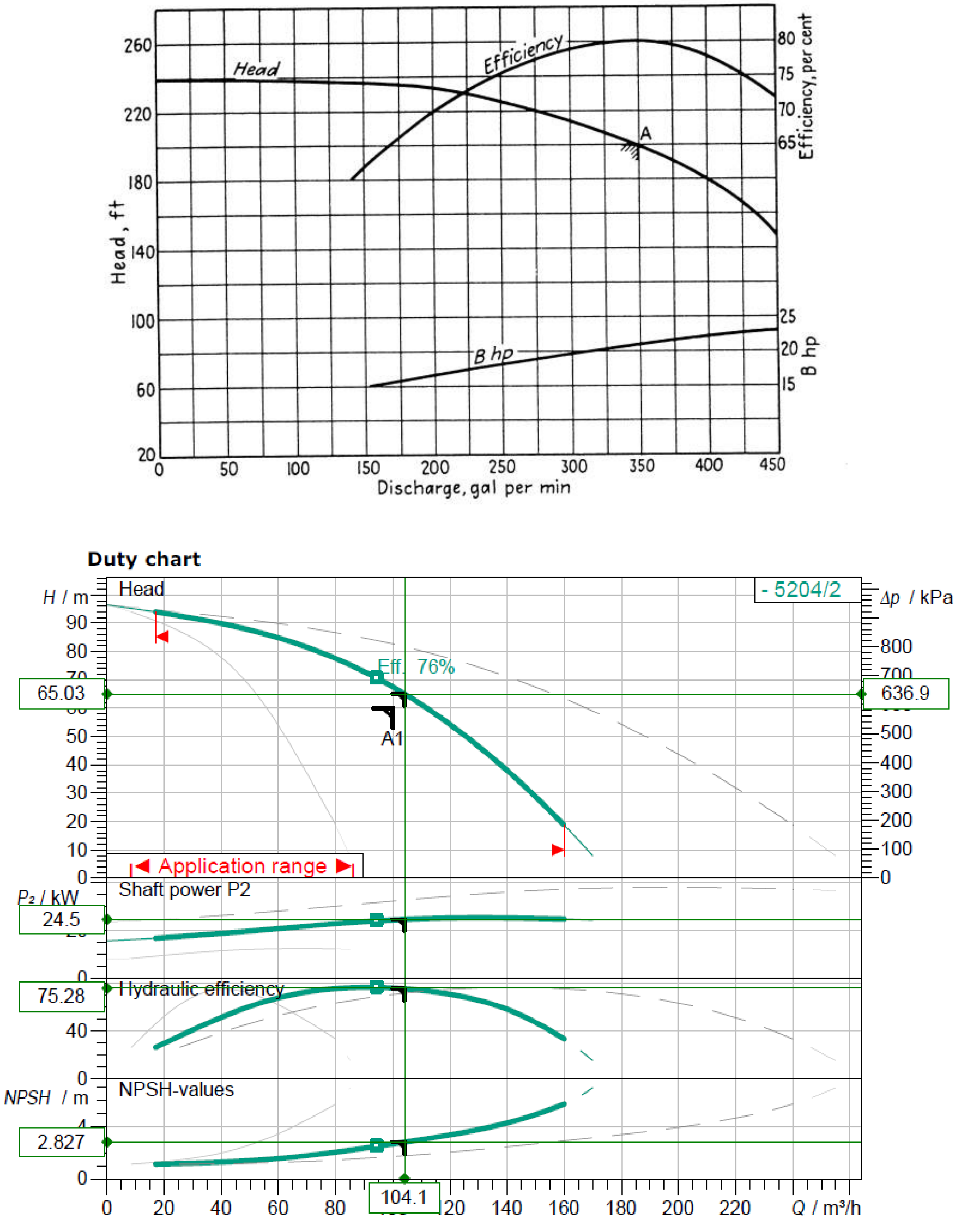


Fig. 2 Characteristic curves of a centrifugal pump at constant speed

Suction lift:

Net positive suction head (NPSH) is total head at pump suction branch over and above the vapor pressure of the liquid being pumped. Also, it's a function of the system design, and represent the force available to drive the flow into the pump.

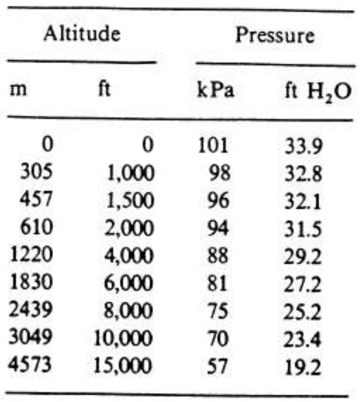
NPSH required (NPSHr) is a function of the pump design and is the lowest value of NPSH at which the pump can be guaranteed to operate without significant cavitation.

Available NPSH is the sum of the barometric pressure and the static head on the pump inlet, less the losses and fittings and vapor pressure of the water.

$$NPSH_{Available} = P_B - h_L - P_v - P_s$$

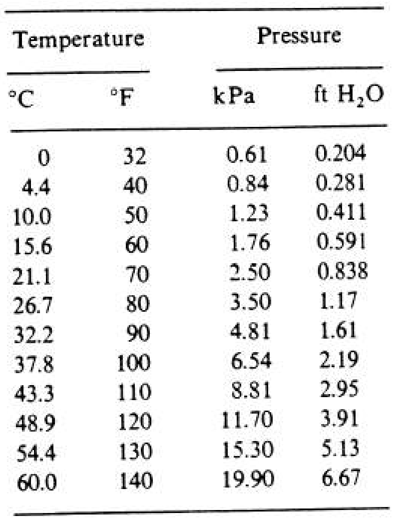
Where; P_B: Barometric Pressure, h_L: Head losses, P_v: Vapor Pressure, Ps: Allowable suction lift of the pump.

Barometric pressure vs. altitude



Altitude		Pressure	
m	ft	kPa	ft H_2O
0	0	101	33.9
305	1,000	98	32.8
457	1,500	96	32.1
610	2,000	94	31.5
1220	4,000	88	29.2
1830	6,000	81	27.2
2439	8,000	75	25.2
3049	10,000	70	23.4
4573	15,000	57	19.2

Vapor pressure of water vs. temperature



Temperature		Pressure	
°C	°F	kPa	ft H_2O
0	32	0.61	0.204
4.4	40	0.84	0.281
10.0	50	1.23	0.411
15.6	60	1.76	0.591
21.1	70	2.50	0.838
26.7	80	3.50	1.17
32.2	90	4.81	1.61
37.8	100	6.54	2.19
43.3	110	8.81	2.95
48.9	120	11.70	3.91
54.4	130	15.30	5.13
60.0	140	19.90	6.67

Cavitation is the formation of vapor cavities in a liquid, small liquid-free zones ("bubbles" or "voids"), that are the consequence of forces acting upon the liquid.

$$\text{NPSH}_{\text{available}} < \text{NPSH}_{\text{required}}$$

Pump stations:

Pumping stations are facilities including pumps and equipment for pumping fluids from one place to another. They are used for a variety of infrastructure systems, such as the supply of water to canals, the drainage of low-lying land, and the removal of sewage to processing sites.

Design consideration for pump stations:

1) Location: the location of pump stations has an important bearing on the pressure maintained in the distribution system.
2) Architecture: the building should be adequate in size and have space for placing additional units as needed. It should be fireproof, pleasing in appearance, and have well-kept grounds. Its architecture can and should be in harmony with surroundings.
3) Capacity and operation: the pumps must have sufficient capacity to care for peak load, and the designer and operator must furnish and the pumping plant may operate 8-12 hours.

SOLVED PROBLEMS

Problem 6.1: Residential campus of 50,000 inhabitants and per capita consumption of 250 liters per day and assume that the water needed to fire demand for 10 hr duration. Determine:

a) All draft water demand and fire demand?

b) Total daily water flow rate and water volume?

c) Dimension of water storage capacity (four parts) each L=8W & D= 5m?

d) NO. of duty and standby pump if available pump in Iraqi Market a centrifugal pump operates at a speed of 1450 r/min and Q =250 m^3/hr, H=40 and efficiency=55%?

e) Calculate the pump power and horse power?

f) Compute the efficiency of the pump, the discharge, head, and power if the pump speed were changed to 2900 r/min?

Solution:

a) $Average\ flowrate = 0.25 \times 50000 = 12500\ m^3/day$

$$Maximum\ daily\ demand = 1.8 \times 12500 = 22500\ m^3/day$$

$$Maximum\ weekly\ demand = 1.48 \times 12500 = 18500\ m^3/day$$

$$Maximum\ monthly\ demand = 1.28 \times 12500 = 16000\ m^3/day$$

$$Maximum\ hourly\ demand = 2.7 \times 12500 = 33750\ m^3/day$$

$$Mimum\ hourly\ demand = 0.9 \times 12500 = 11250\ m^3/day$$

$$G = 1020 \times \sqrt{P} \times \left(1 - 0.01\sqrt{P}\right) = 1020 \times \sqrt{50} \times \left(1 - 0.01\sqrt{50}\right) = 6702.49\ gpm$$

$$= 1522.135 \frac{m^3}{hr} = 36531.247\ m^3/day$$

b) $Total\ flowrate = 22500 + 36531.247 = 59031.247 \frac{m^3}{day} \approx 2460\ m^3/hr$

$$Total\ water\ volume = 22500 + 36531.247 \times \frac{10}{24} = 37721.353\ m^3 \approx 37722\ m^3 > MMD$$

c) $V = \frac{37722}{4} = L \times W \times D = 8W^2 \times 5 \rightarrow W = 15.35\ m\ L = 122.83\ m\ \&\ D = 5m$

d) $NO.of\ duty\ pumps = \frac{2460}{250} \approx 10$

$$NO.of\ standby\ pumps = \frac{10}{3} \approx 3$$

So, use 13 number of pumps (10NO.s duty pump +3NO standby pump)

e) $P_w = \gamma \times Q \times H = 9.81 \times \frac{250}{3600} \times 40 = 27.25 \approx 28\ kW$

$$P_m = \frac{P_w}{\eta} = \frac{28}{0.55} = 50.9\ kW \rightarrow \frac{50.9}{0.745} = 68.34\ hp$$

f) $\frac{Q_1}{Q_2} = \frac{N_1}{N_2} \rightarrow \frac{250}{Q_2} = \frac{1450}{2900} \rightarrow Q_2 = 500\ m^3/hr$

$$\frac{H_1}{H_2} = \frac{N_1^2}{N_2^2} \rightarrow \frac{40}{H_2} = \frac{1450^2}{2900^2} \rightarrow H_2 = 160\ m$$

$$\frac{P_1}{P_2} = \frac{N_1^3}{N_2^3} \rightarrow \frac{50.9}{P_2} = \frac{1450^3}{2900^3} \rightarrow P_2 = 407.2\ kW$$

Sanitary and Environmental Engineering

PART 1: WATER SUPPLY ENGINEERING

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Lecture 7: Plain Sedimentation

Water Treatment-Clarification Process

Process	Function
Chemical sedimentation	Removal of colloidal particles
Aeration, chemical oxidizing agents, and adsorbents	Removal of taste and odors
Softening	Removal of divalent cations-generally calcium and magnesium
Stabilization	Prevent corrosion or deposition in distribution system.

Sedimentation theory:

Sedimentation is the separation of impurities known as discrete particles from the flowing fluid (flow) by the action of natural forces (gravity). This process takes place in a basin known as the sedimentation tank or the clarifier.

Discrete particles are the particles that do not change in size, shape or in weight when falling freely through the flowing flow.

Forces acting on the falling particle:

Assume that the particle is a sphere with diameter (d)

F_i = Impelling (driving) force = $F_g - F_b$

F_g =Gravity force, $F_g = g \times \rho_s \times V$

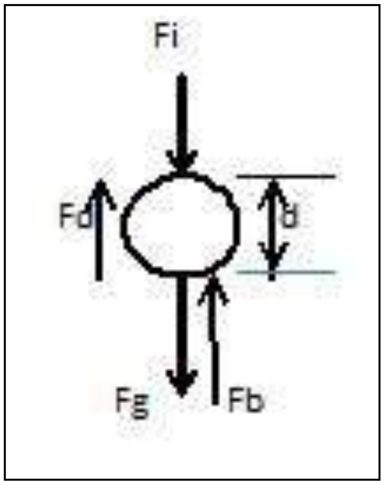
F_b = Buoyancy force, $F_b = g \times \rho_w \times V$

g = Gravity acceleration

Volume = Particle volume, $V = \frac{4\pi r^3}{3} = \frac{\pi d_s^3}{6}$

ρ_s = Density of the solids (Particles)

ρ_w = Density of the fluid (Water)



Then, $F_i = \frac{g \times \pi \times d_s^3}{6} \times (\rho_s - \rho_w)$

F_d = Drag or frictional force, $F_d = \frac{C_D \times V_s \times A \times \rho_w}{2}$

C_D = Drag coefficient.

V_s = Settling velocity of the particle.

A = Projection area of the particle on the direction of the flow, $A = \frac{\pi d_s^2}{4}$

The rate of change in momentum of a particle of mass (m) settling in a still fluid is:

$$\left(\frac{m \times dV_s}{dt} = F_i - F_d\right)$$

When the settling velocity is constant then, $F_i = F_d$

Then, $V_s = \sqrt{\frac{4gd_s(\rho_s - \rho_w)}{3C_D\rho_w}} = \sqrt{\frac{4gd(G_s - 1)}{3C_D}}$ that is known as Newton's settling formula.

G_s = Specific gravity of the particles, $G_s = \frac{\rho_s}{\rho_w}$

For laminar flow conditions; $C_D = \frac{24}{R_e} = \frac{24\mu_w}{V_s \times d_s \times \rho_w}$ $\quad When\ R_e < 0.5$

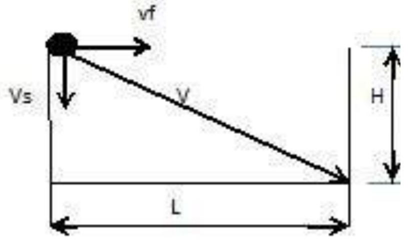
For transition flow conditions; $C_D = \frac{24}{R_e} + \frac{3}{\sqrt{R_e}} + 0.34$ $\quad When\ R_e > 1000$

For turbulent flow conditions; $C_D = 0.4$

Where Re = Reynolds number, $R_e = \frac{V_s \times d_s \times \rho_w}{\mu_w}$ (μ_w = Viscosity of water)

Then, $V_s = \frac{gd_s^2(\rho_s - \rho_w)}{18\mu_w} = \frac{gd_s^2\rho_w(G_s - 1)}{18\mu_w}$ that is known as Stock's settling formula.

The design of sedimentation tanks is generally based upon the concept of the ideal sedimentation as shown in the figure below.



The particle entering the basin will have a horizontal velocity equal to the velocity of the flow (v_f) and a settling velocity (V_s) according to Stocks velocity.

For the particle to be removed from the flow, its resultant velocity should be equal the resultant velocity (V) which will carry it to the bottom of the tank.

Summary: if ($V_{f\ for\ particals} = V_{f\ for\ water\ flow}$) , ($V_{s\ for\ particals} = V_{s\ from\ Stock\ law}$) ,

($V_{R\ for\ particals} = \sqrt{{V_f}^2 + {V_s}^2}$) & $V_{R\ for\ water\ flow} = \sqrt{{V_f}^2 + {V_s}^2}$

So, the particles will be removed 100% if ($V_{R\ for\ particals} = V_{R\ for\ water\ flow}$)

Considering the velocity vector triangle and the dimensions of the tank (L= length, H = height and W = width).

$$Horizontal\ velocity\ \ v_f = \frac{Q}{A_c} = \frac{Q}{W \times H}$$

If particle is to be removed, it's settling velocity and horizontal velocity (v_f) must be such that their resultant, (V), will carry it to the bottom of the tank before the outlet zone is reached, all particles with the same settling velocity will be removed.

$$\frac{v_f}{V_s} = \frac{L}{H}\ then\ settling\ velocity\ V_s = \frac{v_f \times H}{L} = \frac{Q}{W \times H} \times \frac{H}{L} = \frac{Q}{W \times L} = \frac{Q}{A_s}$$

Where:
Q: The discharge entering the tank;
Ac: The cross-sectional area of the tank perpendicular to the flow = $W \times H$
As: The surface area= $W \times L$

Surface overflow rate (SOR or V_s) is numerically equal to the flow divide by the plan or surface area of the basin, but which physically represents the settling velocity of the smallest particle to be 100% removed from the flow or to be settled. Any particle of (V_s) larger or equal to SOR will settle ($V_{s\ for\ particle} \geq SOR$), and while those which settle at lower velocities will be removed in direct proportion to the ratio of their settling velocity to (V_s), so, if V_s is smaller than SOR, it will not settle ($V_{s\ for\ particle} < SOR$).

Weir Overflow Rates is the flow rate of the clarifier effluent per the length of the overflow weir. It is argued that if the weir loading rates exceed the recommended values, the velocity of currents approaching the weirs may be such that excessive solids are carried over the weir.

Ideal sedimentation

Solid liquid separation process in which a suspension is separated into two phases:

1) Clarified supernatant leaving the top of the sedimentation tank (overflow).
2) Concentrated sludge leaving the bottom of the sedimentation tank (underflow).

The following characteristics exhibit in an ideal sedimentation process:

1) The flow is evenly distributed across the tank;
2) The particles are evenly distributed in the flow.

Purpose of Settling:

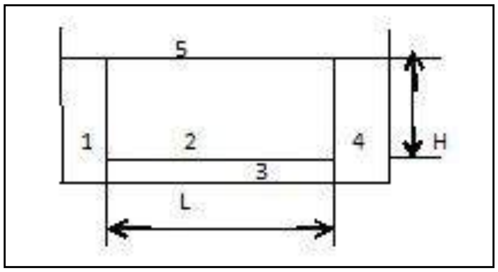
1) To remove coarse dispersed phase.
2) To remove coagulated and flocculated impurities.
3) To remove precipitated impurities after chemical treatment.
4) To settle the sludge (biomass) after activated sludge process / tricking filters.

Principle of Settling:

1) Suspended solids present in water having specific gravity greater than that of water tend to settle down by gravity as soon as the turbulence is retarded by offering storage.
2) Basin in which the flow is retarded is called settling tank.
3) Theoretical average time for which the water is detained in the settling tank is called the detention period.

An ideal sedimentation basin is divided into the five zones:

1) **Inlet zone:** Region in which the flow is uniformly distributed over the cross section such that the flow through settling zone follows horizontal path.
2) **Settling zone:** Settling occurs under quiescent conditions.
3) **Sludge zone:** For collection of sludge below settling zone.
4) **Outlet zone:** Clarified effluent is collected and discharge through outlet weir.
5) **Freeboard zone:** The distance between normal water level and the top of a structure, such as a dam, that impounds or restrains water.



Inlet and Outlet Arrangement:

Inlet devices: Inlets shall be designed to distribute the water equally and at uniform velocities. A baffle should be constructed across the basin close to the inlet and should project several feet below the water surface to dissipate inlet velocities and provide uniform flow;

Outlet Devices: Outlet weirs or submerged orifices shall be designed to maintain velocities suitable for settling in the basin and to minimize short-circuiting. Weirs shall be adjustable, and at least equivalent in length to the perimeter of the tank. However, peripheral weirs are not acceptable as they tend to cause excessive short-circuiting.

Types of Settling:

Type I-Discrete particle settling: Particles settle individually without interaction with neighboring particles.
Type II-Flocculent Particles: Flocculation causes the particles to increase in mass and settle at a faster rate.

Type III-Hindered or Zone settling: The mass of particles tends to settle as a unit with individual particles remaining in fixed positions with respect to each other.

Type IV-Compression: The concentration of particles is so high that sedimentation can only occur through compaction of the structure.

Types of sedimentation tanks:

1) Shape: rectangular, square, circular (cylindrical);
   a) Sedimentation tanks may function either intermittently or continuously. The intermittent tanks also called quiescent type tanks are those which store water for a certain period and keep it in complete rest. In a continuous flow type tank, the flow velocity is only reduced and the water is not brought to complete rest as is done in an intermittent type.
   b) Settling basins may be either long rectangular or circular in plan. Long narrow rectangular tanks with horizontal flow are generally preferred to the circular tanks with radial or spiral flow.
2) Direction of flow: horizontal, radial, vertical;
3) Detention time: full and with draw, continuous flow;
4) Plain sedimentation, sedimentation with chemicals

Design criteria

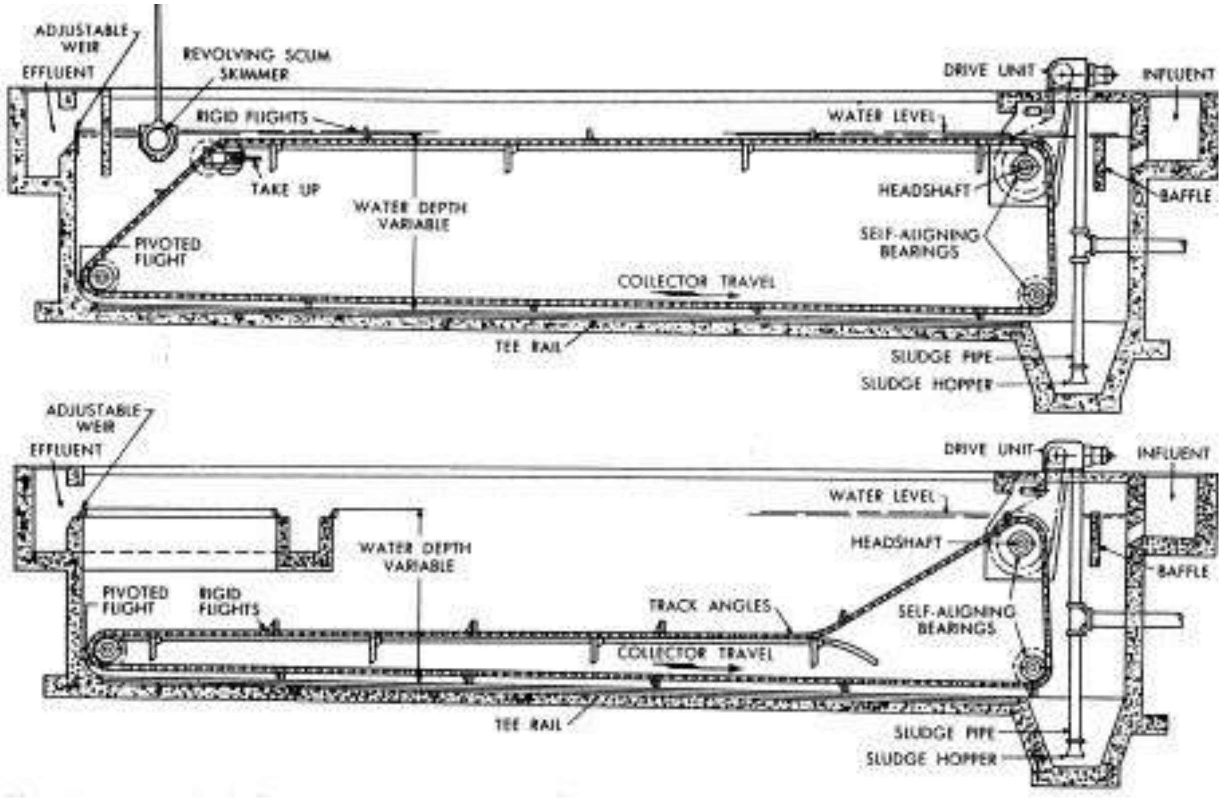
1) SOR: 20 – 80 m/d.
2) Detention time: 2- 6 hr (Average 3 –4 hr).
3) Depth or height (H): 2-6 m (Average 3–4 m).
4) Inlet velocity (v_f) 0.15 – 0.5 m/min.
5) In rectangular tanks L/W = (2/1 – 5/1), L max less than 100 m.
6) In circular tanks Diameter (D) max less than 50m, inlet diameter less than 10%D.
7) Weir loading rate ($\frac{\mathrm{Q}}{\text{Weir Effluent Length}}$): 120 – 250 m^3/m/d.
8) scouring velocity (v_h)

The horizontal velocity that will re-suspend the settled particles, $\mathrm{v_h} = \sqrt{\frac{8\beta g d_s(G_s-1)}{f}}$

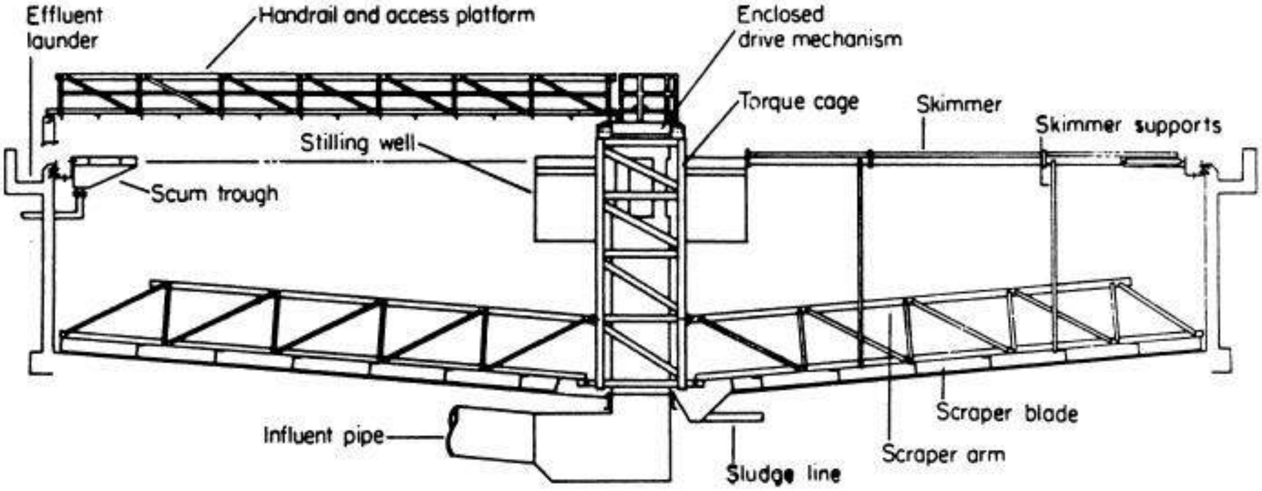
β = 0.04 – 0.06, f = 0.02 – 0.03

Why is it hard to reach ideal settling in sedimentation tanks?

1) Variable inlet flowing velocity.
2) Variable temperature inside the tank that will give different ρ_w and μ_w.
3) Un-similar particle distribution.
4) Short circuiting due to the shape of the tank.



Rectangular clarifiers.



Center flow clarifier.

Solved problems

Problem 7.1: Design a rectangular sedimentation tank to treat 393 m^3/hr flows. Assume the smallest particle to be 100% removed is 0.03 mm in diameter, detention time = 3hr, inlet flowing velocity = 0.15 m/min, temperature= 20 °C and Gs= 1.65. Find:

1) H, W and L of the tank.
2) Volume of the settled sludge per day (m^3/d) if the influent suspended solids (SS) concentration is 200 mg/L , the sludge is 5% in solid content and the removal efficiency is 80% in the tank.
3) Discuss the effect of temperature changes, T=5 °C in winter and T= 33 °C in summer.
4) The dimensions of a circular sedimentation tank for the same example.

Solution:

$$From\ table\ in\ appendex\ page\ 642\ \rho_w = 0.9982 \times 10^3 \frac{kg}{m^3}\ and\ \mu_w = 1.002 \times 10^{-3} \frac{kg}{m.sec}$$

$$V_s = \frac{g\rho_w(G_s-1)d_s^2}{18\mu_w} = \frac{9.81 \times 0.9982 \times 10^3 \times (1.65-1) \times \left(0.03 \times 10^{-3}\right)^2}{18 \times 1.002 \times 10^{-3}} = 3.176 \times 10^{-4} \frac{m}{sec}$$
$$= 27.39 \frac{m}{day} = SOR$$

$$\mathrm{SOR} = \frac{\mathrm{Q}}{\mathrm{A_S}} \Rightarrow \mathrm{A_S} = \frac{\mathrm{Q}}{\mathrm{SOR}} = \frac{393 \frac{\mathrm{m^3}}{\mathrm{hr}} \times 24}{27.39 \frac{\mathrm{m^3}}{\mathrm{day}}} = 344.36\ \mathrm{m^2}$$

$$\mathrm{Q} = \frac{\mathrm{Volume}}{\mathrm{Time}} \Rightarrow \mathrm{Volume} = \mathrm{A_s} \times \mathrm{H} = \mathrm{Q} \times \mathrm{t}$$

$$\mathrm{Volume} = 344.36 \times \mathrm{H} = 393 \frac{\mathrm{m^3}}{\mathrm{hr}} \times 3\mathrm{hr} \Rightarrow \mathrm{H} = 3.42\ \mathrm{m\ O.K}$$

$$\mathrm{v_f} = \frac{\mathrm{L}}{\mathrm{t}} \Rightarrow \mathrm{L} = 0.15 \frac{\mathrm{m}}{\mathrm{min}} \times 3\mathrm{hr} \times 60 = 27\mathrm{m}$$

$$\mathrm{A_s} = \mathrm{L} \times \mathrm{W} \Rightarrow 344.36 = 27 \times \mathrm{W} \Rightarrow \mathrm{W} = 12.75\mathrm{m}$$

Check: $\frac{L}{W} = \frac{27}{12.75} = 2.12\ O.K$

Check: $Weir\ Loading\ Rate\ (WLR) = \frac{Q}{Weir\ effluent\ length}$

Assume effluent length = W = 12.75 m

$$\mathrm{WLR} = \frac{\mathrm{Q}}{\mathrm{Effluent\ length}} = \frac{393 \frac{\mathrm{m^3}}{\mathrm{hr}} \times 24}{12.75\ \mathrm{m}} = 739.76\ \mathrm{m^3/m/day} > 250\ \mathrm{m^3/m/day}\ \ NOT\ O.K$$

$$Assume\ weir\ loading\ rate = 250\ \mathrm{m^3/m/day}\ = \frac{393 \times 24}{\mathrm{Length}} \Rightarrow \mathrm{Length} = 37.7 \approx 38\ \mathrm{m}$$

Check the scouring velocity:

$$\mathrm{Assume}\ \frac{\beta}{\mathrm{f}} = 2$$

$$\mathrm{v_h} = \sqrt{\frac{8\beta g(G_s-1)d}{f}} = \sqrt{8 \times 2 \times 9.81 \times (1.65 - 1)(0.03 \times 10^{-3})} = 0.055\frac{\mathrm{m}}{\mathrm{sec}} = 3.319\ \mathrm{m/min}$$

$$\therefore v_f = 0.15m/\min and\ V_s > v_f\ O.K$$

2) Total amount of suspended solids in the effluent:

$$200\frac{\mathrm{mg}}{\mathrm{L}} \times 393\frac{\mathrm{m^3}}{\mathrm{hr}} \times \frac{10^3}{10^6} \times 24 = 1886.4\ \mathrm{kg/day}$$

80% removal (sedimentation in the tank)

$$\therefore Settled\ solids = 0.8 \times 1886.4 = 1509\frac{kg}{day} = m_s$$

$$Sludge\ amount = Solids\ amount + Water\ amount$$

$$\frac{5}{100} = \frac{w_s}{w_s + w_w} = \frac{1509}{1509 + w_w} \Rightarrow w_w = 28671\ \mathrm{kg/day}$$

$$\mathrm{Volume\ of\ solids} = \frac{m_s}{\rho_s}\ \ \&\ \ \mathrm{Volume\ of\ water} = \frac{m_w}{\rho_w}$$

Assume $\rho_w = 998.2\frac{\mathrm{kg}}{\mathrm{m^3}}$ at $\mathrm{T} = 20^0\mathrm{C} \approx 1000\ \mathrm{kg/m^3}$

$$\mathrm{Volume\ of\ sludge} = \mathrm{Volume\ of\ solids} + \mathrm{Volume\ of\ water} = \frac{1509}{1.65 \times 10^3} + \frac{28671}{10^3} = 29.58\frac{\mathrm{m^3}}{\mathrm{day}}$$

$$\approx 30\frac{\mathrm{m^3}}{\mathrm{day}}$$

$$\Rightarrow To\ be\ used\ in\ size\ calculation\ and\ the\ number\ of\ times\ cleaning\ of\ the\ hopper$$

3) Settling velocity changeability with temperature or water mass density and viscosity and compare it with the design criteria:

Season	Temperature (^{0}C)	Water mass density $\rho_w\ (\frac{kg}{m^3})$	Water viscosity $\mu_w\ (\frac{kg}{m.sec})$
Winter	5	0.9999×10^3	1.519×10^{-3}

Summer	33	0.9957×10^3	0.7983×10^{-3}

So, for sedimentation tank design the best water temperature degree at winter temperature (5^{0}C), and then increase the settling velocity with increase temperature due to ($V_s > SOR$).

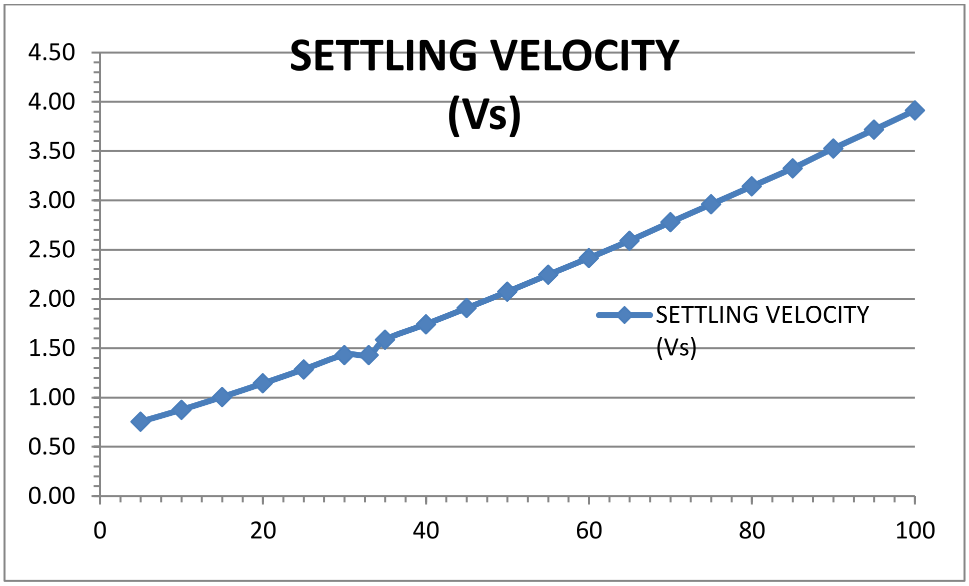


Fig. 1 Settling velocity variation with different temperature.

Problem 7.2: A settling tank if designed for an overflow rate of 4000-liter pr m^2 per hour. What percentage of particles of diameter (a) 0.05 mm (b) 0.02 mm, will be removed in the tank at 10^{0}C? Assume specific gravity of particles S.G = 2.65.

Solution:

$$Settling\ velocity\ V_s = SOR = \frac{Q}{A_s} = \frac{\frac{4000\times10^{-3}}{3600}}{1} = 1.111 \times 10^{-3}\,\frac{m}{sec} = 0.111\ cm/sec$$

(a) For 0.05 mm particles:

S.G = 2.65,

$$From\ table\ \text{at T} = 10^0\text{C}\ \rho_w = 0.9997 \times 10^3\,\frac{kg}{m^3}\ and\ \mu_w = 1.307 \times 10^{-3}\,\frac{kg}{m.sec}$$

$$V_s = \frac{g\rho_w(G_s-1)d_s^2}{18\mu_w} = \frac{9.81 \times 0.9997 \times 10^3 \times (2.65-1) \times \left(0.05 \times 10^{-3}\right)^2}{18 \times 1.307 \times 10^{-3}}$$
$$= 1.719 \times 10^{-3}\,\frac{m}{sec} = 0.1719\,\frac{cm}{sec} = 148.56\,\frac{m}{day} > SOR$$

$\% \, Settled = \frac{0.1719}{0.111} \times 100 = 154.86\% > 100\%$, Hence all the particles of 0.05 mm diameter will settle.

(b) For 0.02 mm particles:

$$V_s = \frac{g\rho_w (G_s - 1) d_s^2}{18\mu_w} = \frac{9.81 \times 0.9997 \times 10^3 \times (2.65 - 1) \times \left(0.02 \times 10^{-3}\right)^2}{18 \times 1.307 \times 10^{-3}}$$
$$= 2.751 \times 10^{-4} \frac{m}{sec} = 0.0275 \frac{cm}{sec} = 23.771 \frac{m}{day} < SOR$$

$\% \, Settled = \frac{0.0275}{0.111} \times 100 = 24.774\% < 100\%$, Hence the particles of 0.02 mm diameter will settle at 24.774%.

Problem 7.3: Find the diameter of the particles with specific gravity of 1.2 removed in a tank having a surface area of 250 m^2 and treating 8 million liters per day (MLD)? Assume temperature 26^{0}C and specific gravity of particles S.G = 2.65.

Solution:

$$Overflow\ rate = \frac{(8 \times 10^6) \times 10^{-3}}{250 \times 24} = 1.333\ m^3/m^2/hour$$

$$\therefore Settling\ velocity\ V_s = \frac{1.333 \times 100}{1 \times 3600} = 0.037\ cm/sec$$

Applying stoke's law:

S.G = 2.65,

$From\ table$ at $\mathrm{T} = 26^0\mathrm{C}$ $\rho_w = 0.99681 \times 10^3 \frac{kg}{m^3}$ and $\mu_w = 0.8746 \times 10^{-3} \frac{kg}{m.sec}$

$$V_s = \frac{g\rho_w (G_s - 1) d^2}{18\mu_w} = 3.702 \times 10^{-4} = \frac{9.81 \times 0.99681 \times 10^3 \times (2.65 - 1) \times d^2}{18 \times 0.8746 \times 10^{-3}} \Rrightarrow d$$
$$= 0.019\ mm$$

$$V_s = \frac{g\rho_w (G_s - 1) d^2}{18\mu_w} = 3.702 \times 10^{-4} = \frac{9.81 \times 0.99681 \times 10^3 \times (1.2 - 1) \times d^2}{18 \times 0.8746 \times 10^{-3}} \Rrightarrow d = 5.4 \times 10^{-5}$$
$$= 5.4 \times 10^{-2} mm$$

Sanitary and Environmental Engineering

PART 1: WATER SUPPLY ENGINEERING

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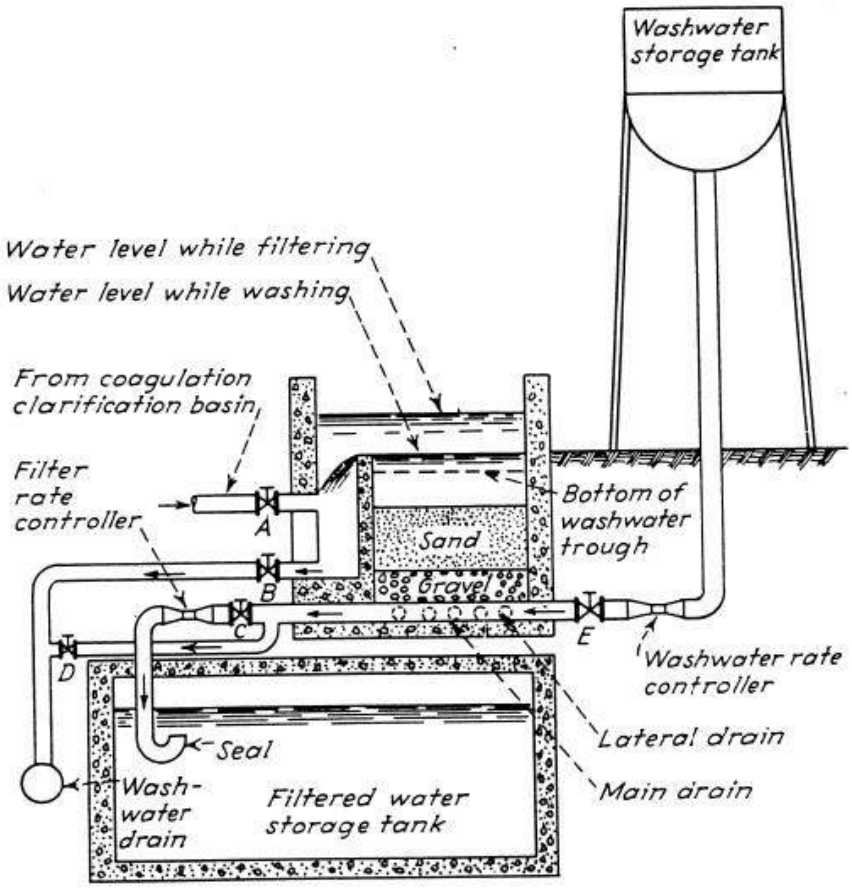
Lecture 9: Filtration

Filtration is the separation of non-settleable solids from water by passing it through a porous media. Filtration also removes from water: color, taste, odor, iron, manganese and microorganisms such as bacteria. In water treatment plants this is done in a unit known as the filter.

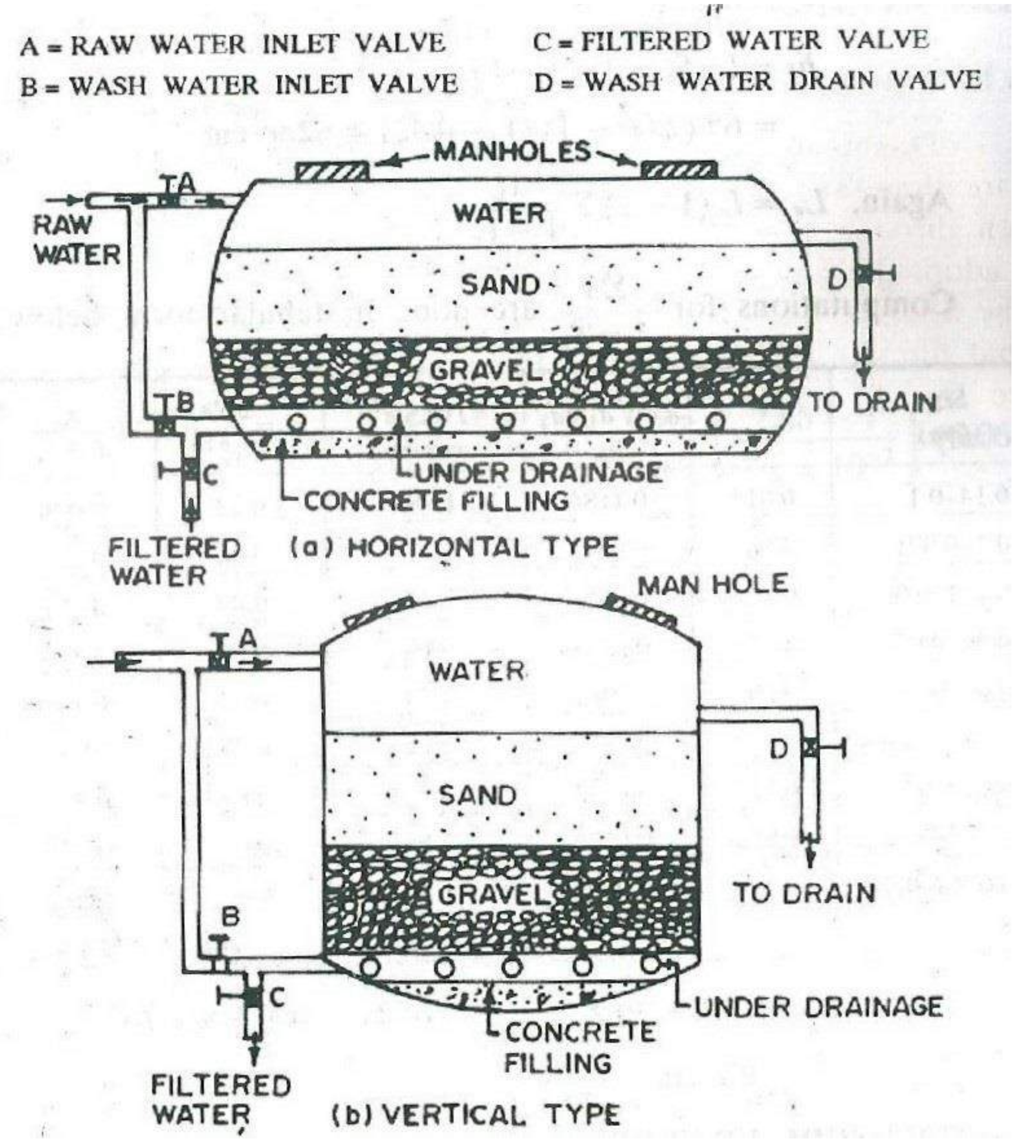
Types of filter according to process workability:

1) Gravity filter:
    a) Slow sand filter: They consist of fine sand, supported by gravel. They capture particles near the surface of the bed and are usually cleaned by scraping away the top layer of sand that contains the particles.
    b) Rapid-sand filter: They consist of larger sand grains supported by gravel and capture particles throughout the bed. They are cleaned by backwashing water through the bed to 'lift out' the particles.



Diagrammatic section of a rapid sand gravity filter.

2) Pressure filter:



Pressure filter.

Types of filter according to media:

1) Mono-media: as sand or anthracite.
2) Multimedia filters: They consist of two or more layers of different granular materials, with different densities. Usually, anthracite coal, sand, and gravel are used. The different layers combined may provide more versatile collection than a single sand layer. Because of the differences in densities, the layers stay neatly separated, even after backwashing.

Filter Media:

The choice of a filter medium is dictated by the durability required, the desired degree of purification, and the length of filter run and ease of backwash sought. The ideal medium should have such a size and be of such material that it will provide a satisfactory effluent, return a maximum quantity of solids, and be readily cleaned with a minimum of wash water.

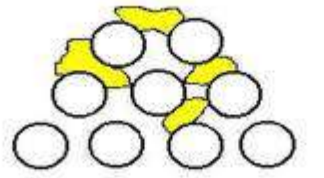
1) **Sand:** Sand, either fine or coarse, is generally used as filter media. The size of the sand is measured and expressed by the term called effective size. The effective size, i.e. D_{10} may be defined as the size of the sieve in mm through which ten percent of the sample of sand by weight will pass. The uniformity in size or degree of variations in sizes of particles is measured and expressed by the term called uniformity coefficient. The uniformity coefficient, i.e. ($D_U = \frac{D_{60}}{D_{10}}$) may be defined as the ratio of the sieve size in mm through which 60 percent of the sample of sand will pass, to the effective size of the sand.
2) **Gravel:** The layers of sand may be supported on gravel, which permits the filtered water to move freely to the underdrains, and allows the wash water to move uniformly upwards.
3) **Other materials:** Instead of using sand, sometimes, anthracite is used as filter media. Anthracite is made from anthracite, which is a type of coal-stone that burns without smoke or flames. It is cheaper and has been able to give a high rate of filtration.

Fine material	Coarse material
1) Better effluent 2) High head loss in upper layers of the bed. 3) Short filter runs. 4) Most difficult in cleaning upon backwash	1) Better utilization of the storage capacity of the filter. 2) Permit deeper penetration of the floc. 3) Longer filter runs. 4) Easier cleaning upon backwash

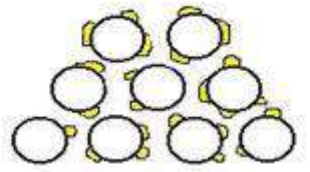
Sand	Anthracite	Gravel
1) Clean, hard, and resistance. 2) Not lose more than 5% by weight after being placed in 40% hydrochloric acid for 24 hr. 3) Sand depth 600-700 mm. 4) Effective size of 0.45-0.55 mm. 5) Uniformity coefficient not less than 1.2 and not exceed 1.7.	1) Anthracite depth 600-700 mm. 2) Effective size of 0.7 mm. 3) Uniformity coefficient of 1.75 or less.	1) Hard, rounded, durable, free from flat, thin, and contain no foreign material. 2) Effective size between 0.8 to 2 mm. 3) Uniformity coefficient of not over 1.7.

Filtration Mechanisms: There are four basic filtration mechanisms:

1) **Sedimentation:** The mechanism of sedimentation is due to force of gravity and the associate settling velocity of the particle, which causes it to cross the streamlines and reach the collector.
2) **Interception:** Interception of particles is common for large particles. If a large enough particle follows the streamline that lies very close to the media surface it will hit the media grain and be captured.
3) **Brownian diffusion:** Diffusion towards media granules occurs for very small particles, such as viruses. Particles move randomly about within the fluid, due to thermal gradients. This mechanism is only important for particles with diameters < 1 micron.
4) **Inertia:** Attachment by inertia occurs when larger particles move fast enough to travel off their streamlines and bump into media grains.



Mechanical Straining



Physical Adsorption

The area of filter medium which is a function of particle size and bed depth, it follows that increasing the size of the medium will require an increase in depth.

Rapid Sand Filters (RSF)

Rapid filtration generally implies a process which includes coagulation, flocculation, clarification, filtration, and disinfection.

Essential characteristics of a rapid filter:

1) The turbidity of water applied to the filters should not exceed 10 units and preferably 5 units,
2) High rate of filtration, 120 to 240 m/day,
3) Washing the filter units by revising flow of filtered water upward through the filter to remove mud and other impurities which have lodged.

Filter component: As shown in the figures above:

A) Water head: Clearance, 600-900 mm in depth.
B) Filer bed (Sand layer), 600- 1200 mm in depth. The objectives of this layer:

    i. Coarse to retain large quantity of solid particles.
    ii. Fine to prevent the passage of small solid particles.
    iii. Deep layer to allow long filtration run.
    iv. Graded to permit good backwash cleaning.
    Properties: Effective size (D_{10})= 0.45 -0.55 mm, Uniformity coefficient (C_u)= D_{60}/D_{10} =1.2-1.7.

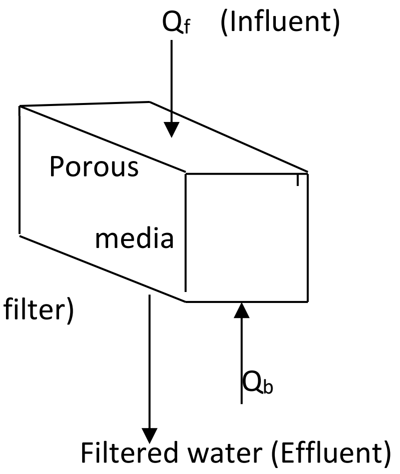
C) Gravel layer (bed), 400-600 mm in depth. The objectives of this layer:

    i. Support the filter bed (sand layer).
    ii. Permit the filtered water to move freely towards the underdrain system.
    iii. Allows the washing water to move uniformly upwards to the filter bed.
    Properties: clean, free from flat and thin shapes, hard, round, durable and ρ_s = 1600 kg/m^3

D) Underdrain system: The objectives of this layer:

    i. To collect the filtered water.
    ii. To distribute the washing water evenly.

Mathematical equation of filtration process:



Q_f = Filtration discharge (flow through one filter)

$$Q_f = A_s \times V_f$$

A_s =surface area of the filter.

V_f = Filtration rate (m/day).

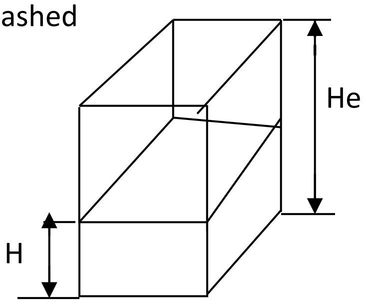
Q_b= Backwash discharge (Washing water for one filter)

$$Q_b = A_s \times v_b$$

v_b = Backwash rate

$$Q_b = \frac{Volume\ of\ washing\ water}{Time\ required\ for\ washing} = \frac{Q}{V}$$

* An expansion space for the filter media to move when backwashed



$$\frac{H_e}{H} = \frac{(1-n)}{(1-n_e)}$$

H = Height of the filter bed.

H_e = Height of the expanded filter bed when backwashed

n = Porosity of the clean filter bed.

n_e = Porosity of the expanded filter bed, $\left(n_e = \left(\frac{v_b}{v_s}\right)^{0.22}\right)$

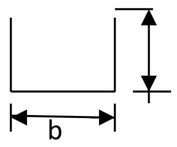
v_b = Backwash velocity rate.

v_s= Settling velocity of the filter media (Stoke 's settling velocity)

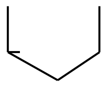
E) Trough: The objectives of this lateral channel to collect the backwash water.

Types of the troughs

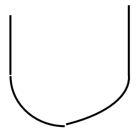
Flat bottom



V- bottom



Rounded



$$y = 1.73 \times \sqrt[3]{\frac{q_t^2}{g \times b^2}}$$

y = Height of the trough (m).

b = Width of the trough (m).

g = Acceleration of gravity (m/ sec^2)

q_t = Backwash discharge reaching one trough (m^3 /sec), $\left(q_t = \frac{Q_b}{No.of\ troughs}\right)$

F) Gullet: The objectives of this main channel to collect the backwash water from the troughs.

Design Criteria

1) Filtration rate (v_f) = 120 -360 m/day.
2) Backwash rate (v_b) = 0.15 -0.9 m/min (216-1296 m/day).
3) Surface area (A_s) = 5 – 200 m^2
4) Washing time = 5 -15 min.
5) Horizontal distance for backwash water to reach the trough is not to exceed one meter.
6) Filtration run = 12-72 hr (if the turbidity of the influent is: a. Low–long run, b. High–short run).
7) Total head loss = 2.7 – 3.7 m

Operation Problems In RSFs

1) **Air Binding:**
    a) When the filter is newly commissioned, the loss of head of water percolating through the filter is generally very small. However, the loss of head goes on increasing as more and more impurities get trapped into it.
    b) A stage is finally reached when the frictional resistance offered by the filter media exceeds the static head of water above the bed. Most of this resistance is offered by the top 10 to 15 cm sand layer. The bottom sand acts like a vacuum, and water is sucked through the filter media rather than getting filtered through it.
    c) The negative pressure so developed, tends to release the dissolved air and other gases present in water. The formation of bubbles takes place which stick to the sand grains. This phenomenon is known as Air Binding as the air binds the filter and stops its functioning.
    d) To avoid such troubles, the filters are cleaned as soon as the head loss exceeds the optimum allowable value.

2) **Mud accumulation:** The mud from the atmosphere usually accumulates on the sand surface to form a dense mat. During inadequate washing this mud may sink down into the sand bed and stick to the sand grains and other arrested impurities, thereby forming mud balls.

3) **Sand Incrustation (cracking of filter):** to solve this problem a-Skimming b-Use anthracite as a top layer on the filter Media-Carbonation. The fine sand contained in the top layers of the filter bed shrinks and causes the development of shrinkage cracks in the sand bed. With the use of filter, the loss of head and, therefore, pressure on the sand bed goes on increasing, which further goes on widening these cracks.

Remedial measures to prevent cracking of filters and formation of mud balls by:

a) Breaking the top fine mud layer with rakes and washing off the particles.
b) Washing the filter with a solution of caustic soda.
c) Removing, cleaning and replacing the damaged filter sand.

Backwashing of Rapid Sand Filter

1) For a filter to operate efficiently, it must be cleaned before the next filter run. If the water applied to a filter is of very good quality, the filter runs can be very long. Some filters can operate longer than one week before needing to be backwashed. However, this is not recommended as long filter runs can cause the filter media to pack down so that it is difficult to expand the bed during the backwash.
2) Treated water from storage is used for the backwash cycle. This treated water is generally taken from elevated storage tanks or pumped in from the clear well.

3) The filter backwash rate has to be great enough to expand and agitate the filter media and suspend the floc in the water for removal. However, if the filter backwash rate is too high, media will be washed from the filter into the troughs and out of the filter.

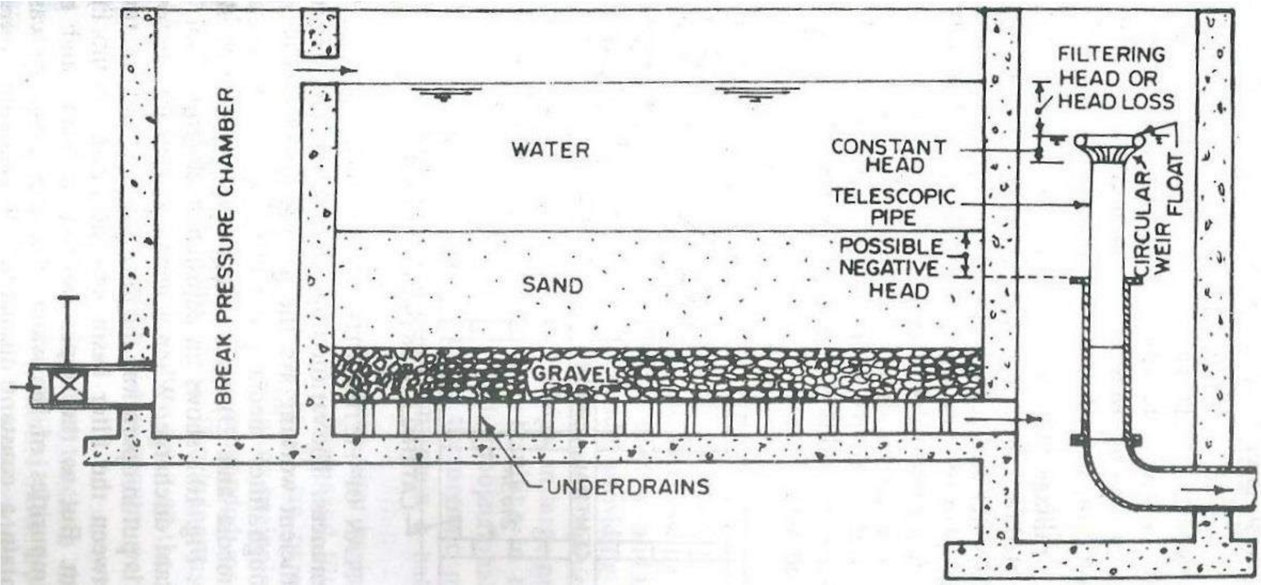
When is Backwashing Needed?

The filter should be backwashed when the following conditions have been met:

1) The head loss is so high that the filter no longer produces water at the desired rate; and/or
2) Floc starts to break through the filter and the turbidity in the filter effluent increases; and/or
3) A filter run reaches a given hour of operation.

Principles of Slow Sand Filtration

1) In slow sand filter impurities in the water are removed by a combination of processes: sedimentation, straining, adsorption, and chemical and bacteriological action.
2) During the first few days, water is purified mainly by mechanical and physical-chemical processes. The resulting accumulation of sediment and organic matter forms a thin layer on the sand surface, which remains permeable and retains particles even smaller than the spaces between the sand grains.
3) As this layer develops, it becomes living quarters of vast numbers of microorganisms which break down organic material retained from the water, converting it into water, carbon dioxide and other oxides.
4) Most impurities, including bacteria and viruses, are removed from the raw water as it passes through the filter skin and the layer of filter bed sand just below. The purification mechanisms extend from the filter skin to approx. 0.3-0.4 m below the surface of the filter bed, gradually decreasing in activity at lower levels as the water becomes purified and contains less organic material.
5) When the microorganisms become well established, the filter will work efficiently and produce high quality effluent which is virtually free of disease carrying organisms and biodegradable organic matter. They are suitable for treating waters with low colors, low turbidities and low bacterial contents.



Slow sand filter.

Sand Filters vs. Rapid Sand Filters

1) **Base material:** In SSF it varies from 3 to 65 mm in size and 30 to 75 cm in depth while in RSF it varies from 3 to 40 mm in size and its depth is slightly more, i.e. about 60 to 90 cm.
2) **Filter sand:** In SSF the effective size ranges between 0.2 to 0.4 mm and uniformity coefficient between 1.8 to 2.5 or 3.0. In RSF the effective size ranges between 0.35 to 0.55 and uniformity coefficient between 1.2 to 1.8.
3) **Rate of filtration:** In SSF it is small, such as 100 to 200 L/h/sq.m. of filter area while in RSF it is large, such as 3000 to 6000 L/h/sq.m. of filter area.
4) **Flexibility:** SSF are not flexible for meeting variation in demand whereas RSF are quite flexible for meeting reasonable variations in demand.
5) **Post treatment required:** Almost pure water is obtained from SSF. However, water may be disinfected slightly to make it completely safe. Disinfection is a must after RSF.
6) **Method of cleaning:** Scrapping and removing of the top 1.5 to 3 cm thick layer is done to clean SSF. To clean RSF, sand is agitated and backwashed with or without compressed air.
7) **Loss of head:** In case of SSF approx. 10 cm is the initial loss, and 0.8 to 1.2m is the final limit when cleaning is required. For RSF 0.3 m is the initial loss, and 2.5 to 3.5 m is the final limit when cleaning is required.

Item	SSF	RSF
Base material	Particle Size: 3-65 mm Depth: 20-75 cm	Particle Size: 30-40 mm Depth: 60-90 cm
Filter sand	D_{10}: 0.2-0.4 mm C_U: 1.8-3	D_{10}: 0.35-0.55 mm C_U: 1.2-1.8
Filtration rate	2.4-4.8 m/day	120-360 m/day
Flexibility	not flexible for meeting variation in demand	flexible for meeting reasonable variations in demand
Post treatment required	water may be disinfected slightly to make it completely safe	Disinfection is a must after RSF
Method of cleaning	Scrapping and removing of the top 1.5 to 3 cm thick layer is done to clean SSF	To clean RSF, sand is agitated and backwashed with or without compressed air
Loss of head	10 cm is the initial loss, and 0.8 to 1.2m is the final limit when cleaning is required	0.3m is the initial loss, and 2.5 to 3.5m is the final limit when cleaning is required

SOLVED PROBLEMS

Problem 9.1: A WTP is designed to treat 48,000 m^3/day. For filtration 12 RSFs are used, each filter is designed with 2 troughs of a square cross-sectional area. The filtration rate is 160 m/day and the backwash rate is 864 m/day. Find

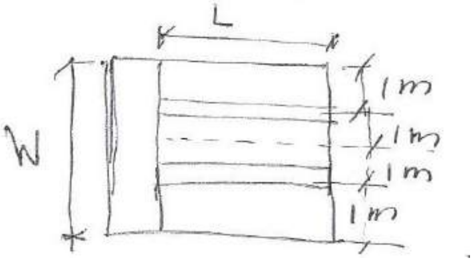
1) Dimensions of each filter, L and W (m) with or without washing? Also filter depth?
2) Dimensions of each trough, y and b (m) and gullet?
3) Volume of water (m^3) to wash 2 filters at the same time for 5 minutes?

Solution:

1) $\because\ n_b = Zero\ (All\ filters\ in\ operation\ i.e\ no\ washing)$

$$Q_f = \frac{Q_{Total}}{n-n_b} = \frac{48\times10^3}{12} = 4000\ m^3/day$$

$$Q_f = v_f \times A_s \Rrightarrow A_s = \frac{4000\,\frac{m^3}{day}}{160\,\frac{m}{day}} = 25\ m^2 = L \times W$$



For two troughs, W=4 m,

Assuming maximum horizontal flow for backwashing water to reach the trough = 1 m

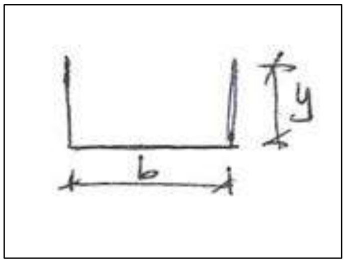
$$\therefore L = \frac{25}{4} = 6.25\ m$$

Check: total filter depth = Freeboard (0.5m) + Water head (1m) + Sand layer (1m) + Gravel layer (0.6m) + Underdrain (0.4m) = 3.5 m

So, use filter dimension (L = 6.25 m, W = 4 m, and H = 3.5 m)

2) Trough dimension:

$$q_t = \frac{Q_b}{No.of\ troughs} = \frac{A_s\times v_b}{2} = \frac{25\ m^2\times\frac{864}{24\times3600}}{2} = 0.125\ m^3/sec$$



Assume y = b (Square cross section of the trough)

$$y = 1.73 \times \sqrt[3]{\frac{q_t^2}{g\times b^2}} = 1.73 \times \sqrt[3]{\frac{q_t^2}{g\times y^2}} \Rrightarrow y^5 = (1.73)^3 \times \frac{q_t^2}{g} = (1.73)^3 \times \frac{0125^2}{9.81} \Rrightarrow y = 0.38\ m = b$$

Gullet dimension:

$$q_G = \frac{Q_b}{No.of\ Gullet} = \frac{A_s \times v_b}{1} = \frac{25\,m^2 \times \frac{864}{24 \times 3600}}{1} = 0.25\,m^3/sec$$

Assume y = b (Square cross section of the gullet)

$$y = 1.73 \times \sqrt[3]{\frac{q_G^2}{g \times b^2}} = 1.73 \times \sqrt[3]{\frac{q_G^2}{g \times y^2}} \Rightarrow y^5 = (1.73)^3 \times \frac{q_D^2}{g} = (1.73)^3 \times \frac{025^2}{9.81} \Rightarrow y = 0.505\,m = b$$

3) $Water\ volume\ for\ washing = Q_b \times t \times n_b = 864\frac{m}{day} \times 25\,m^2 \times \frac{5\,min}{60\times24} \times 2 = 150\,m^3\ (Washing\ water\ for\ two\ filters)$

$$Volume\ for\ one\ filter = \frac{150}{2} = 75\,m^3$$

Note: The ratio of water washing amount for one filter does not exceed 7% of the water filtration amount.

$$Volume\ of\ filtrated\ water = 4000\frac{m^3}{day} \times 1 = 4000\,m^3$$

$$Ratio\ for\ one\ back\ washing = \frac{75}{4000} \times 100 = 1.875\% < 7\%\ \ O.K$$

Problem 9.2: A city has a population of 100,000 capita with an average rate of demand of 160 L/cap.day. Find the area of rapid sand filter? Assume an average filtration rate of 120 m^3/m^2.day of filter area, the filter length equal 1.5 width and filter area not exceed 40 m^2.

Solution:

$$Maximum\ daily\ demand = 1.8 \times 160 \times 10^{-3} \times 100000 = 28800\,m^3/day$$

$$Area\ of\ filters = \frac{28800}{120} = 240\,m^2\ \ No.of\ filters = \frac{240}{40} = 6\,filters$$

$$A_s = 40 = L \times W = 1.5W^2 \Longrightarrow W \approx 5.20\,m\ \&\ L = 7.8\,m$$

Problem 9.3: A flat bottom trough is to receive the wash water from a section of the filter which is 2 m wide and 3 m long. The wash water rate is 700 liters per min per m^2. If the water is to have a depth of 25 cm at the upper end of the trough, what should be the dimensions of the trough?

Solution:

$$Q_b = v_b \times A_s = 700 \times 2 \times 3 = 4200 \frac{L}{min} = 4.2 \frac{m^3}{min} = 0.07\ m^3/sec$$

$$y = 1.73 \times \sqrt[3]{\frac{q_t^2}{g \times b^2}} \Longrightarrow 0.25 = 1.73 \times \sqrt[3]{\frac{0.07^2}{9.81 \times b^2}} \Longrightarrow b = 0.406\ m$$

Assume freeboard = 5 cm, the depth of trough will be = 25 + 5 = 30 cm

Sanitary and Environmental Engineering

PART 1: WATER SUPPLY ENGINEERING

PART 1: WATER SUPPLY ENGINEERING

Lecture 10: Disinfection

The filtered water may normally contain some harmful disease producing bacteria in it. These bacteria must be killed in order to make the water safe for drinking. The process of killing these bacteria is known as Disinfection or Sterilization, or, is the killing of microorganisms causing diseases.

Methods of disinfection

1. Treatment with chlorine (Chlorination).
2. Treatment with Ozone (Ozonation).
3. Treatment with Iodine and Bromine.
4. Treatment with Ultraviolet irradiation.
5. Extreme values of pH.
6. Treatment with ultrasonic waves.
7. Treatment with oxidizing agents (Potassium permanganate).
8. Boiling of water.
9. Chlorine dioxide.

Chlorination:

Chlorination is a cheap and reliable process, not difficult in handling. Chlorine is a disinfect that destroys the enzymes of the bacterial cells and enters to attack the intracellular system. Chlorine may remove colour and iron also the destruction of taste and odour. The efficiency of chlorine is reduced by increasing pH and at low water temperature (less than 24 ^{0}C).

1) **Chlorine in water:** Chlorine and water react:

$$Cl_2 + H_2O \longrightarrow HOCl + H^+ + Cl^- \quad \text{(HOCl hypochlorous acid)}$$

$$HOCl \longrightarrow H^+ + OCl^- \quad (OCl^- \text{ hypochlorite ion})$$

HOCl and OCl accomplish the disinfection process. Since the concentration of these components is a function of the hydrogen ion concentration, so the efficiency of disinfection is affected by pH. Chlorine and its products are most effective at low pH. Under pH = 3, molecular chlorine maybe present. Chlorine existing in water as HOCl, OCl and molecular chlorine is defined as **free available chlorine**.

Free available chlorine will react with nitrogenous compounds like ammonia to form chloramines. The reaction of ammonia with HOCl is shown below:

$NH_3 + HOCl \longrightarrow H_2O + NH_2Cl$ (monochloramine) at pH over 7.5

$NH_2Cl + HOCl \longrightarrow H_2O + NHCl_2$ (dichloramine) at pH 5 – 6.5

$NHCl_2 + HOCl \longrightarrow H_2O + NCl_3$ (nitrogen trichloride) at pH 4.4

Chlorine in water reacting with ammonia, nitrogenous compounds and organic matter is defined as **combined available chlorine.** The above compounds are less active than free available chlorine as disinfecting agents, but may maintain residuals for longer time than free chlorine.

2) **Chlorine Demand =** Chlorine added – Residual chlorine

It is the difference between the mount of chlorine added and the mount present as residual either **Free or Combined** after a contact time.

Chlorine demand to kill microorganism = C X t

Where: C = Concentration of the disinfect (chlorine) and t = Contact time between the microorganism and the disinfect (chlorine).

Residual chlorine is the remaining chlorine after disinfection in water. This may be necessary in the distribution system and storage tanks to protect water in the pipes from any pollution.

3) **What is used in chlorination?**
   - **a)** Chlorine gas, chlorine is obtained in pressurized cylinders ranging in weight from 45 to 1000 kg.
   - b) Hypochlorination, chemical compounds such as Sodium hypochlorite NaOCl (laundry bleach) and Calcium Hypochlorite $Ca(OCl)_2 \cdot 4H_2O$.

Hypochlorites are useful in disinfecting waters of reservoirs and swimming pools also in controlling algae growth. Hypochlorites in water will give free available chlorine:

$Ca(OCl) \longrightarrow Ca^{+2} + 2OCl^-$

$NaOCl \longrightarrow Na^+ + OCl^-$

$OCl^- + H^+ \longrightarrow HOCl$

4) **Chlorination is classified according to its point application as:**
   - a) **Plain Chlorination** is used with no other treatment. Chlorine is added to the water from the source. The dose added depends on the degree of contamination. About 0.5 mg/L or more is used for disinfection and obtain residual chlorine free or combined.
   - b) **Pre-chlorination**, applying chlorine before any other treatment. It may be added to the suction pipe of the raw water pumps or in the flash mixer. Its advantages:
     - i. Improve the coagulation process.
     - ii. Reduce taste and odor caused by organic sludge in sedimentation tanks.
     - iii. Killing algae and other microorganisms, so filters may remain clean to obtain long filtration runs.

     A dose of 5 – 10 mg/L is used to obtain 0.1 – 0.5 mg/L residual chlorine free or combined.

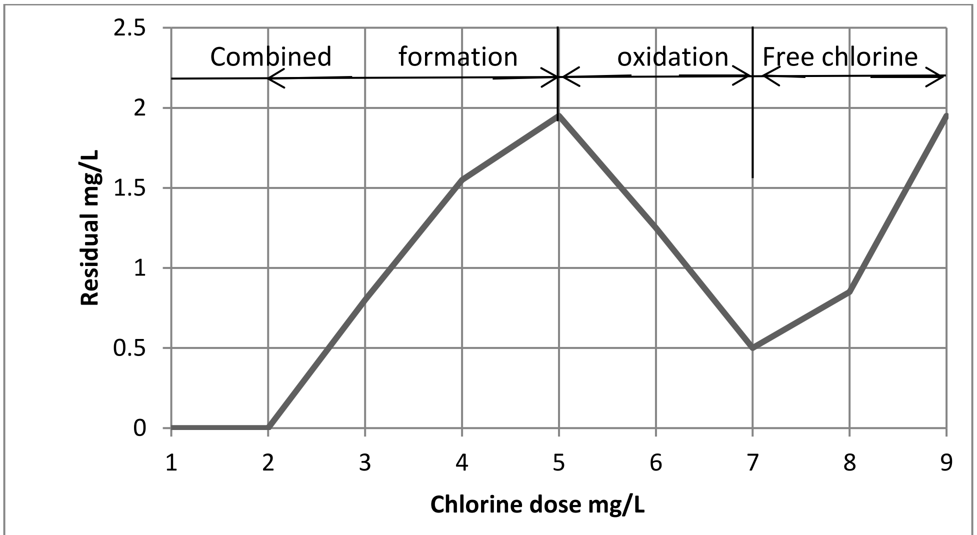
   - c) **Post-chlorination**, applying chlorine at the end of the treatment process. A contact time for about 30 minutes is required for disinfection when adding 0.25 – 0.5 mg/L chlorine dose to obtain 0.1 – 0.2 mg/L residual chlorine. In Iraq a dose of 1 -2 mg/L is added to obtain 0.1 – 0.5 mg/L residual.

The combination of pre-and post-chlorination may be needed if the raw water is highly polluted. Also in some cases chlorine is injected into the distribution system to maintain the desired residual.

d) **Breakpoint chlorination**, chlorine is consumed in oxidizing many compounds present in water. No chlorine can be measured until the initial chlorine demand is satisfied. Then chlorine reacts and may form combined chlorine residual (some are odorous and undesirable like chlorophenols). These combines increase with the addition of chlorine dose until a maximum combined residual is reached. Further addition of chlorine will cause a decrease in the combined residual. This is the Breakpoint chlorination, at this point the combined are oxidized to oxides. After this point, free chlorine residual is present. A chlorine dose of 7 – 10 mg/L in order to obtain free chlorine about 0.5 mg/L.

The breakpoint indicates complete oxidation of the chloramines and other chlorine combinations, and the residual above the breakpoint is mostly free available chlorine.

e) **Super chlorination/Dechlorination,** is adding a high chlorine dose for quick disinfection, this will produce high residues, so dechlorination should be applied afterward. For dechlorination: aeration, adding chemicals, using activated carbon (adsorbent) may be applied.



Break point Chlorination

Ozone: It's manufactured by electrical discharge into cooled dried air. Approximately 1% of the atmospheric oxygen is converted to O_3 at an energy consumption of 0.025 kWh per gram O_3. The mixture of air and ozone is transferred into water either by bubbling it through the bulk solution or by permitting droplets of water to fall through a rising column of gas.

Advantage	Disadvantage
1) It is effective as oxidizing agent and germicide, in destruction of organic matter which might produce tastes and odors, and in oxidation or iron and manganese. 2) More rapid in action than other oxidizing agents with small dose (0.25-1.5) mg/L at low contact time (45 sec-2 min). 3) It is not particularly sensitive to pH within range of pH (5-8), but is significantly affected by temperature.	1) It's restricted of use. 2) On-site generation 3) Its cost relative to chlorine. 4) It is not providing residual disinfection in water distribution system.

Chlorine dioxide: It is produced by the chlorination of sodium chlorite in a ratio of 1 mole chlorine to 2 moles chlorite, and the result compound (ClO_2).

Advantage	Disadvantage
1) It is effective as oxidizing agent and germicide, in destruction of organic matter which might produce tastes and odors, and in oxidation or iron and manganese. 2) It does not react with ammonia. 3) More rapid in action and appears to be able to oxidize organics without formation of halogenated hydrocarbons. 4) It is unaffected by pH within the normal range encountered in water. 5) It is relatively persistent at ordinary temperatures and can provide residual protection.	1) It's restricted of use. 2) On-site generation 3) Its cost relative to chlorine.

SOLVED PROBLEMS

Problem 10.1: Chlorine usage in the treatment of 20 MLD of water is 8 kg/day. The residual after 10-minute contact is 0.2 mg/L. Compute the dosage in milligrams per liter and chlorine demand of water? Design the two chlorine contact tank each L=4W, effective depth = 2 m, detention time = 30 m? Also, calculate the total numbers of paths and baffles if clear width of path not more than 2 m?

Solution:

$$Dosage = \frac{8 \times 10^6 mg/day}{20 \times 10^6\ L/day} = 0.4\ mg/L$$

$$Chlorine\ demand = 0.4 - 0.2 = 0.2\ mg/L$$

$$V = \frac{20000}{2 \times 24 \times 60} \times 30 = 104.15\ m^3 \rightarrow A = \frac{208.3}{2} = 104.15\ m^2 \rightarrow W = \sqrt{\frac{104.15}{4}} = 5.1\ m\ \&\ L = 20.4\ m$$

So, the chlorine contact tank dimensions (20.4 x 5.1 x 2.5) m

$$NO. of\ paths = \frac{5.1}{3} \& NO. of\ baffel = 3 - 1 = 2$$

Problem 10.2: Calculate the quantity of bleaching powder required per day for disinfecting 4 MLD. The dose of chlorine has to be 0.5 ppm and the bleaching powder contains 30% of available chlorine?

Solution:

$$Chlorine\ required = \frac{0.5\frac{mg}{L} \times 4 \times 10^6 L/day}{10^6} = 2\ kg/day$$

$$Amount\ of\ bleaching\ powder = \frac{2 \times 100}{30} = 6.67\ kg/day$$

Problem 10.3: Chlorine usage in the treatment of 25000 m^3/day is 9 kg/day. The residual chlorine after 10 minutes' contact is 0.2 mg/L. Calculate the dosage in milligrams per liter and chlorine demand of the water?

Solution:

$$Water\ treated\ per\ day = 25 \times 10^6\ L/day$$

$$Chlorine\ consumed\ per\ day = 9 \times 10^6\ mg/day$$

$$Chlorine\ used\ per\ liter\ of\ water = \frac{9 \times 10^6}{25 \times 10^6\ L/day} = 0.36\ mg/L$$

$$Chlorine\ demand = 0.36 - 0.2 = 0.16\ mg/L$$

Problem 10.4: A new water main is disinfected using a 50 mg/L chlorine dosage by applying a 2 percent hypochlorite solution. (a) How many kilograms of dry hypochlorite powder, containing 70 percent available chlorine, must be dissolved in 100 liters of water to make a 2 percent (20000 mg/L) solution? (b) At what rate should this solution be applied to the water entering the main (400 L of water volume) to provide a concentration of 50 mg/L? (c) If 34000 liters of water are used to fill the main at a dosage of 50 mg/L, how many liters of hypochlorite solution are used?

Solution:

a) $Kilogram\ of\ hypochlorite\ powder\ for\ 2\%\ solution = \frac{100\ L \times 1\frac{kg}{L} \times 0.02}{0.7} = 2.86\frac{kg}{100\ L}$

b) $Feed\ rate\ for\ 50\frac{mg}{L} = \frac{50\frac{mg}{L}}{20000\frac{mg}{L}} = \frac{1\ Volume\ of\ 2\%\ solution}{400\ Volume\ of\ water} = 2.5 \times 10^{-3}$

c) $Solution\ usage = 34000\ L \times 2.5 \times 10^{-3} = 85\ L$

Problem 10.8: Results of chlorine demand test on a raw water are given below. Determine the break-point dosage and the chlorine demand?

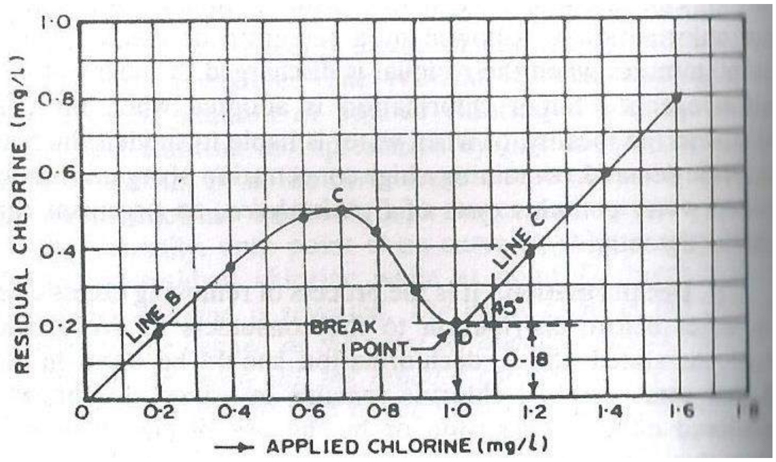
Sample No.	Chlorine dosage mg/L	Residual chlorine after 10 min. mg/L
1	0.2	0.18
2	0.4	0.34
3	0.6	0.48
4	0.8	0.46
5	0.9	0.27
6	1	0.18
7	1.2	0.38
8	1.4	0.58
9	1.6	0.78

Solution:

Sample No.	Chlorine dosage mg/L	Residual chlorine after 10 min. mg/L	Chlorine demand mg/L
1	0.2	0.18	0.02
2	0.4	0.34	0.06
3	0.6	0.48	0.12
4	0.8	0.46	0.34
5	0.9	0.27	0.63
6	1	0.18	0.82

7	1.2	0.38	0.82
8	1.4	0.58	0.82
9	1.6	0.78	0.82

Figure bellows the curve plotted the basis of the above data.



From the curve, we find that break point occurs at point D, at which the applied chlorine = 1 mg/L.

So, the break point dosage = 1.0 mg/L

Chlorine demand at break point = 1.0 -0.18 = 0.82 mg/L

It is observed that since the slope of curve C is 45^0, the chlorine demand (=0.82 mg/L) remains constant after break point, since all additional chlorine added after point D appears as free chlorine.

Even from the data, at a dose of 1.2 mg/L, the residual chlorine = 0.38 mg/L.

Hence, residual chlorine = 1.2 – 0.38 = 0.82 mg/L.

Sanitary and Environmental Engineering

PART 1: WATER SUPPLY ENGINEERING

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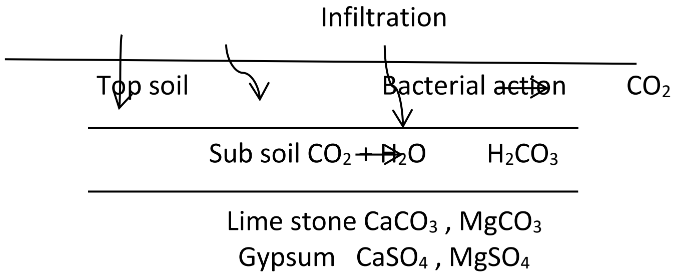
Lecture 11: Special Treatments: Hardness Removal or Water Softening

Water hardness is a measure of the amount of calcium and magnesium salts in water. Calcium and magnesium enter water mainly through the weathering of rocks. The more calcium and magnesium in water, the harder the water. Water hardness is usually expressed in milligrams per liter (mg/L) of dissolved calcium and magnesium carbonate.

The formation of hardness in natural water



$CaCO_3 + H_2CO_3 \longrightarrow Ca(HCO_3)_2$

$MgCO_3 + H_2CO_3 \longrightarrow Mg(HCO_3)_2$

Insoluble Soluble in water

Water with high content of hardness:

1) Leaves a deposit known as the scale when water is heated.
2) Does not lather with soap; this is due to the reaction of Ca^{+2} and Mg^{+2} with soap:

$Ca^{+2} + 2(soap)^{-} \longrightarrow Ca\,(soap)_2 \downarrow$ scum (complex solid)

The soap will not react with the dirt but will form with hardness undesirable precipitates (scum).

So, hardness is defined as the total amount of polyvalent cations mainly Ca^{+2} and Mg^{+2} found in water. Fe, Mn, Al and Zn may cause hardness to a lesser degree (low concentrations in nature). Hence hardness is the total concentration of Ca^{+2} and Mg^{+2} in water expressed in mg/L as $CaCO_3$.

Water is classified according to concentration of hardness:

Hardness in mg/L as $CaCO_3$	**Type of water**
0-75	Soft
75-100	Moderately hard
100-300	Hard
More than 300	Very hard

Hard water: The water containing sulfates, chlorides and bicarbonate of calcium and magnesium dissolved in it. These materials react with soap, causing a precipitation which appears as a scum or curd on the water surface.

Type of hardness:

a) Temporary or carbonate hardness caused by carbonates and bicarbonates of calcium and magnesium: $Ca(HCO_3)_2$ &; $Mg(HCO_3)_2$ (Boiling the water or by adding lime);
b) Permanent or noncarbonated hardness caused by sulfate ($CaSO_4$ & $MgSO_4$) and chloride ($CaCl_2$ & $MgCl_2$). The chlorides of magnesium and calcium are very corrosive to steam boilers and quickly cause pitting and grooving of boiler tubes while the sulfates of calcium and magnesium also cause scale in boilers. In steam boilers the carbonate and bicarbonate will release carbon dioxide and cause corrosion of the tube sodium sulfate may cause foaming in boilers if present in large amounts, (Caustic soda or soda ash).

Softening Processes:
1) Heating the water.
2) Lime-Soda method: Lime CaO, Soda ash Na_2CO_3.
3) Caustic Soda method: NaOH.
4) Ion Exchange of Zeolite method.

The advantage of water softening: Reduction of soap consumption, lowered cost in maintains plumbing fixtures, and improved taste of cooked food.

Ion Exchange method:

It means the reversible interchange of ions between a solid phase and a liquid phase. For water softening, water (liquid) is passed through a filter (solid) containing the ion exchange material. This material could be a natural material known as zeolite (green sand) Na_2O: Al_2O_3: SiO_2 (Na_2X), or synthetic material could be known as resin which is higher in the exchange capacity.

Zeolites are complex of aluminum, silica and soda, some forms of which are synthetic and others are naturally occurring.

The Filtration Process:

$$\underset{\text{Hard water}}{\begin{array}{c} Ca \\ Mg \\ \end{array}\left[\begin{array}{l} HCO_3 \\ SO_4 \\ Cl \end{array}\right.} + \underset{\text{Filter bed}}{Na_2X} \longrightarrow \underset{\text{Exhausted filter}}{\left.\begin{array}{c} Ca \\ Mg \end{array}\right] X} + \underset{\text{Soft water}}{2\,Na \left[\begin{array}{l} HCO_3 \\ SO_4 \\ Cl \end{array}\right.}$$

By this process Ca and Mg (hardness) will be removed from water and replaced by an equivalent amount of Na. When the ion exchange material or filter is saturated with the hardness ions (exhausted from its ions), then the hardness will pass through the filter with no change. This is called as **the Break through point**. At this time the filter needs to be regenerated or to remove the hardness from the filter. This process is to wash the filter with brine water containing Na.

The Regeneration Process:

$$\underset{\text{Exhausted filter}}{\left.\begin{array}{c} Ca \\ Mg \end{array}\right] X} \quad \underset{\text{Brine water}}{+2\,NaCl} \longrightarrow \underset{\text{New Filter}}{Na_2X} + \underset{\text{Industrial Wastewater Treatment}}{\left.\begin{array}{c} Ca \\ Mg \end{array}\right] Cl_2}$$

Advantages of the zeolite method:
a) A compacted unit easy to operate.
b) No sludge is formed.
c) The product is zero hardness water which is used in industry (textile) or in boilers.
d) May remove iron (Fe) from water.

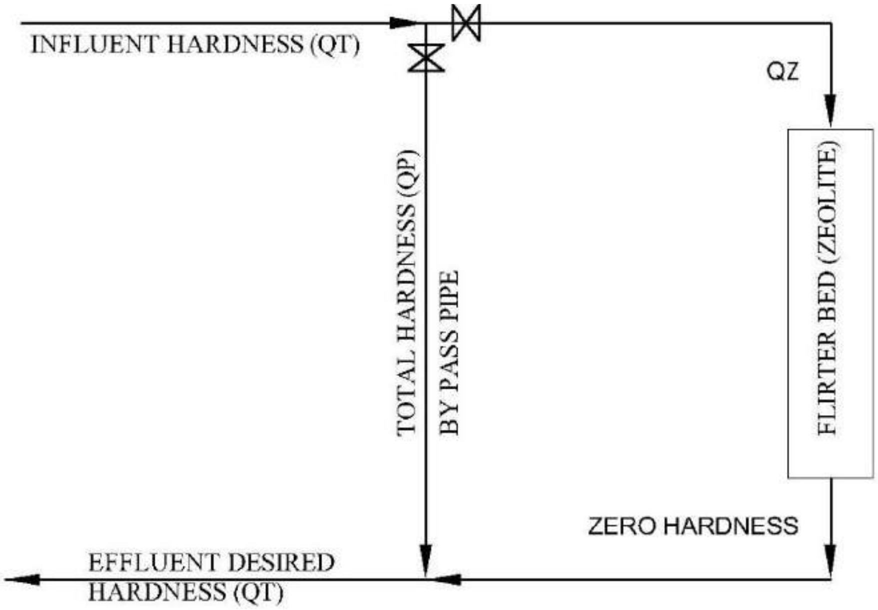
The ion exchange capacity of a filter is expressed in:

a) Kilo grain of hardness to be removed per m^3 volume of the filter, kilo grain / m^3.
b) Equivalent hardness/m^3, eq/m^3.
c) Kilo gram, kg/m^3.

Where, kilo grain = equivalent hardness (kg = 2.2 lb and lb = 7 kilo grain).

Design criteria

1) Height of the filter: 0.5-2.0 m.
2) Filtration rate: 120 -240 m/day
3) Salt for regeneration: 125-150 gm/eq. hardness to be removed.
4) Brine solution: 5-10%
5) Washing time: 5-15 min
6) Capacity of zeolite: 135-250 eq hardness/m^3 or 9-20 kg/m^3.
7) Capacity of Resin: 275-550 eq hardness/m^3 or 20-36 kg/m^3



$$Q_T \times Desired\ hardness = Q_P \times Influent\ hardness\ + Q_Z \times Zero\ hardness$$

$$\frac{Q_P}{Q_T} = \frac{Desired\ hardness}{Influent\ hardness} = \%\ By\ pass$$

Break through point or time mean Ca, Mg will appear in the effluent from the zeolite filter.

$$Break\ through\ time = \frac{V}{Q} = \frac{Total\ volume\ of\ exchange\ in\ the\ zeolite}{Q_Z \times Influent\ hardness}$$

$$Break\ through\ time = \frac{Capacity\ of\ zeolite \times Volume\ of\ the\ zeolite\ filter}{Total\ amount\ of\ hardness\ to\ be\ remove\ at\ time\ (t)}$$

$$Break\ through\ time = \frac{\frac{kilo\ grain\ (eq.hardness)}{m^3} \times m^3}{\frac{m^3}{time} \times \frac{k\ grain\ (eq.hardness)}{m^3}} = time$$

Note: Brine solution (5%) for regeneration represent:

Salt	Water
5	100
138	x

x= Weight of water (kg) for 5% solution.

$$Volume = \frac{x}{\rho_w}$$

Membrane filtration:

It’s a physicochemical separation technique that used difference permeability as a separation mechanism. Water is pumped against the membrane surface resulting in the production of product and waste streams.

Membrane is a synthetic material less than 1 mm thick, semi-permeable (highly permeable to some components in the feed stream and less permeable to others).

Microfiltration (M.F): A membrane separation process in which particles greater than about 20 nanometers in diameter are screened out of a liquid in which they are suspended.

Ultra-filtration (U.F): Filtration through a filter capable of removing very minute (ultramicroscopic) particles. The filtration of a colloidal substance through a semi-permeable medium that allows only the passage of small molecules.

Nanofiltration (N.F): Nanofiltration is a relatively recent membrane filtration process used most often with low total dissolved solids water such as surface water and fresh groundwater, with the purpose of softening and removal of disinfection by-product precursors such as natural organic matter and synthetic. Pressure-driven filtration through a membrane that removes particles of about two nanometers or larger.

Reverse osmosis:

It is perhaps the best demonstrated technology for saline water conservation. If a semi-permeable membrane separates two bodies of water with differing salt concentrations the water will flow from less to more concentrated. The flow can be halted by applying pressure equal to the osmotic pressure of the concentrated solution, and reversed by exceeding that pressure. In practice the pressure ranges from 5-50 times the osmotic pressure of the water. Reverse osmosis systems include the membrane, a support structure, a pressure vessel and a pump.

Reverse osmosis is not easily applied to seawater although development of new membrane materials and configurations and use multistage processes. The cost of treating brackish water by this technique ranged (0.06-0.25) $/m^3.

Advantage:

1. Remove suspended with no coagulation up to a turbidly 200 NTU.
2. Very high removal in microorganisms.
3. Less space required than conventional filtration (1 m^2 floor area=100 m^2 membrane area).
4. High quality of filtered water.

Item	Membrane filter	Conventional filter
Filtration rate (vf)	0.03-0.17 m/hr	5-15 m/hr
Pressure	0.2-2 bar	0.18-0.3 bar
Filtration run	30-90 min	1-4 day
Backwash	1-5 min	5-15 min

1 bar = 14.5 psi = 100 kPa

Solved Problem

Problem 11.1: Design a cylindrical softener to be used in a textile factory. The influent flow is 500 m^3/day with hardness of 246 mg/L as $CaCO_3$. The softener is supplied with zeolite of an exchange capacity of 250 kilograin/m^3. Find height (H) and diameter (D) of the softener (assume H=D)?

Solution:
Assume break through time = 1 day
The amount of hardness to be removed = Flow (Q) X Concentration = Loading rate

$$Loading\ rate = \frac{500\frac{m^3}{day} \times 246\frac{mg}{L} \times 10^3}{10^6} = 123\frac{kg}{day} for\ one\ day\ process$$

$$123 \times 2.2 \times 7 = 1894\ kilo\ grain/day$$

$$Volume\ of\ zeolite\ required = \frac{Loading\ rate}{Zeolite\ exchange\ capacity} = \frac{1894\ kilo\ grain/day}{250\ kilo\ grain/m^3}$$

$$= 7.6\ m^3\ for\ one\ day\ process$$

$$Diameter\ (D) = \sqrt[3]{\frac{7.6\ m^3}{\frac{\pi}{4}}} = 2.13\ m = Height(H)$$

$$Total\ capacity\ of\ exchange = Capacity \times Volume$$

$$Total\ capacity\ of\ exchange = 250\ \frac{kilograin}{m^3} \times 7.6\frac{m^3}{day} = 1900\ kilograin/day$$

In this problem the break through time is one day (24 hours).
Different conditions may appear that will change the break through time:

1) Increasing or decreasing in influent hardness.
2) Increasing or decreasing in influent flow.
3) Increasing or decreasing the exchange capacity (Changing the zeolite material).

For the same problem above, the hardness increased from 246 to 400 mg/L as $CaCO_3$

$$Loading\ rate = \frac{400 \times 500 \times 2.2 \times 7 \times 10^3}{10^6} = 3080\frac{k\ grain}{day} for\ one\ day\ process$$

Available volume for exchange = 7.6 X 250 =1900 k grain/day

$$Break\ through\ time\ (BT) = \frac{Available\ volume\ or\ capacity\ for\ exchange}{Loading\ rate} = \frac{1900\ k\ grain}{3080\frac{k\ grain}{day}} = 0.616\ day$$

$$= 14.90\ hour\ (Every\ 15\ hr\ the\ filter\ must\ be\ regenerated$$

If a new zeolite is used of a capacity from 250 to 480 kgrain/m^3. Find break through time:

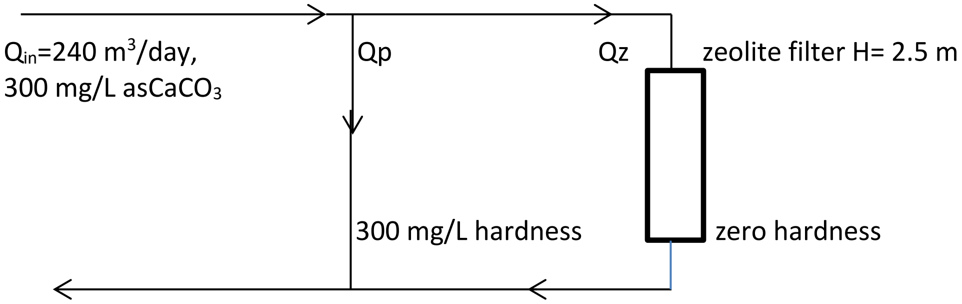
$$Loading\ rate = \frac{500\frac{m^3}{day} \times 246\frac{mg}{L} \times 10^3}{10^6} = 123\frac{kg}{day} for\ one\ day\ process$$

$$123 \times 2.2 \times 7 = 1894\ kilo\ grain/day$$

$$Volume\ of\ zeolite\ required = \frac{Loading\ rate}{Zeolite\ exchange\ capacity} = \frac{1894\ kilo\ grain/day}{480\ kilo\ grain/m^3}$$
$$= 3.945\ m^3\ for\ one\ day\ process$$
$$Diameter\ (D) = \sqrt[3]{\frac{3.945\ m^3}{\frac{\pi}{4}}} = 1.713\ m = Height(H)$$
$$Total\ capacity\ of\ exchange = Capacity \times Volume$$
$$Total\ capacity\ of\ exchange = 480\ \frac{kilograin}{m^3} \times 3.945 \frac{m^3}{day} = 1893.6\ kilograin/day$$
$$Break\ through\ time\ (BT) = \frac{Total\ capacity\ for\ exchange}{Loading\ rate} = \frac{480\ k\frac{grain}{m^3} \times 3.945\ m^3}{1894 \frac{k\ grain}{day}} = 0.999\ day$$
$$= 23.995\ hour\ (Every\ day\ the\ filter\ must\ be\ regenerated$$

Problem 11.2: Design soft water softener as shown in the figure. The zeolite capacity is 235 eq. hardness/m^3. Find:

1) Surface area of the filter?
2) Amount of salt used for regeneration, assume 150 gm/eq. hardness is required?
3) Volume of water for regeneration (10% solution)?



Desired hardness 60 mg/L as CaCO$_3$

Solution:

1) <u>Mass Balance (Output</u>)

$$60 \frac{mg}{L} as\ CaCO_3 \times 240 \frac{m^3}{day} = Q_p \times 300 \frac{mg}{L} + Q_z \times zero$$
$$Q_p = Influent\ flow \times \frac{Desired\ hardness}{Influent\ hardness} = 240 \frac{m^3}{day} \times \frac{60\ mg/L}{300\ mg/L} = 48\ m^3/day$$
$$Q_z = Q_{Influent} - Q_p = 240 - 48 = 192\ m^3/day$$

$$Amount\ of\ hardness\ to\ be\ removed\ (Loading\ rate) = \frac{192 \frac{m^3}{day} \times 300 \frac{mg}{L} \times 10^3}{10^6} = 57.7 \frac{kg}{day}$$
$$= 57.6 \times 2.2 \times 7 = 887.04 \frac{kgrain}{m^3} = 887.04\ eq.hardness\ for\ one\ day\ process$$

$$Volume\ of\ zeolite = \frac{887.04\ eq.hardness/day}{235\ eq.hardness/m^3} = 3.77\ m^3$$

$$Surface\ area\ A_s = \frac{3.77}{2.5} = 1.52\ m^2$$

2) Amount of salt required for regeneration:

$$Amount\ of\ salt\ required\ for\ regeneration = 150\frac{gm}{eq.hardness} \times \frac{887.04\ \frac{eq.hardness}{day}}{1000}$$
$$= 133.056\ kg/day$$

3) Weight of water for 10% brine solution:

Water salt
100 10 x= 1330 kg For ρw = 10^3 kg/m^3
x 133 Volume of water = 1330/10^3 = 1.33 m^3

Problem 10.3: For water softening, a cylindrical zeolite filter is used with capacity 48 kg/m^3, in a water system designed for treating 500 m^3/day. In this system 30% by-pass is used and the influent has hardness 500 mg/L as $CaCO_3$. Find the:

a) Number of filters need if each filter has 0.5 m diameter and 2 m height if the one-day break through time?
b) Determine the desired effluent hardness?
c) Break through time (hr) if the hardness of the influent reach 750 mg/L as $CaCO_3$?

Solution:

$$Q_T = 500\ m^3/day\ ,\ Q_p = 0.3 \times 500 = 150\ m^3/day\ \ ,\ Q_z = 0.7 \times 500 = 350\ m^3/day$$

$$Zeolite\ capacity = 48\frac{kg}{m^3} = 48\frac{kg}{m^3} \times 2.2 \times 7 = 739.2\ kilograin/m^3$$

Assume break through time = 1 day

The amount of hardness (kg/day) = flow rate (m^3/day) * concentration (kg/m^3)

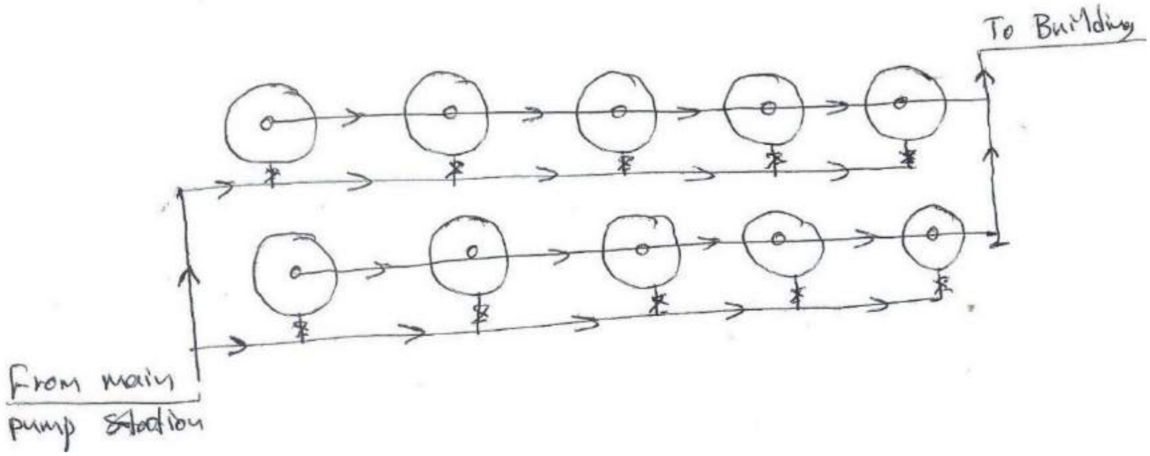
$$Loading\ rate = 350\frac{m^3}{day} \times 500 \times 10^{-3}\ \frac{kg}{m^3} = 175\frac{kg}{day} \times 2.2 \times 7 = 2695\ kilograin/day$$

$$Volume\ of\ zeolite\ required = \frac{Loading\ rate}{Zeolite\ exchange\ capacity} = \frac{2695\ kilograin/day}{739.2\ kilograin/m^3}$$
$$= 3.645\ m^3\ for\ one\ day\ process$$

$$Volume\ of\ each\ \ filter = \frac{\pi}{4} \times 0.5^2 \times 2 = 0.3925\ m^3$$

$$No\ of\ filters = \frac{3.645}{0.3925} = 9.286 \approx 10\ filter$$

$$Effluent\ hardness = \frac{350 \times 0 + 150 \times 500}{500} = 150\ mg/L$$



If the hardness increased to 750 mg/L as $CaCO_3$

$$Loading\ rate = 350\ \frac{m^3}{day} \times 750 \times 10^{-3}\ kg/m^3 \times 2.2 \times 7 = 4042.5 \frac{kilo\ grain}{day} for\ one\ day\ process$$

$$Break\ through\ time\ (BT) = \frac{Available\ volume\ or\ capacity\ for\ exchange}{Loading\ rate}$$

$$= \frac{739.2 \frac{kilograin}{m^3} \times 3.645\ m^3}{4042.5\ \frac{kilo\ grain}{day}} = 0.666\ day$$

$$= 15.996\ hour\ (Every\ 16\ hr\ the\ filter\ must\ be\ regenerated$$

Problem 11.4: For water softening, a cylindrical zeolite filter is used with capacity 60 kg/m^3, in a water system designed for treating 1560 m^3/day. In this system 20% by-pass is used and the influent has hardness 360 mg/L as $CaCO_3$. Find the:

a) Effluent hardness from the softener if the final desired hardness is 75 mg/L as $CaCO_3$?
b) Number of filters need if each filter has 0.8 m diameter and 2 m height if the one-day break through time?
c) Break through time (hr) if the hardness of the influent reach 750 mg/L as $CaCO_3$?

Solution:

$Q_T = 1560\ m^3/day\ ,\ Q_p = 0.2 \times 1560 = 312\ m^3/day\ \ ,\ Q_z = 0.8 \times 1560 = 1248\ m^3/day$

a) $Q_{Effl} \times Hard_{Effl} = Q_p \times Hard_{Effl} + Q_z \times Hard_{Z\ EFFL} \Rightarrow 1560 \times 75 = 312 \times 360 + 1248 \times Hard_{Z\ EFFL}$

Effluent hardness from the softener = 3.75 mg/L

b) Case 1: 20% by-pass:

Zeolite capacity, $Zeolite\ capacity = 60 \frac{kg}{m^3} = 60 \frac{kg}{m^3} \times 2.2 \times 7 = 924\ kilograin/m^3$

Assume break through time = 1 day

The amount of hardness (kg/day) = flow rate (m^3/day) * concentration (kg/m^3)

$$Loading\ rate = 1248 \frac{m^3}{day} \times 360 \times 10^{-3}\ \frac{kg}{m^3} = 499.28 \frac{kg}{day} = 499.28 \frac{kg}{day} \times 2.2 \times 7$$
$$= 6918.912\ kilograin/day$$

$$Volume\ of\ zeolite\ required = \frac{Loading\ rate}{Zeolite\ exchange\ capacity} = \frac{6918.912\ kilograin/day}{924\ kilograin/m^3}$$
$$= 7.488\ m^3\ for\ one\ day\ process$$
$$Volume\ of\ each\ \ filter = \frac{\pi}{4} \times 0.8^2 \times 2 = 1.0048\ m^3$$

$$No\ of\ filters = \frac{7.488}{1.0048} = 7.452 \approx 8\ filter$$

Case 2: 0% by-pass:

Zeolite capacity, $Zeolite\ capacity = 60\frac{kg}{m^3} = 60\frac{kg}{m^3} \times 2.2 \times 7 = 924\ kilograin/m^3$

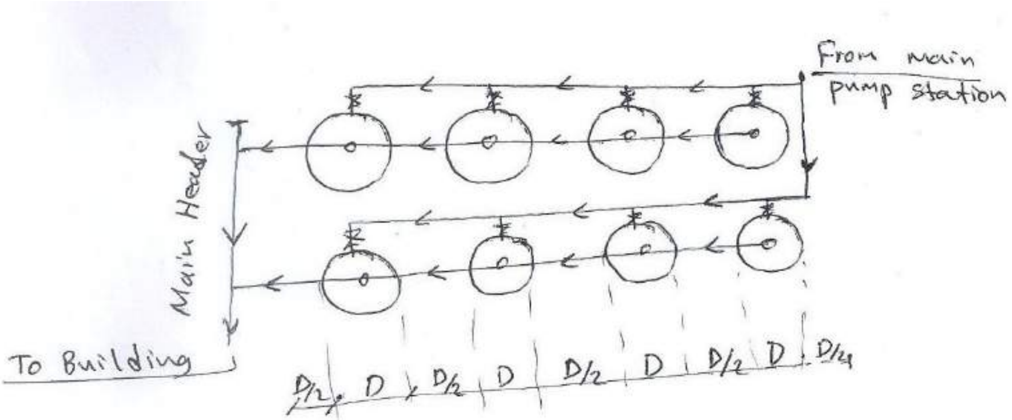
Assume break through time = 1 day

The amount of hardness (kg/day) = flow rate (m^3/day) * concentration (kg/m^3)

$$Loading\ rate = 1560\frac{m^3}{day} \times 360 \times 10^{-3}\ \frac{kg}{m^3} = 561.6\frac{kg}{day} = 561.6\frac{kg}{day} \times 2.2 \times 7$$
$$= 8648.64\ kilograin/day$$

$$Volume\ of\ zeolite\ required = \frac{Loading\ rate}{Zeolite\ exchange\ capacity} = \frac{8648.64\ kilograin/day}{924\ kilograin/m^3}$$
$$= 9.36\ m^3\ for\ one\ day\ process$$
$$Volume\ of\ each\ \ filter = \frac{\pi}{4} \times 0.8^2 \times 2 = 1.0048\ m^3$$

$$No\ of\ filters = \frac{9.36}{1.0048} = 9.315 \approx 10\ filter$$



If the hardness increased to 750 mg/L as $CaCO_3$

$$Loading\ rate = 1248\ \frac{m^3}{day} \times 750 \times 10^{-3}\frac{kg}{m^3} \times 2.2 \times 7 = 14414.4\frac{kilo\ grain}{day} for\ one\ day\ process$$

$$Break\ through\ time\ (BT) = \frac{Available\ volume\ or\ capacity\ for\ exchange}{Loading\ rate} = \frac{924\ \frac{kilograin}{m^3} \times 7.488\ m^3}{14414.4\ \frac{kilo\ grain}{day}}$$
$$= 0.48\ day = 11.52\ hour\ (Every\ 12\ hr\ the\ filter\ must\ be\ regenerated$$

Sanitary and Environmental Engineering

PART 2: WASTEWATER ENGINEERING

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Lecture 1: Introduction to Wastewater or Sewage

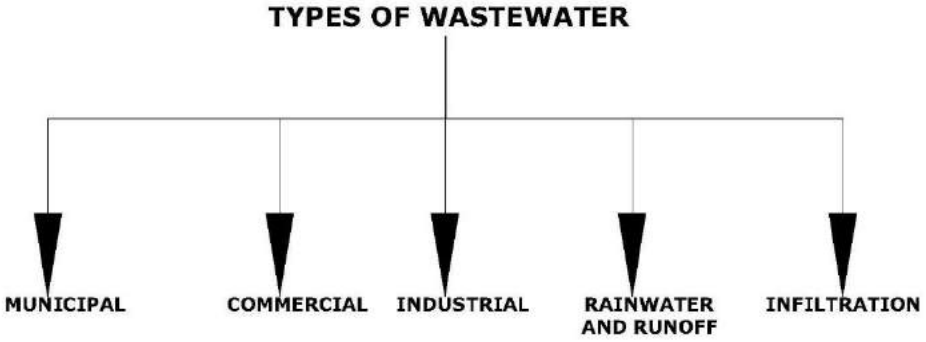
The liquid waste generated from various activities: residential, industrial, commercial and public areas, due to the consumption of potable water. It is collected and transmitted in a pipe system known as the sewer system. This wastewater contains impurities and/ or pollutants in the form of solids, liquids and gases in concentrations harmful to the environment. The management of this wastewater require knowing its quantity and quality.

Sewerage is applied to the art of collecting, treating, and disposing of sewage**.** It implies the collecting of wastewaters from occupied areas and conveying them to some point of disposal. The liquid wastes will require treatment before they can be discharged into a body of water or otherwise disposed of without endangering the public health or causing offensive condition.

Sewerage works are comprehensive terms covering all structures and procedures required for collecting, treating, and disposing of sewage.

Types of wastewater flowing in the sewer system:

1) Municipal-Sanitary-Domestic sewage: The liquid wastes conducted away from residences, business buildings, and institutions.
2) Industrial wastewater: The liquid waste from industrial establishments.
3) Rainwater-Runoff-Storm sewage.
4) Infiltration: Groundwater entering sanitary sewers through defective pipe joints and broken pipes. Infiltration will occur where local groundwater elevation is higher than the sewer pipe. Gravel bedding materials in sewer pipe trenches act as a French drain. Groundwater flows parallel to the sewer until it reaches the area of damaged pipe. In areas of low groundwater, sewage may exfiltration into groundwater from a leaking sewer.



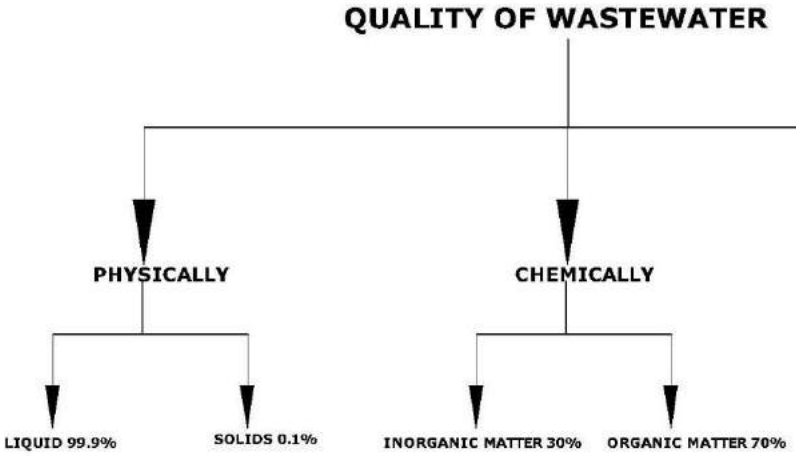
Sewage treatment covers any process to which sewage is subjected in order to remove or alter its objectionable constituents so as to render it less dangerous or offensive. Treatment which produces an effluent containing not more than 20 mg/L BOD_5 and 30 mg/L suspended solids (SS).

Sewage disposal applies to act of disposing of sewage by any method. It may be done with or without previous treatment.

Note: The degree of treatment required depends upon the water quality standards applicable to the receiving stream and the flow and quality of both the stream and waste.

Quality-Characteristic of wastewater:

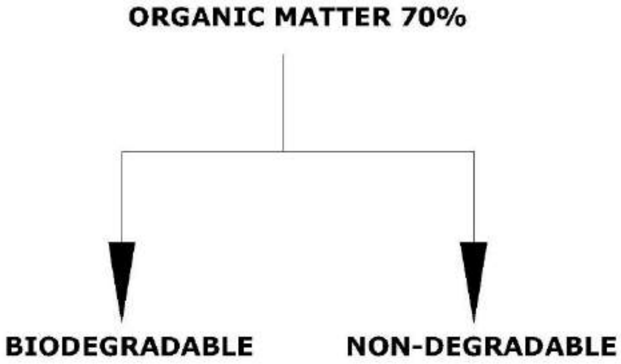
1) *Physically:* 99.9% water and 0.1% solids (Suspended and Dissolved).
2) *Chemically:* 30% inorganic and 70% organic.
3) *Microbiological:* Different types of bacteria, algae, fungi and protozoa.

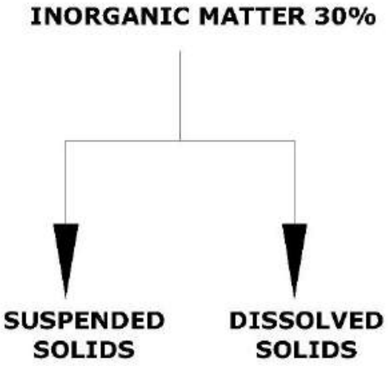


Inorganic materials: The suspended solids are sand, silt, clay particles and grits (glass, gravel, etc.)
The dissolved solids are toxic and nontoxic material.

- **a)** Nontoxic solids are different types of salts: cations: Ca, Mg, Mn, Al, Fe, Na, K and anion: HCO_3, CO_3, SO_4, Cl, PO_4, and NO_3.
- **b)** Toxic minerals: **Hg, Pb, Cd, Cr, Cu, Ag, Ni, Zn.**

Organic materials containing C, H, O, N, S, and P in their compounds. These compounds are suspended and dissolved materials in wastewater, which are solids, liquids and gases. Like: carbohydrates, proteins, fats, oil, grease, phenols, pesticides, detergents, etc.





Gases in wastewater, CO_2, NH_4, CH_4, H_2S

The main content in wastewater is the organic matter which is considered a main pollutant that is harmful to the environment. The environment can deal with these materials by the decomposition process done by the organisms depending on: type of the organic matter, concentration of the dissolved oxygen in the water, type of organisms and the environment conditions.

Decomposition or degradation of organic matter:

1) Aerobic process (with oxygen):

Organic matter + O_2 + organisms $\longrightarrow$ New calls + CO_2 + H_2O + SO_4 + NO_3 + NH_3

2) Anaerobic process (with no oxygen)

Organic matter + organisms $\longrightarrow$ New calls + CO_2 + H_2O + N_2 + NH_3 + CH_4 + H_2S

Determination of the concentration of organic matter:

1) *BOD (Biochemical oxygen demand):* The oxygen utilized by the microorganisms, under aerobic conditions to stabilize (decompose) the biodegradable organic matter. The concentration of organic matter = the amount of oxygen consumed.
2) *COD (Chemical oxygen demand):* The oxygen equivalent of organic matter that is oxidized by a strong chemical oxidant. The organic matter is destroyed by a mixture of chromic and sulfuric acids then are converted to CO_2 and water.
3) *TOC (Total organic carbon):* To convert the organic carbon of the organic matter to carbon dioxide CO_2 by using heat, oxygen, ultraviolet radiation and chemical oxidant.

The relationships between these components are in the following ratios:
If the BOD/COD ratio is 0.5 and greater for the untreated waste ($\frac{BOD}{COD} \geq 0.5$) then the waste is easily degradable by biological means. If the ratio is less than 0.3 ($\frac{BOD}{COD} < 0.3$), the waste may have toxic components.

Type of wastewater	$\frac{BOD}{COD}$	$\frac{BOD}{TOC}$
untreated	0.3 – 0.8	1.2 – 2.0
treated	0.1 – 0.3	0.2 – 0.5

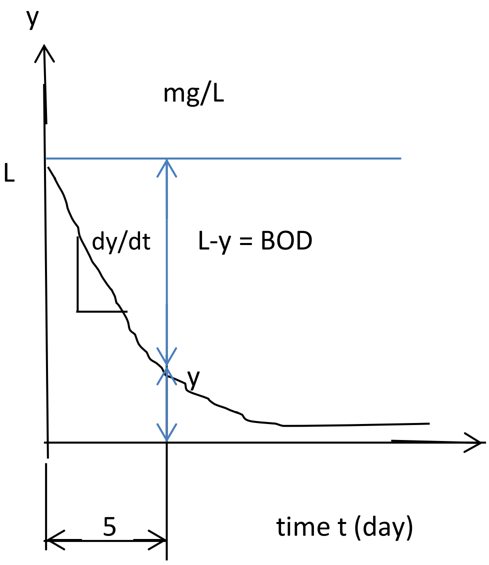
The standard BOD test:

1) *Put the wastewater in 2 bottles.*
2) *Determine the dissolved oxygen in bottle 1 (DO_1)*
3) *Put bottle 2 in the incubator at temperature 20 ^{0}C for 5 days. The bacterial will grow and decompose the organic matter in the wastewater utilizing the dissolve oxygen (DO_1).*
4) *Determine the remaining dissolved oxygen in bottle 2 (DO_2).*
5) *Calculate BOD in the wastewater = $DO_1 - DO_2$*

Note: Dissolved oxygen is measured by a) chemical reaction – using 2 bottles b) Using an electrode – one bottle.

The standard test is at temperature 20 ^{0}C as this is the optimum temperature for the bacteria to grow. The carbonic organic matter needs about 20 days for complete degradation. But through the 5 days' incubation time about 70% of this organic matter will decompose. This is explained by the following:

The decomposition of organic matter is a 1st order reaction:



$$\frac{dy}{dt} = -K_1 y \Rrightarrow \frac{dy}{y} = -K_1 dt$$

By integration:

$$\ln y = -K_1 t + c \qquad (1)$$

At t = 0, y = L

So, c = ln L and eq. 1 will be:

$$\ln y = -K_1 t + \ln L \;\; or \;\; \ln\frac{y}{L} = -K_1 t$$

$$\frac{y}{L} = e^{-K_1 t} \quad or \;\; y = Le^{-K_1 t}$$

y: The non-degradable (remaining) organic matter at time t (mg/L).
L: The ultimate organic matter at time 0 = BOD_u (mg/L).
K_1: Degradation (deoxygenation) rate (day^{-1}).
t: Time for decomposition (day).
The oxygen utilized or organic matter decomposed through time t (day) at temperature T (^{0}C) will be:
$BOD_{t-T} = L - y = L - Le^{-K_1 t} = L(1 - e^{-K_1 t})$, So, the standard test gives $BOD_{5\text{-}20}$:
For domestic sewage K_1 = 0.23 /day at T = 20 ^{0}C
To change the rate from any temperature to 20 ^{0}C by using ($K_{1(T)} = K_{1(20)} \times 1.047^{(T-20)}$).

$$BOD_{5-20} = L(1 - e^{-0.23\times 5}) \Rrightarrow \frac{BOD_{5-20}}{L} \approx 0.68 \approx 70\%$$

Typical domestic sewage characteristic

Parameter mg/L	Weak	Medium	Strong
Total dissolved solids	270	500	860
Total suspended solids	120	210	400
BOD_5	110	190	350
COD	250	430	800
TOC	80	140	260
Total N	20	40	70
Total P	4	7	12
Chloride	30	50	90
Sulfate	20	30	50

Solved problems

Problem 1: The results from the laboratory show that the BOD $_{1\text{-}30}$ = 110 mg/L for a wastewater sample. Find: 1-ultimate BOD (BOD$_u$), 2- BOD $_{5\text{-}20}$?

Solution:

$$K_{1(T)} = K_{1(20)} \times 1.047^{(T-20)} \Rightarrow K_{1(30)} = 0.23 \times 1.047^{(30-20)} = 0.364\ day^{-1}$$

$$BOD_{t-T} = L(1-e^{-K_1 t}) \Rightarrow BOD_{1-30} = L(1-e^{-K_{1-30}t}) \Rightarrow 110\frac{mg}{L} = L(1-e^{-0.364\times 1}) \Rightarrow$$

$$L = 360.527\ mg/L$$

$$BOD_{t-T} = L(1-e^{-K_1 t}) \Rightarrow BOD_{5-20} = L(1-e^{-K_{1-20}t}) \Rightarrow BOD_{5-20} = 360.527 \times (1-e^{-0.23\times 5})$$

$$= 246.371\ mg/L$$

Problem 2: If the waste with ultimate BOD$_u$ equal 190 mg/L and k$_{1\text{-}20}$ = 0.23/day is discharge to a stream at an average temperature 25^{0}C,

a) compute the BOD would be exerted in 3 and 5 days at temperature 20 and 25 ^{0}C, (BOD_{3-20} , BOD_{5-20} , (BOD_{3-25}), BOD_{5-25} ?

b) Compute the organic loading in (kg/day) for BOD_{5-20} if the flowrate 50 m^3/hr?

Solution:

a) $BOD_{5-20} = 190 \times (1-e^{-0.23\times 5}) = 129.939\ mg/L$

$$BOD_{3-20} = 190 \times (1-e^{-0.23\times 3}) = 94.7\ mg/L$$

$$K_{1(25)} = K_{1(20)} \times 1.047^{(T-20)} = 0.23 \times 1.047^{(25-20)} = 0.289\ /day$$

$$BOD_{3-25} = 190 \times (1-e^{-0.289\times 3}) = 110.16\ mg/L$$

$$BOD_{5-25} = 190 \times (1-e^{-0.289\times 5}) = 145.208\ mg/L$$

b) $Oranic\ load\frac{kg}{day} = 129.939 \times 10^{-3}\frac{kg}{m^3} \times 50 \times 24\frac{m^3}{day} = 155.926\ kg/day$

Sanitary and Environmental Engineering

PART 2: WASTEWATER ENGINEERING

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Lecture 2: Quantity of wastewater:

The proportion of the water consumed which reach the sewer will must be decided upon after careful consideration of local conditions. Although the sewage may vary in individual cities from 70-130 percent of the water consumed, designers frequently assume that the average rate of sewage flow, including a moderate allowance for infiltration, equal the average rate of water consumption.

1) Domestic: 80-90% water consumption.
2) Industrial: a) domestic as in 1; b) Water process.
3) Public: as in 1.
4) Runoff or storm sewage: Using the rational formula:

$$Average\ daily\ demand = (70 - 130) \times of\ water\ consumption$$

Fluctuations in Dry Weather Flow

Since dry weather flow depends on the quantity of water used, and as there are fluctuations in rate of water consumption, there will be fluctuations in dry weather flow also. In general, it can be assumed that:

$$Maximum\ sewage\ daily\ demand\ Q_{Max} = (1.5 - 2) \times Average\ sewage\ daily\ demand$$

$$Minimum\ sewage\ daily\ demand\ Q_{Min} = (0.5 - 0.67) \times Average\ sewage\ daily\ demand$$

Observation of fluctuations in various cities indicates that the peak for a small residential area is likely to be 225 percent of the average for that day. For commercial area the peak may reach 150 percent of the average and for industrial areas somewhat less. The flow in the outfall line of a sewer system serving a city having a normal population and commercial and industrial activities will have a peak flow of about 150 percent of the daily average. Some designers use the following formula to estimate the maximum rate of domestic sewage flow from small areas:

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

Where:

M: Ratio of the maximum sewage flow to the average ($M = \frac{Maximum\ sewage\ flow}{Average\ sewage\ flow}$).

P: Population served in thousands.

The maximum sewage flow will be the hourly maximum, or the peak rate of the maximum day plus the maximum infiltration.

IMPORTANT NOTE: The minimum rates of sewage flow are useful in the design of sewage pumping plants and occasionally to investigate the velocities in sewers during low flow periods. Generally, the minimum flow may be taken as 50 percent of the average.

For design purposes (minimum requirements):

Sanitary sewer type	Flow (L per capita/day)
Laterals and sub main sewers	1500 including normal infiltration
Main, trunk, and outfall sewers	950 including normal infiltration + industrial wastes if known to be in large amounts.

Amount of storm sewage:

Rational formula: Q= CiA

Q: Runoff discharge (L^3/t).

A: Catchment area (L^2).

C: Runoff coefficient depending on the surface type and different building areas as shown in the table

Type of surface	C	Description of area	C
Asphalt streets	0.85-0.90	Business	0.7- 0.95
Cement streets	0.80-0.95	Residential –single family	0.30-0.50
Gravel driveways	0.15-0.30	Residential –multi units	0.40-0.75
Lawns, sandy soil	0.05-0.20	Industrial-light	0.50-0.80
Lawns, heavy soil	0.13-0.35	Industrial- heavy	0.60-0.90

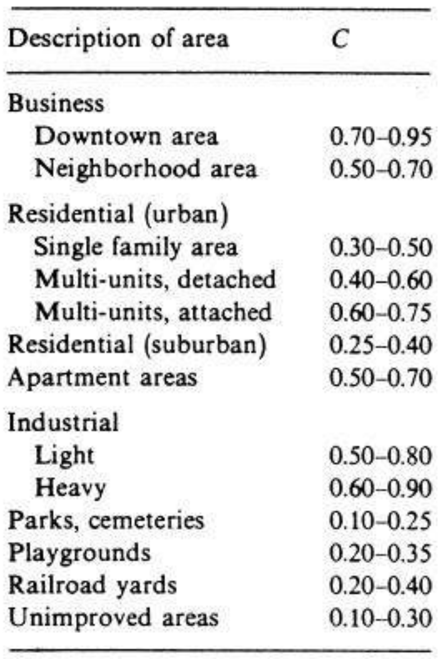
i= Rainfall intensity, using intensity duration curves which are in the form of equations:

$$i_{(\frac{mm}{hr})} = \frac{a}{b+t}$$

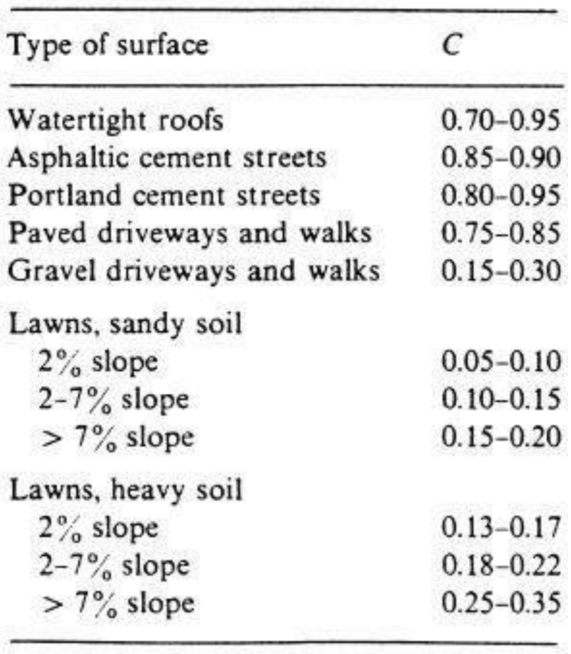
Where:

a & b are constants and t: the duration of storm (minute)

Table 13-2: Runoff coefficients for different areas.



Description of area	C
Business	
Downtown area	0.70–0.95
Neighborhood area	0.50–0.70
Residential (urban)	
Single family area	0.30–0.50
Multi-units, detached	0.40–0.60
Multi-units, attached	0.60–0.75
Residential (suburban)	0.25–0.40
Apartment areas	0.50–0.70
Industrial	
Light	0.50–0.80
Heavy	0.60–0.90
Parks, cemeteries	0.10–0.25
Playgrounds	0.20–0.35
Railroad yards	0.20–0.40
Unimproved areas	0.10–0.30



Type of surface	C
Watertight roofs	0.70–0.95
Asphaltic cement streets	0.85–0.90
Portland cement streets	0.80–0.95
Paved driveways and walks	0.75–0.85
Gravel driveways and walks	0.15–0.30
Lawns, sandy soil	
2% slope	0.05–0.10
2–7% slope	0.10–0.15
> 7% slope	0.15–0.20
Lawns, heavy soil	
2% slope	0.13–0.17
2–7% slope	0.18–0.22
> 7% slope	0.25–0.35

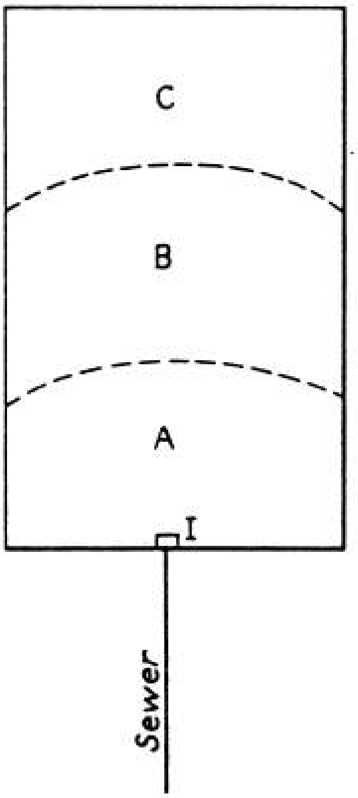
Time of concentration

The runoff time to reach the collection point into the desired sewer.

$$Time\ of\ concentration\ (t_c) = Inlet\ time\ (t_i) + Flowing\ time\ (t_f)$$

Inlet time (t_i): Time required for rainwater to flow from the most remote distance in the catchment area to reach the collecting point and produce the maximum discharge, from figure 13.3 page 325.

Flowing time (t_f): Time required for the flowing discharge inside the sewer to reach the collecting point.



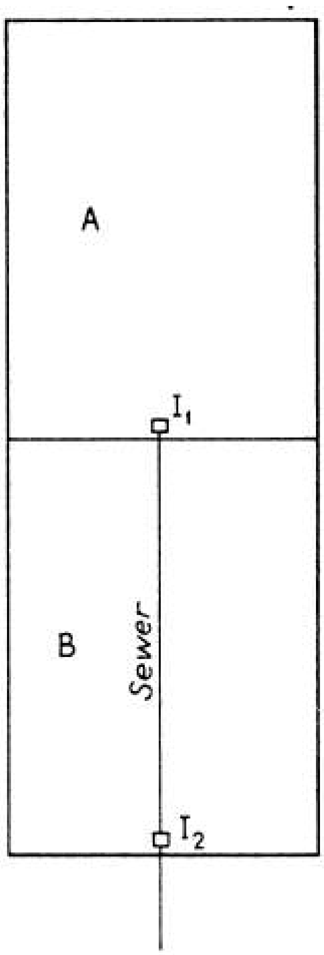


Fig.13-1: Inlet time.

Fig. 13-2: Inlet time and flow time.

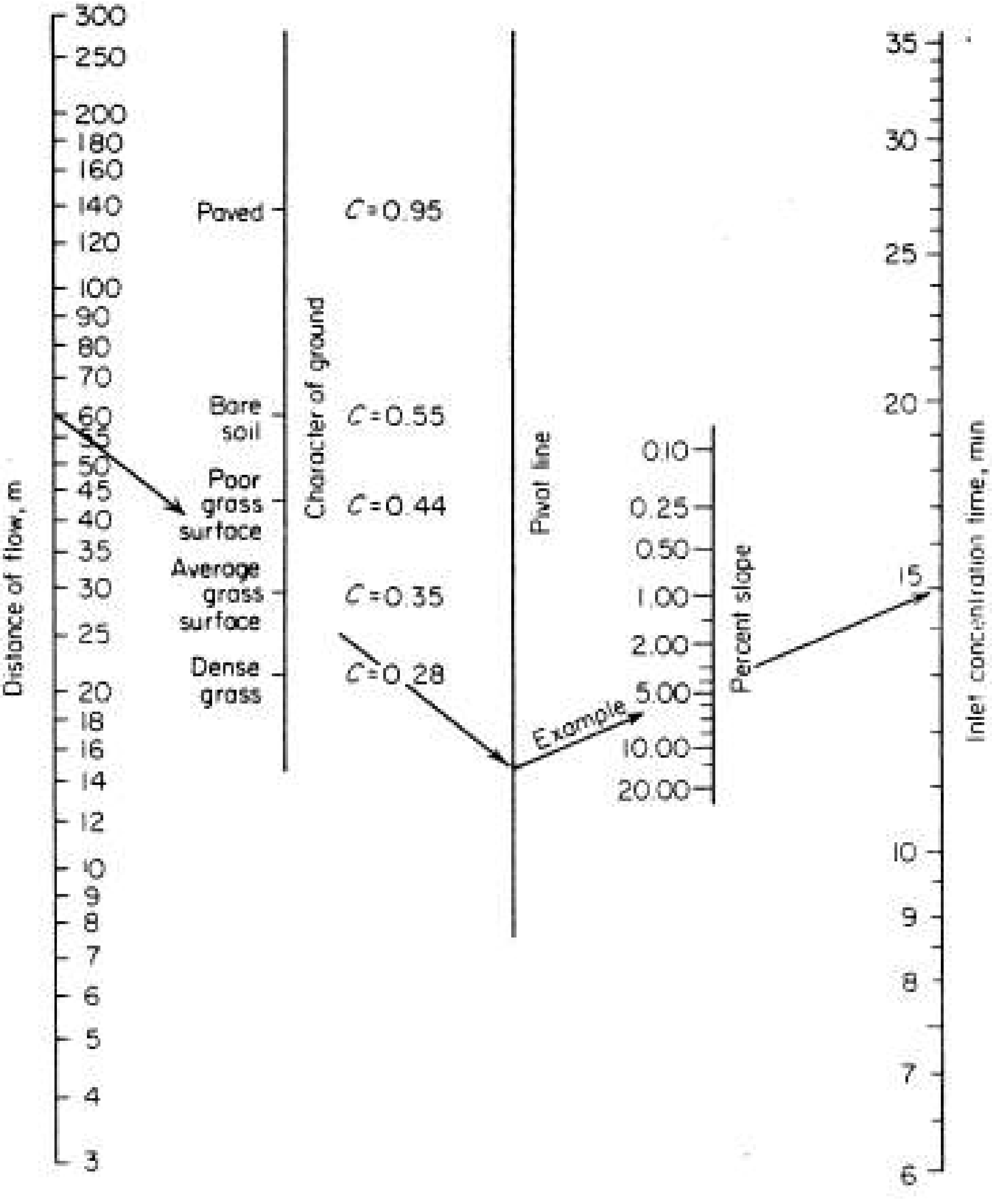


Fig. 13-3 Overland flow time.

Design periods and use of sewage flow data:

Detail	Design period (year)	Design criteria
Design of a sewer system	indefinite	It is necessary to estimate maximum population densities expected in various districts and locations of commercial and industrial districts together with maximum rates of sewage flow per second and maximum infiltration per day.
Sewage pumping plant	10	Rate of flow required are average daily, peak, and minimum flow rates, including infiltration.
Sewage treatment plant	15-20	Flow rate required are average and peak rates, both include infiltration.

Solved problems

Problem 1: Determine the runoff coefficient for an area of 0.20 km^2, 3000 m^2 is covered by building, 5000 m^2 by paved driveways and walks, and 2000 m^2 by Portland cement street. The remaining area is flat, heavy soil, covered by grass lawn? Compute the runoff coefficient for the whole area using the values in Table 13-1?

Solution:

surface	runoff coefficient, C (Table 13-1)	Area ratio ($\frac{A}{A_{total}}$)	runoff coefficient for an area ($C \times \frac{A}{A_{total}}$)
Roofs	0.70 – 0.95	$\frac{3000}{200000}$	$0.0105 - 0.01425$
Driveways and walks	0.75 – 0.85	$\frac{5000}{200000}$	$0.01875 - 0.02125$
Street	0.8 – 0.95	$\frac{2000}{200000}$	$0.008 - 0.0095$
Grass lawn	0.13 – 0.17	$\frac{190000}{200000}$	$0.1235 - 0.1615$
C_{Avg}			$0.16 - 0.21$

Problem 2: Find the runoff discharge (m^3/min) for an area 200,000 m^2 where i= 95 mm/hr. The area consists of: 30% roofs (C =0.7), 20% asphalt pavements (C=0.85), 10% paved sidewalks (C=0.75) and the remaining are lawns (C = 0.18) of heavy soil covered with grass 5%in slope?

Solution:

The average runoff coefficient, $C_{Avg} = 0.3 \times 0.7 + 0.2 \times 0.85 + 0.1 \times 0.75 + 0.4 \times 0.18 = 0.527$

$$Q = CiA = 0.527 \times 0.095 \frac{m}{hr} \times 200000 = 10013 \frac{m^3}{hr} = 166.833\ m^3/min$$

Problem 3: Find the runoff discharge (m^3/sec) for the catchment region of area 100,000 m^2. The remote distance= 60 m, the surface is grass of slope 4%, C= 0.35 and $i = \frac{5230}{t+30}$? Also, the diameter of the sewer pipes to carry this discharge at full flow condition; assume the flowing velocity is 1 m/sec?

Solution:

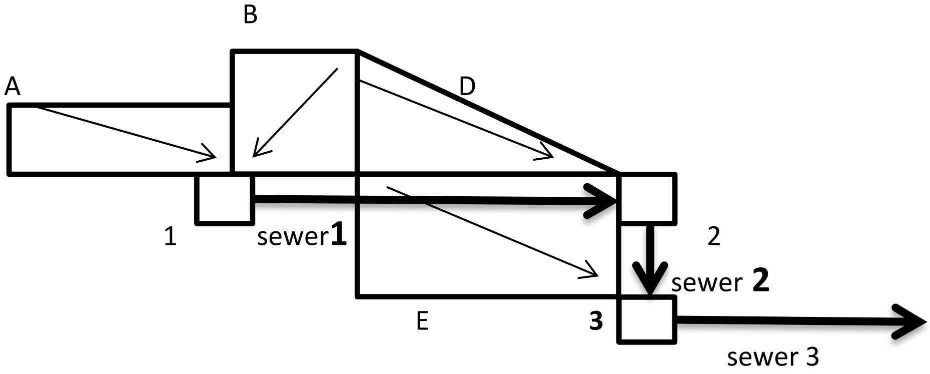
From Fig. 13-3, with distance = 60 m, C = 0.35, Slope = 4%, find the inlet time =15 minute = time of concentration due to flow time = 0.

$$i = \frac{5230}{t + 30} = \frac{5230}{15 + 30} = 116.2\ mm/hr$$

$$Q = CiA = 0.35 \times 0.1162 \frac{m}{hr} \times 100000 = 4067 \frac{m^3}{hr} = 1.129\ m^3/sec$$

$$Q = v \times A = 1.129 = 1 \frac{m}{sec} \times \frac{\pi}{4} \times D^2 \Longrightarrow D = 1.199\ m\ say\ use\ pipe\ diameter\ 1200\ mm$$

Problem 4: For the given plan, find the diameter of the sewer pipe 2 at full flow condition (assume flowing velocity 1m/sec). Use common intensity method if ($i = \frac{5230}{t+30}$).



item	Area (m^2)	C	t_i (min)	pipe	Flowing time (t_f)
A	7000	0.5	10	1	5.5
B	8000	0.6	11	2	1.5
D	10000	0.3	16		
E	15000	0.6	14		

Solution:

To find t for each area where flow will reach points

Area	t_i (min) for roof	t_f (min) for pipe	tc (min)
A	10	Pipe 1: 5.5	15.5
B	11	Pipe 1: 5.5	16.5 (Controlled)
D	16	Pipe 2: 1.5	16
E	14	0	14

$$i = \frac{5230}{t+30} = \frac{5230}{18+30} = 108.958\ mm/hr$$

Runoff discharge reaching collecting points is from area A, B, and C, using common intensity method

$$Q = i \times \sum_{y=1}^{n} A_y \times C_y$$

$$= 0.108958 \times \sum_{y=1}^{n} (7000 \times 0.5 + 8000 \times 0.6 + 10000 \times 0.3 + 15000 \times 0.6) = 2211.847 \frac{m^3}{hr}$$

$$= 0.614\ m^3/sec$$

$$Q = v \times A = 0.614 = 1 \frac{m}{sec} \times \frac{\pi}{4} \times D^2 \Longrightarrow D = 0.884\ \ m\ say\ use\ pipe\ diameter\ 900\ mm$$

Problem 5: An urban area has an area of 100000 m^2 and a runoff coefficient of 0.45. Using duration of 25 min and the curve of Fig. 13-4 compute the runoff resulting from point rainfalls with a recurrence interval of 5 and 15 years?

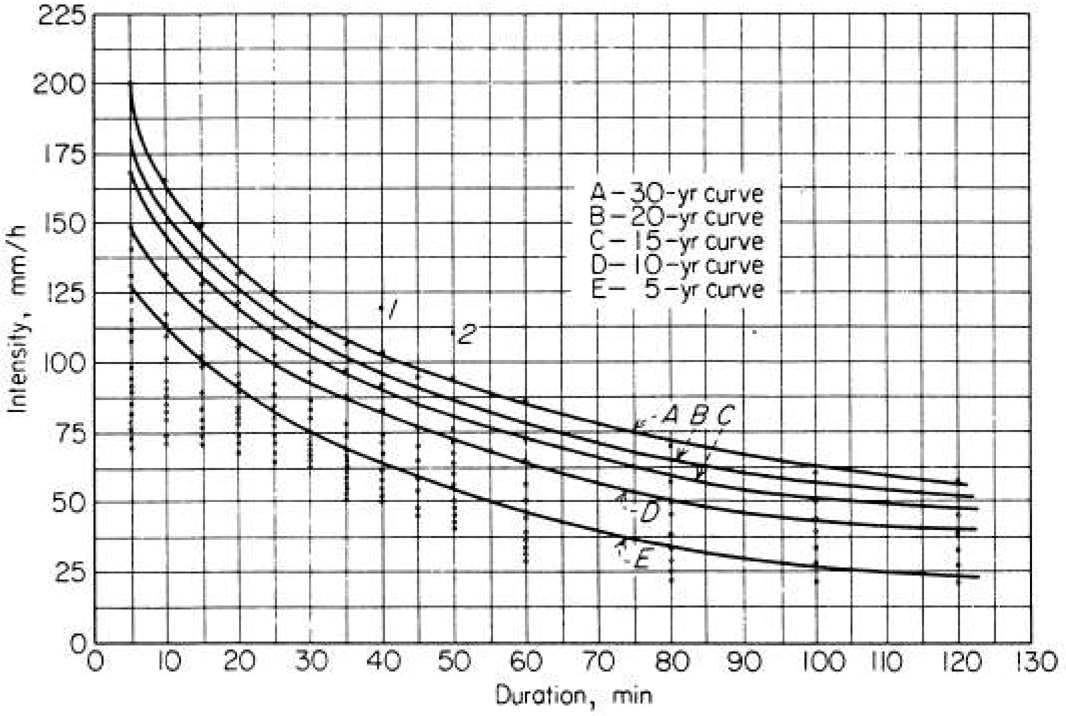


Fig. 13-4: Rainfall curves derived from storm records.

Solution:

A = 100000 m^2, C = 0.45

During 25 min at interval 5 & 15 year

From Fig. (13-4), I = 81.5 mm/hr for 5 year and I = 109.375 mm/hr for 15 year

For 5 year: $Q_{(m^3/sec)} = 0.278 \times C_{(mm/hr)} \times I \times A_{(km^2)} = 0.278 \times 0.45 \times 81.25 \times \frac{100000}{10^6} = 1.016\, m^3/sec$

For 15 year: $Q_{(m^3/sec)} = 0.278 \times C_{(mm/hr)} \times I \times A_{(km^2)} = 0.278 \times 0.45 \times 109.375 \times 0.1 = 1.368\, m^3/sec$

Sanitary and Environmental Engineering

PART 2: WASTEWATER ENGINEERING

PART 2: WASTEWATER ENGINEERING

Lecture 3: Sewer systems

1) *Separate system:* Two pipes are used, one to carry DWF and another pipe to carry WWF.
2) *Combined system:* One pipe to carry both DWF and WWF.

DWF is the dry weather flow and WWF is the wet weather flow.

Comparison of characteristics of combined wastewater with other sources:

Parameter	unit	Rainwater	Runoff	Combined wastewater	Domestic wastewater
Total suspended solids TSS	mg/L	<1	67 - 101	270 -550	120 - 370
BOD	mg/L	1 -13	8 -10	60 -220	120 -380
COD	mg/L	9 -16	40 - 73	260 - 480	260 -900
Coliform Bacteria	MPN/100mL		10^3 -10^4	10^5 -10^6	10^5 -10^7
Nitrate	mg/L	0.05 -1.0	0.43 – 0.91		
Phosphorous	mg/L	0.02 -0.15	0.67 -1.66	1.2 -2.8	4-12
Copper Cu	µg/L		27 - 33		
Lead Pb	µg/L	0 - 70	30 -144	140 - 600	
Zinc Zn	µg/L		135 - 226		

Sewer types according to its sewage type:

1) **Sanitary sewers** carry sanitary sewage and industrial wastes after treatment produced by the community and only such ground, surface, and storm water as may enter through poor joints, around manhole covers, and through deficiencies.
2) **Storm sewers** are designed to carry the surface and storm water which runs in the same conduits.

Sewer types according to its flow type:

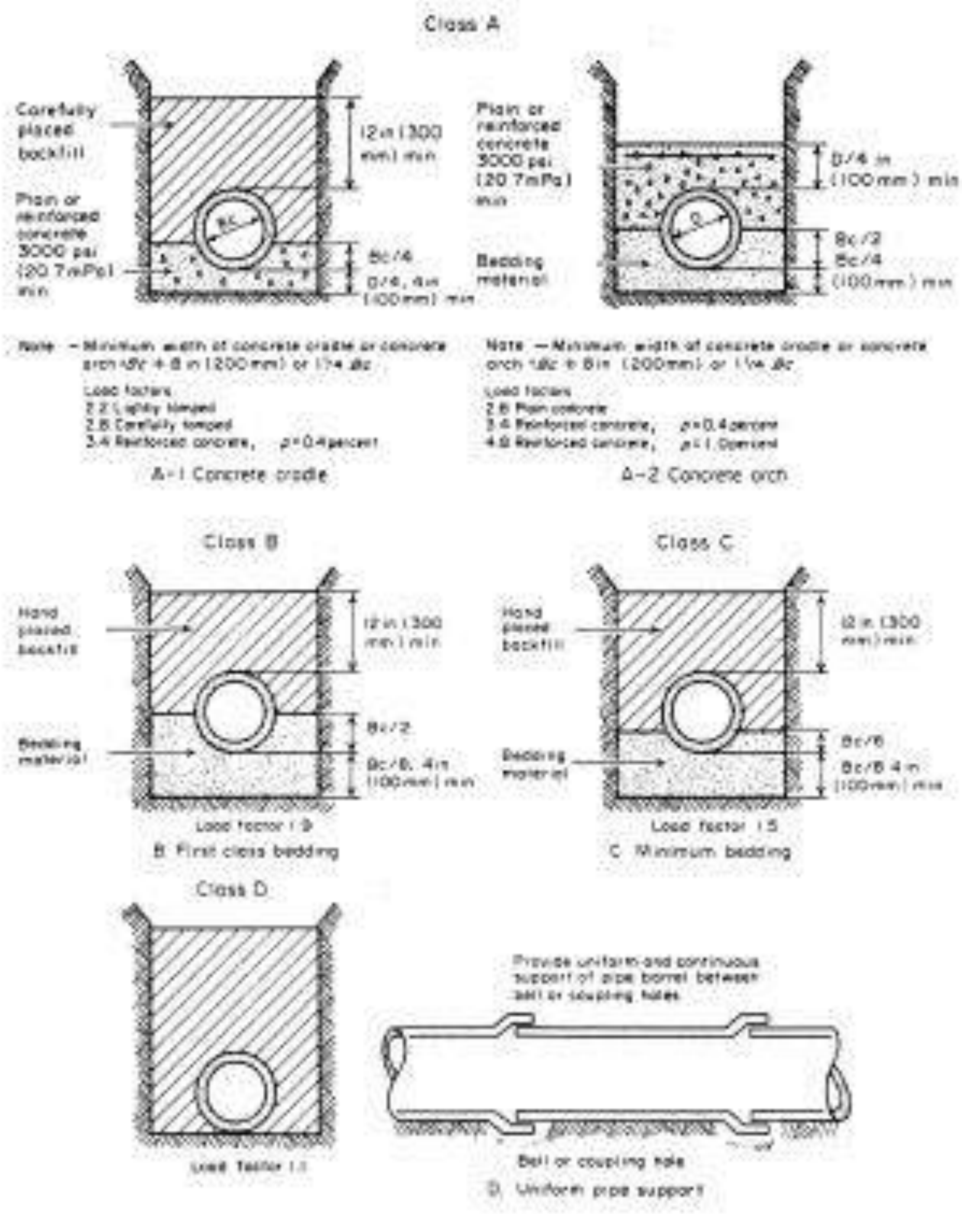
3) **Combined sewers** carry all types of sewage in the same conduits.
4) **Separate sewer:** it is said to be separate sewer if storm sewage is carried separately from the domestic and industrial wastes.

Sewer types according to its location:

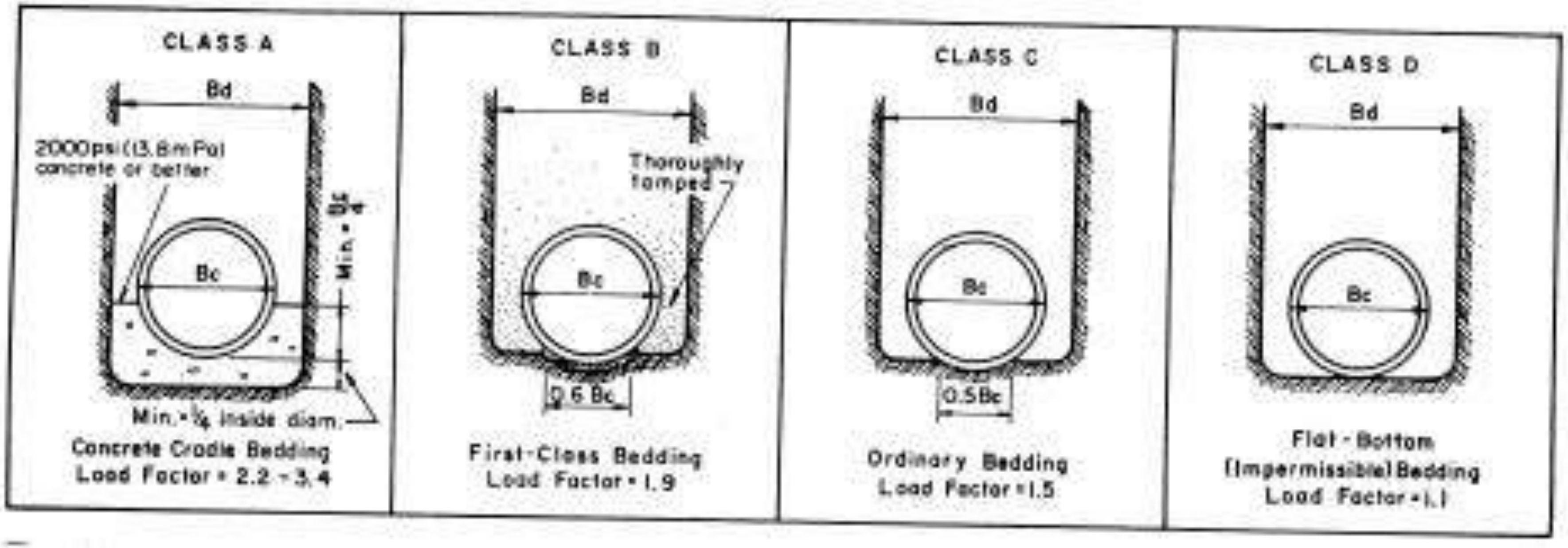
1) **House sewer** is a pipe conveying sewage from the plumbing system of a single building to a common sewer or point of immediate disposal.
2) **Lateral sewer** has no other common sewer discharging into it.
3) **Submain sewer** is one that receives the discharge of a number of lateral sewers.
4) **Main sewer** or trunk sewer receives the discharge of one or more Submain sewers.
5) **Outfall sewer** receives the discharge from the collecting system and conducts it to treatment plant or point of final disposal.
6) **Intercepting sewer** is one that cuts transversely a number of other sewers to intercept dry-weather flow, with or without a determined quantity of storm water, if used in combined system.
7) **Relief sewer** is one that has been built to relieve an existing sewer of inadequate system.

Material types of sewers:

1) Clay sewer pipe.
2) Plain concrete sewer pipe.
3) Reinforced concrete sewer pipe.
4) Asbestos cement pipe.
5) Plastic truss pipe (PI).
6) Un-plasticized polyvinylchloride pipes (uPVC).
7) Cast iron (C.I) and Ductile iron pipe (D.I).
8) Glass reinforced plastic pipe (GRP).
9) High density polyethylene pipe (HDPE)



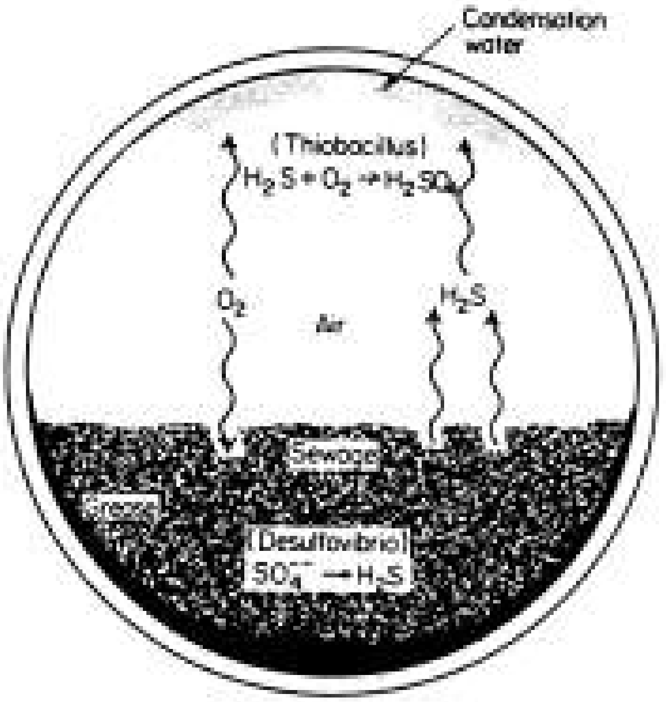
Method of pipe bedding and load factors applicable to strength.



Bedding methods for concrete pipe.

Corrosion in sewers:

Organic matter may accumulate in the sewer pipe as the result of low flowing velocity (d/D less than 0.3) or/and the accumulation of grease and oil on the surface of the wastewater. The organic matter will undergo anaerobic decomposition. This will form fatty acids (low pH) and the reduction of sulfates that will produce H_2S. The thiobacillus can oxidize H_2S into sulfuric acid H_2SO_4 that will lead to the destruction of the crown.



How to control corrosion:

1) Control the flowing velocity where d/D should be more than 0.3.
2) Ventilation.
3) Flushing the sewer system from any depositions.
4) Chlorination.
5) Lining the crown with protective material.

Flow in sewers:

Design requirement of sewer system:

1) Preliminary investigations.
2) The underground survey.
3) The survey and map.
4) Layout of the system.
5) Pipe profile.

For design the flowing velocity is according to Manning formula as the flow in this system is by gravity where the pipes are laid by appropriate slopes.

$$V_{(m/sec)}=\frac{1}{n}\times R^{\frac{2}{3}}\times S^{\frac{1}{2}}$$

V: Flowing velocity (m/sec)
R: Hydraulic radius when full flow, R= D/4 (m)
S: Slope=hydraulic gradient ($S=\frac{h_L}{L}$).
n: Roughness coefficient depending on the material of the pipe.

Material of the pipe	n
Plastic	0.009
Cement	0.01
Cast iron	0.012
Steel, Concrete, Clay	0.013
Brick	0.015

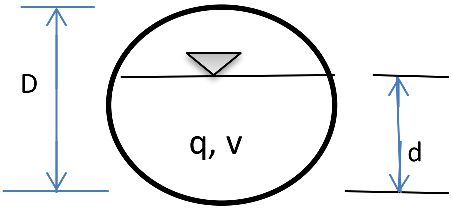
For full flow conditions

$$Q_{(m^3/sec)}=V\times A=\frac{1}{n}\times R^{\frac{2}{3}}\times S^{\frac{1}{2}}\times\frac{\pi}{4}\times D^2=\frac{1}{n}\times\left(\frac{D}{4}\right)^{0.667}\times S^{0.5}\times\frac{\pi}{4}\times D^2=\frac{0.311}{n}\times D^{2.667}\times S^{0.5}$$

For n = 0.013:

$$Q_{(m^3/sec)}=23.385\times D^{2.667}\times S^{0.5}\ \&\ V_{(m/sec)}=30.385\times D^{\frac{2}{3}}\times S^{\frac{1}{2}}$$

For partial flow



q: Actual flowing discharges.

v: Actual flowing velocity.

d: Depth of flow.

The following table is used for partial flow design

Design criteria:

v = and more than 0.6 m/sec for DWF
= and more than 0.9 m/sec for WWF
v_{max} = 2.4 m/sec
d/D = and more than 0.3

Why is the depth of flow (d) important in designing the sewer system?

Answer: to check d/D, to determine the invert drop (branch and main sewer), and also, to reduce the danger of back up.

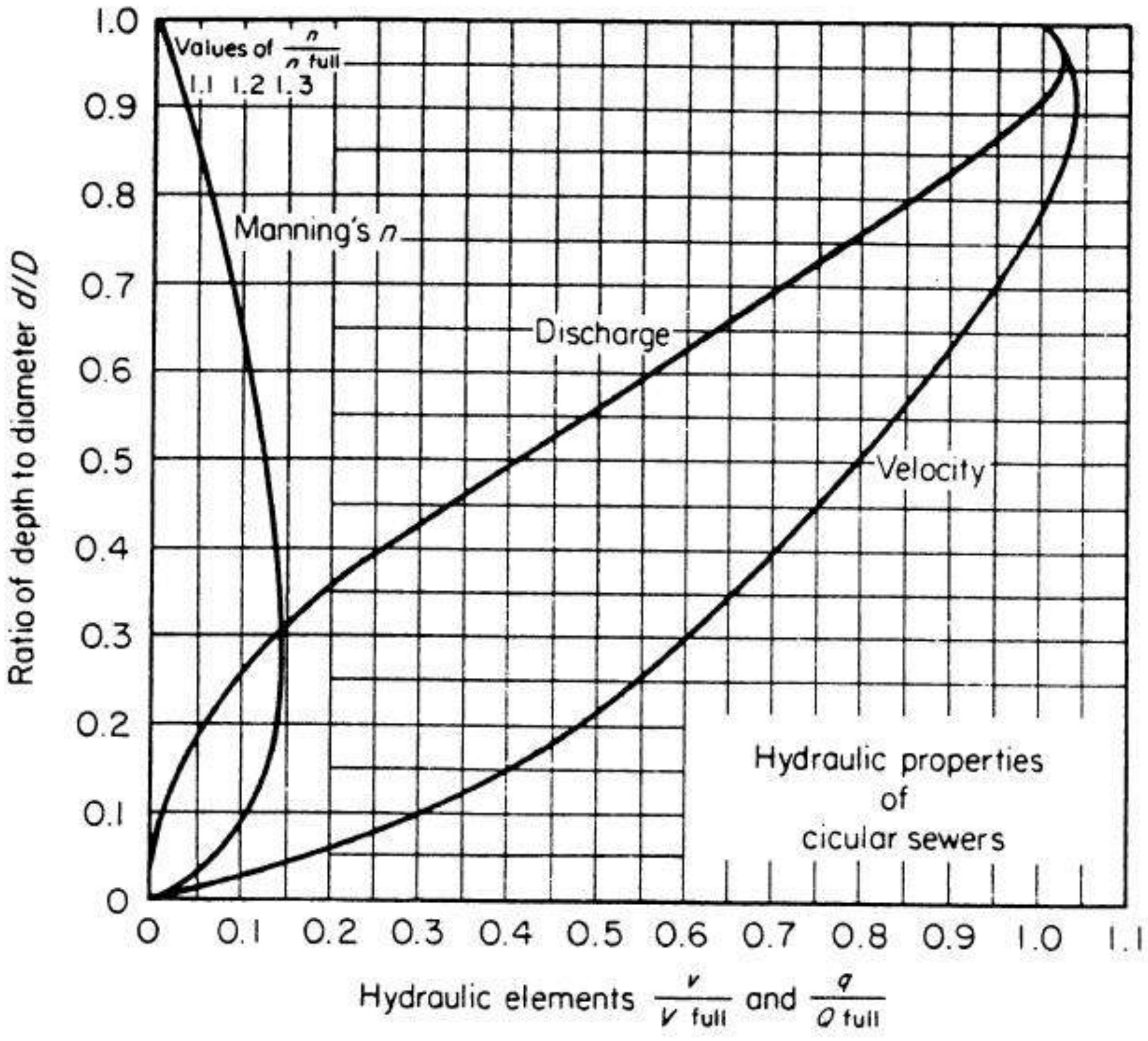


Fig. 15-5: Hydraulic elements of circular pipes.

Table Partial Flow Elements

$\frac{d}{D}$	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
$\frac{q}{Q}$	0.15	0.26	0.42	0.56	0.71	0.85	0.98	1.0
$\frac{v}{V}$	0.6	0.7	0.8	0.88	0.95	1.02	1.04	1.0

Solved problems

Problem 1: Find the diameter of the sewer to carry 3.4 m^3/min at a slope of 0.003 (assume full flow, n=0.013).
Solution:

$$Q = v \times A = \frac{1}{n} \times A \times R^{\frac{2}{3}} \times S^{\frac{1}{2}} = \frac{1}{0.013} \times \left(\frac{\pi}{4} \times D^2\right) \times \left(\frac{D}{4}\right)^{\frac{2}{3}} \times (0.003)^{\frac{1}{2}}$$

$$Q = \frac{3.4}{60}\frac{m^3}{sec} = 0.0567\frac{m^3}{sec} = 1.3125 \times D^{\frac{8}{3}} \Longrightarrow D = 0.307m\ so\ use\ D = 300\ mm$$

Problem 2: A sewer is to carry 25 m^3/min wastewater at a slope of 0.003 (assume n=0.013)
Find 1- Standard size (mm)
2- actual flowing velocity (m/sec)
3-depth of flow

Design diameter	100	150	200	250	300	400	450	500	600
Nominal diameter	110	160	210	255	305	400	450	500	630
Design diameter	700	800	900	1000	1100	1200	1400	1600	1800
Nominal diameter	710	800	900	1000	1100	1200	1400	1600	1800

Solution:

$$Q = 25\frac{m^3}{min} = 0.4167\ m^3/sec$$

$$Q = v \times A = \frac{1}{n} \times A \times R^{\frac{2}{3}} \times S^{\frac{1}{2}} = \frac{1}{0.013} \times \left(\frac{\pi}{4} \times D^2\right) \times \left(\frac{D}{4}\right)^{\frac{2}{3}} \times (0.003)^{\frac{1}{2}}$$

$$Q = 0.4167\frac{m^3}{sec} = 1.3125 \times D^{\frac{8}{3}} \Longrightarrow D = 0.650\ m$$

Check the standard size = 700 mm:

Full Flow:

$$V = \frac{1}{n} \times R^{\frac{2}{3}} \times S^{\frac{1}{2}} = \frac{1}{0.013} \times \left(\frac{0.7}{4}\right)^{\frac{2}{3}} \times (0.003)^{\frac{1}{2}} = 1.318\ m/sec$$

$$Q = V \times A = 1.318 \times \frac{\pi}{4} \times 0.7^2 = 0.507\ m^3/sec$$

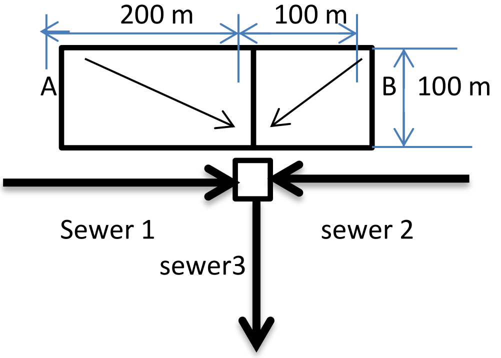
Partial Flow: for 25 m^3/min in pipe diameter 700 mm.

$$\frac{q}{O} = \frac{0.4167}{0.507} = 0.821$$

From hydraulic elements figure or table, find:

$$\frac{d}{D} = 0.77 > 0.3 \ \Longrightarrow d = 0.539\ mm\ \&\ \frac{v}{V} = 1.0 \Longrightarrow v = 1.318\frac{m}{sec} > 0.6\ m/\sec O.K$$

Problem 3: Find the diameter of sewer 3 in the figure given. Use the common intensity method as ($i = \frac{1200}{t+86}$), Assume n=0.013.



Area	**C**		**Slope**
A	0.95		0.5
B	0.55		2
Sewer	**D mm**	**d/D**	**Vfull m/sec**
1	305	0.6	2.0
2	460	0.8	1.5
3	?	0.5	1.53

Solution:
From Fig.13.3

Area	**Distance (m)**	**t_i (min)**	**t_f (min)**	**t_c (min)**
A	142	8.9	0	8.9
B	224	17	0	17 (control

$$i = \frac{1200}{t+86} = \frac{1200}{17+86} = 11.65\ mm/hr$$

Runoff discharge:

$$Q = CiA = 11.65 \times 10^{-3} \times (200 \times 100 \times 0.95 + 100 \times 100 \times 0.55) = 285.425 \frac{m^3}{hr} = 0.079\ m^3/sec$$

Full flow:

For sewer 1: $Q = V \times A = 2 \frac{m}{sec} \times \frac{\pi}{4} \times 0.3^2 = 0.1413\ m^3/sec$

For sewer 2: $Q = V \times A = 1.5 \frac{m}{sec} \times \frac{\pi}{4} \times 0.4^2 = 0.1884\ m^3/sec$

Actual flow:
From Fig. 15-5, find the hydraulic elements of sewer:

sewer 1: $\frac{d}{D} = 0.6 \Rightarrow d = 0.18\ m \Rightarrow \frac{q}{Q} = 0.56 \Rightarrow q = 0.56 \times 0.1413 = 0.0791\ m^3/sec\ \&\ \frac{v}{V} = 0.88 \Rightarrow v = 0.88 \times 2 = 1.76\ m/sec > 0.9\ m/\sec O.K$

sewer 2: $\frac{d}{D} = 0.8 \Rightarrow d = 0.8 \times 0.4 = 0.32\ m \Rightarrow \frac{q}{Q} = 0.85 \Rightarrow q = 0.85 \times 0.1884 = 0.16\ m^3/sec\ \&\ \frac{v}{V} = 1.00 \Rightarrow v = V = 1.5\ m/sec > 0.9\ m/\sec O.K$

sewer 3:

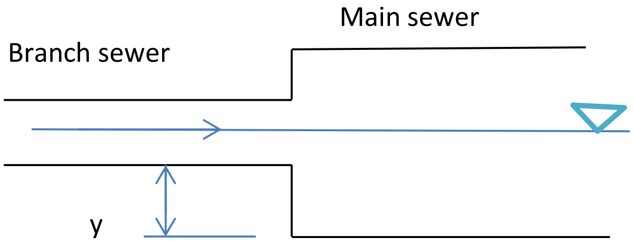
$$q_3 = Runoff + q_1 + q_2 = 0.079 + 0.0791 + 0.16 = 0.3181\ m^3/sec$$

$$\frac{d}{D} = 0.5 \Rightarrow \frac{q}{Q} = 0.4 \Rightarrow Q = \frac{0.3181}{0.4} = 0.7952\ m^3/sec\ \&\ \frac{v}{V} = 0.8 \Rightarrow v = 0.8 \times 1.53 = 1.224\ m/sec > 0.9\ m/\sec O.K$$

$$Q = V \times A \Rightarrow 0.7917 = 1.53 \times \frac{\pi}{4} \times D^2 \Rightarrow D = 0.811\ m\ use\ D = 800\ mm$$

Sewer	Q (m^3/sec)	q (m^3/sec)	D (m)	d (m)	V (m/sec)	V (m/sec)
1	0.1413	0.0791	0.3	0.18	2	1.76
2	0.1883	0.16	0.4	0.32	1.5	1.5
3	0.7952	0.3181	0.8	0.4	1.53	1.224

Problem 4: At what height (y) above the invert of the main should the branch sewer be connected?



Sewer	D (mm)	q (m³/min)	Slope
Main	2130	93.5	0.0002
Branch	760	16	0.001

Solution:

For branch sewer:

Full flow

$$V=\frac{1}{n}\times R^{\frac{2}{3}}\times S^{\frac{1}{2}}=\frac{1}{0.013}\times\left(\frac{0.7}{4}\right)^{\frac{2}{3}}\times(0.001)^{\frac{1}{2}}=0.761\,\frac{m}{sec}$$

$$Q=V\times A=0.761\times\frac{\pi}{4}\times 0.7^{2}=0.292\,\frac{m^{3}}{sec}=17.563\,m^{3}/min$$

For partial flow:

$$\frac{q}{Q}=\frac{16}{17.563}=0.911\ \&\ \frac{d}{D}=0.82\ \Longrightarrow d=0.82\times 0.7=0.574\,m\ \&\ \frac{v}{V}=1.02\Longrightarrow v=1.02\times 0.761$$

$$=0.776\,m/sec>0.6\,m/\sec O.K$$

For main sewer:

Full flow

$$V=\frac{1}{n}\times R^{\frac{2}{3}}\times S^{\frac{1}{2}}=\frac{1}{0.013}\times\left(\frac{2}{4}\right)^{\frac{2}{3}}\times(0.0002)^{\frac{1}{2}}=0.685\,\frac{m}{sec}$$

$$Q=V\times A=0.685\times\frac{\pi}{4}\times 2^{2}=2.151\,\frac{m^{3}}{sec}=129.054\,m^{3}/min$$

For partial flow:

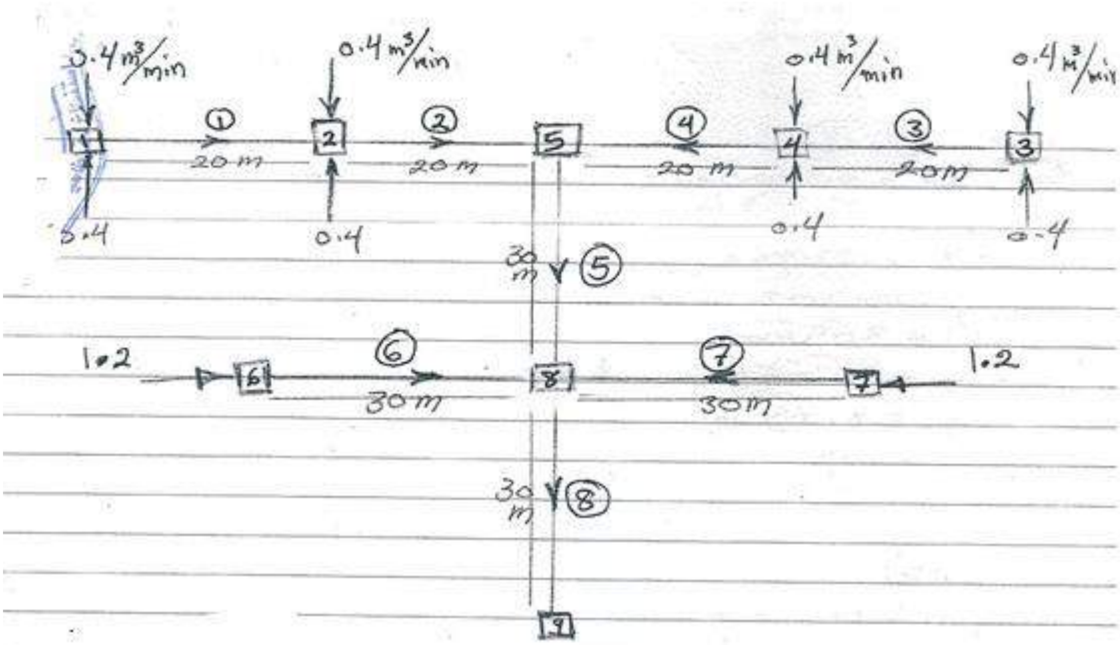
$$\frac{q}{Q}=\frac{93.5}{129.054}=0.724\ \&\ \frac{d}{D}=0.71\ \Longrightarrow d=0.71\times 2=1.42\,m\ \&\ \frac{v}{V}=0.95\Longrightarrow v=0.95\times 0.685$$

$$=0.65\,m/sec>0.6\,m/\sec O.K$$

$$y=d_m-d_b=1.42-0.574=0.846\,m$$

Problem 5: Design the sewage network with all important information supported in figure below. Assume all sewers are at slope 0.003 and roughness coefficient n = 0.13.the hydraulic elements as below:

sewer	$\frac{d}{D}$
1, 2, 3, 4	0.5
5, 6, 7	0.6
8	0.7



a) Design the sewer network?

b) Design the profile sewer (longitudinal section)? Assume invert to invert value = 25 mm.

a) Solution:

sewer	n	S	q, (m^3/min)	q, (m^3/sec)	$\frac{d}{D}$	$\frac{q}{Q}$	Q (m^3/sec)	CAL. D (m)	ASS. D (m)	V (m/sec)	Q (m^3/sec)	$\frac{q}{Q}$	$\frac{d}{D}$	d (m)	$\frac{v}{V}$	v (m/sec)
1 or 3	0.013	0.003	0.800	0.013	0.500	0.410	0.033	0.250	0.25	0.663	0.0325	0.4099	0.5	0.125	0.8	0.530
2 or 4	0.013	0.003	1.600	0.027	0.500	0.410	0.065	0.324	0.3	0.749	0.0529	0.5042	0.56	0.168	0.85	0.636
5	0.013	0.003	3.200	0.053	0.600	0.560	0.095	0.374	0.4	0.907	0.1139	0.4682	0.53	0.212	0.83	0.753
6 or 7	0.013	0.003	1.200	0.020	0.600	0.560	0.036	0.259	0.25	0.663	0.0325	0.6149	0.65	0.1625	0.91	0.603
8	0.013	0.003	5.600	0.093	0.700	0.710	0.131	0.422	0.4	0.907	0.1139	0.8193	0.77	0.308	1	0.907

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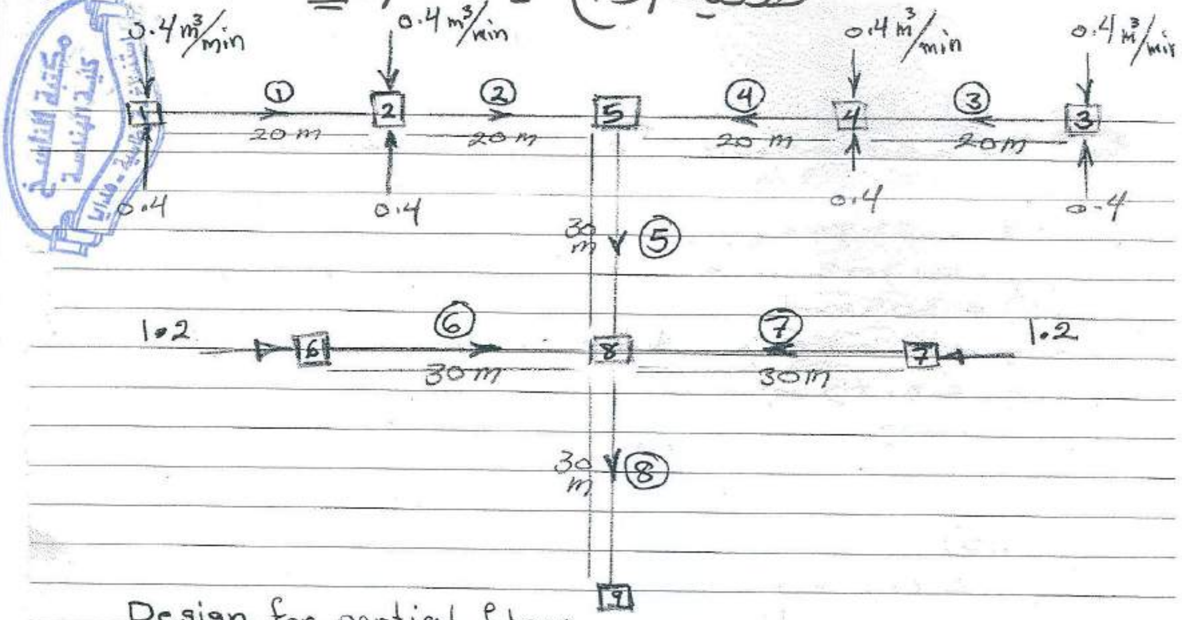


Design for partial flow

sewer	d/D
1,2,3,4	0.5
5,6,7	0.6
8	0.7

All sewers are at $S = 0.003$

assume $n = 0.013$

sewer ①, ③

$q_1 = q_3 = 0.4 + 0.4 = 0.8\ m^3/min = 0.013\ m^3/sec$

for $d/D = 0.5$ $\therefore q/Q = 0.5$ $\therefore Q = 0.026\ m^3/sec$

$0.026 = 23.846\,(D)^{2.67}\,(0.003)^{0.5}$

$\therefore D = 0.233\,m = 233\ mm$ Standard size 200, 25

for $\boxed{D = 200mm}$

$Q_{200} = 23.846\,(0.2)^{2.67}\,(0.003)^{0.5} = 0.018\ m^3/sec$

$V_{200} = 30.385\,(0.2)^{2.67}\,(0.003)^{0.5} = 0.566\ m/sec$

OR $= 4 * 0.018 / \pi * (0.2)^2 = 0.566\ m/sec$

$\therefore q/Q = 0.013/0.018 = 0.72$ $\therefore d/D = 0.64$

$v/V = 1.07$ $\therefore v = 1.07 * 0.566 = 0.61\ m/se$

for $D = 250\ mm$

$Q_{250} = 0.032\ m^3/sec$, $V_{250} = 0.658\ m/sec$

$\therefore q/Q = 0.013/0.032 = 0.41$ $\therefore d/D = 0.44$

$v/V = 0.94$ $\therefore v = 0.94 * 0.658 = 0.62\ m/sec$

1

sewer ②, ④

$q_2 = q_4 = 0.8 + 0.4 + 0.4 = 1.6\ m^3/min = 0.027\ m^3/sec$

for $d/D = 0.5$ $\therefore q/Q = 0.5$ $\therefore Q = 0.054\ m^3/sec$

$0.054 = 23.846\,(D)^{2.67}\,(0.003)^{0.5}$

$\therefore D = 0.302 = 302\ mm$ standard 305 mm

for $\boxed{D = 305\ mm}$

$Q_{305} = 0.055\ m^3/s$ $\therefore q/Q = 0.027/0.055 = 0.49$

$V_{305} = 0.75\ m/s$ $\simeq 0.5$

$\therefore d/D = 0.5$ $v/V = 1$ $\therefore v = 1 * 0.75 = 0.75\ m/sec$

sewer ⑤

$q_5 = 1.6 + 1.6 = 3.2\ m^3/min = 0.053\ m^3/sec$

for $d/D = 0.6$ $\therefore q/Q = 0.67$ $\therefore Q = 0.08\ m^3/sec$

$0.08 = 23.846\,(D)^{2.67}\,(0.003)^{0.5}$

$\therefore D = 0.351\ m = 351\ mm$ Standard 305, 380

for $D = 305\ mm$

$Q_{305} = 0.055\ m^3/s$ $q/Q = 0.053/0.055 = 0.96$ not o.k

for $\boxed{D = 380\ mm}$

$Q_{380} = 0.099\ m^3/sec$ $\therefore q/Q = 0.053/0.099 = 0.54$

$\therefore d/D = 0.53$ O.K

$V_{380} = 0.87\ m/s$ $v/V = 1.02$

$\therefore v = 1.02 * 0.87 = 0.89\ m/sec$

$\therefore D = 380\ mm$

sewer ⑥, ⑦

$q_6 = q_7 = 1.2\ m^3/min = 0.02\ m^3/sec$

for $d/D = 0.6$ $\therefore q/Q = 0.67$ $\therefore Q = 0.03\ m^3/sec$

$0.03 = 23.846\ D^{2.67}\,(0.003)^{0.5}$

$\therefore D = 0.243\ m = 243\ mm$ standard 200, 250

for $D = 200\ mm$

$Q_{200} = 0.018\ m^3/s$ $q/Q = 0.02/0.018 = 1.1$ overflow

$V_{200} =$.

$Q_{250} = 0.032\ m^3/s$ $\therefore q/Q = 0.02/0.032 = 0.63$

$V_{250} = 0.658\ m/s$ $\therefore d/D = 0.58$ o.K

$v/V = 1.05$ $v = 1.05 \times 0.658 = 0.687\ m/sec$

$\therefore D = 250\ mm$

Sewer ⑧

$q_8 = 3.2 + 1.2 + 1.2 = 5.6\ m^3/min = 0.093\ m^3/sec$

for $d/D = 0.7$ $\therefore q/Q = 0.82$ $\therefore Q = 0.113\ m^3/sec$

$0.113 = 23.846\ (D)^{2.67}\ (0.003)^{0.5}$

$\therefore D = 0.401\ m = 401\ mm$ standard 380, 460

for $D = 380\ mm$

$Q_{380} = 0.099\ m^3/sec$ $q/Q = 0.093/0.099 = 0.94$ X

for $\boxed{D = 460\ mm}$

$Q_{460} = 0.164\ m^3/s$ $\therefore q/Q = 0.093/0.164 = 0.57$

$V_{460} = 0.99\ m/s$ $\therefore d/D = 0.53$ ok $v/V = 1.02$

$\therefore D = 460\ mm$ $\therefore v = 0.99 \times 1.02$

$= 1.01\ m/s$ o.k

-3-

Profile invert to invert (25 mm = 0.025 m)

Sewer 1,3

at MH 1,3 depth = 0.5 m

drop = length * slope

= 20 m * 0.003 = 0.06 m

at MH 2,4 depth = 0.5 + 0.06 = 0.56 m

Sewer 2,4

at MH 2,4 depth = 0.56 + 0.025 = 0.585 m

drop = 20 m * 0.003 = 0.06

at MH 5 depth = 0.585 + 0.06 = 0.645 m

Sewer 5

at MH 5 depth = 0.645 + 0.025 = 0.67 m

drop = 30 m * 0.003 = 0.09

at MH 8 depth = 0.67 + 0.09 = 0.76 m

Sewer 6,7

at MH 6,7 depth = 0.5 m

drop = 30 m * 0.003 = 0.09 m

at MH 8 depth = 0.09 + 0.5 = 0.59 m

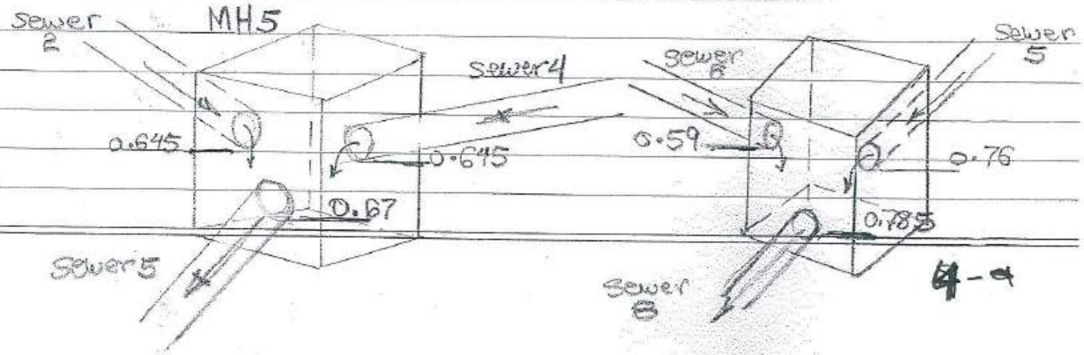
(above 0.76 m o.k)

Sewer 8

at MH 8 depth = 0.76 + 0.025 = 0.785 m

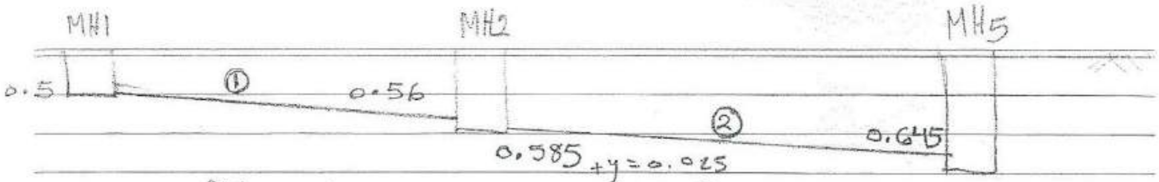
drop = 30 m * 0.003 = 0.09 m

at MH 9 depth = 0.785 + 0.09 = 0.875 m

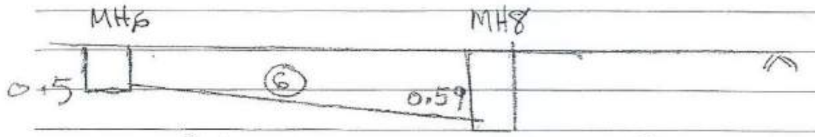


4-4

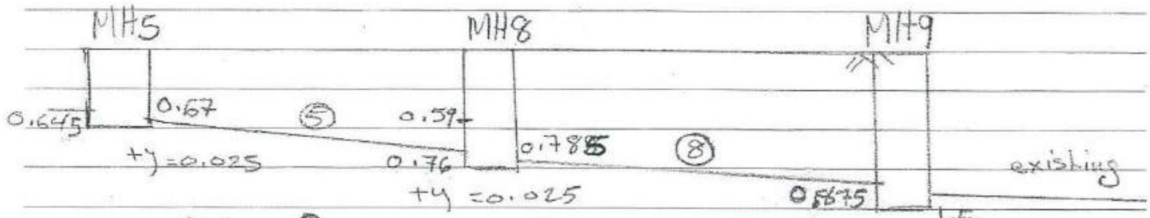
For invert to invert 25mm



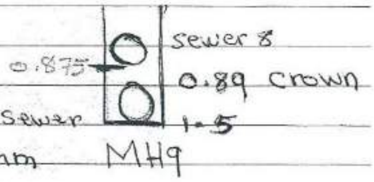
profile A sewer ①, ② ③, ④



profile B sewer ⑥ , ⑦



profile C sewer ⑤, ⑧, existing



1.5 − 0.61 = 0.89 m
elevation of the crown existing sewer
610mm

invert of sewer 8 at 0.875m
abover the crown of existing sewer 0.89 m

Profile checking back up

Sewer	1	and 2
D mm	200	305
d/D	0.64	0.5
d	128	152.5
y	24.5 mm ≈ 0.025 m	

Sewer 1,3

at MH 1,3 depth = 0.5 m drop = 0.06m

at MH 2,4 depth = 0.5 + 0.06 = 0.56m

Sewer 2,4

at MH 2,4 depth = 0.56 + 0.025 = 0.585 m

at MH 5 drop = 0.06 depth = 0.585 + 0.06 = 0.64

Sewer	2	and 5
D mm	305	380
d/D	0.5	0.53
d	152.5	201.4
y	48.9 mm ≈ 0.049 m	

Sewer 5

at MH5 depth 0.645 + 0.049 = 0.694

at MH 8 drop = 0.09 depth = 0.694 + 0.09 = 0.784

Sewer 6,7

at MH 6,7 depth = 0.5 m

at MH 8 drop = 0.09 depth = 0.5 + 0.09 = 0.5

Sewer	6	and 8
D mm	250	460
d/D	0.58	0.53
d	145	243.8
y	98.8 mm ≈ 0.099 m	

5 [illegible]-9

sewer	5	and 8
D mm	380	460
d/D	0.53	0.53
d	201.4	243.8
y	42.4 mm ~ 0.042 m	

Sewer 8

at MH 8 depth 0.784 + 0.042 = 0.826 m

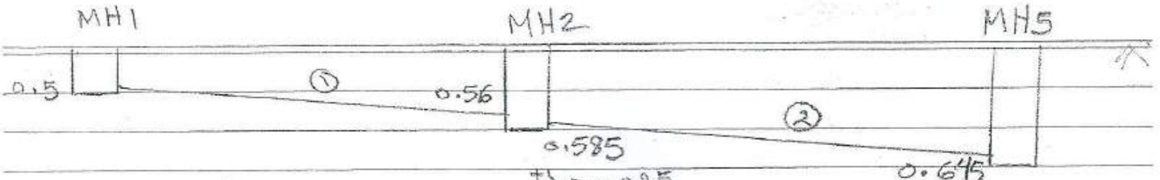
at MH9 drop = 0.09 ∴ depth = 0.826 + 0.09 = <u>0.916 m</u>

check y from sewer 6,7

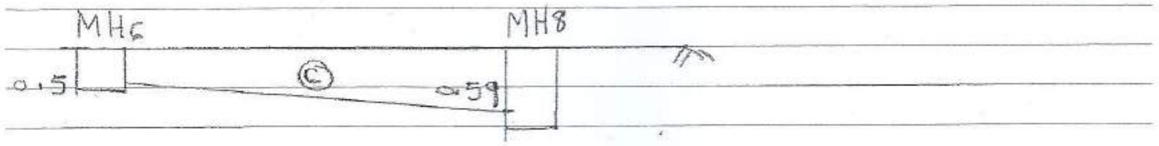
at MH8 depth = 0.590 + 0.099 = 0.689 m

MH9 drop = 0.09 ∴ depth = 0.689 + 0.09 = 0.779 m

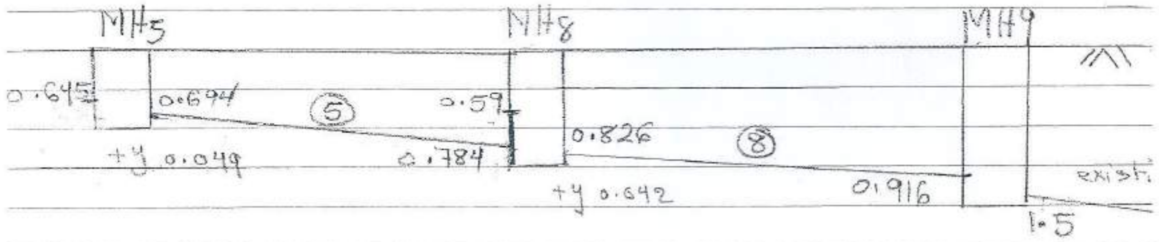
Check back up



profile A

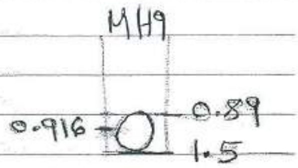


profile B



profile C

For sewer ⑧



correct the drop = 0.89 − 0.826 = 0.064

$\therefore$ slope for ⑧ $= \frac{0.064}{30} = 0.0021$

1.5 = 0.61 = 0.8

$$Q_{460} = 23.846(0.46)^{2.67}(0.0021)^{0.5}$$
$$= 0.138\ m^3/sec$$

$$q/Q = \frac{0.093}{0.138} = 0.67 \quad \therefore d/D = 0.6, \quad v/V = 1.05$$

$$V_{460} = \frac{0.138 \times 4}{\pi(0.46)^2} = 0.83\ m/s \quad \therefore v = 0.872\ m/sec$$

Sanitary and Environmental Engineering

PART 2: WASTEWATER ENGINEERING

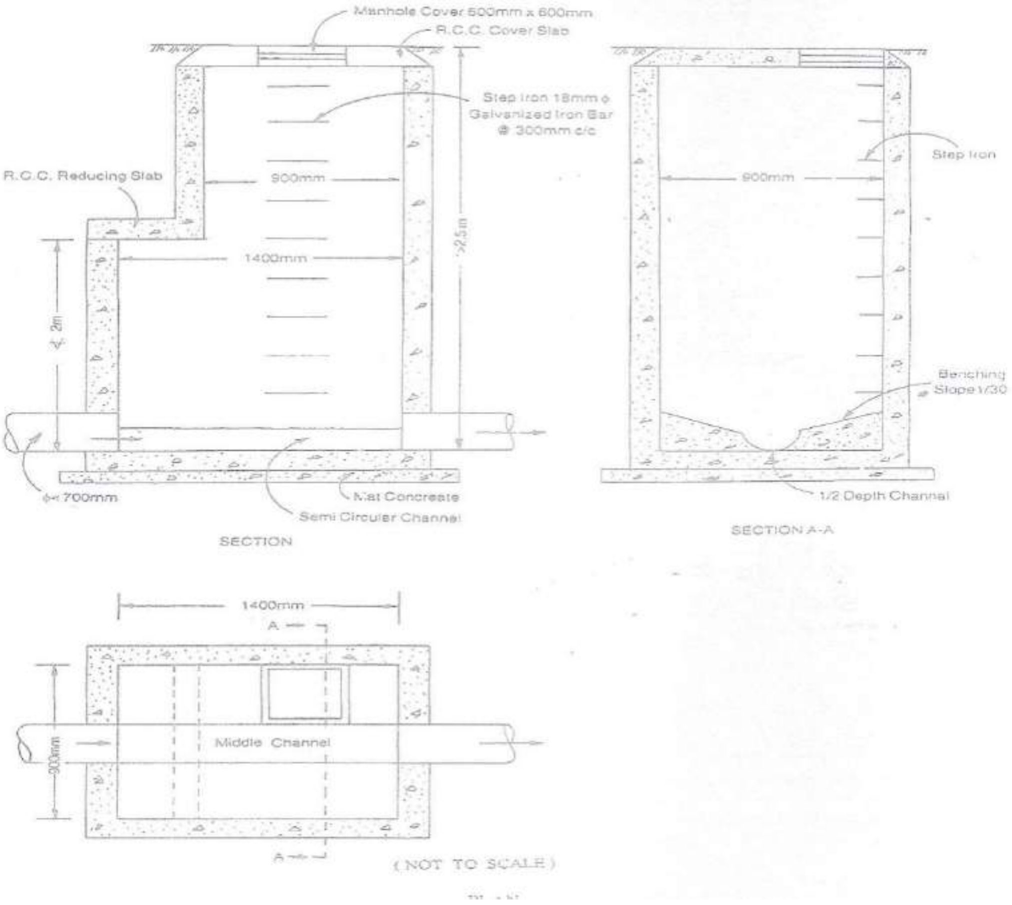
PART 2: WASTEWATER ENGINEERING

Lecture 4: Appurtenances of the sewer system

1) **Manhole (MH):** A chamber for inspection, cleaning and maintenance. It should be located at:
   a) Change in sewer direction;
   b) Change in sewer diameter;
   c) Change in sewer slope;
   d) Has to be located at interval of 50 – 90 m apart.

   Types of manholes:
   a) Shape: Rectangular, Circular.
   a) Material of construction: Brick, Concrete and Plastic.
   b) Size according to depth specifications:

Depth	Size
Up to 0.5 m	0.45 x 0.45 m
0.5 – 1.0 m	0.6 x 0.6 m
1.0 – 2.0 m	0.6 x 0.75 m
2.0 – 3.0 m	0.9 x 1.2 m or 1.1 – 1.2 m in diameter
More than 3.0 m	1.8 m in diameter



2) **Street inlets:** Openings on the street surface, through which runoff water (WWF) is admitted and conveyed to a storm or combined sewer system. Common types: a-Curb, Vertical openings b-Gutter, Horizontal openings.

Sewerage System

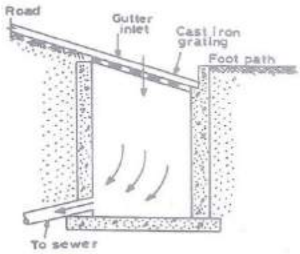


Fig. 3.11. Gutter Inlet

3.7.1.5. Catch Basin

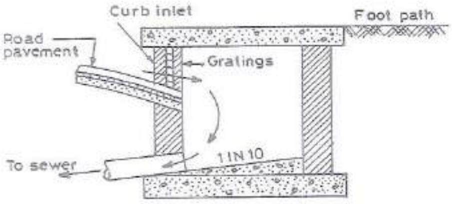


Fig. 3.10. Curb inlet

3) **Catch basin:** Small settling chamber constructed below the street inlet, to allow grit, sand and large solids to settle from the flowing wastewater.

out from the sewers and to retain floating matter.

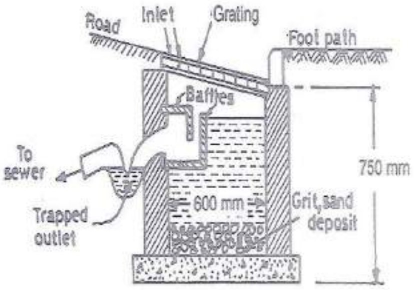


Fig. 3.12. Catch Basin

4) **Oil, grease and sand trap:** Tank or basin used to remove oil, grease, sand, mud…etc., from the sewage of industries, hotels, restaurants, kitchens and car garages. These materials are harmful and may cause clogging in the sewer system.

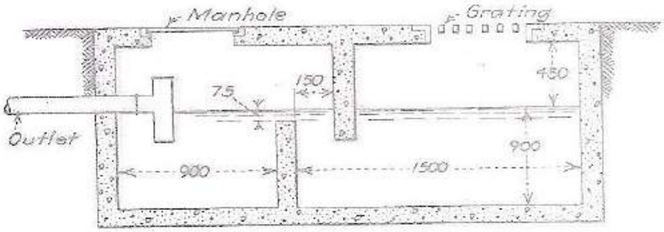
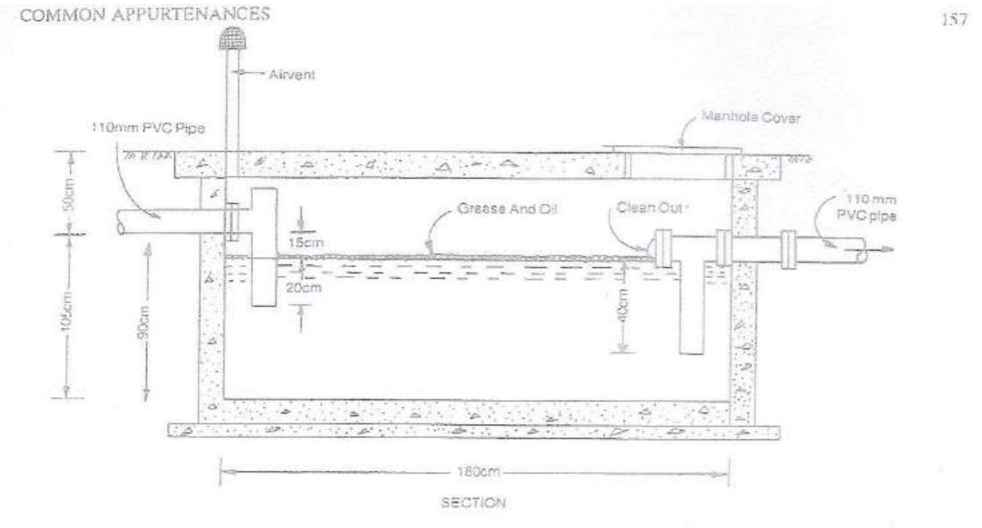
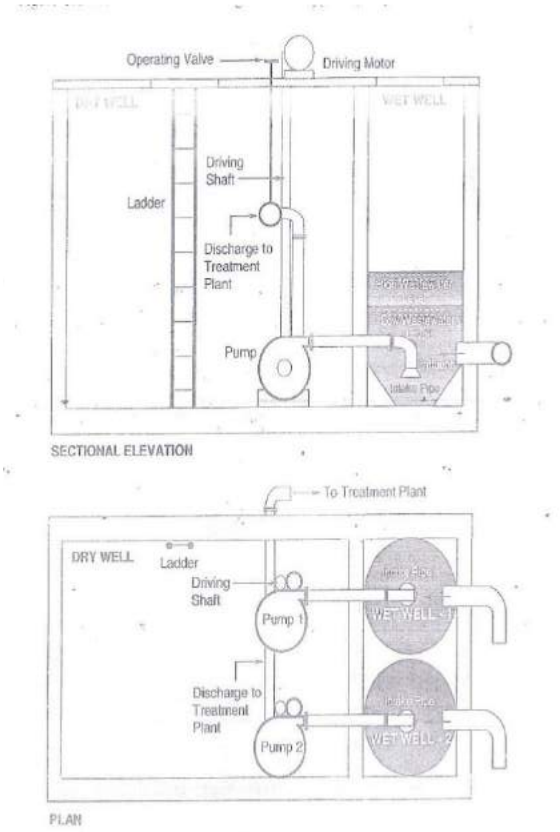


Figure 16-12 Sand and oil trap for garages and car washes. (Dimensions in mm.)



5) **Pumping station:** This station is to lift the level of the flowing wastewater in the sewer system from low elevations to adequate heights. It is constructed of two parts, the wet well (pit) to receive the flowing wastewater, and the dry pit for the installation of the pumps. The size of the wet pit should be small to avoid anaerobic decomposition. The station should be provided with good ventilation.



igure 5.2 Schematic diagram of a typical sewage pump house with wet and dry wells.

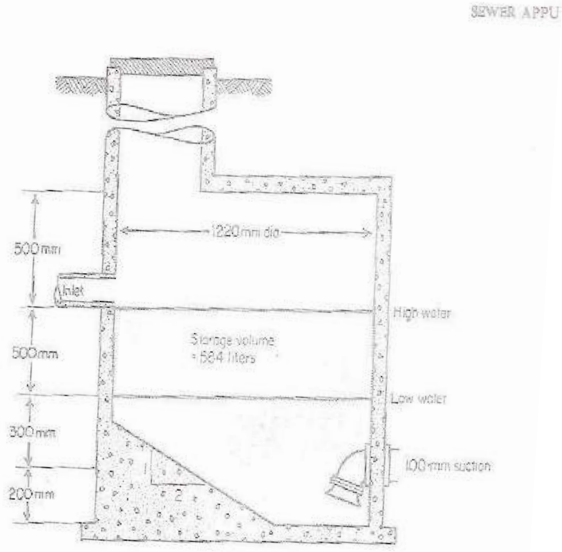


Figure 16-20 Wet pit details.

Sewage pumps station requirements:

a) Self-cleansing velocities = 0.6 m/sec.

b) In small pump stations the pump is sized to meet peak flow and lower flows accumulate in the wet well until sufficient liquid is present to permit the pump to run for at least (running time, t_r = 2min) for sewage flow and (t_r = 4min) for storm flow.

c) In standard practice, the pump will not start than ones in (filling time, t_f = 5min). Peak Flow

$$\text{Peak Flow } D = \frac{Maximum\ Wastewater\ Flow\ (L/day)}{60 \times 24}$$

d) The pump running time: $t_r = \dfrac{V}{D-Q}$, Where:

V: Storage Volume;

D: Pump Discharge;

Q: Influent Flow (Average Wastewater Flow).

e) The filling time, with the pump off: $t_f = \dfrac{V}{Q}$

f) The total cycle time: $t_c = t_r + t_f = \dfrac{V}{D-Q} + \dfrac{V}{Q}$

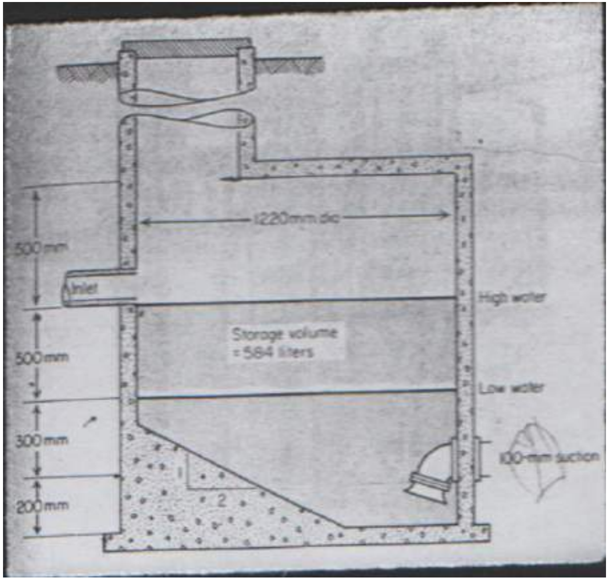
g) Minimum sewage volume when (t_r = 2min), $V = t_r \times D = 2D$ **and** Minimum storm volume when (t_r = 4min), $V = t_r \times D = 4D$

h) The Volume When (t_f = 5min) $\dfrac{V}{D-Q} + \dfrac{V}{Q} = 5\,\text{min}$, so take the volume covers the starting and stopping elevation which is large = V_{Actual}.

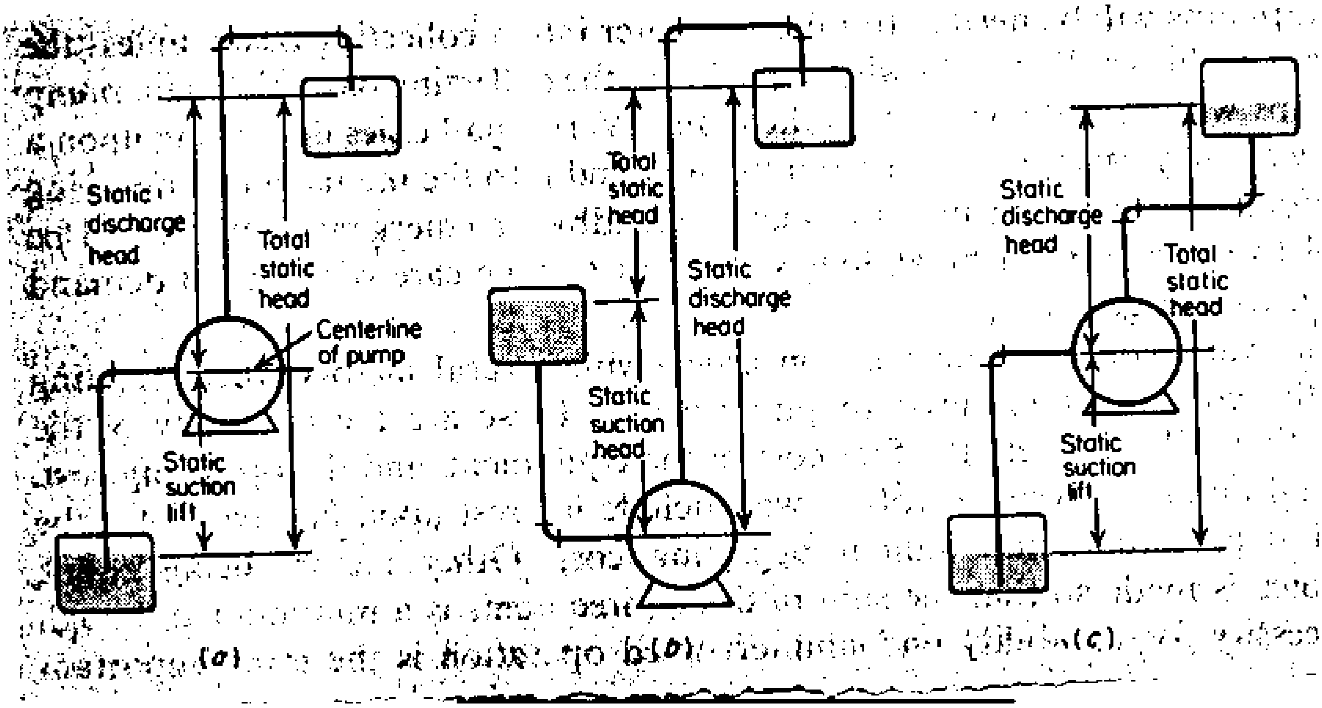
i) The actual cycle time at average flow: $t = \dfrac{V_{actual}}{D-Q} + \dfrac{V_{actual}}{Q}$

j) Assume D = (1.5-2) Q

Total Depth = Pump Suction (200mm) + Submergence (300mm) + Free Board Level (600mm)



Wet pit details.



$$Total\ head\ (H) = Static\ head + Dynamic\ head + Pressure\ required + Total\ head\ loss$$

$$Motor\ power\ (kW) = \frac{\gamma_{(kN/m^3)} \times Q_{(m^3/sec)} \times H_{(m)}}{Overall\ efficiency},$$

$$Motor\ power\ (h_p) = \frac{\gamma_{(kN/m^3)} \times Q_{(m^3/sec)} \times H_{(m)}}{Overall\ efficiency \times 0.7457}$$

Solved problems

Problem 1: Find the volume of the wet well in a pumping station. Assume the pump stops 10 times/hr.
Solution:

$$Total\ cycle\ time = Running\ time + Filling\ time = \frac{V}{D-Q} + \frac{V}{Q}$$

Assume D = 2Q, so, $Total\ cycle\ time = \frac{60\ min}{10\ times/hr} = 6\ min$

$$Total\ cycle\ time = 6 = \frac{V}{2Q-Q} + \frac{V}{Q} = \frac{2V}{Q} \Rightarrow V = \frac{6Q}{2} = 3Q$$

Problem 2: A small subdivision produces an average wastewater flow of 120,000 L/day, the minimum hourly flow is estimated to be 15,000 L/day, and the maximum, 420,000 L/day. Determine the design pumping rate and wet well capacity? Also calculate the pump pipe diameter? Assume the average sewage velocity limitation (1-3) m/sec, use average sewage velocity 1.5 m/sec.

Solution:

The minimum pump delivers: $D_{min} = \frac{15000}{1440} = 10.416\ L/min$

The maximum pump delivers: $D_{max} = \frac{420000}{1440} = 292\ L/min$

The average flow: $Q = \frac{120000}{1440} = 83.333\ L/min$

The pump running time, $t_r = \frac{V}{D-Q}$, the filling time, $t_f = \frac{V}{Q}$, and $t_c = \frac{V}{D-Q} + \frac{V}{Q}$

To assure 2 min running time, the minimum volume: $V = t_r \times D = 2min \times 292 \frac{L}{min} = 548\ L$

To assure a 5 min cycle, at average flow:

$$t_c = \frac{V}{D-Q} + \frac{V}{Q} \Rightarrow 5 = \frac{V}{292-83.33} + \frac{V}{83.333} = \frac{V}{208.667} + \frac{V}{83.333} = \frac{292V}{17388.847} \Rightarrow V = 297.754\ L \approx 298\ L$$

Therefore, the running time governs, the volume between starting and stopping elevations will be about 600 L (approximate volume).

The actual cycle time at average flow:

$$t_c = t_r + t_f = \frac{600}{292-83.333} + \frac{600}{83.333} = 2.875 + 7.2 = 10.075\ min$$

The minimum depth of water below the suction pipe of about 200 mm.

At an intake velocity 0.6 m/sec a submergence of about 300 mm.

Freeboard about anticipated maximum water level = 600 mm

Total minimum depth of the pump station: $= 200mm + 300mm + 500mm + 600mm = 1600mm$

$$Q = v \times A \Rightarrow \frac{420}{24 \times 3600} = 1.5 \times \frac{\pi}{4} \times D_{PUMP}^2 \Rightarrow D = 0.064\ m$$

If use, $D = 50\ mm\ with\ average\ velocity = 2.47\ m/\sec O.K$

So, use, $D = 65\ mm\ with\ average\ velocity = 1.46\ m/\sec O.K$

Problem 3: Determine the total head and the power of the sewage pump according to the information (Sewage flow =100 m^3/hr, static head =15.4 m, total head losses = 7 m, pressure required at the end of the pipe =15 m, dynamic head = 10 m, and overall efficiency of the pump = 64%)? Also calculate the pump pipe diameter? Assume the average sewage velocity limitation (1-3) m/sec, use average sewage velocity 2 m/sec.

Solution:

$$Q = \frac{100\ m^3/hr}{3600} = 0.0278\ m^3/sec$$

$$Total\ head\ (H) = 15.4 + 10 + 15 + 7 = 47.4\ m$$

$$Motor\ power\ = \frac{9.81\frac{kN}{m^3}\times\frac{0.0278m^3}{sec}\times 47.4m}{0.64} = 20.198\ kW \approx 27\ hp,$$

$$Q = v \times A \Longrightarrow \frac{100}{3600} = 2\ \times \frac{\pi}{4} \times D_{PUMP}^2 \Longrightarrow D = 0.133\ m$$

So, $D = 150\ mm\ with\ average\ velocity = 1.57\ m/\sec O.K$

Sanitary and Environmental Engineering

PART 2: WASTEWATER ENGINEERING

PART 2: WASTEWATER ENGINEERING

Lecture 5: Sewage Disposal

Sewage disposal to water bodies such as rivers will be described by Streeter-Phelps equation. This equation calculates the amount of the oxygen deficit in the river due to the degradation of the organic matter that is dumped into it. This equation describes a process known as the self-purification process, where the river can deal with certain amounts (concentrations) of organic matter that are aerobically decomposed and not cause pollution. This equation can give the following information:

1) The critical oxygen deficit (minimum concentration of the dissolved oxygen) in the river.
2) The allowable amount (concentration) of organic matter to be dumped into the river.
3) The location downstream for other waste disposal sites.
4) Drawing the variation of the dissolved oxygen with time or distance after the disposal of the sewage into the river. This curve is known as the oxygen sag curve.

Application of formulas:

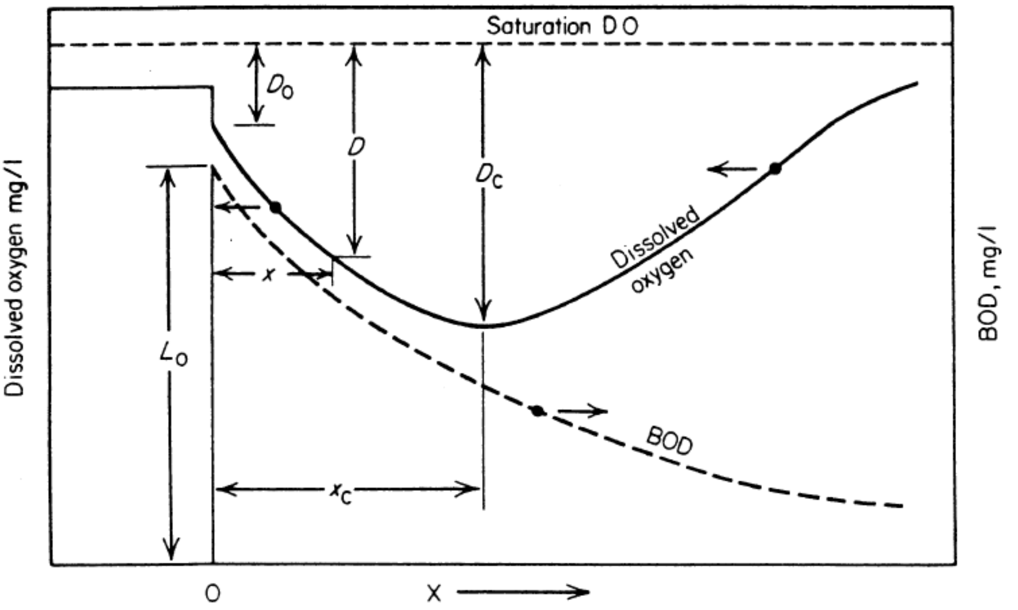
The Streeter-Phelps equation may be used to predict:

1) The effect of a particular waste load upon a receiving stream;
2) Determine the waste load which can be discharged without reducing the dissolved oxygen below some prescribed level.

Effect of stream discharge:

1) Organic matter which enters the stream is broken down by bacteria to ammonia, nitrates, sulfates, carbon dioxide, etc., which are utilized by plants and algae to produce carbohydrates and oxygen.
2) Excessive quantities of waste materials can upset the cycle by causing rapid bacterial growth and resulting depletion of dissolved oxygen in the stream.
3) Self-purification process defines as the stabilization of the excess organic matter through the normal cycle will be re-established.
4) Water quality standards are often based upon maintenance of some minimum dissolved oxygen concentration which will protect the natural cycle in the stream while taking advantage of its natural assimilative capacity.

Streeter-Phelps mathematical model:



Oxygen sag curve and BOD removal

$$D_t = \frac{K_1 L}{K_2 - K_1}\left(e^{-K_1 t} - e^{-K_2 t}\right) + D_i e^{-K_2 t}$$

$$D_c = \frac{K_1 L e^{-K_1 t_c}}{K_2}$$

$$t_c = \frac{1}{K_2 - K_1} ln\left[\frac{K_2}{K_1}\left\{1 - \frac{(K_2 - K_1)D_i}{K_1 L}\right\}\right]$$

D = Oxygen deficit (mg/L) at any time or distance after sewage disposal.

K_1 = Deoxygenation rate (day $^{-1}$) at Temperature of the mix ($K_{1T} = K_{120} \times (1.047)^{T-20}$).

K_2 = Aeration rate (day $^{-1}$) at Temperature of the mix ($K_{2T} = K_{220} \times (1.025)^{T-20}$).

L = Ultimate BOD (mg/L) of the mix.

t = Time for degradation (day).

D_i = Initial oxygen deficit (mg/L).

D_c = Critical oxygen deficit (mg/L).

t_c = Critical time for the critical oxygen deficit (day).

Characteristics of the mix:

$$Q_{Mix} = Q_r + Q_w$$

$$BOD_{Mix} = \frac{BOD_r \times Q_r + BOD_w \times Q_w}{Q_{Mix}}$$

$$L_{Mix} = \frac{L_r \times Q_r + L_w \times Q_w}{Q_{Mix}}$$

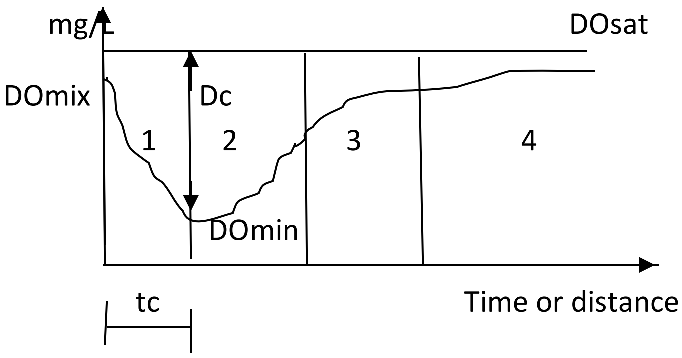
$$DO_{Mix} = \frac{DO_r \times Q_r + DO_w \times Q_w}{Q_{Mix}}$$

$$T_{Mix} = \frac{T_r \times Q_r + T_w \times Q_w}{Q_{Mix}}$$

r = river, w = wastewater (sewage), Q = discharge flow, DO = dissolved oxygen, T = temperature, BOD = concentration of organic matter.

Zones of the self-purification process

1- Zone of degradation
2- Zone active decomposition
3- Zone of recovery
4- Zone of clear water



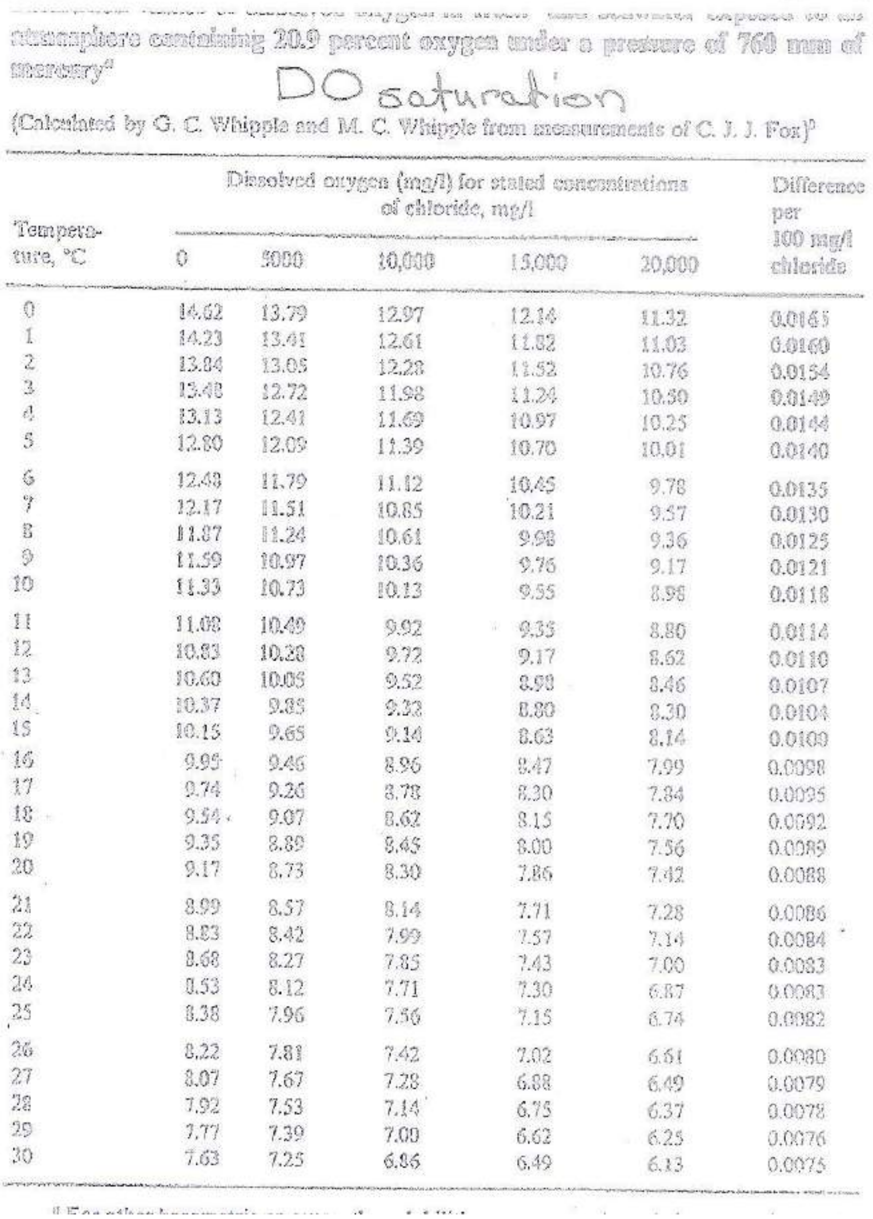
Factors affecting the self-purification process:

1- *Dilution* with a large amount containing substantial dissolved oxygen will reduce of significant pollution effects.
2- *Currents* assist in dispersion of the wastewater in the stream, preventing locally high concentrations of pollutants.
3- Sedimentation, suspended solids, which may contribute to oxygen demand, are removed by settling if the stream velocity is less than the scour velocity of the particles.
4- Bottom deposits and runoff: organic material which has settled or been adsorbed may be re-released or the products of its decomposition may be released.
5- Sunlight acts as a disinfectant and stimulates the growth of algae. The algae produce oxygen during daylight, but utilize oxygen at night. Waters containing heavy algal growths may be supersaturated with dissolved oxygen during daylight hours and be anaerobic at night.

6- *Temperature* affects the solubility of oxygen in water, the rate of bacterial action, and the rate of re-aeration. The critical condition is generally in warm weather when utilization rates are high and availability.

The effect of K_1 and K_2 on the self- purification process

1) Increasing K_1 will increase D_c and displace D_c upstream. Decreasing K_1 will decrease D_c and displace D_c downstream.
2) Increasing K_2 will decrease D_c and displace Dc upstream. Decreasing K_2 will increase Dc and displace D_c downstream.



atmosphere containing 20.9 percent oxygen under a pressure of 760 mm of mercury[a]

DO saturation

(Calculated by G. C. Whipple and M. C. Whipple from measurements of C. J. J. Fox)[b]

Temperature, °C	Dissolved oxygen (mg/l) for stated concentrations of chloride, mg/l					Difference per 100 mg/l chloride
	0	5000	10,000	15,000	20,000	
0	14.62	13.79	12.97	12.14	11.32	0.0165
1	14.23	13.41	12.61	11.82	11.03	0.0160
2	13.84	13.05	12.28	11.52	10.76	0.0154
3	13.48	12.72	11.98	11.24	10.50	0.0149
4	13.13	12.41	11.69	10.97	10.25	0.0144
5	12.80	12.09	11.39	10.70	10.01	0.0140
6	12.48	11.79	11.12	10.45	9.78	0.0135
7	12.17	11.51	10.85	10.21	9.57	0.0130
8	11.87	11.24	10.61	9.98	9.36	0.0125
9	11.59	10.97	10.36	9.76	9.17	0.0121
10	11.33	10.73	10.13	9.55	8.98	0.0118
11	11.08	10.49	9.92	9.35	8.80	0.0114
12	10.83	10.28	9.72	9.17	8.62	0.0110
13	10.60	10.05	9.52	8.98	8.46	0.0107
14	10.37	9.85	9.32	8.80	8.30	0.0104
15	10.15	9.65	9.14	8.63	8.14	0.0100
16	9.95	9.46	8.96	8.47	7.99	0.0098
17	9.74	9.26	8.78	8.30	7.84	0.0095
18	9.54	9.07	8.62	8.15	7.70	0.0092
19	9.35	8.89	8.45	8.00	7.56	0.0089
20	9.17	8.73	8.30	7.86	7.42	0.0088
21	8.99	8.57	8.14	7.71	7.28	0.0086
22	8.83	8.42	7.99	7.57	7.14	0.0084
23	8.68	8.27	7.85	7.43	7.00	0.0083
24	8.53	8.12	7.71	7.30	6.87	0.0083
25	8.38	7.96	7.56	7.15	6.74	0.0082
26	8.22	7.81	7.42	7.02	6.61	0.0080
27	8.07	7.67	7.28	6.88	6.49	0.0079
28	7.92	7.53	7.14	6.75	6.37	0.0078
29	7.77	7.39	7.00	6.62	6.25	0.0076
30	7.63	7.25	6.86	6.49	6.13	0.0075

General requirements:

1. The BOD at distance x: $L_{(X)} = L_0 e^{-(K_1+K_3)\frac{x}{u}} + \frac{L_a}{K_1+K_3}\left(1-e^{-(K_1+K_3)\frac{x}{u}}\right)$,

   Where: L_O = the BOD in the stream at x = 0

2. $$D_{(X)} = D_O e^{-K_2\frac{x}{u}} + \frac{K_1}{K_2-(K_1+K_3)}\left(L_0 - \frac{La}{K_1+K_3}\right)\left(e^{-(K_1+K_3)\frac{x}{u}} - e^{-K_2\frac{x}{u}}\right) + \left(\frac{S_R}{K_2} + \frac{K_1 L_a}{K_2(K_1+K_3)}\right)\left(1-e^{-K_2\frac{x}{u}}\right).$$

Where: D_O = the deficit in the stream at x = 0

3. The critical deficit:

$$X_c = \frac{u}{K_2-(K_1+K_3)}\ln\left[\frac{K_2}{K_1+K_3} + \frac{K_2-(K_1+K_3)}{(K_1+K_3)L_o - L_a}\left[\frac{L_a}{K_1+K_3} - \frac{K_2 D_o - S_R}{K_1}\right]\right]$$

K_1: Deoxygenation Constant

K_2: Reaeration constant = $\left(\frac{D_M U}{H^3}\right)^{1/2}$ $D_M = 2.037\times10^{-5}\, cm/sec$ at 20⁰C

$$K_{2(T)} = K_{2(20c)}(1.025)^{T-20}$$

4. If BOD decreases along a reach and BOD_X is less than predicted by:

   $L_x = L_o e^{-K_1\frac{x}{u}}$, So $L_a = 0$, $L_x = L_O e^{-(K_1+K_3)\frac{x}{u}}$

5. If BOD increases with (x) or decreases less than predicted by:

   $L_x = L_o e^{-K_1\frac{x}{u}}$ So $K_3 = 0$ $L_x = L_O e^{-K_1\frac{x}{u}} + \frac{L_a}{K_1}\left(1-e^{-k_1\frac{x}{u}}\right)$

6. In absence of values (K_3, La, S_R) may be reduced to Classic Streeter- Phelps Formulation:

   $$D_{(x)} = D_o e^{-K_2\frac{x}{u}} + \frac{K_1}{K_2-K_1}L_o\left(e^{-K_1\frac{x}{u}} - e^{-K_2\frac{x}{u}}\right)$$

$$x_c = \frac{u}{K_2-k_1}\ln\frac{K_2}{K_1}\left[1-\frac{K_2-K_1}{K_1 L_o}(D_o)\right],\ t = \frac{x}{u}$$

$$D_{(t)} = D_{(o)}e^{-K_2 t} + L_o\left(e^{-K_1 t} - e^{-K_2 t}\right)$$

$$t_c = \frac{1}{K_2 - K_1} \ln \frac{K_2}{K_1} \left[1 - \frac{K_2 - K_1}{K_1 L_o} (D_o) \right]$$

$$DO \times (Q_s + Q_w) = DO_s \times Q_s + DO_w \times Q_w$$

$$BOD_5 (Q_s + Q_w) = BOD_{5s} \times Q_s + BOD_{5w} \times Q_w$$

Solved problems

Problem 1: A waste effluent of 560 L/s with BOD_5 = 50 mg/L, DO = 3 mg/L and T = 23 ^{0}C is to be discharged into a river of 2.8 m^3/s flow, BOD_5 = 4mg/L, DO = 8.2mg/L and T= 17 ^{0}C. K_1 for the waste is 0.23 day $^{-1}$ at 20 ^{0}C. The average velocity of the river is 0.18 m/s with aeration rate of 0.43 day $^{-1}$ at 20 ^{0}C. 1) Is DO min within the environmental limitations? 2) At what distance is the maximum deficit located. 3) Draw the oxygen sag curve?

Solution:

Characteristics of the mix:

$$Q_{Mix} = Q_r + Q_w = 2.8 + 0.56 = 3.36\, m^3/sec$$

$$BOD_{5Mix} = \frac{BOD_{5r} \times Q_r + BOD_{5w} \times Q_w}{Q_{Mix}} = \frac{4 \times 2.8 + 50 \times 0.56}{3.36} = 11.67 \approx 11.7\, mg/L$$

$$DO_{Mix} = \frac{DO_r \times Q_r + DO_w \times Q_w}{Q_{Mix}} = \frac{8.2 \times 2.8 + 3 \times 0.56}{3.36} = 7.33\, mg/L$$

$$T_{Mix} = \frac{T_r \times Q_r + T_w \times Q_w}{Q_{Mix}} = \frac{17 \times 2.8 + 23 \times 0.56}{3.36} = 18^0 C$$

$$K_{1(18)} = K_{1(20)} \times 1.047^{(T-20)} = 0.23 \times 1.047^{(18-20)} \approx 0.21\, day^{-1}$$

$$K_{2(18)} = K_{2(20)} \times 1.025^{(T-20)} = 0.43 \times 1.025^{(18-20)} \approx 0.41\, day^{-1}$$

$$BOD_{t-T} = L(1 - e^{-K_1 t}) \Rightarrow BOD_{5-18} = L(1 - e^{-0.21 \times 5}) \Rightarrow 11.7 = L(1 - e^{-0.21 \times 5}) \Rightarrow L_{Mix} = BOD_u = ultimate\ BOD = 17.998\, mg/L$$

DO saturation from table, at T_{mix} = 18 ^{0}C = 9.54 mg/L

$$D_i = DO_{Sat.at\ Tmix} - DO_{mix} = 9.54 - 7.33 = 2.21\, mg/L$$

$$t_c = \frac{1}{k_2 - k_1} ln \frac{k_2}{k_1} \left[1 - \frac{k_2 - k_1}{k_1 \times L_0} \times D_0 \right] = \frac{1}{0.41 - 0.21} ln \frac{0.41}{0.21} \left[1 - \frac{0.41 - 0.21}{0.21 \times 17.1} \times 2.21 \right] = 2.69\, day$$

$$D_c = \frac{K_1 L e^{-K_1 t_c}}{K_2} = \frac{0.21 \times 17.1 \times e^{-0.21\times 2.69}}{0.41} = 4.96\ mg/L$$

$$D_{min} = DO_{Sat.at\ Tmix} - D_c = 9.54 - 4.98 = 4.58\frac{mg}{L} > 4\frac{mg}{L}\ O.K$$

$$Distance = Velocity \times time = 0.18\ m/\sec \times \frac{2.95 \times 24 \times 3600}{1000} = 45.878\ km$$

Problem 2: A city discharges 1.25 m^3/sec of wastewater onto a stream whose minimum rate of flow is 8 m^3/sec. The velocity of stream is 0.8 m/sec. The temperature of the wastewater is 20 ^{0}C and that of the stream is 15 ^{0}C. The BOD$_5$ of the wastewater is 183 mg/L, and that of the stream is 2 mg/L. The wastewater contains no dissolved oxygen but the stream is 90 percent saturated upstream of the discharge. At 20 ^{0}C, deoxygenation coefficient (K$_1$) is estimated to be 0.3 per day and reaeration coefficient (K$_2$) is 0.9 per day. Determine the critical-oxygen deficit and its location. Also, estimate the 20 ^{0}C BOD$_5$ of the sample taken at the critical point. Use temperature coefficient of 1.135 for deoxygenation coefficient (K$_1$) and 1.024 for reaeration coefficient (K$_2$). Also, plot the dissolved-oxygen sag curve? Given the saturation concentration at 15 ^{0}C and at mixed temperature = 10.15 mg/L

Solution:

1. Characteristics of the mix:

$$Q_{Mix} = Q_r + Q_w = 8 + 1.25 = 9.25\ m^3/sec$$

DO of the stream, $DO_r = 0.9 \times 10.15 = 9.14\ mg/L$

$$DO_{Mix} = \frac{DO_r \times Q_r + DO_w \times Q_w}{Q_{Mix}} = \frac{9.14 \times 8 + 0 \times 1.25}{9.25} = 7.9\ mg/L$$

$$T_{Mix} = \frac{T_r \times Q_r + T_w \times Q_w}{Q_{Mix}} = \frac{15 \times 8 + 20 \times 1.25}{9.25} = 15.675^0C$$

$$K_{1(15.675)} = 0.3 \times 1.135^{(15.675-20)} \approx 0.173\ day^{-1}$$

$$K_{2(15.675)} = 0.9 \times 1.024^{(15.675-20)} \approx 0.812\ day^{-1}$$

$$BOD_{5Mix} = \frac{BOD_{5r} \times Q_r + BOD_{5w} \times Q_w}{Q_{Mix}} = \frac{2 \times 8 + 183 \times 1.25}{9.25} = 26.46\ mg/L$$

$$BOD_{t-T} = L(1 - e^{-K_1 t}) \Rightarrow BOD_{5-15.675} = L(1 - e^{-0.173\times 5}) \Rightarrow 26.46 = L(1 - e^{-0.173\times 5}) \Rightarrow L_{Mix} = BOD_u = ultimate\ BOD = 45.702\ \frac{mg}{L} = L_{Mix}$$

2. Initial dissolved oxygen deficit at mixed temperature (T$_{Mix}$ = 15.675 ^{0}C)

From table, DO saturation concentration at 15.675 ^{0}C = 10.15 mg/L

Initial deficit: $D_0 = DO_{Sat.at\,Tmix} - DO_{mix} = 10.15 - 7.9 = 2.25\,mg/L$

3. Determination of t_c and x_c:

$$t_c = \frac{1}{K_2 - K_1} \ln \frac{K_2}{K_1} \left[1 - \frac{K_2 - K_1}{K_1 L_o} (D_o) \right] = \frac{1}{0.812 - 0.173} \ln \frac{0.812}{0.173} \left[1 - \frac{0.812 - 0.173}{0.173 \times 45.702} \times 2.25 \right] = 2.105\,da$$

$$x_c = v \times t_c = 0.8 \times \left(\frac{3600 \times 24}{1000} \right) \times 2.105 = 145.497\,km$$

4. Determination of D_c and DO at x_c:

$$D_c = \frac{K_1 L_{0mix}\, e^{-K_1 t_c}}{K_2} = \frac{0.173 \times 45.702 \times e^{-0.173 \times 2.105}}{0.812} = 6.765\,mg/L$$

$DO\ at\ x_c = 10.15 - 6.765 = 3.385\ \frac{mg}{L} > DO_{Min} = 2\,mg/L$ O.K

5. Determination of BOD_5 of a sample taken at x_c:

$$L_t = L_{0Mix} \times e^{-K_1 \times t_c} = 45.702 \times e^{-0.173 \times 2.105} = 31.752\,mg/L$$

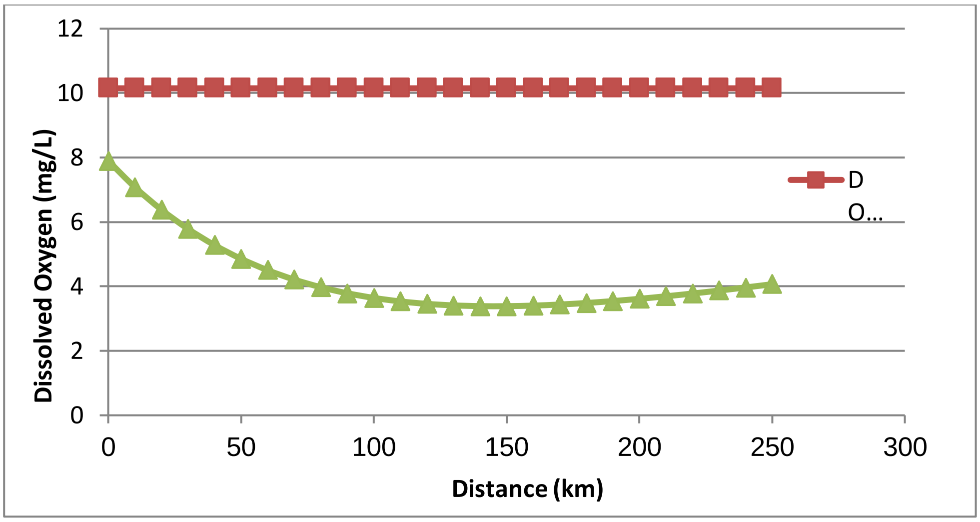
$$BOD_{5-20} = 31.752 \times (1 - e^{-0.3 \times 5}) = 24.667\,mg/L$$

6. Computation of additional points on the oxygen sag curve:

$$D_t = \frac{K_1 L_{0Mix}}{K_2 - K_1} (e^{-K_1 t} - e^{-K_2 t}) + D_i e^{-K_2 t}$$
$$= \frac{0.173 \times 45.702}{0.812 - 0.173} (e^{-0.173 \times t} - e^{-0.812 \times t}) + 2.25 e^{-0.812 \times t}$$

$$DO_t = DO_{Sat} - D_t = 10.15 - D_t$$

DISTANCE (km)	0	50	100	120	140	150	200	250
Time (day)	0.000	0.723	1.447	1.736	2.025	2.170	2.894	3.617
D_t (mg/L)	2.250	5.291	6.506	6.691	6.761	6.762	6.534	6.081
DO_{sat} (mg/L)	10.15	10.15	10.15	10.15	10.15	10.15	10.15	10.15
DO (mg/L)	7.900	4.859	3.644	3.459	3.389	3.388	3.616	4.069



Oxygen sag curve

Problem 3: The domestic sewage of a town is to be discharged into a stream after treatment. Determine the maximum permissible effluent BOD and the percentage purification required in the treatment given the following particulars:

Population of town: 50000 capita, DWF of sewage: 150 liters per capita, BOD contribution per capita: 0.075 kg/day, Minimum flow of stream: 0.20 m^3/sec, BOD of stream: 3 mg/L, Maximum BOD of stream on downstream: 5 mg/L.

Problem 4: A stream, saturated with DO, has a flow of 1.2 m^3/sec, BOD of 4 mg/L and oxygenation rate constant of 0.3 per day (based 10). It receives an effluent discharge of 0.25 m^3/sec having BOD 20 mg/L, DO 5 mg/L and deoxygenation rate constant 0.13 per day (based 10). The average velocity of flow of the stream is 0.18 m/sec. Calculate the DO deficit at point 20 km and 40 km downstream? Assume that the temperature is 20^0C throughout and BOD is measured at 5 days. Take saturated DO at 20^0C as 9.17 mg/L.

Problem 5: A city discharge at the rate of 1200 liters per second, into a stream whose minimum flow is 5000 liters per second, the temperature of both being 20^0C. The 5-day BOD at 20^0C for sewage is 160 mg/L and that of river water is 2 mg/L. The DO content of sewage is zero while that of stream is 90% of the saturation DO. Find out the degree of treatment required if the minimum DO to be maintained in the stream is 4 mg/L? Assume deoxygenation coefficient as 0.1 (based 10) and reoxygenation coefficient as 0.3 (based 10). Given saturation DO at 20^0C as 9.17 mg/L.

Sanitary and Environmental Engineering

PART 2: WASTEWATER ENGINEERING

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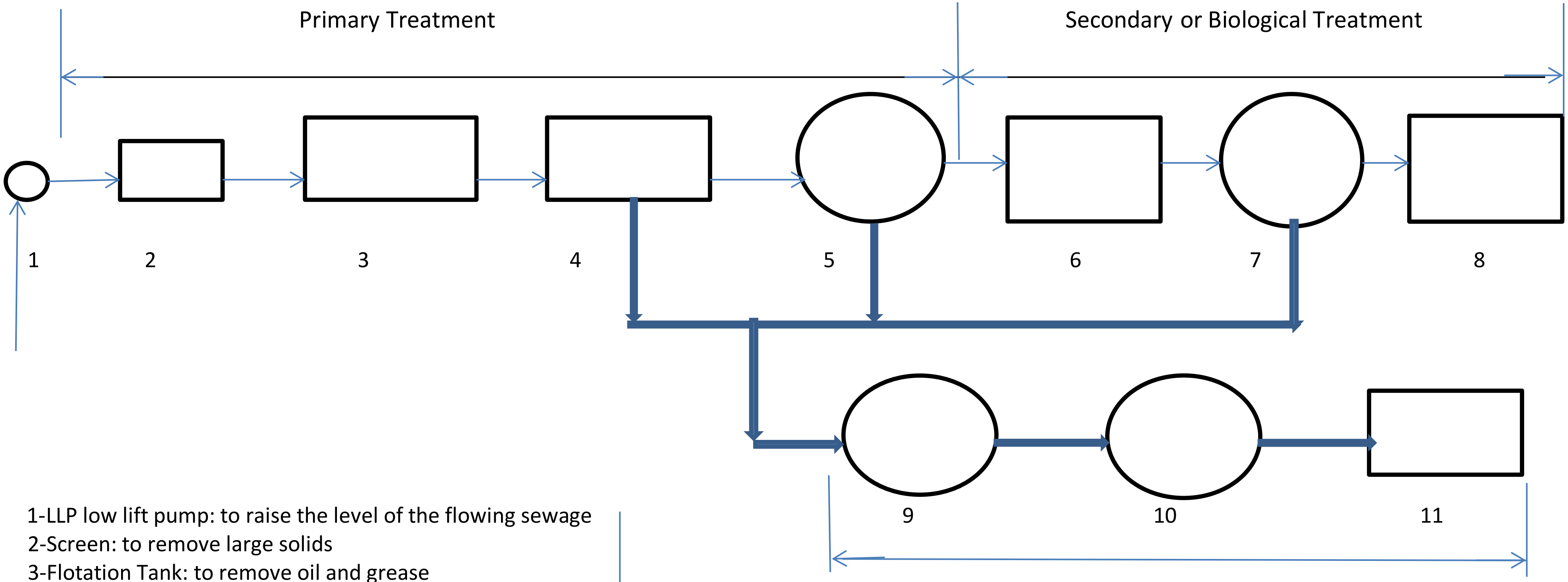
Lecture 6: Flow Diagram for a Wastewater or Sewage Treatment Plant (WWTP)



1-LLP low lift pump: to raise the level of the flowing sewage
2-Screen: to remove large solids
3-Flotation Tank: to remove oil and grease
4-Girt Chamber: to remove grits and sand
5-Primary Sedimentation Tank: to remove suspended solids

For the removal of inorganic matter

6-Biological Tank: for the decomposition of organic matter
7-Secondary Sedimentation Tank: for the settlement of the Flocculent biological mass
8-Chlorination Contact Tank: to kill harmful organisms

For the removal of organic matter

9-Thickner: to reduce the moisture content for sludge
10-Digester: for the decomposition of the organic matter
11-Drying Beds: to dry the sludge

Sludge Treatment

Liquid →

Sludge ➡

Wastewater treatment plant:

First: Primary treatment: a treatment process for wastewater (or sewage) is employed to remove inorganic material that floats or will settle, and pathogens that account for about 30 percent of biological oxygen demand (BOD). It involves:

1. Low lift station
2. Parshall flume.
3. Racks, screens and comminutors.
4. Floatation tank.
5. Grit removal.
6. Primary sedimentation tanks.

Second: secondary or biological treatment: is a treatment process for wastewater (or sewage) to remove organic matter and achieve a certain degree of effluent quality by using a sewage treatment plant with physical phase separation to remove settleable solids and a biological process to remove dissolved and suspended organic compounds.

1. Biological process.
2. Secondary sedimentation tank.
3. Chlorine contact tank (effluent disposal).

Biological process:

Attached growth process:

a) Intermittent sand filter.
b) Trickling filters.
c) Rotating biological contactors.

Suspended growth process:

a) Conventional process.
b) Tapered aeration process.
c) Step aeration process.
d) Completely mixed process.
e) Extended aeration process.
f) Contact stabilization process.

Sludge treatment and disposal:

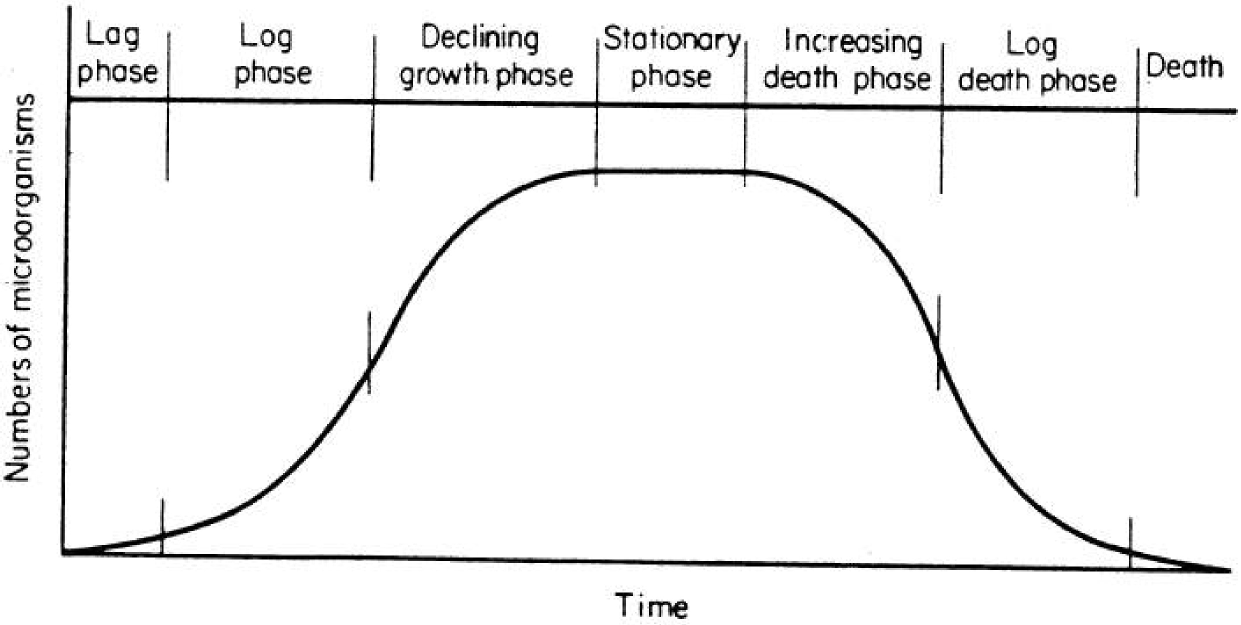
a) Thickener tank.
b) Digester tanks.
c) Dewatering process (drying beds, vacuum filter, pressure filter, centrifuges, pressed filters, drying and combustion).

Third: Tertiary treatment is the final cleaning process that improves wastewater quality before it is reused, recycled or discharged to the environment. The treatment removes remaining inorganic compounds, and substances, such as the nitrogen and phosphorus.

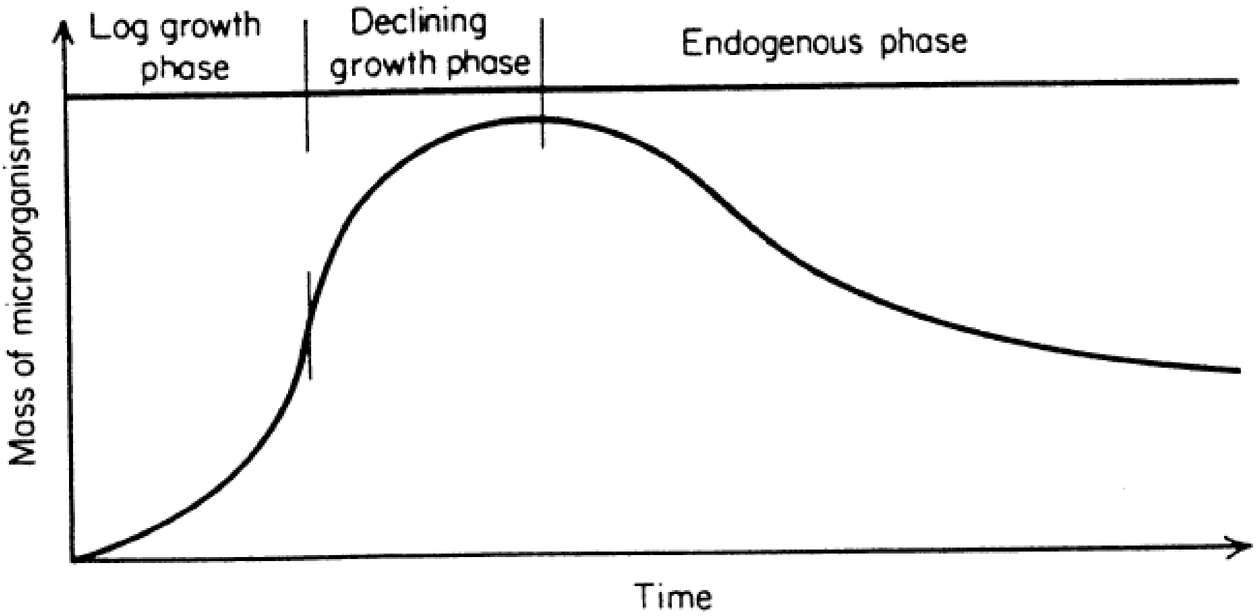
Microbiology of sewage treatment:

Domestic sewage contains enormous quantities of microorganisms which depending upon its age and the quantity of dilution water, bacterial counts in raw sewage may be expected to range $(0.5 \times 10^5 \rightarrow 5 \times 10^6)$ /mL. Viruses, protozoans, worms, etc., are also present but their concentration is seldom important enough to require measurement.

Bacteria are single-celled plants which metabolize soluble food and reproduce by binary fission.



Growth pattern based on number of organisms.



Growth pattern based on mass of organisms.

The declining growth phase is generally used for biological treatment systems.

Sanitary and Environmental Engineering

PART 2: WASTEWATER ENGINEERING

PART 2: WASTEWATER ENGINEERING

Lecture 7: Preliminary and Primary Treatment

Note:

a) The headwork of wastewater plants usually incorporates a flow measurement device as a Parshall flume and mechanical or physic-chemical system designed for removal of large floating solids, grit and perhaps grease (preliminary treatment systems).

b) Primary treatment has traditionally implied a sedimentation process intended to remove suspended organic solids. Chemicals are sometimes added in primary clarifiers to assist in removal of finely divided and colloidal solids or to precipitate.

1. **Low lift sewage pump station:**

2. **Screen**: A protective device used to remove large and coarse materials from the wastewater, which may damage equipment.

   a) Types: rack, mesh

   b) Openings: Coarse more than 50 mm, medium 25-50 mm, fine less than 25 mm.

   c) Flowing velocity through the openings: v_f= 0.6-1.2 m/sec

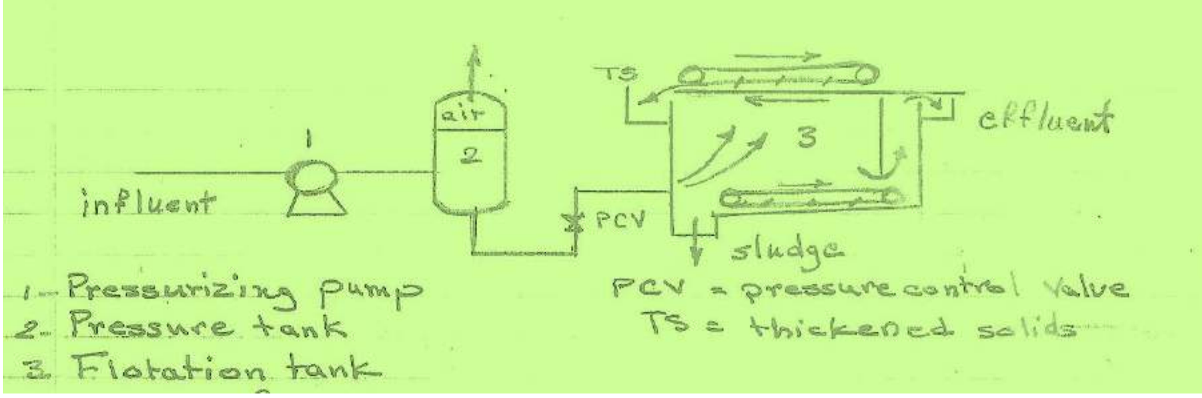
3. **Flotation or Skimming tank:** To separate solid and liquid particles (grease and oil) from the wastewater by the introduction of air bubbles. The particles are attached to the bubbles and by the buoyancy forces, the particles are raised to the water surface. The floating particles are removed by skimming.

   Types: Dissolved air flotation, Dispersed air flotation

Dissolved air flotation (DAF): Air bubbles are introduced by injection of air to the water under pressure. Air is dissolved under 250-450 kPa in the pressure tank, where the water is held for several minutes.

The components of this tank:

1-pressurizing pump 2-pressure tank 3-flotation tank



The performance of this system depends on:

$$\frac{A}{S} = 1.3 \times S_a \times \frac{(f \times P - 1)}{C_i}$$

Temperature °C	0	10	20	30
S_a mL/L	29.2	22.8	18.7	15.7

$\frac{A}{S}$: Air to solid ratio (mL of air/ mg solids)
S_a: Solubility of air depending on temperature (mL/L)
ρ_{air}= 1.3 mg /mL
f = Fraction of dissolved air at pressure (P) = 0.5
P = Pressure, atm ($P_{atm} = \frac{P_{kPa}+101.35}{101.35}$).
C_i = Concentration of suspended solids in the influent (mg/L)

Design criteria:
Detention time = 3-30 min
Flowing velocity = 50-250 mm/sec
$\frac{A}{S}$= 0.005-0.6 mL/mg
Surface loading rate = velocity to rise the suspended solids = 8-160 L/m^2.min ($SLR = \frac{Q}{A_S}$).
The quantities of float skimming = (3-7) % of incoming flow.

Dispersed air flotation: Air bobbles are formed by aeration at atmospheric pressure using revolving impellers or turbine aerators. The advantages of a dispersed-air flotation system are:

a) Compact size,
b) Lower capital cost, and
c) Capacities to remove relatively free oil and suspended solids.

The disadvantages of induced-air floatation:

a) Higher connected power requirements than the pressurized system,
b) Performance is dependent on strict hydraulic control, and
c) Less flocculation flexibility.

4. **Grit Chamber:** A sedimentation tank designed mainly for the removal of grits: sand, gravel, glass and other heavy solids from wastewater. The separation of inorganic and organic solids takes place due to the difference in the specific gravity of these particles. This parameter affects the settling velocity v_s and the scouring velocity v_h for these particles where they are used for design requirements.

As the flow in this tank is turbulent:

All particles are assumed to settle in according with Newton Law: $v_s = \sqrt{\frac{4g\times(G_s-1)\times d}{3C_D}}$ & for sedimentation:

$$v_s = SOR = \frac{Q}{A_s} = \frac{Q}{W \times L}$$

To be scour at velocity: $v_h = \sqrt{8\left(\frac{\beta}{f}\right) \times g \times (G_s - 1) \times d}$ & $v_h = \frac{Q}{A_C} = \frac{Q}{W \times H}$

v_h should be more than v_f (flowing velocity)

For design:
SOR = v_s for inorganic particles:

v_s for organic particles < v_f < v_s for inorganic particles

Types: Aerated grit chambers, Horizontal flow grit chambers.

Aerated grit chamber: The separation of different types of solids is done by producing a rolling flow similar to the spiral flow by using air. The velocity of the flow v_f is to be below v_s for inorganic particles and above v_s organic particles.

$$v_s \text{ for organic particles } < v_f < v_s \text{ for inorganic particles}$$

Design criteria:

1- Detention time= 2-4 min

2- Dimensions for a rectangular tank:

L = 8-20 m, W = 2-5 m, H = 2-7 m

W/H = 1:1 – 5:1

L/H = 3:1 – 5:1

3- Air flow rate = 0.015-0.5 m^3/min/ m length

Horizontal flow grit chamber: It is designed to provide sufficient time for inorganic particles to settle and the flowing velocity will carry out the organic particles. The velocities for design (v_s and v_h) are being controlled for the variation of the wastewater flow.

To control the velocities;

1- Using the proportional weir;

2- Using the parabolic cross section.

Design criteria:

1- Detention time= 30 -90 min

2- v_f = 0.2-0.4 m/sec

3- Dimensions:

L = 3 -25 m, H = 1-2 m

L/H less than 12

4- Quantity of grit = $2.5 \times 10^{-6} - 1.8 \times 10^{-4} \frac{m^3}{m^3} of\ wastewater$

<u>**NOTE:**</u> To assure removal of grit (inorganic matter), while permitting such organic matter as might settle to be re-suspended by scour, the necessary condition may be calculated as follow:

For particles of grit (inorganic matter), with diameter of 0.2 mm and specific gravity = 2.65,

$$v_s = \sqrt{\frac{4g \times (G_s - 1) \times d}{3C_D}} = \sqrt{\frac{4 \times 980 \times (2.65 - 1) \times 0.02}{3 \times 10}} = 2.1\ cm/sec$$

The scour velocity of inorganic particles is given by:

$$v_h = \sqrt{8\left(\frac{\beta}{f}\right) \times g \times (G_s - 1) \times d} = \sqrt{8\left(\frac{0.06}{0.03}\right) \times 980 \times (2.65 - 1) \times 0.02} = 23\ cm/sec$$

For particles of organic matter, with diameter of 0.2 mm and specific gravity = 1.1,

$$v_s = \sqrt{\frac{4g \times (G_s - 1) \times d}{3C_D}} = \sqrt{\frac{4 \times 980 \times (1.1 - 1) \times 0.02}{3 \times 10}} = 0.511\ cm/sec$$

The scour velocity of organic particles is given by:

$$v_h = \sqrt{8\left(\frac{\beta}{f}\right) \times g \times (G_s - 1) \times d} = \sqrt{8\left(\frac{0.06}{0.03}\right) \times 980 \times (1.1 - 1) \times 0.02} = 5.6\ cm/sec$$

Thus, if the basin is designed to have a surface overflow rate of 0.021 m/sec and a horizontal velocity greater than 0.056 m/sec and less than 0.23 m/sec will remove grit (inorganic material) without removing organic material. To assure that the grit is reasonable clean, the horizontal velocity is generally close to the scour velocity of the grit.

5. **Primary sedimentation tank**: To remove settleable solids mainly inorganic. It is designed to remove 50 -70% inorganic particles and 25-40% organic particles. Removal curves for suspended solids (SS) indicating inorganic, and BOD indicating organic particles with SOR of the tank are used for design, (Fig. 23-1 page 470 and Fig. 23-2 page 471) or the readings from these curves are given in the table below.

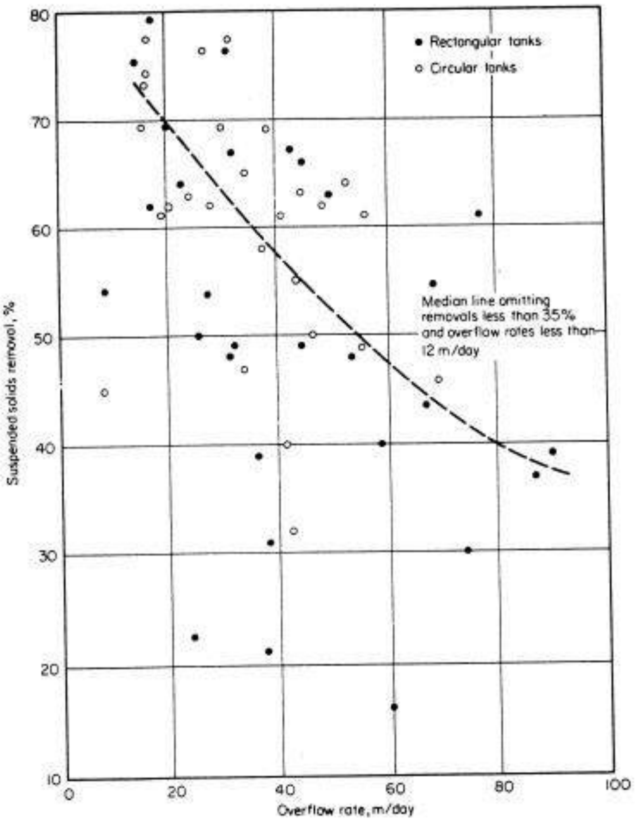


Fig. 23-1: Suspended solids removal in primary clarifiers.

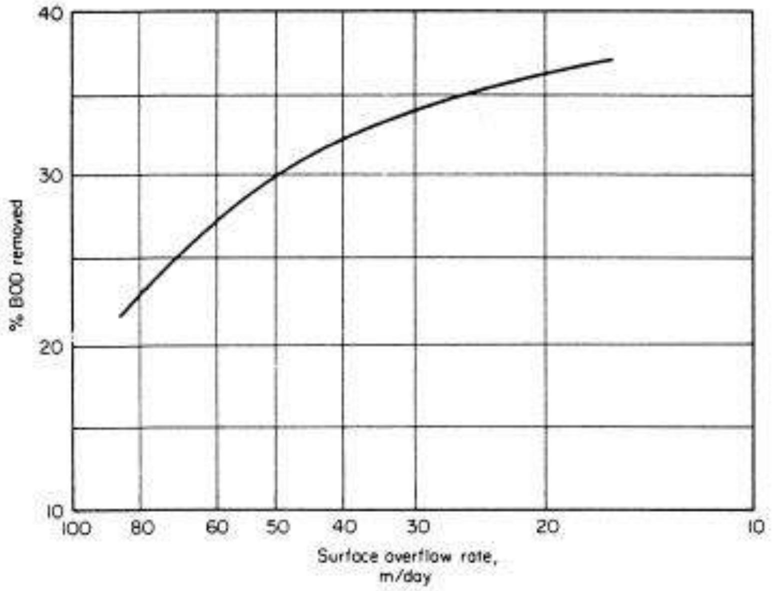


Fig. 23-2: BOD removal in primary clarifiers.

SOR m/d	20	30	40	50	60	70	80
% SS removal	70	65	58	53	48	45	40
% BOD removal	36	34	32.8	30	27.5	25	22.5

Design criteria:

1- Detention time = 1 -4 hr

2- SOR = Average flow 30-50 m/d & maximum flow 80 – 120 m/d

3- v_f = 0.6-3.6 m/min

4- Weir loading rate = 125-600 m^3/m/d

5- Dimensions:

Rectangular L = 15-100 m, H = 3 -5 m, W = 3 -30m

Circular D less than 60m

6- Scum removed average about = $7.5 \times 10^{-6}\frac{m^3}{m^3}\, of\ wastewater\ flow$

Solved problems

Problem 1: A flotation tank is used for thickening suspended solids (oil) of 3000 mg/L, knowing that: $\frac{A}{S}$= 0.008 mL/mg, temperature =20°C, surface loading rate = 8 L/m^2.min, Q= 400 m^3/d. Find: 1-required pressure P (kPa) inside pressure tank, 2- Surface area of the flotation tank (m^2) & 3) The quantities of float skimming if (7)% of incoming flow?
?
Solution:
1) From table at T = 20°C, air solubility S$_a$ = 18.7 mL/L

$$\frac{A}{S} = 1.3 \times S_a \times \frac{(f\times P-1)}{C_i} \Rrightarrow 0.008 = 1.3 \times 18.7 \times \frac{(0.5\times P-1)}{3000} \Rrightarrow P = 3.98\,atm$$

$$\therefore P = 3.98\,atm = \frac{P_{kPa} + 101.35}{101.35} \Rrightarrow P = 302\,kPa$$

2) $A_s = \frac{Q}{Surface\ Loading} = \frac{400\times 10^3\ L/day}{8\times 24\times 60\ L/day} = 34.7\,m^2$

3) $\frac{kg}{day} = 3000\frac{mg}{L} \times 10^{-3} \times 400\frac{m^3}{day} \times 0.07 = 84\,kg/day$

Problem 2: Design a horizontal flow girt chamber (rectangular) of a parabolic section. Knowing that Q$_{max}$ = 18,000 m^3/d, Qavg = 15,000 m^3/d ,Q$_{min}$ = 10,000 m^3/d, Gs inorganic particles = 2.65, Gs organic particles = 1.1, C$_D$ =10, β/f = 2, detention time = 1 min, particles to be removed = 0.2 mm in diameter?
Solution:
For inorganic particles, Gs = 2.65:

$$v_s = \sqrt{\frac{4g\times(G_s-1)\times d_s}{3C_D}} = \sqrt{\frac{4\times 980\times(2.65-1)\times 0.02}{3\times 10}} = 2.076\ \approx 2.1\,cm/sec$$

$$v_s\ for\ inorganic\ particles = SOR = \frac{Q}{A_S}$$

Surface area of grit chamber:

$$A_s = \frac{18000}{0.021\times 24\times 60\times 60} = 9.921\,m^2 = L\times W \qquad (1)$$

$$v_h = \sqrt{8\left(\frac{\beta}{f}\right)\times g\times(G_s-1)\times d_s} = \sqrt{8\left(\frac{0.06}{0.03}\right)\times 980\times(2.65-1)\times 0.02} = 23\,cm/sec$$

For organic particles, Gs = 1.1:

$$v_h = \sqrt{8\left(\frac{\beta}{f}\right)\times g\times(G_s-1)\times d_s} = \sqrt{8\left(\frac{0.06}{0.03}\right)\times 980\times(1.1-1)\times 0.02} = 5.6\,cm/sec$$

For design Ac:

$$v_{h\,organic} < v_{h\,design} < v_{h\,inorganic}$$

Assume $v_{h\,design} = 0.2\,m/sec$

$$\therefore A_c = \frac{Q}{v_h} = \frac{18000}{0.2\times 24\times 60\times 60} = 1.042\,m^2 = H\times W \qquad (2)$$

$$V = Q\times t = \frac{18000\times 1\,min}{24\times 60} = 12.5\,m^3 = L\times W\times H \qquad (3)$$

Solving eq. 1, 2, & 3 and find: $L = 12\,m$, $W = 0.827\,m$, & $H = 1.26\,m$

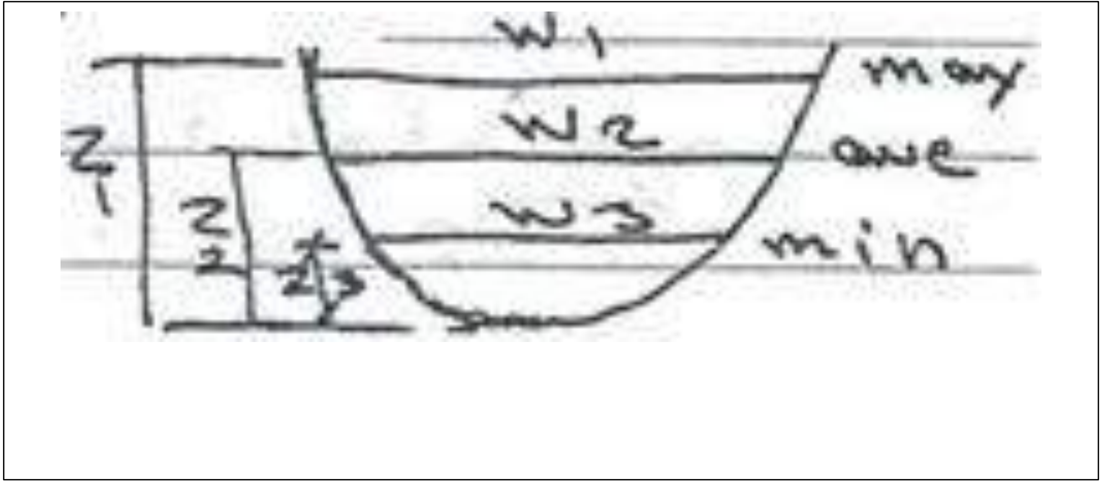
Assume use area of parabolic section = $\frac{2}{3}WZ$, Where:
W: Width at the design level.
Z: Height at the design level.

$$Z = C\times W^2$$

So, for A$_C$, $A_C = \frac{2}{3}WZ \rightarrow 1.042 = \frac{2}{3}\times 0.827\times Z \rightarrow Z = 1.889\,m$

$$Z = C \times W^2 \rightarrow 1.889 = C \times 0.827^2 \rightarrow C = 2.763$$

Q (m^3/day)	A$_C$ (m^2)	W (m)	Z (m)
18000	1.042	0.827	1.889
15000	0.868	0.78	1.68
10000	0.579	0.68	1.289



Problem 3: Design a primary sedimentation tank for 60% SS removal. The influent flow is 10,000 m^3/d, and assume the detention time = 2hr? Also find the SS settling and effluent sludge if the influent SS concentration 2000 mg/L?

Solution:

From Fig. 23-1 or Table for 60% SS removal, SOR = 37 m/day and from Fig. 23-2 or Table 33% BOD removal.

$$SOR = \frac{Q}{A_s} \rightarrow A_s = \frac{Q}{SOR} = \frac{10000}{37} = 270\ m^2$$

If circular tank: $A = \frac{\pi}{4} D^2 \rightarrow 270 = \frac{\pi}{4} D^2 \rightarrow D = 18.5\ m$

$$V = Q \times t = A_s \times H \rightarrow \frac{10000}{24} \times 2\ hr = 270 \times H \rightarrow H = 3.1\ m$$

Weir loading rate, $WLR = \frac{Q}{Weir\ length} = \frac{10000}{\pi \times 18.5} = 172\ m^3/(m.day)$ O.K

If rectangular tank: Assume W = 10 m, so, $A = L \times W \rightarrow 270 = L \times 10 \rightarrow L = 27\ m$

Weir loading rate, $WLR = \frac{Q}{Weir\ length} = \frac{10000}{10} = 1000\ m^3/(m.day)$ not O.K

Assume weir loading rate = 600 m^3/(m.day) & weir length = 17 m

The influent sewage SS, $\frac{kg}{day} = 2000 \times 10^{-3} \times 10000 = 20000\ kg/day$

The influent SS settling sludge, $\frac{kg}{day} = 2000 \times 10^{-3} \times 10000 \times 0.6 = 12000\ kg/day$

The effluent SS, $\frac{kg}{day} = 2000 \times 10^{-3} \times 10000 \times 0.4 = 8000\ kg/day$

Problem 4: A raw wastewater containing 250 mg/L SS and 200 mg/L BOD passes through a clarifier with a surface area of 500 m^2. The flow ranges from 10000 m^3/day to 30000 m^3/day. Estimate the effluent BOD and SS at maximum and minimum flow, and the maximum and minimum rate of sludge production? Assume the sludge is 95 percent water.

Problem 5: Design a primary clarifier system for a community with population of 50000 persons. Size the clarifiers so that the SOR will be not more than 40 m/day at peak flow with one unit out of service and not more than 25 m/day at average flow? The minimum retention time is to be 1 hour and maximum weir loading rate 370 m^3/m.day. Also estimate the quantity of sludge which must be removed? If the sludge pumping cycle is 15 min/hour at maximum, what flow must the pump be sized to handle?

Problem 6: Design a primary clarifier system for a community with population of 50000 persons with a water supply demand 180 L/capita.day, and assuming that 80% of water supplied to the community is converted into sewage. Size the clarifiers so that the SOR will be not more than 40 m/day at peak flow (case 1) and not more than 25 m/day at average flow (case 2) with one unit out of service? The minimum retention time is to be 1 hour and maximum weir loading rate 370 m^3/m.day. Also, find the percent removal SS and BOD for each SOR?

SOR m/d	20	30	40	50	60	70	80
% SS removal	70	65	58	53	48	45	40
% BOD removal	36	34	32.8	30	27.5	25	22.5

Solution:

Average sewage flow: $Q_{Avg} = 0.8 \times 180 \times 50000 = 7200000 \frac{L}{day} = 7200\ m^3/day$

Peak sewage flow: $Q_{Peak} = 0.8 \times 1.8 \times 180 \times 50000 = 12960000\ \frac{L}{day} = 12960\ m^3/day$

Case 1: SOR = 40 m/day at peak flow:

$SOR = \frac{Q}{A_s} \rightarrow A_s = \frac{12960}{40} = 324\ m^2 \rightarrow D = 20.32\ m$ use D = 20 m & $Q = \frac{V}{t} \rightarrow V = 12960 \times \frac{1}{24} = 540\ m^3, H = \frac{V}{A_s} = \frac{540}{324} = 1.667\ m$

$WLR = \frac{Q}{L} = \frac{12960}{\pi \times 20} = 206.369\ \frac{m^3}{m.day} < 370\ \frac{m^3}{m.day}$ O.K

From table at SOR = 40 m/day: SS% removal = 58% and BOD% removal = 32.8%

Case 2: SOR = 25 m/day at average flow:

$SOR = \frac{Q}{A_s} \rightarrow A_s = \frac{7200}{25} = 288\ m^2 \rightarrow D = 19.15\ m$ use D= 19 m & $Q = \frac{V}{t} \rightarrow V = 7200 \times \frac{1}{24} = 300\ m^3,\ H = \frac{V}{A_s} = \frac{300}{288} = 1.041\ m$

$WLR = \frac{Q}{L} = \frac{7200}{\pi \times 19} = 120.683\ \frac{m^3}{m.day} < 370\ \frac{m^3}{m.day}$ O.K

From table at SOR = 25 m/day: SS% removal = 67.5% and BOD% removal = 35%

Population equivalent or unit per capita loading, (PE), in waste-water treatment is the number expressing the ratio of the sum of the pollution load produced during 24 hours by industrial facilities and services to the individual pollution load in household sewage produced by one person in the same time.

The population equivalent (P.E.) of a waste may be determined by divided the total mass per day by the per mass loadings. Population equivalent has been used as a technique for determining industrial waste treatment costs. PE may be determined on the basis of flow, BOD_5, COD, TSS, P, N, etc.

Example: An industrial waste of 1000 m^3/day and having a BOD_5 of 500 mg/L, calculate the population equivalent in terms of BOD_5 if the domestic BOD loading = 95 gm/(cap.day)?

Solution:

$$PE = \frac{0.5 \times 10^3 \ \frac{gm}{m^3} \times 1000 \ m^3/day}{95 \ gm/(capita.day)} = 5263 \ capita$$

Example: A waste with a BOD of 300 mg/L and a flow of 1000 m^3/day L, calculate the population equivalent in terms of BOD_5 if the domestic BOD loading = 80 gm/(cap.day)?

Solution:

$$PE = \frac{0.3 \times 10^3 \ \frac{gm}{m^3} \times 1000 \ m^3/day}{80 \ gm/(capita.day)} = 3750 \ capita$$

Sanitary and Environmental Engineering

PART 2: WASTEWATER ENGINEERING

PART 2: WASTEWATER ENGINEERING

Lecture 8: Secondary Treatment or the Biological treatment

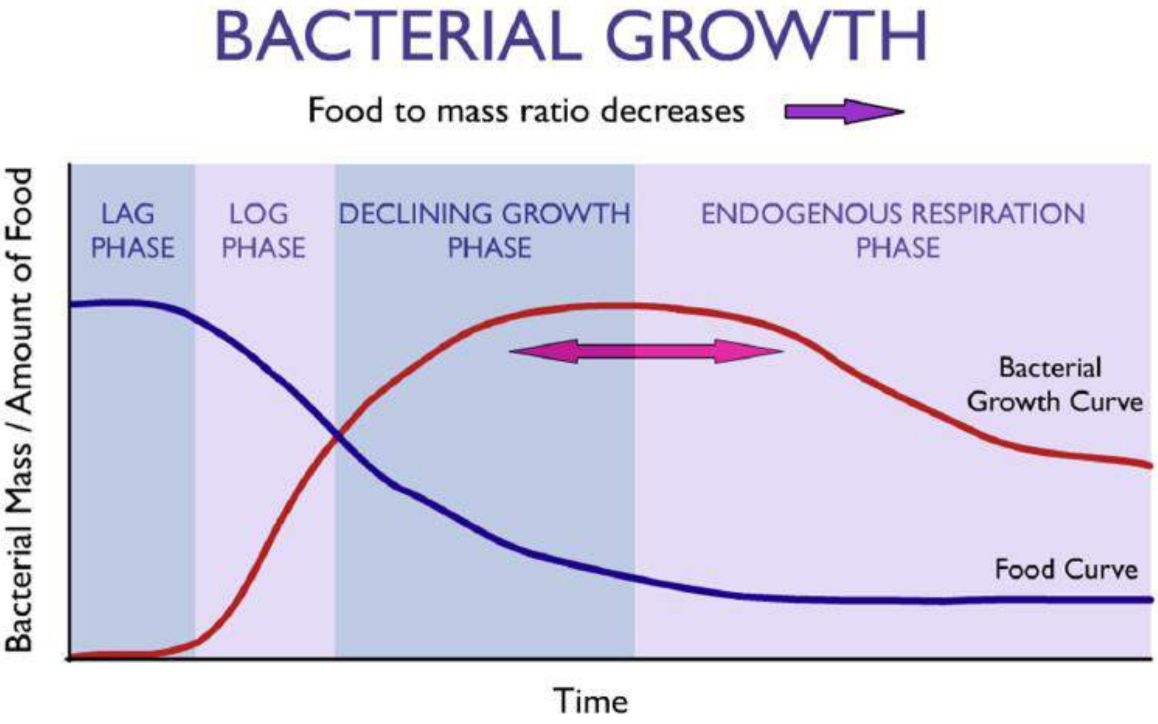
Biological treatment systems are intended to remove soluble and colloidal organic matter from the wastewater which remains after primary treatment. This removal is accomplished biologically using a variety of microorganisms principally bacteria. In this treatment the active microbial growth is in contact with the wastewater so that they may consume the organic matter as food. Biological treatment systems are designed to maintain a large active mass of bacteria within the system confines.

Basic requirements:

a) Mixed population of active microorganisms.
b) Good contact between the microorganisms and the waste (organic matter).
c) Availability of oxygen (for aerobic decomposition).
d) Availability of nutrients (N = 0.05 mg/BOD, P = 0.016 mg/BOD, also S, Na, K, Ca, Mg, Fe).
e) Favorable environmental conditions for microbial activity such as temperature and pH.

Bacterial Growth Pattern (Food-Biomass curve):

Bacteria are single-celled plants which metabolize soluble food and reproduce by binary fission. Bacteria are capable of solubilizing food particles outside the cell mean of extracellular enzymes, and hence can remove soluble, colloidal and solid organic matter from wastewater.



1) *Lag phase:* Time required for the microorganisms to acclimate with the environment.
2) *Log phase (exponential):* The microorganism growth will be at a maximum rate with a rapid consumption of food (organic matter).
3) *Stationary or declining growth phase:* The biomass concentration remains relatively constant where the growth is offset by the death of cells and the lack of food.
4) *Death or endogenous respiration phase:* No growth due to any food.

Summary:

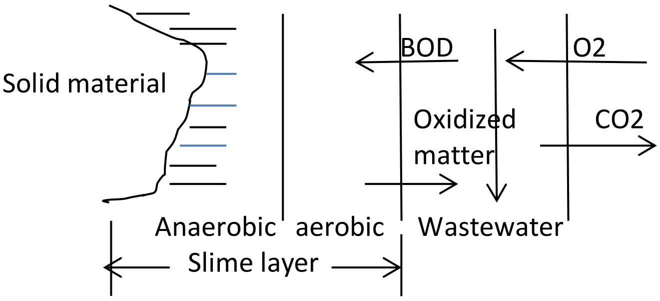
a) The stationary phase is generally used for biological system since the bacteria in the log growth phase of development have a great deal with energy available, have limited accumulation of waste products.
b) The extended aeration and sludge digestion are operated in death phase.

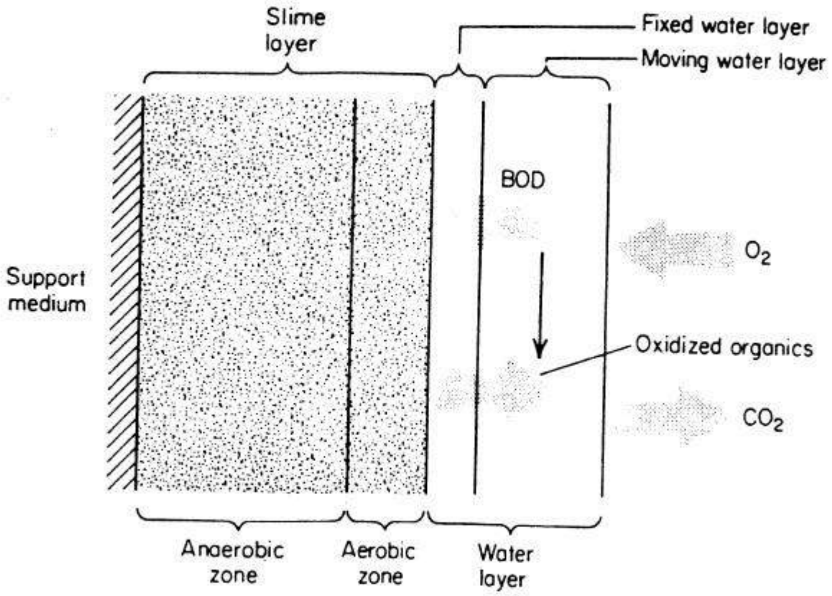
Techniques used in the biological treatment: Attached growth process, Suspended growth process

First: Attached Growth Process:

It is utilizing a solid medium upon which bacteria solids are accumulated in order to maintain a high population. The area available for such growth is an important design parameter, and a number of processes have been developed which attempt to maximize area as well as other limiting factors.

The microorganisms are attached to an inert packing material such as: rocks, gravel, sand, wood and plastic. The organic matter and nutrients are removed from the wastewater flowing through the packed material by the attached growth of microorganisms on the solid material known as the biofilm or slime layer. This illustrated in the figure:





Schematic diagram of attached growth process

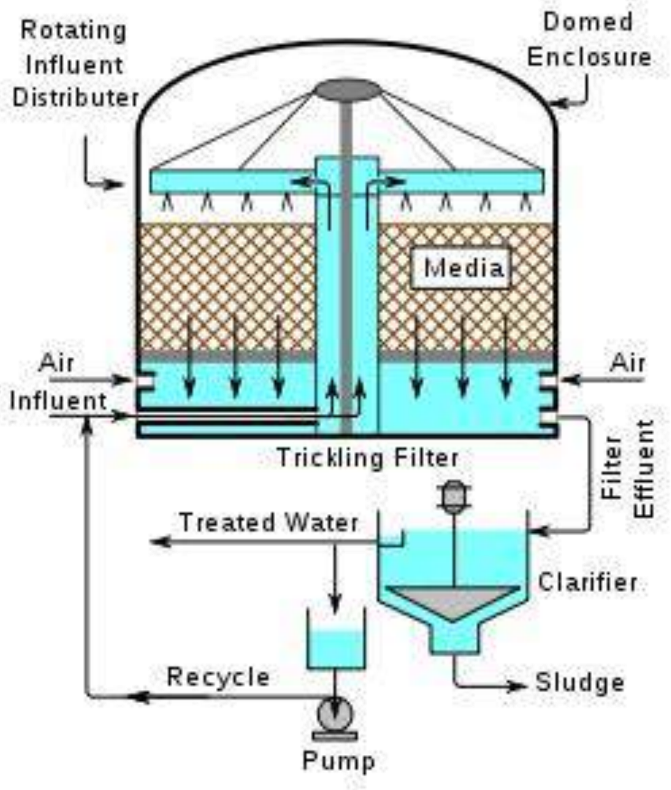
Second: Suspended Growth Process:

The microorganisms are maintained in suspension within the wastewater reactor by appropriate mixing methods (natural or mechanical methods). The microorganisms are mixed with the organic matter so they can grow to form an active mass of microbial flocs (biomass). In most processes the required volume is reduced by returning bacteria from the secondary clarifier in order to maintain a high solids concentration. It includes activated sludge and its various modifications, oxidation ponds and sludge digestion systems.

Attached Growth Process

Surface growth process include intermittent sand filter, trickling filters, rotating biological contactors, and a variety of similar proprietary devices. One of the treatment units designed by this technique is the Trickling Filter.

Trickling filters utilize a relative porous bacteria growth medium such as rock or formed plastic shapes.



Types of Trickling Filters: Low rate, High rate

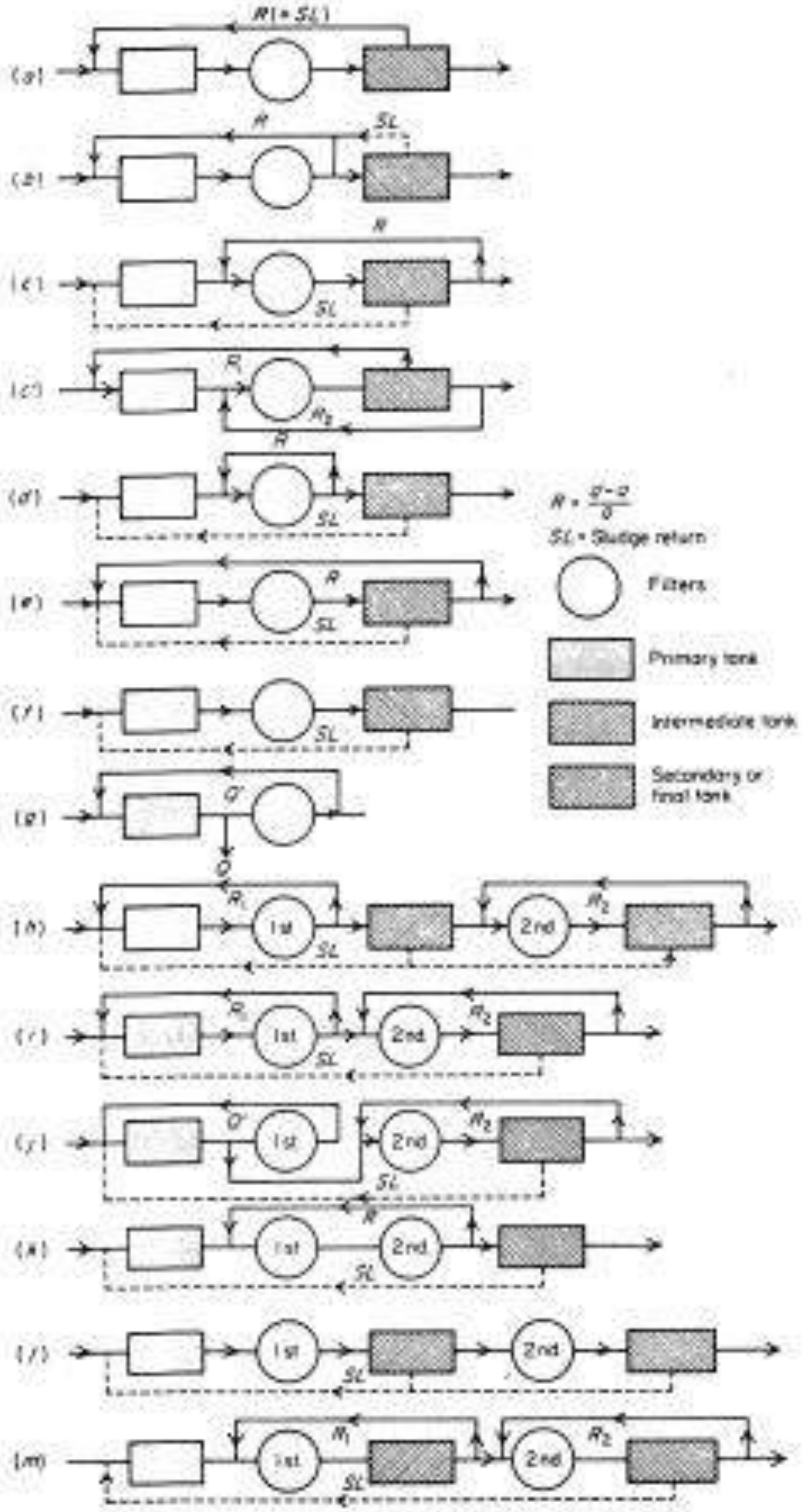
Item	Low Rate	High Rate
Depth (m)	1.5-3.0	1.0-2.0
Hydraulic loading ($m^3/m^2/d$)	1.0 – 4.0	10-40
Organic loading (kg BODu/m^3/d)	0.08 – 1.5	1.5-18.7
Recirculation ratio	0	1.0-3.0
BOD removal (%)	74 -80	80-85
Filter files	many	few

Advantages of recirculation:

a) Increase the biomass in the system;
b) Maintain uniform hydraulic and organic loadings;
c) Dilution of the influent with water of a better quality;
d) Thinning of the slime layer (biofilm).

Techniques of recirculation used in the operation process may vary widely. The different configurations being used are listed in Fig. 24-6 page 483. The basic forms depending on the flow pattern are:

1- Single stage, one Trickling filter is used.
2- Two stages, two filers are used in series.



Flow diagram of one- and two-stage trickling filter plants.

Mathematical model of Trickling filter system:

The design of Trickling filter is based on empirical formulas. One of these formulas is the National Research Council of USA (NRC) formula (based on collected data at military bases in USA during World War 2):

For the design of the single stage filter and the 1st stage of the two-stage system:

$$\frac{C_i - C_e}{C_i} = \frac{1}{1 + 0.532\sqrt{\frac{Q \times C_i}{V \times F}}}$$

Where:

C_i = BOD_5 of the influent (mg/L).

C_e = BOD_5 of the effluent (mg/L).

Q = Wastewater flow rate (m^3/min).

V = Volume of the filter (m^3).

F = Recirculation factor ($F = \frac{1+r}{(1+0.1r)^2}$).

r = Recirculation ratio ($r = \frac{Q_r}{Q}$).

Q_r = Recirculation flow rate.

For the second stage of the two stages filter the formula becomes:

$$\frac{C_e - C_e^{`}}{C_e} = \frac{1}{1 + \frac{0.532}{1 - \left(\frac{C_i - C_e}{C_i}\right)} \times \sqrt{\frac{Q \times C_e}{V^{`} \times F^{`}}}}$$

$C_e^{`}$ = BOD_5 of the effluent from the 2nd filter (mg/L).

$V^{`}$ = Volume of the 2nd filter (m^3).

$F^{`}$ = Recirculation factor of the 2nd filter.

Operating problems:

1. In cold weather a reduction in the efficiency removal is noticed. About 3% for each 1°C decrease in temperature. In such climates fiberglass covers are used to prevent ice formation and also to contain odors that may be produced in the filter.
2. Filter files accumulation breed in low rate filters but are less troublesome in high rate where the high hydraulic rates carry the larvae before they can mature. This problem is solved by flooding the filter for 24 hr with the addition of insecticide.

Solved problems

Problem 1: Calculate the effluent BOD_5 of a two-stage trickling filter with the following flows, BOD_5 and dimensions:

Q =3.15 m^3/min, BOD_5 = 290 mg/L

Volume of filter NO.1 = 830 m^3

Volume of filter NO.2 = 830 m^3

Filter depth = 2 m

Recirculation (filter NO.1) = 125 percent Q & r_1 = 125%

Recirculation (filter NO.2) = 100 percent Q & r_2 = 100%

Solution:

By using NRC formula:

$$\frac{C_i - C_e}{C_i} = \frac{1}{1 + 0.532\sqrt{\frac{QC_i}{VF}}}$$

For the first stage:

$$F_1 = \frac{1 + r_1}{(1 + 0.1r_1)^2} = \frac{1 + 1.25}{(1.125)^2} = 1.78$$

$$\frac{290 - C_e}{290} = \frac{1}{1 + 0.532\sqrt{\frac{3.15(290)}{830(1.78)}}}$$, So, C_e= 85.5 mg/L

For the second stage:

$$F_2 = \frac{1 + r_2}{(1 + 0.1r_2)^2} = \frac{2}{1.21} = 1.65$$

$$\frac{C_e - C_e'}{C_e} = \frac{1}{1 + \frac{0.532}{1 - \left[\frac{(C_i - C_e)}{C_i}\right]_i}\sqrt{\frac{QC_e}{V'F'}}}$$

$$\frac{85.5 - C_e'}{85.5} = \frac{1}{1 + \frac{0.532}{1 - [(290 - 85.5)/290]}\sqrt{\frac{3.15(85.5)}{830 \times 1.65}}}$$

$$C_e' = 38\,mg/L$$

If the effluent BOD_5, from the first stage is assumed to about 100 mg/L, the organic loading rate, including recirculation is:

$$Oraganic\ loading\ rate = Q_{Influent} \times BOD_{5Influent} + (Q_{Influent} + Q_r) \times BOD_{5Effluent}$$

$$Oraganic\ loading\ rate = 3.15\frac{m^3}{min} \times 290\frac{mg}{L} \times 10^{-3} + 3.15 \times 1.25\frac{m^3}{min} \times 100\frac{mg}{L} \times 10^{-3}$$
$$= 1.307\ kg/\min = 1882.44\ kg/day$$

The organic loading rate for each filter: $OLR = \frac{1882\ kg/day}{830\ m^3} = 2.27\frac{kg}{m^3}\ per\ day$

First stage efficiency:= $\frac{C_i - C_e}{C_i} = \frac{290 - 85.5}{290} \times 100 = 70.517\%$

Second stage efficiency:= $\frac{C_e - C_e'}{C_e} = \frac{85.5 - 38}{85.5} \times 100 = 55.55\%$

Overall stage efficiency:= $\frac{C_i - C_e'}{C_i} = \frac{290 - 38}{290} \times 100 = 86.89\%$

By using Velz formula:

$$C_i = \left(\frac{C_i + rC_e}{1+r}\right)e^{-KD}$$

, where:

K = 0.49 for high rate filters & 0.57 for low rate system.

D = filter depth (m)

For first stage:

$$C_e = \left(\frac{290+1.25C_e}{1+1.25}\right)e^{-0.49\times 2} \Rightarrow C_e = 61.1\,mg/l$$

For second stage:

$$C'_e = \left(\frac{61.1+1.0C'_e}{1+1.0}\right)e^{-0.49\times 2} \Rightarrow C'_e = 14.1\,mg/l$$

The Velz equation is valid for BOD removals of 90 percent or less.

Problem 2: Calculate the effluent BOD_5 of a two-stage trickling filter with the following flows, BOD_5 and dimensions, NRC formula? Q = 5000 m^3/day, influent BOD_5 = 280 mg/L, volume of first filter = 1000 m^3, volume of second filter = 800 m^3, filter depth = 2 m, recirculation rate for first filter = 1.5, recirculation rate for second filter = 1.25. Also, calculate organic loading rate (BOD_5 kg/day) and Hydraulic loading ($m^3/m^2/d$) for raw, primary and secondary filter, efficiency of each stage and overall removal efficiency?

Solution:

$$Q = 5000\,\frac{m^3}{day} = 3.472\,\frac{m^3}{min}$$

Organic loading rate for raw sewage: $OLR = 5000\,\frac{m^3}{day} \times 280\frac{mg}{L} \times 10^{-3} = 1400\,kg\,BOD_5/day$

First stage filter: $F_1 = \frac{1+r}{(1+0.1r)^2} = \frac{1+1.5}{(1+0.1\times 1.5)^2} = 1.89$

$$\frac{C_i - C_e}{C_i} = \frac{1}{1 + 0.532\sqrt{\frac{Q \times C_i}{V \times F}}}$$

$$\frac{280 - C_e}{280} = \frac{1}{1 + 0.532\sqrt{\frac{3.472 \times 280}{1000 \times 1.89}}} \rightarrow C_e = 77.328\,mg/L$$

$$\%\,Removal = \frac{280 - 77.324}{280} \times 100 = 72.384\%$$

Organic loading rate for filter 1: $OLR = Q_{in} \times C_{in} + Q_r \times C_r = 5000\,\frac{m^3}{day} \times 280\frac{mg}{L} \times 10^{-3} + 5000\,\frac{m^3}{day} \times 1.5 \times 77.328\,\frac{mg}{L} \times 10^{-3} = 1400 + 579.96 = 1979.96\,kg\,BOD_5/day$

Hydraulic loading rate: $HLR = \frac{Q+Q_r}{A_s} = \frac{5000\times 2.5}{\frac{1000}{2}} = 25\,\frac{m^3}{m^2.day}$

If need, Organic loading rate for effluent sewage from filter1: $OLR = 5000\ \frac{m^3}{day} \times 77.328\ \frac{mg}{L} \times 10^{-3} = 386.64\ kg\ BOD_5/day$

Second stage filter: $F_1 = \frac{1+r}{(1+0.1r)^2} = \frac{1+1.25}{(1+0.1\times1.25)^2} = 1.777$

$$\frac{C_e - \grave{C}_e}{C_e} = \frac{1}{1 + \frac{0.532}{1-\left(\frac{C_i - C_e}{C_i}\right)} \times \sqrt{\frac{Q \times C_e}{\grave{V} \times \grave{F}}}}$$

$$\frac{77.328 - \grave{C}_e}{77.328} = \frac{1}{1 + \frac{0.532}{1-\left(\frac{280 - 77.324}{280}\right)} \times \sqrt{\frac{3.472 \times 77.328}{800 \times 1.777}}} \rightarrow \grave{C}_e = 35.237\ mg/L$$

$$\%\ Removal = \frac{77.328 - 35.237}{77.328} \times 100 = 54.431\%$$

Organic loading rate for filter 2: $OLR = Q_{in} \times C_{in} + Q_r \times C_r = 5000\ \frac{m^3}{day} \times 77.328\ \frac{mg}{L} \times 10^{-3} + 5000\ \frac{m^3}{day} \times 1.25 \times 35.237\ \frac{mg}{L} \times 10^{-3} = 386.64 + 220.231 = 606.871\ kg\ BOD_5/day$

Hydraulic loading rate: $HLR = \frac{Q+Q_r}{A_s} = \frac{5000\times2.25}{\frac{800}{2}} = 28.125\, \frac{m^3}{m^2.day}$

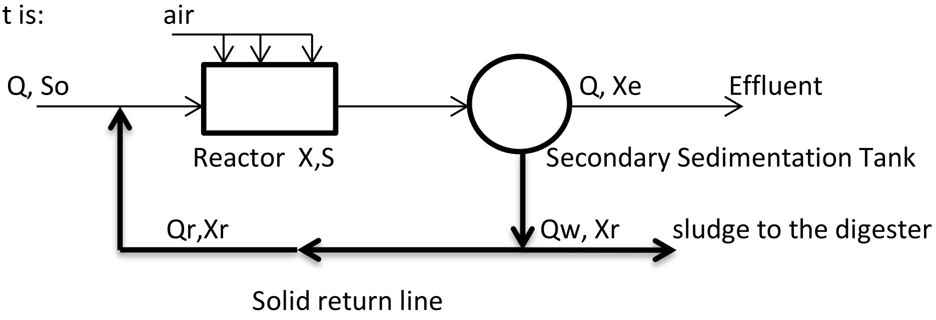
If need, Organic loading rate for effluent sewage from filter2: $OLR = 5000\ \frac{m^3}{day} \times 35.237\ \frac{mg}{L} \times 10^{-3} = 176.185\ kg\ BOD_5/day$

Overall Removal Efficiency: $\%\ Removal = \frac{280-35.237}{280} \times 100 = 87.415\%$

Suspended Growth Process

Suspended growth processes maintain an adequate biological mass in suspension within the reactor by employing either natural or mechanical mixing. In most processes the required volume is reduced by returning bacteria from the secondary clarifier in order to maintain a high solids concentration. Suspended growth processes include, Activated sludge and its various modifications, Oxidation ponds and Sludge digestion system.

One of the treatment units designed by this technique is the Activated Sludge Process. The flow diagram of this unit is:



Where:

Q = Wastewater flow rate (Influent).

Qr = Return (recycle) sludge flow.

Qw = Waste sludge flow.

X = Mixed liquor suspended solids concentration (MLSS).

Xr = underflow suspended solids concentration.

Xe = Effluent suspended solids concentration.

So = Influent BOD_5.

S = Effluent soluble BOD_5.

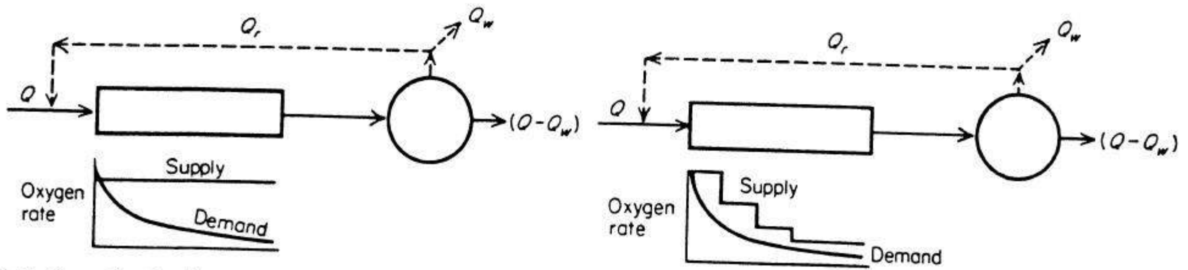
This unit consists of:

1. Reactor or the aeration basin, in which the microorganisms are kept in suspension by mixing or aerating the influent wastewater. The microorganisms grow in this tank to form the biomass which is known as the activated sludge.
2. Secondary sedimentation tank (clarifier) in which the biomass (activated sludge) is settled by gravity.
3. Solid return (recycle) system which is to return a portion of the settled sludge from the bottom of the secondary sedimentation tank back to the reactor as (Qr).

Note: All the settled sludge is returned but a portion is wasted (Qw). The effluent from the secondary sedimentation tank is usually disposed into the river after disinfection but within environmental limits. This effluent could be reused in irrigation or industrial purposes.

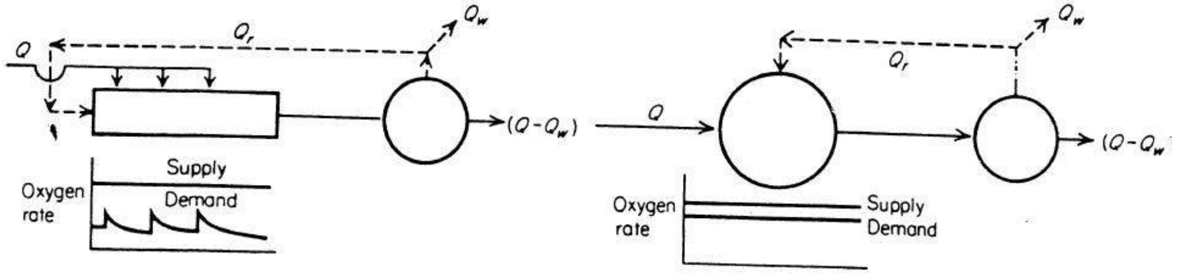
Types of activated sludge processes:

a) Conventional process, the return sludge is mixed with the influent at the head of the reactor. Air is provided uniformly along the reactor.
b) Tapered aeration process, as in 1 but matching of the oxygen supply with the demand is done by introducing more air at the end of the reactor.
c) Step aeration process, distributing the influent flow to a number of inputs along the reactor. This is done to avoid high oxygen demand.
d) Complete mixed process, the influent flow, return sludge and air are applied uniformly at several input points throughout the reactor.



(a) Conventional activated sludge

(b) Tapered aeration activated sludge



(c) Step aeration activated sludge

(d) Completely mixed activated sludge

Activated sludge configuration and effect upon oxygen supply

Operating problems of activated sludge systems:

1. Floating sludge: Air bubbles of nitrogen gas will raise the sludge to the surface of the reactor. This is caused by high retention time where the sludge remains too long in the reactor. Here the bacteria will reduce the nitrate to nitrogen gas (denitrification process). This problem is solved by increasing Qr to reduce the detention time or by increasing Qw to reduce the sludge age.
2. Bulking sludge: Presences of microorganisms which will enter large amounts of water in their cells due to the lack in nutrients or low pH. This problem is solved by providing the lack in nutrients and a neutral pH. The require nutrients are shown in the table.

Elements	N	P	S	Na	K	Ca	Mg	Fe	other
Concentration mg/L	0.05	0.016	0.004	0.004	0.003	0.004	0.003	0.001	trace

Mathematical model of activated sludge systems:

The basic factor in design, control, and operation of suspended growth systems is the mean cell residence time or sludge age (θ_c), which define by:

$$\theta_c = \frac{X}{\left(\frac{\Delta X}{\Delta t}\right)}$$

Where:

X: Total microbial mass in the reactor.

$\frac{\Delta X}{\Delta t}$: Total quantity of solids withdrawn daily.

θ_c^m: The lowest value of sludge age (θ_c) at which operation is possible.

At retention times θ less than θ_c^m organisms are removed more quickly than they are synthesized, the failure will occur $\theta < \theta_c^m$.

θ_c^d: The design value of sludge age (θ_c) and must be significant greater than (θ_c),$\left(\theta_c^d > \theta_c\right)$

$\frac{\theta_c^d}{\theta_c^m}$: The safety factor of the system should be at least 4. In standard process, the safety factor of 20 or more.

For equilibrium system, the quantity of solids produced must equal that lost, and quantity produced per day given by:

$$\mu = \frac{\hat{\mu} \times S}{K_s + S} - k_d$$

Where:

μ: Net specific growth rate or growth per unit mass per unit time.

$\hat{\mu}$: Maximum rate of growth.

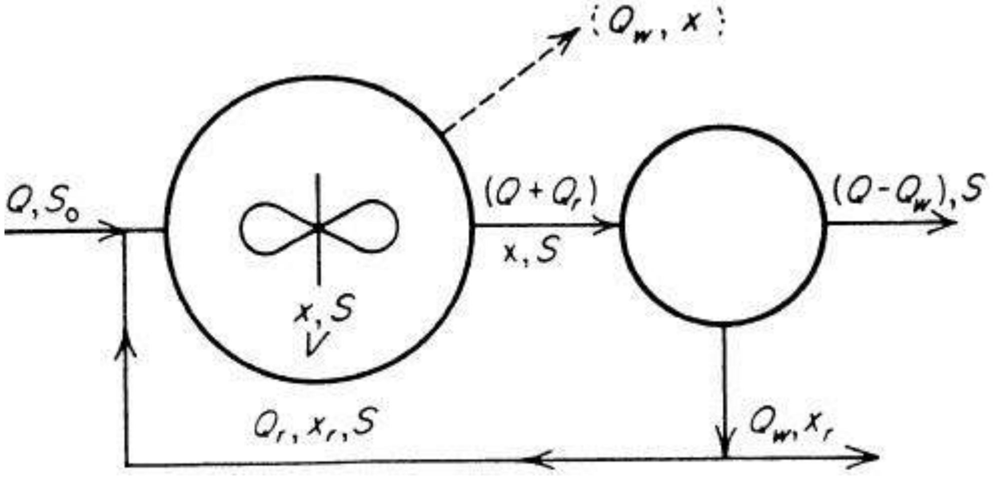
S: Concentration of substrate surrounding the microorganisms.

K_s: Half velocity constant, and

k_d: Microorganisms decay coefficient (mass/unit mass/time) reflecting the endogenous burn-up of cell mass.

Completely mixed process with solids recycle:

All waste utilization occurs in the biological reactor and that the total biological mass in the system is equal to the biological mass in the reactor (mass balance).



Completely mixed biological reactor with solids recycle.

The mean cell residence time, or sludge age:

1) If solids are wasted from the clarifier as shown in the solid line, rather than from the reactor:

$$\theta_c = \frac{X}{\left(\frac{\Delta X}{\Delta t}\right)} = \frac{xV}{Q_w \times x_r + (Q - Q_w)x_e}$$

Where:
Q: Waste flow rate,
x: Mixed liquor suspended solid concentration (MLSS).
x_r: Clarifier underflow suspended solids concentration.
xe: Effluent suspended solids concentration.
Qw: Waste sludge flow rate, and
Qr: Return sludge flow rate.

2) If solids are wasted from the reactor as shown in the dashed line, rather than from the clarifier:

$$\theta_c = \frac{xV}{Q_w \times x + (Q - Q_w)x_e}$$

Lawrence and McCarty have shown the total microbial mass in the reactor is given by:

$$xV = \frac{Y \times Q \times (S_0 - S) \times \theta_c}{1 + k_d \times \theta_c} \quad \& \; x = \frac{\theta_c}{\theta} \times \frac{Y \times (S_0 - S)}{1 + k_d \times \theta_c}$$

Where:
Y: Growth yield coefficient relating cell yield to the material metabolized.
S_0: Influent BOD_5.
S: Effluent soluble BOD_5
θ: Liquid retention time in the reactor, $\theta = \frac{V}{Q}$

Secondary Sedimentation Tank

a) In trickling filters: is used to remove large particles of sloughed slime layer. Its design is similar to Primary Sedimentation tanks. For design requirements:
SOR 25 – 33 m/d and not to exceed 50 m/d at peak flow.

b) In activated sludge process: is used to contribute BOD of the effluent and for thickening the sludge to maintain the required sludge age. For design requirements:
Solid loading rate = 2.5 -6.2 kg/m^2/hr at average flow and not to exceed 10.2 kg/m^2/hr at peak flow
SOR = 16 – 32 m/d at average flow and 40 – 64 m/d at peak flow
Weir loading 125 – 375 m^3/m /d
Depth = 2.5 – 6 m

Solved problems

Problem 1: Design an activated sludge process to yield an effluent of BOD_5 20 mg/L and suspended solids 25 mg/L. The influent BOD_5 after primary treatment is 160 mg/L. The wastewater flow is 10 m^3/min, MLSS =2500 mg/L, underflow SS concentration = 1.5%, yield coefficient (Y) = 0.65, sludge age, $\theta_c = 10\ day$, decay coefficient (k_d) = 0.05/day, BOD_5 of SS effluent = 63% of SS effluent and the solids are 80% volatile.
Solution:

1. The BOD_5 of the effluent solids can be estimated to be 0.63 ($SS_{Effluent}$),
   The soluble effluent BOD_5 must thus be reduced to (20-0.63*25 = 4 mg/L)

2. The total biological mass in the reactor is calculated from Eq. (24-11):

$$xV = \frac{YQ\theta_c(S_o - S)}{1 + k_d\theta_c}$$

Where:
xV: Microbial mass (mg).
V: Volume of the reactor (m^3).
x = Mixed liquor suspended solids concentration (MLSS).
Y: Growth yield coefficient ($\frac{gm\ biomasses\ generated}{gm\ BOD\ or\ COD\ consumed}$).
θ_c: Sludge age or mean cell residence time (day).
k_d: Decay coefficient (day^{-1}).
$S = BOD_5\ of\ the\ effluent - BOD_5\ of\ the\ SS\ effluent.$
θ: Liquid retention time in the reactor ($\theta = \frac{V}{Q}$).

$$xV = \frac{YQ\theta_c(S_o - S)}{1 + k_d\theta_c} = \frac{0.65 \times (10 \times 1000 \times 1440\ L/day) \times (10) \times (160 - 4)\ mg/L}{1 + 0.05 \times (10)} = 9.734 \times 10^9\ mg$$

$$V = \frac{9.73 \times 10^9}{2500} = 3.894 \times 10^6\ L = 3894\ m^3$$

3. The rate of sludge (biological solids) production is obtained from Eq. (24-6) and is:

$$:\frac{dX}{dt} = \frac{xV}{\theta_c} = \frac{9.73 \times 10^9\ mg}{10\ day} = 9.73 \times 10^8\ mg/day = 973.5\ kg/day$$

The solids are 80 percent volatile (organic),

Total production $= \frac{973.5}{0.8} = 1217\ kg/day$

4. The waste sludge flow rate:

Note: ($1\%\ Concentration = 10000\frac{mg}{L}$ & $1.5\%\ Concentration = 15000\frac{mg}{L}$)
The underflow solids concentration is unlikely to exceed 15,000 mg/L and could be less. Assume xr = 15000 mg/L,

$$Q_w = \frac{Total\ Solid\ Production}{x_r}$$

$$Q_w = \frac{1217 \times 10^6\ mg/day}{15 \times 10^3\ mg/L} = \frac{1217 \times 10^6\ mg/day}{15 \times 10^6\ mg/m^3} = 81.1\ m^3/day$$

5. The recirculation flow can be calculated from equation (24-10):

$$Q_r \times x_r = (Q + Q_r)x \Rightarrow Q_r = \frac{Q \times x}{x_r - x} = \frac{10\ m^3/\min \times (2500)\ mg/L}{(15000 - 2500)\ mg/L} = 2\ m^3/\min$$

$$r=\frac{Q_r}{Q}=\frac{2}{10}=0.2$$

6. The hydraulic retention time in the reactor:

$$\theta=t=\frac{V}{Q}=\frac{3.894\times10^{6}L}{10\times1000\times1440\,L/day}=0.27\,days=6.5\,hour$$

7. The oxygen demand required:

The required amount of oxygen which must be provided in suspended growth process is equal to the difference between the ultimate BOD of the waste which is removed and the ultimate BOD of the solids which are wasted.

For ordinary domestic sewage this may be taken as equal to:

$$\mathrm{O_2\ demand}=1.47\times(S_O-S)\times Q-1.14\,X_r\,Q_w$$

$$O_2\,demand=1.47\times(160-4)\times(14.4\times10^{6})-1.14\times15000\times81100=1.915\times10^{9}$$
$$=1915\ kg/day$$

The volume of air required at STP in m^3 may be calculated from:

Assume $\rho_{air}=1.2\ kg/m^3$ and if O_2=23.2% in air

Flow rate of air, $Q_{air}=\frac{\mathrm{O_2\ demand}}{0.232\times1.2}=\frac{\mathrm{O_2\ demand}}{0.278}=\frac{1915}{0.278}=6856\,m^3/day$

If the actual efficiency of oxygen transfer is 7% (7% O_2 mass transfer in water)

Volume of air required or Actual Air Required = $\frac{6856\,m^3/day}{0.07}=97943\,m^3/day=68\,m^3/\mathrm{min}$

The air volume required per unit of BOD_5 removed equal:

$$\frac{97943\,m^3/day}{(160-4)\times10^{-3}kg/m^3(10\times1440)m^3/day}=43.6\,m^3/kg$$

8. Compressor Power requirements to provid airflow:

$$P=\frac{w\times R\times T_1}{8.41\times e}\times\left[\left(\frac{P_2}{P_1}\right)^{0.283}-1\right]$$

P: the power required (kW),
w: the air mass flow (kg/sec),
R: the gas constant (8.314),
T: the inlet temperature (^{0} K),
P_1: the absolute inlet pressure in atm,
P_2: the absolute outlet pressure in atm,
e: the efficiency of the machine (70-80%).

$$Oxygen\ required=\frac{1915\ kg/day}{0.07\times0.232}=117918.72\ kg/day$$

Assume inlet air temperature = 30 ^{0}C=303 ^{0}K
Assume P1 =1atm & P2=1.58 atm

$$P=\frac{w\times R\times T_1}{8.41\times e}\times\left[\left(\frac{P_2}{P_1}\right)^{0.283}-1\right]=\frac{0.71\times8.314\times303}{8.41\times0.75}\times\left[\left(\frac{1.58}{1}\right)^{0.283}-1\right]=39.189\ kW$$

9. Organic loading rate:

$$BOD_u=\frac{BOD_{5Influent}}{1-e^{-k_1\times t}}=\frac{160\ mg/L}{1-e^{-0.23\times5}}\approx235\ mg/L$$

$$Organic\ LoadingRate=\frac{BOD_u\times Q}{V}=\frac{235\times10^{-3}\frac{kg}{m^3}\times10\times1440m^3/day}{3894\ m^3}\approx0.87\frac{kg}{m^3.day}$$

Problem 2: Design a circular secondary sedimentation tank for the reactor of the active sludge process in the last example. Assume depth of the tank = 2.5 m and solid loading rate = 4 kg/m^2/hr?

Solution:

1. Total solids flowing into the secondary sedimentation tank is calculate from equation:

$$x \times (Q + Q_r) = 2500\frac{mg}{L} \times \frac{10(1+0.2)\,m^3}{min} = 2500\,\times \frac{10^{-3}\,kg}{m^3} \times 12 \times \frac{60\,m^3}{hr}$$
$$= 1800\,kg/hr$$

2. Surface area of SST:

$$A_s = \frac{Total\ Solids}{Solid\ Loading} = \frac{1800\,kg/hr}{4\,kg/m^2.hr} = 450\,m^2$$

SST diameter,

$$A_s = 450\,m^2 = \frac{\pi}{4}D^2 \Rrightarrow D \approx 24\,m$$

3. Surface overflow rate of SST (SOR):

$$SOR = \frac{Q}{A_s} = \frac{10 \times 1440\,m^3/day}{450\,m^2} = 32\,m/day$$

4. Weir loading rate of SST (WLR):

$$WLR = \frac{Q}{\pi D} = \frac{10 \times 1440\,m^3/day}{\pi \times 24\,m} = 191\,m^3/m/day$$

5. Detention time:

$$t = \theta = \frac{V}{Q} = \frac{450 \times 2.5m^3}{10 \times 60\,m^3/hr} = 1.875\,hr$$

Problem 3: An activated sludge plant treats a wastewater flow of 5000 m^3/day with a BOD$_5$ of 180 mg/L. the organic loading rate is 0.3 kg BOD$_5$ per kg MLSS and the hydraulic retention time is 5 hours. Determine the concentration of MLSS in mg/L?

Problem 4: If the reactor in problem 17.8 is operated at sludge age of 10 days determine the sludge production rate. If the sludge is concentrated in the clarifier underflow to 1.5 percent solids content, what will be the waste sludge flow?

Problem 5: Design an activated sludge process to treat a waste flow of 15000 m^3/day with a BOD$_5$ of 180 mg/L following primary treatment. The effluent BOD$_5$ and SS are to be less than 20 mg/L. Assume x_r = 15000 mg/L, x = 2500 mg/L, $\theta_c = 10\ days$, Y = 0.60, k_d = 0.05. determine the reactor volume, the sludge production rate, the circulation rate, the hydraulic retention time, and the oxygen required?

Problem 6: A wastewater has an average BOD5 of 250 mg/L. The average flow is 8000 m^3/day, the minimum 3000 m^3/day, and the maximum 13500 m^3/day. Design a complete activated sludge system, including primary and secondary clarifiers, to yield an effluent BOD5 and SS of 30 mg/L at peak flow. What do you estimate the effluent BOD5 will be at average flow? Assume xr = 1percent (min), MLSS = 3000 mg/L, $\theta_c = 7.5\ days$ (min), Y = 0.65, and k_d = 0.04.

Sanitary and Environmental Engineering

SANITARY LABORATORY EXPERIMENTS

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Experiment 1: Turbidity

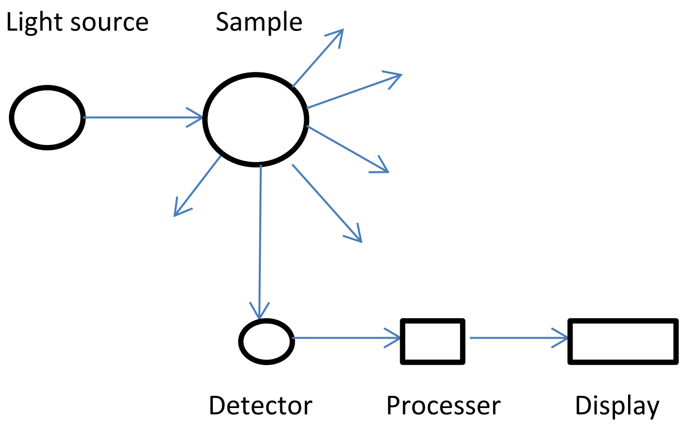
Turbidity is the water property containing suspended solids (SS) that prevents light from moving in straight lines through the sample.

Sources of turbidity

1) Living or dead organisms (algae, fungi, bacteria).
2) Silt and clay.
3) Organic matter from the disposal wastewater into water bodies.

Method of determination: Nephelometric

Turbidity is directly proportional to the intensity of the scattered light



The result is recorded in the display in NTU units (Nephelometric turbidity unit).

Experiment 2: pH

pH is an expression to identify the intensity of the acid or base in a solution which represents the concentration of hydrogen ion or hydrogen activity.

The ionization reaction of water: $H_2O \longrightarrow H^+ + OH^-$

Where the equilibrium is: $Kw = (H^+)\ (OH^-)\ /\ (H_2O)$ eq 1 ( ) = chemical activity, $(H_2O) = 1$

Kw = equilibrium constant = 10^{-14} mole/L at 25 °C

The activity ( ) is also expressed as concentration [ ] so eq 1 is written as

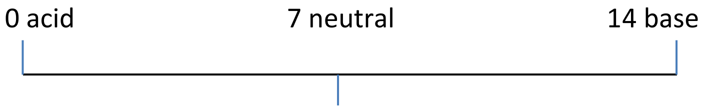
$[H^+]\ \ [OH^-] = 10^{-14}$

$\log[H^+] + \log[OH^-] = \log 10^{-14} = -14$

$-\log[H^+] - \log[OH^-] = 14$

Let –log = p

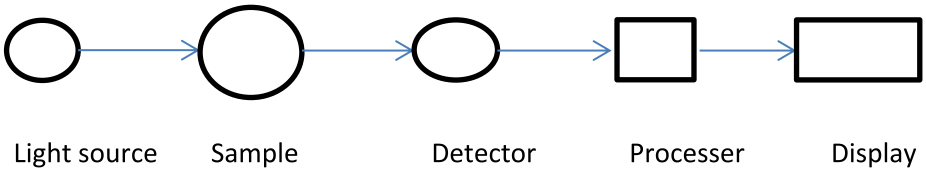
pH + pOH = 14



Methods of determination:

1. *Colemetric method by using indicators:*
   Indicator: organic acid or base compounds which undergo color changes in solutions of different pH.
2. *Photometric:*
   A reagent (chemical compound) is added to the sample where it takes a degree of color that is proportional to the concentration of the parameter to be measured.

The light energy passing through the colored sample with a specific wave length will be absorbed by the tested substance (measured parameter). The photometer determines the coloration by measuring the absorption or transmission of the light by the detector and then through the processer which will calculate the concentration. The result is then recorded in the display.



3. Electrometric method: By using an electrode sensitive to H^+ ions in the pH meter

Experiment 3: Color

Sources of color: present of some ions (Fe, Mg, Mn), organic material, soil particles and industrial and domestic sewage.

Types: 1-True color caused by dissolved matter which can be removed by chemical processes.

2- Apparent color caused by suspended colloidal matter and could be removed by sedimentation and/ or filtration.

Methods of determination

1- Color comparison with standard colors of potassium chloro platinum K_2PtCl_6 , using the lavibond device.

Put 50 mL of the sample in the nessler tube.

Put the tube on the right side in the device.

Put an empty tube on the left side in the device.

Compare the color of the sample with the color on the hazen disc (K_2PtCl_6 colors) and read the result which is expressed as unit color.

Note: if the reading is more than 70 then the sample should be diluted with distilled water.

Color = reading x dilution factor

2- Photometric method, using the wave length for color determination.

Experiment 4: Total Solids, Total Suspended Solids, and Total Dissolved Solids

Is the matter (suspended and dissolved solids) that remain as residue upon evaporating and drying of a known volume of a water sample, at 103 -105 ^{o}C for 24 hrs. It is expressed as TS and reported in mg/L.

Methods of determination:

1. Weight a clean pan (A1) gm.
2. Put a measured volume of sample (Vs) mL.
3. Put the pan in the drying oven at 103-105 ^{o}C for 24 hrs .
4. Weight the pan and residues after drying (A2) gm.
5. Calculate concentration of total solids mg/L, $TS = \frac{(A2-A1)}{Vs} \times 10^6$

Total solids (TS) = Total dissolved solids (TDS) + Total suspended solids (TSS)

Total dissolved solids (Filterable): Consist mainly of inorganic salts present in water and any amounts of dissolved organic matter. It is expressed as TDS and reported as mg/L. It refers to the material that passes through filter paper (no.1) and the filtrated water is dried at 103 -105 ^{o}C for 24 hrs .

Total suspended solids (nonfilterable): The non-dissolved matter such as silt, clay and microorganisms resent in water. It is expressed as TSS and reported as mg/L. It refers to the material that remains on the filter paper and after drying at 103 -105 ^{o}C for 24 hrs .

Methods of determination

1. Weight a clean pan (B1) gm and a clean dry filter paper (no.1) (C1) gm.
2. Filter a measured volume of sample (Vs) mL through the filter paper.
3. Put the pan and filter paper in the drying oven at 103-105 ^{o}C for 24 hrs .
4. Weight the pan and residues after drying (B2) gm.
5. Weight the filter paper and residues after drying (C2) gm.
6. Calculate concentration of total dissolved solids mg/L, $TDS = \frac{(B2-B1)}{Vs} \times 10^6$
7. Calculate concentration of total suspended solids mg/L, $TSS = \frac{(C2-C1)}{Vs} \times 10^6$

Note: The hot pans and filter papers should be kept in a desiccator to cool out before weighing.

Example: Total solids (TS)

A1 = 60.05 gm, A2 = 60.0773 gm, Vs = 50 ml

$$TS = \frac{(A2 - A1)}{Vs} \times 10^6 = \frac{(60.0773 - 60.05)}{50} \times 10^6 = 546\, mg/L$$

Example: Total dissolved and suspended solids (TDS, TSS)

B1 = 60.065 gm, B2 = 60.0894 gm

C1 = 1.005 gm, C2 = 1.0081 gm

Vs = 50 mL

$$TDS = \frac{(B2 - B1)}{Vs} \times 10^6 = \frac{(60.0894 - 60.065)}{50} \times 10^6 = 488\, mg/L$$

$$TSS = \frac{(C2 - C1)}{Vs} \times 10^6 = \frac{(1.0081 - 1.005)}{50} \times 10^6 = 62\, mg/L$$

$$TS = TDS + TSS = 488 + 62 = 550\, mg/L$$

Experiment 5: Electrical Conductivity

It is a numerical expression to represent the ability of a water sample to carry an electrical current. It depends on the concentration and type of the ions existing in the water (TDS). Temperature of water affects the movement and direction flow of the ions, as 1°C increase causes 2% increase in conductivity.

The standard unit of electrical resistance is ohms (Ω), and the conductivity is the reciprocal of the resistivity. It is expressed as 1/ ohms. cm or mhos/cm. in practical the unit used is µmhos/cm (mhos/cm = 10^6 µmhos/cm). Fresh distilled water has 0.5 – 2 µmhos/cm conductivity.

This measurement is used as a rapid estimation of TDS in a water sample:

TDS mg/L= f x conductivity µmhos/cm

Where f is a factor that depends on the ionic component in the solution and on temperature,

f = 0.55 – 0.9.

Methods of determination

Conductivity is measured by an electrical meter, using an electrode sensitive to the ionic concentration of the water.

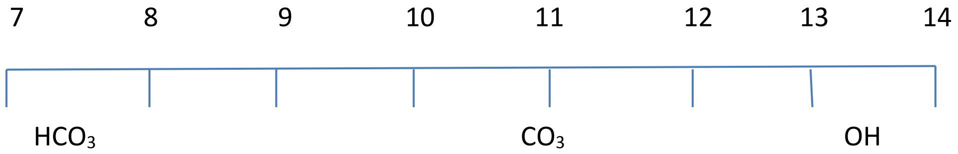
Experiment 6: Alkalinity

Is the quantitative capacity required to neutralize a strong acid to a normal pH and is expressed in concentration mg/L as $CaCO_3$.

Sources of alkalinity

1. The presences of bicarbonate (HCO_3) and carbonate (CO_3) in water from natural weathering of soil.
2. The presences of silicate and phosphate salts.
3. The addition of lime (CaO), soda ash (Na_2CO_3) in water treatment plant.
4. The action of algae in consuming (CO_2) by photolysis which will increase the pH.

pH range of alkalinity is more than 7



Method of determination:

By titration with a standard solution (acid) like (HCl or H_2SO_4), 0.02 N using two indicators, phenolphthalein (ph) and methyl orange (mo).

Titration is the process to determine the amount of the substance to be measured which will completely react with a standard solution. The reaction will end at an equivalent point which is indicated by the indicator added that changes it color at this point.

Standard solution: Whose strength or reacting value per unit volume is known, such as: Molar solution (M) which contains one molecular weight of a substance in one liter of water ($Molarity = \frac{Molecular\ weight}{Volume}$).

Normal solution (N) which contains one equivalent weight of the substance in one liter of water ($Normality = \frac{Equivalent\ weight}{Volume}$).

$$Equivalent\ weight = \frac{Molecular\ weight}{Equivalent\ charge}$$

1. Take 10 mL sample (volume of sample Vs).
2. Add 2 drops ph indicator.
3. Titrate with 0.02 N acid read volume used to reach end point (from pink to colorless) Vph.
4. Add 2 drops mo indicator.
5. Titrate with 0.02 N acid read volume used to reach end point (from yellow to orange) Vmo.

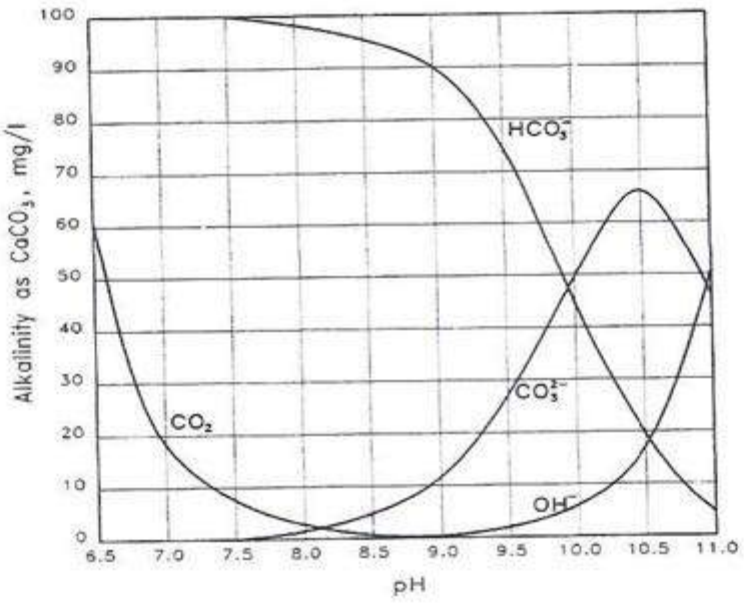
Calculate alkalinity

Alkalinity ph as mg/L CaCO$_3$, $\mathrm{p} = \frac{\mathrm{v_{ph}} \times \mathrm{N} \times \text{Equivalent weight of } \mathrm{CaCO_3} \times 10^3}{\mathrm{V_s}}$

Total alkalinity as mg/L CaCO$_3$, $\mathrm{T} = \frac{(\mathrm{v_{ph}} + \mathrm{v_{mo}}) \times \mathrm{N} \times \text{Equivalent weight of } \mathrm{CaCO_3} \times 10^3}{\mathrm{V_s}}$

Table to identify alkalinity in water to the titration results:

Titration result	OH alkalinity mg/L CaCO$_3$	CO$_3$ alkalinity mg/L CaCO$_3$	HCO$_3$ alkalinity mg/L CaCO$_3$
P = zero	0	0	T
P less than T/2	0	2P	T-2P
P = T/2	0	2P	0
P more than T/2	2P-T	2(T-P)	0
P = T	T	0	0



Experiment 7: Jar Test

The jar test is a common laboratory procedure used to determine the optimum operating conditions for water or wastewater treatment. This method allows adjustments in pH, variations in coagulant or polymer dose, alternating mixing speeds, or testing of different coagulant or polymer types, on a small scale in order to predict the functioning of a large-scale treatment operation.

Jar Testing Apparatus

The jar testing apparatus consists of six paddles which stir the contents of six 1 liter containers. One container acts as a control while the operating conditions can be varied among the remaining five containers. A rpm gage at the top-center of the device allows for the uniform control of the mixing speed in all of the containers.

Jar Test Procedure

The jar test procedure involves the following steps:

1. Fill the jar testing apparatus containers with sample water. One container will be used as a control while the other 5 containers can be adjusted depending on what conditions are being tested. For example, the pH of the jars can be adjusted or variations of coagulant dosages can be added to determine optimum operating conditions.
2. Add the coagulant to each container and stir at approximately 100 rpm for 1 minute. The rapid mix stage helps to disperse the coagulant throughout each container.
3. Reduce the stirring speed to 25 to 35 rpm and continue mixing for 15 to 20 minutes. This slower mixing speed helps promote floc formation by enhancing particle collisions which lead to larger flocs.
4. Turn off the mixers and allow the containers to settle for 30 to 45 minutes. Then measure the final turbidity in each container.

Residual turbidity vs. coagulant dose is then plotted and optimal conditions are determined. The values that are obtained through the experiment are correlated and adjusted in order to account for the actual treatment system.

Experiment 8: Hardness

Methods of determination:

1-The calculation method

Based on water analysis for cations concentrations causing hardness (which could be determined by the photometric method). The hardness is calculated by summing the concentrations of these cations after multiplying each cation by a factor. The result is recorded as:

Element	equivalent wt.
$CaCO_3$	50
M^+	
Ca	20
Mg	12
Fe	26
Mn	27
Al	9
Zn	32

Total hardness mg/Las $CaCO_3 = \sum M^+ \text{ X } \text{Factor}$

$$Factor = \frac{Equivalent\ weight\ of\ CaCO_3}{Equivalent\ weight\ of\ M^+}$$

2-Titration with EDTA

Take 10 mL sample (volume of sample Vs).

Add 10 drops of buffer solution (to make the pH about 10 for the reaction).

Add 2 drops indicator (Eriochrom black T).

Titrate with 0.01 M EDTA read volume (V) used to reach end point (from red to blue).

$$Total\ hardness \frac{mg}{L} as\ CaCO_3 = \frac{V \times B \times 10^3}{V_s}$$

B = mg $CaCO_3$ which is equivalent to 1 ml EDTA =1

Relationship between hardness and alkalinity

1-If total hardness > total alkalinity

Then carbonate hardness = total alkalinity (alkalinity is HCO_3 and CO_3)

Non-carbonate hardness = total hardness – total alkalinity

2-If total hardness < total alkalinity

Then total hardness = carbonate hardness (due to HCO_3)

Alkalinity due to HCO_3 and CO_3 = total hardness

Alkalinity due to OH = total alkalinity - total hardness

Water Quality analysis

The analytical measurements for inorganic elements in a water sample, should qualify the following equation:

The sum of the positive ions (cations) must equal the sum of the negative ions (anions) in the solution:

$$\sum \text{cations} = \sum \text{anions}$$

Check the accuracy of the chemical analysis of the sample is done by calculating the percentage difference which is determined by:

$$Percentage\ difference = \frac{(\sum Cations - \sum Anions)}{(\sum Cations + \sum Anions)} \times 100$$

Where the cations and anions in the sample should be in meq / L

The acceptance criteria is given as

∑ anions meq/L	Percentage difference %
0 -3.0	+- 0.2
3.0 -10.0	+- 2
10 -800	+- 5

Problem: The following analysis has been completed for a water sample. Check the accuracy of the analysis.

Cation	Concentration mg/L	Anion	Concentration mg/L
Ca	42	HCO_3	115
Mg	8.6	SO_4	29
Na	10	Cl	26
K	1.3	NO_3	1.0

Solution:

Cation	Concentration mg/L	Eq.wt. mg/meq	meq/L	Anion	Concentration mg/L	Eq.wt. mg/meq	meq/L
Ca	42	20	2.1	HCO_3	115	61	1.89
Mg	8.6	12	0.72	SO_4	29	48	0.6
Na	10	23	0.43	Cl	26	35.5	0.73
K	1.3	39	0.03	NO_3	1.0	62	0.02
∑ cations			3.28	∑ anions			3.24

$$Percentage\ difference = \frac{(\sum Cations - \sum Anions)}{(\sum Cations + \sum Anions)} = \frac{(3.28 - 3.24)}{(3.28 + 3.24)} = 0.46\%$$

The analysis is acceptable as ∑ anions is in the range 3 -10 and % difference is less than 2.

Experiment 9: Chlorine (Residual)

The determination of chlorine is to ensure the activity of the chlorination process and to determine the residual chlorine concentration in the treated water which is supplied to the consumer.

Methods of determination:

Photometric:

A reagent (DPD) is added to the sample where it takes a degree of color which is proportional to the concentration of the chlorine to be measured. Put the colored sample in the device. Set the device on chlorine and wait for the sign. The result is recorded in the display as mg/L chlorine.

Add DPD_1 + DPD_2 to the sample to determine the residual as free chlorine. Then add DPD_3 to the same sample to determine total residual chlorine. So,

Combined chlorine = Total chlorine – Free chlorine.

Experiment 10: Chloride

Chloride in the form of Cl^{-1} ion is the major anion in water and wastewater. This ion is considered one of the pollution indicators, as one person may produce 15 mg/L chloride in domestic sewage.

Methods of determination

Argentometric method: titration with silver nitrate ($AgNO_3$)

Take 10 ml water sample(Vs)

Add 2 drops potassium chromate indicator K_2CrO_4 indicator

Titrate with standard solution, (0.0141 N) $AgNO_3$to reach end point yellow to reddish brown read the volume required (V)

Calculate chloride concentration as Cl mg/L= $V \times N \times \text{eq. wt of Cl} \times 10^3 / Vs$

$$= V \times 0.0141 \times 35.5 \times 10^3 / Vs$$

Experiment 11: Microbiology of water

Water is a good environment for different types of microorganisms to live, as it supplies the nutrients they require. But potable water should be safe to drink which does not contain microorganisms that cause diseases. By disinfection these microorganisms are killed. To determine the efficiency of the disinfection process and the degree of water pollution, microbiology tests are required.

Laboratory equipment:

1) Glass wares, pipettes, test tubes, bottles, Petri-dishes… etc.
2) Auto clave, for the sterilization of all the glass wares, liquid… etc, this is performed at 121 ^{0}C and 1 bar pressure for 15 minute.
3) Incubator, for providing the proper environment of temperature (desirable and constant) for the growth of the microorganisms.

Samples:

1) Water samples are collected in clean and sterilized bottles.
2) The samples are to be tested immediately. If not they should be stored at 4^{o}C (to stop the microbial activity).
3) Samples should be diluted: river, lake, wastewater and any polluted water.

Dilution procedure:

1) Prepare a number of clean test tubes.
2) Put 9 mL of distilled water in each tube.
3) Cover each tube with cotton.
4) Sterilize the tubes in the auto clave.
5) Dilution proportions.

a) Put 1ml sample to one test tube of 9mL distilled water = 0.1 dilution(1:10).
b) Put 1ml from (a) to another test tube of 9ml distilled water = 0.01 dilution (1:100).
c) Put 1ml from (b) to another test tube of 9ml distilled water = 0,001 dilution and so on (1:1000).

Methods used to determine the microorganisms living in water (bacteria):

1) Total Plate count

This procedure is to determine the density of microorganisms (bacteria) in water. It is an empirical measurement because bacteria grow in pairs, chains and colonies. Also no single media (nutrient) will satisfy all types of bacteria. In this procedure, counting the number of bacteria that will grow on a solid media (ager) is performed.

a) Put 1 mL of the collected sample (or diluted) in a sterilized Petri-dish.
b) Pure a suitable amount of warm and sterilized media (nutrient agar) in the Petri-dish.
c) Put the Petri-dish in the incubator at 35 ^{o}C.

d) After 24-48 hr count the number of colonies that have grown on the agar.

Record the result as CFU/mL (colony forming unit per mL) = reading X dilution factor

Note: neglect the result if the reading is 300 CFU/mL or more

Problem:

Proportion of dilution	1	0.1	0.01	0.001	0.0001
Dilution factor	1	10	100	1000	10000
reading	>300	>300	40	5	zero
CFU/ml	-	-	4000	5000	-

The average reading could be recorded = (4000 + 5000)/2 = 4500 CFU/ml

2) Multiple tube fermentation

A large number of fecal and pathogenic bacteria grow and are transmitted in water. Many of these pathogens are detected by complicated producers. So an indicator organism is used as an index of possible pathogenic pollution. A coliform bacterium is used as the indicator organism. By the determination of this indicator, pollution may be measured and the degree of disinfection is evaluated.

a) Prepare a number of clean test tubes: For drinking water tests, 5 tubes are required with no dilution, for any other water a set of 3 tubes is required for each dilution used.
b) Put 10 mL of a liquid media (MacConky broth) in each test tube. Insert a small tube (Durham) in each tube as invert which should be filled with the liquid media.
c) Cover each tube with cotton.
d) Sterilize the tubes in the autoclave.
e) Inoculate each tube with 1 mL of the water sample (or diluted). Return the cotton on the tube.
f) Put the tubes in the incubator at 35°C.
g) After 24-48 hr record the results:
    i. Positive result (+) indicating bacterial growth, by the change in the color of the media and the appearance of gas in the durham tube.
    ii. Negative result (-) indicating no growth, no change in color or gas formation.
h) The positive results are taken only. The following table is used to record the results as MPN/100mL (most probable number).

Problem: Raw Water

Using 4 sets of tubes (3 tubes in each set)

Dilution proportion	1	0.1	0.01	0.001
Dilution factor	1	10	100	1000
(+) tubes in one set	3	3	2	0

Select the last 3 dilutions for recording the results: 320 and from the table read: 93 MPN/100mL.

The raw water contains: the reading X dilution factor (the middle of the 3 dilutions used)

=93 X 100= 9300 MPN/100mL

Problem: Drinking Water Test

Using 5 tubes inoculated with 1mL of the treated water in each tube.
No (+) result: less than 2.2 MPN/100mL
One (+) results: 2.2 MPN/100mL
Two and more (+) results: 5.1 and more MPN/100mL

Experiment 12: Dissolved oxygen (DO)

The amount of oxygen gas dissolved in water, rivers, potable water, sewage …etc depends mainly on temperature and salt content of the water. In water bodies this amount will vary with: flowing velocity, surface area and depth. The main source of oxygen in water is from the atmosphere.

Methods of determination:

1) DO meter: By using an electrode sensitive to oxygen molecules in water.
2) Azide modification method.
   a) Put the sample in a BOD bottle (250 or 300 mL)
   b) Add 2mL MnSO4 solution
   c) Add 2 mL alkaline –azide solution (NaOH. NaI. NaN3)
   d) Shake the bottle for complete mixing:
      i. IF a- a white color appears- there is no DO in the sample, then stop the test.
      ii. b-a yellow or brown color appears – there is DO in the sample , continue the test.
   e) Leave the sample to settle for 10 to 15 minutes.
   f) Add 2mL H2SO4 and shake the bottle to dissolve the settle matter.
   g) Take 203 mL from the bottle in a flask.
   h) Add 1 mL starch as an indicator, a black color will appear.
   i) Titrate with 0.025 N sodium thiosulfate Na2S2O3.

The reaction will give a blue color and the end point will be a colorless solution. Read the volume used for titration A mL.

The concentration of DO in the sample is calculated from:

DO (mg/L)= A x N x eq.wt of O_2 x 10^3 / volume of sample(with no additions)

= A x 0.025 x 8 x 10^3 / 200 = A

So DO (mg/L)= A(mL)= volume of sodium thiosulfate used for titration.

Why is a volume of 203 mL of the sample used in the titration process:

300x200/(300-4) = 202.7mL OR 250x200/(250-4) = 203.3mL

Where 250 or 300 mL =Volume of the BOD bottle used in the test

4mL = the amount of chemicals added for reaction (2mL $MnSO_4$ = 2mL azide solution)

203mL represents the volume required for titration which is equivalent to the original sample 200mL that satisfies the equation above.

Experiment 13: Biochemical oxygen demand (BOD)

For the determination of organic matter concentrations in different types of water samples. If high amounts of organic matter exist in the sample, the sample should be diluted to provide enough dissolved oxygen for biodegradation.

Dilution water:

1) Take one liter of distilled water.
2) Add 2ml $FeCl_3$ + 2ml $CaCl_2$ + 2mL $MgSO_4$ as nutrients.
3) Aerate the water by shaking or using an air compressor to saturate the water with dissolve oxygen.

Dilute the water sample according to:

a) Put the diluted sample into 2 BOD bottles.
b) Determine the existing dissolved oxygen in bottle 1(by the azide method) DO_2.
c) Put the 2nd bottle in the incubator for 5 days at 20°C temperature, for the biodegradation of the organic matter in the sample.
d) Determine the remaining dissolved oxygen in the 2nd bottle DO_2.
e) Calculate $BOD_{5\text{-}20} = (DO_1 - DO_2)$ x dilution factor (Dilution factor = 1/ proportion of dilution).

Sample	Proportion of Dilution
River water	1:3
Low strength sewage	1:5 – 1:10
High strength sewage	1:50 – 1:100