ENVIRONMENTAL ENGINEERING-1

Water Supply Engineering

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1

Water Systems

1.1. INTRODUCTION

The five essential requirements for human existence are : (*i*) air (*ii*) water (*iii*) food (*iv*) heat and (*v*) light. Contamination of these elements may cause serious health hazards not only to man but also to animal and plant life. *Environmental Engineering* deals with all these essential elements.

The use of water by man, plants and animals is universal. Without it, there can be no life. Every living thing requires water.

Man and animals not only consume water, but they also consume vegetation for their food. Vegetation, in turn, cannot grow without water. Growth of vegetation also depends upon bacterial action, while bacteria need water in order to thrive. The bacterial action can convert vegetable matter into productive soil. New plants, which grow in this soil, grow by sucking nutrients through their roots in the form of solution in water. Thus an ecological chain is maintained. Water maintains an ecological balance – balance in the relationship between living things and environment in which they live.

The use of water is increasing rapidly with our growing population. Already there are acute shortages of both surface and under ground waters in many parts of the country. Careless pollution and contamination of the streams, lakes, reservoirs, wells and other under ground sources has greatly impaired the quality of available water. This pollution results because of improper disposal of waster water —both domestic as well as industrial. Organised community life require twin services of water supply and sewage disposal. Good sanitation cannot be maintained without adequate water supply system. Without proper disposal, the wastes of a community can create intolerable nuisance, spread diseases and create other health hazards. The planning, designing, financing and operation of water and waste water systems are complex undertakings, and they require a high degree of skill and judgement. The work of construction and maintenance of water supply and waste water disposal systems is generally undertaken by Government agencies – mostly through *Public Health Engineering* or *Environmental Engineering Departments* consisting of Civil Engineers.

1.2. HISTORICAL DEVELOPMENT

Man's search for pure water began is prehistoric times. The story of water supply begins with the growth of ancient capital cities, or religious and trade centres. In olden days, most of community settlements throughout the World were made near springs, lakes and rivers from where the water supply for drinking and irrigation purposes was obtained. Rig Veda (4000 years B.C) makes a mention of digging of wells. Similarly, Ramayana, Mahabhartha and Puranas make mention of wells as the principal source of water supply. These wells were mostly of shallow depth, dug near river banks. Water was lifted from the wells through indegenous methods. However, no water treatment or distribution works existed. Apart from India (Bharat), other major civilisations of the World, such as Greece, Egypt, Assyria etc. used wells for their settlements which were located slightly away from springs, lakes and rivers. Joseph's well at Cairo is one of the oldest deep wells excavated in rock to a depth of about 300 feet. These wells, however, caused water supply problems during periods of drought. It became necessary, therefore, to store water. Cisterns were constructed for collecting rain water while reservoirs were constructed to store water from streams and rivers during monsoon period. The stored water was conveyed to towns through masonry conduits and aqueducts. The earlier examples are the aqueducts built by Appius Claudius in about 312 B.C. for water supply to Rome. Lyons in Paris, Metz in Germany and Segovia and Serille in Spain built similar aqueducts and syphons for water supply used for drinking, bathing and other purposes. Sextus Julius Frontinus, Water Commissioner of Rome (A.D. 97) reported the existence of nine aqueducts supplying water to Rome and varying in length from 10 to over 50 miles and in cross-section from 7 to over 50 sq. ft., with an estimated aggregate capacity of 84 mgd. The great sewer, known as the *cloaca maxima* and constructed to drain the Roman Forum, is still in service.

There was practically no improvement in water supply systems in the middle ages. The earlier water supply structures got destroyed with the fall of Rome. In the ninth century, few important water supply structures were constructed by the Moors in Spain. In the twelfth century, small aqueduct was constructed in Paris. In London, spring water was brought by means of lead pipes and masonry conduits in the thirteenth century. In Germany, water works were constructed in 1412 and pumps were introduced in 1527 in Hanover. Franciscan monk constructed aqueduct of Zempola in Mexico in the middle of 16th century. In 1582, a pump was erected on the old London bridge for the supply of water from the Thames. The water was conveyed through lead pipes. In Paris, pumps operated by water power were erected in 1608. Pumps operating from steam were introduced in the 18th century in London and Paris. In the United States, spring water was conveyed by gravity to Boston in 1652. Pumps etc. were introduced at Bethlehem in 1754.

However, purposeful quality control of water supply is quite recent in origin. The scientific discoveries and engineering inventions of the eighteenth and nineteeth centuries created centralised industries to which people flocked for employment. This caused serious water supply and waste disposal problems in the industrial towns. No great schemes of water supply were started until the Industrial Revolution had well passed its first half century. The development of the large impounding reservoir was largely due to the necessity of feeding canals constructed during the first phase of the Industrial Revolution.

The first water filter was constructed in 1804 by John Gibb at Paisley in Scotland. It was a slow sand filter and worked in conjunction with a settling basin and roughening filter. Next successful filters were constructed in 1827 by Robert Thom at Greenock. In 1829, James Simpson built sizable filters for the Chelsea Water Company to improve its supply from the Thames river. By 1870, the mechanical filter of the pressure type began to be employed, the earliest being the Halliday filters installed at Crewe (1888), Bridlington and elsewhere. In 1894 pre-filters were successfully built. In the first decade of 20th century, mechanical pressure filters were introduced, Hastings being an early pioneer with Canndy filters built in 1900. In India, Calcutta was the first city where a modern water supply system was constructed in 1870.

The technique of clarification and filtration soon grew. By 1939, mechanically-sludged sedimentation tanks were in general use. The micro-strainer, for the removal of plankton from the impounded water was developed by Boucher, and was introduced by Glenfield and Kennedy in 1945. Coagulation of water with sulphate of alumina began experimentally in 1827, but was adapted practically only in 1881 to treat Bolton's water supply. Activated silica was introduced by Bayliss in U.S.A. during 1937. The first permanent use of chlorination originated under the direction of Sir Alexander Houston at Lincoln in 1905. In 1917, Paterson Engineering Company installed the first gaseous chlorinator at the Rye Common Works. Super-chlorination and dechlorination was first applied in 1922 at the Deptford works of the Metropolitan Water Board. The art of softening water was also first developed in Great Britain. The first municipal softener was constructed by Plumstead in 1854. Development of the softener took a novel turn in 1912 by the construction, at the Hooten works of the West Cheshire Water Board, of a base exchange softener.

Since India was under British occupation, water supply schemes in India were undertaken practically about the same time as in England, though with a slower rate. In 1870, a water supply system was constructed at Calcutta. Till Independence, only few cities had protected water supply systems.

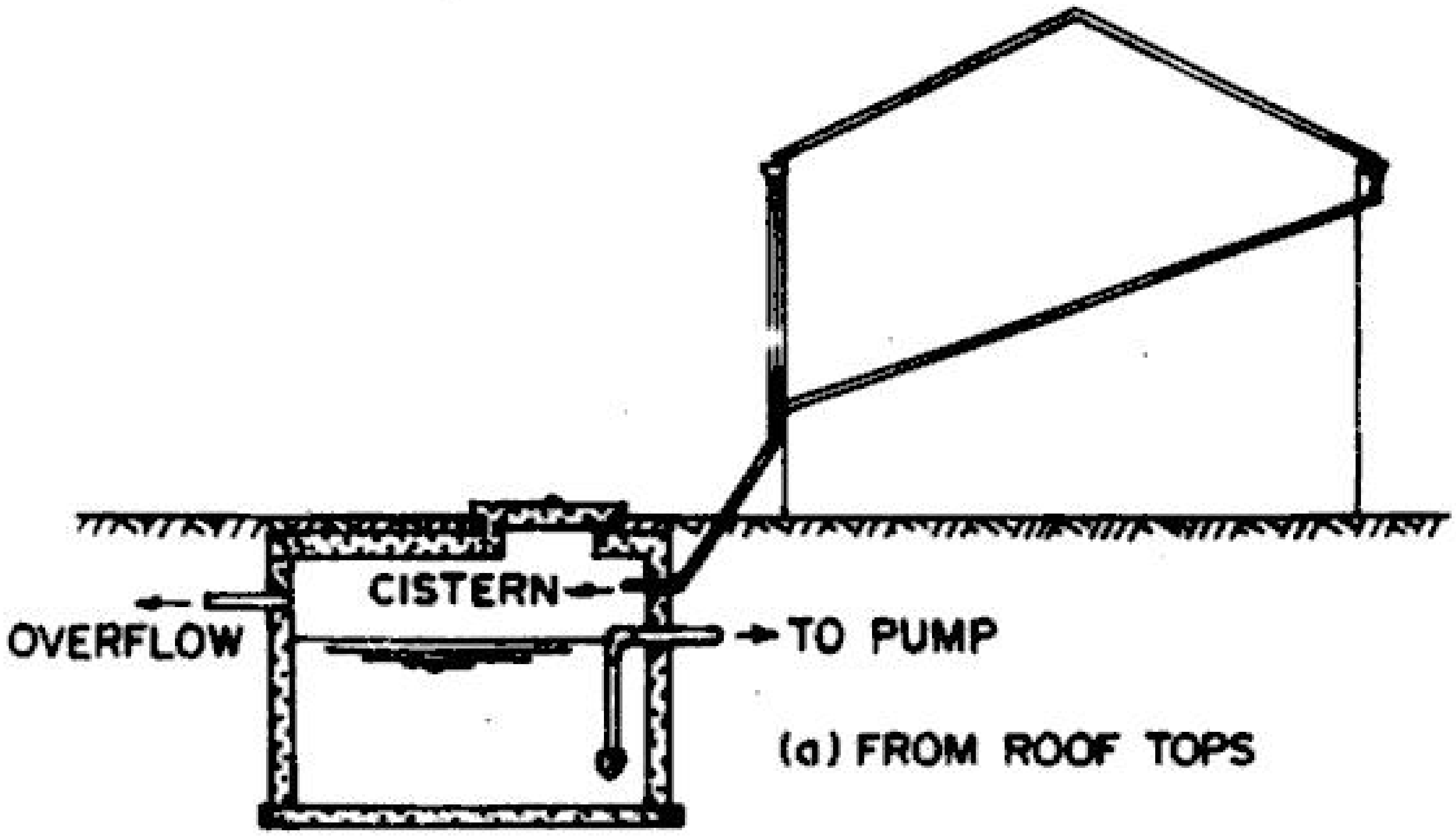
1.3. SOURCES OF WATER

The following are common sources of water

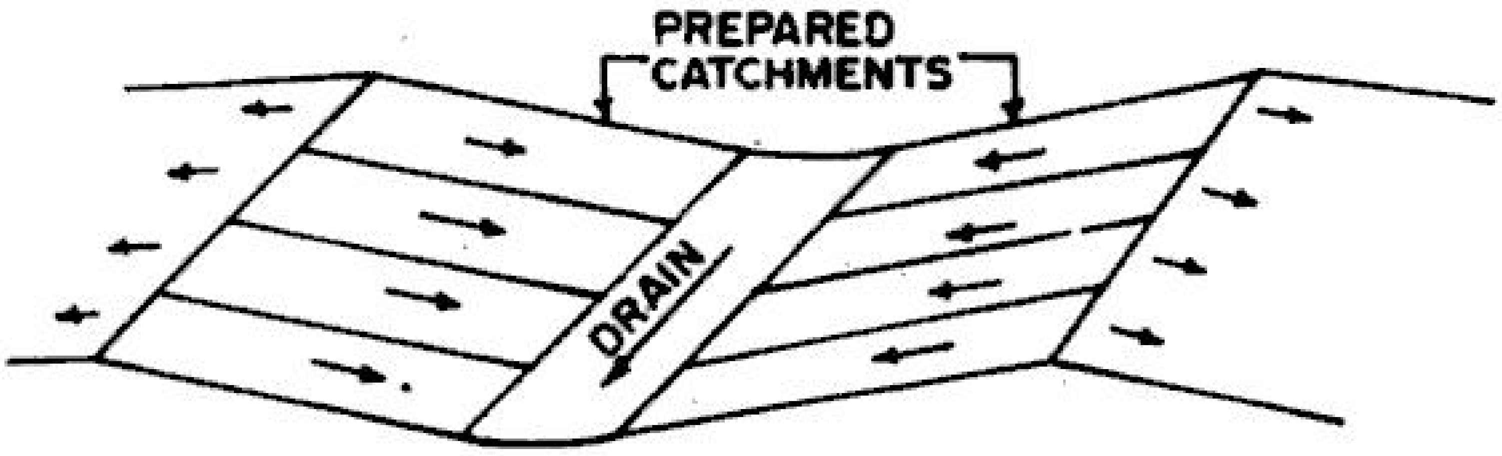
(*i*) Rain water (*ii*) Surface water

(*iii*) Ground water (*iv*) Water obtained from reclamation.

1. Rain Water



(a) FROM ROOF TOPS



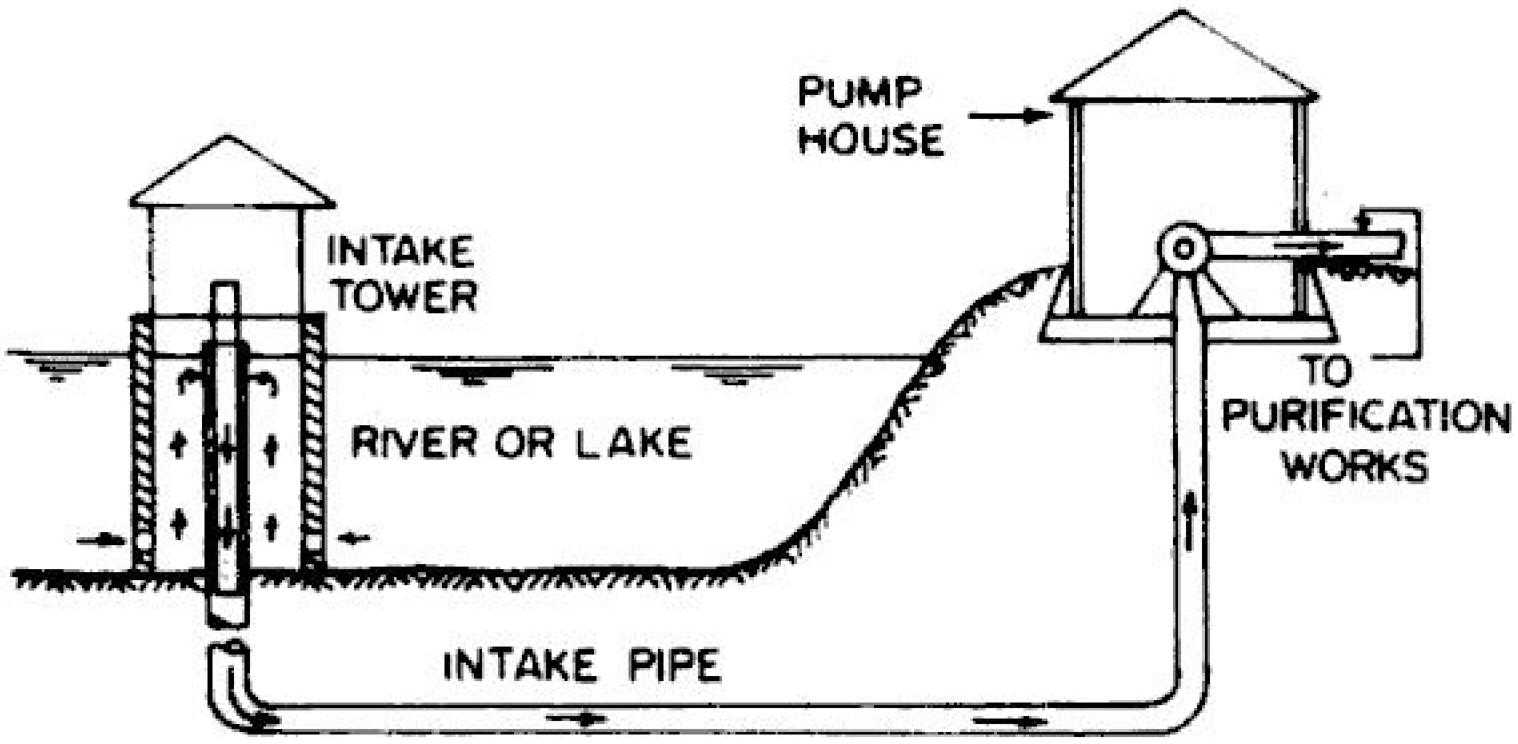
(b) FROM PREPARED CATCHMENTS

FIG. 1.1. DIRECT COLLECTION OF RAIN WATER

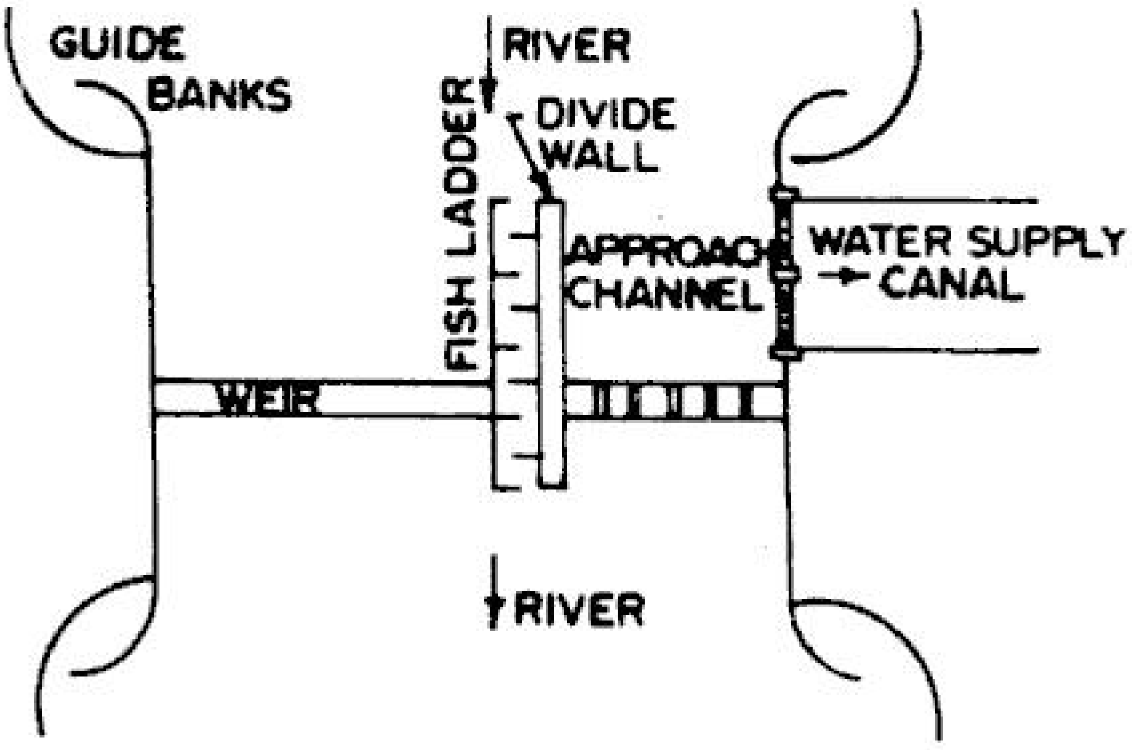
(*a*) ***From roofs of houses and dwellings*** : Water is stored in small underground tank or cistern, for small individual supplies (Fig. 1.1 *a*).

(*b*) ***From prepared catchments*** : The surface of catchments is made impervious by suitable lining material, and suitable slope is given so that water is stored in moderate size reservoirs. This water is used for communal supplies, mostly for drinking purposes.

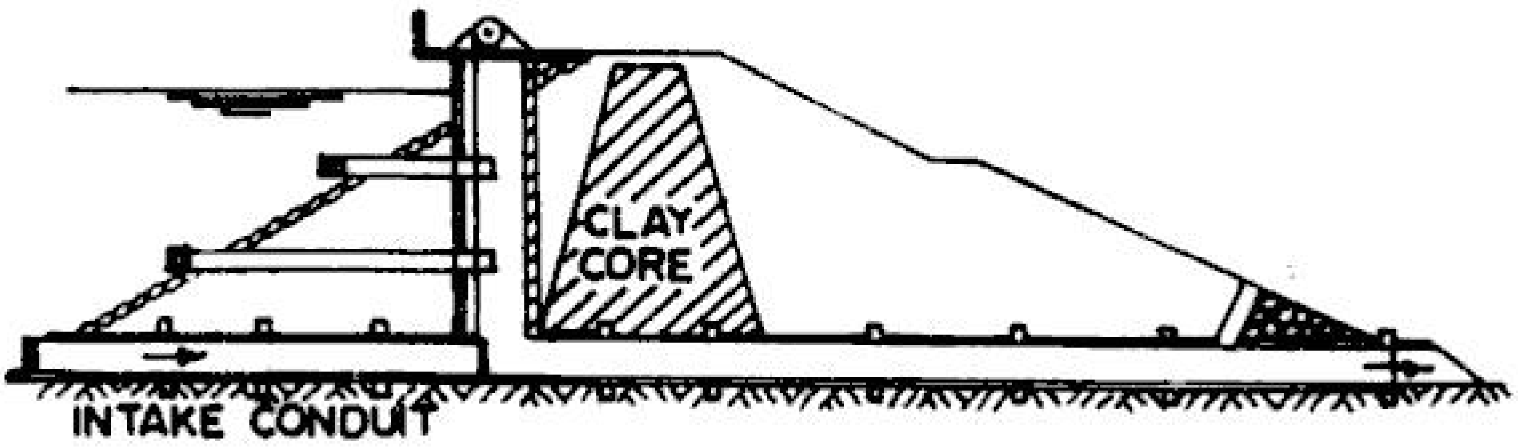
2. Surface Water



(a) CONTINUOUS DRAFT FROM STREAMS OR LAKES



(b) FROM RIVER DIVERSION WORKS



(c) WATER FROM RESERVOIR STORAGE

FIG. 1.2. SOURCES OF SURFACE WATER

Surface water is the one which is available as *run-off* from a catchment area, during rainfall or precipitation. This runoff flows either into streams or into undrained lakes. The runoff water flowing into streams can either be *stored* in a reservoir by constructing a dam across it, or be diverted into a water supply channel. Thus, depending upon the scheme of collection, we get surface water from the following sources.

(*a*) ***From rivers by continuous draft*** : Water may be collected directly from the river, without any diversion work (Fig 1.2 *a*).

(*b*) ***From river diversion.*** A diversion work is constructed across a perennial river and water is diverted into a canal which leads water to the site of water purification works (Fig. 1.2 *b*).

(*c*) ***From reservoir storage.*** Where supply is not ensured throughout the year, dam may be constructed across the river and water stored in the reservoir (Fig. 1.2 *c*).

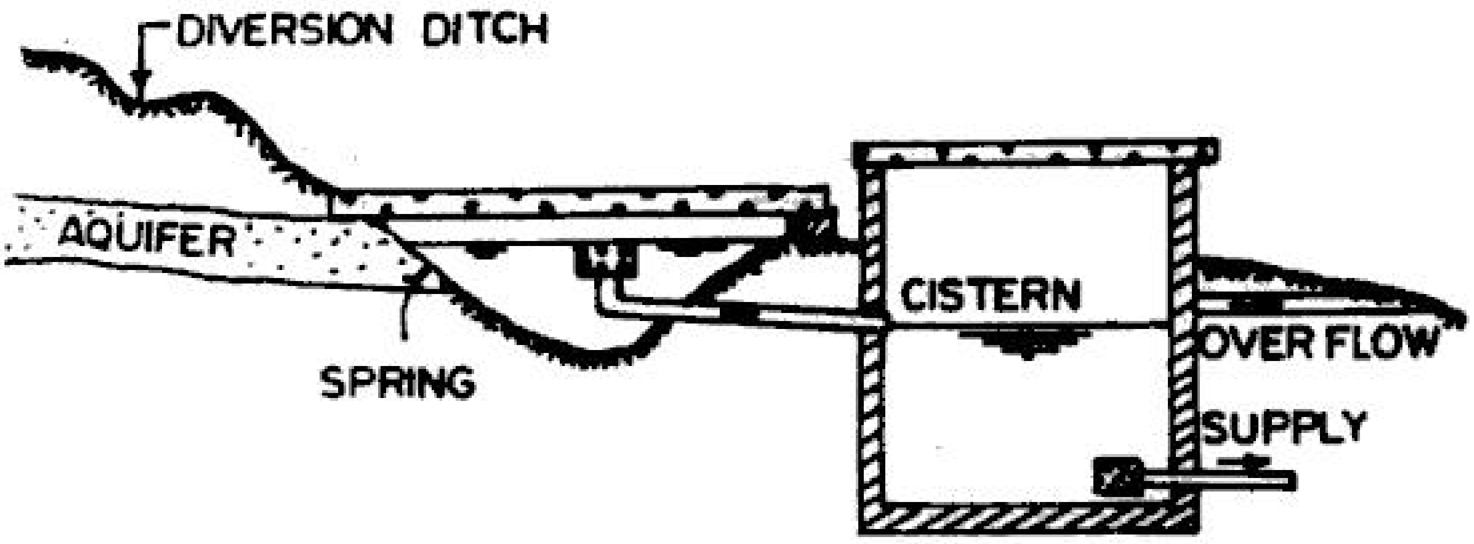
(*d*) ***From direct intake from natural lakes.*** Water may also be obtained through direct intakes from natural lakes which receive surface run-off from the adjoining catchment (Fig. 1.2 *a*).

3. Ground Water

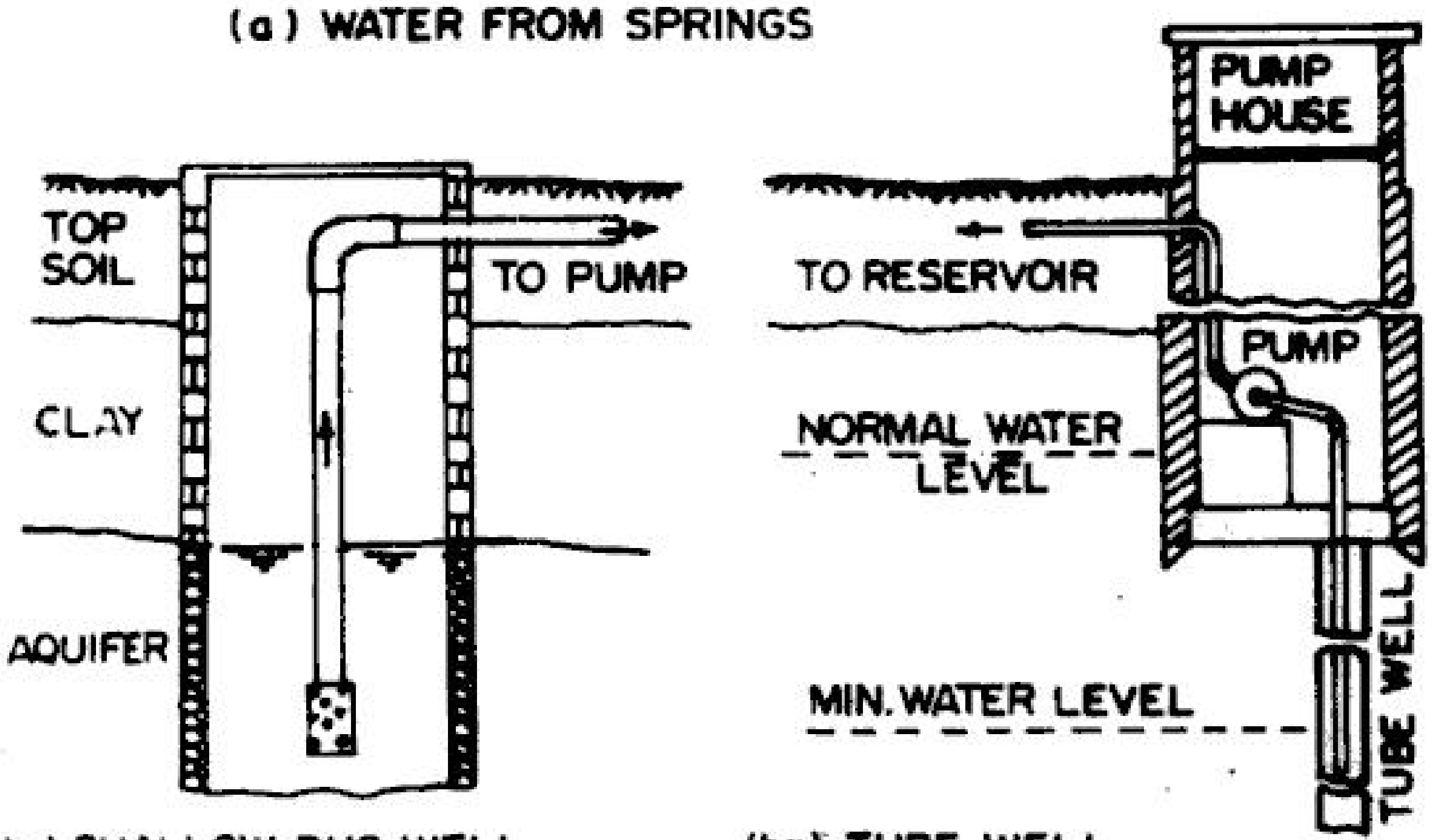
The largest available source of fresh water lies underground. The term *'ground water'* refers to this water, which is stored by nature, under-ground in the water-bearing formation of earth's crust. The total ground water potential is estimated to be one third the capacity of oceans. The main source of ground water is precipitation. A portion of rain falling on the earth's surface infiltrates into ground, travels down and when checked by impervious layer to travel further down, forms ground water. The *ground water reservoir* consists of water held in voids within a geologic stratum. The ground water can be tapped from the following sources.

(*a*) From natural *springs* (Fig. 1.3 *a*).
(*b*) From *wells* and *bore holes* (Fig. 1.3 *b*).
(*c*) From *infiltration galleries*, basins or cribs (Fig. 1.3 *c*).
(*d*) From wells and galleries with flows augmented from some other sources :
  (*i*) spread on surface of the gathering ground
  (*ii*) carried into charging basins or ditches, or
  (*iii*) led into diffussion galleries or wells.

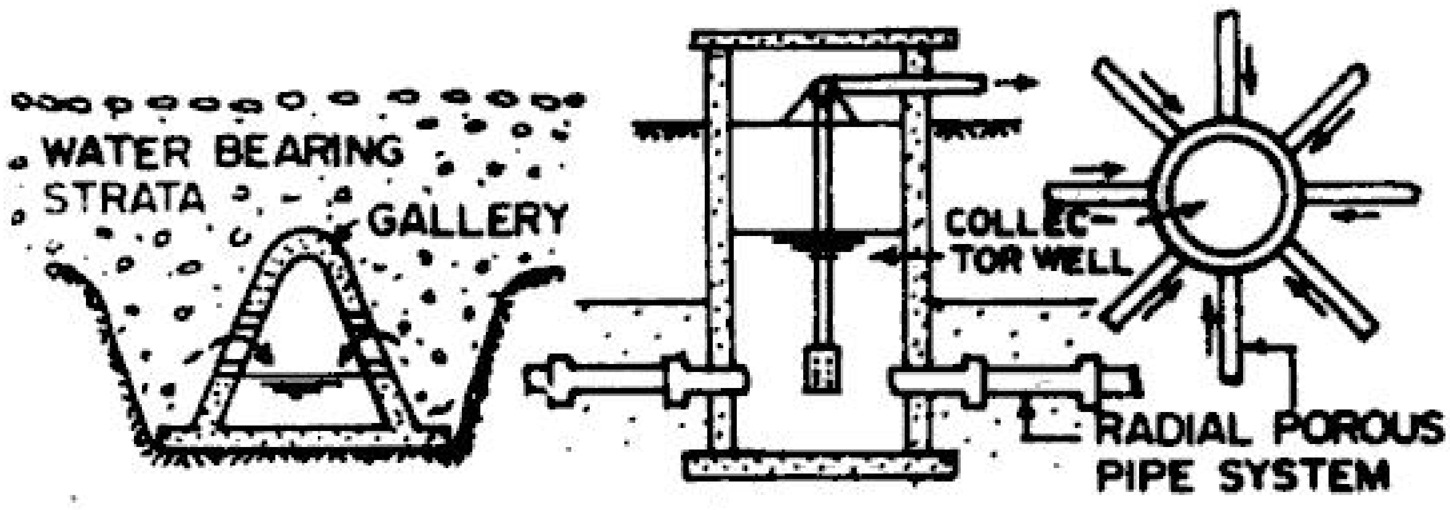
(*e*) From river side *radial collector wells* (Fig. 1.3 *d*)



(a) WATER FROM SPRINGS



(bi) SHALLOW DUG WELL (bii) TUBE WELL



(c) INFILTRATION GALLERY (d) RADIAL COLLECTOR WELL

FIG. 1.3. SOURCES OF UNDERGROUND WATER.

4. Water obtained by reclamation

(*a*) ***Desalination.*** Saline or brakish water may be rendered useful for drinking purposes by installing desalination plants. The common methods used for desalination are: distillation, reverse osmosis, electrodialysis, freezing and solar evaporation.

(*b*) ***Re-use of treated waster water.*** Effluent or waste water can be treated suitably so that it may be re-used. An example of the controlled indirect re-use is the intentional artificial recharge of ground water aquifers by adequately treated waste water.

1.4. WATER SUPPLY SYSTEMS

The primary objective of water treatment for public supply is to take water from the best available source and to subject is to processing which will ensure water of good physical quality, free from unpleasant taste or odour and containing nothing which might be detrimental to health. The treatment of water to improve its quality involves additions to, substractions from, or chemical changes in the raw water. Municipal water systems consist of the following units.

1. Collection works
2. Transmission works
3. Purification works

and 4. Distribution works.

These systems have been shown diagrammatically in Fig. 1.4.

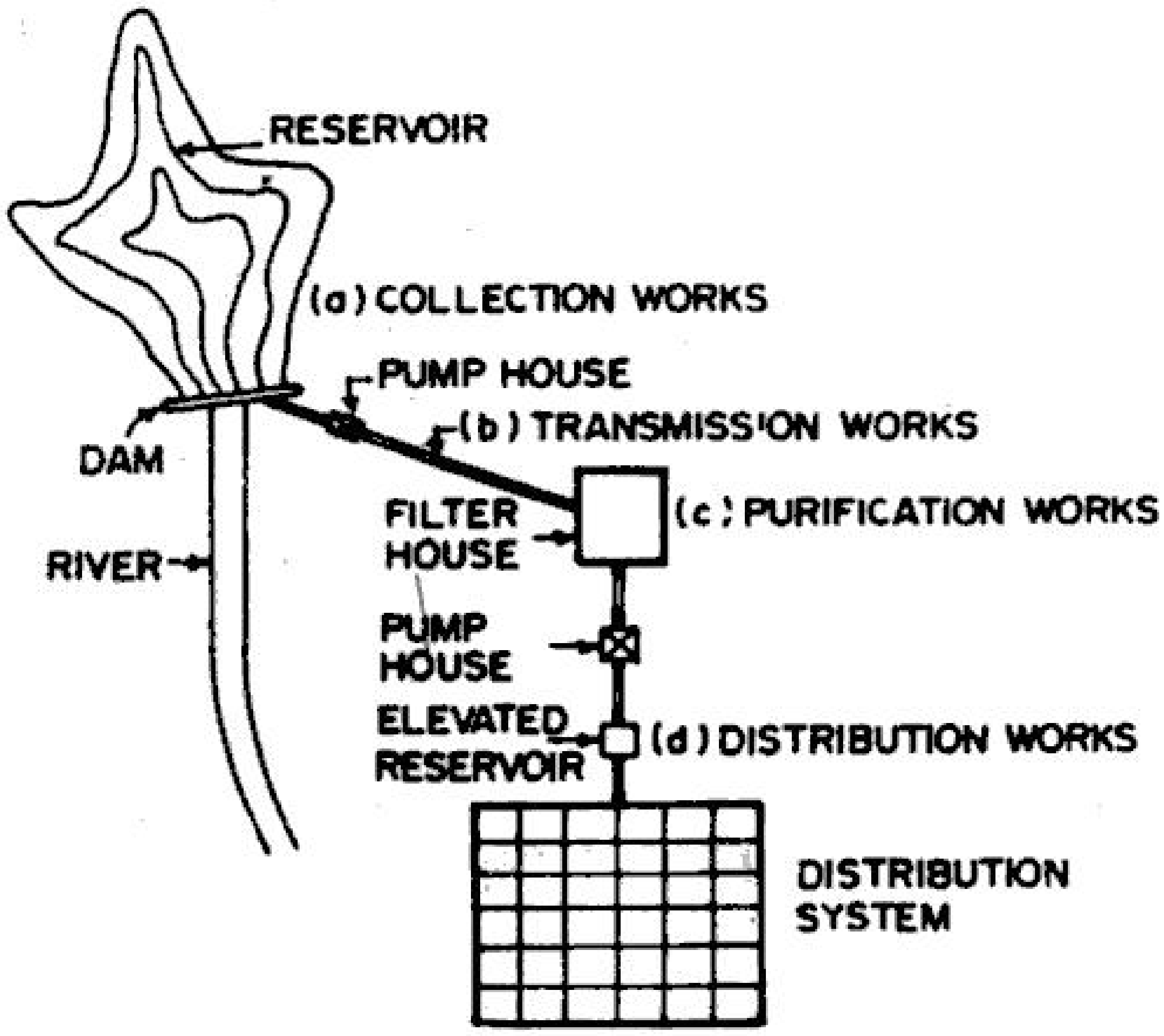


FIG. 1.4. WATER SUPPLY SYSTEMS

1. Collection Works

Collection works are meant for the development of surface water or ground water resources. For major cities, or where water requirements are large, water is collected from a surface source—mostly a river or stream. If the river is perennial, a direct intake structure can be built on the river bank. If, however, river is not perennial, a dam is built across the river so that water is stored in the reservoir. Water is then drawn from the reservoir as per needs. The collection works, therefore, consist of a storage or diversion work, and an intake structure. The planning and design of collection works have been discussed in Chapter 3.

2. Transmission Works

In many cases, the collection works may be far away from the city where water is to be supplied. In that case, water is conveyed to the city through the transmission works. These form the connecting link between the collection works and the purification works. Depending upon the topography of the area between the two sites, the transmission works may be in the form of conduits, canals or aqueducts. For simply gravity flow, canals are generally used. However, if highlands intervene, pumping may have to be resorted to.

3. Purification Works

The water collected directly from the source may not be safe for drinking because of physical, chemical and biological impurities. The municipal water works must deliver to the consumer the water that is : (1) hygienically safe, (2) esthetically attractive and palatable, and (3) economically satisfactory for its intended use. Diseases like typhoid, cholera, dysentery etc. are water borne diseases. The principal aim of the purification works is to supply clean and bacteria free water. The common components of water purification works are : (*i*) *Filtration plants* to remove objectionable colour, turbidity, bacteria and other harmful organisms, (*ii*) *Deferrization and demanganization plants* to remove excessive amounts of iron and manganese, and (*iii*) *Softening plants* to remove excessive amounts of scale forming, soap consuming ingredients like calcium and magnesium ions.

4. Distribution Works

The treated and purified water is finally sent to the consumers through suitable distribution system. In order that water may flow in the water supply pipes under pressure, the purified water is normally stored in an elevated service reservoir. More than one reservoir may be needed in large systems. There are two patterns of water distribution system : (*i*) branching pattern with dead ends, and (*ii*) grid iron pattern. The plan, topography and location of the area with respect to the service reservoir establish the type of distribution system and its character of flow. The design of distribution system has been discussed in Chapter 16.

2

Hydrology

2.1. THE WATER CYCLE

Hydrology is the science which deals with the occurrence, distribution and movement of water on the earth, including that in the atmosphere and below the surface of the earth. Water occurs in the atmosphere in the form of vapour, on the surface as water, snow or ice and below the surface as ground water occupying all the voids within a geologic stratum.

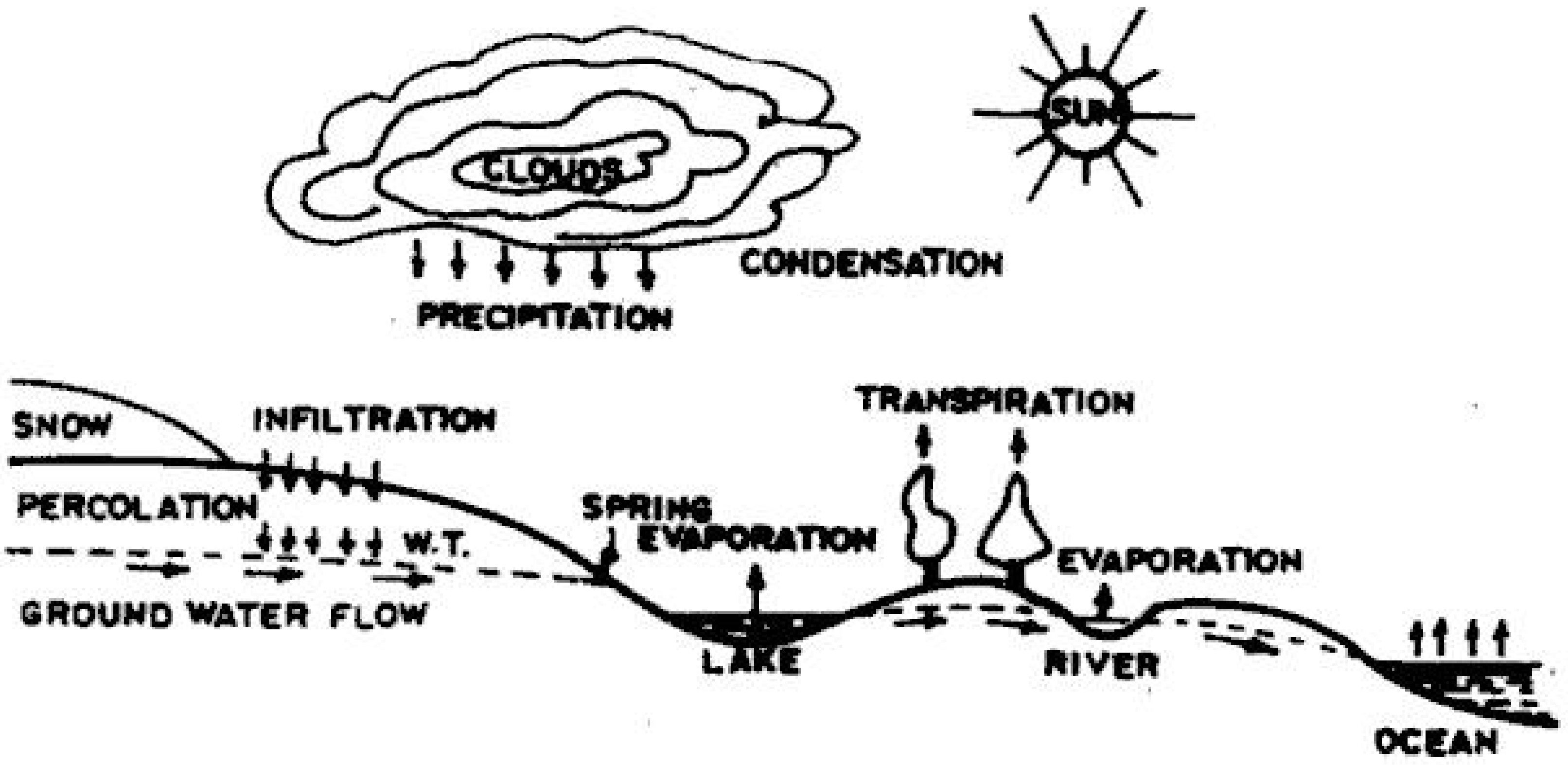


FIG. 2.1. THE WATER CYCLE

Except for the deep ground water, the total water supply of earth is in constant circulation from earth to atmosphere, and back to the earth. The earth's water circulatory system is known as the *water cycle* or the *hydrologic cycle*. Water circulates naturally through five principal realms—(*i*) oceans, (*ii*) atmosphere, (*iii*) lakes and rivers, (*iv*) ice caps and glaciers, and (*v*) underground. Hydrology concerns water and its behaviour in all these realms. *Hydrologic cycle* or the *water cycle* is the process of transfer of moisture from atmosphere to the earth in the form of precipitation, conveyance of the precipitated water by streams and rivers to ocean and lakes etc., and evaporation of water back to the atmosphere. Fig. 2.1 illustrates, diagrammatically, the complete hydrologic cycle.

The hydrologic cycle consists of the following processes:

1. Evaporation and Transpiration (*E*)

The water from the surfaces of ocean, rivers, lakes and also from the moist soil evaporates. The vapours are carried over the land by air in the form of clouds. Transpiration is the process of water being lost from the leaves of the plants from their pores. Thus, the total evaporation (*E*), inclusive of the transpiration consists of :

(*i*) Surface evaporation
(*ii*) Water surface evaporation
  (*a*) From river surface
  (*b*) From oceans
(*iii*) Evaporation from plants and leaves (transpiration)
and (*iv*) Atmospheric evaporation.

2. Precipitation (*P*)

Precipitation may be defined as the fall of moisture from the atmosphere to the earth surface in any form. Precipitation may be in two forms:

(*a*) *Liquid Precipitation* : *i.e.* rainfall.
(*b*) *Frozen Precipitation* : This consists of :
  (*i*) Snow (*ii*) Hail
  (*iii*) Sleet (*iv*) Freezing rain.

3. Run-off (*R*)

Run-off is that portion of precipitation that is not evaporated. When moisture falls to the earth's surface as evaporation, a part of it is evaporated from the water surface, soil and vegetation and through transpiration by plant, and the remainder precipitation is available as run off which ultimately runs to the ocean through surface or sub-surface streams. Thus, run off may be classified as follows :

(1) *Surface run off*

Water flows over the land and is first to reach the streams and rivers, which ultimately discharge the water to the sea.

(2) *Interflow or sub-surface run off*

A portion of precipitation infiltrates into surface soil and, depending upon the geology of the basin, runs as sub-surface run-off and reaches the streams and rivers.

(3) *Ground water flow or base flow*

It is that portion of precipitation, which after infiltration, percolates down and joins the ground water reservoir which is ultimately connected to the ocean.

Thus, the hydrologic cycle may be expressed by the following simplified equation.

$$\text{Precipitation} = \text{Evaporation} + \text{Run off}$$

$$(P) = (E) + (R)$$

provided adjustment is made for the moisture held in storage at the beginning and at the end of the period.

2.2. PRECIPITATION

To the hydrologist, precipitation is the general term for all forms of moisture emanating from the clouds and falling to the ground. The following are the essential requirements for precipitation to occur :

1. Some mechanism is required to cool the air sufficiently to cause condensation and droplet growth.

2. Condensation nuclii are also necessary for formation of droplets. They are usually present in the atmosphere in adequate quantities.

3. Large scale cooling is essential for significant amount of precipitation. This is achieved by lifting of air. Thus a meteorological phenomenon of lifting of air masses is essential to result precipitation.

Types of Precipitation

Precipitation is often classified according to the factors responsible for lifting. Broadly speaking, there are four types of precipitation.

(1) Cyclonic precipitation.
(2) Convective precipitation
(3) Orographic precipitation
(4) Precipitation due to turbulent ascent.

1. Cyclonic Precipitation

Cyclonic precipitation results from lifting of air masses converging into low pressure area or cyclone. The cyclonic precipitation may be divided into (*a*) frontal precipitation, and (*b*) non-frontal precipitation.

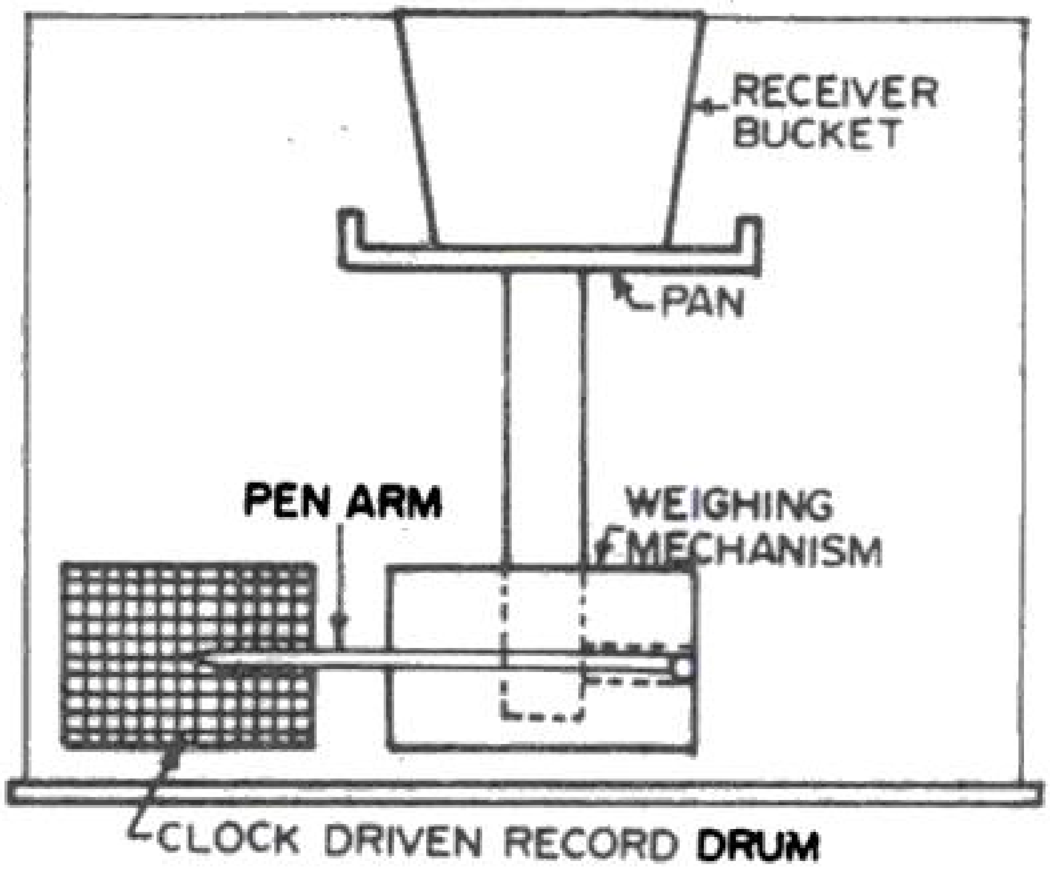


FIG. 2.3. WEIGHING BUCKET RAIN GAUGE

3. Tipping Bucket Type Rain-gauge

The tipping bucket type rain-gauge consists of a 30 cm diameter sharp edge receiver. At the end of the receiver is provided a funnel. A pair of buckets are pivoted under the funnel in such a way that when one bucket receives 0.25 mm (0.01 inch) of precipitation it tips, discharging its contents into a reservoir bringing the other bucket under the funnel. Tipping of the bucket completes an electric circuit

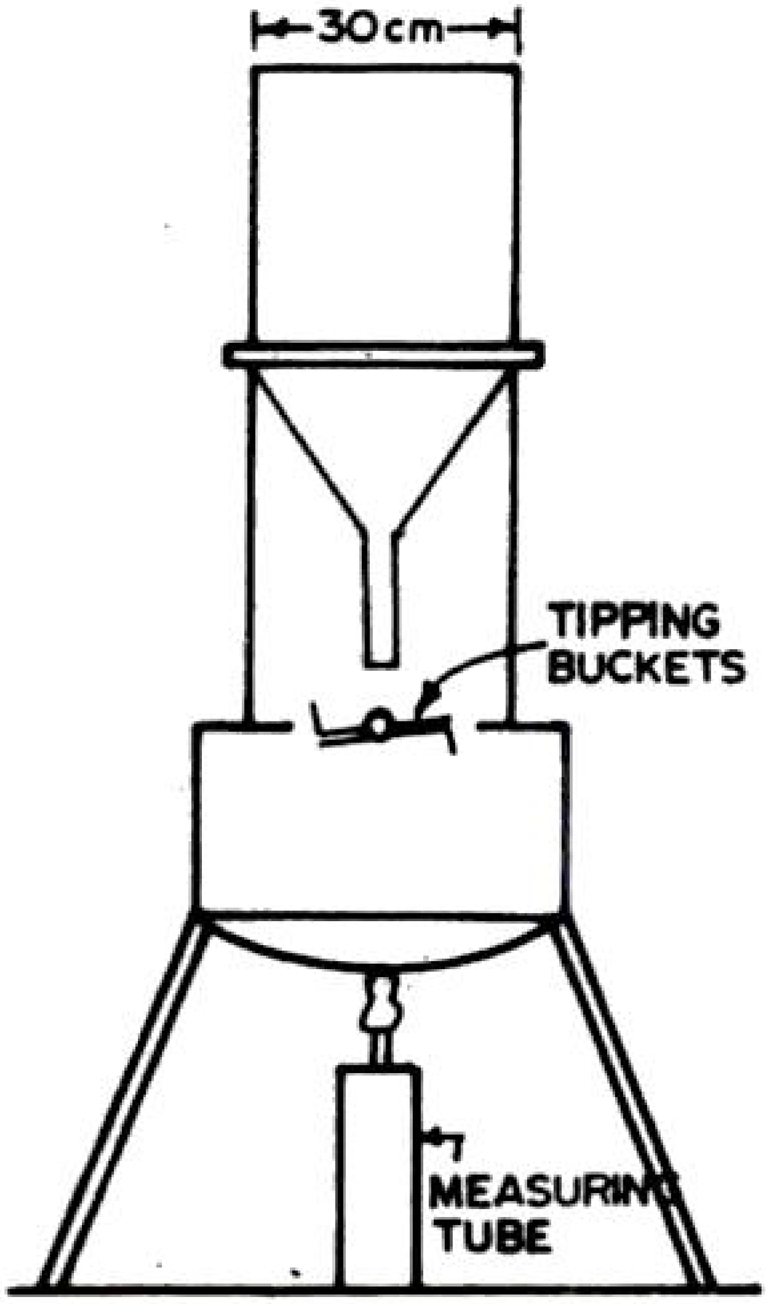


FIG. 2.4. TIPPING BUCKET TYPE RAIN-GAUGE

causing the movement of pen to mark on clock driven revolving drum which carries a record sheet.

4. Float Type Automatic Rain-gauge

The working of a float type rain-gauge is similar to the weighing bucket type gauge. A funnel receives the rain water which is collected in a rectangular container. A float is provided at the bottom of the container. The float is raised as the water level rises in the container, its movement being recorded by a pen moving on a recording drum actuated by a clock-work. When the water level in the container rises so that the float touches the top, the siphon come into operation, and releases the water; thus all the water in the box is drained out.

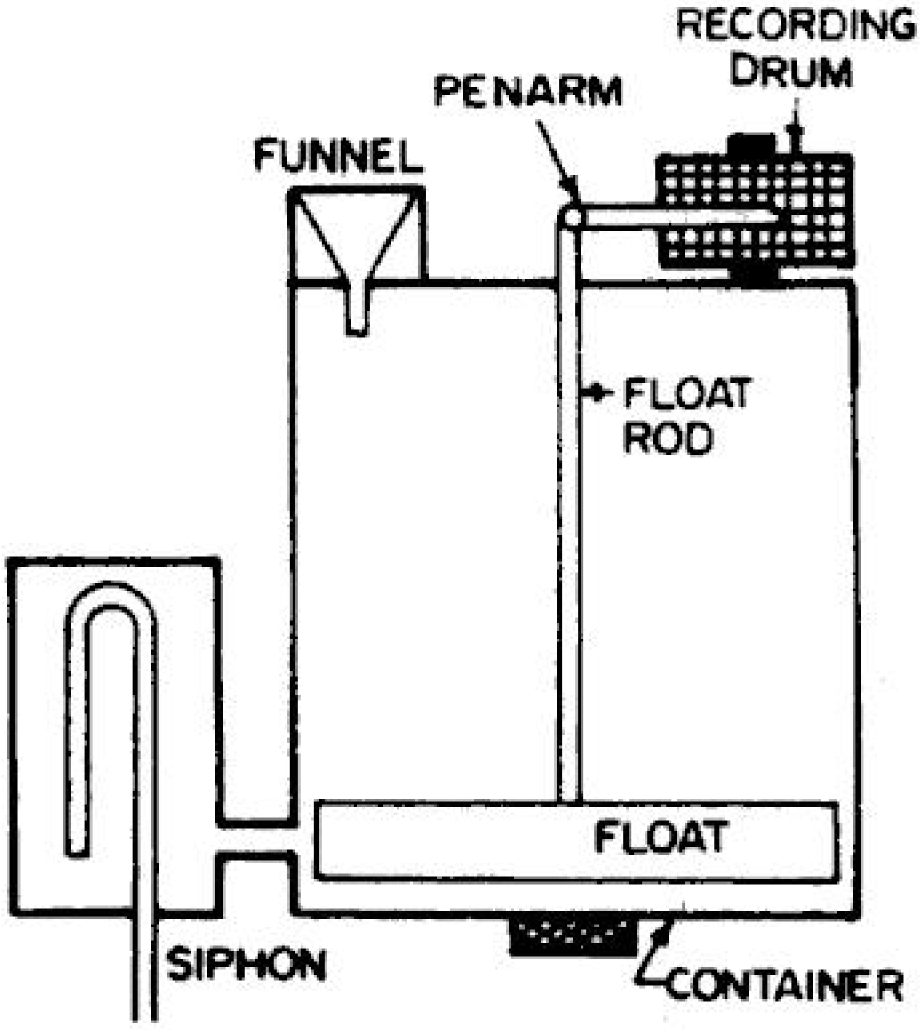


FIG. 2.5. FLOAT TYPE AUTOMATIC RAIN-GAUGE

2.4. COMPUTATION OF AVERAGE RAINFALL OVER A BASIN

If a basin or catchment area contains more than one raingauge station, the computation of average precipitation or rainfall may be done by the following methods :

1. Arithmetic average method.
2. Thiessen polygon method.
3. Isohyetal method.

1. Arithmetic Average Method

If the rainfall is uniformly distributed in its areal pattern, the simplest method of estimating average rainfall is to compute the arithmetic average of the recorded rainfall values at various stations.

Thus, if $P_1, P_2, P_3, \ldots\ldots, P_n$ etc., are the precipitation or rainfall values measured at n gauge stations, we have

$$P_{av} = \frac{P_1 + P_2 + \ldots\ldots + P_n}{n} = \frac{\Sigma P}{n} \qquad \ldots(2.1)$$

2. Thiessen Polygon Method

The arithmetic average method is the most approximate method since rainfall varies in intensity and duration from place to place. Hence the rainfall recorded by each rain-gauge station should be weighted according to the area it is assumed to represent.

Thiessen polygon method is a more common method of weighting the rain-gauge observations according to the area. Thiessen polygon method is also called weighted mean method and is more accurate than the arithmetic average method.

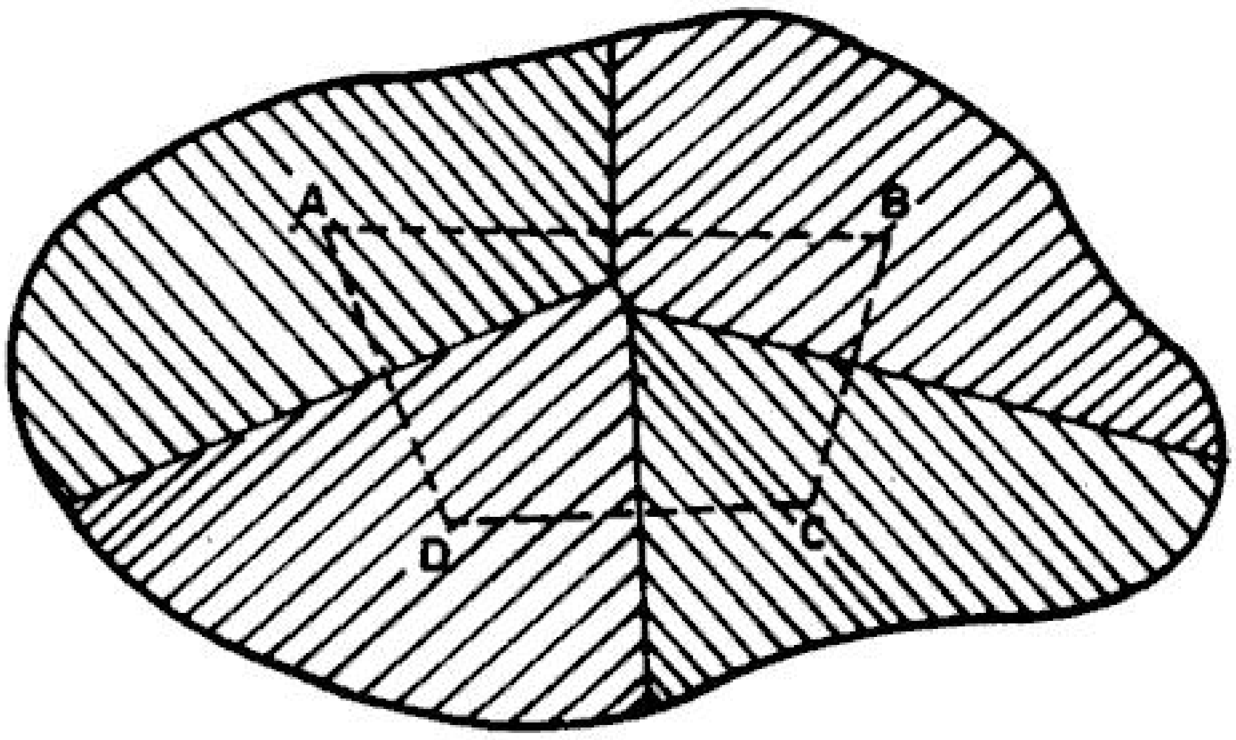


FIG. 2.6. THIESSEN POLYGON METHOD

Procedure

1. Join the adjacent rain-gauge stations A, B, C, D etc., by straight lines.

2. Construct the perpendicular bisectors of each of these lines.

3. A Thiessen network is thus constructed. The polygon formed by the perpendicular bisectors around a station encloses an area which is everywhere closer to that station than to any other station. Find the area of each of these polygons shown hatched in Fig. 2.6.

4. Multiply the area of each Thiessen polygon by the rain-gauge value of the enclosed station.

5. Find the total area (ΣA) of the basin.

6. Compute the average precipitation or rainfall from the equation

C = Constant, the value of which may be taken as 1.1 for deep bodies of water and 1.5 for shallow bodies of water.

K = Constant, the value of which may be upto 16.

2. ***Rohwer's Formula***

$$E = C' (1.465 - 0.00732 p_a)(0.44 + 0.0732 W)(V - v) \quad ...(2.4\,b)$$

where E = Evaporation loss in cms per day (24 hours)

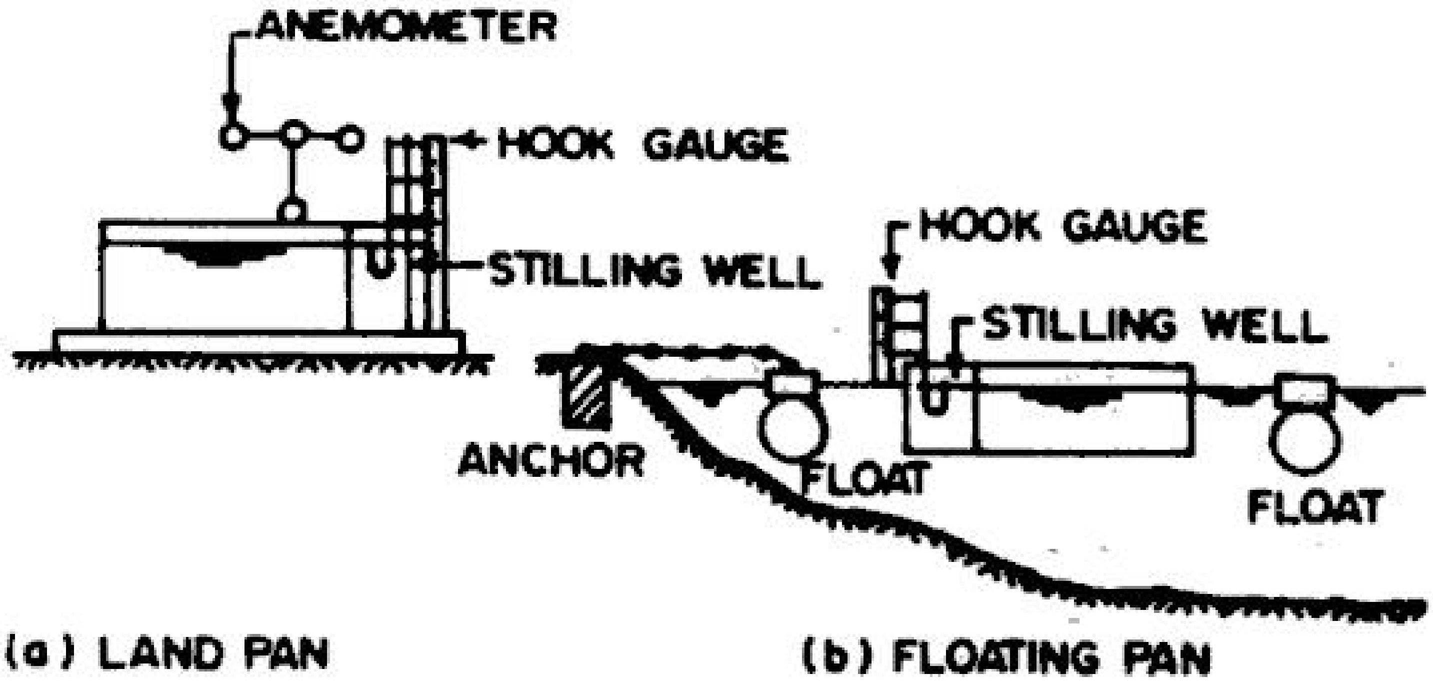
C' = Constant, the value of which may be found out from the available evaporation data; it may be approximately taken equal to 0.75.

p_a = Atmospheric pressure in cms of mercury, at 0°C.

V, v and W have the same meaning as in the Meyer's formula.

Determination of Evaporation from Field Measurements

The evaporation from water surface is generally measured by exposing pans of water to the atmosphere. The water pan may be either installed on the ground near the lake (Fig. 2.8 *a*), or it may be suppported on water surface with the help of floats (Fig. 2.8 *b*). The former is known as *land pan*, while the later is known as a *floating pan*. In each case, the Standard Class *A* pan is a galvanised iron pan of 4′ (1.20 m) diameter and 10″ (25 cm) depth. The land pan is supported on a grid of 5 cm × 10 cm timbers to raise it above the ground to promote air circulation. The floating pan, however, is kept floating on water surface with the help of floats. The change in the water level is measured by a hook gauge provided over a stilling well. Temperature, rainfall, wind speed etc. are also measured. The evaporation measured in the pan in multiplied by a suitable coefficient to get the actual evaporation of the lake surface. This coefficient, known as *pan evaporation coefficient*, varies from 0.6 to 0.8.



(a) LAND PAN (b) FLOATING PAN

FIG. 2.8. MEASUREMENT OF EVAPORATION

2.6. RUN-OFF

The run-off of a catchment area in any specified period is the total quantity of water draining into a stream or into a reservoir in that period. This can be expressed as (*i*) centimetres of water over a catchment, or (*ii*) the total water in cubic-metres or hectare-metres for a given catchment.

The rainfall is disposed of in the following manner:

1. Basin Recharge.
2. Direct run-off (or simply, run-off).
3. Percolation down to ground water.
4. Evaporation.

1. *Basin Recharge*

The basin recharge consists of the following :

(*i*) Rain intercepted by leaves and stems of vegetation.

(*ii*) Water held up in surface depressions, commonly known as the depression storage.

(*iii*) Soil moisture held as capillary water in pore spaces of soil or as hygroscopic water absorbed on the surface of soil particles.

2. *Direct Run-off*

Direct run-off is that water which reaches the stream shortly after it falls as rain. Direct run-off consist of:

(*i*) Over land flow (a surface run-off).

(*ii*) Inter-flow (Influent stream).

Overland flow is that portion of water which travels across the ground surface to the nearest stream. However, if the soil is permeable, water percolates into it, and when it becomes saturated, flows laterally in the surface soil to a stream channel. The essential condition for inter-flow is that the surface soil is permeable, but the subsoil is relatively impermeable so that water does not percolate deep to meet the ground water.

3. *Percolation down to Ground Water (base flow)*

If the subsoil is also permeable, water percolate deep down-wards to meet the ground water. Much of the low water flow in rivers is derived from the ground water. Stream channels which are below the ground water are called effluent streams.

2.7. COMPUTATION OF RUN-OFF

The run-off from a catchment can be computed daily, monthly or yearly. Following are some of the methods of computing the run-off :

(*a*) By formulae and tables.
(*b*) By Infiltration method.
(*c*) By Unit Hydrograph.
(*d*) By Rational method.

(*a*) RUN-OFF BY FORMULAE AND TABLES

1. Run-off Coefficient

The run-off and the rainfall can be inter-related by run-off coefficient, by the expression

$$R = k\,P \qquad \text{...(2.5)}$$

where R = run-off in cm,
P = rainfall in cm,
k = run-off coefficient.

The run-off coefficient naturally depends upon all the factors which affect the run-off. This method is used only for small water control projects, and should be avoided for the analysis of major storms. The customary values of k are given below :

Area	k
Urban residentials	
Single Houses	... 0.3
Garden apartments	... 0.5
Commerical and Industrial	... 0.9
Forested areas, depending on soil	... 0.05–0.2
Parks, farm land, pasture	... 0.05–0.3
Asphalt or concrete pavement	... 0.85.

Barlow's Table

T.G. Barlow carried out studies of catchments mostly under 140 sq. km in the United Provinces (U.P.) and gave the following values of k (in percentage) for various classified catchments :

TABLE 2.1 BARLOW'S TABLE

Class	*Description of Catchment*	*Run-off per cent*
A	Flat, cultivated and black cotton soils	10
B	Flat, partly cultivated, various soils	15
C	Average	20
D	Hills and plains with little cultivation	35
E	Very hilly and steep, with hardly any cultivation	45

The above values of run-off percentages are for average monsoon. These are to be multiplied by the following coefficients (Table 2.9) according to the nature of the season.

TABLE 2.2 BARLOW'S COEFFICIENT

Nature of season	*Class of Catchment*				
	A	*B*	*C*	*D*	*E*
(1) Light rain, no heavy downpour	0.70	0.8	0.8	0.8	0.8
(2) Average or varying rainfall, no continous downpour	1.0	1.0	1.0	1.0	1.0
(3) Continuous downpour	1.50	1.50	1.60	1.70	1.80

2. Strange's Tables and Curves

W.L. Strange gave tables and curves for run-off resulting from rainfall in the plains of South India. The following tables and graphs are for the former Bombay Presidency. Strange's tables and curves give run-off for daily rainfall, and take into account three types of catchment, (*i.e.* good, average and bad), and three surface conditions, (*i.e.* dry, damp and wet) prior to the rain.

TABLE 2.3 DAILY RUN-OFF ACCORDING TO STRANGE
NOTE : FOR GOOD OR BAD CATCHMENT, ADD OR DEDUCT UPTO 25% OF YIELD.

Daily rainfall in (mm)	*Run-off percent and yield when the original state of ground was*					
	Dry		*Damp*		*Wet*	
	%	*Yield in mm*	*%*	*Yield in mm*	*%*	*Yield in mm*
6.25					8	0.5
12.5			6	0.75	12	1.5
18.75			8	1.50	16	3.0
25	3	0.75	11	2.75	18	4.5
31.25	5	1.56	14	4.37	22	6.88
37.5	6	2.25	16	6.00	25	9.37
43.75	8	3.5	19	8.31	30	13.13
50	10	5.0	22	11.00	34	17.00
62.5	15	9.37	29	18.15	43	26.90
75	20	15.00	37	27.75	55	41.25
100	30	30.00	50	50.00	70	70.00

3. Inglis's formula

C.C. Inglis gave the following formulae, derived from data collected from 37 catchments in the Bombay Presidency :

For Ghat areas

$$R = 0.85\,P - 12'' \quad \text{(when } R \text{ and } P \text{ are in inches)} \qquad \ldots(2.6\ a)$$

$$R = 0.85\,P - 30.5 \quad \text{(when } R \text{ and } P \text{ are in cm)} \qquad \ldots(2.6)$$

For Non-Ghat areas

$$R = \left(\frac{P-7}{100}\right) \times P \quad \text{(when } R \text{ and } P \text{ are in inches)} \qquad \ldots(2.7\ a)$$

$$R = \frac{(P-17.8)}{254} \times P \quad \text{(when } R \text{ and } P \text{ are in cm)} \qquad \ldots(2.7)$$

4. Lacey's formula

$$R = \frac{P}{1 + \dfrac{120\,F}{PS}} \quad \text{(when } P \text{ and } R \text{ are in inches)} \qquad ..(2.8\ a)$$

and

$$R = \frac{P}{1 + \dfrac{304.8\,F}{PS}} \quad \text{(when } P \text{ and } R \text{ are in cm)} \qquad \ldots(2.8)$$

In both the expressions,
S = A catchment factor and F = Monsoon duration factor.

Corresponding to the five classes of catchment, defined by Barlow, Lacey gave the following values of the catchment factor (S) :

TABLE 2.4

Class of Catchment	*Value of S*
A	0.25
B	0.60
C	1.00
D	1.70
E	3.45

Lacey also divided the monsoon into three classes, depending upon its duration and gave the following values of monsoon duration factors :

TABLE 2.5

No	*Class of monsoon*	*Monsoon duration factor (F)*
1	Very Short	0.50
2	Standard Length	1.00
3	Very Long	1.50

(5) Khosla's formula

$$R = P - \frac{T-32}{9.5} \text{ (when } R \text{ and } P \text{ are in inches)} \quad \ldots(2.9\,a)$$

$$R = P - \frac{T-32}{3.74} \text{ (when } R \text{ and } P \text{ are in cm)} \quad \ldots(2.9)$$

where T = mean temperature in °F on the entire catchment.

The temperature introduced in the formula takes into account various factors affecting losses by evaporation, transpiration, sunshine and wind velocity.

(6) Parker's formulae

(*i*) $R = 0.94\,P - 14$ for catchments in the British Isles
(*ii*) $R = 0.94\,P - 16$ for catchments in Germany ...(2.10)
(*iii*) $R = 0.80\,P - 16.5$ for catchments in East U.S.A.

where R and P are in inch units.

(*b*) RUN-OFF BY INFILTRATION METHOD

Infiltration is defined as the movement of water through the soil surface and into the soil. The capacity of any soil to absorb water from rainfall falling continuously at an excessive rate goes on decreasing with time, until a minimum rate of infiltration is reached. At any instant, the *infiltration capacity* of a soil is the maximum rate at which water will enter the soil in a given condition. The *infiltration rate* is the rate at which water actually enters the soil during a storm, and is equal to the infiltration capacity or the rainfall rate, whichever is less. The *infiltration capacity* of soil can be determined experimentally by subjecting an experimental plot to rainfall rates

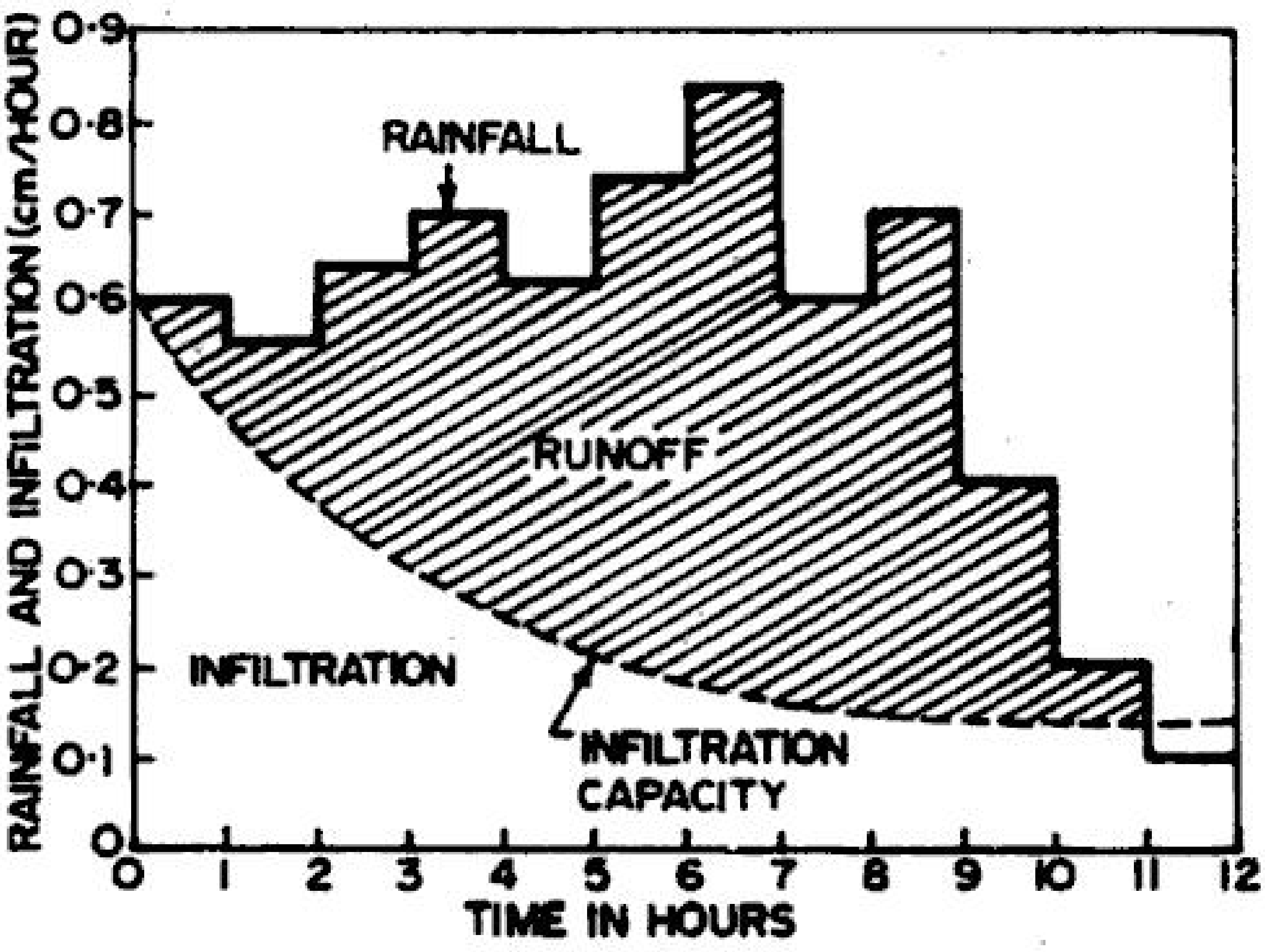


FIG. 2.9. INFILTRATION CAPACITY CURVE AND RUN-OFF

in excess of infiltration capacity, and by measuring the surface run-off (Fig. 2.9). For small areas having uniform infiltration characteristics, the run-off volume can be estimated by subtracting infiltration from the design rainfall.

Infiltration Index

Infiltration index is the average rate of loss such that the volume of rainfall in excess of that rate will be equal to direct run-off. Estimates of run-off volume from large areas, having heterogeneous infiltration and rainfall characteristics, are made by use of *infiltration indices.*

There are two types of infiltration indices.

(*i*) *Average infiltration rate* or W-index and (*ii*) ϕ index.

The W-index is calculated from the expression

$$W = \frac{P - R}{t_r} \text{ cm/hr} \qquad \text{...(2.11)}$$

where t_r = duration of rainfall in hours.

The ϕ-index is defined as the rate of rainfall above which the rainfall volume equals the run-off volume (Fig. 2.10).

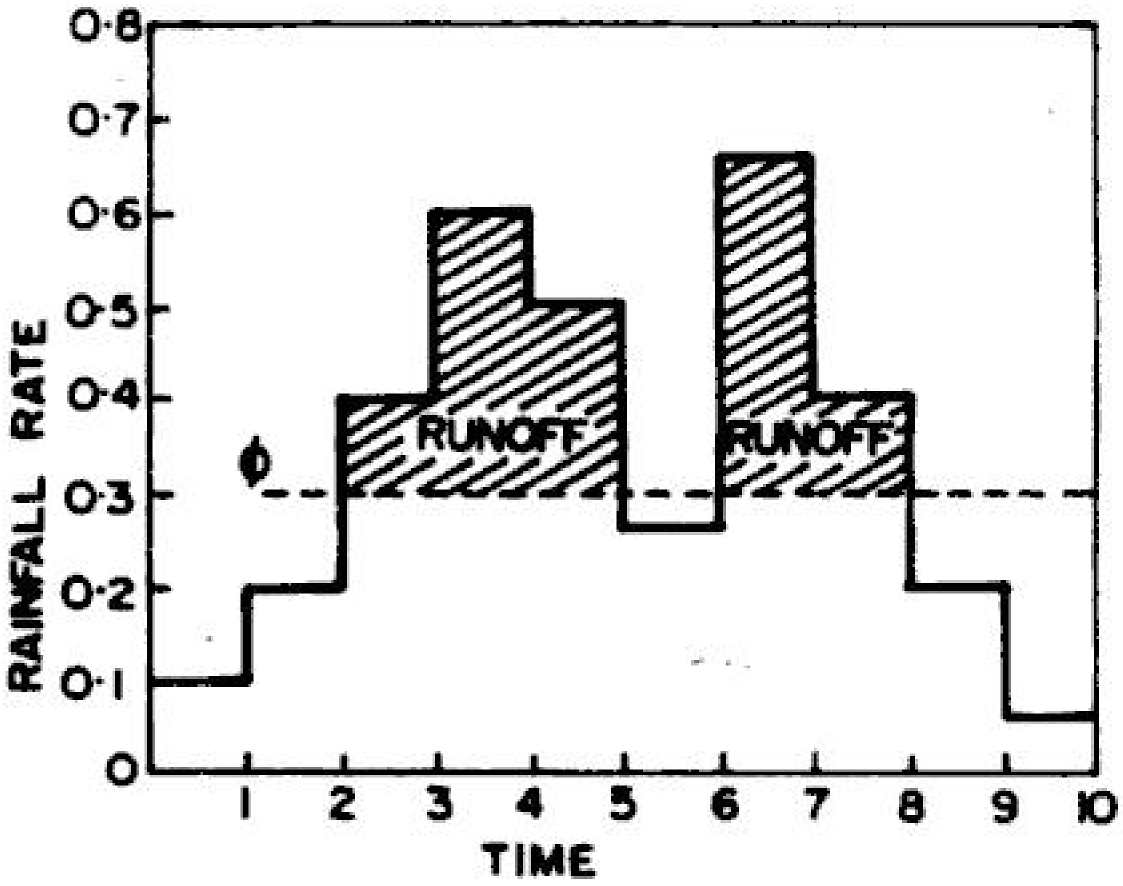


FIG. 2.10. ϕ INDEX AND RUN-OFF

For flood forecasting, appropriate index values must be derived by correlation with those factors which determine the index at any time. In such approach, there seems to be no advantage over the method discussed earlier, in which the run-off and rainfall are correlated. However, the infiltration index can be used to estimate the run-off co-efficient (k) from the relation

$$k = \frac{i - W}{i} \qquad \text{...(2.12)}$$

where i = rainfall intensity (cm/hr)

2.8. ESTIMATION OF MAXIMUM RATE OF RUN-OFF OR FLOOD DISCHARGE

The estimation of peak flow or flood can be made by the following methods :

(*a*) By physical indication of past floods.
(*b*) By flood discharge formulae.
(*c*) By flood frequency studies.
(*d*) By unit hydrograph

The method of flood frequency and the method of unit hydrograph can be found in any standard text book on Hydrology.

(*a*) PHYSICAL INDICATION OF PAST FLOODS

Ancient monuments, etc., situated on river banks always bear past flood marks. Old persons in the villages situated on the bank of the river may be contacted to know the maximum water level attained in the past 35 years. The cross-section of the river may be plotted, and the water line corresponding to the highest flood can be drawn on it. From such a cross-section, the water-flow-area, wetted perimeter and hydraulic mean depth can be calculated. By longitudinal sectioning with the help of levelling slightly to the upstream and downstream of the site where cross-section has been plotted, the longitudinal slope of the bed of the river can be determined. Assuming this to be the same as the hydraulic slope during the past flood, the mean velocity of flow can be computed by Chezy's or any other suitable hydraulic formula. This velocity can be multiplied with the probable area of water section at the time of past flood to calculate the flood discharge. This procedure should be repeated at several villages or water marks, to get consistent results.

(*b*) FLOOD DISCHARGE FORMULAE

Some of the empirical formulae for estimating the flood discharge are given below. Most of these are in the form

$$Q = CA^n \qquad \text{...(2.13)}$$

where Q = flood discharge
A = catchment area
n = flood index
C = flood coefficient.

Both C and n depend upon various factors, such as (*i*) size, shape and location of catchment, (*ii*) topography of the catchment, and (*iii*) intensity and duration of rainfall, and distribution pattern of the storm over the basin.

1. Dicken's Formula

In metric unit,

$$Q = CA^{3/4} \qquad ...(2.14)$$

where Q = discharge in cumecs
A = area of basin in sq. km.

In F.P.S. units

$$Q_1 = C_1 A_1^{3/4} \qquad ...(2.14\ a)$$

where Q_1 = discharge in cusecs
A_1 = area of basin in sq. miles.

The constants C and C_1 depend upon the catchment and may be obtained from Table 2.6.

TABLE 2.6

Region	C	C_1
Northern India	11.4	825
Central India	13.9–19.5	1000 to 1400
Western Ghats	22.2–25	1600 to 1800

2. Ryve's Formula

For Madras catchments

$$Q = CA^{2/3} \qquad ...(2.15)$$

(where Q is in cumecs and A in sq. km.)

and $$Q = C_1 A_1^{2/3} \qquad ...(2.15\ a)$$

(where Q_1 is in cusecs and A_1 in sq. miles)

values of C and C_1 may be obtained from Table 2.7

TABLE 2.7

	Location of the Catchment	C	C_1
1.	Areas within 24 km (15 miles) from the coast.	6.75	450
2.	Area within 24 km –161 km (25 to 100 miles) from the coast)	8.75	563
3.	Limited areas near hills	10.1	672

3. Inglis Formula

Inglis formula is applicable for catchments of former Bombay Presidency.

Expressed in metric units,

$$Q = \frac{123\,A}{\sqrt{A + 10.4}} \approx 123\,A^{1/2} \qquad \text{...(2.16)}$$

Expressed in F.P.S. units,

$$Q_1 = \frac{7000\,A_1}{\sqrt{A_1 + 4}} \approx 7000\,A_1^{1/2} \qquad \text{...(2.16 } a\text{)}$$

4. Nawab Jang Bahadur Formula

For catchments of old Hydrabad State :

Expressed in metric units,

$$Q = CA^{\left(0.993 - \frac{1}{14}\log A\right)} \qquad \text{...(2.17)}$$

where C varies from 48 to 60.

Expressed in F.P.S. units,

$$Q_1 = C_1 A_1^{\left(0.92 - \frac{1}{14}\log A_1\right)} \qquad \text{...(2.17 } a\text{)}$$

where C_1 varies from 1600 to 2000.

5. Fanning's Formula

For American catchments,

In metric units,

$$Q = CA^{5/6} \qquad \text{...(2.18)}$$

where average value of C_1 may be taken equal to 2.54.

In F.P.S. units,

$$Q_1 = C_1 A_1^{5/6} \qquad \text{...(2.18 } a\text{)}$$

where average value of C_1 may be taken equal to 200.

6. Creager's Formula

Applicable for American catchments

Expressed in F.P.S. units,

$$Q_1 = 46\,C_1 A_1^{(0.894 A_1 - 0.048)} \qquad \text{...(2.19)}$$

The constant varies from 30 to 100.

7. Fuller's Formula

This formula takes into account the flood frequency also. Expressed in metric units,

$$Q_{max} = CA^{0.8}(1 + 0.8 \log T)(1 + 2.67A^{-0.3}) \qquad ...(2.20)$$

where T = Number of years after which such a flood is to reoccur.

Q = Maximum flood (in cumecs) during any part of the day that could occur in T-years.

A = Area of drainage basin, in sq. km.

C = Constant varying from 0.185 to 1.3.

Expressed in F.P.S. units,

$$Q_1 = C_1 A_1^{0.8}(1 + 0.8 \log T)(1 + 2A_1^{-0.3}) \qquad ...(2.20\ a)$$

where C_1 varies from 14 to 98.

PROBLEMS

1. Explain with the help of diagram the hydrologic cycle.
2. What do you understand by precipitation ? Explain various types of precipitation.
3. Explain any one type of automatic raingauge.
4. Describe various methods of computing average rainfall over a basin.
5. What are the methods of computing run-off from a catchment area? Give various formulae stating clearly the area for which each is applicable.
6. Explain various methods of determining flood discharge in a stream.
7. Give various flood discharge formulae applicable for Indian catchments.

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$$
\begin{aligned}
\text{Yearly demand} &= 475 \times 365 \text{ cumec-days} \\
&= 475 \times 365 \times 8.64 = 1497960 \text{ ha-m} \\
&\approx 1.5 \text{ million ha-m.}
\end{aligned}
$$

In Fig. 3.8, tangents T_1, T_2, T_3, T_4, T_5 etc. are drawn to the curve at the apexes, and parallel to the demand curve which has a slope

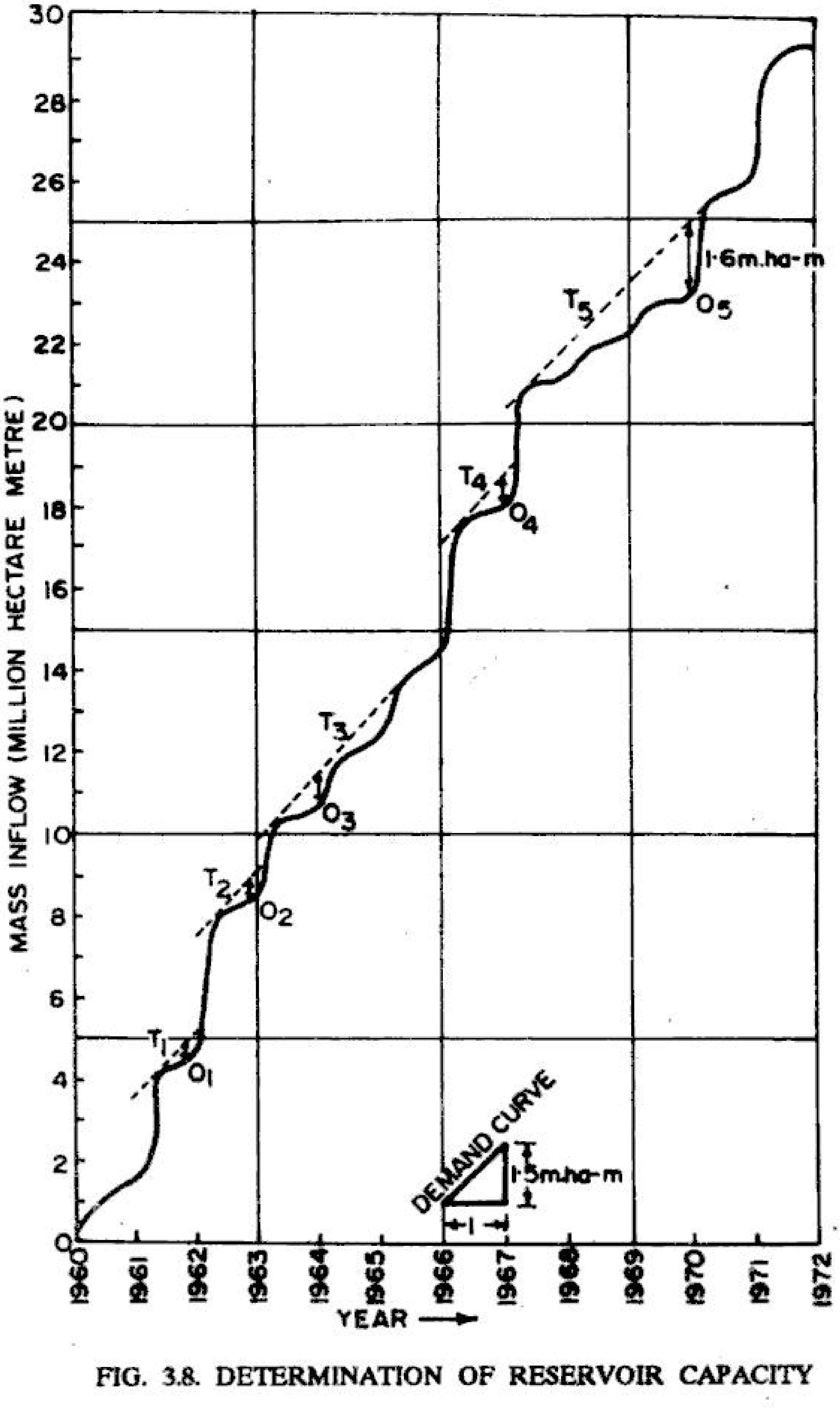


FIG. 3.8. DETERMINATION OF RESERVOIR CAPACITY

of 1.5 m ha-m in 1 year. The ordinates $O_1, O_2, ...O_5$ indicate the deficiencies during the dry periods, assuming that the reservoir was full at the beginning of the period. The maximum of these ordinates (*i.e.* O_5 = **1.6 m ha-m**) gives the desired reservoir capacity.

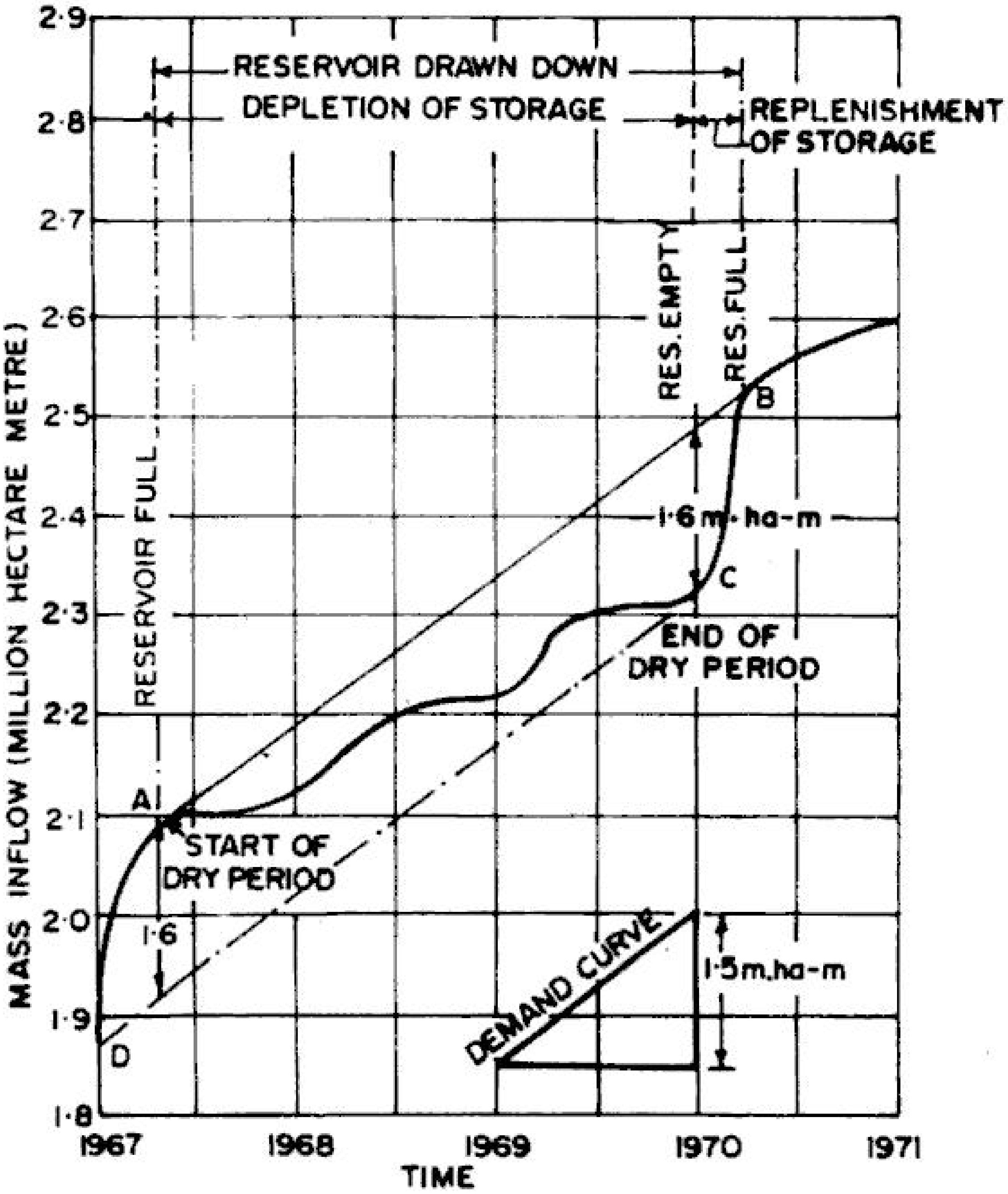


FIG. 3.9. DETERMINATION OF RESERVOIR CAPACITY

Fig. 3.9 shows the enlarged view of the curve from period 1967 to 1971 during which maximum storage is required. Line *AB* is drawn parallel to demand curve and tangential to the mass inflow curve at point *A*. At point *C* of the curve, a storage capacity of 1.6 million hectare metres is required. It is essential that the demand line *AB* should meet the inflow curve at point *B*, so that reservoir becomes full at *B* ; otherwise it will never be full. Similarly, if a line *CD* is drawn parallel to the demand curve, and tangential to the mass-inflow curve at *C*, then it should intersect the curve at *D* so that the reservoir becomes full at the start of the dry period.

Example 3.4 *Draw the mass inflow diagram and compute the storage needed for an impounding reservoir, for a constant draft of 300 million litres per month, with the following recorded mean monthly runoff values :*

TABLE 3.7.

Order of the month	*1*	*2*	*3*	*4*	*5*	*6*	*7*
Observed monthy mean run off (million litres)	*1200*	*1500*	*600*	*100*	*80*	*30*	*10*
Order of the month	***8***	***9***	***10***	***11***	***12***	***13***	***14***
Observed monthly mean runoff (million litres)	*0*	*20*	*60*	*800*	*1000*	*1200*	*700*

Solution.

The computations are shown in Table 3.8, where Q denotes the runoff (or discharge) in the stream and D denotes the draft required, per month.

TABLE 3.8. COMPUTATION FOR STORAGE

Order of month	*Run off* Q	*Draft* D	*Cumulative run off* ΣQ	*Deficiency* $(D - Q)$	*Cumulative Deficiency* $\Sigma (D - Q)$	*Reservoir state*
(1)	*(2)*	*(3)*	*(4)* $= \Sigma$ *(2)*	*(5)* = *(3)* − *(2)*	*(6)* $= \Sigma$ *(5)*	*(7)*
1	1200	300	1200	− 900	0 (2400)	
2	1500	300	2700	− 1200	0 (1500)	
3	600	300	3300	− 300	0 (300) ↑	Res. Full at the beginning of dry period.
4	100	300	3400	200	200	
5	80	300	3480	220	420	
6	30	300	3510	270	690	
7	10	300	3520	290	980	
8	0	300	3520	300	1280	
9	20	300	3540	280	1560	
10	60	300	3600	240	1800*	* Max. deficiency
11	800	300	4400	− 500	1300	
12	1000	300	5400	− 700	600	
13	1200	300	6600	− 900	0 (300)	Res. Full
14	700	300	7300	− 400	0 (700)	

2. ARCH DAMS

An arch dam (Fig. 3.12) is a dam curved in plan and carries a major part of its water load horizontally to the abutments by arch action. This part of water load depends primarily upon the amount of curvature. The balance of the water load is transferred to the foundation by cantilever action. The thrust developed by the water load carried by arch action, essentially requires strong side walls of the canyon to resist the arch forces. The weight of arch dam is not counted on to assist materially in the resistance of external loads. For this reason, uplift on the base is not an important design factor.

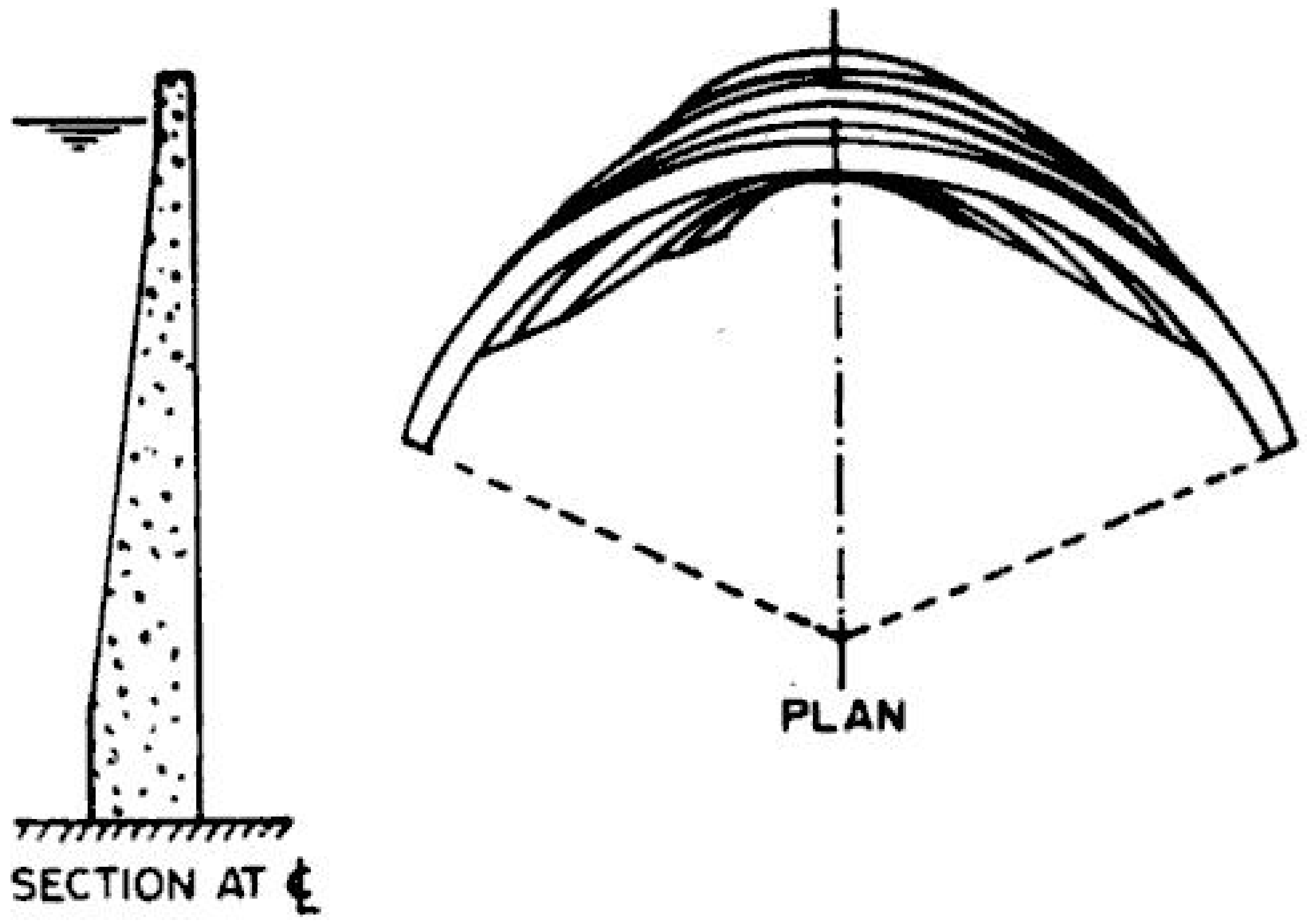


FIG. 3.12. ARCH DAM

3. BUTTRESS DAMS

A buttress dam (Fig. 3.13) consists of a number of buttresses or piers, dividing the space to be dammed into a number of spans. To hold up water and retain the water between these buttresses, panels are constructed of horizontal arches or flat slabs. When the panels consist of arches, it is known as *muliple arches type buttress dam.* If the panels consist of flat slab, it is known as *deck type buttress dam.*

4. STEEL DAMS

Steel dams are constructed with a framework of steel with a thin skin plate as deck slab on the upstream side. In India, no such dam has been constructed. However, in United States three such dams have been constructed : Ash Fork Dam in Arizona (1898), Redridge Dam in Michigan (1905) and Hauser Lake Dam in Montana (1901). Out of these, the first two dams gave satisfactory results

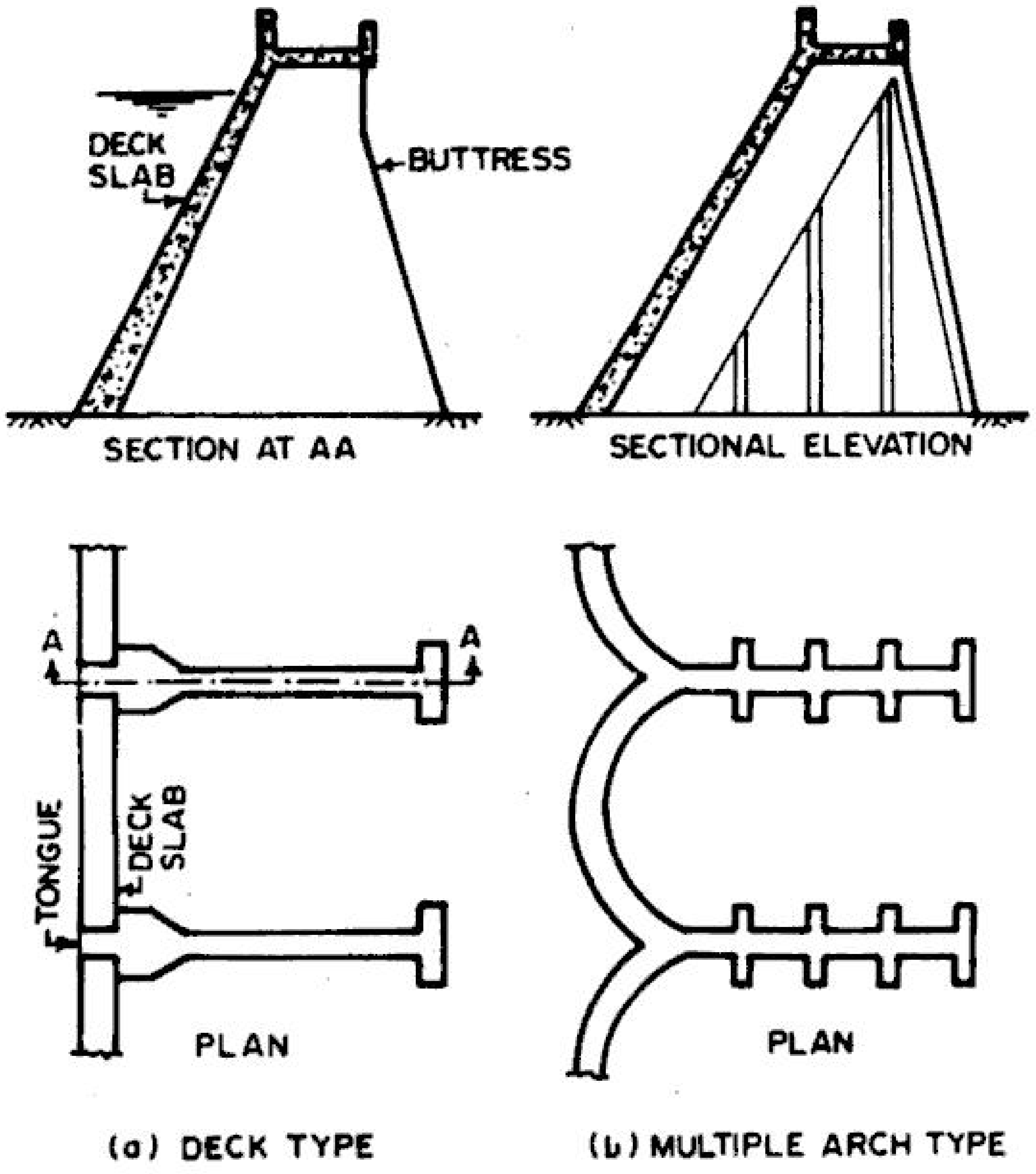


FIG. 3.13. BUTTRESS DAMS

while the third dam failed only after one year of service. The failure was mainly due to undermining of the foundation by leakage through or under the steel sheet pile.

Steel dams (Fig. 314) are generally of two types : (*i*) *direct strutted type* and (*ii*) *cantilever type.* In the direct strutted type, the load on the deck plate is carried directly to the foundations through

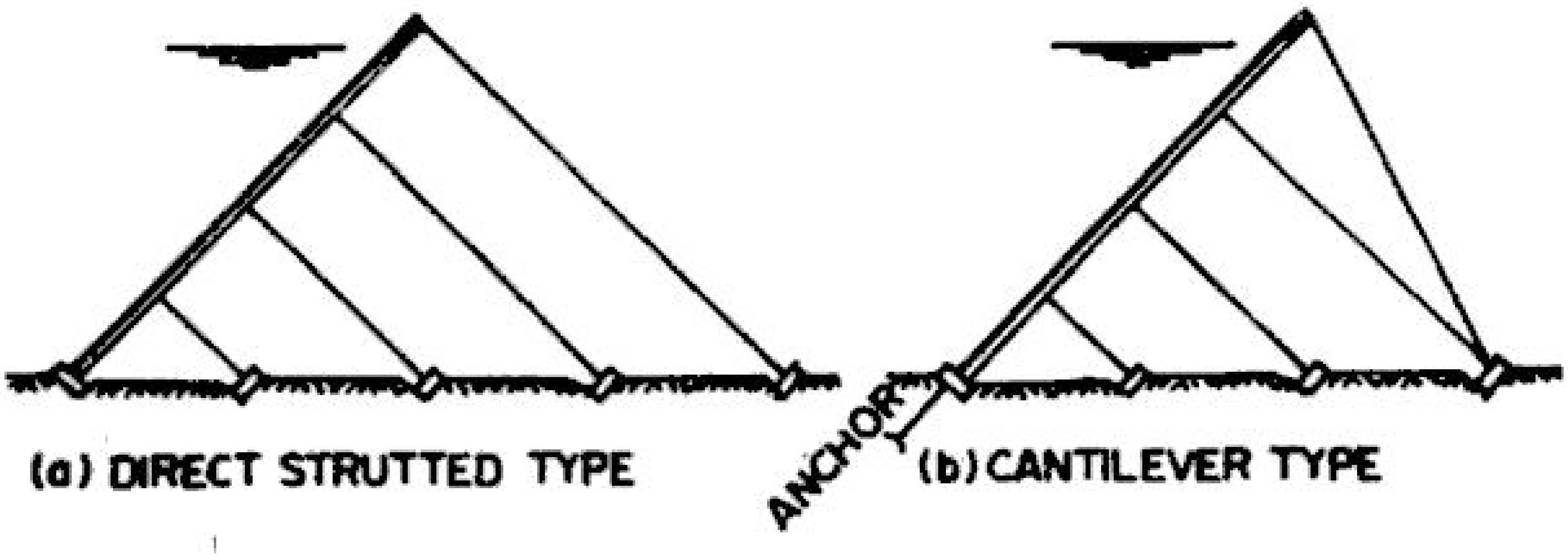


FIG. 3.14. STEEL DAMS

inclined struts. In the cantilever type, the section of the bent supporting the upper part of the deck is formed into a cantilever truss. This arrangement introduces a tensile force in the deck girders which is resisted either by anchoring the deck girder into the foundation at the upstream toe or by framing the entire bent rigidly together so that the moment of the weight of the water on the lower part of the deck may be utilized to offset the moment of the cantilever.

5. TIMBER DAMS

A timber dam is constructed of framework of timber struts and beams, with timber plank facing to resist water pressure. A timber dam is an ideal temporary dam, though a well designed, constructed and maintained timber dam may last 30-40 years. They are suitable to places where timber can be available in plenty.

Timber dams are normally found to be of three types :

1. A-frame type (Fig. 3.15).
2. Rock-filled crib type (Fig. 3.16)
3. Beaver type.

1. A frame type timber dam

Fig. 3.15 shows a typical A-frame type timber dam. It consists of five component parts : (*a*) sills (*b*) struts (*c*) wales (*d*) studs and (*e*) lagging. The sills should be fastened to the ledge rock by wedge bolts or anchor bolts. The lagging should not be of less than 5 cm thickness.

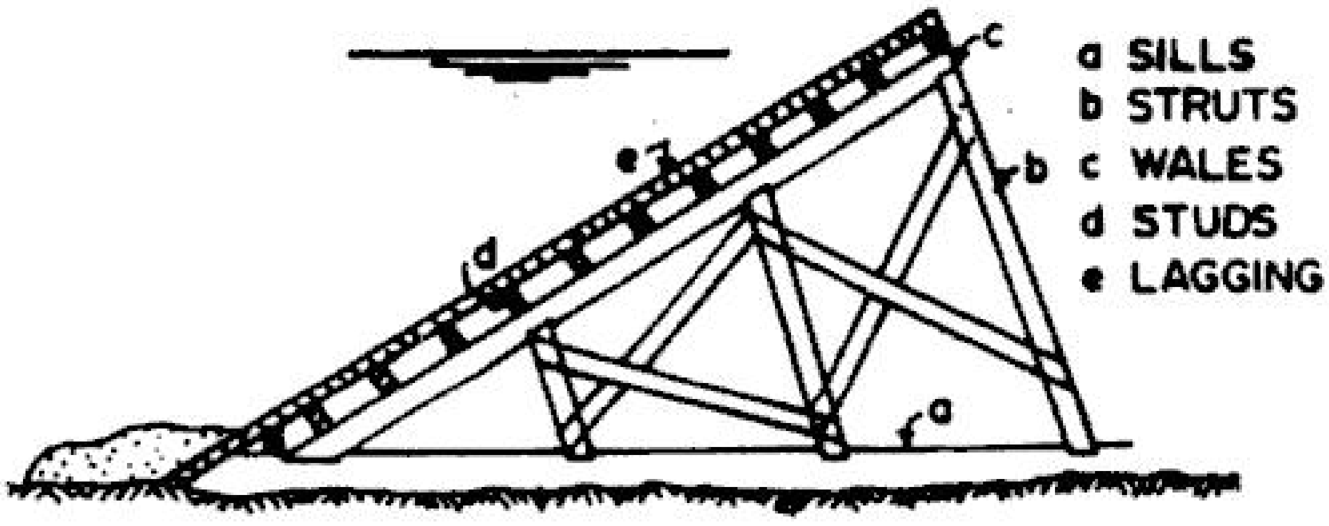


FIG. 3.15. A-FRAME TYPE TIMBER DAM

2. Rockfilled crib type timber dam

Fig. 3.16 shows such a type of dam in which cribs of square or round timber are drift-bolted together. The timbers are spaced about 2–2.5 m centre to centre both ways. The space between them is filled with rock fragments or boulders. In the case of rock foundation, the bottom cribs are pinned to the rock foundation. If, however, the dam is constructed on earth foundation, sheet piling is provided both at the u/s as well as d/s side as shown in Fig. 3.16.

1. The site should be so selected that it may admit water even under worst condition of flow in the river, or under lowest possible water level in a lake or reservoir. If possible, intake should be located sufficiently inside the shore line.

2. Its site should be as near to the treatment work as possible.

3. It should be so located that it admits relatively pure water free from mud, sand or other floating materials. It should be located at a place protected from rapid currents.

4. It should be so located that it is free from the pollution. River intakes should be constructed well upstream of points of discharge of sewage and industrial wastes. If located near a city, it should be located to the upstream of the city so that water is not contaminated.

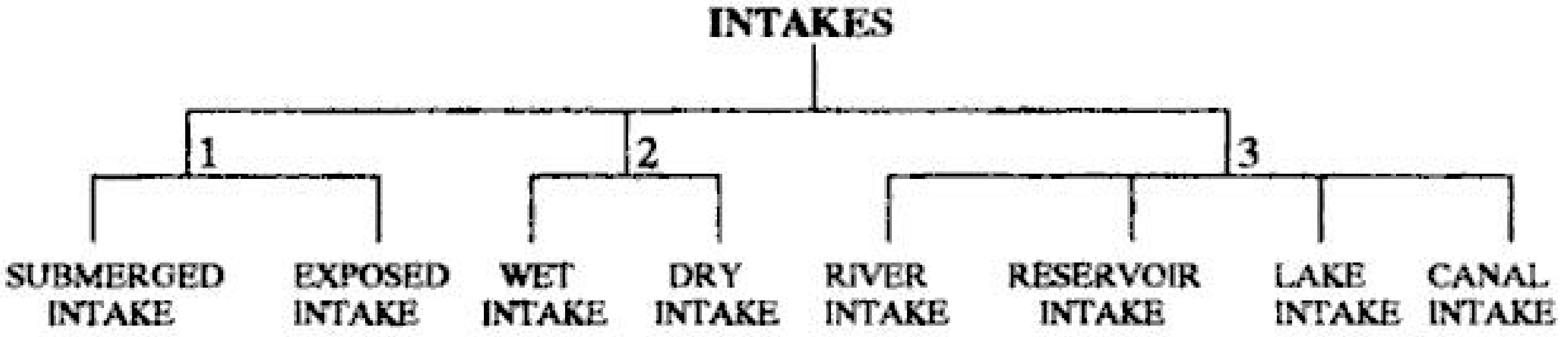
5. It should not interfere with river traffic, if any.

6. The intake should be so located that good foundation conditions are available and the possibility of scouring is the least.

7. It site should be so selected that its further expansion is possible.

Types of Intakes

Intakes are classified under three heads as under :



Submerged intake is the one which is constructed entirely under water. Such an intake (Fig. 3.23) is commonly used to obtain supply from a lake. An *exposed intake* is in the form of a well or tower constructed near the bank of a river, or in some cases even away from the river banks. Exposed intakes are more common due to ease in its operation (Fig. 3.19). A *wet intake* is that type of intake tower in which the water level is practically the same as the water level of the sources of supply. Such an intake is sometimes known as jack well (Fig. 3.20) and is most commonly used. In the case of *dry intake,* however, there is no water in the water tower (Fig. 3.21 *c*). Water enter enters through entry port directly into the conveying pipes. The dry tower is simply used for the operation of valves etc.

River Intakes

A river intake is located to the upstream of the city so that pollution is minimized. They are either located sufficiently inside the river so that demands of water are met with in all the seasons of the year, or they may be located near the river bank where a

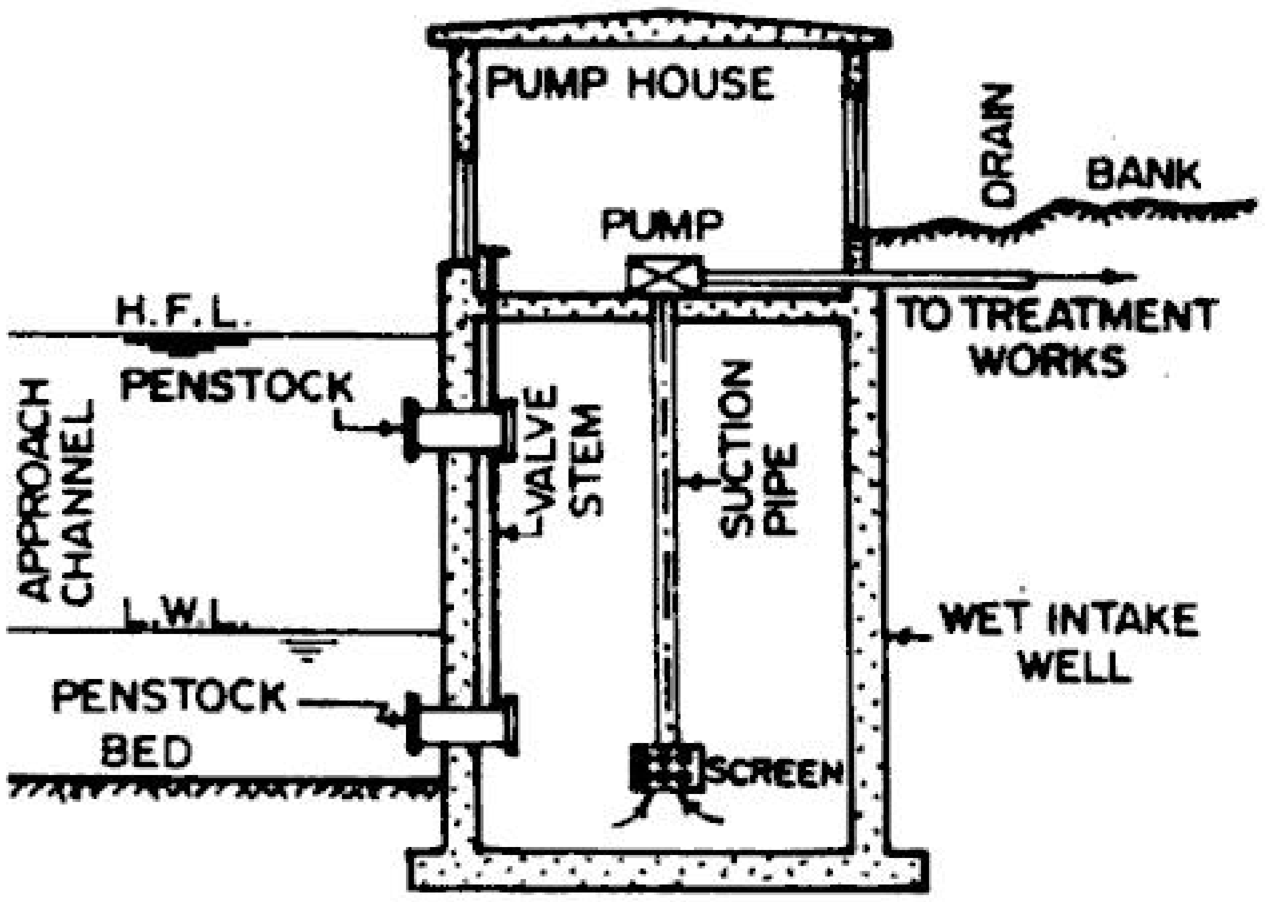


FIG. 3.19. RIVER INTAKE.

sufficient depth of water is available. Sometimes, an approach channel is constructed and water is led to the intake tower. If the water level in the river is low, a weir may be constructed across it to raise the water level and divert it to the intake tower.

Fig. 3.19 shows a wet type intake well founded on river bed. The intake tower permits entry of water through several entry ports located at various levels to cope with the fluctuations in the water level during different seasons. These entry ports are sometimes known as *penstocks* and are provided with suitably designed screens to exclude debris and floating material from entry. The entry ports contain valves which can be operated from the upper part of the well.

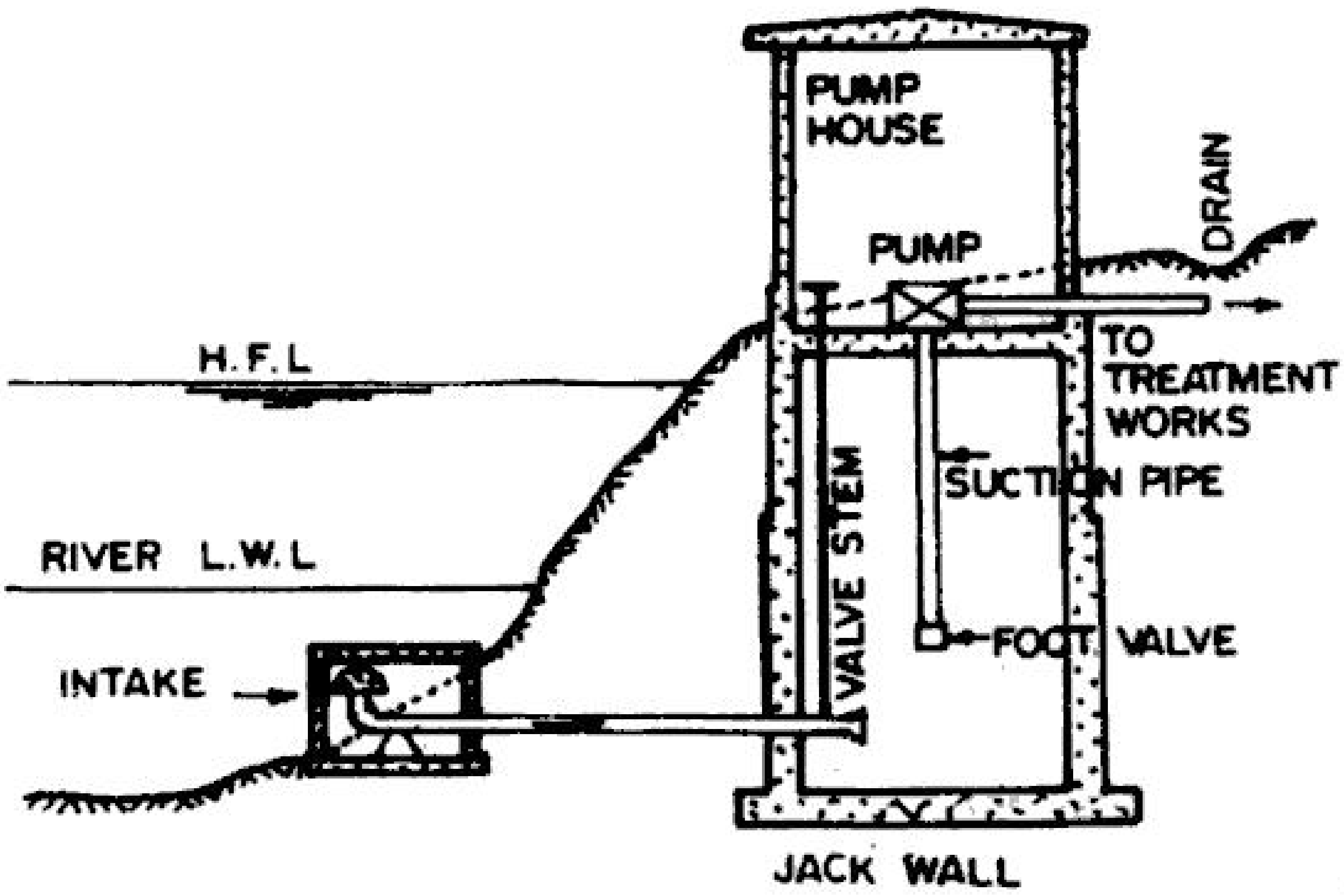


FIG. 3.20. RIVER INTAKE.

The lowest entry is placed below the low water level of the river so that water is available in the jack well during summer season also when river carries minimum discharge. The intake well should be founded on sound footing, to a depth deeper than the scour depth. The upper part of the well serves as the pump house. The suction pipe admits water through a screen.

Where river bed is soft or unstable, the intake tower may be founded slightly away from the river bed, as shown in Fig. 3.20. The intake is kept submerged under the low water level of the river. It essentially consists of a rectangular or circular entry chamber with a strong grill at its top. The pipe conveying water from the intake to the jack well has a bell-mouth entry with a screen, and is supported on a concrete support. While the entry of debris and floating material is checked by the top grill, the entry of mud or coarse sand etc. is checked by the screen provided at the bell-mouth entry. Water enters to the jack well through a valve which can be controlled from the pump house.

Reservoir Intake

When the flow in the river is not guaranteed throughout the year, a dam is constructed across it to store water in the reservoir so formed. The reservoir intakes are practically similar to the river intake, except that these are located near the upstream face of the dam where maximum depth of water is available. Their design depends upon the type of dam. Fig. 3.21 (*a*) shows a typical intake for an earth dam with several entry ports. The intake is constructed near the toe of the dam. The access to the intake tower is provided through a foot bridge. Water may enter the well through a number of entry ports located at various elevations so that relatively clear top water is admitted at all seasons. The water level in the well is practically the same as the reservoir level. The valves of the entry ports are operated from the gate house located at the top of the well. From the well, water is led to the down-stream through a suitably designed conduit which passes through the body of the earth dam.

Fig. 3.21 (*b*) shows the dry type intake well with a trash-rack structure which is located below the minimum reservoir level. The entry of water is controlled through a valve operated from the upper portion of the well. Fig. 3.21 (*c*) shows an alternative form of the dry well in which water from different entry ports is led directly to the outlet pipe. The well remains dry. In each case, however, the outlet pipe or conduits passes through the main body of the earth dam. This pipe, commonly known as the sluice way, should have projecting collars at regular intervals. These collars increase the path of water seeping along the boundary of the sluice way. The length of the seepage path should be more so that no damage

In the above expression, R, commonly known as *radius of zero drawdown*, is the radius, measured from the centre of the well to a point where the drawdown curve meets the original water table tangentially. *In practice, the selection of the radius of influence R is approximate and arbitrary, but the variation in Q is small for a wide range of R.* Suggested values of R fall in the range of 100 to 300 metres.

Alternatively, R may be computed from the following approximate expression given by Sichardt :

$$R = 3000\, s \sqrt{k}$$

where R and s are in metres, and k is in m/sec.

If there are two observation wells at radial distance r_1 and r_2 $(r_2 > r_1)$, and if the depths of water in them are h_1 and h_2 respectively, Eq. 4.4 can also be expressed in the form :

$$Q = \frac{\pi k(h_2^2 - h_1^2)}{\log_e \frac{r_2}{r_1}} \qquad \text{...(4.4 } a)$$

or

$$Q = \frac{1.36\, k\, (h_2^2 - h_1^2)}{\log_{10} \frac{r_2}{r_1}} \qquad \text{...(4.4 } b)$$

If the drawdown (s) is measured at the well, we have

$$s = H - h$$

and

$$H = s + h, \text{ or } H + h = (s + 2h)$$

Then, from Eq. 4.4,

$$Q = \frac{\pi k\, (H - h)\, (H + h)}{\log_e \frac{R}{r}} = \frac{\pi\, ks\, (s + 2h)}{\log_e \frac{R}{r}}$$

or

$$Q = \frac{\pi ks\, (s + 2L)}{\log_e \frac{R}{r}} = \frac{2\pi ks\, (L + s/2)}{\log_e \frac{R}{r}}$$

where $h = L$ = Length of the strainer

or

$$Q = \frac{2.72\, ks\, (L + s/2)}{\log_{10} \frac{R}{r}} \qquad \text{...(4.5)}$$

Assumptions and Limitations of Dupuit's Theory

Dupuit's theory of flow for unconfined aquifer is based on the following assumptions :

1. The velocity of flow is proportional to the tangent of the hydraulic gradient instead of sine.
2. The flow is horizontal and uniform everywhere in the vertical section.

3. Aquifer is homogeneous, isotropic and of infinite aerial extent.

4. The well penetrates and receives water from the entire thickness of the aquifer.

5. The co-efficient of transmissibility is constant at all places and at all times.

6. Natural ground water regime affecting an aquifer remains constant with time.

7. Flow is laminar and Darcy's law is applicable.

Out of these, assumptions (1), (2) and (7) are of particular importance. The flow is not horizontal, especially near the well. Also, the piezometric surface attains greater slope as it approaches the well boundary, with the result that assumption 1 is an approximation. Due to these reasons, the parabolic form of piezometric surface computed from the Dupuit's theory deviates from the observed surface. This deviation is large at the well face, resulting in the formation of seepage face. In addition to these, the velocity near the well increases and the flow no longer remains laminar. Thus, Darcy's law equation is not valid near the well face.

2. Confined Aquifer

Fig. 4.5 shows a well fully penetrating a confined or artesian aquifer. Let (x, y) be the coordinates of any point P on the drawdown curve, measured with respect to the origin O. Then, from Darcy's law, flow crossing a vertical plane through P is given by

$$Q = k\, i_x A_x$$

where A_x = cross-sectional area of flow, measured at P

$= 2\pi x b$

b = thickness of confined aquifer

i_x = hydraulic gradient at $P = \frac{dy}{dx}$

$$\therefore \quad Q = k\left(\frac{dy}{dx}\right)(2\pi x b)$$

$$\therefore \quad Q\frac{dx}{x} = 2\pi k b . dy.$$

Integrating between the limits (R, r) for x and (H, h) for y, we get

$$Q\int_r^R \frac{dx}{x} = 2\pi k b \int_h^H dy$$

$$\therefore \quad Q\left[\log_e x\right]_r^R = 2\pi k b \left[y\right]_h^H$$

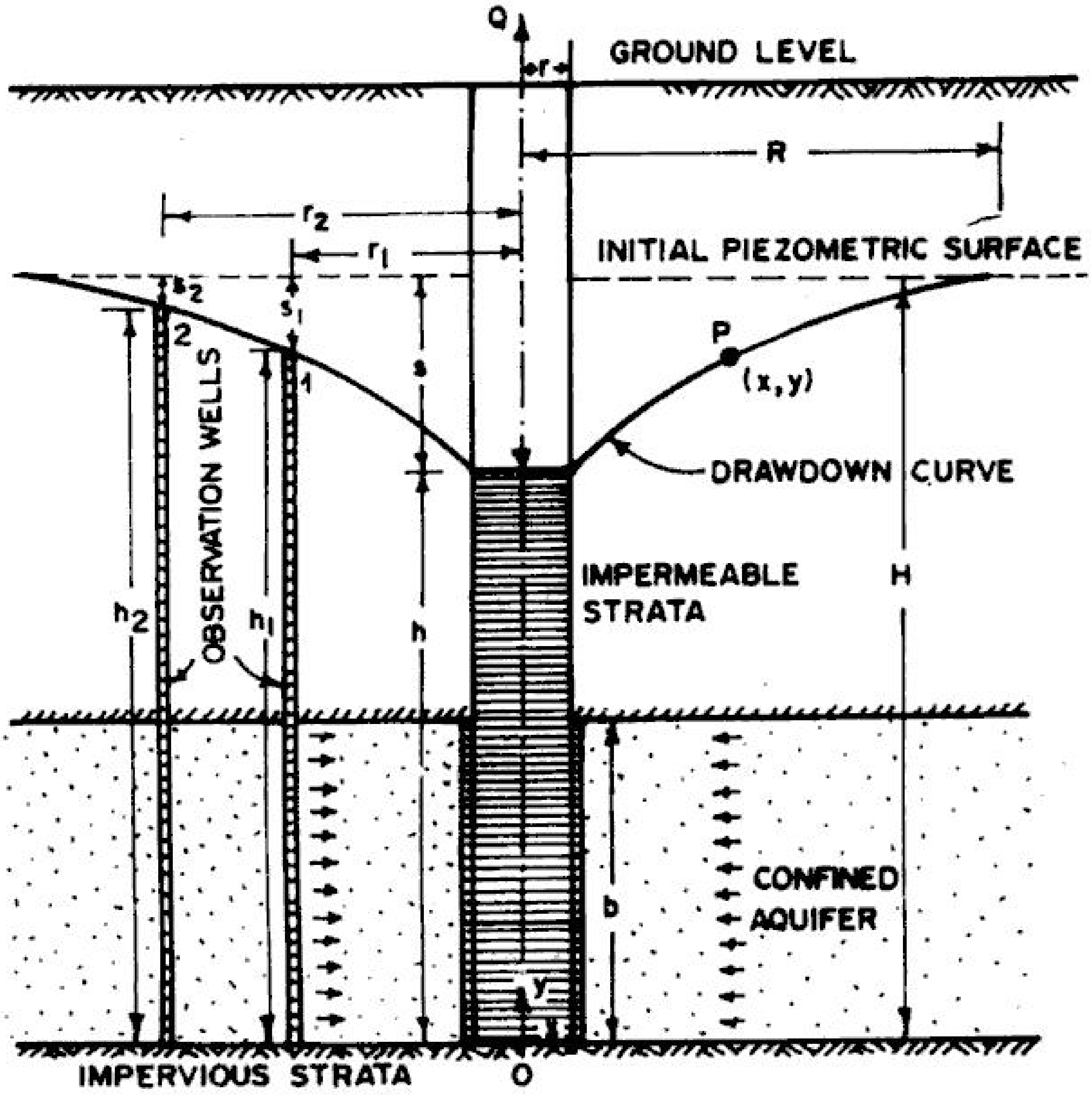


FIG. 4.5. CONFINED AQUIFER

From which

$$Q = \frac{2\pi k b\,(H-h)}{\log_e \dfrac{R}{r}} \qquad \text{...(4.6 } a\text{)}$$

$$= \frac{2.72\, bk\,(H-h)}{\log_{10} \dfrac{R}{r}} \qquad \text{...(4.6 } b\text{)}$$

$$= \frac{2\pi\, b\, k\, s}{\log_e \dfrac{R}{r}} = \frac{2.72\, b\, k\, s}{\log_{10} \dfrac{R}{r}} \qquad \text{...(4.6 } c\text{)}$$

$$= \frac{2.72\, T s}{\log_{10} \dfrac{R}{r}} \qquad \text{...(4.6 } d\text{)}$$

where T = coefficient of transmissibility = bk

s = drawdown at the well.

Eq. 4.6 (*a*) is known as the *equilibrium equation* or Thiem equation.

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the tube. The slots are wide inside and narrower outside, as shown in Fig. 4.14. The gauge of slots depends on the coarseness of sand, and varies from 0.15 to 0.4 mm.

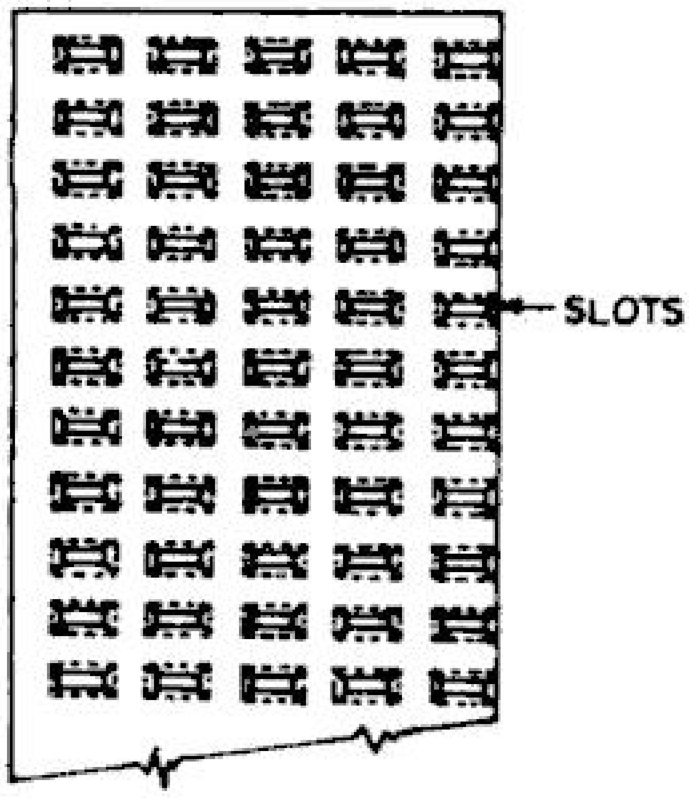

FIG. 4.14. COOK STRAINER

(*ii*) ***Tej strainer***

It is similar to cook strainer, but is manufactured in India. It consists of a brass tube constructed of a brass sheet bent round to form the tube, the vertical joint being brazed. The slots are cut in the sheet before it is bent. The strainer is generally manufactured from 7.5 cm diameter upwards, and is made in $2\frac{1}{2}$ metre lengths.

The individual lengths of the strainer are then joined together by means of screwed collars of brass.

(*iii*) ***Brownlie strainer***

The Brownlie strainer is made of a polygonal convoluted steel plate having perforations. A wire mesh surrounds the steel tube, as shown in Fig. 4.15. The mesh consists of heavy parallel copper wires woven with copper ribbons. Since the wire mesh is slightly away from the perforated tube, it is known as the best type of strainer.

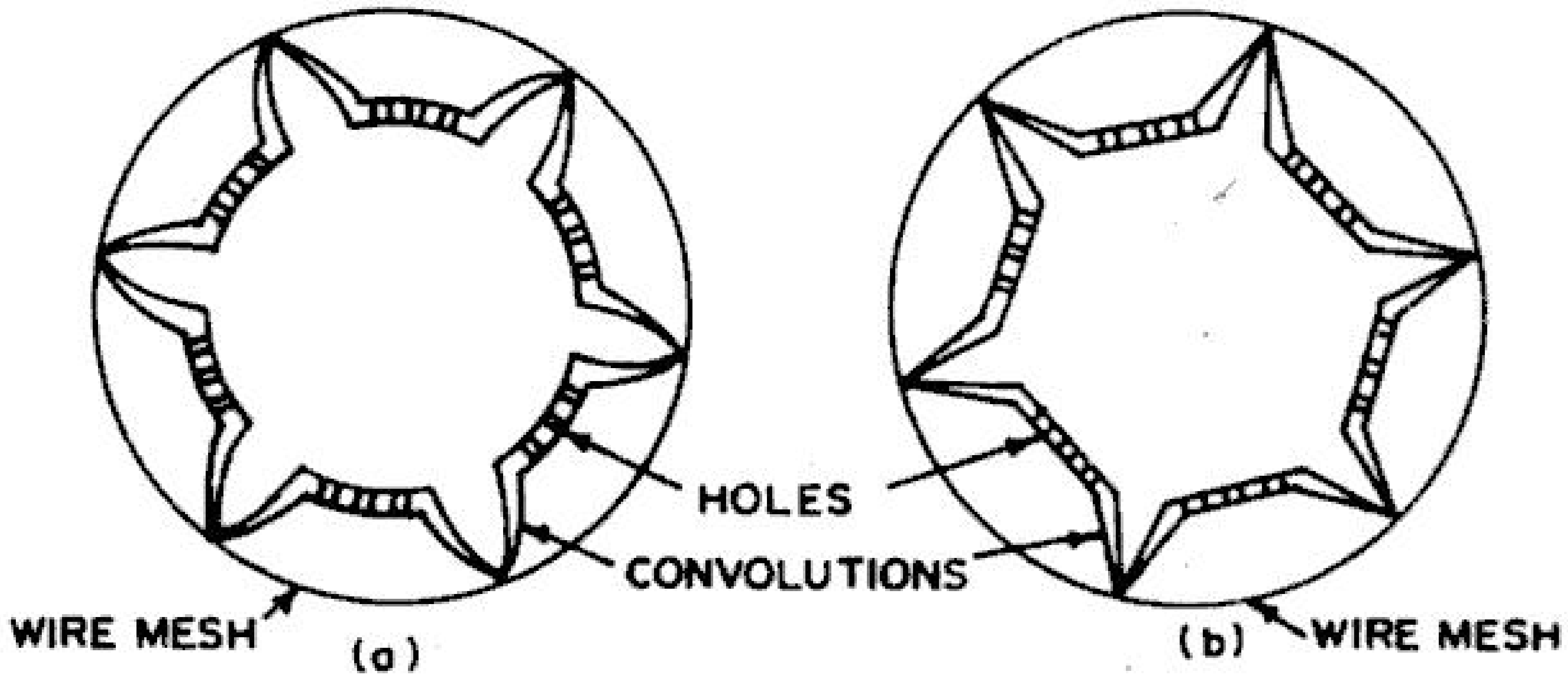

FIG. 4.15. BROWNLIE STRAINER

(*iv*) ***Ashford strainer***

This is very delicate strainer and consists of perforated tube with a wire round it over which a wire mesh is soldered. The wire

keeps the mesh away from the tube. The wire mesh is protected and strengthened by a wire net around it, as shown in Fig. 4.16.

(*v*) ***Leggett strainer***

It is expensive type of strainer in which a cleaning device is provided. The cleaning device is in the shape of cutters which can be turned in the slits. The cutters are operated from the top (ground surface). These cutters clean the strainer clogged by the solid matter.

(*vi*) ***Phoenix strainer***

It is a mild steel tube in which the openings are made by cutting slits from inside. The tube is cadmium plated to keep it free from danger of chocking and corrosion caused by chemical action.

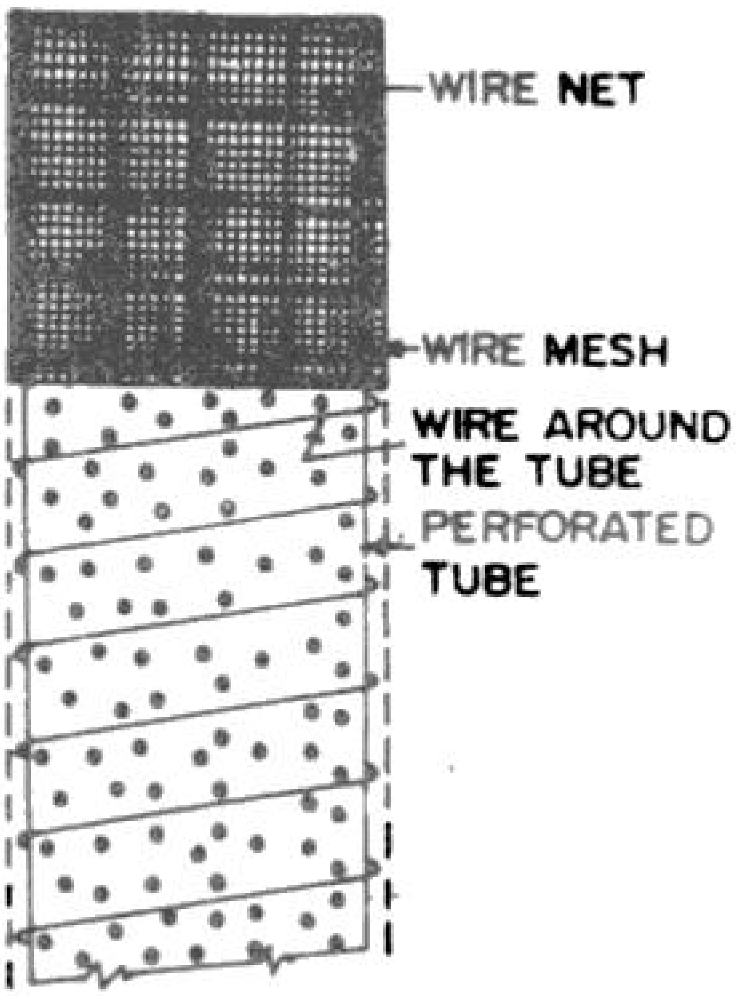

FIG. 4.16. ASHFORD STRAINER

(*vii*) ***Layne and Bowler strainer***

It is a robust type strainer manufactured in America. It consists of wedge-shape steel wire wound to suitable pitch round a slotted or perforated steel or wrought iron pipe. The joint of the strainer pipes are made by screwed collars.

Chocking of Strainers

The strainer of a tube well may get chocked due to two actions: (1) Mechanical action, (2) Chemical action.

(1) ***Mechanical chocking.*** Mechanical chocking may result from the chocking of slits with sand and other particles. This may however, be prevented by providing such slits which expand inwards. The pulsating action of the centrifugal pump may also remove the chocking. To safeguard against chocking, proper screening or shrouding should be provided. Another method of eliminating chocking is to permit inflow velocity lesser than the critical.

(2) ***Chemical chocking.*** The strainer may be chocked due to chemical action of salts present in water. The chemical action may also deteriorate a strainer by corrosion. If calcium bicarbonate present in water exceed by an amount of 15 parts per million parts of water, carbon dioxide is released when pressure is reduced due to pumping and calcium carbonate is precipitated on the strainer. The cumulative action of such precipitation reduces the yield. The chemical chocking

by deposition of carbonates is reduced by providing large slit area and having low inflow velocity. Similarly, sodium salts may attack mild steel and cast iron strainer pipes causing chocking. Sodium bi-carbonate may attack copper to a certain extent, though brass is not readily attacked.

(2) CAVITY TYPE TUBE WELL

This is a special type of tube well in which water is not drawn through the strainer, but it is drawn through the bottom of the well where a cavity is formed. The tube well pipe penetrates a strong clay layer which acts as strong roof. Thus, a cavity tube well is similar to deep well. However, a deep well draws water from the first aquifer below the *mota* while the cavity well need not do so. The essential condition for a cavity tube well to function efficiently is to have confined aquifer of good specific yield, and the aquifer should have a strong impervious material above it. In the initial stage of pumping with the help of centrifugal pump or an air lift pump, fine sand comes with water and consequently a hollow or cavity is formed. As the spherical surface area of cavity increases outwards, the radial critical velocity decreases, the sand particles stop entering the well. At this stage, an equilibrium in the cavity formation is established and clean water continues to enter the well on further pumping at the same constant discharge. After the formation of the cavity, the velocity of entry of water at the bottom of the pipe is lesser than the critical.

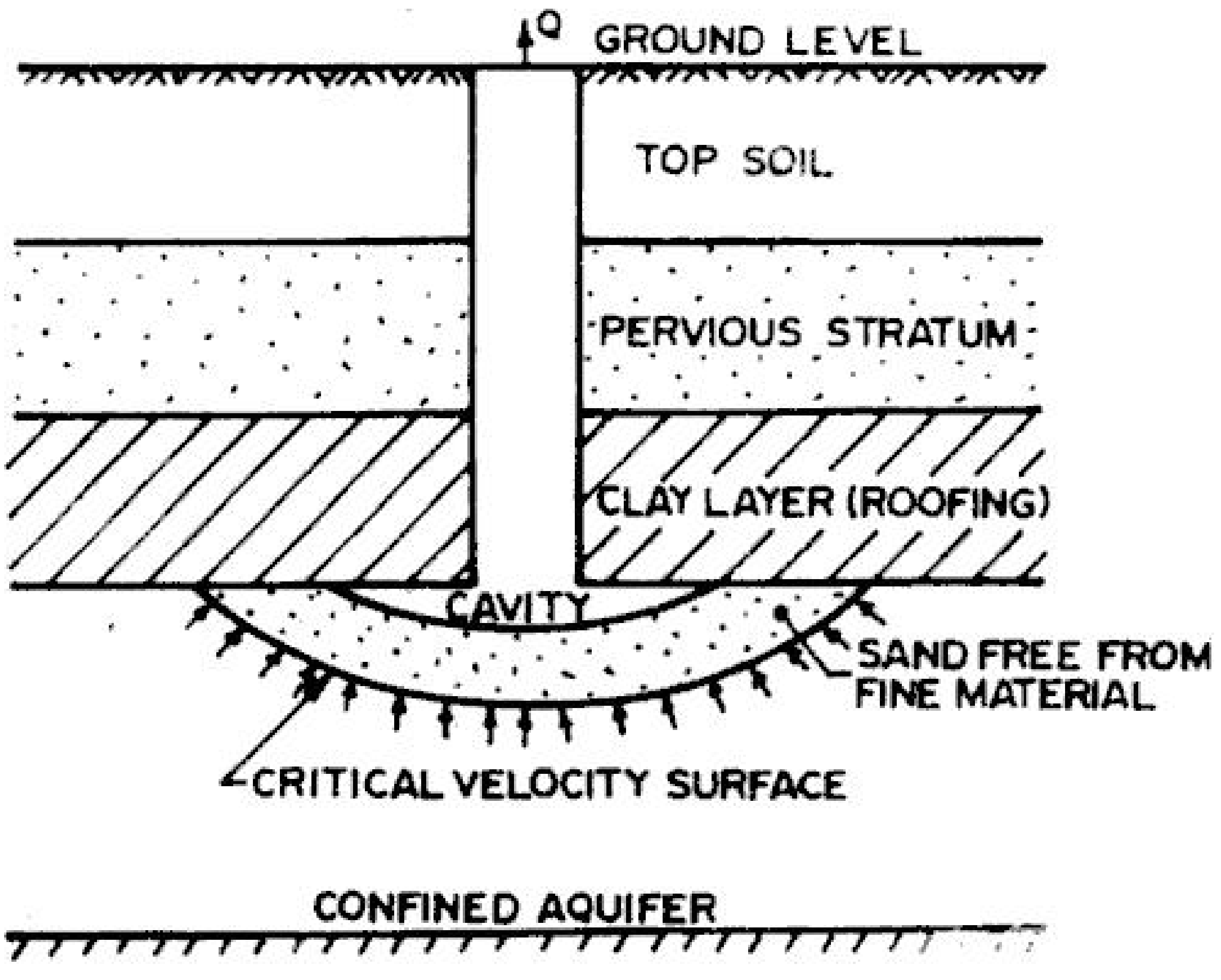

FIG. 4.17. CAVITY TUBE WELL

of a 'sludger' or 'sand pump'. The sludger, almost similar to the bailer, is a steel pipe 2 to 4 metres long, having a cutting shoe of hard steel riveted to its bottom. A flap valve at its lower end permits the entry of the cut material. The sludger is inserted in the casing pipe and is worked up and down by means of a rope the other end of which passes over a pulley fixed centrally to a tripod. A platform is attached to the upper end of the casing pipe, and the weight placed on the platform drives the casing pipe slowly into the hole. When the sludger is full with the cutting paste, it is taken out and emptied. The process goes on till the required level is reached. However, if the rock formations are hard, the sludger is unable to cut it and a string of drilling bits, shown in Fig. 4.20, is inserted and operated by an engine.

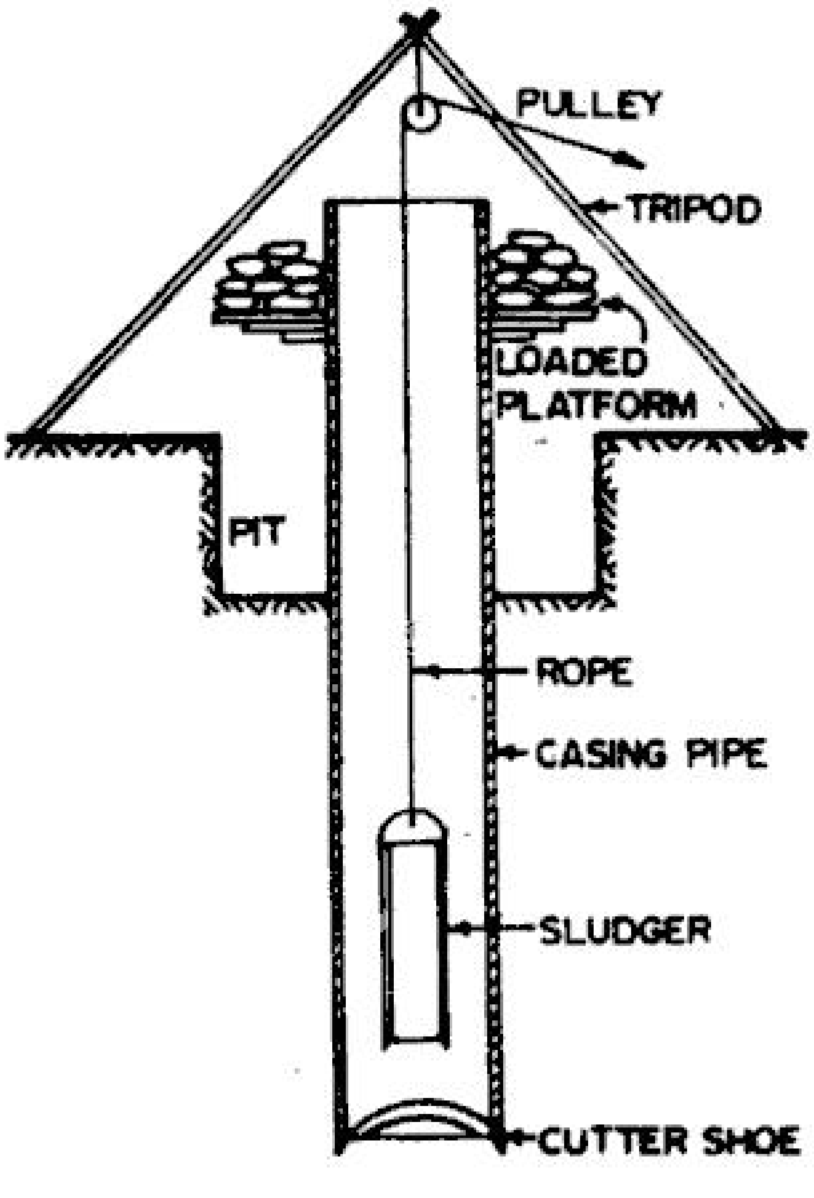

FIG. 4.21. PERCUSSION BORING BY MANUAL LABOUR.

In both the methods, the record of the material collected by bailer or sludger is kept. A bore log is then plotted to know the depths of various formations. The well pipe with strainers at determined levels of aquifer is then lowered to the desired depth. The well is then shrouded, and the casing pipe is taken out when the shrouding is done in steps. Wells by cable-tool method have been drilled in diameters upto 30 cm and to depths as great as 1600 metres.

3. HYDRAULIC ROTARY METHOD

Hydraulic rotary method, sometimes known as *rotary boring method*, is used for drilling large bores in unconsolidated strata. This is the fastest method and has been used for well upto 45 cm diameter (upto 150 cm with a reamer), and for depth over 1600 m. Oil wells over 7000 metre deep have been drilled by this method.

In this method, the boring is done with the help of a drilling bit attached at the end of a string of hollow pipes (Fig. 4.22). A mixture of clay and water, known as *drilling mud*, is continuously circulated through the drill shaft in the hole. Material loosened by the bit is carried upward in the hole by the rising mud. Ordinarily, no casing is required since the drilling mud forms a clay lining and supports the walls of the hole.

The drill bits have hollow shanks and one or more centrally located orifices for jetting the mud into the bottom of the hole. The drill rod, made of heavy pipe, carries the drill bit at one end, and is screwed to a square section known as *kelly*. A rotating table, which fits closely around the kelly, rotates the drill rod which slides downward as the hole deepens. The rising drilling mud carrying rock fragments is taken to the settling basin where the cuttings settle. The mud is recirculated to the hole. To maintain the required consis-

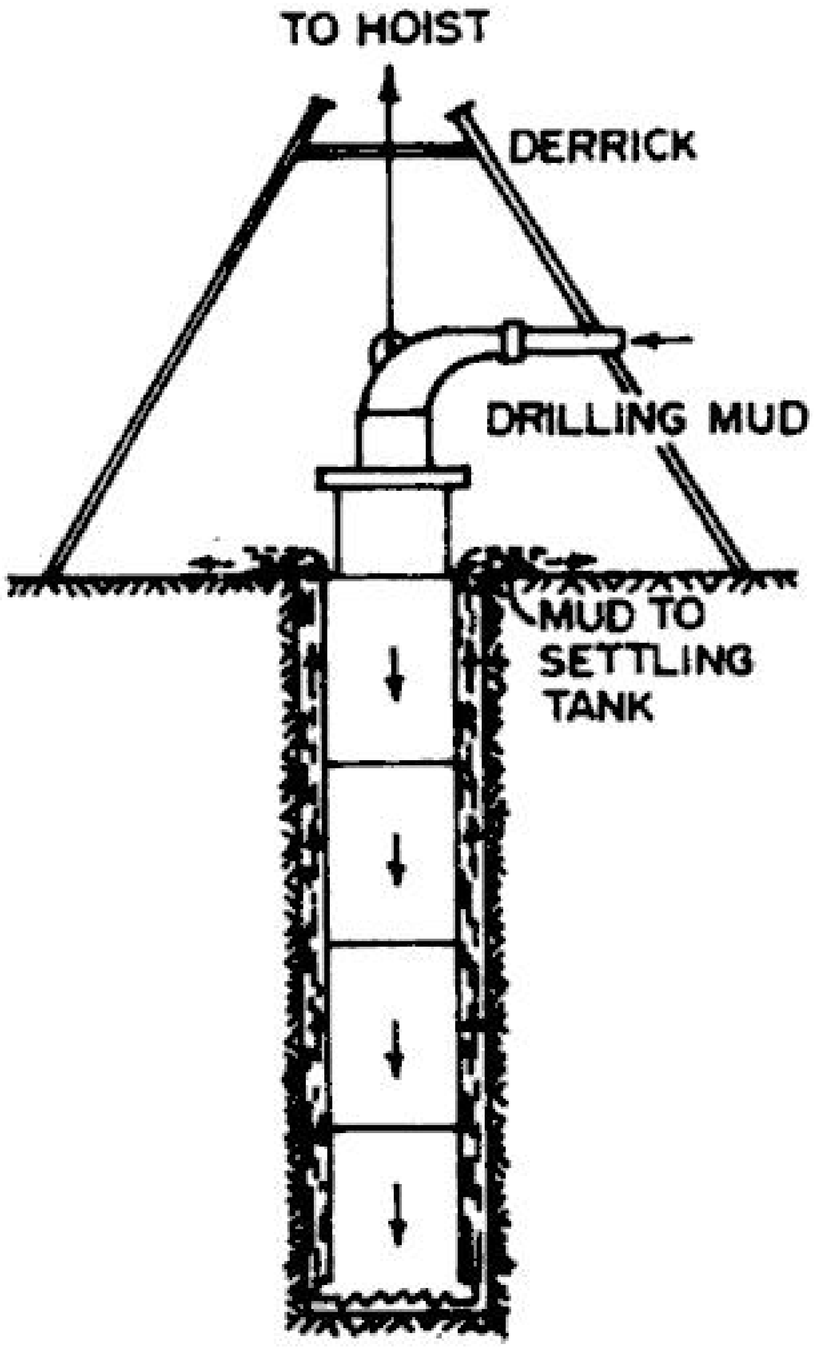

FIG. 4.22. HYDRAULIC ROTARY METHOD

tency, clay and water is added to the circulating mud from time to time. A complete boring record is maintained to know the type of formations at various depths. When desired level is reached, the drill rod etc. are taken out and the well pipe containing strainer pipes at appropriate locations (opposite aquifers) is lowered. Since the well walls are coated with clay, it should be washed to get more discharge. Back washing is done by lowering the drill pipe and bit in the well pipe, and forcing water containing calgon (sodium-hexa-meta phosphate). Calgon has the property of dispersing clay colloids. A collar, of the size of well-pipe is attached to the drill rod just above the bit. This forces the water through the strainer causing washing action on the clay wall. At the same time, the drill rod is plunged up and down causing surging action. When washing in the bottom is done, the bit is raised through some distance and the operation is repeated.

4. REVERSE ROTARY METHOD

Reverse rotary method, similar to the hydraulic rotary method, is very much used in Europe. In this method, the cuttings are used for this purpose. A mixture of water and fine grained material is circulated in the hole. The procedure is essentially a suction dredging method. The walls of the hole during drilling are supported by hydrostatic pressure acting against the film of fine-grained material deposited on the walls by the drilling water. The method of recirculation of drilling water containing fine-grained particles and cleaning the well after inserting the well-pipe is similar to that of hydraulic rotary methods.

4.13. WELL SHROUDING AND WELL DEVELOPMENT

(a) *Well Shrouding*

Well shrouding is a process of interposing coarse material such as gravel and coarse sand between the well-pipe (strainer pipe) and the aquifer soil to prevent finer particles of soil coming in contact with the strainer and chocking it. This is essential in sandy and unconsolidated formations of aquifer. This is also essential in slotted type tube well where a strainer is not used. Such tube well is also sometimes known as a *gravel-packed well.* The shrouding increases the effective well diameter, acts as a strainer to keep fine material out of the well, and protects the well-pipe from caving of surrounding formations. A gravel packed well has a greater specific capacity than one of the same diameter not surrounded by a gravel. A minimum thickness of 40 cm gravel pack is necessary to make it effective. The proper gain size distribution of the shrouding material depends upon the mechanical analysis of the aquifer and upon the perforations or screen slot size.

The amount of shrouding material per 30 cm length of the casing pipe can be calculated accurately before hand. In the beginning, the shrouding material for 60 cm length is shovelled in from the top between the tube-pipe and casing. The casing is raised by 30 cm with the help of jack. Then the quantity for each 30 cm is added and the casing pipe is withdrawn 30 cm at a time till the strainer is covered. The shrouding material is sometimes placed through small pipes or pilot holes around the tube-pipe which feed the material down into position. A further refinement is the use of bladeless pumps for pumping the shrouding material into place.

(*b*) Well Development

Well development is the process of removing fine material from the aquifer formation surrounding the strainer pipe, and is aimed at (*i*) increasing the specific capacity of the well, (*ii*) preventing sand flowing in, and (*iii*) obtaining maximum economic well life. The actual yield of the well can be known only after well development. Thus, it also helps in determining the required characteristics of the pump and power unit to be installed. Depending upon formation characteristics of the aquifer, a well may be developed by one of the following methods.

1. Development by pumping.
2. Development by surging.
3. Development by compressed air.
4. Development by back washing.
5. Development by dry ice.

1. Development by pumping

In this method, a variable speed pump is used. The method is based on the principle that irregular and non-continuous pumping agitates the fine material surrounding the well so that it can be carried into the well and pumped out. Initially, the pump is started with a very low discharge. The fine particles start coming. This low speed is maintained till clear water comes. The discharge is then increased in steps until maximum discharge or well capacity is reached. The pump is then stopped and levels permitted to increase till it comes to normal. The pump is then again started and the procedure repeated, till no fine particles come.

2. Development by Surging

In this method, surging effect is created by up and down movement of a hollow surge block or a bailer. Calgon (sodium-hexametaphosphate) is added to water, so that it acts as dispersing agent for fine grained particles. When the surge block is moved up, it sucks water in. When it is moved down, it forces water-calgon solution back in the formation. Further upward motion bring with it fine material. The surge block is connected to a string of hollow pipe from which the water charged with fine particles is pumped out continuously. The procedure is repeated by increasing the speed of surging till clear water comes out.

3. Development by Compressed Air.

In this method, the development is done with the help of an air compressor, a discharge pipe and an air pipe. The air pipe is put into the discharge pipe and is lowered into the well tube, till the assembly reaches near the bottom of the strainer-pipe section. The lower end of the air pipe is kept emerging out of the discharge pipe by a small length. The air entry to the air pipe is first closed and the compressor is then started till a pressure of 6 to 10 kg/cm^2 is built up. The air is then suddenly made to enter the pipe, at this pressure, with the help of suitable quick-opening valve. This sudden entry of air into well creates a powerful surge within the well causing loosening of fine material surrounding the perforations. When the pressure decreases, water enters the well bringing the loosened particles with it. The continuous air injection creates an air lift pump, and the water carrying fine particles is pumped out. The process is repeated till clear water comes. The pipe assembly is then lifted up, and the surging is again created. The operation is repeated at intervals along the screen section till the well is fully developed.

4. Development by Back Washing

In this method, in addition to the compressor, a discharge pipe and an air pipe, and additional small air pipe is used. The well is sealed at its top so that is becomes air-tight. The discharge pipe and air pipe assembly is lowered in the well, as in the previous method, but the end of the air pipe is kept inside the discharge pipe. A small air pipe is fitted at the top of the air-tight cover and is provided with a three-way cock. With the help of the three-way cock, air can be admitted to the well either through the long air pipe (but inside the discharge pipe) or through the long air pipe fitted at the top. Air is first made to enter the long air pipe. This forces air and water out of the well through the discharge pipe. When clear water comes the valve is closed, and water level is allowed to increase in the well. The valve is then turned to the other side so that air enters through the discharge pipe and at the same time agitates the fine particles surrounding the well. Calgon is often added to water. When air starts escaping from the discharge pipe, the valve is turned so that air enters the long air pipe, and the assembly works as an air-lift pump and the water is pumped out. The procedure is repeated till clear water comes and the well is fully developed.

5. Development by dry ice (solid sodium dioxide)

In this method, well is developed with the help of two chemicals : hydrochloric acid and solid sodium dioxide (known as dry ice). First of all, hydrochloric acid is poured into the well. The well is capped at the top and compressed air is forced into the well. The pressure of the compressed air forces the chemical into the formation. The cap is then removed and blocks of dry ice are dropped into the well. The sublimation releases gaseous carbon dioxide, and a high pressure of this gas is built up in the well. On releasing the pressure the muddy water is forced up in the form of a jet and is automatically thrown out of the well. Explosion of mud and water extending 40 metres into the air from a well in Utah (U.S.A) was observed when the well was developed with dry ice.

4.14. OPEN WELLS

As stated earlier, an open well is essentially of a bigger diameter than of a tube well, and derives its water only from one pervious stratum. Since a tube well, in general, may derive water from more than one aquifer formation, it has greater depth than an open well. The economically feasible depth of an open well is limited to 30 metres below the ground surface. In a lined open well, the entry of water is from the bottom and *not* from the sides.

An open well is classified as :

(*i*) Shallow well. (*ii*) Deep well.

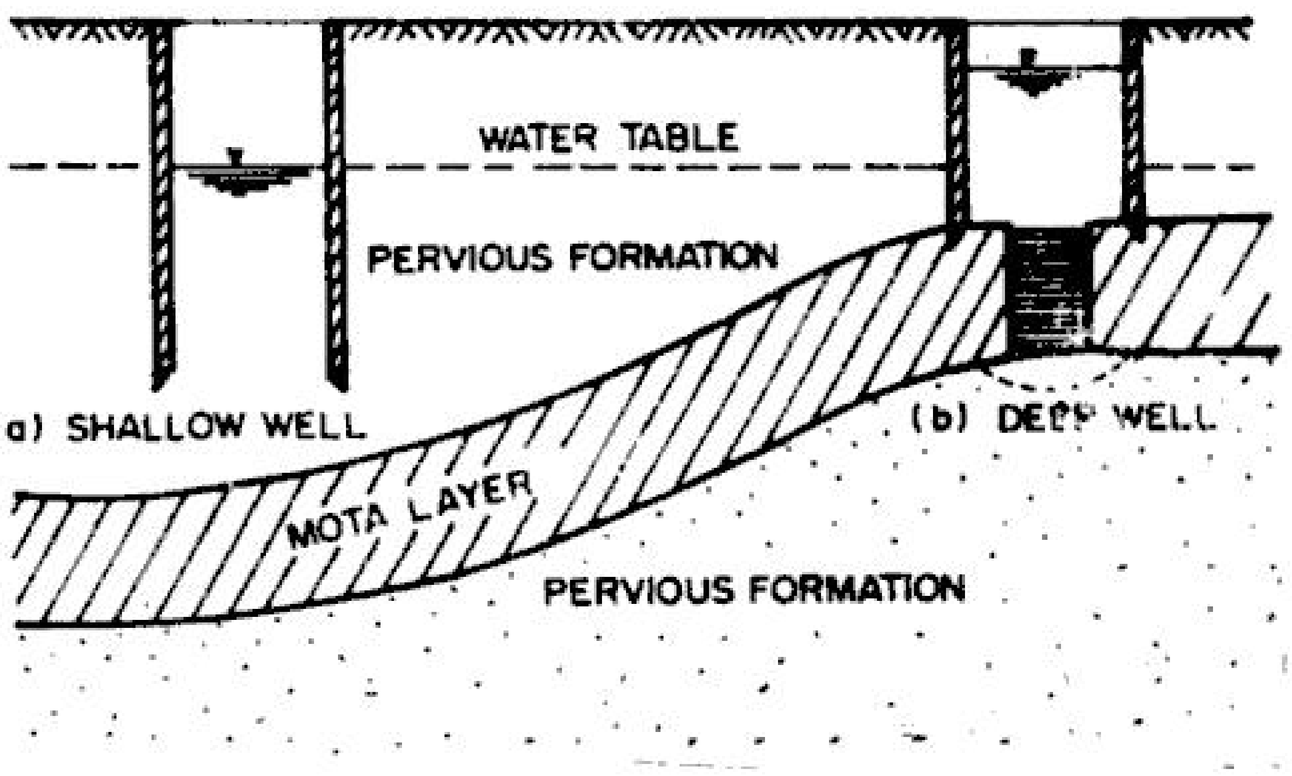

FIG. 4.23. SHALLOW AND DEEP WELLS

The nomenclature of shallow and deep well has nothing to do with the actual depth of the well. A *deep well* is a well which is supported on a *mota* layer and draws its water supply, through a hole bored in it, from the pervious formation below the *mota* layer.

A *shallow well,* on the other end, penetrates the pervious stratum only and draws its water supply through it. The term *mota* layer also sometimes known as *matbarwa* or *nagasan,* refers to a layer of clav, cemented sand, kankar or any other hard material. The *mota* layer gives structural support to the open well, and is found throughout the Indo-Gangetic plain. These mota layers may either be continuous, or may be localised and may be found in different thicknesses and depths at different places.

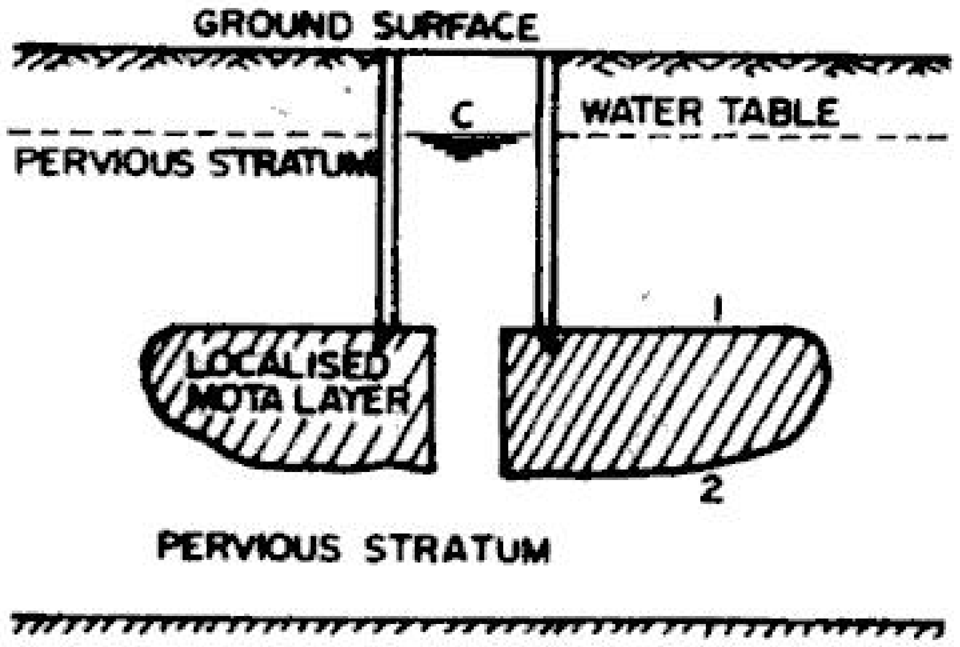

FIG. 4.24. DEEP WELL ON LOCALISED MOTA LAYER

Fig. 4.23 (*a*) shows a shallow well which derives water from the pervious stratum, and does not rest on a mota layer. Fig. 4.23 (*b*) shows a deep well resting on a continuous mota layer. Fig. 4.24 shows a deep well resting on a localised mota layer and deriving its water from the second pervious stratum. Actually, a shallow well can be deeper than a deep well. However, since a shallow well draws water from the first pervious stratum (*i.e.* top formation), the water in it is liable to be contaminated by rain water percolating in the vicinity, and may take with it mineral organic matter such as decomposing animals and plants. The water in a deep well is not liable to get such impurities and infections. Also, the pervious formation below a mota layer normally has greater water content and specific yield. Hence discharge from a deep well is generally more than a shallow well.

The open well may further be classified as :

(*i*) Kachha well or unlined well.

(*ii*) Well with impervious lining.

(*iii*) Well with pervious lining.

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$$\therefore \qquad R = (6.1352)^{\frac{1}{0.4868}} = (6.1352)^{2.0542}$$

$$\therefore \qquad R = \mathbf{41.52\ m}$$

(*b*)

$$Q = 2000 \text{ litres/min.}$$

$$= 2 \text{ m}^3/\text{min.}$$

$$(\text{since 1 litre/min.} = 0.001 \text{ m}^3/\text{min.})$$

$$R = 41.52 \text{ m} \,;\, H = 40 \text{ m},$$

$$h = 36 \text{ m} \,;\, r = 10 \text{ m}$$

$$\therefore \qquad Q = 2 = \frac{1.36\,k\,(40^2 - 36^2)}{\log_{10} \dfrac{41.52}{10}}$$

$$\therefore \qquad k = \frac{2}{1.36\,(40^2 - 36^2)} \log_{10} \frac{41.52}{10}$$

$$= 0.003 \text{ m/min.}$$

$$= \mathbf{4.31\ m/day.}$$

(*c*) Depth of water in the well is given by

$$2 = \frac{1.36 \times 0.003\,(40^2 - H_0^2)}{\log_{10} \dfrac{41.52}{0.30}}$$

$$\therefore \qquad 1600 - H_0^2 = \frac{2}{1.36 \times 0.003} \times \log_{10} \frac{41.52}{0.30}$$

$$= 1049.58$$

$$\therefore \qquad H_0 = 23.46 \text{ m.}$$

Hence drawdown at the well

$$= 40 - 23.46$$

$$= \mathbf{16.54\ m}$$

(*d*) ***Specific capacity*** is defined as the discharge per unit drawdown. Let it be designated by S_c. It is not constant, but decreases as the discharge increases (see §4.6). Let us assume that the yield is directly proportional to the drawdown or to the radius of zero-drawdown.

$$\therefore \qquad Q \infty R$$

or

$$Q = C.R \text{ where } C \text{ is a constant.}$$

For a given data,

$$C = \frac{Q}{R} = \frac{2}{41.52} = 0.04817.$$

$$\therefore \qquad R = \frac{Q}{C} = \frac{Q}{0.04817} \text{ in general.}$$

Now corresponding to drawdown of 1 m, discharge $Q = S_c$. Hence radius of zero drawdown

$$= \frac{S_c}{C} = \frac{S_c}{0.04817}$$

Hence $$Q = S_c = \frac{1.36\,k\,(40^2 - 39^2)}{\log_{10}\dfrac{R}{r}}$$

$$= \frac{1.36 \times 0.003\,(40^2 - 39^2)}{\log_{10}\left(\dfrac{S_c}{0.04817 \times 0.3}\right)}$$

or $$S_c \log_{10}(69.2\,S_c) = 0.3223.$$

Solving this by trial and error, we get

$$S_c = 0.258 \text{ m}^3/\text{min}.$$

Hence, specific capacity of the well is **0.258 m³/min/m** depression head.

(e) maximum rate of discharge Q_m will be obtained when drawdown in the well is equal to H, *i.e.*, when H_0 = zero.

$$\therefore \qquad Q_m = \frac{1.36 \times 0.003\,(40^2 - 0^2)}{\log_{10}\dfrac{Q_m}{0.04817 \times 0.3}}$$

or $$Q_m \log_{10} 69.2\,Q_m = 6.528$$

Solving this by trial and error, we get

$$\mathbf{Q_m = 2.85\ m^3/min.}$$

4.18. UNSTEADY FLOW

The analysis of flow towards wells discussed in the previous articles is based on the assumption that steady state of flow is developed immediately after pumping is started. Actually, the cone of depression fluctuates with time. The gradual approach of cone of depression towards a steady state is produced primarily by the removal of water from storage as cone deepens. Hence a *storage co-efficient* comes into play. The storage co-efficient is a dimensionless constant of the aquifer and may be interpreted as the amount of water in storage *released from storage* from a column of aquifer of unit cross-section under a unit decline of head.

Equations developed for unsteady or transient well flow normally show how the drawdown s of the piezometric surface or water table is related to the time of pumping the well. In Fig. 4.31,

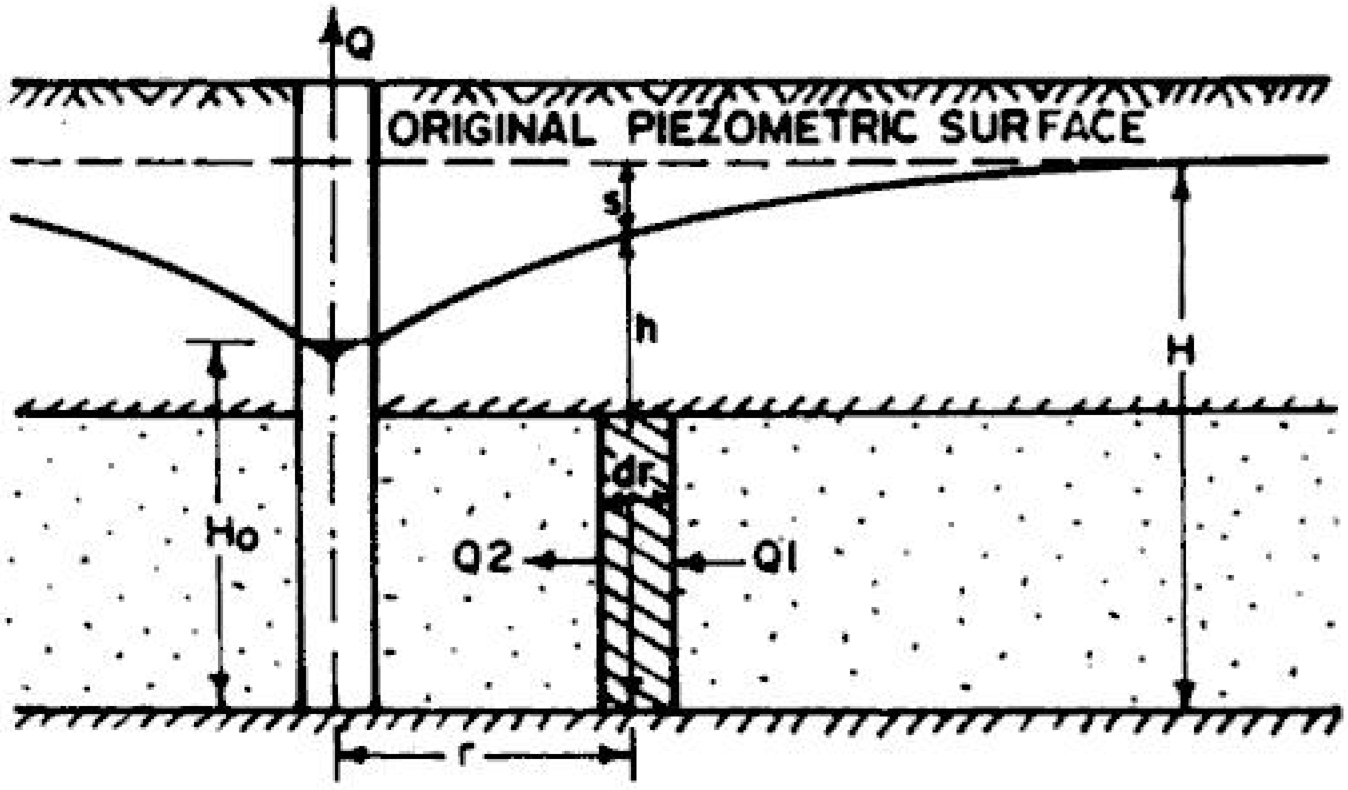


FIG. 4.31. TRANSIENT FLOW

consider an annular cylinder of thickness *dr* and radius *r*. Due to unsteady flow conditions, water will be released from storage of this elementary cylinder. According to definition of storage co-efficient *S*, the rate $\frac{\partial V}{\partial t}$ at which a certain volume *V* is released from storage over an aquifer of area *A* is given by

$$\frac{\partial V}{\partial t} = -\frac{\partial h}{\partial t} . SA \qquad ...(1)$$

where V = volume of water released per horizontal area A of aquifer.

h = height of piezometric surface or water table above lower boundary of aquifer.

S = storage co-efficient.

A = area of the aquifer to which $\frac{\partial V}{dt}$ applies.

t = time.

Since *h* decreases with time *t*, minus sign with $\partial h/\partial t$ has been used. For the elementary cylinder of the aquifer, $A = 2\pi r\, dr$. Hence

$$\frac{\partial V}{\partial t} = -2\pi r\, dr . S . \frac{\partial h}{\partial t} \qquad ...(2)$$

Let Q_1 = discharge entering the outer face of the cylinder.

Q_2 = discharge leaving the inner face of the cylinder.

∴ Change in discharge

$$\delta q = Q_1 - Q_2.$$

Since the rate of increase in q can be expressed as $-\frac{\partial q}{\partial r}$, the increase in discharge over the annular area $= -\frac{\partial q}{\partial r}\,dr$. Substituting this for $\frac{\partial V}{\partial t}$ in (2), we get

$$-\frac{\partial q}{\partial r}\,dr = -\,2\pi r\,dr\,.\,S\,.\,\frac{\partial h}{\partial t} \qquad ...(3)$$

For confined radial flow, we get from Darcy law,

$$q = 2\pi r\,T\,\frac{\partial h}{\partial r}$$

$$\therefore \qquad \frac{\partial q}{\partial r} = 2\pi\,T\,\frac{\partial}{\partial r}\left(r\,\frac{\partial h}{\partial r}\right)$$

$$= 2\pi\,T\left(\frac{\partial h}{\partial r} + r\,\frac{\partial^2 h}{\partial r^2}\right) \qquad ...(4)$$

Substituting in (3), we get

$$2\pi\,T\left(\frac{\partial h}{\partial r} + r\,\frac{\partial^2 h}{\partial r^2}\right) = 2\pi r\,.\,S\,\frac{\partial h}{\partial t}$$

which simplifies to

$$\frac{\partial^2 h}{\partial r^2} + \frac{1}{r}\,\frac{\partial h}{\partial r} = \frac{S}{T}\,\frac{\partial h}{\partial t} \qquad ...(4.24)$$

This is the basic equation for unsteady flow towards the well. Theis (1935) obtained a solution for this equation based on the analogy between ground water flow and heat conduction, by assuming that the well is replaced by mathematical sink of constant strength such that $h = H$ before pumping begins and that $h \rightarrow H$ as $r \rightarrow \infty$ after pumping begins ($t \geq 0$). The solution is

$$H - h = \frac{Q}{4\pi\,T}\int_{u = r^2 S/4Tt}^{\infty} \frac{e^{-u}}{u}\,du \qquad ...(4.25\ a)$$

or

$$s = \frac{Q}{4\pi\,T}\,W(u) \qquad ...(4.25)$$

where

$$W(u) = \int_{u}^{\infty} \frac{e^{-u}}{u}\,du = \text{ well function.}$$

Eq. 4.25 is known as the *non-equilibrium equation* or *Theis equation*. The integral in the above equation is a function of the lower limit, and is known as exponential integral. It can be expressed as a convergent series so that Eq. 4.25 can also be expressed as

$$s = \frac{Q}{4\pi\,T}\left[-\,0.5772 - \log_e u + u - \frac{u^2}{2\,.\,2\,!} + \frac{u^3}{3\,.\,3\,!} - .....\right] \qquad ...(4.25\ b)$$

TABLE 4.3. VALUES OF WELL FUNCTION W(u)

($u = N \times 10^n$)

$u =$ → $N =$ ↓	$N \times 10^{-15}$	$N \times 10^{-14}$	$N \times 10^{-13}$	$N \times 10^{-12}$	$N \times 10^{-11}$	$N \times 10^{-10}$	$N \times 10^{-9}$	$N \times 10^{-8}$
1	33.96	31.66	29.36	27.05	24.75	22.45	20.15	17.84
2	33.27	30.97	28.66	26.36	24.06	21.76	19.45	17.15
3	32.86	30.56	28.26	25.96	23.65	21.35	19.05	16.75
4	32.57	30.27	27.97	25.67	23.36	21.06	18.76	16.46
5	32.35	30.05	27.75	25.44	23.14	20.84	18.54	16.23
6	32.17	29.87	27.50	25.26	22.96	20.66	18.35	16.05
7	32.02	29.71	27.41	25.11	22.81	20.50	18.20	15.90
8	31.88	29.58	27.28	24.97	22.67	20.37	18.07	15.76
9	31.76	29.46	27.16	24.86	22.55	20.25	17.95	15.65

$u =$ → $N =$ ↓	$N \times 10^{-7}$	$N \times 10^{-6}$	$N \times 10^{-5}$	$N \times 10^{-4}$	$N \times 10^{-3}$	$N \times 10^{-2}$	$N \times 10^{-1}$	N
1	15.54	13.24	10.94	8.633	6.332	4.038	1.823	0.2194
2	14.85	12.55	10.24	7.940	5.639	3.355	1.223	0.04891
3	14.44	12.14	9.837	7.535	5.235	2.959	0.906	0.01305
4	14.15	11.85	9.550	7.247	4.948	2.681	0.702	0.003779
5	13.93	11.63	9.326	7.024	4.726	2.468	0.560	0.001148
6	13.75	11.45	9.144	6.842	4.545	2.295	0.454	0.000360
7	13.60	11.29	8.990	6.688	4.392	2.151	0.374	0.000116
8	13.46	11.16	8.856	6.555	4.259	2.027	0.311	0.0000377
9	13.34	11.04	8.739	6.437	4.142	1.919	0.260	0.0000125

Wenzel tabulated the values of *W(u)* for various values of *u* ranging from 10^{-15} to 9.9. Table 4.3 gives the values of $W(u)$ for *u* ranging from 10^{-15} to 9.

From the table, when

$u = 5$, $W(u) = 0.001148$

and when $u = 5 \times 10^{-3}$, $W(u) = 4.726$.

The values of *formation constants* *S* and *T* can be found by measuring drawdowns in observation wells when the well under study is pumped at a constant rate of discharge *Q*. There are several methods of determining *S* and *T*, but we will discuss here following three methods :

(*a*) Theis method,
(*b*) Jacob's method,
(*c*) Chow's method.

(*a*) THEIS METHOD

Theis proposed a curve-fitting method for finding formation constants *S* and *T* from a pump out test. From Eq. 4.25, we observe that

$$s = \left[\frac{Q}{4\pi T}\right] W(u) \text{ or } \log W(u) = \left[\log \frac{4\pi T}{Q}\right] + \log s \qquad \text{....(4.26 } a)$$

and

$$\frac{r^2}{t} = \left[\frac{4T}{S}\right] u \text{ or } \log u = \left[\log \frac{S}{4T}\right] + \log \frac{r^2}{t} \qquad \text{...(4.26 } b)$$

If a constant withdrawal rate (*Q*) is maintained, the bracketed portions of the above two equations are constant for a given pumping test. It is to be noted that *s* is related to r^2/t in a manner that is similar to the relation of $W(u)$ to *u*. Hence if a plot or *data curve* is made between *s* and r^2/t on logarithmic co-ordinates tracing paper (Fig. 4.32 *a*) to the same scale as the *type curve* *W(u)* versus *u* (Fig. 4.32 *b*), the data curve will be similar to the type curve.

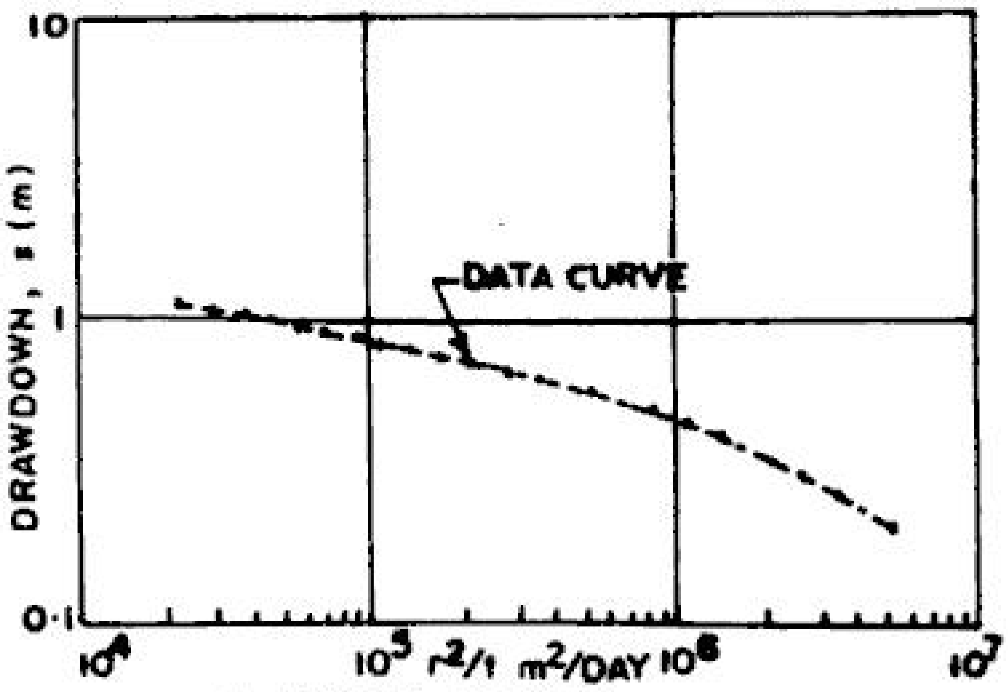
Procedure :

(*i*) In the *observation well* situated at a radial distance *r* from the main well, observe *s* and *t*.

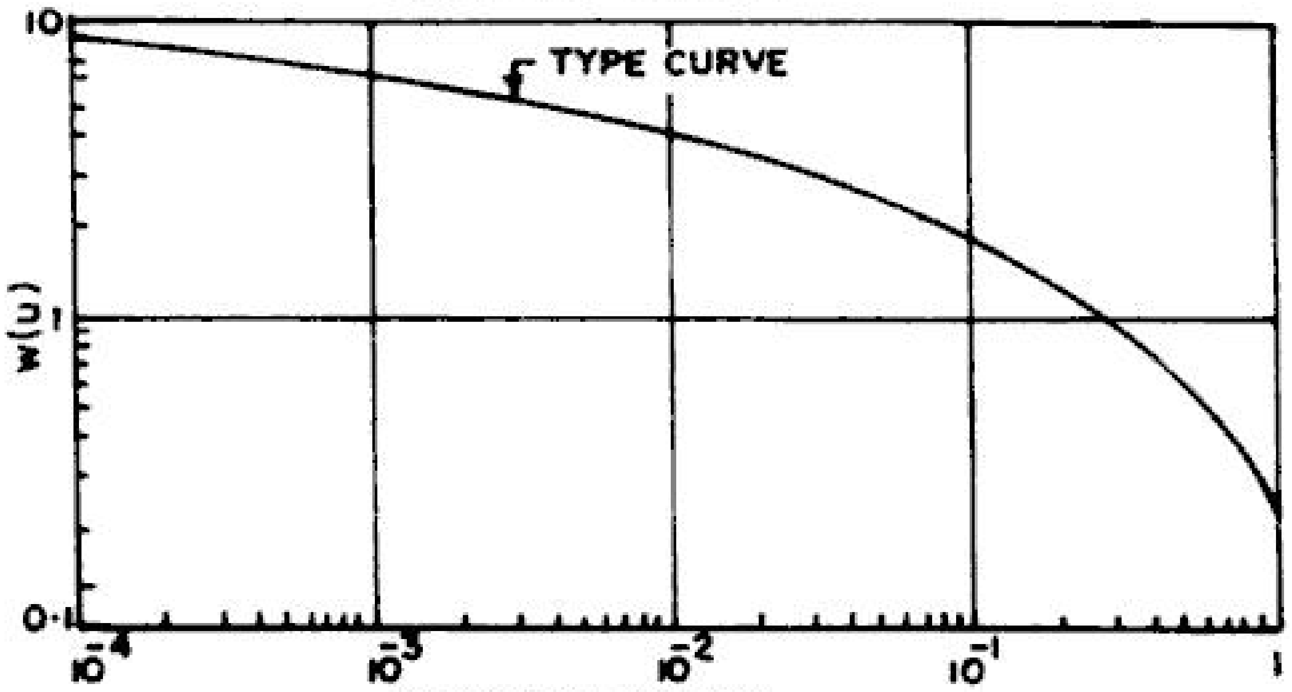
(*ii*) Plot *s* versus r^2/t on a log-log tracing paper (Fig. 4.25 *a*).

(*iii*) Plot *W*(*u*) versus *u* on log-log graph paper (Fig. 4.32 *b*).

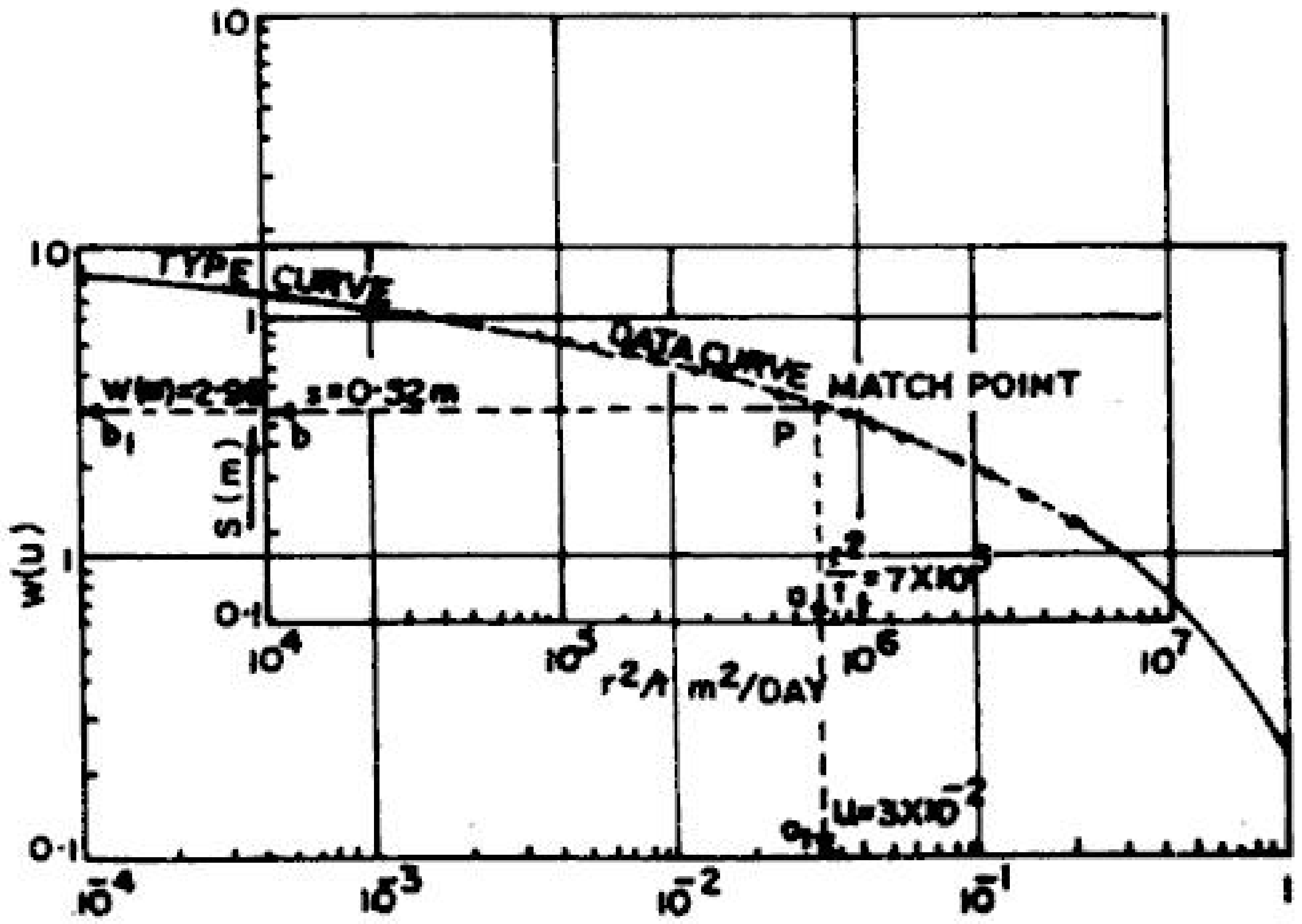
(*iv*) Keep Fig. 4.32 *a* on Fig. 4.32 *b*, and adjust it in such a way that when the co-ordinate axes are held parallel, the data



(a) DATA CURVE



(b) TYPE CURVE



(c) CURVE FITTING

FIG. 4.32. THEIS METHOD.

curve is oriented in a position which represents the best fit of the field data to the type curve.

(*v*) With both graph sheets at the best match position, an arbitrary point P on the top curve is selected and pricked through.

(*vi*) The co-ordinates (a, b) and (a_1, b_1) of the match point are noted from both the curves.

Thus the pair of values $[s, W(u)]$ and $[r^2/t, u]$ are known. The aquifer constants are then calculated from the relations :

$$T = \frac{Q}{4\pi s} W(u) \qquad \text{...(4.27 } a)$$

and

$$S = \frac{4\,uT}{r^2/t} \qquad \text{...(4.27 } b)$$

(*b*) JACOB'S METHOD

Jacob suggested a method which completely avoids curve fitting. He observed that when r is *small* and t is large, u may be small and hence all terms after 2nd term of the series expansion (Eq. 4.25 *b*) of $W(u)$ may be neglected.

Thus
$$W(u) = -\,0.5772 - \log_e u$$

$\therefore$
$$s = \frac{Q}{4\pi\,T}[-\,0.5772 - \log_e u]$$

or
$$s = \frac{Q}{4\pi\,T}\left[\log_e \frac{4\,Tt}{r^2 S} - 0.5772\right] \qquad \text{...(4.28)}$$

For the same observation well, if s_1 and s_2 are the observed drawdown at times t_1 and t_2 since pumping started, we have

$$\Delta s = s_2 - s_1$$

$$= \frac{2.303Q}{4\pi T}\log_{10}\frac{t_2}{t_1} \qquad \text{...(4.29)}$$

If a plot is made between s and $\log_{10} t$ (Fig. 4.33) then for one log cycle of time difference (*i.e.* $t_2 = 10\,t_1$), we get

$$\Delta s = \frac{2.303Q}{4\pi\,T}$$

or
$$T = \frac{2.303Q}{4\pi\,\Delta s} \qquad \text{...(4.30)}$$

Extrapolating the straight line of the curve to intersect with the zero drawdown axis permits the calculation for S. Let t_0 be

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1. Arithmetical increase method.
2. Geometrical increase method.
3. Incremental increase method.
4. Decreased rate of growth method.
5. Graphical extension method.
6. Graphical comparison method.
7. Zoning method or master plan method.
8. Ratio and correlation method.
9. Growth composition analysis method.

1. Arithmetical Increase Method

This is the most simple method of population forecast, though it generally gives lower results. In this method, the increase in population from decade to decade is assumed constant. Mathematically, this hypothesis may be expressed as

$$\frac{dP}{dt} = K \qquad \text{...(5.1 } a\text{)}$$

where $\frac{dP}{dt}$ is the rate of change of population and K is a constant. From the census data of past 3 or 4 decades, the increase in population for each decade is found, and from that an *average* increment is found. For each successive future decade, this average increment is added. The future population P_n after n decades is thus given by

$$P_n = P + nI \qquad \text{...(5.1)}$$

where P_n = future population at the end of n decades
P = present population,
I = average increment for a decade.

This method should be used for forecasting population of those large cities, which have reached their saturation population.

2. Geometrical Increase Method or Uniform Percentage Growth Method

In this method, it is assumed that the *percentage increase* in population from decade to decade is constant. From the population data of previous three or four decades, the percentage increase in population is found and its average is found. If I_g is the average percentage increase per decade, or r_g is the increase per decade expressed as ratio, the population P_n after n decades is given by

$$P_n = P\left(1 + \frac{I_g}{100}\right)^n = P\,(1 + r_g)^n \qquad \text{...(5.2)}$$

Eq. 5.2 can be derived very easily as under :

Let P be the present population and P_1 be the population after one decade.

Then, $$P_1 = P + \frac{I_g}{100}P = P\left(1 + \frac{I_g}{100}\right)^1 \qquad \text{...}(i)$$

Similarly, population P_2 after two decades is

$$P_2 = P_1 + \frac{I_g}{100}P_1 = P_1\left(1+\frac{I_g}{100}\right)^1 = P\left(1+\frac{I_g}{100}\right)^2 \qquad \text{...}(ii)$$

Hence $$P_n = P\left(1 + \frac{I_g}{100}\right)^n$$

While the arithmetical average method is analogous to the 'simple interest method', this method is analogous to the compulation of income by the 'compound interest method'. This method gives *higher results* since the percent increase never remains constant but, instead, decreases when the growth of the city reaches to saturation.

The value of r_g can be found from the expression

$$r_g = \left(\frac{P_n}{P}\right)^{1/n} - 1 \qquad \text{...(5.2 }a)$$

Alternatively, r_g can be determined by computing the *average* of growth rates of several known decades of the past :

$$r = \frac{\text{increase in population}}{\text{original population}} \text{ for each decade.}$$

Knowing r_1, r_2 ..... r_n for each decade, the average value r_g can be found either by *arithmatic average method* or by *geometric average method* :

(*i*) By arithmatic average method :

$$r_g = \frac{r_1 + r_2 + \ldots\ldots r_n}{n} \qquad \text{...(5.2 }b)$$

(*b*) By geometric average method

$$r_g = \left(r_1 \,.\, r_2 \ldots\ldots r_n\right)^{1/n} \qquad \text{...(5.2 }c)$$

The field engineers use the arithmatic average method for computing r_g (or I_g) since it gives slightly *higher* (and hence safer) values. However, the Manual on water supply and treatment recommends to use the *geometric mean method.*

3. Incremental Increase Method

This method combines both the arithmetic average method and the geometrical average method. From the census data for the past several decades, the *actual increase* in each decade is first found. Then the *increment* in *increase for each decade* is found. From these, an *average increment of the increases* (known as incremental increase) is found. The population in the next decade is found by adding to the present population the average increase plus the *average incremental increase* per decade. The process is repeated for the second future decade, and so on. Thus the future population at the end of n decades is given by :

$$P_n = P + nI + \frac{n(n+1)}{2} r \qquad \text{...(5.3)}$$

where P = present population

I = average increase per decade

r = average *incremental increase*

n = number of decades.

Eq. 5.3 can be easily derived as under :

Let P be at the present population. The population P_1 after one decade will be

$$P_1 = P + I + 1\,r \qquad \text{...}(i)$$

Similarly, population P_2 after 2 decades is

$$P_2 = P_1 + I + 2r = P + 2I + 3r = P + 2I + \frac{2(2+1)}{2} r$$

Population P_3 after 3 decades is

$$P_3 = P_2 + I + 3r = P + 3I + 6r = P + 3I + \frac{3(3+1)}{2} r$$

Hence, population P_n after n decades is

$$P_n = P + nI + \frac{n(n+1)}{2} r$$

4. Decreased Rate of Growth Method or Logistic Method

It is found that the rate of increase of population never remains constant, but varies. Fig. 5.1 shows a plot between the population P and the time T for a developing city. The population of a city will grow until it reaches a saturation population which is established by limit of economic opportunity. All populations thus grow according to the *logistic* or *S-curve.* The curve *ABC* (Fig. 5.1 *a*) starts with

Arithmetical Increase	*Geometrical Increase*
$P_L = 57500$	$\log_{10} P_L = 4.7597$
$P_E = 41500$	$\log_{10} P_E = 4.6181$
$P_L - P_E = 16000$	$\log_{10} P_L - \log_{10} P_E = 0.1416$
$0.625\,(P_L - P_E)$	$0.625\,(\log_{10} P_L - \log_{10} P_E)$
$= 10000$	$= 0.0885$
$\therefore \quad P_M = 57500 + 10000$	$\log_{10} P_M = 4.7597 + 0.0885$
$= \mathbf{67500}$	$= 4.8482$
	$\therefore \quad P_M = \mathbf{70500}$

Note. The geometric estimates are higher for post-censal years and lower for inter-censal years.

Example 5.4. *A city has following recorded population :*

1951	*50000*
1971	*110000*
1991	*160000*

Estimate : (*a*) *the saturation population, and* (*b*) *expected population in 2011.*

Solution :

Here $n = 20$ years

$$P_0 = 50{,}000\,;\; P_1 = 1{,}10{,}000\,;\; P_2 = 1{,}60{,}000$$

Hence from Eqs. 5.9 (*a*), (*b*) and (*c*)

$$P_s = \frac{2 \times 50000 \times 110000 \times 160000 - (110000)^2(50000 + 160000)}{50000 \times 160000 - (110000)^2}$$

$$\approx \mathbf{190488}$$

$$a = \log_e \frac{190488 - 50000}{50000} = 1.033$$

$$b = \frac{1}{20} \log_e \frac{50000\,(190488 - 110000)}{110000\,(190488 - 50000)} = -\,0.0673$$

Hence the equation of the logistic curve is

$$P = \frac{190488}{1 + e^{1.033 - 0.0673\,t}}$$

In 2011, $t = 2011 - 1951 = 60$ years.

$$\therefore \qquad P_{2001} = \frac{190488}{1 + e^{1.033 - 0.0673 \times 60}}$$

$$= \frac{190488}{1 + 0.04954} \approx \mathbf{181500}$$

5.6. WATER DEMAND

An average person may consume no more than 5 to 8 litres a day in liquid and solid foods, including 3 to 6 litres in the form of water, milk and other beverages. However, the per capita consumption of water drawn from public supply is quite large. Total water requirements may be divided into the following five categories:

1. Residential or domestic use.
2. Institutional use.
3. Public or civic use.
4. Industrial use.
5. Water system losses.

1. Residential or domestic use

The residential or domestic use includes water requirements for drinking, cooking, bathing, washing of clothes, utensils and house, and flushing of water closets. Provision is sometimes made for domestic animals. IS : 1172–1957 recommends a per capita water consumption of 135 litres per day. Table 5.5 gives the break up of water requirements for domestic purposes, which forms about 50% of the total water requirements per head per day, for all the five categories mentioned above. Table 5.6 gives the water requirements for domestic animals. It should be noted that water required for lawn sprinkling and for residential gardens is over and above the values given in Table 5.5.

TABLE 5.5.
WATER REQUIREMENTS FOR DOMESTIC PURPOSES

S.No.	*Description*	*Amount of water in litres per head per day*
1	Bathing	55
2	Washing of clothes	20
3	Flushing of W.C.	30
4	Washing the house	10
5	Washing of utensils	10
6	Cooking	5
7	Drinking	5
	Total	**135 litres**

TABLE 5.6.
CONSUMPTION OF WATER FOR DOMESTIC ANIMALS AND LIVE-STOCKS

S.No.	*Animals*	*Water consumption in litres per animal per day*
1	Cow and buffalo	40 to 60
2	Horse	40 to 50
3	Dog	8 to 12
4	Sheep or goat	5 to 10

The Manual on water supply and treatment, prepared by the Ministry of Urban Development (MUD) New Delhi recommends the following rates in litres per capita per day (lcpd) for domestic and non-domestic needs (Table 5.7) :

TABLE 5.7.
WATER FOR DOMESTIC AND NON-DOMESTIC NEEDS

Description	*Amount of water (lcpd)*
1. For communities with population upto 20,000	
(*a*) Water supply through stand post	40 (min.)
(*b*) Water supply through house service connection	70 to 100
2. For communities with population 20,000 to 100,000	100 to 150
3. For communities with population above 100,000	150 to 200

2. Institutional Use

The manual on water supply and Treatment recommends the values of water requirements for *institutional needs* as given in Table 5.7.

3. Public or Civic use

Water required for public or civic uses may be for the following purposes : (*i*) Road washing, (*ii*) Sanitation, (*iii*) Public parks, and (*iv*) Fire fighting. For road washing in the municipality area, a provision of 5 litres per head per day is made. Similarly, for sanitary purposes, such as cleaning public sanitary blocks, flushing sewer systems etc., a provision of 3 to 5 litres per head per day may be made. Water required for maintaining public parks etc. may be 2 to 3 litres per square metre per day.

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5. Water System Losses

Losses from a water distribution system consists of (*i*) leakage and over-flow from service reservoirs, (*ii*) leakage from main and service pipe connections, (*iii*) leakage and losses on consumer's premises when they get un-metered house-hold supplies (*iv*) under-registration of supply meters, and (*v*) large leakage or wastage from public taps. Losses in the supply lines are mainly due to defective pipe joints, cracked pipes, and loose valves and fittings. In the case of a well maintained, and fully metered water distribution system, the losses may hardly exceed 20% of the total consumption. In a system where the supply is partly metered (for domestic connections) and partly un-metered (for municipal taps), the losses may be upto 50% of the total supply.

Example 5.5. *Compute the 'fire demand' for a city having population of 1,40,000 using various formulae.*

Solution. P = population in thousands = 140.

1. ***Kuichling's formula***

$$Q = 3182\sqrt{P} = 3182\sqrt{140} = \mathbf{37650\ litres/min.}$$
$$= \mathbf{0.627\ cumecs.}$$

2. ***Buston's formula***

$$Q = 5663\sqrt{P} = 5663\sqrt{140} = \mathbf{67000\ litres/min.}$$
$$= \mathbf{1.117\ cumecs.}$$

3. ***Freeman's formula***

$$Q = 1136\left(\frac{P}{5} + 10\right) = 1136\left(\frac{140}{5} + 10\right)$$
$$= \mathbf{43168}\ \text{litres/min} = \mathbf{0.719}\ \text{cumecs.}$$

No. of fire streams,

$$F = 2.8\sqrt{P} = 2.8\sqrt{140} \approx \mathbf{33.}$$

4. ***National Board of Fire Underwriter's formula***

$$Q = 4637\sqrt{140}\,(1 - 0.01\sqrt{140})$$
$$= \mathbf{48{,}374\ litres/min. = 0.806\ cumecs.}$$

5.7. FACTORS AFFECTING RATE OF DEMAND

The average per capita water consumption varies widely in Indian cities, the demand figures varying from 80 litres to 160 litres Such variation depends upon a number of factors summarised below:

1 ***Size and type of community*.** The fluctuations in demand depends upon the size of the city. In a large city, the fluctuations

in demand may be narrow. In a small city, per capita water consumption is to be expected to be small because there are only limited uses for water in a small town. The fluctuation in use in an industrial community is normally much smaller than in a residential community.

2. ***Standard of living.*** The higher the standard of living is, the higher the demand and greater the variation in demand.

3. ***Climatic conditions.*** If the community is located in hot climate, water use will be increased by bathing, lawn sprinkling and use in parks and recreation fields. Similarly, in extreme cold climates, water may be wasted at the faucets to prevent freezing of pipes, resulting in increased consumption. Extremes of heat and cold cause variations in demand.

4. ***Quality of water.*** Poor-quality water results in a reduction in use.

5. ***Pressure in the supply.*** High pressure results in increased use while low pressure results in decreased use.

6. ***System of supply.*** Water supply may either be continuous, (*i.e.* for all the 24 hours) or intermittent. Generally, intermittent supply will reduce rate of demand.

7. ***Sewerage.*** Use of water increases when sewers are installed in an area previously without them.

8. ***Metering.*** Use of water decreases when the supplies are metered.

9. ***Water rates.*** An increase in water rates will reduce consumption in metered areas.

10. ***Age of community.*** Older, more stable communities use less water than rapidly developing communities where new homes are being constructed and owners are planting new lawns.

11. ***Lawn sprinkling.*** Enforcement of lawn sprinkling regulations can reduce peak demands significantly.

5.8. VARIATIONS IN RATE OF DEMAND

The average daily per capita consumption is obtained by dividing the quantity of water supplied during the year by the number of days in the year and the number of persons served. This per capita consumption or demand varies not only from year to year and from season to season, but more important from day to day and hour to hour. These variations are expressed as percentage of the annual average daily consumption. Some common values are as under :

1. ***Maximum seasonal consumption.*** 130% of annual average daily rate of demand.

2. ***Maximum monthly consumption.*** 140% of annual average daily rate of demand.

3. ***Maximum daily consumption.*** 180% of annual average daily consumption.

4. ***Maximum hourly consumption.*** 150% of average for the day.

A convenient formula for estimating consumption is given by R.O. Goodrich :

$$p = 180\, t^{-0.10} \qquad \text{...(5.23)}$$

where, p = percentage of annual average consumption for time t days from $\frac{2}{24}$ to 360.

Substituting $t = 1$, we get $p = 180$, *i.e.*, the consumption for maximum is 180% of the average.

Substituting $t = 30$ days, we get

$$p = 180\,(30)^{-0.10} = 128\%.$$

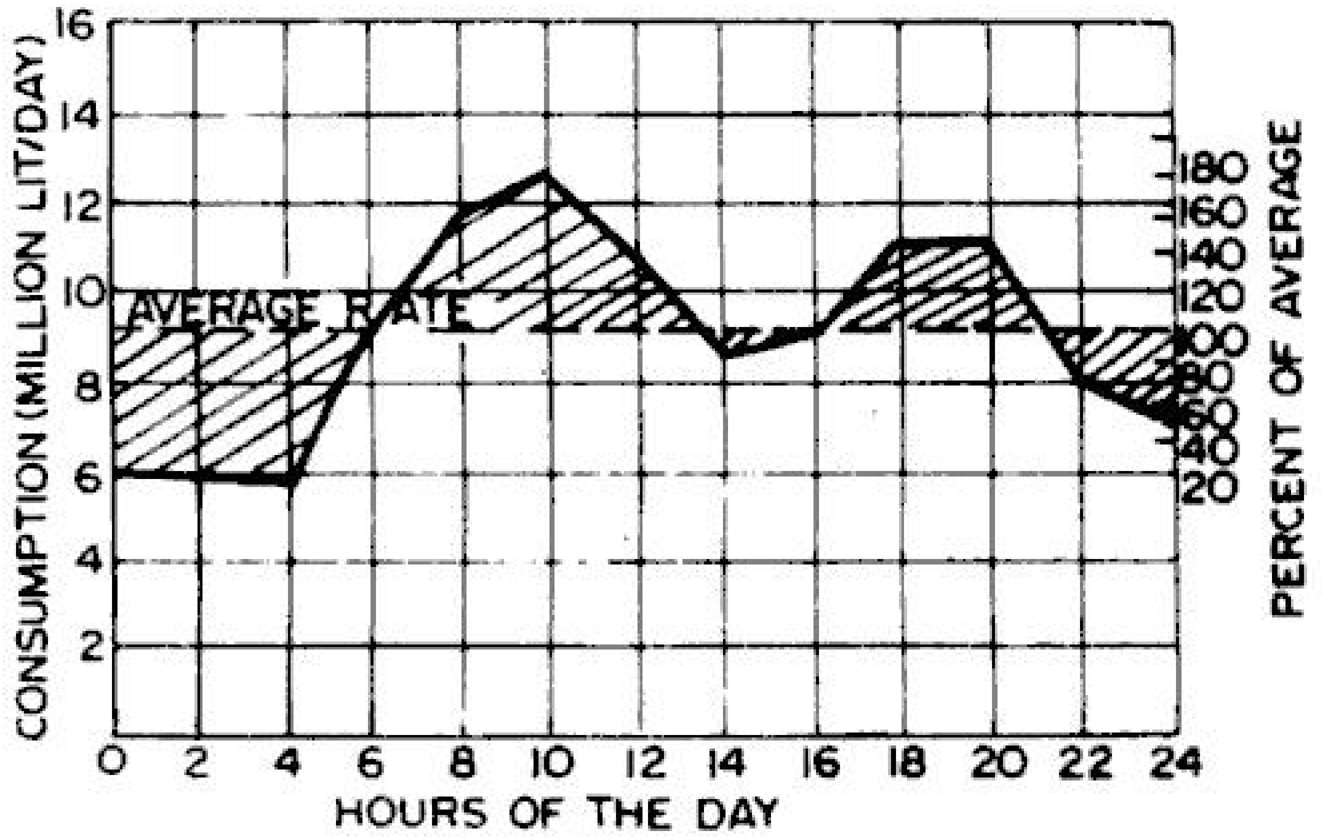


FIG. 5.4. VARIATION IN CONSUMPTION THROUGHOUT THE DAY.

Fig. 5.4 shows the variation in consumption throughout the day. In most of the Indian cities, the peak demand occurs in the morning and evening. During the night hours, the consumption is below the average. The average consumption is shown by the dotted lines. A term *absolute maximum hourly demand* is used to indicate the consumption of maximum hour on maximum day of maximum month of maximum season. Let the annual average consumption in a city be 150 litres per capita per day. The rate of consumption on maximum day will be = 150×1.8 =270 litres. Maximum hourly consumption will be $=\frac{150}{24}\times 1.8\times 1.5=16.9$ litres/hour. The absolute maximum hourly demand (adopting the monthly and seasonal factors suggested above) will be $=\frac{150}{24}\times 1.30\times 1.40\times 1.80\times 1.50=\frac{150}{24}\times 4.914=30.7$ litres/hour (against an annual average of $\frac{150}{24}=6.25$ litres hour).

In order to cope with the hourly variations in the demand, either the pumps may be run at variable speed (which is difficult and cumbersome) or else the pumps may be run at average speed and store the water during the period of less consumption. The second alternative is generally followed. If the pumps are run for all the 24 hours, the rate of delivery by the pumps will be equal to average demand. If they are run only for t hours in a day, the rate of pumping will be $\frac{24}{t}$ times the average consumption. The excess water during the slack demand period is stored in a service reservoir, to be consumed during the periods of peak demand.

Effect of Variation in Consumption on Design

A water supply system has several units, and design of each unit should match with the hourly, daily and seasonal variations in the demand. The design principles taking into account the effect of variation in the consumption are given below :

1. ***Filters and pumps.*** The filter units as well as pumping units are designed for 1.50 times the average daily demand. For example, if the annual average consumption is 150 litres/capita/day, and the population is 50,000, the filter units are designed for $1.50\times 50{,}000=75{,}000$ litres capacity. Similarly, the pumps are designed to deliver 75,000 litres of water in 24 hours. If, however, the pumps are worked only for 12 hours, their hourly discharge will e $\frac{75000}{12}=6250$ litres per hour.

2. ***Distribution mains.*** Distribution mains are designed for the maximum hourly demand of the maximum day. Adopting the factors suggested above, the multiplying factor for the supply will be $=1.8\times 1.5=2.7$.

3. ***Sedimentation tanks and water reservoirs.*** The sedimentation tanks and the clean water reservoirs are designed for the average daily rate of consumption.

PROBLEMS

1. Explain in brief different methods used for prediction of future population of a city, with reference to the design of a water supply system.

2. What do you understand by 'per capita demand' of water ? How is it determined ?

3. Discuss various factors that affect the rate of demand.

4. Explain in brief various expressions used to determine the 'fire demand'.

5. Explain in brief various factors that affect population growth.

6. Write a note on variations in rate of demand. Explain clearly how you take into account these variations in the design of various units.

7. The following data shows the variation in population of a town from 1922 to 1972. Estimate the population of the city in the year 2002. Use various methods :

Year	1922	1932	1942	1952	1962	1972
Population	72,000	85,000	1,10,500	1,44,000	1,84,000	2,21,000

[**Ans.** (*i*) Arithmetical increase method : 3,10,400
(*ii*) Geometrical increase method : 4,35,000
(*iii*) Incremental increase method : 3,46,400

8. In two periods, each of 20 years, a city population grew from 30000 to 172000 to 292000. Find (*a*) saturation population (*b*) the coefficients of logistic equation and (*c*) expected population in the next 20 years.

[**Ans.** (*a*) 3,12,800 (*b*) $a = 2.243$ $b = -0.122$ (*c*) 310800

6

Quality of Water

6.1. INTRODUCTION

The primary objective of water treatment and purification is to collect water from best available source and subject it to processing which will ensure water of good physical quality, free from unpleasant taste or odour and containing nothing which might be detrimental to health. It is not possible to find absolutely pure water in nature. Chemically pure water is the one which contains two parts of hydrogen and one part of oxygen. Water drops just emitting out of the clouds may be considered as chemically pure water, but as the drops fall down they absorb some gases and other matter. It is however not essential to have chemically pure water. Sometimes presence of some minerals give good taste to water. Supplies of water may be obtained by drawing directly upon rivers, lakes, or springs. Depending largely upon the sources from which they are derived, the various waters so obtained may differ greatly in purity and suitability for the purposes for which they are required. Underground water may contain much matter dissolved from state through which it passes. Shallow wells are much more prone to bacterial pollution than are deep wells. Streams and rivers collect impurities from the earth and through the discharge of sewage and industrial effluents and these are carried to ponds, lakes and reservoirs. In the past, there have been many cases of recurring epidemics of enteric fevers traceable to drinking water supplies.

The following are important requirements of water for *domestic use* :

1. It should be colourless, and sparkling clear. It must be free from solids in suspension and must not deposit a sediment on standing.
2. It should be of good taste, free from odour.
3. It should be reasonably soft.
4. It should be plentiful and cheap.
5. It should be free from disease producing bacteria or organisms.
6. It should be free from objectionable dissolved gases, such as sulphuretted hydrogen. It should, however, have sufficient quantity of dissolved oxygen.
7. It should be free from harmful salts.
8. It should be free from objectionable minerals, such as iron, manganese, lead, arsenic and other poisonous metals.
9. It should be free from radio-active substance such as radium, stronsium etc.
10. It should be reasonably free from phenolic compounds, chlorides, fluoride and iodine.
11. It should not lead to scale formation and should be non-corrosive.

The above requirements are only qualitative. The quantitative requirements are given in the subsequent articles.

6.2. COMMON IMPURITIES IN WATER AND THEIR EFFECT

Impurities in water are classified into three heads :

(*i*) Suspended impurities.
(*ii*) Dissolved impurities.
(*iii*) Colloidal impurities.

Suspended impurities are those which normally remain in suspension. They are microscopic and make water turbid. Dissolved impurities are not visible, but they are large since water is good solvent. They cause bad taste, hardness and alkalinity. Sometimes, they are harmful. Colloidal impurities are electrically charged. Due to this, the colloidal particles, usually very small in size, remain in constant motion and do not settle. Table 6.1 gives summary of various suspended and dissolved impurities.

TABLE 6.1. SUSPENDED AND DISSOLVED IMPURITIES

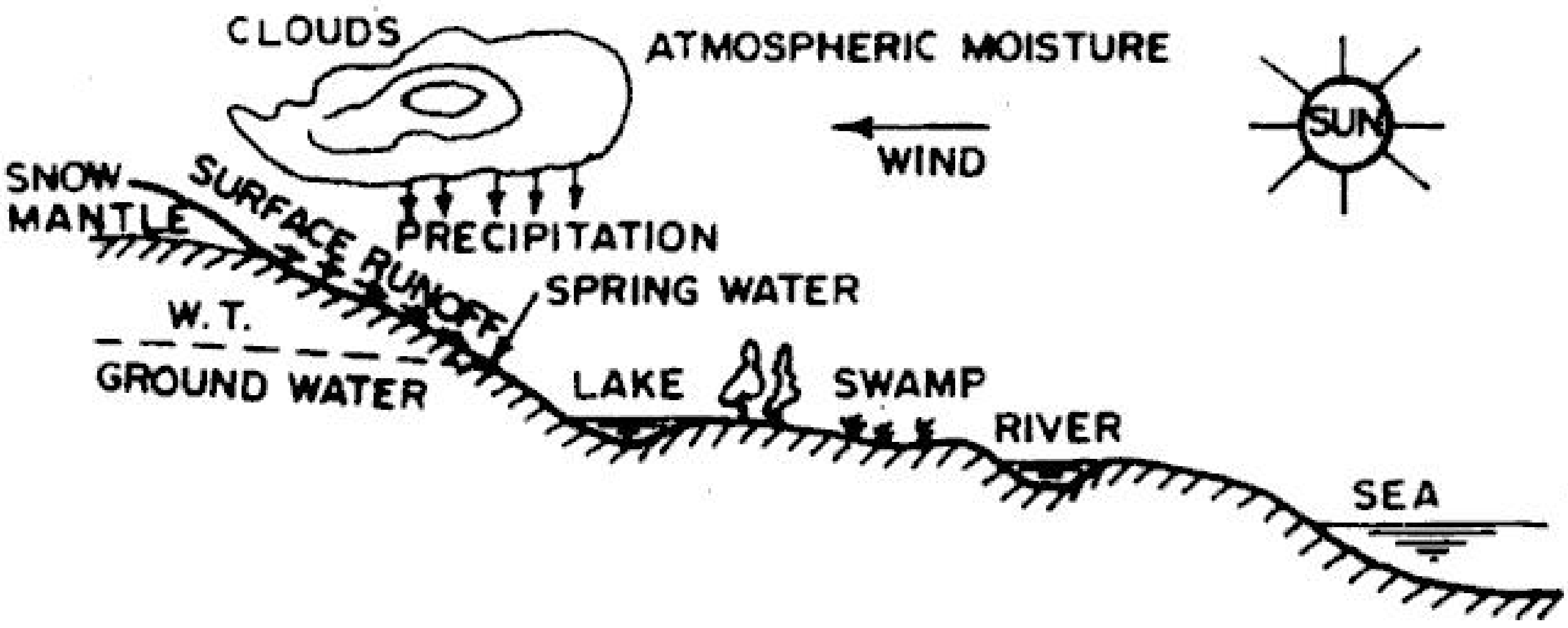
Type	*Constituents*	*Effect*
1 **Suspended Impurities**	(*a*) Bacteria	Some cause diseases
	(*b*) Algae, Protozoa	Odour, colour, turbidity
	(*c*) Silts	Murkiness or turbidity
2 **Dissolved Impurities**	(*a*) *Salts*	Alkalinity
	(*i*) Calcium and Magnesium —Bicarbonate	Alkalinity, hardness
	—Carbonate	Hardness
	—Sulphate	Hardness, corrosion
	—Chloride	
	(*ii*) Sodium —Bicarbonate	Alkalinity, softening effect
	—Carbonate	Alkalinity, softening effect
	—Sulphate	Foaming in boilers
	—Fluoride	Dental flurosis or mottled enamel
	—Chloride	Taste
	(*b*) *Metals and Compounds*	
	(*i*) Iron oxide	Taste red colour, corrosiveness, hardness
	(*ii*) Manganese	Black or brown colour
	(*iii*) Lead	Cumulative poisoning
	(*iv*) Arsenic	Toxicity, poisoning
	(*v*) Barium	Toxic effect on heart, nerves
	(*vi*) Cadmium	Toxic, illness
	(*viii*) Cyanide	Fatal
	(*viii*) Boron	Affect central nervous system
	(*ix*) Selenium	Highly toxic to animals, fish
	(*x*) Silver	Discolouration of skin; eyes
	(*xi*) Nitrates	Blue baby conditions; infant poisoning; colour; acidity
	(*c*) *Vegetable dyes*	
	(*d*) *Gases* —Oxygen	Corrosiveness to metals
	—Carbon dioxide	Acidity, corrosiveness
	—Hydrogen sulphide	Odour, acidity, corrosiveness

6.3. QUALITY OF SOURCE

Natural water is available from the following sources :

(*i*) Pure water
(*ii*) Rain water
(*iii*) Surface runoff water
(*iv*) River water.
(*v*) Lakes and pond water, swamp water.
(*vi*) Ground water

Fig. 6.1 Shows the characteristic properties of natural water in hydrologic cycle.



CHARACTERISTICS OF NATURAL WATERS

1. SEAWATER	:	SALTS
2. PRECIPITATION	:	GASES, VAPOURS, PARTICULATES, SALT NUCLEI, RADIO-ACTIVE FALL OUT
3. SURFACE RUNOFF	:	PARTICULATES, ORGANIC MATTER, NITRATES, PHOSPHATES, BIOCIDES
4. GROUND WATER AND SPRING WATER	:	CARBONATES, CHLORIDES AND SULPHATES OF CALCIUM AND MAGNESIUM; IRON AND MANGENESE ; SO_2 ; H_2S
5. LAKE WATER	:	ALGAE, ODOURS, TASTES
6. SWAMP WATER	:	COLOUR, ODOURS, TASTES
7. RIVER WATER	:	PARTICULATES ; ORGANIC MATTER; WASTE WATERS

FIG. 6.1. CHARACTERISTIC PROPERTIES OF NATURAL WATERS IN HYDROLOGIC CYCLE

(*ii*) ***Mesolimnion (or Thermocline)***. It is a zone, 3 to 6 m deep, having rapidly changing temperature at the rate of 1° C or more per 1 m change in depth. The primary disadvantage of this zone is that the oxygen content is depleted through microbial activity. Since there is no opportunity of reaeration, anearobic conditions are set in with an increase in iron, carbon dioxide, manganese, H_2S taste and odour. All these are potentially detrimental to water quality.

(*iii*) ***Hypolimnion*** : This is the bottom most zone which is cold. It has practically no dissolved oxygen. It has very high dissolved iron and manganese. Anaerobic conditions exist in this zone. Also, there is an increase in CO_2 and decrease in pH.

6.4. WATER ANALYSIS

Various types of impurities present in water can be determined by water analysis. This analysis is done both for raw water as well as treated or purified water. The examination of raw water will enable us to determine the outline or processes of water purification. Water analysis of purified water is done to know whether the degree of purification has reached the required standards or not. Following are the purposes of water analysis of raw water and purified water:

1. To classify the water with respect to general level of mineral constituents.
2. To determine the degree of clarity and ascertain the nature of matter in suspension.
3. To determine the chemical and bacteriological pollution of water.
4. To determine the presence/absence of an excess of any particular constituent affecting potable quality and general use.
5. To determine the level of organic impurities.
6. To set the outlines of purification process and specify various stages in it.
7. To ascertain whether purification of water has reached the required standards or not.

Thus to summarise, examination of water is used to classify, prescribe treatment, control treatment and purification processes and maintain public supplies of an appropriate standard of organic quality, clarity and palatability.

The examination of water may be divided into three classes:

1. Physical Examination. 2. Chemical examination.
3. Microbiological examination.

Sampling. The proper collection of water sample entails the most careful consideration of all relevant circumstances, as regards choice of sampling point, method of collection and frequency of testing. Frequency of testing directly depends upon the variability of quality of raw water and to the hazards of pollution. For deep underground water, where chances of pollution are less, the testing may be less frequent. In the case of impounded water, where chemical and physical data are intimately related to biological investigation, seasonal factors and pollutional aspects determine how often a river water is to be examined. The samples should be collected in clean and sterilized bottles having stoppers. The water and the bottle should not be touched while collecting the sample.

6.5. PHYSICAL EXAMINATION

These consist of (*i*) Colour test (*ii*) taste and odour test (*iii*) temperature test, and (*iv*) turbidity test.

1. COLOUR TEST

Water gets its colour due to leaves, peat, logs and other organic matter either in true solution or in colloidal suspension. Colour can be measured against various standards or scales such as Hazen or Platinic chloride scale, Burgess scale or cobalt scale using a tintometer. The apparent colour of water due to turbidity is first removed by centrifuging. This is achieved by placing the sample of water in a centrifuge and rotating it at a very high speed so that the vortex caused due to centrifuge action spills out suspended matter. The sample is then ready for determination of its true colour, using a tintometer. The instrument has an eye-piece with two holes. A the slide of *standard coloured water* is seen through one hole, while the slide of the water to be tested is seen through the other hole. A number of slides of standard colour in water are kept ready for comparison. The intensity of colour in water is measured in terms of arbitrary *unit of colour* on the cobalt scale. The unit of colour is that produced by one milligram of platinum cobalt in one litre of distilled water. For public water supply, the number on cobalt scale should not exceed 20, and should preferably be less than 10. According to IS 3025-1964, if 1.245 g of potassium chloroplatinate and 1.0 g of cobalt chloride are dissolved in distilled water and diluted to 1 litre, it has a colour of 500 Hazen units.

2. TASTE AND ODOUR TESTS

Taste and odour tests are of prime importance in assessing the palatability of water. They may be there due to the presence of mineral salts, tarry substances, industrial wastes, domestic sewage, decomposing organic matter, certain types of microscopic organisms or chemical compounds such as phenols etc. However, taste and odour are the most difficult physical characteristics to measure in any numerical sense because of personal factors related to taste and odour, atmospheric conditions of impurity, temperature and humidity. Odour is meaured in terms of *thresh hold odour number.* This is numerically equal to the volume of sample of water in cubic centimetres required to be added to 100 c.c. of odour free fresh water when the mixure just starts giving the typical smell in the diluted sample. For example, if 6 c.c. of water added to 100 c.c. of fresh water does not give any odour, while if more than 6 c.c. of water when added to 100 c.c. of water gives odour in the diluted sample, the thresh hold number will be 6. For public water supply, thresh hold number should not be more than 3. The test is conducted at 20°C. *Alternatively,* the odour test may be carried out by inhaling through two tubes of an instrument called *Osmoscope.* It consists of two 10 mm dia. glass tubes 30 cm long kept about 8 mm apart. The lower end of each tube is plain and is held slightly above water surface while the upper beveled end can be inserted in the nostril. Diluted water samples in the ratio of 1:2, 1:4, 1:8, 1:16 etc. are prepared. One tube is kept in a flask containing one of the above diluted water sample, while the other tube is kept in the flask containing original water sample. The mixure with greatest dilution is tested first, and then others with lesser and lesser dilution. The mixure giving first detectable odour will be taken as *thresh hold odour,* while the number of additions of fresh water in such mixure is known as *odour-intensity* or pO *value.* Following table gives the odour interpretation of the pO values :

pO value	*Meaning*
0	No perceptible odour.
1	Very faint odour, detectable only to trained observer.
2	Faint odour, detectable to average person only if he is told.
3	Distinct odour, readily detectable by most people.
4	Very distinct odour.
5	Strong and intense odour.
6	Extremely strong odour.

3. TEMPERATURE TEST

The temperature of water to be supplied should be between 10°C to 20°C. Temperature higher than 25°C is considered objectionable. It can be measured with ordinary thermometers graduated in

0.1°*C*, range 0 to 50°. In case of large bodies of surface water, where it is required to record temperature at a certain depth, *'broken capillary thermometer'* is used. At depths greater than 15 m, a *thermo-couple* may be used. Temperature measurements are sometimes important to identify the magnitude of density, viscosity, vapour pressure and surface tension of water. Other factors such as saturation values of solids and gases dissolved in it, B.O.D. values and the biological activities are dependent on temperature of water.

4. TURBIDITY TEST

Turbidity is imparted by the colloidal matter present in water. The colloidal matter may be clay and loam or microscopic organisms. It is a measure of the resistance of water to the passage of light through it. Hence turbidity is estimated against standard suspensions of a siliceous material such as Fuller's earth, the silica scale being based on standardization by photometric means. The *standard unit* of turbidity is the turbidity produced by one part of Fuller's earth in a million parts of distilled water. Turbidity is expressed in terms of parts of suspended matter per million parts of water by weight, abbreviated as p.p.m. One unit or 1 p.p.m. is equivalent to 1 mg per litre. The permissible turbidity of domestic water may be between 5 to 10 p.p.m.

The following are common methods of measuring turbidity of water : (*i*) by turbidity rod (*ii*) by Jackson's turbidimeter (*iii*) by Baylis turbidimeter (*iv*) by Nephelometers.

(*i*) **Turbidity Rod.** Turbidity rod is used for measuring turbidity of water in the field. It consists of a graduated aluminium rod, about 20.3 cm in length, at the upper end of which is attached a graduated non-stretchable tape of about 12.2 cm long. At the lower end of the aluminium rod, a screw containing a platinum needle (of 1 mm diameter and 2.5 cm length) and a nickel ring is inserted. The graduated tape has a mark at its top end specifying the position of eye during the test. In order to find the turbidity, the lower end of the rod is gradually immersed in water whose turbidity is to determined. Eye is kept constantly at the marked position and the platinum needle is watched. The rod is moved slowly in water till the platinum needle just disappears from the vision due to turbidity of water. The reading of the graduated tape near the water surface directly gives turbidity in p.p.m. The rod gives only rough value of the turbidity of water.

(*ii*) **Jackson's Turbidimeter.** This is a laboratory apparatus which is used to measure turbidity when it is more than 25 p.p.m., and preferably when it is more than 100 p.p.m. It consists of a

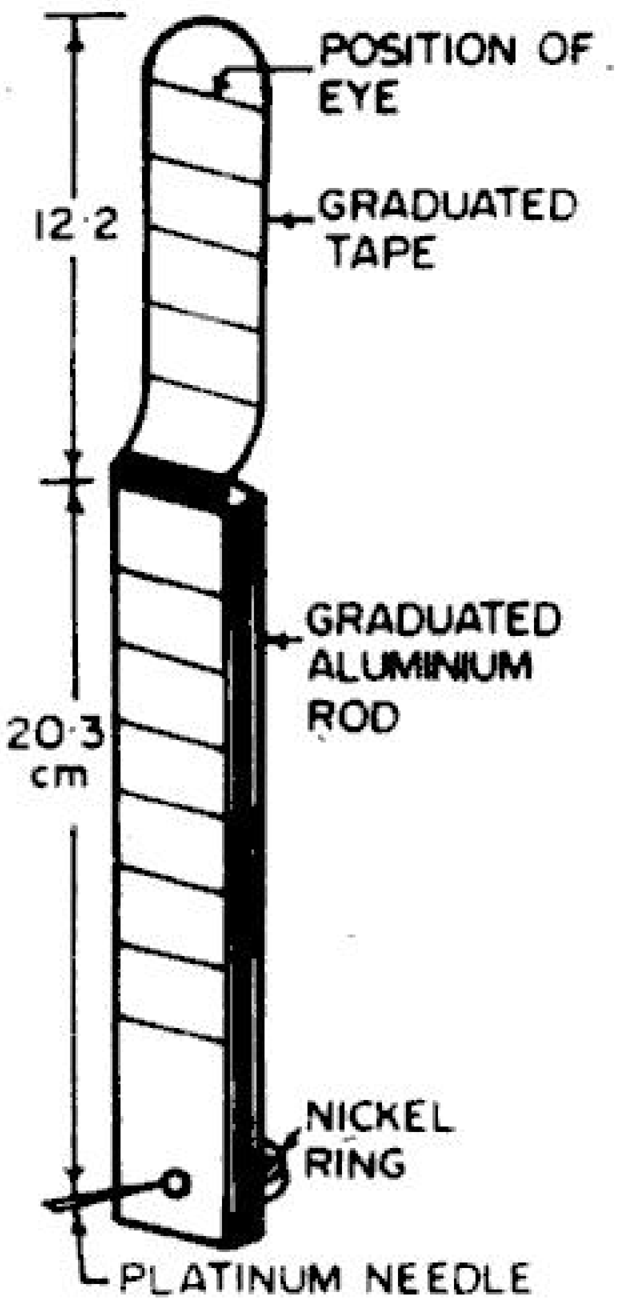


FIG. 6.4. TURBIDITY ROD

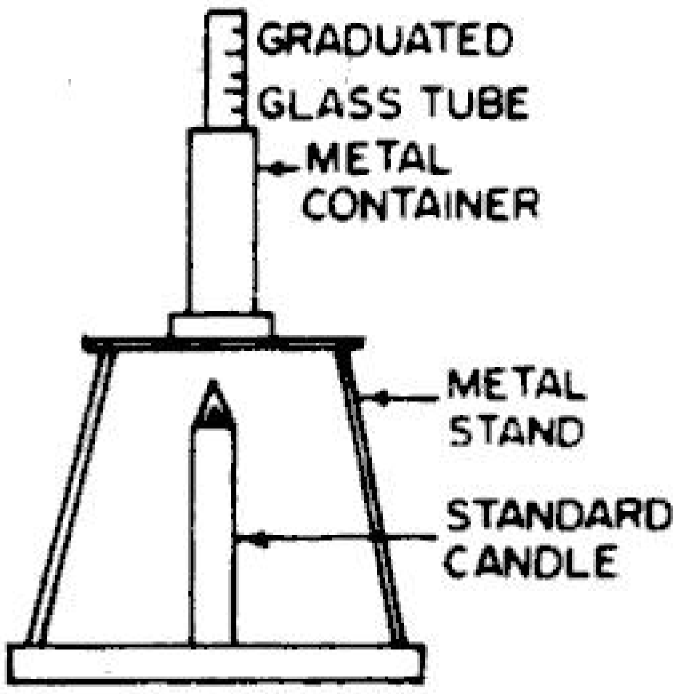


FIG. 6.5. JACKSON'S TURBIDIMETER

metal stand holding a metal container and a graduated glass tube in it, as shown in Fig. 6.5. A standard candle is placed below the stand. The water sample is poured in the sample and the image of the flame of the standard candle is seen through the turbid water in the glass tube. The level of water in the glass tube is gradually increased till the image of the flame ceases to be seen. The height of water column, measured in the graduated glass tube provides the measure of the turbidity of the water. The longer the light path, the lower the turbidity; a light path of 21.5 cm corresponds to 100 JTU (Jackson Turbidity Unit) while light path of 10.8 cm corresponds to 200 JTU where 1 JTU = 1 p.p.m.

(*iii*) **Baylis Turbidimeter.** This is a very accurate instrument and is preferred when turbidity of the sample is less than 5 units. It consists of a galvansied iron box in which two glass tubes are kept at one end and a 250 watt bulb with reflector is placed at the other end as shown in Fig. 6.6. One tube contains standard solution of known turbidity while in the other tube the water sample is kept. The tube is held firmly in a platform with beveled holes at its bottom end. The tubes are surrounded on all its four sides by blue cobalt plates, and at its bottom by a white opal glass plate. Because of blue cobalt plates, blue light is cast in both the tubes, and a comparison is made. If the light differs, another tube containing standard solution of different turbidity is introduced in the place of the first one till the colour in both the tubes match. The standard solution at this stage give the turbidity of the given water sample. The turbidity is expressed either as p.p.m. or BTU (Baylis' Turbidity Unit), both being equivalent.

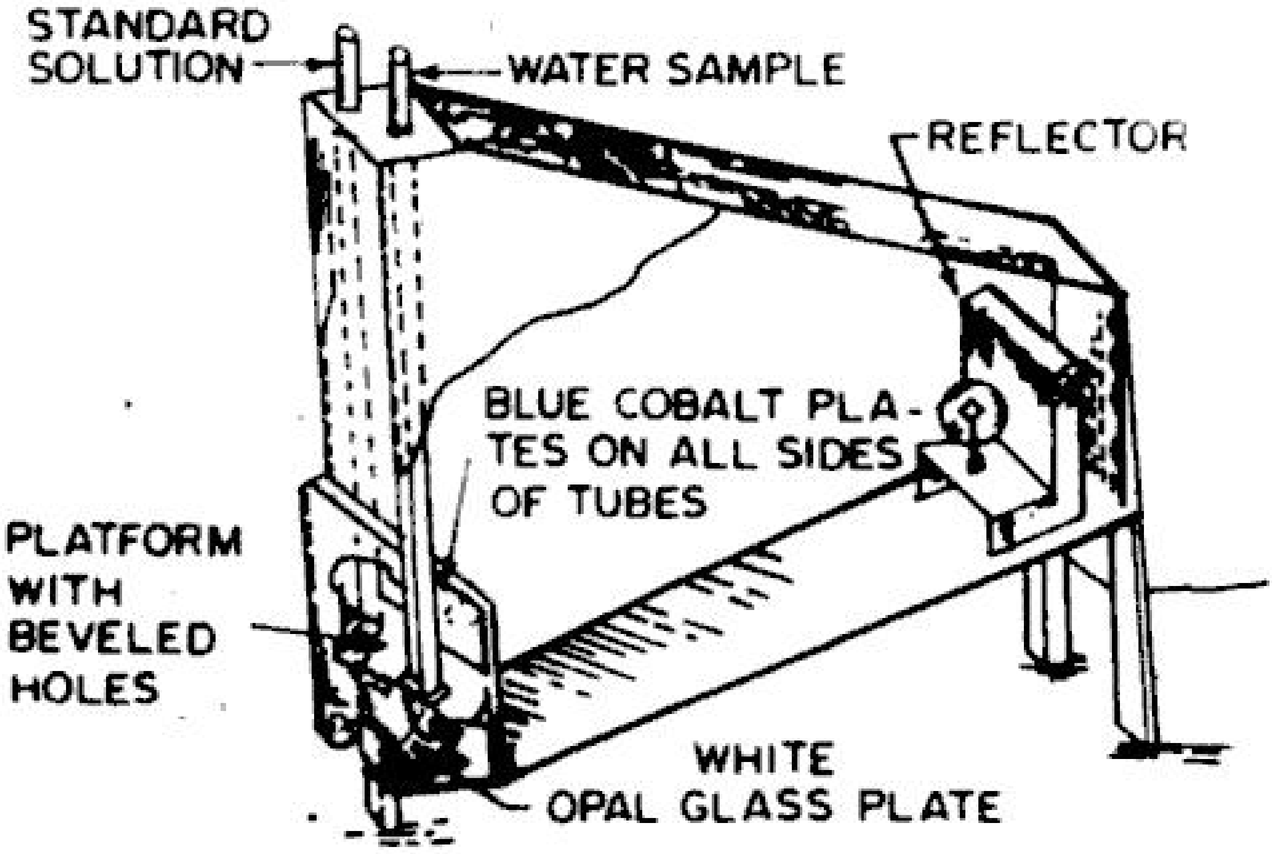


FIG. 6.6. BAYLIS' TURBIDIMETER

(*iv*) **Nephelometers** : Nephelometers are the *modern commerical turbidimeters* in which the light intensity is measured at right anlges to the incident ray in contrast to the conventional turbidimeters (described earlier) in which the light intensity is measured after the same passes straight through the turbid water. To achieve this, nephelometers use photometers. A photometer measures the intensity of light passing through the turbid water, after the same is scattered at right angles to the incident ray. The value of turbidity measured by a nephelometer is generally expressed as NTU (Naphelometric turbidity unit). Since formazin polymer is used as reference turbidity standard suspension, naphelometric turbidity units are also sometimes referred to as FTU (Formazin turbidity unit).

6.6. CHEMICAL EXAMINATION

Chemical tests are carried out to determine the following :

(*i*) Total solids. (*ii*) Chlorides. (*iii*) Hardness. (*iv*) pH value. (*v*) Metals and other chemical substances. (*vi*) Nitrogen and its compounds. (*vii*) Dissolved gases.

1. TOTAL SOLIDS

In this test, the suspended, dissolved and colloidal solids are determined separately and then added together to get total solids.

$$= -\log_{10}[50.5 \times 10^{-7}] = -\log_{10} 50.5 - \log_{10} 10^{-7}$$
$$= -1.703 + 7 \approx 5.297 \approx \mathbf{5.3}$$

Example 6.5. *Find out the pH of a mixture formed by mixing the following two solutions :*

Solution A : volume 300 ml ; pH = 7
Solution B : volume 700 ml ; pH = 5

Solution

$$[H^+]_A = 10^{-7} \text{ moles/litre}$$

and $$[H^+]_B = 10^{-5} \text{ moles/litre}$$

For solution A

Concentration of $[H^+]$ in 300 ml $= 10^{-7} \times \frac{300}{1000}$

$$\therefore \quad [H^+]_A = 3 \times 10^{-8} \text{ moles/litre.}$$

For solution B

Concentration of $[H^+]$ in 700 ml $= 10^{-5} \times \frac{700}{1000}$

$$\therefore \quad [H^+]_B = 7 \times 10^{-6} \text{ moles/litre.}$$

$\therefore$ Concentration of $[H^+]_{mix} = [H^+]_A + [H^+]_B$

$$= [3 \times 10^{-8}] + [7 \times 10^{-6}] \text{ moles/litres}$$
$$= [0.03 \times 10^{-6}] + [7 \times 10^{-6}] \text{ moles/litres.}$$
$$= 7.03 \times 10^{-6} \text{ moles/litre}$$

$$\therefore \quad pH = -\log_{10}(7.03 \times 10^{-6}) = -0.847 + 6$$
$$= \mathbf{5.153}$$

5. METALS AND OTHER CHEMICAL SUBSTANCES

(*i*) Iron and Manganese

Iron in water causes hardness, bad tastes, discolouration of clothes and plumbing fixures and incrustation in water mains. Iron in water is frequently accompanied by heavy growths of *crenothrix* which exaggerates the staining, pipe clogging and other troubles. A water which contains iron is termed ferryginous. Estimation of iron is a simple calorimetric procedure and thiocyanate or thioglycollic acid are most usually employed for the purpose. Manganese imparts a brownish or purplish colour to water and laundered goods (when oxidised) and stains plumbing fixures. It also impairs tastes of coffee and tea. Manganese is estimated by matching the pink colour produced on oxidation to permanganate. Sodium paraperiodate or ammonium persulphate is used to colour both the water sample and manganese colour standards.

(*ii*) Lead and Arsenic

Lead is a well known cumulative poison. Lead taken in body by either brief or prolonged exposures can seriously injure health or cause death. Under normal conditions, the concentration of lead should be less than 0.05 mg/l. The presence of lead can be detected by six drops of sulphuric acid in a glass of water, which will yield white precipitate. The newer organic reagents have provided in dithizone another method for estimation of lead in addition to the older determination as sulphide. The toxicity of arsenic is well known. Chronic arsenic poisoning is difficult to diagnose, and can be quite disabling prior to detection. Arsenic can be removed from water in ion exchange equipment using activated alumina or bone char. Arsenic can be determined in trace amounts by heteropoly blue calorimetric method and the Gutzeit method of comparing stains produced on treated paper strips.

(*iii*) Fluorides and Iodides

If present in small concentrations, both fluoride and iodide may be useful to human beings. Iodine upto 1 p.p.m. keep away goiter while fluoride upto 1.2 p.p.m. will prevent dental caries of the children. Excess concentrations (more than 3 p.p.m.) can cause dental fluorosis or mottled enamel in children. Fluorides may be estimated calorimetrically by developing a colour with zirconium – alizarin reagent. Iodide can be determined in water supplies by utilizing its ability to catalyze the reduction of ceric ions by arsenious acid, the effect being proportional to the amount of iodide present.

(*iv*) Barium and Boron

Barium can have serious toxic effects on the heart, blood vessels, and nerves. It occurs naturally in some mineral springs as the carbonate salt. Ingestion of large amounts of boron can affect the central nervous system, and protracted ingestion may cause *borism*. Its concentrations can be determined by calorimetric and titrimetric methods. In the presence of boron, a solution of carmine or carminic acid in concentrated sulphuric acid changes from bright red to a bluish red or blue, depening on the concentration of boron present.

(*v*) Cadmium and Hexavalent Chromium

Cadminum is highly toxic. It can become a water contaminant through careless discharge of electroplating plant washes or from galvanizing operations in which cadmium is a contaminant. Cadmium ions from an intense red compound with diphenyl-thiocarbzone (dithizone) which can be extracted into carbon tetrachloride and used for the chlorimetric determination of cadmium. Chromium, when inhaled can be cause of cancer in man, though the results of ingestion are not known. The likely sources of hexavalent chromium in water could be pollution by wastes from chromium plating shops. Soluble hexavalent chromium in the absence of interfering amounts of substances such as molytodenum vanadium and mercury may be determined colorimetrically by reaction with diphenyl-carbazide in acid solution.

(*vi*) Sodium and Potassium

Biological regulatory mechanism in the body of a normal human maintains a constant sodium content in the body despite wide variations in sodium intake. Excess quantities are excreted, primarily in the urine. However, some patients of heart, kidney or lever diseases are unable to rid themselves of sodium resulting in a condition known as *edema* in which large amounts of sodium-containing fluid accumulate. The concentration of sodium can be determined either by the flame photometric method by the grammetric method. Sodium and potassium may however by estimated together and expressed as sodium chloride. In special circumstances, sodium may be separately determined as sodium zinc uranyl acetate. Potassium may be estimated as the perchlorate which is sparingly soluble in organic solvents or as dipotassium cobalt-nitrite.

(*vii*) Phenols

Water receives phenols from trade wastes, washing from tarmac road, gas liquours and creosoted surfaces. Trace phenol concentrations of the order of 0.001 ppm can impart an objectionable taste to a water following marginal chlorination. The removal of phenolic tastes from a water supply offers a serious challenge at the treatment plant. Phenols can be determined by amino antipyring method or by Gibbs colorimetric method.

6. NITROGEN AND ITS COMPOUNDS

Nitrogen can be present in water in the following four different forms :

(*i*) Ammoniacal Nitrogen (free and saline ammonia)
(*ii*) Albuminoid Nitrogen.
(*iii*) Nitrite
(*iv*) Nitrate.

(*i*) Ammoniacal Nitrogen

The occurrence of *free ammonia* indicates the direct inclusion of organic matter, particularly those arising from the excrement (urine) of animal and human species. Surface water may also get polluted

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1. Record the quantities tested in order of magnitudes, total number of tests and number of positive results.
2. Record ratio of positive to total successive ratios (column 4).
3. Find the difference between successive ratios (column 5)
4. Find the product of each difference and the reciprocal of the larger quantity used in computing the ratio from which difference was taken (column 6).
5. The *E-Coli index* is the sum of these products.

TABLE 6.5. E-COLI INDEX

Quantities c.c. (1)	*No. of tests* (2)	*Number positive* (3)	*Ratio (3) : (2)* (4)	*Difference between lines in col. (4)* (5)	*Product* (6)
10	30	20	0.667		
1	30	10	0.333	0.334	0.0334
0.1	30	5	0.167	0.166	0.166
0.01	30	4	0.1333	0.0337	0.337
0.001	30	1	0.05	0.0833	8.333
				Sum	8.869

∴ E-coli-Index = 8.87.

The above method of finding E-coli index has several anamolies, if the observations in column 3 do not fall on the probability curve to get consistent values in column 6. For example, if the 5 th line in colum 3 has a value 2 (instead of 1), the value in column 6 would by 3.333 instead of 8.333. Obviously, for a greater number of positive results, we should have higher index. Thus, error results from assuming a value of 2 which falls off the probability curve in column 3.

The Most Probable Number (M.P.N.)

The shortcomings of the Coliform Index or E-Coli Index have caused it to become obsolete and it has been replaced by *most probable number* (M.P.N.).

The *most probable number* (M.P.N.) of coliform or E-coli is defined as that bacterial density, which if it had been actually present in the sample under examination, would more frequently than any other, have given the observed analytical result. The M.P.N. is determined by applying laws of probabilities to the available test data. Standard tables are available to determine M.P.N. for various combination of test results. Table 6.6 gives M.P.N. of organisms per

100 ml of sample and confidence limits using 1 tube of 50 ml and 5 tubes of 10 ml. For other Tables, reference may be made to IS: 1622–1964.

TABLE 6.6
MPN OF ORGANISMS PER 100 ML OF SAMPLE USING 1 TUBE OF 50 ml AND 5 TUBES OF 10 ml

Number of positive tubes		*MPN per 100 ml*	*Limits within which MPN per 100 ml can lie*	
50 ml tubes	*10 ml tubes*		*Lower limit*	*Upper limit*
0	1	1	<0.5	4
0	2	2	<0.5	6
0	3	4	<0.5	11
0	4	5	1	13
1	0	2	<0.5	6
1	1	3	<0.5	9
1	2	6	1	15
1	3	9	2	21
1	4	16	4	40

Sampling for Bacteriological Tests

Bacteriological samples should be collected in sterile, clear glass bottles having ground glass stoppers which are protected with a piece of sterile linen, oiled silk or paper tied around the neck of the bottle. Before collecting the samples, the bottle should be sterilized in hot air oven at 160°C for one hour or in an autoclave at a pressure of at least 1 kg/cm^2 (121°C), maintained for 20 minutes. The bottle should not be opened until immediately before the sample is to be taken and the stopper, when withdrawn from the bottle should be kept in its covering and held pointing downwards. If the water to be sampled contains or is likely to contain chlorine, a minute crystal of sodium thiosulphute or 0.1 ml of sodium thiosulphate solution (0.3%) should be added to the bottle before sterilization. Bacteriological testing of water should be commenced as soon as possible after collection. Where this is not possible, the samples should be kept in ice or in a refrigerator till they are taken up for analysis. All such iced or chilled samples should be taken up for tests within 48 hours after collection.

1. TOTAL COUNT OR AGAR PLATE COUNT TEST

This test is carried out to find the number of bacteria colonies growing on a specified medium incubated for a specified period of time and at a specified temperature. The enumeration of the total bacteria is therefore known as *Colony Count*. Various methods are available, but the following three methods are in general use for water testing.

(*i*) Colony Count in *nutrient agar* after 3 days of growth at 20 – 22°C.

(*ii*) Colony Count in nutrient agar after 24 hours incubation at 37°C.

(*iii*) Colony Count after continuation of the 37°C incubation for a further 24 hours.

Out of this, the second method is more common. A suitable volume of water sample (or dilution of water sample with sterilised sample) is transferred with aseptic precautions to a sterile glass covered *Petridish* (100 mm diameter and 15 mm deep) by means of a sterile pipette. Molten nutrient agar, cooled to 45°C, is added and thoroughly mixed with the sample in the Petridish when the mixture solidifies, the Petridish is transferred to incubator (37°C), inverted and left for 24 hour for incubation. The bacterial colonies which are formed are then counted with the help of a magnifying glass. Sometimes a specially constructed box (such as Quebec colony counter) is used, which allows the examination of the plates by combined reflected and transmitted light against a dark background in a blacked out room. The results of the colony count are computed for 1 ml of the water sample. For potable water, the total count should not exceed 100 per ml.

2. THE COLIFORM TEST

As defined earlier, the coliform group includes all of the aerobic and facultaive anaerobic Gram-negative, non-spore forming, rod shaped bacteria which ferment lactose with gas formation within 48 hours at 35°C. Two standard test techniques are available for the coliform group.

(*a*) Multiple tube fermentation technique.
(*b*) Membrane filter technique.

(*a*) **Multiple tube fermentation technique.** This consists of the following tests in three stages :

(*i*) Presumptive test.
(*ii*) Confirmed test.
(*iii*) Completed test.

(*i*) **Presumptive test.** The presumptive test consists of inoculating measured amounts of water sample into tubes of Mac Conkey bile salt lactose broth and incubating the cultures at 37°C for 24 to 48 hours. If acid and gas are formed, the test is positive. If they are not formed, the test is negative. If all the tubes are negative, the conclusion is that coliform bacteria are not present in the sample. Positive results require further testing for *confirming* the presence of coliform bacteria in the sample.

The amount of water sample used in the test depends upon the anticipated bacterial purity of water. Volume of water samples in different tubes are taken in multiples of ten, such as 0.01 ml, 0.1 ml, 1 ml, 10 ml, 50 ml etc. Following are the combinations of tubes of media suggested for various types of water :

1. Raw river water–
   5 tubes of 1 ml, 5 tubes of 0.1 ml and 5 tubes of 0.01 ml.
2. Stored water, rapid filtered water or polluted well water–
   1 tube of 50 ml, 5 tubes of 10 ml and 5 tubes of 1 ml.
3. Samples from distribution system –
   1 tube of 50 ml, 5 tubes of 10 ml.

The 50 ml tubes contain 50 ml of double strength Mac Conkey broth medium, 10 ml tubes 10 ml of the double strength medium while 1 ml tubes and tubes of dilutions of 1 ml contain 5 ml of single strength medium.

All the tubes showing acid and sufficient gas to fill the concavity at the top of the Durham tube are recorded as '*presumptive positive*'. The absence of formation of acid and gas at the end of 48 hours incubation constitutes a negative test. The most probable number (MPN) of coliform organism in 100 ml of sample is then determined from the probability tables.

(*ii*) **Confirmed test.** The results obtained in the above test are only presumptive evidence of the presence of *Bact. Coli* and are in fact only presumptive evidence of the presence of any member of the coliform group or coli-aerogenes group of bacteria. Presumptive positive tubes from any sample of chlorinated water should be confirmed for the actual presence of coliform organisms. Such confirmation may not be necessary in the case of unchlorinated samples. Confirmed test is carried out either using *solid medium* or a *liquid medium.*

In the confirmed test with *liquid medium*, the presumptive culture is plated out on MacConkey agar or Eosin-methylene blue agar of Endo agar and incubated at 37°C for 24 hours in order to ensure the growth of single, discrete and pure colonies. Coliform bacteria from deeply coloured nucleated colonies with or without metallic surface luster on Eosin methylene blue agar. On Endo agar, coliform agar, coliform colonies are nucleated dark red with or with or without metallic sheen. Atypical colonies are unnucleated opaque and pink in these media. The colonies of coliform bacteria are pink in colour in MacConkey agar medium. The appearance of typical colonies on the plates within the incubation period constitutes a positive confirmed test.

In the confirmed test with liquid medium, a small portion of lactose broth showing positive presumptive test is carefully transferred to another fermentation tube containing *brilliant green bile lactose broth.* The tube is incubated at 37°C for 48 hours. The formation of gas in any amount in the inverted tube of the fermentation tube constitutes a confirmed test for the coliform organism.

(*iii*) **Completed test.** This test is applied to a typical or atypical colonies found upon the plates of solid medium or to the brilliant green lactose bile broth fermentation tubes showing gas used for the confirmed test. In the latter case, the culture from the brilliant green lactose bile broth tubes is plated out on Endo or Eosin methylene blue plates, as soon as possible after the appearance of gas. The plate is incubated at 37°C for 24 hours. From each of these plates, or from each of the plates used for confirmed test using solid medium, pick up two or more typical coliform colonies and transfer each to a MacConkey broth tube and a nutrient agar slant. If no typical colonies are present, two or more colonies considered most likely to consist of the organism of the coliform group, are picked up for the transfer. The broth tubes are incubated at 37°C for 48 hours and examined for gas production. The tubes showing gas are recorded as positive. The agar slants are incubated at 37°C for 24 to 48 hours and examined for gram-stained preparations of the growths on slants corresponding to the broth tubes that shows gas. The formation of gas in the lactose broth tube, and the demonstration of Gram-negative, non-spore forming, rod shaped bacteria in the agar culture is considered a completed positive test.

(*b*) **Membrane filter technique.** Membrane filter technique is relatively a recent development of field measurement of bacteria. It was first developed in Germany during World War II and subsequently introduced in U.S.A. by Goetz and Tsuneishi in 1951. The method permits field filtration of the water sample and shipment of filters to the laboratory on a preservative medium. The method provides a means for improved control of water quality in water treatment plant and other laboratories since it has a higher degree of precision (reproductibility) than the multiple-tube fermentation test, and it permits the examination of much larger volumes of sample than the multiple-tube-test, with increased sensitivity in coliform detection. Also, definitive results are obtained in a shorter time than with the multiple tube method.

The method consists of filtering the sample of water through a filter membrane of uniform microscopic pores small enough to retain all the coliforms and other bacteria. The membrane filter paper used is 5 cm in diameter and of 150 micron thickness. It is made of biologically inert cellulose esters containing over 500 million capillary pores of uniform dimension. The filter holding assembly consists of a seamless funnel which fastens to a receptable bearing a porous plate for support of the filter membrane. Filter membranes are sterilized prior to use. Using sterile foreceps, the sterile filter membrane is placed over the porous plate of the unit. The filtration procedure consists of passing the measured quantity of water sample through the membrane under partial vacuum. After the sample has been filtered, the funnel is rinsed three times with 20-30 ml volumes of sterile buffered dilution water. The membrane is than lifted up with sterile foreceps, gently rolled, grid side up, onto the surface of an absorbent pad containing M-endo medium. The membrane cultures are incubated at 37°C for 20 hours, in an inverted position in the incubator with 100% humidity. The colony counting exhibit a metallic-appearing surface luster. The colony countig is done either with the help of a low power microscope or with the help of any suitable colony counter. The volume of water sample to be used in the test will be governed by the expected bacterial density. When less than 20 ml of water sample is to be used, it is diluted with sterile buffered dilution water to a minimum of 30 ml just before filtration. The estimated Coliform density of the sample is recorded in terms of coliform per 100 ml sample, using relation :

$$\text{Coliform colonies per 100 ml} = \frac{\text{Coliform colonies counted} \times 100}{\text{ml sample filtered}}$$

The method is not suitable for waters with turbidity due to algae or other material.

3. Test for E-coli

As stated earlier, *E-coli* is the most frequent type present in human and animal intestine. The presence of *E-coli* in a water supply therefore indicates pollution by faecal contamination. Due to this reason, *E-coli* is also sometimes used as the *indicator organism.* The method consists of transferring a small portion of the culture from each positive presumptive tube (coliform presumptive test) as soon as gas is noticed, to one tube of brilliant green bile lactose broth, and one tube of peptone water (preheated to 44°C) and incubating these for 6 to 24 hours in a water bath maintained at 44°C. After the incubation period, the broth tubes are examined for production of gas. Half the contents of the peptone culture is mixed to it gently. The immediate appearance of the peptone culture is transferred to a test tube, and 0.2 to 0.3 ml of Kovac's regent

TABLE 6.7. (CONTD.)

	I.C.M.R. Committee		*W.H.O. International*		*U.S.P.H.S*		*A.W.W.A.*	*Environmental Hygiene Committee*	
	(P)	*(E)*	*(P)*	*(E)*	*(P)*	*(E)*	*(P)*	*(P)*	*(E)*
Toxic									
Arsenic (as As) mg/l		0.2	—	0.2	0.01	0.05	—	—	0.2
Barium (as Ba) mg/l	—	—	—	—	—	1.0	—	—	—
Chromium (as hexa-valent) mg/l	—	0.5	—	0.05	—	0.2	—	—	0.01
Cynides mg/l	—	0.01	—	0.01	0.01	0.2	—	—	0.01
Lead (Pb) mg/l	—	0.1	—	0.1	—	0.05	—	—	0.1
Selenium (as Se) mg/l	—	0.05		0.05	—	0.01	—	—	0.05
Radio-activity									
Alfa Emmitter (μc/ml)	—	10^{-9}	—	10^{-9}	—	—	—	—	10^{-9}
Beta Emmitter (μc/ml)	—	10^{-8}	—	10^{-8}	—	—	—	—	10^{-8}
Radium^{226} (μμc/ml)	—	—	—	10	—	3	—	—	1
Strontium^{90} (μμc/ml)	—	—	—	30	—	10	—	—	—
Bacteriological Five 10 ml portions	MPN < 1 per 100 ml. No. sample should show an MPN of coliform bacteria in excess of 10 per 100 ml.		Not more than 10% of all portions examined shall show presence of coliform bacteria. MPN ≤ 1 per 100 ml. No two consecutive samples shall have the presence of coliform bacteria.		Not more than 10% of all samples shall show the presence of coliform. MPN not to exceed 1 per 100 ml.		Zero or less than 1.0 MPN per 100 ml. None of the samples should contain an MPN of coliform exceeding 10 per 100 ml	No coliform organisms.	

4. *Bacillary dysentery.* This is caused by bacteria of genus *shigella, Sh. dysentrial, Sh. flexneri, Sh. boydii* and *Sh. sonnei*. This disease is mostly contracted due to ingestion of food contaminated by flies or by unhygienic food handlers. It may sometimes be contracted by consuming water contaminated by human faeces.

(*b*) **Protozoal Diseases**

The two important diseases that come under this category are : *Amoebiasis and amoebic dysentery.* These are caused by the protozoon *entamoeba histolytica*. They live in the human large intestine forming cyst which are excreted in the bowel discharges of infected persons and which will live for long periods in water. Infection takes place due to ingestion of these cysts which are carried by water or flies or even human fingers.

(*c*) **Virus Diseases**

Any virus extracted and capable of causing infection when ingested could be transmitted by water. Those viruses that can multiply in the intestinal wall and that are discharged in large numbers in faeces are of more concern. The recognized viruses that are of serious concern are the following members of the *enteric virus* group : *polio viruses, coxsakie viruses, ECHO viruses, the viruses of infectious hepatitis,* the *adenoviruses* and the *reoviruses.* The poliovirus persists in the intestines of infected persons for a short time after infection which probably takes place from contaminated fingers directly or on food. *Poliomyelitis* is common where sanitation and food hygiene are poor. Delhi experienced the waterborne outbreak of *infectious hepatitis* in 1955-56.

(*d*) **Helminthic (worm) disease**

Worm infections are sporadic and occur only under grossly insanitary conditions or through gross mismanagement of sewage disposal system. The eggs and larvae of intestinal worms are discharged into faces. The common diseases that are spread are *schistosomiasis* and *swimmer's itch.* The causes of schistosomiasis is a group of trematode worms (flukes) that inhibit the viens of bladder or large intestines and discharge eggs into the urine or faeces.

6.11. STANDARDS OF PURIFIED WATER

Table 6.7 gives the physical, chemical and bacteriological standards for water quality, as suggested by the following agencies :

(*i*) Indian Council of Medical Research (I.C.M.R.) Committee.
(*ii*) World Health Organisation (W.H.O.) International.
(*iii*) United States Public Health Society (U.S.P.H.S).
(*iv*) American Water Works Association (A.W.W.A.).
(*v*) Environmental Hygiene Committee.

Recommendations of Ministy of Urban Development (MUD)

The manual on Water Supply and Treatment prepared by the Central Public Health and Environmental Engineering organisation, under the Ministry of Urban Development (MUD) India, lays down the following standards for water.

(*a*) Physical and chemical standards

The physical and chemical quality of water should not exceed the limits shown in Table 6.8.

TABLE 6.8
PHYSICAL AND CHEMICAL STANDARDS (MUD, INDIA)

S.N. (*1*)	*Characteristics* (*2*)		*Acceptable** (*3*)	*Cause for** rejection* (*4*)
1	Turbidity (units on J.T.U. scale)		2.5	10
2	Colour (units of platinum cobalt scale)		5.0	2.5
3	Taste and odour		unobjectionable	unobjectionable
4	pH		7.0 to 8.5	6.5 to 9.2
5	Total dissolved solids	(mg/l)	500	1500
6	Total hardness (as $CaCO_3$)	(mg/l)	200	600
7	Chlorides (as Cl)	(mg/l)	200	1000
8	Sulphates (as SO_4)	(mg/l)	200	400
9	Fluorides (as F)	(mg/l)	1.0	1.5
10	Nitrates (as NO_3)	(mg/l)	45	45
11	Calcium (as Ca)	(mg/l)	75	200
12	Magnesium (as Mg)	(mg/l)	$\ngtr 30$***	150
13	Iron (as Fe)	(mg/l)	0.1	1.0
14	Manganese (as Mn)	(mg/l)	0.05	0.5
15	Copper (as Cu)	(mg/l)	0.05	1.5
16	Zinc (as Zn)	(mg/l)	5.0	15.0
17	Phenolic compounds (as phenol)	(mg/l)	0.001	0.002
18	Anionic detergents (as MBAS)	(mg/l)	0.2	1.0
19	Mineral oil	(mg/l)	0.01	0.3

TABLE 6.9
GUIDELINE VALUES FOR BACTERIOLOGICAL QUALITY

Organism (1)	*Unit* (2)	*Guide line Value* (3)	*Remarks* (4)
(A) Piped water supplies			
1. Treated water entering the distribution system :			Turbidity < 1 NTU for disinfection with chlorine ; pH preferably 8.0 ; free chlorine residual 0.2–0.5 mg/l following 30 minutes (minimum) contact
(*i*) Faecal coliforms :	Number/100 ml	0	
(*ii*) Coliform organisms :	Number/100 ml	0	
2. Untreated water entering the distribution system :			
(*i*) Faecal coliforms	Number/100 ml	0	
(*ii*) Coliform organisms	Number/100 ml	0	In 98% of samples examined throughout the year–in the case of large supplies when sufficient samples are examined
(*iii*) Coliform organisms	Number/100 ml	3	In an occasional sample but not in consecutive sample.
3. Water in distribution system			
(*i*) Faecal coliforms	Number/100 ml	0	
(*ii*) Coliform organisms	Number/100 ml	0	In 95% of samples examined throughout the year–in the case of large supplies when sufficient samples are examined
(*iii*) Coliform organisms	Number/100 ml	3	In an occasional sample but not in consecutive sample
(B) Unpiped water supplies			
(*i*) Faecal coliform	Number/100 ml	0	
(*ii*) Coliform organisms	Number/100 ml	10	Should not occur repeatedly ; if occurence is frequent and if sanitary protection cannot be improved an alternative source should be found, if possible.
(C) Emergency water supplies			
(*i*) Faecal coliforms	Number/100 ml	0	
(*ii*) Coliform organisms	Number/100 ml	0	Advise public to boil water in case of failure to meet guide line values.

PROBLEMS

1. Write a note on common impurities found in water.

2. Describe in brief various tests conducted for physical examination of water.

3. Describe in brief various important tests conducted for chemical examination of water

4. (*a*) What do you understand by Coliform index ? How do you determine it.

(*i*) Write a note on significance of MPN in water quality.

5. What objections could be there if (*i*) turbidity (*ii*) chlorides (*iii*) nitrates, and (*iv*) hardness are present in excess in water.

6. Write a note on bacteriological analysis of water.

7. Write a note on aquatic micro-organisms.

8. Classify various types of bacteria. What do you understand by the coli-aerogeneous group of bacteria ? Explain its significance.

9. Describe the Agar plate count test.

10. Describe the multiple tube fermentation technique for the determination of coliforms.

11. What do you understand by membrane filter technique. Describe.

12. What do you understands by E-coli ? How do you determine its presence in water

13. Describe various 'nuisance bacteria'

14. Write a note on various water borne diseases.

15. Give in a tabular form the standards of drinking water as described by (*i*) A.C.M.R. (*ii*) W.H.O. (*iii*) A.W.W.A. (*iv*) Manual by M.U.D.

16. Write short notes on the following.
- (*i*) Hardness of water
- (*ii*) pH value and its determination.
- (*iii*) Coliform index.
- (*iv*) M.P.N.
- (*v*) Nitrogen and its compounds.
- (*vi*) E-coli.

17. Differentiate between the following :
- (*i*) Temporary and permanent hardness
- (*ii*) B-coli and E-coli.
- (*iii*) Virus and bacteria.
- (*iv*) Free ammonia and albuminoid ammonia.

7

Unit Operations

7.1. INTRODUCTION

The term '*unit operations*' first became popular in chemical engineering, in which any chemical process may be resolved into a coordinated series of the so called 'unit actions'. In chemical engineering, the term 'unit operation' is applied to procedures in which changes produced are essentially *physical*. The unit operation approach has been employed in chemical engineering education for years, with considerable success.

However, in water and waste water treatment, *unit operations are those procedures in which the changes produced are physical, chemical or biological*. The unit operation approach in water and waste water treatment has the following advantages.

1. It gives better understanding of the processes inherent in the treatment, and of the capabilities of these processes in attaining the objective.
2. It helps in the development of mathematical and physical modes of treatment mechanisms and the consequent design of treatment plants.
3. In helps in the coordination of effective treatment procedure to attain desired plant performance and effluent quality.

The analysis of the physical, chemical and biological unit operations is generally achieved by (*i*) development of mathematical models and (*ii*) construction of simple physical model to produce the desired reaction.

7.2. IMPORTANT UNIT OPERATIONS

The following are important unit operations employed in water and waste water treatment.

1. ***Gas Transfer*** : Aeration.
2. ***Ion Transfer*** :
   - (*a*) Chemical coagulation
   - (*b*) Chemical precipitation
   - (*c*) Ion exchange
   - (*d*) Adsorption.
3. ***Solute Stabilization***
   - (*a*) Chlorination
   - (*b*) Liming.
   - (*c*) Recarbonation
   - (*d*) Super-chlorination
4. ***Solids Transfer***
   - (*a*) Straining
   - (*b*) Sedimentation
   - (*c*) Floatation
   - (*d*) Filtration
5. ***Nutrient or molecular Transfer***
6. ***Interfacial contact***
7. ***Miscellaneous operations*** :
   - (*a*) Disinfection
   - (*b*) Copper sulfating
   - (*c*) Fluoridation
   - (*d*) Thermal desalination.
8. ***Solid concentration and stabilization***
   - (*a*) Thickening
   - (*b*) Centrifuging
   - (*c*) Chemical conditioning
   - (*d*) Elutriation

(*e*) Biological floatation
(*f*) Vacuum filtration
(*g*) Air drying
(*h*) Heat drying
(*i*) Sludge digestion
(*j*) Incineration
(*k*) Wet combustion.

The aims of some of these operations one described in subsequent articles.

7.3. GAS TRANSFER

In this unit operation, the gases are either released/desorbed from water or are dissolved/absorbed in water. This is achieved by exposing the water through spray or bubble aeration to either air or other atmospheres, under normal, increased or reduced pressures. This is an important unit operation which is carried out for water purification to achieve the following.

(*i*) Removal of objectionable gases such as carbon dioxide, hydrogen sulfide and other volatile odorous substances, by spray or bubble aeration.

(*ii*) Deferrization and/or demanganisation of water, through the addition of oxygen by spray or bubble aeration.

(*iii*) Addition of ozone from ozone generators or chlorine gas from chlorine dispensers, either for the disinfection of waters or for the destruction of odors and tastes in waters.

(*iv*) Addition of carbon dioxide from the gas or carbon dioxide generators to *recarbonate* lime softened water.

(*v*) Removal of corrosion-promoting oxygen as well as other gases (degasification) by spraying water into vacuum chamber at ordinary temperatures or at elevated temperatures.

The first two operations of gas transfer, mentioned above, are commonly called *aeration*. In most instances, the shared engineering objective of aeration is either the *removal* of gas and other volatile substances from water or their *addition* to water, or both at the same time. Commonly used aerators fall under four categeries :

(*i*) Gravity aerators
(*ii*) Spray aerators
(*iii*) Diffusers
and (*iv*) Mechanical aerators.

7.4. ION TRANSFER

In this unit operation, ion transfer takes place by means of (*i*) chemical coagulation, (*ii*) chemical precipitation, (*iii*) ion exchange, or (*iv*) adsorption.

Chemical coagulation is aimed at the removal of colloidal particles from water. For this purpose, floc forming chemicals, known as coagulants (such as aluminium or iron salts) are added to water which results in the increase in particle size. Organic clusters of different size, shape and weight are formed, and they settle more rapidly. The coagulants themselves are soluble, but they are precipitated by transfer of their ions to substances in the water.

While chemical coagulation removes the suspended impurities from water, *chemical precipitation* throws out of solution the dissolved substances through ion transfer. The chemicals added to achieve this purpose dissolve in water, and their released ions react with the ions in water, resulting in the precipitation of dissolved impurities. The common examples of chemical precipitation are (*i*) flocculation of iron from water, by the addition of lime, (*ii*) precipitation of carbonate hardness by addition of lime (*iii*) precipitation of non-carbonate hardness by addition of soda-ash and (*iv*) precipitation of fluoride ions from water by addition of tricalcium phosphate.

Ion exchange is the reversible interchange of ions between water and a solid ion-exchange media. *Water softening* by ion exchange is an important example. To be effective, solid ion exchange must (*i*) contain ions of their own, (*ii*) be insoluble in water and (*iii*) provide enough space in their porous structure for ions to pass freely in and out of the solid. The three main groups of ion exchange processes are (*i*) water softening, (*ii*) dealkalization, and (*iii*) demineralization. Softening serves to remove the salts of calcium and magnesium. Dealkalization consists of removal of the hydroxides, carbonates and bicarbonates of calcium, sodium and magnesium. Demineralization serves the total removal of all ions. In industry, both cation and anion exchangers are employed to prepare boiler feed water and deionize or demineralize process-waters.

Adsorption is the process of ion-transfer in which *interfacial forces* remove the ions and molecules (known as adsorbates) from solution (water) and concentrate at the interface of adsorbents. The use of granular carbon or powdered activated carbon applied ahead of water treatment plant clarifiers or filters, for the adsorption of odor-and taste-producing ions and molecules, is well known. Organic contaminants are removed from water by the process of adsorption which results from the attraction and accumulation of one substance on the surface of another. The chemical nature of the carbon surface is of relatively minor significance in the adsorption of organics from water and secondary to the magnitude of the surface area of carbon available. Hence high surface area is of prime importance in adsorption. Granular activated carbons typically have surface area of 500-1400 m^2/g. The major contribution to surface area is located in the pores of molecular dimensions.

7.5. SOLUTE STABILIZATION

In this operation, water is stabilized such that objectionable solutes are converted into unobjectionable forms without removal. The common forms of solute stabilization are :

(*i*) *Chlorination* : By chlorination, hydrogen sulfide in water is oxidised into sulfate.

(*ii*) *Liming* : By passing the water through chips of marble, lime stone or dolomites, carbon dioxide in excess of equilibrium requirements is converted into soluble bicarbonate.

(*iii*) *Recarbonation*. By recarbonation of water softened by excess-time treatment, excess lime is converted into bicarbonate.

(*iv*) *Superchlorination* : By addition of chlorine dioxide to water, odour-producing substances are oxidised.

7.6. SOLIDS TRANSFER

In this unit operation, solids are removed from water. Depending upon the size of the solids, this transfer can take place by straining, sedimentation, floatation and filtration– adopted in order of the decreasing size of the solids. *Straining* removes the floating and suspended solids (such as leaves, sticks, debris etc.) that are larger in size than the openings of screens and racks. Straining out of algae is accomplished by microscreens. *Sedimentation* permits the removal of suspended solids from water, in settling tanks in which the carrying and scouring power of flowing water are reduced until suspended particles settle by gravity. *Floatation* is obtained by addition to water of flotation agents that attach themselves to suspended solids or attach suspended solids to bubbles of air and lift the particles to the surface, which are then removed from the floatation devices. *Filtration* is one of the most important operation of solids transfer which combines straining, sedimentation and interfacial contact where-in the suspended solids or flocs are transferred onto grains of the filter media containing sand, coal or other granular materials.

7.7. WATER TREATMENT PROCESSES

Raw water may contain suspended, colloidal and dissolved impurities. The purpose of water treatment is to remove all those impurities which are objectionable either from taste and odour point of view or from public health point of view. Following are the purposes of water treatment :

(*i*) to remove colour, dissolved gas and murkiness of water.

(*ii*) to remove objectionable tastes and odour.

(*iii*) to remove the disease producing micro-organisms so that water is safe for drinking purposes.

(*iv*) to remove hardness of water.

(*v*) to make it suitable for a wide variety of industrial purposes such as steam generation, brewing, dyeing etc.

For surface waters, following are the treatment processes that are generally adopted.

1. **Screening.** This is adopted to remove all the floating matter from surface waters. It is generally provided at the intake point.

2. **Aeration.** This is adopted to remove objectionable tastes and odour and also to remove the dissolved gases such as carbon dioxide, hydrogen sulphide etc. The iron and manganese present in water are also oxidised to some extent. The detailed description of this process is given in chapter 8. This process is optional and is not adopted in cases where water does not contain objectionable taste and odour.

3. **Sedimentation with or without coagulants.** The purpose of sedimentation is to remove the suspended impurities. With the help of plain sedimentation, silt, sand etc. can be removed. However, with the help of sedimentation with coagulants, very fine suspended particles and some bacteria can be removed.

4. **Filtration.** The process of filtration forms the most important stage in the purification of water. Filtration removes very fine suspended impurities and colloidal impurities that may have escaped the sedimentation tanks. In addition to this, the micro-organisms present in water are largely removed.

5. **Disinfection.** It is carried out to eliminate or reduce to a safe minimum limit, the remaining micro-organisms, and to prevent the contamination of water during its transit from the treatment plant to the place of its consumption.

6. **Miscellaneous processes** : These include water softening, desalination, removal of iron, manganese and other harmful constituents.

Fig. 7.1 gives the schematic layout of a water treatment plant.

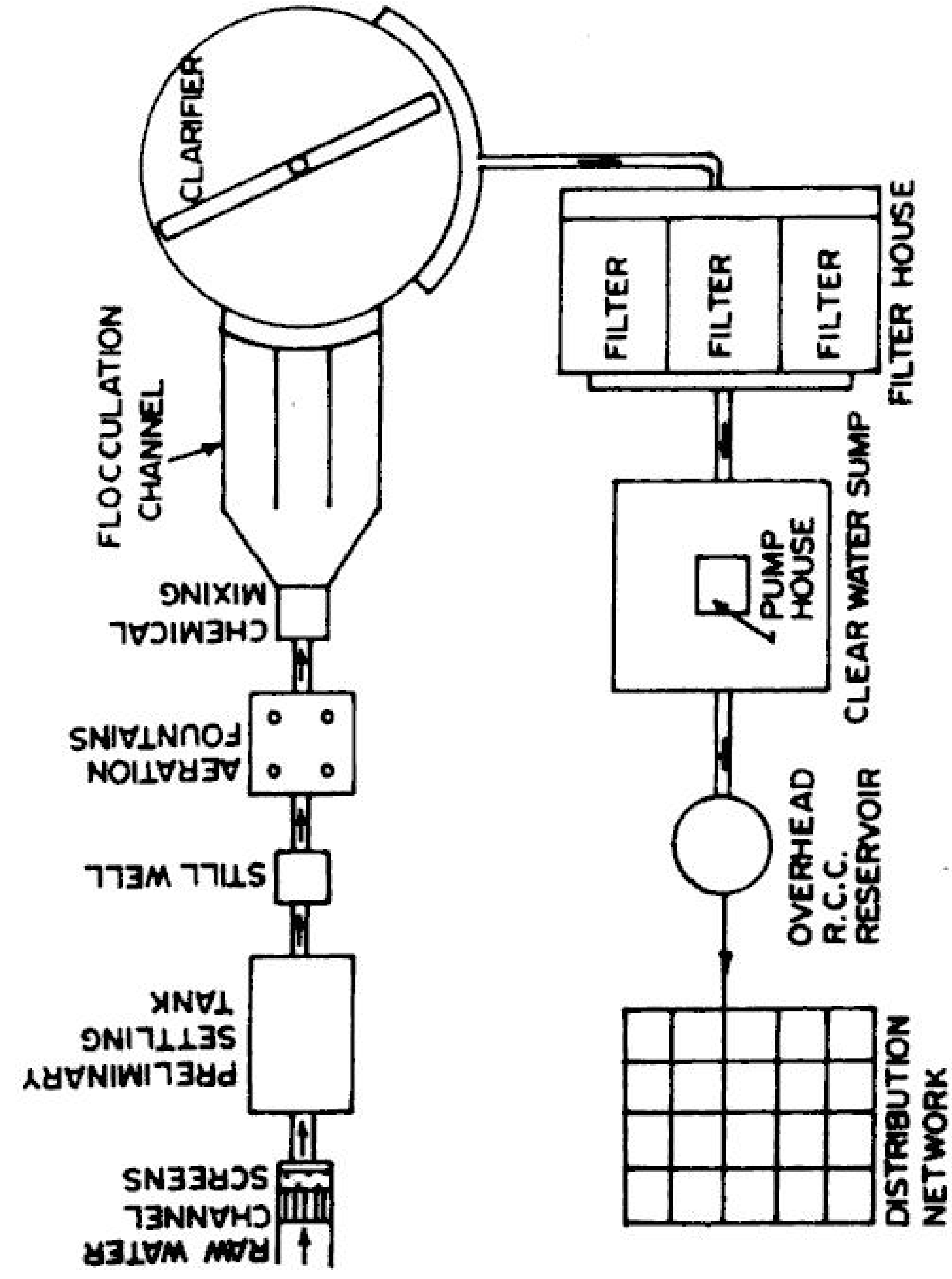


FIG. 7.1 SCHEMATIC LAYOUT OF WATER TREATMENT PLANT.

8

Screening and Aeration

8.1. SCREENING

Water, when derived from the surface sources, may contain suspended matter which may range from floating debris such as sticks, branches, leaves etc. to fine particles such as sand, silt etc., causing turbidity. Screens serve as a protective device for the remainder of the plant rather than as a treatment process.

Screens may be of two types :

(*i*) Coarse screens
and (*ii*) Fine screens.

8.2. COARSE SCREENS OR BAR SCREENS

Coarse screens or bar screens are intended to intercept only grosser floating material. They are mostly in the form of bar grill. The bars are generally of 25 mm size and are spaced at 75 to 100 mm centres. Mostly, bars are kept inclined so that they can be cleaned easily with a rake. Trash racks are often included in dams and other intake structures. For the purposes of cleaning, they are placed on a slope of 3 to 6 vertical to 1 horizontal. Fig. 8.1 shows three intake structures equipped with more or less self cleaning screens.

In Fig. 8.1 (*a*), the vertical screen is constructed of vertical wires or bars. The strained water enters the screen box, leaving behind leaves and other debris which drops below as sediment. In Fig. 8.1 (*b*), straining is achieved by upward flow, leaving behind debris or waste. The inclined screen can be lifted into vertical position (shown

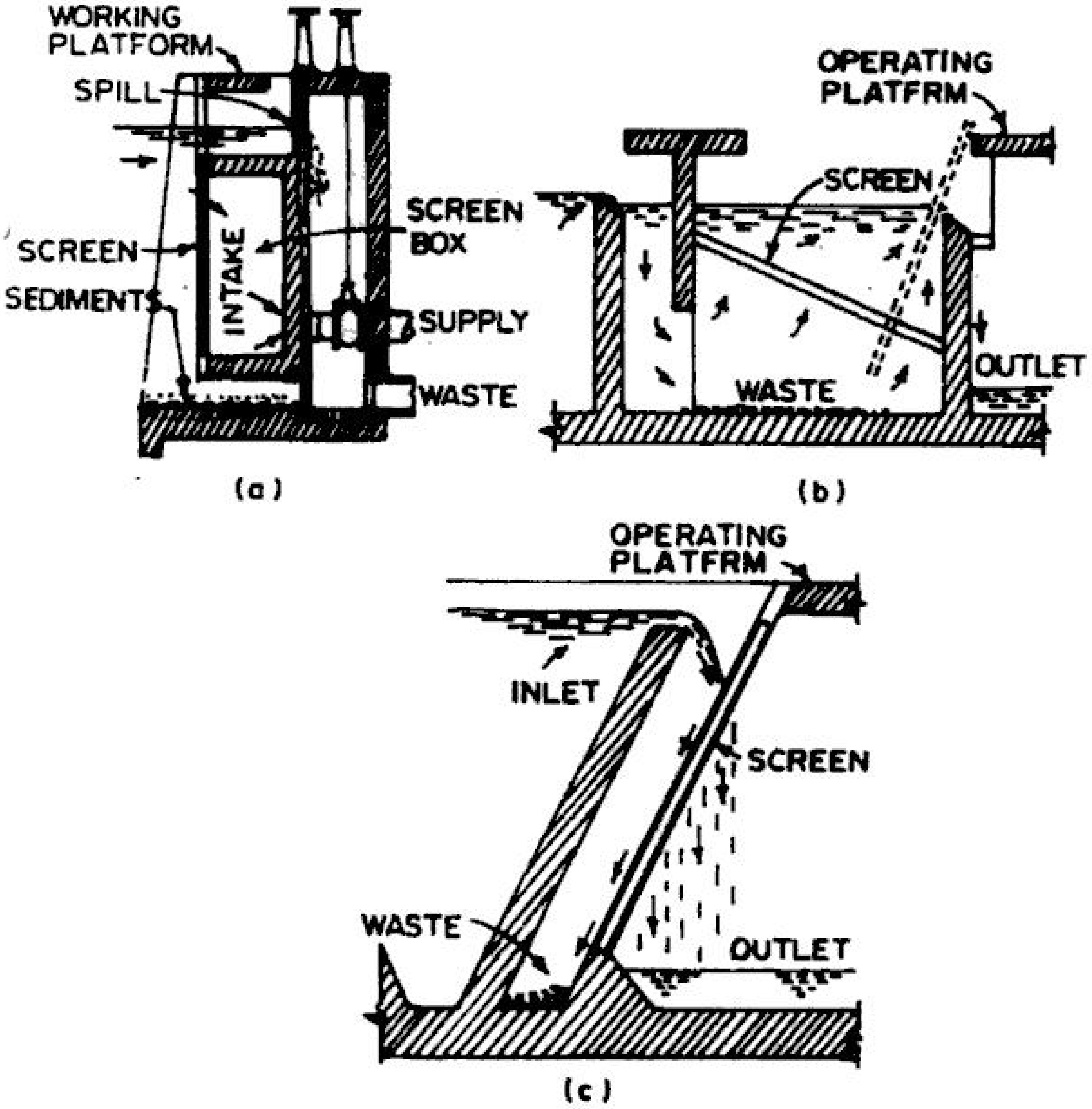


FIG. 8.1. RACKS AND SCREENS OF WATER INTAKES.

by dotted lines), for cleaning. Fig. 8.1 (*c*) shows a self flushing inclined screen. In most of the cases, the common arrangement is to slide a pair of removable screens into vertical grooves in the walls and bottom of the inlet channel.

Head loss through racks and screens

The head loss through unobstructed screens depends upon the nature of their construction (open area, blocked area, shape of screen elements etc.), as well as the approach velocity. Kirschmer (1926) has suggested the following empirical relationship and coefficients for racks with different shaped bears

$$h = \beta \, (w/b)^{4/3} \, h_v \, . \sin \theta \qquad \text{...(8.1)}$$

where h = head loss

h_v = approach velocity head,

b = minimum width of clear opening between pairs of bars

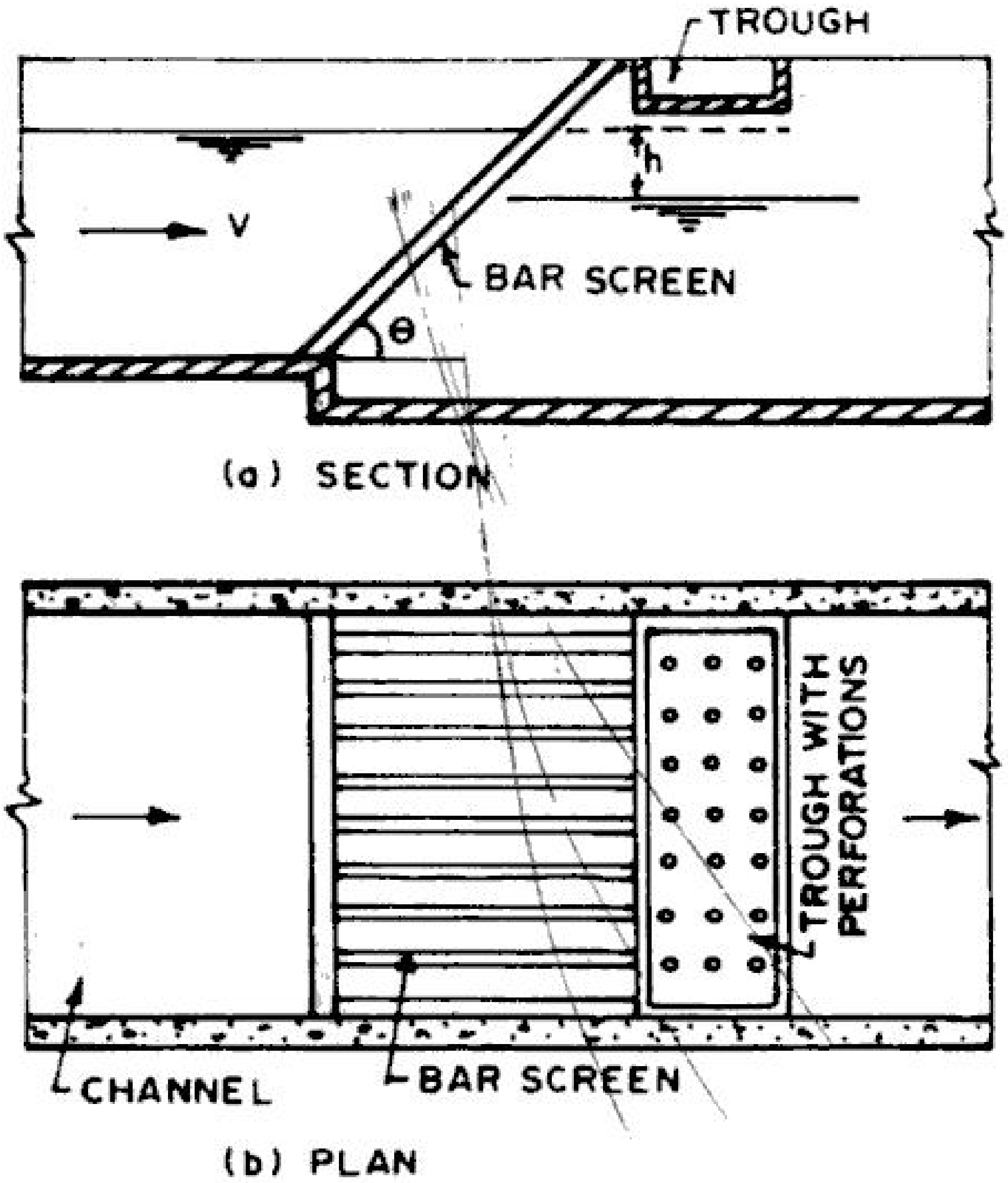


FIG. 8.2.

θ = angle of rack with horizontal

β = a shape factor

= 2.42 for sharp-edged rectangular bars

= 1.83 for rectangular bars with semi-circular upstream face

= 1.79 for circular rods.

Some manufacturers recommend dropping the channel 150 to 300 mm across a bar screen. The maximum head loss through clogged racks and screens is generally below 80 cm.

8.3. FINE SCREENS

Fine screens are used at surface water intakes, sometimes alone, some times following a bar screen. In order that fine screens do not get clogged up, some device is setup to clean it continuously. Due to this reason, fine screens are usually arranged as end less bands or drums of material perforated with holes of about 6 mm

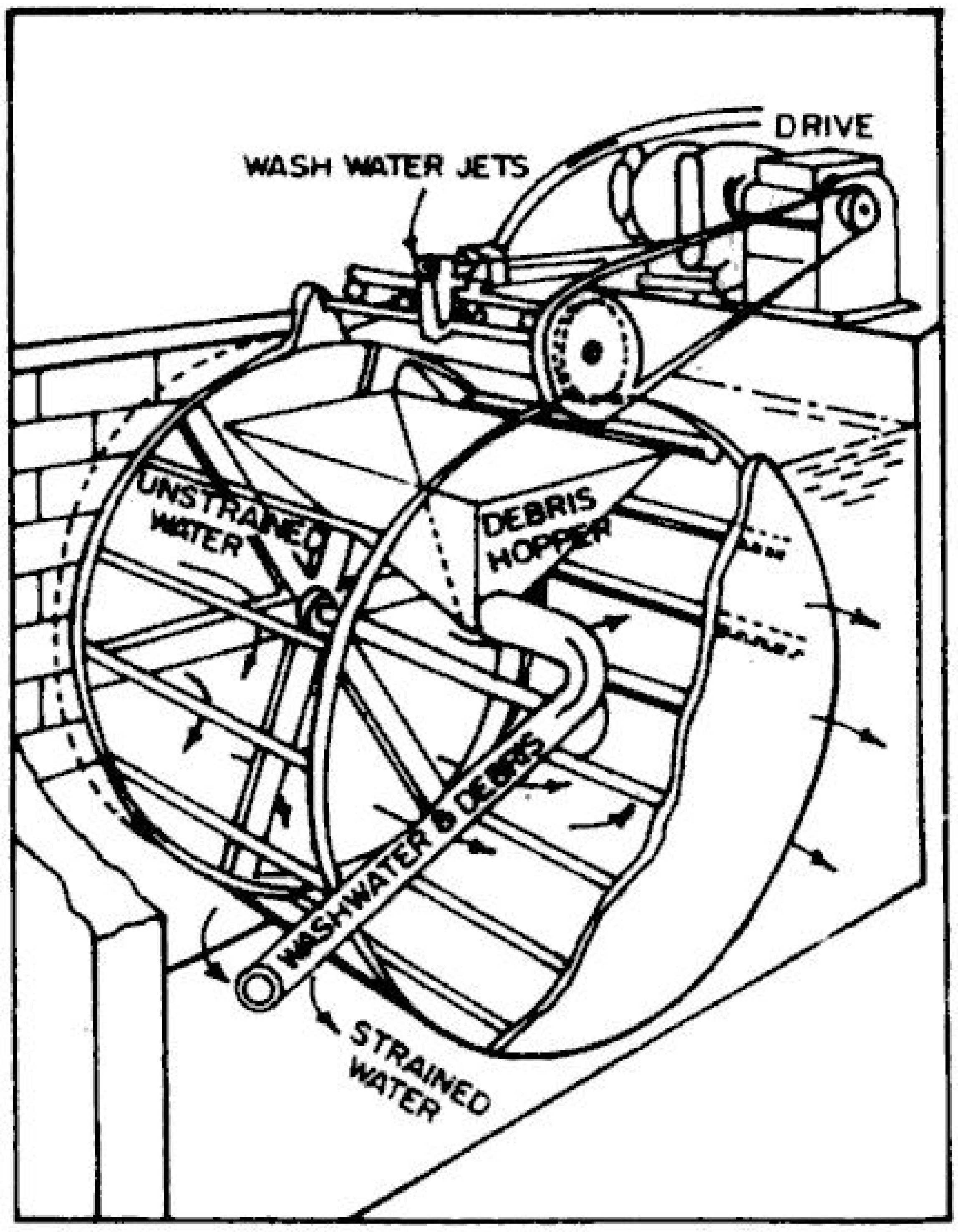


FIG. 8.3. ROTARY DRUM STRAINER.

diameter. The process is therefore known as *'automatic straining'*. Automatic strainers are self cleaning and they work continuously, eliminating solids from the liquid flowing through and disposing of these solids, also continuously. In automatic strainers, a pair of *straining fabric* is submerged in water to be strained while the remainder of the fabric remains above the water. Fig. 8.3 shows a *rotary drum strainer*, while Fig. 8.4 shows the *band screen strainer.*

8.4. MICRO-STRAINERS

A micro-strainer incorporates a specially woven stainless steel wire cloth mounted on the periphery of a revolving drum filled with continuous back washing arrangements. Two of the grades widely used have limiting apertures of 23 microns and 35 microns respectively. Micro-strainers are useful for screening stored waters which do not contain a large amount of suspended matter but which contain plankton, algae and other microscopic sized particles. They are installed to the upstream of rapid gravity or slow sand filters whose output may there by increased by as much as 50%. In operation, the revolving drum is kept submerged in the flowing water to approximately two thirds of its depth. Raw water from lake or reservoir

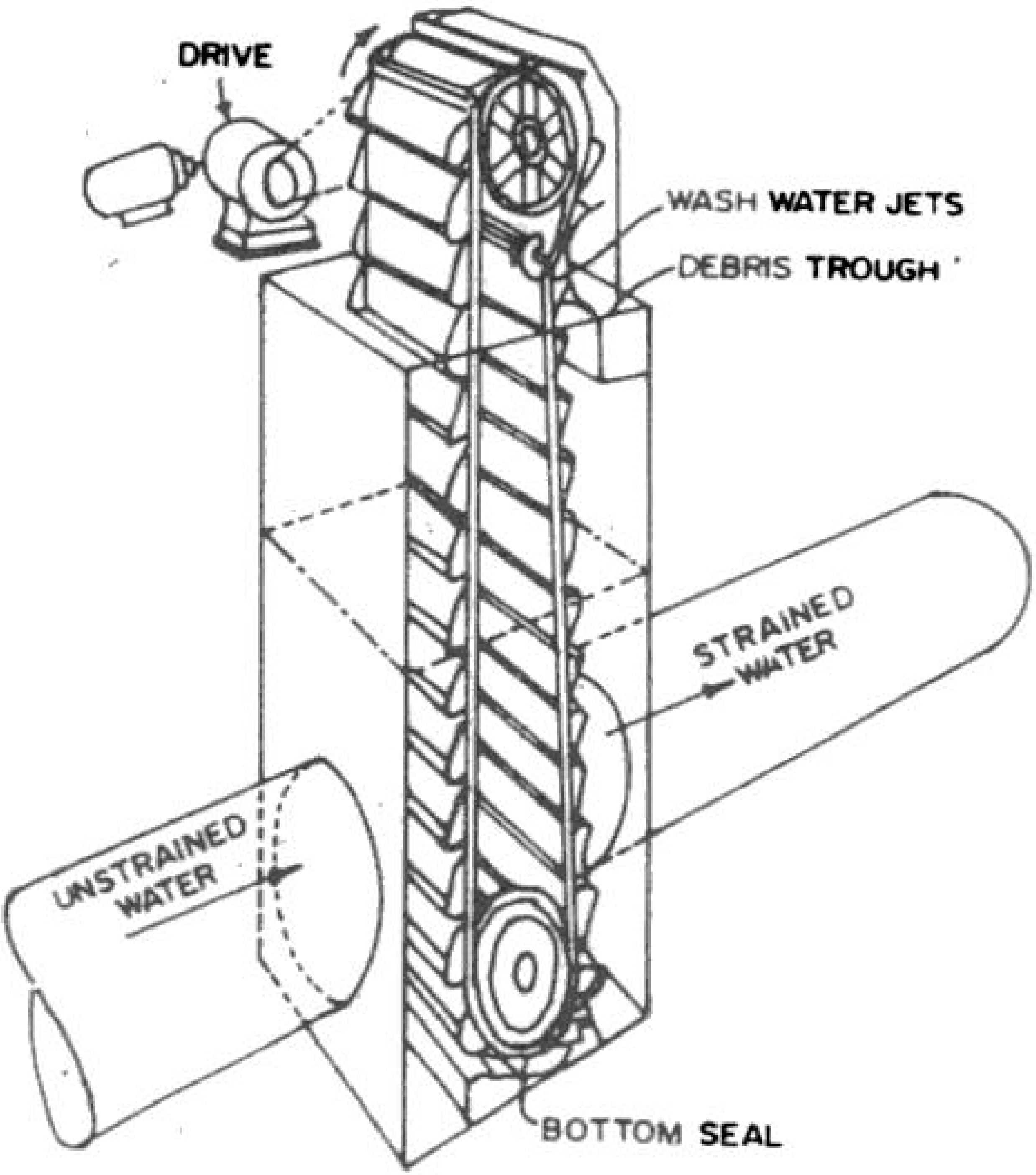


FIG. 8.4. BAND OR TRAVELLING SCREEN STRAINER.

enters through the upstream open end of the drum and flows radially outwards through the microfabric, leaving behind the suspended solid content. The solids so strained are carried upwards on the side of the fabric, with the help of a row of wash water jets which may use about 1% of total quantity of water strained. A single unit of 3 m diameter × 3 m wide drum may deal with about 50000 to 80000 litres per hour. It should be noted that microstraining cannot remove colour or finely divided matter such as clay.

8.5. AERATION

Aeration is one of the important unit operation of gas transfer. The aim of the aeration is to create extensive, new, and self-renewing interfaces between air and water, to keep interfacial films from building up in thickness

Objectives : Aeration of water is done to accomplish the following objectives.

1. It removes tastes and odours caused by gases due to organic decomposition.
2. It increases the dissolved oxygen content of the water.

3. It removes hydrogen sulphide, and hence odour due to this is also removed.
4. It decreases the carbon dioxide content of water, and thereby reduces its corrosiveness and raises its pH value.
5. It converts iron and manganese from their soluble states to their insoluble states, so that these can be precipitated and removed.
6. Due to agitation of water during aeration, bacteria may be killed to some extent.
7. It is also used for mixing chemicals with water, as in the Aeromix process and in the use of diffused compressed air.

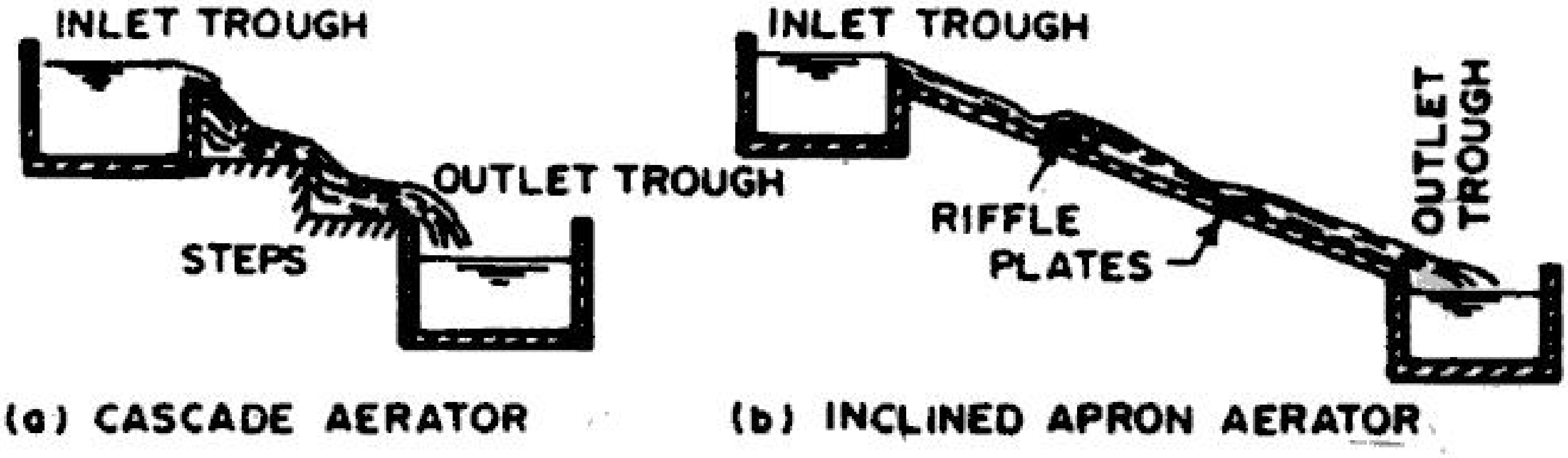
8.6. TYPES OF AERATORS

Aeration is done by the following main types of aerators :

(*a*) Free fall aerators or gravity aerators
  (*i*) Cascade aerators
  (*ii*) Inclined apron aerators
  (*iii*) Slat tray aerators
  (*iv*) Gravel bed aerators (trickling beds)
(*b*) Spray aerators
(*c*) Air diffuser basins.

1. **Cascade aerators.** Cascade aerators are the simplest of the free fall aerator. Weirs and waterfalls of any kind are cascade aerators (Fig. 8.5 *a*). A simple cascade consists of a series of three or four steps – of concrete or metal. Water is allowed to fall through a height of 1 to 3 metres, and due to this it comes into close contact with air. The cascades can be either in open air, or may be in a room which has plenty of louvred air inlet. The reduction of CO_2 is usually in the range of 50 to 60%.

2. **Inclined apron aerator with riffle plates :** In this type of aerator, water is allowed to fall along an inclined plane/apron



(a) CASCADE AERATOR (b) INCLINED APRON AERATOR

FIG. 8.5. GRAVITY AERATORS.

which is usually studded with riffle plates in herring bone fashion. The breaking-up to the sheet of water will cause agitation of water and consequent aeration (Fig. 8.5 *b*)

3. **Slat tray aerators.** This is most commonly used. It consists of a closed round or square structure containing a series of closely-stacked superimposed wood-slat trays (Fig. 8.6). Water enters the top of the aerator and is evenly distributed over the topmost tray. The slats in the trays are staggered so that the films of water raining over the edges of the slat in one tray fall on the centres of the slats in the tray just below. Air is supplied to the bottom of the aerator with the help of a blower, which blows it upward. A ventilator is provided at the top, which discharges air and gases to the atmosphere. Water is collected in the collector pan at the bottom, from where it flows to a catch basin or reservoir.

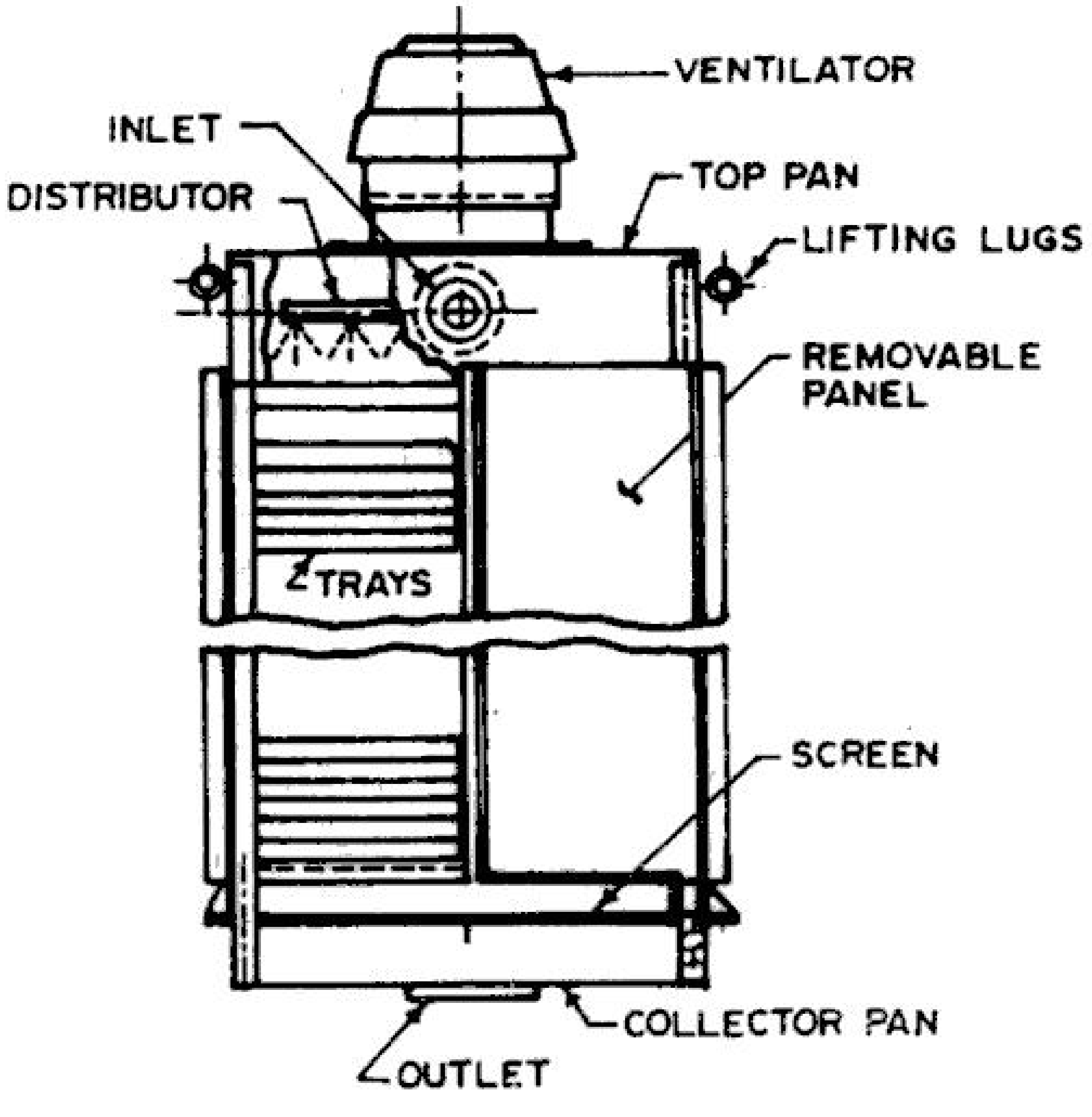


FIG. 8.6. SLAT TRAY AERATOR.

4. **Gravel bed aerators (Trickling beds).** Cascading through beds of coke, lime-stone or anthracite is believed to have more efficient CO_2 removal than other methods. Fig. 8.7 shows a typical gravel bed aerator in which water is applied at the top and trickles

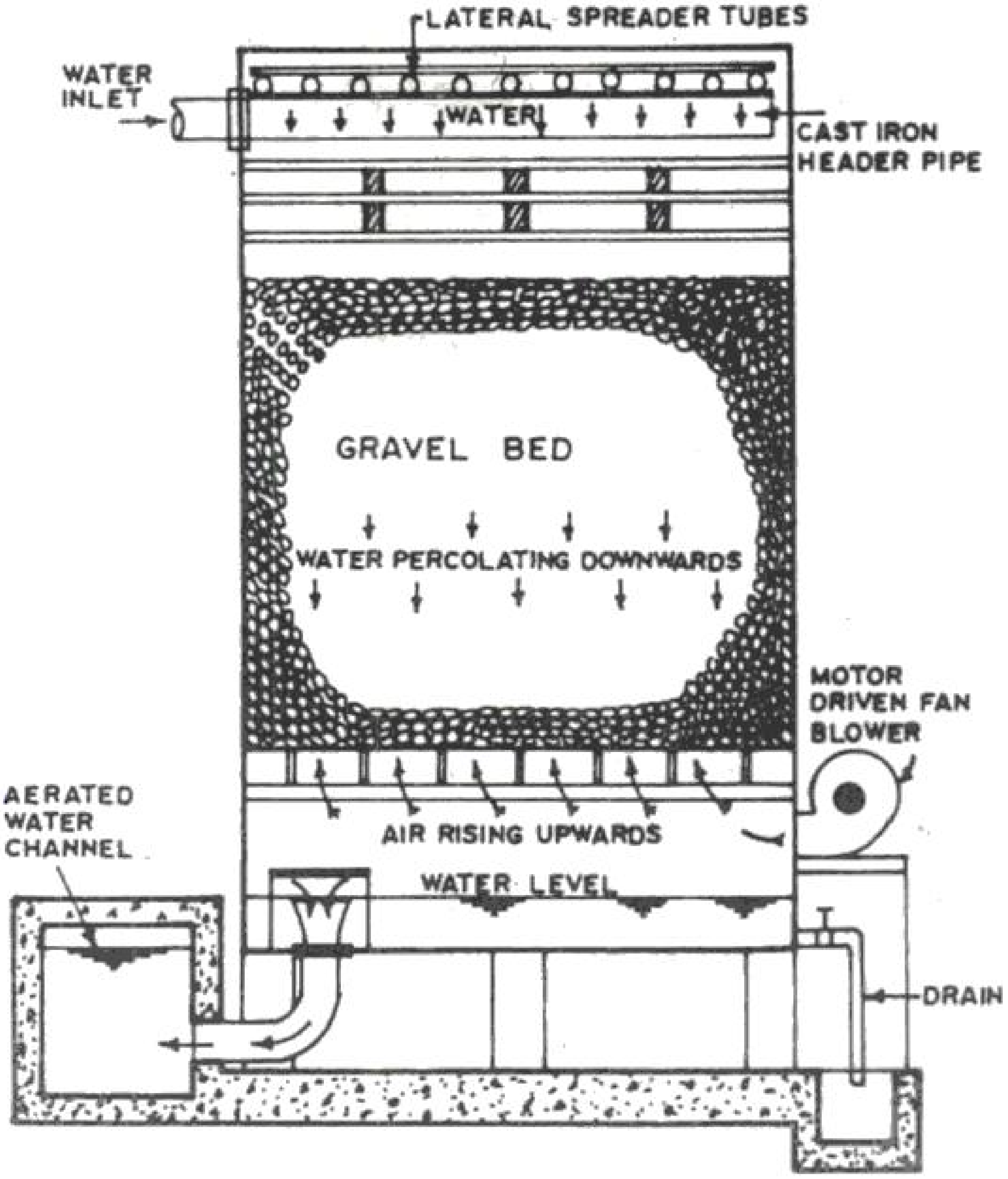


FIG. 8.7. GRAVEL BED AERATOR

down while air is blown upwards. The thickness of gravel bed may be from 1 to 1.5 m.

In another form, commonly known as *trickling beds,* three or four trays filled with coke, slag or stone are used. The thickness of bed in each tray is kept about 0.5 to 0.6 m, while the vertical distance between the bed is kept about 0.5 m. Water is applied from top through a perforated distribution pipe as shown in Fig. 8.8. During the trickling process, aeration takes place.

4. Spray Aerators. Spray aerators divide the water flow into fine streams and small droplets which come into intimate contact with the air in their trajectory. Water is sprinkled in fine jets through nozzles. It requires considerable head (0.75 to 1.5 kg/cm^2), but it reduces carbon dioxide by 70 to 90% or more.

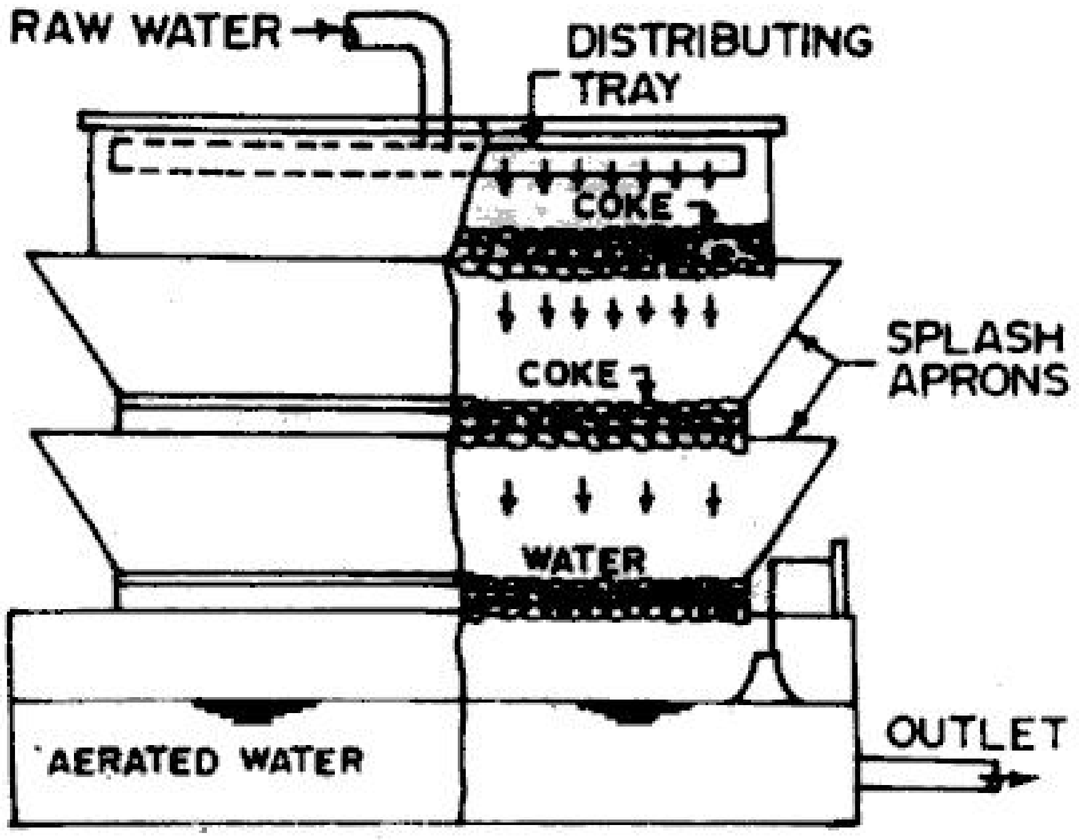


FIG. 8.8. TRICKLING BED AERATOR

5. Air Diffusion. In this method, perforated pipe network is installed at the bottom of the aeration tank, and compressed air is blown through these pipes. The air bubbles travel upward through water, thus causing aeration. Air diffuser basins have a retention period of about 15 minutes and a depth of 3 to 5 metres.

Alternatively, compressed air may be injected into the flow of water in a pipe, or air at atmospheric pressure may be drawn in the pipe where a constriction, such as the throat of a venturi tube reduces the water pressure below atmospheric. However, aeration under pressure does not remove CO_2.

8.7. FACTORS GOVERNING AERATION OR GAS TRANSFER

(*a*) Rate of gas absorption

If it is postulated that the rate of gas absorption is proporticnal to the degree of *under saturation* (or *saturation deficit*) in the absorbing liquid, we have

$$\frac{dC}{dt} = K_g (C_s - C_t) \qquad ...(8.2)$$

where $\frac{dC}{dt}$ = change in concentration, or rate of absorption, transport or transfer at time t.

C_s = saturation concentration of gas in water, at a given temperature (*i.e.* maximum concentration in absorption).

C_t = concentration at time t.

K_g = proportionality factor for existing condition of exposure.

Integrating between the limits C_0 at $t = 0$ to C_t at $t = t$, we get the basic equation.

$$C_t - C_0 = (C_s - C_0)\,[1 - \exp(-K_g t)] \qquad \text{...(8.3)}$$

In Eq. 8.3, the proportionality factor K_g increases with temperature and the *degree of mixing* of the gas with liquid. Since the molecules of gas must pass through the gas-liquid interface, K_g is a function of A/V ratio.

$$\therefore \qquad K_g = k_g \frac{A}{V} \qquad \text{...(8.4)}$$

where k_g = *gas transfer coefficient*, having dimension of velocity.

A/V = area of interface per unit volume of liquid

A = largest practicable area of interface between a given water volume V and air (or gas).

For absorption $C_0 < C_t < C_s$ and both $(C_t - C_0)$ and $(C_s - C_0)$ are positive

For desorption, $C_s < C_t < C_0$ and both $(C_t - C_0)$ and $(C_s - C_0)$ are negative.

The value of $K_g\,(= k_g A/V)$ must normally be determined experimentally and verified in plant-scale tests.

(*b*) **Rate of gas desorption**

Eq. 8.3 derived for rates of absorption applies also to the rate of desorption. In contrast to absorption, the rate of desorption, precipitation, release, or dissolution of a gas from a liquid becomes proportional to its degree of *oversaturation* in the liquid, or the *saturation surplus.*

(*c*) **Factors governing aeration or gas transfer.**

From Eq. 8.3, we have

$$C_t = C_0 + (C_s - C_0)\,[1 - \exp\{-k_g (A/V)\,t\}] \qquad \text{...(8.3 a)}$$

From this, we observe that gas transfer can be optimised by:

(*i*) Generating the largest practicable area A of interface between given volume (V) of water, and air or gas

(*ii*) Preventing the build-up of thick interfacial film, thereby keeping the value of k_g higher.

(*iii*) Inducing longer time (t) of exposure

(*iv*) Ventilating the aerator and its components.

Hence to ensure proper aeration it is necessary :

(*a*) to increase the area of water in contact with air *i.e.* if the water is sprayed, the smaller the droplets produced, the greater will be the area available. Similarly, if the water is being made to fall as a film over packing material in a tower, the smaller the size of the packing material, the greater will be the area available ;

(*b*) to keep the surface of the liquid constantly agitated so as to reduce the thickness of the liquid film which would govern the resistance offered to the rate of exchange of the gas ; and

(*c*) to increase the time of contact of water droplets with air or increase the time of flow which can be achieved by increasing the height of jet in spray aerator and increasing the height of tower in the case of packed madia.

Introducing factor $f = \left(\frac{A}{V}\right) t$, let us compare the efficiencies of spherical bubbles of air, rising up through water, with that of spherical droplets of water falling under gravity.

Spherical droplets of water

In the case of spray aerators, let the spherical droplets of water fall down after attaining a height *h*. Then, we have

$$h = \frac{1}{2} g t^2$$

$$\therefore \quad t = \sqrt{\frac{2h}{g}}$$

$$\therefore \quad \text{Factor } f_s = \left[\left(\frac{A}{V}\right) t\right]_{\text{spherical}} = \frac{6}{d}\sqrt{\frac{2h}{g}} \qquad \text{...(8.5)}$$

(since ratio *A*/*V* for a sphere is equal to 6/*d* where *d* is the dia. of spherical droplets).

Spherical bubbles of air

Let the bubbles rise throgh water column of height *h*, with a velocity *v*.

$$\therefore \quad \text{Time taken, } t = \frac{h}{v}$$

$$\therefore \quad \text{Factor } f_b = \left[\left(\frac{A}{V}\right) t\right]_{\text{bubble}} = \frac{6}{d} \cdot \frac{h}{v} \qquad \text{...(8.6)}$$

Hence, ratio $$\frac{f_b}{f_d} = \left(\frac{6}{d}\frac{h}{v}\right) \div \left(\frac{6}{d}\sqrt{\frac{2h}{g}}\right)$$

1. Orifice and nozzle behaviour

Spray aerators are normally composed of perforated or nozzle pipes which create a spray pattern. The *initial spray velocity* (v) is given by

$$v = C_v \sqrt{2gh} \qquad \text{...}(i) \quad \text{...}(8.10)$$

where C_v = velocity coefficient (≈ 0.95)

h = orifice head or driving head.

For a pipe having multiple openings,

$$\therefore \qquad Q = C\,(\Sigma a)\sqrt{2gh} \qquad \text{...}(ii) \quad \text{...}(8.11)$$

where Q = rate of discharge.

C = discharge coefficient

= 0.8 for rounded openings

= 0.85 to 0.92 for nozzles

= 0.6 for sharp edged openings

Σa = total area of openings

If there are n openings, each of equal area a,

$$\Sigma a = na \qquad \text{...}(iii)$$

The water rises either vertically or at an angle (α) and falls onto a collecting apron, after moving along a trajectory.

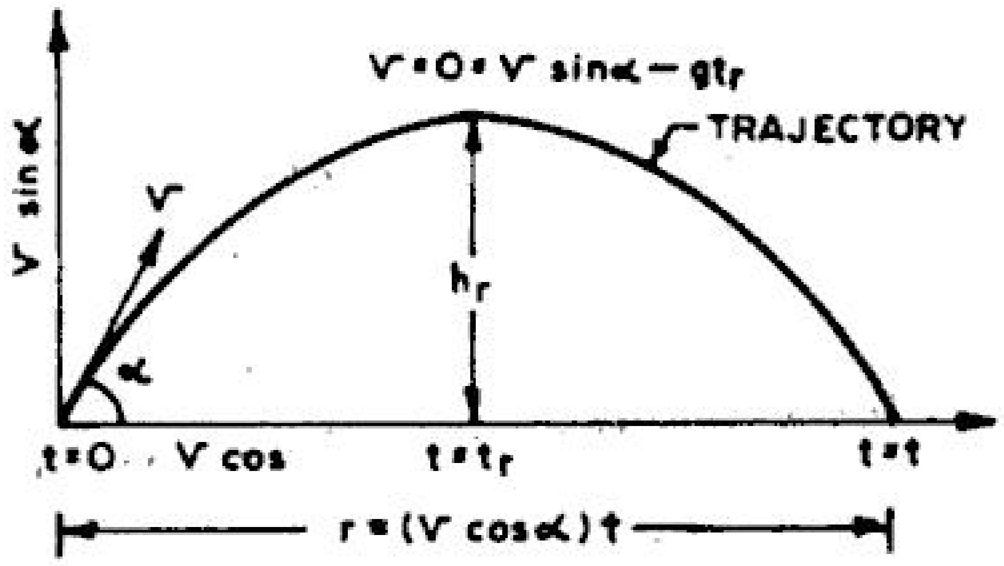


FIG. 8.9.

Let h = driving head

t_r = time of rise of spray

t = total time of exposure $= 2\,t_r$

α = inclination of jet.

At the time of rise t_r, $v = 0$

$$\therefore \qquad v = 0 = v \sin\alpha - g\,t_r$$

In the absence of any control device built into the system, it is reasonable to assume that flow at the lower end of the launder will be at its critical depth. Then

$$h = \left[\frac{Q^2}{b^2 g} \right]^{1/3} \qquad \text{...(9.18 } b\text{)}$$

where Q is the total discharge in the launder $= qL$

Sometimes, multiple troughs or launders are preferred to provide uniform distribution at low flows. There is a growing trend towards the use of effluent launders or troughs covering a good part of the surface of the settling basins. These are spaced at a distance of one tank depth between the troughs. The use of maximum feasible weir length in the tank from the outlet towards the inlets assists greatly in controlling density currents. Normal weir loadings are up to 300 m^3/day/m length.

5. Sludge removal. The particles settled in the basin constitute the sludge which can be removed either manually or mechanically. In the former case, the basin has to be put out of service, drained and the sludge dug out from the bottom. This method may be satisfactory where the quantity of suspended matter is small, requiring sludge clearance once in 2 or 4 months. However, where the quantity is large, the frequency of clearance will be very often, and it will not be economical to remove the sludge through manual labour. In addition to this, if the impurities settled are organic in nature, likely to decompose if allowed to accumulate for longer time, the resulting oxygen demand may cause septic conditions and produce objectionable odours. Hence mechanical or hydraulic methods are used for sludge removal. The sludge is collected and removed from the tanks as a fluid and caused to flow to its final point of disposal before being de-watered. Generally, travelling or rotating scrapers are installed to maintain the fluidity, and to transport the sludge to one or more outlet points from where it is sucked out through sludge pipe connected to sludge pump, as shown in Figs. 9.10 and 9.11.

9.9. SETTLING TANK EFFICIENCY

The hydraulic efficiency of the settling tank is the *flow through period* (t_d) which can be expected to approach the *critical detention period* (t_0) only in ideal tanks. Settling tank efficiency is reduced by the following currents :

(*i*) *Eddy currents*, set up by the inertia of the in-coming fluid

(*ii*) *Surface currents*, induced due to wind in open tanks

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