Bari Sir

Keya Madam

Problem

Environmental Engineering-1

CE3141

Lecture -1

Cycle-2, C-day

07-02-2018

Water Collection and Transportation

- **Collection of water** is the drawing of water from the sources of water, commonly known as intakes.
- The **collection system** is a set of engineering works designed to convey water from a source to a distribution system via treatment plant and includes intakes, suction pipes, delivery pipes and pumping stations.
- **Transportation of water** is the conveyance of water from the intake to the treatment plant and also to the distribution systems.

What is Intake?

The intake is a device placed in a surface water source to permit the withdrawal of water from the source and then discharge it into intake pipe through which it will flow into the water-works system.

It consist of the opening, strainer, or grating through which the water enters, and the conduit conveying the water, usually by gravity, to a well or sump.

From the well the water is pumped to the mains or treatment plant.

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

Considerations for Designing and Locating Intake

The following must be considered in designing and locating intakes:

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

Intake Velocity and Depth

Intake entrance should lie 3 to5 m below the water surface but 1 to 2 m above the river, lake or reservoir floor to keep bottom sediments out of intakes.

The entrance velocities are kept down to 7.6 to 10 m/sec. At such low velocities, vegetation, debris and other materials are not entrained in the flowing water, fish and other aquatic lives are well able to escape from the intake current.

Gratings or screens of 1 to 3 meshes to a centimeter are provided at the intake entrance.

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Intake Pipe

- Intakes are connected to the banks of rivers or to the shores of lakes and reservoirs (i) by pipelines (often laid with flexible joints) or (ii) by tunnels blasted through rock beneath the floor.
- The pipelines are generally laid in a trench on the riverbank or on the lake or reservoir floor and covered after completion.
- Pipe passing through the foundation of dam are subjected to heavy loads and to stresses caused by consolidation of the foundation.
- Intake pipes are designed to operate at self-cleansing velocities, 14 to 19 cm/sec.
- Flow may be by gravity or by suction.

Pumping Station

- Pump wells are generally located on shore or banks.
- Suction lift including friction should not exceed 5 to 6 m accordingly.
- Pump wells are often quit deep.
- The determining factor is the elevation of water level in the river, lake or reservoir in times of drought.

Design Considerations

Following are the important considerations for design of an intake:

i. Selection of a particular type for the given source.

ii. The magnitude of the external forces (waves, currents and blows from floating and submerged objects) to be resisted by the intake.

iii. Consideration of the total lift from the source to the treatment plant and selection of a suitable pumping unit.

iv. Determination of the total length of suction and delivery mains, head losses due to friction and small bends, enlargement and reduction.

Design Considerations

v. Selection of a suitable screen to provide around the intake pipe not to permit entry of large and small objects, such as logs, stones, aquatic lives and vegetation.

vi. Installation of intake valves or porthole at 2 or 3 different levels to get the best available quantity of water, eliminating seasonal fluctuation of waster levels.

vii. Determination of cost-benefit ratio. To reduce the cost, the intake elevation is often made higher so that the water flows to the treatment plant by gravity.

viii. Assurance of the safety of the intake structure, provision future extension and installation of standby unit of pumps.

Types of Intakes

The intakes are mainly of the following four types:

(i) Canal intakes

(ii) Reservoir or lake intakes

(iii) River intakes and

(iv) Portable intakes.

Canal Intakes

An intake chamber is constructed in the canal section. This results in the reduction of waterway which increases the velocity of flow. It therefore becomes necessary to provide pitching on the downstream and upstream portions of canal near intake.

The entry of water in the intake chamber takes through the coarse screen and the top of outlet pipe is provided with fine screen. The inlet to outlet pipe is of bell-mouth shape with perforations of fine screen on its surface.

The outlet valve is operated from the top and it controls the entry of water into the outlet pipe from where it is taken to the treatment plant.

Canal Intakes

As the water level in the canal section practically remains constant, it is not necessary to provide intake pipe at various levels. To reach up to the bottom of intake, the steps should be provided in zigzag manner starting from manhole.

The flow velocity through the outlet pipe is usually kept as about 1.50 m/sec and it helps in determining the area and diameter of the withdrawal conduit.

For designing the area of coarse screen, the flow velocity is limited to as low as about 150 mm/sec or so. The flow velocity through the bell-mouth shaped inlet is limited to about 300 mm/sec or so.

Lake and Reservoir Intake

Lake intakes are sited with due reference to sources of pollution, prevailing winds and surface currents.

Reservoir intakes resemble lake intakes but generally lie closer to bank in the deepest part of the reservoir. They are often incorporated into the impounding structure itself.

Portable Intakes

In case of emergencies such as festival sites, wars, etc., it sometimes becomes necessary to draw water with the help of a movable intake.

It essentially consists of a truck fitted with a pumping plant. The truck is brought to the site and it is placed in such a position that it becomes possible to immerse the suction pipe of the pump. The end of the suction pipe is kept just above the bed level of water source and it is also provided with screen or strainer.

Thus the water lifted by the portable intake is relatively free from suspended solids. The water is then conveyed through the discharge pipe of the pumping plant.

River Intake

Understandably, river intakes are constructed well upstream from points of discharge of sewage and industrial wastes.

Optional location will take advantages of deep water, a stable bottom, and favorable water quality, all with proper reference to protection against floods, debris and river traffic.

Where the river bed shifts or depth of flow varies greatly, intake pumps may be mounted on carriage that are moved up and down on the river bank to stay within desired suction lift as flows rise and fall.

River Intake

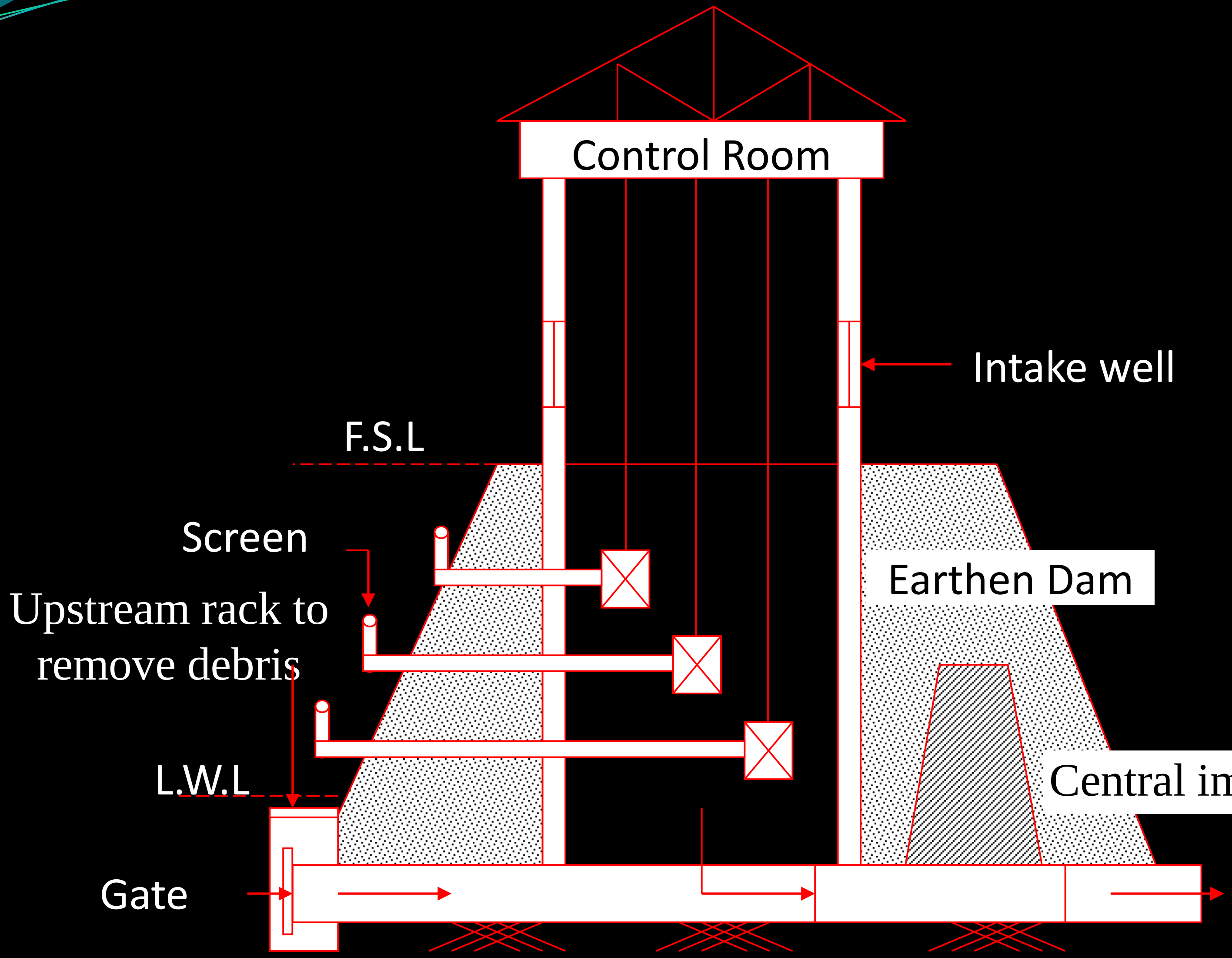

Figure: Intake well for earthen dam

Transportation of Water

The arrangement for the transportation of water from the source of supply to the treatment plants and subsequent distribution to the consumers form an important part of the water-works system.

The source of supply usually being at some distance away from towns and cities, it is necessary to construct structures for the transportation of water.

These structures are known as **pipes** and **conduits**.

There are two general classes of pipes; (i) **Pressure pipes** in which the water flows under hydraulic pressure, and (ii) **Gravity pipes** (open channels) in which the water flows in gravity.

Transportation of Water

- A pressure pipe is also defined as a pipe flowing full.
- Such pipes are often less costly than open channels (canals and flumes) because they can generally follow a shorter route.
- If water is scare, pressure pipes may be used to avoid loss of water by seepage and evaporation, which generally occurs in open channels.
- Pressure pipes are preferable for public water supplies because of the reduced opportunity for pollution.
- The open channel may take the form of canal, flume, tunnel, aqueduct or partly filled pipe.
- Open channels are characterized by a free water surface, in contrast to pressure pipes, which always flow full.

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17-02-2018

Desirable Qualities of Pressure Pipes

The desirable qualities of pressure pipes are as follows:

i. They should be made of durable materials so that no leakage develops causing wastage of water.

ii. They should be strong and of sufficient thickness to withstand both internal and external stresses.

iii. The inner surface of the pipe should be very smooth so that the resistance to flow is minimum.

iv. The pipe materials should not impart any physical and chemical effects to water.

Desirable Qualities of Pressure Pipes

The desirable qualities of pressure pipes are as follows:

v. The pipes should be light so that transporting, handling and laying the pipe under different conditions of topography, geology and communication become easier.

vi. Low initial cost and maximum service period of pipes are desirable.

vii. The pipe materials should be so selected that annual maintenance cost is low, joints can be made easily, offer adequate resistance to the corrosive characteristics of soil and water and highly skilled labour is not required for their laying and construction.

viii. The pipe sections should possess good hydraulic properties.

Corrosion of Metal Pipe

Generally pipes are made by various materials like steel, cast iron, concrete, asbestos cement, vitrified-clay, PVC, GI, copper, wrought iron, plastic, asphaltic fiber and lead. But metal pipes are subject to corrosion.

Corrosion is a phenomenon by which metals and their alloys are attacked by the environment consisting of chemicals.

There are mainly two types of corrosion in pressure water pipes, (i) External corrosion and (ii) Internal corrosion.

The external agents like biological action, oxygen, etc. cause external corrosion, and the internal corrosion is generally attributed primarily to the nature of water, which flows through pipes.

Corrosion of Metal Pipe

The chemical attacks of an environment upon a result in the oxidation of the metals and the formation of corrosive products, usually the oxides, hydroxides, carbonates, sulphides, etc.

In most cases, corrosion product is insoluble in the environment and forms a separate phase on or adjacent to the metal.

Hence, corrosion may be defined simply as the process by which the metals and their alloys are destroyed by chemical or electrochemical means.

Causes of Corrosion of Metal Pressure Pipes

Following are the important causes of corrosion of metal pressure pipes:

i. **Pitting:** Localized pitting is usually caused in metal pipes by the concentration of electric currents resulting from the potential differences on the metal surface, which accelerates. This process is also accelerated by dissolved oxygen content of flowing water.

ii. **Influence of acids and alkalies:** Acidity and alkalinity of water passing through pipes will help vigorously to corrode pipes.

iii. **Influence of sulphurous compounds:** The influence of sulphurous compounds on metal pipes is harmful. It has been reported that the presence of sulphides particles raised the proportions causing rust from 22 to 90%. The effect of sulphides is almost due to liberation of hydrogen sulphides which accelerates the attack of acids on the pipe metal.

Causes of Corrosion of Metal Pressure Pipes

iv. **Biological action:** Soil contains various types of bacteria both aerobic and anaerobic. Certain anaerobic bacteria are capable of rendering the oxygen present in sulphates, nitrates and carbonates available for the free oxygen and thereby corrosion will proceed pace. The most important sulphate reducing bacteria (*Vibrio desulphuricans*) which can cause serious attack on buried pipes when three conditions are satisfied:

(a) absence of oxygen as in many clayey soil,

(b) presence of proper food (organic matter) and other environmental conditions needed for the growth of bacteria and

(c) presence of large amount of sulphates.

Causes of Corrosion of Metal Pressure Pipes

v. **Cavitation:** The effects of cavitations are similar to those of corrosion but are due more to erosion. The sudden and alternate making and breaking of high vacuum and the creation and condensation of water vapour cause a bombardment of the surrounding surfaces with particles of water and water vapour moving at a high velocity thereby accelerating corrosion.

vi. **Temperature:** The increase in temperature accelerates the rate of corrosion. The rate of corrosion in water pipes may be increased in three or four fold by raising the temperature from 60 to 150° F.

vii. **Velocity of flowing water:** As the velocity of the water in the pipe increases from linear to turbulence type, the rate of corrosion is sharply increased.

Effect of Corrosion

- Corrosion of water pipes causes a great economic loss. Both direct and indirect losses resulting from corrosion are vast and undesirable.
- Replacing a corroded leaky water main by the roadside is very difficult and costly.
- Corrosion greatly reduces the pressure head and results in increases cost of pumping and short life of the water mains.
- Leakage in domestic plumbing fixtures due to corrosion involves not only the replacement but also repairing damages to walls, floors, etc.
- Rusty water due to corrosion causes strain in cloth after washing, produces unsightly marks on the plumbing fixtures and unsuitable for domestic uses.

Control of Corrosion

- Corrosion of metal pipes may be reduced or eliminated by protection coatings of paint, galvanizing, bituminous compounds, or cement linings.
- Red lead paint or zinc pigments offer some protection and are used on the exterior or exposed metal pipes.
- Other metallic protective coatings are tin coatings, nickel coatings, chromium coatings and copper coatings.
- Galvanizing by dipping the pipe molten zinc is an effective corrosion control except for highly acid waters. Galvanized pipe is widely used for small service lines in distribution systems but is too expensive for large pipe.
- Large pipes are usually protected by non-metallic coatings, such as bituminous coatings or cement linings. Numerous commercial bituminous compounds are available for both hot and cold application.

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09-05-2017

Scale Formation in Pressure Pipes

- Scale formation in water pipes is mainly due to the presence of dissolved mineral matter and gases under favorable conditions of temperature and pressure.
- Scale formation caused water pipes to wear out and burst out very soon as the cross-sections of the pipes are reduced and this also causes insufficient discharge through pipes.
- Scaling also causes water unfit for domestic and industrial uses.

Scale Formation in Pressure Pipes

- The impurities which are mainly responsible for scale formation in water pipes may be classified under two heads: (i) Dissolved mineral matter and (ii) Dissolved gases.
- Dissolved mineral matter include the hardness producing substances, i.e., carbonates, bicarbonates, sulphates and chlorides of calcium and magnesium, and silica.
- Dissolved gases include carbon dioxide, oxygen, nitrogen, hydrogen sulphides and methane.

Control of Scale Formation

- To control scale formation in pressure pipes, water is softened.
- The chief objective of water softening is to remove dissolved mineral compounds which constitute the hardness and which deposit scales in water pipes, boilers and hot water heating system cause serious difficulties in many processes including textile finishing, dying, canning, paper making, cold drinks preparation, tanning and others.
- Following are the effective processes by which scale forming minerals and gases are removed from water: (i) Lime-soda process, (ii) Zeolite process, (iii) Phosphate process and (iv) Lime process.

Forces Acting on Pipes

Pipes carrying water under pressure must be designed to withstand stresses caused by internal and external loads, and temperature changes, and to satisfy the structural and hydraulic requirements.

The forces acting on pipes are:

a. Internal forces due to static head
b. Internal forces due to water hammer
c. Forces at bends and changes in cross-section
d. Forces due to temperature changes
e. External forces in the form of backfill, traffic and own weights.

Internal Forces due to Static Head

Internal forces due to static head create hoof stress (transverse stress or circumferential stress) and longitudinal stress.

Hoof stress, $S_h = pd/2t$

Where, S_h = hoof stress per linear length in inch of the pipe.

P = intensity of static pressure in psi = wh, in which h is the static head and w is the unit weight of water.

d = pipe diameter in inch.

t = thickness of the pipe shell in inch.

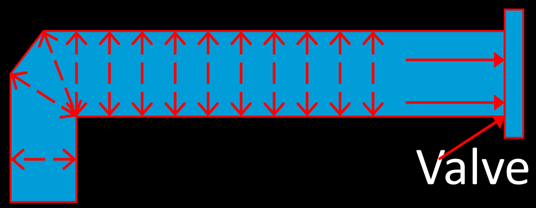
Longitudinal (tensile) stress, $S_t = pd^2/[4t(d+t)] = pd/4t$ (approximately)

Internal Forces due to Water Hammer

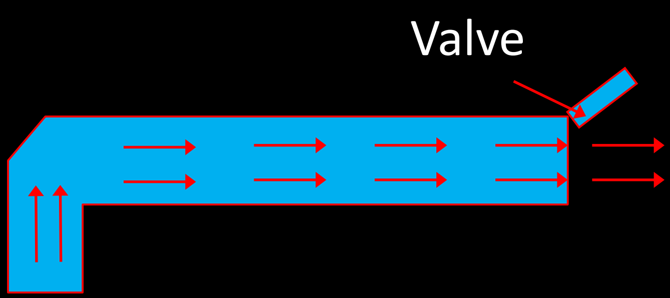
One of the most damaging factors to a water piping system is water hammer action. In addition to its effect on the piping system, water hammer causes banging noises in the system that is very disagreeable to occupants in the building. Water hammer occurs when a column of water flowing through a pipe line and discharging at an open outlet, is suddenly stopped by closing the outlet. Since flowing water has force, tremendous pressure result at the point of closure and pressure surges move along the pipe.

Internal Forces due to Water Hammer

Phase 1: The valve on the line is closed and the water contained in the line is at rest. Water pressure is thus exerted in all directions in an equal way.

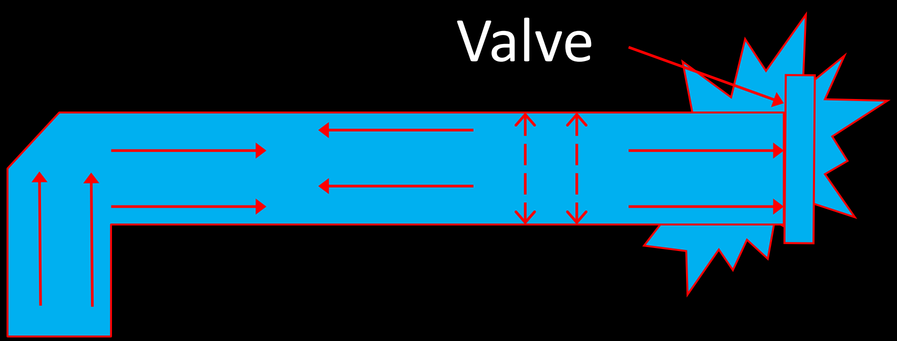


Phase 2: The valve on the line is open and water flow freely through the open outlet. Now the water pressure is utilized to force the water out of the open end of the pipe. Arrows indicate the direction of force in the column of water.



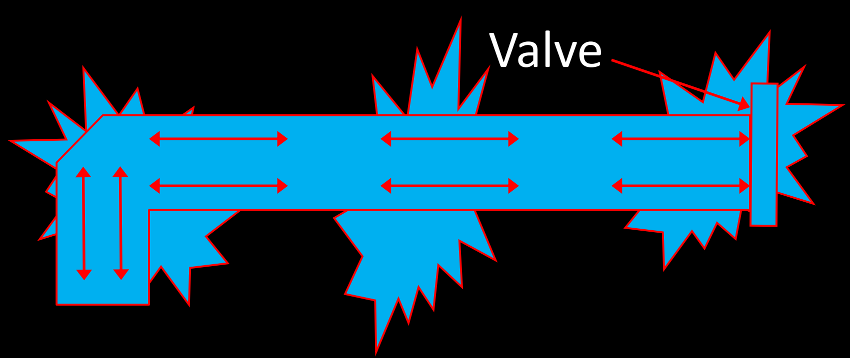
Internal Forces due to Water Hammer

Phase 3: When the valve is quickly closed, the column of freely flowing water is suddenly stopped; excessively high pressures are generated at the point of stoppage. This creation is the same as would result of a steel bar moving through the line at the velocity of water were suddenly stopped by the valve.



Internal Forces due to Water Hammer

Phase 4: In an effort to equalize the pressure build-up of the water, a shock wave will travel back along the branch line until a large diameter pipe is reached. This will allow the shock wave to dissipate itself. Arrows denote the direction of force towards the valve and then its reversal as a shock wave towards the point of relief. Since the shock wave travels at speeds in excess of 4000 fps, it causes a piping clatter all along its route. Often the shock wave will oscillate back and forth between the valve and the point of relief until the pressure is stabilized with the branch line.



Internal Forces due to Water Hammer

The pressure generated by the shock wave can expand and often rupture the piping. Although piping clatter is normally associated with water hammer, you cannot assume that when these noises do not occur, that the shock wave is non-existent. Quite often, water hammer takes place without any physical sounds. Therefore it is very important that piping system be designed with all due consideration given to the means that compensate for the action of water hammer.

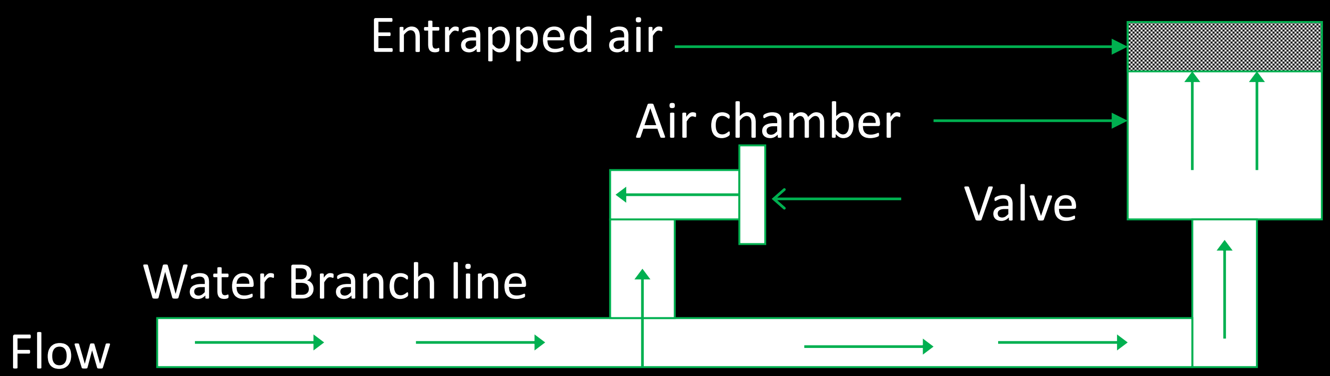
Methods of controlling water hammer

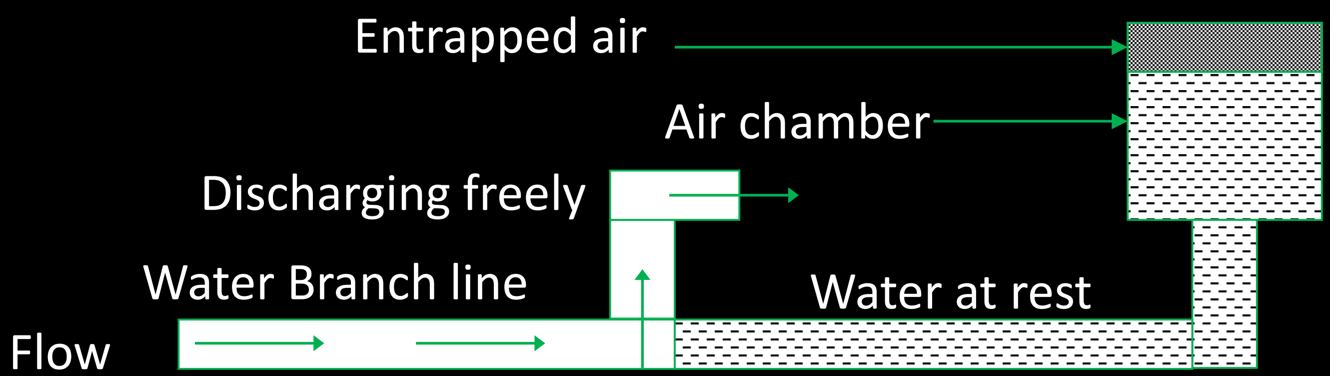
In order to eliminate the damage and piping clatter that results from water hammer, it is important that certain steps be taken in piping system design to compensate for the excessive pressures that are generated when a column of flowing water is suddenly stopped.

The consideration is needed in some means or device that will provide flexibility in the system to absorb the initial shock wave of water hammer thereby conflicting the action to a given section of piping.

Air is the most effective medium for absorbing the shock wave caused by water hammer for (i) water is non-compressible; (ii) air can be compressed to considerable pressure when the water compresses the air, it also fills the void offered by the displaced air. Because water has this flexible means to expend its force, the shock wave that would otherwise result, is quickly absorbed.

Methods of controlling water hammer





Forces at Bends and Changes in Cross-section

A change in direction magnitude of flow velocity is accompanied by a change in the momentum of water. The force required to produce this change in momentum covers from pressure variation within the water and from the forces transmitted to the water from the pipe walls. For a pipe bend of the uniform section,

Longitudinal force = $s(\pi d)t$

Where, *s* is the unit stress, *d* is the diameter of the pipe and t the wall thickness of the pipe.

Similar expression for forces developed for a pipe bend of non-uniform cross-section, pipe contraction and enlargement can be computed. These stresses can be eliminated or reduced by providing an efficient anchorage at the bend, contraction and enlargement.

Forces due to Temperature Changes

Longitudinal stress of considerable magnitude may develop in pipes exposed to large changes in temperature. The change in length δ of a pipe length **L** when subjected to a temperature change ΔT is,

$$\delta = \alpha L.\ \Delta T$$

where α is the coefficient of thermal expansion of the pipe material. If this change in length is prevented, longitudinal stress will develop. From the principles of mechanics of materials it is known that in the elastic range, $\delta = E\varepsilon = E\delta/L$

where ε is the unit strain (elongation per unit length) and **E** is the thermal modulus of elasticity and δ is the resulting unit stress. Combining these two equations, it gives,

$$\alpha = E\alpha\Delta T$$

This indicates the longitudinal stress that would result when a pipe with fixed ends is subjected to a temperature change. Expansion joints are usually provided to reduce temperature stresses.

External Forces

Pipes are often placed in an excavated trench which is back filled, or they are laid on ground surface and covered with earth. In either case a vertical load is imposed on the pipe. If a load is superimposed on the fill, a portion of it will be transferred to the buried pipe. The magnitude of the load thus produced depends on the rigidity of the pipe, the type of bedding and the character of the fill material.

1. According to Anson Marston, for rigid pipes in narrow trenches, the load **w** in pound per foot of pipe has been found to be,

$$w = c\gamma B^2$$

where B is the trench width at the top of the pipe, γ is the specific weight of the fill material, and c is the coefficient characteristics of the fill material, and the ratio of cover depth to the width of the trench, and is generally termed as load calculation coefficient.

External Forces

If the pipe is flexible type, and the soil at the sides is well compacted, the side fills may be expected to carry their proportional share of the total load. The empirical formula for the load on the buried flexible pipe in a narrow trench is,

$w = c\gamma BD$

where D is the out side diameter of the pipe.

If the pipe is placed on undisturbed ground and covered with fill (as highway culvert), the equation for the load on buried pipe under embankment conditions is,

$w = c_p \gamma D^2$

value of c_p depends on the type of the pipe and characteristics of the foundation and backfill.

External Forces

The formula for load due to super imposed concentrated load is given in the following form by D.H.Holl's integration of Bousainesq's formula as,

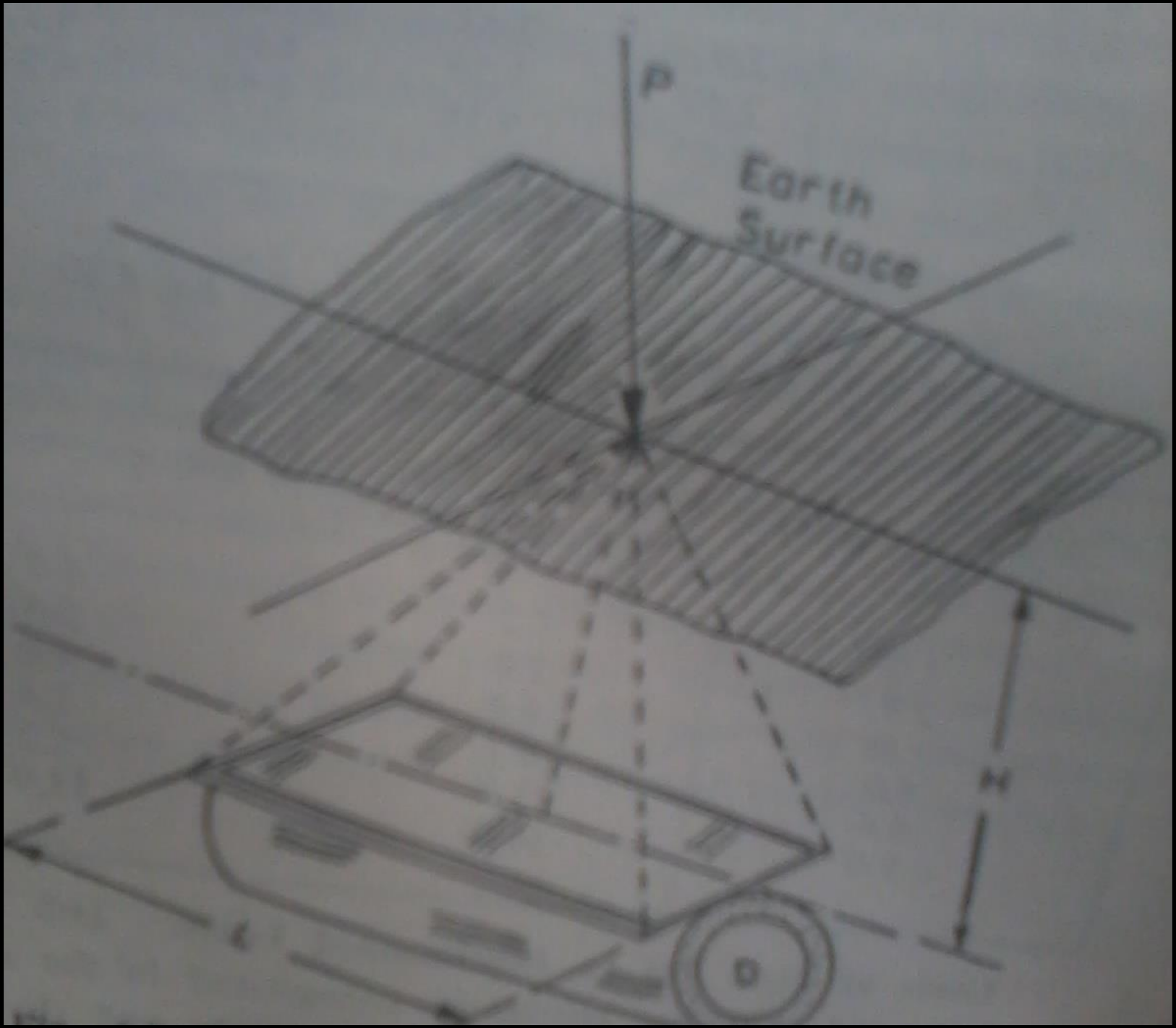
$w_{sc} = (c_1 p_{sc})\ I/L$

where w_{sd} is superimposed concentrated load on pipe in lb/ft, p_{sc} is the superimposed concentrated load in lb, I is the impact factor and c_1 is the load coefficient which is a function of $D/2H$ and $L/2H$. In case distributed superimposed load, the expression for load on the pipe is,

$w_{sd} = c_1 pID$

where w_{sd} is the superimposed distributed load on the pipe in lb/ft length, p is the intensity of distributed load in lb/sq.ft, c_1 is the load coefficient which is function of N/2H and M/2H.

External Forces



Environmental Engineering-1

CE3141

Lecture -4

Cycle-4, D-day

25-02-2018

WATER DISTRIBUTION SYSTEM

The distribution system is that part of the water-work which receives the water from the pumping station or from treatment plant and delivers it throughout the district to be served.

It includes, such as, reservoirs for purposes of storage, equalizing pressures and subsequent distribution, together with pipes, valves, hydrants and other appurtenance for carrying water, service pipe and meters etc.

Objectives of Distribution System

The main purposes of the construction of water distribution system are:

- To make water available in close proximity to the consumer;
- To supply water in adequate quantities according to the demand of the consumers;
- To supply water with adequate pressure;
- To regulate water supply as per requirement.

General consideration for water distribution system

The general considerations of the planning of distribution system should be observed in its design:

Circulation of water: The layout of distribution system should be such that there is free circulation of water and that the number of dead ends should be very few. Where dead ends are unavoidable, the hydrants will be provided to act as washouts.

Construction and design: The construction and design of water distribution system should be such that ample water is available at all times at desired pressure in all portion of the distribution system.

General consideration for water distribution system

The minimum residual pressures at ferrule point for direct supply to single-storeyed, two-storeyed and three-storeyed buildings should be respectively 7 m, 12 m and 17 m. If the available pressure in the pipe is low, it has to be boosted up with the help of fire engines in case of fire occurrence. In general, the construction and design of the water distribution system be such that it can be relied upon even for meeting future expansion programs.

Earth cushioning: The mains, which are laid under roads, should be provided with a minimum earth cushioning of 900 mm height from the top of mains. At other places, the cushioning may be of 750 mm height.

General consideration for water distribution system

Construction by sewage: The water pipe line should be laid above the sewers at a vertical distance of about 2 m and the horizontal distance between the water pipes and sewer should be at least 3 m. If the physical configuration of country does not permit the provision of this minimum spacing, extraordinary precautions should be taken to make the distribution system watertight to the maximum possible extent.

Fire demand: The distribution system should be so laid that the water for fire demand is available in required quantity at desired pressure at number of points along it.

General consideration for water distribution system

Economy: The layout and design of distribution system should be economical. The cost of distribution system forms a substantial part to extent of about 90% of the total cost of the water supply project. Hence the water distribution system should be carefully designed by taking into various factors such as pumping head, type of pipes, storage requirement, pipe diameter, etc.

Gradients: It is not necessary to lay mains at constant gradients. But the gradients of mains should in general follow the natural contour of ground. The gradient line should not rise above the hydraulic gradient line which means that at every points along the mains, there should be a positive pressure gradient than the atmospheric pressure.

General consideration for water distribution system

Leakage: The distribution system should be fairly watertight and the loss of water due to leakage should be brought down to the minimum possible extent.

Repairs: The distribution system should be so laid as to permit easy repairs. The broken or worn out parts of the equipments for various operations should be properly replaced.

Safety from pollution: The layout of distribution system should be such that it does not contribute to the pollution of water flowing in it.

Sanitation: The sanitation of area through which the distribution system is passing should be good so that there are no chances for water to be polluted during repairs or replacements of pipe lines.

Unsafe cross connection: The distribution system should not have any unsafe cross connection from which there are chances for contaminated water to enter it.

Methods of Distribution System

Depending upon the topography of the country, the distribution system may be classified as (i) Gravity system, (ii) System with direct pumping and (iii) System with pumping and storage.

Gravity system: A gravity system is adopted where the source of supply such as a lake or an impounding reservoir, is at a sufficient elevation with respect to the city in order to produce adequate pressure for fire and domestic service. This method, evidently, is the safest and most reliable. In this system, water is conveyed through pipe by gravity only. In case of fire, the motor pumps may be used to develop high pressures for fire fighting purpose.

Methods of Distribution System

System with direct pumping: In this system, water is directly pumped into the mains. Consumption is the only outlet. This method is least desirable, a failure in the power supply means breakdown of the system. Also, pressures in the mains vary with the consumption, so that under varying consumption, several pumps may be required to conform to the supply, adding to cost.

Methods of Distribution System

System with pumping and storage: This is also called the direct-indirect or dual system. In this, when the demand-rate exceeds the rate pumping, the flow into the distribution system is both from the pumping station as well as the elevated reservoir. When , however, the reverse condition exist i.e., pumping is more than the demand, the excess of water is stored in the reservoir. This system, obviously, is the most economical and reliable. It provides for a uniform rate of pumping. The pumps can be operated at their rated capacities, resulting in higher efficiency and economy of operation. Also, the water stored serves as a reserve to take care of fire demands and pump breakdowns.

Service Reservoir

The service or distribution reservoirs are generally provided in the distribution system to store the clean treated water before it is dispatched to the consumers. These reservoirs may be constructed of brick masonry, cement concrete – plain, reinforced or pre-stressed, steel or stonemasonry.

The common materials used are cement concrete and steel. According to the situation with respect to ground, the service reservoirs are classified in the following three types: (i) surface reservoirs, (ii) elevated reservoirs and (iii) stand pipes.

Purposes of Service Reservoir

Following are the purposes served by the service or distribution reservoir:

- If pumps are used, the provision of these reservoirs makes it possible to run pumps at uniform rate.
- In case of gravity system of supply, the provision of these reservoirs will result in mains of smaller diameters.
- They furnish the facility of storage of water for meeting fluctuating hourly demand of water.
- They maintain constant pressure in the mains. The pressure in mains, without service reservoirs, will fall as the demand of water will increase.
- They make the design and construction of treatment unit and distribution system economical.
- They serve as storage for emergencies such as breakdown of pumps, bursting of mains, heavy fire demand, interruption in power supply, temporary floods, etc.

Pressure Requirements in Distribution System

- In designing water distribution systems pressure requirements for ordinary use, and for fire fighting must be considered.
- In residential districts fires pressure of 60 psi at the hydrant are recommended.
- In commercial districts minimum pressure of 75 psi is tolerable, but higher pressures must be provided in districts with tall buildings.
- The maintenance of high pressure in mains means increased pumping cost and usually also increased leakage.
- Some large cities have installed dual systems in business districts, a low-pressure system for ordinary use and a high-pressure system (150 to 300 psi) for fire fighting only.

Pipe Systems

The pipe system comprises of the following four units:

The supply main: the supply main or main is the direct conveyor of water from the pumping plant or the gravity conduit. It should be of sufficient size to carry the flow.

The sub-mains: The sub-mains are the secondary feeders connected to either side may be placed at about 300 m apart and should be of sufficient size to discharge domestic supply and fire flow.

Minor distributors: The minor distributors or branches make up the grid iron of pipes and supply water to the fire hydrant and service pipes of the residences and other buildings. For fire service, minimum diameter of pipe should be 150 mm and for domestic service alone 100 mm and less.

Valves: Valves are needed to operate the control the pipe system. These should be sufficient in number and suitably located.

Environmental Engineering-1

CE3141

Lecture -5

Cycle-5, B-day

28-02-2018

Methods of Water Supply

Following are the two systems of supply of water which are based on the duration of supply:

Continuous Method:

In this system of supply, the water is supplied to the consumers for 24 hours of day. This is the most ideal system of supply and it should be adopted as far as possible.

The only disadvantage of this system is that considerable wastage of water occurs, if consumers do not possess civic sense regarding the importance of treated water. One recommended way to avoid this defect of the system is to supply water through meters. Whether it is desirable to install meters or not is a debatable question and hence the decision to install meters should be taken after careful considerations and deliberations.

Methods of Water Supply

Intermittent Method:

In this system of supply, the water is supplied during certain fixed hours of day only. The usual period is about 1 to 4 hours in the morning and about the same period in the afternoon. This system of supply of water proves to be useful for the following two conditions:

(a) the available pressure is poor and

(b) the quantity of water available is not sufficient to meet with various demands for water.

The working of the system is very simple. The distribution area is divided into several zones and the timings of each zone are so adjusted that good working pressures are maintained in each zone.

Continuous and Intermittent Supply

A continuous method of supply is always better than the intermittent method because of the following reasons:

(a) When the supply of water is only for a few fixed hours of the day, consumers are compelled to store water for use during the non-supply hour. The domestic storage tanks built for the purpose may suffer for want of proper maintenance and attention for a lone time, resulting in a possible contamination of the water supply.

(b) The unused water of storage tanks is most likely to be thrown out to be replaced, during the supply hour, by fresh supply of water. Evidently, this is a wasteful use of water. Also, where the supply is not metered, there is a tendency on the part of consumers to leave the taps open for all hours, resulting in additional wastage of water. The receptacles so left under public hydrants and faucets may remain overflowing, without being attended to, for a long time.

Continuous and Intermittent

(c) In case of fire breaks out during the non-supply hours, considerable damage would have resulted before the supply could be turned on and fire extinguished.

(d) During the non-supply hours, pressure in the distribution mains may fall below atmospheric pressure, causing partial vacuum, sucking in air or other harmful gases from sewers running close-by and resulting in a possible contamination of the water supply.

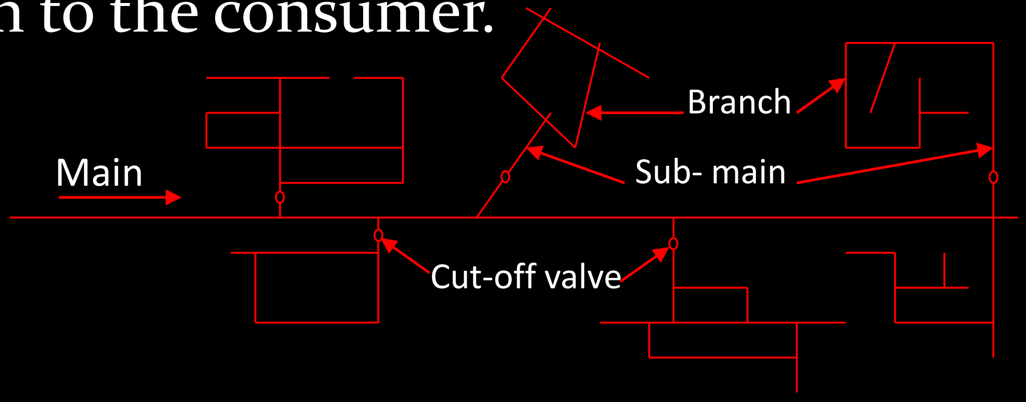
Methods of Layout of Distribution Pipe

Four methods of distribution pipe laying are discussed. It may however be noted that each system has its own merits and no locality strictly adopts only one of these methods. The necessary combinations of the above methods are usually made to suit the local conditions of the area.

Dead-end method

This is also known as the tree system of layout and it consist of one supply main from which sub-mains are taken. The sub-mains again divide into several branch lines from which service connections are given to the consumer.

This system is suitable to old towns and cities which have been irregularly developed having no definite pattern of roads and streets.



Methods of Layout of Distribution Pipe

Advantages

Following are the advantages of the dead-end method:

i. It is possible to work out accurately the discharge and pressure at any point in the distribution system. The design calculations are simple and easy.

ii. The cut-off valves required in this system of layout are comparatively less in number.

iii. The diameters of mains are to be designed for the population likely to be served by them only. This fact may make the system cheap and economical.

iv. The laying of water pipe is simple.

v. By suitably locating valves, water supply can be so regulated that by closing any valve, a section of the system can be cut off for repairs without affecting the rest.

Methods of Layout of Distribution Pipe

Disadvantages

Following are the disadvantages of the dead-end method:

i. During repairs, the large portion of distribution area is affected. It results into great inconvenience to the consumers of that area.

ii. There are many dead-ends in this system. The pipes terminate at the dead-ends and hence there is no free circulation of water. There are chances for water to be polluted due to stagnation and it may endanger the public life. For this purpose, the scour valves are provided at dead-ends and water from dead-ends is removed periodically by the operation of these valves. This measures proves to be costly as treated water is thrown to waste and it also requires careful attendance and operation of the scour valves.

iii. The water available for fire fighting will be limited in quantity as the discharge from mains is also limited. This may prove to be serious in some roads.

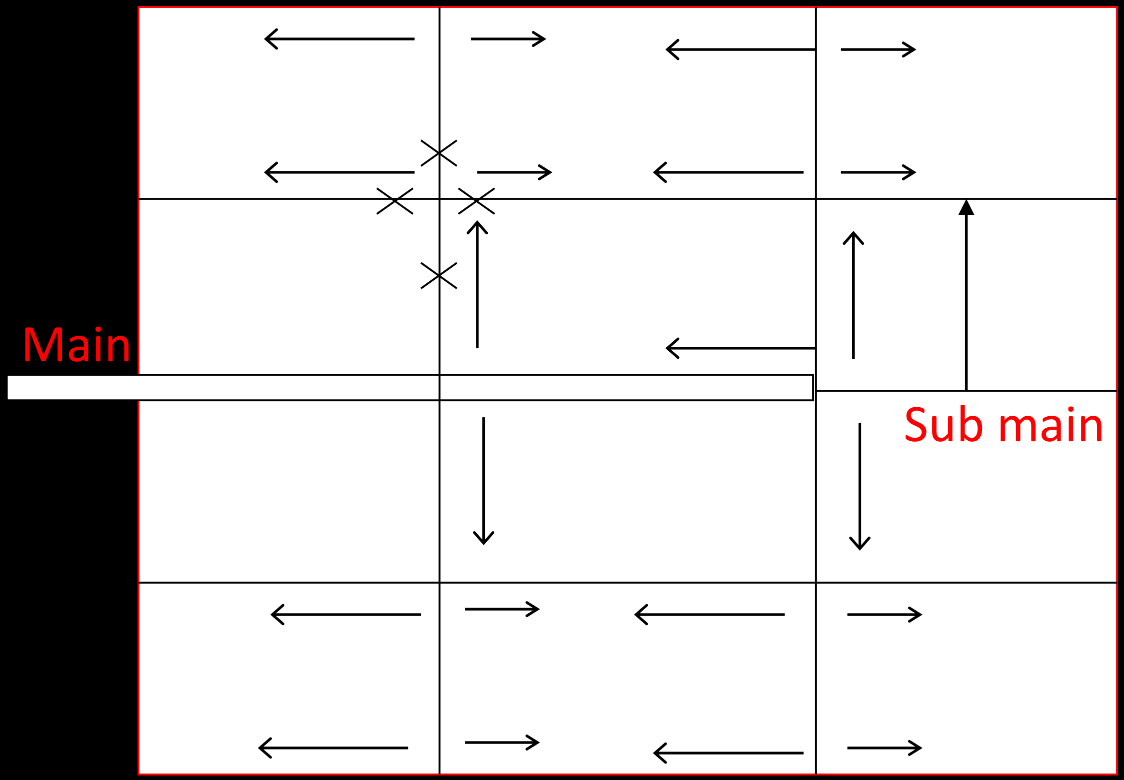
Methods of Layout of Distribution Pipe

Grid-iron method

This is also known as the interlaced system or reticulation system. The mains, sub-mains and branches are interconnected with each other as shown in Figure

It is an improvement over the Dead End System, caused by connecting the ends of the various mains so as to eliminate the dead ends. The water then circulate freely throughout the system.

Such a system is very useful for a city laid out on a rectangular plan.



Methods of Layout of Distribution Pipe

Advantages

Following are the advantages of the grid-iron method:

i. In case of repairs, a very small portion of the distribution area will be affected.

ii. There is free circulation of water and hence it is not liable for pollution due to stagnation.

iii. The water is delivered at every point of distribution system with minimum loss of head.

iv. When a fire occurs, plenty of water is available for the fighting purpose and by manipulating the cut-off valves, whole supply of water may be concentrated for this purpose.

Methods of Layout of Distribution Pipe

Disadvantages

Following are the disadvantages of grid-iron method:

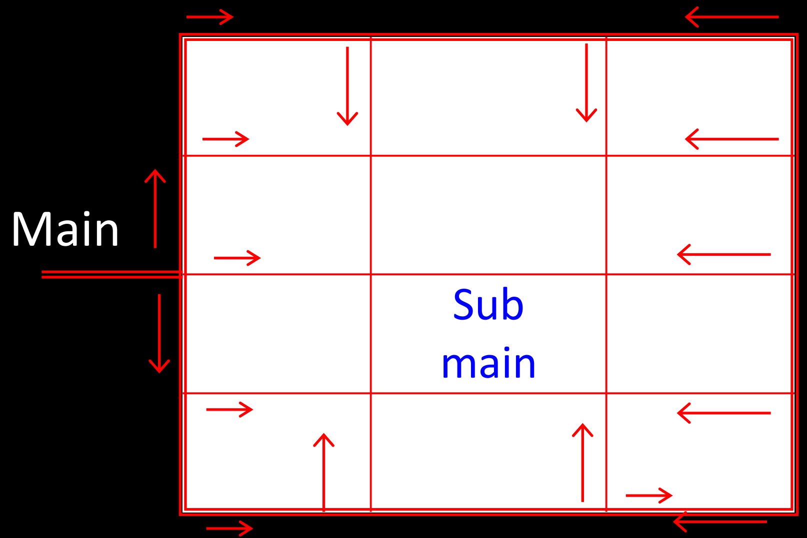
i. The cost of laying water pipes is more.
ii. The grid-iron system of layout requires longer length of pipes.
iii. The procedure for calculating the sizes of pipes and for working out pressures at various points in the distribution system is laborious, complicated and difficult.
iv. The valves required in this system are more in number and in fact, four valves are to be installed at every cross junction.

Use: The grid-iron system of layout is more suitable for towns having well-planned roads and streets. However the principle of grid-iron system can be applied to the dead-end system of layout by removing dead-ends.

Methods of Layout of Distribution Pipe

Circular method

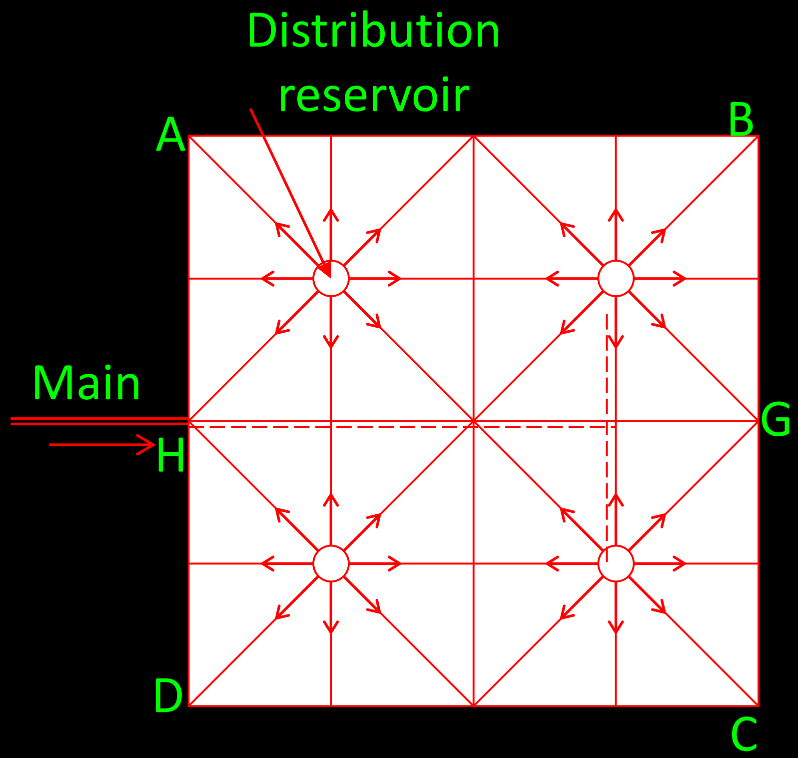
This is also known as the ring system and a ring of mains is formed around the distribution area as shown in Figure. This system possesses advantages and disadvantages as those of grid-iron system. The distribution area in divided into rectangular or circular blocks and the water mains are laid on the periphery of these blocks. The ring system of layout is most suitable for towns having well-planned roads and streets.



Methods of Layout of Distribution Pipe

Radial method

This method of layout is just the reverse of the ring method. In this system, the water is taken from the mains and pumped into the distribution reservoirs which are situated at centers of different zones as shown in Figure. The water is then supplied through radially laid pipes. The radial method of layout gives quick service and the calculations for design of size of pipes are simple. The radial method is most suitable for towns having roads laid out radially.



Maintenance of Distribution System

The distribution system of water supply should be maintained so that the equipment employed and the processes followed can work smoothly without interruption. A well designed and properly laid water distribution system may prove to be highly inefficient in its operation for want of proper maintenance. Following are the important items which are to be attended to during the maintenance of distribution system of water supply:

i. The flushing of water pipes should be carried out wherever necessary especially in case of dead-ends on lines and at places where water is supplied without the process of filtration.

ii. The hydrants, valves and various other appurtenances installed on the water mains should be checked regularly and should be maintained in perfect running order.

Maintenance of Distribution System

iii. The records regarding the lengths of pipes laid, lengths of pipe repaired or replaced, expenditure incurred, number of fire hydrants, number of service connections and all other relevant data in connection with the distribution system should be well maintained for ready reference.

iv. The wastage of water especially of leakage through pipe joints should be brought down to the minimum possible extent by adopting suitable preventive measures.

v. The water pipes should be cleaned periodically by the use of scraping devices and the incrustations formed on the inside surface of water pipes should be removed.

vi. The meters installed on the distribution system should be checked from time to time and the defective or unreliable meters should be immediately repaired or replaced.

vii. The maps showing the up-to date latest layout of distribution of water pipes in the locality should be maintained in the office of the responsible authority.

Environmental Engineering-1

CE3141

Lecture -6

Cycle-5, C-day

03-03-2018

Design of Distribution System

Factors to be considered in the design of a distribution system are –

(a) Type of flow – whether continuous or intermittent.

(b) Method of distribution – whether by gravity or by pumping.

(c) Probable future demand based on prospective increase in population. This also includes the industrial demand as well as the fire fighting requirements.

(d) Period to be considered to be the life of pipes used. The system should be designed anticipating the future of the condition that will obtain near the end of the time when the amounts set aside for depreciation would have returned the first cost.

Design of Distribution System

The Hazen Williams equation can conveniently be used for the hydraulic design of the branched or looped networks of pipes. The Hazen Williams equation can be written in the form:

$$Q = 3.7 \times 10^{-6}\, CD^{2.63}(H/L)^{0.54} \text{ ------------------------------------- } (1)$$

Where, Q = flows, lps

C = roughness coefficient (100 – 140 for rough to smooth pipes)

D = diameter, m

H = head loss, m

L = length of pipe, m

Design of Distribution System

For a definite value of C, the equation (1) can be written as:

$H/L = 1.39 \times 10^{-6} Q^{1.85}/(D^{4.87})$ ----------------- (2) [For C = 130]

$H/L = 1.59 \times 10^{-6} Q^{1.85}/(D^{4.87})$ ----------- (3) [For C = 120]

The process involved in the design is to make a pipe layout, assume the pipe size and then work out the terminal pressure head which could be made available at the end of the each pipe section when discharging the peak flow. The available pressure heads are checked to see if they correspond to permissible residual pressure heads. If not, the pipe size is changed and the system is reinvestigated until satisfactory conditions are obtained.

Design of Distribution System

Branched Network

The following design procedure may be adopted for branched network:

i. Collect/ prepare a map of the area to be served with roads, streets and other features and make a layout of mains, sub-mains and branches including location of valves and other appurtenances.

ii. Estimate the pick flow at different points and determine the quantity flowing through each section of the pipe. Peak flow = average daily flow × peak factor.

iii. Assume pipe sizes of all the pipes in the network (to calculate approximate pipe size the velocity may be assumed to be around 1 m/s).

iv. Calculate frictional head loss per unit length of the pipe and then multiply by pipe length of the pipe to find the total head loss.

v. Determine the terminal pressure head taking the change in the elevation of the pipe into account.

vi. In case of a difference between the computed terminal pressure and the permissible pressure head, revise the pipe size.

Design of Distribution System

Looped Network

The determination of probable flow in each pipe in a pipe networks requires complicated and tedious trial and error solutions. In any looped pipe network two conditions must be satisfied:

i. The flow entering a junction must equal the flow leaving it.

ii. The algebraic sum of the pressure drop (head loss) around any closed loop must be zero.

Analysis of Distribution System

Frequently, it become necessary to analyze a given distribution system in order to determine through a quick and approximate check, the pressures and flows available in any section of the system and to suggest ways to improve upon the same, if found inadequate. A few important methods are briefly discussed as below:

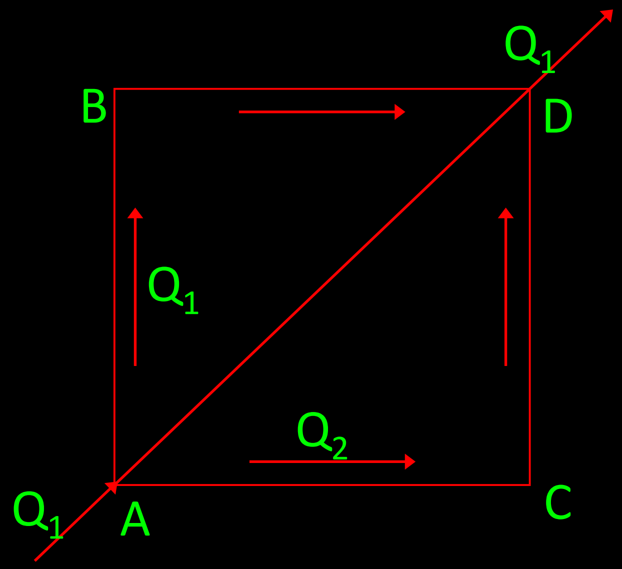
- Equivalent pipe method
- Method of sections
- Hardy-Cross method

Analysis of Distribution System

Equivalent pipe method

This method is useful in rendering a complex network of pipes into an equivalent pipe system giving the same discharge and loss of head as in the complex system. For purpose of analysis, the entire network of pipes is considered to be split up into two portions: (i) pipe in series and (ii) pipe in parallel.

Pipe in series: Pipes carry arbitrarily chosen values of discharge Q_1 flowing through branches AB and BD and Q_2 flowing through AC and CD in Figure. It is assumed that the loss of head for pipe in series is additive.



Analysis of Distribution System

Knowing discharge (say Q_1) and diameters of pipe-lines AB and BD through which it flows, it is possible to determine the loss of head H_1 in their total length (AB+BD). Here, use is made of the nomograph for Hazen-Williams Formula in which C=100. A single length of equivalent pipe AD of known diameter can then be selected to give the same values of discharge Q_1 and loss of head H_1.

Pipes in parallel: In this case, head through pipes in parallel i.e., ABD and ACD is the same. If a certain loss of head (say H_1) is one assumed to occur in either arm length ABD and ACD, flows through the arms can be worked out and added together to see that the total flow corresponds nearly to the original flow Q. The size and length of a single pipeline can then be calculated to give the same discharge and loss of head.

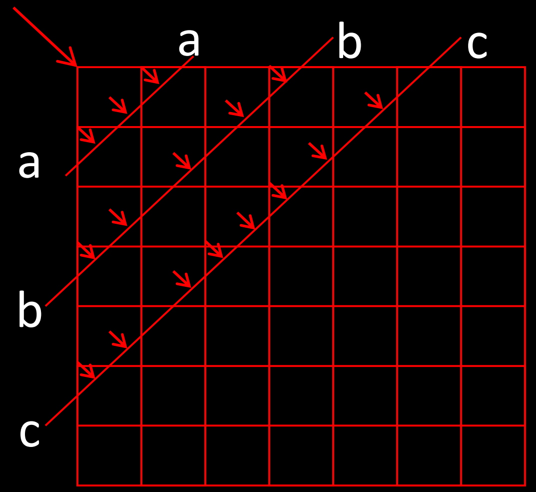
Analysis of Distribution System

Method of sections

This is an approximate method but gives a quick check and is simple to follow. The method may be described in the following steps:

i. Cut the network of pipes by a number of sections a-a, b-b, c-c, etc. at right angles to the assumed direction of flow in Figure. Consider a proper sequence of pipes and character of district served (residential or commercial).

ii. Calculate the quantity of water to be supplied beyond each section.



Method of sections

iii. Study the average available gradient. Velocity allowed in pipes is 2 to 4 ft/sec. Permissible gradient should be 1 to 3 ft per 100 ft.

iv. Find out the number of pipes cut by each section.

v. Calculate the total discharge of water available at the end of each section by determining the discharge capacities and number of pipes cut at section and summing these up.

vi. Difference between the required discharge and the calculated discharge are made up by providing suitable number of additional pipes at allowable gradients at each end of section.

Analysis of Distribution System

Hardy-Cross method

Pipe network problems in water distribution systems are usually solved by methods of successive approximation, since any analytical solution requires the use of many simultaneous equations, some of which are nonlinear. We know,

$H/L = 1.59 \times 10^{-6}\, Q^{1.85}/(D^{4.87})$ ---------- (3) [For C = 120]

The equation (3) can be written for a particular pipe as:

$H = kQ^{x}$ --------(4)

In which k is the constant depending on the length, diameter and roughness of the pipe as well as the fluid property, x is the component equal to 1.85 for Hazen Williums' equation and 2 for the Manning equation.

Analysis of Distribution System

Hardy-Cross method

Hardy Cross developed a method of successive approximation in which the circuits are balanced, distribution of flow is determined and the above two conditions of flow are satisfied. The solutions for pipe network problem suggested by Hardy Cross requires that the flow in each pipe be assumed so that the principle of continuity is satisfies in each junction. A correction, Δ, to the assumed flow is computed successively for each pipe loop in the network until the correction is required to an acceptable value. If Q_a is the assumed flow, then the correction is $Q - Q_a$ i.e. $Q = Q_a + \Delta$ --------(5)

Inserting the value of Q from equation (5) and applying the condition that the head loss around any closed loop is zero the head loss equation (4) gives: $\sum k\,(Q_a + \Delta)^x = 0$ ----------- (6)

Analysis of Distribution System

Dardy-Cross method

Expanding equation (6) $\sum kQ_a{}^x + x\sum k\Delta Q_a{}^{x-1} + (x-1)/2\sum k\Delta^2 Q_a{}^{x-2}$ + ---- + ----- = 0 -----------------(7)

If Δ is small compared to Q, the third and all successive terms of the equation (7) may be neglected, hence,

$\sum kQ_a{}^x + \Delta x\sum kQ_a{}^{x-1} = 0$ ---(8)

Solving equation (9) for Δ:

$\Delta = -\sum kQ_a{}^x / x\sum kQ_a{}^{x-1}$ or, $\Delta = -\sum H / (x\sum H/Q_a)$ -------------- (9)

Analysis of Distribution System

The procedure for Hardy Cross method

i. Carefully examine the network and assume reasonable rates of flow in each pipe such that inflow equals outflow at each junction.

ii. In each loop, determine the head loss, H and H/Q for all pipes.

iii. With due attention to sign, compute the total head loss around each circuits.

iv. Compute $\sum H/H$ for the same circuit without giving any consideration to sign.

v. The correction, Δ is computed for each loop by equation (9). The minus sign may be disregarded if the correction so obtained is made by inspection.

Analysis of Distribution System

The procedure for Hardy Cross method

vi. Apply the correction to each pipe in each loop. When the sign of Δ is negative (-), decrease the clockwise flows and increase the counterclockwise flows. When the sign is positive (+), increase clockwise flows and decrease counterclockwise flows. Pipes that are common to two loops require double correction.

vii. With adjusted flows, repeat the procedure for the second approximation. The procedure is continued until the desired accuracy is attained.

WATER QUALITY AND TREATMENT

The water required for public water supply scheme should be potable or wholesome water i.e. fit for drinking purposes. It is however not essential to have physically or chemically pure water. The presence of some minerals in water is required to give some taste to the water i.e. to make it palatable and they also assist in food assimilation. It will be difficult, time consuming and costly to have complete purification of the water.

WATER QUALITY AND TREATMENT

The impurities in water are to be removed to a certain extent only so that it does not prove harmful to the public health. The term *wholesome* water is used to indicate the water which is not chemically pure, but does not contain anything harmful to the human body i.e. the water in which there are no pathogenic bacteria, no toxic substances and no excessive organic matter. The term *pure* water is a relative term and it has to be interpreted in relation to the use of water.

Water Quality Parameters and Standards

The standards of water quality parameter depend on the purposes of use. These parameters are briefly described as below:

Turbidity: The term turbidity is applied to water containing suspended matter that interferes with the passage of light through the water or in which visual depth is restricted. The turbidity may be caused by a wide variety of suspended materials which range in size from colloidal to coarse dispersions, depending upon the degree of turbulence. Because of the wide variety of materials that cause turbidity in natural waters, it has been necessary to use an arbitrary standard. The standard chosen was 1 mg of SiO_2 in 1 liter distilled water and the silica used must meet certain specifications as to particle size. Now, 1 unit of turbidity = 1 mg SiO_2/L.

Water Quality Parameters and Standards

Total dissolved solids (TDS): Total dissolved solids comprise inorganic salts and small amount of organic matter. The common dissolved mineral salts are claimed to affect the taste, hardness, corrosion and encrustation. Dissolved inorganic substances may exert adverse effects on aquatic animals and plants and many cause irrigation problem. The amount of dissolved solid present in water is an important consideration in its suitability for domestic use. In general, water with a total solids content of less than 500 mg/l is most desirable for such purposes.

Water Quality Parameters and Standards

Depending on the TDS water is often classified as follows:

Excellent	TDS < 300 mg/l
Good	300 – 600 mg/l
Fair	600 – 900 mg/l
Poor	900 – 1200 mg/l
Unacceptable	TDS > 1200 mg/l

Water Quality Parameters and Standards

Colour: Water become coloured due to addition of various suspended matter in water. Tannins, acid and humates, from the decomposition of lignin, are considered to be the principal colour bodies. Iron is sometimes present as ferric humate and produces a colour of high potency. Colour caused by suspended matter is referred to as apparent colour and is differentiated from colour due to vegetable or organic extracts that are colloidal and which is called true colour. In water analysis it is important to differentiate between "apparent" and "true" colour.

Waters containing natural colour are yellow-brownish in appearance. Through experience, it has been found that solution of potassium chloroplatinate ($K_2 Pt Cl_6$) tinted with small amounts of cobalt chloride yield colours that are very much like the natural colours. The colour produced by 1 mg/L of platinum (as $K_2 Pt Cl_6$) is taken as the standard unit of colour.

Water Quality Parameters and Standards

Tastes and odour: The words taste and odour are often used loosely and interchangeably. Actually there are four tastes: sour, salt, sweet and bitter – strictly confined in their perception to the taste buds of the tongue. Odours appear to be without limit in number and are known to change in quality as the concentration of the odourous compounds, or the intensity of their small, is varied.

Tastes and odours are associated with (i) decaying organic matter, (ii) living algae and other microorganisms containing essential oils and other odourous compounds, (iii) iron and manganese and other metallic products of corrosion, (iv) industrial wastes, particularly phenolic substances, (v) disinfecting chlorine and its substitute compounds and (vi) biologically nondegradable synthetic organics.

Water Quality Parameters and Standards

Temperature: The most desirable range of temperatures for a public water supply is between 40 and 50°F. Natural waters are seldom found below 40°F. As the temperature rises above 50°F, the water becomes less palatable and less suited to certain uses. Temperatures above 80 °F are undesirable, and above 90° to 95°F the water is unfit for a public supply.

Hydrogen ion concentration, pH: PH is a term used rather universally to express the intensity of the acid or alkaline condition of a solution. More exactly, it is a way of expressing the hydrogen ion concentration. In the field of water supplies, it is a factor that must be considered in chemical coagulation, disinfection, water softening and corrosion control. In sewage and industrial waste treatment employing biological processes, pH must be controlled within a range favourable to the particular organisms involved.

Water Quality Parameters and Standards

Alkalinity: The alkalinity of water is a measure of its capacity to neutralize acids. The alkalinity is due primarily to salts of weak acids and strong bases. Such substances act as buffers to resist a drop in pH resulting from acid addition. Alkalinity is thus a measure of the buffer capacity. Three major classes of minerals cause most of the alkalinity in natural waters: bicarbonate, carbonates and hydroxides. Other salts of weak acids, such as borate, silicates and phosphates may be present in small amounts. A few organic acids, such as humic acid, add to the alkalinity of natural waters. Excessive or insufficient alkalinity interferes with water treatment (coagulation).

Specific conductance: The specific conductance of water is the reciprocal of the resistance in ohms of a column of the water 1 cm long and having cross section of 1 sq.cm at a specific temperature, usually 25°C. It is commonly reported in ohms. The specific conductance is used as a measure of the quality of the water.

Water Quality Parameters and Standards

Hardness: Hardness in water is that characteristic which prevent the lathering of soap. It is caused principally by the solution in water of carbonates, bicarbonates and sulphates of calcium and magnesium, although the chlorides and nitrates of these two elements and sometimes of iron and of aluminum are effective to a lesser degree in causing hardness.

The term hardness is defined as the ability of the water to cause precipitation of insoluble calcium and magnesium salts of higher fatty acids from soap.

Total hardness is expressed in various ways, the standard practice being in parts per million by weight in terms of calcium carbonate.

Water Quality Parameters and Standards

Various ranges of hardness of water

Class	1	2	3	4
Hardness, ppm	0 – 55	56 – 100	101 – 200	201 – 300
Degree of hardness	Soft	Slightly hard	Moderately hard	Very hard

Water Quality Parameters and Standards

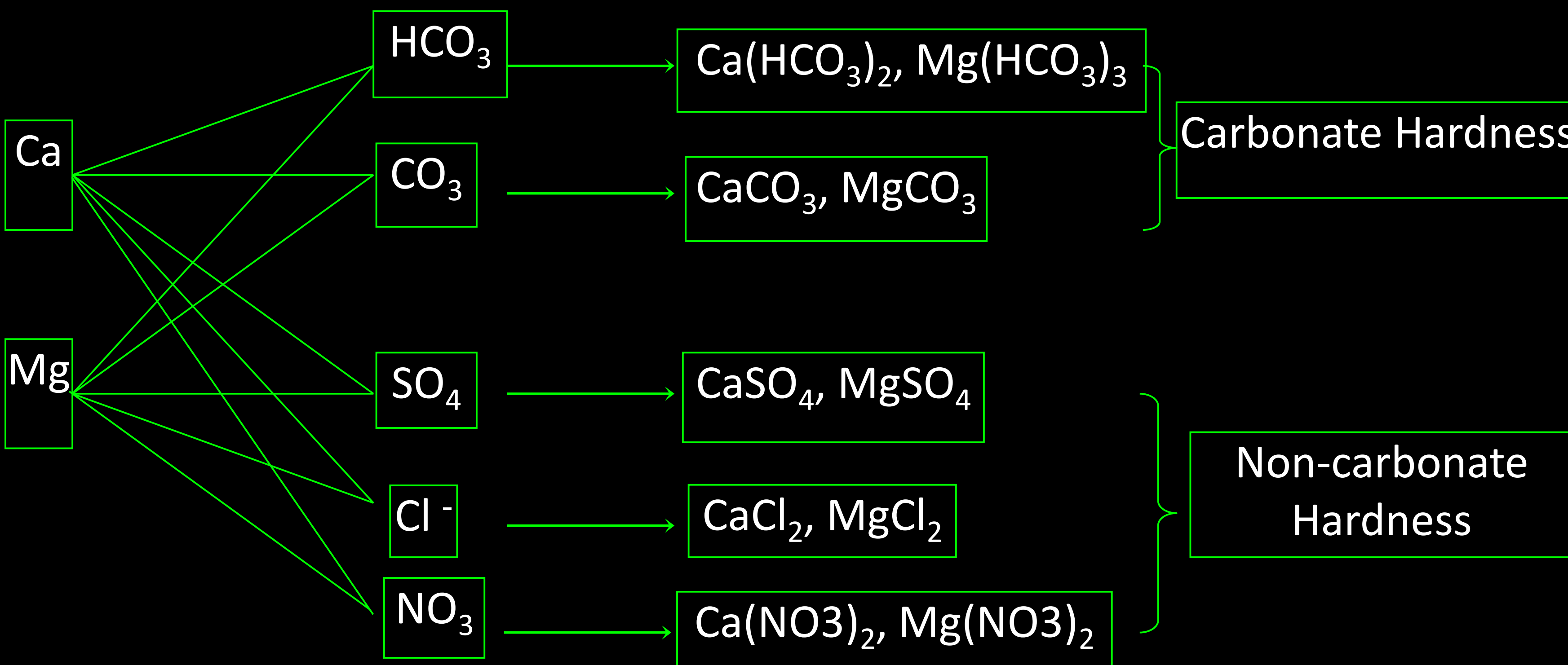
Classification of hardness: Depending on soap destroying power hardness is classified as temporary hardness and permanent hardness.

The temporary hardness is also known as the carbonate hardness and it is mainly due to the presence of bicarbonate of calcium and magnesium. It can be removed by boiling or by adding lime to the water.

The permanent hardness is also known as the non-carbonate hardness and it is due to the presence of sulphates, chlorides and nitrates of calcium and magnesium. It cannot be removed by simply boiling the water. It requires special treatment of water softening.

Water Quality Parameters and Standards

The diagram of carbonate hardness and non-carbonate hardness is presented in Figure.



Water Quality Parameters and Standards

Principal bad effect of hardness: The principal bad effect of hardness is given below: Enough consumption of soap.

Clogs skin, discolours porcelain, stains and shortens fabrics, toughens and discolours vegetables.

Gives difficulty in textile and paper manufacture, tannery and other industrial processes.

Forms scales in boilers, resulting in great heat transfer losses and danger of boiler failure.

Water Quality Parameters and Standards

Arsenic: In Bangladesh the presence of arsenic in groundwater was first detected in 1993 at Baroghoria union of Chapai Nawabganj district. It has been observed that about one in every three shallow tube wells is producing water with arsenic in acceptable limits. The concentration of arsenic in drinking water in excess of permissible limit is toxic to human body. According to the WHO guideline value the desirable maximum concentration of arsenic in drinking water would be 0.01 mg/l. In Bangladesh the maximum acceptable concentration in drinking water is considered to be 0.05 mg/l. Symptoms of arsenic toxicity leading to cancer may occur due to excessive intake of arsenic in the human body over a longer period of time.

Water Quality Parameters and Standards

Drinking water standard

Parameters		Standard (mg/l)	
		USPHS	WHO
Physical quality	Total Dissolve Solid (TDS)	500	500
	Turbidity	10	5
	Color	20	15
	Temperature	50^{o} F	50^{o} F
	Tastes and odor	Water should be completely free from taste and odor	

Water Quality Parameters and Standards

Drinking water standard

Parameters		Standard (mg/l)	
		USPHS	WHO
Chemical quality	Arsenic (As)	0.01	0.01
	Cyanide (Cn)	0.01	0.01
	Lead (Pb)	0.05	0.05
	Barium (Ba)	1.00	1.00
	Selenium (Se)	0.01	0.01
	Cromium (Cr)	0.05	0.05
	Cadmium (Cd)	0.05	0.05
	Silver (Ag)	0.05	0.05

Water Quality Parameters and Standards

Drinking water standard

Parameters		Standard (mg/l)	
		USPHS	WHO
Chemical quality	Detergent (ABS)	0.50	0.50
	Chloride (Cl)	250	200
	Copper (Cu)	1.00	1.00
	Carbon-Chloroform extract (CCE)	0.20	0.20
	Iron (Fe)	0.25	0.25
	Manganese (Mn)	0.05	0.05
	Iron (Fe) + Manganese (Mn)	0.30	0.30
	Nitrate as NO3	45	45
	Phenols	0.001	0.001

Water Quality Parameters and Standards

Drinking water standard

Parameters		Standard (mg/l)	
		USPHS	WHO
Chemical quality	Sulphate (SO_4)	250	250
	Zinc (Zn)	5.00	5.00
	Calcium (Ca)	80.0	75.0
	Magnesium (Mg)	80.0	75.0
	Total hardness	150	100
	Total alkalinity	120	100
	Fluoride (F^-)	0.60	0.50
	pH	7 to 8	7 to 8

Environmental Engineering-1

CE3141

Lecture -7

Cycle-5, D-day

04-03-2018

Treatment of water

Natural water contains impurities in different forms. The presence of these impurities in excess of acceptable limits make the water unfit for domestic supplies. The main objectives of water treatments are to make water potable i.e. to make water safe to drink, pleasant to taste and suitable for domestic uses.

Groundwater is usually hard but free from pathogenic bacteria and can be supplied for drinking purpose without treatment. Some tubewell water in Bangladesh may contain iron, arsenic and hardness in excess of acceptable levels, and may therefore require specific treatment. Surface water is turbid, colored and contaminated by pathogenic microorganisms and needs extensive treatment involving sedimentation, coagulation with sedimentation, filtration and disinfection.

Treatment of water

The type of treatment required depends on the physical chemical and biological characteristics of water. The most common steps in water treatment are clarification and disinfection.

Clarification involves removal of suspended and colloidal particles including color-producing substances by plain sedimentation, sedimentation with coagulation and filtration to remove visible impurities and make the water attractive to the consumers.

Disinfection means destruction of pathogenic organisms to make the water safe.

Sometimes invisible dissolved minerals and gases present in groundwater are required to be removed by specific treatment processes to make the water potable.

Treatment of water

The common water treatment methods are:

(i) Plain sedimentation,

(ii) Coagulation with sedimentation,

(iii) Filtration and

(iv) Disinfection.

Some of the treatments process/unit operations for removal of specific impurities are (i) aeration, (ii) water softening, (iii) arsenic removal, (iv) iron removal, (v) activated carbon application (vi) fluoridation and defluoridation, (vii) demineralization and (viii) desalinization.

One or a combination of more than one treatment method is employed for water treatment depending on the quality of raw water.

Plain Sedimentation

Principle of particle settling in water

This is a process causing the organic or inorganic particles heavier than water to settle by retaining water in a tank or basin. These particles are held in suspension in natural water mainly by turbulence or current and when the current is retarded, the suspended particles settle at the bottom of the basin.

A particle having specific gravity of more than 1, i.e. heavier than water, tends to move downward in relatively quiescent water by the force of gravity, accelerating until the frictional resistance (drag) of the water equals the gravitational force acting upon the particle. Thereafter the particle travels with a constant vertical velocity called the '*terminal velocity*' or '*settling velocity*' of the particle.

Plain Sedimentation

The settling velocity of the particle depends upon:

- Horizontal flow velocity of water
- Shape and size of the particle
- Specific gravity of the particle
- Viscosity of water
- Density of water
- Temperature of water

Plain Sedimentation

The settling velocity of spherical particles under laminar flow conditions is given by the simplified equation:

$$v_s = \frac{g}{18}(S-1)\frac{d^2}{\gamma}$$

Where, v_s is the settling velocity, g is acceleration due to gravity, S is specific gravity of the particle, d is diameter of the particle and γ is kinematic viscosity of water.

Plain Sedimentation

The above equation is called Stoke's Law. Stoke's Law holds good only for particle size 0.1 cm in diameter and Reynold's number 1 or less. For large particles having diameter greater than 1 cm and Reynold's number above 2000, Newton's Law for frictional resistance or drag applies:

$$v_s = \sqrt{\frac{4g}{3C_D}(S-1)d}$$

Where C_D is the Newton's coefficient of drag.

Plain Sedimentation

The particles in between the above mentioned size or Reynold's numbers are in transition settling. Stoke's Law is valid for computation of settling velocity of discrete particles. Discrete particles are those which do not change size, shape and mass during settling and which do not influence each other by being too close. Particles settling under this conditions is called *discrete settling*.

In case of closely packed particles, the water displaced by the particles may cause additional friction and the settling velocity is reduced. This is termed as *hindered settling*. Hidered settling becomes noticeable when the concentration of suspended solids is greater than 2000 mg/l. This situation of high concentration of suspended solids may happen in river water during high flooding and heavy rainfall.

Sometimes settling particles may adhere to each other and grow in size and thus deviate from the settling characteristics represented by Stoke's Law. This may occur in settling of algae or freshly formed flock by the process of flocculation with coagulant. These particles/flocks and tent to stick together and form new bigger particles which settle at a faster rate. This type of settling is called *flocculent settling*. Discrete, hinder and flocculent settling are shown in Figure.

Plain Sedimentation

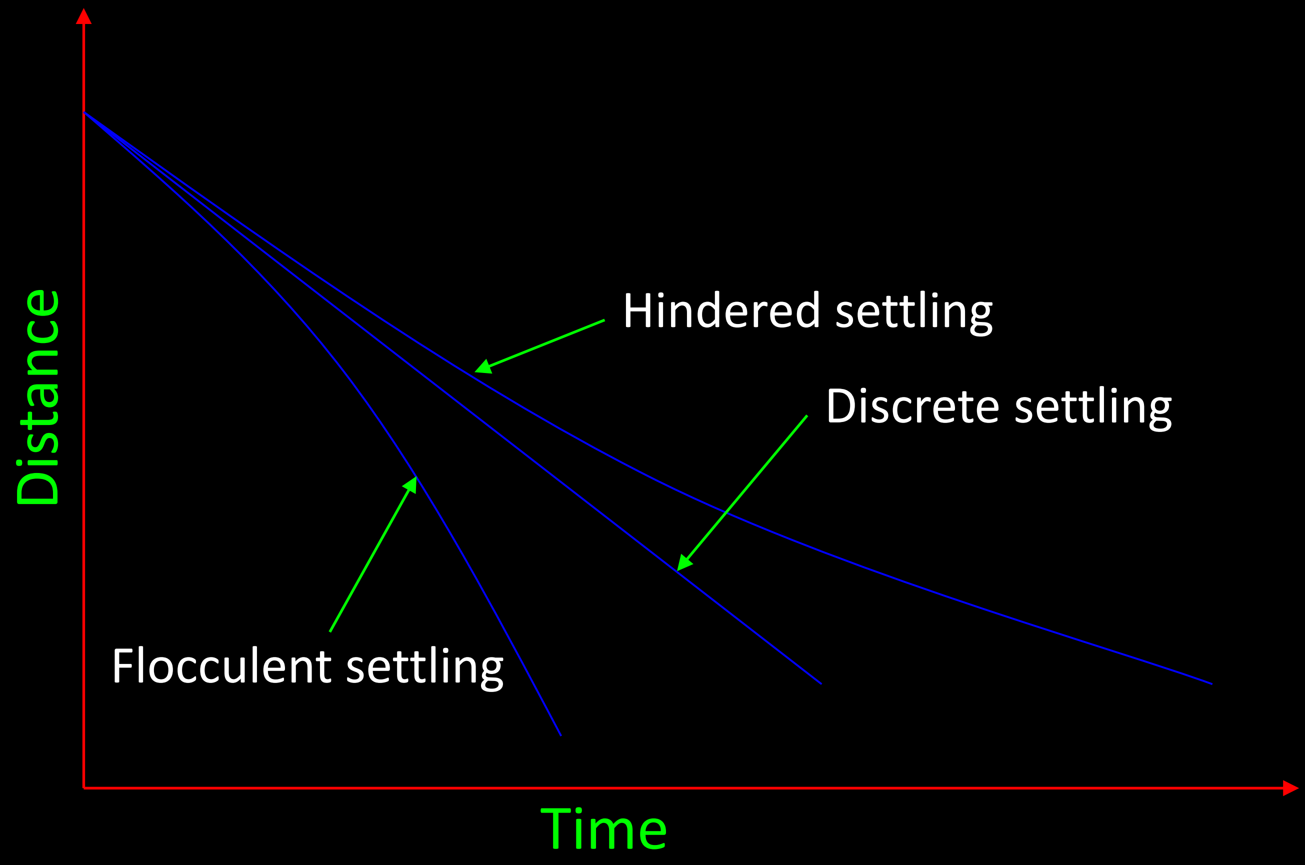


Figure: Settling of different types of particles in water

Plain Sedimentation

Design of sedimentation tanks

A rectangular sedimentation tank can be subdivided into four different areas comprised of an inlet, settling, outlet and sludge accumulation zones. The inlet zone serves to provide even flow distribution over the full cross section, the outlet zone collects the clarified water over the full tank width. Sludge is accumulated at the tank bottom where it is stored and removed periodically.

The efficiency of the settling tank in the removal of suspended particles can be determined using limiting settling velocity v_o of a particle which will just travel the full depth (H) of the tank within the detention time (T). Using the dimensions and notations used in Figure, the following equations can be written: $v_o = H/T$ and $T = V/Q = BLH/Q$

Plain Sedimentation

from these two equations:

$v_o = Q/BL = Q/surface\ area.$

Where, Q = flow rate, B, L and V are width, length and volume of the sedimentation zone of the settling tank.

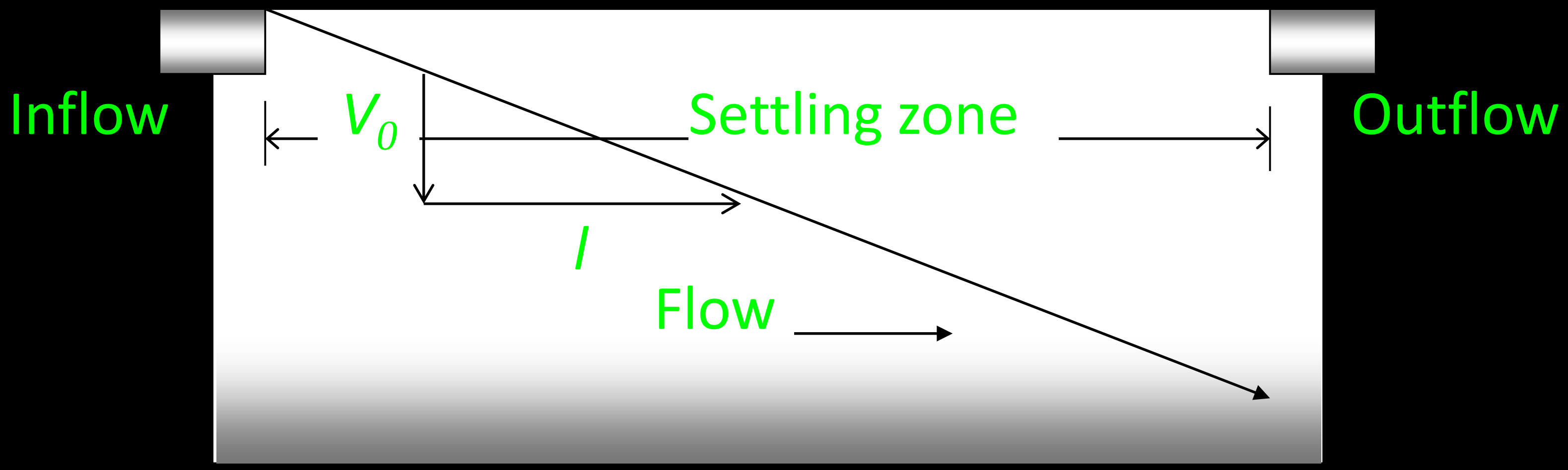


Figure: Rectangular horizontal flow settling tank

Plain Sedimentation

The tank will remove all the particles having settling velocity $v_s > v_o$ and the particles with settling velocity $v_s < v_o$ will be removed in the proportion $v_s : v_o$. The above analysis shows that the settling efficiency depends on the ratio between the influent flow rate Q and the surface area of the tank BL, which is called the "*surface loading*". Hence the efficiency of the settling tank is independent of the depth of the tank.

The higher the surface area the greater is the efficiency.

Plate settlers and tube settlers have been designed to provide a larger surface area and achieve higher efficiency.

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CE3141

Lecture -8

Cycle-7, C-day

25-03-2018

Coagulation and Flocculation

The removal of very fine light colloidal impurities from water is difficult to achieve in practice by the process of plain sedimentation. This can be greatly expedited by the addition to water of certain chemical compound which where thoroughly mixed form wooly masses of flocculent precipitate enmeshing the suspended particles become heavier and finally settle out. These substances are called *coagulants* and their process of action is *coagulation*.

Coagulation and Flocculation

Principle of Coagulation

i. **Flock formation:** When coagulants are dissolved in water and thoroughly mixed with it, they produce a thick gelatinous precipitate. This precipitate is known as the flock and this flock has got the property of arresting the suspended impurities in water during its downward travel towards the bottom of tank.

ii. **Electric charges:** The ions of flock are found to possess positive electric charge. Hence they will attract the negatively charged colloidal particles of clay and thus they cause the removal of such particles from water.

Coagulation and Flocculation

Factors Influencing Coagulation

Many factors influence the coagulation of waters. Among them, the following are important:

i. Kind of coagulant

ii. Quantity of coagulant

iii. Characteristics of water (suspended matter, pH and temperature) and

iv. Time of mixing, flocculation and coagulation.

Coagulation and Flocculation

Dosage of Coagulants

This depends upon number of factors such as:

i. Turbidity of water

ii. Colour of water

iii. pH value of water

iv. Time of settlement and

v. Temperature of water

Coagulation and Flocculation

Chemical Reactions

1. $Al_2(SO_4)_3 + 3Ca(HCO_3)_2 = Al_2(OH)_6 + 3CaSO_4 + 6CO_2$

2. $FeSO_4 + Ca(HCO_3)_2 = Fe(HCO_3)_2 + CaSO_4$

$Fe(HCO_3)_2 + 2Ca(OH) = Fe(OH)_2 + 2CaCO_3 + 2H_2O$

$4Fe(OH)_2 + O_2 + H_2O = 2Fe(OH)_3$

Filtration

The process of passing the water through the beds of granular materials is known as the filtration.

The filtered water is potable and palatable and it is free from various undesirable impurities like colour, odour, turbidity, pathogenic bacteria, etc.

During the process of filtration, the following effects occur on water:

i. The suspended and colloidal impurities which are present in water in a finely divided state are removed to great extent.

ii. The chemical characteristics of water are altered.

iii. The number of bacteria present in water is also considerably reduced.

Filtration

The theory of filtration

It can be explained based on the following four actions:

i. Mechanical straining: The suspended particles which are unable to pass through the voids of sand grains are arrested and removed by the action of mechanical straining.

ii. Sedimentation: The voids between the sand grains of filter act more or less like small sedimentation tanks. The particles of impurities, arrested in this voids, adhere to the particles of sand grains mainly for the following two reasons:

a. Due the presence of a gelatinous film or coating developed on sand grains by previously caught bacteria and colloidal matter;

b. Due to the physical attraction between the two particles of matter.

Thus the suspended impurities are removed by filter by the action of sedimentation.

Filtration

iii. Biological metabolism: The growth and life process of the living cells is known as the biological metabolism. When bacteria are caught in the voids of sand grains, a zoological jelly or film is formed around the sand grains. This film contains large colonies of living bacteria. The bacteria feed on the organic impurities contained in water. They convert such impurities into harmless compounds by the complex biological reactions.

iv. Electrolytic changes: When two substances with opposite electric charges are brought into contact with each other, the electric charges are neutralized and in doing so, new chemical substances are formed.

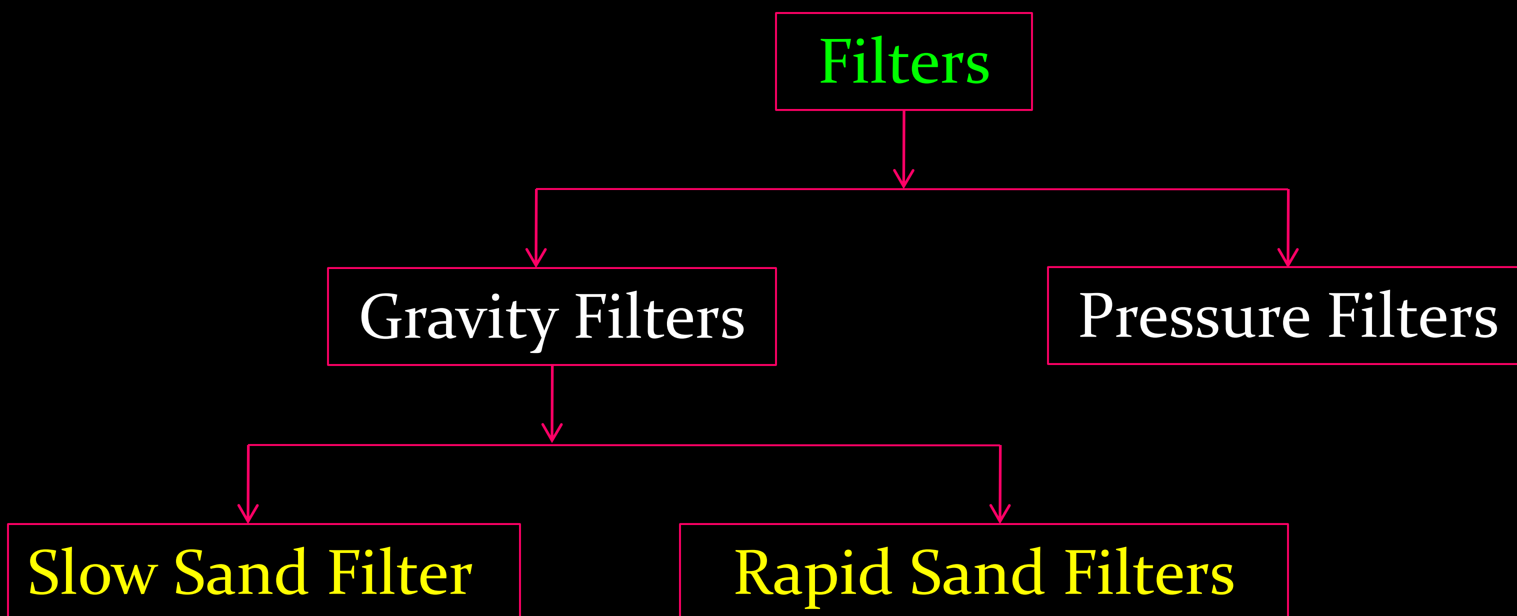
Filtration

Classification of Filters

i. Slow sand filter

ii. Rapid sand filter

iii. Pressure filter



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Lecture -10

Cycle-5, C-day

11-07-2017

Filtration

Slow sand filter: A slow sand filter consists of the following parts:

Enclosure tank:

a. Made of brick masonry coated with waterproof material.

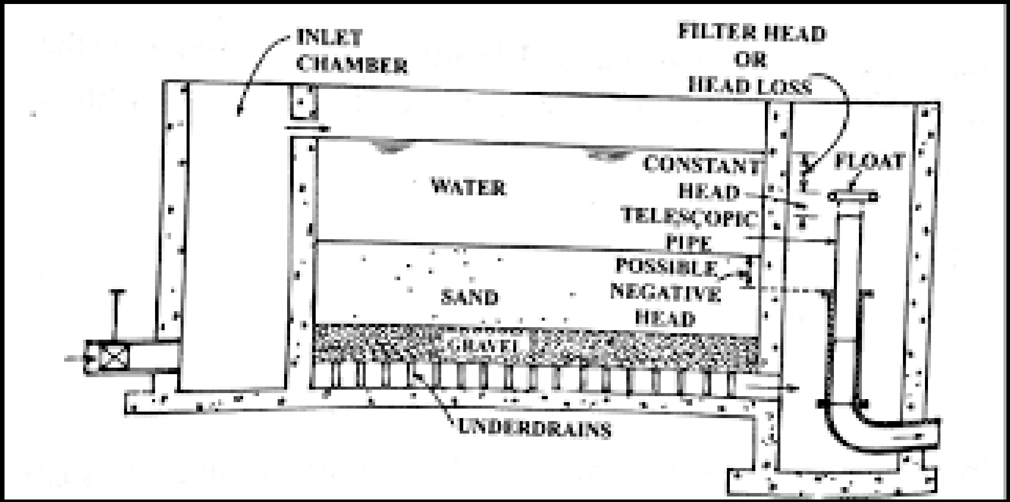
b. Bed slope is about 1 in 100 to 1 in 200 toward the central drain.

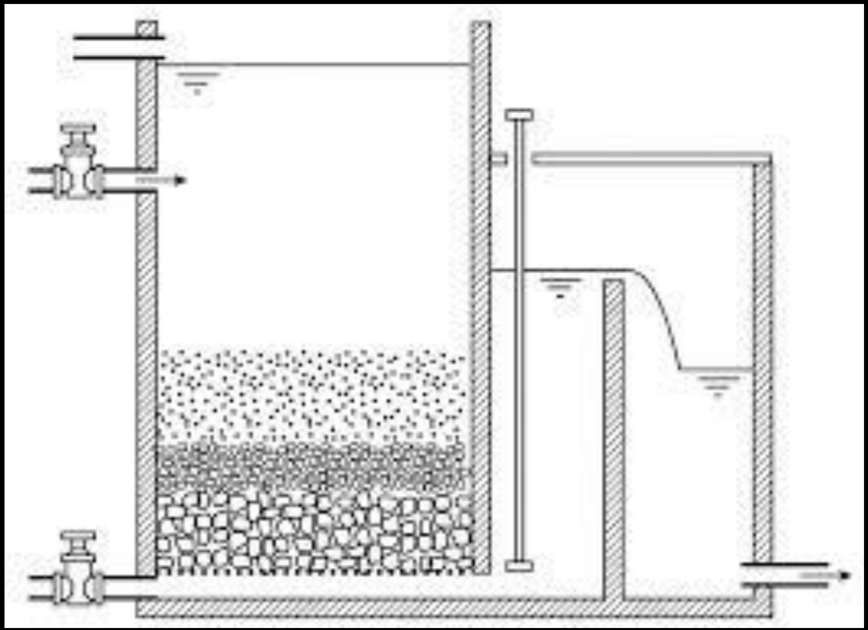
c. Depth of tank is about 2.5 m to 3.5 m.

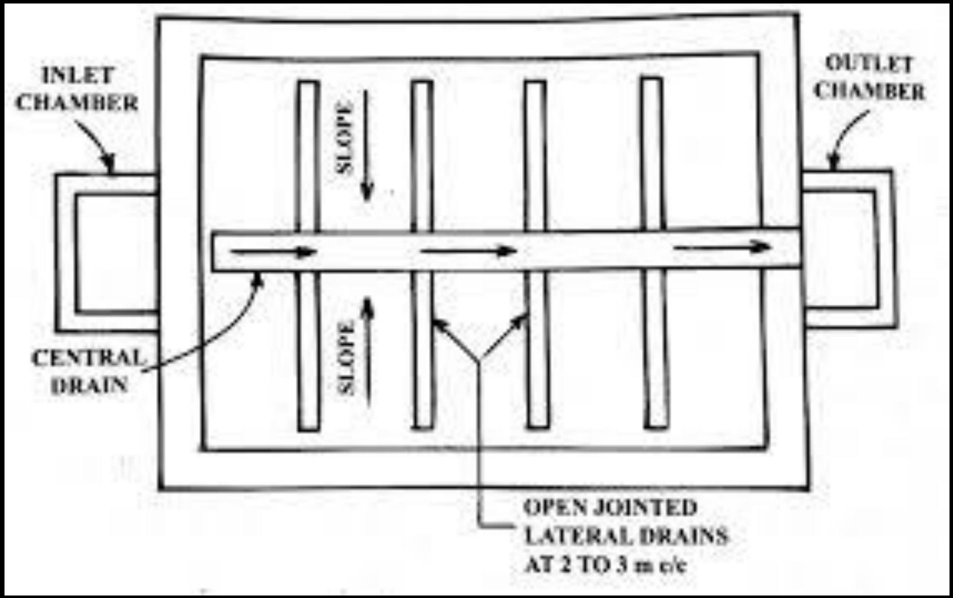
d. Surface area vary from 30 m^2 to 2000 m^2 even more.

Filtration

Slow sand filter







Filtration

Slow sand filter: A slow sand filter consists of the following parts:

Under drainage system:

a. Consists of central drain and lateral drains.
b. Spacing of lateral drain is about 2.5 m to 3.5 m

Base material:

The base material is gravel and it is placed on the top of under drainage system.

Its depth varies from 300 mm to 750 mm in a graded layer of 150 mm from bigger size at the bottom to smaller size of gravel at topmost layer.

Filtration

Slow sand filter: A slow sand filter consists of the following parts:

Base material:

Layer	Depth (mm)	Material size (mm)
Topmost	150	3 to 6
Intermediate	150	6 to 20
	150	20 to 40
Lowest	150	40 to 65
Total	600	

Filtration

Slow sand filter: A slow sand filter consists of the following parts:

Filter media of sand:

- Depth of sand layer – 600 to 900 mm
- Effective size – 0.20 mm to 0.30 mm
- Uniformity coefficient – 2 to 3
- The finer the sand, the better will be the efficiency of filter regarding removal of bacteria.
- But in this case, the output from filter is lowered.

Filtration

Slow sand filter: A slow sand filter consists of the following parts:

Appurtenances:

- Vertical air pipe passing through layer of sand. It helps in proper functioning of filtering layers.
- The device for measuring loss of head, for controlling depth of water above sand layer and for maintaining rate of flow.
- In order to maintain a constant discharge through the filter, an adjustable telescopic tube is usually adopted.

Filtration

Rapid sand filter:

Rules for designing under drainage system

- The ratio of length of lateral drain to its diameter should not exceed 20.
- The cross-sectional area of central drain should be about twice the cross-sectional area of lateral drain.
- The total cross-sectional areas of perforations should be about 0.20% of the total filter area.
- The cross-sectional area of a lateral drain should be about 2 to 4 times the total cross-sectional areas of perforations in it.
- The perforations in the lateral drain should be of diameter 6 mm to 12 mm.
- The spacing of perforations in the lateral drain should vary from 75 mm to 200 mm centre to centre.

Filtration

Rapid sand filter:

Base material:

The base material is gravel and it is placed on the top of under drainage system.

The gravel to be used for base material should be cleaned and free from clay, dust, silt and vegetable mater.

The gravel particles should be durable, hard, round and strong.

Its depth varies from 450 mm to 600 mm in a graded layer of 150 mm from bigger size at the bottom to smaller size of gravel at topmost layer.

Filtration

Rapid sand filter:

Base material:

Layer	Depth (mm)	Material size (mm)
Topmost	150	3 to 6
Intermediate	150	6 to 12
	150	12 to 20
Lowest	150	20 to 40
Total	600	

Filtration

Difference between Slow Sand Filter and Rapid sand filter:

Item	SSF	RSF
Base material of gravel	Varies from 3 to 65 mm in size and 300 to 750 mm in depth.	Varies from 3 to 40 mm in size and 600 to 900 mm in depth.
Coagulation	Not required	Essential
Compactness	Required large area for its installation	Required small area for its installation
Construction	Simple	Complicated as underdrainage system is to be properly designed and constructed.
Cost of operation	Low	High
Economy	High initial cost of both land and material	Cheap and quite economical.
Efficiency	Very efficient in the removal of bacteria but less efficient in the removal of colour and turbidity.	Less efficient in the removal of bacteria but more efficient in the removal of colour and turbidity.
Loss of head	150 mm to 750 mm	3 m to 3.5 m

Filtration

Difference between Slow Sand Filter and Rapid sand filter:

Item	SSF	RSF
Filter material of sand	Effective size varies from 0.2 to 0.3 mm and uniformity coefficient is about 2 to3.	Effective size varies from 0.35 to 0.6 mm and uniformity coefficient is about 1.2 to 1.7.
Flexibility	Not flexible for meeting variation in demand	Quite flexible for reasonable fluctuations in demand.
Period of cleaning	1 to 3 months	2 to 3 days
Method of cleaning	Scraping of top layer of 15 mm to 25 mm thickness. Long and laborious method.	Agitation and back-washing with or without the help of compressed air. Short and speedy method.
Rate of filtration	100 to 200 L/hr/m^2 of filter area	3000 to 6000 L/hr/m^2 of filter area
Skilled supervision	Not essential	Essential
Suitability	Suitable for small towns/villages where land is cheap.	Suitable for big cities where land is high and demand of water is considerable.

Filtration

Pressure filter

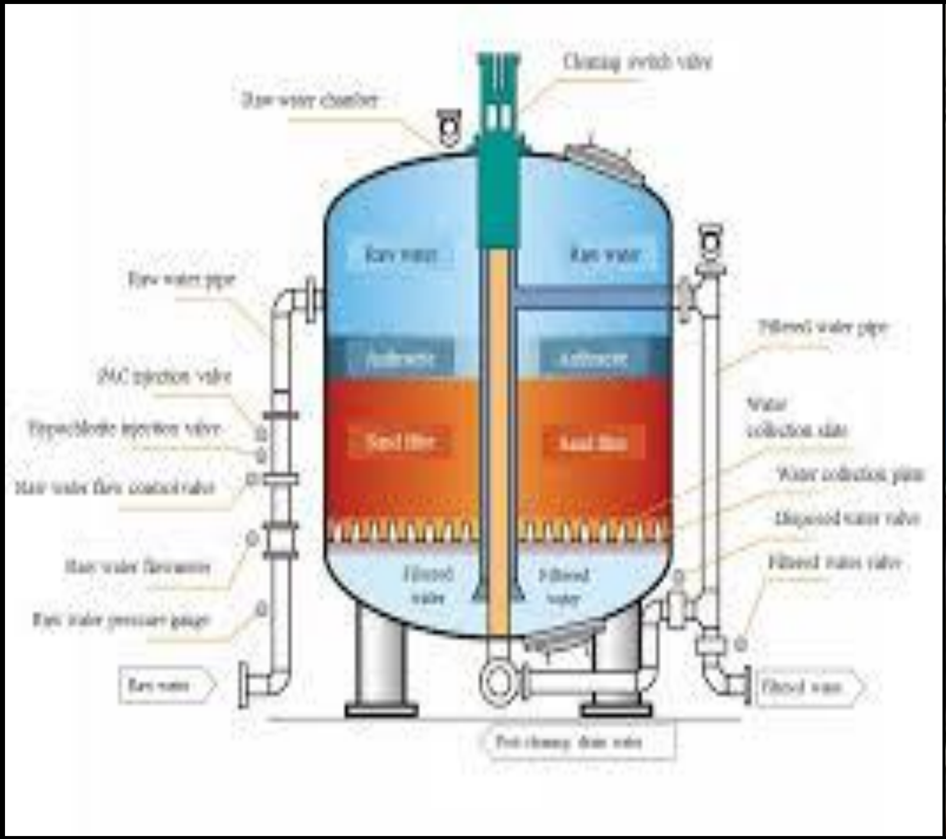
A filter is enclosed in space and the water passes under pressure greater than atmospheric pressure. This pressure can be developed by pumping and it may vary from 0.3 to 0.7 N/mm^2.

Construction: It is closed steel cylinders either horizontal or vertical. The diameter varies from 1.5 to 3.0 m and length or height varies from 3.5 m to 8.0 m. The manholes are provided at top for inspection.

Working: The water mixed with coagulant is directly admitted to the pressure filter. Thus the flocculation takes place inside the filter itself. In normal working condition, all valves are closed except those for raw water and filtered water. Filtered water is collected in storage tank through central drain.

Filtration

Pressure filter



Filtration

Pressure filter

Cleaning: The compressed air may be used to agitate sand grains. The valves for raw water and filtered water are in closed position and those for wash-water and wash-water drain are in open position. The cleaning of pressure filters may be required more frequently.

Rate of filtration: The rate of filtration of pressure filters is high as compared to that of rapid sand filters. It is about 6000 to 15000 litres/hr/m^2 of filter area as compared to that of 3000 to 6000 litres/hr/m^2 of rapid sand filters.

Efficiency: Less efficient than the rapid sand filters in terms of bacterial load, colour and turbidity.

Suitability: Not suitable for public water supply project.

Filtration

Pressure filter

Advantages:

- The unit is compact.
- Modern filter unit does not require manual operation and supervision.
- Flexible in operation because the rate of filtration can be altered by changing the compressed air pressure.
- Dose not require further pumping as the filter water comes out under pressure.
- It is ideal for small estate.
- Require less number of fittings.
- Require small space for installation.
- The sedimentation and coagulation tanks are not required.

Filtration

Pressure filter

Disadvantages:

- It is difficult to keep close watch on the performance.
- It is difficult to repair.
- The overall capacity is small.
- They are costly and hence they cannot be recommended for treating large quantity of water.
- They posses poor efficiency in the removal of bacteria and turbidity.
- They require additional pumps for pumping the water in them.

Environmental Engineering-1

CE3141

Lecture -9

Cycle-8, C-day

02-04-2018

Disinfection of water

The water should be disinfected before it enters the distribution system. The main purpose of disinfection is to prevent contamination of water during its transit from the treatment plant to the place of its consumption.

Disinfection is the process of destruction of pathogenic bacteria which is harmful for health.

On the other hand, Sterilization is the process of destruction and removal of all harmful or harmless bacteria.

The destruction and removal is brought about in several ways:

i. Physical removal through coagulation, sedimentation and filtration
ii. Natural die-away of the organisms in an unfavorable environment during storage,
iii. Destruction by chemicals

Disinfection of water

The materials or substances which are to be used for disinfection are called the disinfectant and the requirement of a good disinfectant are as follows:

i. Its dose should be such that some residual concentration is obtained to grant protection against contamination in the water during its conveyance and retention.
ii. It should be effective in killing all the harmful pathogenic organisms from the water and make it perfectly safe for use.
iii. It should be harmless, unobjectionable, economical and easily available.
iv. It should be of such a nature that its strength or concentration in the treated water can be quickly determined.
v. It should not require skilled labour and costly equipment for its application.
vi. It should take only reasonable time in killing the harmful pathogenic organisms at normal temperature.

Disinfection of water

Minor methods of disinfection:

1. Boiling method
2. Excess Lime treatment
3. Iodine and bromine treatment
4. Ozone treatment
5. Potassium permanganate
6. Silver treatment
7. Ultra-violet ray treatment

In addition to the above chemicals, certain other chemical agents can also be used as disinfectants and they include alcohols, soaps and synthetic detergents, dyes, hydrogen peroxide, various alkalies and acids, etc.

Disinfection of water

Following factors must be considered in applying the disinfection agents:

i. Concentration and type of chemical agent
ii. Contact time
iii. Intensity and nature of physical agent
iv. Nature of suspending liquid
v. Number and types of organisms
vi. Temperature

Disinfection of water

Chlorination

It is the treatment for disinfection by applying chlorine. It is widely used in large scale because of the following factors:

i. It is easy to apply due to relatively high solubility of about 7000 mg/L.
ii. It is readily available as gas, liquid or powder.
iii. It is very toxic to most of the microorganisms and thus metabolic activities are stopped.
iv. It leaves harmless residue in solution, but it provide protection in the distribution system.
v. It produces desired effects which last for a long time.
vi. The treatment by chlorination is cheap and reliable.

Disinfection of water

Chlorination

When chlorine is added to water , it reacts according to the following equation:

$Cl_2 + H_2O \leftrightarrow HOCl + HCl$

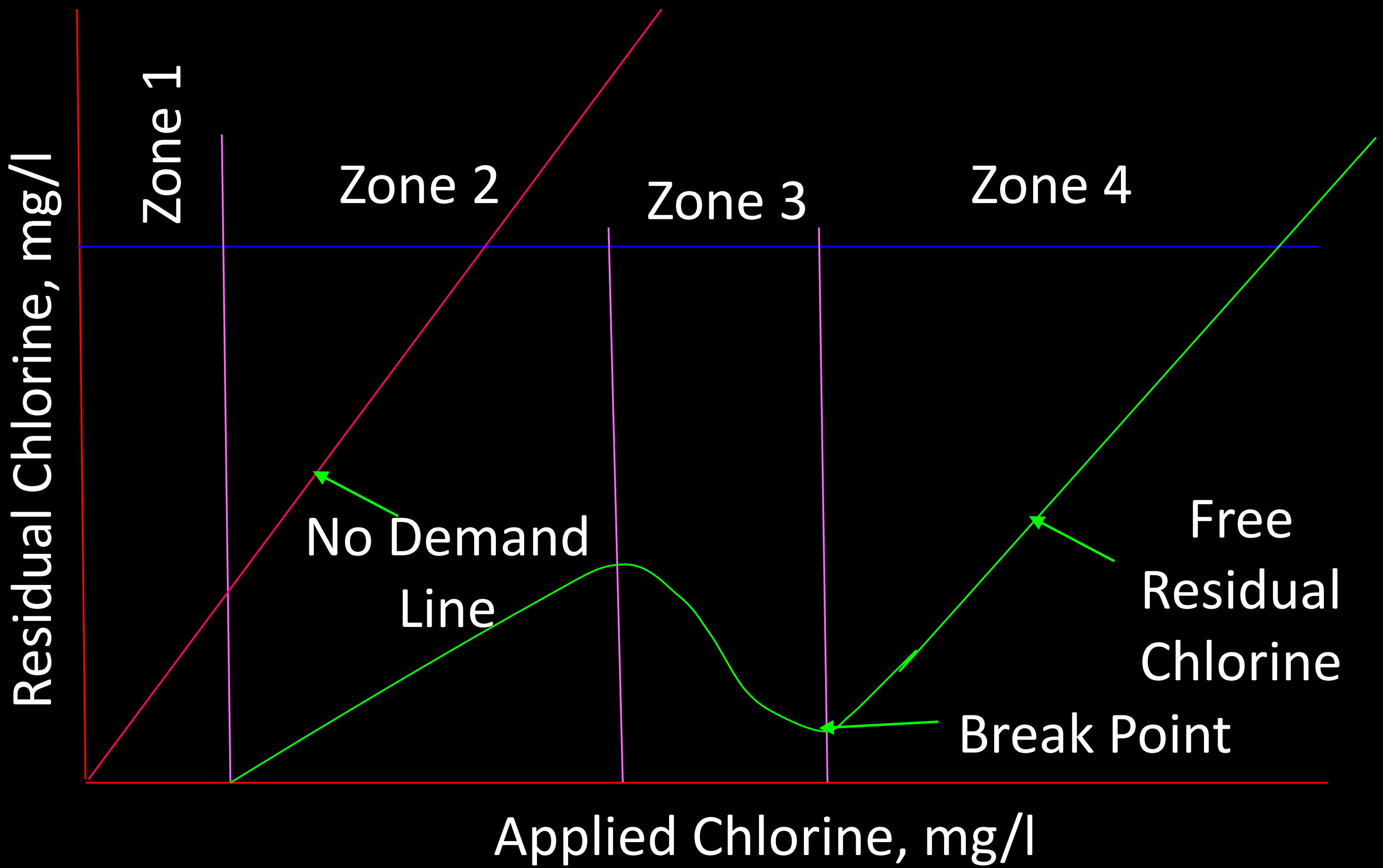
$HOCl \leftrightarrow H^+ + OCl^-$

The hypochlorous acid HOCl dissociates by a reversible equation into hydrogen ions. Both hypochlorous acid HOCl and hypochlorite ions Ocl are responsible for the disinfection of water.

The action of chlorine is dependent to the pH value of water.

The disinfection by chlorine is rapid when pH value of water is below 7.00.

Disinfection of water



Disinfection of water

Chlorination

Zone-1: Destruction of chlorine by reducing compounds.

Zone-2: Formation of chloro-organic compounds and chloramines.

Zone-3: Destruction of chloro-organic compounds and chloramines.

Zone-4: Formation of free available chlorine.

The addition of chlorine at the break (or dip) is termed as break point chlorination or free residual chlorination.

Disinfection of water

Application of Chlorine

Post Chlorination: Chlorine is generally applied after all other treatments have been given to the water supply. This may be termed as post chlorination.

Pre-chlorination: It is the application of chlorine before filtration. Pre-chlorination reduces the bacterial load on filters resulting in increase filter runs, and oxidized excessive organic matter thus removing taste and odour.

Double chlorination: It is the application of chlorine at two points in the treatment process. It is essentially pre-chlorination and post chlorination.

Disinfection of water

Application of Chlorine

The advantages of double chlorination are:

i. Decrease load on the filter
ii. Greater removal of bacteria
iii. Greater factor of safety due to maintaining two chlorination
iv. Control of algae and slimy growth in coagulating basins and filters.

Super chlorination: It is the application of chlorine to water of an excess amount of chlorine. The method is effective in destroying high concentration of tastes and odours in water. Bacterial removal is also high.

Iron and Manganese Removal from water

Iron present in water either as $Fe(HCO_3)_2$ or $FeSO_4$. Manganese is often associated with the iron present in water. Iron and Manganese when present in amounts greater than 0.3 ppm are objectionable because of

i. unpleasant taste and odour
ii. colouring of water
iii. deposits of iron precipitations in pipes.

The method of removal of iron and manganese are based on converting the soluble ferrous and manganese compounds to the insoluble ferric and manganese compounds and removing the precipitates so formed.

Iron and Manganese Removal from water

Removal process

Aeration:

It is an effective method in precipitating out iron when present as $Fe(HCO_3)_2$ and subsequent removal through the process at sedimentation and filtration.

If, however, iron is present as $FeSO_4$, it would be necessary to add lime.

Base exchange process:

The zeolite bed is made of manganese zeolite obtained by treating the base exchange material with manganese sulfate and potassium permanganate.

Chlorination:

Some times chlorination employed either alone or in combined with aeration.

Environmental Engineering-1

CE3141

Lecture -10

Cycle-9, C-day

16-04-2018

Softening of water

There are three general methods used for water softening:

Lime process: This process reduces only carbonate hardness. The principle involved is to neutralize the CO_2 with $Ca(OH)_2$, forming normal carbonates which precipitate out when present in excess and are removed by sedimentation and filtration. This process in known as Clark Process.

Reactions

$CO_2 + Ca(OH)_2 = CaCO_3 + H_2O$

$Ca(HCO_3)_2 + Ca(OH)_2 = 2CaCO_3 + 2H_2O$

$Mg(HCO_3)_2 + Ca(OH)_2 = CaCO_3 + MgCO_3 + 2H_2O$

Softening of water

Lime and Soda ash process: Lime has no effect on sulphates of Ca and Mg, which are responsible for causing most of the non-carbonate hardness found in natural water. However, by the use of soda ash (Na2CO3), the non-carbonate hardness can be removed.

Reactions

(i) $CaSO_4 + Na_2CO_3 = CaCO_3 + NaSO_4$

(ii) $MgSO_4 + Ca(OH)_2 = Mg(OH)_2 + CaSO_4$

$CaSO_4 + Na_2CO_3 = CaCO_3 + NaSO_4$

Softening of water

Base Exchange process:

In this process, hard water is passed through a bed of zeolite sand (complex silicates of aluminium and sodium) whereby it exchanges its Ca and Mg for the Na in zeolite until Na become exhausted.

Reactions

CaSO4

MgSO4 Na_2CO_3

MgCO3 + $Na_2Al(SiO_3)_2$ = $[Ca/Mg]Al(SiO_3)_2$ + Na_2SO_4

CaCO3 2NaCl

CaCl

[Ca/Mg]Al(SiO3)2 + 2NaCl = Na2Al(SiO3)2 + [CaCl/MgCl]

Desalinization of water

There are so many processes of removing salts from the water. These are:

1. Distillation
2. Freezing
3. Demineralization
4. Electrodialysis
5. Membrane filtration

Arsenic Removal

Co-precipitation and Adsorption Processes

Water treatment with coagulant such as $Al_2(SO_4)_3.18H_2O$, $FeCl_3$, $Fe_2(SO4)_3.7H_2O$ are effective in removing arsenic from water.

In the coagulation-flocculation process Al2(SO4)3.18H2O, FeCl2 or Fe2(SO4)3.7H2O is added and dissolve in water under efficient stirring for at least 1 minute micro-flocs are formed rapidly.

The water is then gently stirred for few minutes for agglomeration of micro-flocs into larger, easily settable flocs.

During this flocculation process al kinds of micro-particles and negatively charged ions are attached to the flocs by electrostatic attachment.

Arsenic is also adsorbed onto coagulated flocs.

Arsenic Removal

Co-precipitation and Adsorption Processes

Oxidation of As(III) to As(v):

$H_3AsO_3 + HOCl = HAsO_4^{--} + Cl^- + 3H^+$

$3H_3AsO_3 + 2KMnO_4 = 3HAsO_4^{--} + 2MnO_2^+ + 4K^+ + H_2O$

Alum dissolution:

$Al_2(SO_4)_3.18H_2O = 2Al^{+++} + 3SO_4^{++} + 18H_2O$

Aluminium precipitation (acidic):

$2Al^{+++} + 6H_2O = 2Al(OH)_3 + 6H^+$

Co-precipitation:

$H_2AsO_4^- + Al(OH)_3$ = Al-As (complex) + Other products

Arsenic adsorbed on $Al(OH)_3$ as Al-As complex is removed by sedimentation. Filtration may also be required to ensure complete removal of all flocs.

Arsenic Removal

Other Methods of Arsenic Removal

i. Lime treatment
ii. Naturally occurring iron
iii. Sorptive media (Activated alumina, Al_2O_3)
iv. Ion exchange
v. Membrane techniques
vi. Microbial process
vii. Chemical packages
viii. Cartridge filters

Arsenic Removal

Lime treatment

- Water treatment by addition of CaO or $Ca(OH)_2$ also removes arsenic.
- The precipitated $Ca(OH)_2$ acts as a sorbing flocculent for arsenic.
- The arsenic removal by lime is relatively low, usually between 40 to 70%.
- The highest removal is achieved at pH 10.6 to 11.4.
- Obviously water treated by lime would require secondary treatment in order to adjust pH to an acceptable level.

Arsenic Removal

Naturally occuring iron

- The iron precipitates [$Fe(OH)_3$] formed by oxidation of dissolved iron [$Fe(OH)_2$] present in ground water have an affinity for the adsorption of arsenic.
- Only aeration and settling of tubewell water rich in dissolved iron has been found to remove up to 25% of the arsenic present.
- The iron removal plants (IRPs) in Bangladesh constructed on the principles of aeration, sedimentation and filtration have been found to remove arsenic without any added chemical.
- The conventional IRPs more or less work as arsenic removal plants (ARPs) as well.

Arsenic Removal

Sorptive media (Activated alumina, Al_2O_3)

- Activated alumina, Al_2O_3, having a good sorptive surface, is an effective medium for arsenic removal.
- When water passes through a packed column of activated alumina, the impurities, including arsenic, present in water are adsorbed on the surfaces of activated alumina grains.
- The column become gradually saturated with impurities and needs to regenerate.

Arsenic Removal

Ion exchange

- This process is similar to activated alumina, but the medium is a synthetic resin of more well-defined ion exchange capacity.
- The process is normally used for removal of specific undesirable cations or anions from water.
- As the resin become exhausted, it needs to be regenerated.

Arsenic exchange:

$2R\text{-}Cl + HAsO4 = R2HAsO4 + 2Cl$

Regeneration:

$R2HAsO4 + 2Na + 2Cl = 2R\text{-}Cl + HAsO4 + 2Na$

Arsenic Removal

Membrane Techniques

- Membrane techniques like reverse osmosis and electrodialysis are capable of removing all kinds of dissolved solids including arsenic from water.
- In this process water is allowed to pass through special filter media which physically retain the impurities present in water.
- The water, for treatment by membrane techniques, must be free from suspended solids and the arsenic in water must be in pentavalent form.

Arsenic Removal

Microbial process

Microbial removal of aresenic is based on:

- Microbial oxidation of As(III) to As(V) to facilitate its removal by conventional arsenic removal process.
- Bio-accumulation of arsenic from the surrounding water environment.

There are number of micro-organisms capable of oxidizing arsenic at neutral pH. The common iron bacteria used to oxidize ferrous iron to ferric iron can oxidize as well as absorb arsenic. Removal of trace metals from water through accumulation in algae is well-recognized.

Arsenic Removal

Chemical packages

- Different types of chemical packages are available in the form of tea bags, small packets and powder form for the removal of arsenic from drinking water.
- The principles involved in arsenic removal by these chemicals involve oxidation, sorption and co-precipitation.

Arsenic Removal

Cartridge filters

Filter units with cartridges filled with sorptive media or ion-exchange resins are already available in market.

- These units remove arsenic like any other dissolved ions present in water.
- These units are not suitable for water having high impurities or containing iron.
- Presence of iron having higher affinity than arsenic can quickly saturate the media, requiring regeneration or replacement.
- The initial and operating costs are high and beyond the reach of the common people.

Leak Detection & Control

Lecture -17

Cycle-13, C-day

10-07-2018

Leak Detection

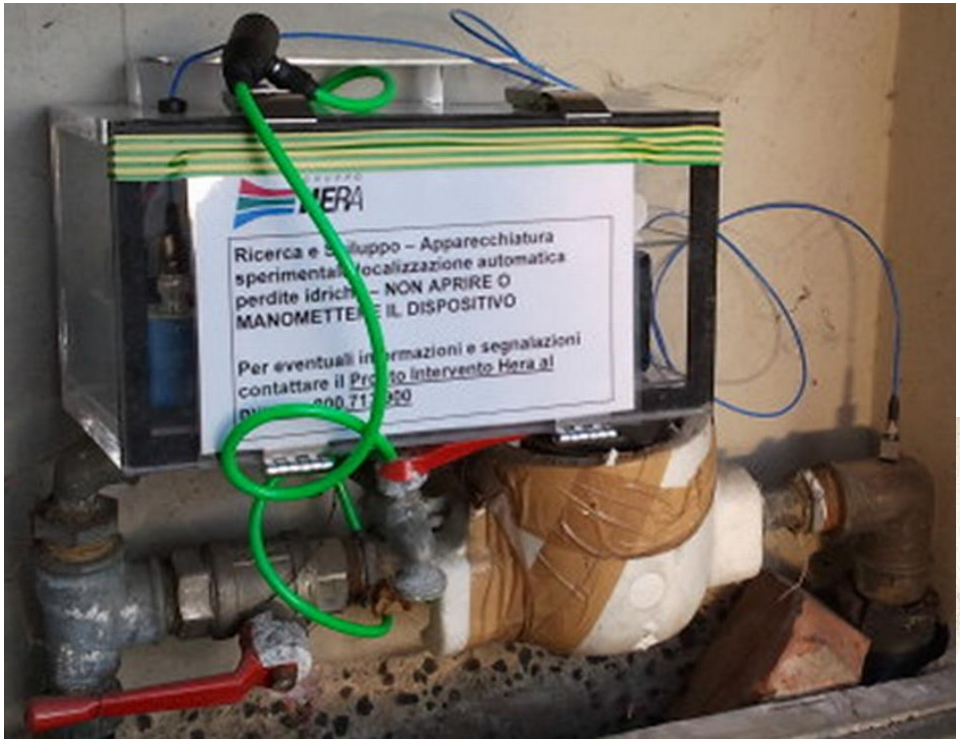
- Early detection of invisible leaks in a water distribution network is of great significance to most water utilities.
- The delay in detection and repair of a failed water main can lead to a large amount of water loss and serious damage to infrastructure near the failure.
- The control of water leaks in water distribution networks represents a critical issue for all utilities involved in drinking water supply.

Leak Detection

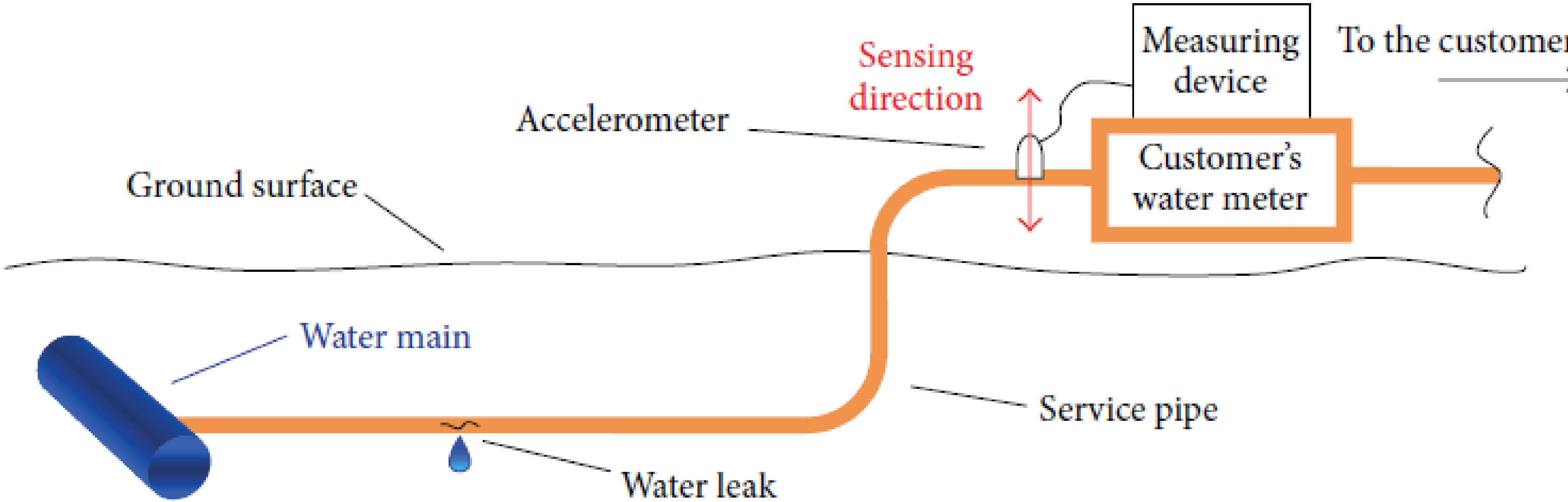
Methods of Leak Detection

- Head Loss Ratios
- Vibration monitoring
- Sound monitoring
- *Water Audits*

Leak Detection



Leak Detection



Leak Detection

Water Audits

Water audits determine the amount of water loss in the distribution system. They can be performed on a network-wide basis or district by district.

Network-wide audits provide an overall picture of water losses in the distribution system as a whole. These audits require detailed accounting of water flow into and out of the distribution system, usually based on past meter records and flow meter accuracy checks.

The comprehensive nature of network-wide audits entails significant effort, especially for large systems.

Leak Detection

For district audits, the distribution system is divided into small districts or zones having approximately 20 to 30 km of water main.

Districts are isolated individually by turning off the appropriate valves except at control points where portable flow meters are installed to measure water flow over a 24-hour period.

To determine if excessive leakage exists, the ratio between the nighttime minimum rate and the average daily rate of water flow is compared to “normal” ratios or to previously measured ratios of the same district.

Flow rates from any 24-hour commercial use should be subtracted from the measured flow rates.

Leak Detection

Alternatively, if all service connections in the water system are metered, more accurate information about leakage can be obtained by monitoring water flow and usage in the isolated district over an extended time period.

Areas of excessive leakage in a district can be bracketed by "step testing."

This is done by subdividing the district itself and then measuring flow rates while turning off valves to cut off different subdivisions in succession.

A significantly lower flow rate indicates excessive leakage in the last subdivision that is shut off.

Leak Detection

Leak-Detection Surveys

- In areas that have been identified as having excessive leakage, leaks are commonly pinpointed using acoustic devices.
- These devices detect the sound or vibration induced by water leaking from pressurized pipes.
- Leak sounds are transmitted through the pipe itself over significant distances (depending on pipe size and type), and through the surrounding soil in the immediate area of the leak.

Leak Detection

Leak-Detection Surveys

- Initially, leak detection crews roughly bracket leaks in water-distribution systems by listening on all accessible contact points with the distribution system such as fire hydrants and valves.
- Suspected leaks are then pinpointed by listening on the ground surface directly above the pipe at very close intervals (about 1 m).
- Alternatively, suspected leaks can be pinpointed automatically by using modern leak noise correlators, which have become popular in recent years.
- Normally, leak noise correlators are more efficient and more accurate than listening devices.

Leak Detection

Leak-Detection Surveys

- Leaks could also be detected using several non-acoustic techniques such as tracer gas, infrared imaging, and ground-penetrating radar.
- The use of these techniques for this purpose, however, is still very limited and their effectiveness is not as well established as that of acoustic methods.

Detection Equipment and Techniques

Listing Devices

- These devices include listening rods, aquaphones, and geophones or ground microphones, and may be either mechanical or electronic.
- They use sensitive mechanisms or materials such as piezoelectric elements to sense leak-induced sound or vibration.
- Modern electronic devices have signal amplifiers and noise filters to make the leak signal stand out.
- The operation of listening devices is usually straightforward, but their effectiveness depends on the experience of the user.

Detection Equipment and Techniques

Typical listening devices include listening rods

Ground microphones

Leak Noise Correlators

- These are portable microprocessor-based devices that pinpoint leaks automatically based on the cross-correlation method (Figure 1).
- In this method, acoustic leak signals are measured with vibration sensors or hydrophones at two pipe contact points (usually fire hydrants or valves) that bracket the location of a suspected leak.
- Leak signals are transmitted from the sensors to the correlator wirelessly.
- The leak is in most cases located asymmetrically between measurement points and consequently there is a time lag between the measured leak signals.

Leak Noise Correlators

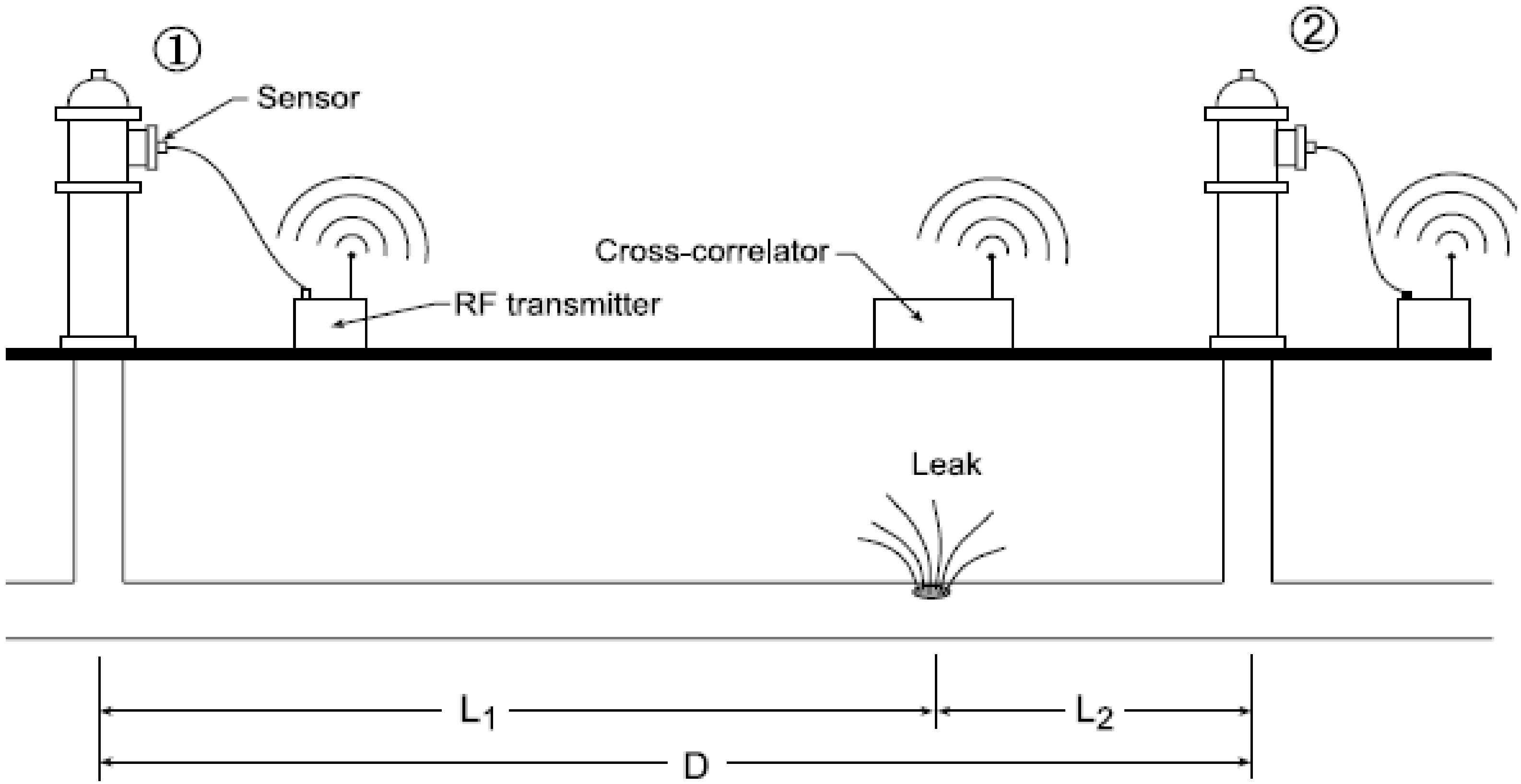
- The time lag is found from the cross-correlation function of the leak signals.
- In the presence of a leak, the cross correlation function has a distinct peak at the time shift between leak signals.
- The location of the leak is calculated based on an algebraic relationship between the time lag, the sensor-to-sensor distance, and the propagation velocity of sound waves in the pipe (Figure 2).
- The distance between sensors is measured on site or read from distribution system maps.
- Propagation velocities for various pipe types and sizes are usually available in most commercial devices, or they can be measured easily on site.

Leak Noise Correlators

A leak noise correlator is a portable microprocessor-based device that pinpoints leaks automatically.

Figure -1

Leak Noise Correlators



Arrival time of signal ① = $T_1 = L_1 / V$, where V is sound propagation velocity in pipe

Arrival time of signal ② = $T_2 = L_2 / V$

Time lag between signals ① and ② = $\Delta T = T_2 - T_1 = (L_2 - L_1) / V$

$L_2 = D - L_1 \;\; \rightarrow \;\; \Delta T = (D - 2\,L_1) / V \;\; \rightarrow \;\; L_1 = (D - V\Delta T) / 2$

Figure -2

Leak Detection

Tracer Gas Technique

- With this technique, a non-toxic, water insoluble and lighter-than-air gas, such as helium or hydrogen, is injected into an isolated segment of a water pipe.
- The gas escapes at a leak opening and then, being lighter than air, permeates to the surface through the soil and pavement.
- The leak is located by scanning the ground surface directly above the pipe with a highly sensitive gas detector.

Leak Detection

Thermography

The principle behind the use of thermography for leak detection is that water leaking from an underground pipe changes the thermal characteristics of the adjacent soil, for example, making it a more effective heat sink than the surrounding dry soil.

The resulting thermal anomalies above pipes are detected with handheld, or vehicle or airplane-mounted infrared cameras.

Leak Detection

Ground-penetrating Radar

- Radar can be used to locate leaks in buried water pipes either by detecting voids in the soil created by leaking water as it circulates near the pipe, or by detecting segments of pipe which appear deeper than they are because of the increase in the dielectric constant of adjacent soil saturated by leaking water.
- Ground-penetrating radar waves are partially reflected back to the ground surface when they encounter an anomaly in dielectric properties, for example, a void or pipe.
- An image of the size and shape of the object is formed by radar time-traces obtained by scanning the ground surface.
- The time lag between transmitted and reflected radar waves determines the depth of the reflecting object.

Solar Stills

Solar Still Background

The first known use of stills dates back to 1551 when it was used by Arab alchemists. Other scientists and naturalists used stills over the coming centuries including Della Porta (1589), Lavoisier (1862), and Mauchot (1869).

The first "conventional" solar still plant was built in 1872 by the Swedish engineer Charles Wilson in the mining community of Las Salinas. This still was a large basin-type still used for supplying fresh water using brackish feedwater to a nitrate mining community. The plant used wooden bays which had blackened bottoms using logwood dye and alum. The total area of the distillation plant was 4,700 square meters. On a typical summer day this plant produced 4.9 kg of distilled water per square meter of still surface, or more than 23,000 liters per day. This first stills plant was in operation for 40 years!

Solar Stills

Emergency Survival Tool

Fortunately, there is an emergency survival technique for gathering water from our driest deserts during their most brutal seasons. **It is commonly known as the solar still**.

Make your own distilled water from stream or lake water, salt water, or even brackish, dirty water, using Solar Still Plans. With just a few basic building materials, a sheet of glass and some sunshine, you can purify your own water at no cost and with minimal effort.

Solar Still Background

Still Operation

- A solar still operates on the same principle as rainwater: evaporation and condensation.
- The water from the oceans evaporates, only to cool, condense, and return to earth as rain.
- When the water evaporates, it removes only pure water and leaves all contaminants behind.
- Solar stills mimic this natural process.

Solar Stills

The solar still functions under the general principle of the "greenhouse effect". Solar energy heats the ground by passing through a clear plastic barrier. Moisture from the soil then evaporates, rises and condenses on the underside of the plastic barrier above.

The still also has the ability to purify tainted water. In fact, it condenses pure water from just about anything. Even urine will produce clean, drinkable water.

CAUTION: One fluid never to be used is radiator fluid, as its toxins will vaporize and poison the water.

How to Make a Solar Still

Example-1:

Materials

There are only 2 essential components to constructing the solar still- a container to catch the water and a 6 x 6-foots sheet of clear plastic. A shovel or trowel, a length of plastic tube and tape are all optional.

The container can be a collapsible cup, an empty plastic bottle, a small cooking pot or just about anything with a large enough opening to catch falling drops of water.

The sheet of clear plastic can be a ground cloth used under tents when backpacking or a thin painting drop cloth. Both work well as long as there are no tears or holes.

A 6-foot length of flexible plastic tubing will allow you to drink accumulated water without needing to break down the solar still, inevitably affecting its efficiency.

How to Make a Solar Still

Construction

❖Dig a pit approximately 4 feet wide and 3 feet deep. Use a shovel, hand trowel, a digging stick or even your hands in soft soil or sand. Look for a sandy wash or a depression where rainwater might collect.

❖In the center of the pit, dig another small hole deep enough for the water container.

❖Place the container inside, then run the tubing from the container to the outside of the pit. If there is tape available, tape the tubing to the inside of the container.

❖Blanket the pit with the plastic sheet, evenly on all sides, but not touching the bottom of the pit. Anchor the corners with rocks.

How to Make a Solar Still

Construction

❖Find a small rounded rock to place in the center of the sheet, over the water container. This will keep the plastic centered and control any flapping from the wind. Gently push down on the center weight until the sides slope to a 45^{o} angle.

❖Next, secure the edges of the plastic sheet with rocks and dirt. Make sure there are no places where moisture can escape.

❖Close the tubing end with a knot, or double it and tie it closed.

How to Make a Solar Still

Within two hours, the air inside the still will become saturated with moisture and begin to condense onto the underside of the plastic sheeting. Because of the angle of the plastic, water will run down towards the center. Finally, drops will gather and fall from the apex down into the water container. As the container fills, simply sip fresh, sterile water from the plastic tubing.

How to Make a Solar Still

The solar still only takes about an hour to build. If constructed correctly, it can yield about a quart of water a day. And although the palm trees may be noticeably absent, you will have made your very own oasis in the desert, quicker than Hollywood could.

CAUTION: Solar stills are not a primary water source, nor a substitute for carrying adequate amounts of water in the desert. Always carry a minimum of one gallon per day per person.

How to Make a Solar Still

Example-2

A solar still works like a salt evaporation pond, except that the water that invisibly evaporates is extracted from the air; the minerals and other impurities are left behind and discarded. As the hot, moisture-laden air rises up to the slanting sheet of relatively cool glass sealed to the box, water condenses out in the form of small droplets that cling to the glass. As these droplets get heavier, they roll down the glass to the collector tube at the bottom and then out to the jug.

How to Make a Solar Still

The box is built from 3/4 " BC-grade plywood, painted black on the inside to absorb heat. We used a double layer of plywood on the sides to resist warping and to help insulate the box, with an insulated door at the back and a sheet of glass on top.

How to Make a Solar Still

Example-3

We chose to paint the inside black and use two large glass baking pans to hold the water. Glass baking pans are a safe, inexpensive container for dirty or salty water, and they can easily be removed for cleaning. We used two 10 x 15" pans, which hold up to 8 quarts of water when full. To increase the capacity of the still, just increase the size of the wooden box and add more pans.

The operation of the distiller is simple. As the temperature inside the box rises, water in the pans heats up and evaporates, rising up to the angled glass, where it slowly runs down to the collector tube and then out to a container.

How to Make a Solar Still

Example-3

The runoff tube is made from 1" PEX tubing. Stainless steel can also be used. However, use caution with other materials—if in doubt, boil a piece of the material in tap water for 10 minutes, then taste the water after it cools to see if it added any flavor. If it did, don't use it.

Turn undrinkable water into pure, crystal-clear distilled water with a home-built solar still.

View step-by-step photos of how to make a solar still in the Image Gallery as well as this PDF of the DIY Solar Still Plans.

How to Make a Solar Still

Example-3

1. **Mark and cut the plywood pieces** according to the cutting list. Cut the angled end pieces with a circular saw or tablesaw set to a 9 degree angle.

2. **Cut the insulation** the same size as the plywood base, then screw both to the 2 x 4 supports with 2 1/2" screws.

3. **Screw the first layer** of front and side pieces to the base and to each other, then add the back piece. Predrill the screws with a countersink bit.

4. **Glue and screw** the remaining front and side pieces on, using clamps to hold them together as you predrill and screw. Use 1 1/4" screws to laminate the pieces together and 2" screws to join the corners.

How to Make a Solar Still

Example-3

5. **Glue and screw** the hinged door pieces together, aligning the bottom and side edges, then set the door in position and screw on the hinges. Add a pull or knob at the center.

6. **Paint the inside of the box** with black high-temperature paint. Cover the back and the door with reflective foil glued with contact cement. Let the paint dry for several days so that all the solvents evaporate off.

7. **Apply weatherseal around** the edges of the hinged door to make the door airtight.

8. **Drill a hole** for the PEX drain. The top of the PEX is 1/2" down from the top edge. Clamp a scrap piece to the inside so the drill bit doesn’t splinter the wood when it goes through.

How to Make a Solar Still

Example-3

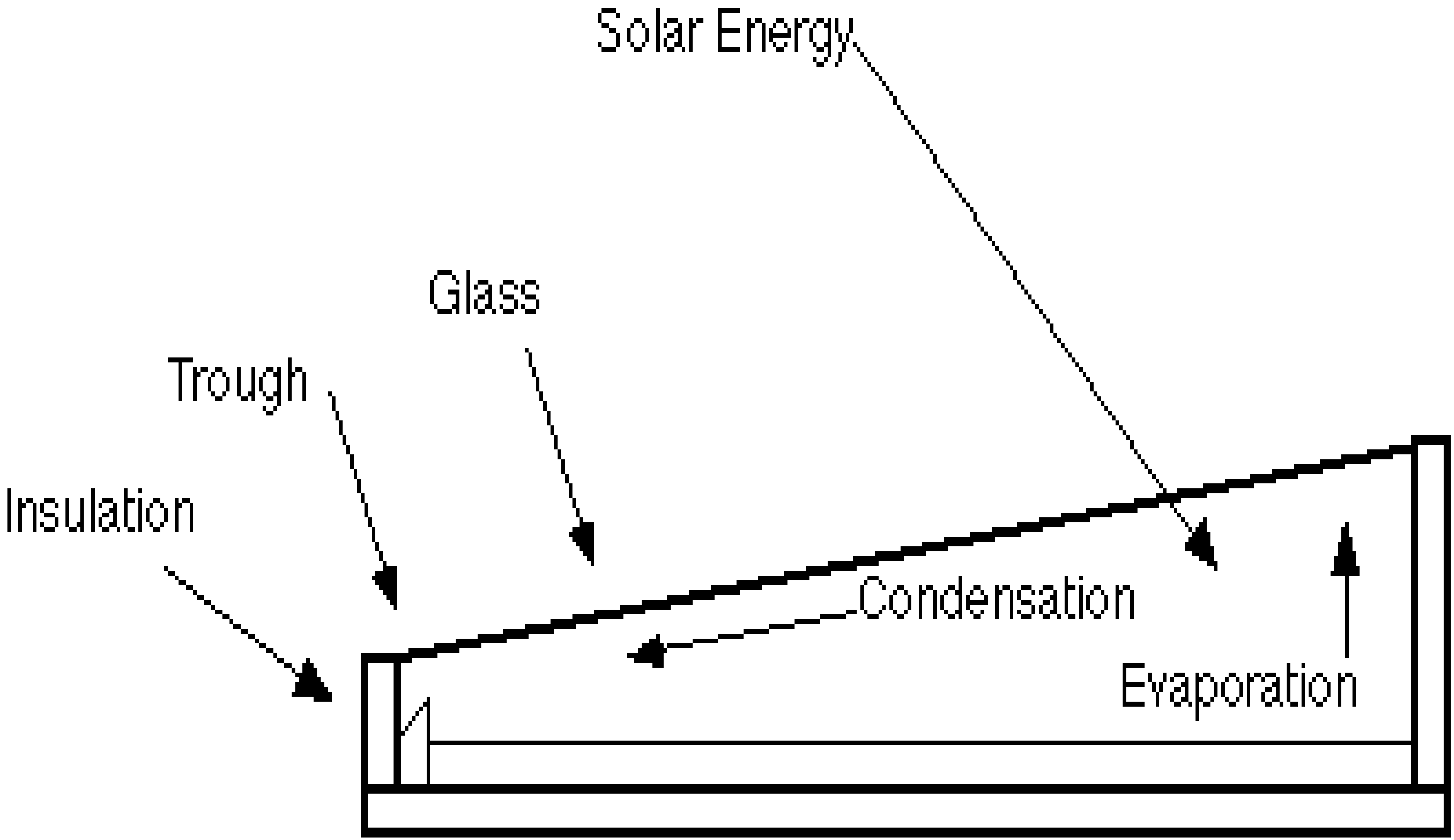
9. **Mark the first 19" of PEX,** then cut it in half with a utility knife. Score it lightly at first to establish the cut lines.

10. **Drill three 1/8" holes** in the side of the PEX for screws, then insert the PEX through the hole. Butt it tight against the other side, then screw it in place, sloping it about 1/4".

11. **Wipe a thick bead of silicone caulk** along the top edge of the PEX to seal it against the plywood.

12. **Shim the box level** and tack a temporary stop to the top edge to make it easy to place the glass without smearing the caulk. Spread a generous bead of caulk on all the edges, then lay the glass in place. Tape it down around the edges with painter’s tape, then let it set up overnight.

Solar Stills



Have a Break for Next Day

SOCIAL FACTORS ON WATER SUPPLY AND SANITATION

Introduction

12.1 Socio-Economic aspects of WSS

The socio-economic factors of an area, a community or the country as a whole are considered very much essential elements in planning and designing the development projects. Development efforts obviously target a section of people; a particular community or area based masses. It is, therefore, is an urgent issue to know the clientele's socio-economic status, need of the clienteles, demands of them, their absorptive capacity, the gender issues involved and the potential adverse impact on vulnerable groups. If these factors are known to the planners, decision-makers and project designers, it becomes easier to set the project goal, target and strategic approaches.

12.2 Socio-Economy of rural and urban Bangladesh

With an area of only 144000 sq.kms. and a population of 130 million, Bangladesh is one of the most densely populated countries of the world (WB, 1997). In this semitropical, predominantly rural country about 48% of rural and 44% of the urban population live below the poverty line, a reduction by almost half from 15 years ago (BBS 1993). Per capita Gross National Product in 1993 was US$220 (WB, 1993). Household spend 59% of their income on food, and 60% of children below 5 years of age are malnourished (WB). The country ranked 147 out of 173 countries in the world as per UNDP human development index.

The annual growth of the Gross National Product in the 1980 – 1991 period exceeded the population growth rate and averaged 4.3%, an increase from 2.3% in the previous decade (WB, 1993). While agricultural production averaged 2.6% growth annually during that period, its share shrank from 55% to 36% of GDP, while industry grew from 9% to 16% and services from 37% to 48% IWB, 1993).

12.3 Demographic characteristics

The population in1996 was approximately 111 million, and expected to grow to 131 million by the year 2000. Already an estimate shows that it has further increased to 121.8 million of which 62.4 million male and 59.4 million female (estimated as on January 1996). The overall growth rate has dropped from 2.7% in 1970 – 1980 to 2.2% between 1980 – 1991 and 1.84% (estimated as on

January 1996). The urban population has been growing faster than the rural population due to migration, increasing from 8% of the population in 1970 to 17%, or 21.6 million people, by 19991 (WB, 1993). About 48% of the urban populations live in Dhaka, Chittagong, Khulna and Rajshahi, and 40% live in the 180 pourashavas.

12.4 Power structure

12.5 Cultural issues (traits)

12.6 Influence of socio-economic aspects on community water supply and sanitation

12.7 Rural leadership

12.8 Local government structure

Local administration is called the local government. Local government structure is framed in four stages. These are started from root level: village level, union level, upa-zila (thana) level and zila (district) level administration. There are roughly 85650 villages, about 4450 unions, about 470 upa-zilas outside the city corporation areas and 64 zilas in Bangladesh.

The Zila parishad (district councils) are involved in planning, implementation and monitoring of development activities in districts, including approval of plans from the thana level. Thana Unnayan Samannay Committees (thana development coordination committees), consisting of union parishad chairmen and government officers, are responsible for planning, implementation and monitoring at the thana level. The union parishads (union councils) are elected bodies responsible for promoting development activities.

The six major cities are governed by City Corporations, and there are about 300 other urban areas government by municipal authorities called pourashavas. Pourashavas are elected municipal councils under the administrative control of the Ministry of Local Government and Rural Development (MLGRD). The new local government structure is increasing the number of levels of directly elected local authorities and decentralizing responsibilities to a greater extent under the Local Government Act.

12.9 Concept of community participation

Community participation means the involvement of general peoples of the community in any development and social activities in the community. The ownership of community people of any development project ensures the community participation. Without active and direct participation of community any project must be failed. So the concept of community participation describes the participation of community people in every steps of management i.e., planning, implementation, operation and maintenance of the project.

12.9.1 Community participation in WSS

People participation in decision making and local ownership results in effective and sustainable development process. This belief has played a central part in the shift in institutional strategies from supply-driven to demand-driven approaches, which response to the felt needs and aspirations of users, especially the poor. However, quantitative evidence of the efficacy of participation in determining project effectiveness, relative to other factors, has been missing.

In most developing countries, public sector agencies provide rural infrastructure. Poor public sector performance has led to a widespread search for institutional alternatives and means to increase the accountability of the public sector. In the rural water sub-sector, the search has been for strategies to increase users' "exit" and "voice" options and to restructure the sector so that suppliers have incentives to match the demand of users.

The problematic issue therefore is not technology and construction but rules and regulations-institutions and organization. The first challenge for agencies is to create an incentive for staff to work in partnership with hundreds of communities. The second task for agencies staff is to enable communities to make informed choices, and manage and choose from a menu of water supply options (technology and management) that the agency offers.

Although the agency task has changed dramatically over the years, that fact has seldom been recognized or acknowledged by the agencies themselves. Hence, the agencies and their competency, organizations, structure, and management-by-blueprint style have remained largely the same. The mismatch between the task and the mandate, ability and competence of the agency has resulted in many successful government attempts to induce participation. The key question, then, is: How can organizations change to induce participation in collective action?

Beneficiary participation can be brought about in several ways: directly, through participation, in decision making; indirectly, through leaders; or through representation on committees or boards. Participation of beneficiaries can be facilitated through extension workers, local government units, non-governmental organizations (NGOs), and the private sector. Many factors influence beneficiary participation, including the immediate and broader policy context; client characteristics (including felt need); and agency characteristics, such as flexibility, responsiveness to client, and willingness to invest in the sound organization of communities. In this regard, the following questions are to be addressed:

- Does people's participation contribute to project effectiveness?
- How important is this contribution, relative to other factors?
- What factors and strategies influence participation in collective action?
- What are the lessons for the design of large-scale project?
- What are the implications for policy reform?

12.9.2 What is participation?

Participation is a process through which stakeholders influence and share control over development initiatives and the decision and resources, which affect them. Participation is also a voluntary process by which people, include the disadvantaged (in income, gender, ethnicity, or education), influence or control the decisions that affect them. The essence of participation is exercising voice and choice.

~~Definitions of participation about (Cohen and Uphoff 1977, Korten 1980; Paul 1987; and Ghai and Hewit de Alacantara 1990);~~ All of them include in some measure the notions of contributing, influencing, sharing, or redistributing power and of control, resources, benefits , knowledge, and skills to be gained through beneficiary involvement in decision making. There is also many debates among participations and in the literature about whether participation is a means or an end, or both (World Bank 1992; Picciotto 1992).

12.9.3 Defining community participation

How community participation is defined will depend upon on the objectives and the needs and possibilities for participation. No matter whether a maximum or minimum community involvement is developed, some central questions will have to be answered and tested in the field; who participates in which phases and decisions, how and to what degree, and what effects are desired for the programme and the community. These questions are not yet generally considered by the agencies solely responsible for the planning and implementation of water supply and sanitation programmes.

Some of the international organizations have taken the working definition of community participation from three dimensions. These are involvement of all those affected in *decision making* about what should be done and how; Mass contribution to the development effort, i.e. to the *implementation* of the decision; and sharing in the *benefits* of the programmes (WB, 1976). Since equitable share of benefits is essential, community participation can be defined by *involvement of* *the local population activity in the decision-making concerning development projects or in their implementation.*

Involvement of population in the physical works of implementation of project is also sometimes treated as participation. This kind of involvement prescribed passive acceptance of services and provision of supports in cash or kind, in giving money for a pump, digging a well for a water supply, or laying bricks for a health center. The dynamics of changing society, however, demand much more than mere acceptance, allegiance, and unpaid labour. The new type of involvement requires *identification with the movement*, which grows only out of *involvement in thinking, planning, deciding*.

This conception does not assume that there is an ideal level of participation varies, but over the long run sustainability will depend on minimizing transaction cost in horizontal and vertical interactions. Participation is viewed as a *means to defined ends, not as an end in it*; the goal therefore is to optimize participation to achieve the desired project goals, not simply to maximize participation. The desired goals in rural water supply projects include achieving improved water supply systems and developing the human, organizational and management capacity to solve problems as they arise in order to sustain the improvements.

The principle underlying participation to give people a voice is constant, yet the choices that people make vary infinitely. Thus a community may decide to subcontract maintenance to an independent mechanic rather than to undergo training and take turns doing the work. A water user' group may choose to dissolve the organization or to define new goals after the first ones have been met. For example, when construction is complete, a water committee may transform itself to undertake sanitation itself to undertake sanitation construction, to build a football field, or to branch into children's education, depending on the community group may divide into smaller, functional sub-groups, with the larger group meeting only occasionally. Alternatively, people may informally nominate leaders to represent their interests.

12.9.4 Necessity of community participation

The reasons that advanced the concept of and necessity of community participation in all the development interventions including WSS are:

The reasons advance for community participation

(i) With participation, more will be accomplished

(ii) With participation, services can be provided more chiefly

(iii) Participation has an intrinsic value for participants
(iv) Participation is a catalyst for further development
(v) Participation encourage a sense of responsibility
(vi) Participation guarantees that a felt need is involved
(vii) Participation ensures things are done the right way
(viii) Participation uses valuable indigenous knowledge
(ix) Participation frees people from dependence on other's skills
(x) Participation makes people more conscious of the cause of their poverty and what they can do about it.

The form of community Participation

(i) Consultation
(ii) A financial contribution by the community
(iii) Self-help projects by groups of beneficiaries
(iv) Self-help projects by involving the whole community
(v) Community specialized workers
(vi) Mass action
(vii) Collective commitment to behaviour change
(viii) Endogenous development
(ix) Autonomous community projects
(x) Approaches to self-sufficiency

12.9.5 Level of participation

12.10 Participatory planning

The success and failure of any activities or programs mainly depends on planers. It is the necessity to make plan according to Mission and Vision and the practical situation. The participation of representatives from root or field level to top level is essential for planning. The planning by such type of participation is called the participatory planning. The participatory planning may be shown by a cycle called participatory planning cycle (Figure 12.). This cycle accelerates the trends of woks.

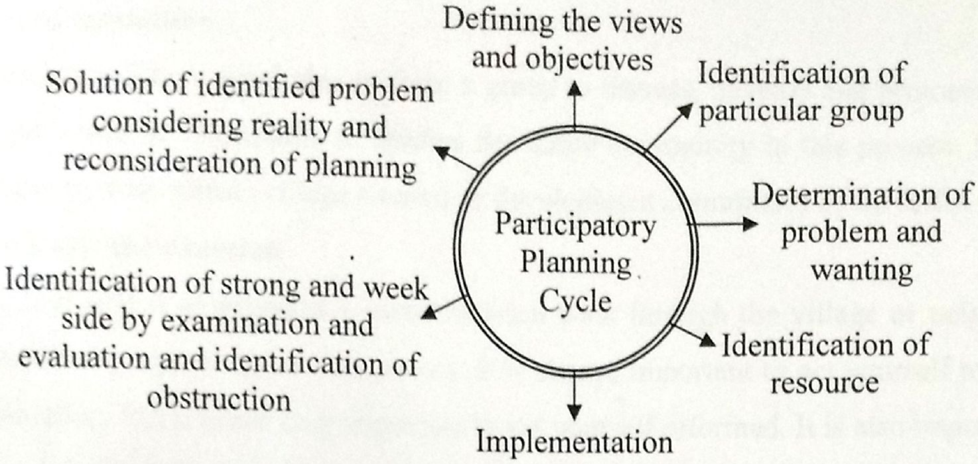


Figure 12.2: Participatory planning cycle

12.11 Community organization

12.12 Community mobilization

Community mobilization involves the creation of social movement for a particular programme by mobilizing many of allies at the national, regional and community level. The aim is to create a demand and activate a wide range of groups, for instance for sanitation improvements. Community mobilization is the glue that binds advocacy activities to more planned and behaviour-oriented communication activities. Steps of community mobilization are as follows:

Contact with local leaders

One of the first steps will be to introduce yourself to the local authorities and /or local leaders. Inform them of your plans and ask for their approval and support. This is not only a matter of good work easier. They can give you a first impression of the community, the various socio-economic groups, problems and needs, possible constraints, and so on. They can tell you what they think of the new project and in what way they are willing and able to contribute. Together you can discuss possible way to improve the community.

Formal meeting

One of the next steps may be to call a formal meeting to inform the entire community of the programme and your plans. The organization of a formal meeting is useful to give general information and to ask for general support. It is however not the right place to discuss things in detail. In general meetings most people will only be listeners as there will neither be the time no chance for everybody to give her/his view.

Formation of committee

In most cases it will be a good idea to form a group to discuss, develop and promote specific local activities, as it will be impossible to involve the entire community in this process. Sometimes the main community committee (village council or development committee) could fulfill this task.

Getting to know the situation

Another initial step is to organize a demonstration walk through the village or neighbourhood by asking interested people to show you around. It is always important to get yourself informed on the existing situation. But it is not only important to get yourself informed. It is also important to get the community informed of your intentions.

Involvement of the professional workers

At the start of community mobilization we should also try to inform and involve other professionals in the area. They can give you information. They may give you input in your programme or you may take part in their activities.

12.13 Sustainable development approach

To achieve their development objectives interventions in water supply and sanitation have to be sustainable. The main thrust of the concept is that activities by the present generation should not compromise the resources, or the environmental conditions of future generations. A water supply or sanitation system is sustainable when it:

- provides an efficient and reliable service at a level which is desired;
- can be financed or co-financed by the users with limited but feasible external support and technical assistance;
- is being used in an efficient and effective way, without negatively affecting the environment.

The definition of sustainability involves four crucial dimensions: the user community wanting an efficient and reliable service, the technology that has to provide it, the institutional environment and efficient use of the service (Figure 12.1). These issues will be briefly described here.

12.14 Gender issues conceptual frame

A gender approach means that the different knowledge, responsibilities, roles, resources, needs and attitudes of men and women are taken into account. A gender approach requires an openmindness and aims at the fullest possible participation of both women and men. It takes account of wealth, age, ethic group: rich and poor, young and old. The main feature of a gender approach is that it focuses on men and women and reviews the needs and possibilities to fully involve both groups in water and sanitation improvements.

The word 'Gender' generally is used about something to mean either male or female. But in social science, gender is used to indicate the difference between male and female in society. Though it is synonymous word but in practical it has a great difference from sex. This is why every body should have a clear conception about the term 'Gender' and 'Sex' in social science.

Sex is the character indicating difference between male and female created by natural and biological means; or, difference of male and female on the basis of physical characteristics; or, physiological characteristics of male and female which is selected by birth and unchangeable.

Gender is the identification of male and female which is socially buildup, socially selected relationship between male and female, socially fixed up role of male and female which is changeable and it differs with society and culture.

The term 'gender' is used to describe a set of qualities and behaviours expected from men and women by their society. These form his or her social identity. Gender expectations arise from the notion that certain qualities, behaviours, characteristics, needs and role are 'natural' for me. It refers to the social construction of female and male identity. It includes the ways in which those differences, whether real or perceived, have been valued, used and relied upon to classify women and men and to assign roles and expectations to them. The significance of this is that the lives and experiences of women and men, including their experience of the legal system, occur within complex sets of differing social and cultural expectations.

Gender refers to the roles and responsibilities of women and men that are created in our families, our societies and our cultures. These roles and expectations are learned. They can change over time and they vary within and between cultures. This concept is vital because if facilities gender analysis revealing how women's subordination is socially constructed. As such, the subordination can be changed. It is not biologically predetermined nor is it enshrined in granite.

Now the term 'gender' is not confined within male and female. In modern definition it identify the disadvantageous, unprogressed, neglected, suppressed stream of the society. This stream is the issue of gender. It can be men or women, children or young, young or old, etc.

12.15 Women empowerment

Women empowerment is a conflicting and debatable term. Once upon a time, social development planners thought that the absence of direct participation of women in development project is the main obstacle of social development. Their argument is that women are about 50% of total manpower and these parts are inactive and neglected in society. So they want to give them chance in all steps. They want to utilize them as a worker in development project besides the men. This concept is called the women empowerment. But upto certain period they cannot understand that anyhow every woman is engaged to develop their society with honour and respect.

12.16 Gender auditing

The term audit means the check, examination, assessment, etc. about anything. Gender auditing was basically the examination or assessment of involvement or scope of participation of male and female in any activity. But at present the concept of male and female has been changed by neglected or disadvantageous stream of the community or society in gender issue. So, for the case of gender auditing, it is the auditing of involvement or participation or scope of participation of neglected or disadvantageous stream and advantageous or influential stream of community or society in any activity.

Gender auditing may be performed in any stage of the project by questioner survey. There are some indicators must have by which auditing cam be performed. Here some questions are mentioned as for example for auditing gender (considering male and female) in any organization:

Key questions

- What is the ratio of male and female staff at all levels of your organization?
- What are the procedures for the selection of staff? How do they encourage/discourage men or women?
- What are the recruitment procedures for various categories of staff? How do they encourage/discourage men or women?
- What are the criteria used and procedures following for posting? How do they encourage/discourage men or women from joining the service?
- What are the criteria and the procedures followed for the transfer of staff? How do they affect women? How do they affect men? What are the differences? What are their causes? What are the consequences (eg. Staff drop-outs)?
- What are the criteria and processes followed for the training of staff?
- How do they affect men and women? What is the present ratio of staff receiving training (men/women; in-service/outside)
- What are the criteria and procedures being used for promotions? How do they differentially affect women and men? What are the male/female ratios in some of the recent promotions?

For each of the above points identify or suggest ways in which the negative impact on men or women could be remedied.

12.17 Gender balance and sensitivity

Gender balance is the minimization of difference in involvement of people of different status of society in any activity. Here qualification, capacity, capability, eligibility must be considered and judged without any external influence for their involvement and participation. The balancing of gender is the great sensitive issue for development of society.

Course Title : Environmental Engineering-I

Course No. CE 3141

Credits : 3

Conduct hours/week : 3

Course Content:

- Water quality and treatment
- Water quality parameters and standards,
- Water treatment:
- Plain sedimentation, flocculation and settlement, filtration, disinfection, and other treatment methods
- Small scale iron and arsenic removal units
- Other low-cost treatment methods for rural communities
- Monitoring and sanitary protection of water supply distribution system

Reference Books:

1. **Water Supply and Sanitation**
   - M.F Ahmed
   - M.M.Rahman
2. **Environmental Engineering**
   - H.S.Peavy
   - D.R.Rowe
   - G. Tchobanoglous
3. **Environmental Engineering**
   - M.L Davis
   - D.A Cornwell
4. **A Text book of Water Supply Engineering**
   - M.A.Aziz
5. **Water Supply and Sanitary Engineering**
   - Rangwala

Wholesome water/ Potable water:

The term wholesome water is used to indicate the water which is not chemically pure, but does not contain anything harmful to the human body i.e. the water in which there is no pathogenic bacteria, no toxic substances and no excessive organic matter. Thus the wholesomeness is a must while the palatability of water is desirable.

Requirements for potable water:

- It should be clear, odorless and colourless
- It should be free from harmful and disease producing bacteria
- It should be free from all objectionable substances
- It should be fresh and cool
- It should be palatable i.e. aesthetically attractive

Water quality parameters:

Water quality parameters are three types:

- Physical parameters
- Chemical parameters
- Biological parameters

Physical Parameters:

- Colour
- Taste and odour
- Temperature
- Turbidity
- Suspended solids

Colour

Pure water is always colourless, but in nature is often coloured by foreign substances. Water whose colour is partly due to suspended matter is said to have apparent colour. Colour contributed by dissolved solids that remain after removal of suspended matter is known as true colour

Source:

- Algae metabolism
- End products of degraded organic matter
- Discharge of untreated and partially treated waste water from various industries like food processing, textile industries, paper production etc
- Divalent species containing iron and manganese etc

Impact:

- Coloured water is not aesthetically acceptable to the general public
- Highly coloured water is unsuitable for laundering, dyeing, papermaking, beverage manufacturing, dairy production and other food processing and textile and plastic production
- Coloured water effects its marketability for both domestic and industrial use

Measurement

One true colour unit (TCU) corresponds to the amount of colour exhibited under the specified test conditions by a solution containing 1.0 mg of platinum per litre of water in the form of chloroplatinate ion. Most consumers can detect a colour of 15 TCU (WHO guideline value and Bangladesh standard for drinking water) in a glass of water.

Taste and Odour

Many substances with which water comes into contact in nature or during human use may impart perceptible taste and odour. These include minerals, metals and salts from soil end products from biological reactions and constituents of wastewater.

Impact:

Consumers find taste and odour aesthetically displeasing for obvious reasons. It is unpleasant for drinking and bad for health

Measurement:

Measurement of taste and odour casing organics can be made using gas or liquid Chromatography. As Chromatographic analysis is time consuming and requires expensive equipment it is not routinely performed on water samples, but should be done if problem organics are suspected.

Quantitative tests that employ the human senses of taste and smell can be used for this purpose. An example of this test is the threshold odour number (TON).

- Varying amounts of odourous water are poured into containers and diluted with enough odour free distilled water to make a 200 ml mixture

- An assembled panel of five or ten noses is used to determine in which the odour is just barely detectable to the sense of smell. For public water supply the threshold water number should not be more than 3

- The TON of that sample is then calculated using the formula

$$TON = \frac{A + B}{A}$$

Where,

A= volume of odourous water
B= Volume of odourfree water required to produce a 200 ml mixture

Temperature:

Source:

The temperature of natural water systems respond to many factors. The ambient are the most universal. Generally shallow bodies of water are more affected by ambient temperatures than are deeper bodies. Removal of forest canopies and irrigation return flows can also result in increased stream temperature

Impact:

- If the temperature is increased, biological activity increases. An increase of 10 degree is usually sufficient to double the biological activity
- Temperature changes affects the reaction rates and solubility levels of chemicals
- Temperature also affects other physical properties of water. The viscosity of water increases with decreasing temperature.

Measurement:

The measurement of temperature is done with the help of ordinary temperature

Turbidity:

The colloidal materials present in water interferes with passage of light and thus imparts turbidity to the water.

Source:

I. Erosion of colloidal material such as clay, silt, rock fragments and metal oxides from the soil.
II. Vegetables fibers and microorganisms may also contribute to the turbidity.
III. Household and industrial waste water may contain a wide variety of turbidity producing material.

iv. Soaps, detergents and emulsifying agents produce stable colloids that result turbidity

Impact:

- The cloudy appearance developed in water due to turbidity is aesthetically unattractive and it may also be harmful to the consumers.
- It also disturbs the disinfection process because the solids may partially shield the organisms from the disinfection

Measurement:

Turbidity is measured by photo metrically by determining the percentage of light of a given intensity that is either absorbed or scattered. The original measuring apparatus, called a **Jackson Turbidity Meter.** 1 JTU being equal to the turbidity produced by 1 mg SiO_2 in IL of distilled water**.**

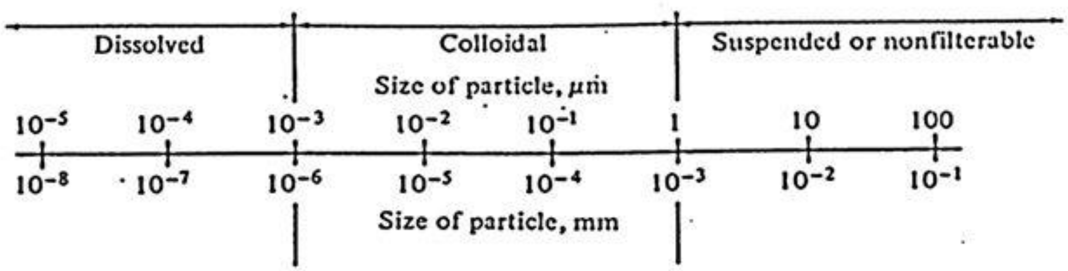
Formaizin, a chemical compound, provides more reproducible standards than SiO_2 and has replaced it as a reference. Turbidity meter readings are now expressed as formazin turbidity units or **FTUs**. The term neophelometry turbidity units **(NTU)** is often used to indicate that the test was run according to the scattering principle

Suspended solids:

Solids – settleable, suspended, dissolve

Three categories:

- settleable
- suspended
- fine or dissolved solids



size classification of solids in water

Source:

I. Inorganic solids such as clay, silt and other soil constituents
II. Organic material such as plant, fibers and biological solids such as algal cells. Bacteria etc.
III. Industrial waste, domestic waste, oils greases etc

Measurement:

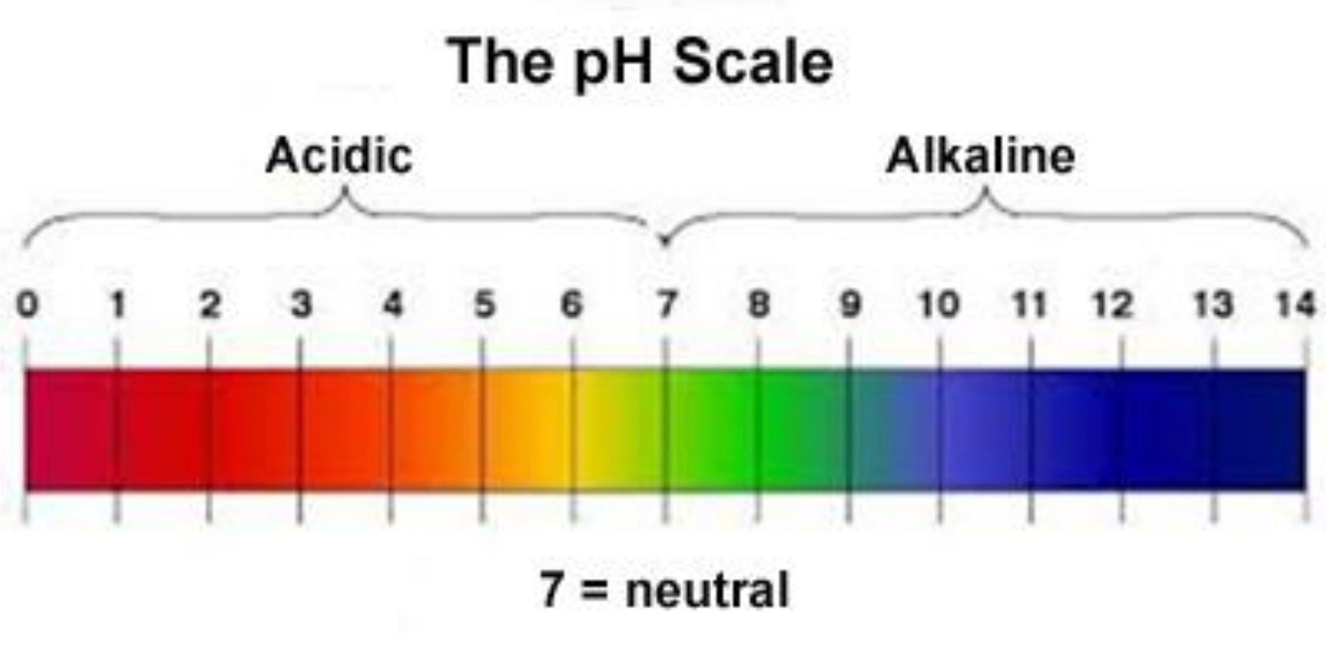
There are several tests available for measuring solids. Most are gravimetric tests involving the mass of residue.

Chemical water quality parameter:

- pH
- Alkalinity
- Acidity
- Total solids
- Hardness
- Chlorides
- Dissolved gases
- Metals
- Nitrates and its compounds

1) pH:

The acidity or alkalinity of water is measured in terms of its pH value or H-ion concentration



Impact:

- For potable water, the pH value should be between 6.5 and 8.5 and it is found that if this range is maintained, there is no direct effect on health of human being.
- If pH value of water is below 4, it will produce a sour taste and if it is more than 8.5 it will impart a bitter taste.
-
- The higher value of pH induces the formation of Trihalomethane which are responsible for casing cancer in human beings
- The lower value of pH starts corrosion in pipes and thereby the toxic metals like Zn, Pb, Cu are released.

Measurement:

There are two methods for measurement

1. Electrometric method
2. Colorimetric method

1. Electrometric method:

In this method the potentiometer is used to measure the electrical pressure exerted by positivity charged H-ions. The pH value then correspondingly expressed.

ii) Colourimetric method:

In this method the chemical reagents or indicators are added in water and the colour produces id compared with standard colours of known pH value

Problem 1:

A factory discharges 50 m^3/day of waste having pH =11. If the waste contains KOH only, find out the quantity of KOH in kg/day

Solution:

pH + pOH = 14

11 + pOH = 14

pOH = 14-11 = 3

Thus the concentration of hydroxyl ion in the given water is 10^{-3} moles/litre i.e. $[OH^-]$ = 10^{-3} moles/litre

The molecular weight of KOH in 1 liter solution works out to

(39 + 16 + 1) = 56 g

KOH in g/litre = 56 x 10^{-3}

KOH in kg/litre = 56 x 10^{-6}

Now, Discharge = Q = 50 m^3/day = 50 x 10^3 litres/day

Quantity of KOH in given waste
= (56 x 10^{-6}x 50 x 10^3) kg/day = 2.80 kg/day

2) Alkalinity:

Alkalinity is defined by as the quantity of ions on water that will react to neutralize hydrogen ions. Alkalinity is thus a measure of the ability of water to neutralize acids.

Source:

The alkalinity is due to the presence of bicarbonate (HCO_3^-), carbonate (CO_3^{--}) and hydroxide (OH^-)

$$CO_2 + H_2O \rightleftarrows H_2CO_3^*$$

$$H_2CO_3^* \rightleftarrows H^+ + HCO_3^-$$

$$HCO_3^- \rightleftarrows H^+ + CO_3^{2-}$$

$$CO_3^{2-} + H_2O \rightleftarrows HCO_3^- + OH^-$$

Impact:

- In large quantities, alkalinity imparts bitter taste to water.
- The principle objection to alkaline water, however is the reactions that can occur between alkalinity and certain cations in the water. The resultant precipitate can foul pipes and other water systems appurtenances.

Measurement:

The alkalinity is measured by volumetric analysis. The various types of indicators are available for this purpose. The commonly adopted two indicators are-

1) Phenolphthalein
2) Methyl orange

1) Phenolphthalein :

Pink above pH 8.2 and
Colourless below pH 8.2

2) Methyl orange:

Red below pH 4.5 and
Yellow-orange above pH 4.5

Alkalinity on pH scale is represented as follows:

- The range of total alkalinity is 4.5 to 14
- The range of bicarbonate i.e. HCO_3^- alkalinity is from 4.5 to 8.2
- The range of carbonate i.e. CO_3^- alkalinity is from 8.2 to 10
- The range 0 to 4.5 indicates no alkalinity

Alkalinity Relationships

Result of P/M or P/T Alkalinity Test*	Hydrate Alkalinity (OH) Equals:	Carbonate Alkalinity (CO_3) Equals:	Bicarbonate Alkalinity (HCO_3) Equals:
P = 0	0	0	M
P = M	M	0	0
P = ½ M	0	M	0
P < ½ M	0	2P	M–2P
P > ½ M	2P–M	2(M–P)	0

P = Phenolphthalein
M= Methyl orange
T= Total alkalinity

Problem: From the following data of volumetric analysis, calculate the hydroxide, carbonate and bicarbonate alkalinities:

Sample 100 ml	Total ml of titrant to reach end point	
	Phenolphthalin	Mythyl orange
A	10	15.5
B	14.4	38.6
C	8.2	8.4
D	0	12.7

Solution:

Sample A:

$$P = \frac{10 * 1000}{100} = 100$$

$$T = \frac{15.5 * 1000}{100} = 155$$

0.5 T = 77.5

As P > 0.5 T, OH^{-} alkalinity = 2P-T= 200-155 = 45

CO_3^{--} alkalinity = 2(T-P) = 2(155-100) = 110

HCO_3^{-} alkalinity = 0

Sample B:

$$P = \frac{14.4 * 1000}{100} = 144$$

$$T = \frac{38.6 * 1000}{100} = 386$$

0.5 T = 193

As P < 0.5 T, OH^{-} alkalinity = 0

CO_3^{--} alkalinity = 2P = 288

HCO_3^{-} alkalinity = T-2P = (386-288) = 98

Sample C:

$$P = \frac{8.2 * 1000}{100} = 82$$

$$T = \frac{8.4 * 1000}{100} = 84$$

0.5 T = 42

As P > 0.5 T, OH^{-} alkalinity = 2P-T = 164-84 = 80

CO_3^{--} alkalinity = 2(T-P) = 2(84-82) = 4

HCO_3^{-} alkalinity = 0

Sample D:

P = 0

$$T = \frac{12.7 * 1000}{100} = 127$$

As P = 0, OH^{-} alkalinity = 0

CO_3^{--} alkalinity = 0

HCO_3^{-} alkalinity = T = 127

Cycle 6
Day A

3) Acidity:

The term acidity with reference to the water and waste water is defined as the capacity of substances contained in the water to take up hydroxyl ions (OH^-) to reach a defined pH value (0 to 8.2)

The acidity are the following two types:

(i) Carbon dioxide acidity
(ii) Mineral acidity

(i) Carbon dioxide Acidity:

This acidity is due to the presence of CO_2 in ground water and surface water

(ii) Mineral Acidity:

The mineral acidity is due to the presence of HCl, H_2SO_4, HNO_3 and strong organic acids

Measurement:

The mineral acidity is determined by titrating or neutralizing sample with strong base NaOH to pH 4.3. The carbon dioxide acidity in the sample is calculated by neutralizing completely by continuing the titration till pH of 8.2 is reached.

The amount of acidity is expressed in terms of C_aCO_3, Then

Total acidity (as C_aCO_3) = mineral acidity + CO_2 acidity

Impact:

(i) It affects the aquatic life
(ii) It affects the biological treatment of sewage
(iii) It corrodes pipes
(iv) It interferes in the treatment of water as in case of water softening
(v) The water having acidity more than 50 mg/l can not be used for R. C. C. construction

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(v) The water having acidity more than 50 mg/l can not be used for R. C. C. construction

Cycle 6
Day A

4) Total Dissolved Solids:

The material remaining in the water after filtration for the suspended solids analysis is considered to be dissolved

Total solids = Total dissolved solids + Total Suspended solids

Sources:

(i) Dissolved material results from the solvent action of water on solids, liquids, and gases
(ii) Materials from decay products of vegetation, from the organic chemicals and from the organic gases are common organic dissolved constituents of water.

Impact:

(i) Dissolved minerals, gases and organic constituents may produce aesthetically displeasing colour, test and odour

(i) Some chemicals may be toxic, and some of the dissolved organic constituents have been shown to be carcinogenic

In general water with a total solid content of less than 500 mg/l are most desirable for such purposes. Water with a higher solids content have a laxative and sometimes the reverse effect upon people.

Depending on the TDS water is often classed as follows:

Excellent	TDS < 300 mg/l
Good	300-600 mg/l
Fair	600-900 mg/l
Poor	900-1200 mg/l
Unacceptable	1200 mg/l

(5) Hardness:

The term hardness is defined as the ability of the water to cause precipitation of insoluble calcium and magnesium salts of higher fatty acids from soap.

Source:

Hardness is two types:

(i) Temporary hardness (Carbonate hardness)

(ii) Permanent hardness (Non Carbonate hardness)

Temporary hardness is mainly due to the presence of bicarbonates of calcium and magnesium. It can be removed by boiling or by adding lime to the water.

The permanent hardness is due to the presence of sulphates, chlorides, and nitrates of calcium and magnesium. It can not be removed simply by boiling the water

Impact:

(i) Soap is more consumed due to hardness of water

(ii) Boiler scale, the result of the carbonate hardness precipitate may cause considerable economic loss

(iii) Corrosion may occur.

Measurement:

Hardness can be measured by using spectrophotometric techniques or chemical titration to determine the quantity of calcium and magnesium in a given sample. Hardness can be measured directly by titration with ethylenediamine tetra acetic acid (EDTA) using eriochrome black T (EBT) as an indicator. The EBT reacts with the divalent metallic cations, forming a complex that is red in colour. The EDTA replaces the EBT in the complex, and when the replacement is complete, the solution changes from red to blue. If 0.01 M EDTA is used 1.0 ml of the titrant measures 1.0 mg of hardness as C_aCO_3

Cycle 6
Day A

Standard Value:

Soft < 50 mg/L as C_aCO_3
Moderately Hard 50-150 mg/L as C_aCO_3
Hard 150-300 mg/L as C_aCO_3
Very Hard > 300 mg/L as C_aCO_3

The public health service standards recommended a maximum of **500 mg/L** of hardness in drinking water.

Cycle 6
Day A

Total Hardness = Carbonate Hardness or alkalinity + Non-carbonate hardness

Or T.H = C.H. + N.C.H

Following two rules should be remembered in this connection

1. When alkalinity < T.H then
   C.H = Alkalinity

2. When alkalinity > T.H. then
   C.H = T.H

Cycle 6
Day A

Hardness in mg/l as C_aCO_3 = M^{++} (mg/l) x $\frac{equivalent\ wt.of\ C_aCO_3}{equivalent\ wt.of\ M^{++}}$

Where M represents any ion or radical

Now, Equivalent weight = $\frac{Molecular\ weight}{X}$

Where X = for acids, the number of moles of H^{++} obtainable from 1 mole acid

X = for bases, the number of moles of H^{++} with which 1 mole of base will react

Thus the equivalent weight of C_aCO_3 = $\frac{(40+12+3*16)}{2}$ =50

Cycle 6
Day A

The equivalent weights of $Ca^{++}, Mg^{++}, Sr^{++}$ will be respectively as follows

Ca = 40/2 = 20
Mg = 24.4/2 = 12.2
Sr = 87.6/ 2 = 43.8

For measuring alkalinity, the reading of only CO_3^{--} or HCO_3^{-} will be required and expressed as percentage, they can be found out found out from the following relations

Total Alkalinity as HCO_3 in mg/L = Bicarbonate Alkalinity x 1.22

Total Alkalinity as CO_3 in mg/L = Carbonate Alkalinity x 0.60

Molecular wt of HCO_3 = (1+12+3*16) =61

Molecular wt of CO_3 = 12+3*16 = 60

Cycle 6
Day A

Problem : The analysis of water from a bore shows the following results in mg/l

Ca = 60, Mg = 48, Na = 103.5 K =19.5
HCO_3 = 224, SO_4 =220.8 Cl = 78.1

Find out the total hardness, carbonate hardness and non carbonate hardness

Solution:

Total Hardness = (60 x 50/20 + 48 x 50/12.2) = 150 + 196.72 = 346.72 mg/L as C_aCO_3

Cycle 6
Day A

Total alkalinity as HCO_3 in mg/L = Bicarbonate alkalinity x 1.22

244 = Bicarbonate alkalinity x 1.22

Bicarbonate alkalinity = 244/1.22 = 200 mg/L

In this case alkalinity< T. H.

C.H = Alkalinity = 200 mg/L

Then N.C.H = T.H – C.H
= 346.72 – 200 = 146.72 mg/L

6) Chlorides:

The chlorides contents, especially of sodium chlorides or salt.

Measurement:

I. 50 c.c of sample of water is taken by pipette in a porcelain dish
II. Two or three drops of potassium chromate solution are added to the sample of water
III. The chloride contents are then determined by filtering with standard solution of silver nitrate

$$AgNO_3 + NaCl \rightarrow AgCl + NaNO_3$$

$$2AgCl + K_2C_{r2}O_7 \rightarrow Ag_2C_{r2}O_7 + 2KCl$$

Impact:

The presence of chlorides can corrode and such water cannot be used boilers because of formation of hydrochloride acid due to the presence of magnesium chloride in water.

Cycle 6
Day D

Water Treatment

1. Plain Sedimentation
2. Sedimentation with coagulants/ flocculation
3. Filtration
4. Disinfection

Cycle 6
Day D

Some of the treatment operations for removal of specific impurities are:

- Aeration
- Water softening
- Arsenic removal
- Iron removal
- Activated Carbon application
- Fluoridation and defluoridation
- Demineralization
- Desalinization

Cycle 6
Day D

Plain Sedimentation:

Principles of particle settling in water:

This is a process causes the organic or inorganic particles heavier than water to settle by retaining water in the tank or basin. These particles are held suspension in natural water mainly by turbulence or current.

The particles travels with a constant vertical velocity called the terminal velocity or settling velocity of the particle.

Cycle 6
Day D

The settling velocity of the particle depends upon

- Horizontal flow velocity of the water
- Shape and the size of the particle
- Specific gravity of the particle
- Viscosity of the water
- Density of the water
- Temperature of the water

Cycle 6
Day D

The settling velocity of spherical particles under laminar flow conditions is given by the equation

$$v_s = \frac{g}{18}(S - 1)\, d^2/\Upsilon$$

Where

v_s = settling velocity
g = acceleration due to gravity
S = specific gravity pf the particle
d = diameter of the particle
ϒ = kinematic viscosity of water

Cycle 6
Day D

This equation is called Stoke's Law. Stoke's Law holds good only for particle size 0.1 cm in diameter and Reynold's number 1 or less. For larger particles having diameter greater than 1 cm and Reynold's number above 2000 Newton's Law for frictional resistance or drag applies

$$Vs = \sqrt{\frac{4g}{3Cd}(S-1)d}$$

Where

Cd is the Newton's coefficient of drag

Cycle 6
Day D

Stoke Law is valid for computation of settling velocity of discrete particles. Discrete particles are those which do not change size, shape and mass during settling and which do not influence each other by being too close. Particle settling under this condition is called discrete settling. The expression for discrete particle is

V= 418 (S - S_1) d^2(3T + 70)/100

Where
V = velocity of settlement in mm per second
S= specific gravity of the particle
S_1= Specific gravity of water
d = diameter of the particle in mm
T = temperature in ^{o} C

Cycle 6
Day D

This equation is applicable for particles having diameter smaller than 0.1 mm or so. For particles of greater diameter than 0.1 mm he found that the velocity of settlement was proportional to the first power of diameter and not to the second power of diameter. Hence for bigger particles

$$V = 418\ (S - S_1)\ d(3T + 70)/100$$

Cycle 6
Day D

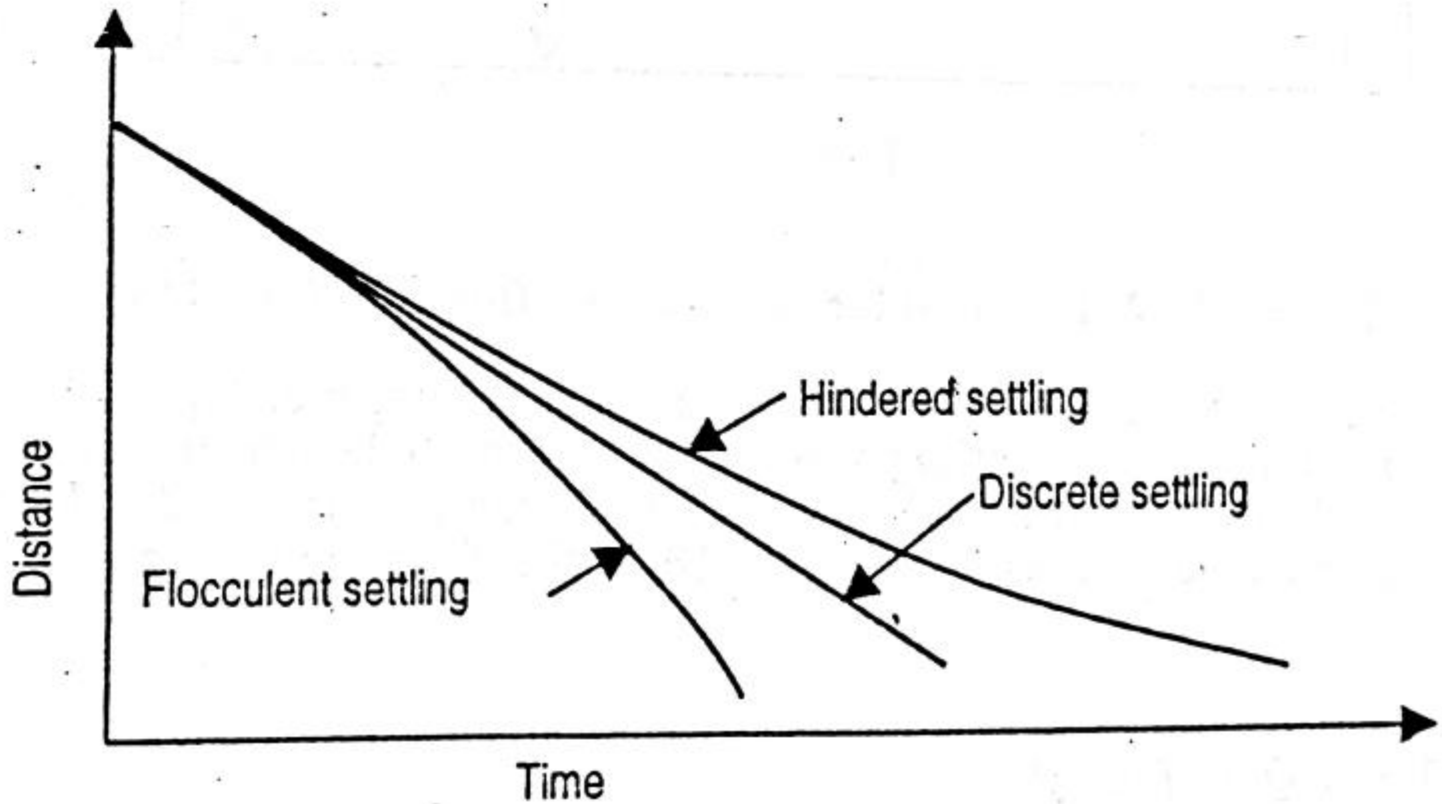


Figure 18.1 Settling of different types of particles in water

Cycle 6
Day D

Hindered Settling:

In case of closely packed particles the water displaced by the particles may cause additional friction and the settling velocity is reduced. This termed as the **Hindered Settling.** Hindered settling becomes noticeable when the concentration of suspended solids is greater than 2000 mg/l

Flocculent Settling:

Sometimes settling particles may adhere to each other and grow in size and thus deviate from the settling characteristics represented by Stoke's Law. This may occur in settling of algae or freshly formed floc by the process of flocculation with coagulant. These particles/flocs tend to stick together and form new bigger particles which settle at a faster rate. This type of settling is called flocculent settling.

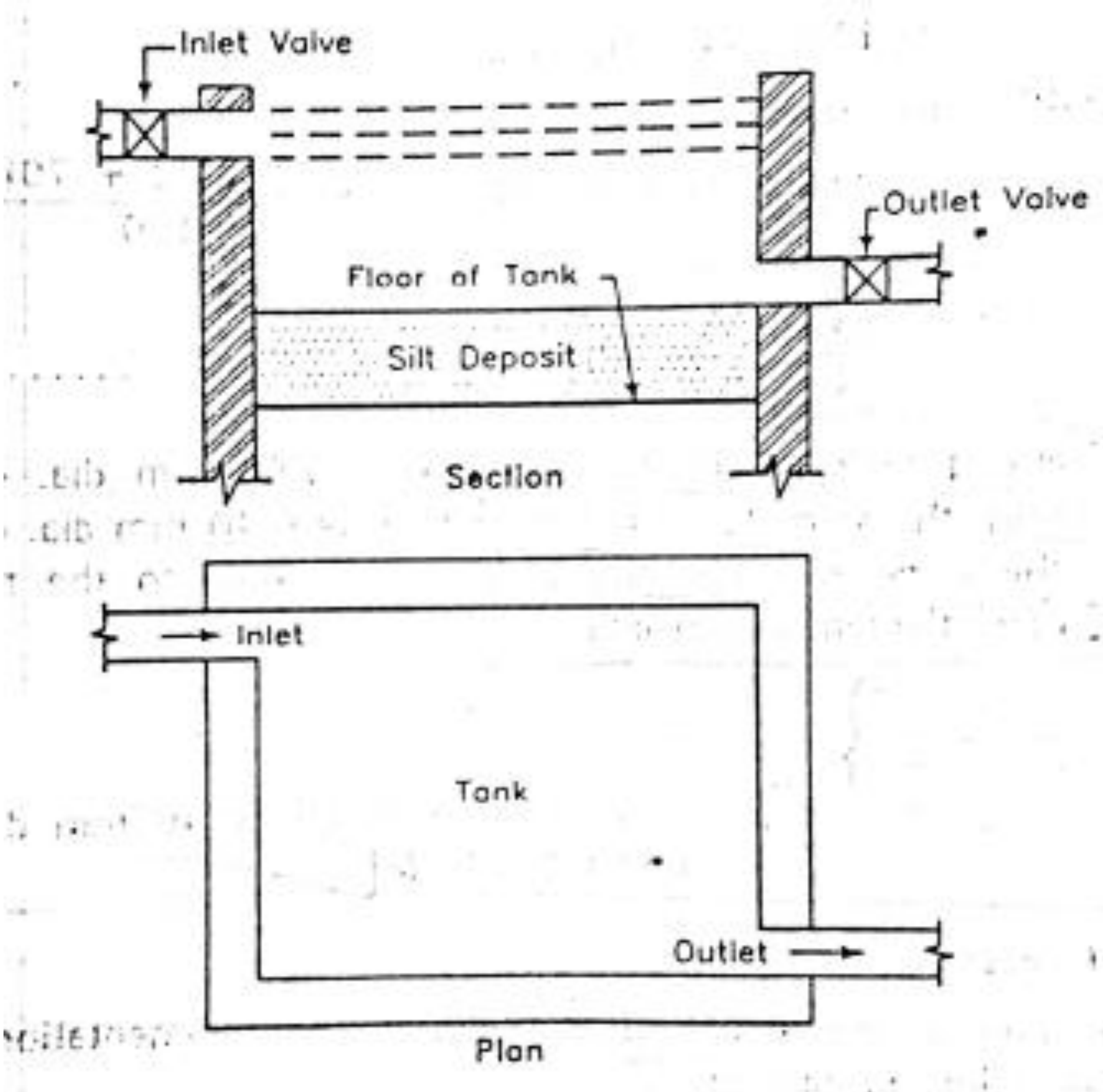
Cycle 6
Day D

Types of sedimentation tank:

Depending upon the nature of working the sedimentation tanks are the following two types.

1. Fill and draw type
2. Continuous flow type

Cycle 6
Day D



Fill and draw type tank

Cycle 6
Day D

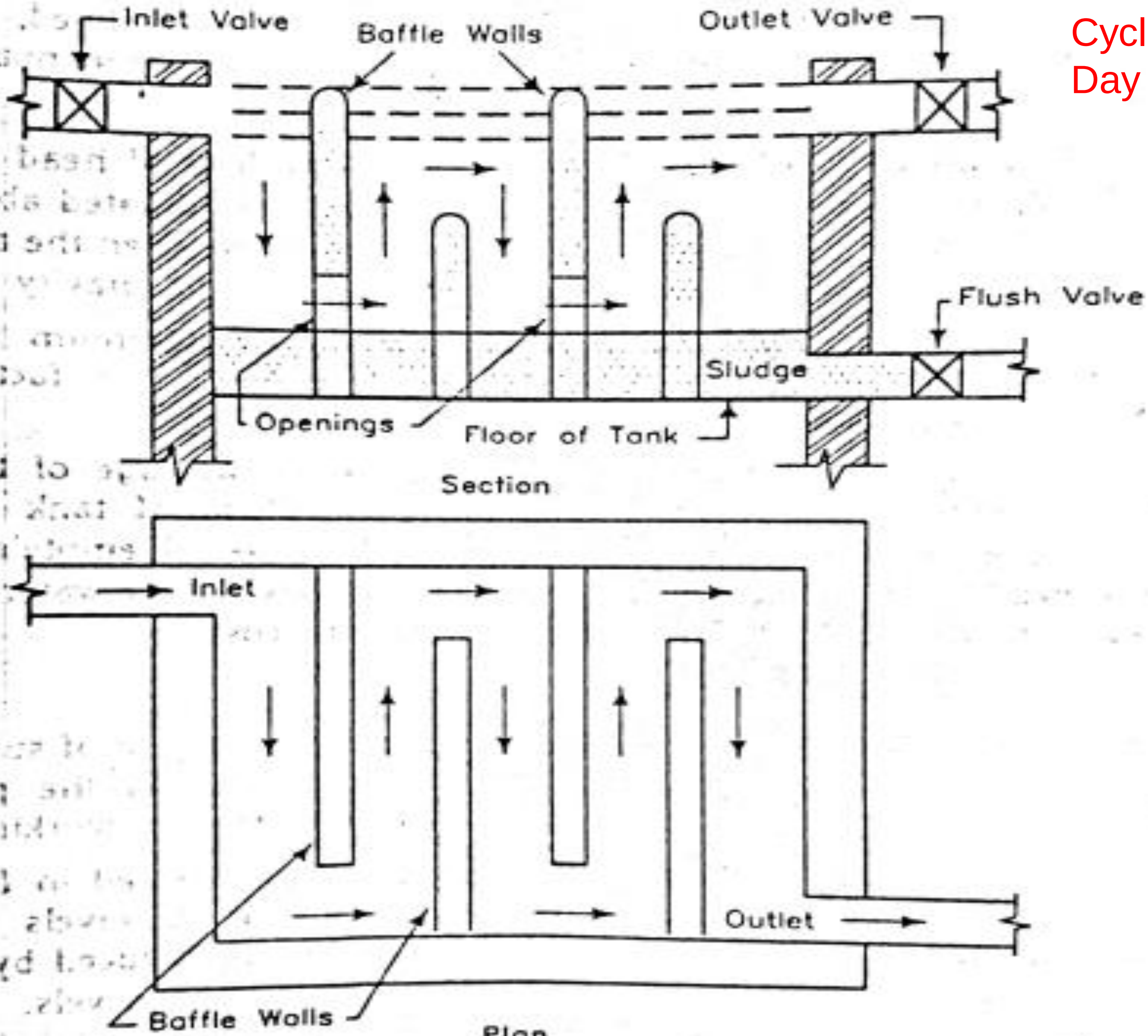
Working:

The working of the tank is simple. The water is filled the tank and it is then allowed to rest for a certain time. During the period of rest, the particle sin suspension will settle down at the bottom of the tank. The clear water is then drawn off and the tank is cleaned of silt and filled again.

Cycle 6
Day D

The usual period of rest to cause settlement of particles is about 24 hours or so. If time required for inlet, outlet emptying and cleaning operations is added , a period of about 30 to 36 hours is required to put the tank again in working condition.

Cycle 6
Day D



Continuous flow type tank

Cycle 6
Day D

Design aspects of continuous flow type of sedimentation tank:

Following are the design aspects

1. Velocity of flow
2. Capacity of tank
3. Inlet and outlet arrangements
4. Shapes of the tanks
5. Miscellaneous considerations

Cycle 6
Day D

1. Velocity of flow:

It should remain uniform through out the tank and it is generally not allowed to exceed 150 mm to 300 mm per minute.

2. Capacity of the tank:

Following are the two methods by which the capacity of sedimentation tank is determined

I. Detention period
II. Overflow rate

Cycle 6
Day D

I) Detention period:

The relation between capacity of a tank and the detention period can be established as follows

Let C = Cubical contents or capacity of tank
Q = Discharge or rate of flow per hour
T= Detention period in hours

Then $T = C/Q$

Or $C = Q \times T$

For plain sedimentation tanks, the detention period is vary from 4 to 8 hours and when coagulants are used it may vary from 3 to 4 hours

Cycle 6
Day D

ii) Overflow rate:

In this method it is assumed that the settlement of a particle at the bottom of the tank does not depend on the depth of tank. But it depends on the surface area of tank

Let L = Length of tank
B = Breadth of tank
D = Depth of tank = Side Water Depth = S. W.D
C = Capacity of tank
T = Detention period
Q = Discharge of rate of flow
V = velocity of descend of a particle to the bottom of tank
= Surface Overflow Rate = S.O.R

Cycle 6
Day D

Then T = C/Q = (L x B x D)/Q…………………….(1)

Also, T = Distance of descend/ Velocity of descend

= D/V = S.W.D/S.O.R ………………….(2)

Equation 1 and 2

(L x B x D)/Q = D/V

Or S.O.R = V = Q/(L x B)…………………..(3)

The equation 3 shows that the velocity of descend of particle is independent of the depth of tank and it inversely varies as the surface area of the tank

Cycle 7
Day A

Introduction to environmental engineering

General:

Environmental engineering has been defined as the branch of engineering that is concerned with the protecting the environment from the potentially deleterious effect of human activity, protecting human populations from the effect of adverse environmental factors, and improving environmental quality for human health and well being

Cycle 7
Day A

Environmental Engineering:

The Environmental Engineering Division of the American Society of Civil Engineers (ASCE) has published the following statement of purpose

Environmental engineering is manifest by sound engineering thought and practice in the solution of problems of environmental sanitation, notably in the provision of safe, palatable and ample public water supplies, the proper disposal of or recycle of waste water and solid wastes, the adequate drainage of urban and rural areas for proper sanitation and control of water, soil and atmospheric pollution, and the social and the environmental impact of these solutions.

Furthermore it is concerned with engineering problems in the field of public health, such as control of arthropod-borne diseases, the elimination of industrial health hazards and the provision of sanitation in urban, rural and recreational areas and the effect of the technological advances on the environment

Cycle 7
Day A

From the definition the topic of environmental engineering are

- Provision of safe, palatable and ample public water supplies
- Proper disposal of recycling of waste water and solid wastes
- Control of water, soil and atmospheric pollution

Cycle 7
Day A

Sanitation:

Sanitation may be defined as the science and practice of effecting healthful and hygienic condition and involves the study and use of hygienic measures such as

- Safe, reliable water supply
- Proper drainage of waste water
- Proper disposal of human waste
- Prompt removal of all refuse

Cycle 7
Day A

The principle objectives of providing sanitation facilities are

- To have improved public health
- To minimize environmental pollution

Classification of wastes:

In general wastes can be classified as follows:

Human waste or human excreta:

Refers to only human faeces and urine and usually are not combined with liquid or solid wastes. They are also widely known as night soil when collected without dilation in large volumes of water

Municipal sewage/Wastewater:

It is the liquid waste conveyed by a sewer and may include domestic and industrial discharges as well as storm water, groundwater infiltration and inflow

Domestic Sewage:

It is the liquid waste which originates in the sanitary conveniences, eg water closets, urinals , baths, sinks etc of dwellings, commercial facilities and institutions in a community. Sometimes it is referred to as sanitary sewage.

Cycle 7
Day A

Sullage:

It is the liquid discharges from kitchens, wash basins etc. and excludes human excreta. Sullage is less foul than domestic sewage and can be discharged through open surface drains in unsewered areas.

Industrial wastes:

It includes the liquid discharges from spent water in different industrial processes such as manufacturing and food processing

Cycle 7
Day A

Strom Water:

It is the surface runoff during and immediately after the rain fall, which enters sewers through inlets. Strom water is not as foul as sanitary or industrial sewage and hence can be carried through open drains or channels and disposed of in natural rivers or streams without any treatment.

Solid wastes:

It includes all materials which are normally solid and are discharged as useless or unwanted during human activities

Cycle 7
Day A

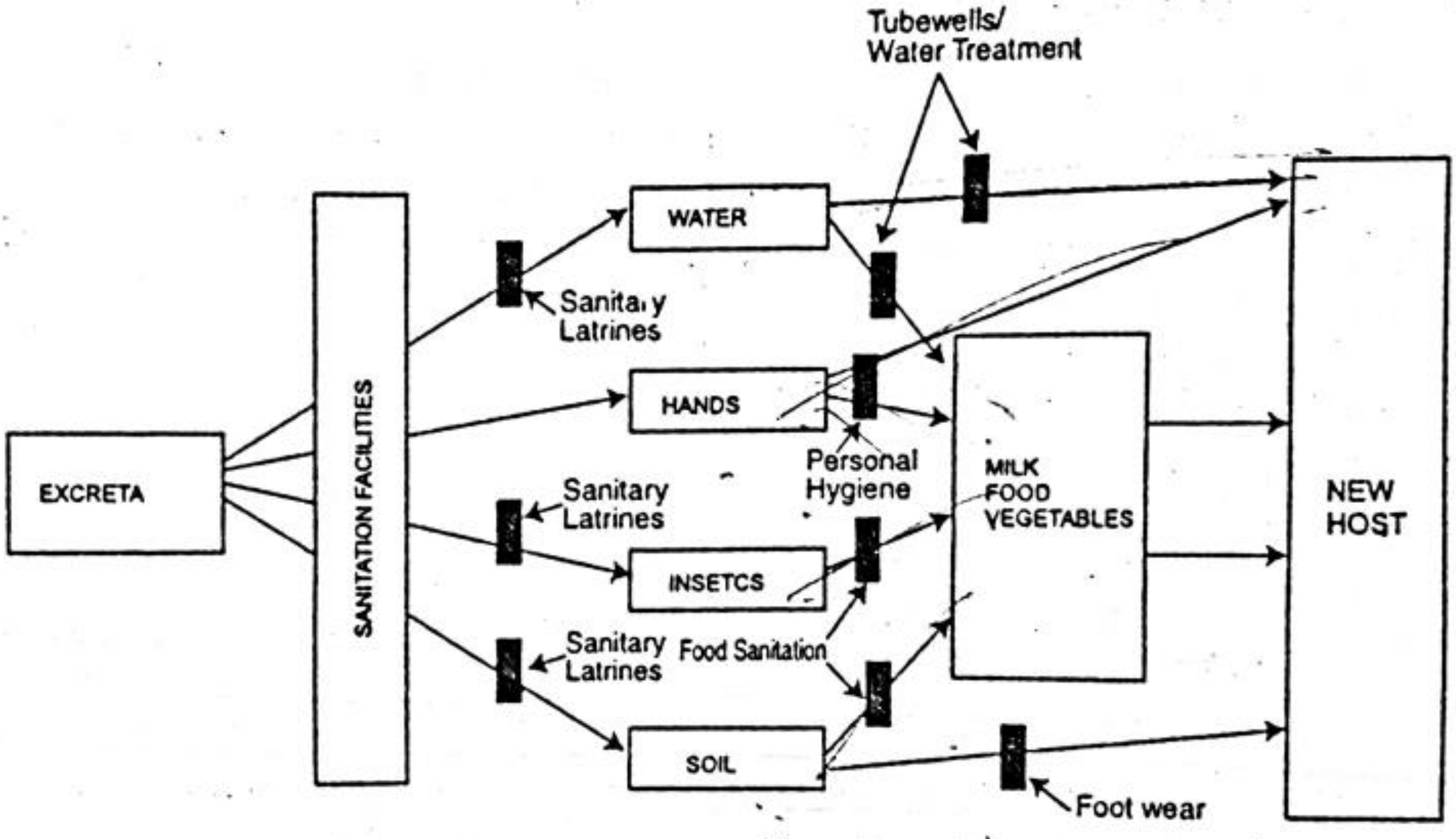


Figure 8.2 Disease Transmission and Sanitation

Cycle 7
Day A

Sanitation system:

A sanitation system is involves all arrangements necessary to store, collect, process and deliver human wastes back to nature in a safe manner. Sanitary systems with respect to human waste management may be considered to have following functions:

- Excretion and storage
- Collection and transportation
- Process /treatment
- Disposal/recycle

Cycle 7
Day A

Classification of sanitation system:

- Onsite sanitation system
- Offsite sanitation system

The classification of on-site or offsite sanitation system depends on whether the waste is stored, treated and disposed at the point of generation or transported to somewhere else for treatment and /or disposed.

- Wet system
- Dry system

Sanitation system can also be classified as wet or dry systems based on the methods of collection and conveyance of wastes produced in a community. Wet and dry systems can be either on site or off site systems.

On site systems:

When the wastes are collected, treated and disposed of at the point of generation it is called an on site system. On site systems are widely used in rural areas of both developed and developing countries and in absence of more costly sewerage system, is also extensively used in urban areas of developing countries.

The simplest onsite sanitation system is a pit latrine, which consists of manually dug pit covered by a concrete, wooden or bamboo slab with a squatting hole. Some form of super structure is erected over the pit to ensure privacy to the user.

Cycle 7
Day C

The onsite sanitation system has over the years been developed and improved into a lot of different designs e.g ventilated improved pit latrines, pour flush single and double pit latrines, aqua privies, septic tank and so on.

The basic principles of on site systems however remain the same, liquids infiltrate into the soil and the solids are retained, anaerobically digested and have to be removed, or a new pits have to be dug at the regular intervals. The basic on site systems are primarily designed to dispose of human excreta.

Off – site systems:

When the waste is collected and transported to somewhere else for treatment and disposal, the system is called off site e.g. bucket latrine systems and conventional sewerage system. The basic elements of off site sanitation systems therefore include collection, transportation, treatment, disposal and /or reuse.

- The waste is collected either through house sewers or manually using buckets or vaults
- These are transported either by cart, truck or sewer system to a suitable distance place where it is treated prior to disposal or reuse

Cycle 7
Day C

Dry System:

In dry system no water is used for the dilution of the waste. They are usually applied to the unsewered areas with no piped water supply, e g, pit latrine systems (onsite), bucket latrine system (off site)

Wet system:

In the wet system the waste is diluted with flushes of water. Wet systems are suitable where piped water supply systems are available, e.g, septic tank system (on site), conventional sewerage systems (off site)

Cycle 7
Day C

Sometimes a sanitation system can be classified as either permeable or confined depending on whether the system allows infiltration.

- Confined systems do not allow infiltration of the liquid portion of the wastes into the ground, e.g aqua privies, septic tank

- In the unconfined permeable system the liquid part of the wastes is allowed to infiltrate , causing potential pollution of the ground water e.g pit latrines

Cycle 7
Day C

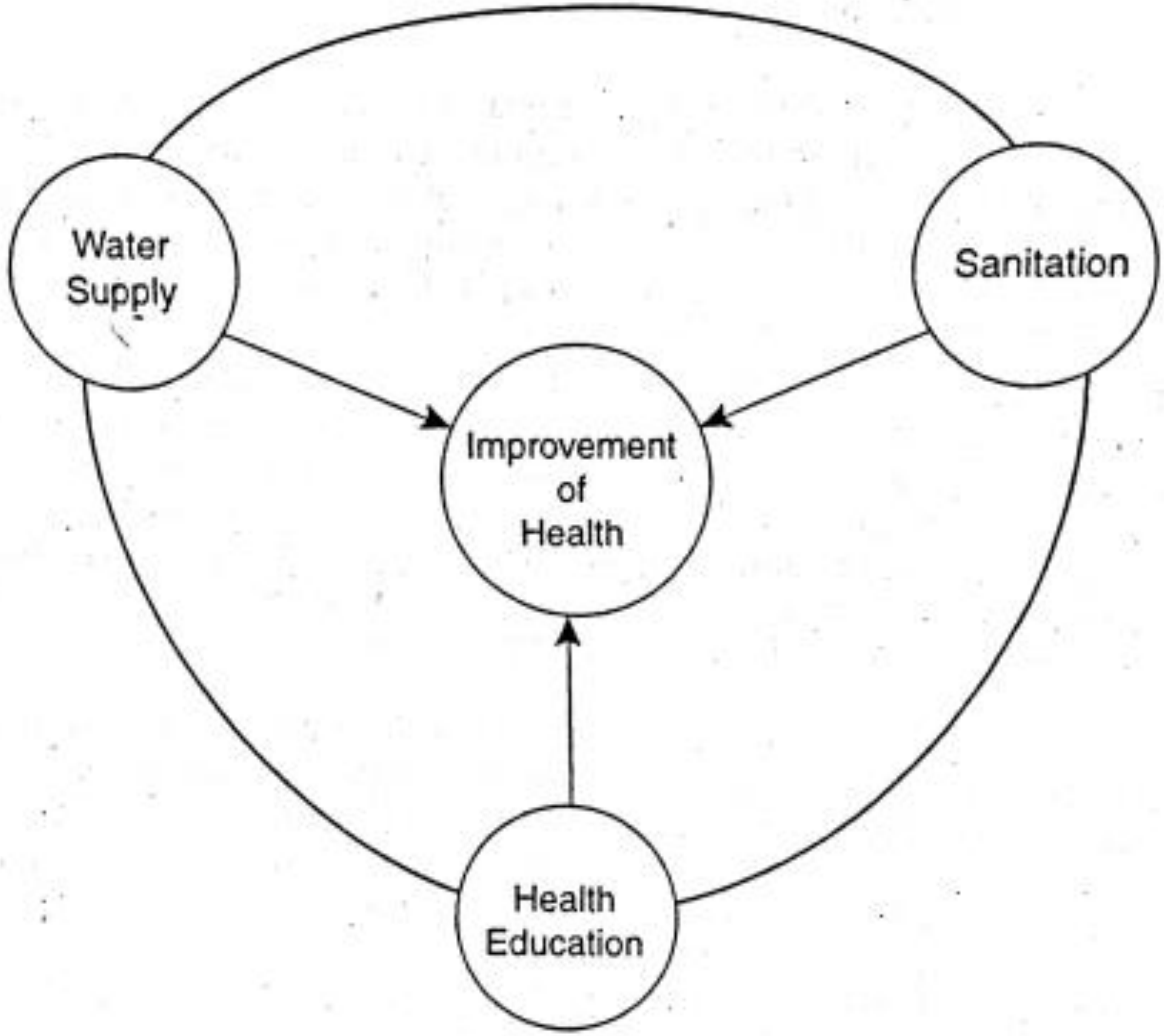


Fig : Interrelationship between water, sanitation and health education

Cycle 7
Day C

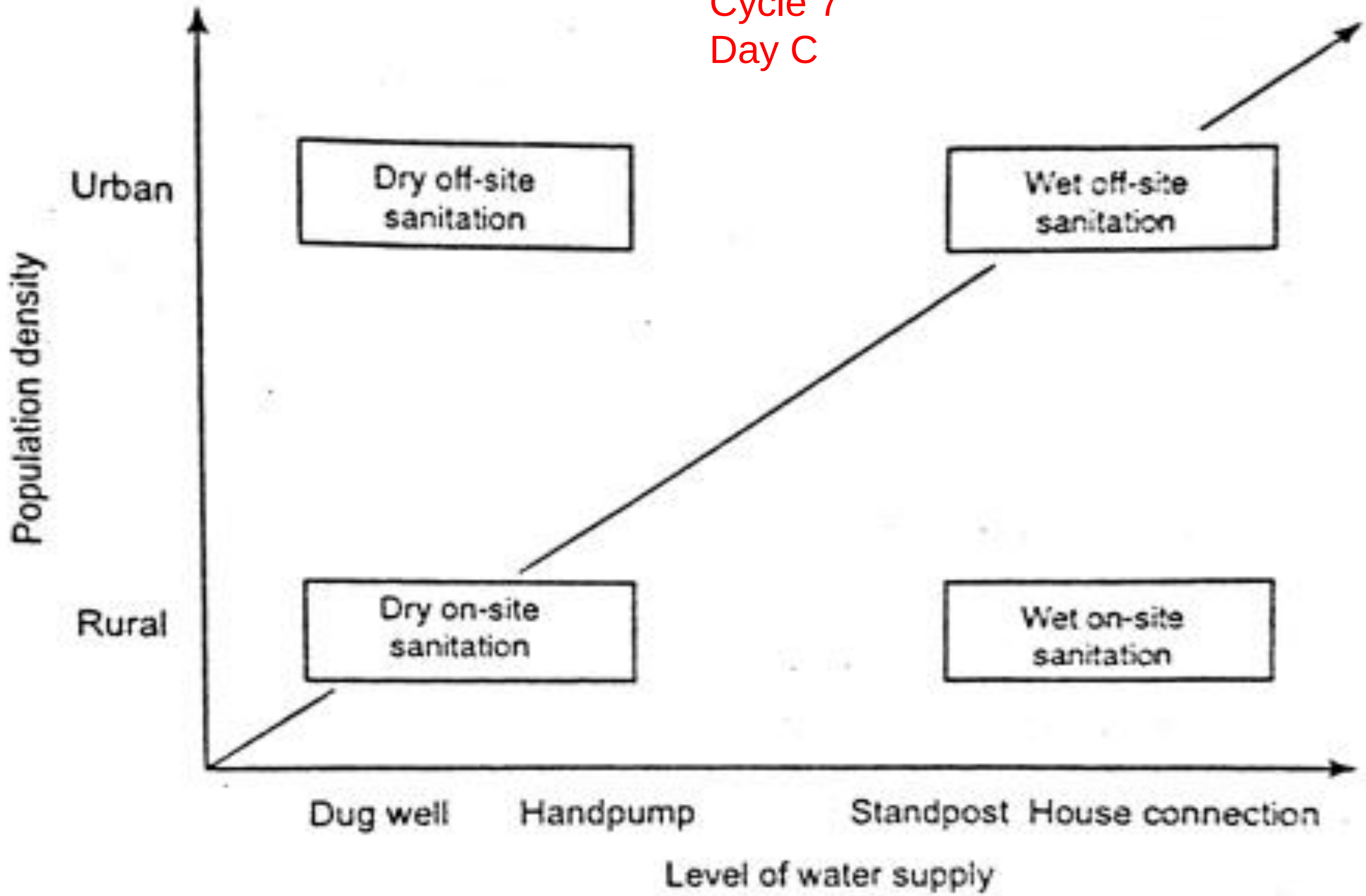


Figure 8.3 **Various sanitation systems based on water consumption level and population density (Source: Veenstra et al., 1997)**

Cycle 7
Day C

Important factors for sanitation in Bangladesh :

Housing density:

- Simple single pit latrines are suitable for use in rural areas and low density urban areas up to about 300 people per hectare.

- At higher densities alternating double pit latrines may be feasible, but other options such as small bore sewers, community latrine cum biogas plant etc may be more appropriate solutions

Cycle 7
Day C

Water supply and service level :

In areas where water use is low (say less than 30l/c/d) and where water has to be hand carried from public stand posts, tube wells or communal wells, pit latrines of one type or another are technically feasible sanitation option.

Difficulties associated with pit latrines:

Digging pits in loose and unconsolidated soils (e.g sand or fine grained alluvium) is difficult and the lining must nit prevent the seepage of faecal liquids out of the pit into the surrounding soil. Pit latrines are vulnerable.

Cycle 7
Day C

Soil Permeability:

Soils with permeability below 2.5 mm per hour are unsuitable for pit latrines, as the liquid fraction of the excreta in unable to infiltrate into the soil, thereby leading to overflow of the pits.

Infiltration capacity of the pits also greatly reduced where the water table is high e.g in the coastal region of the country.

Cycle 7
Day C

Ground water pollution:

The deposition of excreta in pits may pollute water sources, particularly wells, tube wells pond etc located nearby. The danger of pollution increases if the pit is dug down to the water table . Bacteria will not penetrate more than 1.2 m in most unsaturated soil but they have been known to travel over 100 m in gravel below the water table

So, users are required to locate pits at least 10 m away from tube wells or other water sources to avoid potential pollution.

Cycle 7
Day C

What is Water Supply Engineering

A branch of engineering concerned with the development of sources of supply, transmission, distribution, and treatment of water. The term is used most frequently in regard to municipal water works, but applies also to water systems for industry,

irrigation and other purposes.

Objectives of Water Supply

- Supply water in adequate quantity
- Supply safe and wholesome water to the consumers
- Make water easily available to consumers

Cycle 7
Day C

Elements of Water Supply System:

The planning of water supply system requires that the probable water use and its variation be estimates as accurately. The first step in the design of a water supply is the determination of the quantity of water that will be required with provision for the estimate requirements of the future. Next a reliable source of water must be located and finally a collection system, a treatment plant and a distribution system must be provided. The essential elements of a water supply system are (i) Source of supply, (ii) Collection system, (iii) Treatment of purification and (iv) Distribution system

Cycle 7
Day C

Source of supply:

All fresh waters on earth come in the form of precipitation evaporated mainly form oceans, seas and other surface water sources. A part of this precipitated water flows as surface water and remains trapped in depressed areas and another part enters in to the earth to form groundwater. Therefore, surface water and groundwater are the main sources of water for water supply. Rainwater being relatively free from contamination, can be a good source of water supply. The main considerations for the selection of a source of water supply are quantity, quality and cost.

Collection system:

The collection system is dependent on the water supply. An intake with pumping facilities is required if the water is to be collected from a surface water source. Dug wells or tube wells are common collection devices for groundwater, while a permanent roof or an uncontaminated ground surface is needed for collection of rain water for water supply.

Cycle 7
Day C

Treatment:

Natural waters usually contains impurities, which require treatment to make the water suitable for domestic water supplies. The type and degree of treatment processes involve removal of turbidity, color, taste and odour and removal and destruction of pathogenic (disease producing) micro-organisms. Groundwaters are relatively free from disease-producing bacteria but rich in mineral substances and may require removal of iron, hardness, arsenic,, fluoride etc. If the dissolved minerals in groundewaters are within acceptable limits, the water may be supplied without any treatment. The most common methods used for treatment include screening, sedimentation, aeration, treatment with treatment with chemicals, filtration, demineralization and disinfection.

Distribution system:

A distribution system is needed to deliver water to individual consumers. The piped water supplies require distribution networks of pipes with storage reservoirs, pumping devices, stand posts, valves and other appurtenances.

• In unpiped water supplies, the sources of water are to be distributed to make them easily accessible to the consumers.

Cycle 7
Day C

- The rural water supply based on manually operated tubewells does not require a distribution network but the tubewells are required to be distributed over the area in such a way that the distance from the households are reasonable and each tubewell serves an optimum number of household.

- In the location of community type treatment plants, accessibility and distances from the community are required to be taken into considerations.

Planning and Design Considerations

- Quality of water should not deteriorate below WHO standard
- Water in adequate quantity in convenient location
- Traditional sources should be selected for water supply development
- Construction, operation, maintenance and repair should be within reach of the available skills
- The equipment should be robust, reliable and locally available
- Construction and operation cost should be minimum
- Use of pumping and chemicals should be minimum
- System should be planned together with the community
- Women should be involved and consulted to address their needs

Cycle 7
Day C

Planning and Design Considerations

- In built system for monitoring of performance
- Provision for preventing deterioration of water quality
- Sustainability should be preferred in planning, designing and pricing the water supply

Cycle 8
Day A

Population and Water requirement

Demand of water is generally done by the help of the two factors

1. Probable population estimated at the end of the design period
2. Rate of water supply per capita per day

The design period is the period into the future for which the estimate is to be made. The period should neither be too long so that full financial burden is not thrown on the present generation, not it should be too short so as to avoid the design becoming uneconomical. In practice it varies from 25 to 40 years

Cycle 8
Day A

The per capita consumption of water is the total consumption divided by the population and the number of days in the year. It is generally expressed as liter per capita per day . Thus

Water consumption per capita per day = Total consumption/(Population X 365)

Cycle 8
Day A

Factors affecting the consumption of water:

I. Size of the city:

The per capita use of water tends to be higher in larger cities than in small towns. The difference results from great industrial use, more parks and other public facilities , greater commercial use and perhaps more loss and waste in the larger cities

ii) Characteristics of the people:

Water use is influenced by the economic status of the people. The per capita use of the slum areas will be much less than that in the high class residential areas, because of extra water use for gardening, car washing, air conditioning and for some ornamental display

iii) Climate condition:

Water is used is warm, dry climates than humid and cold climates for bathing, lawn and garden watering, air conditioning etc. In extremely cold countries water may be wasted as faucets to prevent the freezing of pipes

iv) Commerce and industries

v) Pressure of the water:

The rate of use of water increases when the pressure on the distribution system is increased. Increases in the rates of use of water with pressure have been known to reach 30 % for a change from 1.76 to 3.16 kg/cm^2. This fact should be lead to the designer to provide the lowest pressure that will give satisfactory service . Excess pressure means wastage of water

Cycle 8
Day A

vi) Quality of water

vii) Sewerage facilities

viii) Water rates and metering

ix) Nature of supply

x) Availability of private supply

xi) Efficiency in management

x) Number of inhabitants

Cycle 8
Day A

Water quantity:

Water is required by a community for domestic and other purposes. Usually a water supply system is designed for a future population. The total quantity of water is required by the community is computed by:

$Q_f = P_f$ x q

Where, Q_f is the quantity of water requirement per day,
P_f is the projected population estimated at the end of the design period

q is the rate of water consumption per capita per day

Cycle 8
Day A

Population projection:

The following seven methods are generally used for predicting future population.

Uniform growth rate method:

In this method a constant increment of growth is added for a period based on the population growth of the same period in the recent past. For the example if the population increased from 10,000 to 11,000 over the last five years, it would increase by an increment of 1000 in the next five years.

Uniform percentage growth rate method:

In this method a constant percentage of growth is assumed for an equal period of time. For example, if the population of the community increased from 10,000 to 11,000 during the last five years, it would increase another 10% to a population of 12100 during the next five years.

Decreasing growth rate method:

The method is similar to the uniform growth rate method but with an assumption of a decreasing rather then the uniform rate of increase.

For example, if the population of a community increased from 10000 to 11000 during five year period, it would increase by the decreasing rate say 900 rather then 1000 at the end of next five year period. The decreasing growth may result when growth potential is reduced.

Graphical extension or curvilinear method:

In this method the population time curve is extended by an eye estimation to obtain the future population as shown in figure

Cycle 8
Day A

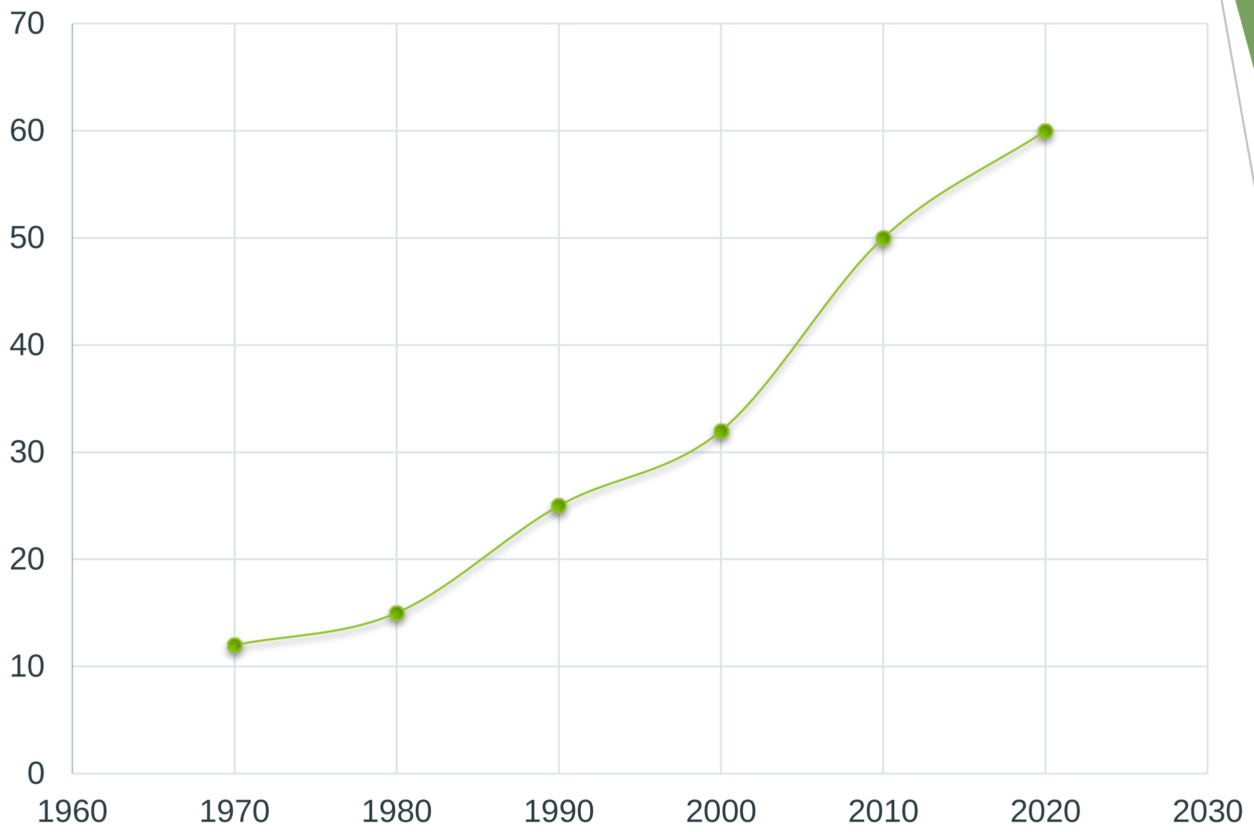


Fig : Population projection by graphical method

Cycle 8
Day A

Geometric progression method:

Geometric progression method is the most widely used method which is also known as empirical method suggested by Hardenbarg. The future population can be estimated by this method using the equation.

$$P_f = P_p(1+r)^n$$

Where P_f= future population

P_p= present population

n = number of years to be considered

r = rate of yearly population growth

Cycle 8
Day A

The population growth rate is available from the census report. The rate of population growth can also be estimated from population data of two years of n years interval in the recent past using the expression:

$$r = \sqrt[n]{\frac{P_2}{P_1}} - 1$$

Where P_2 and P_1 are the population of two dates of n number of years apart.

Cycle 8
Day A

Least square parabola method:

In this method the population of the population-time curve is determined by the least square method using the suitable data.

Let the variables X and Y denote the years and the population figures of those years respectively. The equation of the least square parabola fitting the time population data is given by:

$Y = a + bX + cX^2$ ....(1)

Cycle 8
Day A

Where a, b, c are constant to be found more normal equations by applying the actual population data:

$$\sum Y = aN + b\sum X + c\sum X^2 \text{ ………………………(2)}$$

$$\sum XY = a\sum X + b\sum X^2 + c\sum X^3 \text{ … … … … ….(3)}$$

$$\sum X^2Y = a\sum X^2 + b\sum X^3 + c\sum X^4 \text{ … … … ….(4)}$$

Where N= number of observation of set of data

If the population data of a community for a number of year is known the values of constants a, b, c in equation 1 can be computed by solving the equations 2, 3, 4

There are several methods also

Zoning Method:

This is rather the modern and the most useful method of population forecast. In this method the master plan of the city is prepared and it is divided into several zones such as residential zone, industrial zone, commercial zone etc.

Cycle 8
Day A

Ratio and correlation method:

It is evident that the population growth of a small area is related to some extent to the population growth of a wide area. Thus the rate of population growth of a town is related to some extent to the rate of population growth of state or nation. Hence it is possible to estimate the population of the town under consideration by considering the rate of the population growth of state or nation

Cycle 8
Day A

Growth composition analysis method:

There are only three ways by which change in the population occurs.

1) Through births
2) Through deaths
3) Through migration

If the above three factors for area under consideration are properly analyzed, the estimated population of some future period can be obtained by the following equation

Estimated population = present population + natural increase or decrease + migration

Cycle 8
Day A

Estimated natural increase = T $(I_B \mathrm{P} - I_D \mathrm{P})$

Where. T = Forecast period in years

I_B = rate of birth per year

I_D = rate of death per year

P = present population

Cycle 8
Day C

In the rural areas of Bangladesh, the water requirement for various purposes has been estimated as follows:

Drinking	:	2 – 3 lpcd
Washing cloths	:	8 – 10 lpcd
Washing utensils	:	6 – 8 lpcd
Cooking foods	:	3 – 5 lpcd
Bathing	:	14 - 20 lpcd
Others	:	9 –14 lpcd

Cycle 8
Day C

The typical water requirement for domestic purposes are given below:

Table 17.1: Water requirements for domestic purposes

Types of water supply	Water consumption range, lpcd	Typical water consumption, lpcd
Village open/tube wells and standpost		
Distance, > 1000m	5 - 10	7
Distances, 500 - 1000 m	10 - 15	12
Distance , 250 - 500 m	10 - 20	15
Distance , 50 - 250 m	15 - 40	25
Distance , < 50 m	20 - 50	35
Water supply source in yard		
Well in yard	20 - 60	40
Single tap in yard	30 - 80	50
House connection		
Single tap	30 - 60	50
Multiple tap	70 - 250	150

Cycle 8
Day C

Water requirement for other purposes

Nature of consumption	Range of consumption, lpcd	Average consumption, lpcd
Commercial use	10 – 150	40
Industrial use	30 – 450	120
Public uses	10 –100	25
Livestock	10 – 35	20
Loss and wastage	20 – 150	40

Cycle 8
Day C

Water requirements in rural and urban areas in Bangladesh

Areas	Water consumption, lpcd
Rural areas	50
Upazila towns	100
Zilla town	120
City corporation	180

Cycle 8
Day C

Essential elements for designing a water supply system for a city:

1. The number of years, design period for which the proposed water supply system and its component structures and equipment are to be adequate.
2. The number of people or the design population to be served
3. The rates of water use, or design flows, in terms of per capita water consumption including industrial, commercial and fire equipment.
4. The area to be served, or design area, and the allowances to be made for population density and a real water consumption from residential and industrial districts

Cycle 8
Day C

Design periods are chosen with the following factors in mind

1. Useful life of component structures and equipment taking into proper account of wear and tear
2. Ease of difficulty of extending or adding to the existing and planned works, including a considerations of their location
3. Anticipated rate of population growth, including possible shifts in community, industrial and commercial development
4. Availability of fund
5. Performance of the works during their early years when they will not be loaded to the capacity

Sources of water supply

Cycle 8
Day C

Water on Earth

Seawater	96.5%
Ice and Snow	1.76%
Atmospheric Water	0.001%
Sub-Total	**98.26%**
Freshwater Available	1.74%
Groundwater	1.7%
Lakes	0.013%
Rivers	0.002%
Total	**100%**

Cycle 8
Day C

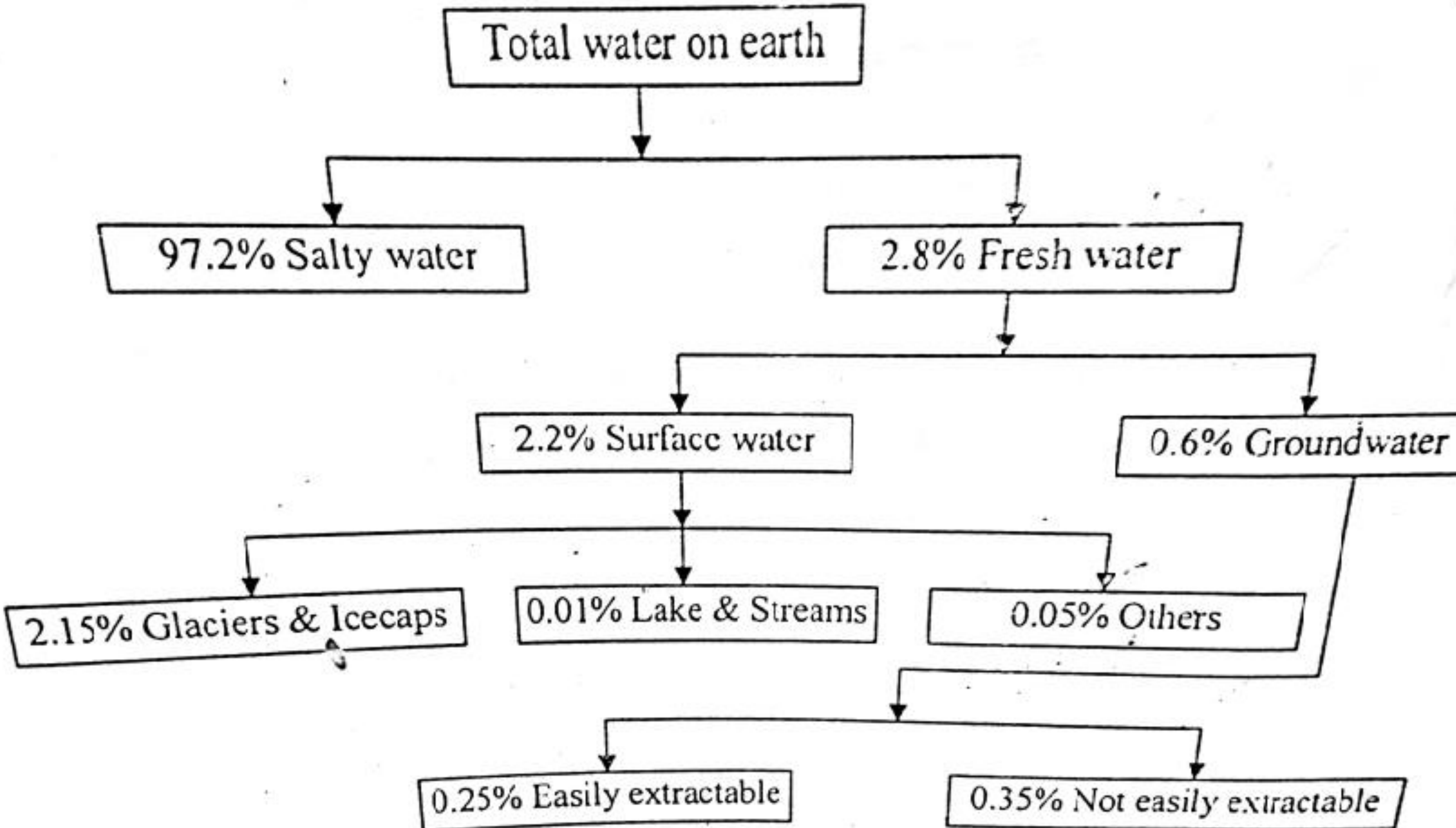


Figure 5.1: Water Distribution on Earth

Cycle 8
Day C

Average Renewal Time for Various Water Resources

Atmospheric Water	8 days
River Water	16 days
Soil Water	1 year
Wetlands Water	5 years
Lake Water	17 years
Groundwater	1,400 years

Sources of Groundwater:

The following are the major sources of groundwater:

- Meteoric water
- Connate water
- Magmatic water

Meteoric water:

This is the main source of groundwater. This water is received in the form of rain and snow. Water from tanks, lakes, rivers and seas is again received by earth after vaporization. Water is received by melting of snow or rain, hence it is called ‘meteoric’ or ‘shooting star water’. From the surface of the earth, this water infiltrates down below through rock joints, pores and fissures of rocks and is stored at the level of impermeable rocks in the form of groundwater.

Connate Water:

Water contained in pores and cavities of sedimentary rocks under seas and lakes is called connate water. It is also called ‘sediment water’. It is the second important source of groundwater. This is the water that is entrapped in the interstices of sedimentary and volcanic rocks at the time of deposition. Connate water is highly mineralized and salty and does not mix readily with meteoric groundwater. Connate water is usually found deep down in the lower layers of the zone of saturation.

Cycle 8
Day C

Magmatic Water:

Hot magma enters rocks due to volcanic action after which its vapor drops are condensed and converted into water. This is called magmatic water. Apart from it, other sources are those in which groundwater becomes again available on the surface of the earth.

They are mainly springs, wells, and geysers. Such water is considered to have been generated in the interior of the earth. It has consequently travelled to the upper layers of the earth’s surface for the first time; this is also known as magmatic water.

Groundwater aquifer:

An **aquifer** is an underground layer of water-bearing permeable rock, rock fractures or unconsolidated materials (gravel, sand, or silt) from which groundwater can be extracted using a water well.

Aquifers are permeable formations having structures which permit appreciable quantity of water to move them under ordinary field conditions. Aquifers are mainly two types.

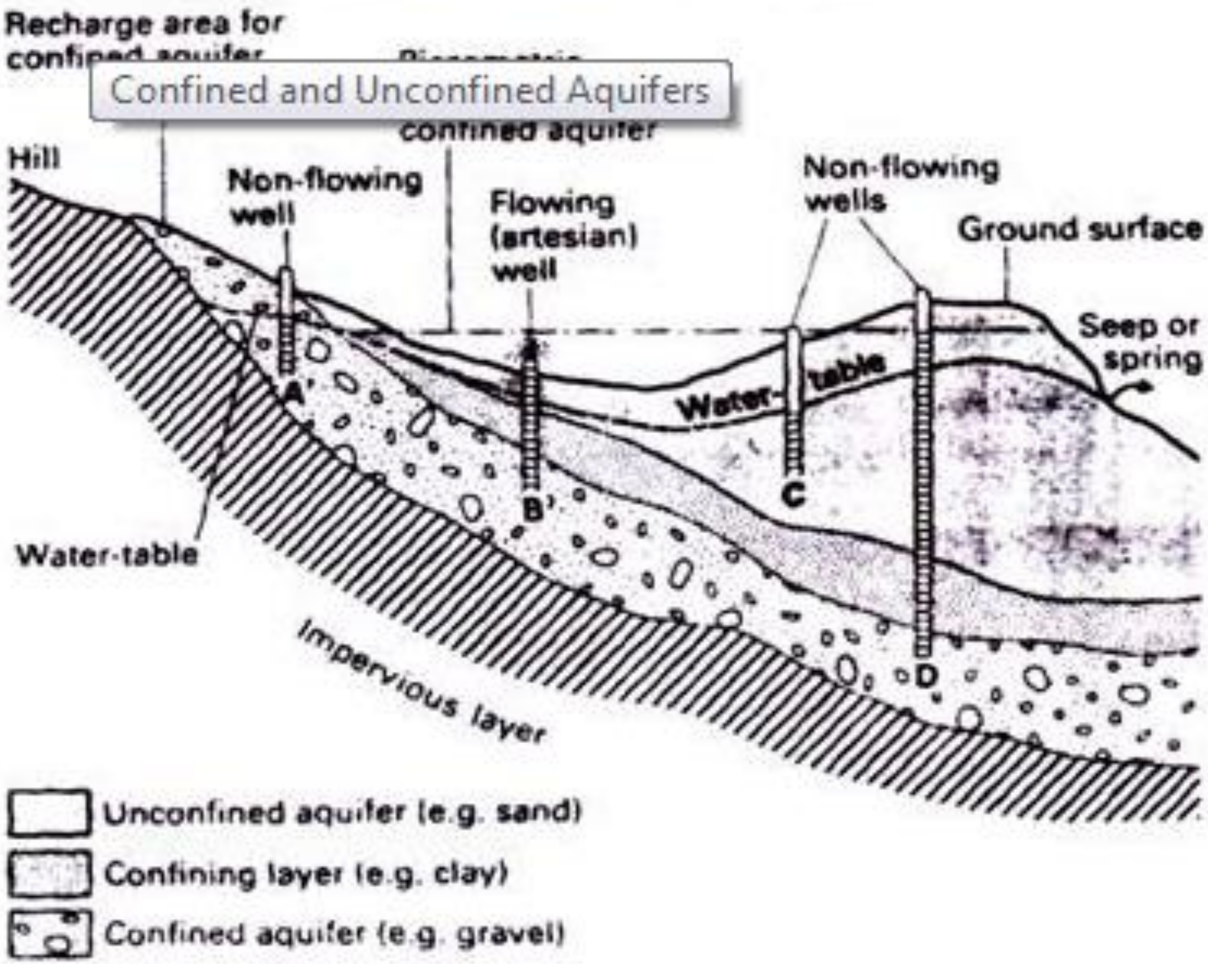
- Unconfined aquifer
- Confined aquifer

Unconfined aquifers are sometimes also called *water table* or *phreatic* aquifers, because their upper boundary is the water table or phreatic surface. Typically (but not always) the shallowest aquifer at a given location is unconfined, meaning it does not have a confining layer (an aquitard or aquiclude) between it and the surface.

Confined aquifers are aquifers that are overlain by a confining layer, often made up of clay. The confining layer might offer some protection from surface contamination.

Cycle 8
Day C

Figure 4.5
Confined and Unconfined Aquifers



Forms of Groundwater sources:

There is four forms of groundwater sources are found

1. Infiltration galleries
2. Infiltration wells
3. Springs
4. Wells

Infiltration galleries:

When water can be obtained within a reasonable distance below ground level, for example, below the river bed, horizontal porous pipes with open joints can be laid under the ground. It is apparent that a very large proportion of the groundwater will be intercepted by galleries than by a vertical well.

Cycle 8
Day C

The intercepted water can be collected at suitable points in vertical collecting wells and can be used. Inverted filter surrounding the pipes can be provided to prevent clogging of pipes. Horizontal galleries laid at a depth more than 8 meters are uneconomical. Figure 1.5 shows cross-section and longitudinal section of the infiltration gallery.

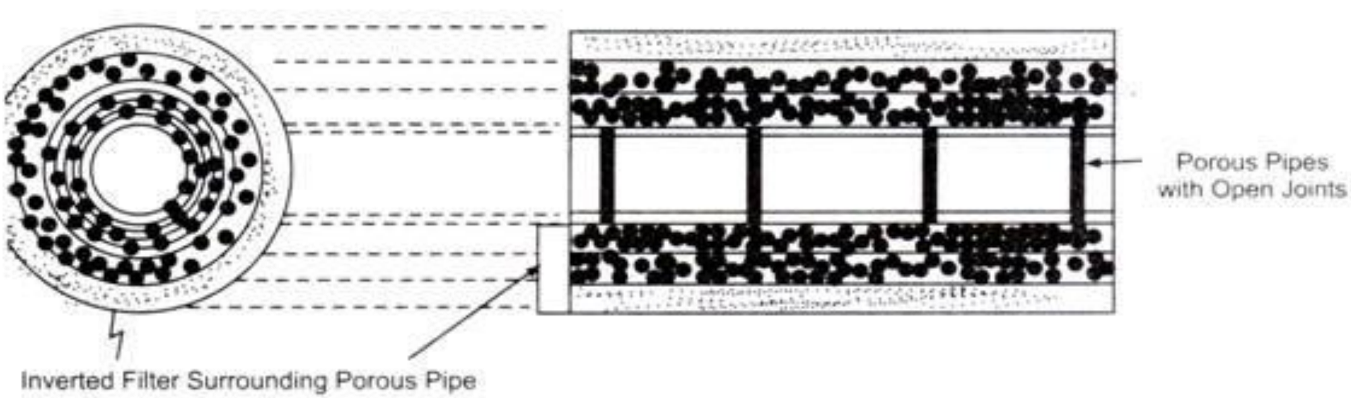


Fig. 1.5. Infiltration gallery

Cycle 9
Day A

Infiltration Wells:

Sometimes water can be made available by sinking infiltration wells in the porous soil, for example, in the dry bed of a river. The infiltration well can be joined to vertical collecting wells or jack wells sunk on the bank of the river by means of horizontal underground porous pipeline. It intercepts water also and is called Infiltration gallery. Thus it can be recognized that infiltration wells and galleries supplement each other. Figure 1.6 shows the cross-sectional elevation and plan of an infiltration well.

Cycle 9
Day A

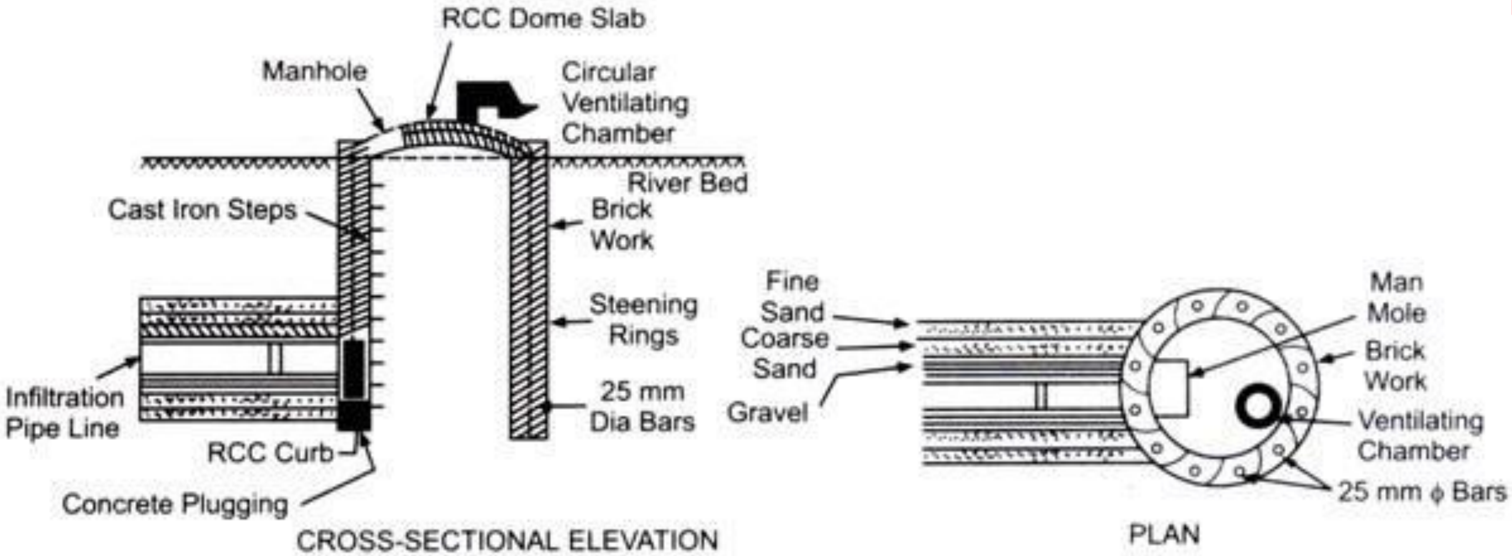


Fig. 1.6. Infiltration well

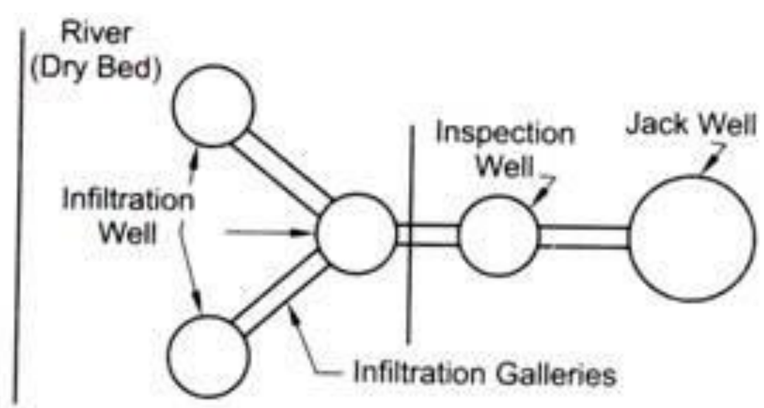


Fig. 1.7. Plan showing arrangement of wells and galleries

Springs:

When groundwater appears at the surface for any reason, the springs are formed. They serve as source of water supply for small towns, specially near hills or based of hills. Some springs discharge hot water due to the presence of sulfur and other minerals in their formations. These hot springs cannot be used to supply water for domestic purposes. But the hot water is found to cure some of the human disorders. Springs are mainly three types.

1) Artesian Spring
2) Gravity Spring
3) Surface Spring

1)Artesian Springs:

In this type of spring, the groundwater comes to the surface under pressure. The artesian spring may also be formed due to the presence of fissure or crack in impervious layer. The fissure or crack should be continued up to the ground surface.
The artesian springs give practically uniform quality of water throughout the year.

2)Gravity Springs:

This type of springs develop due to overflowing of water table. The flow from a gravity spring is variable with the rise or fall of water table. In order to meet with such fluctuations, a trench may be constructed near such a spring. The trench act as a storage reservoir.

3) Surface Springs:

This type of spring is formed when subsoil water is exposed to the ground surface by the obstruction of an impervious layer. The quality of water available from surface springs is quite uncertain and the cut-off walls, may be constructed to develop such springs. It is found that the quality of water depends on geological and topographical conditions and it may be hard or soft, pure or polluted.

Cycle 9
Day A

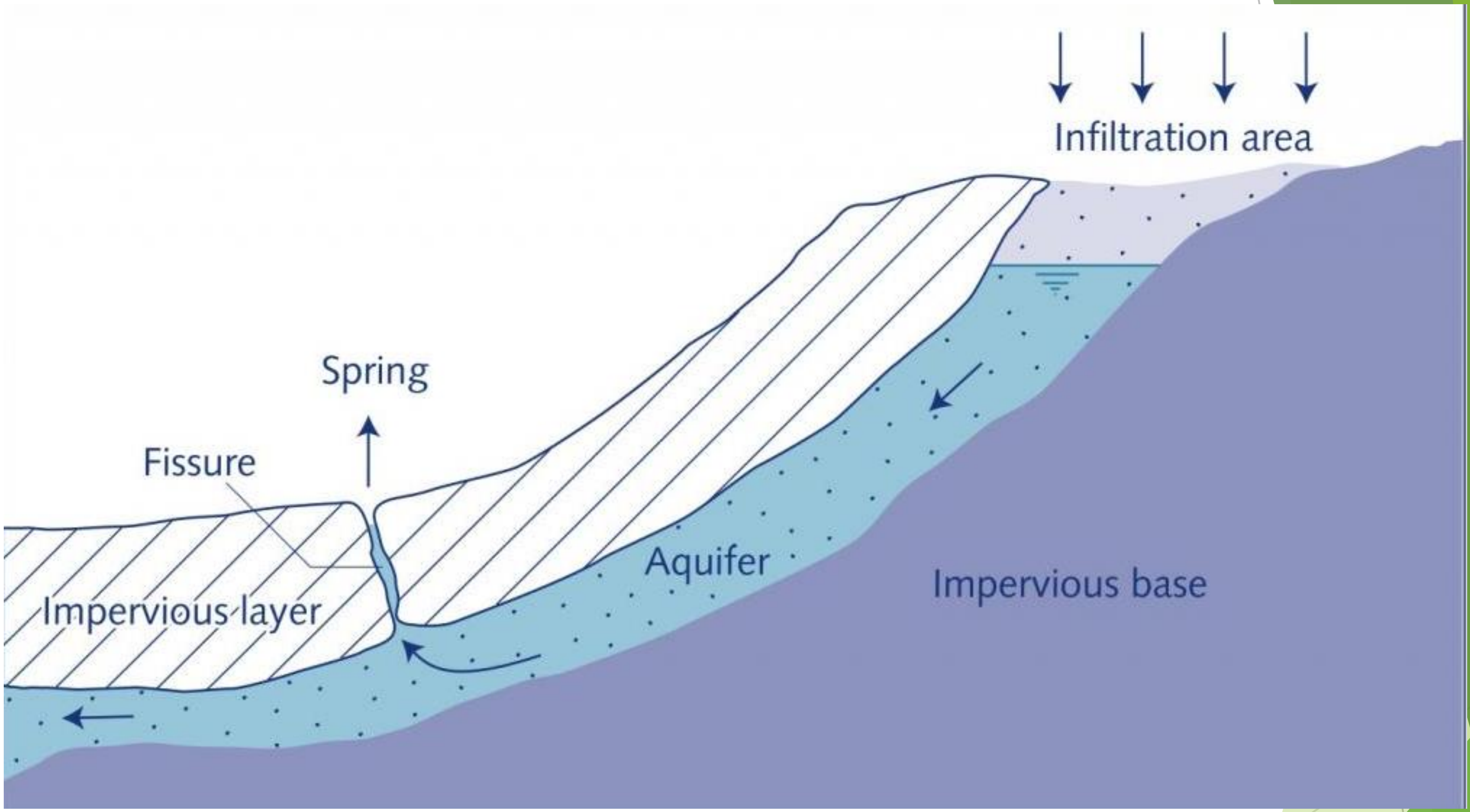


Fig: Artesian Spring

Cycle 9
Day A

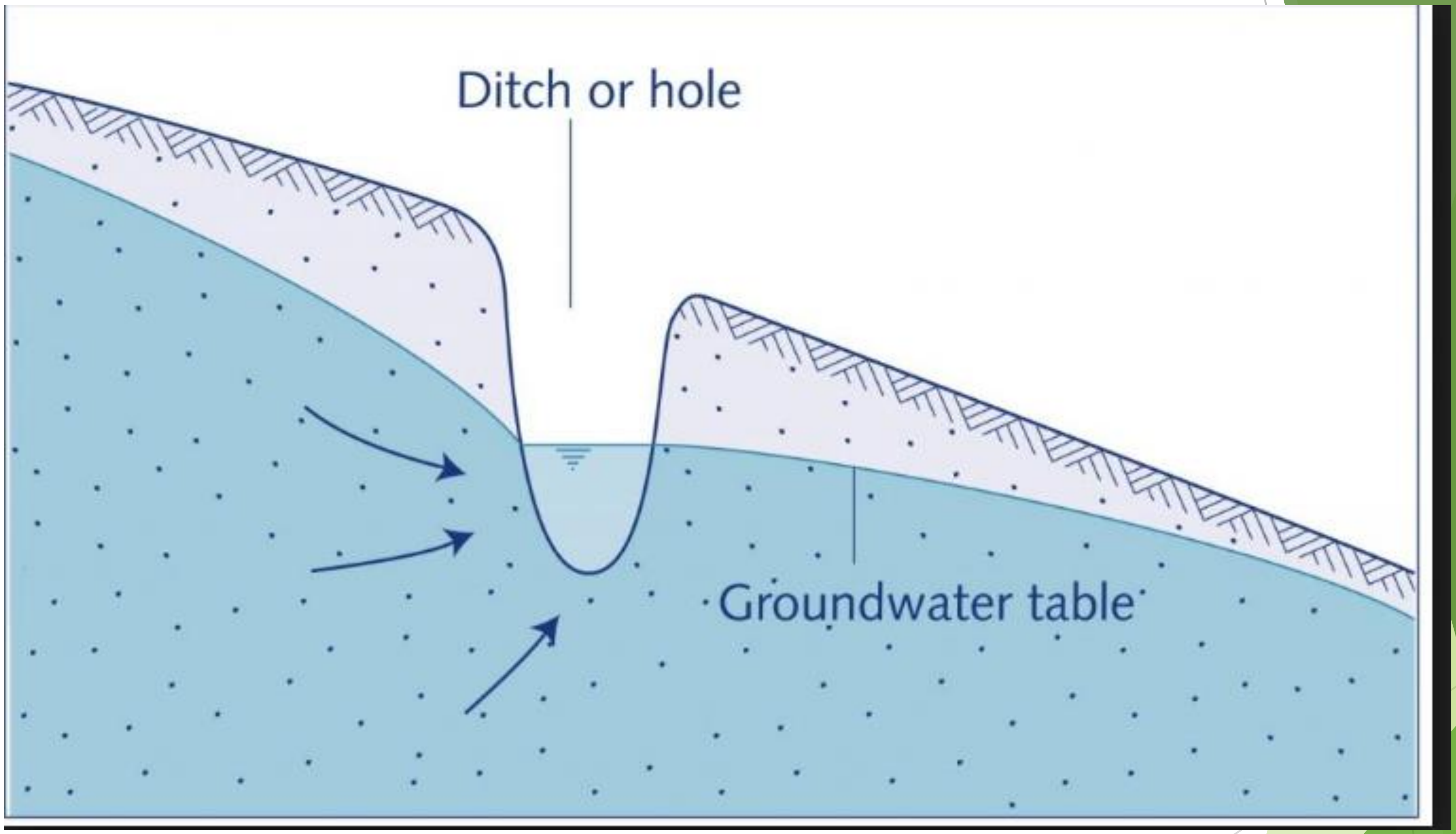


Fig: Gravity Spring

Cycle 9
Day A

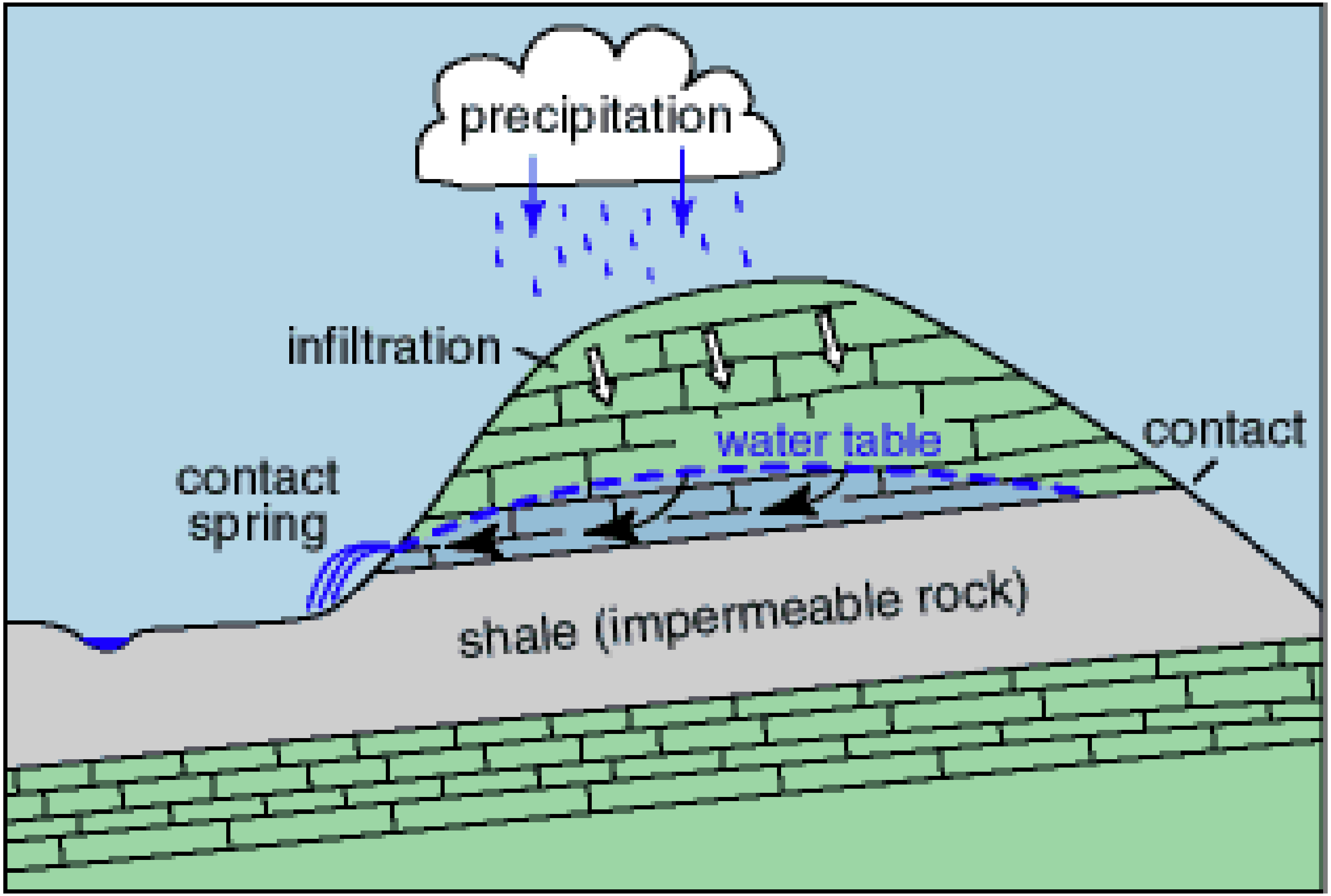


Fig: Surface Spring

Wells:

A well is defined as an artesian hole or pit made in the ground for the purpose of tapping water. The holes made for tapping oil are also known as well. But in the general sense, a well indicates a source of water. Three factors govern the theory of wells. These are geological conditions of the earth surface i.e., slope of the water bearing strata, porosity of various layers, and quantity of water which is absorbed and stored in different layers. The wells are generally classified as Shallow wells, Deep wells, Tube wells and Artesian wells.

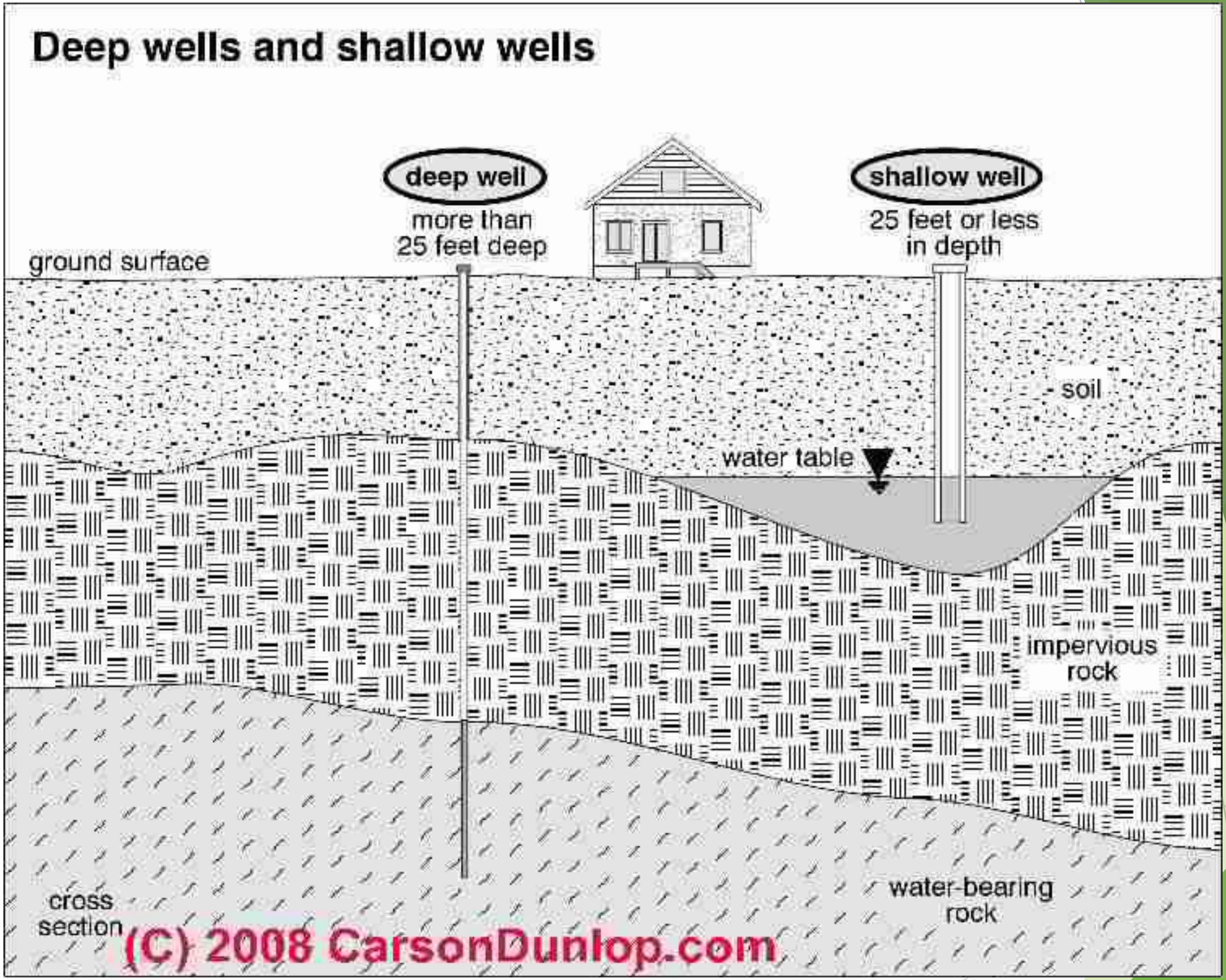
Shallow wells:

The shallow wells are constructed in the upper most layer of the earth's surface. They obtain the quota of water supply from the ground water table. The diameter of the shallow wells varies from 2 to 6 m. They may be lined or unlined from inside. The lining is also called the steining and its thickness varies from 30 to 50 cm. These wells are also sometimes referred to as draw wells or gravity wells or open wells or dug wells or percolation wells.

Deep wells:

The deep wells obtain their quota of water from an aquifer below an impervious layer. The theory of deep well is based on the travel of water from the outcrop of the site of deep well. The outcrop is the place where the aquifer is exposed to the atmosphere. The entry of rainwater takes place at outcrop and it reaches the site of deep well. During travel the water gets thoroughly purified. But it dissolves certain salts and may therefore become hard. In such cases, some treatment would be necessary to remove the hardness of water.

Cycle 9
Day A


Deep wells and shallow wells
deep well
more than
25 feet deep
shallow well
25 feet or less
in depth
ground surface
soil
water table
impervious
rock
water-bearing
rock
cross
section
(C) 2008 CarsonDunlop.com

Tubewells:

A tubewell is a deep well having a diameter of about 50 to 200 mm obtains it quota of water from s number of aquifers. The blind pipes are placed against the impervious layers.

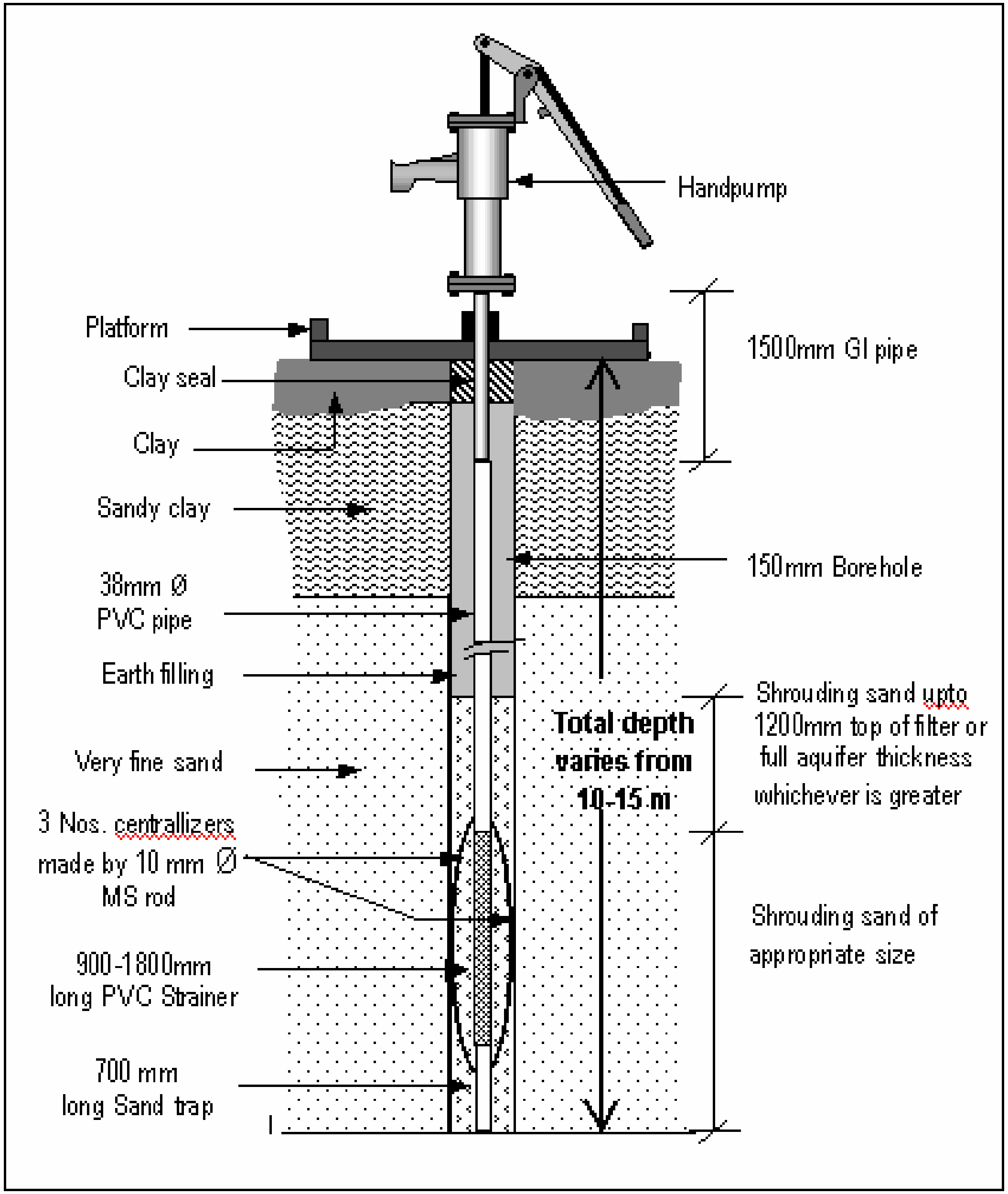


Fig Tube well

Cycle 9
Day A

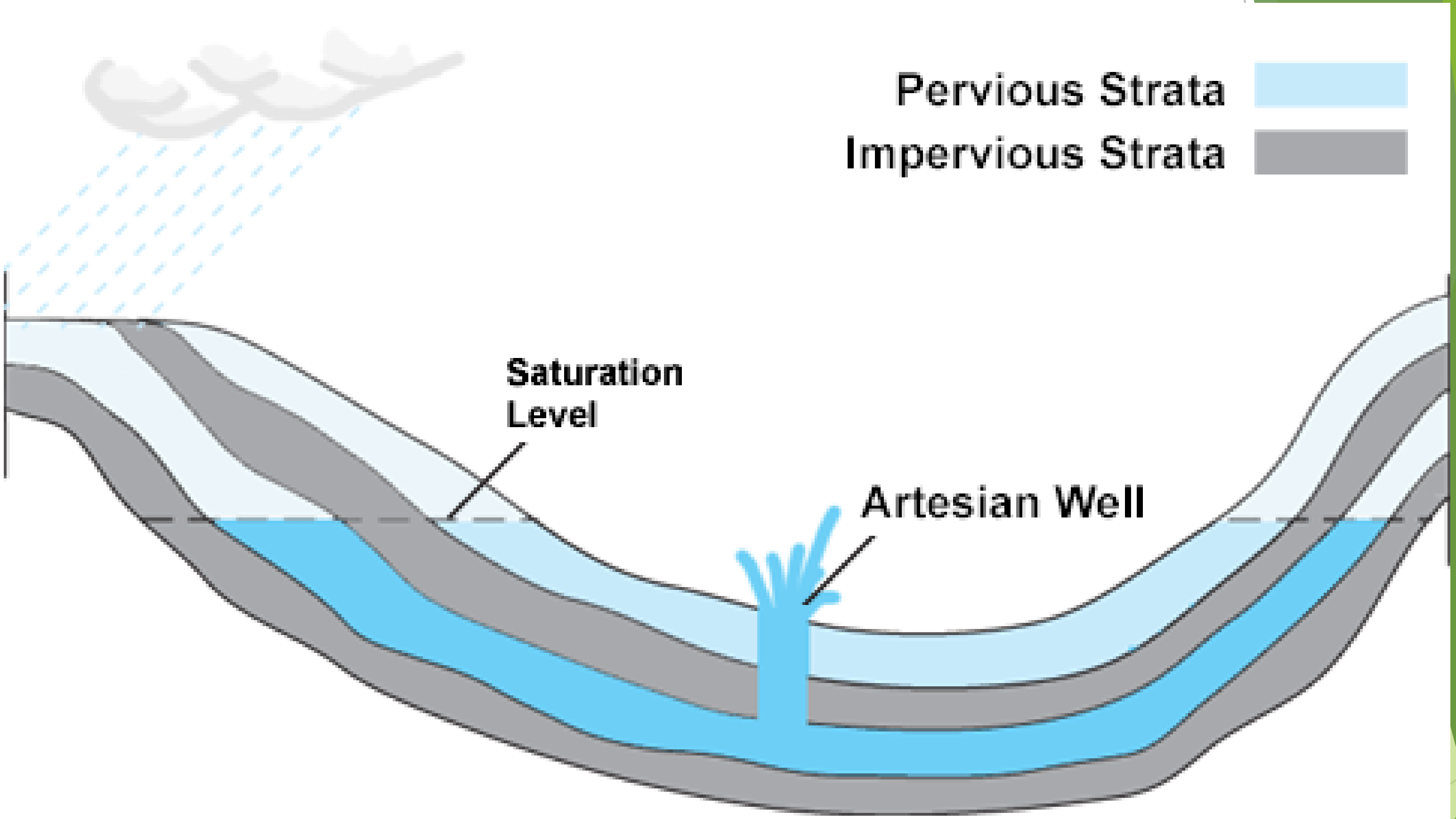


Fig: Artesian Well

Cycle 9
Day C

Groundwater movement:

The groundwater moves due to action of gravity and molecular attraction of surface tension . The velocity of flow depends upon on three factors

1. Slope of the groundwater surface
2. Hydraulic properties of soil through which it flows
3. Temperature of water

Hazen's gave a formula for finding the velocity of flow, is as follows

$$V = Cd^2S \ldots\ldots\ldots\ldots\ldots\ldots..(1)$$

Where, V is the average velocity of flow in meters per day,

C is coefficient varying from 400 to 1000

D is effective size of soil grains in mm and

S is slope of water table.

The discharge of aquifer can then be obtained easily by the multiplication of area of cross- section of aquifer and velocity of flow of groundwater. Mr. H Darcy enunciates a law governing the flow through soil, According to this law,

Q = KIA……………………………………..(2)

Where Q = discharge, I = hydraulic gradient
A = area of cross-section and
K = proportionality constant known as the coefficient of permeability

The discharge velocity, V = Q/ A…………………(3)

Permeability:

The capacity of transmission of water of the entire soil of full width and full depth is represented by permeability. It is defined as the average velocity of flow that will occur through the total cross sectional area of soil under unit hydraulic gradient.

Transmissibility:

The term transmissibility is used to indicate the same physical meaning of the permeability, but differing only mathematically. The capability of the soil unit width and full depth (i.e . Width = 1 and hence A = depth) is known as the Transmissibility

Specific yield:

The specific yield of an aquifer is defined as the ratio, expressed as percentage of the volume of water after being saturated, can be drained by gravity to the total volume of aquifer.

Specific yield = (Volume of water drained by gravity)/Total volume

Specific retention

The specific retention of an aquifer is the ratio expressed as a percentage of the volume of water it will retain after saturation against the force of gravity to its own volume.

Specific retention = (Volume of water to be retained)/ Volume of water

Storage coefficient

The water yielding capacity of a confined aquifer can be expressed in terms of its storage coefficient. Storage coefficient is defined as the volume of water that an aquifer releases per unit surface area of aquifer per unit change in the component of head normal to that surface. In most confined aquifers, the value of storage coefficient ranges 0.00005 to 0.005

Cycle 9
Day C

Unconfined steady flow: Dupits's formula for the flow into an ordinary well is based on Darcy's law, presented in equation 5.2. The hydraulics of flow for a well through an unconfined aquifer is presented in Figure 5.11. It is assumed that the flow of groundwater is horizontal and radial towards the center of the well. The slope of the cone of depression and area of flow at a distance x from the center of

Well is represented by

$$i = \frac{dy}{dx} \quad \text{and} \quad A = 2\pi xy$$

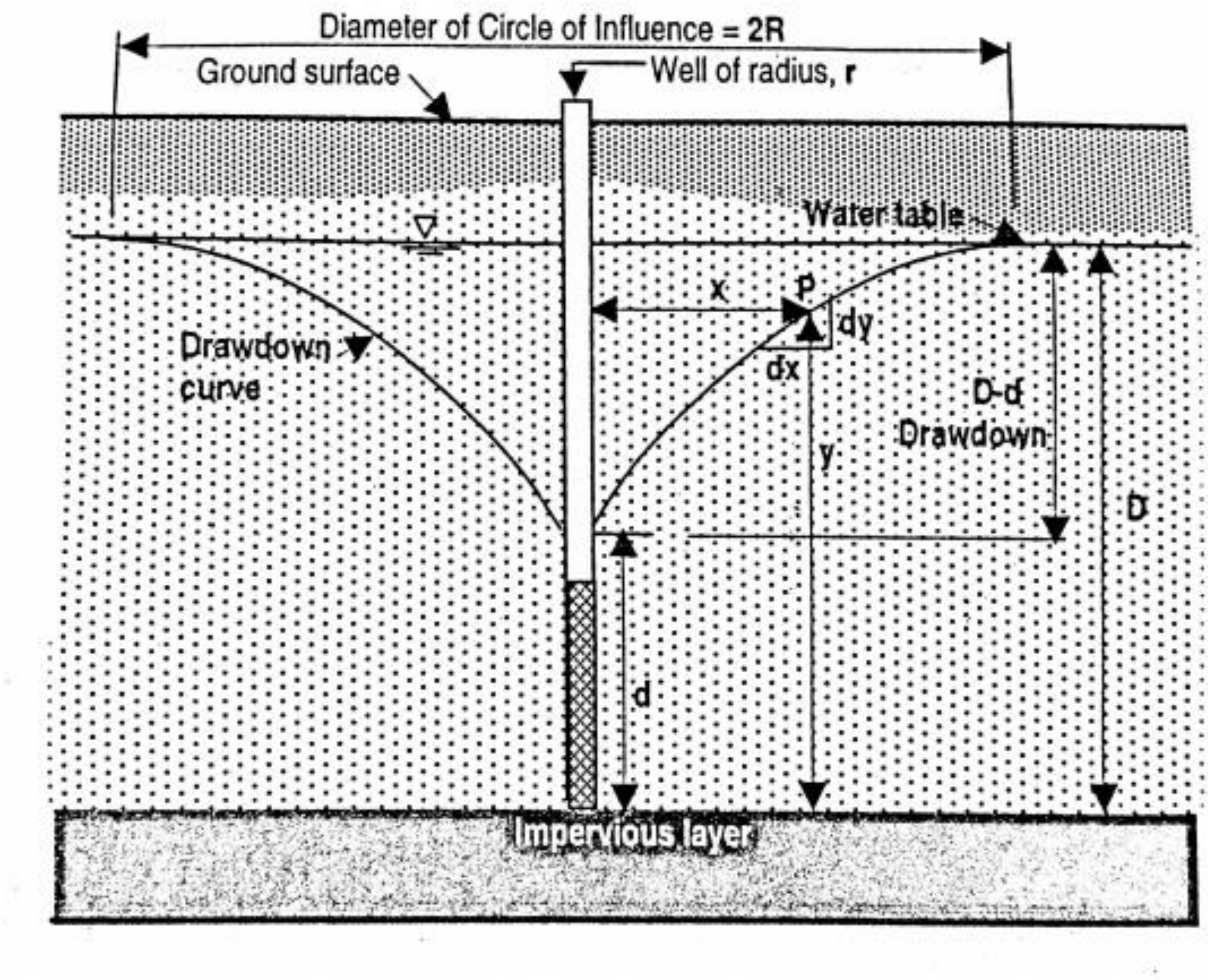
Substituting these values in equation

$$Q = 2K\pi xy \frac{dy}{dx}$$

or $$Q\frac{dx}{x} = 2\pi Kydy$$

by integration $Q \operatorname{Log}_e x = \pi K y^2 + C$ where C is a constant.

Cycle 9
Day C



Diameter of Circle of Influence = 2R
Ground surface
Well of radius, r
Water table
x
P
dy
dx
Drawdown curve
D-d
Drawdown
y
D
d
Impervious layer

For $y = d$ at $x = r$, r being the radius of the well and $y = D$ at $x = R$, R being the radius of the circle of influence or distance of the outer boundary from the centre of the well, the equation (16.6) becomes:

$$Q = \frac{\pi K(D^2 - d^2)}{\log_e(\frac{R}{r})} \tag{16.7}$$

Where
Q = well discharge, m^3/d
K = coefficient of permeability, m/d
D = depth of the aquifer, m
d = static head, m
R = radius of circle of influence, m
r = radius of the well, m

The equation 16.7 shows that if the term (D-d) is small as compared to (D+d), the flow Q varies approximately as (D-d) for a well installed in a particular aquifer. This linear relationship between the rate of flow and drawdown leads to the definition of *specific capacity* of a tubewell installed in an aquifer. *Specific capacity* of a tubewell, therefore, is the rate of flow per unit drawdown, which may be expressed as $m^3/d/m$ of drawdown.

Cycle 9
Day C

Confined steady flow: The flow condition in an artesian or pressure well has been illustrated in Figure 5.12. In an artesian tube well the drawdown occurs in the piezometric surface and the depth of flow remains constant and equal to the thickness of the pressure or artesian aquifer m. In the case of an artesian well:

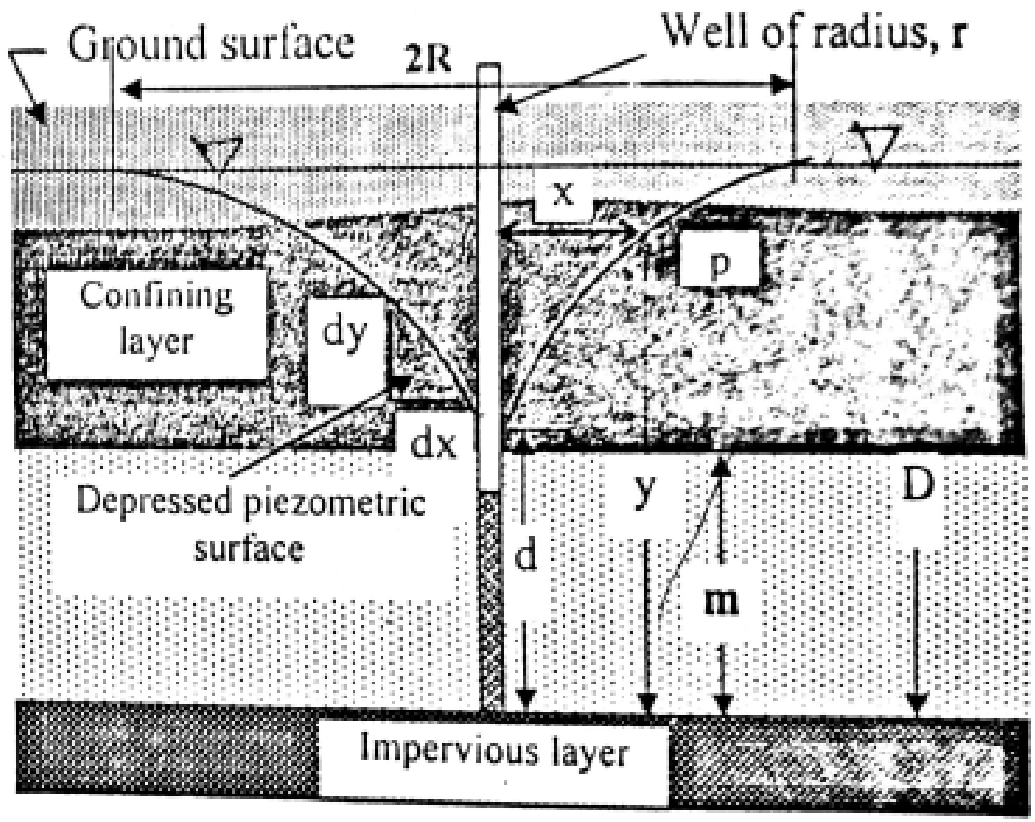


Figure 5.12: Well in confined aquifer

$i = \frac{dy}{dx}$ *and* $A = 2\pi x m$ Substituting the value of i and A in equation 5.2.

$$Q = 2K\pi x m \frac{dy}{dx} \quad or \quad Q\frac{dx}{x} = 2K\pi x m dy \qquad (5.9)$$

Where m is the thickness of the confined aquifer. Integrating equation (5.9) between the limits x = r for y = d and x = R for y = D:

$$Q = \frac{2\pi Km(D - d)}{\log_e(\frac{R}{r})} \text{ --(5.10)}$$

The rate of flow Q of an artesian well sunk in a confined aquifer is proportional to the drawdown (D-d).

Interference of well: Interference between two or moiré tube wells occurs when their cone of depression overlap. Interference reduces the discharges of the interfering tube wells. Ideally the tube wells should be spaced enough to avoid interference. Small diameter tube wells installed in fine sand will produce too little drawdown to cause interference.

Problems of groundwater development in Bangladesh:

Groundwater is the main source of water supply in urban and rural areas of Bangladesh. Groundwater in Bangladesh is available abundantly, but the available groundwater for drinking purposes has become a problem for the following reasons.

- Arsenic in groundwater
- Excessive dissolved iron
- Salinity in costal areas
- Lowering of groundwater level
- Rock/stony layers in hilly areas

Arsenic in groundwater:

The concentration of arsenic in drinking water in excess of permissible limit is toxic to human body. According to the WHO guideline value the desirable maximum concentration of arsenic in drinking water would be 0.01 mg/l. In Bangladesh the maximum acceptable concentration in drinking water is considered to be 0.05 mg/l. Symptoms of arsenic toxicity leading to cancer may occur due to excessive intake of arsenic in the human body over a longer period of time. In Bangladesh the presence of arsenic in groundwater first detected in 1993 at Baroghoria union of Capai Nawabganj district. Appreciating the gravity of the problem, water sample testing activities started in 1995 by various organizations and agencies. It has been observed that about one in every three shallow tube wells is producing water with arsenic in acceptable limits.

Excessive dissolved iron:

In Bangladesh the permissible limit of iron in groundwater is 1 mg/l but iron content upto 5 mg/l is acceptable for rural water supply. It has been observed that iron content exceeds this limit in many handpump tubewells. It has been observed from a survey in 1993 1230 unions in Bangladesh have an iron content more than 5mg/l. It may be observed that dissolved iron in shallow tubewell water in about 67% areas of Bangladesh is in excess of 2 mg/l. However iron content in deep tubewell water is comparatively lower. In urban areas iron removal plants have been constructed and in rural areas community-type iron removal units attached to handpumps are provided.

Salinity in coastal areas:

The concentration of dissolved minerals in groundwater is higher than that of surface water. The coastal belt of Bangladesh, extended over 86 upazilas, is identified as a problem area where complex hydrological conditions and adverse water quality make water supply difficult as compared to other parts of the country. Unlike other of Bangladesh, groundwater of acceptable quality at relatively shallow depths, which can be easily withdrawn by conventional handpump tubewells is not available in most parts of the coastal area. In some places low salinity water has been found in deep aquifers. Exploratory drillings with borehole logging have been conducted in many places to locate sweet water aquifers. In rural water supply chlorine content upto 1000 ppm is acceptable for coastal belts where the normal acceptable limit is 250 ppm

Lowering of groundwater level:

Although groundwater in Bangladesh is said to be abundant, a considerable areas of the country faces scarcity of groundwater within suction limit in the dry season

Due to the over-exploration of groundwater for irrigation purposes, the water level declines, rendering thousands of suction mode no.6 tubewells inoperable. In the low water table areas the force mode Tara and Tara-II pumps are provided.

Rocky/stony layers in hilly areas:

In Chittagong Hill Tracts Districts of Bangladesh. Drilling of tubewells for water supply is difficult due to the presence of hard formations in the subsurface. In most cases conventional drilling methods for the installation of handpumps cannot penetrate these hard rock formations. These situations also prevails in some part of Chittagong, Cox’s Bazar, Sylhet, in the north of

Mymansing, Netyrokona and Panchagar districts. To overcome the situatuation and provide portable water through handpump tubewells, mechanical drilling rings are used. These however increases the installation cost of the water point.

Storage reservoirs

An artificial lake formed by the construction of dam across a valley is termed as a storage reservoir. Whatever may be the size or use of a reservoir, the main object or function of a reservoir is to store water and thus it stabilises the flow of water. The most important physical characteristic of a reservoir is therefore its storage capacity. The topographic survey of the dam site is carried out and a contour map is prepared. The capacity of reservoir is then worked out with the help of the contour map.

A storage reservoir essentially consists of the following three parts:

(i) A dam to hold water.

(ii) A spillway to allow the excess water to flow and

(iii) A gate chamber containing necessary valves for regulating the flow of water.

Selection of site: Following are the factors, which are to be taken into consideration while selecting the site for a storage reservoir:

(i) Area of land to be submerged by the construction of reservoir;

(ii) Availability of construction materials and possibilities of using local materials for the construction of dam;

(iii) Availability of good foundation bed for dam;

(iv) Availability of skilled labour for the construction of dam;

(v) Chances of biological troubles;

(vi) Characteristics of catchment area;

(vii) Density of population over the catchment area;

(viii) Distance between the proposed site and the point of distribution;

(ix) Elevation of reservoir level;

(x) Facilities of transport for men and materials;

(xi) Geological conditions of basin of storage area;

(xii) Nature of land to be acquired;

(xiii) Possibilities of earthquake occurrences due to the storage of water;

(xiv) Quality of water available;

(xv) Quality of water likely to come to the reservoir site;

Storage capacity of the reservoir:

Following are the two methods, which are used to compute the storage capacity of reservoir

1. Analytical method
2. Graphical or mass curve method

Tubewell Technology:

A large variety of tubewells have been designed for abstraction of groundwater and are being used all over the world. The tubewells designed and developed, including those being used in Bangladesh, may be grouped under 3 categories:

- Shallow tubewells
- Deep-set intermediate technology
- Deep tubewells

Shallow tubewell technology:

In shallow tubewell technology, hand pumps are operated in a suction mode. A suction pump draws water from a shallow depth by creating a vacuum in the suction pipe. The suction handpumps can practically extract water from up to a depth of 7.5 m static water level. This category handpumps include:

- No. 6 handpump tubewell
- Rower pump tubewell
- Disco pump tubewell

Cycle 10
Day C

No. 6 Handpump tubewell: In Bangladesh the most common and popular technology used for abstraction of groundwater is the No. 6 handpump tubewell. The name of the tubewell is based on its barrel diameter in inches. About 3-4 million public and private No.6 handpump tubewells are already in use throughout the country and a very high percentage of these tubewells is in operational condition at any time. Before the lowering of the water table was encountered, the No.6 handpump was considered to be the only low-cost option for potable drinking water supply in rural, peri-urban and urban areas where piped water supply system were not introduced. A typical No.6 handpump tubewell is shown in Figure 21.2. There are also No.4 and No.2 handpump tubewells but their use is limited to private tubewells within family premises.

Cycle 10
Day C

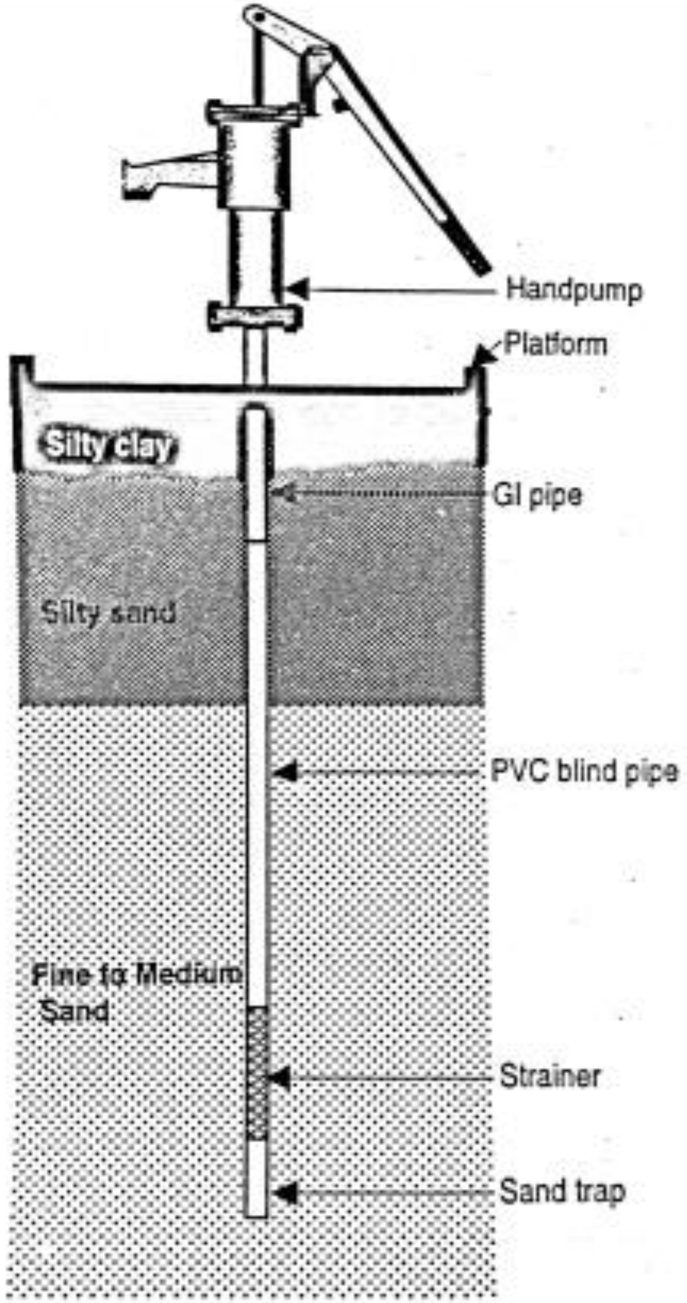


Figure 21.2 A No.6 handpump tubewell

- The average discharge of the pump is 30-40 litres/minute.
- A No. 6 handpump, if installed properly, can serve for 15 to 20 years.
- The general components of a No. 6 handpump tubewell are

➢ Handpump
➢ Blind pipe (rising pipe)
➢ Strainer(screen)
➢ Sand trap

Cycle 10
Day C

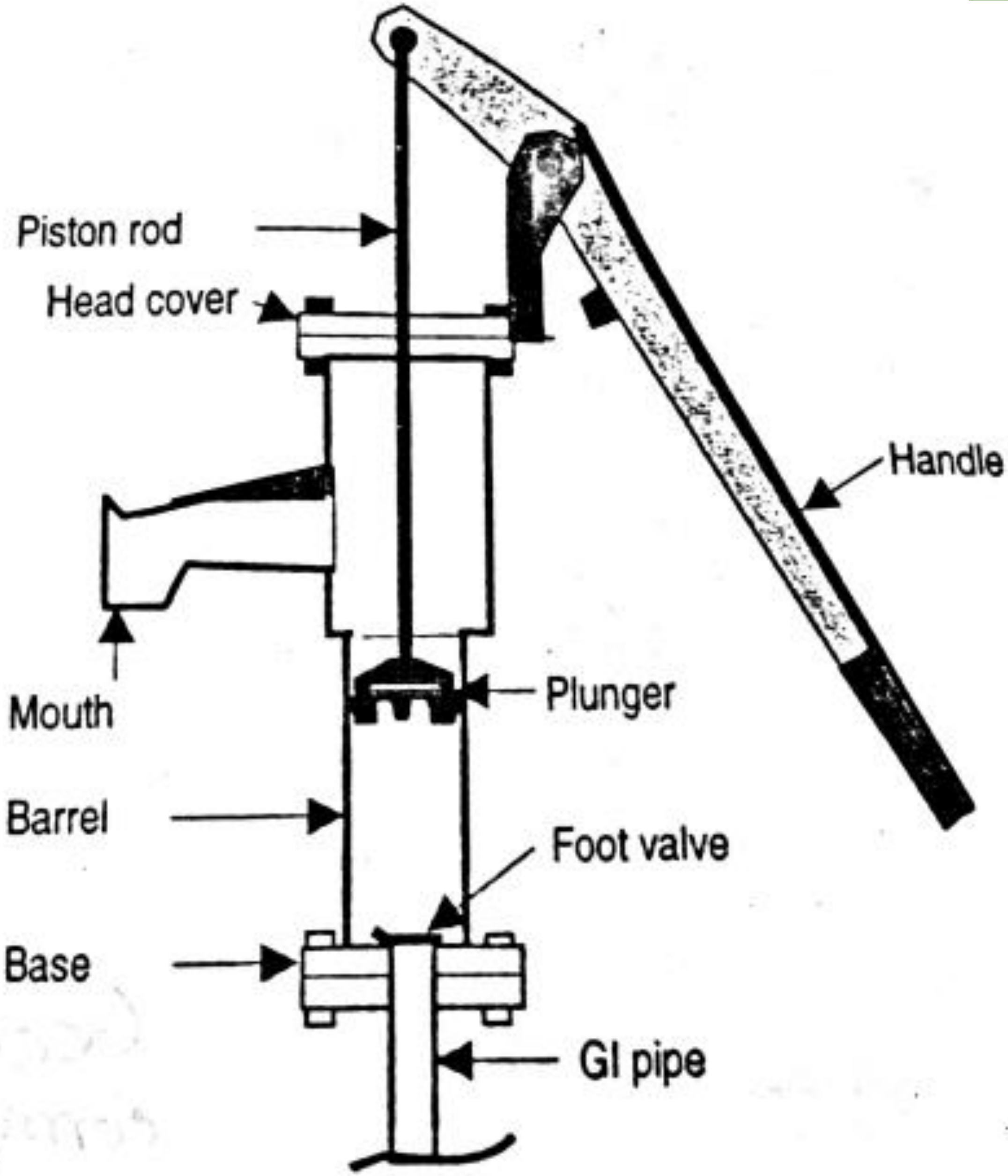


Figure 21.3 : The No.6 Handpump

Cycle 10
Day C

Rower pump tubewell:

- The rower pump is a manually operated reciprocating pump with a 54 mm diameter PVC pipe as the pump cylinder
- The operation of rower pump is rowing a boat, hence the name rower pump
- In the high water table areas rower pump is used for irrigation purposes and occasionally for domestic water supply

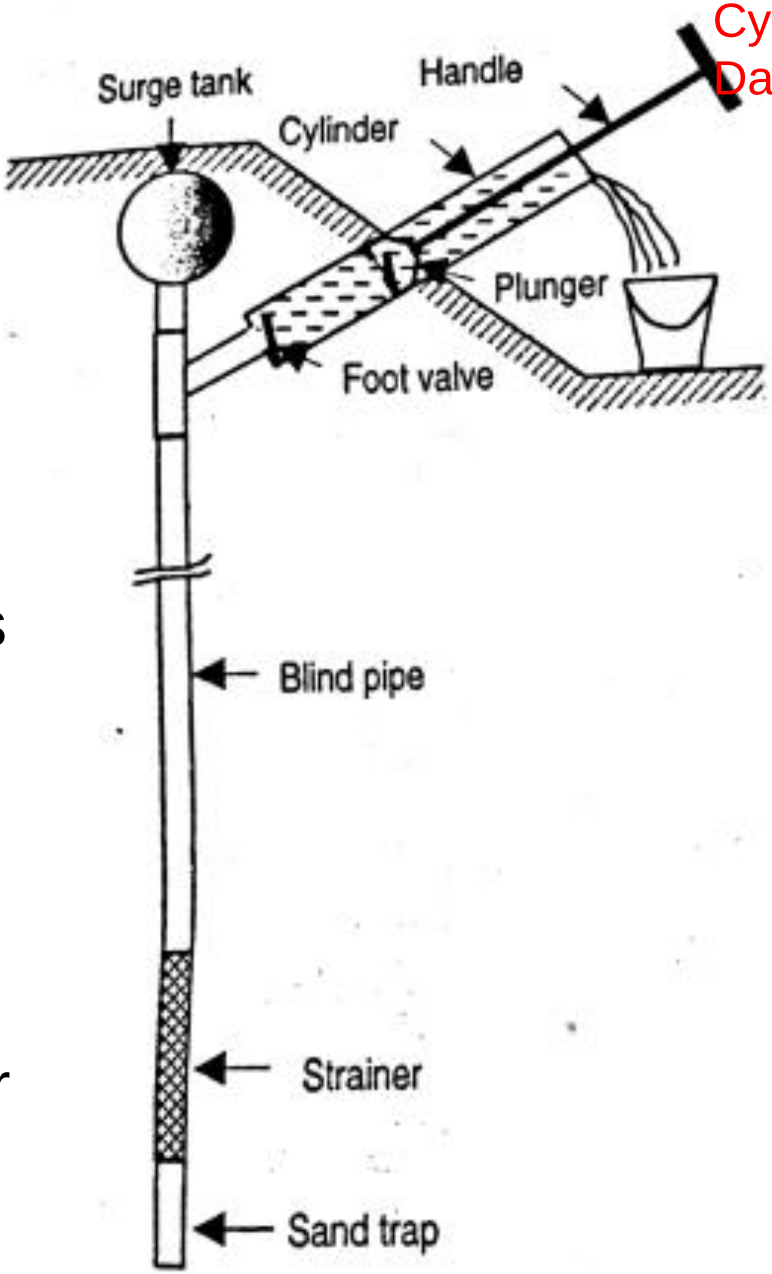


Figure 21.4 A rower pump

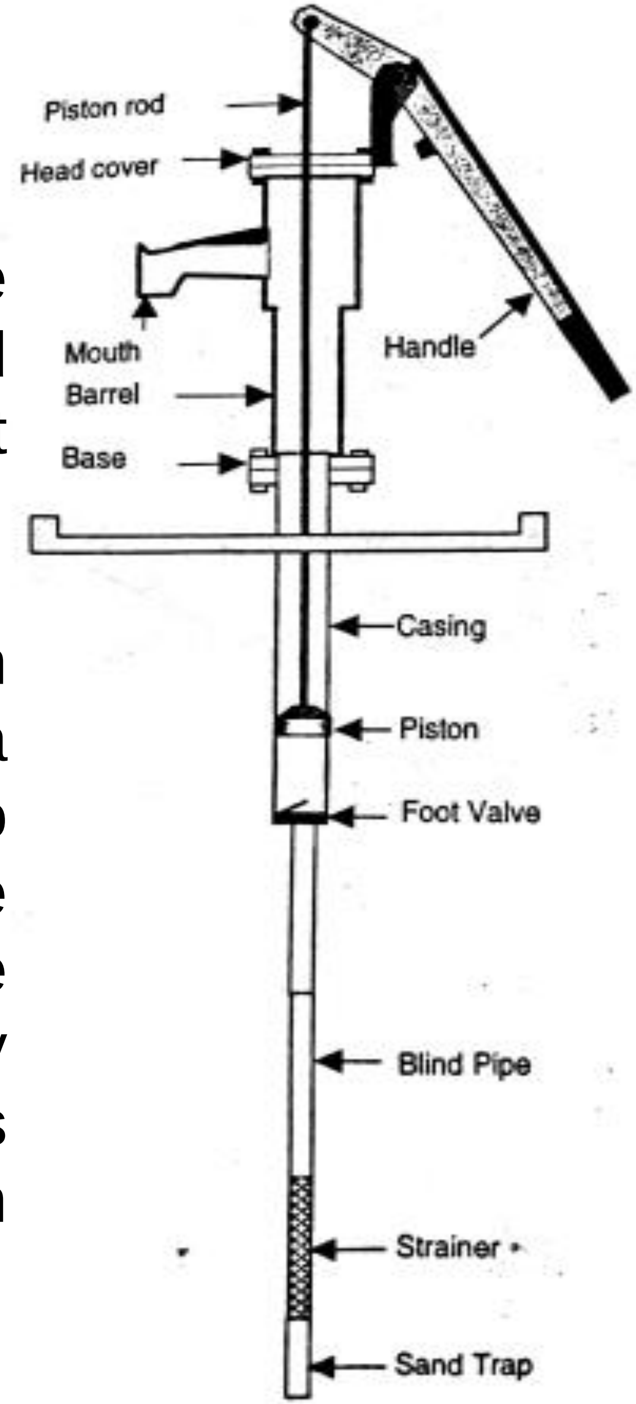
- Rower pumps can lift water upto a maximum suction lift of 8.5 m.
- The discharge capacity is 0.8 lps
- The pump is not very suitable for domestic water supply due to poor sanitary protection
- The life time of rower pump is 3- 5 years

Cycle 10
Day C

Disco pump tubewell:

- There are some areas where the water level goes beyond the suction limit for a short duration.

- The lowest level reached such a depth that if the piston of a no. 6 pump is extended to about 3 m, the pump could be kept operational for the whole year. To meet the water supply requirements of these areas the disco pump has been developed locally



- In Gazipur, it is known as half cylinder pump

- The 75 mm diameter GI pipe is used as the casing upto 3 m below the ground surface.

- The discharge from the pump is equal to that of No. 6 hand pump but requires comparatively more force to raise the water

- The limitation of the disco pump is that it can only be used where the water level will remain within 10 m from the ground surface

Deep-set intermediate technology:

- Shallow tubewells operated under suction mode are not able to withdraw water in low water table areas.
- Water can be abstracted from a depth beyond the suction limit using the intermediate technology.
- Deep set handpumps can abstract water from a high as 30 m from the static water level.

Hand pumps in this category include:

- Tara handpump tubewell
- Moon handpump tubewell
- Bangla handpump tubewell
- Mark –II handpump tubewell
- Other locally produced, improvised deep-set pumps

Cycle 10
Day C

Tarahandpump tubewell:

- The no.6 handpump tubewell is inoperable in dry season, to overcome this problem the Tara handpump has been developed in Bangladesh by UNICEF, and the UNDP World Bank program, to tap water from upto 15 m below the ground surface.

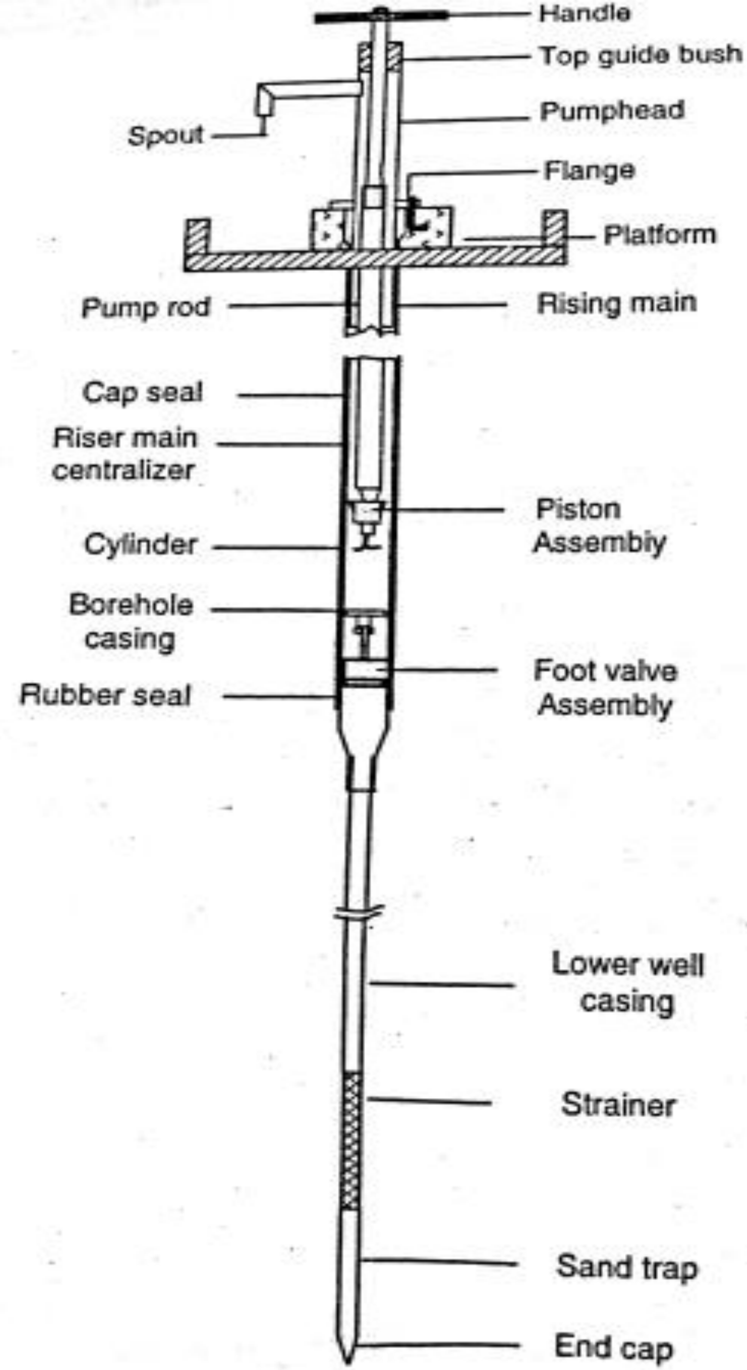


Figure 21.6 Tara handpump tubewell

- A properly installed and maintained Tara handpump should work about 10 years without major problems

- Because of the direct action, the force is to be applied directly by the operator without having any mechanical advantage

- Tara handpump provides moderate output max 4 m^3/day for 7m lift and very low output max 1.5 m^3/day for 12 m lift

Tara –II hand pump:

- In some areas of the country; the water level in dry season goes down below 15m which is the lowest normal functioning range of Tara handpump.

- In order to withdraw water a deeper aquifer, the standard tara pump is modified

- The piston assembly is set at 30 m and all other aspects of the tara hand pump is remain the same

Cycle 10
Day C

Moon hand pump tubewell:

- The moon hand pump tubewell is the modified version of Tara hand pump
- The direct action tara hand pump tubewell is found to be uncomfortable to the usurers particularly to woman

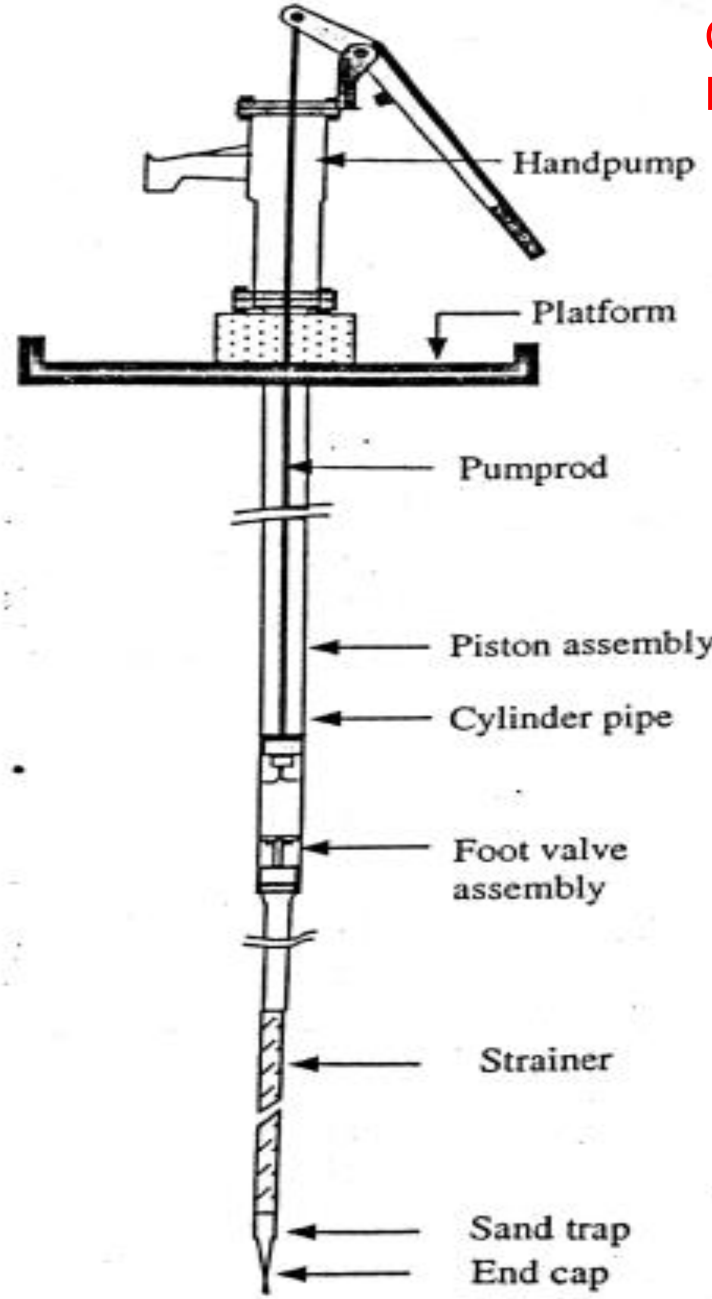


Figure 21.7 Moon handpump tubewell

- Considering this difficulties, the head of the tara hand pump tubewell has been replaced by that of a no. 6 hand pump to get the advantage of the lever action and PVC pump rod has been replaced by steel rod in the moon pump

- A good number of moon hand pumps have been installed in Naogaon, Chapai Nawabganj and Manikganj under the Dutch assisted 18 district town project

- The maximum discharge of a moon hand pump is 0.6 lps.

- It is suitable for lifting water upto 25 m

Bangla handpump tubewell:

- This is a modified version of the moon hand pump
- The difference with the moon pump is that the upper well casing is of smaller diameter PVC pipe.
- The lifting capacity of the pump is limited to 15 m
- A performance evaluation of the bangla pump was conducted for a short duration of two years and it was found to be functioning satisfactiory.

Cycle 10
Day C

Mark –II hand pump:

- The mark –II handpump tubewell, widely known as the Indian Mark –II is the most popular tubewell in India and widely used in many other countries in the world
- It has a deep-set pump capable of lifting water from a depth of over 30 m
- Although the Mark-II handpump tubewell has not yet been installed and familiarized in Bangladesh, it may serve the demand of a robust hand tubewell in low water table areas

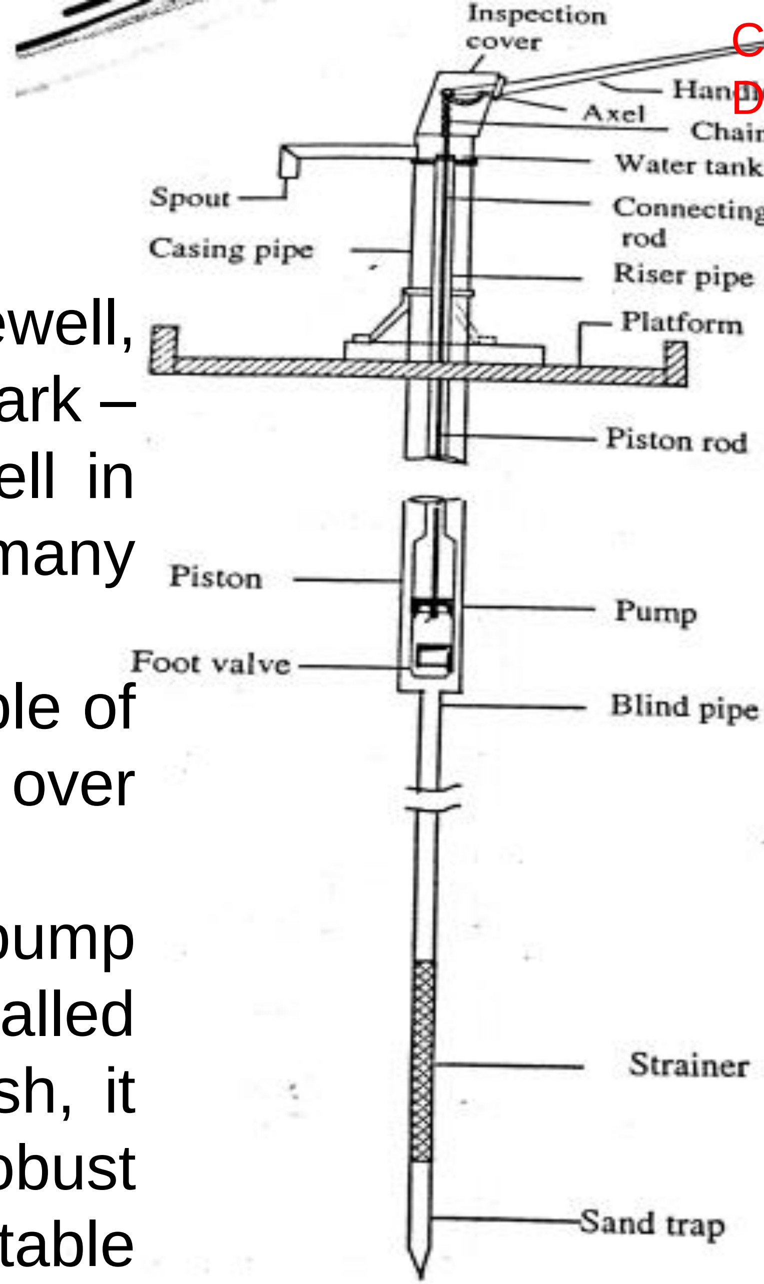


Figure 21.8 Mark II handpump tubewell

Cycle 11
Day A

21.5 ALTERNATIVE WATER SUPPLY TECHNOLOGIES

Technological options

The type of handpump technology suitable for a particular area depends on the groundwater level, water quality and hydrogeological conditions. There are some areas like the coastal belt in the southern part of Bangladesh, where the conventional shallow and deep tubewells technologies are not successful due to the high salinity. Alternative water supply options are needed for those areas. The important alternative water supply technologies include:

- Shallow shrouded tubewell (SST)
- Very shallow shrouded tubewell (VSST)
- Pond sand filter (PSF)
- Household filters
- Solar desalination
- Rainwater harvesting.

Cycle 11
Day A

Shallow shrouded tubewell (SST)

In the high salinity coastal areas it has been found that fresh water is available in shallow aquifers composed of fine sand at 15 to 20 m depth. This is due to accumulation of rainwater in the topmost aquifer. However, the particle size of soil and the depth of the aquifer are not suitable for installing a normal tubewell. To get water through these very fine-grained aquifers, an artificial sand packing is required around the screen of the tubewell. This artificial sand packing, called shrouding, increases the yield of the tubewell and prevents entry of fine sand into the screen. The 15 to 20 m deep tubewells, installed with this technology are called shallow shrouded tubewells (SST). The SSTs are fitted with No.6 handpumps operating under suction mode. A typical SST with components is shown in Figure 21.10.

Cycle 11
Day A

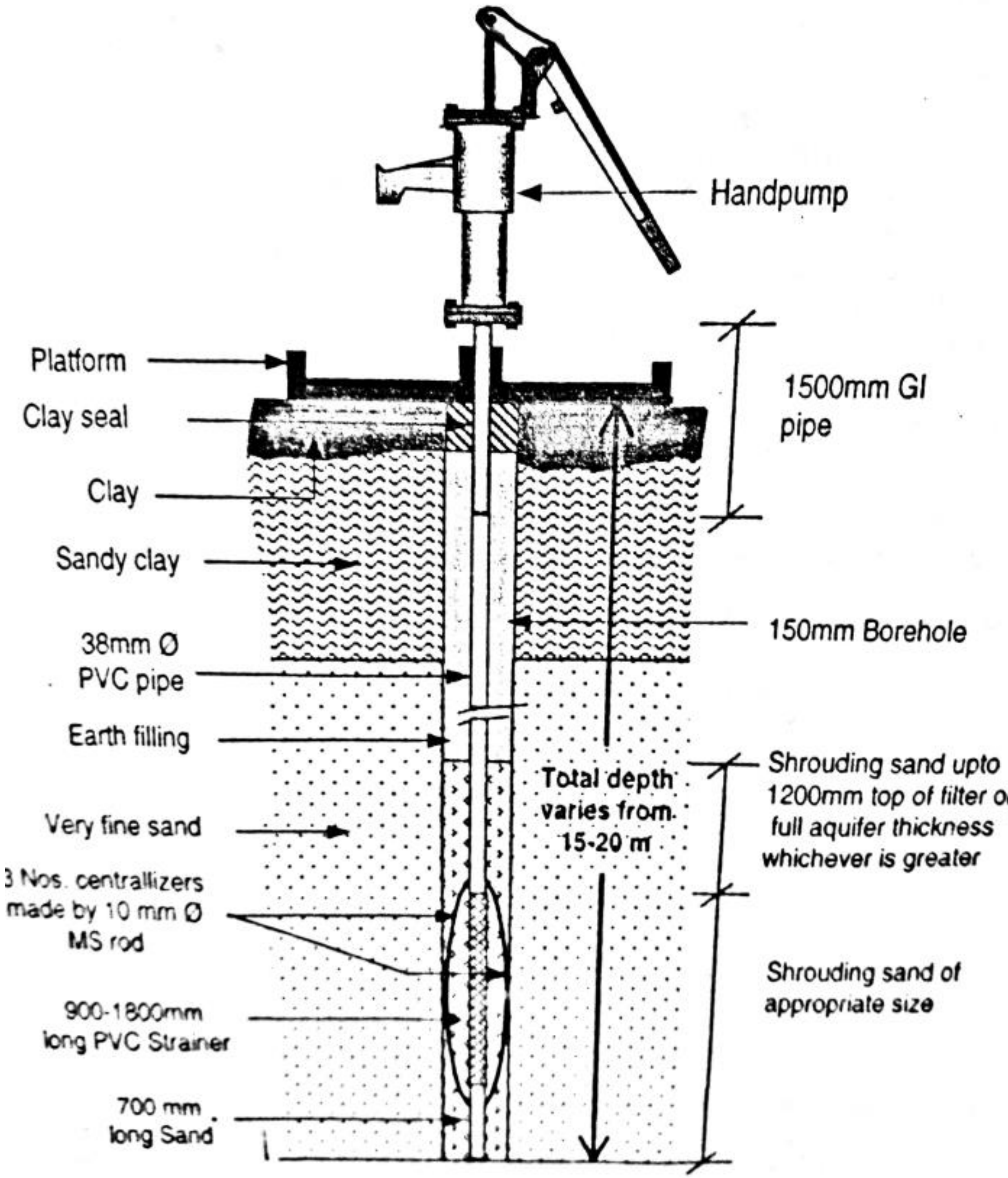


Figure 21.10 Shallow and very shallow shrouded tubewell

Very shallow shrouded tubewell (VSST):

- This is a low-cost handpump tubewell about 8 m in depth with a 2 m strainer shrouded with coarse sand.

- It is designed to collect water from very shallow aquifers formed by displacement of saline water by a continuous flow of accumulated fresh water.

- A VSST is a convenient method for withdrawal of fresh water in limited quantities.

Very shallow shrouded tubewell (VSST):

- This system is considered suitable for drinking water supply for settlements where water demand is low.
- A VSST is not different from an SST expect in the depth of well

Cycle 11
Day A

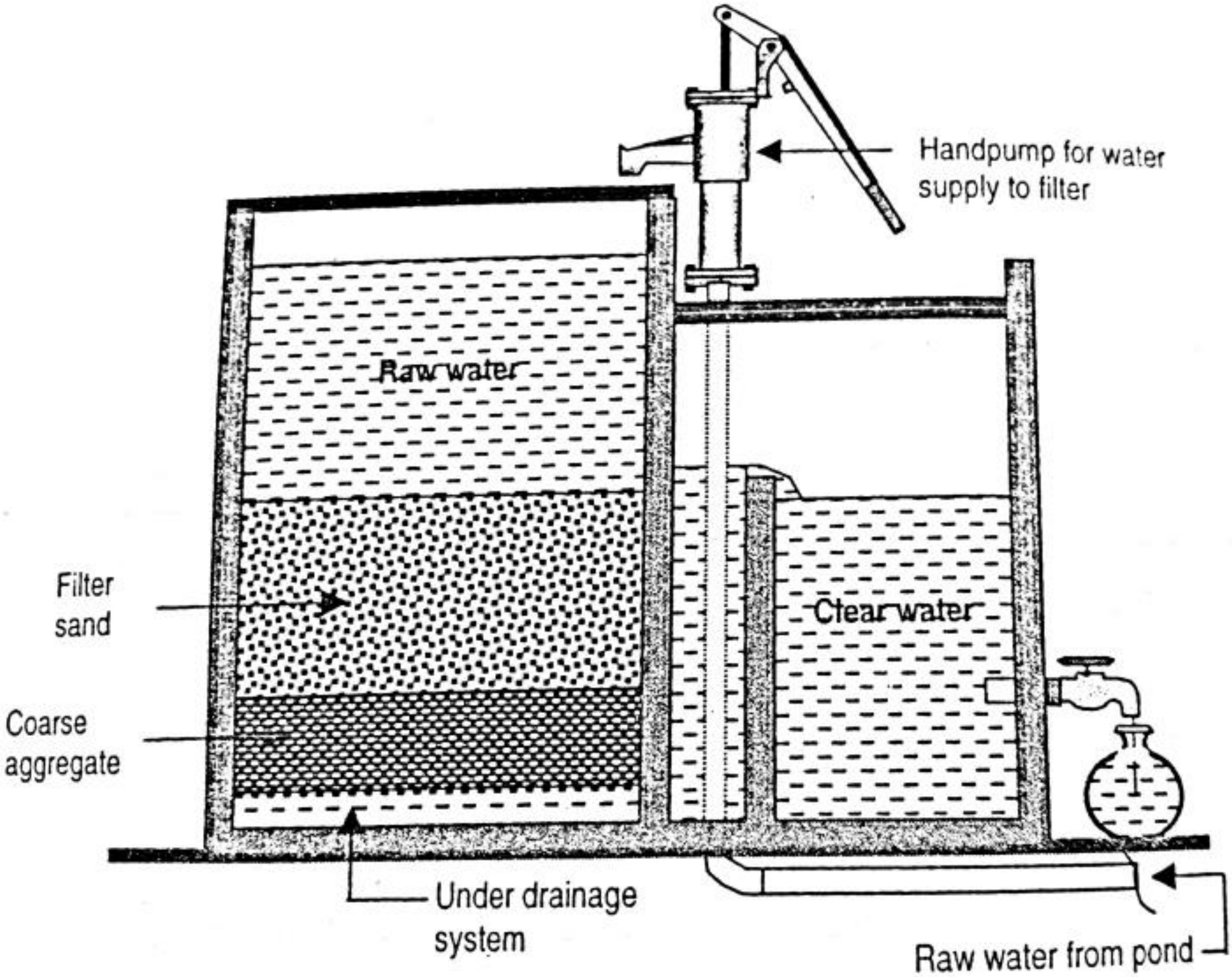


Figure 21.11 Pond sand filter

Pond Sand Filter (PSF):

- An alternative and popular option of potable water supply in coastal areas in the pond sand filter (PSF).

- The water from the pond is pumped by a manually operated hand tubewell to feed the filter bed, which is raised from the ground and the treated water is collected through taps

- On average the operating period of a PSF between cleaning is usually 2 months

Cycle 11
Day A

Pond Sand Filter (PSF):

- The problems encountered are low discharge and difficulties in washing the filter bed
- It is very low cost technology with very high efficiency in turbidity and bacterial removal
- Though it has a very high bacterial removal efficiency, it may not remove 100% of the pathogens from heavily contaminated surface water.

Household filters

Surface water containing impurities can be clarified by a pitcher filter unit or a small sand filter at the household level. It is an old method of water purification, once widely used in rural areas of Bangladesh. These processes of water treatment at household level have been phased out with the introduction of tubewells for village water supply. Pitcher filters are constructed by stacking a number pitchers (Kalshis), one above the other, containing different filter media as shown in Figure 21.12. Raw water is poured in the top Kalshi and filtered water is collected from the bottom one. In this process, water is mainly clarified by the mechanical straining and adsorption depending of the type of filter media used.

Cycle 11
Day A

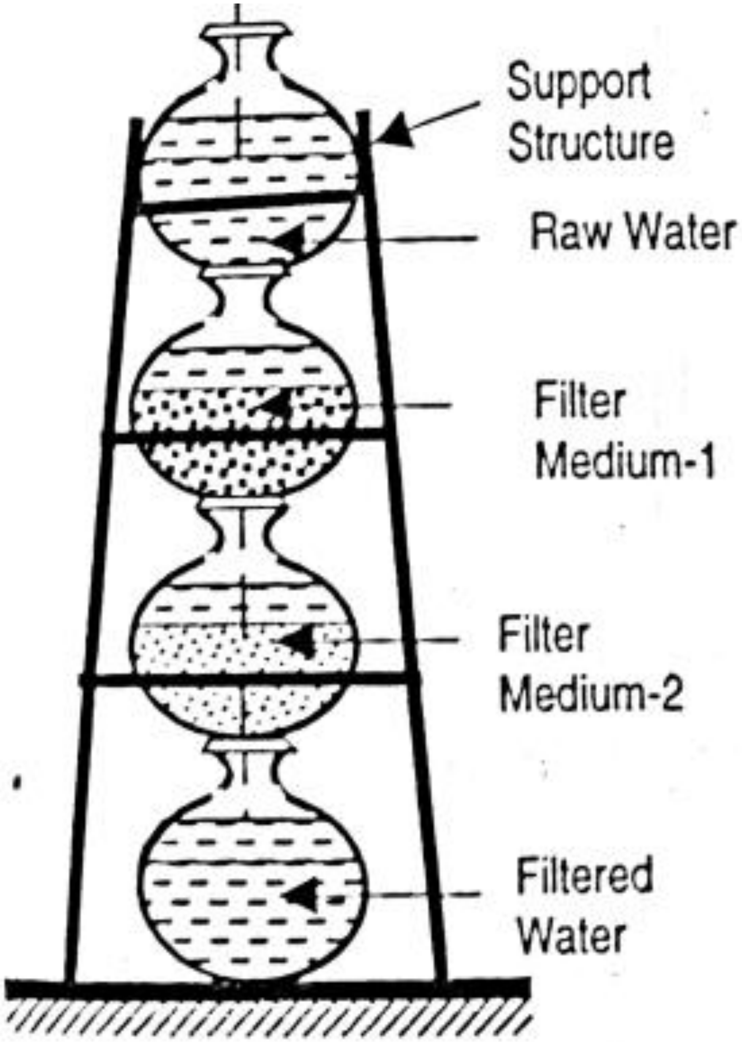


Figure 21.12 Pitcher (Kalshi) filter

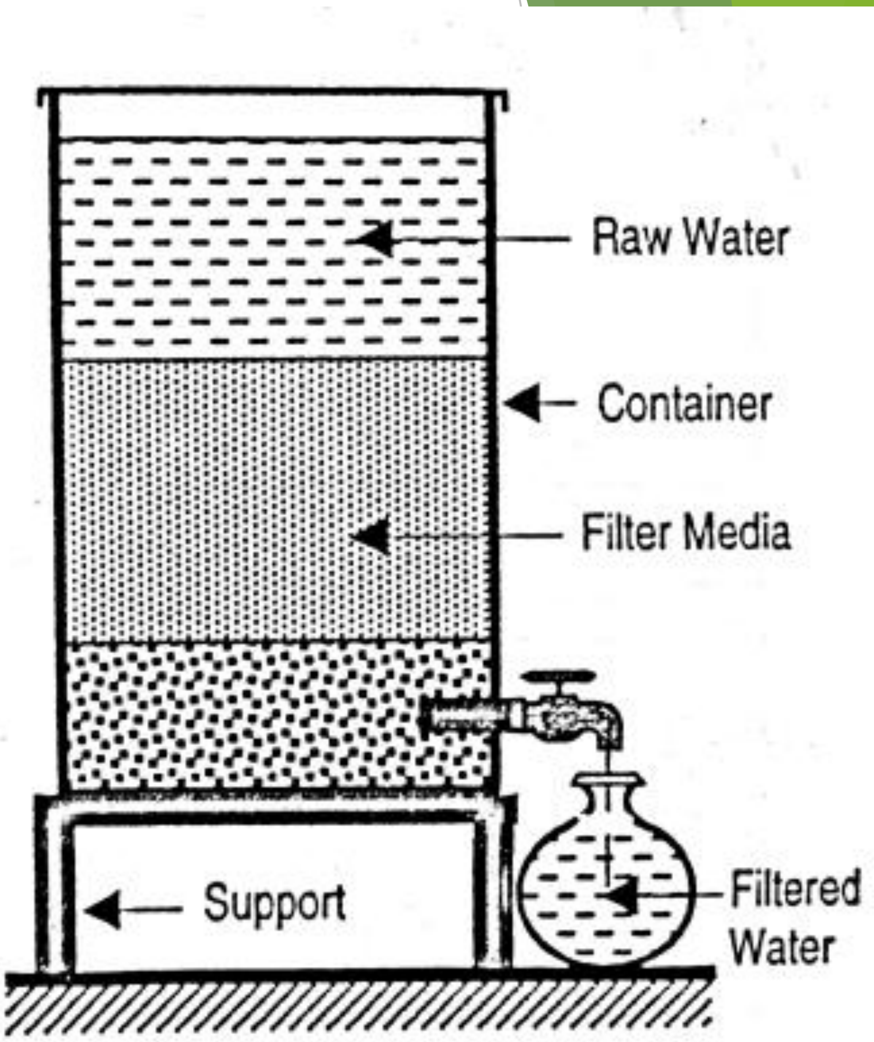


Figure 21.13 Small household filter

Cycle 11
Day A

Small household Filters can be constructed by stacking about 300-450 mm thick well graded sand on a 150-250 mm thick coarse aggregate in a cylindrical container.

The important characteristics of household filters are

- Suitable for surface water treatment
- Remove turbidity, colour and micro-organisms
- Complete removal of pathogenic micro-organisms is not guaranteed
- Not suitable for high turbid water
- Difficulty in cleaning and keeping the system operational

Cycle 11
Day A

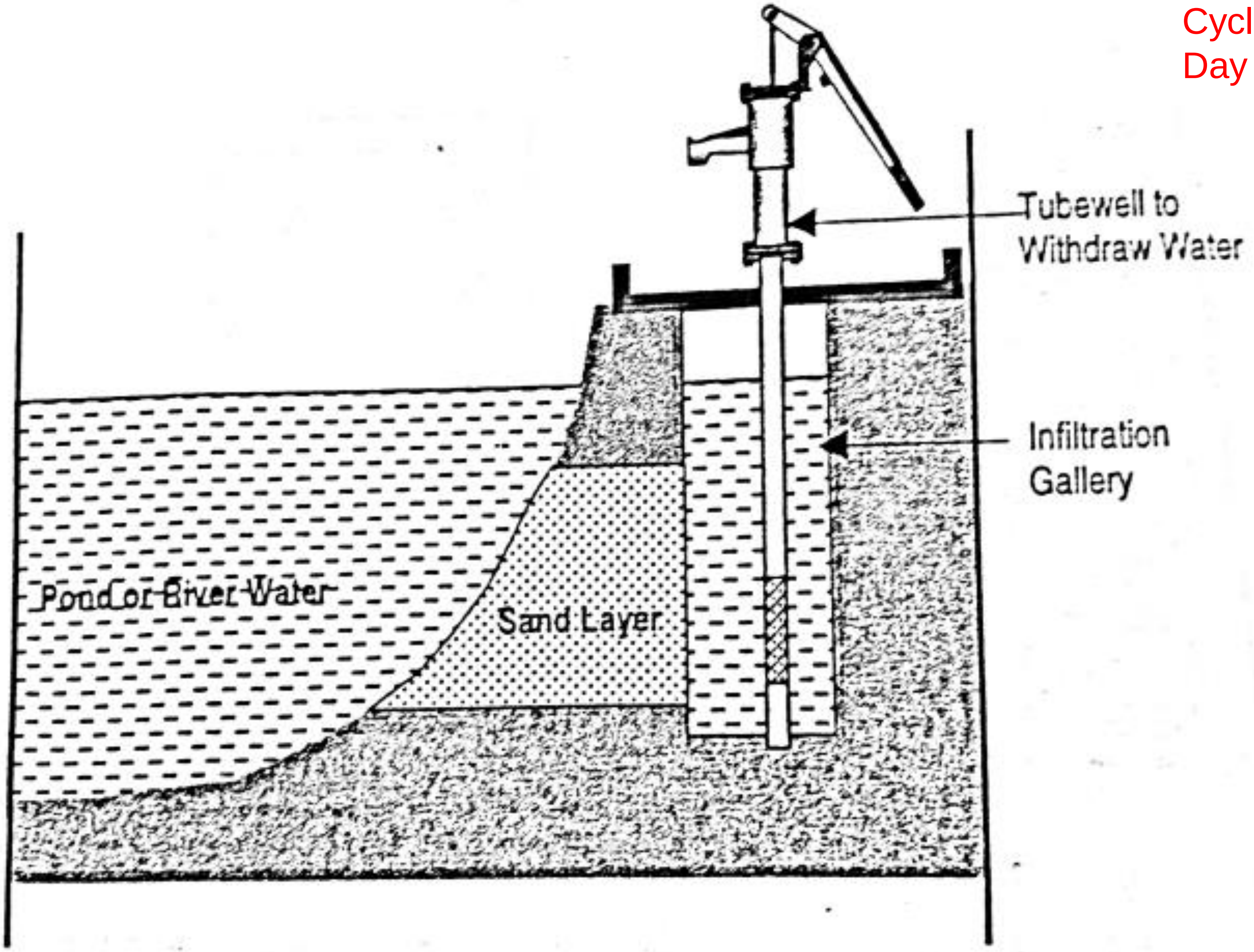


Figure 21.14 Infiltration well/gallery constructed along pond/river

Infiltration well/gallery:

- Installation of infiltration wells/galleries along banks of rivers and ponds provides clean water for domestic purposes

- The main problem encountered with infiltration galleries is the sanitary protection of water. Water without proper sanitary protection becomes soon contaminated.

Rainwater harvesting:

- The average annual rainfall in the costal and hilly regions is more than 3000 mm, against an average rainfall of about 2400 mm in Bangladesh

- The collection and storage of rainwater is an alternate option of water supply in these areas

- Rainwater harvesting is a potential water supply option in the acute arsenic affected areas in Bangladesh

Cycle 11
Day A

The two main constrains in developments of a completely rainwater based water supply system:

- Availability of suitable catchment area
- The need for larger storage tank

Table 21.1: Advantages and disadvantages of rainwater collection system

Advantages	Disadvantages
• The quality of rainwater is comparatively good • The system is independent and therefore suitable for scattered settlements • Local materials and craftsmanship can be used in construction of rainwater system • No energy costs are incurred in running the system • Ease in maintenance by the owner/user • The system can be located very close to the consumption points	• The initial cost may prevent a family from installing a rainwater harvesting system • The water availability is limited by the rainfall intensity and available roof area • Mineral-free rainwater has a flat taste, which may not be liked by many • Mineral-free water may cause nutrition deficiencies in people who are on mineral deficient diets • The poorer segment of the population may not have a roof suitable for rainwater harvesting

The available rainwater can be estimated by the equation:

Q= CIA

Where, Q = total quantity of rainwater available in $m^3/year$

C = coefficient of available runoff

I = rainfall intensity

A = catchment area in m^2

The minimum catchment area A required for the collection of rainwater for N number of people supplied with q litres per capita per day of water can be derived from this equation

$$A = 0.365\ q\ N / (CI)$$

About 25 % of the rainwater may be assumed to be lost by evaporation and by washing the catchment area. The catchment area is usually washed using first rain, which produces inferior quality rainwater. For an average annual rainfall of 2.4 m/yr, and a coefficient of runoff 0f 0.75. equation can be written in the following form

$$A = 0.203\ q\ N$$

The minimum volume of the storage rainwater tank V, required for rainwater can be computed by the equation

$$V = 0.365\ f\ q\ N$$

However, if only drinking and cooking water is harvested, the sizes of the storage tank and the catchment area would be smaller and within an affordable range for a family. Substituting f =0,4 in equation the minimum volume of storage tank required for rainwater becomes

$$V = 0.146\ q\ N$$

Design, construction and maintenance of tubewells:

The design of tubewells mainly involves the selection of length, diameter, and slot opening of the screen and design of the shroud materials, on the basis of the available aquifer characteristics.

The size corresponding to 10% finer is called the effective size, which governs the permeability of the aquifer. The ratio of the size corresponding to 60% finer and the effective size is called the uniformity coefficient, which defines the uniformity of the grain size. Thus uniformity coefficient can be expressed as:

$$U = D_{60}/D_{10} \tag{21.1}$$

Where U = uniformity coefficient, D_{60} = grain size corresponding 60% finer and D_{10} = grain size corresponding 10% finer, i.e. effective size. The aquifer materials having twice the effective size and a similar uniformity coefficient have four times higher permeability.

The best aquifer for screening is determined by comparison of the grain size analysis curve of the materials along the depth of bore hole. The slot of the screen should be such that it will retain only 40 to 60% aquifer sand. The grain size corresponding to 60% or 50% finer as shown in Figure 21.9. will determine the slot size of the strainer. If the slot size is required to be increased, the tubewell has to be shrouded with coarse-grained materials.

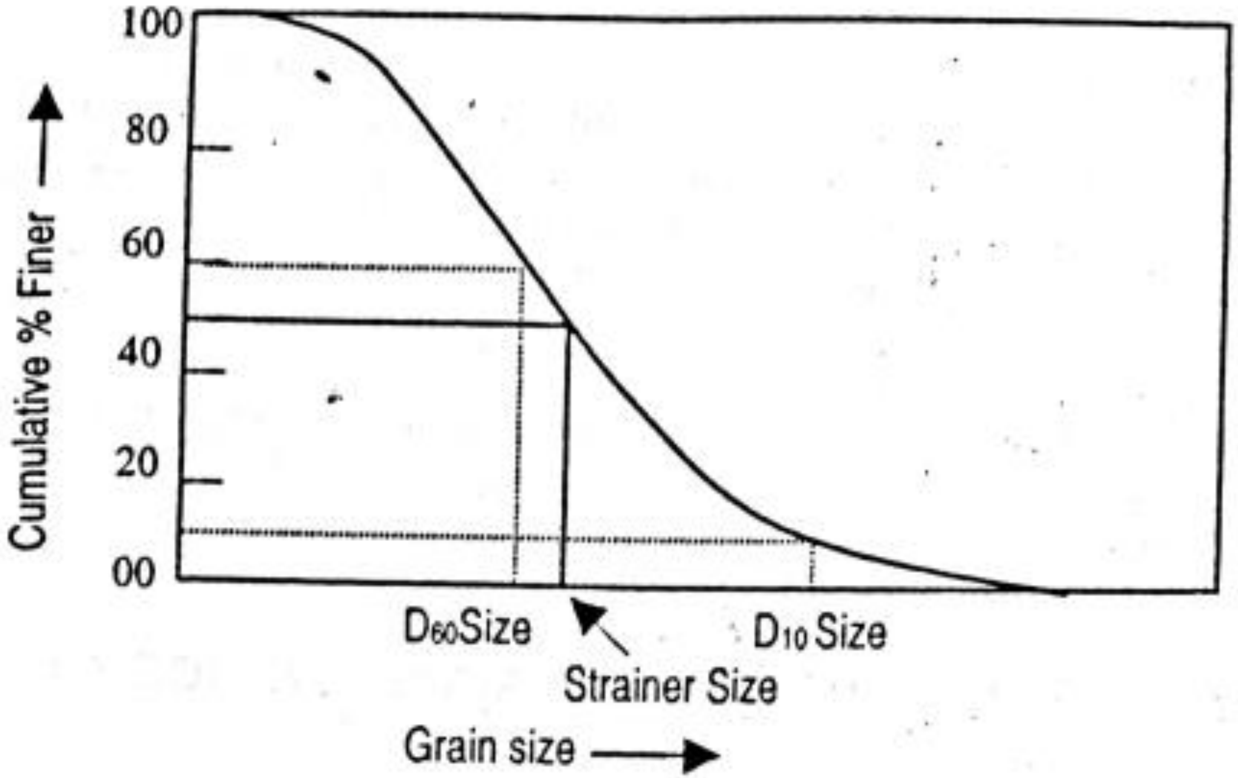


Figure 21.9 Grain size distribution curve

The length and diameter of the strainer will be determined on the basis of permissible entrance velocity, percent opening of the strainer and thickness of the aquifer. The length and diameter of the strainer required may be calculated from the expression:

$$Q = \pi D L (0.01p) v_e \tag{21.2}$$

Where Q = design discharge of the tubewell, D = diameter of the screen, L = length of the screen, p = percent opening of the screen, generally dependent on slot size and available from specification, v_e = permissible entrance velocity.

The entrance velocity while the water is entering into the screen should not exceed 0.03 m/s. An entrance velocity less than 0.03 m/s results in low frictional head loss in the screen, low encrustation and corrosion. The entrance velocity should be maintained in the range between 0.03 and 0.010 m/s.

The length of the strainer should also satisfy the requirement corresponding the thickness of the aquifer. Only about 70 to 80% of a homogeneous artesian aquifer can be screened provided that the pumping water level does not fall below the top of the aquifer. In the case of a water table aquifer the bottom one-third to half of the aquifer can be screened.

If the particle size of the aquifer material is very fine or the depth of the aquifer is insufficient to provide a suitable screen, the tubewell may be shrouded. The tubewell may also be shrouded to obtain higher discharge. The shrouding materials retain the aquifer sand and prevent it from entering into the large slotted strainer. Theoretically, the maximum pore size of spherical granular material is one-sixth of its size. The shrouding materials selected must be 4 to 6 times larger than the size of the aquifer sand corresponding to 70% retained or 30% finer, and must have a uniformity coefficient less that 2.5. The procedure for the design of the shrouding materials is as follows:

- Draw particle size distribution curves of all the visible different strata of the aquifer sand where strainer of the tubewell will be located. Select the particle size distribution curve of the finest sand for the design of the shrouding material.
- Multiply the 70% retained size of the sand by a factor 4 if the sand is fine and uniform or 6 if the sand is coarser or non-uniform. Locate the result of multiplication as 70% retained size of the shrouding material. This is the first point of the curve that represents the artificial shrouding material.
- Draw a smooth curve representing a shrouding material with a uniformity coefficient of 2.5 or less through this 70% retention point of the material. It is to be done by trial and error.

- Prepare specifications for the shrouding material by selecting four or five standard sieve sizes that cover the spread of the curve and then set down the permissible range for percent retained on each of the selected sieves. The permissible range may be 8% above and below the percent retained at any point on the curve.
- Finally, select the size of the slot of the screen that will retain 90% or more of the shrouding material.

A very thin layer of shrouding material less than 25 mm thick can retain the aquifer sand, but it is impractical to place a thin gravel pack in a well and expect the material to completely cover the surrounding of the screen. A minimum thickness of 75 mm is considered practical for installation in the field to ensure an envelope of shrouding material around the screen along the entire length.

In case of shallow and very shallow shrouded tubewells a shrouding sand having D_{10} size or 90% retention size of 0.008 inches. Fineness modulus of 1.6 and uniformity coefficient 0f 1.5 is performing well in case of No. 8 (0.008 inch) slot screen in Bangladesh

Sinking of handpump tubewells:

- Sludger method
- Rotary drilling method

Design of distribution system:

The Hazen Williams equation can conveniently be used for a hydraulic design of the branched or looped network. The equation can be written in the form of:

$$\boldsymbol{Q = 3.7 * 10^{-6}\, CD^{2.63} \left(\frac{H}{L}\right)^{0.54}} \quad \text{…………………}(1)$$

Where, Q= Flow lps,

C= roughness coefficient (100-140 for rough to smooth pipes)

D= diameter, in m

H= head loss in m

L= length of pipe in m

For a definite value of C the equation can be written as

$$\frac{H}{L} = \mathbf{1.39} \times \frac{10^6 Q^{1.85}}{D^{4.87}} \text{ ....(2)}$$

For C=130

$$\frac{H}{L} = \mathbf{1.59} \times \frac{10^6 Q^{1.85}}{D^{4.87}} \text{.............(3)}$$

For C=120

The process involved in the design is to make a pipe layout, assume the pipe size then work out the terminal pressure head which could be made available the end of each pipe section when discharging the peak flow. The available pressure heads are checked to see if they correspond to permissible residual pressure heads. If not the pipe size is changed and the system is reinvestigated until satisfactory conditions are obtained.

Branched network:

The following design procedure may be adopted for branched network:

1) Collect / prepare a map of the area to be served with roads, streets and other features and

make a layout of mains, sub-mains and branches including location of valve and other appurtenances/

2)Estimate the peak flow at different points and determine the quantity flowing through each section of the pipe. Peak flow = average daily flow x peck factor

3)Assume pipe sizes of all the pipes in the network(to calculate the approximate pipe size the velocity may be assumed to be round 1m/s.

4) Calculate the frictional head loss per unit length of pipe and then multiply by pipe length of pipe to find the total head loss

5) Determine the terminal pressure head taking the change in the elevation of the pipe into account.

6) In case of a difference computed terminal pressure and the permissible pressure head revise the pipe size.

Looped Network:

In any looped pipe network two condition must be satisfied.

1) The flow entering in a junction must equal the flow leaving it.
2) The algebraic sum of the pressure drop(head loss) around any closed group must be zero.

Pipe network problems in water distribution systems are usually solved by methods of successive approximation

The equation is $\boldsymbol{H = kQ^{x}}$……………….(4)
In which k is the constant developing on the length, diameter, and roughness of the pipe as well as the fluid property, x is the component equal to 1.85 for Hazen Williams' equation and 2 for the Manning equation

Hardy Cross developed a method of successive approximation in which circuits are balanced, distribution of flow is determined and the above two conditions of flow is satisfied . The solutions for pipe network problems suggested by Hardy Cross requires that the flow in each pipe be assumed so that the principle of continuity is satisfied at each junction. A correction Δ, to the assumed flow is computed successively for each pipe loop in the network until the correction is reduced to an acceptable value. If Qa is the assumed flow and Q is the actual flow, then the correction is

$Q - Q_a$ i.e

$Q = Q_a + \Delta$……………………(5)

Inserting the value of Q from equation (5) and applying the condition that the head loss around any closed loop is zero the head loss equation (4) gives

$0 = \sum k(Q_a + \Delta)^x$………………..(6)

Expanding the equation

$$\sum kQ_a{}^{x} + \mathbf{x}\sum kQ_a^{x-1} + \frac{x-1}{2}\sum k\Delta^2 Q_a^{x-2} + \ldots\ldots + = 0 \ldots\ldots\ldots (7)$$

If Δ is small compared to Q the third and all successive and all successive terms of the equation (7) may be neglected, hence

$$\sum kQ_a{}^{x} + x\sum kQ_a^{x-1} = 0 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (8)$$

Solving equation (8) for Δ

$$\Delta = -\sum \mathrm{k}Q_a{}^{x} / x\sum kQ_a^{x-1} \text{ or}$$

$$\Delta = -\frac{\sum H}{\frac{x\sum H}{Q_a}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (9)$$

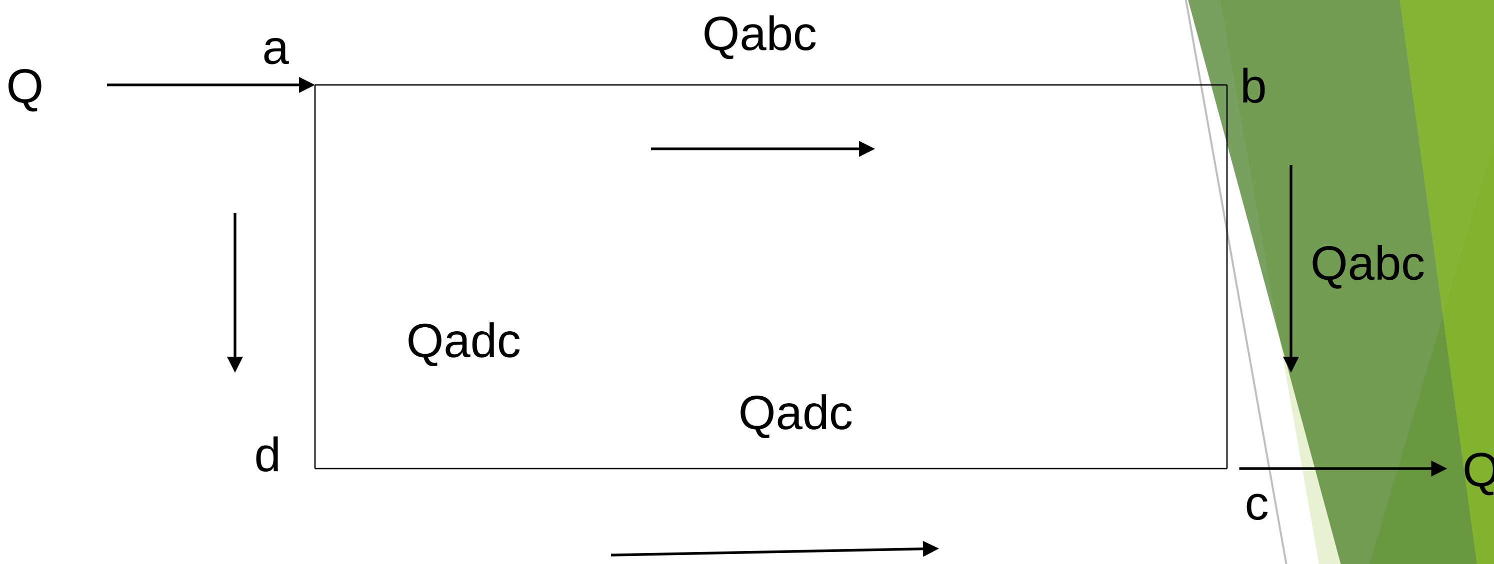


Fig: A looped network for flow analysis

Consider the simple loop shown in figure . The arrowheads showing the assumed direction of flow and head loss , the correction Δ can be calculated by

$$\boldsymbol{\Delta = -[(H_{ab} + H_{bc}) - (H_{ad} + H_{dc})]/x\left[\frac{(H_{ab}+H_{bc})}{Q_{abc}} + \frac{(H_{ad}+H_{dc})}{Q_{adc}}\right]}$$

The procedure for the Hardy Cross Method of pipe network analysis is given below.

1) Carefully examine the network and assume reasonable rates of flow in each pipe such that inflow equals outflow in each junction
2) In each loop, determine the head loss, H and H/Q for all pipes
3) With due attention to sign compute the total head loss around each circuit
4) Compute ∑H/Q for the same circuit without giving any consideration to sign
5) The correction Δ is computed for each loop by the equation (9). The minus sign may be disregarded if the correction so obtained is made by inspection.

6) Apply the correction to each pipe in each loop. When the sign of Δ is negative decrease the clockwise flow and increase the counterclockwise flow. When the sign is positive there will be vice versa. Pipes that are common to two loops require double correction

7) With adjusted flows repeat the procedure for the second approximation. The procedure is continued until the desired accuracy is attained.

Example 1: Calculate the flow in each of the pipes in the following pipe network.

Solution: First trial with assumed flow Q_o

Step 1

Pipe	Length m	Dia mm	Q_o lps	Ho/L mm	Ho m	Ho/Qo	Δ lps	Q_1 lps
AB	800	200	+25	0.0038	+3.04	0.1216	0.54	25.54
BC	400	150	+13	0.0046	+1.84	0.142	0.54	13.54
AD	400	150	-20	0.0102	-4.08	0.204	0.54	-19.46
DC	800	150	-8	0.0018	-1.44	0.18	0.54	-7.46
					Sum= -0.64	Sum= 0.647		

$$\mathbf{\Delta = } -\frac{\sum H}{\frac{x\sum H}{Q_a}} = 0.64/(1.85*0.647) = 0.54$$

Step 2:

Pipe	Length m	Dia mm	Q_o lps	Ho/L mm	Ho m	Ho/Qo	Δ lps	Q_1 lps
AB	800	200	25.54	0.0039	+3.12	0.122	0.12	25.54
BC	400	150	13.54	0.0049	+1.96	0.145	0.12	13.54
AD	400	150	-19.46	0.0097	-3.88	0.2	0.12	-19.46
DC	800	150	-7.46	0.0017	-1.36	0.18	0.12	-7.46
					Sum= -0.14	Sum= 0.649		

$$\boldsymbol{\Delta} = -\frac{\sum H}{\frac{x\sum H}{Q_a}} = 0.14/(1.85*0.649) = 0.12$$

(ii) to collect information for location of an industry, its future expansion, availability of labour, marketing and distribution of the product, etc.;

(iii) to provide data to the transportation industry;

(iv) to work out requirements for other public utilities such as telephones, electric power, etc.

Following are various methods of population forecasts or population projections and the selection of method will naturally depend on the available data:

(1) Arithmetical increase method
(2) Geometrical increase method
(3) Incremental increase method
(4) Graphical method
(5) Comparative method
(6) Zoning method
(7) Ratio and correlation method
(8) Growth composition analysis method
(9) Logistic curve method.

A brief description of each of the above method of forecasting population will now follow:

(1) *Arithmetical increase method:* In this method, the average increase of population for the last *three* or *four* decades is worked out and then for each successive future decade, this average is added. This method gives low results and it is to be adopted for large cities which have practically reached their maximum development.

Problem 2-3.

The census records of a city show population as follows:

Present.................................... 50000
Before one decade 47100
Before two decades 43500
Before three decades .... 41000.

Work out the probable population after *one*, *two* and *three* decades by using arithmetical increase method.

Solution:

The average increase of population between successive decades upto present is worked out as follows:

Present and first decades	=	50000 − 47100	=	2900
First and second decades	=	47100 − 43500	=	3600
Second and third decades	=	43500 − 41000	=	2500
		Total		9000

Average increase per decade $= \frac{9000}{3} = 3000.$

The population after each successive future decade is obtained by adding this average as follows:

Population after *one* decade = 50000 + 3000 = 53000
Population after *two* decades = 53000 + 3000 = 56000
Population after *three* decades = 56000 + 3000 = 59000.

(2) ***Geometrical increase method:*** In this method, it is assumed that the percentage increase in population from decade to decade remains constant. From the available census records, this percentage is fixed and then population of each future successive decade is worked out. The fixation of percentage in case of developing cities should be done carefully. Otherwise this method is likely to give very high results. This method gives better results for old cities which are not undergoing further development.

As the increase in population is compounded over the existing population every decade, this method is sometime also referred to as the *uniform increase method*.

The assumed average growth rate can be computed in the following *two* ways:

(i) *Arithmetic average:* The average of the percentage growth rates of the several known decades of the past are worked out and then, by taking the arithmetic mean, the constant increase per decade is obtained. Let a_1, a_2, a_3, etc. be the percentage increase in population from decade to decade.

Then, $$a = \frac{a_1 + a_2 + a_3 + \ldots\ldots}{n}$$

where a = arithmetic average

n = number of decades.

(ii) *Geometric average:* In this method, the average growth rate is obtained by the geometric average as follows:

$$a = \sqrt[n]{a_1 \times a_2 \times a_3 \times \ldots\ldots}$$

Problem 2-4.

Work out problem 2-3 by using the geometrical increase method.

Solution:

(1) *By arithmetic average:* The percentage increase in population from decade to decade is worked out as follows:

Present and first decades $= \frac{50000 - 47000}{47100} \times 100 = 6.16$

First and second decades $= \frac{47100 - 43500}{43500} \times 100 = 8.28$

Second and third decades $= \frac{43500 - 41000}{41000} \times 100 = 6.10$

Total $= 20.54$

$\therefore$ Average percentage $= \frac{20.54}{3} = 6.85$, say 7.

The population after each successive future decade is obtained by adding this percentage increase as follows:

Population after *one* decade $= 50000 + 50000 \times 0.07 = 53500$

Population after *two* decades $= 53500 + 53500 \times 0.07 = 57245$

Population after *three* decades $= 57245 + 57245 \times 0.07 = 61252.$

(2) *By geometric average:* In this case,

average population $= \sqrt[3]{6.16 \times 8.28 \times 6.10}$

$= \sqrt[3]{311.13}$

$= 6.78$, say 7.

Hence the population after each successive future decade will be same as above.

(3) *Incremental increase method:* This method combines the above *two* methods. The population of each successive future decade is first worked out by the arithmetical increase method and to these values, the incremental average per decade is added. It thus combines the advantages of both the above methods and hence it gives satisfactory results.

Problem 2-5.

Work out problem 2-3 by using the incremental increase method.

Solution:

The incremental increase between successive decades is worked out as follows:

Period	Increase of population	Incremental increase
Second and third decades	2500	
		+ 1100
First and second decades	3600	
		− 700
Present and first decades	2900	
		Net + 400

$\therefore$ Average incremental increase $= \frac{400}{2} = 200.$

Average arithmetical increase as worked out in problem 3 $\Big\} = 3000.$

The population after each successive future decade is obtained by adding arithmetical increase and this average incremental increase as follows:

Population after *one* decade $= 50000 + 3000 + (1 \times 200)$
$= 53200$

Population after *two* decades $= 53200 + 3000 + (2 \times 200)$
$= 56600$

Population after *three* decades $= 56600 + 3000 + (3 \times 200)$
$= 60200$

(4) *Graphical method:* In this method, a curve of population against time is drawn for the city under consideration. The known census records are put up on the graph to get the shape of the curve. The curve is then carefully extended from present to future decades and the population after each successive future decade is read from the curve. The extension of curve in future decades should be based on personal judgement of the designer and it should be assisted by probable future conditions and past history of the city.

It may be noted that graphical method indicates the graphic representation of previous mathematical methods. The nature of extension will determine the mathematical method.

Problem 2-6.

Work out problem 2-3 by using the graphical method.

Solution:

The population in thousands is plotted against decades as shown in fig. 2-2. The population after each successive future decade is obtained by extending the curve, shown by dotted line in fig. 2-2.

The curve of cost in Rs. (million) per unit of storage against height of dam is drawn as shown in fig. 3-26 and it indicates that the most economic height of dam with respect to the construction cost only is 70 m.

Problem 3-3.

Calculate the discharge of a tube well of diameter 80 cm. The thickness of water bearing strata and drawdown are respectively 10 m and 4 m. Assume the radius of circle of influence as 300 m and permeability as 20 m^3 per unit area per day.

Solution:

$$Q = \frac{\pi P (H^2 - h^2)}{2.303 \log_{10} \frac{R}{r}}$$

In the given problem,

$P = 20$ m^3 per unit area per day

$H = 10$ m

$H - h = 4$ m

or $h = H - 4 = 10 - 4 = 6$ m

$R = 300$ m

$r = 40$ m.

Substituting,
$$Q = \frac{\pi \times 20 \times (10^2 - 6^2)}{2.303 \times \log_{10} \frac{300}{0.40}}$$
$$= \frac{\pi \times 20 \times 16 \times 4}{2.303 \times 2.8751}$$
$$= 607.30 \text{ m}^3 \text{ per day.}$$

Problem 3-4.

The diameter of a tube well is 50 cm. It is constructed in an aquifer of thickness 14 m. The radius of circle of influence is 225 m. Assuming permeability as 30 m^3 per unit area per day, calculate the drawdown when the yield of well is 1900 m^3 per day.

Solution:

$$Q = \frac{\pi P (H^2 - h^2)}{2.303 \log_{10} \frac{R}{r}}$$

In the given problem:

$Q = 1900$ m^3 per day

$P = 30$ m^3 per unit area per day

$H = 14$ m

$R = 225$ m

$r = 25$ cm.

Substituting,
$$1900 = \frac{\pi \times 30 \times (14^2 - h^2)}{2.303 \log_{10} \frac{225}{0.25}}$$

$\therefore$ $1900 \times 2.303 \times 2.4771 = \pi \times 30\ (14^2 - h^2)$

$\therefore$ $115 = 14^2 - h^2$

$\therefore$ $h^2 = 81$

$\therefore$ $h = 9$ m

$\therefore$ Drawdown $= (H - h) = 14 - 9 = 5$ m.

Problem 3-5.

In a recuperation test, the following results were obtained:

Initial depression head	= 8 m
Final depression head	= 5 m
Time of recuperation	= 2 hours
Diameter of well	= 4 m.

Calculate the specific capacity of well and yield under a head of 3 m.

Solution:

$$K = 2.303 \frac{A}{T} \log \frac{H_1}{H_2}.$$

In the given problem,

$A = \frac{\pi \times 4^2}{4} = 4\pi \text{ m}^2$

$T = 2$ hours

$H_1 = 8$ m

$H_2 = 5$ m.

Substituting, $K = 2.303 \frac{4\pi}{2} \log \frac{8}{5}$

$= 2.95$ m^3 per hour under the head of one metre.

Then, $Q = K.H$

$= 2.95 \times 3 = 8.85$ m^3 per hour.

Scanned by CamScanner

Problem 3-6.

Find the diameter of an open well to give a yield of 360 litres per minute. The working depression head is 4 m and the aquifer soil consists of coarse sand.

Solution:

$$Q = \left(\frac{K}{A}\right) A.H.$$

In the given problem,

Q = Yield of well in m^3 per hour

$= \frac{360 \times 60}{1000} = 21.6\ m^3$ per hour

$\frac{K}{A}$ = 1 for coarse sand expressed in m^3 per hour per m^2 of area of well under one metre depression head

$A = \frac{\pi d^2}{4}$ where d is the diameter of well in m

H = 4 m.

Substituting,

$$21.6 = 1 \times \frac{\pi d^2}{4} \times 4$$

$$\therefore \quad d^2 = \frac{21.6}{\pi} = 6.88$$

$\therefore \quad d$ = 2.62 m, say 2.70 m.

Problem 3-7.

From the following data, work out the most economical size of tube well:

Yield of well	: 7200 m^3 per day
Thickness of confined aquifer	: 30 m
Radius of circle of influence	: 300 m
Drawdown	: 5 m
Permeability	: 60 m^3 per unit area per day.

Solution:

$$Q = \frac{2.72\ Pt\ (H-h)}{\log \frac{R}{r}}$$

In the given problem,

Q = 7200 m^3 per day

P = 60 m^3 per unit area per day

t = 30 m

$(H-h)$ = 5 m

R = 300 m.

Substituting,

$$7200 = \frac{2.72 \times 60 \times 30 \times 5}{\log \frac{300}{r}}$$

$$\therefore \quad \log \frac{300}{r} = 3.40 \qquad \therefore \frac{300}{r} = 2512$$

$$\therefore \quad r = \frac{300}{2512} \times 100 = 11.94 \text{ cm, say } 120 \text{ mm.}$$

Problem 3-8.

A tube well having diameter of 20 cm taps an artesian aquifer of thickness 25 m. If drawdown is 4.50 m and permeability is 40 m^3 per unit area per day, calculate the yield of tube well in litres per hour. Assume the radius of circle of influence as 300 m.

Solution:

$$Q = \frac{2.72\ Pt\ (H-h)}{\log \frac{R}{r}}$$

In the given problem,

P = 40 m^3 per unit area per day

t = 25 m

$(H-h)$ = 4.5 m

R = 300 m

$r = \frac{10}{100} = 0.10$ m.

Substituting,

$$Q = \frac{2.72 \times 40 \times 25 \times 4.5}{\log \frac{300}{0.10}}$$

$$= \frac{12240}{3.4771} = 3520.17\ m^3 \text{ per day}$$

$$= \frac{3520.17}{24} \times 1000$$

$$= 146673.75, \text{ say } 147700 \text{ litres per hour.}$$

Power for pumps:

Following machines are used to develop power for the working of pumps:

(1) Steam engine
(2) Diesel engine
(3) Gasoline engine
(4) Electric motor.

Each of the above machine will now be briefly described.

(1) *Steam engine:* The steam engine is clumsy and old fashioned. It is practically out of date at present. It consumes more fuel and takes more time to come to the working stage. Also there is considerable loss of energy when it is being stopped for cooling. It is however reliable and can be used for large installations where fuel is cheaply available. The efficiency of steam engine is about 60% to 70%.

(2) *Diesel engine:* The initial cost of this type of engine is high and it produces noise during working. It requires skilled supervision. The engine is not so clumsy as steam engine and it takes up the load at once. It is reliable and consumes less quantity of fuel. The efficiency of diesel engine is about 70% to 80%.

(3) *Gasoline engine:* In this type of engine, the gasoline or petrol is used as fuel. It is therefore very costly and hence it is rarely adopted. It is suitable for working as stand-by units of pumps.

(4) *Electric motor:* This is the modern machine to create power for pumps and it is used for small and medium plants. The motor is compact in design and can be switched on immediately. It runs smoothly and it is free from dust and smoke. It is found to be cheap at places where electricity can be developed economically. However, in case of sudden failure of electric plant, the whole system of water supply comes to a standstill and causes great hardship. Hence it is advisable to keep diesel engines or gasoline engines as stand-by units along with electric motors. The efficiency of electric motors is about 90% to 95%.

The above are the *four* machines which are adopted for developing power for the pumps and depending upon the facilities, fund, design, nature of pump, etc., a suitable machine is recommended.

Horse-power of pumps:

The horse-power required for the pumps is worked out as follows:

Let W = weight of water in kg per second
H = total head in m.

Then, $$\text{H.P.} = \frac{WH}{75}.$$

This is known as the *water horse-power* or W.H.P. and the brake horse-power or B.H.P. is obtained by dividing W.H.P. with the efficiency of pump and motor.

Let E = combined efficiency of pump and motor.

Then, $$\text{B.H.P.} = \frac{WH}{75E}.$$

In this equation, the value of H is obtained by the following equation:

$$H = h + h_f$$

where h = total static head or difference in level between the lowest water level in the well and full supply level of the tank

h_f = head lost in friction.

Let f = Coefficient of friction

l = Length of pipe in m

v = Velocity of flow in m per second

g = Acceleration due to gravity = 9.81 m/sec^2

d = Diameter of pipe in m

Q = Discharge in m^3 per second.

Then, $$Q = \frac{\pi d^2}{4} \cdot v$$

or $$v = \frac{4Q}{\pi d^2}.$$

Then, according to the modified Darcy-Weibach formula,

$$h_f = \frac{flv^2}{2gd} = \frac{fl}{2 \times 9.81 \times d} \times \left(\frac{4Q}{\pi d^2}\right)^2$$

$$= \frac{flQ^2}{12.1 d^5}.$$

Note: In working out the total head, the loss of head due to friction only is considered as other minor losses of head are generally negligible.

Problem 4-1.

A city has a population of 60000. It is to be supplied with water at 250 litres per head per day. Calculate the B.H.P. of motor to raise the water to an overhead tank 60 m high. The length and diameter of the rising main are 300 m and 30 cm respectively. The efficiency of motor is 95% and that of pump is 60%. Assume f = 0.04 and peak hour demand as 1.50 times the average demand.

Solution:

Average demand $= 60000 \times 250$

$= 15000000$ litres per day

$= 15000\ m^3$ per day

$= \dfrac{15000}{24 \times 60 \times 60}\ m^3$ per second

$= 0.1736\ m^3$ per second.

Peak hour demand $= 1.50 \times 0.1736$

$= 0.26\ m^3$ per second.

$\therefore$ $W = 1000 \times 0.26 = 260$ kg per second

$h = 60$ m

$$h_f = \frac{flQ^2}{12.1 d^5}$$

$$= \frac{0.04 \times 300 \times (0.26)^2}{12.1 \times (0.30)^5} = 27.82 \text{ m}$$

$H = h + h_f$

$= 60.00 + 27.82 = 87.82$ m

$E = 0.95 \times 0.60 = 0.57.$

Hence, B.H.P. $= \dfrac{WH}{75E}$

$= \dfrac{260 \times 87.82}{75 \times 0.57}$

$= 534.$

Problem 4-2.

Design a pumping station to raise water from an intake well to a sedimentation tank with the following data:

Water to be raised per day	$= 18000\ m^3$
Length of suction pipe	= 40 m
Length of rising main	= 150 m
Coefficient of friction	= 0.04
Diameter of pipe (suction and rising)	= 50 cm
Shifts of working of pumps	= 2
Duration of each shift	= 8 hours
Combined efficiency of motor and pump	= 80%
Static head through which water is to be raised	

Solution:

Water requirement per day $= 18000\ m^3$

Total pumping hours $= 2 \times 8 = 16.$

Required pumping capacity per hour $= \dfrac{18000}{16} = 1125\ m^3.$

Note: Since requirement of water per day is 18000 m^3 and the pumps are to work only for 16 hours per day, the required pumping capacity per hour is obtained by dividing daily requirement by 16 and not by 24.

Q per second $= \dfrac{1125}{60 \times 60} = 0.31\ m^3$ per second

$W = 1000 \times 0.31 = 310$ kg per second

$h = 21$ m

$$h_f = \frac{flQ^2}{12.1 . d^5} = \frac{0.04 \times (40 + 150) \times (0.31)^2}{12.1 \times (0.50)^5}$$

$= 1.95$ m

$H = h + h_f = 21 + 1.95 = 22.95$ m

$E = 0.80.$

Hence, B.H.P. $= \dfrac{WH}{75E}$

$= \dfrac{310 \times 22.95}{75 \times 0.80} = 118.60.$

Provide 4 units of pumps, each having B.H.P. of 30. Thus the total B.H.P. would be 120. Provide 2 such other additional units of pumps as stand-by units.

Problem 4-3.

The water is to be pumped from a jack well to the service reservoir situated 2 km away. Work out the capacity of pumps and motors from the following data:

Water to be raised per day	$= 8000\ m^3$
Total hours of pumping	= 16
R.L. of water level in jack well	= 322.00
R.L. of F.S.L. of reservoir	= 342.00
Depression in water level during pumping	= 1.50 m
Friction in rising main	= 1 m per km length
Other losses of head due to bends, valves, etc.	= 1 m
Combined efficiency of pump and motor	= 63%.

Solution:

Water to be raised per day $= 8000\ m^3$

Total pumping hours $= 16$

Required pumping capacity per hour $= \frac{8000}{16} = 500\ m^3$.

$$Q = \frac{500}{60 \times 60} = 0.14\ m^3 \text{ per second}$$

$$W = 1000 \times 0.14 = 140 \text{ kg per second}$$

$$h = (342 - 322) + \text{depression head}$$

$$= 20 + 1.50 = 21.50 \text{ m}$$

$$h_f = 1 \times 2 = 2 \text{ m.}$$

Other losses $= 1$ m

$$H = h + h_f + \text{other losses}$$

$$= 21.50 + 2 + 1 = 24.50 \text{ m}$$

$$E = 0.63.$$

Hence, B.H.P. $= \frac{WH}{75E}$

$$= \frac{140 \times 24.50}{75 \times 0.63}$$

$$= 72.59.$$

Adopt 2 pumps of 40 B.H.P. each together with one stand-by unit of the same capacity.

Problem 4-4.

The water is to be raised from a tube well to an overhead storage tank. Work out the B.H.P. of the electro-motor pumping set from the following data:

Discharge of tube well	= 30 litres per second
Length of rising main	= 200 m
Coefficient of friction	= 0.072
R.L. of ground level	= 200.00
R.L. of water level in the well	= 192.00
Depression head	= 5 m
R.L. of water level in overhead tank	= 215.00
Combined efficiency of electro-pumping set	= 75%
Velocity in rising main	

Solution:

$$Q = \frac{\pi d^2}{4} \times v$$

where Q = discharge

$= 0.03\ m^3$ per second

d = diameter of rising main

v = velocity

$= 180$ cm per second.

$$\therefore \quad 0.03 = \frac{\pi d^2}{4} \times 1.80$$

$$\therefore \quad d = \sqrt{\frac{4 \times 0.03}{\pi \times 1.80}}$$

$$= 0.1456 \text{ m, say } 0.15 \text{ m.}$$

$$W = 1000 \times 0.03 = 30 \text{ kg per second}$$

$$h = (200 - 192) + (215 - 200) + \text{depression head}$$

$$= 8 + 15 + 5 = 28 \text{ m}$$

$$h_f = \frac{flQ^2}{12.1 d^5}$$

$$= \frac{0.072 \times 200 \times (0.03)^2}{12.1 \times (0.15)^5} = 14.22 \text{ m.}$$

$$H = h + h_f$$

$$= 28 + 14.22 = 42.22 \text{ m}$$

$$E = 0.75.$$

Hence, B.H.P. $= \frac{WH}{75E}$

$$= \frac{30 \times 42.22}{75 \times 0.75}$$

$$= 22.52.$$

Adopt 3 units of 10 B.H.P. each together with one stand-by unit of the same capacity.

QUESTIONS

1. Why is pumping of water required in water supply projects?
2. What considerations govern the choice of a particular type of pump?
3. Discuss in detail the utility of air lift pumps in water supply projects.

is observed. The difference between the total amount of soap solution and the lather factor indicates the hardness of water.

The water, having hardness of about 5 degrees, is reasonably soft water and a very soft water is tasteless. Hence, for potable water, the hardness should preferably be more than 5 degrees but less than 8 degrees or so.

For the purpose of convenience, a tentative scale of hardness may be framed as shown in table 5-1.

TABLE 5-1
SCALE OF HARDNESS

No.	Nature of water	Hardness in degrees
1	Extremely soft	1
2	Very soft	2
3	Soft	3
4	Moderately soft	6
5	Moderately hard	7
6	Hard	9
7	Very hard	11
8	Excessively hard	15
9	Too hard for use	17

The hardness is normally expressed in terms of calcium carbonate. Now the chemical analyses for individual ions are usually given in terms of that ion. It will thus be necessary to convert the analytical results to the common denominator.

$$\text{Hardness in mg/l as } CaCO_3 = M^{++}\ (\text{mg/l}) \times \frac{\text{equivalent wt. of } CaCO_3}{\text{equivalent wt. of } M^{++}}$$

where M represents any ion or radical.

Now, $\text{Equivalent weight} = \dfrac{\text{Molecular weight}}{X}$

where

X = for acids, the number of moles of H^{++} obtainable from 1 mole of acid

X = for bases, the number of moles of H^{++} with which 1 mole of base will react.

Thus, Equivalent weight of $CaCO_3 = \dfrac{(40 + 12 + 3 \times 16)}{2} = 50.$

For solving problems on hardness, the following facts should be noted:

(1) The alkalinity will be caused by positively charged Ca^{++}, Mg^{++} and Sr^{++} ions and negatively charged CO_3^{--}, and HCO_3^- ions only. The readings of other metals should be ignored.

(2) The equivalent weights of Ca^{++}, Mg^{++} and Sr^{++} will be respectively as follows:

$$Ca = 40/2 = 20;$$
$$Mg = 24.4/2 = 12.2; \text{ and}$$
$$Sr = 87.6/2 = 43.8.$$

(3) For measuring alkalinity, the reading of only CO_3^{--} or HCO_3^- will be required and expressed as percentage, they can be found out from the following relations:

$$\left.\begin{array}{l}\text{Total alkalinity}\\ \text{as } HCO_3 \text{ in mg/l}\end{array}\right\} = \left\{\begin{array}{c}\text{Bicarbonate alkalinity}\\ \text{in mg/l}\end{array}\right\} \times 1.22$$

$$\left.\begin{array}{l}\text{Total alkalinity}\\ \text{as } CO_3 \text{ in mg/l}\end{array}\right\} = \left\{\begin{array}{c}\text{Carbonate alkalinity}\\ \text{in mg/l}\end{array}\right\} \times 0.60$$

$$\left[\begin{array}{l}\text{Molecular wt of } HCO_3 = (1 + 12 + 3 \times 16) = 61\\ \text{Molecular wt of } CO_3 = (12 + 3 \times 16) = 60\end{array}\right]$$

Problem 5-1.

The analysis of water from a bore shows the following results in mg/l:

$Ca = 60$, $Mg = 48$, $Na = 103.5$, $K = 19.5$
$HCO_3 = 244$, $SO_4 = 220.8$, $Cl = 78.1$.

Find out the total hardness, carbonate hardness and non-carbonate hardness.

Solution:

$$\text{Total hardness} = (60 \times 50/20 + 48 \times 50/12.2)$$
$$= (150 + 196.72) = 346.72 \text{ mg/l as } CaCO_3.$$

$$\left.\begin{array}{l}\text{Total alkalinity}\\ \text{as } HCO_3 \text{ in mg/l}\end{array}\right\} = \left\{\begin{array}{c}\text{Bicarbonate alkalinity}\\ \text{in mg/l}\end{array}\right\} \times 1.22$$

$\therefore$ $244 = \text{Bicarbonate alkalinity} \times 1.22$

$\therefore$ Bicarbonate alkalinity $= 244/1.22 = 200$ mg/l.

In this case,

Alkalinity < T.H.

$\therefore$ C.H. = Alkalinity = 200 mg/l.

Then, N.C.H. = T.H. − C.H.

= (346.72 − 200) = 146.72 mg/l.

Problem 5-2.

The analysis of a sample of water shows the following results in mg/l:

Na	= 20	Cl	= 40
K	= 30	HCO_3	= 67
Ca	= 5	SO_4	= 5
Mg	= 10	NO_3	= 10

The concentration of stroncium (Sr) is equivalent to a hardness of 2.29 mg/l and the carbonate alkalinity in this water is zero. Calculate the total hardness, carbonate hardness and non-carbonate hardness in mg/l as $CaCO_3$.

Solution:

$$\text{Total hardness} = \frac{Ca^{++} \times 50}{20} + \frac{Mg^{++} \times 50}{12.2} + 2.29$$

$$= \frac{5 \times 50}{20} + \frac{10 \times 50}{12.2} + 2.29$$

$$= (12.5 + 40.98 + 2.29)$$

$$= 55.77 \text{ mg/l as } CaCO_3.$$

Bicarbonate alkalinity = 67/1.22 = 54.92 mg/l as $CaCO_3$.

In this case,

Alkalinity < T.H.

∴ C.H. = Alkalinity = 54.92 mg/l as $CaCO_3$.

Then, N.C.H. = T.H. − C.H.

= (55.77 − 54.92) = 0.85 mg/l as $CaCO_3$.

Problem 5-3.

The results obtained from a sample of water are as follows in mg/l:

Na	= 20	Sr	= 2
K	= 30	Cl	= 40
Ca	= 6	HCO_3	= 72
Mg	= 11	SO_4	= 5

Find out T.H., C.H. and N.C.H. in mg/l as $CaCO_3$.

Solution:

Total hardness = (6 × 50)/20 + (11 × 50)/12.2 + (2 × 50)/43.8

= (15 + 45.08 + 2.28)

= 62.36 mg/l as $CaCO_3$.

Bicarbonate alkalinity = 72/1.22 = 59.02 mg/l as $CaCO_3$.

In this case,

Alkalinity < T.H.

∴ C.H. = Alkalinity = 59.02 mg/l as $CaCO_3$.

Then, N.C.H. = T.H. − C.H. = (62.36 − 59.02)

= 3.34 mg/l as $CaCO_3$.

Problem 5-4.

The total hardness value obtained from the complete analysis of a water sample is found to be 116 mg/l. The analysis further shows that the concentrations of all the *three* principal cations causing hardness are numerically the same. If the value of C.H. is 58 mg/l, calculate the following:

(1) the value of N.C.H.;
(2) the concentrations of principal cations; and
(3) the value of total alkalinity in mg/l.

Solution:

T.H. = C.H. + N.C.H.

In the given problem,

T.H. = 116 mg/l and C.H. = 58 mg/l.

∴ 116 = 58 + N.C.H.

∴ N.C.H. = (116 − 58) = 58 mg/l as $CaCO_3$.

Let P = Concentration of principal cations.

Then, T.H. = (P × 50)/20 + (P × 50)/12.2 + (P × 50)/43.8

∴ 116 = P (2.5 + 4.1 + 1.14)

∴ P = 116/7.74 = 14.99, say 15 mg/l

∴ Concentrations of Ca^{++}, Mg^{++} and Sr^{++} are 15 mg/l.

In this case,

Alkalinity < T.H.

∴ Total alkalinity = C.H. = 58 mg/l as $CaCO_3$.

(4) *Hydrogen-ion concentration (pH value)*: The acidity or alkalinity of water is measured in terms of its pH value or H-ion concentration.

The pure water (H_2O) consists of positively charged hydrogen or H-ions combined with negatively charged hydroxyl or OH-ions. But the process of dissociation takes place in pure water and hence it contains some uncombined positively charged H-ions and some for ready reference. This test is simple and hence it is commonly carried out in the public health laboratories. The usual indicators are Benzol yellow, Methyl red, Bromphenol blue, etc. for acidic range and Thymol blue, Phenol red, Tolyl red, etc. for alkaline range.

Problem 5-5.

In a water treatment plant, the pH values of entering and leaving waters are respectively 7.5 and 8.5. Assuming linear variation of pH with time, find out the average pH value of water.

Solution:

By definition,

$$pH = -\log_{10} H$$

For entering water,

$$pH = 7.5$$

$$\therefore \quad 7.5 = -\log_{10} H$$

$$\therefore \quad H = 10^{-7.5}$$

For leaving water,

$$pH = 8.5$$

$$\therefore \quad 8.5 = -\log_{10} H$$

$$\therefore \quad H = 10^{-8.5}$$

$$\therefore \quad \text{Average value of } H = \frac{10^{-7.5} + 10^{-8.5}}{2}$$

$$= 10^{-8.5}\left[\frac{10+1}{2}\right] = 5.50 \times 10^{-8.5}$$

$$\therefore \text{ Average pH value of water} = -\log_{10} H$$

$$= -\log_{10} (5.50 \times 10^{-8.5})$$

$$= (8.50 - 0.7404) = 7.7596.$$

Problem 5-6.

Calculate pH and OH values of freshly prepared distilled water.

Solution:

As per equation (3),

$$[H^+] \times [OH^-] = 10^{-14} \text{ mole/l}$$

In freshly distilled water, the concentrations of both the ions will be the same say, C moles/litre.

Then,

$$C \times C = 10^{-14}$$

$$C = 10^{-7}$$

$$[H^+] = 10^{-7} \text{ and } [OH^-] = 10^{-7}$$

Now,

$$pH = -\log_{10} [H^+]$$

$$= -\log_{10}[10^{-7}] = -[-7] = 7$$

Similarly

$$pOH = 7.$$

Problem 5-7.

A factory discharges 50 m^3/day of waste having pH = 11. If the waste contains KOH only, find out the quantity of KOH in kg/day.

Solution:

$$pH + pOH = 14$$

$$11 + pOH = 14$$

$$pOH = 14 - 11 = 3.$$

Thus the concentration of hydoxycl ion in the given water is 10^{-3} moles/litre i.e. $[OH^-] = 10^{-3}$ moles/litre.

The molecular weight of KOH in 1 litre solution works out to

$$(39 + 16 + 1) = 56 \text{ g.}$$

$$\text{KOH in g/litre} = 56 \times 10^{-3}$$

$$\text{KOH in kg/litre} = 56 \times 10^{-6}$$

Now,

$$\text{Discharge} = Q = 50 \text{ m}^3\text{/day} = 50 \times 10^3 \text{ litres/day.}$$

$$\text{Quantity of KOH in given waste} = (56 \times 10^{-6} \times 50 \times 10^3) \text{ kg/day}$$

$$= 2.80 \text{ kg/day.}$$

Problem 5-8.

Calculate the pH of 1000 mg/l of $Ca(OH)_2$.

Solution:

The molecular weight of $Ca(OH)_2$ works out to $[40 + 2(16+1)]$

$$= 74 \text{ g/l.}$$

Now, 74 g/litre contains 2 moles of $[OH^-]$.

$$1 \text{ g/litre contains } \frac{2}{74} \text{ moles of } [OH^-]$$

Thus, $[OH^-] = \frac{2}{74}$

$\therefore$ $pOH = -\log_{10} [OH^-]$

$= -\log_{10} [2/74]$

$= -\log_{10} [0.027] = 1.5686.$

Now, $pH + pOH = 14$

$\therefore$ $pH = 14 - pOH = (14 - 1.5686)$

$= 12.4314$

$\therefore$ pH of 1000 mg/l i.e. 1 g/l of $Ca(OH)_2 = 12.4314.$

Problem 5-9.

Find out the pH of the following mixture:

	Volume	pH
Solution A ..	500 ml	6
Solution B ..	500 ml	5

Solution:

As the volume of both the solutions in the mixture i.e. A and B is the same, the morality of the mixture will be half in 1000 ml.

pH of A = 6 i.e. $[H^+]$ of A $= 10^{-6}$ mole/litre

pH of B = 5 i.e. $[H^+]$ of B $= 10^{-6}$ mole/litre

pH of new mixture $= \left(\frac{1}{2} \times 10^{-6}\right) + \left(\frac{1}{2} \times 10^{-5}\right)$

$= (0.5 \times 10^{-6}) + (5 \times 10^{-6})$

$= 5.5 \times 10^{-6}$ mole/litre

$\therefore$ pH of new mixture $= -\log_{10} [H^+]$

$= -\log_{10}(5.5 \times 10^{-6})$

$= (-0.7404 + 6)$

$= 5.2596.$

Problem 5-10.

Find out the pH of the mixture which will be formed by mixing the following *two* volumes:

	Volume	pH
Solution A ..	100 ml	6
Solution B ..	900 ml	5

Solution:

In this case, the volumes of A and B are not the same in 1000 ml mixture.

pH of A = 6 i.e. $[H^+]$ of A $= 10^{-6}$ mole/litre

pH of B = 5 i.e. $[H^+]$ of B $= 10^{-5}$ mole/litre

For solution A:

The volume of solution A is 100 ml and concentration of $[H^+]$ is 10^{-6} mole/litre i.e. 10^{-6} moles per 1000 ml.

$\therefore$ Concentration of $[H^+]$ in 100 ml $= 10^{-6} \times \frac{100}{1000}$

$\therefore$ $[H^+]_A = 10^{-7}$ mole/litre.

For solution B:

The volume of solution B is 900 ml and concentration of $[H^+]$ is 10^{-5} mole/litre i.e. 10^{-5} moles per 1000 ml.

$\therefore$ Concentration of $[H^+]$ in 900 ml $= 10^{-5} \times \frac{900}{1000}$

$\therefore$ $[H^+]_B = 9 \times 10^{-6}$ mole/litre.

Concentration of $[H^+]$ in mixture $= [H^+]_A + [H^+]_B$

$= [10^{-7}] + [9 \times 10^{-6}]$

$= (0.1 \times 10^{-6}) + (9 \times 10^{-6})$

$= 9.1 \times 10^{-6}$ mole/litre

$\therefore$ $pH = -\log_{10} (9.1 \times 10^{-6})$

$= (-0.959 + 6) = 5.041.$

Problem 5-11.

There are *three* samples *X*, *Y* and *Z* of water having pH values of 4.5, 5.5 and 6.5 respectively. Calculate how many times *X* is acidic than *Z*

Solution:

$pH = -\log_{10} [H^+]$

Sample *X*:

$pH = 4.5$

$\therefore$ $4.5 = -\log_{10} [H^+]$

$\therefore$ $[H^+] = 10^{-4.5}$ mole/litre.

Sample Z:

$$pH = 6.5$$

$$6.5 = -\log_{10}[H^+]$$

$$\therefore \quad [H^+] = 10^{-6.5} \text{ mole/litre.}$$

$$\therefore \quad \frac{\text{Concentration of } [H^+] \text{ ion in } X}{\text{Concentration of } [H^+] \text{ ion in } Z} = \frac{10^{-4.5}}{10^{-6.5}} = 100$$

Thus sample X is 100 times acidic than sample Z.

Problem 5-12.

What would be the pH of a solution containing 1.70×10^{-8} g of hydroxide per litre? Assume molecular weight of OH as 17 g.

Solution:

$$[OH^-] = \frac{17 \times 10^{-9}}{17} = 10^{-9} \text{ mole/l}$$

$$\therefore \quad pOH = -\log(10^{-9}) = 9$$

Now, $pH + pOH = 14$

$$\therefore \quad pH = (14 - 9) = 5.$$

Problem 5-13.

If the hydrogen concentration is 3×10^{-2} mole/l, calculate the hydroxide concentration.

Solution:

As per equation (3),

$$[H^+] \times [OH^-] = 10^{-14} \text{ mole/l}$$

$$\therefore \quad [3 \times 10^{-2}] \times [OH^-] = 10^{-14} \text{ mole/l}$$

$$\therefore \quad [OH^-] = \frac{1}{3} \times 10^{-12} \text{ mole/l}$$

(5) ***Alkalinity:*** The term *alkalinity* with reference to the water and waste water is defined as the capacity of substances contained in the water to take up hydroxium (H_3^+O) to reach a defined pH value (4.3 to 14). The alkalinity is due to the presence of bicarbonate (HCO_3^-), carbonate (CO_3^{--}) or hydroxide (OH^-).

The determination of alkalinity is very useful in waters and wastes because it provides buffering to resist the changes in pH value. The alkalinity is usually divided into the following *two* parts:

(1) Total alkalinity i.e. above pH 4.5

(2) Caustic alkalinity i.e. above pH 8.2.

The alkalinity is measured by the volumetric analysis. The various types of indicators are available for this purpose. The commonly adopted *two* indicators are as follows:

(1) Phenolphthalein : pink above pH 8.2 and colourless below pH 8.2

(2) Methyl orange : red below pH 4.5 and yellow-orange above pH 4.5.

The bromcresol green-methyl red indicator may be preferable to methyl orange as the colour change from greenish-blue above pH 4.5 to light pink below pH 4.5 is more definite.

The amount of alkalinity is expressed in terms of $CaCO_3$. If the strength of titrant solution is N/50, 1 ml of titrant solution will be equal to 1 mg $CaCO_3$ because the equivalent weight of $CaCO_3$ is 50.

Thus, $$\text{Alkalinity in mg/l as } CaCO_3 = \frac{\text{Total reading}}{\text{Vol. of sample in ml}} \times 1000.$$

If the strength of titrant solution is N/40, the equation will be as follows:

$$\text{Alkalinity in mg/l as } CaCO_3 = \frac{\text{Total reading} \times (0.025 \times 50 \times 1000)}{\text{Vol. of sample in ml}}$$

$$= \frac{\text{Total reading}}{\text{Vol. of sample in ml}} \times 1250.$$

The neutralization of OH^- is complete at pH 8.2. The neutralization of CO_3^{--} is only half complete at pH 8.2 and not fully completed until a pH value of 4.5 is reached. Thus alkalinity on pH scale is represented as follows:

(1) The range of total alkalinity is 4.5 to 14.

(2) The range of bicarbonate i.e. HCO_3^- alkalinity is from 4.5 to 8.2.

(3) The range of carbonate i.e. CO_3^{--} alkalinity is from 8.2 to 10.

(4) The range 0 to 4.5 indicates no alkalinity.

Table 5-2 can be used for finding out the alkalinity of a sample with the process of titration. The letter P indicates phenolphthalein reading and the letter T indicates total alkalinity.

LAB

TABLE 5-2

DETERMINATION OF ALKALINITY

Result of titration	Hydroxide alkalinity as $CaCO_3$ (OH^-)	Carbonate alkalinity as $CaCO_3$ (CO_3^{--})	Bicarbonate alkalinity as $CaCO_3$ (HCO_3^-)
P = 0	0	0	T
P < 0.5 T	0	2P	T − 2P
P = 0.5 T	0	2P	0
P > 0.5 T	2P − T	2(T − P)	0
P = T	T	0	0

Following rules should be kept in mind while finding out the alkalinity of a sample:

(1) For simplicity, it is assumed that HCO_3^- and OH^- alkalies cannot come together in the same sample.

(2) The alkalinities of other than carbonate, bicarbonate and hydroxide origin are absent.

(3) OH^- alone gives initial pH of about 10.

(4) CO_3^{--} will be present at pH $\geq$ 8.2

(5) OH^- and CO_3^{--} together give initial pH of about 10.

(6) CO_3^{--} and HCO_3^- can exist together.

(7) HCO_3^- alone gives initial pH < 8.2.

The determination of alkalinity of water is helpful for the following *reasons*:

(i) It assists in finding out the quantity of lime and soda-ash required for the removal of hardness.

(ii) It helps to neutralise the acids produced during flocculation.

(iii) It leads to the reactions which may occur between alkalinity and certain cations in the water. The resultant precipitate can foul pipes and other appurtenances of the water distribution systems.

(iv) The excess alkalinity is harmful for irrigation which leads to the soil damage and reduce crop yields.

(v) The highly alkaline water is usually unpalatable.

(vi) The large amount of alkalinity imparts a bitter taste to the water.

(vii) The natural waters are sometimes rendered alkaline to control corrosion due to acids.

(viii) The waste waters containing excess caustic alkalinity are not to be discharged into natural streams or sewers.

(ix) The water having alkalinity less than 250 mg/l is desirable for domestic consumption and for R.C.C. construction.

Problem 5-14.

From the following data of volumetric analysis, calculate the hydroxide, carbonate and bicarbonate alkalinities:

Sample 100 ml	Total ml of titrant to reach end point	
	Phenolphthalein	Mythyl orange
A	10	15.5
B	14.4	38.6
C	8.2	8.4
D	0	12.7

Solution:

Sample A:

$$P = \frac{10 \times 1000}{100} = 100$$

$$T = \frac{15.5 \times 1000}{100} = 155$$

$$0.5\ T = 77.5$$

As P > 0.5 T, from table 5-2,

$$OH^- \text{ alkalinity} = 2P - T = (200 - 155) = 45$$

$$CO_3^{--} \text{ alkalinity} = 2(T - P) = 2(155 - 100) = 110$$

$$HCO_3^- \text{ alkalinity} = 0.$$

Sample B:

$$P = \frac{14.4 \times 1000}{100} = 144$$

$$T = \frac{38.6 \times 1000}{100} = 386$$

$$0.5\ T = 193$$

As P < 0.5 T, from table 5-2,

$$OH^- \text{ alkalinity} = 0$$

$$CO_3^{--} \text{ alkalinity} = 2P = 288$$

$$HCO_3^- \text{ alkalinity} = T - 2P = (386 - 288) = 98.$$

Sample C:

$$P = \frac{8.2 \times 1000}{100} = 82$$

$$T = \frac{8.4 \times 1000}{100} = 84$$

$$0.5\ T = 42$$

$\therefore$ As $P > 0.5\ T$, from table 5-2,

OH^- alkalinity $= 2P - T = (164 - 84) = 80$

CO_3^{--} alkalinity $= 2(T - P) = 2(84 - 82) = 4$

HCO_3^- alkalinity $= 0$.

Sample D:

$$P = 0$$

$$T = \frac{12.7 \times 1000}{100} = 127$$

As $P = 0$, from table 5-2,

OH^- alkalinity $= 0$

CO_3^{--} alkalinity $= 0$

HCO_3^- alkalinity $= T = 127$.

Problem 5-15.

Find out the total and individual (species-wise) alkalinity from the following laboratory record:

Strength of titrant : N/40 → ধরলে N=50L

Volume of sample : 50 ml $= 50 \times 10^3$ mL

Phenolphthalein end point : 5.6 ml

Methyl orange end point : 13.5 ml.

Also estimate the pH value of sample. $\frac{50 \times 1000}{40}$ (N)

Solution:

As strength of titrant is N/40,

$$P = \frac{5.6 \times 1250}{50} = 140$$

$$T = \frac{5.6 \times 1250}{50} = 337.5$$

$$0.5\ T = 168.75$$

As $P < 0.5\ T$, from table 5-2,

OH^- alkalinity $= 0$

CO_3^{--} alkalinity $= 2P = 280$

HCO_3^- alkalinity $= T - 2P = (337.5 - 280) = 57.5$.

As CO_3^{--} and HCO_3^- exist together, the pH value of sample will be between 8.2 to 10.

Problem 5-16.

A water sample with pH 9 had a caustic alkalinity of 70 mg/l, total alkalinity of 230 mg/l and total hardness of 300 mg/l, all as $CaCO_3$. Calculate the amounts of the various forms of alkalinity present and the amount of non-carbonate hardness.

Solution:

The pH of sample is 9 and hence the neutralization of CO_3^{--} is only half completed.

$\therefore$ Carbonate i.e. CO_3^{--} alkalinity $= 2 \times$ Caustic alkalinity

$= (2 \times 70)$

$= 140$ mg/l.

Now, Total alkalinity $= CO_3^{--}$ alkalinity $+ HCO_3^-$ alkalinity

$\therefore$ $230 = 140 + HCO_3^-$ alkalinity

$\therefore$ HCO_3^- alkalinity $= (230 - 140) = 90$ mg/l.

Also, Total hardness $=$ Alkalinity $+$ Non-carbonate hardness

$\therefore$ N.C.H. $= (300 - 230)$

$= 70$ mg/l.

(6) *Acidity:* The term *acidity* with reference to the water and waste water is defined as the capacity of substances contained in the water to take up hydroxyl ions (OH^-) to reach a defined pH value (0 to 8.2).

The acidity are of the following *two* types:

(i) Carbon dioxide acidity

(ii) Mineral acidity.

Now, out of *three* forces which oppose the tendency of settlement of particle, the attempts are made to control the first and second forces in purification process of water. The third force, namely, the viscosity of water is unpracticable to control as it is dependent on temperature. The control of temperature of a huge quantity of water becomes unreasonable and uneconomical.

The velocity of flow can be decreased by increasing the length of travel and thus a particle is allowed to stay for a longer period in the sedimentation tank. The particle is thus given maximum opportunity to come down and settle at the bottom of tank.

The size and shape of the particle are altered by the addition of certain chemicals in water. These chemicals are known as the *coagulants* and their action of making sedimentation tanks effective will be discussed at length in the next chapter.

Problem 6-1.

Find out the velocity of settlement of spherical discrete particles of 0.08 mm diameter with specific gravity 2.67 in water at 20°C.

Solution:

As per Stokes's law,

$$v = 418\,(s - s_1)\,d^2\,\frac{3T + 70}{100}$$

where d = diameter of discrete particles = 0.08 mm

T = 20°C

s = specific gravity of the particle = 2.67

s_1 = specific gravity of water = 1.00

v = velocity of settlement in mm per second.

Substituting

$$v = 418\,(2.67 - 1.00)\,(0.08)^2\,\frac{(3 \times 20) + 70}{100}$$

$$= 5.808 \text{ mm per second.}$$

Problem 6-2.

A settling tank is designed to remove spherical particles of 0.80 mm diameter with specific gravity 1.20 from the water at 22°C. Determine the removal of spherical discrete particles of 0.40 mm diameter with specific gravity 1.20 by this tank. Assume ideal settling conditions.

Solution:

As per Stokes's law,

$$v = 418\,(s - s_1)\,d^2\,\frac{3T + 70}{100}$$

For 0.80 mm diameter particles,

$$v_1 = 418\,(1.20 - 1.00)\,(0.80)^2\,\frac{(3 \times 22 + 70)}{100}$$

$$= 113.70\,(0.80)^2 \text{ mm per second.}$$

For 0.40 mm diameter particles,

$$v_2 = 418\,(1.20 - 1.00)\,(0.40)^2\,\frac{(3 \times 22 + 70)}{100}$$

$$= 113.70\,(0.40)^2$$

$$\frac{v_1}{v_2} = \frac{(0.80)^2}{(0.40)^2} = 4.00 \quad \text{.........................} \quad (1)$$

The settling tank is designed to remove 0.80 mm dia. discrete particles and if the same tank is used to remove 0.40 mm dia. discrete particles, the removal of latter will be proportional to the ratio of velocities of settlement. From equation (1),

$$\frac{v_2}{v_1} = \frac{1}{4}$$

$= 25\%$ = percentage removal of 0.40 mm diameter discrete particles.

Types of sedimentation tanks:

Depending upon the nature of working, the sedimentation tanks are of the following *two* types:

I. Fill and draw type tanks

II. Continuous flow type tanks.

Each of the above type of sedimentation tank will now be briefly described.

I. Fill and draw type tanks:

Working: These are also known as the *quiescent type or intermittent type* sedimentation tanks. The working of tank is simple. The water is filled in the tank and it is then allowed to rest for a certain time. During the period of rest, the particles in suspension will settle down at the bottom of tank. The clear water is then drawn off and the tank is cleaned of silt and filled again.

period is found to vary from 4 to 8 hours and when coagulants are used, it may vary from 3 to 4 hours.

In this method, the depth of settling tank is to be taken into consideration. The depths usually provided for settling tanks vary from 3.50 to 6.00 metres.

(ii) *Overflow rate:* In this method, it is assumed that the settlement of a particle at the bottom of tank does not depend on the depth of tank. But it depends on the surface area of the tank. This assumption can be theoretically justified as follows:

Let L = Length of tank
B = Breadth of tank
D = Depth of tank = Side water depth = S.W.D.
C = Capacity of tank
T = Detention period
Q = Discharge or rate of flow
V = Velocity of descend of a particle to the bottom of tank = Surface overflow rate = S.O.R.

Then,
$$T = \frac{C}{Q} = \frac{L \times B \times D}{Q} \quad \ldots\ldots (1)$$

Also,
$$T = \frac{\text{Distance of descend}}{\text{Velocity of descend}}$$
$$= \frac{D}{V} = \frac{\text{S.W.D.}}{\text{S.O.R.}} \quad \ldots\ldots (2)$$

Equating (1) and (2),
$$\frac{L \times B \times D}{Q} = \frac{D}{V}$$

or
$$\text{S.O.R} = V = \frac{Q}{L \times B} \quad \ldots\ldots (3)$$

The equation (3) shows that the velocity of descend of a particle is independent of the depth of tank and it inversely varies as the surface area of the tank.

The quantity of water passing per hour per unit area of settling tank is known as the *surface overflow rate* and the design of settling tank is made on the basis of overflow rate. The depth of tank is assumed as about 180 cm to 360 cm. For plain rectangular sedimentation tanks, the overflow rate is about 500 to 750 litres per hour per m^2 and when coagulants are used, it is about 1000 to 1250 litres per hour per m^2.

Problem 6-3.

The overflow rate of a flocculator clarifier is 60 l/m^2-min. Express this rate in m^3/m^2-d. If S.W.D. is 3.50 m, calculate the detention period.

Solution:

$$\text{S.O.R.} = 60 \text{ l/m}^2\text{-min}$$
$$= 60 \times 10^{-3} \text{ m}^3/\text{m}^2\text{-min}$$
$$= 60 \times 10^{-3} \times 60 \times 24 \text{ m}^3/\text{m}^2\text{-d}$$
$$= 86.40 \text{ m}^3/\text{m}^2\text{-d.}$$

$$\text{Detention period} = T = \frac{\text{S.W.D.}}{\text{S.O.R.}} = \frac{3.50}{86.40}$$
$$= 0.04 \text{ day} = 0.96 \text{ hour.}$$

Problem 6-4.

Calculate the diameter and S.W.D. of a settling tank to treat a flow of 20 × 10^6 ml/d. The maximum S.O.R. is 18 m^3/m^2–d and detention period is 3 hours.

Solution:

$$Q = 20 \times 10^6 \text{ ml/d}$$
$$= \frac{20 \times 10^6}{1000} \text{ m}^3/\text{d}$$
$$= 20000 \text{ m}^3/\text{d.}$$

$$\text{Detention period} = T = \frac{\text{S.W.D.}}{\text{S.O.R.}}$$

$$\therefore \text{S.W.D.} = \frac{3}{24} \times 18 = 2.25 \text{ m.}$$

$$\text{Surface area of settling tank} = \frac{\text{Discharge}}{\text{S.O.R.}} = \frac{20000}{18}$$

$$\therefore \frac{\pi}{4} (\text{Diameter})^2 = 1111.11 \text{ m}^2$$

$$\therefore \text{Diameter of settling tank} = 37.61, \text{ say } 38 \text{ m.}$$

Problem 6-5.

Calculate the dimensions of a rectangular plain sedimentation tank to treat 12 × 10^6 ml/day. Assume a detention period of 6 hours and the velocity of flow as 20 cm per second.

Solution:

$$Q = \text{Discharge or rate of flow}$$

$$= \frac{12 \times 10^6}{24} \times \frac{1}{1000} = 500 \text{ m}^3.$$

Now, Velocity of flow $= 20$ cm/minute $= 0.20$ m/minute

Length of tank required = Velocity of flow × Detention period

$$= 0.20 \times (6 \times 60) = 72 \text{ m}.$$

Also,

$$T = \frac{L \times B \times D}{Q}$$

$$\therefore \quad 6 = \frac{72 \times B \times D}{500}$$

$$\therefore \quad B \times D = \text{Cross-sectional area of tank}$$

$$= \frac{500 \times 6}{72} = 41.67 \text{ m}^2.$$

Assuming the effective depth of tank as 4 m and total depth with freeboard of 0.50 m as 4.50 m,

$$\text{Width of tank required} = \frac{41.67}{4} = 10.42 \text{ m, say } 10.50 \text{ m}.$$

Provide the sedimentation tank of dimensions 72 m × 10.50 m × 4.50 m.

Problem 6-6.

A rectangular tank with length of 15 m, width of 6 m and depth 3 m is to treat 2.40×10^6 ml/d. Calculate:

(1) Detention period of tank

(2) Average flow velocity through tank

(3) S.O.R.

Solution:

(1) *Detention period of tank:*

$$Q = 2.40 \times 10^6 \text{ ml/d}$$

$$= \frac{2.40 \times 10^6}{24} \text{ ml/hour}$$

$$= 100 \times 10^3 \text{ litres per hour} = 100 \text{ m}^3/\text{hour}.$$

Then,

$$T = \frac{L \times B \times D}{Q}$$

$$= \frac{15 \times 6 \times 3}{100}$$

$$= 2.70 \text{ hours}.$$

(2) *Average flow velocity through tank:*

$$\text{Flow velocity} = \frac{\text{Discharge}}{\text{Cross-sectional area}} = \frac{100}{6 \times 3} \times \frac{100}{60} \text{ cm/minute}$$

$$= 9.26 \text{ cm/minute}.$$

(3) *S.O.R.:*

$$\text{S.O.R.} = \frac{Q}{L \times B} = \frac{100}{15 \times 6} \times 1000$$

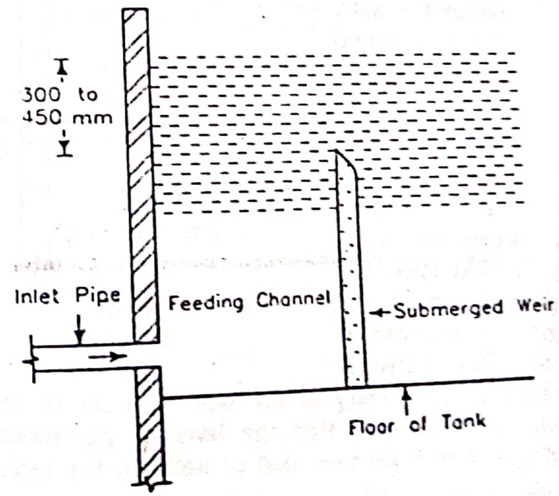
$$= 1111 \text{ litres per hour per m}^2.$$

---

(3) ***Inlet and outlet arrangements:*** The *inlet* is a device which is provided to distribute the water inside a tank and the *outlet* is a device which is meant to collect outgoing water. These arrangements should be properly designed and located in such a way that they do not form any obstruction or cause any disturbance to the flowing water.

If rate of inflow is equal to the rate of outflow, the water level of sedimentation tank is controlled. If *two* rates are different, the eddies or currents will be set up and they will greatly affect the working of tanks.

Some of the typical arrangements for inlets and outlets are shown in fig. 6-3 to fig. 6-6.



Inlet

Fig. 6-3

Fig. 6-3 shows a submerged inlet. It consists of a feeding channel which extends for the whole width of the tank and a submerged weir.

(6) *Sodium aluminate:* The chemical composition of this coagulant is $Na_2Al_2O_4$. This coagulant, when dissolved and mixed with water, reacts with salts of calcium and magnesium. Following are the chemical reactions:

$$Na_2Al_2O_4 + Ca(HCO_3)_2 = CaAl_2O_4 + Na_2CO_3 + CO_2 + H_2O \quad \ldots (1)$$
$$Na_2Al_2O_4 + CaCl_2 = CaAl_2O_4 + 2NaCl \quad \ldots (2)$$
$$Na_2Al_2O_4 + CaSO_4 = CaAl_2O_4 + Na_2SO_4 \quad \ldots (3)$$

This coagulant removes carbonate or temporary hardness as seen from equation (1) and it also removes non-carbonate or permanent hardness as seen from equations (2) and (3). The effective range of pH value for this coagulant is 6 to 8.50 and as pH value of normal water lies within this range, the adjustment of pH value is not important when this coagulant is used. However this coagulant is costly and hence it cannot be adopted for treating water on a large scale. It may be adopted for treating water to be used for boilers or for water in which floc by aluminium sulphate is not easily formed.

Problem 7-1.

Find out the quantity of alum required to treat 9 million litres of water per day. The dosage of alum is 14 mg per litre. Also work out the amount of CO_2 released per litre of water treated.

Solution:

$$\text{Quantity of alum per day} = \frac{14 \times 9 \times 10^6}{10^6} = 126 \text{ kg.}$$

The chemical reaction is as follows:

$$Al_2(SO_4)_3 . 18H_2O + 3Ca(HCO_3)_2 = 2Al(OH)_3 + 3CaSO_4 + 18H_2O + 6CO_2.$$

Now, Molecular wt. of alum $= (2 \times 26.97) + (3 \times 32.066) + (36 \times 1.008) + (30 \times 16)$

$= (53.94 + 96.198 + 36.288 + 480)$

$= 666.426$, say 666.

Molecular wt. of CO_2 $= (1 \times 12.01) + (2 \times 16)$

$= 44.01$, say 44.

Thus 666 mg of alum will release 6×44 mg of CO_2

$$\therefore \text{ 14 mg of alum will release} = \frac{14 \times 6 \times 44}{666}$$

$= 5.55$ mg of CO_2.

Problem 7-2.

15

A water treatment plant consumes ferrous sulphate and lime as coagulant at the rate of 10 mg of ferrous sulphate per litre of water. Find out the quantities of ferrous sulphate and lime required to treat 9 million litres of water.

Solution:

The chemical reactions involved are as follows:

$$FeSO_4 ,\ 7H_2O + Ca(OH)_2 = Fe(OH)_2 + CaSO_4 + 7H_2O$$
$$Ca(OH)_2 = CaO + H_2O.$$

Now, Molecular wt. of $FeSO_4$, $7H_2O$ $= (1 \times 55.85) + (1 \times 32.066) + (11 \times 16) + (14 \times 1.008)$

$= 278.028$, say 278.

Molecular wt. of CaO $= (40.08 + 16)$

$= 56.08$, say 56.

$$\text{Quantity of ferrous sulphate} = \frac{10 \times 9 \times 10^6}{10^6} = 90 \text{ kg.}$$

Also, 278 kg of ferrous sulphate will react with 56 kg of lime.

$\therefore$ Quantity of lime required corresponding to 90 kg of ferrous sulphate $= \dfrac{50 \times 90}{278}$

$= 18.13$ kg.

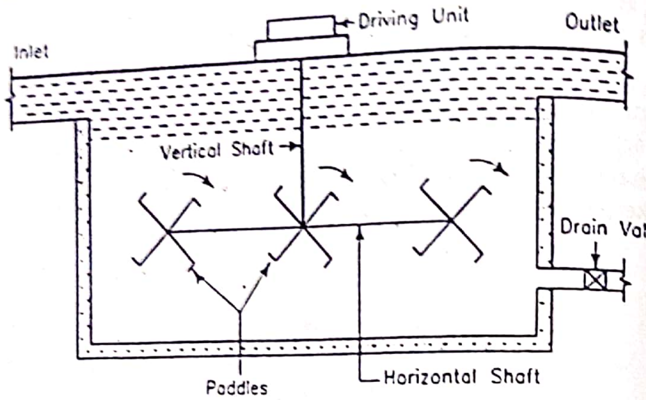
Feeding the coagulants:

The coagulants may be fed or allowed to enter water either in powder form or in solution form. The former is known as the *dry feeding* and the latter is known as the *wet feeding*. Each method of feeding will now be briefly described.

(1) ***Dry feeding:*** In dry feeding, the coagulant is stored in a powder form and it is then allowed to fall in the mixing channel in measured quantity. The dry feeding is desirable for various reasons as mentioned below:

(1) It is simple in operation.
(2) It requires relatively less space for its working.
(3) It grants freedom from corrosion.
(4) It becomes possible to maintain neatness.

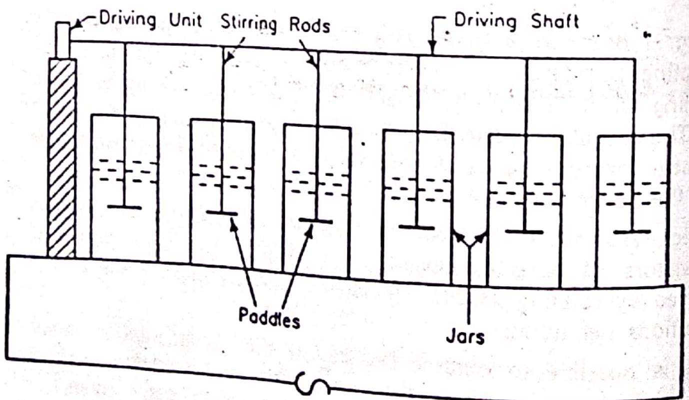
sedimentation tank. The detention period for floc chamber is about 15 to 40 minutes and that for tank, it is about 3 to 4 hours. The other details and aspects of design for such a tank are similar to those of a plain sedimentation tank. The cleaning of tanks is usually carried out at intervals of about 3 to 6 months.



Flocculator
FIG. 7-9

Jar test:

In order to determine approximately the dosage of coagulant, the usual test which is performed in the laboratory, is the *jar test*. The water for which dosage of coagulant is to be determined is placed in a number of jars or beakers having capacity of about 500 to 1000 millilitres.



Jar test
FIG. 7-10

Fig. 7-10 shows jar test with *six* jars. The various amounts of coagulant are then added in each jar. The driving unit is then started. The driving shaft rotates the paddles situated at lower ends of the stirring rods as shown in fig. 7-10. The paddles are rotated for 5 minutes at the speed of 30 to 40 R.P.M. and then slowly for about 20 minutes. The formation of floc in each jar is noted. The amount of coagulant in jar which produces good floc with the least amount of coagulant is preferred. The speed of paddles as well as time of mixing may be varied for different tests.

It is necessary to carry out this test frequently so as to determine optimum dose of coagulant and thus to achieve economy in its use. If the quality and characteristics of water are changing, the jar test should be done continuously.

Problem 7-3. 15

Design a coagulation sedimentation tank to treat 9 million litres of water per day. Make suitable assumptions where necessary.

Solution:

$$\text{Average daily water to be treated} = \frac{9 \times 10^6}{10^3} = 9000 \text{ m}^3.$$

$$\text{Average hourly water to be treated} = \frac{9000}{24} = 375 \text{ m}^3.$$

Assuming maximum hourly demand to be 1.50 times the average demand of water,

$$\text{maximum hourly demand} = (375 \times 1.50) = 562.50 \text{ m}^3.$$

Assume detention period of settling tank as 4 hours.

Then, $C = Q \times T$

where C = Capacity of settling tank in m^3

Q = Rate of flow per hour = 562.50 m^3 per hour

T = Detention period in hours = 4 hours.

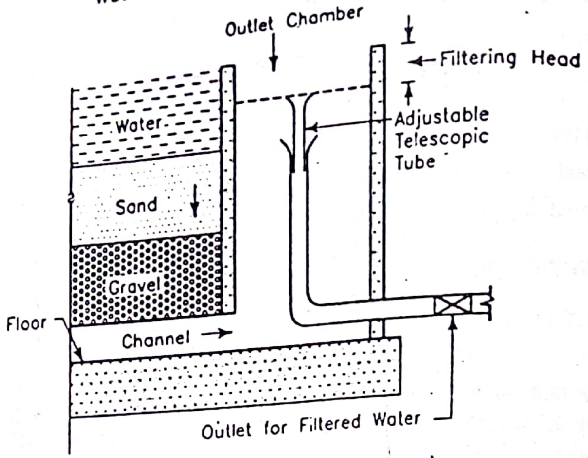
Substituting, $C = (562.50 \times 4) = 2250 \text{ m}^3$.

Assume effective depth of settling tank as 4 m.

$$\text{Sectional area} = \frac{2250}{4} = 562.50 \text{ m}^2.$$

Alternatively adopt overflow rate of 1000 litres per hour per m^2 of tank area.

$$\text{Then, sectional area} = \frac{562.5 \times 10^3}{1000} = 562.50 \text{ m}^2.$$



Cross-section through outlet chamber
FIG. 8-2

The depth of water on filter is to be carefully decided. It should neither be too small nor too high. Generally it is kept as equal to the height of filter media of sand.

The slow sand filters are usually worked for a maximum filtration head of 750 mm or so. But from the view point of safety, the maximum filtration head desired is obtained by multiplying the depth of filtering sand with a factor of 0.67 to 0.80. Thus, when specified limit of filtration head is reached, the filter requires cleaning.

For the purpose of cleaning, the top layer of sand is scraped or removed through a depth of about 15 mm to 25 mm. The water is then admitted to the filter. But the purified water is not taken into use until the formation of film around sand grains occurs. This may require a period of about *one* or *two* days.

When cleaning of filter had been done for a number of times, the effective depth of filter media of sand is reduced. In order to maintain the efficiency of filter, a fresh layer of about 150 mm depth of graded sand is then added to the filter. At places, where there is acute shortage of filtering sand, the scraped sand obtained during cleaning operations is washed and stored for future replacement.

The interval between *two* successive cleanings depends mainly on the nature of impurities present in water to be treated and the size of filtering sand. It usually varies from 1 to 3 months.

Rate of filtration: The rate of filtration for a normal slow sand filter varies from 100 to 200 litres per hour per m² of filter area

Efficiency of slow sand filters: The efficiency of slow sand filters is as follows:

(1) *Bacterial load:* The slow sand filters are highly efficient in the removal of bacterial load from water. It is expected that they remove about 98 to 99 per cent of bacterial load from raw water and this percentage may be as high as 99.50 to 99.90 when pre-treatment has been given to the raw water. However, for complete removal of bacteria, the disinfection is essential.

(2) *Colour:* The slow sand filters are less efficient in the removal of colour of raw water. It is estimated that they remove about 20 to 25 per cent colour of raw water.

(3) *Turbidity:* The slow sand filters can remove turbidity to the extent of about 50 p.p.m. For water having greater turbidity than 60 p.p.m., it is necessary to give preliminary treatment and bring down its turbidity below 50 p.p.m.

Problem 8-1.

Find the area of slow sand filters required for a town having a population of 15000 with an average rate of demand as 160 litres per head per day.

Solution:

Maximum daily demand $= (15000 \times 160 \times 1.50)$

$= 3600000$ litres.

Assuming a rate of filtration as 150 litres per hour per m² of filter area,

$$\text{area of filter required} = \frac{3600000}{150 \times 24} = 1000 \text{ m}^2.$$

Let the size of one unit be 16.00 m × 12.50 m. Then provide 6 such units of slow sand filters including one unit as stand-by. The units may be arranged in series with 3 units on either side.

II. Rapid sand filters (gravity type):

Purpose: The great *disadvantage* of a slow sand filter is that it requires considerable space for its installation. This requirement makes it uneconomical for places where land values are high. As seen in problem 1, the area required for slow sand filter, only for a moderate town of 15000 population, works out to be 1000 m² and with future expansion, other additional equipment, etc., the area required for water supply project would be about 2000 m² or so.

and the working of filter is seriously disturbed. This phenomena is known as the *air binding* as air binds the filter and stops its working. The rate of filtration is consequently greatly reduced.

In case of rapid sand filters, the allowable loss of head is about 3 m to 3.50 m and the allowable negative head is about 1200 mm. The filter is to be washed when this limit of the allowable loss of head has been reached. It is usually cleaned after 2 to 3 days.

Troubles in operation: Following *two* troubles are generally encountered in operating rapid sand filters:

(1) Mud balls

(2) Cracking of filters.

(1) *Mud balls:* The mud balls are generally formed near the top of filter media. They may even be formed and distributed throughout the filter. The mud balls are caused due to insufficient washing of sand grains. The gelatinous film formed during filtration is not separated out from sand grains during washing. The mud balls interfere with the normal working of the filter and their size is about 25 mm to 50 mm.

(2) *Cracking of filters:* The fine material contained in the top layer of filter shrinks and this shrinkage tends to form cracks in the filter. These cracks are prominent near wall junctions.

To remove these troubles, the following *remedies* are adopted:

(1) The mud balls are broken with the help of rakers or some such equipment.

(2) The working of filter is carried out with high velocity of water.

(3) The damaged portion of filter media is replaced.

Rate of filtration: The chief *advantage* of a rapid sand filter is that its rate of filtration is very high. It is about 3000 to 6000 litres per hour per m^2 of filter area. The high rate of filtration results in considerable saving of space for the installation of filter.

Efficiency of rapid sand filters: The efficiency of rapid sand-filters is as follows:

(1) *Bacterial load:* The rapid sand filters are less effective in the removal of bacterial load. It is expected that they remove about 80 to 90 per cent of bacterial impurity present in water.

(2) *Colour:* The rapid sand filters are highly efficient in colour removal and the intensity of colour can be brought down below 10 on cobalt scale.

(3) *Turbidity:* The rapid sand filters can remove turbidity to the extent of 35 to 40 p.p.m. As water entering rapid sand filter is invariably given the treatment in coagulation sedimentation tank, it possesses less turbidity. This turbidity is easily brought down to the permissible limits by rapid sand filters.

Problem 8-2.

Find the area of rapid sand filters required for a town having a population of 80000 with an average rate of demand as 200 litres per head per day.

Solution:

Maximum daily demand $= (80000 \times 200 \times 1.50)$

$= 24000000$ litres.

Assuming a rate of filtration as 5000 litres per hour per m^2 of filter area,

$$\text{area of filter required} = \frac{24000000}{5000 \times 24} = 200\ m^2.$$

Let the size of one unit be 8 m x 5 m. Then provide 6 such units of rapid sand filters including one unit as standby. The units may be arranged in series with 3 units on either side.

Problem 8-3.

A rapid sand filter is required to treat a flow of 0.50 m^3 per second. The filtration rate is 120 m^3 per day per m^2 of filter area and it is provided that the filtration rate with one filter washing is not to exceed 150 m^3 per day per m^2 of filter area. Determine the number of units and the area of each unit to satisfy these conditions.

Each filter is washed for 5 minutes every 24 hours at a wash rate of 10 mm per second per m^2 of filter area. The filter remains out of operation for a total interval of 30 minutes per day. Calculate the percentage of filter output used for washing.

Solution:

Maximum flow rate $= (0.5 \times 60 \times 60 \times 24)\ m^3$ per day

$= 43200\ m^3$ per day.

Note: The rate of filtration can also be expressed as m^3 per day per m^2 of filter area because of the relation 1 m^3 = 1000 litres.

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Thus 120 m^3 per day per m^2 of filter area is equivalent to $\left(\frac{120 \times 1000}{24}\right)$

= 5000 litres per hour per m^2 of filter area.

$$\left.\begin{array}{l}\text{Filter area on the basis}\\ \text{of maximum filtration rate}\end{array}\right\} = \frac{43200}{150} = 288 \text{ m}^2.$$

$$\left.\begin{array}{l}\text{Filter area on the basis}\\ \text{of minimum filtration rate}\end{array}\right\} = \frac{43200}{120} = 360 \text{ m}^2.$$

$$\text{Area of one filter unit} = (360 - 288) = 72 \text{ m}^2.$$

$$\text{Total nos. of filters} = \frac{\text{Maximum filter area}}{\text{Area of one unit}} = \frac{360}{72} = 5.$$

Now, each unit of filter is working at the filtration rate of 120 m^3 per day per m^2 of filter area and the operation of filter is out of order for a period of 30 minutes.

Hence the total working period per day of each filter is

$$(24 - 0.5) = 23.5 \text{ hours.}$$

$$\left.\begin{array}{l}\text{Output of each unit}\\ \text{per day}\end{array}\right\} = \text{Area} \times \text{Filtration Rate} \times \frac{\text{Working period}}{24}$$

$$= \left(72 \times 120 \times \frac{23.5}{24}\right)$$

$$= 8460 \text{ m}^3.$$

$$\text{Wash rate} = 10 \text{ mm per second per m}^2 \text{ of filter area}$$

$$= (10 \times 10^{-3} \times 60) \text{ m per minute per m}^2 \text{ of filter area.}$$

$$\text{Washing period} = 5 \text{ minutes}$$

$$\therefore \left.\begin{array}{l}\text{Wash-water required}\\ \text{per day}\end{array}\right\} = \text{Area} \times \text{Wash rate} \times \text{Washing period}$$

$$= 72 \times (10 \times 10^{-3} \times 60) \times 5$$

$$= 216 \text{ m}^3.$$

$$\left.\begin{array}{l}\text{Percentage of filter out-}\\ \text{put used for washing}\end{array}\right\} = \frac{\text{Wash-water required}}{\text{Output of each unit}} \times 100$$

$$= \frac{216}{8460} \times 100$$

$$= 2.55\%.$$

Comparison between slow sand filters and rapid sand filters (gravity type):

We have so far discussed at length slow sand filters and rapid sand filters (gravity type) separately. In order to bring out the points of differences between similar items of slow sand filters and rapid sand filters, a chart as shown in table 8-1, is prepared.

TABLE 8-1

DIFFERENCE BETWEEN TWO TYPES OF FILTERS

No.	Item	Slow sand filters	Rapid sand filters (gravity type)
1	Base material of gravel	Varies from 3 to 65 mm in size and 300 to 750 mm in depth.	Varies from 3 to 40 mm in size and 600 to 900 mm in depth.
2	Coagulation	Not required.	Essential.
3	Compactness	Requires large area for its installation.	Requires small area for its installation.
4	Construction	Simple.	Complicated as underdrainage system is to be properly designed and constructed.
5	Cost of operation	Low.	High.
6	Depreciation of plant	Low.	High.
7	Economy	High initial cost of both, land and material.	Cheap and quite economical.
8	Efficiency	Very efficient in the removal of bacteria but less efficient in the removal of colour and turbidity.	Less efficient in the removal of bacteria but more efficient in the removal of colour and turbidity.
9	Filter media of sand	Effective size varies from 0.20 to 0.30 mm and uniformity coefficient is about 2 to 3.	Effective size varies from 0.35 to 0.60 mm and uniformity coefficient is about 1.2 to 1.7.
10	Flexibility	Not flexible for meeting variations in demand.	Quite flexible for reasonable fluctuations in demand.
11	Loss of head	150 mm to 750 mm.	3 m to 3.50 m.
12	Method of cleaning	Scraping of top layer of 15 mm to 25 mm thickness. Long and laborious method.	Agitation and back-washing with or without the help of compressed air. Short and speedy method.
13	Period of cleaning	1 to 3 months.	2 to 3 days.
14	Rate of filtration	100 to 200 litres per hour per m^2 of filter area.	3000 to 6000 liters per hour per m^2 of filter area.
15	Skilled supervision	Not essential.	Essential.
16	Suitability	The filter can be constructed of local labour and material. It is suitable for small towns and villages where land is cheaply available.	It is suitable for big cities where land cost is high and variation in demand of water is considerable.

(6) It should take only reasonable time in killing the harmful pathogenic organisms at normal temperature.)

It has been universally recognised that the chlorine is an ideal material for the disinfection for treating water on a large scale. The disinfection at present therefore is mainly carried out by chlorination. However, there are other minor methods of disinfection. We will discuss these methods before we take up the discussion on chlorination.

Theory of disinfection:

The rate of kill in general is expressed by the Chick's law as follows:

$$\frac{dN}{dt} = -KN_t \qquad \ldots (1)$$

where K = Reaction rate constant for particular disinfectant

N = Number of viable organisms

N_t = Number of organisms at any time t.

Integrating equation (1),

$$\int \frac{dN}{dt} = -\int KN_t$$

$$\therefore \int \frac{dN}{N_t} = -\int K\,dt$$

$$\therefore \log_e N_t = -Kt + C \qquad \ldots (2)$$

At $t = 0$, $N_t = N_o$ = Number of organisms initially.

$$\therefore \log_e N_o = C.$$

$\therefore$ Equation (2) will be,

$$\log_e N_t = -Kt + \log_e N_o$$

$$\therefore \log_e \frac{N_t}{N_o} = -Kt$$

$$\therefore t = \frac{1}{K}\log_e \frac{N_o}{N_t}$$

Changing to base 10,

$$t = \frac{1}{K}\log_{10} \frac{N_o}{N_t}$$

Since N_t will never reach zero, it is the usual practice to specify kill as a percentage e.g. 99.7%.

The most popular disinfectant of water is chlorine and it does [not] obey the equation mentioned above. But it follows the relation [gi]ven by the expression:

$$\frac{dN}{dt} = -KN_t t$$

$$\int \frac{dN}{N_t} = \int -Kt\,dt$$

$$\log_e N_t = -\frac{Kt^2}{2} + C.$$

$$t = 0,\ N_t = N_o$$

$$C = \log_e N_o$$

$$\log_e \frac{N_t}{N_o} = -\frac{Kt^2}{2}$$

$$t^2 = \frac{2}{K}\log_{10}\frac{N_o}{N_t}$$

$$t = \sqrt{\frac{2}{K}\log_{10}\frac{N_o}{N_t}}$$

[Pro]blem 9-1.

To obtain 99.70% ~~kill~~ of bacteria, the ozone is to be used in [w]ater with a residual of 0.6 mg/l. The reaction constant under these [c]onditions is 3×10^{-2} per second. Calculate the contact time period.

[S]olution:

Now, 99.70% of bacteria are killed i.e. 0.3% of bacteria remains in the water after ozonisation.

In the water, the concentration of bacteria is 100 mg/l and after ozonisation, it is 0.30 mg/l. But residual of 0.60 mg/l is given.

N_o = Number of organisms initially

$$= 100 \times \frac{0.60}{0.30} = 200 \text{ mg/l}$$

N_t = Number of organisms at time t

$$= 0.60 \text{ mg/l}.$$

Now,
$$t = \frac{1}{K}\log_{10}\frac{N_o}{N_t}$$

$$= \frac{1}{3 \times 10^{-2}}\log_{10}\frac{200}{0.60} = 84 \text{ seconds.}$$

The process of hypo-chlorination is carried out at present by commercial compounds such as HTH (high test hypochlorite), Pittchlor, Pittcide, Hoodchlor, Perchloron, etc. instead of bleaching powder.

These compounds have a chlorine content of about 65 to 70 per cent. The other *advantages* of such compounds are as follows:

(i) They are available in powder form in small packings.

(ii) Their chlorine content does not decrease with storage.

(iii) They can be applied to the water in dry condition or as solution.

The process of hypo-chlorination is not adopted for public water supply projects. It may however be adopted for small installations such as swimming pools, etc.

Problem 9-3.

A town having population of about 50000 is to be supplied water at the rate of 150 litres per capita per day. The disinfection of water is to be carried out with bleaching powder containing 30 per cent of active chlorine. If the chlorine dose required for infection is 0.3 p.p.m. or 0.3 mg/l, calculate the quantity of bleaching powder per year.

Solution:

$$\text{Total requirement of water} = (150 \times 50000) \text{ litres/day} = 7.5 \times 10^6 \text{ litres/day}$$

$$\text{Chlorine dose required for disinfection} = 0.3 \text{ mg/l}$$

$$\therefore \text{Quantity of chlorine required} = 0.3 \times \frac{7.5 \times 10^6}{10^6} \text{ kg/day} = 2.25 \text{ kg/day.}$$

Now, the bleaching powder contains 30 per cent of active chlorine. It means that 30 kg of chlorine is available from 100 kg of bleaching powder.

$$\therefore \text{Quantity of bleaching powder required} = \frac{2.25}{30} \times 100 \text{ kg/day} = 7.5 \text{ kg/day.}$$

$$\text{Requirement of bleaching powder per year} = (365 \times 7.5) \text{ kg} = 2737.5 \text{ kg, say } 2750 \text{ kg.}$$

(2) *Chloramines:* It is found that chlorine alone is not stable in water. But when it is mixed in water with ammonia, it forms compounds,

known as the *chloramines*. These compounds are stable in water and are found to possess disinfecting properties. They also remove odour from water to a certain extent. Following are the reactions involved:

$$H_2O + Cl_2 = HCl + HOCl$$

$$NH_3 + HOCl = H_2O + NH_2Cl \text{ (Monochloramine)}$$

$$NH_2Cl + HOCl = H_2O + NHCl_2 \text{ (Dichloramine)}$$

$$NHCl_2 + HOCl = H_2O + NCl_3 \text{ (Nitrogen trichloride).}$$

The monochloramine NH_2Cl predominates at pH value over 7.50; dichloramine $NHCl_2$ at pH of 5 to 6.50; and nitrogen trichloride NCl_3 at pH below 4.40.

The ammonia is added to the water generally in the ratio of one-half to one-fourth of chlorine amount. The ammonia dissolves quickly in water. But it does not diffuse easily in water. Hence it is necessary to mix it with the help of mechanical means. The ammonia may be used in the form of gas or as solution or as ammonium sulphate or as ammonium chloride.

One precaution is necessary when water is treated with the chloramines. The water, after treatment is completed, should be supplied to the consumers after an interval of about 20 to 60 minutes.

Following are the *advantages* of adding ammonia to the water before the addition of chlorine:

(i) It is more effective than chlorine alone.

(ii) The effect produced lasts for a longer period.

(iii) The quantity of chlorine required becomes less.

(iv) There is less irritation to nose and eyes. Hence it is more useful for treating water for swimming pools.

(v) There is no danger of overdose.

(3) *Free chlorine gas:* The chlorine can be applied in gaseous form or in liquid form. In the former case, the chlorine gas is dissolved in a small quantity of water and then the solution is fed to the water under treatment of disinfection. In the latter case, the chlorine gas is converted into liquid form by applying pressure on it. It is found that the chlorine gas when subjected to a pressure of about 0.7 N/mm^2 is converted into liquid. The pressure will however depend on the temperature.

The liquid chlorine is stored in pressure cylinders or drums and is supplied for use. The liquid chlorine is highly corrosive when damp by the simple method of aeration or exposure to sunshine. But this is not a practical or desirable measure for final waters which are intended for public supply purposes.

Problem 9-4.

The quantity of chlorine used to treat 20000 m^3 of water per day is 8 kg. The residual chlorine after contact period of 10 minutes is found to be 0.20 mg/l. Calculate the dosage in mg/l and the chlorine demand of the water.

Solution:

Water treated per day = 20000 m^3

= 20000 × 10^3 litres

= 20 × 10^6 ml.

Chlorine consumed per day = 8 kg = 8 × 10^6 mg

$\therefore$ Chlorine used per litre of water = $\frac{8 \times 10^6}{20 \times 10^6}$ = 0.40 mg/l.

The given chlorine dosage is therefore 0.40 mg/1.

Now, Residual chlorine left = 0.20 mg/l.

Hence the actual chlorine dosage which has reacted in water i.e. the chlorine demand of water is equal to (0.40 − 0.20) = 0.20 mg/l.

Problem 9-5.

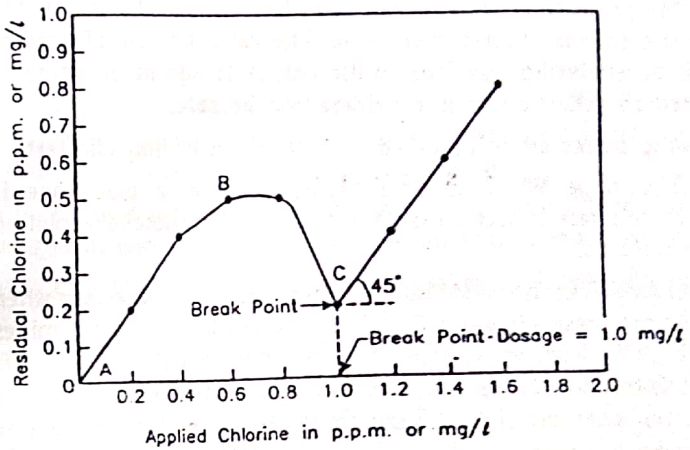
The results obtained from chlorine demand test on samples of raw water are as follows:

Sample no.	Chlorine dosage in mg/l	Residual chlorine after 10 minutes in mg/l
1	0.20	0.19
2	0.40	0.36
3	0.60	0.50
4	0.80	0.48
5	1.00	0.20
6	1.20	0.40
7	1.40	0.60
8	1.60	0.80

Draw the chlorine demand curve and find out the break point dosage. Also calculate the chlorine demand at a dosage of 1.20 mg/l.

Solution:

The chlorine demand curve or in the strict sense, the chlorine residual curve is drawn as shown in fig. 9-2. The break point is represented by point C and hence the break point dosage is 1.00 mg/l.



Break point chlorination

FIG. 9-2

Then, Chlorine demand at break point = Applied chlorine − Residual chlorine

= (1.00 − 0.20) = 0.80 mg/l.

The chlorine demand after the break point becomes constant and all added chlorine subsequently appears as free chlorine.

$\therefore$ Chlorine demand at a dosage of 1.20 mg/l = 0.80 mg/l.

Note. The result is in line with given data of chlorine residual of 0.40 mg/l with a dosage of 1.20 mg/l giving the chlorine demand of (1.20 − 0.40) = 0.80 mg/l.

Tests for chlorine:

Following are the usual tests which are applied for determining the amount of free and combined chlorine in a sample of water:

(1) Orthotolidin test

(2) Starch-iodide test.

The orthotolidin test is very common and as the starch-iodide test is costly and time consuming, it is adopted only when the orthotolidin test becomes unsuitable.

Problem 9-2.

Compare the contact times necessary to obtain 99.99% kill of bacteria in water under the following conditions:

(1) Free chlorine residual of 0.15 mg/l and $K = 1.1 \times 10^{-2}$ per second.

(2) Combined chlorine residual of 2 mg/l with $K = 1.2 \times 10^{-5}$ per second.

Solution:

(1) *Free chlorine residual:*

The bacteria to be killed are 99.99% i.e. 0.01% bacteria remain in water.

$\therefore$ 0.01 mg/l bacteria remain from 100 mg/l of its concentration.

$\therefore$ 0.15 mg/l bacteria remain from $100 \times \frac{0.15}{0.01} = 1500$ mg/l concentration.

Thus, $N_o = 1500$ mg/l;

$N_t = 0.15$ mg/l

and $K = 1.1 \times 10^{-2}$ per second.

Then,
$$t = \sqrt{\frac{2}{K} \log_{10} \frac{N_o}{N_t}} = \sqrt{\frac{2}{1.1 \times 10^{-2}} \log_{10} \frac{1500}{0.15}}$$
$= 26.97$, say 27 seconds.

(2) *Combined chlorine residual:*

In this case,

$N_t = 2.00$ mg/l

$K = 1.2 \times 10^{-5}$ per second

$$N_o = 100 \times \frac{2.00}{0.01}$$
$= 20000$ mg/l.

Then,
$$t = \sqrt{\frac{2}{K} \log_{10} \frac{N_o}{N_t}} = \sqrt{\frac{2}{1.2 \times 10^{-5}} \log_{10} \frac{20000}{2.00}}$$
$= 816.50$ seconds.

Minor methods of disinfection:

Following are the *seven* minor methods of disinfection:

(1) Boiling method
(2) Excess lime treatment
(3) Iodine and bromine treatment
(4) Ozone treatment
(5) Potassium permanganate
(6) Silver treatment
(7) Ultra-violet ray treatment.

Each of the above minor method of disinfection will now be briefly described.

(1) ***Boiling method:*** When the water is boiled above a certain temperature, the bacteria are killed. The boiling of water is the most effective method of disinfection. But to boil water on a large scale is impracticable. However, in case of epidemic, the consumers may be advised to boil the water before use for drinking and domestic purposes as the boiling of water is an absolute safeguard against infection by bacteria of the water-borne diseases. Most of these bacteria are destroyed when the water has attained a temperature of about 80°C, but it is safer to raise the temperature to boiling. However it will only be sufficient to bring the water to the boiling level and prolonged boiling is unnecessary and wasteful.

(2) ***Excess lime treatment:*** The treatment of lime is given to the water for the removal of dissolved salts. But it was found that if excess lime is added to the water, it will also work in addition as a disinfecting material. The action of excess lime is based on the fact that lime increases pH value of water. The extreme acidity or alkalinity is detrimental to bacteria. Hence, when pH value of water is about 9.50 or so, the bacteria can be removed to the extent of 99.93 or 100 per cent. But when this treatment is adopted for disinfection, the excess lime is to be removed by any suitable method of recarbonation after disinfection.

(3) ***Iodine and bromine treatment:*** When water is treated with iodine or bromine, it is disinfected. The dosage of iodine or bromine is about 8 p.p.m. and the contact period with water is about 5 minutes. The iodine and bromine are also available in the form of pellets or small pills. The use of iodine or bromine is limited to small water supplies such as swimming pools, troops of army, private plants, etc.

$$^{P}f = {}^{P}P(1+r)^{n} \qquad (2.7)$$

where ^{P}f = future population

^{P}P = present population

r = probable rate of yearly or per decade increase

n = number of years to be considered.

When the population data of the past decades are available the average value of r can be computed from the following expression

$$r = \sqrt[n]{\frac{P_2}{P_1}} - 1$$

where P_1 and P_2 are the population data at two dates of n number of years.

This method should be used carefully as it may give erroneously high results when applied to young and rapidly advancing cities having expansion of short duration only. This method is also known as the Geometrical Progression method.

Example : The population of a city was 124,000 in 1960 and 156,000 in 1970. (a) What was the annual rate of increase? (b) What will be the probable population in 1980?

Solution: (a) Here, P_1= 124,000, P_2= 156,000 and n= 10.

$$r = \sqrt[10]{\frac{156{,}000}{124{,}000}} - 1 = 1.023 - 1 = 0.023$$

(b) In this case, P_p =156,000, r=0.023, and n=20

P_f =156.000 $(1+0.023)^{20}$=246.000 Ans:

(7) Least Square Parabola Method: In this method, the population-time curve is assumed to be parabolic.

Let the variables X and Y denote respectively the year and the population during that year. The equation of the least square parabola fitting the data (census data for a number of decades) is

$$Y=a+bX+cX^2 \quad \text{.......} \quad (2.9)$$

where a, b and c are constants and are to be found from the following normal equations by applying the actual data :

$$\Sigma Y=aN+b\Sigma X+c\Sigma X^2 \quad \text{.......} \quad (2.10)$$

$$\Sigma XY=a\Sigma X+b\Sigma X^2+c\Sigma X^3 \quad \text{.......} \quad (2.11)$$

$$\Sigma X^2Y=a\Sigma X^2 + b\Sigma X^3 +c\Sigma X^4 \quad \text{.......} \quad (2\ 12)$$

where N = number of observations or sets of data.

If the population data of the city under study for a number of decades are known, then by solving the simultaneous Eqs. 2.10, 2.11 and 2.12 with the given data, the values of the constants a, b, and c in the Eq. 2.9 can be computed, and the desired equation of the least square parabola can be found out. Then from this equation, the population at any future date can be computed.

$$P=\frac{HQ}{3.960} \quad \text{.....................} (5.1)$$

in which P= theoretical horsepower required to operate the pump (W.H.P. = Water Horse Power);

H = total lift or head of the pump;

Q = volume of water to be pumped, gpm.

If the head to be pumped against is given in psi, the formula becomes.

$$P=\frac{QP}{1.715} \quad \text{.........................} (5.2)$$

in which P = intensity of pressure, psi.

The actual horsepower required depends on the efficiency of the pump and may be found by relation.

$$P_1=\frac{P}{E} \quad \text{.........................} (5.3)$$

in which P_1 = actual horsepower required to operate the pump (B.H. P = Break Horse Power)

E = efficiency of the pump.

PROBLEM 1 : It is required to pump water at the rate of 6.750 gpm from a reservoir whose surface is at an elevation of 180 ft. to a tank whose bottom is at an elevation of 372 ft. The pump is placed at an elevation σf 192 ft.. the diameter of the suction pipe is 30 inch. the length of the pipe from the pump to the tank is 290ft. and the estimated size of this pipe is 24 inch. The sum of the minor head losses in the suction and discharge pipe may be taken as 1.5 ft. If the maximum

depth of water in the tank is to be 25 ft., what is the required horsepower of a pump for which the overall efficiency is 67 per cent ?

Assume head loss due to friction in 290 ft.+1.5 ft.

Neglect all other head losses.

Solution :

Elevation of water surface in the tank = 372 + 25=397 ft.

Discharge lift or the vertical distance from the centre of the pump to that surface =397 -192 =205 ft.

Suction lift, or the vertical distance from the water surface in the reservoir to the centre of the pump = 192-180 = 12 ft.

Since that pump is above the water surface this lift a positive.

Total head H = 205+12+1.5+1.5 = 220 ft.

$$P=\frac{HQ}{3.960}=\frac{220\times 6.750}{3.960}=375(\text{W.H.P})$$

$$P_1=\frac{375}{0.67}=560(BHP)Ans$$

PROBLEM 2 : Design a suitable set of pumping unit to deliver 4.50.000 gph from an intake well of a river bank to the treatment plant. Total length of rising main from the intake well to the treatment plant is 800ft. and the static head is 60 ft-Design also the cast iron main.

Assume: Velocity of water = 12 fps

friction factor = 0.0075

efficiency = 70%

Solation.

$Q = 4{,}50{,}000 \div 60 = 7.500$ gpm

again $Q = \dfrac{4.50.000}{60 \times 60 \times 6.24} = 20$ cfs

Cross-sectional area $= \dfrac{20}{12} = 1.667$ sq. ft.

$\dfrac{\pi d^2}{4} = 1.667$ (d = diameter of pipe)

$d = \sqrt{\dfrac{1.667 \times 4}{\pi}} = 1.5$ ft $= 18$ inch

Frictional head loss h_f

$= \dfrac{4flv^2}{2gd} = \dfrac{4 \times 0.0075 \times 800 \times (12)^2}{2 \times 32.2 \times 1.5} = 36$ft.

Velocity head, $h_v = v^2/2g = (12)^2/2 \times 32.2 = 2.24$ft.

Total head $H = h_s + h_f + h_v = 60 + 36 + 2.24 = 98.24$ ft

$P = \dfrac{HQ}{3{,}960} = \dfrac{98.24 \times 7.500}{3.960} = 188$ (WHP)

$P_1 = \dfrac{180}{0.70} = 265$ (BHP) Ans.

PROBLEM 3 : Water is supplied from an impounding reservoir 30 miles away to a service reservoir near the town. A cast iron main is to be designed to supply 425 mgd. Loss of head due to friction in the pipe is estimated to be 300 ft. All other head looses are neglected. What size cast iron pipe would you use?

Assume $f = 0.0075$

Solution :

$h_f = \dfrac{4 \times 0.0075 \times (30 \times 5280) \times v^2}{2 \times 32.2 \times d} = 300$

$\dfrac{v^2}{d} = 4.06 \therefore V^2 = 4.06 \times d$....1)

Again. $Q = 425$ mgd $= 425 \times 1.547 = 787$ cfs

[1 mgd = 1.547 cfs]

$Q = av = \dfrac{\pi d^2}{4} \times v \therefore v = \dfrac{4Q}{\pi d}$

$\therefore v^2 = \dfrac{16Q^2}{(\pi d)^2}$

Substituting the value of v^2 from Eq. 2 to Eq. 1.

$\therefore \dfrac{16Q^2}{\pi^2 d^5} = 4.07$

$d^5 = \dfrac{16 \times (787)^2}{\pi^2 \times 4.07^2} = 246{,}800$

$\therefore d = 11.98 \simeq 12$ft. Ans.

Problem 4 : Design a pumping unit capable of lifting 5 mgd of water from an intake well to the treatment plant against a static head of 60 ft ; length of suction main is 120 ft and that of rising main is 400 ft. The pump will work in two shifts of eight hours each.

Assume : velocity of flow = 6 fps

friction factor = 0.01

efficiency = 75%

Solution :

Total length of pips = 120+400 = 520 ft.

Discharge. Q =5mgd = $\frac{5\times10^6}{6.24} = 8\times10^5$ cu ft/day

Since total pumping time is 16 hrs/day.

Pumping capacity = $\frac{8\times10^5}{16} = 50.000$ cuft/hr.

$= \frac{50.000}{60\times60} = 13.9$ cfs

$Q = \frac{\pi d^2}{4} = 13.9$

$\therefore d = \sqrt{\frac{4\times13.9}{\pi\times6}} = 1.7 \simeq 21.75$ft =21 inch

$h_f = \frac{4\times0.01\times520\times6^2}{2\times32.2\times1.75} = 6.7\,ft$

$h_v = \frac{6^2}{2\times32.2} = 0.56\,ft$

$H = h_s + h_f + h_v = 60+6.7+0.56 = 67.26$ft

$Q = \frac{50.00.000}{16\times10} = 5.220\,gpm$

$P = \frac{HQ}{3,960} = \frac{67.26\times5,220}{3,960} = 88.5$ (WHP)

$P_1 = \frac{P}{E} = \frac{88.5}{0.75} = 118$ (BHP) Ans.

Problem 5 : Design the transmission main and the pumping unit from the following data :

Water supply rate = 40 gpcd

Estimated population=85.000

Ground R. L. = at the pum house = 102.50 ft.

Treatment plant R. L. = 193.00 ft.

Velocity through pipes = 8 fps

Pumping time = 10 hrs. daily

Total length of pipe = 3.500 ft.

Friction factor = 0.01

Efficiency = 65%

Solution :

Total water required = 40×85.000 =34,00,000 gpd

$= \frac{34,00,000}{6.24} = 5.45\times10^5$ cuft/day

Pumping rate = $\frac{5.45\times10^5}{10} = 5.45\times10^5$ cuft/day

$= \frac{5.45\times10^5}{60\times60} = 15.15$ cfs

$Q = \frac{\pi d^2}{4} = \times v = 15.15$

$\therefore d = \sqrt{\frac{4\times15.15}{\pi\times8}} = 1.56$ ft.

Use a 21 inch diameter pipe d = 1.75 ft.

Static head = 193.00 -102.50 = 90.50

Friction head = $\frac{4\times0.01\times3.00\times8^2}{2\times32.2\times1.75}$ =80.0 ft

Velocity head= $\frac{v8^2}{2\times32.2}$ 1.0 ft

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Total head, H = 90.5+80.0+1.0=171.5ft

Discharge, $Q = \frac{34,00,000}{10 \times 60} = 5,667$ gpm

$\therefore P = \frac{HQ}{3,960} = \frac{171.5 \times 5667}{3,960} = 246$ (WHP)

$\therefore P_1 = \frac{P}{E} = \frac{246}{0.65} = 380$ (BHP) Ans.

QUESTIONS

1. State the purposes or pumps and pumping machinery in water supply systems.

   Water supply to a small town with ultimate population of one lakh (1,00,000) supplied with 60 gped has to be arranged from a river flowing nearby. Design the economical section of the rising main and the necessary pumping unit from the following data;

Static head	60 ft
Total length of pipe	520 ft.
Coefficient of friction	0.01
Velocity of water in the pipe	6 ft/sec
Pump efficiency	70%

   The pumps will work in two shifts of six hours each in a day. (BUET, 1972)

2. Write the characteristics of centrifugal pumps. Design a suitable pumping unit and the size of the transmission main delivering water from a source 500 yds. away to a treatment plant or a small town having the design. Population of 2.5 millions supplied with 50 gpcd against a frictional head of 70ft. The pump will operate only 8 hrs. in a day. Take pump efficiency =65% and f= 0.01. Neglect all other head losses. (BUET, '71).

3. State the general considerations for pumping installation. What is the theoretical horsepower required for a pump to

F-14

raise 1800 gpm of water against a total head of 150 ft. including all losses ?

Which type of pump do you suggest and why? (BUET, '68)

4. What is the total lift of a pump ?

Design a pumping unit to transmit water from a source to a treatment plant of a snail town Having ultimate oppulation of 80,000 supplied with 50 gped of water.

Given : R. L. of the ground at pump hours = 98.20 ft.

R. L. of the entry site ground to a treatment plant = 154.60ft.

Length of pipe line = 2000ft.

Velocity of water through the pipe = 8 fps.

Friction factor = 0.0075

Pump efficiency = 65% (BUET, '67)

5. A multistoried building requires 15,000 gpd of water. The water will be supplied by a 3 inch diameter well. Design a suitable pumping unit from the following data : Suction head =12 ft. Delivery head= 180 ft. Size of the suction and delivery pipe = 2 inch Velocity of water through pipes = 6 ft/sec. Friction factor = 0.01 Assume reasonable values of data not supplied. (BUET, 1972)

6. Write explanatory notes on :

Deep well turbine Pump, Air Lift Pump, Submersible pump. Suction lift.

A typical rectangular sedimentation tank is shown in Fig. 8.3.

Example 1 : One million gallons of water per day (1 mgd) passes through a sedimentation tank which is 20 ft. wide, 50 ft. long, and 10 ft. deep, (a) Find the detention time for this basin. (b) What is the average velocity of flow through the basin? (e) If the suspended solids content of the water averages 40 ppm, what weight of dry solids will be deposited every 24 hours assuming 75% removal in the basin, (d) What is the over flow rate?

Solution : (a) Detention time = $\dfrac{\text{Volume of tank}}{\text{Flow per unit time}}$

$$= \frac{20 \times 50 \times 10}{1 \times 106} \times 7.48 \times 24$$

= 1.8 hrs.

(b) Velocity, $V_S = \dfrac{Q}{A} = \dfrac{1 \times 10^6}{20 \times 10 \times 7.48 \times 60 \times 60 \times 24}$

= 0.0077 fps.

(c) Totat solids deposited $= \dfrac{40 \times 10^6}{10_6} \times 8.34 \times 0.75$

= 250 lb/day

(d) Over flow rate $= \dfrac{Q}{BL} = \dfrac{1 \times 10^6}{50 \times 20} = 1000 gpd / sft.$

Example 2 : A rectangular sedimentation tank is to treat 4,00,00 gpd of raw water. The detention period is to be 4 hours, 'the velocity of flow 3 inch per minute and the depth of water and sediment is 14 ft. If an allowance of 4 ft. for sediment is made, what should be the length and width of the sedimentation tank?

Solution :

Velocity of flow = 3 inch/min–0.25 fpm

Total length of the tank =240 × 0.25×60 ft.

Volume of water to be treated in

$$4 \text{ hours} = \frac{4,00,000 \times 4}{24} = 66,700 \text{ gallons}$$

$$= 8,920 \text{ cft.}$$

Cross-sectional area of the tank

$$= 8,920/60 = 148.7 \text{ sft.}$$

Effective depth of the tank- 14-4 =10 ft.

Width of the tank= 148.7/10= 14.87= 15 ft.

Circular Sedimentation Tank: Sometimes, circular sedimentation tanks are used, as shown in Fig. 8.4. The diameter of the circular tank depends on the overflow rate, volume and depth. As for circular tanks, equipment is made in certain standard sizes, generally, the tank bottom is cone-shaped with a slope of about 1 inch vertical in 1ft. horizontal. With these conditions as basin the volume of circular sedimention tank is found by the formula

$$V=D^2 (0.011D+0.786H)$$

in which V = volume of circular tank in cft

D = diameter of the tank in ft

The filter beds are usually rectangular in size, having the ratio of length and breadth as 3 : 2.

Example 1 : A rapid sand filterplant is to be designed for a capacity of 6 mgd. What should be the number and size of the filter units? What should be the percentage of filtered water required to wash the filter beds? What should be the capacity of wash water tank?

Assume: (1) Rate of filtration : 2 gpm per sq. ft.

(2) Rate of washing : 15 gpm per sq. ft.

(3) Length of filter run : 24 hrs including 5 min for washing the filter bed and 10 min for resettlement of sands.

Solution :

(a) The plant will operate only 23 hr and 45 min.

Filtration rate = 2×60×23.75=2850 gallons per day per sq ft.

Filter area required = (6,000,000/2,850)=2,100 sq ft.

No. of units. $N=2.7\sqrt{Q}=2.7\sqrt{6}=7$

Area of each units =2100/7 = 300 sq ft.

Size of each unit = 20' × 15'

(b) Total wash water required=15×2100×5= 157,500 gallons per day

Percentage of filtered water required

=(157,500/6,000,000) ×100 = 2.625%

(c) Capacity of wash water tank-1,57,500 gallons.

Example 2 : A filter bed has an area of 360 sq ft. If the washing for 5 min at the rate of 24 inch per min is contemplated, how much wash water will be required?

Solution : 24 inch rise of water is attained by applying the wash water at the rate of 15 gpm per sq ft of filter area. Wash water requirement-15 × 360×5=27,000 gallons.

Example 3 : A rapid sand filter operating at 2 gpm per sq ft needs washing after 24 hr of operation. The filter has an area of 350 sq ft and it needs washing at the rate of 15 gpm per sq ft for 5 min. The time required for resettlement of sand is 10 minutes.

What per cent of the water that is filtered will be required for wash water.

Solution : Filtration rate = 2×60 ×23.75=2,850 gpd ped sq ft. Capacity of the plant=2,850×350=997,5000 gpd. Wash water requirement=15×5×350=26,250 gpd percent of filtered water for washing the filter=(26,250/997,500) × 100=2.63

Efficiency of Rapid Sand Filter: With proper pretreatment of the water, rapid sand filters are applicable for treatment of any surface water. Rapid sand filters are very efficient in removing bacteria, colour, odour, turbidity, iron and manganese. The following are the percentage removal of the above mentioned water quality.

within narrow limits of accuracy. To little chlorine is ineffective, too much may cause tastes and odours. Operation of chlorinator should be automatic so that the amount of chlorine fed to water should be proportional to the volume of flow and the chlorine demand of water.

Chlorine Dosage : The amount of chlorine required to be added to the water supply can be determined in the laboratory by adding varying dosages of chlorine to equal proportions of the waste? sample and finding the amount of residual after a contact period of 10-20 minutes.

Example : It is required to disinfect 5,00,000 gpd of water with 0.3 mg/l of chlorine. If bleaching powder is used (which coatains. $33^1/_3$ percent of available chlorine), how many pounds of bleaching powder are needed to treat the daily flow of water ?

Solution : Available chlorine in the bleaching powder is $33^1/_3\%$

∴ 1 lb of chlorine is available in 3 lbs of bleaching powder.

Since 1 mg/l of chlorine = 8.34 lbs of chlorine per million gallons of water.

∴ Amount of bleaching powder required per million gallon = 3 × 8.34=25-02 lbs.

∴ Amount of bleaching powder required for a flow of 500,000 gpd and a chlorine dosage of 0.3 mg/l.

$$= 25.02 \times \frac{500{,}000}{1{,}000{,}000} \times 0.3 = 3.75 \text{ lbs}$$

Minimum chlorine Residuals for drinking water at 20°C

pH Valve	6-7	7-8	8-9	9-10
Free available chlorine, mg/l after 10 mins	0.2	0.2	0.4	0.8
Combined available chlorine, mg/l after 60 mins	1.0	1.3	1.5	1.8

Special Methods : Chlorine is generally applied after all other treatments have been given to the water supply. This may be termed as post chlorination and is the standard treatment at all waterworks. There are, however, other special methods of chlorination to be used depending upon the particular purposes to be gained.

(a) Pre-chlorination is the application of chlorine preceding filtration, either added into the suction pipes of raw water pumps or to the water as it enters the mixing basin. Pre-chlorination reduces bacterial load filters resulting in increased filter runs, and oxidizes excessive organic matter thus removing taste and odour.

(b) Double chlorination is the application of chlorine at two points in the treatment process. It is essentially pre-chlorination with an added treatment to the final effluent from the filters. Advantages claimed with this method of chlorination are (i) decrease in the load on filters, (ii) greate

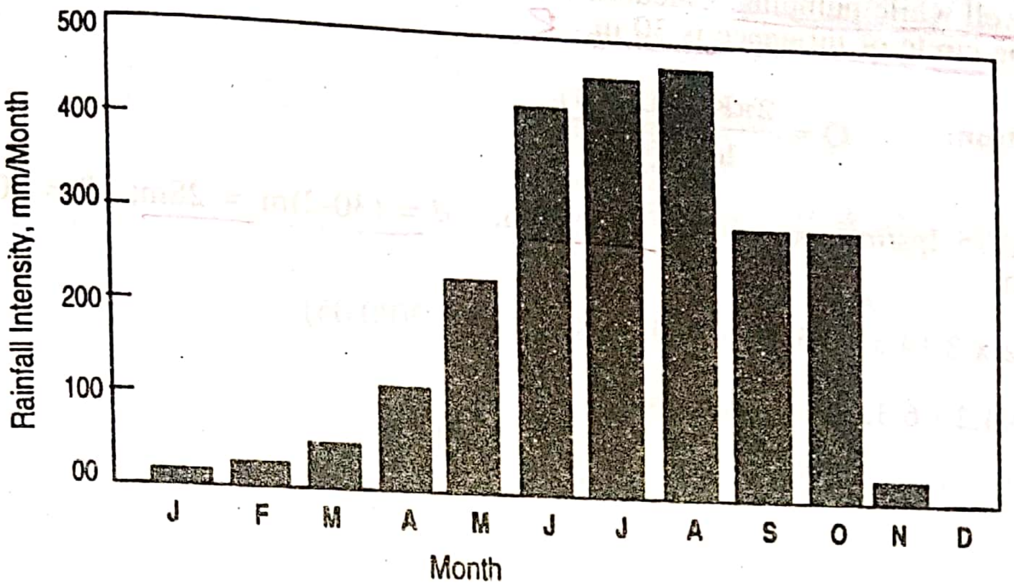

Figure 16.12 Monthly distribution of rainfall in a normal Year in Bangladesh

16.6 WORKED EXAMPLE

Example - 1

A 100 mm diameter tubewell is sunk 35 m below static groundwater level. The depth of water in the tubewell while pumping is 33 m. The radius of drawdown is 30 m and the coefficient of permeability of the aquifer is 0.5 l/s/m². Calculate the probable discharge of the well.

Solution: $Q = \dfrac{\pi K(D^2 - d^2)}{\text{Log}_e(\frac{R}{r})}$

$K = 0.5$ lps/m²; $D = 35$m; $d = 33$m; $R = 30$m; $r = 0.05$m

Substituting the values in the above equation:

$Q = 3.14 \times 0.5 \times (35^2 - 33^2) / \text{Log}_e (30/0.05)$

$Q = 3.14 \times 0.5 \times 136/6.3969 = 33.4$ lps

Example - 2

A 100 mm diameter tubewell is sunk to withdraw water from a 10 m thick confined aquifer having coefficient of permeability equal to 0.75 lps/m². The depth of water below the piezometric level is 30 m and it falls 2 m in the

D D-d
drawdown

tubewell while pumping. Calculate the discharge of the tubewell when the radius of the circle of influence is 30 m. R

Solution: $$Q = \frac{2\pi Km(D-d)}{\log_e(\frac{R}{r})}$$

$K = 0.75$ lps/m^2; $m = 10$m; $D = 30$m; $d = (30\text{-}2)$m $= 28$m; $R = 30$m; $r = 0.05$m

$Q = 2 \times 3.14 \times 0.75 \times 10\ (30 - 28)\ /\ (\log_e 30/0.05)$

$Q = 94.2\ /\ 6.3969 = 14.73$ lps

Questions:

1. Make a comparative statement of different sources of water for water supply in Bangladesh. Mention the importance of groundwater for water supply in Bangladesh.

2. State the hydraulics of groundwater flow in wells. Deduce mathematical expressions for yield of artesian as well as ordinary wells.

3. Describe the aquifer characteristics and groundwater situation in Bangladesh.

4. Explain the problems in groundwater development in Bangladesh.

17 5. A 150 mm diameter tubewell produces 100 lps with a drawdown of 3 m and a circle of influence of 120 m in diameter. The static depth of water in the well is 40 m. Calculate the coefficient of permeability of the aquifer in which the tubewell is sunk.

r Q (D−d) R K d

K = 0.943 lps/m²

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Disinfection of tubewell

Tubewells are contaminated during construction mainly due to the use of contaminated surface waters for sinking. Sometimes contaminating materials are used to stabilize the boreholes against collapse and caving. Tubewells in Bangladesh are also contaminated due to submergence in floods or under storm surges. The contaminated tubewells need disinfection by the following procedure:

(a) Prepare about 50 l of chlorine solution with a chlorine concentration of 50 mg/l (dissolve 0.150 gms of bleaching powder containing 33% chlorine in 1 litre of water).

(b) Open the base of the tubewell and pour the chlorine solution in the pipe slowly. The chlorine solution will fill the pipe and then enter into the aquifer through the strainer.

(c) Dismantle the tubewell and submerge/wash smaller components in the chlorine solution and wipe all the surfaces of larger components with the same chlorine solution. Then reassemble and fix the tubewell.

(d) Wait for at least six hours and then pump out the water until traces of chlorine can be smelled in water produced by the tubewell.

(e) When the pumping of the tubewell is complete, the disinfected tubewell is ready for regular use.

18.6 WORKED EXAMPLES

Example - 1

Find the dimension of a settling tank to treat 45 m^3 of raw water per hour when the overflow rate is 0.5 m/hr, and the detention time is 3 hours.

Solution : Overflow rate or surface loading = Q/LB = 0.5 m/hr

or $45/LB = 0.5$ m/hr

Therefore, $LB = 45/0.5 = 90\ m^2$ (18.4)

Detention time $T = LBH/Q = LBH/45 = 3$ hrs

Or $LBH = 135\ m^3$ (18.5)

From equation (18.4) and (18.5)

$H = 1.5$ m

Let $L = 4$ B

Substituting the value of L in equation (18.4)

$4B^2 = 90$

$B = (90/ 4)^{1/2}$

or $B = 4.74$ m, $L = 90/ 4.74 = 18.97$ m

The size of the settling tank may be taken as 4.75 m x 19 m x 1.5 m

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$0.04\sqrt{Q}$ L : B = 3 : 2

Example - 2

14

A rapid sand filter is to be designed for a capacity of 27,000 m^3/day. What should be the number and size of the units. Calculate the percentage of filtered water required to wash the filter bed and the capacity of the wash water tank.

Assume: Rate of filtration : 5 m^3/hr/m^2
Rate of washing : 35 m^3/hr/m^2
Length of the filter run : 24 hrs. including 5 min. for filter washing and 10 min. for resettlement of sand bed.

T

Solution: Filtration period = 24 hrs. – 15 min. = 23 hrs. 45 mins. = 23.75 hrs.

Filtration rate = 5 x 23.75 = 118.75 m^3/day/m^2
A·2 Filter area required = 27,000/(118.75) = 227.37 m^2 [say 227 m^2]
Number of filter units required = 0.04√ 27,000 = 6.57 [say 7 units]
Area of each unit = 227/7 = 32.42 m^2
W = $\sqrt{\frac{A}{1.5}}$ Width of the filter unit =√ [32.42/(2/3)] = 4.66 ; length = 4.66 x 1.5 = 6.99 ?
[say 7 m]
Actual area provided = 4.66 x 7 x 7 = 228.34
Wash water required = 35 x 5 x 228.34/60 = 665.99 m^3 ≅ 666 m^3
Minimum capacity of wash water tank = 666 m^3
Percentage of filtered water required for washing = 666 x 100/27,000 = 2.47%

Questions:

1. What are the objectives of water purification? Name the methods of water purification indicating their specific application.

2. What are the factors influencing settling velocity of particles in water? Define different types of settling with examples.

3. Show that the efficiency of a settling tank is dependent on surface area, not depth of tank.

4. Calculate the dimension of a rectangular settling tank to treat 100 m^3 of water per hour when the overflow rate is 0.75m/hr and the detention time is 2 hours. 5.8 X 23.1 X 1.5

5. Explain the process of coagulation-sedimentation. How does it differ from plain sedimentation?
Draw neat sketch of a mechanical mixing device.

6. What is filtration of water? Describe briefly the theories of filtration.

7. What are the purposes of roughing filtration? Explain the mechanism of roughing filtration.

8. Mention the important characteristics of slow and rapid sand filtration. What are the limitations of slow sand filtration?

9. Explain the operating difficulties of rapid sand filtration. How can these difficulties be removed?

10. What are the factors influencing disinfection of water? What are the characteristics of a good chemical disinfectant?

11. Explain the process of disinfection by chlorination.

12. Draw a typical chlorination curve and explain the reaction zones. Explain breakpoint chlorination.

16 13. A slow sand filtration unit produces 1,000 m^3 of water per day. How much bleaching powder with 30% available chlorine will be required per day to treat this water with a chlorine dose of 0.5 mg/l?

1.67 kg/day.

14. Discuss the procedure of disinfection of a contaminated handpump tubewell.

15. Explain the recommended processes of disinfection of water reservoir, pipe network and plumbing system.

16. Describe the process of disinfection of a dug well water by chlorine?

public standpipes should operate at low pressure. Water collected from the public standpipe is carried home in a container for consumption. The quality of water, particularly the bacteriological quality, may deteriorate during collection, transportation, storage and consumption. The water consumption from a standpipe is generally low, between 20 and 30 litres, because the water uses for purposes other than drinking, cooking, etc. are curtailed depending on the distance and the number of users of a standpipe. In spite of many shortcomings, the public standpipe is the most suitable low-cost option for water supply for large numbers of people who cannot afford to have house or yard connections.

20.6 WORKED EXAMPLE

Example - 1

Calculate the peak water demand of a rural community having a present population of 500 for a design period of 5 years. The average per capita water consumption is 50 lpcd with a peak factor of 3, the population growth rate is 2% (r=0.02) and the loss and wastage is 20%.

Solution:

Applying equation (20.2) $Q = f\, q\, P_p\, (1 + r)^n / (1 - 0.01w)$

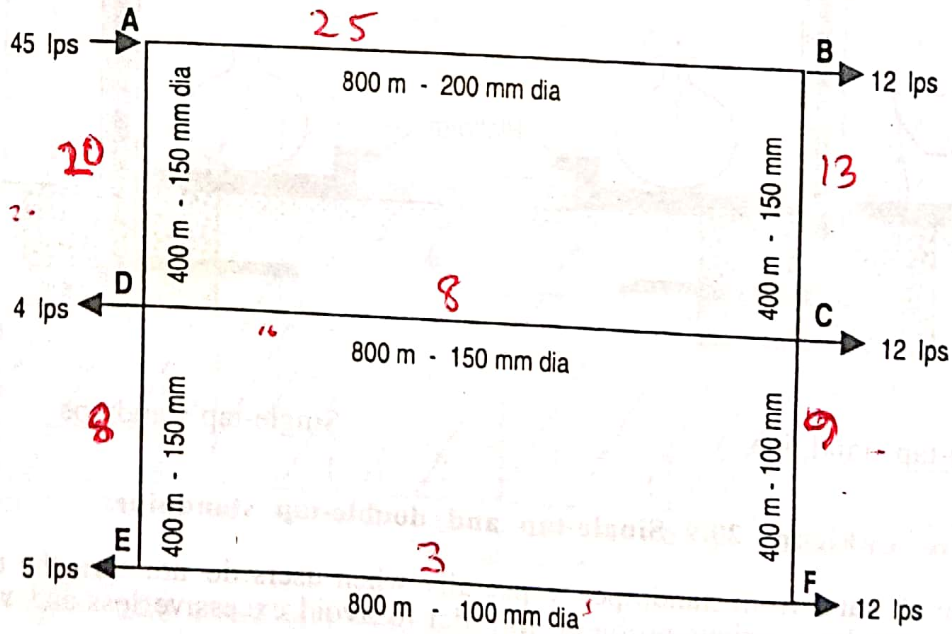
$Q = 3 \times 50 \times 500\, (1 + 0.02)^5 / (1 - 0.01 \times 20)$

or $Q = 75{,}000\, (1.02)^5 / (0.80)$

or $Q = 103{,}508$ lpd $= 103.508$ m^3/day

Example - 2

Calculate the flow in each of the pipes in the following looped pipe network:



$H_f = 1.59 \times 10^6 \dfrac{Q^{1.85}}{D^{4.87}} \times L$

A flow of 45 lps enters at joint A from a source. This water is drawn at nodes B, C, D, E and F, at the rate of 12, 12, 4, 5, and 12 lps respectively for consumption as shown.

<1.5 → OK
>1.5 → Trial

Solution: First trial with assumed flow, Q_o

Circuit	z	Length, m	Dia. mm	Q_o Lps	H_o/L m/m	H_o M	H_o/Q_o	Δ lps	Q_1 lps
I	AB	800	200	+25	0.0038	+3.04	0.1216	+0.54	+25.54
	BC	400	150	+13	0.0046	+1.84	0.1415	+0.54	+13.54
	AD	400	150	-20	0.0102	- 4.08	0.2040	+0.54	- 19.46
	DC	800	150	- 8	0.0018	- 1.44	0.1800	+0.54* +1.89*	- 5.57
					Sum :	-0.64	0.6471		
	Correction Δ = - ΣH/(xΣH/Q) = - (- 0.64)/ (1.85 x 0.6471) = + 0.54								
II	DC	800	150	+ 8	0.0018	+1.44	0.1800	- 0.54* - 1.89*	+5.57
	CF	400	100	+ 9	0.0170	+6.80	0.7556	- 1.89	+7.11
	DE	400	150	- 8	0.0018	- 0.72	0.0900	- 1.89	- 9.89
	EF	800	100	- 3	0.0022	- 1.76	0.5867	- 1.89	- 4.89
					Sum :	5.76	1.6453		
	Correction Δ = - ΣH/(xΣH/Q) = - (+5.76)/(1.85 x 1.6453) = - 1.89								

-8+.54+1.89

Common pipe DC required correction from both the circuits

Second trial with corrected flow, Q_1

Corcuit	Pipe	Length, m	Dia. mm	Q_1 Lps	H_1/L m/m	H_1 m	H_1/Q_1	Δ lps	Q_2 lps
I	AB	800	200	+25.54	0.0040	+3.20	0.1253	- 0.46	+25.08
	BC	400	150	+13.54	0.0050	+2.00	0.1477	- 0.46	+13.08
	AD	400	150	- 19.46	0.0097	- 3.88	0.1994	- 0.46	- 19.92
	DC	800	150	- 5.57	0.0010	- 0.80	0.1436	- 0.46* - 0.11*	- 6.14
					Sum :	+ 0.52	0.6160		
	Correction Δ = - ΣH/(xΣH/Q) = - (+0.52)/ (1.85 x 0.6160) = - 0.46								
II	DC	800	150	+5.57	0.0010	+0.80	0.1436	+ 0.46* + 0.11*	+ 6.14
	CF	400	100	+7.11	0.0109	+4.36	0.6132	+ 0.11	+7.22
	DE	400	150	- 9.89	0.0028	- 1.12	0.1133	+ 0.11	- 9.78
	EF	800	100	- 4.89	0.0055	- 4.40	0.8998	+ 0.11	- 4.78
					Sum :	- 0.36	1.7699		
	Correction Δ = - ΣH/(xΣH/Q) = - (- 0.36)/ (1.85 x 1.7699) = + 0.11								

*Common pipes require correction from both the circuits

$$\Delta = -\frac{\Sigma H_o}{x \times \Sigma \frac{H_o}{Q_o}}$$

x ↓ 1.85

The corrected flow after the second trial is Q_2. More trials will give more accurate flows in the pipe network.

Question

1. What are the main purposes of transmission and distribution systems? What are the different water supply systems? Write the advantages and disadvantages of each water transmission and distribution system.

2. What are the main types of distribution networks? What are their relative advantages and disadvantages?

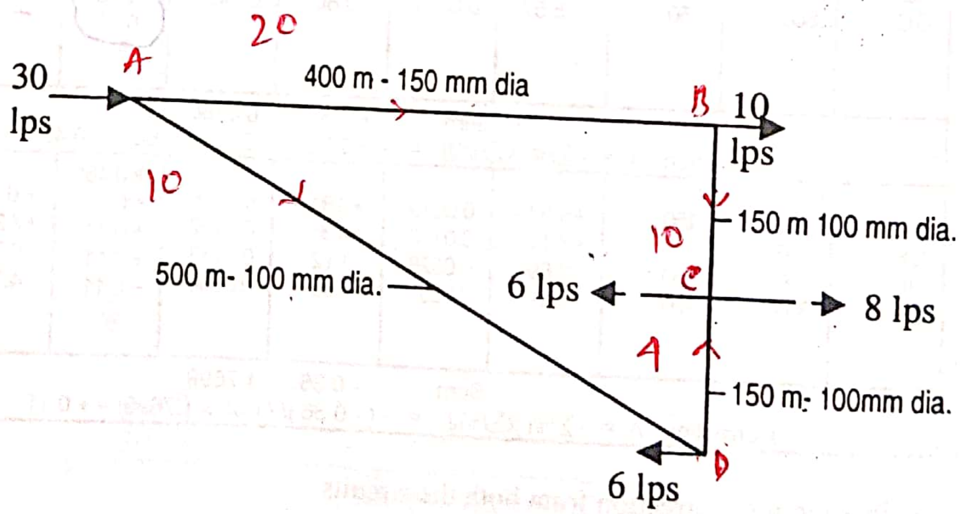
3. How can water demand of a community be estimated? Mention the water consumption data and the corresponding recommended peak factor for the design of water supply systems in Bangladesh.

4. Calculate the peak water demand for a design period of 10 years of a rural village having a present population of 1,500. The average per capita water consumption is 50 lpcd with a peak factor of 3, the population growth rate is 1.5% and the loss and wastage is 30%.
   346,270 m3/d

5. State systematically the process of design of branched as well as looped network of distribution systems.

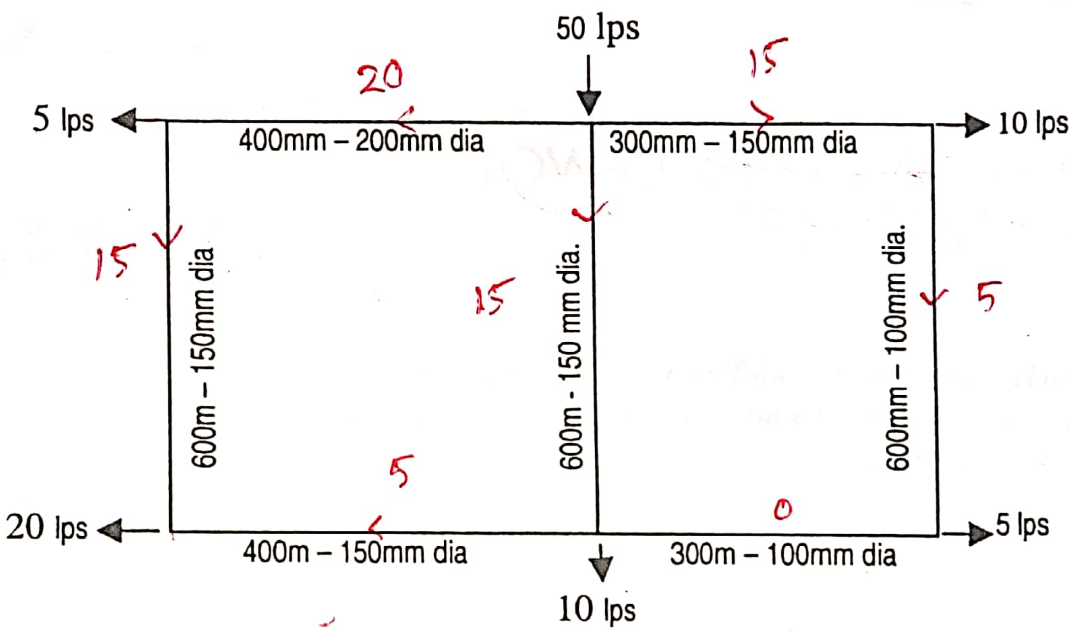
6. How can you calculate the flow in a looped network using the Hardy Cross method?

7. Calculate the flow in the following pipe network for the inflow and outflow shown in the diagram using the Hardy Cross method.



H.W

8. Find out the approximate flow in the pipes in the network shown below:



9. What are the different types of service connections? Draw a neat sketch of a yard connection.

21.6 WORKED EXAMPLE

Example - 1 17

Design a strainer for a 38 mm diameter tubewell to be operated by a No.6 handpump at the rate of 40 lpm. Slot No.10 strainer having a 40% open area is to be used. The entrance velocity should be around 0.01 m/sec.

Solution: The open area per metre of 38 mm strainer = $\pi\, d \times l \times 0.4 = 3.14 \times 0.038 \times l \times 0.4 = 0.0477\ m^2/m$

Q = 40 lpm = $(40/60) \times 10^{-3}\ m^3/sec. = 0.67 \times 10^{-3}\ m^3/sec.$

Entrance Velocity, $v_e = 0.01$ m/sec.

Entrance area required = $Q/v_e = 0.67 \times 10^{-3}/0.01 = 0.067\ m^2$

Length of the strainer required $L = 0.067/0.0477 = 1.4$ m

Strainer length of 1.5 m may be provided.

The length of the strainer can also be computed using equation 21.2 directly.

Example - 2 17

Calculate the rainwater available for a family having a roof area of 20 m^2 in the central region of Bangladesh, where rainfall intensity is 2.0 m per year. Assume a runoff coefficient of 0.75.

Solution : Quantity of rainwater available, $Q = C\, I\, A$

$C = 0.75$; $I = 2.0$ m/yr; $A = 20\ m^2$

$Q = 0.75 \times 2.0 \times 20 = 30\ m^3/yr$

Example - 3

The average rainfall intensity in Bangladesh is 2.4 m/yr. and the runoff coefficient is 0.70. Calculate the minimum catchment area required for a family of 7 persons to be supplied with 15 lpcd of water.

Solution: The catchment are required is, $A = 0.365\, q\, N/(C\, I)$

q = 15 lpcd; $N = 7$; $C = 0.7$; $I = 2.4$ m/yr

$A = 0.365 \times 15 \times 7/(0.70 \times 2.4) = 22.8\ m^2$

Example - 4

Calculate the minimum capacity of the storage tank required for a family of 8 persons to be supplied with 10 lpcd of rainwater. The yearly rainfall intensity is 2.5 m and the rainfall distribution is such that at least 35% of the rainwater must be stored for uninterrupted water supply throughout the year. Also calculate the minimum catchment area required when the coefficient of runoff is 0.7.

Solution: Storage volume required · $V = 0.365\, f\, q\, N$

$f = 0.4$; $q = 10$ lpcd; $N = 8$; $C = 0.7$; $I = 2.5$ m

$V = 0.365 \times 0.4 \times 10 \times 8 = 11.68 \text{ m}^3$

$A = 0.365\, q\, N/(C\, I)$

$A = 0.365 \times 10 \times 8/(0.7 \times 2.5) = 29.2/1.75 = 16.69 \text{ m}^2$

Example - 5

Calculate the per capita water available and the capacity of the storage tank required for a family of 6 persons having a roof area of 20 m² with a runoff coefficient of 0.8. The family lives in a part of Bangladesh having a yearly rainfall of 2.5 m. The distribution demands 40% storage requirement for full utilization of rainwater.

Solution: Available rainwater, $Q = C\, I\, A$

$C = 0.80$; $I = 2.5$ m/yr; $A = 20 \text{ m}^2$; $f = 0.4$

Available rainwater = $20 \times 2.5 \times 0.8 = 40 \text{ m}^3$

Storage volume required = $40 \times 0.4 = 16 \text{ m}^3$

Available rainwater per capita per day = $40 \times 1000/(6 \times 365)$ = 18.26 lpcd.

$\boxed{1 \text{ ppm} = 8.34 \text{ lb/mgd}}$

$40 \times 8.34 \frac{\text{lb}}{\text{mgd}} \times 1 \text{ mgd} \times 0.75$

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Questions

1. What are the present practices of water supply in the rural areas of Bangladesh? Name the suitable alternative low-cost technological options for water supply in Bangladesh.

2. Describe the construction procedure of dug wells. How can you protect dug well water from contamination?

3. Why is the No.6 handpump tubewell the most popular in Bangladesh? Draw the sectional elevation of a No.6 handpump and label the different components of the pump.

4. Why is the Tara handpump tubewell developed and being promoted in Bangladesh? Compare the Tara and Mark-II handpump tubewells with necessary sketches.

5. What are the reasons for local development of Bangla and Moon handpumps? Draw a neat sketch of a Bangla handpump tubewell.

6. Design a strainer for a 30 mm diameter tubewell to be operated by a No.6 handpump at the rate of 25 lpm. A slot No. 12 strainer having 50% open area is to be used. The entrance velocity should not exceed 0.015 m/sec. 58.95 cm

7. Describe a household filter for the treatment of surface water. Mention the important characteristics of a household filter.

8. Describe the working principles of a pond sand filter with a neat sketch. What are problems encountered in the operation and maintenance of pond sand filters?

9. What is the potential for rainwater harvesting in Bangladesh? What are the advantages and disadvantages of rainwater harvesting in Bangladesh?

10. The average rainfall intensity and distribution in Bangladesh in 1999 shows that the total rainwater available in that year is 2200 mm excluding the losses, and a storage volume of 30% of the available rainfall is required for a year-round supply. What are the minimum storage and catchment area required for drinking and cooking at the rate of 10 lpcd for a family of 8persons? $V = 11.68\ m^3$ $A = 13.27\ m^2$

11. Compare the costs, advantages and disadvantages of a No.6 handpump, Tara handpump, and manually operated deep tubewell.

12. Draw a clear diagram of a very shallow shrouded tubewell. Discuss the usefulness of SST and VSST under conditions in the coastal areas of Bangladesh.

Sedimentation and Flotation

Problem 1:

Find the settling velocity of spherical discrete particles 0.05 mm in diameter, given that their specific gravity is 2.6 settling in water at a temperature of 20º C.

Solution:

Stoke's law states that: $V = \frac{g(s-1)d^2}{18\gamma}$

Where:

v = Settling velocity, m/s.

g = Gravitational acceleration, m/s^2.

d = Diameter of spherical particle, m.

S(s.g.)= Specific gravity .

γ = Kinematic viscosity, m^2/s

Sedimentation and Flotation

Given in the problem:

$d = 0.05$ mm $= 5x10^{-5}$ m,

s.g. = 2.6, and from tables of viscosity of water at 20^o C, the kinematic viscosity is 1.003x10-6 m^2/s.

Substituting in the above equation, then:

$V = 9.81(2.6-1)(5\times10^{-5})^2 /18\times1.003\times10^{-6}$

$= 2.7 \times10^{-3}$m/s

Stoke's law is valid for laminar flow, then Reynolds number is less than 0.5. Therefore, Re should be checked:

$Re = v.D/\gamma = 2.17x10^{-3}x5x10^{-5}/1.003x10^{-6} = 0.118$ (less than 0.5, OK.).

Sedimentation and Flotation

Problem 2:

A sample from a river was analyzed for suspended solids. The analysis revealed that it contains 160 g/m^3 suspended solids. The sanitary engineer responsible suggested clarifying this water by the sedimentation process. When the clarification rate was determined experimentally, the cumulative frequency distribution for the settling velocities of these particles showed a straight line relationship:

10 % of the particles have a settling velocity greater than 1.5 m/hr.

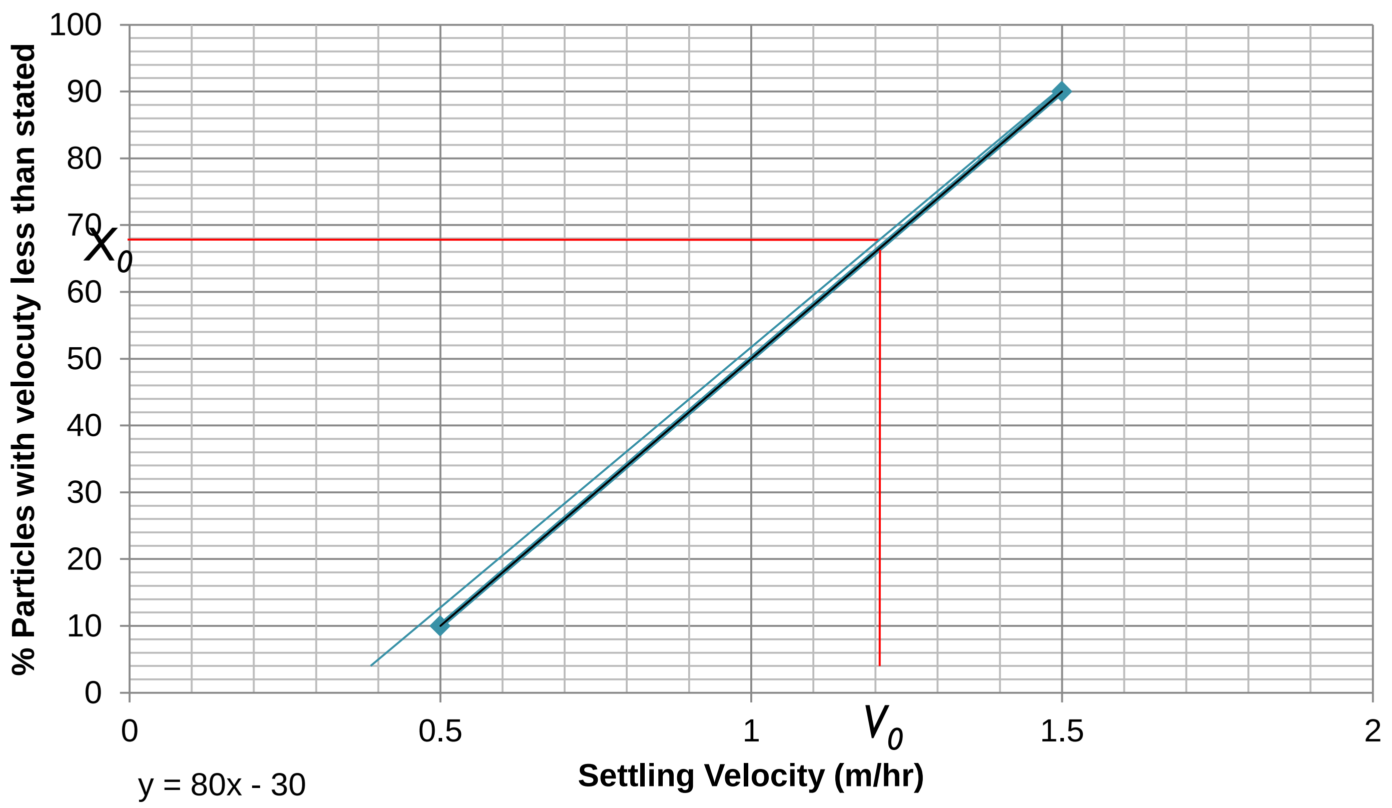
10 % of the particles have a settling velocity less than 0.5 m/hr.

The amount of water to be purified is 1500 m^3/hour. Thus, the sanitary engineer proposed designing a sedimentation basin of length 50 m, width 25 m and depth of 3 m. assuming that this tank will work as an ideal sedimentation basin, find the suspended solids content of the effluent water.

Sedimentation and Flotation

Solution:

From the data given above, plot the cumulative frequency distribution curve for the settling velocities of these particles as shown in figure below:



Sedimentation and Flotation

The overall removal X_T is given by:

$$X_T = 100 - x_o + \frac{1}{v_o} \int_0^{x_o} v_t \, dx$$

Where:

X_T = overall removal

v_o = design settling velocity

x_o = removal corresponding to v_o.

Sedimentation and Flotation

- Given that the discharge Q =1500 m^3/hour and since settling velocity v_o = discharge/area = Q/A and A = width (W) × length (L),

Then, $v_o = Q/WL = 1500/(25x50) = 1.2$ m/hour

From figure for v_o = 1.2 m/hr, then X_o = 66%

Therefore, the overall removal:

$X_T = 100 – X_o$ + removal

$= 100 – 66 +(1/2)(0.37 + 1.2)x66/1.2 = 77\%$

Given that the suspended solids concentration entering the basin C_i = 160 g/m^3, then the concentration of suspended solids in the effluent C_e could be found as:

$C_e = (1 – X_T)C_i = (1 – 0.77)x160 = 36.8$ g/m^3

Sedimentation and Flotation

Problem 3: Settling column tests performed on discrete particle suspension gave the following tabulated results:

Sampling depth (cm)	Sampling time (min)	% Suspended solids remaining in sample
100	60	60
	180	43
	360	29
200	60	63
	180	55
	480	36
300	60	65
	120	63
	360	50

By using this data plot the cumulative batch settlement curve and calculate the total removal for a horizontal settlement basin with surface area of 100 m^2 when incoming flow rate is 4.8 m^3/min. Calculate also the removal to be expected for a similar basin when the flow is 0.02 m^3/sec.

Sedimentation and Flotation

Solution:

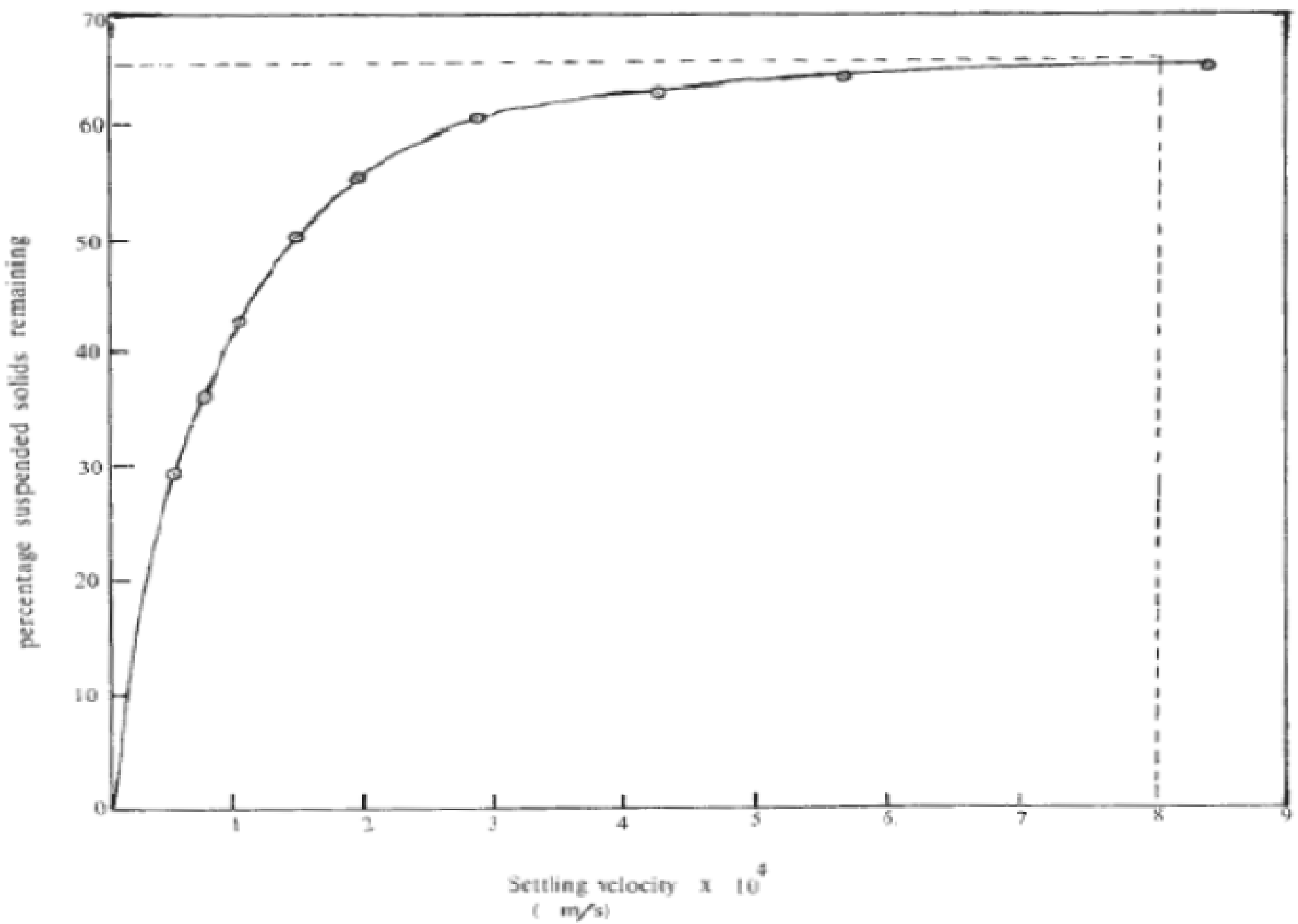
From the given data the settling velocities could be found for each depth and sampling time knowing that:

Settling velocity = sampling depth/sampling time

Sampling depth (m)	Sampling time (sec)	Settling velocity (m/s x 10^4)	% Suspended solids remaining in sample
1	3600	2.78	60
	10800	0.93	43
	21600	0.46	29
2	3600	5.56	63
	10800	1.85	55
	28800	0.69	36
3	3600	8.33	65
	7200	4.17	63
	21600	1.39	50

Sedimentation and Flotation

Using this data achieved above, the cumulative batch settlement curve could be plotted as shown in figure.



Sedimentation and Flotation

For a flow of 4.8 m^3/min, the settling velocity

$V_1 = Q/A = 4.8/(60 \times 100) = 8\times10^{-4}$ m/s

From the graph for a settling velocity of 8×10^{-4} m/s

$x_o = 66$ %

To find the total removal X_T, the integral v.dx should be found. This could be done as follows:

Sedimentation and Flotation

Interval	dx	V_{st}	$V_{st}.dx$
0 – 10	10	0.05	0.5
10 – 20	10	0.15	1.5
20 – 30	10	0.35	3.5
30 – 40	10	0.65	6.5
40 – 50	10	1.05	10.5
50 – 60	10	1.75	17.5
60 – 66	6	4	24
$\Sigma v.dx$			64

Sedimentation and Flotation

$$X_T = 100 - x_O + \frac{1}{v_O}\int_0^{xo} v_t \, .dx$$

Thus, X_T = 100 – 66 + 64x10^{-4}/8 x10^{-4} = 42 %

Using a similar technique for water flow of 0.02 m^3/sec, total removal could be found to be approximately 60%.

Filtration

Problem 4:

Making use of the equations developed by Rose and Carman-Kozeny compute the head loss through a 0.8 m sand bed. Assume that the bed consists of uni-sized, spherical sand particles having a diameter of 0.5 mm, the kinematic viscosity is equal to 1.003×10^{-6} m^2/s and the bed porosity is 40 percent while the rate of filtration is 250 l/m^2.min.

Solution:

Rose equation states that:

$$h = 1.067 C_D \frac{1}{p^4} \frac{v^2}{g d \phi} L$$

Filtration

Where,

h = Head loss in bed, m.

ν = kinematic viscosity = $1.003 \times 10^{-6}\ m^2/s$

v= Face velocity, m/s.

d = diameter of the particle, m. = 0.5 mm (0.5×10^{-3} m)

g = Gravitational acceleration, m/s^2. = $9.81\ m/s^2$

φ = Particle shape factor, dimensionless = (surface area of equivalent volume of sphere/actual surface area) = 1

p = bed porosity, dimensionless (= volume of voids/total volume) = 0.4

L = bed depth, m. = 0.8 m

Rate of filtration = 250 l/m^2.min. = 4.17x10-3 m^3/m^2.sec

Filtration

C_D = Newton's drag coefficient.

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

$$\mathbf{Re} = \frac{v\, d}{\nu}$$

$$\mathbf{Re} = \frac{v\, d}{\nu} = \frac{4.17x10^{-3}\, x0.5x10^{-3}}{1.003x10^{-6}} = 2.079$$

$$C_D = \frac{24}{\mathbf{Re}} + \frac{3}{\sqrt{\mathbf{Re}}} + 0.34 = \frac{24}{2.079} + \frac{3}{\sqrt{2.079}} + 0.34 = 13.96$$

$\therefore$ Head loss,

$$h = 1.067 C_D \frac{1}{p^4} \frac{v^2}{g d \phi} L = 1.067x13.96x \frac{1}{0.5^4} \frac{\left(4.17x10^{-3}\right)^2}{9.81x0.5x10^{-3}\, x1} x0.8 = 1.65 m$$

Filtration

Using Carman-Kozeny equation the head loss is given by:

$$h_1 = E\frac{1-p}{p^3}\frac{v^2}{\phi dg}.L$$

Where:

$$E = \frac{150(1-p)}{\mathbf{Re}} + 1.75$$

Therefore,

$$E = \frac{150(1-p)}{\mathbf{Re}} + 1.75 = \frac{150(1-0.4)}{2.079} + 1.75 = 45.04$$

Hence,

$$h_1 = E\frac{1-p}{p^3}\frac{v^2}{\phi dg}.L = 45.04\frac{1-0.4}{0.4^3}\frac{\left(4.17x10^{-3}\right)^2}{1x0.5x10^{-3}x9.81}x0.8 = 1.2m$$

Filtration

Problem 5:

Determine the clear water head loss in a filter bed that consists of two layers of filter media; a uniform anthracite layer of depth of 0.6 m with an average particle size of 1.6 mm and a specific gravity of 1.5; the other layer is composed of uniform sand 40 cm deep with an average particle size of 0.6 mm and a specific gravity of 2; for a rate of filtration of 150 $L/m^2/min$. The operating temperature was found to be 15^o C and the porosity is 0.35. Use Rose equation for evaluating the clear water head loss.