Basrah University

College of Engineering

Department of Civil Engineering

Lectures in Water Supply Engineering

4th Class Course

3hours/week

15 weeks course

Ch.1 Quantity of Water

1-1 Water Consumption

Water is used for domestic, commercial, industrial, agricultural and public purposes.

Domestic Water Use

Domestic (or residential) water demand covers uses of water by households, both inside and outside the confines of the residence and typically includes washing, cooking, bathing, laundry and gardening. Average water demand for domestic use is dependent on;

1. Climatic condition.
2. Living standards.
3. The extent by which the area is sewered.
4. Metering of water supply.
5. Water cost.
6. Other factors like;
    a. Water pressure.
    b. Water quality.
    c. Water management.

Generally, the average water demand for domestic use is 200 to 500 liter per capita per day.

Commercial Water Use

Commercial use consists of water used by warehouses, stores and shopping centres, restaurants, cinemas hotels and related activities. Water demand for commercial use is dependent on number of employees in the commercial area and water demand of each employee. Per employee water demand is estimated to be 20% of per capita water demand for domestic use.

Industrial Water Use

Industrial water demand is dependent on industry type and production rate.

For examples steel industry requires 28.6 m^3 per ton of steel produced and paper industry requires 60 m^3 per ton of pulp produced. Then, if steel factory produces 40 tons steel per week it will consume water at a rate of 163.4 m^3/day. Table (1) gives examples water demand for some industries.

Table (1) Water demand for some industries

Industry type	Water demand (ton water/ton production)
Fertilizer	80-200
Leather	40
Paper	200-400
Petroleum refinery	1-2
Sugar	1-2
Textile	80-140

Agricultural Water Use

Agricultural demand is taken to cover all irrigation and livestock purposes. Water demand for irrigation use is dependent on crop type and the planted area. For example, grass crop grown in a sub-humid climate with a mean temperature of 30°C needs 7.5 mm of water per day.

For livestock, the following tables give water demands for drinking and meat processing of some livestock species.

Table (2) drinking water demand for chicken

Chicken age (weeks)	Water requirement (litre /1000 birds/week)	
	21°C	32°C
1-4	50-206	50-415
5-8	345-470	550-770

Table (3) Drinking water demand small ruminants

Small ruminants	Daily requirements (liter/head)
Adult sheep	2-6
lambs	4-10

Table (4) Drinking water demand for cattle when the daily high temperature is 32°C

Type of cattle	Daily liters required per 45kg of body weight
Cow	4
Cow-calf pair	8
Bull	4

Public Water Use

Public water use includes the water used for public buildings like schools, universities and jails. The water demand for public use is estimated to be 50 to 75 litter per capita per day.

Water Losses

In addition to the above water uses the total water demand is increased by 10 to 30% to include the water losses due to;

- Leaks of pipes.
- Evaporation from open tanks.
- Unauthorized connections.

Total Water Demand

The total average water demand is the sum of all above water demands in addition to losses. It is obtained as;

Total average water demand= (domestic water demand+ commercial water demand+ industrial water demand+ agricultural water demand+ public water demand)+ water loss

Example 1.1

Estimate the average water demand for a city has a population of 30000. The city has a textile factory which produces 30 ton textile per week. It has a total surface area of $3km^2$, 5% of it is planted with grass. The number of employees in the commercial area of the city is 800. Also, the city contains poultry farm which has a capacity of 2000 birds.

<u>Solution:</u>

Total average water demand= (domestic water demand+ commercial water demand+ industrial water demand+ agricultural water demand+ public water demand)+ water loss

Domestic water demand;

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

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

Industrial water demand;

For textile industry, water requirement=100 ton/ton production

$$\therefore industrial\ water\ demand = 100 \times 30 = 3000\ ton\,water/week$$

$$= 428.6 m^3/day$$

Agricultural water demand;

$$Planted\ area = 3 \times \frac{5}{100} \times 10^6 = 150000\ m^2$$

Water demand=7.5mm per day

$$\therefore Plants\ watering\ demand = \frac{7.5}{10^3} \times 150000 = 1125\ m^3/day$$

For chicken;

Water demand=$\frac{2000}{1000} \times 700 = 1400\ litre/week$

$$= 1.4\ m^3/week = 0.2\ m^3/day$$

$\therefore Agricultural\ water\ demand = 1125$ +0.2=1125.2 m^3/day

Commercial water demand;

$Commercial\ water\ demand = \frac{20}{100} \times \frac{300}{1000} \times 800$ =48000 litre/day= $48\ m^3/day$

$$Public\ water\ demand = \frac{75}{1000} \times 30000 = 2250 m^3/day$$

$$\therefore total\ water\ demand = 9000 + 48 + 428.6 + 1125.2 + 2250$$

$$= 12851.8\ m^3/day$$

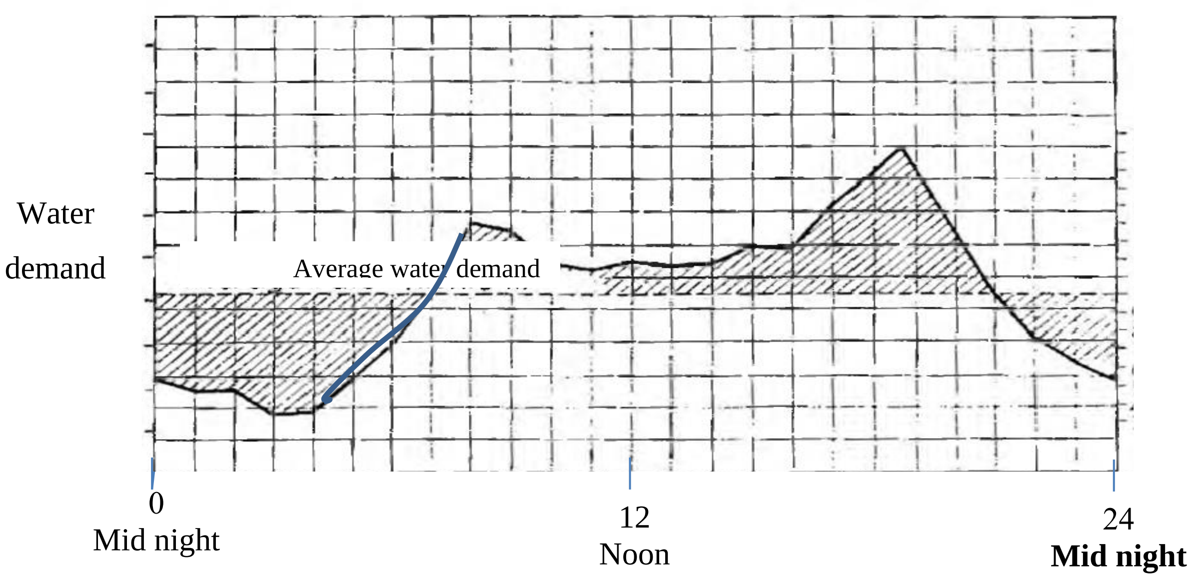
Assume water loss=20%

$$\therefore Total\ water\ demand\ inclding\ the\ losses = 1.2 \times 12851.8$$

$$= 15422.2\ m^3/day$$

1-2 Variation in Rates of Water Consumption

Climatic conditions and the working day cause wide variations in rates of water consumption. For specific city, the variation of water consumption with hours of the day is shown below;



Variation of water consumption rate with hours of the day

The percentage of maximum water consumption during t duration (in days) to average water demand is obtained as;

$$p = 180\ t^{-0.1}$$

Where;

$$p = \frac{maximum\ water\ demand\ during\ t}{average\ water\ demand} \times 100$$

<u>Maximum daily water demand;</u>

t=1 day and p=180, then;

maximum daily water demand=1.8×avg. water demand

<u>Maximum weekly water demand;</u>

t=7 days and p=148, then;

Maximum weekly water demand=1.48×avg. water demand

Maximum hourly water demand;

It is obtained as;

maximum hourly water demand=1.5× maximum daily water demand

or;

maximum hourly water demand=2.7× avg. water demand

The design capacity of any water project is maximum daily water demand

Example 1.2

A town consumed water at an average rate of 20000 m^3/day. Determine the maximum daily, weekly and hourly water demands of the town.

Solution

Maximum daily water demand;

Maximum daily water demand=1.8×avg. water demand

Maximum daily water demand=1.8×20000=36000 m^3/day

Maximum weekly water demand;

Maximum weekly water demand=1.48×avg. water demand

Maximum weekly water demand=1.48×20000=29600 m^3/day

Maximum hourly water demand;

Maximum hourly water demand=2.7× avg. water demand

Maximum hourly water demand=2.7×20000=54000 m^3/day

1-3 Design Period

It’s the period of time during which the water project serves the city before it is abandoned or upgraded. The design period is dependent on project type and life of construction materials. For examples;

- For pumping stations, the design period is 5 to 10 years.
- For treatment plants, the design period is 10 to 20 years.
- For water networks, the design period is dependent mainly on average life of the used pipes. For example, if the network is constructed using ductile

iron pipes which have average life of 75 years, the design period can be 75 years.

1-4 Population Estimate Methods

The design capacity of any water or sewage project must be obtained at the end of design period. Subsequently, the served population must be estimated at the end of design period. For example, if the project will serve specific city till the year 2040, then, the population of the city must be obtained in the year 2040. The population is estimated using census records

There are different methods for population estimate. These include;

1- Arithmetic method.
2- Geometric method.
3- Declining growth method.
4- Ratio method.

1. Arithmetic Growth Method

In this method, the rate of population growth (dP/dt) is assumed to be constant;

$$\frac{dP}{dt} = k$$

By integration;

$$P_t = P_o + k\Delta t$$

Where;

P_t = population in the year t

P_o = population in the base year o

k = constant calculated using known population records as;

$$k = \frac{P_2 - P_1}{\Delta t}$$

Where P_1 and P_2 are two population records separated by time duration of Δt.

Example 1.3

The population records of city A for three censuses are as given below, estimate the city population in the year 2040.

Year	1990	2000	2010
population	35000	38500	42150

Solution:

$$P_t = P_o + k\Delta t$$

$$P_{2040} = P_{2010} + k \times 30$$

$$K = \frac{\Delta P}{\Delta t}$$

$$K_1 = \frac{38500 - 35000}{10} = 350/year$$

$$K_2 = \frac{42150 - 38500}{10} = 365/year$$

$$K = K_{avg} = \frac{350 + 365}{2} = 357.5/year$$

$$P_{2040} = 42150 + 357.5 \times 30 = 52875$$

2. Geometric Growth Method

In this method, the rate of population growth is proportional to population, i.e;

$$\frac{dP}{dt} = kP$$

By integration;

$$Ln\ P_t = Ln\ P_0 + k\Delta t$$

Where;

P_t = population in the year <u>t</u>

P_0 = population in the base year <u>o</u>

k = constant calculated using known population records as;

$$k = \frac{Ln\ P_2 - Ln\ P_1}{\Delta t}$$

Where P_1 and P_2 are two population records separated by time duration of Δt.

<u>Example 1.4</u>

Solve example 1.3 using geometric growth method. Also, estimate the design flowrate for a water treatment plant serves city A till the year 2040. Assume city A is a residential area.

Solution:

$$Ln\ P_t = Ln\ P_0 + k\Delta t$$

$$Ln\ P_{2040} = Ln\ P_{2010} + k \times 30$$

$$k = \frac{Ln\ P_2 - Ln\ P_1}{\Delta t}$$

$$k_1 = \frac{Ln\ 38500 - Ln\ 35000}{10} = 0.00953$$

$$k_2 = \frac{Ln\ 42150 - Ln\ 38500}{10} = 0.00906$$

$$K = K_{avg} = \frac{0.00953 + 0.00906}{2} = 0.009295$$

$$Ln\ P_{2040} = Ln\ 42150 + 0.00925 \times 30$$

$$Ln\ P_{2040} = 10.93 \rightarrow P_{2040} = e^{10.93} = 55826.28 \rightarrow P_{2040} = 55827$$

<u>Design flowrate</u>

Let domestic water demand=250 liter/capita/demand

Let public water demand=60 liter/capita/demand

$$\therefore total\ water\ demand = \left(\frac{250 + 60}{1000}\right) \times 55827 = 17306.4\ m^3/day$$

$$assume\ water\ losses = 30\%$$

$$\therefore total\ water\ demand\ including\ the\ losses = 1.3 \times 17306.4$$

$$= 22498.3\ m^3/day$$

$design\ flowrate\ (capacity) of\ watertreatment\ plant$

$$= max. daily\ water\ demand = 1.8 \times 22498.3$$

$$= 40497\ m^3/day$$

3. Declining Growth Method

In this method, the rate of population growth is proportional to population deficit, i.e;

$$\frac{dP}{dt} = k(P_{sat} - P)$$

By integration;

$$P_t = P_0 + (P_{sat} - p_0)(1 - e^{-k\Delta t})$$

Where;

P_t = population in the year <u>t</u>

P_0 = population in the base year <u>0</u>

P_{sat}= population at saturation

$$P_{sat} = population\ density \times\ city\ area$$

k = constant calculated using known population records as;

$$k = -\frac{1}{n} \ln \frac{p_{sat} - p}{p_{sat} - p_0}$$

Where P and P_o are two population records <u>n</u> years apart.

<u>Example 1.5</u>

The population records of city B for four censuses are as given below. Estimate the city population in the year 2035. City B has an area of 16 km^2 and a population density of 10000/km^2.

Year	1980	1990	2000	2010
population	75600	83150	90550	97260

<u>Solution</u>

$$P_t = P_0 + (P_{sat} - p_0)(1 - e^{-k\Delta t})$$

P_{sat}= *population density× city area*

P_{sat}= *10000× 16=160000*

$$k = -\frac{1}{n}\ln\frac{p_{sat}-p}{p_{sat}-p_0}$$

$$k_1 = -\frac{1}{10}\ln\frac{160000-83150}{160000-75600} = 0.009371$$

$$k_2 = -\frac{1}{10}\ln\frac{160000-90550}{160000-83150} = 0.010125$$

$$k_3 = -\frac{1}{10}\ln\frac{160000-97260}{160000-90550} = 0.010161$$

$$k_{avg} = \frac{k_1+k_2+k_3}{3} = \frac{0.009371+0.010125+0.010161}{3} = 0.009886$$

$$P_t = P_0 + (P_{sat} - p_0)(1-e^{-k\Delta t})$$

$$P_{2035} = P_{2010} + (P_{sat} - p_0)(1-e^{-k\times 25})$$

$$P_{2035} = 97260 + (160000-97260)(1-e^{-0.009886\times 25}) = 110998.6$$

$$P_{2035} = 110999$$

4. Ratio Method

The ratio method of forecasting depends upon the population projection of the governorate and the assumption that the city (which is located in that governorate) in question will maintain the same trend in the change of the ratio of its population to that of the governorate. If we consider city B which is located in governorate A and the population of city B is required to be predicted in the year t, then;

$$\frac{P_{A_t}}{P_{A_0}} = \frac{P_{B_t}}{P_{B_0}}$$

Where;

P_{A_t} *and* P_{A_0}= populations of governorate A in the years t and 0, respectively.

P_{B_t} *and* P_{B_0}= populations of city B in the years t and 0, respectively.

Note: the previous population records of governorate A are present, while the population of city B is known at the present time only.

Example 1.6

Governorate (A) has a population in the year 2000 equals 700,000. The rate of population growth of this governorate is assumed to be constant and equals 17500/ year. In this governorate, city (B) is located and has population in the year 2018 equals 45000. Estimate the population of city (B) in the year 2039.

Solution:

$$\frac{P_{A_t}}{P_{A_0}} = \frac{P_{B_t}}{P_{B_0}}$$

$$\frac{P_{A_{2039}}}{P_{A_{2018}}} = \frac{P_{B_{2039}}}{P_{B_{2018}}}$$

Since the rate of population growth is constant, then, use arithmetic growth method;

$$P_t = P_o + k\Delta t$$

$$P_{A_{2018}} = P_{A_{2000}} + k \times 18$$

$$P_{A_{2018}} = 700000 + 17500 \times 18 = 1015000$$

$$P_{A_{2039}} = P_{A_{2000}} + k \times 39$$

$$P_{A_{2039}} = 700000 + 17500 \times 39 = 1382500$$

$$\frac{1382500}{1015000} = \frac{P_{B_{2039}}}{45000}$$

$$P_{B_{2039}} = 61293.1 \rightarrow P_{B_{2039}} = 61294$$

1-5 Fire Demand

Fire demand is the quantity of water required for fire fighting. It can be obtained as;

$$F = 223\ C\ A^{0.5}$$

Where;

F= fire demand (l/min.)

C= coefficient depends of construction type.

For ordinary constructed buildings, C=1

For wood- framed buildings, C=1.5

For fire resistive buildings, C=0.6

A= total area of all building floors, excluding the basement, (m^2).

Note: for fire resistive buildings;

- If the vertical openings are protected;

A= the total area of largest three successive floors.

- If the vertical openings are not protected;

A= the total area of largest six successive floors.

Limits of the above formula;

For any single fire;

F_{max}≈23000 l/min for one-story buildings

F_{max}≈31000 l/min for multi-story buildings

To protect nearby buildings;

F_{max}≈46000 l/min

In all above cases;

F_{min}≈1890 l/min

For residential areas, the fire demand is obtained using the following table (see p.18 in text book);

Table (5) Residential fire flow

Distance between adjacent units (m)	Required fire flow (l/min)
>30.5	1890
9.5-30.5	2835-3780
3.4-9.2	3780-5670
≤3.0	5670-7560#

for continuous construction use 9450l/min.

Fire Storage

Fire storage is the volume of tank required to store the water of fire fighting. It is obtained as;

$$Fire\ storage = \ F \times fire\ flow\ duration$$

The fire flow duration is dependent on fire demand and can be obtained using the following table (see p.19 in text book);

Table (6) Fire flow duration

Required fire flow (l/min.)	Duration (hour)
<3780	4
3780-4725	5
4725-5670	6
5670-6615	7
6615-7560	8
7560-8505	9
>8505	10

The fire demand of a city can be estimated based on city population using one of the following formula;

1-Kuichling Formula

$$\mathrm{Q} = 3182\sqrt{P}$$

2-Freeman Formula

$$Q = 1136\left[\frac{P}{10} + 10\right]$$

Where,
Q = quantity of water required in liters/minute.
P = Population in thousands.

Example 1.7

A street has three types of buildings with the characteristics given in the table below. It is required to design a water pumping station to supply the fire demand of this street. Find the design flowrate of this pumping station.

Building type	Construction type	Number of floors	Area of each floor (m^2)
A	Ordinary	10	180
B	Ordinary	6	300
C	Fire resistive with protection of vertical openings	10	400

Solution:

$F = 223\, C\, A^{0.5}$

For building A;

C=1, A=10×180=1800m^2

$F = 223 \times 1 \times 1800^{0.5} = 9461.1\, liter/min.$

For building B;

C=1, A=6×300=1800m^2

$F = 223 \times 1 \times 1800^{0.5} = 9461.1\, liter/min.$

For building C;

C=0.6, A=3×400=1200m^2

$F = 223 \times 0.6 \times 1200^{0.5} = 4635\, liter/min.$

Design flowrate of the pumping station is the maximum fire demand=9461.1 liter/min.

Example 1.8

Find the fire storage required for;

1- A residential area in which the distance between the adjacent units is 5m.
2- A residential area composed of attached houses.

Solution:

For residential area, F is obtained from Table 5;

1- Distance between the adjacent units =5m
   For distance between the adjacent units=3.4m, F=5670 litre/min.
   For distance between the adjacent units=9.2m, F=3780 litre/min.
   For distance between the adjacent units=3.4m;

$\frac{3780-5670}{9.2-3.4} = \frac{F-5670}{5-3.4} \rightarrow$ F=5148.6 litre/min.

For F=5148.6 litre/min.,

From Table 6; fire duration=6hrs.

$$Fire\ storage = F \times fire\ flow\ duration$$

$$Fire\ storage = \frac{5148.6}{1000} \times 6 \times 60 = 1853.4m^3$$

2- For attached houses, F=9450 litre/min.

From Table 6, fire duration=10 hrs.

$$Fire\ storage = \frac{9450}{1000} \times 10 \times 60 = 5670\ m^3$$

Example 1.9

Estimate the fire demand for a city has a population of 35000.

1-Kuichling Formula

$$\mathrm{Q} = 3182\sqrt{P}$$

$$\mathrm{Q} = 3182\sqrt{35} = 18825\, litre/min.$$

2-Freeman Formula

$$Q = 1136\left[\frac{P}{10} + 10\right]$$

$$Q = 1136\left[\frac{35}{10} + 10\right] = 15336\, litre/min.$$

Ch.2 Piping Materials

The complete water works system has the following components:

1- Water source (river, lake, impounding reservoir, ground water, or sea).
2- Intake structure.
3- Transmission system (pipe line or open channel that is used to transport water from the source to the water treatment plant).
4- Water treatment plant.
5- Water distribution system (or water network) completed with storage tanks and pumping stations.

In all of above components piping materials are required. The types of pipes, fittings, and valves are discussed in this chapter. The emphasis throughout this chapter is on pipe 100 mm (4 in.) in diameter and larger.

2.1 Ductile Iron Pipe (DIP)

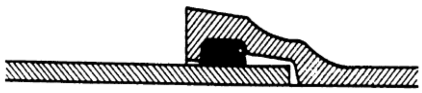
DIP is widely used in water distribution systems. It is commonly used for both smaller distribution mains and larger transmission mains.

2.1.1 Materials. DIP is a cast-iron product. Cast-iron pipe is manufactured of an iron alloy centrifugally cast in sand or metal molds. Ductile iron is produced by the addition of magnesium to molten low sulfur base iron, causing the free graphite to form into spheroids and making it about as strong as steel.

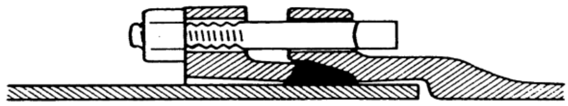
2.1.2 Joints. For DIP, rubber gasket push-on and mechanical joints (Fig.1- a & b) are the most commonly used for buried services. These joints allow for some pipe deflection (about 2-5° depending on pipe size) without sacrificing water tightness. Neither of these joints is capable of resisting thrust across the joint and requires thrust blocks or some other sort of thrust restraint at bends and other changes in the flow direction.

Flanged joints (Fig.1c) are sometimes used at fitting and valve connections. Victaulic (or grooved end joints, Fig.1d) are normally used for exposed service and are seldom used for buried service. Flanged joints are rigid and victaulic joints are flexible and they are used for exposed pipes, which are subjected to vibration.

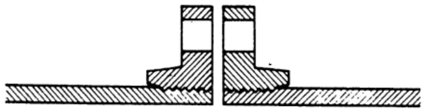
Ball joints (Fig.1e) are used for joining pipelines on river beds, where settlement will occur after the pipe is laid.



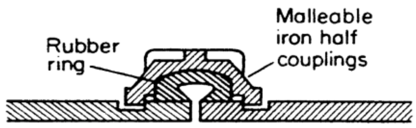
(a) Push-on joint



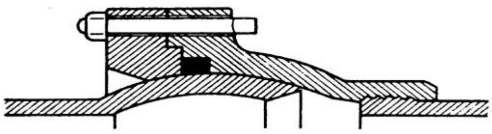
(b) Mechanical joint



(c) Flanged joint



(d) Victaulic joint



(e) Ball joint

Fig.1 DIP joints

2.1.3 <u>Gaskets</u>. Gaskets for ductile iron push-on and mechanical joints are natural or synthetic rubber. Natural rubber is suitable for water pipelines but deteriorates when exposed to raw or recycled wastewater.

2.1.4 <u>Fittings</u>

A list of standard fittings is given below:

- Bends (90°,45°, 22.5°, 11.25°)
- Base bends
- caps
- crosses
- Blind flanges

- Reducers
- Tees
- Wyes

Fittings are designated by the size of the openings, followed by the deflection angle. A 90° bend (or elbow) for 250 mm pipe would be called a 250 mm 90° bend. Reducers, reducing tees, or reducing crosses are identified by giving the pipe diameter of the largest opening first, followed by the sizes of other openings in sequence. Thus, a reducing tee on a 300 mm line for a 150 mm fire hydrant run might be designated as a 300 mm×150 mm×300 mm tee.

2.1.5 Linings.

DI pipes are usually lined to protect them against the formation of rust tuberculation. Examples of lining materials include:

- Cement mortar
- Glass
- Epoxy
- Polyethylene

Considering its low cost, long life, and sustained smoothness, cement mortar lining for DIP in water distribution systems is the most useful and common. Although cement-mortar lining is normally very durable, it can be slowly attacked by very soft waters with low total dissolved solids content (less than 40 mg/L), by high sulfate waters, or by waters under saturated in calcium carbonate.

2.1.6 Coatings. Although DIP is relatively resistant to corrosion, some soils may attack the pipe. In corrosive soils, the following coatings may be appropriate for protecting the pipe:

- Plastic wrapping
- Hot-applied coal-tar enamel
- Hot-applied coal-tar tape
- Coal-tar epoxy

• Cold-applied tape

2.2 Polyvinyl Chloride (PVC) Pipe

It is used in both water and wastewater service, *polyvinyl chloride* (PVC) is the most commonly used plastic pipe for municipal water distribution systems. Because of its resistance to corrosion, its light weight and high strength to weight ratio, its ease of installation, and its smoother interior wall surface.

2.2.1 Materials. PVC is a polymer extruded under heat and pressure into a thermoplastic that is nearly inert when exposed to most acids, alkalis, fuels, and corrosives. Generally, PVC should not be exposed to direct sunlight for long periods. The impact strength of PVC will decrease if exposed to sunlight and should not be used in above-ground service.

2.2.3 Joints. For PVC pipe, a rubber gasket bell and spigot type joint is the most commonly used joint. The bell and spigot joint allows for some pipe deflection without sacrificing water tightness. This joint is not capable of resisting thrust across the joint and requires thrust blocks or some other sort of thrust restraint at bends and other changes in the direction of flow.

2.2.4 Gaskets. As with gaskets for DIP, gaskets for PVC pipe are natural rubber or synthetic rubber. Natural rubber is suitable for water pipelines but deteriorates when exposed to raw or recycled wastewater.

2.2.5 *Fittings.* Ductile iron fittings are used in all available sizes of PVC pipes. Although not widely used, PVC fittings, in configurations similar to ductile iron fittings, are also available for smaller line sizes.

2.2.6 Linings and Coatings. PVC pipe does not require lining or coating.

2.3 Steel Pipe

Steel pipe is available in any size, from 100 m through 3600 mm, for use in water distribution systems. Though rarely used for pipelines smaller than 400 mm, it is widely used for transmission pipelines in sizes larger than 600 mm. The principal advantages of steel pipe include high strength, the ability to deflect without breaking, the ease of installation, shock resistance, lighter weight than ductile iron pipe, the ease of fabrication of large pipe, the availability of special configurations by welding, the variety of strengths available, and the ease of field modification.

2.3.1 Materials. There are two types of steel pipes: (1) mill pipe and (2) fabricated pipe. *Mill pipe* includes steel pipe of any size produced at a steel pipe mill to meet finished pipe specifications.

Fabricated pipe is steel pipe made from plates or sheets. It can be either straight or spiral. Steel pipe may be manufactured from a number of steel alloys with various yield and ultimate tensile strengths.

2.3.3 Joints. For buried service, *bell* and *spigot joints* with rubber gaskets or mechanical couplings are common. *Welded joints* are also common for pipe 600 mm and larger.

2.3.4 Gaskets. Gaskets for steel flanges are usually made of cloth-inserted rubber either 1.6 mm (or 3.2 mm thick and are of two types:

- ring (extending from the ID of the flange to the inside edge of the bolt holes)
- full face (extending from the ID of the flange to OD)

Gaskets for mechanical and push-on joints for steel pipe are the same as those of ductile iron pipe.

2.3.5 Fittings. Specifications for steel fittings can generally be divided into two classes, depending on the joints used and the pipe size:

- Ranged, welded
- Fabricated

2.3.6 Linings and coatings. Steel pipes are subjected to corrosion. Thus, they must be lined and coated. Cement mortar is an excellent lining for steel pipe.

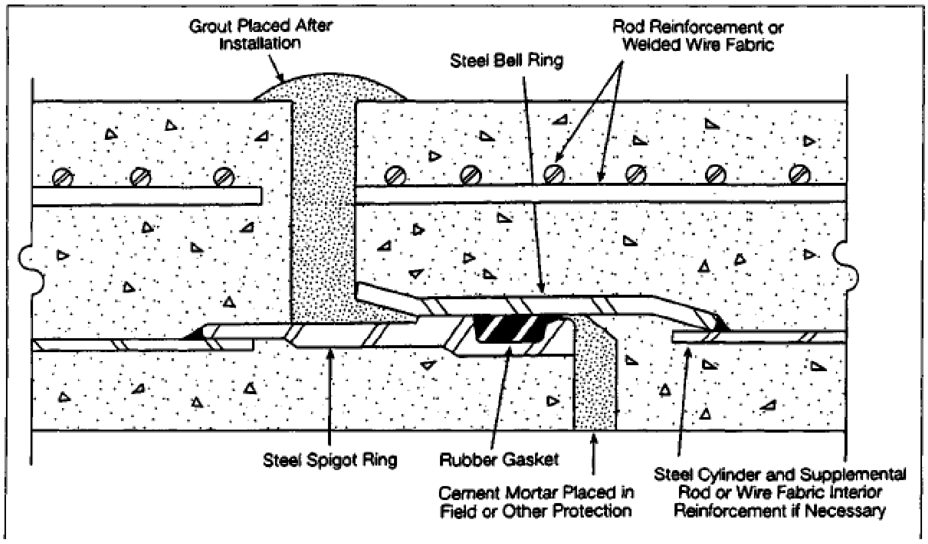
In corrosive soils, the following coatings may be appropriate for protecting steel pipe:

- Hot-applied coal-tar enamel

- Cold-applied tape system
- Fusion-bonded epoxy
- Coal-tar epoxy

2.4 Reinforced Concrete Pressure Pipe (RCPP)

Several types of RCPP are manufactured and used. These include steel cylinder, prestressed, steel cylinder, and non cylinder. Some of these types are made for a specific type of service condition and others are suitable for a broader range of service conditions.



Cross section in concrete pipe

2.5 High-Density Polyethylene (HDPE) Pipe

HDPE pipes are used in transmission and distribution system applications. HDPE pipe is gaining acceptance for use in municipal water systems because of; <u>its resistance to corrosion, its light weight and high strength to weight ratio, its resistance to cracking, its smoother interior wall surface, and its demonstrated resistance to damage during seismic events.</u>

2.5.1 Materials. Low-density polyethylene was first used for cable coatings. Pipe grade resins were developed in the 1950s and have evolved to today's high-density, extra-high-molecular weight materials.

2.5.3 Joints. HDPE pipe can be joined by thermal butt-fusion, flange assemblies, or mechanical methods as may be recommended by the pipe manufacturer. HDPE is not to be joined by solvent cements, adhesives (such as epoxies), or threaded-type connections.

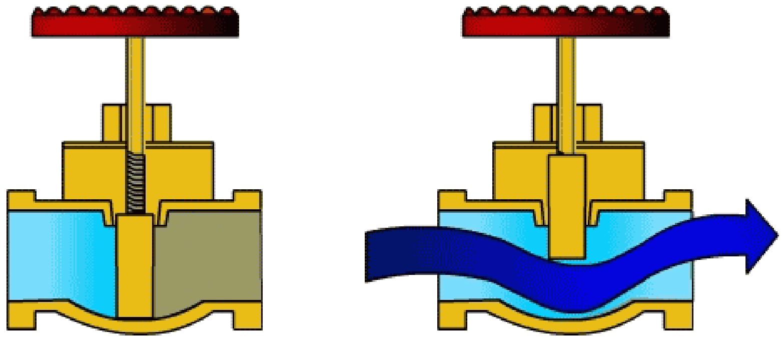
Thermal butt-fusion is the most widely used method for joining HDPE piping. This procedure uses portable field equipment to hold pipe and/or fittings in close alignment while the opposing butt-ends are faced, cleaned, heated and melted, fused together, and then cooled under fusion parameters recommended by the pipe manufacturer and fusion equipment supplier. For each polyethylene material there exists an optimum range of fusion conditions, such as fusion temperature, interface pressure, and cooling time.

2.6 Glass Reinforced Plastic Pipes (GRP Pipes)

These plastic pipes are reinforced with glass fibers. They are resistive to UV rays and available in diameters reach 4m. The most common methods of RGP pipes connection are adhesion, laminating, bell & spigot and assembly of flanged connections.

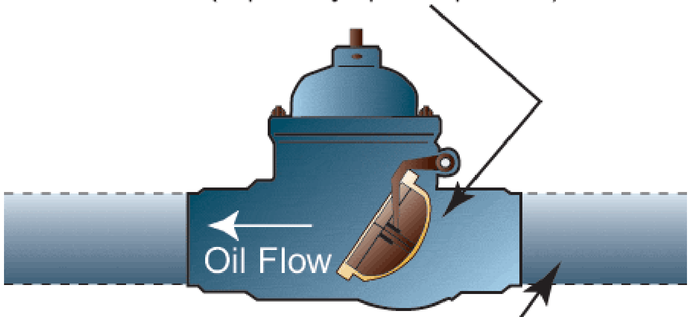
2.7 Valves

Gate valves: They are used to shut off the water when distribution pipes are needed to be repaired. Gate valves are placed at street corner where lines intersect and at max. spacing of 150m for high value areas and 250m for other areas. Gate valves are installed into manholes.

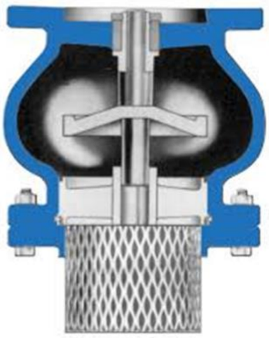


Gate Valve Closed Gate Valve Opened

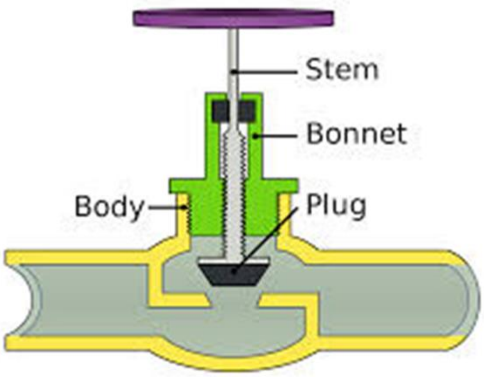
Check valves: They are installed on discharge pipes of pumps. They permit water to flow in only one direction and are generally used to prevent flow reversal when pumps are shut down.



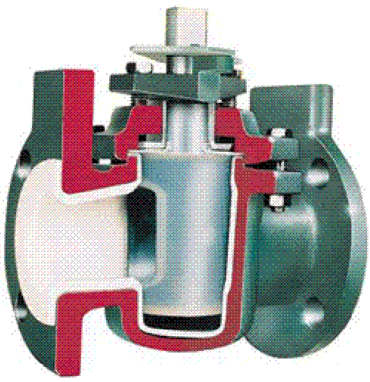
Foot valves: They are check valves installed at the end of pump suction line and they prevent drainage of the suction pipe when the pump is shut down.



Globe valves:They are seldom used in water distribution systems because of their high head loss. The primary application of these valves is in houses plumbing where their low cost outweighs their poor hydraulics.

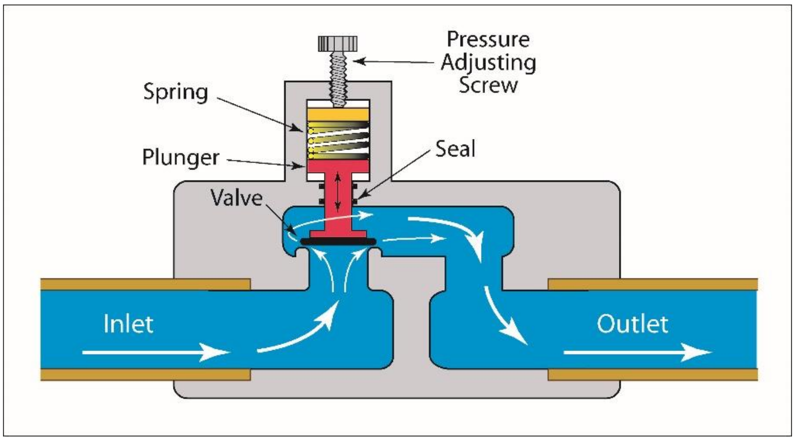


Plug (or cone) valves: These valves have tapered plug which turns in a tapered seat. They are used for water piped under high pressure.

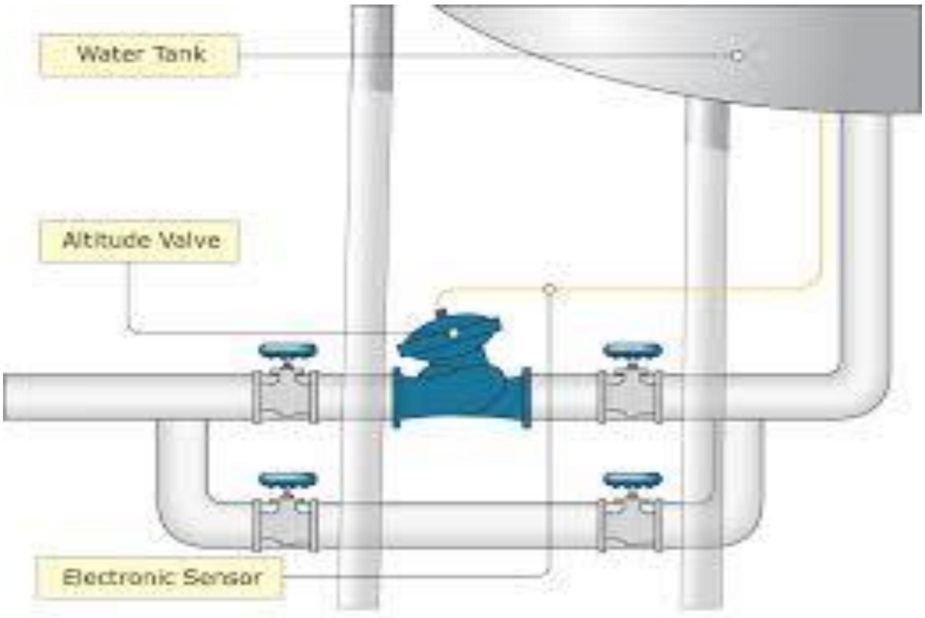


Butterfly valves: They are used in low pressure applications. They have many advantages over gate valves in large pipes including; lower cost, compactness, minimum head loss and ease of operation.

Pressure regulating valves: These valves automatically reduce the pressure on the downstream side to any desirable value and they are used on pipe lines entering low areas of a city where without such reduction, water pressure would be too high.



Altitude valves: These valves automatically close a supply line to an elevated water storage tank when the tank is full.



Sluice gates: They are vertically sliding valves used to open or close openings into walls.

Ch.3 Water Distribution System

A water distribution system is needed to deliver water to the individual consumer in the required quantity and under a satisfactory pressure.

3-1 Distribution System Components

A water distribution network is a collection of;

1. Pipes

Pipes are used to convey water. The direction of flow is from the end at higher head to that at a lower head.

2. Junctions

Junctions (also called nodes) are points where the pipes are connected and where water enters or leaves the network. Nodes may be points of water withdrawal (demand nodes), locations where water is introduced to the network (source nodes), or locations of tanks (storage nodes).

3. Pumps

Pumps are used to increase the hydraulic head of water.

4. Tanks

Tanks are storage nodes where the volume of water can vary with time.

3-2 Types of Water Distribution Systems

Water distribution systems may be classified as branched system, looped (or grid) system, or a combination of the two. The configuration of the system is influenced by street patterns, topography, and location of treatment and storage works.

1. Branched system (or dead ends system)

Branched or dead-ends system (Fig.1) has low construction cost. However, its performance is unsatisfactory because of;

- Stagnant water near the ends of the system which may cause accumulation of sediments and result in water of bad taste and odor.
- If repairs are necessary, a large area must be cut off from the water.

- With high water demand, the head loss may be excessive.

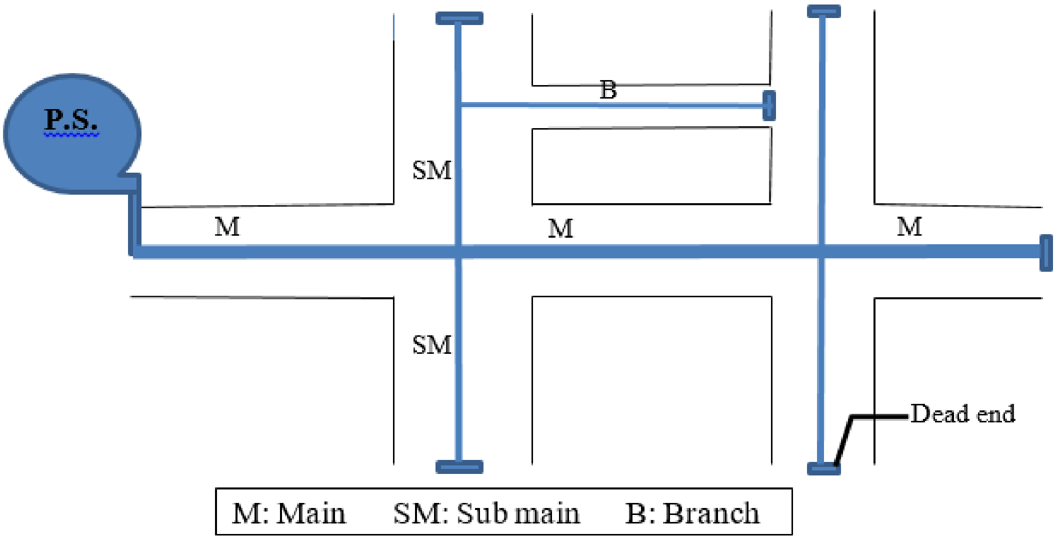


Fig.1 Layout of branched water network in a city sector

2. Looped (or grid) System

Grid systems (Fig.2) are usually preferred to branched systems, since they can supply a withdrawal point from at least two directions. However, they have high construction cost because of increasig the length of pipes and number of gate valves.

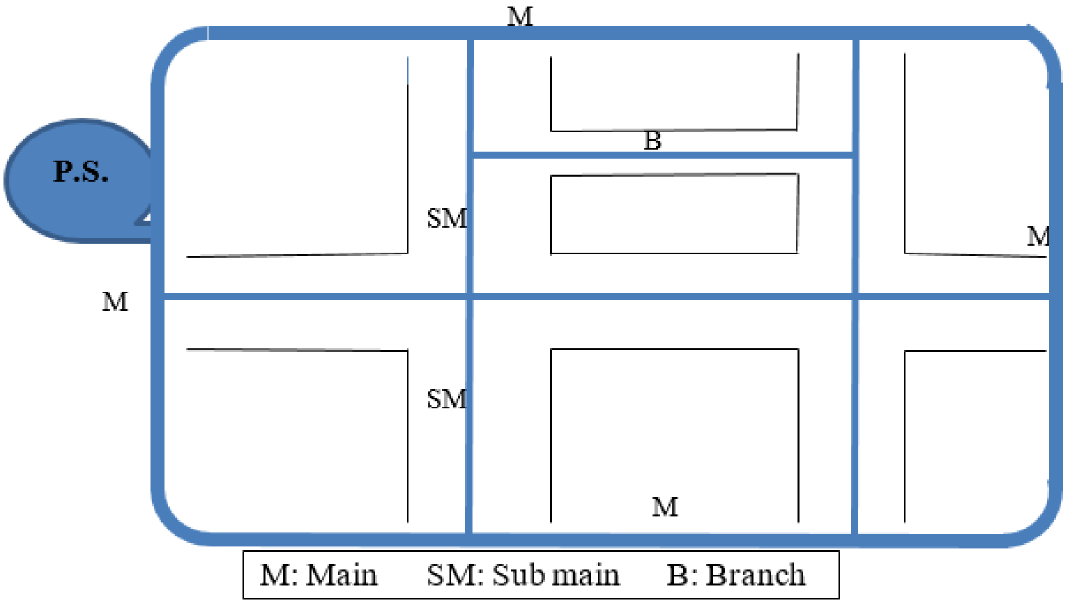


Fig.2 Layout of looped water network in a city sector

Both of the above systems are classified into; single-main system and double-main system. In single-main system there is a single main serves both sides of a street. While, in a double-main system, there is a main on each side of the street. The chief advantage of the two-main system is that repairs can be made without interfering with traffic and without damage to the pavement.

3-3 Flow in Pipes

The flow in water networks pipes is due to pressure and, thus, governed by Hazen–Williams and Darcy–Weisbach formulas. Hazen–Williams equation is the commonly used equation for analysis and design of water networks.

Hazen–Williams equation

For circular conduits flowing full, Haze- William equation is;

$$Q = 0.278\ C\ D^{2.63} S^{0.54}$$

Where;

Q= water flowrate, m^3/sec.

C= Hazen-William roughness coefficient.

D= pipe diameter, m.

S= slope of hydraulic grade line $=h_L/L$

h_L= total head loss, m.

L= pipe length, m.

The value of Hazen- William roughness coefficient (C) is dependent on Pipe material, *see Table (6-1), p.117*. Examples;

For plastic pipes, C=150.

For new DI pipe, C=130 to 140

For old DI pipe, C= 75 to 100

Generally, for design and analysis of water networks, C values are considered to be 90 and 150 for DI and plastic pipes, respectively.

3-4 Flow Velocity in Water Pipes

Recommended flow velocity in water pipes varied over the range 1 to 2m/sec.

3-5 Analysis of Water Networks (Loop type) Using Hardy Cross Method

Hardy Cross method of network analysis permits the computation of flow rates through a network and the resulting head losses in the system. It is a relaxation method by which corrections are applied to assumed flows until an acceptable hydraulic balance of the system is achieved.

The Hardy Cross analysis is based on the following two principles:

1. In any flow system continuity must be preserved, i.e., $\sum Q_{in} = \sum Q_{out}$.
2. The pressure (or head) at any junction of pipes has a single value.

The above two principles can be explained using the simple flow system shown in Fig.3. In this system, there is one loop (I), four pipes (1,2,3 &4) and four junctions (or nodes).

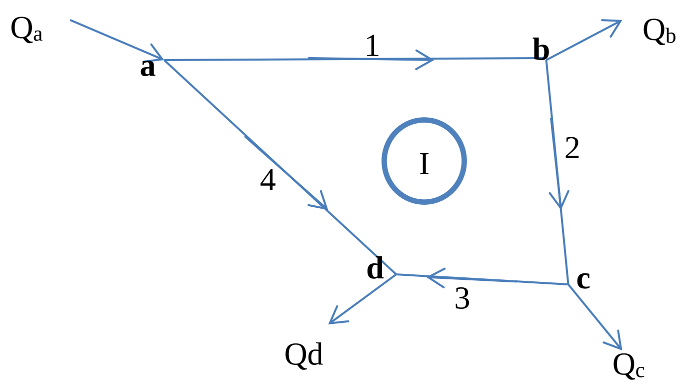


Fig.3 Simple flow system of one loop

Principle 1 means;

For the whole flow system, $Q_a = Q_b + Q_c + Q_d$

At junction No.a, $Q_a = Q_1 + Q_4$

At junction No.b, $Q_1 = Q_b + Q_2$

Principle No.2 means;

head at node d=head at node a $-h_{L1} - h_{L2} - h_{L3}$

or;

head at node d=head at node a $-h_{L4}$

To apply Hardy-cross method, at first, the system must be defined in terms of pipe size, length, and roughness. Then, the following procedure is adopted:

1. Arbitrary divide the inflow into components so that;

$$\sum Q_{in} = \sum Q_{out}$$

2. Find the head loss of each pipe using Haze-William equation;

$$h_L = KQ^{1.85}$$

Where;

$$K = \frac{L}{(O.278\ C\ D^{2.63})^{1.85}}$$

3. Find summation of head losses for each loop;

$\sum_i h_{Li}$; i= pipe number

4. Find $\frac{h_L}{Q}$ for each pipe.
5. Find $\sum_i \frac{h_{Li}}{Q_i}$ for each loop ; i= pipe number
6. Find the correction of Q for each loop;

$\Delta Q_I = -\frac{\sum_i h_{Li}}{1.85\ \sum_i \frac{h_{Li}}{Q_i}}$; ΔQ_I= correction of Q values for pipes in loop No.I

7. Correct Q of each pipe;

$$Q_{i\ new} = Q_{i\ old} \mp \Delta Q$$

8. Repeat steps 2 to 7 until accepted absolute error ($|\Delta Q|$) or relative error ($\frac{|\Delta Q|}{Q_{new}}$) is reached, i.e.;

$(|\Delta Q|)_{max} \leq$ specified value like 10^{-5}

or;

$(\frac{|\Delta Q|}{Q_{new}})_{max} \leq$ specified value like 1%

<u>Notes</u>:

- Number the pipes and loops in clockwise direction.
- Give positive sign for clockwise flows (and head losses) and negative sign for anticlockwise flows (and head losses).
- Common pipes of two loops receive both corrections with given attention to sign conversion.

The application of Hardy Cross method on any flow system is done using the following tables:

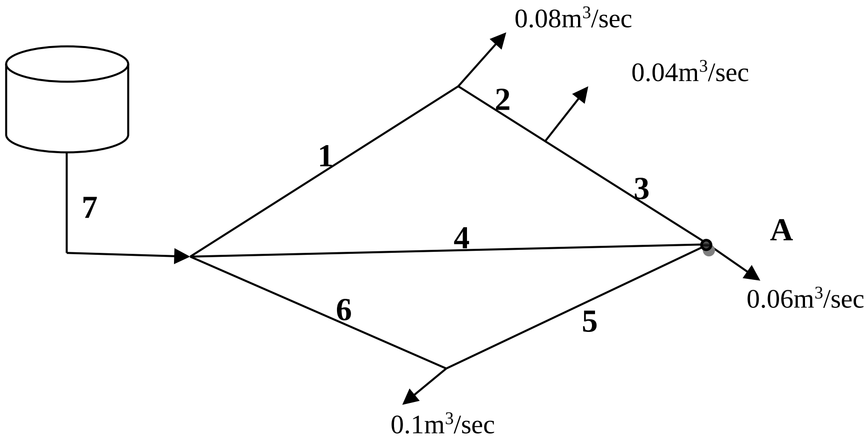
Pipe No.					

Pipe length (m)					
Pipe diameter (m)					
K					

Loop No.	Pipe No.	Q (m^3/sec)	h_L (m)	$\sum h_L$	$\frac{h_L}{Q}$	$\sum \frac{h_L}{Q}$	ΔQ	Q_{new}

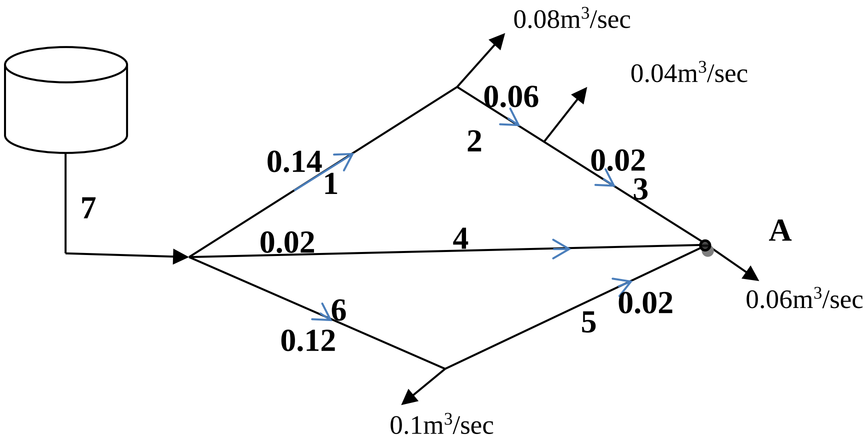
Example 3.1

The water network shown below is fed with water using an elevated storage tank. If the head at junction (A) is 25m, find the water level in the tank. Assume all the pipes are PVC and placed at a level of -1m. Hint: Maximum allowable absolute error is 10^{-5} m^3/sec.



Pipe No.	1	2	3	4	5	6	7
Length (m)	600	200	400	1000	550	550	200
Dia. (mm)	300	300	300	400	250	250	500

Solution



Pipe No.	1	2	3	4	5	6	7.0

L(m)	600	200	400	1000	550	550	200
D (mm)	300	300	300	400	250	250	500
K	211.3	70.4	140.9	86.9	470.4	470.4	5.9

Trial No.1

Loop No.	Pipe No.	Q (m^3/sec)	h_L	$\sum h_L$	h_L /Q	$\sum h_L$ /Q	ΔQ	Qnew
I	1	0.140	5.56	5.99	39.7	54.4	-0.05953	0.08047
	2	0.060	0.39		6.4			0.00047
	3	0.020	0.10		5.1			-0.03953
	4	-0.020	-0.06		3.1			-0.13261
II	4	0.020	0.06	-9.59	3.1	97.6	0.05308	0.13261
	5	-0.020	-0.34		16.9			0.03308
	6	-0.120	-9.31		77.6			-0.06692

Trial No.2

Loop No.	Pipe No.	Q (m^3/sec)	h_L	$\sum h_L$	h_L /Q	$\sum h_L$ /Q	ΔQ	Qnew
I	1	0.08047	2.00	-0.43	24.8	49.6	0.00468	0.08515
	2	0.00047	0.00		0.1			0.00515
	3	-0.03953	-0.36		9.0			-0.03485
	4	-0.13261	-2.07		15.6			-0.12935
II	4	0.13261	2.07	-0.23	15.6	88.8	0.00142	0.12935
	5	0.03308	0.86		25.9			0.03450
	6	-0.06692	-3.16		47.2			-0.06550

Trial No.3

Loop No.	Pipe No.	Q (m^3/sec)	h_L	$\sum h_L$	h_L /Q	$\sum h_L$ /Q	ΔQ	Qnew
I	1	0.08515	2.22	-0.04	26.0	50.2	0.00040	0.08555
	2	0.00515	0.00		0.8			0.00555
	3	-0.03485	-0.28		8.1			-0.03445
	4	-0.12935	-1.98		15.3			-0.12977
II	4	0.12935	1.98	-0.13	15.3	88.5	0.00082	0.12977
	5	0.03450	0.93		26.9			0.03532
	6	-0.06550	-3.04		46.4			-0.06468

Trial No.4

Loop No.	Pipe No.	Q (m^3/sec)	h_L	$\sum h_L$	h_L /Q	$\sum h_L$ /Q	ΔQ	Qnew
I	1	0.08555	2.24	-0.02	26.1	50.4	0.00025	0.08580
	2	0.00555	0.00		0.9			0.00580
	3	-0.03445	-0.28		8.0			-0.03420
	4	-0.12977	-1.99		15.3			-0.12959
II	4	0.12977	1.99	-0.01	15.3	88.6	0.00007	0.12959
	5	0.03532	0.97		27.4			0.03539
	6	-0.06468	-2.97		45.9			-0.06461

Trial No.5

Loop No.	Pipe No.	Q (m^3/sec)	h_L	$\sum h_L$	h_L /Q	$\sum h_L$ /Q	ΔQ	Qnew
I	1	0.08580	2.25	-0.0002	26.2	50.4	0.00002	0.08582
	2	0.00580	0.01		0.9			0.00582
	3	-0.03420	-0.27		8.0			-0.03418
	4	-0.12959	-1.98		15.3			-0.12961
II	4	0.12959	1.98	-0.007	15.3	88.6	0.00004	0.12961
	5	0.03539	0.97		27.5			0.03543
	6	-0.06461	-2.96		45.8			-0.06457

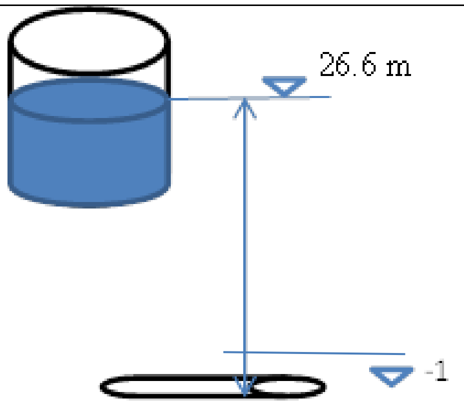
Trial No.6

Loop No.	Pipe No.	Q (m^3/sec)	h_L	$\sum h_L$	h_L /Q	$\sum h_L$ /Q	ΔQ	Qnew
I	1	0.08582	2.25	-0.0001	26.2	50.4	0.00001	0.08583
	2	0.00582	0.01		0.9			0.00583
	3	-0.03418	-0.27		8.0			-0.03417
	4	-0.12961	-1.98		15.3			-0.12960
II	4	0.12961	1.98	-0.0001	15.3	88.6	0.00000	0.12960
	5	0.03543	0.97		27.5			0.03543
	6	-0.06457	-2.96		45.8			-0.06457

Max. $|\Delta Q|$ =0.00001 m^3/sec …O.K.

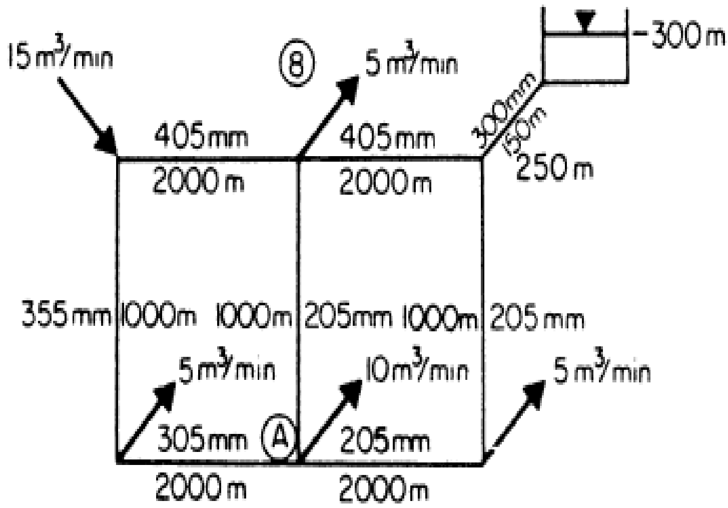
Head at feed point= head at A+ h_{L4}
$h_{L4}= K_4 Q_4^{1.85}$
$h_{L4}= 86.9\times 0.12960^{1.85} = 1.98$m

Head at feed point=25+1.98= 26.98m
Water level in the tank=-1+26.98+ h_{L7}
$h_{L7}= K_7 Q_7^{1.85}$
$h_{L7}= 5.9\times 0.28^{1.85} = 0.56$m
Water level in the tank=-1+26.98+ 0.56 =26.54m ≈ 26.6m

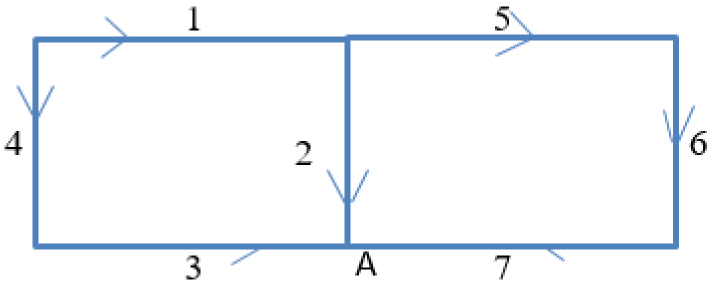


Example 3.2

Find the pressure at junction A of the flow system shown below. Assume all the pipes are PVC and the maximum allowed absolute error is 0.0001.



Q(m3/min)	Q(m3/sec)
9	0.150
2	0.033
-1	-0.017
-6	-0.100
-2	-0.033
2	0.033
12	0.200
7	0.117



Pipe No.	1	2	3	4	2	5	6	7
L (m)	2000	1000	2000	1000	1000	2000	1000	2000
D(mm)	405	205	205	355	205	405	205	205
K	163.6	2246	4492	155.3	2246	163.6	2246	4492

Trial No.1

Loop no.	Pipe	Q (m^3/sec)	HL	$\sum h_L$	h_L /Q	$\sum h_L$ /Q	ΔQ	Qnew
	1	0.150	4.89		32.6			0.1423
	2	0.033	4.16		124.7			0.0993
	3	-0.017	-2.31		138.4			-0.0244
I	4	-0.100	-2.19	4.55	21.9	317.6	-0.0077	-0.1077
	2	-0.033	-4.16		124.7			-0.0993
	5	0.033	0.30		9.1			-0.0404
	6	0.200	114.37		571.9			0.1263
II	7	0.117	84.39	194.91	723.3	1429.0	-0.0737	0.0429

Loop no.	Pipe	Q (m^3/sec)	HL	$\sum h_L$	h_L /Q	$\sum h_L$ /Q	ΔQ	Qnew

	1	0.142	4.44		31.2			0.1147
	2	0.099	31.33		315.4			0.0879
	3	-0.024	-4.67		191.4			-0.0519
I	4	-0.108	-2.52	28.57	23.4	561.3	-0.0275	-0.1353
	2	-0.099	-31.33		315.4			-0.0879
	5	-0.040	-0.43		10.7			-0.0565
	6	0.126	48.85		386.8			0.1102
II	7	0.043	13.28	30.37	309.3	1022.2	-0.0161	0.0269

Trial No.3

Loop no.	Pipe	Q (m^3/sec)	HL	$\sum h_L$	h_L /Q	$\sum h_L$ /Q	ΔQ	Qnew
	1	0.115	2.98		26.0			0.1107
	2	0.088	24.97		284.2			0.0951
	3	-0.052	-18.87		363.5			-0.0560
I	4	-0.135	-3.83	5.24	28.4	702.0	-0.0040	-0.1393
	2	-0.088	-24.97		284.2			-0.0951
	5	-0.056	-0.80		14.2			-0.0678
	6	0.110	37.98		344.6			0.0989
II	7	0.027	5.58	17.79	207.7	850.7	-0.0113	0.0156

Trial No.4

Loop no.	Pipe	Q	HL	$\sum h_L$	h_L /Q	$\sum h_L$ /Q	ΔQ	Qnew
	1	0.111	2.79		25.2			0.1064
	2	0.095	28.93		304.1			0.0930
	3	-0.056	-21.68		387.4			-0.0603
I	4	-0.139	-4.05	5.99	29.1	745.7	-0.0043	-0.1436
	2	-0.095	-28.93		304.1			-0.0930
	5	-0.068	-1.12		16.6			-0.0699
	6	0.099	31.09		314.3			0.0967
II	7	0.016	2.04	3.07	130.6	765.6	-0.0022	0.0134

Trial No.5

Loop no.	Pipe	Q	HL	$\sum h_L$	h_L /Q	$\sum h_L$ /Q	ΔQ	Qnew
	1	0.106	2.59		24.3			0.1056
	2	0.093	27.72		298.2			0.0940
	3	-0.060	-24.89		412.8			-0.0611
I	4	-0.144	-4.29	1.13	29.8	765.1	-0.0008	-0.1444
	2	-0.093	-27.72		298.2			-0.0940
	5	-0.070	-1.19		17.0			-0.0717
	6	0.097	29.84		308.4			0.0949
II	7	0.013	1.54	2.47	115.0	738.7	-0.0018	0.0116

Trial No.6

	1	0.106	2.55		24.2			0.1049
	2	0.094	28.28		300.9			0.0936
	3	-0.061	-25.50		417.4			-0.0618
I	4	-0.144	-4.33	1.00	30.0	772.5	-0.0007	-0.1451
	2	-0.094	-28.28		300.9			-0.0936
	5	-0.072	-1.25		17.4			-0.0721
	6	0.095	28.82		303.5			0.0946
II	7	0.012	1.18	0.47	101.7	723.5	-0.0003	0.0112

Trial No.7

Loop no.	Pipe	Q	HL	$\sum h_L$	h_L/Q	$\sum h_L/Q$	ΔQ	Qnew
	1	0.105	2.52		24.1			0.1047
	2	0.094	28.08		300.0			0.0938
	3	-0.062	-26.05		421.5			-0.0619
I	4	-0.145	-4.37	0.19	30.1	775.6	-0.0001	-0.1453
	2	-0.094	-28.08		300.0			-0.0938
	5	-0.072	-1.26		17.5			-0.0724
	6	0.095	28.62		302.6			0.0943
II	7	0.011	1.11	0.39	99.1	719.1	-0.0003	0.0110

Trial No.8

Loop no.	Pipe	Q	HL	$\sum h_L$	h_L/Q	$\sum h_L/Q$	ΔQ	Qnew
	1	0.105	2.52		24.0			0.1046
	2	0.094	28.17		300.4			0.0937
	3	-0.062	-26.15		422.2			-0.0620
I	4	-0.145	-4.38	0.16	30.1	776.8	-0.0001	-0.1454
	2	-0.094	-28.17		300.4			-0.0937
	5	-0.072	-1.27		17.6			-0.0724
	6	0.094	28.46		301.8			0.0942
II	7	0.011	1.06	0.08	96.9	716.6	-0.0001	0.0109

Head at junction B=300-250-h_L of feed pipe

$$K = \frac{L}{(O.278\, C\, D^{2.63})^{1.85}}$$

$$K = \frac{150}{(O.278\ \times 150 \times\ 0.3^{2.63})^{1.85}} = 52.83m$$

h_L= K Q $^{1.85}$=52.83×(10/60)$^{1.85}$=1.92m

Head at B=300-250-1.92= 48.08m

Head at A= Head at B-h_{L5}-h_{L2}

Head at A = 48.08-163.6×0.0724$^{1.85}$-2246×0.0937$^{1.85}$=18.68m

Ch.4 Water Pumping Stations

4-1 Purpose and Types of Water Pumping Stations

The main purpose of water pumping stations is to transfer water from low points to higher points. The main types of water pumping stations are:

a- Distribution pumping stations.

b- Surface water pumping stations.

a. <u>Distribution pumping stations</u>

The main components of distribution pumping stations (Fig.1) are:

1. Dry pumps (connected at parallel).
2. Suction pipe.
3. Storage and distribution tank.
4. Delivery (or discharge) pipe.
5. Valves.
6. Surge vessel (air chamber for water hammer protection).
7. Chlorination tank and chlorine injection pump.
8. Stand by generator and its fuel tank.
9. Main electricity distribution panel and control.
10. Service building.

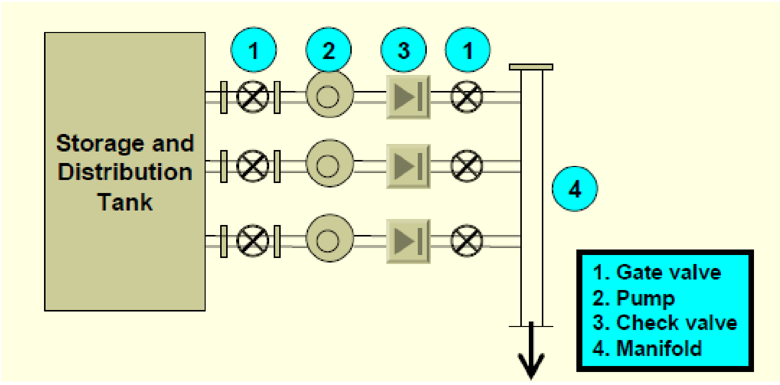


Fig.1 Typical layout of distribution pumping stations

<u>Control of distribution pumping stations</u>

To control the operation of distribution pumping stations, the followings are required:

1. Pressure switch at the discharge side of the pipe

If the pressure in the network increases above a preset value (for example 6bar), the pumps will be shut down one after the other. The pressure on the delivery pipe increases at low demands when many connections are closed.

2. Level switch connected to the water distribution tank.

If the water level in the water distribution tank drops to a pre assigned minimum level, the pumps are shut off one after the other with a pre assigned intervals. The pumps will be started again one after the other when the water in the tank reaches a pre assigned level. An ultra-sound level detector is usually used for water level detection.

b. <u>Surface water pumping stations</u>

The main components of surface water pumping stations are:

1. Submersible or dry pumps (connected at parallel)
2. Suction pipe
3. Delivery pipe
4. Valves
5. Stand by generator and its fuel tank
6. Main electricity distribution panel and control
7. Service building

4-2 Types of pumps

Generally, Pumps are classified into two main categories:

1. Kinetic pumps; like centrifugal pumps (radial, axial, mixed flow). Centrifugal pumps (Fig.2) are the most used type for water pumping.

2. Positive displacement pumps; like rotary pumps (Fig.3)

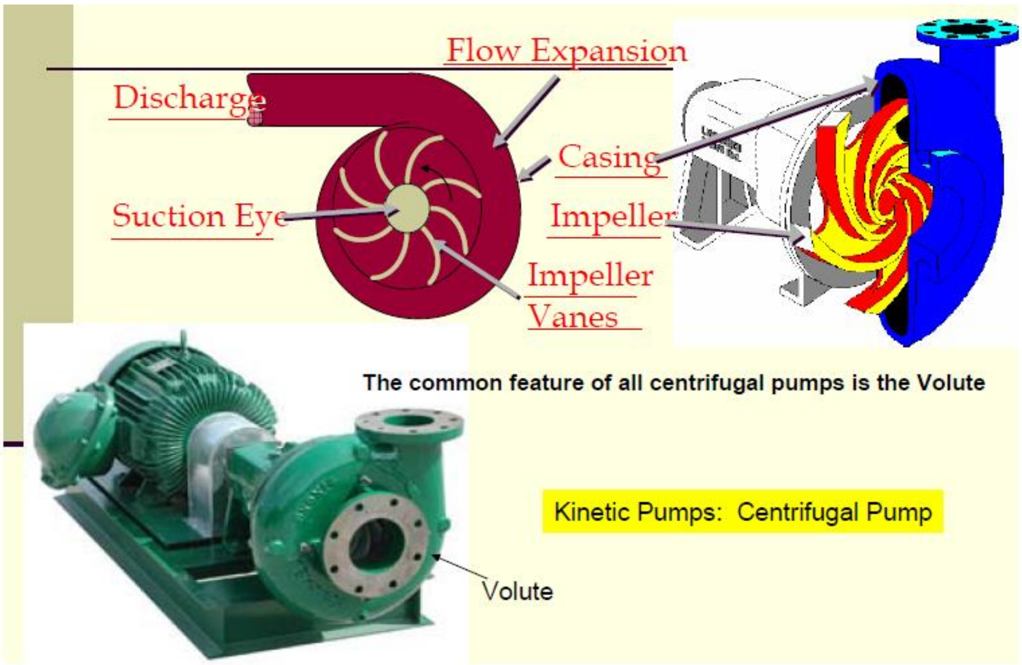


Fig.2 Centrifugal pump

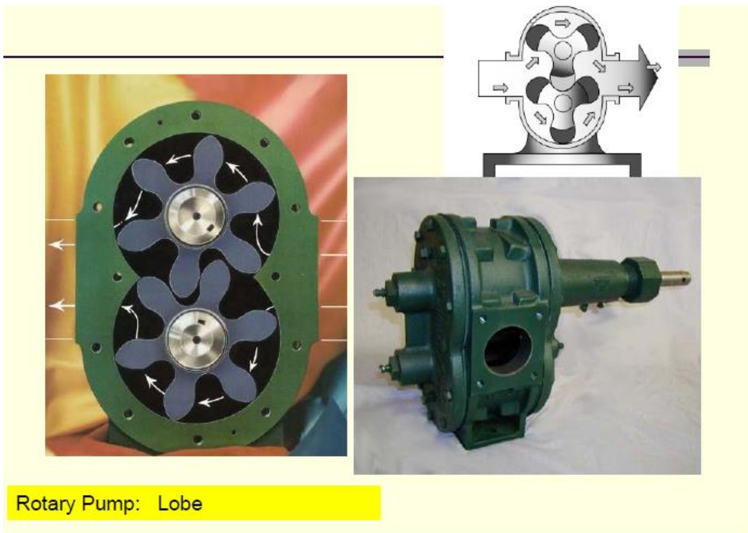


Fig.3 Rotary pump

Advantages of Centrifugal Pump

1. Small in size, space saving & less capital costs

2. Easy for maintenance
3. No danger creates if discharge valve is closed while starting
4. Deal with large volume
5. Able to work medium to low head
6. Able to work medium to low viscous fluid

Disadvantage of Centrifugal pump

1. Extra priming pump is required.
2. Cannot be able to work high head.
3. Cannot deal with high viscous fluid.

Pump priming

A centrifugal pump is said to be primed when there is a positive pressure of water on the suction side of the pump, and the volute is full of water. Thus, when the impeller starts moving, water starts moving, and a flow can be established. If the upstream side of the pump is dry, the pump is said to become "un-primed". It must then be primed before it can be operated again. A positive displacement pump is used to develop a negative pressure in the upstream pipework sufficient to draw water into the suction side of the pump and establish water flow. Priming pump is not provided in the following cases:

1. If the suction head of a centrifugal pump is positive
2. If the pump is used for circulating purpose.

When the suction head is positive; as soon as the suction valve is opened, the suction line is filled with water and this water expels the air in the suction pipe.

4-3 Power of Pumping

The power required to operate a pump (electrical power or motor power) is calculated as;

$$P_m = \frac{\gamma Q H}{E_o}$$

Where;

P_m= Motor power, kW

γ= specific weight of water = 9.81 kN/m^3

H= Total dynamic head, m

E_o= overall efficiency= pump efficiency × motor efficiency

Total Dynamic Head (TDH)

$$TDH = TSH + h_{LT}$$

Where; TSH is the total static head and h_{LT} is the total head losses.

Total Static Head

Case-A: If the pump withdraws the water from a water level lower than the level of its centerline, Fig.5;

$$TSH = discharge\ head + suction\ lift$$

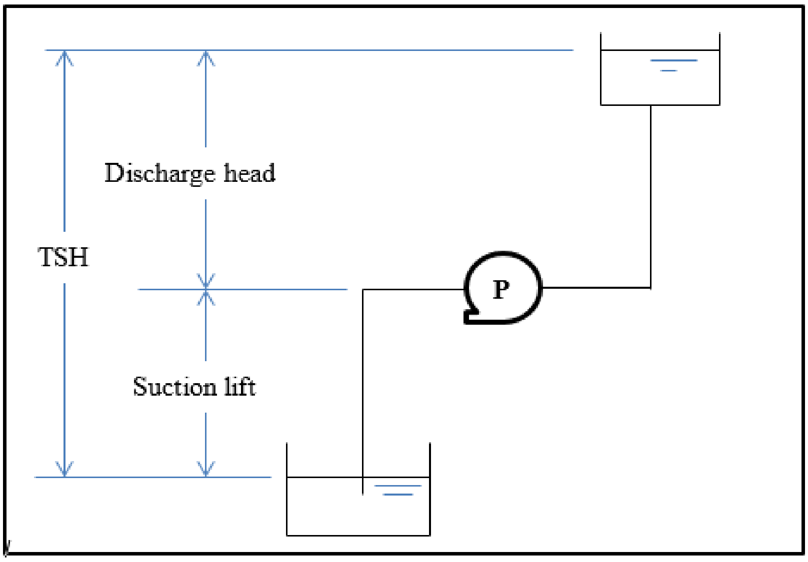


Fig.5 Case-A of pump location

Case-B: If the pump withdraws the water from a water level higher than the level of its centerline, Fig.6;

$$TSH = discharge\ head - suction\ head$$

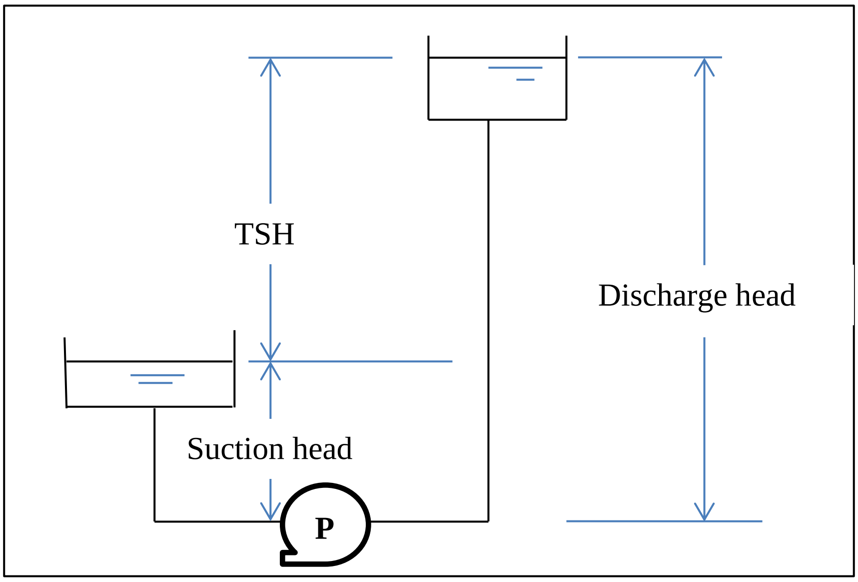


Fig.6 Case-B of pump location

Total head loss

The total sum of head losses is the sum of major head losses (due to friction in pipelines) and minor head losses (due to various pipe fittings) in suction and discharge pipes;

$$h_{LT} = h_l + h_m$$

where; h_l is the major head loss and h_m is the minor head loss. The major head loss is obtained using Hazen- William equation or Darcy-Wisbach equation;

Hazen- William equation

$$h_l = K\,Q^{1.85} \quad ; \quad K = \frac{L}{(0.278CD^{2.63})^{1.85}}$$

Darcy-Wisbach equation

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

Where;

f = friction coefficient which is dependent on Re (Reynolds No.) and $\frac{e}{D}$ (pipe roughness/ diameter) and can be obtained using Moody diagram.

L= pipe length, m.

D= pipe diameter, m.

V= flow velocity, m/sec.

The minor head loss is due to pipe fittings and it can be obtained as;

$$h_m = k\frac{V^2}{2g}$$

where k is minor loss coefficient and its value is dependent on fitting type and diameter as shown in Table 1(Table 7.1, p.156). Or, h_m is obtained using equivalent length method by which the minor head loss is expressed into major head loss in a straight pipe of length equals equivalent length. In this method;

$$h_{LT} = KQ^{1.85};$$

$$K = \frac{L + \sum L_{eq}}{(0.278CD^{2.63})^{1.85}}$$

Where;

$\sum L_{eq}$ *is summation of equivalent lengths for all the incorporated fittings.*

$L_{eq.}$ is dependent on fitting type and diameter and can be obtained from Table 2 (Table 7.2, p.157).

Table 1: k values of typical fittings verses pipe diameter (in

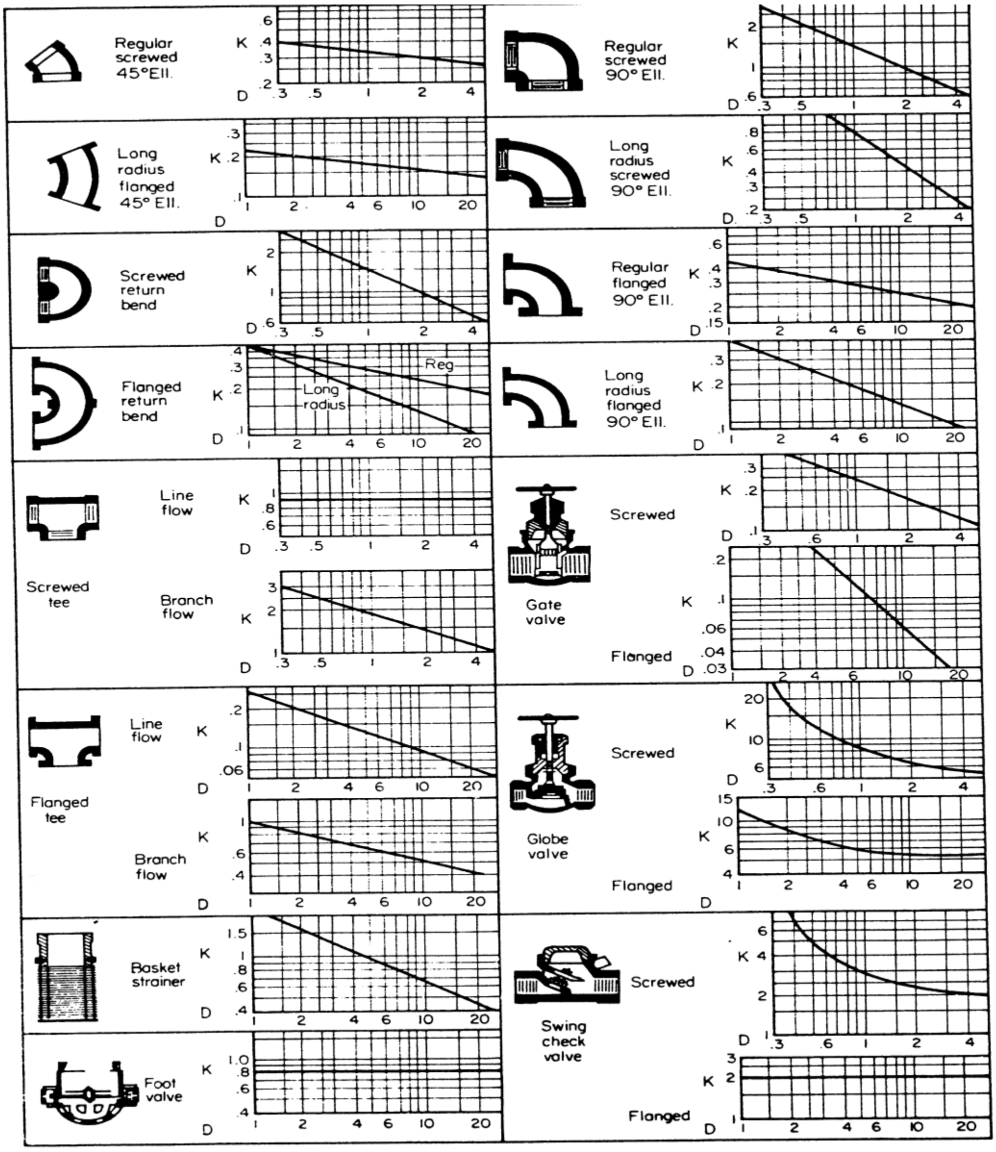


Table 2: Equivalent lengths (foot) for different fittings

Pipe size, in†	Standard ell	Medium radius ell	Long-radius ell	45° ell	Tee	Gate valve, open	Globe valve, open	Swing check, open
1	2.7	2.3	1.7	1.3	5.8	0.6	27	6.7
2	5.5	4.6	3.5	2.5	11.0	1.2	57	13
3	8.1	6.8	5.1	3.8	17.0	1.7	85	20
4	11.0	9.1	7.0	5.0	22	2.3	110	27
5	14.0	12.0	8.9	6.1	27	2.9	140	33
6	16.0	14.0	11.0	7.7	33	3.5	160	40
8	21	18.0	14.0	10.0	43	4.5	220	53
10	26	22	17.0	13.0	56	5.7	290	67
12	32	26	20.0	15.0	66	6.7	340	80
14	36	31	23	17.0	76	8.0	390	93
16	42	35	27	19.0	87	9.0	430	107
18	46	40	30	21	100	10.2	500	120
20	52	43	34	23	110	12.0	560	134
24	63	53	40	28	140	14.0	680	160
36	94	79	60	43	200	20.0	1000	240

Example 4.1

Find the total head loss in a DI pipeline transports water at a flowrate of 1080m^3/hr. the pipeline has a length of 1km and a diameter of 500 mm. It contains 5 gate valves, 3 standard 90° elbows and 1 check valve. Use the two methods for minor losses calculation.

Solution

$$h_{LT} = h_l + h_m$$

$$h_l = K\,Q^{1.85}$$

$$K = \frac{L}{(0.278CD^{2.63})^{1.85}}$$

For DI pipe, C=90

$$K = \frac{1000}{(0.278 \times 90 \times 0.5^{2.63})^{1.85}} = 75.48$$

Q=1080 m^3/hr. = 0.3 m^3/sec

$$h_l = 75.48 \times 0.3^{1.85} = 8.14m$$

$$h_m = k\frac{V^2}{2g}$$

For D=500mm (20in);

For gate valve, k= 0.03

For standard 90˚ elbows, k= 0.21

For check valve, k=2

$$V = \frac{Q}{A} = \frac{0.3}{\left(\frac{\pi \times 0.5^2}{4}\right)} = 1.528m/sec$$

$$h_m = (5 \times 0.03 + 3 \times 0.21 + 1 \times 2) \times \frac{1.528^2}{2 \times 9.81} = 0.33m$$

$$\therefore h_{LT} = 8.14 + 0.33 = 8.47m$$

If equivalent length method is used for minor losses calculation;

$$h_{LT} = KQ^{1.85};$$

$$K = \frac{L + \sum L_{eq}}{(0.278CD^{2.63})^{1.85}}$$

The equivalent lengths for fittings of 500mm (20 in) diameter are:

For gate valve, L_{eq}= 12 ft = 20/3.28 = 3.65m

For standard 90˚ elbows, L_{eq}= 52 ft = 52/3.28 = 15.9m

For check valve, L_{eq}= 134 ft = 134/3.28= 40.9m

$$\sum L_{eq} = 5 \times 3.65 + 3 \times 15.9 + 1 \times 40.9 = 106.9m$$

$$K = \frac{1000 + 106.9}{(0.278 \times 90 \times 0.5^{2.63})^{1.85}} = 83.54m$$

$h_{LT} = K\ Q^{1.85} = 83.54 \times 0.3^{1.85} = 9.01m$

4-4 System Head Curve

It is a curve represents the relation between total dynamic head (TDH) and the discharge (Q) of the flow system, see Fig.6.

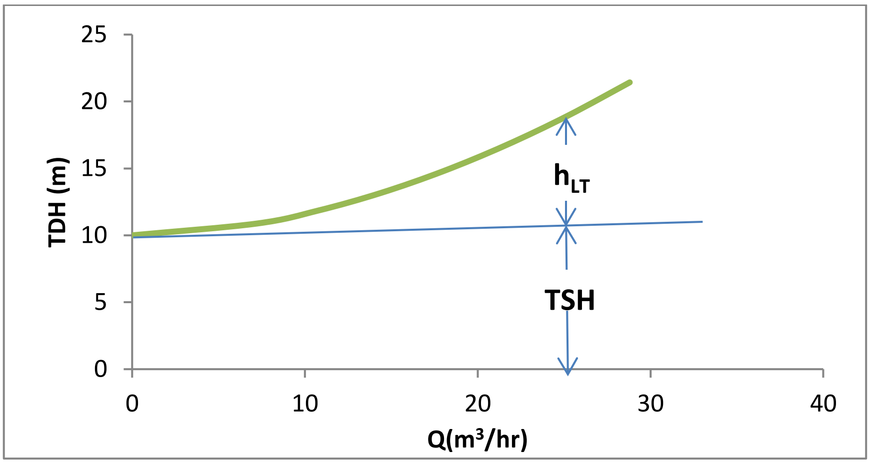


Fig.7 System head curve

4-5 Pump Characteristic Curves

Pumps characteristic curves (or pump performance curves) are a set of curves which represent the relations between total dynamic head, power and efficiency of pump verses water discharge. These curves are given for a specific pump by pump manufacturer. An example of pump characteristic curves is shown in Fig.8.

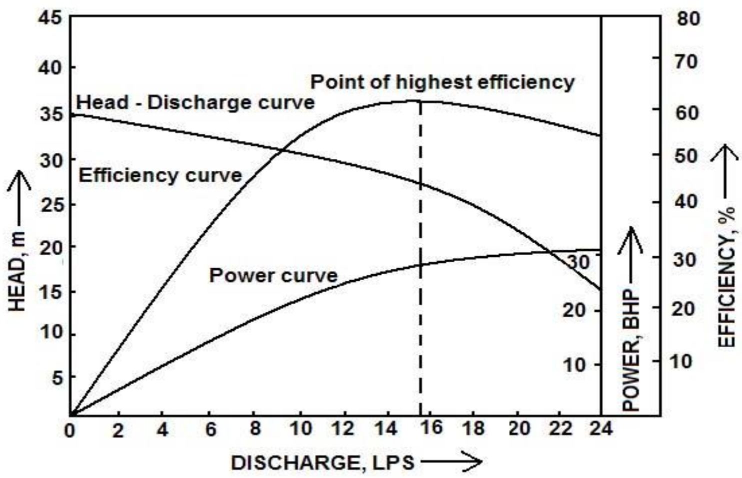


Fig.8 Pump characteristic curves

4-6 Pumps Selection

Pumps are selected for specific application by plotting the system head curve and pump head curve on the same graph, Fig.9. The intersection point of the two curves is called operation point. The x-coordinate and y-coordinate of this point give the operating flowrate and total dynamic head (Q_o and H_o), respectively.

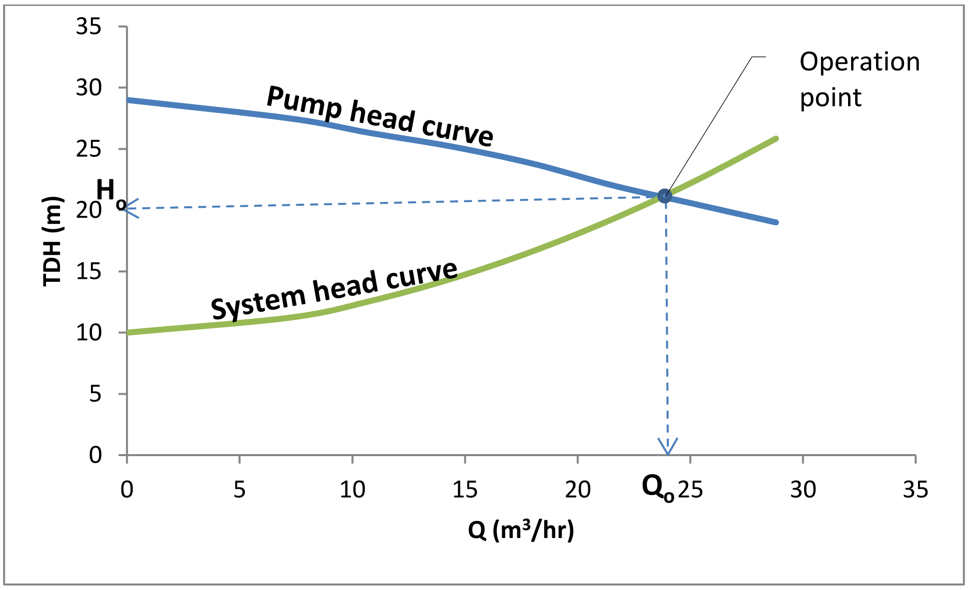
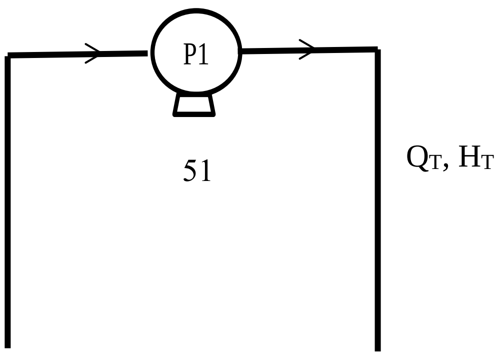


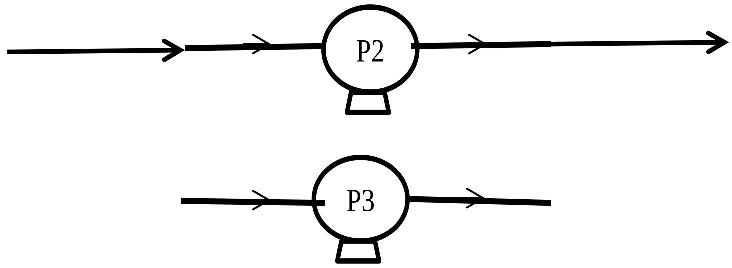
Fig.9 Specification of operating point

The pump is selected if $Q_o \geq Q_{req.}$ *and* $H_o \geq H_{req.}$.While, if $Q_o < Q_{req.}$ *or* $H_o < H_{req}$, then, the pump is declined (not selected).

4-7 Pumps Connection

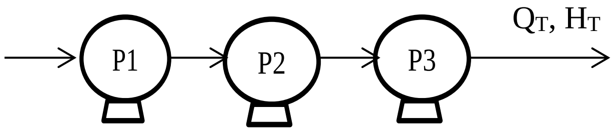
In a pumping station, the pumps may be connected in parallel or series to increase the capacity or head of the pumping station. For three operating pumps connected in parallel;





$Q_T = Q_1 + Q_2 + Q_3$ and $H_T = H_1 = H_2 = H_3$

For three operating pumps connected in series;



$$Q_T = Q_1 = Q_2 = Q_3$$

$$H_T = H_1 + H_2 + H_3$$

The system head curve is not changed when two or more pumps are connected. However, the pump head curve is changed according to the number of connected pumps and their method of connection as shown in Figs. 10 and 11.

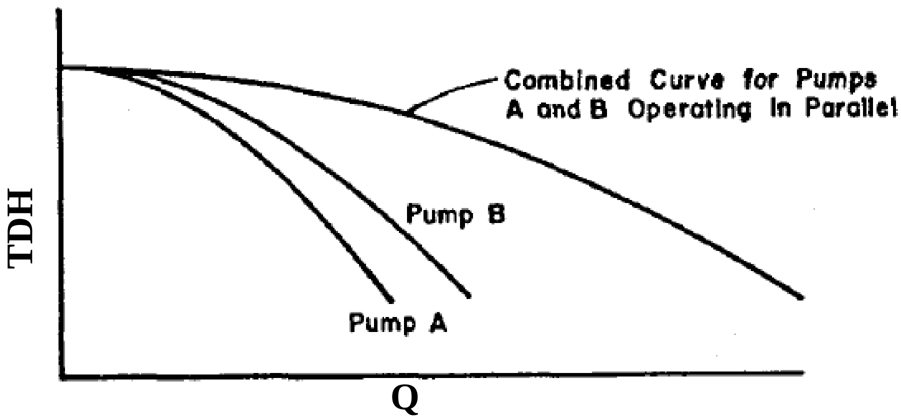


Fig.10 Pump head curve for two pumps connected in parallel

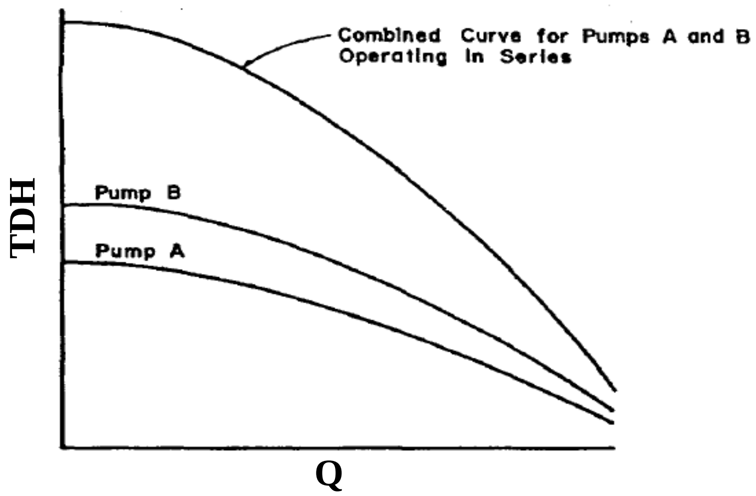


Fig.11 Pump head curve for two pumps connected in series

Example 4.2

A pumping station composes of three identical pumps connected at parallel (2 working and 1 standby) is designed to have total static head of 20m. The pumping station shall serve a village has maximum daily water demand of 3600 m^3/day. The used pump has the characteristics given below. Does the pumping station satisfy the village requirement of water quantity and pressure?. Assume the discharge pipe is PVC and has a length of 1000m and a diameter of 200mm and neglect the minor losses.

Q (m^3/hr)	21.6	32.4	43.2	54	64.8	75.6	86.4	97.2	108
TDH (m)	27.5	26.3	25.2	23.8	22	20.5	19	17	15.2

Solution

Draw the system head curve;

$TDH = TSH + h_{LT}$

By neglecting the minor head losses;

$h_{LT} = KQ^{1.85}$

$TDH = TSH + KQ^{1.85}$

$$K = \frac{L}{(0.278CD^{2.63})^{1.85}}$$

For PVC pipe, C=150

$$K = \frac{1000}{(0.278 \times 150 \times 0.2^{2.63})^{1.85}} = 2532.7$$

∴ The equation of system head curve is;

$$TDH = 20 + 2532.7Q^{1.85}$$

For Q=21.6 m3/hr;

$$TDH = 20 + 2532.7 \times \left(\frac{21.6}{3600}\right)^{1.85} = 20.2\text{m}$$

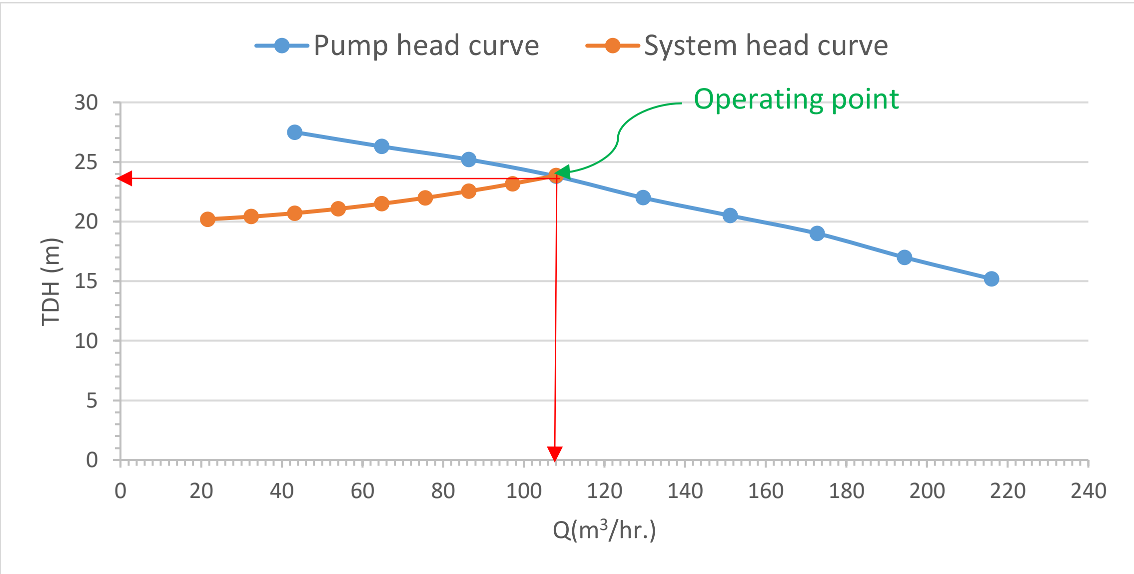
<u>System head curve;</u>

Q (m3/hr)	21.6	32.4	43.2	54	64.8	75.6	86.4	97.2	108
TDH (m)	20.2	20.4	20.7	21.1	21.5	22.0	22.6	23.2	23.9

<u>Pump head curve for two working pumps connected at parallel;</u>

Q (m^3/hr)	43.2	64.8	86.4	108	129.6	151.2	172.8	194.4	216
TDH (m)	27.5	26.3	25.2	23.8	22	20.5	19	17	15.2

Draw the system and pump head curves on the same graph;



From the graph;

Q_o=108m^3/hr. and H_o=23.9m

$Q_{req.}$= 3600 m^3/day =150m^3/hr.

To find the required TDH, substitute the required flowrate in system head curve equation, thus;

$$TDH_{req.} = 20 + 2532.7 \times \left(\frac{150}{3600}\right)^{1.85} = 27.08m$$

$\because$ Q_o<$Q_{req.}$ and TDH_o< TDH_{req}

$\therefore$ The pumping station doesn't satisfy the village requirements of water quantity and pressure

Example 4.3

A pumping station is composed of two identical pumps connected at parallel (1 working and 1 standby). The station is used to lift water from a level of 2m to a level of 28m at a flowrate of 24m^3/hr. Two pump types are available with the characteristics given below. Select the suitable pump type and give the reason behind your selection. The discharge pipe is DI and has a length of 500m and a diameter of 100mm. It contains four gate valves, one check valve and 5 90^o bends.

Q(m^3/hr)		0	7.2	10.8	14.4	18	21.6	25.2	28.8
TDH (m)	Type (A)	29	27.5	26.3	25.2	23.8	22	20.5	19
	Type (B)	46	45	44	42	40	38	35	33

Solution

TDH=TSH+h_{LT}

TSH=28-2=26m

h_{LT}= $KQ^{1.85}$

$$K = \frac{L + \sum L_{eq}}{(0.278CD^{2.63})^{1.85}}$$

The equivalent lengths of fittings of 100mm diameter are:

$L_{eq.}$ for gate valve= 2.3ft=2.3/3.28=0.7m

$L_{eq.}$ for check valve= 27 ft=27/3.28= 8.23m

$L_{eq.}$ for bend= 11ft= 11/3.28= 3.35m

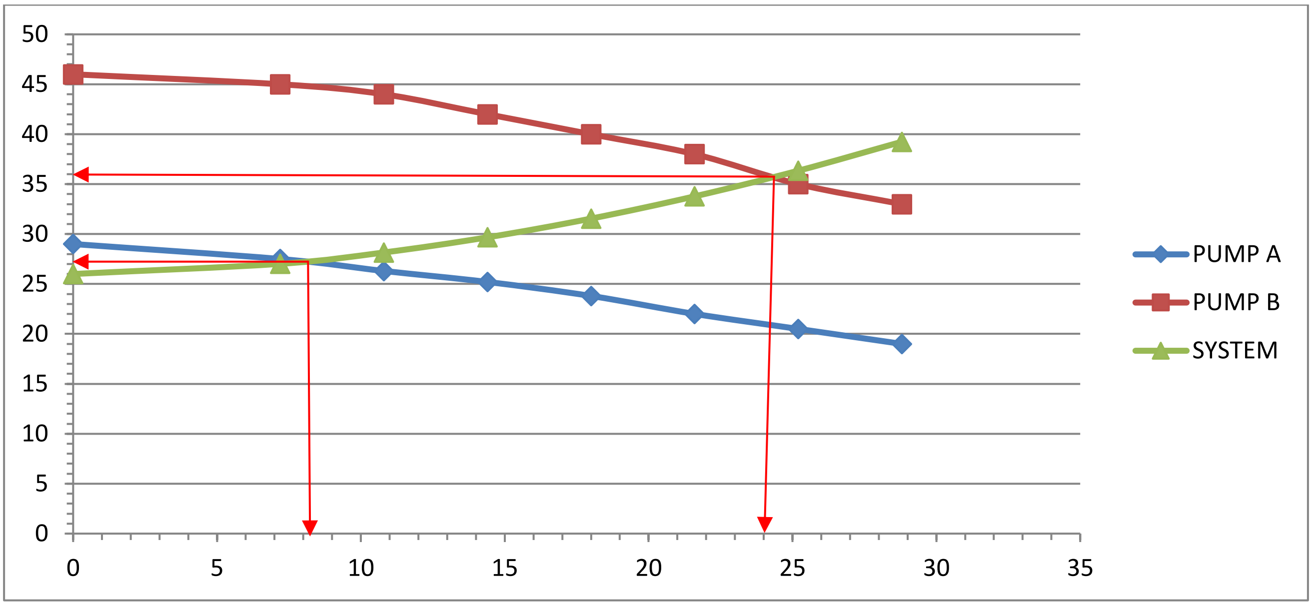
$\sum L_{eq.}$= 4×0.7+ 1×8.23+5×3.35 = 27.78m

$$K = \frac{500+27.78}{(0.278\times 90\times 0.1^{2.63})^{1.85}} = 100258.55$$

TDH=26+100258.55×$Q^{1.85}$

Draw system head curve and pumps head curves on the same graph

Q (m^3/hr)	TDH (m)		
	A	B	System
0	29	46	26.0
7.2	27.5	45	27.0
10.8	26.3	44	28.2
14.4	25.2	42	29.7
18	23.8	40	31.5
21.6	22	38	33.8
25.2	20.5	35	36.3
28.8	19	33	39.2



$Q_{req.}$=24 m^3/hr

TDH_{req}=26+100258.55 $(24/3600)^{1.85}$

TDH_{req}= 35.45m

If pump type A is selected; Q_O= 8.2 m^3/hr and Ho= 27.2m

If pump type B is selected; Q_O= 24 m^3/hr and Ho=35.8 m

Select pump type B because it can give the required TDH and Q

Example 4.4

Use the following data to determine the power required for operating a pumping station composes of 4 similar pumps (3 working & 1 standby) connected at series.

Use the following data:

- Discharge head = 15m.
- *Suction lift=2m.*
- Length of suction pipe=100m.
- Diameter of suction pipe= 150mm.
- Length of discharge pipe=750m.
- Diameter of discharge pipe= 150mm.
- The suction pipe contains one foot valve, two 90° standard elbow, and two gate valves.

- The discharge pipe contains one check valve, three 90° standard elbow, and two gate valves.
- The characteristics of one pump is as given in the following table:

Q(m^3/hr)	0	10	20	30	40	50	60	70	80
TDH(m)	53	52.5	51	50	48	45	42	35	26
Efficiency (%)	0	20	36	51	64	70	66	60	53

Solution

TDH=TSH+h_{LT}

TSH=15+2=17m

$$h_{LT} = h_{LT}\, for\ suction\ pipe + \ h_{LT}\ for\ discharge\ pipe$$

For suction pipe;

$$h_{LT} = h_l + \ h_m$$

h_L= $KQ^{1.85}$

$$K = \frac{L}{(0.278CD^{2.63})^{1.85}}$$

Assume the pipes are ductile iron, C=90

$$K = \frac{100}{(0.278 \times 90 \times 0.15^{2.63})^{1.85}} = 2641.8$$

h_L= $2641.8Q^{1.85}$

$$h_m = k\frac{V^2}{2g} = k\frac{\left(\frac{Q}{A}\right)^2}{2g} = k\frac{\left(Q/\left(\frac{\pi D^2}{4}\right)\right)^2}{2g}$$

For pipe of 150mm diameter;

For foot valve, k= 0.8

For 90 standard elbow, k=0.18

For gate valve, k= 0.14

$$\therefore h_m = k\frac{\left(\frac{Q}{\left(\frac{\pi D^2}{4}\right)}\right)^2}{2g} = (1 \times 0.8 + 2 \times 0.18 + 2 \times 0.14)\frac{\left(Q/\left(\frac{\pi \times 0.15^2}{4}\right)\right)^2}{2 \times 9.81}$$

$h_m = 235.03Q^2$

$\therefore$ For suction pipe;

$h_{LT} = 2641.8Q^{1.85} + 235.03Q^2$

For discharge pipe;

$h_{LT} = h_l + h_m$

$h_L = KQ^{1.85}$

$$K = \frac{L}{(0.278CD^{2.63})^{1.85}}$$

$$K = \frac{750}{(0.278 \times 90 \times 0.15^{2.63})^{1.85}} = 19813.3$$

$h_L = 19813.3Q^{1.85}$

$$h_m = k\frac{V^2}{2g} = k\frac{\left(\frac{Q}{A}\right)^2}{2g} = k\frac{\left(Q/\left(\frac{\pi D^2}{4}\right)\right)^2}{2g}$$

For pipe of 150mm diameter;

For check valve, k= 2

For 90 standard elbow, k=0.18

For gate valve, k= 0.14

$$\therefore h_m = k\frac{\left(\frac{Q}{\left(\frac{\pi D^2}{4}\right)}\right)^2}{2g} = (1 \times 2 + 3 \times 0.18 + 2 \times 0.14)\frac{\left(Q/\left(\frac{\pi \times 0.15^2}{4}\right)\right)^2}{2 \times 9.81}$$

$h_m = 460.3Q^2$

$\therefore$ For discharge pipe;

$h_{LT} = 19813.3Q^{1.85} + 460.3Q^2$

For suction and discharge pipes;

$$h_{LT} = 2641.8Q^{1.85} + 235.03Q^2 + 19813.3Q^{1.85} + 460.3Q^2$$

$$\therefore h_{LT} = 22455.1Q^{1.85} + 695.33Q^2$$

For system head curve;

TDH=17+$22455.1Q^{1.85} + 695.33Q^2$

For Q=10 m^3/hr;

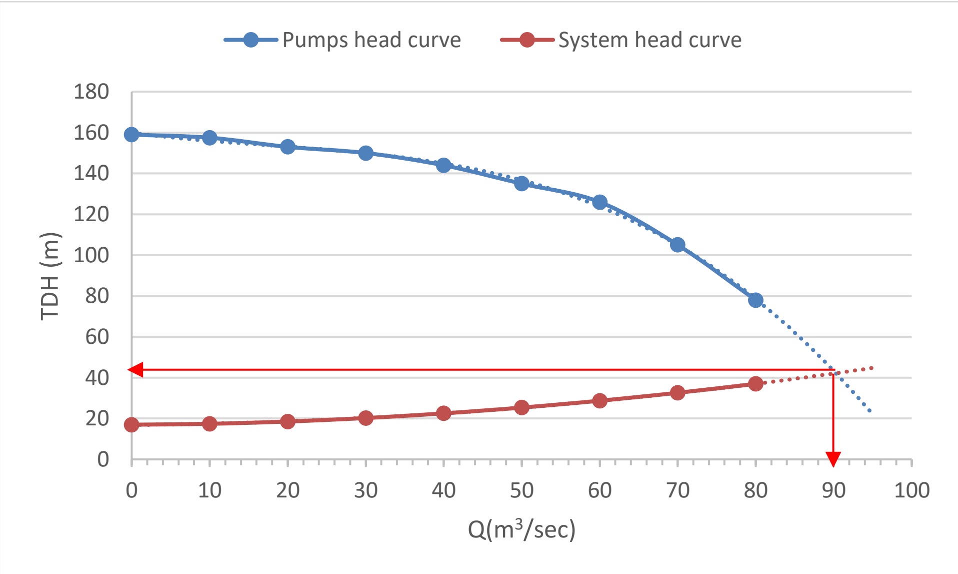
$$TDH = 17 + 22455.1 \times \left(\frac{10}{3600}\right)^{1.85} + 695.33 \times \left(\frac{10}{3600}\right)^2 = 17.4m$$

Q(m3/hr)	0	10	20	30	40	50	60	70	80
TDH (m)	17.0	17.4	18.5	20.2	22.5	25.4	28.7	32.6	37.0

<u>For three working pumps connected at series</u>

Q(m^3/hr)	0	10	20	30	40	50	60	70	80
TDH(m)	159	157.5	153	150	144	135	126	105	78

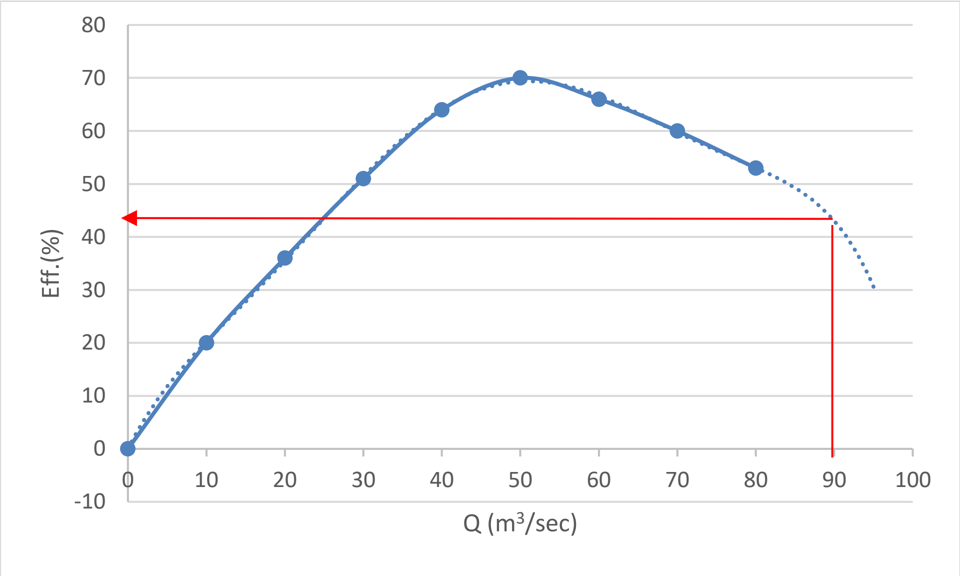
Draw the system and pump head curves on the same graph;



From the graph;

Q_o=90m^3/hr. and H_o=41.5m

Draw pumps efficiency verses water flowrate and find the efficiency at Q=90 m^3/hr.;



From the above graph;

At Q=90m^3/hr., pumps efficiency=43%

$$P_m = \frac{\gamma Q H}{E_o} = \frac{9.81 \times \left(\frac{90}{3600}\right) \times 41.5}{0.43} = 23.7 \text{ kW}$$

4-8 Cavitation

Cavitation is the phenomenon of cavities formation in pump impeller. Pump cavitation occurs when the absolute pressure in the pump inlet drops below the vapor pressure of the liquid. As the net positive suction head is reduced, a point is reached where cavitation becomes detrimental (vapor bubbles form at the inlet of the pump and are moved to the discharge of the pump where they collapse, often taking small pieces of the pump impeller with them, Fig.12). This point is called minimum net positive suction head ($NPSH_{min}$). The value of $NPSH_{min}$ is dependent on pump type and water flowrate. It is given by pump manufacturer.

Fig.12 Cavities in pump impeller

To avoid cavitation, the vertical distance between the surface of the liquid in the supply tank (or any water source) and the centreline of the pump (Z) is obtained as;

$$Z = \frac{P_a - P_v}{\gamma} - NPSH_{min} - h_l$$

Z= the vertical distance between the surface of the liquid in the supply tank and the centerline of the pump, m.

P_a = atmospheric pressure at the location of pump installation, kPa.

P_v= vapor pressure of water, kPa.

h_l= total head loss of suction pipe, m.

$NPSH_{min}$ = minimum net positive suction head as obtained from pump manufacturer, m.

If the calculated value of Z is positive, then, the pump must be installed at a vertical distance not exceeding Z above the water level of supply tank, Fig.13a. While, if the calculated value of Z is negative, then, the pump must be installed at a vertical distance not less than $|Z|$ below the water level of supply tank, Fig.13b.

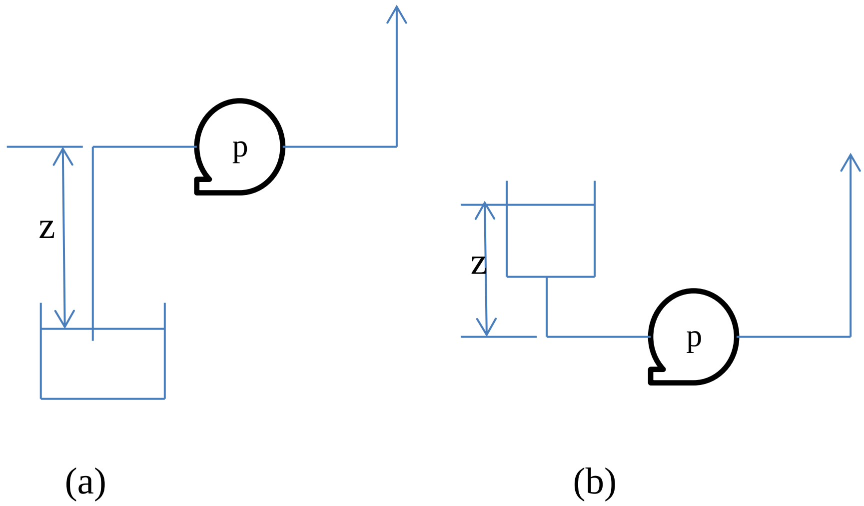


Fig.13 Pump installation cases

The value of atmospheric pressure (P_a) is dependent on altitude of pump installation area and can be obtained from Table 3 (Table 7.3, p.162). *Hint: the obtained P_a value from Table 3 is reduced by 3.5 kPa to account for pressure drop during storm events.*

Table 3: Parametric pressure verses altitude

Altitude (m)	0	305	457	610	1220	1830	2439
Pressure (kPa)	101	98	96	94	88	81	75

The value of vapor pressure of water is dependent on water temperature and can be obtained from Table 4 (Table 7.4, p.163).

Table 4: Vapor pressure of water verses temperature

Temperature (°C)	Vapor pressure (kPa)
0	0.61
4.4	0.84
10	1.23
15.6	1.76
21.1	2.5
26.7	3.5
32.2	4.81
37.8	6.54

Example 4.5

A pumping station is composed of 2 pumps (1W+1S) connected at parallel. It is constructed at an altitude of zero and used to lift water from a level of -2m to a level of 58m. The suction pipe has a length of 50m and a diameter of 600mm. The discharge pipe has a length of 10km and a diameter of 600mm. Find the level of pumps installation if the water temperature varies over the range (15-28)°C. Neglect the minor losses and take C=90. The used pumps have the characteristics given below.

$Q(m^3/hr)$	0	200	300	400	500	600	700	800
TDH (m)	79	76	74	72	69	65	60	55
NPSH (m)	2.0	2.0	2.0	2.0	2.4	2.4	3.2	4.3

Solution

$$Z = \frac{P_a - P_v}{\gamma} - NPSH_{min} - h_l$$

At altitude of zero, Pa=101kPa

To consider storm effect, Pa=101-3.5=97.5 kPa

Pv is dependent on water temperature.

For T=15°C;

At T=10 °C Pv=1.23 kPa

At T=15.6°C Pv=1.76 kPa

By using interpolation; Pv=1.703 kPa

For T=28

At T=26.7 °C Pv=3.5 kPa

At T=32.2°C Pv=4.81 kPa

By using the interpolation; Pv=3.81 kPa

NPSHmin and hl are function of Qo. To find Qo , the system head curve and pump head curve must be plotted on the same graph.

System head curve

TDH=TSH+ h_{l_T}

TSH=58-(-2)=60m

$$h_{l_t} = h_{l_{suction}} + h_{l_{discharge}}$$

$h_{l_{suction}}$=K $Q^{1.85}$; K=$\frac{L}{(0.278 \times C \times D^{2.63})^{1.85}}$

$h_{l_{suction}} = \frac{50}{(0.278 \times 90 \times 0.6^{2.63})^{1.85}} Q^{1.85}$=$1.554Q^{1.85}$

$h_{l_{discharge}} = \frac{10000}{(0.278 \times 90 \times 0.6^{2.63})^{1.85}} Q^{1.85}$=$310.9Q^{1.85}$

$h_{l_T} = 1.554Q^{1.85}$+ $310.9Q^{1.85}$=$312.5Q^{1.85}$

TDH=60+ $312.5Q^{1.85}$

To draw system head curve;

at Q=0…… TDH=60m

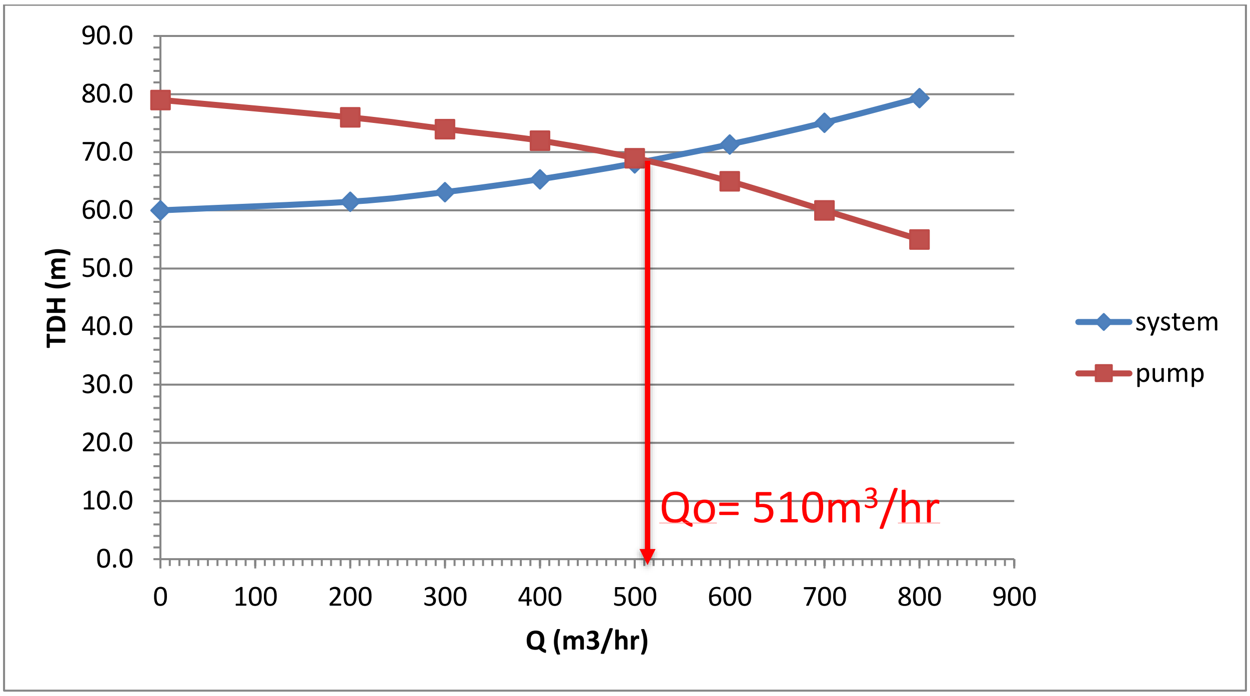
at Q=200m^3/hr …… TDH=60+ 312.5×$(200/3600)^{1.85}$ =61.5m

For different Q values;

Q (m^3/hr)	0	200	300	400	500	600	700	800
TDH (m)	60.0	61.5	63.1	65.4	68.1	71.4	75.1	79.3

To draw pump head curve;

Q (m^3/hr)	0	200	300	400	500	600	700	800
TDH (m)	79	76	74	72	69	65	60	55



From the graph Q_o=510m^3/hr

From the given table…for Q=510m^3/hr……NPSHmin=2.4m

$h_{l_{suction}}$=1.554$Q^{1.85}$ = 1.554× $(\frac{510}{3600})^{1.85}$=0.042m

$$Z = \frac{P_a - P_v}{\gamma} - NPSH_{min} - h_l$$

For T=15°C;

$$Z = \frac{97.5 - 1.703}{9.81} - 2.4 - 0.042 = 7.32m$$

For T=28°C;

$$Z = \frac{97.5 - 3.81}{9.81} - 2.4 - 0.042 = 7.11m$$

∴ THE pump must be placed at a level not exceeding 7.11m above the water level in the supply tank

∴ pumps level=-2+7.11=5.11m

Ch.5 Water Intakes

5-1 Definition of Intakes

Intakes are structures constructed in or adjacent to lakes, reservoirs, or rivers for the purpose of withdrawing water. In general, they consist of an opening with a screen or strainer through which the water enters, and a conduit (an open channel or a pipe line) to conduct the water a low-lift pumping station. The water is pumped by the low-lift pumping station to the water treatment plant.

5-2 Key Requirements of Intake Structures

The key requirements of the intake structures are

1. Reliable.
2. Of adequate size to provide the required quantity of water.
3. Located to obtain the best quality of water.
4. Protected from objects that may damage equipment.
5. Easy to inspect and maintain.
6. Designed to minimize damage to aquatic life.
7. Located to minimize navigational hazards.

5-3 Design Elements of Intake Structures

5-3-1 Reliability

Reliability is an essential feature of intake structures. The water supply system ceases to function when the intake system fails. For larger systems, current design practice provides for duplicate intake structures that include multiple inlet ports, screens, conduits, and pumping units.

5-3-2 Capacity

Because the intake structures are very difficult to expand to provide additional capacity, a design life of the intake structures in the range of 20 to 40 years should be considered.

5-3-3 Location

The major factors to be considered in locating the intake are:

Water quality

The water quality in water sources is effected by;

- water currents,
- wind and wave impacts, and
- water depth due to stratification.

Water depth

The followings are considered;

- maximum available,
- adequate submergence over inlet ports, and
- ice problems avoidance.

Treatment facility

Minimize conduit length to treatment plant.

Cost

Minimize operation & maintenance requirements.

5-4 Types of Intakes

Intake structures may be classified into two categories; exposed intakes and submerged intakes. Many varieties of these types have been used. The selection of the intake type is highly dependent on water source type.

Exposed intakes:

- Tower in lake or impounding reservoir (applicable to large systems and expensive)
- Shore inlet (design for floating debris and/or ice)
- Floating or movable (good access for operation and maintenance)
- Siphon well (applicable to small systems, flexible, and easy to expand)

Submerged intakes:

- Plain-end pipe or elbow (applicable to small systems)
- Screened inlet crib (no navigational impact, no impact from floating debris or ice, not flexible and difficult operation and maintenance)
- Gravel-packed well (no navigational impact, no impact from floating debris or ice and must have favorable geology)
- Horizontal collection systems or infiltration bed (no navigational impact, no impact from floating debris or ice, and must have favourable geology)

5-4-1 Lakes and reservoirs intakes

Because of their navigational impacts as well as severe winter weather and consequent difficulties in their operation and maintenance, exposed structures are not often used in the cold-climate lakes. On the other hand, exposed intake structures have been widely used in warm-climate lakes and in reservoirs. A classic tower design (Fig.1) includes multiple intake ports at different elevations, screens for each port, and access for maintenance. It is accessed by a bridge or boat. Submerged intake structures avoid many of the problems of the exposed systems but are significantly more difficult to maintain because of lack of access. A typical submerged inlet structure is shown in Fig.2. With a favourable geologic stratum of sand and gravel on the shore or the bottom of the lake or reservoir, either an infiltration gallery as shown in Fig.3 or a horizontal collection system under the lake bottom (Fig.4) may be appropriate.

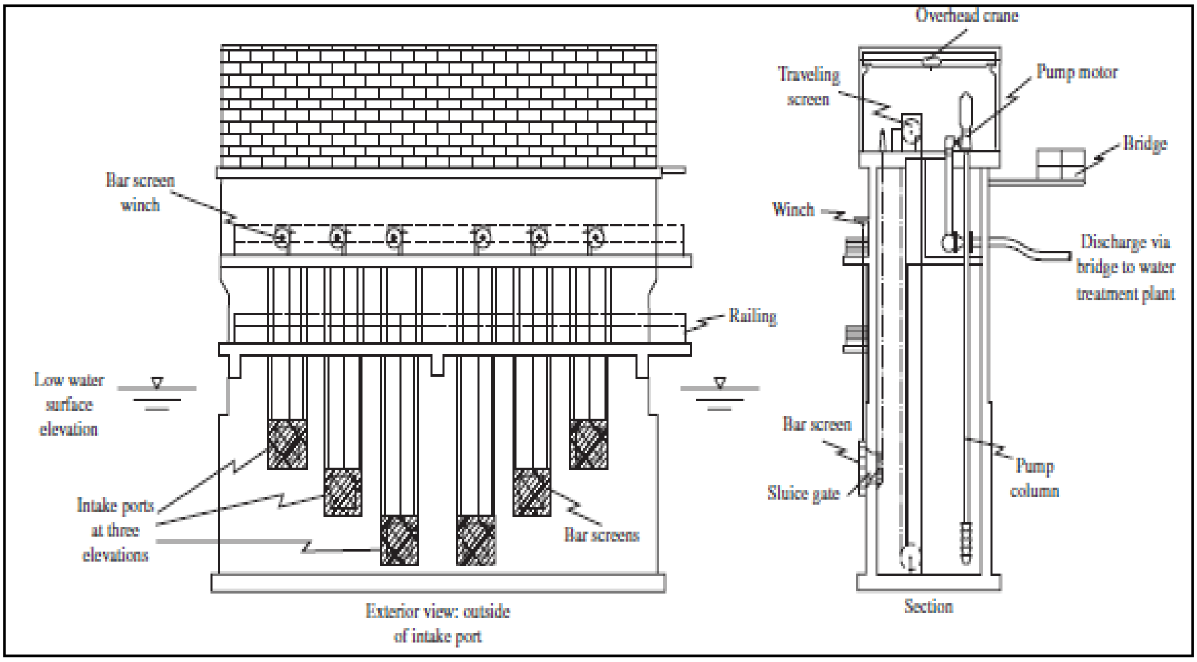


Fig.1 Tower intake.

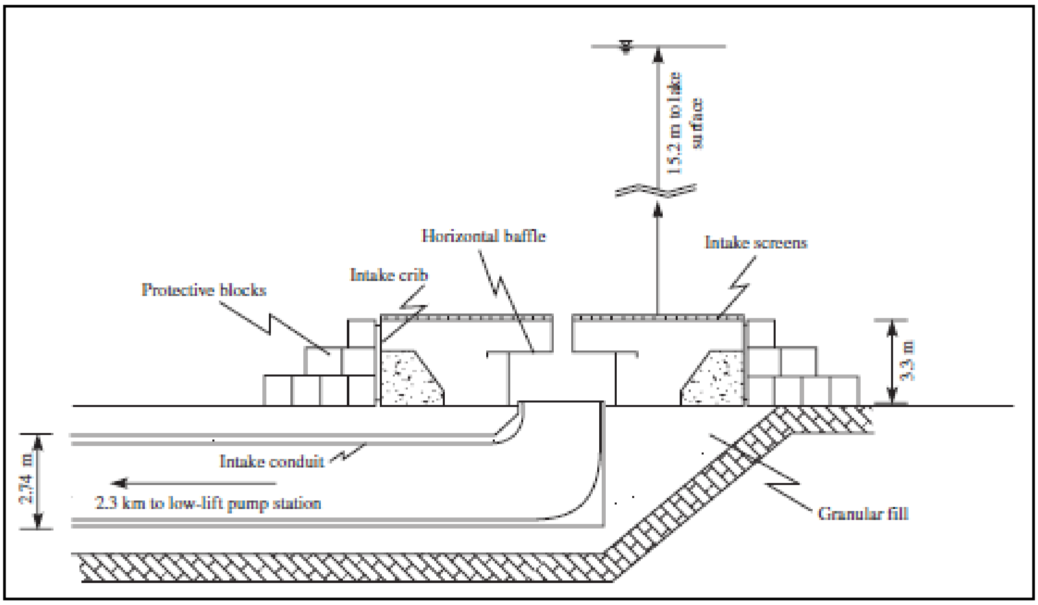


Fig.2 Lake intake crib. Crib is steel octagonal.

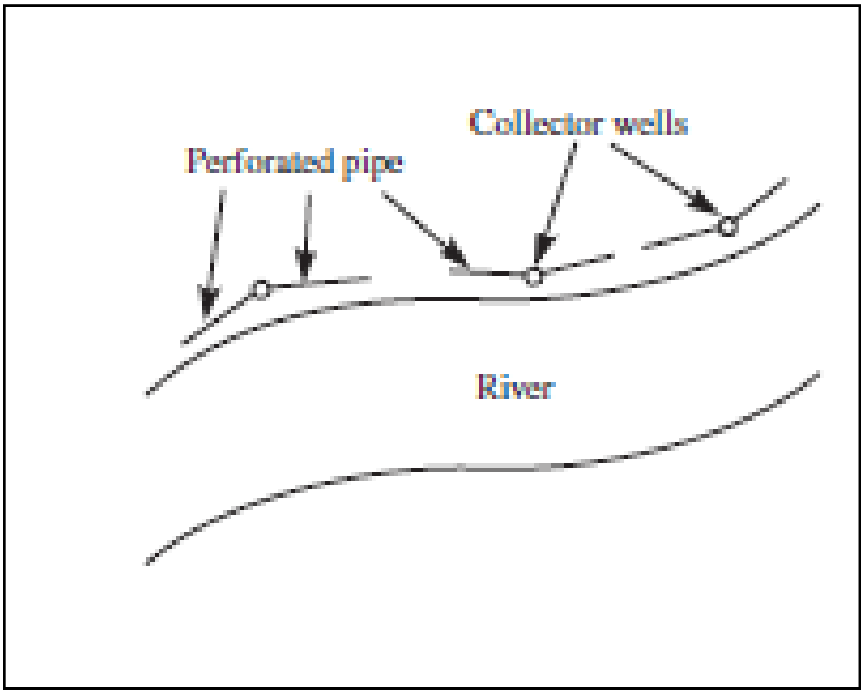


Fig.3 Infiltration gallery with wells.

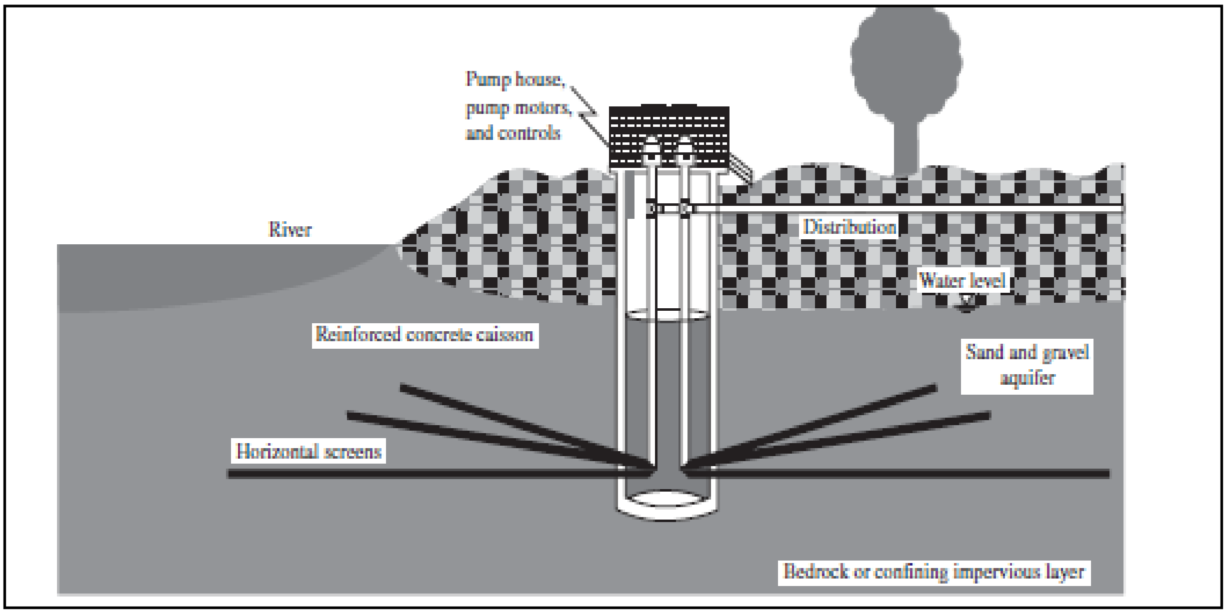


Fig.4 Collector well with horizontal groundwater collection screens.

5-4-2 River intakes

Both exposed and submerged inlet structures have been used in rivers. In large rivers that are controlled by locks and dams, the variation in flow and consequent variation in water surface elevation are of less concern than in unregulated waterways. For most water supplies, Unlike lakes and reservoirs, special consideration must be given to the impact of floods and droughts on river intakes. In the first instance, structural stability, availability of power, and access must be considered in the design. In the second instance, provision must be made for alternative access to water when drought conditions lower the water level below the lowest intake port. While a reservoir or lake will have suspended matter during high wind events, it will seldom have the quantity or quality of the grit produced during flood events on rivers. The river intake structure must be designed to protect the pumps and valves in the transmission system from wear by grit.

5-5 Design Criteria

5-5-1 Layout

Division of the intake system into two or more independent or parallel components is recommended for all but the smallest systems. This enhances reliability, provides flexibility in operation, and simplifies maintenance. The area of the operating deck (also called the operating floor and pump station floor) should be sufficient to allow for the installation and servicing of the pumps, intake gates, and screens. Overhead cranes are an essential feature.

5-5-2 Intake tower

Location: Intake towers should be located as close to the shore as possible, consistent with the variation in water depth. With the exception of very small intakes, the minimum depth should be 3 m.

Intake Ports: Gated ports are provided at various depths to allow for changes in water elevation and changes in water quality due to wind action and stratification. Typical design criteria of intake ports are listed in Table (1).

Table (1) Intake ports design criteria

Criterion	**Typical recommendations**
Number	Multiple: three minimum
Vertical spacing	3 to 5m maximum
Depth of lowest port	0.6 to 2m above bottom depending on muck quality
Depth of top port	Variable: 5 to 9m below surface to avoid wave action
Ice avoidance	At least one port 6 to 9m below the surface
Port flow velocity	Gross area of ports at same elevation sized to limit velocity less than 0.3m/sec. To avoid ice buildup, limit velocity to less than 0.1 m/sec

Gates: Sluice gates may be used on either the interior or exterior of the tower. Historically, gate valves have been preferred because the other valves become fouled with debris.

Coarse Screens: Also known as bar racks, these screens are provided to prevent leaves, sticks, and other large pieces of debris from entering the tower.

Fine Screens. A fine screen is placed downstream of the coarse screen. Its purpose is to collect smaller material that has passed through the coarse screen but is still large enough to damage downstream equipment. Generally, it is placed in the low-lift pump station ahead of the pump intake.

Example 5-1

Design a tower intake to be placed in a reservoir in warm climate. The design conditions are;

- Design flowrate= 40000 m^3/day
- Maximum water level= 20m.
- Minimum water level= 1.7m.
- Reservoir bottom level= -1.3m.

- Ports are placed at 3 levels to consider the fluctuations in water level

Specify the followings:

1- Number and diameter of intake ports.
2- Spacing of ports.
3- Depth of lowest ports.
4- Depth of top ports.

<u>Solution</u>

Q=40000 m^3/day= $\frac{40000}{24\times3600} = 0.463\ m^3/sec$

Q=V.A ; Assume V=0.28 m/sec

$$\therefore A = \frac{Q}{V} = \frac{0.463}{0.28} = 1.65\ m^2$$

Use two ports at the same level and place the ports at three levels, then the area of one port is;

$$A_{one} = \frac{1.65}{2} = 0.825\ m^2 = \frac{\pi D^2}{4}$$

$$\therefore D = \sqrt{\frac{4A}{\pi}} = \sqrt{\frac{4 \times 0.825}{\pi}} = 1.03m$$

∴ Use 6 ports, each of 1.03m diameter and arranged at three levels, so that;

Spacing of ports=4m

Depth of lowest port= (20-(-1.3))-1= 20.3 m

Depth of top port= 21.3 – (1+4×2)= 12.3m\

Example 5-2

An intake tower located in a cold climate reservoir is being designed for a winter design flow rate of 6,000 m^3 /day. The tower will have three ports at three different elevations in each cell. Each port must be able to deliver the design flow rate operating alone. Determine the area of each port opening.

Solution. For a cold climate reservoir, the intake velocity should be limited to less than 0.10 m/sec.

Q=6000 m^3/day= $\frac{6000}{24\times3600} = 0.0694\, m^3/sec$

Q=V.A

Assume V=0.08 m/sec

$$\therefore A = \frac{Q}{V} = \frac{0.0694}{0.08} = 0.868m^2$$

5-5-3 Intake crib

Location: The desired location of the intake crib is in deep water where it will not be buried by sediment, be washed away, be a navigational hazard, or be hampered by problems associated with ice. The minimum suggested depth is 3 m from the surface. In rivers, where the depth exceeds 3 m, the top of the intake should be 1 m above the river bottom. In cases where the water depth is less than 3 m, the crib is buried 0.3 to 1 m.

Structure: octagonal or circular shape is used. The intake is protected by riprap or a concrete slab.

Intake Ports: In warm climates, the intake crib ports are sized to provide a maximum velocity of less than 0.3 m/s. In cold climates, where ice is anticipated, the intake velocity is limited to less than 0.1 m/s.

Screens: Submerged intakes are screened with coarse screens.

Conduit: The conduit may be designed to flow by gravity. It is sized to carry the maximum design flow rate. To minimize the accumulation of sediment the flow velocity should be greater than 1 m/s.

5-5-4 Shore intake

Location: The minimum water depth for a shore intake should be about 2 m. For river intakes, a stable channel is preferred.

Intake Bay: The structure should be divided into two or more independent inlets to provide redundancy. The inlet velocity may be as high as 0.5 m/s in warm climates but should be reduced to 0.3 m/s or less if large amounts of debris are expected. In cold climates, inlet velocities below 0.10 m/s are used to minimize ice buildup.

Screens: Trash racks are used to remove large objects, Fig.5. These are followed by fine screens to protect the pumps. Screenings from the fine screen are collected in a roll-off box and disposed of in a municipal solid waste landfill. The maximum head loss from clogging of the trash racks should be limited to between 0.75 and 1.5 m. As shown in Fig.5, a mechanical cleaning device is used to remove the debris from the trash rack.

Wet Well: The wet well should be divided into cells so that a portion can be taken out of service for inspection and maintenance of the equipment.

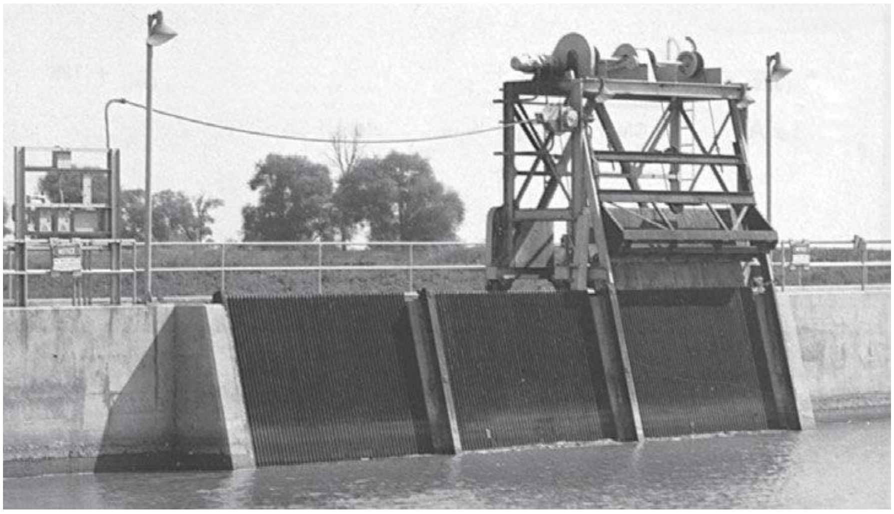


Fig.5 Coarse bar screen, mechanically cleaned

Dimensions: The area of the wet well must be large enough to accommodate the fine screen and pumps. Sufficient space must be provided to service or remove the mechanical equipment. The overhead space above the operating deck must be sufficient to raise the equipment from the wet well to the deck. The depth of the wet well is governed by hydraulic considerations. The high water level is set at the highest elevation of the lake or reservoir or at the 500-year flood level for rivers. The bottom of the wet well must be low enough to allow drawdown of the wet well while pumping at the design flow rate when the source water elevation is at its minimum level. In addition, there must be enough depth to maintain the pump manufacturer’s required submergence to prevent cavitation of the pump.

Ch.6 Water Treatment

6-1 Purpose of Water Treatment

Water is a colorless, tasteless and odorless transparent liquid. However, in natural water resources, water may contain many impurities, see Table 1. Table 1 shows that water impurities are divided into suspended and dissolved impurities. It also shows the effects of these impurities. The purpose of water treatment is to remove most of water impurities and, thus, produce water that is chemically and microbiologically safe for human consumption and free from unpleasant tastes and odors.

Table 1 Water impurities and their effect

Type of impurities	**Constituent**			**Effect**
Suspended impurities	Bacteria			Some cause disease
	Algae			Odor, color and turbidity
	Protozoans			Some cause disease
	Viruses			Some cause disease
	Silt			Turbidity
	Clay			Turbidity
	Colloids			Color and turbidity
Dissolved impurities	Salts	Cations	Calcium	Hardness
			Magnesium	Hardness
			Iron	Color and hardness
			Manganese	Hardness
			Others	Dissolved solids
		Anions	Bicarbonate	Alkalinity
			Carbonate	Alkalinity
			Sulfate	Laxative
			Chloride	Taste
			Fluoride	Tooth mottling
	Organics			Color, taste, odor, toxicity
	Gases	Oxygen		Corrosive and oxidizing agent
		Carbon dioxide		Acid
		Hydrogen sulfide		Acid
		Nitrogen		None
		Ammonia		Caustic

6-2 Conventional Water Treatment

Water treatment is said to be conventional if the water source is river which is the most preferable water source. The flowsheet of a conventional treatment plant is shown in Fig.1.

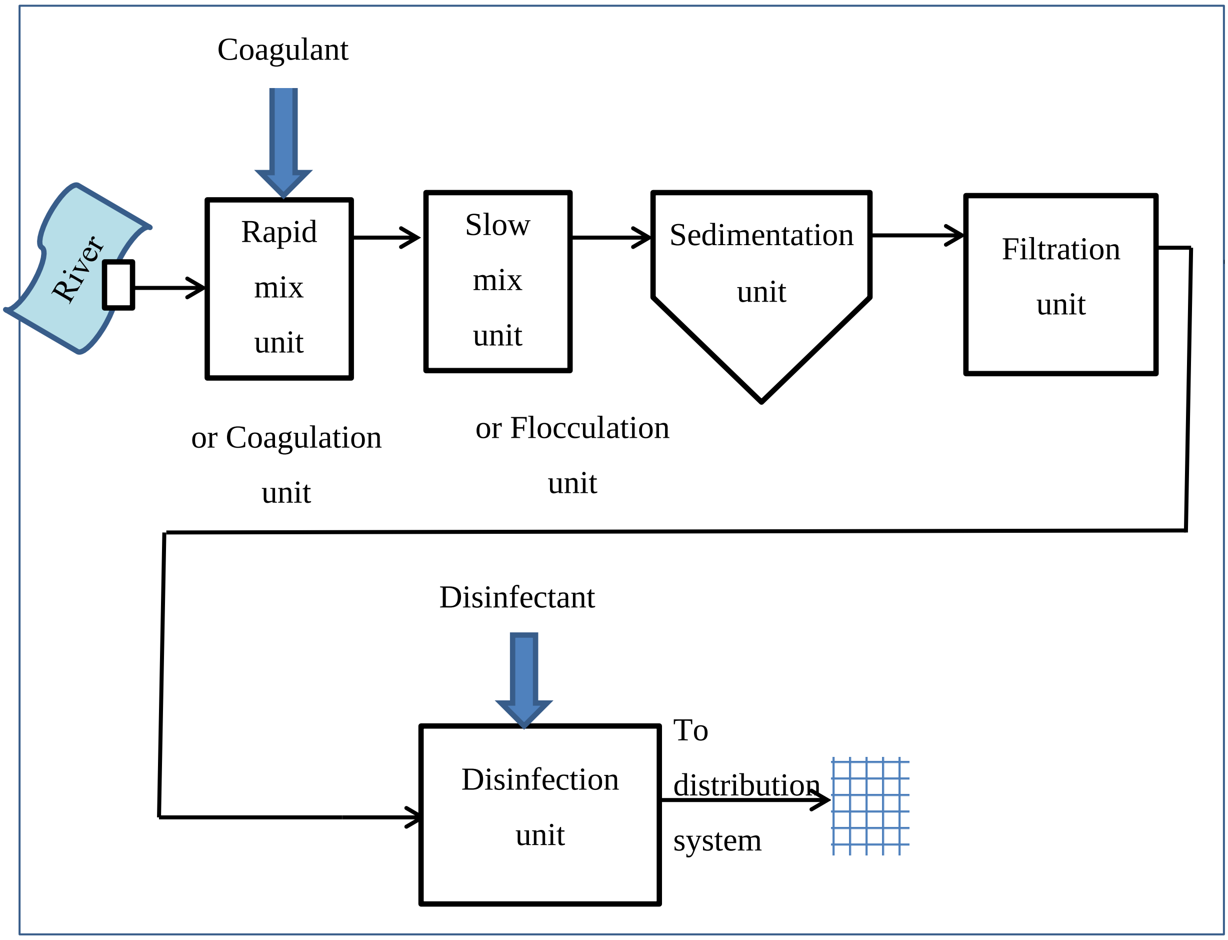


Fig.1 Flowsheet of conventional water treatment plant

6-2-1 Rapid (or Flash) Mix Unit

The aim of rapid mix unit is to dissolve chemicals (coagulants) into water. Rapid mixing can be achieved using many devices such as;

1. Mechanical mixers, Fig.2.
2. Recirculating pumps, Fig.3.
3. Diffused air, Fig.4.

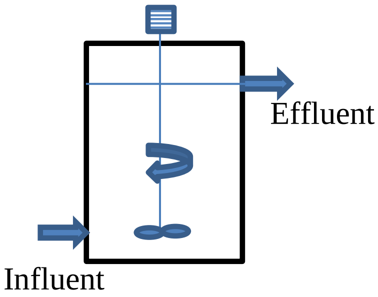


Fig.2 Mechanical mixer

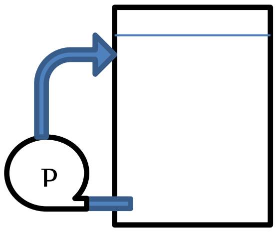


Fig.3 Recirculating pump

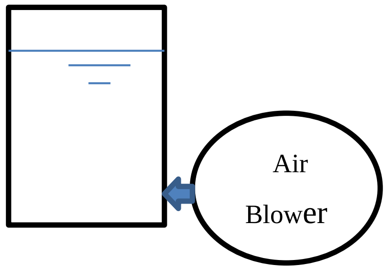


Fig.4 Diffused air

6-2-1-1 Design of Rapid Mix Unit

Rapid mix unit is usually composed of circular tanks. The unit design includes determination of tanks number, dimensions and power of mixing devices.

<u>Design criteria</u>

1. Detention time (t): 10-30 sec
2. Velocity gradient (G): 600-1000 sec^{-1}

Other design considerations include:

- Water depth (d) to diameter (D) ratio= 0.5 to 1.1
- Four baffles are provided at the tank periphery, each baffle extends a distance in to the tank equals 0.1D, Fig.5.
- If two mixers are used in one tank, the total power=1.9 the power of one mixer.

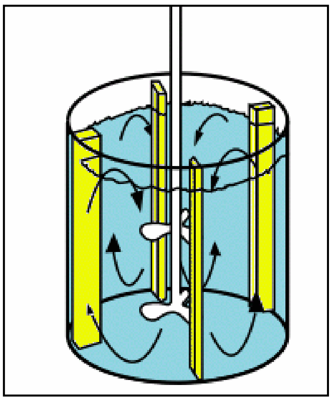


Fig.5 Baffles of rapid mix tank

Detention time is the time required for a small amount of water to pass through a tank at a given flow rate. Mathematically, detention time is given by the following formula:

$$t = \frac{V}{Q} \qquad \ldots (1)$$

Where:

t = detention time

V = water volume

Q = water flowrate

Velocity gradient (G) is a measurement of the intensity of mixing in the tank. The velocity gradient determines how much the water is agitated in the tank, and also determines how much energy is used to operate the flash mixer. It is obtained as;

$$G = \sqrt{\frac{P}{\mu V}} \quad \ldots (2)$$

Where;

P= water power, Watt.

μ= absolute water viscosity, N.sec/m^2.

V= water volume in one tank, m^3.

Design Steps

1. Determine the total water volume in rapid mix unit using Eq.1.
2. Assume the number of tanks =1.
3. Calculate water power using Eq.2.
4. Calculate motor power ($P_{motor} = \frac{P_{water}}{motor\ efficiency}$)
5. If the calculated motor power is available, then use one tank. While, if the calculated motor power is not available, then increase the number of tanks in accordance to the available motor power.
6. Assume water depth to tank diameter ratio.
7. Calculate the tank diameter.
8. Calculate the water depth.
9. Calculate the total depth of tank (h) by adding a free board of 10% water depth. The free board must not be less than 30cm.

Example 6.1

Design rapid mix unit for a water treatment plant has a design capacity of 120,000 m^3/day. The powers of the available mixers are given in the following table. Assume the efficiency of mixer motor is 90%.

Mixer model	Power (kW)
JTQ 25	0.18
JTQ 50	0.37
JTQ 75	0.56
JTQ 100	0.75
JTQ 150	1.12
JTQ 200	1.50
JTQ 300	2.24
JTQ 500	3.74
JTQ 750	5.59
JTQ 1000	7.46
JTQ 1500	11.19

Solution:

Assume t= 15 sec (Detention time (t) : 10-30 sec)

$$Q = \frac{V}{t} \rightarrow V = Q \times t \rightarrow V = \frac{120000}{(24 \times 3600)} \times 15 = 20.83 m^3$$

Assume number of tanks=1

∴ volume of water in one tank= 20.83m^3

Assume G= 800 sec^{-1} (Velocity gradient (G): 600-1000 sec^{-1})

$$G = \sqrt{\frac{P}{\mu V}}$$

$800 = \sqrt{\frac{P}{1.012 \times 10^{-3} \times 20.83}}$ →P=13491Watt (water power)

Mixer power $= \frac{water\ power}{mixer\ efficiency} = \frac{13491}{0.9} = 14990\ Watt = 14.99\ kW$

Check whether a mixer with power of 14.99 kW is available

The maximum available mixer power is 11.19kW, then, increase the number of tanks.

Assume the number of tanks=2

$$V_{one\ tank} = \frac{20.83}{2} = 10.415 m^3$$

$800 = \sqrt{\frac{P}{1.012 \times 10^{-3} \times 10.415}}$ →P=6745.6 Watt (water power)

Mixer power $= \frac{6745.6}{0.9} = 7495\ Watt = 7.5\ kW$

Select mixer model JTQ 1000 which has a power of 7.46 kW. This power is slightly less the required power. Check G;

G= $= \sqrt{\frac{7.46 \times 1000 \times 0.9}{1.012 \times 10^{-3} \times 10.415}}$= 798.12 sec^{-1} (600-800 sec^{-1})…..O.K.

Determine tank dimensions;

Assume water depth to diameter ratio (d/D)=1→D=d

$V_{one} = \frac{\pi D^2}{4} \times d = \frac{\pi D^3}{4}$=10.415

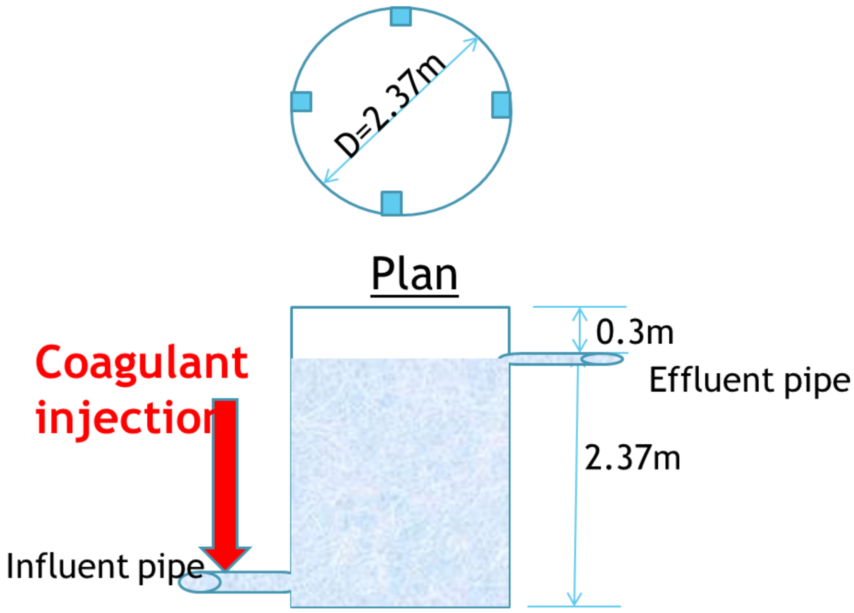
D=2.37m →d=2.37m

$$Free\ board = 0.1 \times d = 0.1 \times 2.37 = 0.237m\ < 0.3m$$

Use free board of 0.3m (30cm)

∴ h=2.37+0.3=2.67m (total tank depth)

∴ Use two circular tanks, each of 2.37m diameter and 2.67m depth and provided with a mixer of 7.46 Kw power.



Vertical profile

Plan view and vertical profile of rapid mix tank

6-2-1-2 Coagulation

Coagulation is a method to alter the colloids so that they will be able to approach and adhere to each other to form larger floc particles. Technically, coagulation applies to the removal of colloidal particles.

The particles in the colloid range are too small to settle in a reasonable time period and too small to be trapped in the pores of a filter. Most colloids are stable because they possess a negative charge that repels other colloids particles before they collide with one another.

Since colloids are stable because of their surface charge, in order to destabilize the particles, we must neutralize this charge. Such neutralization can take place by addition of an ion of opposite charge to the colloid.

<u>Coagulants</u>

During coagulation a positive ion is added to water to reduce the surface charge to the point where the colloids are not repelled from each other. A coagulant is the substance (chemical) that is added to the water to accomplish coagulation. There are three key properties of a coagulant:

<u>1. Trivalent cation:</u> As indicated above, the colloids most commonly found in natural waters are negatively charged, hence a cation is required to neutralize the charge. A trivalent cation is the most efficient cation.

<u>2. Nontoxic:</u> This requirement is obvious for the production of safe water.

<u>3. Insoluble in the neutral pH range:</u> The coagulant that is added must precipitate out of solution so that high concentrations of the ion are not left in the water. Such precipitation greatly assists the colloid removal process.

<u>Types of Coagulants</u>

Coagulants are chemicals used to accomplish coagulation. They include the following types:

1- Aluminum Sulfate [$Al_2(SO_4)_3.18H_2O$]

It is a very commonly used coagulant and can be bought as powder, lumps, or liquid. The main characteristics of alum are;

- Its reaction when it is added to water is with the natural or added alkalinity.
- It is readily soluble.
- Its pH range is 5.5-8.
- Alum solution is corrosive and needs to be stored in tanks with a corrosive- resistance lining.

Alum Reactions

In all alum reactions, aluminum hydroxide $Al(OH)_3$ (floc) is formed according to the alkalinity present:

For alum reaction with natural alkalinity;

$$Al_2(SO_4)_3 + 3Ca(HCO_3)_2 \rightleftharpoons \underline{2Al(OH)_3} + 3CaSO_4 + 6CO_2$$

When lime is added;

$$Al_2(SO_4)_3 + 3Ca(OH)_2 \rightleftharpoons \underline{2Al(OH)_3} + 3CaSO_4 + 6CO_2$$

When soda ash is added;

$$Al_2(SO_4)_3 + 3Na_2CO_3 \rightleftharpoons \underline{2Al(OH)_3} + 3Na_2SO_4 + 3CO_2$$

In the above reaction equations, the underline products are insoluble (precipitates) compounds.

Alum Dose

Generally, the appropriate alum dose is determined by jar test, Fig.6. This test is performed by two steps.

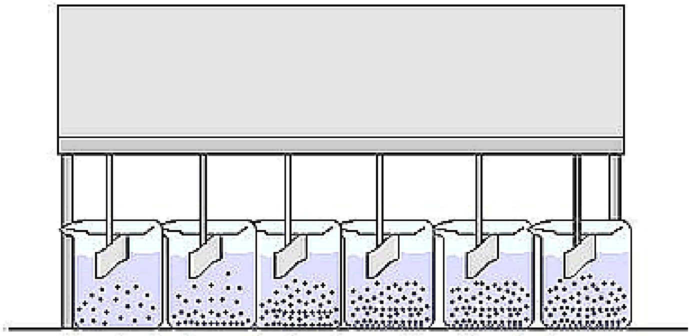
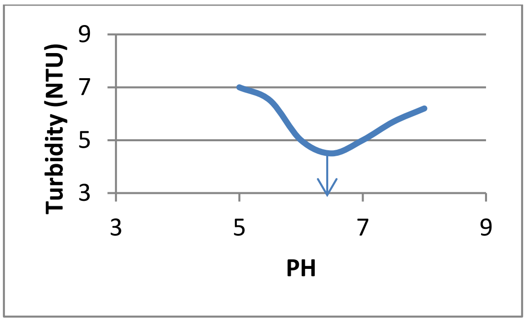


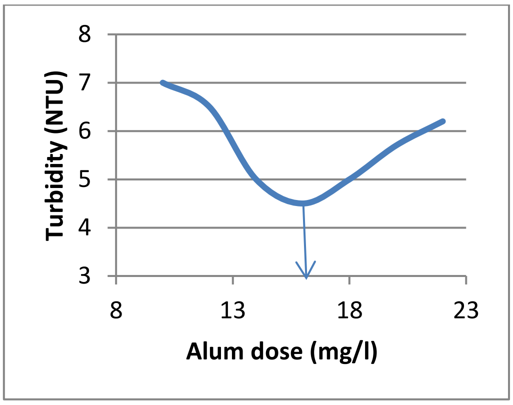
Fig.6 Jar test instrument

1- Constant alum dose with variable pH samples. This step is performed to find the optimum pH value. Where the optimum pH value is that gives minimum turbidity value.



NTU= Nephelometric Turbidity Unit

2- Constant pH (equals the optimum value obtained in step-1) with variable alum doses. This step is performed to find the optimum alum dose.

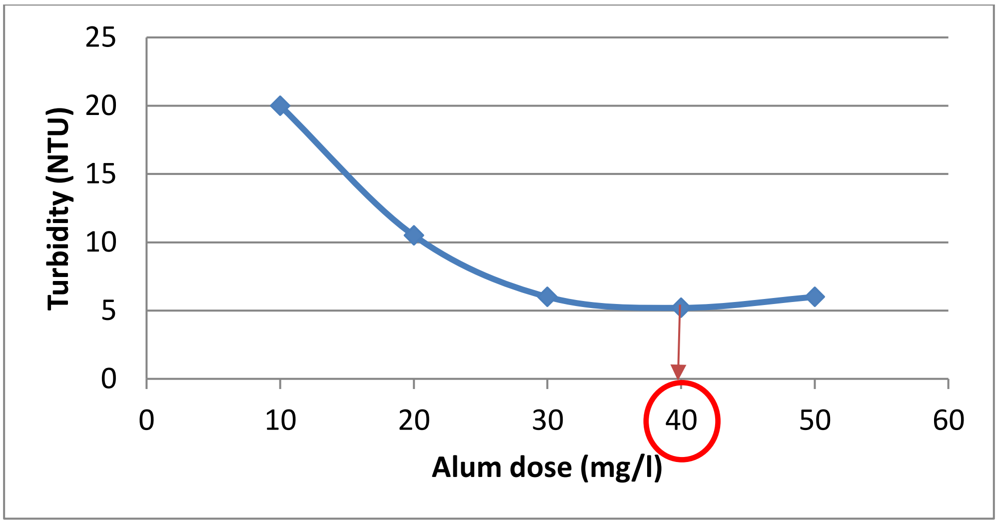


Example 6.2

Determine the optimum alum dosage using the results of Jar test given in the table below.

Sample No.	1	2	3	4	5
Alum dose (mg/l)	10	20	30	40	50
Turbidity (NTU)	20	10.5	6	5.2	6

Solution:



Optimum alum dose=40mg/l

Alkalinity Requirements

Theoretically;

- 1mg/l of alum will react with 0.45 mg/l natural alkalinity.

- 1mg/l of alum will react with 0.35 mg/l hydrated lime [$Ca(OH)_2$].
- 1mg/l of alum will react with 0.28 mg/l quick lime [CaO].
- 1mg/l of alum will react with 0.48 mg/l soda ash [Na_2CO_3].

2- Sodium Aluminate

It is a compound of sodium oxide and aluminum oxide. It is a white powder almost invariably used in conjunction with alum. The two are never mixed before dosing. Sodium aluminate is always being put about 30sec before alum. The used dose is 5-10% of alum dose.

3- Iron Salts

Iron salts can be used coagulants and when available they are normally cheaper, produce heavier flocs and operate over a wider pH range than alum. However, they normally require to be used with lime. When iron salts are used, the following problems might be listed:

a. The storage containers must be lined with corrosion resistive material.
b. Iron salts tend to cake in humid locations.
c. Iron salts are dirty to handle, causing staining.
d. Sludge is more difficult to dispose.
e. Lime must generally be added.

Iron salts include;

1- Ferrous sulfate [$FeSO_4.7H_2O$]
2- Ferric chloride [$FeCl_3$]
3- Ferric sulfate [$Fe_2(SO_4)_3$]

Ferric sulfate is an expensive product and difficult to dissolve but it has many advantages such as;

- Decolorization of low pH waters.

- Removal of manganese at high pH.
- Clarification of high hardness water.
- It can operate at very low temperature.

Example 6.3

If the optimum alum dose is 30mg/l, estimate the monthly requirement of alum for a water treatment plant has a design capacity of 120,000 m^3/day. If the natural alkalinity of the water source is 7mg/l, do we need to add alkalinity?. Select the alkalinity type if it is needed and find the monthly requirement.

Solution:

$$Daily\ requirement\ of\ alum = 120000 \times \frac{30}{10^3} = 3600\frac{kg}{day}$$

$$monthly\ requirement\ of\ alum\left(\frac{ton}{month}\right) = \frac{3600kg}{10^3\left(\frac{kg}{ton}\right)} \times 30\left(\frac{day}{month}\right)$$

$$= 108\frac{ton}{month}$$

Check the adequacy of natural alkalinity;

$Required\ natural\ alkalinity = 30 \times 0.45 = 13.5mg/l > 7\ mg/l$, then, we need to add alkalinity

At first we must find the reacted alum dose;

1mg/l of alum will react with 0.45 mg/l natural alkalinity.

X mg/l of alum will react with 7 mg/l natural alkalinity.

$$X = \frac{1 \times 7}{0.45} = 15.56mg/l$$

$$\therefore non\ reacted\ alum\ dose = 30 - 15.56 = 14.44\frac{mg}{l}$$

If soda ash is added, then, required soda ash dose=0.48×14.44=6.93 mg/l

$$\therefore monthly\ requirement\ of\ soda\ ash = 120000 \times \frac{6.93}{10^3} \times \frac{30}{10^3}$$

$$= 24.95\ ton/month$$

Solution Feed System

Coagulants are usually added to water as a solution. Solution feed system is composed of;

1- Solution tank which should hold 24hours supply and be duplicated.

2- Mixer (to avoid the risk of coagulant settlement).

3- Metering pump.

Strength of solution

5% to 8% solution strength is used

5% means 5kg alum to 95 kg water.

Example 6.4

Design alum feed system for a water treatment plant has a design capacity of 120,000 m^3/day.

Solution:

From example 6.4, the daily requirement of alum=3600kg/day

Assume solution strength=6%

6kg alum ……………..94kg water

3600 kg alum………….. X kg water

x= $\frac{3600 \times 94}{6} = 56400\ kg\ water$

$$\therefore volume\ of\ alum\ solution\ tank = \frac{56400\ kg}{1000 kg/m^3} = 56.4 m^3$$

Use two tanks, each of 28.4m^3 capacity

6-2-2 Flocculation Unit

Flocculation is aggregation by chemical bridging between particles. In this manner, very small suspended solid particles (colloids) agglomerate into larger heavier particles or flocs which can be settled in sedimentation unit. Flocculators are classified into two types;

1. Mechanical (or paddle) flocculators in which slow mixing is mainly achieved using revolving paddles.
2. Hydraulic (or baffled) flocculators in which slow mixing is achieved using baffles.

In paddle flocculators, Paddle wheels can be mounted on vertical or horizontal shafts, Figs 7 and 8. The paddle shafts can be located transverse or parallel with the flow.

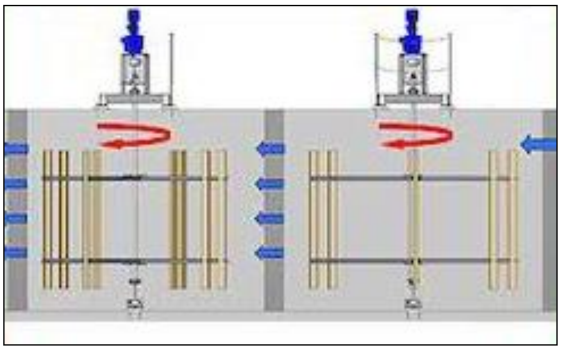


Fig.7 Paddle flocculator provided with vertical shaft paddles

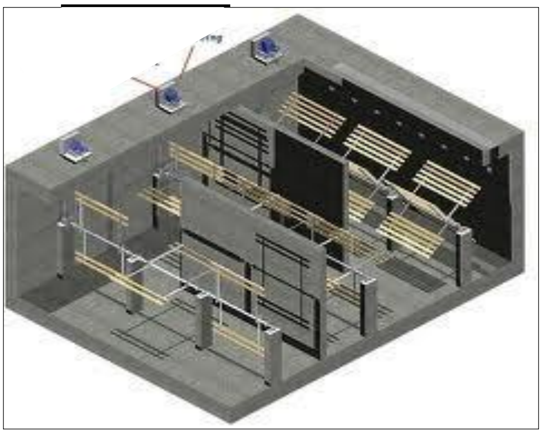


Fig.8 Paddle flocculator provided with horizontal shaft paddles

Baffled flocculators are of two types; horizontal flow (or round-the-end) baffled flocculator (Fig.9) and vertical flow (or over- and- under) baffled flocculator (Fig.10).

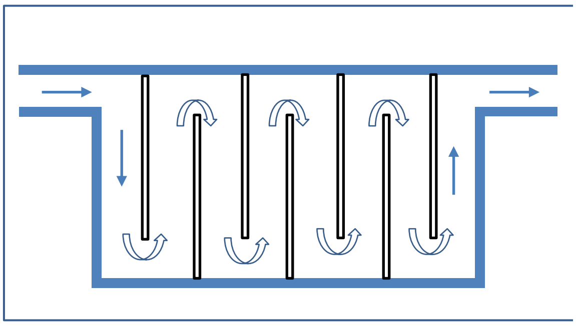


Fig.9 Plan view of horizontal flow baffled flocculator.

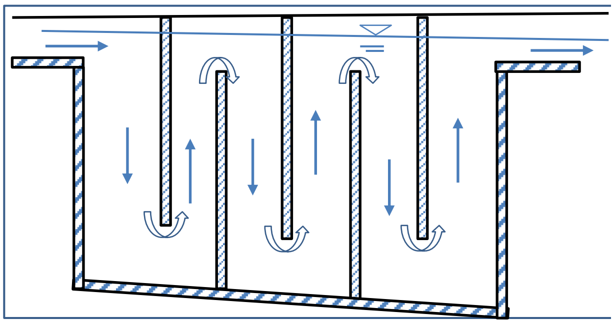


Fig.10 Vertical profile of vertical flow baffled flocculator.

Flocculation is directly proportional to the velocity gradient established in the water by a stirring action (G). The mean velocity gradient is given by:

$$G = \sqrt{\frac{P}{\mu V}} \qquad \ldots(1)$$

where:

G= velocity gradient, sec^{-1}

P= input power, Watt

μ= absolute viscosity, $N.sec/m^2$

V= water volume in one tank, m^3

Considering that the rate of floc formation is directly proportional to velocity gradient, the time of floc formation should decrease with increasing values of G. There is however a maximum size of floc particle associated with each velocity gradient.

Input power for paddle flocculator

For paddle flocculator, the power input is directly related to the drag force on paddles. The input power is obtained as;

$$P = F \times v \qquad \dots(2)$$

Where;

F= drag force, N

v = relative paddle velocity, m/sec; $v = 2\pi krn$

K= 0.75; r= paddle wheel radius (m); *n*=rotational speed of paddle (rps)

The drag force is obtained as;

$$F = \frac{C_D\ A\ \rho\ v^2}{2} \qquad \dots(3)$$

Substituting Eq.3 into Eq.2 gives;

$$P = \frac{C_D\ A\ \rho\ v^3}{2} \qquad \dots(4)$$

Substituting Eq.4 into Eq.1 gives;

$$G = \sqrt{\frac{C_D\ A\ \rho\ v^3}{2\mu V}} \qquad \dots(5)$$

where; A= total area of paddles, m^2

$$A = N_P \times N_b \times A_b \qquad \dots\ (6)$$

N_p= number of paddles

N_b= number of blades in one paddle

A_b= blade area, m^2

$$A_b = L_b \times W_b$$

L_b = blade length, m
W_b = blade width, m
ρ= water density, kg/m^3

C_D= drag coefficient

The value of C_D is dependent on L_b / W_b;

C_D=1.2 for L_b / W_b=5
C_D=1.5 for L_b / W_b=20
C_D=1.9 for L_b / W_b>>20

Input power for baffled flocculator

For baffled flocculator, the input power is due to head loss of baffles and it is obtained as;

$$P = \gamma Q h_L \qquad \ldots(7)$$

Where;

P =input power, kW

γ= specific weight of water, kN/m^3

Q= water flowrate, m^3/sec

h_L=total head loss due to baffles, m

The head loss due to baffles is obtained as;

$$h_L = n_b k \frac{V^2}{2g} \quad \ldots(8)$$

Where; n_b= number of baffles

K= head loss coefficient= 1 to 3

V= flow velocity through the baffle slot, m/sec

Substituting Eq.7 into Eq.1 gives;

$$G = \sqrt{\frac{1000 \times \gamma Q h_L}{\mu V}} \quad \ldots(9)$$

6-2-2-1 Design of Flocculation Unit

Design Criteria

1. Detention time (t)=20 to 30min
2. Velocity gradient (G)= 30 to 60 sec^{-1}
3. G.t=10^4 to 10^5
4. Water depth= 3 to 4.5m

Design of paddle flocculator;

For paddle flocculator, there are other design requirements;

a- Paddles area should not exceed 15 to 20% of the cross sectional area of the flow.

b- Blade width (W_b)=10 to 15cm.

c- The tank is divided into two or more compartments using baffles provided with orifices uniformly distributed over the vertical surface of baffle.

d- Baffles are designed to provide orifice ratio of 3% to 6% of a velocity of 0.27m/sec.

e- The top of baffles is slightly submerged (1 to 2 cm) and the bottom should have a space of 2 to 3cm above the tank floor to allow for tank drainage.

f- Water depth is 1m greater than wheel diameter.

g- Clearness between wheel and walls is 0.3 to 0.7m.

h- Spacing between wheels on adjacent shafts= 1m.

i- Flocculation tanks provided with vertical shaft paddles are divided into square compartments with maximum dimensions of 6m.

j- Flocculation tanks provided with horizontal shaft paddles are divided into rectangular compartments of 6 to 30m long and 3 to 5m width.

Design steps

1. Assume detention time.
2. Determine total water volume in flocculation unit (V_T=Q×t).
3. Assume number of tanks.
4. Find water volume in one tank.
5. Assume water depth (h).
6. Find total tank depth by adding a free board of 0.1D.
7. Find surface area of tank (A_T=V/D).
8. Divide the tank into compartments.
9. Assume paddles installation method (vertical or horizontal)
10. Find the length (L) and width (W) of each compartment.
11. Assume A/(W.D) ratio (see point No.a).
12. Find paddles area (A).
13. Assume number of paddles and number of blades in one paddle
14. Find blade length and width.
15. If blade width is greater than 15cm, increase the number of paddles or blades.
16. Assume G so that; G.t within the range 10^4 to 10^5.
17. Find relative paddle velocity using Eq. 5
18. Find rotational speed of paddles.

Design of baffled flocculator;

For baffled flocculator, other design requirements include;

Flow velocity through the slots= 0.1 to 0.3 m/sec.

Design steps

1. Assume detention time.
2. Determine total water volume in flocculation unit (V_T=Q×t).
3. Assume number of tanks.

4. Find water volume in one tank.
5. Assume water depth (D).

19.Find total tank depth by adding a free board of 0.1D.

6. Find surface area of one tank.
7. Assume tank length to width (L/W) ratio. Hint; L/W≥2:1
8. Find tank dimensions (L and W).
9. Assume flow velocity through slots.
10. Find slot dimensions.
11. Assume G so that; G.t within the range 10^4 to 10^5.
12. Find h_L using Eq.9.
13. Find the number of baffles using Eq.8.

Example 6.5

Design flocculation unit using baffled flocculators for a water treatment plant has a design capacity of 120,000 m^3/day using;

(a) Paddle flocculators.

(b) Baffled floccculors.

Solution

Design of paddle flocculators

Assume t=30min.

$$Q = \frac{V}{t} \quad \rightarrow V_T = Q \times t = \frac{120000}{24 \times 60} \times 30 = 2500m^3$$

Let the number of tanks=2

$$V_{one} = \frac{2500}{2} = 1250m^3$$

Assume water depth (D) = 4m

Total tank depth=4+0.1×4=4.4m

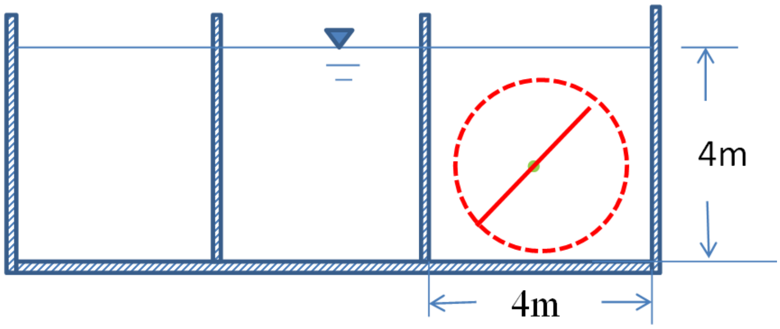
Find surface area of tank;

$$A_T = \frac{V_{one}}{D} = \frac{1250}{4} = 312.5m^2$$

Select horizontal paddles and divide the tank into 3 rectangular compartments.

$$A_{one} = \frac{312.5}{3} = 104.2m^2$$

Let compartment width=water depth=4m



$Compartment\ length = \frac{A_{one}}{Compartment\ width} = \frac{104.2}{4}$=26.05m (<30m..O.K.)

Let $\frac{A}{Cross\ sectional\ area\ of\ flow}$=0.17

Cross sectional area of flow=26.05×4=104.2m²

$$\therefore \frac{A}{104.2} = 0.17 \quad \rightarrow A = 17.71m^2$$

$A = N_P \times N_b \times A_b$

Let N_P=6 and N_b=6 → $17.71 = 6 \times 6 \times A_b \rightarrow A_b = 0.492m^2$

$A_b = L_b \times W_b$

Let W_b= 15cm= 0.15m→ $L_b = \frac{0.492}{0.15}$= 3.28 m

Check wheels spacing (s);

$$S = \frac{26.04 - 6 \times 3.28}{6} = 1.06m \cong 1m \;\; \ldots O.K$$

Use 6 paddles in each compartment, each paddle has six blades and each blade is of 3.28m length and 0.15 m width.

Wheel diameter=water depth-1=3m

Assume uniform G distribution (G values of the three compartments are equal)

Let G=40 sec^{-1}

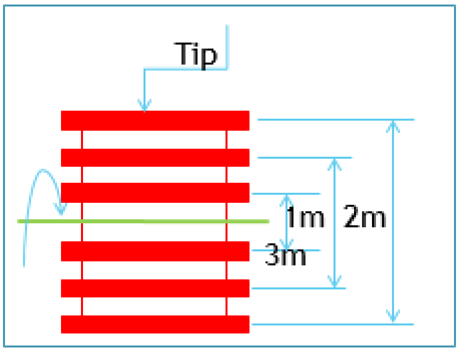
Check G.t;

G.t= 40×60×30=72000 **(10,000-100,000)…O.K.**

$$G = \sqrt{\frac{C_D\ A\ \rho\ v^3}{2\mu V}}$$

L_b / W_b=3.28/0.15=21.9 >20→ $C_D = 1.9$

$$G = \sqrt{\frac{C_D\ \rho\ (\frac{A}{3} v_1^3 \;\; + \frac{A}{3} v_2^3 \;\; + \frac{A}{3} v_3^3 \;\;)}{2\mu V}}$$



Paddle with 6 blades

$$G = \sqrt{\frac{C_D\ \rho \frac{A}{3}(v_1^3 \;\; + v_2^3 \;\; + v_2^3 \;\;)}{2\mu V}} = \sqrt{\frac{C_D\ \rho \frac{A}{3}((\pi k n D_1)^3 + (\pi k n D_2)^3 + (\pi k n D_3)^3)}{2\mu V}}$$

$$40 = \sqrt{\frac{1.9 \times 1000 \times \frac{17.71}{3} \times \pi^3 \times 0.75^3 \times n^3 (1^3 + 2^3 + 3^3)}{2 \times 1.012 \times 10^{-3} \times \frac{1250}{3}}}$$

n=0.063 rps =3.81rpm

Hint: Peripheral speed of outside blade (v)

v = wheel periphery x n (rpm)

<u>Design of Baffled flocculators</u>

Assume t=30min.

$$Q = \frac{V}{t} \quad \rightarrow V_T = Q \times t = \frac{120000}{24 \times 60} \times 30 = 2500 m^3$$

Let the number of tanks=2

$$V_{one} = \frac{2500}{2} = 1250 m^3$$

Assume water depth (D) = 4m

Total tank depth=4+0.1×4=4.4m

Find surface area of tank;

$$A_T = \frac{V_{one}}{D} = \frac{1250}{4} = 312.5 m^2$$

Assume tank length to width ratio (L/W)=3, then;

L=3W

$$A_T = L \times W \rightarrow 312.5 = 3W \times W \rightarrow W = 10.21 m$$

$$\therefore L = 3 \times 10.21 = 30.63 m$$

Assume flow velocity through the slot =0.2m/sec

Use vertical baffles;

$Q = V \times A$

Assume flow velocity through the slot =0.2m/sec

Use vertical baffles;

$$Q = V \times A$$

$$\therefore A = \frac{Q}{V} = \frac{120000/(2\times24\times3600)}{0.2} = 3.47m^2 \ (slot\ area)$$

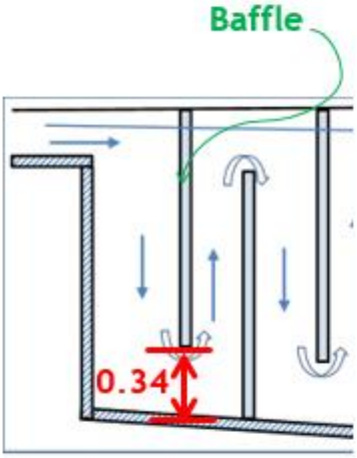
For vertical baffles;

Slot area=slot length × slot width

$$3.47 = 10.21 \times slot\ width$$

$$\therefore slot\ width = \frac{3.47}{10.21} = 0.34m$$

Note:
For horizontal baffles;
Slot length= water depth



Let G=40 sec^{-1}

Check G.t;

G.t= 40×60×30=72000 (10,000-100,000)…O.K.

$$G = \sqrt{\frac{1000\gamma Q h_l}{\mu V}}$$

$$40 = \sqrt{\frac{1000 \times 9.81 \times \frac{120000/2}{24 \times 3600} \times h_l}{1.012 \times 10^{-3} \times 1250}}$$

$$h_l = 0.297m$$

$$h_L = n_b k \frac{V^2}{2g}$$

Assume k=2

$0.297 = n_b \times 2 \times \frac{0.2^2}{2\times 9.81} \rightarrow \; n_b = 72.84$ →Use 73 baffles

6-2-3 Sedimentation Unit

The aim of sedimentation unit is to remove suspended solid particles from a suspension by settling under gravity.

6-2-3-1 Classification of Settling Behavior

Four classes of settling behavior can be distinguished on the basis of ;

- Characteristics of the Particles (discrete or flocculant particals).
- Concentration of Particles in suspension (dilute or concentrated suspensions).

Discrete particles

Particles which don't change in size, shape, or mass during settling.

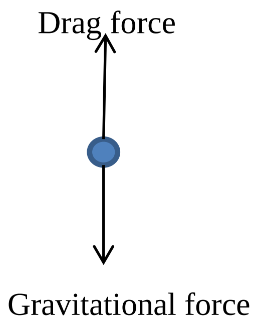
Flocculant particles

Particles which agglomerate during settling and thus they don't have constant characteristics.

Class	Characteristics	Example
Class-I	Settling of discrete particles in dilute suspensions.	Settling of sand particles in pre-sedimentation tanks.
Class-II	Settling of flocculant particles in dilute suspension.	Settling of suspended particles in sedimentation tanks preceded by coagulation-flocculation processes.
Class-III	Hindered or zone settling. Settling of intermediate concentration of flocculant particles.	Settling of flocculant particles in sedimentation tanks of sewage treatment.
Class-IV	Compression settling. Settling of particles of high concentration.	Occurs in sludge thickening units.

6-2-3-2 Theory of Sedimentation

When a particle settles in a fluid, it accelerates until the drag force due to its motion is equal to the submerged weight of the particles. At this point the particle will reach its terminal settling velocity, V_S .



$$Gravitational\ force = (\rho_s - \rho)gV$$

where;

ρ_s= density of solid particle.

ρ= fluid density.

V= particle volume.

$$Drag\ force = C_D\ A_C \rho \frac{V_S^2}{2}$$

where;

C_D= Newton's drag coefficient.

A_c= cross sectional area of particle.

V_S= settling velocity of particle.

Equating gravitational and drag forces;

$$(\rho_s - \rho)gV = C_D\ A_C \rho \frac{V_S^2}{2}$$

Then;

$$V_S = \sqrt{\frac{2gV(\rho_s - \rho)}{C_D\ A_C \rho}}$$

For spherical particles;

$$V = \frac{\pi d^3}{6} \qquad and \qquad A_C = \frac{\pi d^2}{4}$$

where; d is particle diameter.

Then;

$$V_S = \sqrt{\frac{4gd(\rho_s - \rho)}{3C_D\,\rho}}$$

Put $\frac{\rho_s}{\rho} = S$; S= sp. gr. of solid particles

$$V_S = \sqrt{\frac{4gd(S-1)}{3C_D}}$$

Newton's law for settling velocity

The value of C_D is dependent on Reynolds number (Re=$\frac{\rho V_S d}{\mu}$)

For Re <0.5	$C_D = \frac{24}{Re}$
For 0.5 <Re<10^4	$C_D = \frac{24}{Re} + \frac{3}{\sqrt{Re}} + 0.34$
For 10^3<Re<10^5	$C_D \cong 0.4$

When $C_D = \frac{24}{Re}$;

$$V_S = \frac{gd^2(S-1)}{18\nu}$$

Stoke's law for settling velocity

When $C_D \cong 0.4$;

$$V_S = \sqrt{3.3gd(S-1)}$$

Example 6.6

Find the settling velocity for a suspended solid particle has the following characteristics;

a- d=0.5mm and S=2.65.

b- d=0.1mm and S=1.1.

a- d=0.5mm and S=2.65.

Assume Re<0.5 → $use\ Stoke's\ law\ for\ settling\ velocity$;

$V_S = \frac{gd^2(S-1)}{18v}$; $\quad v = \frac{\mu}{\rho} = \frac{1.012\times10^{-3}}{1000} = 1.012 \times 10^{-6} m^2/sec$

$V_S = \frac{9.81\times(\frac{0.5}{1000})^2\times(2.65-1)}{18\times1.012\times10^{-6}}$=0.2221 m/sec

Use the above value of V_S to check Re;

Re=$\frac{\rho V_S d}{\mu} = \frac{V_S d}{v} = \frac{0.2221\times0.5\times10^{-3}}{1.012\times10^{-6}}$ =109.8>0.5 ..Not O.K.

Use the above value of Re to calculate C_D;

For 0.5 <Re<10^4

$$C_D = \frac{24}{Re} + \frac{3}{\sqrt{Re}} + 0.34$$

$\therefore C_D = \frac{24}{109.8} + \frac{3}{\sqrt{109.8}}$+0.34=0.8450

Use C_D to calculate V_S using Newton's law for settling velocity;

$V_S = \sqrt{\frac{4gd(S-1)}{3C_D}} = \sqrt{\frac{4\times9.81\times0.5\times10^{-3}(2.65-1)}{3\times0.845}}$ =0.1130m/sec

Compare the last calculated VS value with the previous calculated V_S value if they are approximately equal, then the last Vs value is the required settling velocity. If they are not, then, use the last V_S value to calculate Re, then use Re to calculate CD and so on… i.e., the solution is done by iteration.

Since; $0.1130(the\ last\ V_S\ value) \neq 0.2221(the\ previous\ V_s\ value)$

then, use the last V_S value to calculate Re;

Re=$\frac{0.1130\times0.5\times10^{-3}}{1.012\times10^{-6}}$=55.8→ $C_D = \frac{24}{55.8} + \frac{3}{\sqrt{55.8}}$+0.34=1.1714

The remaining calculation results are arranged in the following table:

V_S (m/sec)	Re	CD
0.2221	109.8	0.8450
0.1130	55.8	1.1714
0.0960	47.4	1.2817
0.0918	45.3	1.3150
0.0906	44.8	1.3247
0.0903	44.6	1.3274
0.0902	44.5	1.3283
0.0901	44.5	1.3285
0.0901		

$\therefore V_S$=0.0901m/sec

d=0.1mm and S=1.1

Assume Re<0.5 → $use\ Stoke's\ law\ for\ settling\ velocity$;

$V_S = \frac{gd^2(S-1)}{18v}$;

$V_S = \frac{9.81\times(\frac{0.1}{1000})^2\times(1.1-1)}{18\times1.012\times10^{-6}}$=0.0005 m/sec

Use the above value of V_S to check Re;

Re=$\frac{\rho V_S d}{\mu} = \frac{V_S d}{v} = \frac{0.0005\times0.1\times10^{-3}}{1.012\times10^{-6}}$ =0.1<0.5 … O.K.

$\therefore V_S = 0.0005 m/sec$

6-2-3-3 Ideal Settling Tank (Plain Sedimentation)

Plain sedimentation tank can be considered as an ideal settling tank. Plain sedimentation tank is not preceded by coagulation and flocculation units. Fig. 11 illustrates an ideal rectangular clarifier (settling tank) with an inlet zone for transition of influent flow to uniform horizontal flow, a settling zone where the particles settle out of suspension by gravity, an outlet zone for transition of uniform flow in the sedimentation zone to rising flow for discharge, and a sludge zone where the settled particles collect.

Assumptions of ideal settling tank

1. Quiescent condition in settling one.
2. Uniform flow across the settling zone.
3. Uniform solid concentration as flow enters the settling zone.
4. Solids entering the sludge zone are not resuspended.

When a particle enters the settling zone, it will have horizontal velocity component (V_h) equals that of water;

$$V_h = \frac{Q}{W.h} \quad ; \quad \text{W=tank width} \quad \text{and} \quad \text{h=water depth}$$

and a vertical velocity component equals to its terminal settling velocity, V_S.

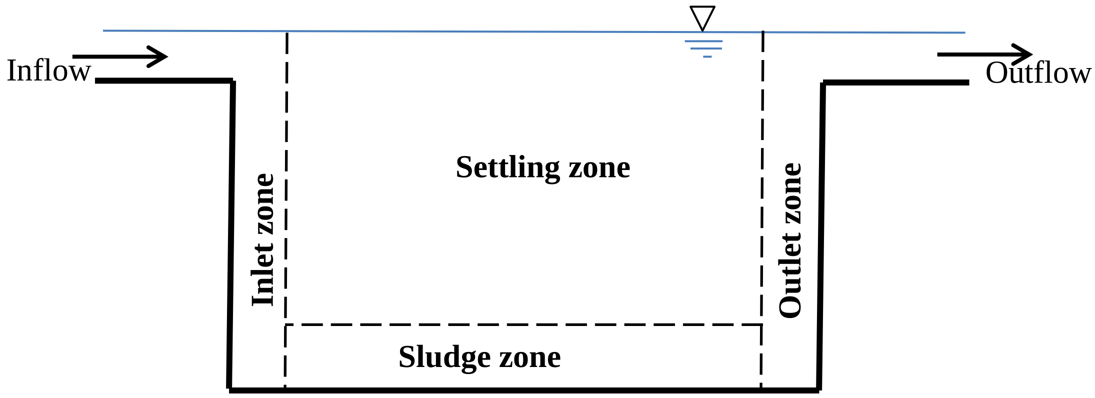


Fig.11 An ideal rectangular settling tank

The particle shall be removed from water if the resultant velocity takes it to the bottom of the tank (the sludge zone) before the outlet zone is reached, Fig.12.

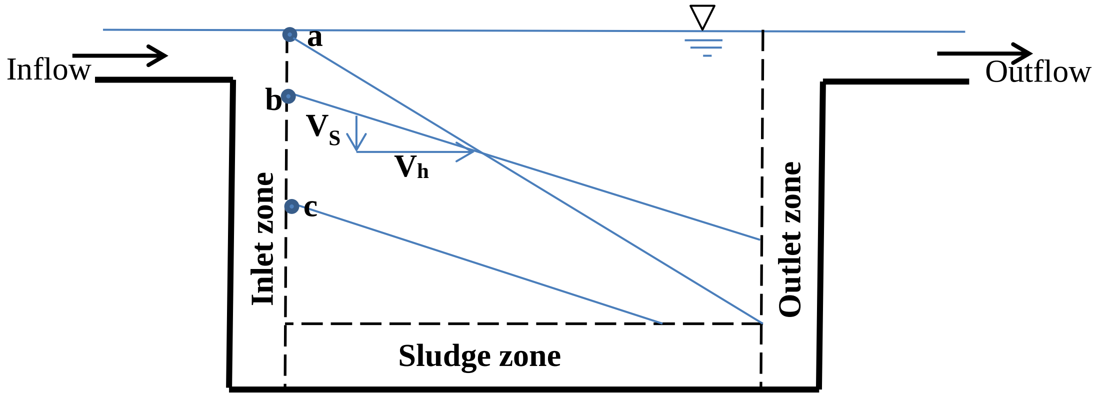


Fig.12 Path of suspended solid particle in the settling zone.

Look at Fig.12 and specify the removed particles (a, b, or c)?

6-2-3-4 Surface Overflow Rate

Surface overflow rate (SOR) is numerically equal to the flowrate divided by the plan area of the basin (SOR=Q/A). Physically, it represents the settling velocity of slowest settling particles removed at 100%. The particles which have settling velocity greater than SOR will be entirely removed, while, those have settling velocity less than SOR will be removed at a ratio equals to their settling velocity (V_S) to SOR.

Thus, the fraction of all removed particles (or tank efficiency) shall be;

$$F = (1 - X_S) + \int_0^{X_S} \frac{V_S}{SOR} dx$$

Or;

$$F = (1 - X_S) + \frac{1}{SOR} \sum_1^n V_{Si} \Delta x_i$$

Where;

X_S= Fraction of particles with settling velocity less than SOR.

n= number of slips (at least n=6)

i= slip number

Δx_i= thickness of slip No.i

V_{Si}= settling velocity at the center of slip No.i

In order to find F, it is required to conduct particles size distribution analysis for the treated water. After that the curve of cumulative distribution of particle settling velcity is drawn as shown in Fig.13. *See example on page 212.*

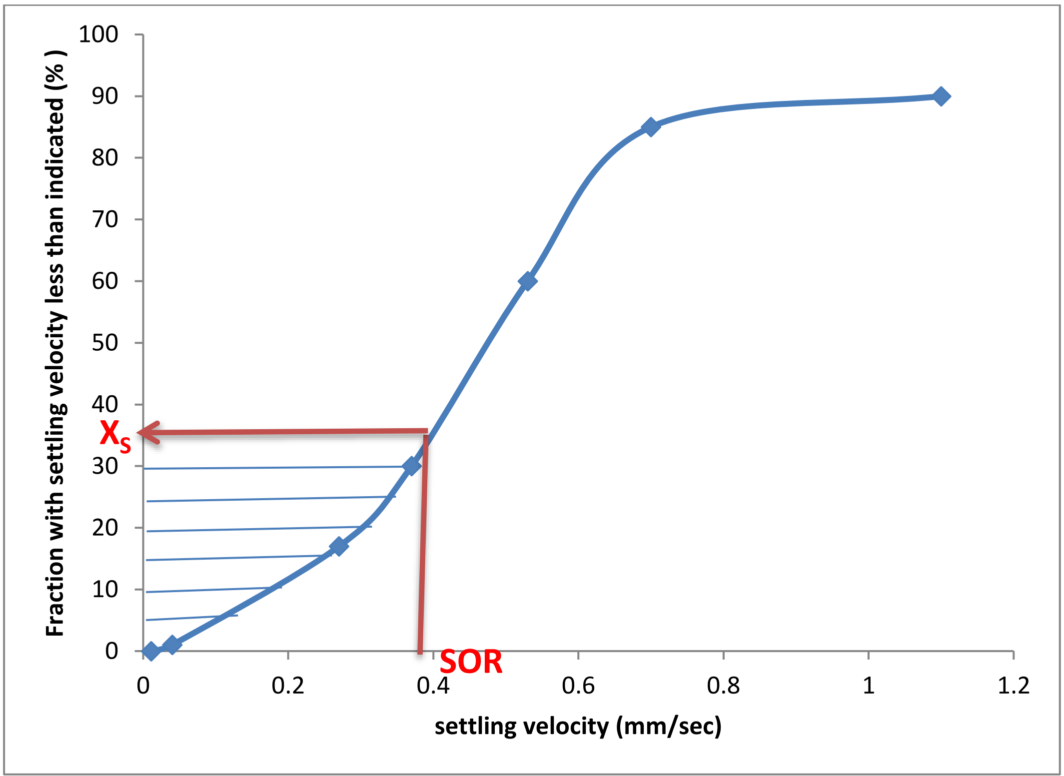


Fig.13 Cumulative distribution of particle settling velocity

6-2-3-5 Flocculant Settling

To find the efficiency of a sedimentation tank preceded by coagulation and flocculation processes, a settling analysis must be performed. This analysis is done in a settling column of 300mm diameter and has a height equals that of the proposed settling tank, Fig.14. Water samples are drawn from taps distributed at different heights and at different times. Then, the con centration of suspended solids in each sample is measured. The analysis results are arranged as shown in Table 1. After that the removal percentages are obtained and the results are arranged as shown in Table 2.

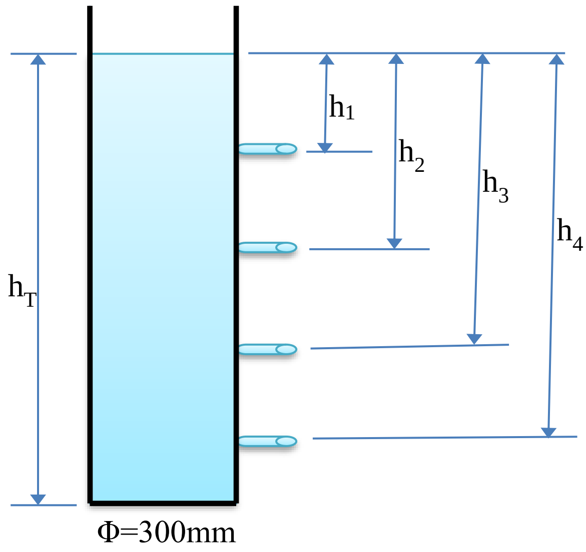


Fig.14 Settling column

Table 1 Data sheet of settling column analysis

Time	SS Concentration (mg/l) at indicated depth			
	h_1	h_2	h_3	h_4
0				
10				
20				

Table 2 Calculations sheet of settling column analysis

Time	% of SS removal at indicated depth			
	h_1	h_2	h_3	h_4
0				
10				
20				

$$\% \ of \ SS \ removal = \frac{SS_0 - SS_t}{SS_0} \times 100$$

SS_0= initial suspended solids concentration

SS_t= Suspended solids concentration at time=t

The calculated SS removal percents are plotted verses time and depth and a contour map of percent removals is plotted as shown in Fig.15.

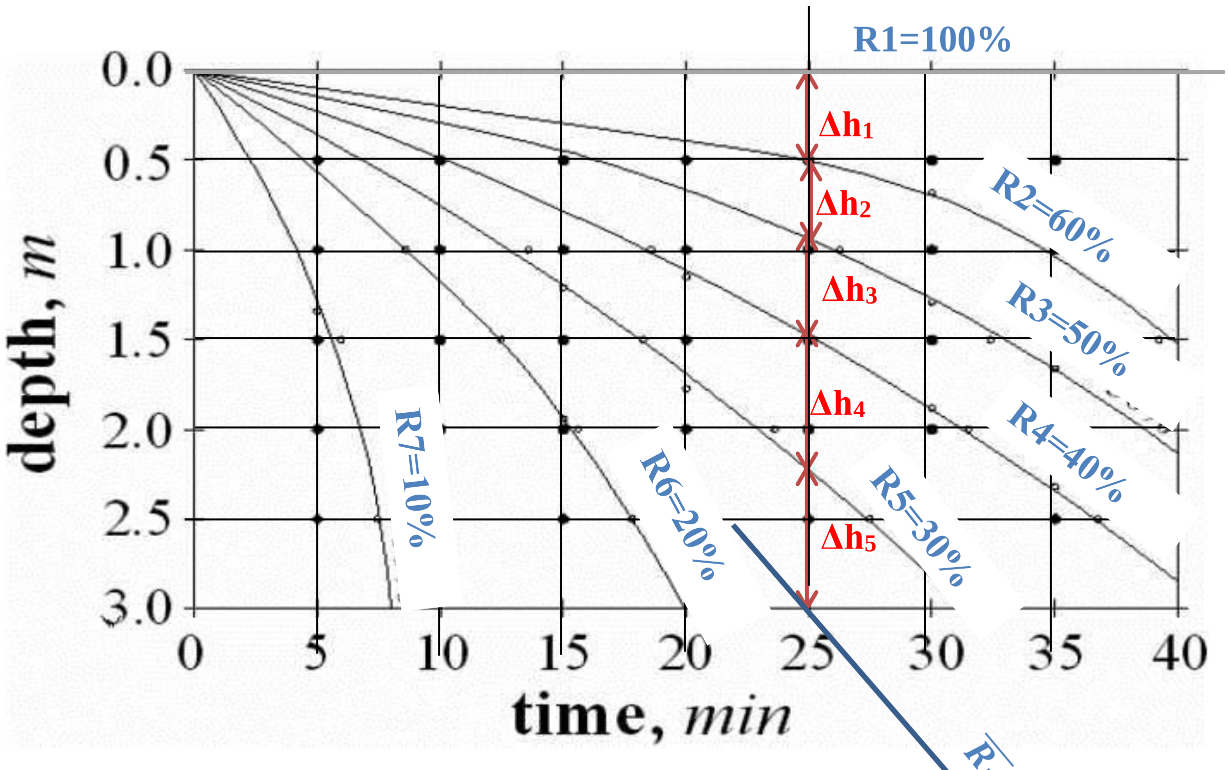


Fig. 15 Settling of flocculant particles

From the plot in Fig.15, the overall removal percentage (F) is determined at a given detention time as;

$$F = \frac{R_1 + R_2}{2} \times \frac{\Delta h_1}{h_T} + \frac{R_2 + R_3}{2} \times \frac{\Delta h_2}{h_T} + \frac{R_3 + R_4}{2} \times \frac{\Delta h_3}{h_T} + \frac{R_4 + R_5}{2} \times \frac{\Delta h_4}{h_T} + \frac{R_5 + \overline{R_5}}{2} \times \frac{\Delta h_5}{h_T} + \cdots..$$

Where; R_1, R_2, R_3,…….= removal percentage of contour lines

Δh values are obtained by plotting a vertical line from the given value of detention time as shown in Fig.15 (detention time= 25min as an example)

Example 6.7

A rectangular sedimentation tank has a length of 40m and a width 10m. It treats water at a flow rate of 9600m$^{3/}$day. The tank is designed to have a side water depth of 4m. Find the overall removal of suspended solid particles using the results of settling column analysis given below.

Time (min.)	% of Suspended solids removal at indicated depth				
	0.8m	1.6m	2.4m	3.2m	4.0m
40	41	21	11	6	3
80	63	45	34	26	21
120	78	59	48	41	36
160	83	71	59	52	46
200	87	76	69	60	55
240	90	81	74	69	65

Solution:

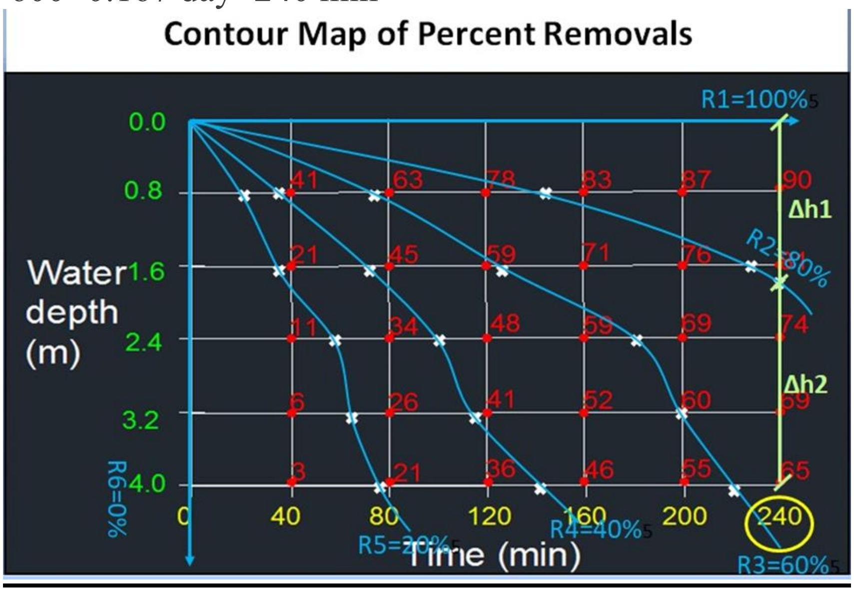
Determine detention time;

Q=V/t→ $t = V/Q$

$V = L \times W \times H$

$V = 40 \times 10 \times 4 = 1600 m^3$

$t = 1600/9600$=0.167 day=240 min



$$F = \frac{R_1+R_2}{2} \times \frac{\Delta h_1}{h_T} + \frac{R_2+R_3}{2} \times \frac{\Delta h_2}{h_T} + \frac{R_3+R_4}{2} \times \frac{\Delta h_3}{h_T} + \frac{R_4+R_5}{2} \times \frac{\Delta h_4}{h_T} + \frac{R_5+\overline{R_5}}{2} \times \frac{\Delta h_5}{h_T} + \cdots ..$$

$$F = \frac{R_1 + R_2}{2} \times \frac{\Delta h_1}{h_T} + \frac{R_2 + \overline{R_2}}{2} \times \frac{\Delta h_2}{h_T}$$

$$F = \frac{100 + 80}{2} \times \frac{1.8}{4} + \frac{80 + 65}{2} \times \frac{2.2}{4}$$

$$F = 80.4\%$$

<u>Question</u>

If the sludge in settling tank of example 6.7 is not withdrawn periodically and it accumulated in the tank and its depth reached a value of 0.8m. Does the tank efficiency increase or decrease due to sludge accumulation and at what percentage?

6-2-3-6 Types of Sedimentation Tanks

Sedimentation tanks may have rectangular, circular or square shapes.

<u>Rectangular sedimentation tanks</u>

Fig. 16 shows plan view and a vertical profile of a rectangular sedimentation tank. Rectangular sedimentation tanks are usually designed to be long and narrow with the following characteristics;

- The flow is along the long axis.
- Length to width ratio (L/W)=3 to 6
- Tanks dimensions are selected to match the requirements of the chosen sludge collection equipment (scraper). Generally;
- Maximum tank length=100m
- Maximum tank width= 13.5m
- If scraper is used, bottom slope=1:24 to 1:12 (V:H).

- If scraper is not use (in case of small sedimentation tanks), bottom slope of 1:1 is usually adopted.
- The influent is discharged behind a baffle (inlet baffle).
- The overflow is flowing over an outlet weir into the effluent launder.

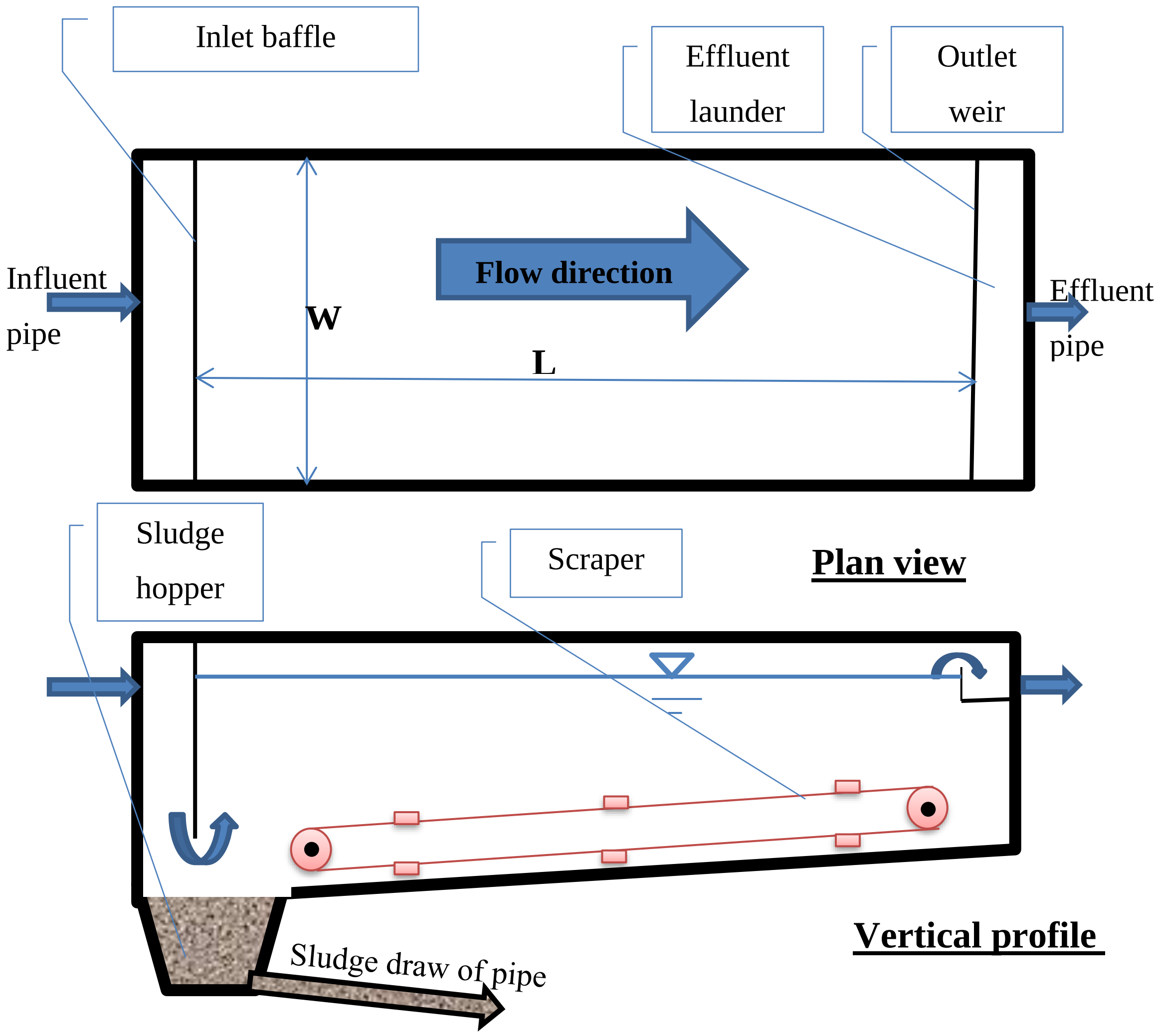


Fig.16 Plan view and vertical profile of a rectangular sedimentation tank

Circular sedimentation tanks

Fig.17 shows a plan view and a vertical profile in a circular sedimentation tank. The inlfluent in discharged at the centre of the tank in a cylinder called stilling well. The main characteristics of circular tanks are;

- Maximum diameter= 40m.
- Stilling well diameter = 0.1 to 0.2 of tank diameter and extends 1 to 2 m below the water surface.
- The overflow is flowing over an outlet weir and collected into the effluent launder at the tank periphery.
- Bottom slope as that of rectangular tanks.

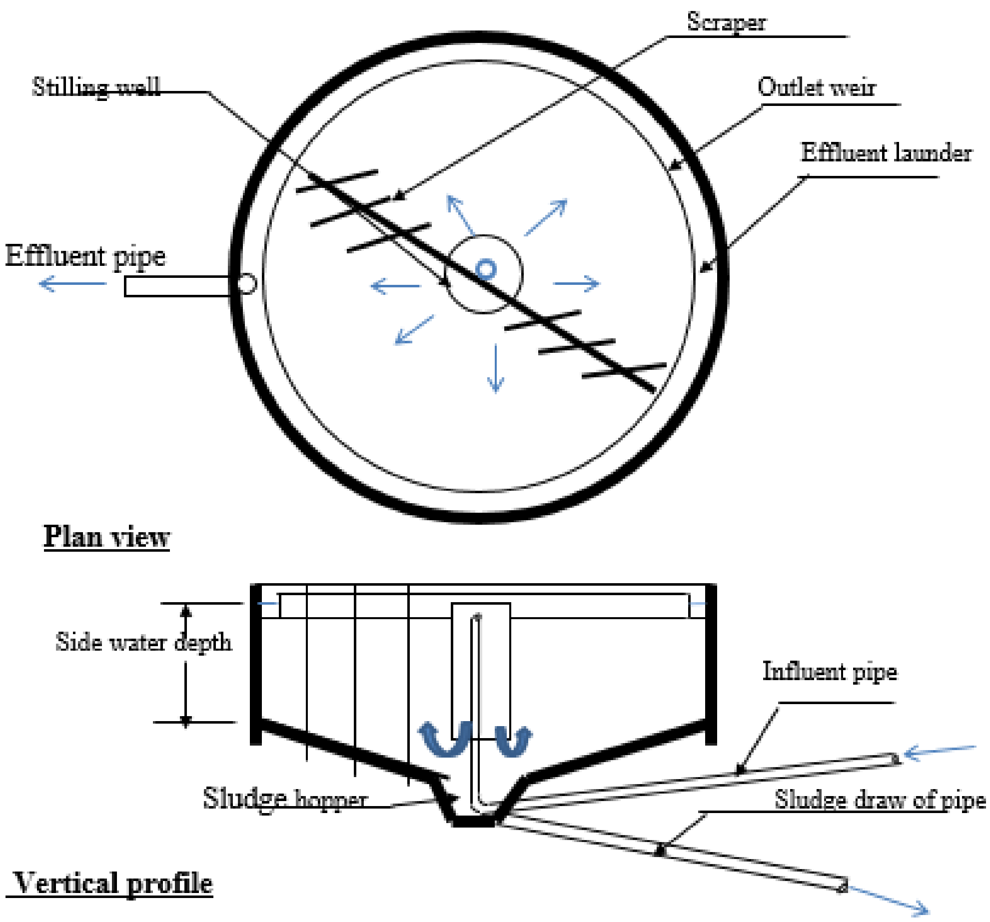


Fig.17 Plan view and vertical profile of a circular sedimentation tank

6-2-3-7 Design of Sedimentation Unit

Design criteria

For cedimentation unit preceded by coagulation and flocculation processes, the following criteria are adopted:

- Surface overflow rate (SOR)= 20 to 33 m^3/m^2.day
- Detention time (t)= 2 to 8 hours.
- Weir loading rate≤ 250 m^3/m.day

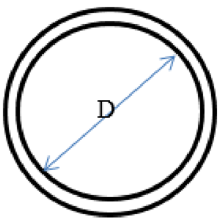
The weir loading rate is equal to;

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

Where;

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

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



For rectangular tanks;

If one effluent launder is used; weir length=W

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

Design steps

1. Assume SOR

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

$$A = Q/SOR$$

3. Assume the number of tanks=2

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

5. Find the dimensions of tank;

   If circular tanks are used;

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

   Where; D_S= diameter of stilling well

   if rectangular tanks are used;

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

6. If D>40m, increase the number of tanks (circular tanks)
7. If L >100m or W>13.5m, increase the number of tanks (retangular tanks)
8. Find Q_{one}
9. Check weir loading rate.

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

   Use v-notches or zig-zag weir, as in Fig.18

Fig.18 V-noches outlet weir

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

Increase the number of effluent launders to n and find n by putting weir loading rate = 250m^3/m.day.

Important note:

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

10. Assume detention time (t).
11. Find the water volume in one tank as;

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

12. Find the side water depth (SWD);

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

Note: h must be between 3 and 5m

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

Design of effluent launder

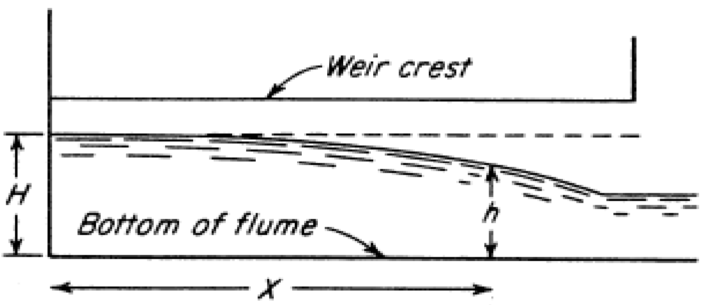
a- Find the minimum water depth in the effluent launder (h);

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

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

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

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



Where;

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

x= length of water path.

For circular tanks;

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

For rectangular tanks;

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

and;

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

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

Example 6.8

Design sedimentation unit for a water treatment plant has a design capacity of 120,000 m^3/day using;

1. Circular tanks.
2. Rectangular tanks.

Solution:

1. Circular tanks

Assume SOR=24 m/day

$$SOR=\frac{Q}{A}\ \rightarrow A=\frac{Q}{SOR}=\frac{120000}{24}=5000\ m^2$$

Assume the number of tanks=2

$$A_{one}=\frac{5000}{2}=2500m^2$$

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

Let $D_s=0.15D\rightarrow\ \ 2500=\frac{\pi}{4}(D^2-(0.15D)^2)$

D=57.06m >40m → increase the number of tanks

Let the number of tanks=3→$A_{one}=\frac{5000}{3}=1666.67m^2$

$$1666.67=\frac{\pi}{4}(D^2-(0.15D)^2)$$

D=46.6m >40m → increase the number of tanks

Let the number of tanks=4→$A_{one}=\frac{5000}{4}=1250m^2$

$$1250=\frac{\pi}{4}(D^2-(0.15D)^2)$$

D=40.35m >40m → increase the number of tanks

Let the number of tanks=5→$A_{one}=\frac{5000}{5}=1000m^2$

$$1000=\frac{\pi}{4}(D^2-(0.15D)^2)$$

D=36.1m <40m → O.K.

D_s=0.15×36.1=5.42m

Or; Assume D=40m→$A_{one}=\frac{\pi}{4}(D^2-(0.15D)^2)$

$$A_{one}=\frac{\pi}{4}(40^2-(0.15\times 40)^2)=1228.36m^2$$

$$\therefore number\ of\ tanks=\frac{5000}{1228.36}=4.07\rightarrow use\ 5\ tanks$$

$$Aone=\frac{5000}{5}=1000m^2\rightarrow D=36.1m$$

$Check\ weir\ loading\ rate$;

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

$$Q_{one}=\frac{Q_{total}}{number\ of\ tanks}=\frac{120000}{5}=24000m^3/day$$

Weir length= $\pi D=\pi\times 36.1=113.41m$

$$weir\ loading\ rate=\frac{24000}{113.41}=\frac{211.62m^3}{m.day}<250\frac{m^3}{m.day}\rightarrow O.K.$$

Assume detention time (t)=4 hours

Find the water volume in one tank as;

$$V_{one}=Q_{one}\times t=\frac{24000}{24}\times 4=4000m^3$$

Find the side water depth (SWD);

$SWD = \frac{V_{one}}{A_{one}} = \frac{4000}{1000} = 4m\ (3-5m) \rightarrow O.K.$

Find the total tank depth ;

Total tank depth=1.1×4=4.4m

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

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

Assume b=400mm=0.4m

$h = \sqrt[3]{\frac{Q_{one}^2}{gb^2}} = \sqrt[3]{\frac{\left(\frac{24000/2}{24\times3600}\right)^2}{9.81\times0.4^2}}$ =0.23m

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

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

q= $\frac{211.62}{24\times3600}$ =0.00245m^3/m/sec

$$X = \frac{\pi D}{2} = \pi \times \frac{36.1}{2} = 56.71m$$

$$H = \left[0.23^2 + \frac{2 \times 0.00245^2 \times 56.71^2}{9.81 \times 0.4^2 \times 0.23}\right]^{0.5} = 0.4m$$

Add free board of 15cm

Total depth of effluent launder=0.4+0.15=0.55m

<u>2. Rectangular tanks</u>

Assume SOR=24 m/day

$$SOR = \frac{Q}{A} \rightarrow A = \frac{Q}{SOR} = \frac{120000}{24} = 5000\ m^2$$

Assume the number of tanks=2

$$A_{one} = \frac{5000}{2} = 2500m^2$$

$A_{one} = W \times L$

Let L/W=4 → L=4W

$2500 = W \times 4W$→ W=25m (>13m Not. O.K.) →L=4×25=100m→ increase the number of tanks

Let the number of tanks=3→$A_{one} = \frac{5000}{3} = 1666.67m^2$

$1666.67 = 4W^2 \rightarrow W$ =20.4m>13.5m

Let W=13m → L=4×13=52m → $A_{one} = 13 \times 52 = 676m^2$

Number of tanks=$\frac{5000}{676} = 7.4 \quad \rightarrow use\ 8\ tanks$

$$A_{one} = \frac{5000}{8} = 625m^2$$

$\therefore 625 = 4W^2 \rightarrow W = 12.5m\ and\ \ L = 4 \times 12.5 = 50m$

$Check\ weir\ loading\ rate$;

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

$$Q_{one} = \frac{Q_{total}}{number\ of\ tanks} = \frac{120000}{8} = 15000\ m^3/day$$

If one effluent launder is used →Weir length= $W = 12.5m$

$weir\ loading\ rate = \frac{15000}{12.5} = \frac{1200m^3}{m.day} > 250\frac{m^3}{m.day} \rightarrow Not\ O.K.$

Let the number of effluent launders=n

Weir length=(2n-1)W

$250 = \frac{15000}{(2n-1)\times 12.5}$ →n=2.9 use 3 effluent launders

Weir loading rate=$\frac{15000}{(2\times 3-1)\times 12.5}$=240 $\frac{m^3}{m.day}$

Assume detention time (t)=4 hours

Find the water volume in one tank as;

$V_{one} = Q_{one} \times t = \frac{15000}{24} \times 4 = 2500m^3$

Find the side water depth (SWD);

$SWD = \frac{V_{one}}{A_{one}} = \frac{2500}{12.5 \times 50} = 4m\ (3 - 5m) \rightarrow O.K.$

Find the total tank depth;

Total tank depth=1.1×4=4.4m

<u>Example 6.9</u>

A water treatment plant includes settling unit composed of five circular settling tanks. Each tank has a diameter of 35m and a side water depth of 4.0 m. The plant treats water at a flow rate of $1.5m^3/sec$ (a) Check the unit design (b) Design the effluent launder. (c) If the treated water contains solid particles of density equals $1400\ kg/m^3$, find the size of the solid particles removed at 60% ($\rho = 1000\ kg/m^3$ and $\mu = 1.012 \times 10^{-3} N.sec/m^2$).

<u>Solution</u>

(a) Check the unit design

Check of unit design means checking if the adopted design criteria values are within the recommended values range.

<u>Check of SOR</u>

$$A_{one} = \frac{\pi}{4}(D^2 - D_S^2) = \frac{\pi}{4}\left(35^2 - 3.5^2\right) = 952.5m^2$$

$$Q_{one} = \frac{1.5}{5} = 0.3m^3/se$$

$SOR = \frac{Q}{A} = \frac{0.3 \times 24 \times 3600}{952.5} = 27.2\ m/day$ →O.K. since it is within the rang (20-33) m/day

Check of detention (t);

$V_{one} = A_{one} \times SWD = 952.5 \times 4 = 3810 m^3$

$Q = \frac{V}{t} \rightarrow \quad t = \frac{V}{Q} = \frac{3810}{0.3\times3600} = 3.53 hr.$ →O.K. since it is within the rang (2-8) hours.

Check of weir loading rate;

$weir\ loading\ rate = \frac{Q_{one}}{weir\ length} = \frac{0.3\times24\times3600}{\pi\times35}$=235.7$m^3$/m/day< 250$m^3$/m/day…O.K.

Since all the design criteria values are within the recommended values, then, the design of sedimentation unit is O.K.

(c) Determining the size of solid particles removed at 60%

Since, the particles which have settling velocity greater than SOR will be entirely removed, while, those have settling velocity less than SOR will be removed at a ratio equals to their settling velocity (V_S) to SOR.

$\therefore \frac{V_S}{SOR} = 0.6$

SOR= 27.2 m/day=3.15×10^{-4} m/sec

$V_S=0.6\times3.15\times10^{-4} = 1.89\times10^{-4}$ m/sec

Assume Re<0.5→ $V_S = \frac{gd^2(S-1)}{18v}$

S=$\frac{\rho_s}{\rho_w} = \frac{1400}{1000} = 1.4;$ $\quad \boldsymbol{v = \frac{\mu}{\rho} = \frac{1.012\times10^{-3}}{1000} = 1.012\times10^{-6} m^2/sec}$

$$1.89\times10^{-4} = \frac{9.81d^2(1.4-1)}{18\times\mathbf{1.012\times10^{-6}}}$$

$d=2.96\times10^{-5}$ m

Check Re;

$$Re=\frac{\rho V_S d}{\mu}=\frac{V_S d}{v}=\frac{1.89\times10^{-4}\times 2.96\times10^{-5}}{\mathbf{1.012\times10^{-6}}}==\mathbf{0.005<0.5 \text{ .. O.K.}}$$

∴ *$d=2.96\times10^{-5}$ m=0.03mm (size of solid particles removed at 60%)*

6-2-4 Filtration Unit

Filtration unit is used to separate non-settleable suspended solids from water by passing it through a porous medium. The aim of filtration is to;

- Produce clear water, and
- Remove color, taste, odors, iron and manganese.

6-2-4-1 Filters Classification

A number of classification systems are used to describe granular filters including filtration rate, media type, flow mechanism, washing technique, filtration rate control. Based on filtration rate filters are classified into rapid sand filter and slow sand filter. The characteristics of rapid sand filter include:

- It is used to high turbidity water which is requiring chemical pretreatment (coagulation).
- Filtration rate = 120 to 240 m/day.
- Beds are cleaned by backwash process.

Where;

$$filtration\ rate=\frac{Q}{surface\ area\ of\ filter}$$

The characteristics of slow sand filter include;

- It is used for low turbidity water which is not requiring chemical pre-treatment.
- Filtration rate=2.6 to 6m/day.
- It is effective in reducing bacteria and color.
- It has high construction cost because it requires large surface area.
- Beds are cleaned by scraping off a thin top layer of sand.

Based on flow mechanisms, filters are classified into;

1. Gravity filters (open rectangular concrete box).
2. Pressure filters (closed cylindrical metal tank).

Fig.19 shows schematic diagram of gravity sand filter

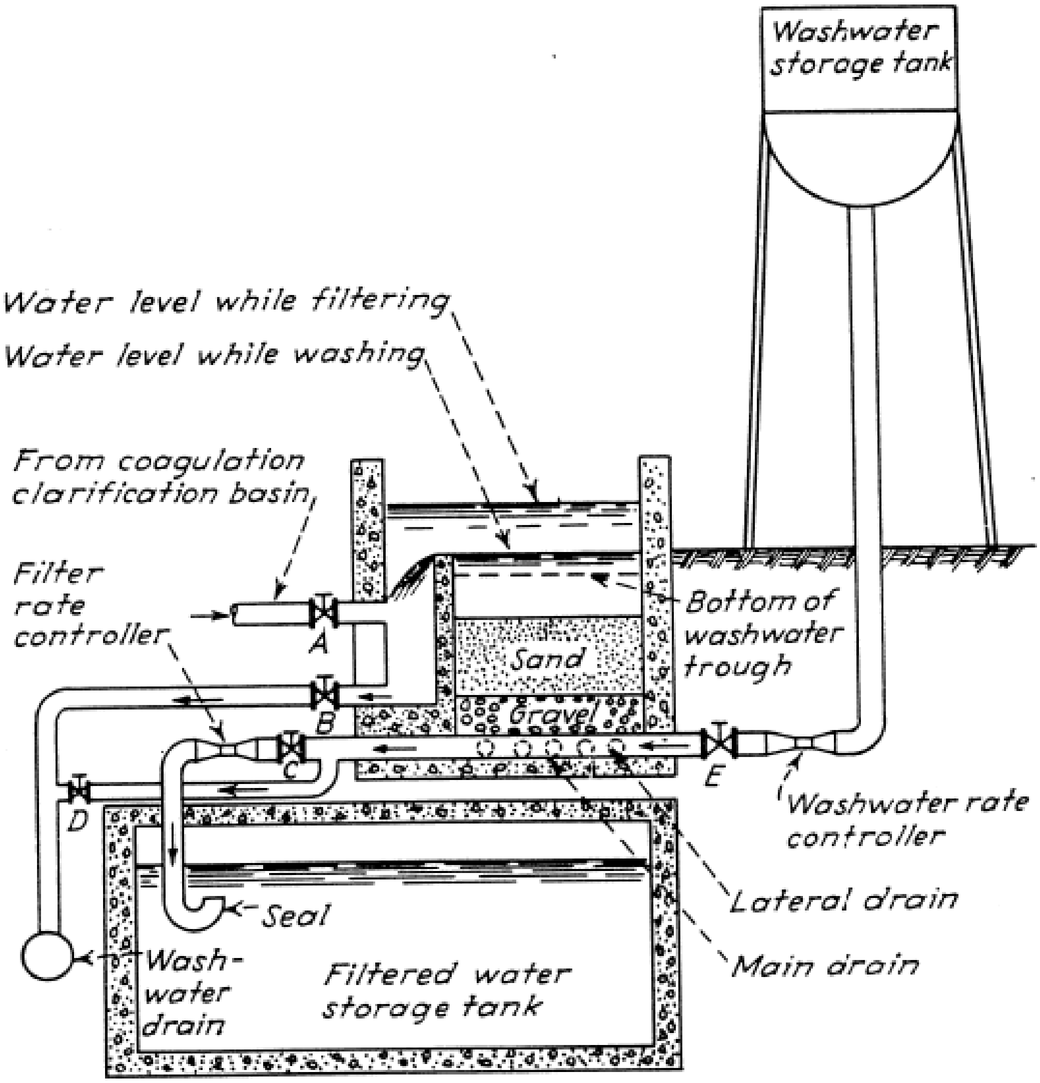


Fig.19 Schematic diagram of gravity sand filter

6-2-4-2 Filter media

Filters may be of single medium, dual-media or triple-media. In single medium, sand or anthracite is used alone and the specifications of sand are;

- Effective size (D_{10})=0.45 to 0.55mm.
- Uniformity coefficient (D_{60}/D_{10})= 1.2 to 1.7.
- Sand depth=600 to 700.

The specifications of anthracite are;

- Effective size (D_{10}) $\geq$ 0.7mm.
- Uniformity coefficient (D_{60}/D_{10}) $\leq$ 1.75.
- Anthracite depth= water depth.

Dual-media beds normally contain 0.15 to 0.3m of sand has effective size of 0.45 to 0.55mm overlaid by 0.46 to 0.76m of anthracite has effective size of 0.8 to 1.2 mm. A triple-media contains 5 to 10cm garnet has effective size of 0.15 to 0.35mm, 0.15 to 0.3m of sand has effective size of 0.35 to 0.5mm and 0.5 to 0.6m anthracite size of 0.8 to 1.2.

6-2-4-3 Gravel

The sand is underlain by 400 to 600mm of gravel which serves to;

- Support the sand
- Allow the washwater to move more uniformly upward to the sand.

Gravel is placed into five to six layers with the finest size on top. The arrangement of gravel layer is as follows;

Gravel size	Layer thickness
2.5 to 5 mm	60 to 80 mm
5 to 10 mm	60 to 80 mm
10 to 20 mm	80 to 120 mm
20 to 40 mm	80 to 120 mm
40 to 60 mm	120 to 200 mm

Total depth = 400 to 600 mm

6-2-4-4 The Underdrain System

The underdrain collects the filtered water from the gravel and distribute the washwater during the washing process. A widely used type is the perforated-pipe system. It consists of a central ductile iron manifold or header into a number of laterals can be attached. The perforations are placed alternately on the underside but 30° off center, see Fig.20. The following criteria are adopted for the design of perforated pipe system:

1. L/D $\leq$ 60 where L is lateral length and D is lateral diameter.
2. Diameter of perforations in the lateral= 6 to 12.5mm
3. Spacing of perforations along the lateral =75mm for 6mm holes and 200mm for 12.5mm holes.
4. $\frac{\sum A_{perforations\ in\ the\ underdrain\ system}}{\sum A_{laterals}} \leq$

   $0.5\ for\ 12.5mm\ perforations\ or \leq 0.25\ for\ 6mm\ perforations$
5. $\frac{\sum A_{perforations\ in\ the\ underdrain\ system}}{filter\ surface\ area} \geq 0.002$
6. Spacing of laterals $\leq$300mm

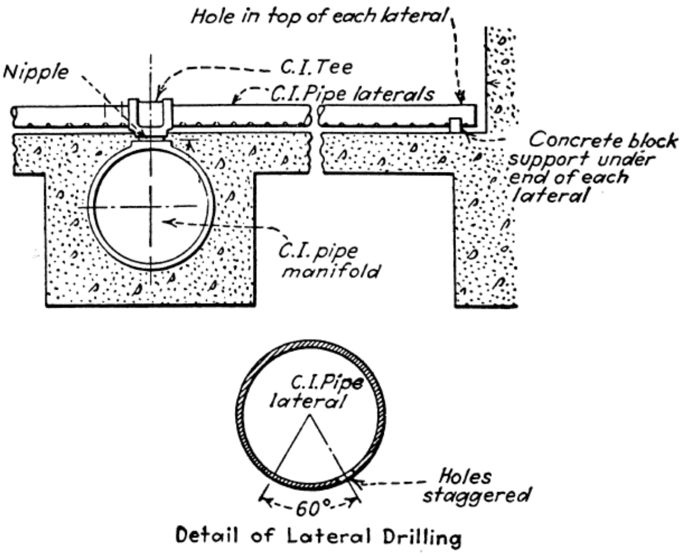


Fig.20 Perforated pipe underdrain system

6-2-4-5 The Washing Process

Washing consists of passing filtered water upward through the bed at such velocity that causes the sand bed expansion. The cleaning of a granular bed during backwash is a result of the shear produced by the rising water and of the abrasion resulting from contacts between particles in the fluidized bed. The backwash velocity (V_b) is;

$$0.3m/min < V_b < V_t$$

$$V_t = 10D_{60} \text{ for sand } (V_t \, in \, m/\text{min} \, and \;\; D_{60} \, in \, mm)$$

$$V_t = 4.7D_{60} \text{ for anthracite}$$

It was found that the maximum abrasion occurs when the bed is 10 percent expanded or;

$$V_b = 0.1V_t$$

Thus for sand;

$$V_b = D_{60}$$

And for anthracite;

$$V_b = 0.47\, D_{60}$$

The rates above are for a temperature of 20°C but can be corrected for other temperature by;

$$V_{b(T)} = V_{b(20)} \times \mu_T^{-1/3}$$

Where;

$V_{b(T)}$ is backwash velocity at temperature = T.

μ_T is water viscosity at temperature=T

<u>Example 6.10</u>

Determine the backwash rate for a sand medium with an effective size of 0.5mm and a uniformity coefficient of 1.5 at 5 and 35°C.

6-2-4-6 Filter Unit and Washwater Troughs

A filter consists of two or more units of sizes depend on plant capacity. Usually all the units are of the same capacity. The units may be placed in one or two rows.

A unit of gravity filter (Fig.21) is concrete box open at the top with a depth of 3m or more.

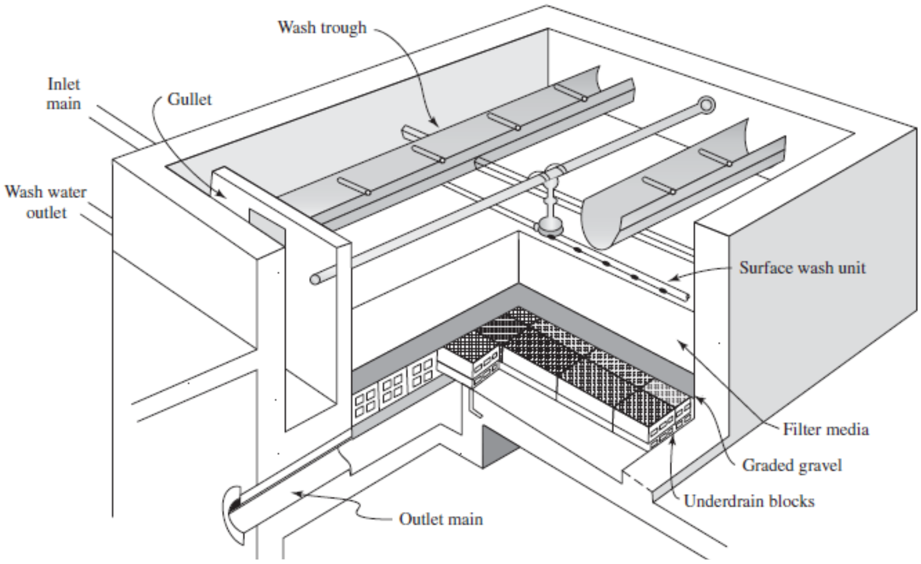


Fig.21 Perspective of gravity filter

$$Total\ filter\ depth = Thickness\ of\ sand + thickness\ of\ gravel + water\ depth + dia.of\ lateral + free\ board$$

The influent pipe discharges behind a baffle wall so that the water currents will not disturb the sand. The sidewalls of filter unit are usually roughened to avoid streaming of water between the sand and the walls.

The rising washwater, after passing through the media, flows into washwater troughs. The top edges of the troughs are horizontal and are placed at the same height, usually at a distance of 600 to 900mm above the sand level. the spacing of troughs (from edge to edge) is ≤ 2m.The troughs can be arranged as shown in Fig.22.

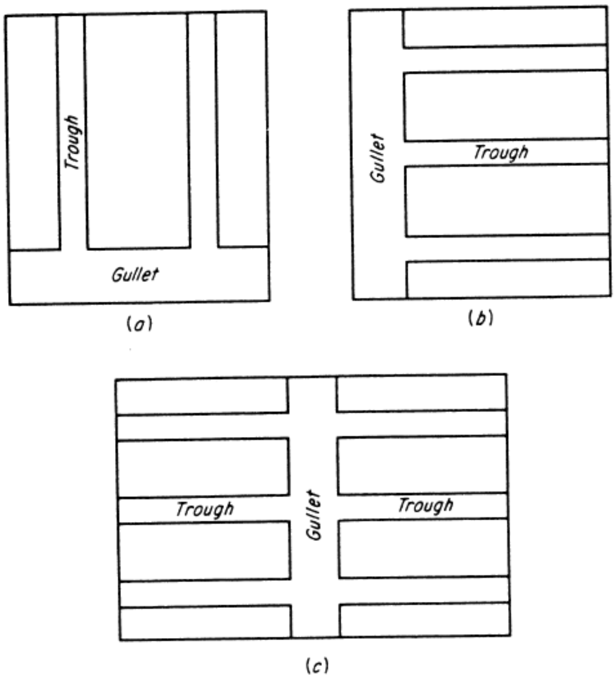


Fig.22 Washwater troughs arrangement

The troughs may have rectangular section or may have vertical walls with V-shape bottom. For rectangular troughs, the dimensions are obtained as;

$$\boldsymbol{y = 1.73}\sqrt[3]{\frac{Q^2}{gb^2}}$$

Where y is the water depth (m) and b is the trough width (m) and Q is the washwater received by one trough (m^3/sec). A free board of 50 to 100mm is usually added to y. for V-shape bottom, equivalent area to the rectangular is used.

<u>Backwash duration</u>

The duration of washing one filter unit is 5 to 10 minutes. This duration is necessary for estimating the required volume of washwater.

Example 6.11:

Design filtration unit for water treatment plant has a design capacity of 120000m^3/day. Use gravity rapid sand filters and assume water temperature is 20°C.

Solution

Assume filtration rate=180m/day

Filtration rate= Q/A

A_{total} =120000/180=666.67m^2

Let number of units=2

A_{unit} = 666.67/2=333.33m^2

Check filtration rate during backwash process;

Filtration rate= $\frac{120000}{(2-1)\times 333.33}$ = 360m/day>240m/day…not O.K

Let number of units=n

A_{unit} = 666.67/n

Put filtration rate during washing=240m/day

$$\frac{120000}{(n-1)\times\frac{666.67}{n}} = 240$$

n= 4

use 4 units $\longrightarrow$ A_{unit} =666.67/4= 166.67 m^2

let L/W=2:1

L=2W $\longrightarrow$ W ×2W=166.67 $\longrightarrow$ W=9.13m $\longrightarrow$ L= 2×9.13=18.26m

Increase the length of unit by 10% (= 1.83m) to consider the inlet section.

Let thickness of sand layer =600mm

Let effective size of sand (D_{10})=0.5mm

Let uniformity coef.= 1.4 =D_{60}/D_{10}

D_{60}=0.5×1.4=0.7mm

Design of washwater troughs

For sand medium;

V_b= D_{60} $\longrightarrow$ V_b =0.7m/min

Let number of washwater troughs =2

Let width of trough=0.6m

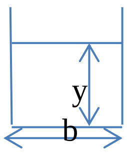
Check spacing of troughs

Spacing of troughs= $\frac{9.13-2\times0.6}{2} = 3.97m > 2m \ldots.not\ O.K.$

Let number of troughs=4

Spacing of troughs= $\frac{9.13-4\times0.6}{4} = 1.68m < 2m \ldots.O.K.$

Use rectangular troughs



$$\boldsymbol{y = 1.73}\sqrt[3]{\frac{Q^2}{gb^2}}$$

$Q_{backwash}$= V_b×A_{unit} =0.7×166.67=116.67m^3/min=116.67/60=1.944$m^{3/}$sec

The flowrate received by one trough=1.944/4=0.486 $m^{3/}$sec

$$\boldsymbol{y = 1.73}\sqrt[3]{\frac{0.486^2}{9.81\times0.6^2}} = 0.7m$$

Use free board of 50mm

Total depth of trough= 0.7+0.05=0.75m

Design of underdrain system (perforated-pipe system)

Let $\frac{L_L}{D_L}$=50

L_L= 9.13/2 =4.57m

D_L =4.57/50 =0.091m $\longrightarrow$ use commercial dia. of 0.1m (4in)

Let dia. of perforations=12.5mm

$A_P=\frac{0.0125^2\pi}{4}$

Spacing of perforations= 200mm

Number of perforations in one lateral (N_P) = 4.57/0.2=22.85

Use 23 perforations in one lateral

Let spacing of laterals=250mm=0.25m

Number of laterals (N_L) = $2\times\frac{18.26}{0.25} = 146.08 \quad use\ 146\ (73\ in\ each\ side\ of\ manifold)$

Check $\frac{\sum A_{perforations}}{\sum A_{laterals}} = \frac{146\times23\times\frac{0.0125^2\pi}{4}}{146\times\frac{0.1^2\pi}{4}} = \frac{0.412}{1.15} = 0.36 < 0.5 \ldots\ldots O.K.$

Check $\frac{\sum A_{perforations}}{Aera\ of\ filter} = \frac{0.412}{166.67} = 0.00247 > 0.002 \ldots\ldots O.K.$

Determination of unit depth

Unit depth=(sand thickness + gravel thickness +water depth +lateral dia.)×1.1

Let thickness of sand layer=600mm=0.6m

Let thickness of gravel layer=600mm=0.6m

Let water depth=1.5m

Unit depth=(0.6 + 0.6 +1.5 +0.1)×1.1= 3.08m

Question

A water treatment plant has a capacity of 24000m^3/day. The filtration system includes pressure filters each has a diameter of 4m. The filter medium is anthracite has effective size of 0.8mm and uniformity coefficient of 1.5. (a) Find the number of units assuming filtration rate when all units are in operation is 200m^3/m^2.day. (b) Check filtration rate when one unit being backwashed. (c) Design the wash water collection system using troughs of semi-circular shape.

6-2-5 Disinfection Unit

Water used for drinking and cooking should be free of pathogenic (disease causing) microorganisms that cause such illnesses as typhoid fever, dysentery, and cholera. Whether a person contracts these diseases from water depends on the type of pathogen, the number of organisms in the water (density), the strength of the organism, the volume of water ingested, and the susceptibility of the individual. Purification of drinking water containing pathogenic microorganisms requires specific treatment called disinfection.

Disinfection reduces pathogenic microorganisms in water to levels designated safe by public health standards. This prevents the transmission of disease. An effective disinfection system kills or neutralizes all pathogens in the water. It is automatic, simply maintained, safe, and inexpensive. An ideal system treats all the water and provides residual (long term) disinfection. Chemicals should be easily stored and not make the water unpalatable.

6-2-5-1 Water Test for biological quality

The biological quality of drinking water is determined by tests for coliform group bacteria. These organisms are found in warm-blooded animals and in soil.

Their presence in water indicates pathogenic contamination, but they are not considered to be pathogens. The standard for coliform bacteria in drinking water is "less than 1 coliform colony per 100 millilitres of sample" (< 1/ 100ml). Water systems are required to test regularly for coliform bacteria.

Coliform presence in a water sample does not necessarily mean that the water is hazardous to drink. A positive result (more than 1 colony per 100 ml water sample) means the water should be retested. The retested sample should be analysed for fecal coliform organisms. A high positive test result, however, indicates substantial contamination requiring prompt action. Such water should not be consumed until the source of contamination is determined and the water purified.

6-2-5-2 Methods of Water Disinfection

Water can be disinfected using; (1) chlorination process, (2) ozonation process, and (3) ultraviolet (UV) radiation.

1. <u>Chlorination of Water</u>

Chlorination is the most commonly used method for water disinfection. It is effective against many pathogenic bacteria, but at normal dosage rates it does not kill all viruses and worms. Chlorine is most often available commercially as chlorine gas cylinders, as sodium hypochlorite (household bleach) and as calcium hypochlorite. Chlorine gas is most often employed in large water treatment plants because of its lower cost; however, chlorine gas is difficult to handle since it is toxic, heavy, corrosive, and an irritant. At high concentrations, chlorine gas can even be fatal.

Chlorine readily combines with chemicals dissolved in water, microorganisms, plant material, tastes, odors, and colors. These components "use up" chlorine and comprise the **chlorine demand** of the treatment system. It is important to add sufficient chlorine to the water to meet the chlorine demand and provide residual disinfection.

Reactions of Chlorine

At the same time that chlorine is being used up by compounds in the water, some of the chlorine reacts with the water itself. The reaction depends on what type of chlorine is added to the water as well as on the pH of the water itself. When chlorine gas enters the water, it reacts with water and breaks down into **hypochlorous acid** (HOCl) and hydrochloric acid (HCl).

$Cl_2 + H_2O \leftrightarrow HOCl + HCl$

Hypochlorous acid may further break down, depending on pH:

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

Hypochlorous acid may break down into a hydrogen ion and a hypochlorite ion, or a hydrogen ion and a hypochlorite ion may join together to form hypochlorous acid. The concentration of hypochlorous acid and hypochlorite ions in chlorinated water will depend on the water's pH. A higher pH facilitates the formation of more hypochlorite ions and results in less hypochlorous acid in the water. This is an important reaction to understand because hypochlorous acid is the most effective form of **free chlorine residual**, meaning that it is chlorine available to kill microorganisms in the water. Hypochlorite ions are much less efficient disinfectants. So disinfection is more efficient at a low pH (with large quantities of hypochlorous acid in the water) than at a high pH (with large quantities of hypochlorite ions in the water.)

The chlorine that does not combine with other components in the water (like ammonia or organic nitrogen) is **free** (residual) chlorine. An ideal system supplies free chlorine at a concentration of 0.3-0.5 mg/l.

Breakpoint chlorination

The breakpoint is the point at which the chlorine demand has been totally satisfied; the chlorine has reacted with all reducing agents, organics, and ammonia in the water. When more chlorine is added past the breakpoint, the chlorine reacts with water and forms hypochlorous acid in direct proportion to the amount of chlorine added. This process, known as breakpoint chlorination, is the most common form of chlorination, in which enough chlorine is added to the water to bring it past the breakpoint and to create some free chlorine residual.

The dosage can produce breakpoint chlorination is determined by drawing chlorine residual curve (or chlorine demand curve). This curve represents the relationship between chlorine dosage applied and chlorine residual, Fig.23.

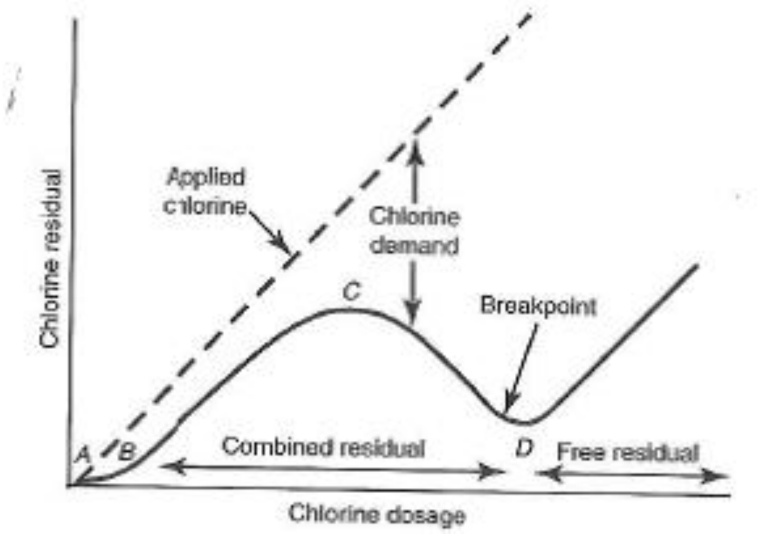


Fig.23 Chlorine residual curve

Hypochlorites

Instead of using chlorine gas, some plants apply chlorine to water as a **hypochlorite**, also known as a **bleach**. Hypochlorites are less pure than chlorine gas, which means that they are also less dangerous. However, they have the major disadvantage that they decompose in strength over time while in storage. Temperature, light, and physical energy can all break down hypochlorites before they are able to react with pathogens in water. There are two main types of hypochlorites; sodium hypochlorite and calcium hypochlorite.

- **Sodium hypochlorite** ($NaOCl$) comes in a liquid form which contains up to 12% chlorine.
- **Calcium hypochlorite** ($Ca(OCl)_2$), is a solid which is mixed with water to form a hypochlorite solution. Calcium hypochlorite is 65-70% concentrated.

Hypochlorites work in the same general manner as chlorine gas. They react with water and form the disinfectant hypochlorous acid. The reactions of sodium hypochlorite and calcium hypochlorite with water are shown below:

$$Ca(OCl)_2 + 2\ H_2O \leftrightarrow 2\ HOCl + Ca(OH)_2$$

$$NaOCl + H_2O \leftrightarrow HOCl + NaOH$$

In general, disinfection using chlorine gas and hypochlorites occurs in the same manner. The differences lie in how the chlorine is fed into the water and on handling and storage of the chlorine compounds. In addition, the amount of each type of chlorine added to water will vary since each compound has a different concentration of chlorine.

Chloramines

Some plants use **chloramines** rather than hypochlorous acid to disinfect the water. To produce chloramines, first chlorine gas or hypochlorite is added to the water to produce hypochlorous acid. Then ammonia is added to the water to react with the hypochlorous acid and produce a chloramine. Three types of chloramines can be formed in water; monochloramine, dichloramine, and trichloramine. Monochloramine is formed from the reaction of hypochlorous acid with ammonia:

$NH_3 + HOCl \leftrightarrow NH_2Cl + H_2O$

Monochloramine may then react with more hypochlorous acid to form a dichloramine:

$NH_2Cl + HOCl \leftrightarrow NHCl_2 + H_2O$

Finally, the dichloramine may react with hypochlorous acid to form a trichloramine:

$NHCl_2 + HOCl \leftrightarrow NCl_3 + H_2O$

Chloramines are used as a secondary disinfectant in water distribution system after primary disinfection is achieved by the use of free chlorine, ozon or UV in treatment plant. The concentration of chloramine required will depend on size of distribution system and the decay rate of the residual. The maximum chloramine concentration allowed is 4mg/l.

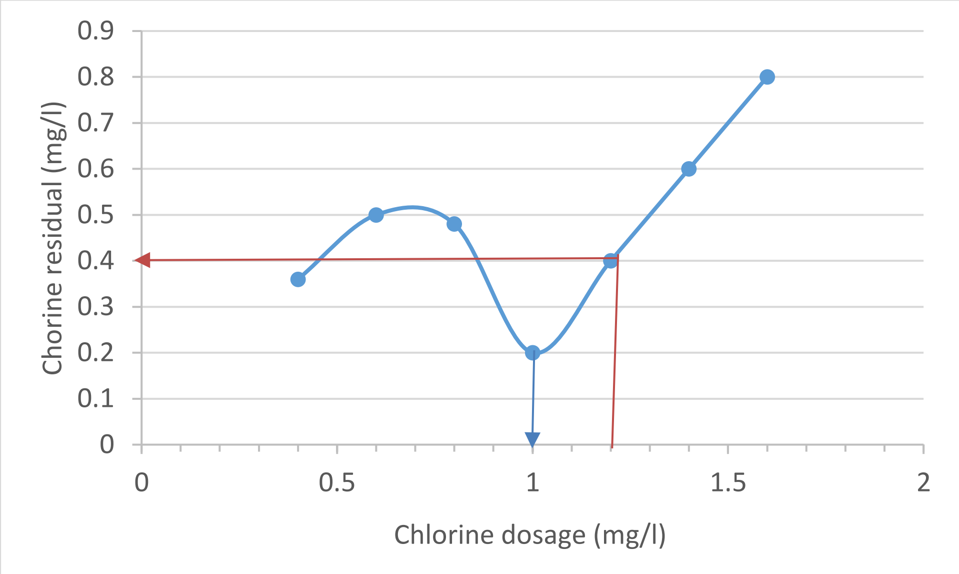
Example 6.12

Use the data given below to find breakpoint dosage and chlorine demand at a dosage of 1.2.

Sample No.	Chlorine dosage (mg/l)	Residual chlorine (mg/l)
1	0.2	0.19
2	0.4	0.36
3	0.6	0.50
4	0.8	0.48
5	1.0	0.20
6	1.2	0.40
7	1.4	0.60
8	1.6	0.80

Solution

Draw the relation between chlorine dosage (x-axis) and chlorine residual (y-axis).



From the graph;

Break point dosage=1 mg/l

At chlorine dosage of 1.2mg/l, from the graph;

Chlorine residual= 0.4mg/l

Chlorine demand=1.2-0.4=0.8 mg/l

Contact time with microorganisms

The **contact (retention) time** (Table 1) in chlorination is that period between introduction of the disinfectant and when the water is used. A long interaction between chlorine and the microorganisms results in an effective disinfection process. Contact time varies with chlorine concentration, the type of pathogens present, pH, and temperature of the water.

Contact time must increase under conditions of low water temperature or high pH (alkalinity). Complete mixing of chlorine and water is necessary, and often a holding tank is needed to achieve appropriate contact time. An alternative to the holding tank is a long length of a pipe to increase contact between water and chlorine.

Table (1) is used for calculating the contact time using the highest pH and lowest water temperature expected for the treated water. For example, if the highest pH anticipated is 7.5 and the lowest water temperature is 42 °F, the "K" value (from Table 1) to use in the formula is 15. Therefore, a chlorine residual of 0.5 mg/l necessitates 30 minutes contact time. A residual of 0.3 mg/l requires 50 minutes contact time for adequate disinfection.

Table 1. Calculating Contact Time

minutes required = K / chlorine residual (mg/l)			
Highest	*Lowest Water Temperature (degrees F)*		
pH	*> 50*	45	< 40
6.5	4	5	6
7.0	8	10	12
7.5	12	15	18
8.0	16	20	24
8.5	20	25	30
9.0	24	30	36

Example 6.13

Pipeline transports chlorinated water from a water treatment plant at flow rate of 756 liter/min. The pipeline diameter is 305mm. Find the pipeline length between chlorine injection point and the first consumer, if the required free residual chlorine concentration is 0.1 mg/l and the water pH and temperature values vary over the ranges (6.6-9.0) and (53.6-78.8)°F, respectively.

Chlorination levels

If a system does not allow adequate contact time with normal dosages of chlorine, superchlorination followed by dechlorination (chlorine removal) may be necessary. **Superchlorination** provides a chlorine residual of 3.0-5.0 mg/l, 10 times the recommended minimum breakpoint chlorine concentration. Retention time for superchlorination is approximately 5 minutes. Activated carbon filtration removes the high chlorine residual. **Shock chlorination** is recommended whenever a water system is new, repaired, or found to be contaminated. This treatment introduces high levels of chlorine to the water. Unlike superchlorination, shock chlorination is a "one time only" occurrence, and chlorine is depleted as water flows through the system; activated carbon treatment is not required. If bacteriological problems persist following shock chlorination, the system should be evaluated.

2. Water Ozonation

Ozone is an unstable gas that can destroy bacteria and viruses. It is formed when oxygen molecules (O_2) collide with oxygen atoms to produce ozone (O_3). Ozone is generated by an electrical discharge through dry air or pure oxygen and is generated onsite because it decomposes to elemental oxygen in a short amount of time. After generation, ozone is fed into a contact tank containing the water to be disinfected. From the bottom of the contact tank, ozone is diffused into fine bubbles that mix with the water, see Figs. 24 and 25.

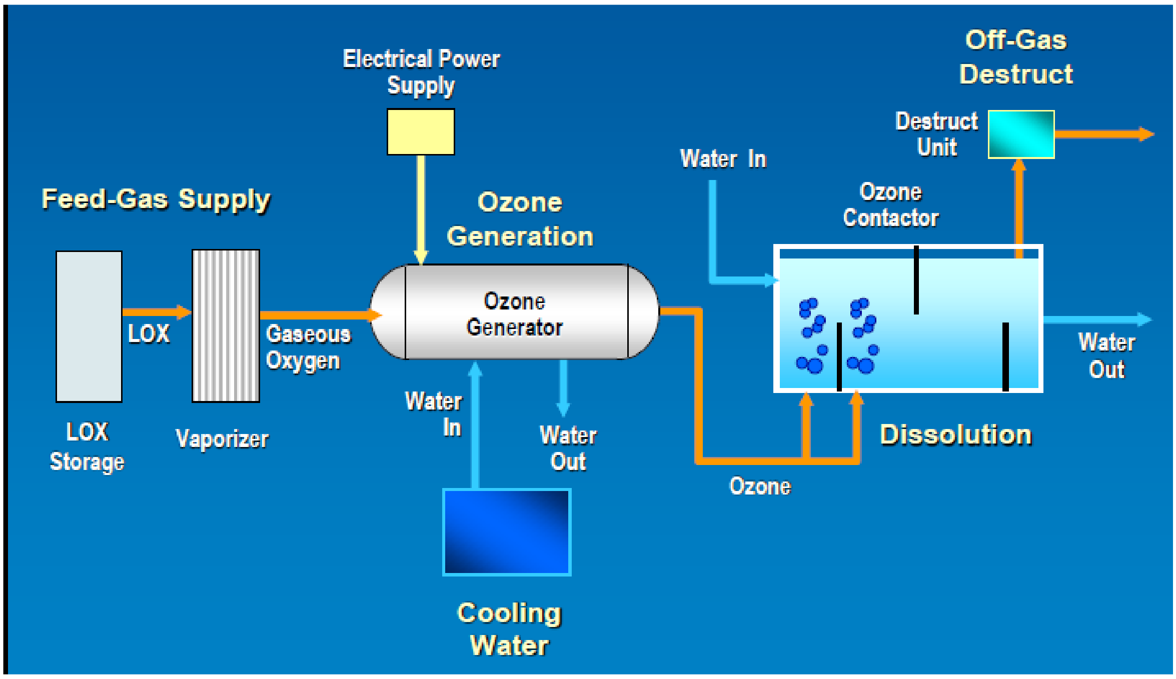


Fig.24 Ozone system component

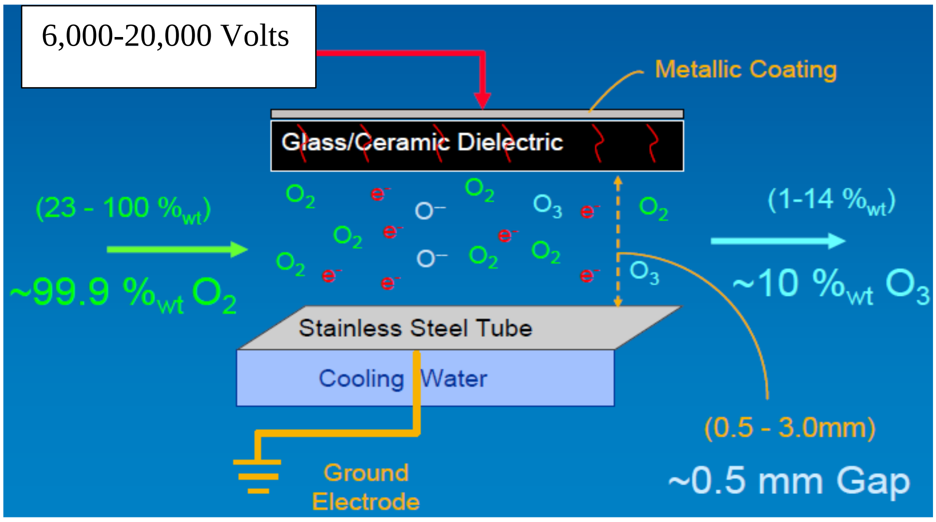


Fig.25 Ozone generator

Advantages of ozone

- Ozone is more effective than chlorine in destroying viruses and bacteria.
- The water needs to be in contact with ozone for just a short time (approximately 10 to 30 minutes).

• Ozone decomposes rapidly, and therefore, it leaves no harmful residual that would need to be removed from the water after treatment.

• Ozone is generated onsite, and thus, there are fewer safety problems associated with shipping and handling.

Disadvantages of ozone

• Low dosages may not effectively inactivate some viruses, spores, and cysts.

• Ozone is very reactive and corrosive, thus requiring corrosion-resistant material, such as stainless steel.

• Ozone is extremely irritating and possibly toxic, so off-gases from the contactor must be destroyed to prevent worker exposure.

• The cost of treatment is relatively high, being both capital- and power-intensive.

• There is no measurable residual to indicate the efficacy of ozone disinfection.

3. UV Disinfection

UV is invisible light radiation with a wavelength between 200-300 nanometres, Fig.26. Unlike chemical approaches to water disinfection, UV provides rapid, effective inactivation of microorganisms through a physical process. When bacteria, viruses and protozoa are exposed to the germicidal wavelengths of UV light, they became incapable of reproducing and infecting. UV light has demonstrated efficacy against pathogenic organisms, including those responsible for cholera, typhoid, hepatitis and other bacterial, viral and parasitic diseases.

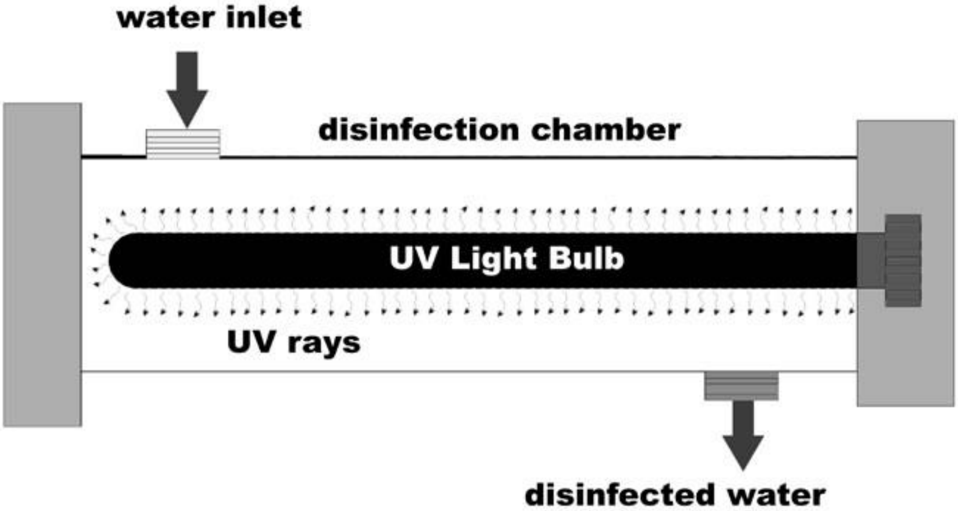


Fig.26 UV disinfection system

Advantages of UV

- Cheap and effective disinfection.
- Chemical free.
- Relatively simple to install, operate and maintain.
- Inactivates protozoa.
- Compact footprint.
- Minimal concerns over by-products.

Disadvantages of UV

- No disinfectant residual.
- It requires water to have low levels of colour and turbidity.
- It is ineffective if the dose and contact time are not correct.