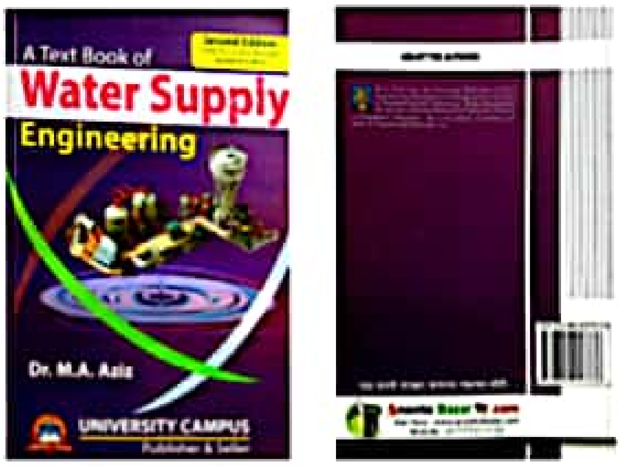
A Text book

Of

Water Supply Engineering



Dr. M.A. Aziz

B.Sc Engg.(Civil), M. Engg. (U.S.A). Ph. D. (U.K),

Professor of Civil Engineering Bangladesh University

Of Engineering and Technology Dhaka, Bangladesh

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MOHAMMAD IMRAN NEWAZ (SIFAT)

A Text book of Water Supply Engineering

Dr. M.A. Aziz
B. Sc. Engg. (Civil), M. Engg. (U.S.A.), Ph. D. (U.K.),
Professor of Civil Engineering Bangladesh University of Engineering and Technology Dhaka, Bangladesh.

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PREFACE

This book is an outgrowth of certain lectures and notes which have been used in the author's classes in the Department of Civil Engineering, Bangladesh University of Engineering and Technology, Dhaka, for the past several years. Its preparation was undertaken to meet the need in engineering education and profession which was felt for a general textbook covering the fundamentals of water supply engineering in a comparatively concise and thoroughly modern manner.

This book on water supply engineering is designed for use as a textbook to the students pursuing engineering and technical education in Engineering Universities, Engineering Colleges, Polytechnic Institutes, and other Technical Institutions. This book is also a unique offer to the practising engineers, architects, contractors and to those preparing themselves for AMIE Examinations,

The discussions of various essential elements of a water supply system follow a general systematic pattern which aims at presenting in a simple, concise and lucid form the best of the most up to-date informations regarding water supply engineering. Numerous illustrative diagrams, worked-out examples, tables, and monographs have been presented in this book to enable the readers to get insight into the subject-matter easily requiring no previous knowledge in this field.

The author cannot make a pretence of being a specialist in all of the fields which are covered in the various chapters of this book. This work is therefore to a large degree a compilation of data and opinion from many different sources.

The author takes pleasure in acknowledging; his great indebtedness to the large number of engineers, architects, and Govt. officials who have privately or by their writings contributed much to make up this volume, A large number of technical books and journals have been frequently consulted and freely used in preparing this text. An effort has been made to always acknowledge the source of information so obtained and if any error of omission has been commited in this respect, it has been committed inadvertently, not by intention. Care has been taken to give accurate information in textbook, however, in a subject so varied as water supply engineering, mistakes may occur. I will appreciate having called to my attention.

I express my heartiest thanks, to Dr. S. H. K. Eusufza Professor and Head, Civil Engineering Department, Bangladesh University of Engineering and Technology, Dhaka, who inspired me in many ways to write this book.

I wish to acknowledge with thanks the valuable encourage¬ment and assistance from my respected teachers, colleagues, friends? and students. Special acknowledgement is made to Mr. M. Firoze Ahmed, Assistant Professor of Civil Engineering, BUET, Dhaka, for his interest, encouragement and valuable assistance in proof ¬reading and in the final preparation of this book.

I wish to express my gratitude to my wife, Saleha, for her unfailing inspiration, support and help during writing this book. Sincere appreciation is also extended to my family for allowing me to take so much of their time to write this book.

Bangladesh University of Engineering
and Technology
Dhaka, Bangladesh.
November, 1975.

M. A. Aziz

Dedicated to the sweet memory
of my beloved parents

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I
INTRODUCTION

1.1 General Considerations :

Water is absolutely essential to all life, both animal and plant. In order to survive, all animals and plants must have ample supply of water. A water supply which is differ for human consumption is essential to all life.

It is difficult to imagine any clean and sanitary environment without water. Invariably, the progress of sanitation throughout the world has been closely associated with the availability of water ; and, the larger the quantity and the better the quality of the water, the more rapid and extensive has been the advance of public health. The history of public health is filled with both tragic and glorious milestones in which water was the important factor.

Man uses water not only for drinking and curlinary purposes, but also for bathing, washing, laundering, heating and air-conditioning, for agricultures, stock raising and gardens, for industrial processes and cooling, for water power and steam power, for fire protection, for disposal of wastes, for fishing, swimming, boating and other recreational purposes, for fish and wild life propagation, for navigation and for engineering constructions. Therefore, every activity of man involves some use of water. Yet man's assessment of the value of water is very low until he finds himself without it.

While man has always recognized the importance of water for his internal bodily needs, his recognition of its importance to health is a more recent development, dating back only a century or so. Since that time, much has been learned about the role of inadequate and contaminated water supplies in the spread of water-borne diseases. Among the first diseases recognized to be water-borne were cholera and typhoid fever. Later, dysentery, gastro-enteritis and other diarrhoeal diseases were added to the list. More recently, water has also been shown to play an important role in the spread of certain virus diseases such as infectious hepatitis (Jaundice).

Water is involved in the spread of communicable diseases in essentially two way. The first is the well known direct ingestion of the infectious agent when drinking contaminated water (e.g. dysentery, typhoid and other gastro-intestinal diseases). The second is due to a lack of sufficient water for personal hygiene purposes. Inadequate quantities of water for the maintenance of personal hygiene and environmental sanitation have been shown to be major contributing factors in the spread of epidemic diseases. Adequate supplies of water for personal hygiene also diminish the probability of transmitting some of the gastro-intestinal diseases mentioned above. The latter type of interaction between water and the spread of diseases has been recogeized by various public health organizations in developing countries which have been trying to provide adequate quantities of water of reasonable, though not entirely satisfactory, quality.

Health problems related to the inadequacy of water supplies are universal but, generally, of greater magnitude and significance in the underdeveloped and developing nations. It has been estimated that about two-thirds of the population of the developing countries obtain their water from contaminated sources. The World Health Organization (WHO) estimates that each year 500 million people suffer from diseases associated with unsafe water supplies. Due largely to poor water supplies, an estimated 5,000,000 infants die each year from diarrhoeal diseases.

In addition to the human consumption and health requirements, water is also needed for agricultural, industrial and other purposes. Though all of these needs are important, water for human consumption and sanitation is considered to be of greater social and economic importance since the health of the people influences all other activities.

1.2 Engineering Aspects of Water Supply :

The planning, design, construction, supervision and maintenance of water supply systems to supply potable water to communities have long been the responsibility of Civil Engineers in every country of the world. Civil Engineers with an understanding of water quality management and sewage treatment and disposal, are known as Sanitary Engineers, but they are also called Public Health Engineers or Environmental Engineers.

The important role that the Public Health Engineers have to play is to provide communities with adequate potable water supplies, facilities for sewage and refuse collection, treatment and disposal, safe recreational areas, and a healthy environment within homes and the places of employment. Many other factors concerned with aesthetics, economic, recreation, and other elements of better living are important consideration and have become part of the responsibilities of the modern Public Health Engineers.

Even in its most specific sense in an engineering enterprise, satisfactory development of water supply and waste-water schemes depends upon hydrologic, and geologic information that can be made available only by decades of institutionalization, orderly observations, recording, and analysis, and also upon the sound knowledge of hydraulics, structural engineering chemistry and microbiology. The planning, design, construction and operation of modern urban water supply and wastewater systems are complex undertakings. The entire works must legally, hygienically, aesthetically and economically defined. Since science, engineering and technology have advanced very much there must be support from teaching research and professional institutions deeply concerned with the advancements of the underlying sciences, engineering and technologies of water supply for the successful planning, design, construction, and operation of modern water supply systems.

1.3 History and Development of Water Supplies:

Water supply has its history, its archeology, its literature, its science, its engineering and technology. There was no truer sign of civilization and culture without good water supply. A knowledge of history and development of water supplies is highly desirable to emphasize changes in practice. The story of water supply begins with the growth of ancient capital cities, or religious or trade centres. Constructed as works of considerable magnitude and complexity, their remnants are monuments to sound, yet daring feats of early Civil Engineers. The relatively recent development of present day water supply systems is as ancient as the history of man. Water-works structures are found in excavation of prehistoric ruins. The remains of Lake Moeris in Egypt indicates its construction about 2000 B.C. It was the largest of the reservoirs of the Nile Valley which is believed to supply water for 20,000,000 people.

The water supply of towns in very early times was derived from large tanks excavated on minor drainage lines which collected and scored the rainfall in the wet season to provide a supply during the dry periods. Especially notable are the structures of water supply, drainage, sewerage and swimming pool of Mohenjodaro civilization in the Indus Valley. In Egypt, Babilonia, and Assyria flat countries traversed by rivers subject to floods, water was supplied by means of open canals with large storage basins. Wells were also used in many countries in ancient times to utilize the underground waters which were collected from them by simple mechanical devices, still to be seen in Egypt and India. Wells are also known to have bean used at remote periods in ancient Greece and Italy and artesian wells were sunk in China in very early times. Ancient China possesses the deepest well in the world as it was knwon to be 1500 ft deep. In ancient Greece and Egypt, deep wells as deep as 300 ft below ground level and wells ranging in depth from 100 to 200 ft even now exist in India.

The numerous conduits which supplied water to ancient Jerusalem are very old, no exact date can be assigned to their construction but they probably go back to the times of the Kings of Judah, 600 to 900 B.C. The two most important of these conduits were carried at different levels to the city from a large reservoir consisting of the three pools of Solomon, built in three terraces, the height was 40 ft. above the ground level. The conduits were rock-cut canals partly built in masonry. Valleys were crossed by syphons formed of large pierced stones embedded in rubble masonry.

The water supply for the city of Rome is one of the marvels of ancient times. The water was brought from the surrounding hills in aqueducts totalling about 385 miles in length. The first aqueduct, the Appia, was 10 miles long and was built in 312 B. C. All of these aqueducts were constructed along the hydraulic grade line in order to avoid the necessity for building pressure conduits. Iron pipe was unknown at that time. Lead was the only material available to carry water under pressure as lead was not suitable for high pressure, it was necessary to convey water at atmospheric pressure in aqueducts. The Greeks were very skilful in their methods of bringing water to their towns in conduits along the contour, lines of hills or through tunnels.

Distribution through pipes was probably unknown to the ancients. London was perhaps the first modern city in the world which at the end of the 16th century used lead pipes for conveyance or distribution of water. After this, for many years wood pipes bored out of logs came to be used and in some parts of Europe such pipes are still in use.

Most of the early water supplies were contaminated by various impurities and the people did not know the science and technology required to purify water to make it safe. People suffered much from various water-borne diseases. The men who first raised the objection to drink impure water and tried to purity it included engineers, scientists. doctors, lawyers, writers and statesmen. Notable among engineers were John Gibb (designed and constructed the first water filter at Paisley in Scotland, U.K. in 1804). James Simpson (designed and built a sizeable water filter for the Chelsa Water Company to improve its supply from the Thames River, England, in 1829) and James P. Kirkwood (designed and built the first sizeable water filter at Poughkeepsie in New York; USA, in 1871).

Among the lawyers were Sir Edwin Chadwick (England. 1842) and Lemuel Snattuck (Boston. USA, 1850; who first told the public not to drink polluted water.

Famous among the doctors were Sir (Dr.) John Simon (first medical officer of health of London City, 1842) and Dr. Stephen Smith (general practitioner of New York City, 1850) who demanded the purity of public water supplies. Two researchers of medical profession of England also accelerated the revolution against drinking of polluted waters: they were Dr. John Snow who in 1849 demonstrated to the world not yet blessed by the discoveries of Louis Pasteur, the role of faecal pollution of drinking water in the epidermicity of cholera and Dr. William Budd who from 1857 onward investigated the water-borne diseases specially, Typhoid fever, its nature, mode of spreading and prevention.

Among writers, Charles Dickens commented on the polluted water of the Thames River and urged the government to adopt necessary steps to purify the water supplied to the public in England (1850).

Therefore, the science, engineering and technology of modern wafer supply systems were contributed by the combind efforts of engineers, chemists, biologists, doctors, geologists, economists and other specialists in the natural and social sciences.

The modern renaissance in water-works design, construction and operation was marked at the start of the nineteenth century by invention of steam-driven pumping machinery and of cast iron. The first steam-driven pumping engine is said to have been installed in London in 1887. Previous to this, pump-driven by the river current have been used. The first steam-pumping engine in the United States was installed in Philadelphia water-works in 1800. Cast iron pp conveyance of water was laid in Philadelphia in 1804 and in London in 1807. Public water supplies in the United States date from 1652 at Boston.

Ancient water supply systems did not have proper treatment methods. Although some cities were able to collect safe water from uninhabitated regions and thereby reduce water-borne diseases to some extent ; many others found their supplies dangerously polluted and that the danger was increasing as population increased upon watersheds. Accordingly, some treatment methods, such as sedimentation, were developed, which when properly applied, eliminated the hazard to some extent. The theory of water filtration was conceived by the engineers early in the 19th century, but city councils were slow to be convinced of the necessity for spending money to save lives, and the water treatment methods were not widely adopted until about 1909. When slow-sand filters were introduced in England in 1906, an immediate reduction in Typhoid fever occurred. This epidemic was further checked by disinfection of filtered water with chlorine. A still creater decrease was accomplished after 1920 by careful control over infected persons, who had become carriers.

As progress and civilization advance, the difficulty of obtaining water becomes greater, because the population of towns increases to such an extent that the existing sources of natural supplies are neither within an easy reach of the majority of the public, nor are they sufficient. The requirements for municipal, trade and manufacturing purposes grow considerably. The sources were inadequate and got contaminated in dry season and became sources of danger. Cholera, typhoid, dysentery and diarrhoea were found in epidemic form. Hence, public water supply schemes, which could deliver pure water to the houses, industries, public places and trade centres, were urgently felt by the authority concerned to be installed.

In our country, water supply on modern lines is comparatively of recent origin. The first water works for the supply of water to Dacca City was completed by the Nawab of Dacca (Sir Nawab Abdul Ghani) in early eighteenth century. After this, water-works were constructed by the Government at Chandpur, Chittagong and other places. The water-works in Calcutta was completed in 1870 and those of Bambay, Madras and Poona 1875, 1880, 1890 respectively.

1.4 Objectives of Water Supply Systems:

The broad objectives underlying any water supply system are: (1) to supply safe and wholesome water to consumers; (2) to supply water in adequate quantity; (3) to make water easily available to consumers so as to encourage personal and household cleanliness.

In order that water should be safe and wholesome it, must satisfy the criteria of being, least harmful upon consumption. A wholesome water is usually one which is unpolluted, free from toxic substances as well as excessive amounts of mineral and organic matter that may impair the quality of water. Standards of quality for drinking water have been established by many countries now to ensure that the water supplied is really safe and wholesome. The first objective as outlined above is fairly satisfying. To supply water in adequate quantity would mean that the source of water supply must be so selected as to ensure that quantities of water as required by the community would be amply available, further, capacities of units required to store requisite quantity of water for continuous supply must be sufficient. To make water within the easy reach of consumers, would mean planning a well laid-out system of distribution involving pipes, valves and other fixtures of adequate design and capacity so that the system could be fully relied upon to meet the continual requirements of consumers at all hours of the day.

1.5 Elements of Water Supply System :

The water requirement of a modern city is so great that a system capable of supplying a sufficient quantity of potable water is necessary. The first step in the design of a water supply system is the determination of the quantity of water that will be required with provision for the estimated requirements of the future. Next a reliable source of water must be located, and finally a collection system, a treatment plant and a distribution system must be provided.

Water use varies from city to city depending on the population, climatic conditions, industrialization and other factors. In a given city, water use varies from season to season and even from hour to hour The planning of a water supply system requires that the probable water use and its variation be estimated as accurately as possible.

The essential elements of a water supply system are as follows : (1) Source of supply ; (2) Collection system; (3) Treatment or purification plant; and (4) Distribution system.

1) Source of Supply : All waters come in the form of precipitation. It is evaporated from the ocean, condenses to form clouds and finally precipitates over land. As the water falls in the form of rain or snow or sleet or hail, it acts as a vacuum cleaner picking up all the dusts and dirt particles in air. Needless to say, the first water that falls from the clouds picks up the greatest concentration of contaminates. After a short period of all, the precipitation is relatively free of contaminants. When the water hits the ground, a portion of its runsoff across the surface of the ground and a portion of its sinks into the ground. Therefore, the source of supply is of two types : a) Surface water supply and (b) Ground water supply.

(a) Surface Water Supply : The water running across the surface of the ground has been designated as surface water. It picks up many substances as it flows back to the ocean like Micro-organisms, organic matter, minerals and other polluting substances. Surface supply is generally obtained from streams, lakes, ponds, reservoirs and oceans. Since surface water is highly polluted it needs extensive treatment.

(b) Ground Water Supply : The surface water which seeps into the ground is designated as ground water or sub-surface water. As it travels through the surface layers of the earth, it picks up-some minerals and a few organics in solution. The micro-organisms and particulate matter find themselves being filtered out in the upper layers. Thus, it is found that most ground waters taken far below the earth's surface are free of microorganisms. These waters are usually relatively low in mineral and organic contaminants. Needless to say ground waters are usually preferred as sources of drinking water to surface waters. Springs, wells and galleries form the chief ground water supply.

Suitability of Sources with Regards to Quantity and Quality :

Quantity : As the effect of rainfall is most direct on the surface sources of water supply, the quantity of water available is abundant. However, since the rainfall may not be uniformly spread throughout the year especially in the case of tropical countries like Bangladesh and India, considerable variations in the flow of surface waters are likely. Thus, the flow in the streams or rivers may vary from a maximum during the rainy season, ancient to result in floods to a minimum during dry months, sufficient to cause long droughts. In case of impounding reservoirs, in addition to the rainfall and run off, the topography of the catchment area is important. It should be such as to-drain off water from all remote points.

As regards the underground sources, the quantity of water available is usually less than that in the case of surface sources ; the effect of rainfall now being most indirect, and depending upon the available underground storage and the geological formations of the substrata i.e. permeable or non-permeable. Incase of shallow wells and springs, it is easier to get supplies by tapping the upper water bearing strata but such a storage may be temporary and fall off during dry season resulting in the failure of the-source. The underground supplies drawn from greater depths i.e deep wells, are more constant in their yield and hence more reliable.

Quality : Impuritities in water normally are of two types, suspended and dissolved. The surface waters are characterized by the suspended impurities whereas the ground waters are generally free from the suspended matter but are likely to contain a large amount of the dissolved impurities, which they gather during the course of their travel in the underground strata comprising of rocks and minerals. The suspended matter often contains the pathogenic or diseases producing bacteria ; as such surface waters are not considered to be safe for water supply without the necessary treatment. Ground waters are comparatively safer and fit for use with or without minor treatment only.

The rain water is soft, has a flat taste and is free from contamination. As, however, rain falls through the air, it collects dusts and gases from atmosphere and becomes impure. Where rain water is collected in storage tanks, it may pick up impurities and therefore requires to be disinfected before use or drinking. However, because of the soft nature of the water, it find; an excellent use for washing purposes.

The river water varies in quality. This variation is caused by the great difference or the maximum and minimum flow. The maximum flow is caused by high floods, resulting in an increase in turbidity and bacteria due to the surface wash brought into the river. The minimum flow due to the flow of ground water into the river, resulting in the decrease of turbidity but increase of dissolved impurities. The river water is also usually found to be contaminated with sewage or industrial wastes from towns and cities. The river water, therefore, must be throughly treated before supplying for public use.

An impounding reservoir stores water by the construction of a dam across a natural water course. The storage provided may be as much as 60 days or less. This long storage enables the suspended matter to settle down and be removed. There is also considerable reduction in harmful bacteria and in colours present. Long storage is, however, objectionable in one way and this is that it creates-growth of microscopic organisms in water impairing its general quality. Aeration and chlorination are thus normally required before water is considered fit for supply.

The quality of ground water is comparatively much better. This is due to the fact that water gets strained during its passage through the porous underground strata. The geological formations with which water comes into contact also impart to it certain qualities like softness or hardness. The bacterial content of waters from springs, infiltration galleries and deep wells is usually low due to the straining action involved. In general, ground water is good in quality but may require some treatment to improve its chemical characteristics.

Choice of a Source for Water Supply : Considerations in the selection of a particular source for supplying of water are : (a) Quantity, (b) Quality and (c) Cost

The quantity of water available from the source should be sufficient to cater for the needs of the town or city regarding domestic service, industrial demands, fire fighting requirements and other public uses. The quantity of water supplied should also include the design requirements, which means the calculated quantity would be somewhat higher than the bare needs.

The quality of water should be wholesome, safe and free from pollution of any kind. The health of the public should in no way be endangered due to epidemics associated with water-borne diseases.

The quantity and quality of water are prime considerations in the selection of any source of supply. Cost considerations regarding the development and operation of water supply are also significant. The cost of supply would depend whether the system of supply is such that the water flows by gravity from the source or it has to be pumped first before supplying. Cost would, naturally, be less in the first case. The cost shall also depend upon the distance between the source of supply and the distribution system. Longer distance means water cost of conduits and other appurtenances required. In short, the cost of water supply must be reasonable compared to the number of people served and must bear a fair relation to the value of property served so that by equitable taxes and reasonable charges for water, the original cost of the system can be repaid at the end of the design period, which is usually 20 to 30 years.

(2) Collection systems : The collection system is a sort of engineering works designed to convey from a source to a treatment plant. The following are essential units in a collection system: (a) Intake, (b) Intake main, (c) Aqueduct or Transmission main, and (d) Pumping station.

Intake : An intake is a device or a structure placed in a surface water source to permit the withdrawl of water from this source and its discharge into an intake conduit or pipe (intake main) through which it will flow into the water-works system. Intakes are of primary importance. In general, they consist of the opening protected by a strainer or grating through which the water enters to the intake main. Types of intake structures consist of intake towers submerged intakes, intake pipes or conduits, movable intake and shore intakes. Intake structures of the inlet end of inlet conduit is necessary to protect against current and wave action, floods, stoppage, navigation, pollution and other interference with the proper functioning of the intake. Intake should be so located and designed that the possibility of interference with the supply is minimised to the greatest possible extent. Where uncertaintity of continuous serviceability exists intakes should be duplicated. The following factors must be considered in designing and locating intakes : (i) Location of the best quality of water available, (ii) Possibility of wide fluctuations of water level, (iii) Characteristics of intake surrounding, i.e , depth of water, character of the river bottom, navigation requrements, the effects of waves, currents, floods and storms upon the intake structure and in scouring the river bed and banks, (iv) Formation of shoals and bars, (v) Possible sources of pollution, and (vi) Provision for excluding possible floating materials like logs and vegetations.

Intake Main and Transmission Main : A pipe line must be used to deliver the water from the source to the treatment plant. The kind of pipe to be used-cast iron steel, concrete, asbestos cement or even wood will depend to some extent on local conditions and local costs, which should be determined on an engineering basis. Similarly, the size of the pipe is fixed by the volume of water to be delivered and the pressure or head of water. In general, the design of the pipe line will be governed by the principles of engineering economics, perhaps to a great extent than any other part of the water supply system.

Pumping Station : A pumping station is essential for pumping water from the source through the intake to allow water to flow by gravity through the transmission main to the treatment plant.

(3) Treatment Plant : Water, as it is drawn from streams, reservoirs, or wells, is rarely suitable for use. It must usually be treated before it reaches the consumers. In case of surface waters, the treatment procedure may involve the removal of turbidity, colour, taste, odour and bacteria. Ground water from wells may be treated to reduce hardness, iron, corrosive qualities, and sometimes bacteria. The methods used for treatment include screening, sedimentation, treatment with chemicals, filtration through sand beds, and disinfection to kill microorganisms. Most streams are polluted by industrial or domestic wastes to some degree and treatment of the water from a stream must be designed to provide a safe, clear and palatable water under any condition. The engineer must, on the basis of a careful examinanon of local data and conditions, plan. design and supervise the construction of a plant that is capable of producing satisfactory water. He must also so design the plant that the correct operation is easiest and most logical. Adequate treatment is necessary because the use of polluted water can be an important factor in the spread of many diseases.

4) Distribution System: The distribution system is needed to deliver water to the individual consumer in the required quantity end under a satisfactory pressure. The distribution system is often a major investment of a municipal water-works. This includes: (a) Various pipes that convey the water to the consumer, (b) Storage Reservoirs that are provided to aid in the distribution of water, (c) Pumps and necessary equipments, and (d) Fire hydrants, valves, meters and other appurtenances.

Classification of Distribution Systems : There are three general methods or systems for furnishing satisfactory water pressure in the distribution pipes, namely, the gravity system, the distributing reservoir system, and the direct pressure system. In a gravity system the source of supply is at such an elevation with respect to the community to be served that adequate pressure in the pipe is obtained directly from the head. In a distributing reservoir system the water is pumped to a reservoir called a storage reservoir or distributing reservoir which is at such an elevation that the water flows by gravity through the mains. In a direct pressure system, the water is pumped directly into the mains. These methods may be combined, or may merge into one another and therefore, the classification is not absolute.

The procedure in designing a distribution system is essentially the same in each case. The details of the system are determined largely by cost and local topographic characteristics.

Pressure Requirements in Distribution Pipes : For general domestic purposes, the necessary pressure of water in the pipes, which is usually indicated by an ordinary pressure gauge is about 50 to 60 psi. However, in communities built in hilly or mountainous districts, considerable variation in pressure may be necessary in the pipe lines. In general, the pressure for domestic purposes should not fall below 20 psi and should not exceed, 120 psi. Where the conditions are such that, variation in pressure would otherwise exceed the given limits, it is necessary to layout the distribution system in separate districts of high and low pressure. To supply water to all floors in a very high multi-storied building, it is necessary to install separate pumps and roof top tanks in the building.

Fire service requires pressure higher than that provided for domestic use because the water is used at a greater rate and, to be effective in fighting fire the water must issue from the nozzle of a fire hose with a sufficient velocity to a considerable height. When the pipe pressure is dependent on the forcing water through a hose to a fire, a pressure of at least 80 to 100 psi is needed in the pipes. Therefore it is often cheaper, especially in large cities, to use fire engines for providing extra pressure in case of fire, and to provide a lower pressure in the pipes for ordinary occasions. Like many other engineering problems, the question of pipe pressure requires for its solution a careful weighing of conditions and costs.

Distribution Reservoirs : These reservoirs provide storage of treated water to meet the water requirement of the consumers and also to provide fire storage and to stabilize pressures in distribution system. The reservoir of steel or concrete. They may be cylindrical, rectangular or square. The reservoirs should be located a close to the centre of use as possible. The water level in the reservoir must be high enough to permit gravity flow at satisfactory pressures to the pipe system it serves.

Pumping Station: A pumping station is needed to pump water to the reservoir (overhead water tank) to allow the water to flow by gravity to the distribution system.

Distribution Pipes : The basic requirements of distribution system are adequate strength and maximum corrosion resistance. Cast iron, concrete, asbestos-cement, PVC / polyvinyl chloride) and G.I. (Galvanized Iron) pipes compete in .smaller sizes, while steel, and R.C.C. (Reinforced Cement Concrete) pipes compete in larger sizes.

All the Essential elements of a water supply system for a city in diagrammatically shown in the Fig. 1.1.

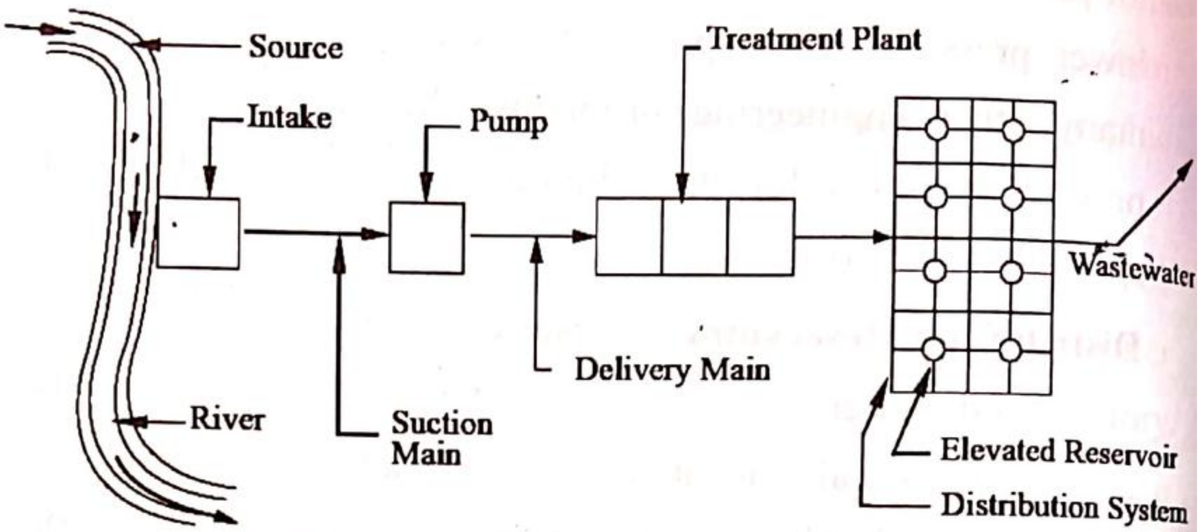


Fig. 1.1 Essential Elements of a Water Supply System

1.7 Planning a Municipal Water Supply System :

In general the following must be done in planning a municipal water supply system;

(1) Estimate of the future population of the community and study of local conditions to determine the quantity of water which must be provided.

(2) Location of a reliable source of water of adequate quality.

(3) Design of a suitable collection system.

(4) Provision for the necessary storage of water, and design of the works required to deliver the water from its source to the community.

(5) Determination of the physical, chemical and biological characteristics of the water.

(6) Design of various units of the treatment plant.

(7) Design of the distribution system, including distribution reservoirs, pumping station, elevated storage, layout and location of fire hydrants.

(8) Provision for the establishment of an organization which will maintain and operate the supply, distribution and treatment facilities.

1.8 Conclusion :

Modern civilization is far more dependent on water than were the civilization of the past. Modern water supply and sewerage engineering together with modern medical science reduced death rates and increased life expectancy. Modern standard of living requires much more water than was used only hundred years ago. Increasing population demands more attenion to water supply, storm drainage and sewerage. Industrial Progress finds increasing uses of water in process industries, for steam generation, and also for electric power production. The emphasis of water supply engineering shifts more or less steadily and continuously.

The development of civilization has increased the importance of water supply engineering and there is no prospect of a line of activity in this field in the foreseeable future. More efficient methods in planning and design and better construction materials and process for water supply systems for urban and rural areas must be utilized to make the systems more efficient and to reduce costs so thất water supply projects may become economically feasible.

QUESTIONS

1. Explain the importance of water supply engineering upon city life. BUET, 1962,68.
2. Write a short essay on the history and development of water supply'engineering. BUET, 1965.
3. Discuss briefly the engineering aspects of water supply. BUET, 1964.
4. Enumerate and explain briefly the essential elements of a water supply system for a city with the help of a neat sketch. BUET, 1963, 66, 69, 70, 72
5. Enumerate the factors to be considered in planning a manicipal water supply system. BUET, 1970, 72.
6. Write an essay on "Water Supply Engineering and Public Health". BUET, 1966, 69, 70, 73.

2
WATER REQUIREMENTS

2.1 Introduction :

Of prime importance in the design of a water supply system is the framing of an estimate giving the total quantity of water that will be required by the community after the completion of the works. The estimate enables the engineer for the determination of sizes and capacities of all the constituent parts of the water supply system. This is generally done by the help of two factors: (1) Probable population estimated at the end of the design period, and (2) Rate of water supply per capita per day.

The design period is the period into the future for which the estimate is to be made. The period should neither be too long so that full financial burden is not thrown on the present generation, nor should it be too short so as to avoid the design becoming uneconomical. In practice, a period varying from 25 to 40 years is considered sufficient for design purposes.

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

$$= \frac{\text{Total consumption in gallons}}{\text{Population} \times 365} \qquad 2.1$$

2.2 Factors Affecting Per Capita Consumption of Water:

The various factors and the way they affect the per capita water consumption are as follows :

(1) Size of the city: The per capita use of water tends to be higher in larger cities than in small towns. Average use In the larger cities is about 100 gpcd as compared with 60 gpcd for the country as a whole. The difference results from greater industrial use, more parks and other public facilities, greater commercial use and perhaps more loss and waste in the larger cities.

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

(3) Climatic Conditions : More water is used in warm, dry climates than humid and cold climates for bathing, lawn and garden watering, air conditioning, etc. In extremely cold climates, water may be wasted at faucets to prevent freezing of pipes.

(4) Commerce and Industries : Industry uses large volumes of water in its manufacturing processes and in supporting operations. Indeed, the production of foodstuffs, metals, chemicals and other basic commodities calls for a tonnage of water that far exceeds the combined tonnage of other raw materials. The actual amount depends on the extent of the manufacturing and the type of industry. Table 2.4 shows the water use in some selected industries. Some industries develop their own water supply and place little or no demand on the municipal water supply system. Zoning of the city affects the location of industries and may help in estimating future industrial demands. Commercial areas include office buildings, warehouses and stores. The per capita demand in such areas is not high, averaging about 10 gpcd per full time employee. The amount of water used by industry varies widely. Table 2.5 shows the water consumption by different industries based on the percentage of total water that is drawn up through the intake.

About 80 per cent of industrial water is for cooling and need not to be of high quality. Water used for process purposes or for boiler feed must be of good quality. In some cases, industrial water must have a lower content of dissolved salts than can be permitted in drinking water. The location of industry is often much influenced by the availability of water supply. However, when other factors dictate the plant location, water requirements, may be reduced far below the industry average.

Commercial consumption of water is sometimes taken as 50,000 to 100,000 gallons per acre per day.

(5) Pressure of Water : The rate of use of water increases when the pressure on the distribution system is increased.

This is due, in part, to the greater loss through leaks and the greater amount run to waste through open faucets. Increases in the rates of use of water with pressure have been known to reach 30 per cent for a change from 25 to 45 psi. This fact should lead the designer to provide the lowest pressure that will give satisfactory service. Excess pressure means wastage of water.

(6) Quality of Water : Improvement of the quality of the water supply will result in increased use of water in part because of the availability of the water for more uses and a feeling of safety on the part of the public in using it. The water quality also influences the industrial uses. If the water is soft and meets the standards of industrial water, the rate of consumption will be high.

(7) Sewerage Facilities : The effect of the installation of sewers in a city is to increase the rate of water consumption because of increase is plumbing facilities. In an unsewered area water consumption generally does not exceed 15 gpcd while in a sewered area the consumption will be equal or exceed 50 gpcd.

(8) Water Rates and Metering : If the cost of water is high, people may become more conservative in water use and industries will often develop their own supply to obtain cheaper water. Metered consumers are more likely to repair leaks and use water with discretion. The installation of meters in some countries has reduced water use by as much at 40 per cent.

(9) Nature of Supply: If the water is supplied intermittently (only for some parts of the day), the rate of water consumption much less than when it is supplied continuously. This may due to decrease in losses and other wasteful uses.

(10) Availability of the Private Supplies : The demand for municipal water is reduced to a great extent if the people and industrialists develop their own private supplies from wells, springs etc. High cost and poor quality of municipal supply often compel the users to go for own supplies.

(11) Efficiency of the Management: The efficiency of the management of a water-works will affect consumption by controlling loss and waste. There is always some leakage in the collection, transportation and distribution system of water-works. In well managed water-works, loss and-waste of water through leaks and faulty joints are carefully controlled. Therefore, an efficient water-works management will keep the loss and waste of water to a minimum.

(12) Number of Inhabitants : This would affect the extent to which use is made of private water supply. Thus, in large cities, the public water supply is almost a necessity while in small towns and villages, the private supplies may remain in use even long after the introduction of the public water supply. Generally per capita consumption is found to increase with increase of population.

2.3 Consumption of Water for Various Purposes :

Water consumption varies from city to city, depending upon the population, climatic conditions, industrialization and other factors. In a given city, water consumption varies from season to season and from hour to hour. Municipal consumption of water may be broken down into the following four general classes :

(1) Domestic Use : This includes water furnished to houses or private buildings for purposes of drinking, bathing, cooking sanitation and other purposes. This varies according to the living standard of the consumers. The standard recommended for use in many countries ranges from 20 to 50 gallons per capita per day.

(2) Industrial and Commercial Use : This includes water required to the offices, stores, hotels, factories, breweries, sugar mills, refineries, tanneries and other industries. This consumption will vary greatly with the character of the city. In small residential communities, the commercial and industrial use may be as low as 10 gpcd, but in big industrial cities it may run as high as 100 gpcd.

(3) Public Use : This includes water used for public buildings such as schools, colleges, universities, hospitals, cinema and theatre halls, jails, mosques, etc., parks, gardens, fire fighting, sprinkling streets, flushing sewers, public fountains and ornamental displays. This varies between 5 to 20 gpcd.

(4) **Loss and Wastes :** This is the water unaccounted for and may be due to bad plumbing, faulty connections, breakage of pipes, meter slippage unauthorised connections, leaky trains and other wastes. This can be minimised by careful maintenance and universal metering. It is often estimated to be 10 to 40 gpcd.

Table 2.1 Normal Consumption of Water

Nature of Consumption	Quantity, gpcd	
	Normal Range	Average
Domestic Use	20-50	45
Commercial and Industrial use	10-100	65
Public Use	5-20	15
Loss and Wastes	10-40	20
Total	45-210	145

Table- 2.2 Special Consumption of Water

Nature of Consumption	Quantity, gpcd unless otherwise specified
Office buildings	10-15
Educational Institutions	10-25
Hostels and halls of residences	30-40
Hospitals	130-350 gallons per bed per day
Hotels	150-250 gallons per day per occupied room
Factories	10-20
Cinema and theatre halls	5-10
Single family houses	35-50
Multifamily houses	50-90
Restaurants	0.5-5 gallons per meal per person
Laundries	3-6 gallons per pound of clothes washed
Domestic animals	3-15 gallons per animal per day

Table 2.1 and 2.2 show the normal and special consumption of water for various purposes.

A breakdown of domestic water consumption for various uses is as follows: 41% for flushing toilet; 37% for washing and bathing ; 6% for kitchen use; 5% for drinking ; 4% for washing clothes ; 3% for general household cleaning ; 3% for watering the gardens and 1 % for washing family cars.

Table 2.3 shows the distribution (approximate) of total consumption of water in a city.

Table 2.3 Distribution of Total Consumption of Water

Nature of Consumption	Percentage of total Consumption
Domestic	42.80
Commercial	31.60
Industrial	12.20
Public and other	13.40
	Total : 100.00

In our country especially in the districts of Dhaka and Chittagong, present water use rate at street faucet varies between 2.5 to 10 gpcd. Presently, the majority of the urban population in Bangladesh live in sub-standard housing and rely upon street taps and hand pump tube-wells for their drinking and cooking water. Estimated present water use for the water works of Dhaka and Chittagong ranges from 30 to 40 gpcd and in 1985 it will range between 40 to 50 gpcd.

Table 2.4 shows the water requirements for some selected industries.

Table 2.4 Water Requirements of Some Selected Industries

Industry	Unit of Production	Gallons per Unit
Manufacturing Products		
Paper	ton	2,000-100,000
Paper pulp	ton	4,000- 60,000
Leather (tanned)	1000 sq. ft. of	1,200- 60,000

	hide	
Cotton goods	1,000 Ib	20.000-100,000
Rayon hosiery	ton	12,000-25,000
Woolens	ton	100,000-140,000
Automobles	each	8,000- 10,000
Food Products		
Meat (slaughtering and packing)	1,000 Ib live weight	600-3,500
Beverage alcohol	gallon	125-170
Vegetables (canned)	case	5-250
Beer	Barrel	470
Soft drinks (coke, seven-up, sprite, etc.)	Ton	3,600 -4,500
Mineral Products		
Aluminum (electrolytic smelting)	ton	35,000-56,000
Copper		
Smelting	ton	10,000
Fabricating	ton	4,000
Refining	ton	200-1,000
Petroleum	Barrel of crude oil	800-3,000
Steel	ton	1,500-50,000
Oil Refining	Barrel	600-800

2.4 Fire Demand :

Fire protection is an important function of a water-works. The total amount of water used in a year for extinguishing fires is usually a negligible part of the total use, but during a fire the rate of demand of it so great as to be the deciding factor, in all but the largest communities, in designing the pumps, reservoirs and distribution systems. The fire demand is a function of population, with a minimum limit, because the greater the population the greater the number of buildings and the greater the risk of the fire. The minimum limit of the fire demand is the amount and the rate of supply that are required to extinguish the largest probable fire that could be started in a community.

At least four streams should be available at all points with in the area protected. Each stream should be capable of delivering at least 175 gpm of water in-low-risk districts and 250 gpm in high-risk districts. The above quantity of water should be available for at least 5 hours. The pressure to be provided at the fire hydrant should, in general, not be less than about 80 psi where mobile fire pumping engines are used, and 80 to 100 psi otherwise.

Authoritative recomendations concerning allowance to be made for peak fire demands in water works design are listed in the Table 2.5. The maximum rate of demand to be provided for design is the sum of the fire demand and the general service demand occurring simultaneously.

Table 2.5 : Empirical Formula For Computing Rates of Fire Demand

Name of the Authority	Formula	Rate in gpm for 100,000 population
National Board of Fire Underwriters	$Q= 1{,}020\sqrt{P}\,(1-0.1\sqrt{P})$	9,180
Kuichling (on basis of fire streams of 250 gpm)	$Q=7000\sqrt{P}$	7,000
Freeman, John R.	$Q=250\left(\frac{P}{5}+10\right)$	7,500
Where, Q= Fire demand in gpm and P= Population in thousands		

The National Board of Fire Underwriters requirements of fire protection water vary between 1,000 gpm for 1,000 persons and 12,000 gpm for 200,000 persons, with a maximum of 20,000 gpm. There must be enough water in the reservoir to provide for a 5-hr fire for towns of less than 2,500 persons and a 10 hr-fire for larger cities.

Based on the experience of water supply engineers and fire departments, a number of, empirical formula for the required number of fire streams have been devised. The following formula devised by Kuichling is most commonly used.

$$F= 2.8\sqrt{P} \tag{2.2}$$

in which F= number of simultaneous fire streams, P= population in thousands.

Example : Calculate the total stream flow in gpm for a town having a population of 10,000. Assume that each stream will spray 250 gpm on the fire simultaneously.

Solution: $F=2.8\sqrt{10}=9$

Total stream flow = 250×9 = 2,250 gpm.

But the amount recommended by the National Board of Fire Underwritets is 3,000 gpm (See Table 2.6). Therefore, the values in the Table 2.6 are generally used.

Table 2.6 : Fire Flow Required by the National Board of Fire Underwriters

Population	Recommended fire flow, gpm	Population	Recommended fire flow, gpm
1,000	1,000	28,000	5,000
2,000	1,500	40,000	6,000
4,000	2,000	80,000	7,000
6,000	2,000	100,000	8,000
10,000	3,000	125,000	9,000
13,000	3,500	150.000	10,000
17,000	4,000	200,000	12,000
22,000	4,500		

Example : What fire flow and storage are required for fire protection (10 hr) in a city of 200,000 population according to the recommendation of the National Board of the Fire Underwriters?

Solution : Total fire flow= 13,000 × 60 ×10= 7,200,000 gallons.

This amount should be kept in the storage reservoir in addition to the reserve for normal use during the fire.

The computed quantity of water to be provided for fire demand is then divided by the estimated population of the city or town to obtain the provision to be made per. person for fire fighting. The quantity of water required for fire fighting should always be stored in the distribution reservoir below its normal low water level. The required number of hydrant are to be located at suitable point in the distribution system to allow the specific number of streams to play upon the fire at a time. Fire engines are to be kept in readiness at different parts in the city. They are summoned to draw the water from the hydrants and to deliver it in the form of streams on the fire. Sometimes, they are to carry pumps to booster the supply pressure available at the hydrant level.

2.5 Fire Hydrants :

A hydrant is an outlet from a water main and is provided chiefly for the purpose of forming a connection for fire hose. One type is shown in the Fig. 2-1. Hydrants are usually of post type, but flush hydrants set in boxes below the footpath or street corner are occasionally used in some cities. Hydrants are usually placed at or near street intersections and at intermediate points also. Since fire hose cannot deliver an effective stream at distances greater than about 400 ft hydrants should be so spaced that each section of the community can be conversed with hose from two or more hydrants. The National Board of Fire Underwriters recommends a spacing of about 200 ft. for a community of 25,000 to 30,000 population requiring a fire flow of 5,000 gpm and a spacing of not more than 300 ft for small copmmunites requiring only 1,000 gpm for fire flow. Hydrants are generally placed near the curb line but far enough from it to be protected from normal traffic hazards.

Fire hydrants are usually made of cast iron with bronze surface. It is frequently desired that a gate valve be placed on the connection to the distribution' system, in addition to the main valve on the hydrant. A rain for emplying the barrel of the hydrant when it is closed is essential in a cold climate to prevent the freezing of the hydrant. The drain should be connected to a drainage channel (storm sewer).

The National Board of Fire Underwriters requires that the hydrants shall be able to deliver 600 gpm with a loss of not more than 2.5 psi in the hydrant and a total loss of not more than 5 psi between the street main and the outlet ; they shall not have less than 2.5 inch outlets and also a large suction connection where engine service is necessary. They shall be of such design that when the hydrant barrel is broken off the hydrant will remain closed. Street connections should not be less than 6 inch in diameter and shall be gated.

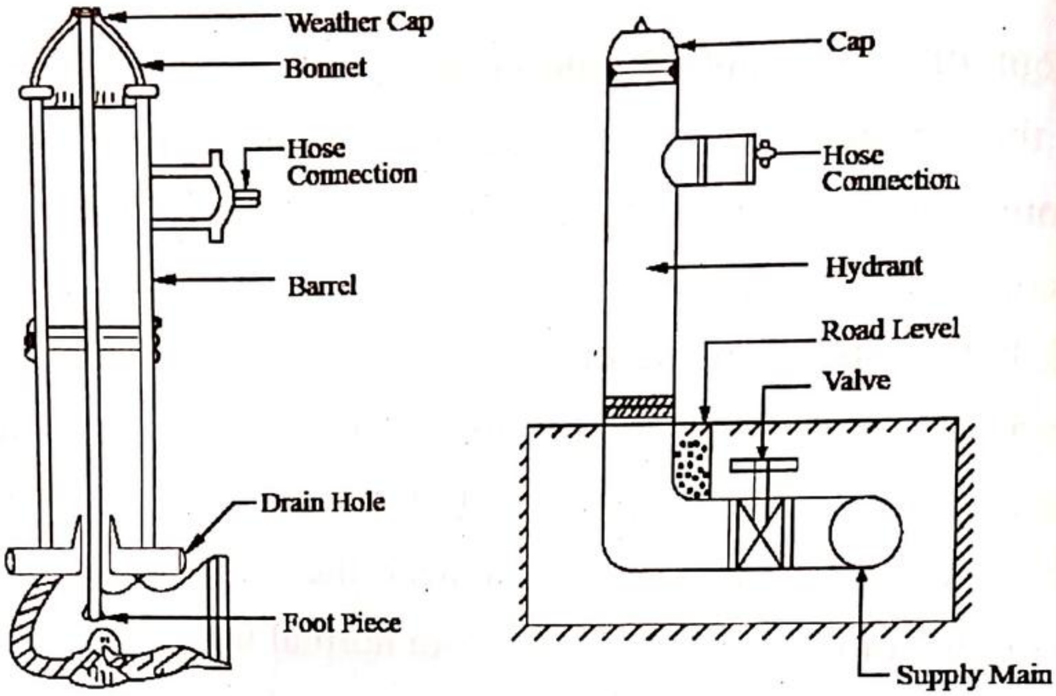


Fig. 2.1 Fire Hydrant

The size of fire hydrant is designated in terms of the minimum opening of the seat ring of the main valve. It must be at least 4 inch for two–$2^1/_2$ inch nozzles, at least 5 inch for three-$2^1/_2$ inch nozzles, and at least 6 inch for four-$2\frac{1}{2}$ inch nozzles. The rate of discharge from a hydrant can be approximately determined by the expression;

$$Q=27dp^{0.5}$$

where

Q=flow, gpm.

d = diameter of the hydrant nozzle, inch,

p = gauge reading, psi.

Fire hose $2^1/_2$ inch in diameter and 100 ft long with a $1\frac{1}{8}$ inch nozzle will deliver about 240 gpm with a nozzle pressure of 60 psi and a hydrant pressure of about 85 psi.

The flow will be less for longer lines or for lower hydrant pressures. To deliver 240 gpm, a hydrant pressure of about 105 psi must be provided for 200. ft long hose, 125 psi for 300 ft and 145 psi for 400 ft.

2.6 Variations in Water Use :

Water consumption changes with the seasons, the days of the week, and the hour of the day. There are major seasonal peaks during summer heat and drought when large volumes of water are drawn to refresh man and his domestic animals, watering lawns and gardens, and to cool air-conditioning equipments. In midwinter the average daily use is usually about 20 per cent lower than the annual daily average, while in summer it may be 25 to 40 per cent above the daily average. Seasonal industries (such as canneries) may cause wide variation in water demand during the year. Within any day there is much less use at night than during daytime hours (Fig. 2.3;. Day to day variations reflect household and industrial activities. Hour to hour fluctuations produce a peak between 6 to 10 A.M. and a trough (lowest consumption; between midnight to 5 A.M. Normal variations in water use must be known if supply pipes, service reservoirs and distribution pipes are to be properly dimensioned. Moreover, there must be suitable allowances for sudden, heavy and unpredictable water demand for fire fighting. The annual volume of water used for fire fighting is small, but during fires the rate of use may be quite high.

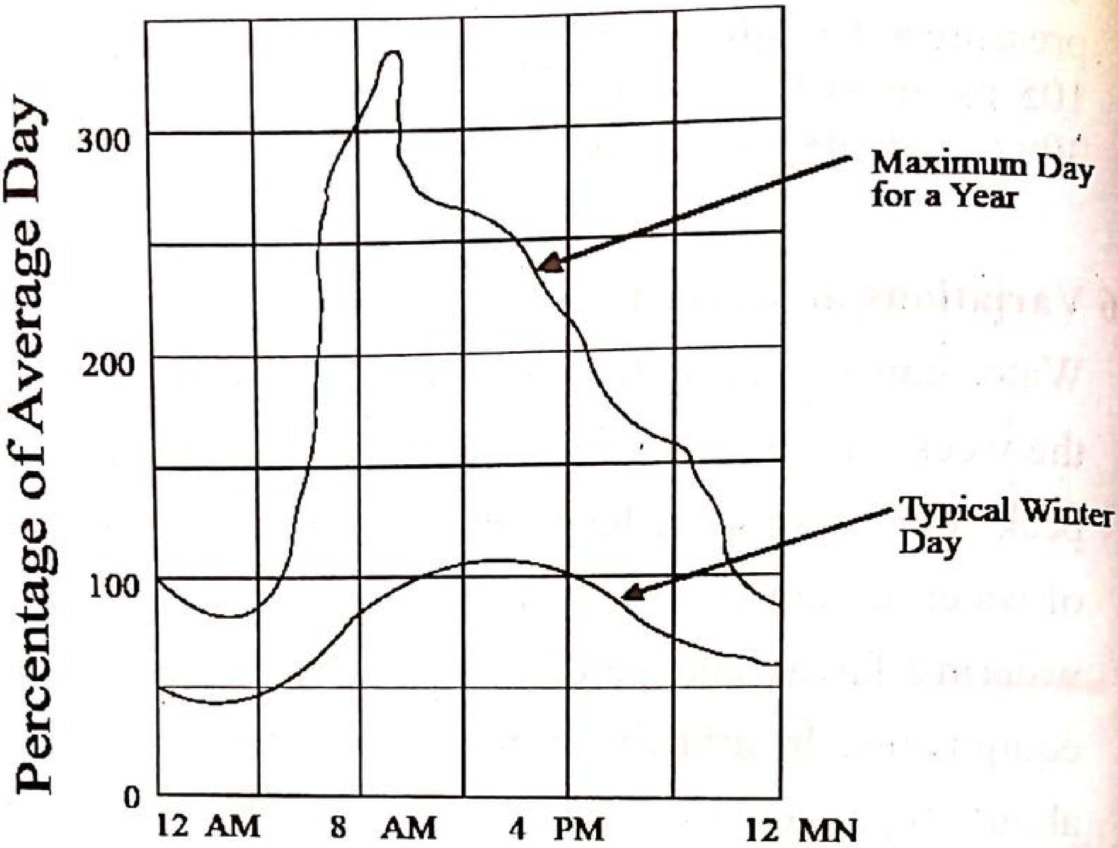


Fig. 2.2 Typical Hourly Variation In Water Consumption

Variations in water use are conveniently expressed as follows :

Maxinum daily flow = 1.5 to 2.0×average daily flow (2.4)

Maximum hourly flow= 2.0 × maximum daily flow or

2.0 to 3.0×average hourly flow (2.5)

Minimum daily flow= 1/3 to 2/3×average daily flow. (2.6)

2.7 Estimates of Water Use :

The first step in the design of a water-works system is an estimate of the requirement for water. Ordinarily an average of 145 gpcd is assumed, but this figure may be altered considerably by local conditions. Previous records in the city under study or data from similar cities in the area are the best guide in selecting a value of per capita use for design.

After deciding on an average per capita requirement, an estimate of the future population of the city must be made to determine average total use. The economic aspects of the problem determine how far into the future the population estimate should be projected. The basic question to be answered is : Is it cheaper in the long run to design and build the system to meet the demand expected at some future date, or to build now for a short time and plan to make additions as future needs develop ?" The answer is often a compromise. Some portions of the project may be more economically built to ultimate size immediately.

2.8 Prediction of Population :

The present population may be obtained from recent census with reasonable alterations. Future prediction is based on the knowledge of the city, and its environment, commerce and industries and their expansion development of surrounding countries, etc. Helpful in predictions will be the study of population trends of similar cities and consultations with local people and officials. The following seven methods are generally used for predicting population :

(1) Uniform Growth Rate Method : In this method, a constant increment of growth is added periodically. For example, if the population increased from 90,000 to 100,000 in a period of five years, it would increase by an increment of 10,000 in the next five year period. This method is also known as the Arithmetical Progression Method.

(2) Uniform Percentage Growth Rate Method : In this method, a constant percentage of growth is assumed for equal periods of time. For example, if the population increased from 100,000 to 110,000 during the past decade, it would increase 10 per cent to 1,21,000 during the next decade.

(3) Decreasing Growth Rate Method : This method is similar to the uniform percentage growth but with an arbitrary assumption of a decreasing rather than a constant rate of increase.

(4) Graphical Extension Method : In this method, the population time curve is extended into future date by eye-estimation. This method is also known as the Curvelinear method.

(5) Graphical Comparison Method : This method involves the extension of the population-time curve of the city under study into the future based on a comparison with population time curves of similar cities. In the this method, the population-time curves are plotted as indicated in Fig. 2.3 with the curves for all cities passing through the same point, represented by the present population of the city for which the predication is to be made. Projections based on studies of migrations, and on other predicted factors may be used to modify the results of the graphical comparison method. This method gives promise of a reliable prediction as it is based on a logical study of the past and future conditions. This method is also known as the Modified Curvelinear Method.

In Fig. 2.3. the population-time curve for the city A, is plotted upto the year 1960 in which its population was 40,000. City B reached 40,000 in the year 1930, so its curve is plotted from the year 1930 onwards. Similarly, curves are drawn, for cities C, D, and B from the year they reached As population of 1960 ; i.e., 40,000. The curve of the city A can now be continued allowing it to be influenced by the rates of growth of cities B, C, D and E.

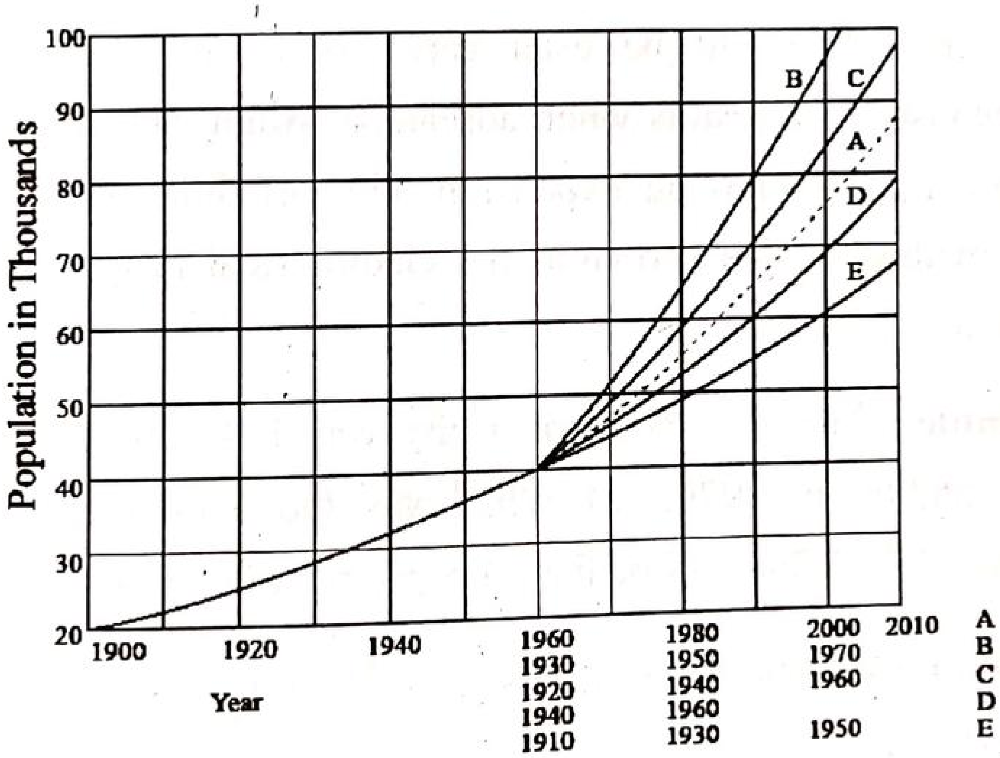


Fig. 2.3 Graphica Comparison Method of Population Prediction

(6) **Empirical Method:** The following empirical formula was suggested by Hardenberg :

$$^{P}f = {^{P}P}(1+r)^{n} \tag{2.7}$$

,where ^{P}f =future population

^{P}P =present population

r = probable rate of yearly or per decade increase

n =number of years to be considered.

When the population data of the past decades are available the average value of r can be computed from the following expression

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

where P_1 and P_2 are the population data at two dates of n number of years.

This method should be used carefully as it may give erroneously high results when applied to young and rapidly advancing cities having expansion of short duration only. This method is also known as the Geometrical Progression method.

Example : The population of a city was 124,000 in 1960 and 156,000 in 1970. (a) What was the annual rate of increase? (b) What will be the probable population in 1980?

Solution: (a) Here, P_1= 124,000, P_2= 156,000 and n= 10.

$$r = \sqrt[10]{\frac{156,000}{124,000}} - 1 = 1.023 - 1 = 0.023$$

(b) In this case, P_p =156,000, r=0.023, and n=20

$P_f = 156{,}000\,(1+0.023)^{20} = 246{,}000$ Ans:

(7) Least Square Parabola Method: In this method, the population-time curve is assumed to be parabolic.

Let the variables X and Y denote respectively the year and the population during that year. The equation of the least square parabola fitting the data (census data for a number of decades) is

$$Y=a+bX+cX^2 \quad \text{......} \quad (2.9)$$

where a, b and c are constants and are to be found from the following normal equations by applying the actual data :

$$\sum Y=aN+b\sum X+c\sum X^2 \quad \text{......} \quad (2.10)$$

$$\sum XY=a\sum X+b\sum X^2+c\sum X^3 \quad \text{......} \quad (2.11)$$

$$\sum X^2Y=a\sum X^2 + b\sum X^3 +c\sum X^4 \quad \text{......} \quad (2\ 12)$$

where N = number of observations or sets of data.

If the population data of the city under study for a number of decades are known, then by solving the simultaneous Eqs. 2.10, 2.11 and 2.12 with the given data, the values of the constants a, b, and c in the Eq. 2.9 can be computed, and the desired equation of the least square parabola can be found out. Then from this equation, the population at any future date can be computed.

F-4

The following worked-out example will give a clear picture of this method.

Example : Table 2.7 shows the population of a country during the years 1870-1970, in ten yean intervals, (a) Find the equation of the least square parabola fitting the data (b) Compute the trend values for the years given in the Table 2.7 and compare with the actual values, (c) Estimate the population in 1980 and 1990.

TABLE 2.7

Year	1870	1880	1890	1900	1910	1920	1930	1940	1950	1960	1970
Population (million	23.2	31.4	39.8	50.2	62.9	76.0	92.0	105.7	122.8	131.7	=151.1

Solution: It is convenient to chose X so that the middle year 1920 corresponds to X=0, and the years 1930, 1940, 1950, 1960, 1970, and 1910, 1900, 1890. 1880, 1870 correspond to 1, 2, 3. 4, 5 and -1, -2, -3, -4. -5, respectively. With this choice, $\sum X$ and $\sum X^3$ are zero and the Eq. 2.10, 2.11, and 2.12 are modified.

The work involved in computation has been shown in Table 2.8.

TABLE 2.8

Year	X	Y	X^2	X^3	X^4	XY	X^2Y
1870	-5	23.2	25	-125	625	-116.0	580.0
1880	-4	31.4	16	-64	256	-125.6	502.4
1890	-3	39.8	9	-27	81	119.4	358.2
1900	-2	50.2	4	-8	16	-100.4	200.8
1910	- 1	62.9	I	-1	1	-62.9	62.9
1920	0	76.0	0	0	0	0	0
1930	1	92.0	1	1	I	92.0	92.0
1940	2	105.7	4	8	16	211.4	422.8
1950	3	122.8	9	27	81	368.4	1105.2
1960	4	131.7	16	64	256	526.8	2107.2
1970	5	151.1	25	125	625	755.5	3777.5
Sum $\sum X=0$ $\sum Y= 886.8$ $\sum X=110$ $\sum X^3=0$ $\sum X^4=1958$ $\sum XY= 1429.8$ $\sum X^2Y= 92090$							

Using the results in the Table 2.8, the normal Eqs. 2.10, 2.11, and 2.12 become

$11a+110c = 886.8$ (2.13)

$110b = 1429.8$ (2.14)

$110a+1958c=9209.0$ (2.15)

From the Eq, 2.14, b = 13.00, and from the Eqs. 2.13 and 2.15 a = 76.64,

and a=76.64 and c= 0.3974.

(a) The required equation is

$Y= 76.74 +13.00X+0.3974X^2$ .... (2.16) where the origin X=0 is first January, 1920 and the unit of X is ten years.

(b) The trend values, obtained by placing X= - 5, -4, - 3, -2, -1, 0, 1, 2, 3, 4,5 in the Eqs. 2.16 are shown in the Table 2.9-together with the actual values. It is seen that the agreement is satisfactory.

TABLE 2.9

Year	X = -5 1870	X = -4 1880	X = -3 1890	X = -2 1900	X = -1 1910	X = 0 1920	X = 1 1930	X = 2 1940	X = 3 1950	X = 4 1960	X = 5 1970
Trend Value	21.6	31.0	41.2	52.2	64.0	76.6	90.0	104.2	119.2	135.0	151.6
Actual Value	23.2	31.4	39.8	50.2	62.9	76.0	92.0	105.7	122.8	131.7	151.1

(C) 1980 corresponds to X=6, for which

$$Y = 76.64 + 13.00\,(6) + 0.3974\,(6)^2 = 168.9 \text{ million}$$

1990 corresponds to X =7. for which

$$Y = 76.64 + 13.00\,(7) + 0.3974\,(7)^2 = 187.0 \text{ million}$$

2.9 Population Distribution and Density:

Estimates of the total population of a city or a town are needed in the design and management of water supply system. Distribution of water and collection of waste-water within the area call, in addition, for estimates of population density and nature of occupancy and use of component areas of the city. Population density is generally expressed as the number of persons per acre. A classification of areas and expected population densities is shown in the Table 2.10.

Table 2.10 : Typical Population Densities

Area Classification	Density persons per acre
1. Residential Areas	
(a) Single family residences, large blocks	10-20
(b) Single family residences, small blocks	20-40
(c) Multiple family residences	40-130
(d) Apartments or tenement houses	100-1000 or more
2. Commercial areas	15-40
3. Industrial areas	10-20
4. Total, excepting parks, playgrounds, graveyards, etc.	10-50

In Bangladesh, the following is the population density :

Residential Areas Population Density :

Single family high cost pucca residences : 25-50 per acre
Single family low cost pucca residences : 50-100 per acre
Katcha type residential areas : 75-150 per acre:
Multi-family high cost residences : 100-200 per acre
Multi-family low cost residences : 150-400 per acre

Gross City-Wise Population Density in Bangladesh :

Metropolitan cities like Dhaka and Chittagong 100-150 per acre

Smaller cities like Comilla, Khulna, Rajshahi, Mymensingh,Faridpur and Sylhet	70-120 per acre
Towns like Natore, Bhola	50-100 per acre

Population density is expected to vary from 25 persons to 400 persons per acre in various types of residential areas all over Bangladesh.

2.10 Essential Elements for Designing a Water Supply System for a City :

Most water supply systems include relatively massive structures (dam, intakes, reservoirs, treatment plants, distribution systems including overhead water tanks, etc.) that require a long time in construction and are not easily and readily expanded. Accordingly, the principal system components are purposely made large enough to satisfy community needs for a reasonable number of years. Selecting the initial or design capacity is not very simple. It calls for skill in predicting social and economic trends and a sound judgment in analyzing past experience and predicting future requirements. Among the needed estimates are the following :

(1) The number of years, or design period for which the proposed water supply system and its component structures and equipment are to be adequate.

(2) The number of people, or design population, to be served.

(3) The rates of water use, or desing flows, in terms of per capita water consumption including industrial, commercial and fire requirements.

(4) The area to be served, or desing area$_t$ and the allowance to be made for population density and area water consumption from residential, commercial and industrial districts.

2.11 Design Periods and Water Consumption Data Required :

Design periods are chosen with the following factors in mind :

(1) Useful life of component structures and equipment taking, into proper account of wear and tear.

(2) Ease or difficulty of extending or adding to the existing and planned works, including a consideration of their location.

(3) Anticipated rate of population growth, including possible shifts in community, industrial and commercial development.

(4) Fund available.

(5) Performance of the works during their early years when they will not be loaded to the capacity.

Thus, it is seen that the economical period of design of a structure of a water works is related to its length of life, initial cost, ease and cost of increasing its capacity, and possibility of obsolescence. In connection with the design, water consumption at the end of the design period must also be estimated. Different units of waterworks require different rates of consumption for design.

Design periods and water consumption data required commonly employed in practice are as follows :

(1) Source Structures : The design period will depend upon-the type of source. In case of groundwater, it is easy to drill additional wells, the design period be short, usually 10 to 15 years. If a dam or an impounding reservoir is to be constructed for surface-supply, it is very difficult and costly to enlarge and therefore, the design period is considered to be 30 to 50 years. The design consumption will be the annual average.

(2) Intakes and Transmission Lines from Source to Treatment Plants : The design period will depend upon the length of the life, of the pipes and the type of intake structure used. It is very difficult to enlarge or replace these units, and generally design periods are long, 30 to 50 years. The design consumption will depend upon the amount of storage provided in the city. Generally the average annual consumption is used.

(3) Water Treatment Plants : Design period is usually 20 to 30 years, as additions can generally be made easily. Consumptions required are annual average, maximum daily, and sometimes maximum weekly.

(4) Pumping Plants : The design period is generally in between 10 to 15 years, as additions and alterations can be done easily. Water consumptions needed are maximum hourly including are demand and minimum hourly.

(5) Distribution Systems : The design period is 20 to 30 years as they are easy (comparatively) to extend. Sometimes, distribution systems are designed for indefinite period and the capacity of the system is made adequate for the highest development of the portion of the city it serves. Maximum hourly consumption including fire demand is required.

(6) Overhead Storage Tanks : The design period is generally 50 to 80 years as the construction cost is very high but sometimes they are designed for 30 to 60 years because it is easy to construct additional ones when needed. Consumptions needed are annual average, fire demand, maximum weekly and maximum hourly.

(7) Pipes : For pipes more than 12 inch diameter, the design period is 20 to 30 year as replacement of smaller pipes is more costly in the long run. For pipes less than 12 inch in diameter, the design period is upto full development because requirements may change fast in the limited areas. Maximum hourly rate of water consumption is needed.

QUESTIONS

1. What are some of the principal factors affecting water use ? BUTE, 62, 66, 69, AMIE, 64, 70
2. Discuss briefly how the quantity of water in a water supply scheme for a city is determined pointing out clearly the influence of each-factor contributing to its consumption. BUET, 70, 12.
3. What are the different uses of water ? What is meant by fire demand and how it is computed ? AMIE, 63, BUET, 68, 72, 73.
4. Express in terms of percentage of the average consumption, the usual mximum daily consumption and maximum hourly consumption. BUET, 62, 70, 74.
5. State the methods of making a population estimate. Which one of the methods, do you think best and why? BUET, 64, 66, 70, 72, 73.
6. Explain briefly the types of variations in the rate of water consumption and point out clearly their effects in the design of a water-works, AMIE, 64, 68, RUET, 65, 70, 72.
7. Explain the importance of population figure in the design of a water supply scheme for a city. How is the prediction of population made for a city and how it is utilized ? BUET, 70, AMIE, 68.
8. The average daily consumption per capita in a town of 30,000 population is 45 gallons for domestic purposes, 30 gallons for industrial and commercial purposes, 12 gallons

for public purposes, 25 gallons for loss and waste. Calculate the average daily consumption, neglecting consumption for fires. BUET, 68

Ans. 8,960,000 gals.

9. The population of was 200,000 in 1955 and 233,000 in 1965. Find the probable population in 1990. BUET, 64

Ans. 338,000

10. According to Kuichling, how many fire streams may be called into use at the same time in a city with a population of 64,000 ? BUET, 72.

Ans. 23

11. Supplied below a chart of population growth of cities A, B, C, D and E. Applying graphical comparison method estimate the probable population for the city A in 1995. If the per capita water consumption (average) is estimated to be 100 gallons per day, determine the capacity (in mgd) for which the water-works-for the city A is to be designed. And 10% for fire requirement. BUET, 72

City	Year 1945	1950	1955	1960	1965	1970
	Population					
A	6,000	10,500	14,400	21,600	27,500	45,400
B	16,000	23,000	30,400	45,600	52,200	61,500
C	36.000	46,000	54.300	62,000	71,000	80,000

D	32,500	45,800	50,000	59,400	66,300	74,000
E	21,000	29,800	46,100	49,600	56,500	68,400

12. The population data for a city is supplied below. Calculate the probable population in 1990 by the following methods : (a) Graphical extension and (b) Empirical method. BUET 70

Year	1930	1940	1950	1960	1970
Population (thousands)	2.6	4.8	12.7	18.6	26.0

13. Tabulated below are the census figures for a town in Bangladesh for the period 1930 to 1970; Estimate the population for 2000 by the least square parabola method. BUET 66.

Year : 1930 1940 1950 1960 1970

Population : 25.0 28.0 33.8 55.8 80.2 (thousands)

14. What are the various classes of water uses? Give the normal range and the average water comsumption rate for each of these classes.

15. Calculate the average, maximum and minimum daily consumption of water for a city with a design population of 0-5 million. BUET. 1972.

3
GROUND WATER

3.1 Introduction :

Ground water forms the major portion of earth's fresh water supply about 97% of the earth's fresh water supply is stored in the underground formations. With the increase in population, the demand for water is also increasing throughout the world. Effective management of ground water system is essential to meet the increasing demand for water. Ground water can be used as a reliable source of water supply irrespective of the climate. In the monsoon areas of South-East Asia, ground water may become an important source of supply especially for irrigation purposes in the dry months. An assured supply of water can be obtained from the underground sources even in the desert areas if the system is properly designed and maintained. The New Valley Project of Egypt is a good example of proper use of ground water where surface water was not available. There are numerous such examples. 94% of all the water-works use ground water and they supply 77% of the total water consumed.

Ground water is a vital source of water supply, especially in area where dry summers or extended droughts cause stream flow to slop. Both the surface water and ground water sources are dependent of each other. Many surface streams receive a major portion of their flow from ground water. Elsewhere, water from the surface streams is the main source of recharge for the ground water. The two sources of supply are definitely interrelated, and use of one may affect the water available from the other source. Both surface and ground water problems should be considered together is planning for the development of water supply systems.

Ground water is a vital source of water supply for Bangladesh. Bangladesh is almost entirely underlain by water bearing formations at depths varying from zero to forty feet below the ground surface (Fig. 3.1). Almost entire supply of drinking water and a large part of the irrigation water come from the underground sources. There are about 1,86,000 Govt hand-pump tubewells in the rural areas of Bannladesh. Besides, there are large diameter tubewells in the municipal areas and numerous privately owned tubewells. Moreover, there are many large diameter deep tubewells owned by the Agricultural Development Corporation for irrigation purposes all over Bangladesh.

History of the water wells is as old as the history of ground water iteself. Egyptians were known to have used wells 2100 B.C. Wells are also known to have been used at remote periods in ancient Greece, Italy and India, and artesian wells were sunk in China in very early times. Ancient China possesses the deepest well in the world and it was known to be 1500 ft deep. In ancient Greece, Italy, Egypt and India deep wells as deep as 300 ft were used to collect ground water for various purposes.

3.2 Sources of Ground Water :

The following are the major sources of ground water : Meteoric water, Connate Water and Juvenille Water.

Meteoric Water : This includes rain, sleet, snow, hail, and other forms of precipitation. A total of about 26,000 cubic miles of water falls on the continents each year. It is this water which fills the soil and upper crust of the earth. It is the most important source of water used by man.

Connate Water : This is the sea water or fresh water trapped in sediments when they are deposited on sea bottoms or lakes. Since most sediments originate in sea water, it is usually salty, it is this source which supplies water found deeper in sedimentary strata of the crust. Connate water is often found in rock units with oil. This oil floats on it and rises upward until it is trapped.

Juvenille Water (Magmatic Water) : This water is produced from volcanic and magmatic activity and during the processes of crystallization of rock molecules. It is hard to determine how much of this water is coming to the surface of the earth at present. Many volcanoes are located under water, and many more are found around the margins of the oceans. It may be that a large part of the water gets into the volcanic vents from the ground or oceans. More, than 90 per cent of all materials coming out of volcanoes is steam. It has not yet been possible to estimate how much of this water that has been recycles back into the volcano from the surface.

3.3 Need for Proper Development and Management of Ground Water Resources :

While some ground-water reservoirs are being, replenished year after year by infiltration from precipitation, rivers, canals and so on, other are being replenished to much lesser degrees or not at all. Extraction of water from these latter reservoirs results in the continued depletion or mining of the water.

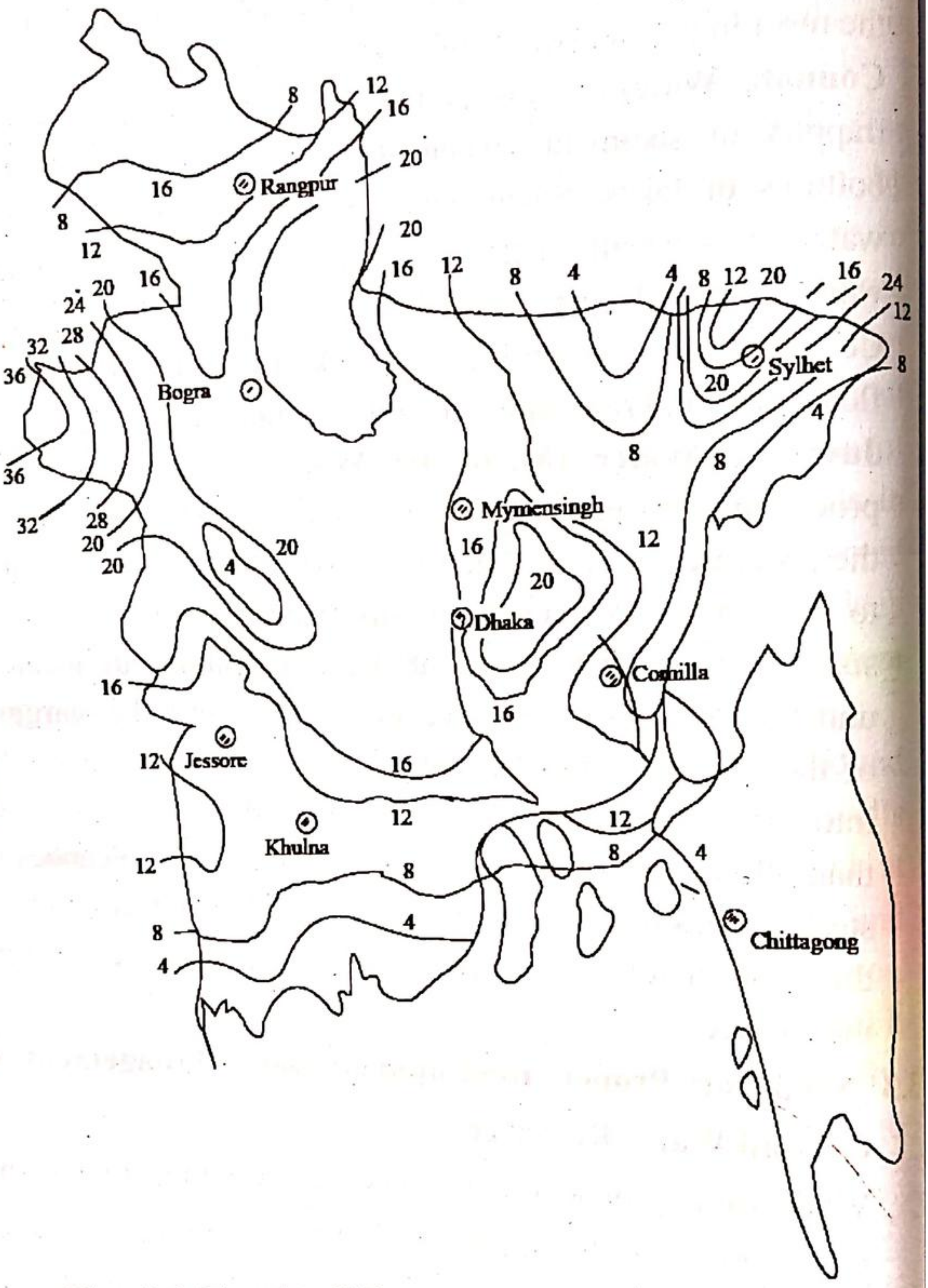


Fig. 3.1 Depth of Groundwater in Bangladesh Measured from Ground Surface (Note: Contour Interval :4 ft) Scale : 1″-48 miles.

Ground water also often seeps into streams, thus providing the low flow (base flow) that is sustained through the driest period of the year. Conversely, if the surface water levels in streams are higher than those in ground-water reservoirs, then seepage takes in the opposite direction, from the streams into the ground water reservoirs. Uncontrolled use of ground water can, therefore, affect the levels of streams and lakes and consequently the uses to which they are normally put.

Ground-water development presents special problems. The lack of solutions to these problems have, in the past, contributed to the mystery that surrounded ground-water development and the limited use to which ground water has been put. The proper development and management of ground-water resources requires a knowledge of the extent of storage, the rates of discharge from and recharge to underground reservoirs, and the use of economical means of extraction. It may be necessary to devise artificial means of recharging these reservoirs where no natural sources exist or to supplement the natural recharge. Research has, in recent years, considerably increased our knowlegde of the processes involved in the origin and movement of ground water and has provided us with better methods of development and conservation of ground-water supplies. Evidence of this increased knowledge is to be found in the greater emphasis being placed on ground-water development.

3.4 Origin, Occurrence and Movement of Ground Water :

An understanding of the processes and factors affecting the origin, occurrence, and movement of ground water is essential to the proper development and use of ground water resources. Of importance in determinating a satisfactory rate of extraction and suitable uses of the water are a knowledge of the quantity of water present, its origin, the direction and rate of movement to its point of discharge, the discharge rate and the rate at which it is being replenished, and the quality of the water. These points are considered in this chapter in as simplified and limited a form as the aims and scope of this book permit.

Hydrologic Cycle. The hydrologic cycle is the name given to the circulation of water in its liquid, vapour or solid state from the oceans to the air, air to land, over the land surface or underground, and back to the oceans (Fig. 3.2).

Evaporation, taking place at the water surface of oceans and other open bodies of water results in the transfer of water vapour to the atmosphere. Under certain conditions, this water vapour condenses to form clouds which subsequently release their moisture as precipitation in the form of rain, hail, sleet, or snow. Precipitation may occur over the oceans returning some of the water directly to them or over land to which winds have previously transpoorated the moistureladen air and clouds. Part of the rain falling to the earth evaporates with immediate return of moisture to the atmosphere. Of the remainder, some, upon reaching the ground surface, wets it and runs off into surface streams finally discharging in the ocean while another part infiltrates into the ground and then percolates to the ground-water flow through which it later reaches the ocean. Evaporation returns some of the water from the wet land-surface to the atmosphere while plants extract some of that portion in the soil through their roots and by a process known as transpiration, return it through their leaves to the atmosphere.

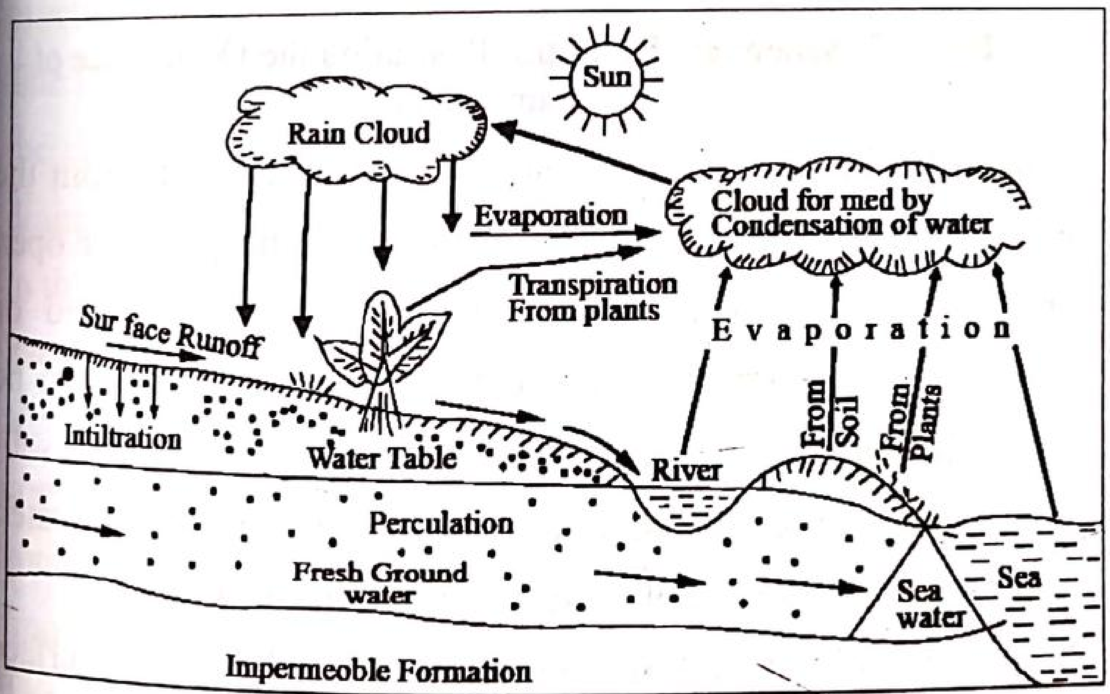


Fig. 3.2 Hydrologic Cycle

Sub-surface Distribution of Water : Sub-surface water found the interstices or pores of rocks may be divided into two main zones. (Fig.3.3). These are the zone of aeration and the zone of saturation.

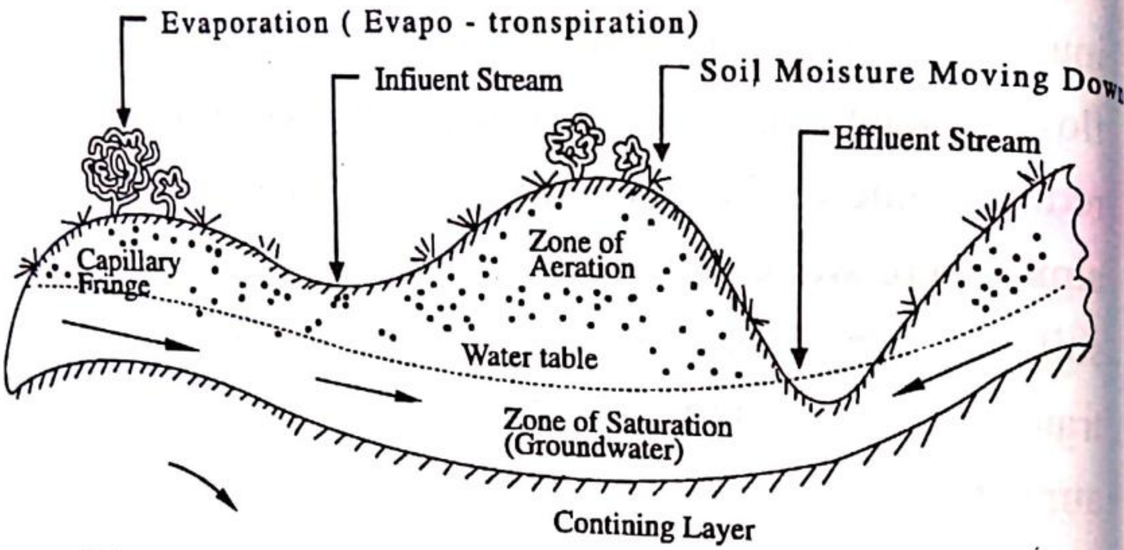


Fig. 3.3 Schematic Diagram Illustrating the Occurrence of Groundwater

Zone of Aeration : The zone of aeration extends from the land surface to the level at which all of the pores or open spaces in the earth's materials are completely filled or saturated with water. A mixture of air and water is to be found in this zone pores and hence its name. It may be sub-divided into three belts. These are (1) the belt of soil water, (2) the intermediate belt and (3) the capillary fringe.

The Belt of Soil Water lies immediately below the surface and is that region from which plants extract, by their roots, the moisture necessary for growth. The thickness of the belt differs greatly with the type of soil and vegetation, ranging form a few feet in grasslands and field crop areas to several feet in forests and lands, supporting deap-rooted plants.

The Capillary Fringe occupies the bottom portion of the one of aeration and lies immediately above the zone of saturation. Its name comes from the fact that the water in this belt is suspended by capillary forces similar to those which cause water to rise in a narrow or capillary tube above the level of the water in a larger Vessel into which the tube has been placed upright. The narrower the tube or the pores, the higher the water rises. Hence, the thickness of the belt depends upon the texture of the rock or soil and may be practically zero where the pores are large.

The Intermediate Belt lies between the belt of soil water and the capillary fringe. Most of its water reaches it by gravity drainage downward through the belt of soil water. The water in this belt is called in intermediate (vadose) water.

Zone of Saturation: Immediately below the zone of aeration lies the zone of saturation in which the pores are completely filled or saturated with water. The water in the zone of saturation is known as Ground Water and is the only from of subsurface water that will flow readily into a well. The object of well construction is to penetrate the earth into this zone with a tube, the bottom section of which has openings which are sized such as to permit the inflow of water from the zone of saturation but to exclude its rock particles. Formations which contain ground water and will readily yield it to wells are called aquifers.

Impervious formations or strata are termed as aquicludes. Aquiclues are hard, compact and cemented rocks such as limestones and related calcareous rocks, shales, sandstones and conglomarates. quarzites, slates, schists, granites and gneiss. They contain very little ground-water.

The amount of ground water which can be obtained in any area depends on the character of the underlying aquifer and the extent and the frequency of recharge. The capacity of an aquifer to contain water is measured by porosity of the formation. Pores vary, in size from microscopic openings in clay and shale to large caverns, fractures, joints, faults and tunnels in limestones and lava. The Table 3.1 indicates the variation in porosity for the more common formation materials.

Table- 3.1 Approximate Average Porosity, Specific Yield, and Permeability of Various Formation Materials.

Materials	Porosity, per cent	Specific Yield per cent	Permeability gpd/sq. ft.
Clay	45	3	1
Sand	35	25	800
Gravel	25	22	15,000
Gravel and Sand	20	16	2,000
Sandstone	15	8	700
Limestone and slate	5	2	1
Quarzite, granite, slate schist, and gneiss		0.5	0.1

A high porosity does not indicate that an aquifer will yield large volumes of water to a well. The only water which can be obtained from the aquifer is that which will flow by gravity.

3.5 Types of Aquifers : Ground-water aquifers may be classified as either water table or artesian aquifers.

A **Water-table aquifer** is one which is not confined by an upper impermeable layer. Hence, it is also called an unconfined aquifer. Water in these aquifers is virtually at atmospheric pressure and the upper surface of the zone of saturation is called the water table (Fig. 3.3). The water table marks the highest level to which water will rise in a well constructed in a water table aquifer. The upper aquifer in Fig. 3.4 is an example of a water table aquifer.

An artesian aquifer is one in which the water is confined under a pressure greater than atmospheric by an overlying, relatively impermeable layer. Hence, such aquifers are also called confined or pressure aquifers. The name artesian owes its origin to Artois, the northernmost province of France, where the first deep wells to tap confined aquifers were known to have been drilled. Unlike water table aquifers, water in artesian aquifers will rise in wells of levels above the bottom of the upper confining layer. This is because of the pressure created by that confining layer and is the distinguishing feature between the two types of aquifers.

The imaginary surface to which water will rise to wells located throughout an artesian aquifer is called the Piezometric Surface. This surface may be either above or below the ground surface at different parts of the same aquifer as is shown in Fig. 3.4. Where the piezometric surface lies above the ground surface, a well tapping the aquifer will flow at ground level and is referred to as a flowing artesian well. Where the piezometic surface lies below the ground surface a non-flowing artesian well results and some means of lifting water, such as a pump, must be provided to obtain water from the well. It is worthy of note here that the earlier usage of the term artesian will reerred only to the flowing type while current usage includes both flowing and non-flowing wells, provided the water level in the well rises above the bottom of the confining layer or the top of the aquifer.

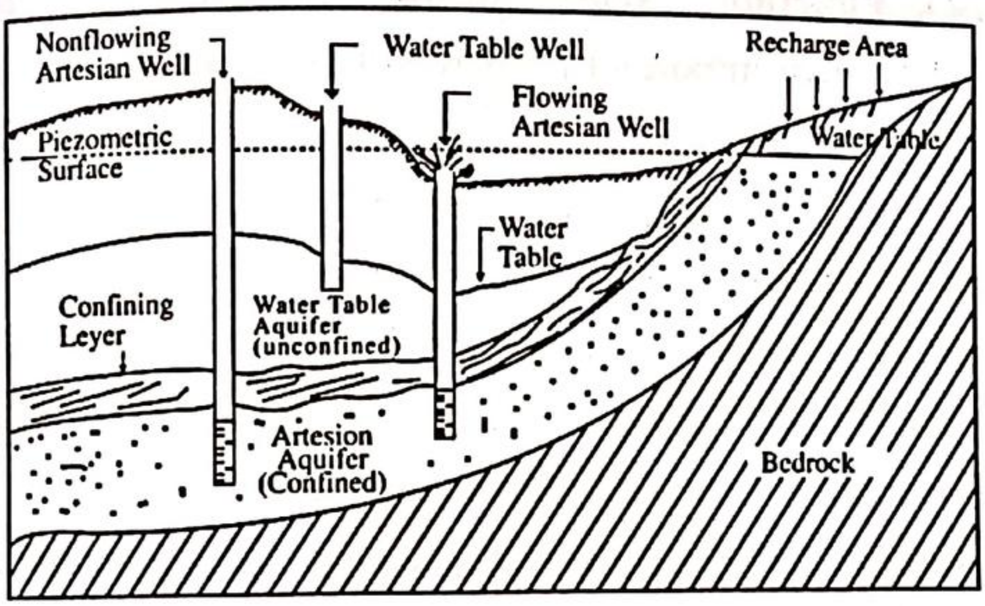


Fig. 3.4 TYPES OF AQUIFERS.

Water usually enters an artesian aquifer in an area where it rises to the ground surface and is exposed (Fig 3.4;. Such exposed area is called a recharge area and the aquifer in that area, being unconfined, would be of the water table type. Artesian aquifers may also receive water underground from leakage through-the confining layers and at intersections with other aquifers, the recharge areas of which are at ground level.

Aquifer Functions : The openings and pores in a water bearing formation may be considered as a network of interconnected pipes through which water flows at very slow rates, seldom more than a few feet per day, from areas of recharge to areas of discharge. This network of pipe, therefore, serves to provide-both storage and flow or conduit functions in an aquifer.

Storage Function : Related to the storage function of an aquifer are two important properties known as porosity and specific yield.

The porosity of a water-bearing formation is that percentage of the total volume of the formation which consists of openings or pores. For example, the porosity of one cubic foot of sand, which contains 0.25 cubic foot of open spaces is 25 per cent. It is, therefore, evident that porosity is an index of the amount of ground water that can be stored in a saturated formation.

The amount of water yeilded by or that may be taken from a saturated formation is less than that which its hold; and is therefore, not represented by the porosity. This quantity is related to the property known as the specific yield and defined as the volume of water released from a unit volume of the aquifer material when allowed to drain freely by gravity. The remaining volume of water not removed by gravity drainage is held by capillary forces such as found in the capillary fringe and by other forces of attraction. It is called the specific retention and like the specific yield, may be expressed as a decimal fraction or percentage. As defined, porosity is, therefore, equal to the amount of the specific yield and the specific retention. An aquifer with a porosity of 0.25 or 25 per cent and a specific yield of 0.10 or 10 percent would, therefore, have a specific retention of 0.15 or 15 per cent. One million cubic feet of such an aquifer would contain 250,000 cubic feet of water of which 100,000 cubic feet would be yielded by gravity drainage.

Conduit function : The property of an aquifer related to its conduit function is known is the permeability.

Permeability is a measure of the capacity of an aquifer to transmit water. It is related to the pressure difference and velocity of flow between two points under laminar or non-turbulent conditions by the following equation known as Darcy's Law (After Henry Darcy, the French Engineer who developed it;.

$$V= \frac{K(h_1 - h_2)}{l} \quad \text{.......} \quad (3.1)$$

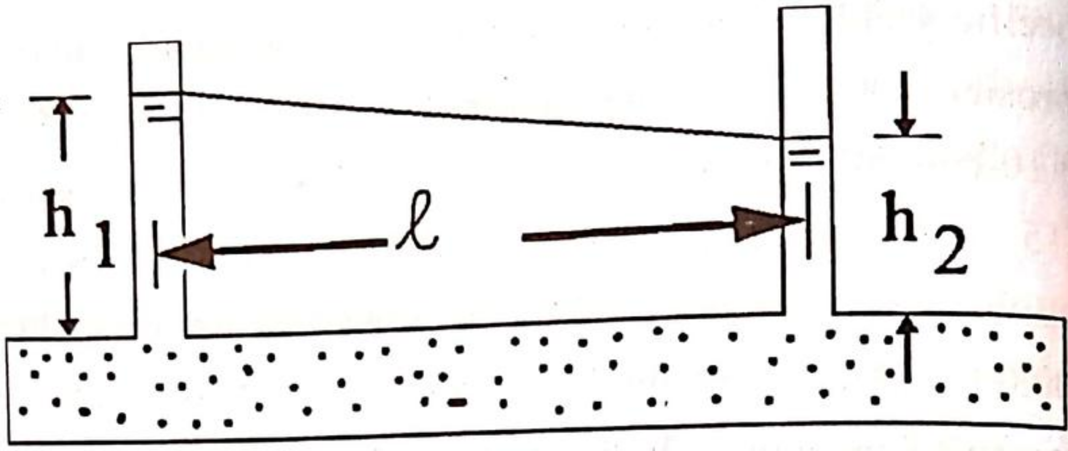
where V is the velocity of flow in feet per day,

h_1 is the pressure at the point of entrance to the section of pipe under consideration in feet of water,

h_2 is the pressure at the point of exit of the same section in feet of water.

l is the lenth of the section of pipe in feet, and

K is a constant known as the coefficient of permeability but often referred to simply as the permeability.



(Fig. 3.5)

Equation 3.1 may be modified to read as

$$V=K_2S$$

where $S = \frac{h_1 - h_2}{1}$ and is called the hydraulic gradient.

The quantity of flow per unit of time through a given cross sectional area may be obtained from Fig. 3.2 by multiplying the velocity of flow by that area. Thus,

$$Q=AV=KAS \qquad (3.3)$$

where Q is the quantity of flow per unit of time and A is the cross sectional area.

Based on Eq. 3.3 the co-efficient of permeability may, therefore be defined as the quantity of water that will flow through a unit cross sectional area of porous material in unit time under a hydraulic gradient of unity (or S = 1.0) at a specified temperature usually taken as 60°F. In ground water problems, Q is usually expressed in gallons per day (gpd), Ain square feet (sq ft.), and K, therefore, in gallons per day per square foot (gpd./sq ft.) The coefficient of permeability can also be expressed in the metric systenf using units of liters per day per square meter under a hydraulic gradient of unity and at a temperature of 15.5=C.

It is importent to note that Darcy's Law in the form shown in Eq. 3.3 states that the quantity of water flowing under laminar or non-turbulent conditions varies in direct proportion to the hydraulic gradient and therefore, the pressure difference (h_1-h_2) causing the flow. This means that doubling the pressure differencs will result in doubling the flow through the same cross-sectional area. By definition. the hydraulic gradient is seen to be equivalent to the slope of the water table for a water table aquifer or of the piezometric surface for an artesian aquifer.

Considering a vertical cross-section of an aquifer of unit width and having a total thickness m, a hydraulic gradient,. S, and an average coefficient of permeability, K, we see from Eq. 3.3 that the rate of flow, q, through this cross section is given by.

$$q=K_mS \qquad (3.4)$$

The product K_m of Eq. 3.4 is termed the coefficient of transmissibility or transmissivity, T, of the aquifer. By further considering that the total width of the aquifer is W, then the rate of flow, Q,. through a vertical cross section of the aquifer is given by.

$$Q=qW=TSW \text{ ....} \qquad (3.5)$$

The coefficient of transmissibility is, therefore, defined as that of flow through a vertical cross-section of an aquifer of unit width and whose height is the total thickness of the aquifer when the hydraulic gradient is unity. It is expressed in gallons per day per foot (gpd/fc) and is equivalent to the product of the coefficient of permeability and the thickness of the aquifer.

Factors Affecting Permebilily: Porosity is an important factor affecting the permeability and, therefore, the capacity of an aquifer for yielding water. This is clearly evident since an aquifer can yeild only a portion of the water that it contains and the higher the porosity, the greater is the volume of water that can be stored. Porosity must, however, be considerd together with other related factors such as particles size arrangement and distribution, continuity of pores, and formation or stratification.

The value of the permeability, K, can also be determined from the laboratory analysis of the soil samples from the aquifer by the following formula :

$$K = Cd^2 \quad \text{...} \quad (3.6)$$

where d effective size of the particles= D

and C = a constant, value varies between 100-120.

The Table 3.2 shows the value of the coefficient of permeability for various effective sizes and porosities.

Spring: A Springs may be regarded as outcrops of ground water which often appear as small water holes at the foot of the hills or along river banks. Springs are generally of two types (1) Gravity Spring and (2) Artesian Spring.

Table 3.2 Co-efficient of permeability

Effective size mm	Porosity, per cent	Co-efficient of gpd/sft	permeability
0.10	25	60	70
	30	130	145
	35	250	275
	40	450	500
0.20	25	240	280
	30	520	580
	35	1,000	1,100
	40	1,500	2,000
0.30	25	540	630
	30	1,170	1,305
	35	2,250	2,475
	40	4,050	4,500
0.40	25	980	1,120
	30	2,080	2.320
	35	4,000	4.400
	40	7,200	8,000
0.50	25	500	1,750
	30	3,250	3,625
	35	6,250	6,875
	40	11,250	12,500

The gravity spring may result either from the out-cropping of an impervious stratum underneath the water bearing formation (Fig. 3.6) or from the overflow of the water table by the continuous rise in ths water table into the sides of the valley (Fig. 3-7)

The yeild of the gravity springs varies with the position of the water table or of rainfall and is, therefore uncertain.

The artesian spring (Fig. 3.8) is one resulting from the water bearing stratum being under pressure, underlain and overlain by impervious strata. Water flows to ihe surface through the weakers spots in the upper impervious strata. either some fault or cervice in the rock.. The yield of the artesian spring is more uniform and almost constant throughout the year.

Hot Springs : Springs that bring warm or hot water to the earth surface are called hot springs, thermal springs or warm spiings. A springs is usually regarded as a thermal or hot springs if the temperature of its water is about 15°F higher than the mean temperature of the air. There are many hot springs all over the world. In Bangladesh, there is a hot spring at Sitakundu in the Chandranath Hill Range, Chittagong. Most of the hot springs derive their heat from masses of magma that have pushed their way into the crust almost to the surface and are now cooling. .In some areas. the circulation of ground water carries it to depths great enough for it to be warmed by the normal increase in earth's interior heat.

Geysers : A geyser (guyzir) is a special type of hot spring that ejects water intermittently with considerable force. The word "geyser" comes from a name of a spring of this type in Iceland geyser, probably based on the verb "geysa" meaning to rush furiously". Geysers result when water accumulates in vertical underground chambers where it is heatced. The pressure of the overlying water causes the boiling point near the bottom to rise The heating also causes the column of water to expand and spill over near the top. This reduces the pressure, and the superheated water at the bottom flashes into steam and causes the geyser to erupt. This process is repeated more or less periodically in a geyser. All ground water contains dissolved minerals and the hot water contains, in general, more, Thus, hot spring and geysers commonly deposit calcite and other minerals.

The Old Faithful Geyser in Yellowstone National Park in America is a very famous one. Since its discovery in 1870 this geyser has regularly spurted forth about 10,000 to 15,000, gallons of steam water and to an average height of 150 ft about once every two hours. The entire display lasts about four minutes. Very few geysers are as regular as the Old Faithful, but many other geysers are known, and the nature of their activity is similar. Outside America, other areas of geyser activity are found in, Iceland, New Zealand and Australia.

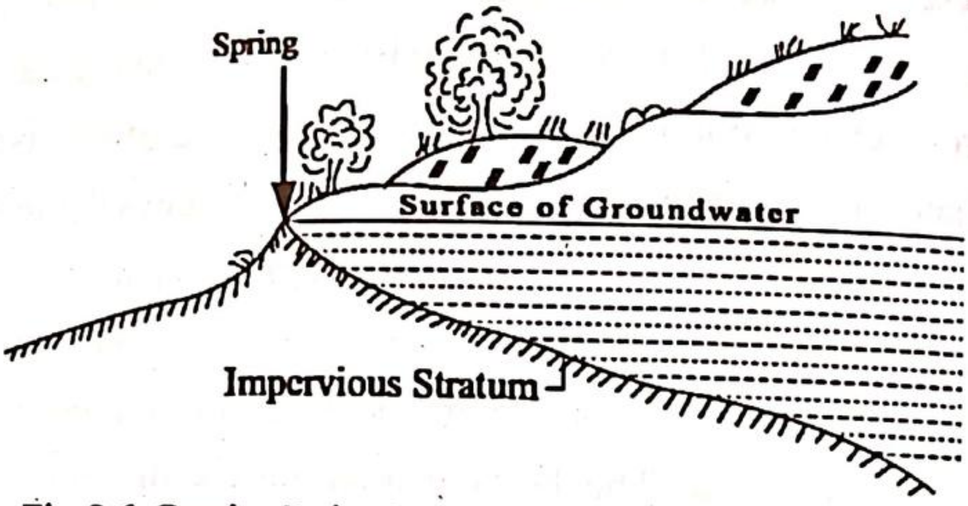


Fig. 3.6 Gravity Spring Resulting from the Outcropping of Impervicus Stratum

Fig. 3.7 Gravity Spring Resulting from the Overflow of Rising Water Table

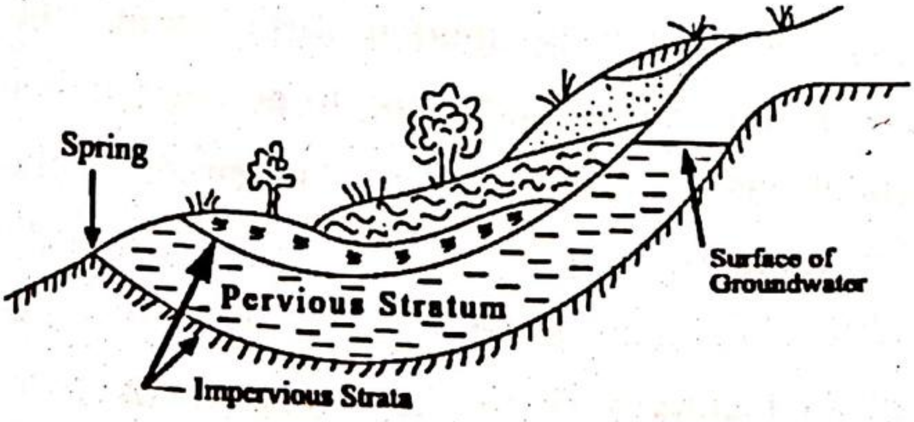


Fig. 3.8 Artesian Spring

Ground Water Geology : The wide variation in texture and stratigraphy of the earth's crust is reflected in the manner of occurrence of both free and confined ground waters. The water table may be several thousand feet down. Ground water may flow through caverns, crevices, and solution passages at velocities comparable to the velocities of turbulent surface streams (1 or more fps), or it may move in laminar flow through the capillary interstices of soils and rocks at velocities of only a few feet a year. Aquifers may be thick and isotropic (possessing the same properties in all directions) as well as homogeneous, or they may consist of a variety of layers, lenses, and tortuous bands of different materials. Detailed acquaintance with the geology of ground water areas is essential to a knowledge of the capacity of water-bearing formations. Surface geology and exposures by mining, quarrying, and related operations must be supplemented by well logs. These are records of the nature and depth of the strata encountered in sinking wells. Combined with measurements of capacity, logs furnish the most important information to be had without the aid of test-well or geophsyical reconnaissance.

Geologically, the earths crust is made up of rocks, and soils. The rocks are igneous, sedimentary, and metamorphic in origin, the soils are derived from the rocks by weathering of rock exposures.

Water-bearing Rocks : The intrusive igneous rocks are dense in texture and would be barren of water. The extrusive igneous rocks may be very porous and contain cracks, boles and extensive caverns. Some lava formations yield water in abundance.

Of the four common varieties of sedimentary rocks (Limestones and related calcareous rocks, shales, sandstones, and conglomerates), the limestones are usually dense and, impervious. However, they are the most soluble of all rocks and where they have been subjected to the leaching action of water containing dissolving carbon dioxide or organic acids, they are honeycombed by solution passages and caverns. Underground streams and lakes are formed in the course of time and these may overflow at the surface to create large springs. Shales, produced by the consolidation of clays, are generally impervious and act as aquicludes. Sandstones, by contrast may be very pervious. Their water-bearing capacity depends upon the extent to which the pores of constituent sand grains are filled with cementing materials. Quartzites, composed of silica sands completely filled with cementing siliceous materials, are like granites in density and imperviousness; loosely cemented sandstones are among the most productive aquifers. The water-bearing capacity of the consolidated or cemented heterogenous mixtures of materials that constiute conglomerates varies considerably. As a rule, they are quite tight. Good aquifers are sometimes encountered in limestones and sandstones at depths in excess of a mile. However most ground water developments are less than 2000 ft deep.

None of the metamorphic rocks is an important water producer. Marble, like the limestone from which it is created, is soluble and may yield water from solution channels. Slates and schists, which originate in shales, are relatively impervious but they transmit some water along joints, cleavage cracks, and fractures. Gneiss resembles, in its structural and water-bearing properties, the intrusive granites from which is generally derived.

Water-bearing soils : Although the water-bearing rocks are important source of water, the areas served by them are small within Bangladesh as a whole. Greater yields of water are derived from the soils of the over burden strata in which free and artesian conditions of flow exist.

Sands and gravels are by far the most important water-bearing soils. They have high specific yields and permeablities and are ordinarily so situated that replenishment is rapid. Uniform or well-sorted sand and gravels are the most productive; mixed materials containing clay are least, for example, boulder clay deposited beneath ice sheets. Transported material is generally more permeable than material in immediate contact with the mother rock, Most sand and gravel beds have been deposited in shallow, active water : (1) In seas, lakes, or river beds as alluvia, (2) at the mouth of canyons as outwash cones, or (3) along, the edge ot retreating ice sheets as outwash plains. Because the origin of these soils and the depth and motion of the transporting water have varied in time, the deposits generally include alternating layers of varying size and grading. Beds deposited is lakes and seas are often extensive; outwash cones or river channels usually contain relatively small lenses of sand and gravel confined between layers of less previous materials.

Clays and silts, although porous, are generally quite impervious. They are poor aquifers and significant only (1) when they confine or interfere with the movement of water through the more pervious soils, and (2) when they supply water to permeable formation by consolidation.

Where rock outcrops at the surface, the rate of water intake is likely to be small, if, on the other hand, the rock is covered by porous and permeable soils, the rate of infiltration is often good, and the overburden becomes, in a sense, a reservoir from which water feeds steadily into the underlying rock. The thicker the water bearing mantle, the greater, in general, is the safe yield from rorks as well as from the soil itself. Topography is also important. The steeper the slope, the more effectively does it shed rainfall and run off from melting snow. Valleys and outwash plains and cones not only accumulate the heaviest overburden but they are, ordinarily, areas of least slops and so in a position to intercept and retain abundant quantities ground water. In Bangladesh, hard rock formation is scare. It is found in some parts of Sylhet (Sunamganj) and Chittagong (Sitakundu and Mirersarai) and in major parts of Chittagong Hill Tracts. The rest of Bangladesh possesses deltaic formation of alluvial deposits. Three mightly rivers the Padma (Ganges), the Brahmaputra and the Meghna with their network of tributaries pass through the deltaic region of Bangladesh filling the lands with alluvial deposits and other unconsolidated materials such as sand and gravel which these river carry with them draining the Himalayan ranges. The thickness of these stratified alluviun in the entire area exeeds 100 ft. The continuous layers, although containing occasional lenss of clay occur at depth varying from 50 to 300 ft. Gravel is frequently marked with fine to medium sand. Coarse sand is found rarely and is thin layers. Within the stratified aquifer, occurence of medium sand is the maximum. The average rainfall in Bangladesh is 98 inches of which 90% of rain falls during 6 months commencing from May to October, inundating all the farmland and sometimes about 90%; of land by flood. The average slope of the country varies about 16 inches per mile north to south in the northern zone and about 3 inches per mile in the rest of the plain areas. The land elevation is about 200 ft in the northern plains and 50 ft in the middle plains and a few feet above mean sea level at the coastal plains.

Having all these topographical and hydrological conditions, the soil strata in Bangladesh contain water more or less everywhere below water table. But all strata do not contain sufficient water that can be pumped or may not give water for a long time. By proper exploration programme, it is possible to get information about the aquifer characterstics and to locate best aquifers in major parts of Bangladesh.

3.7 Groundwater Hydrology :

In the study of a particular groundwater source, surface collection areas and underground conduits and reservoirs must be identified and hydrologic behavior of the system must be discovered. Estimates of safe yield require evaluation of the following factors: (1) the quantities of water added to the formation by infiltration of rain, melting snow and ice, and surface waters, (2) the volume of water stored within the isolated system as measured by the porosity, thickness and areal extent of the water-bearing soil or rock formation, (3) the rate at which the water moves through the ground and can be withdrawn from it, which is a function of its permeability and available hydraulic gradients, and (4) the amount of water lost from the ground by evaporation and transpiration, by effluent seepage into streams and other surface bodies of water by flow from springs and by underground routes of escape. At the same time, the effect of pumping, or other induced with drawal of water from the ground must be taken into account. Withdrawal upets the natural hydrologic and hydraulic balance, water levels fall, directions of movement change, return of water to the surface or to the atmosphere by natural processes may be reduced, and infiltration may be increased.

General Hydrologic Equation : Hydrological equilibrium is expressed by the following equation :

$$\sum R = \sum D + \Delta S \quad \text{............................} \quad (3.7)$$

where $\sum R$ denotes the various hydrological factors of recharge and $\sum D$ those of discharge; ΔS being the associated change in storage volume. More specifically, the recharge is composed of :

1. Natural infiltration derived from rainful.and snowmelt.
2. Infiltration from surface bodies of water.
3. Underflow.
4. Leakage through confining layers or water displaced from them by compression.
5. Water derived from diffusion, charging and water-spreading operation.

Conversly, the discharge includes :

1. Evaporation and transpiration.
2. Seepage into surface bodies of water.
3. Underflow.
4. Leakage through confining layers or absorbed by then through the reduction of compression.

5. Water withdrawn through wells and infiltration galleries. The hydrological inventory of a water-producing area includes, in addition to evaluation of the three terms in Eq. 3.7, a consideration of rainfall and surface run off. The larger the area, the greater are the difficulties of obtaining accurate measurements or close estimates of inventory components.

Recharge and Discharge : When the bulk of the water received by an aquifer is derived from surface streams by infiltration, the progressive reduction in surface flow along the water course is the principal measure of recharge. The intake of rain and melting snow is more difficult to determine. It requires a knowledge of losses by evaporation and transpiration and of the water needed to satisfy the field moisture capacity or specific retention of the soil.

The amount of water entering the ground from diffusion, charging, spreading, and recycling operations is generally a matter of record or can be made so. Discharge of groundwater by evapotranspiration is confined to formations in which the capillary fringe rises to the root zone or the surfae. Discharge of ground, water by seepage sustains the dry weather flow of streams and may be determined from changes in dry weather flow along a stream course. The amount of water withdrown through ground water works is read from records of draft. Underflow and leakage may either recharge or discharge, or both. The difficulty of evaluating the degree of leakage and underflow does not lessen its importance.

Storage : The volume of water within a saturated formation of rock or soil equals its pore space. This is generalized in terms of the porosity. Table 3.1 shows the porosity of various type of water-bearing rocks and soils. Not all the water stored in geological formationg can be withdrawn by nomral engineering operation. Accordingly, there is a difference between total storage and a useful storage. The quantity that will drain off gravitty is called the specific yield, its counterpart is the specific retention. Specific yields vary from zero for plastic clays to 30% or more for uniform sands and gravels. Most aquifers have specific yields of 10 to 20%

The variation in the storage of an artesian basin is generally small. It is sometimes expressed as a storage coefficient or ratio of the volume of water released from the full depth of the aquifer through a unit area of its base when the piezometric surface of the basin drop, drops by a unit of height. The range of values lies between 0.00005 and 0.005. Along with leakage though aquicludes, the relative volumes are small, but the associated absolute magnitudes may be appreciable when areas and pressure difierences are large. Storage coefficients and specific yields become substantially identical when ground-water conditions are free.

Ground storage is relatively large and deficits may be incurred over many years. Ultimately, however, they must be offset by recharge, if the source is not to fail.

3.8 Quality of Ground Water : Generally, the opening through which water flows in the ground are very small. This considerably restricts the rate of flow while at the same time, providing a filtering action against particles in suspension in the water. These properties, it will be seen, considerably afffect the physical, chemical and microbiological qualities of ground water.

Physical Quality : Physically, ground water is generally clear, colourles, with little or no suspended matter, and has a relatively constant temperature. This is attributable to its history of slow percolation through the ground and the resulting effects earlier mentioned. In direct contrast, surface waters are very often turbid and contain considerable quantities of suspended matter, particularly when these waters are found near populous areas. Surface water are also subject to wide variations of temperature. From the physical point of view, ground water is, therofore, more readily usable than surface water, seldom requiring treatment before use. The exceptions are those ground waters which are hydraulically connected to nearby surface waters through large openings such as fissures and solution channels and the interstices of some gravels. These openings may permit suspended, matter to enter into the aquifer. In such, cases, tastes and odours from decaying vegetation may also be noticeable.

Microbiological Quality : Ground waters are generally free from very minute organisms (microbes) which cause diseases and which are normally present in large numbers in surface waters. This is another of the benefits that result from the slow filtering action provided as the water flows through the ground. Also, the lack of oxygen and nutrients in ground water makes it an unfavourable environment for disease producing organisms to grow and multiply. The exceptions to this rule are again provided by the fissures and solution channels found in some consolidated rocks and in those shallow sand and gravel aquifers where water is extracted in close proximity to pollution sources, such as privies and cesspools. This latter problem has been dealt with in more detail at the end of this chapter where the sanitary protection of ground water supplies is discussed, Poor well construction can also result in the contamination of ground waters.

Chemical Quality : The chemical quality of ground water is also considerably influenced by its relatively slow rate of travel through the ground, water has always been one of the best solvents known to man. Its relatively slow rate of percolation through the earth provides more than ample time for many of the minerals that make up the earth's crust to be taken into solution. These minerals have varying rates of solution in water, depending upon a number of conditions which themselves may vary widely within a small region. As a result, there may be appreciably wide variations in the chemical quality of ground water found in regions of relatively limited area extent.

The uses to which ground water can be put depend on its mineral content. Where this content exceeds the recommended limit, treatment should be provided to remove the excessive amounts of the minerals concerned. There are satisfactory methods available for the removal of excessive quantities of the important minerals usually found in ground waters. Expert technical advice should always be sought on the need for and use of these methods.

The mineral content of water is most commonly expressed in parts per million (ppm) which means the number of parts, by weight of the mineral found in one million parts of the solution. For example, a concentration of 10 ppm of iron means that in every million pounds (or kilograms) of the water examined, there will be found 10 pounds (or killograms) of iron.

Another very common form of expression is that of milligrams per liter (mg/l) which is the number of milligrams of the mineral found in one liter of water. This liter unit differs so little from the former that they are for all practical purposes, considered equal and are commonly used interchangeably.

The following are among the more important chemical subs-tances and properties of gound waters which are of great interest to the public health engineers: iron, manganese, chloride, fluoride nitrate, sulphate hardness, total dissolved solids, pH, and dissolved gases such as oxygen, hydrogen sulphide, and carbon dioxide.

Iron and manganese are usually considered together because of their resemblances in chemical behaviour and occurrence in ground water. It is important to note that iron and manganese, in the quantities usually found in ground water, are objectionable because of their nuisance values rather than as a threat to mans health. They both cause staining (reddish brown in the cast of iron and black in the case of manganese) of plumbing futures and clothes during laundering. Iron deposits may accumulate to well screens and pipes, reitricting the flow of water through them. Iron-containing waters also have a characteristic taste which some people find unpleasant. Such waters, when first drawn from a tap or pump, may be clear and colourless, but upon allowing the water to stand, the iron settles out of solution giving a cloudy appearance to the water and later accumulating in the bottom as a rust coloured deposits.

Chlorides occur in very high concentrations in sea water, usually of the order of 20,000 mg/l. Rainwater does not contain chlorides. Aquifers containing large chloride concentrations are usually coastal ones directly connected to the sea or which were so connected sometime in the past. Excessive pumping of wells in aquifers directly connected to the sea or to brackish-water rivers will cause these high chloride, containing waters to move into the otherise fresh water zones of the aquifers. Expert technical advice should be sought on the possibility of such an occurrence.

Water with a high chlorids content usually has an unpleasant taste and may be objectionable for some agricultural purposes. The level at which the taste is noticeable varies from person to person but is generally of the order of 250 mg/l. A great deal depends however, on the extent to which people have been accustomed to using such waters. Animals usually can drink water with much more chloride than humans can tolerate. Cattle have, reportedly, been known to consume water with a chloride content ranging from 3000 mg/l to 4000 mg/l.

Fluoride concentrations in ground water are usually small and mainly derived from the leaching of igneous rocks. Notable among the few cases of concentrations is the reported 12 mg/l from flowing wells in some part of Bangladesh and 32 mg/l in some parts of the world.

When present in concentrations left than 1.0 mg/l in water, fluoride generally reduces tooth decay in small children and is desirable excessive concentrations, however, result in a brown discolouration and pitting of the teeth called dental fluorosis. This condition is particularly noticeable in children but can also occurs in adulrs. The level of concentration at which this adverse effect occurs varies from one community to another depending upon factors such as temperature and fluoride intake to the body through food. It is also likely that continued consumption, of waters containing fluoride in excess of 4 mg/l are usually not recommended for drinking water supplies.

Nitrate content in ground waters various considerably and is often unrelated to the rocks formations in the area. High nitrate concentrations are very often due to the percolation of surface waters containing human wastes and/or animal and other agricultural waste products into aquifers or to the direct flow of contaminated surface runoff into wells. Precautions must therefore be taken in the location and construction of shallow wells in areas where privies and cesspools are to be found.

High concentrations of nitrate in water produce an effect known as cyanosis (methemoglobinemia) in infants. This condition which is characterized by a bluish discolouration of the skin restlessness and drowsiness can be fatal. For this reason, water containing nitrate in excess of 45 mg/l should not be used in preparing food for babies under six months of age. It should be noted that the boiling of such water will only serve to increase the nitrate concentration.

Sulphate in ground water is derived mainly from the leaching of natural deposits of magnesium sulphate or sodium sulphate both of which, in sufficient quantities, may produce laxative effects.

Hardness is that property of water best demonstrated by the readiness with which it dissolves soap to produce suds. No suds are produced in a hard water until the minerals causing the hardness have been removed by chemical combination with constituents of the soap. The greater the hardness, the more soap is required to produce suds.

The hardness produced by the bicarbonate of calcium and magnesium can be virtually removed by boiling the water and is called temporary hardness. The hardness caused mainly by the sulphates and chlorides of calcium and magnesium cannot be removed by boiling and is called permanent hardness. Total hardness is the sum of the temporary and permanent hardness.

The removal of temporary hardness by heat causes the deposition of calcium and magnesium carbonates as a hard scale in kettles, cooking utensils, beating coils, hot water pipes and boiler tubes which result, in a waste of money.

Total dissolved solids refer to the sum total of all the minerals such as chlorides, sulphates etc. found dissolved in the water. A water with a high total dissolved solids content would therefore be expected to present the taste, laxative and other problems associated with the individual mineral. Such waters are usually corrosive to well screens and other parts of the well structure.

PH is a measure of the hydrogen ion concentration in water and indicates whether the water is acid or alkaline. It ranges in value from 0 to 14 with a value of 7 indicating a neutral water, value between 7 and 0 increasingly acid and between 7 and 14 increasingly alkaline waters. Most ground waters in Bangladesh have pH values ranging from about 5.3 to 8. Determination of the pH value is important in the control of corrosion and many processes in water treatment.

Dissolved oxygen content of ground waters is usually low particularly in waters found at great depths. Oxygen speeds up the corrosive attack of water upon iron, steel, galvanized iron, and brass. The corrosive process is also more rapid when the pH is low.

Hydrogen sulphide is recognizable by its characteristic odour of rotten eggs. It is very often found in ground waters which contain iron. In addition to the odour, which is noticeable at as low a concentration as 0.5 mg/l, hydrogen sulphide combines with oxygen to produce a corrosive condition in wells and also combines with iron to form a scale deposit of iron sulphide in pipes. Most of the hydrogen sulphide can be removed from ground water by spraying if into the air or allowing it to cascade in thin layers over a series of trays.

Carbon dioxide enters water in appreciable quantities as the water percolates through soil in which plants are growing. Dissolved in water, it forms carbonic acid which, together with the carbonates and bicarbonates, controls the pH value of most ground waters. A reduction of pressure, such as caused by the pumping of a well, results in the escape of carbon dioxide and an increase in the pH value of the water. Testing of ground-water samples for carbon dioxide content and pH therefore, requires the use of special techniques and should be done at the well site. The escape of carbon dioxide from waters may also be accompanied by the settling out of calcium carbonate deposits.

While the above list includes those chemical substances that are likely to be of greatest general concern in case of small wells, it is by no means an exhaustive one nor intended to be such. Conditions peculiar to specific areas may require analyses of ground waters for other substances. The group of elements often referred to as the trace elements because of the very low concentrations in which they are usually found in waters are here worth mentioning. Among these are arsenic, barium, cadmium, chromium, lead and selenium, all of which are considered toxic to man at very low levels of intake the order of a fraction of 1 mg/L. Since the rare of passage of some of these elements through the body is very slow, the effects of repeated doses are additive and chronic poisoning occurs.

Trace elements generally are not present in objectionable concentrations in ground waters but may be so in a few specific areas. It has been reported for example, that arsenic has been found in sufficiently high concentrations in ground waters in some parts of the world (Mexico and Argentina) to be considered injurious to health. Problems are most likely to arise in areas where waste discharges from industries such as electroplating, and overland run-off containing high concentrations of pesticides (insecticides and herbicides) enter aquifers.

The presence of these tracs elements in drinking water are generally not detectable by taste or smell or physical appearance of the water. Proper chemical analyses are required for their detection.

In some parts of Bangladesh, ground water sometimes contains excessive amount of iron, copper, manganese, chloride. fluoride, calcium and magaesium. These make water unpotable. In coastal areas of Bangladesh, potable water is obtained from wells at depths between 700 to 1000 ft. Above these depths, waters from all aquifers are extremely saline (about 2500 to 3000 mg/l; ground water contains iron in Perojpur, Gopalgonj, Rajbari, Tangail, Rangpur, Maijdi, Chittagong and Chiltagong Hill-Tracts sub-divisions. Iron is excessive only in shallow wells of Bhola and Patuakhali. Again ground water contains carbonate in Rajshahi, Sylhet and Chittagong more than allowable limits Ground water in all other areas of Bangladesh have got acidic characteristic.

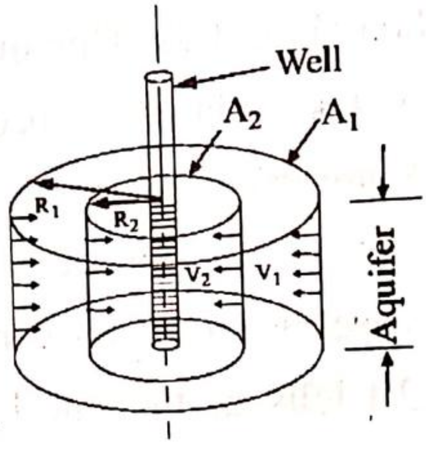
3.9 Well Hydraulics :

Groundwater movements towards a well can be formulated in accordance will the principles of Dupuit and Forchheimer. When the well penetrates to the whole of the aquifer, the flow is steady, and the water table or the piezometric surface, as the case may be, horizontal. Actually the water table is rarely horizontal and the flow is seldom steady. Changes in pumping and recharge rates and in amounts of water stored in the aquifer interfere. However, the usefulness of Dupuit's formulation can be expanded by introducing potential flow theory to cover confined aquifers in which the piezometric surface is inclined. The theory of steady flow has also been extended to leaky confining becomes situations.

Flow Towards Wells

Converging flow : When a well is at rest, that is, when there is no flow taking place from it the water pressure within the well is the same as that in the formation outside the well. The level at which water stands within the well is known as the static water level. This level coincides with the water table for a water-table aquifer or the piezometric surface for an artesian aquifer. Should the pressure be lowered within the well, by a pump for example, then the greater pressure in the aquifer on the outside of the well would force water into the well and flow thereby results. This lowering of the pressure within the well is also accompained by a lowering of the water level in and around the well-water flows through the aquifer to the well from all directions in what is known as converging flow. This flow may be considered to take place through successive cylindrical sections which become smaller and smaller as the well is approached (Fig. 3.9). This means that the area across which the flow takes place also becomes sucessively smaller as the well is approached. With the same quantity of water flowing accross these sections, it follows from Eq 3.3 that the velocity increases as the area becomes smaller.

Darcy's Law (Eq.3.2.) tells us that the hydraulic gradient varies in direct proportion of the velocity. The increasing velocity towards the well is, therefore accompained by an increasing hydraulic gradient. Stated in other terms, the water surface or the piezometric surface develops an increasingly steeper slope towards the well. In an aquifer of uniform shape and texture the dispression of the water table or piezometric surface in the vicinity of a pumped or freely flowing well takes the form of an inverted cone. This cone, known as the cone of depression (Fig.3.10), has it apex at the water level in the well during pumping is known as the pumping water level. The difference in levels between the static water level and the surface of the cone of depression is known as the drawdown. Drawdown, therefore, increases from zero at the outer limits of the cone of depression to a maximun in the pumped well. The radius of influence is the distance from the center of the well to the outer limit of the cone of depression;



$$R_1 = 2R_2 \qquad A_1 = 2A_2$$

$$V_2 = 2V_1$$

Fig. 3.9 Flow converges towards a Well, passing through Imaginary cylindrical surface that are Successively smallar as the well is approached

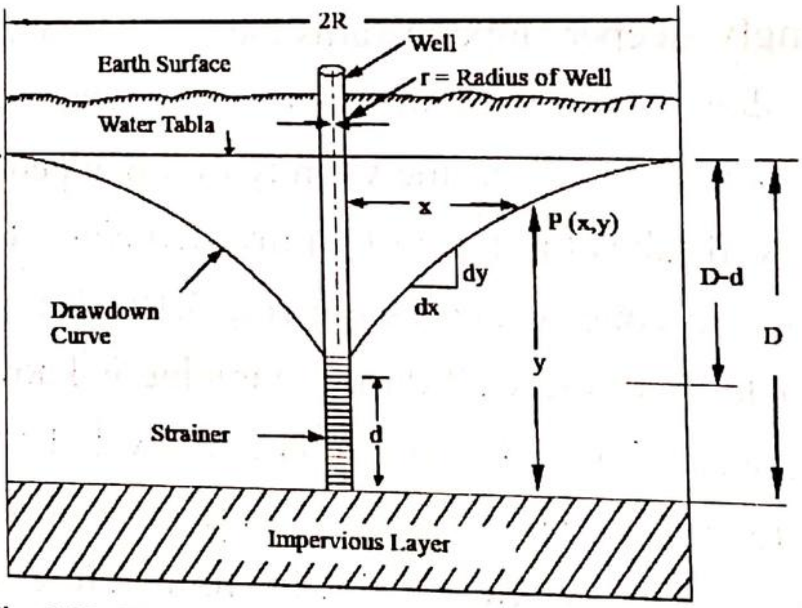


Fig. 3.10 Hydraulics of flow for a Well through an uncon fined aquifer.

Uoconfined Steady Flow : Dupuit's formula for the flow *into* an ordinary well is based on the Eq.3.3. It is assumed that the direction of flow of ground water Is horizontal and that it flows radially towards the centre of the wall shown in Fig 3.10. Hence, it flows through the surface of a right. vertical cylinder so that

$A = 2\pi xy$ and $S = dy/dx$

$Q = KAS = K2\pi xy\ (dy/dx)$ ....(3.8)

or $Q\dfrac{dx}{x} = 2\pi Ky\ dy$

By integration

$Q = \log_e x = \pi K y^2 + C$ .... (3.9)

where C is a constant.

For y = d at x = r (r being the radius of the well) and

y = D at x = R (R being the radius of circle of influence or distance of the outer boundary from the centre of the well) the Eq. 3.9 becomes

$$Q = \frac{\pi K(D^2 - d^2)}{\log_e (R/r)}$$ .... (3.10)

where, Q = well discharge in gpd.

K = coefiicient of permeability in gpd/sft

D= depth of the aquifer in ft.

d = static head in ft.

R = radius of the circle of influence in ft.

r = radius of well in ft.

Beacuse D is constant, the quantity, $(D^2 - d^2)$ increases at a declining rate as d is reduced. Thus, successive increases in draw-down, (D–d) reduce the specific capacity of the well. For a constant value of R, the logarithmic ratio of the radius of the circle of influence to the radius of the well and its inverse relation to the yield indicate that increasing the size of a well does not greatly increase its yield. For example, a 2 ft diameter well will yield only 15 to 30 per cent more water than a 3 inch diameter well.

The rate of flow Q from a well, the drawdown (D–d), and the radius of the circle of influence R are interrelated as indicated in Eq. 3.10. In order that the value of R may be constant the rate of replenishment of underground water must be equal to Q. Hence, for the purpose of approximate computation in the solution of well problems with Dupuit's formula it be assumed that

$$R = CQ \quad \text{.....................................} \quad (3.11)$$

or in substitution in Eq.3.11,

$$Q = \frac{\pi K(D^2 - d)^2}{(\log_e(CQ/r)} \quad \text{...........................} \quad (3.12)$$

Values of C and K for a well can be determined by measurements of Q and of d under two or more different rates of flow. It is evident from Eq. 3.12 that where the drawdown (D–d) is small compared with (D+d) the value of Q varies approximately as (D–d). This linear relationship between the rate of flow and drawdown leads to the definition of the specific capacity of a well as the rate of flow per unit of drawdown, usually expressed in gallons per minute per foot of drawdown.

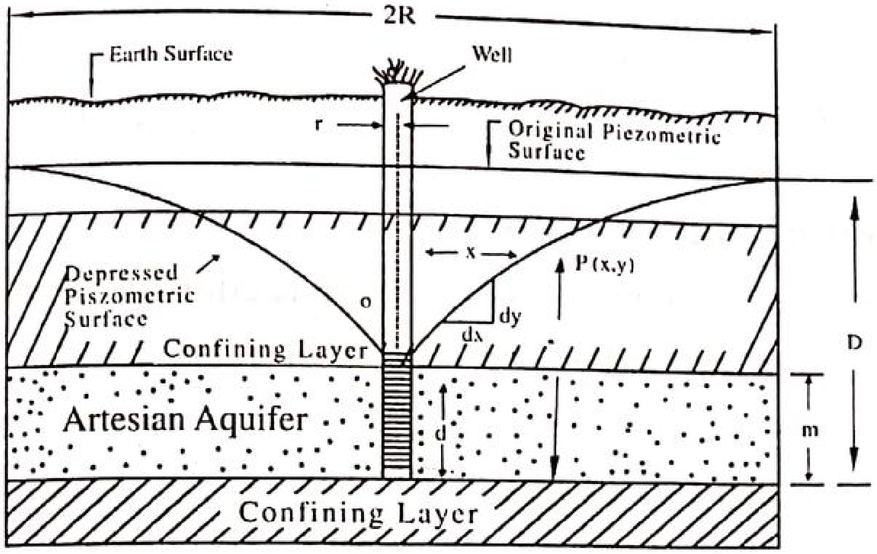


Fig. 3.11 Hydraulics of flow for a Well through an Artesian aquifer.

Confined Steady Flow: The Hypothetical conditions of flow into a pressure or artesian well are illustrated in Fig 3.11. The formula for the rate of flow into a pressure well can be derived similarly to Eq.-3 10, starting with the expression Q = KAS and A =2πxm, hence

$Q = K2\pi xm\,(dy/dx)$ .... (3.13)

where m is the thickness of the confined water bearing stratum.

Integrating between the limits x = r for y = d and x = R for y = D, the Eq; 3.13 becomes

$$Q=\frac{2\pi Km(D-d)}{\log_e(R/r)} \quad \text{.......................................} \quad (3.14)$$

The rate of flow Q is seen to be proportional to the draw-down (D-d) = h. The yield per unit drawdown, or specific capacity of artesian wells has been observed to remain fairly constant at all reasonable values of drawdown. The Eq. 3.14 may be rewritten as follows :

$$h = \frac{Q \log_e(R/r)}{2\pi K m} \quad \text{.......................................} \quad (3.15)$$

3.10 Interference of Wells :

Under some conditions the construction of a single large well may be either impractical or very costly while the installation of a group of small wells may be readily and economically accomplished. Factors such as the inaccessibility of the area to the heavy equipment required for drilling the large well and the high cost of transporting large diameter pipes to the site may be among the important considerations in a situation such as this small wells can be grouped in a proper pattern to give the equivalent performance of a much larger single well.

The grouping of wells however, presents problems due to interference among them when operating simultaneously. Interference-between two or more wells occurs when their cones of depression overlap, thus reducing the yield of the individual well. The drawdown at any point on the composite cone of depression is equal to the sum of the drawdowns at that point due to each of the wells being pumped separately. In particular, the drawdown for a specific discharge in a well affected by interference is greater than the unaffected value by the amount of drawdown at that well contributed by the interfering wells.

In other words, the discharge per unit of drawdown commonly called the specific capacity of the well is reducesd. This means that pumping must take place from a greater depth in the well, at a greater cost, to produce the same quantity of water from the well if it were not subject to interference.

Ideally, the solution would be to space the wells far enough apart to avoid the mutual interference of one on the other. Very often this is not practical for economic reasons and the wells are spaced far enough apart, not to eliminate interference, but to reduce it to acceptable proportions. ' For wells used for water supply purposes, spacings of 250 to 500 feet between wells have been found to be satisfactory. Spacing may be less in fine sand formations, in thin aquifers or when the drawdown is not likely to exceed 5 feet. Greater spacings may be used where the depth and thickness of the aquifer are such as to permit the use of screen lengths in excess of 10 feet. For two similar wells drawing water from the same aquifer situated at weft apart, their rate of discharge shall be

$$Q_1=Q_2=\frac{K(D^2-d^2)}{\log_e(R^2/rw)} \quad \text{.........................} \quad (3.16)$$

Exemple-I : With a well of 12 inch in diameter having a depth of 100 ft below the level of water table, the depth of water when the well is being pumped is 80 ft. As indicated by the test on a sample the effective size of the soil in the water bearing stratum is 0.30 mm and the porosity is 30% and the corresponding coefficient of permeability is 1260 gpd/sft. The radius of drawdown is assumed to be 1000 ft. What is the probable state of discharge of the well in gpm?

Solution : Applying Eq. 3.10

$$\mathbf{Q} = \frac{\pi \times 1260}{2.3026} \times \frac{(100^2 - 80^2)}{\log \frac{1,000}{0.5}}$$

$$= 1,875 \text{ gpd} = \frac{1,875}{24 \times 60}$$

$$= 1300 \text{ gpm}$$

Example 2 : At what rate could water be pumped from the two wells 24 inch diamter each, static depth of water, 200 ft and drawdown 30ft. if both wells were being pumped together at a distance of 24 ft ?

Assume : R = 10 25 ft, K=0.099 gpm sft

Solution: Applying Eq. 3.16

$$Q_1 = Q_2 = \frac{(0.099)(200^2 - 170^2)}{\log_{10}(1025^2 / (1 \times 24))} = 240 gpm$$

Combined rate of flow from two wells = 2× 240 = 480 gpm

3.11 Infiltration Gallery:

An infiltration gallery may be an open trench, a buried porous pipe, or a line of wells closely spaced, placed across or normal to the direction of underground flow in an aquifer. The infiltration galleries are suitably constructed (1) as marginal drains along hillsides, (b) at right angles to the underflow of valleys, (3) parallel to streams towards which upland flow is travelling and (4) above sea- level on islands and along coasts where salt water intrusion is to be aovided. Diameters 2 to 5 ft and manholes at intervals of 200 to 300 ft facilitate inspection and maintenance. Design velocities seldom exceed 2 ft/sec. Water entering a gallery flows to a - collection pump where it is pumped for use. Yields from infiltration galleries vary widely depending upon local coditions; However, infiltration rates of 700 to 3500 gpm. per 100 ft of gallery length are not unusual.

Well Design: A tubewell is simply a pipe sunk into the ground fitted with a strainer in the water bearing strata. The design of a tubewell is the design of its screen principally. For the design of the screen, the following information is necessary: (1) Geological formation of the underground strata, and (2) Grain size analysis results of the aquifer.

The soil samples collected by test boring are analysed by-passing through a series of stand and sieves from 0.01 inch to 1 inch and from the grain size distribution curve, the effective size and the uniformity coefficient are calculated to determine the screen slot openings. Generally, well screens are designed to retain 30 to 50% of the formation materials depending upon the aquifer conditions. For protection of the screen against corrosion and incrustation, the entrance velocity is generally considered as 0.1 fps. The entrance velocity is determined by dividing the desired yield of the well in cfs by the total area of the screen openings-in sft.

The total area of screen openings is the area of the openings provided per foot of the screen multiplied by the selected length of screen in ft. Most manufacturers provide tables showing the open area per foot of screen for each size of screen diameter and for various widths of slot openings. Table 3.3 is an example of one of these. From this table, it is seen that a No. 20 slot 2-inch diameter screen contains 25 square inch of open area per foot of screen length. A 10 ft length of such a screen would therefore contain 250 sq inch of total open area.

There are mainly two sizes of screens : (1) Telescope Size (TS),. and (2) Pipe Size (PS) or ID-Size. The telescope size screens are desinged for cases where the diamaeter of the screen is just sufficiently smaller than the inside diameter of the corresponding size of standared pipe to permit the screen to be freely lowered through the pipe. The pipe size or ID-size screens have the same inside diameter as the corresponding size of the standard pipe.

This type of screen is used when it is desired to maintain the same diameter throughout the full depth of the well.

Table : 3-3 Intake areas of screen openings for selected widths of slot openings.

Nominal Screen size (ID) inch	Intake areas in sq inch per linear foot of screen			
	Slot No. 10 (0.01″)	Slot No. 20 (0.02″)	Slot No. 40 (0.04″)	Slot No. 60 (0.06″)
$1^1/_4$–TS	10	16	26	32
$1^1/_2$–PS	13	22	36	45
2–PS	14	25	41	50
3–TS	15	26	42	52
$2^1/_2$–PS	17	30	48	59
3–PS	20	34	54	68
4–TS	21	35	50	71
4–PS	23	44	68	86

Example : Design the screen of a well from the following data:

Thickness of the aquifer	= 60 ft
Slot size of the screen	= 20
Discharge	= 334 gpm
Entrance Velocity	= 0.1 fps
Well diameter	= 2 inch

Solution:

$Q = AV = 334$ gpm $= 0.74$ cfs.

$\therefore A = \frac{0.74}{0.1} = 7.4 sft = 1066 sq$ inch.

From Table 3.3 open area for 2"-PS screen size and Slot No.20. the open area per foot length of *2* inch dia well **is** 25 sq inch.

Length of screen =1066/25 = 42.64 ft, say 43 ft

3.13 Groondwater Collection :

Discovery and development of groundwater supplies is a very complicated engineering undertaking. It requires understanding of pertinent geological, hydrological and hydraulic actors. The purpose of this section is to discuss the essential constructional features of groundwater developments, the hydraulics associated with the draft of groundwater, and the available means for the maintenance and care of collecting woks (tubewell) and for the conservation of the source.

Common Features of Collection Works : Pumping is the central feature of the most groundwater works. For satisfactoy performance the suction lift, including entrance and pipe losses, must be held below 25 ft. When the water table is further down than the collective pipe leading to the pump and the pumping unit itself must be lowered below ground level, or individual well must be equipped with deep-well pumps.

Well-sinking is a specialized art that has evolved along a number of more or less regional lines. The engineer must give his due attention to the drilling operations as to the adequacy suitability and economy of the proposed developments and the location of the collection works. In addition, he is called on (1) to select the size, number and arrangement of wells, (2) to specify the pumping and appurtenant equipment, (3) to make sure that a reliable contractor is employed, (4) to supervise the testing and development of completed wells, (5) to see that the wells and pipes are properly disinfected before being placed in service, and (6) to assure the prevention of contamination of the operating supply from both surface and underground sources of pollution.

The size, number, and arrangement of wells are determined by the amount and depth of water to be taped, the hydrology and hydraulics of available aquifers, and proposed methods of pumping. Well diameters should be dimensioned in concordance with drawdown and yield. However, well-sinking methods and space requirements for pumping machinery actually govern well size more often than do hydraulic considerations, Strainers or screens and riser pipes should be large enough to keep entrance looses and other sow resistances within reasonable limits at maximum pumping rates. Riser pipe velocities are commonly held down to 2 to 3 ft/sec.

Construction methods depend primarily on the nature of the soil or rock to be penetrated. Costs of construction vary with size, depth and design, and with the equipment and experience of local drillers. Efficient design and operation of well systems are question of depth, spacing, diameter, pumping rate, and other pertinent matters, as well as the geology, hydrology and hydraulics of the sources

In general, tubewells are of two types is shallow tubewells and deep tubewells.

Shallow tubewells are those which are sunk upto the bottom of the first aquifer and do not penetrate the aquiclude below. Deep tubewells are those which cross more than one aquifers. Therefore, the shallow tubewells of one locality might be the deep tubewells of another and vice-versa. Of course, conventionally tubewells under 100ft deep are classed as shallow tube-wells. According to the convention of the Directorate of Public Health Engineering (Bangladesh), shallow tubewells are less than 200 ft deep and deep tubewells are more than 200 ft deep.

Tubewells are extensively used for urban and rural water supplies, industrial water supplies and for agricultural water supplies, Public water supply tubewells varies in size. generally $1^1/_4''$, $1^1/_2''$, $2''$, $3''$ to $6''$ diameter. But irrigation tubewells are generally made7″ to 10″ diameter.

Part of Municipal Tubewell : The following are the main parts of a municipal tubewell (Fig.3.12)

Sump pipe : The small length of plain pipe generally 4' to 5' in length used at the bottom of lowest stainer is called

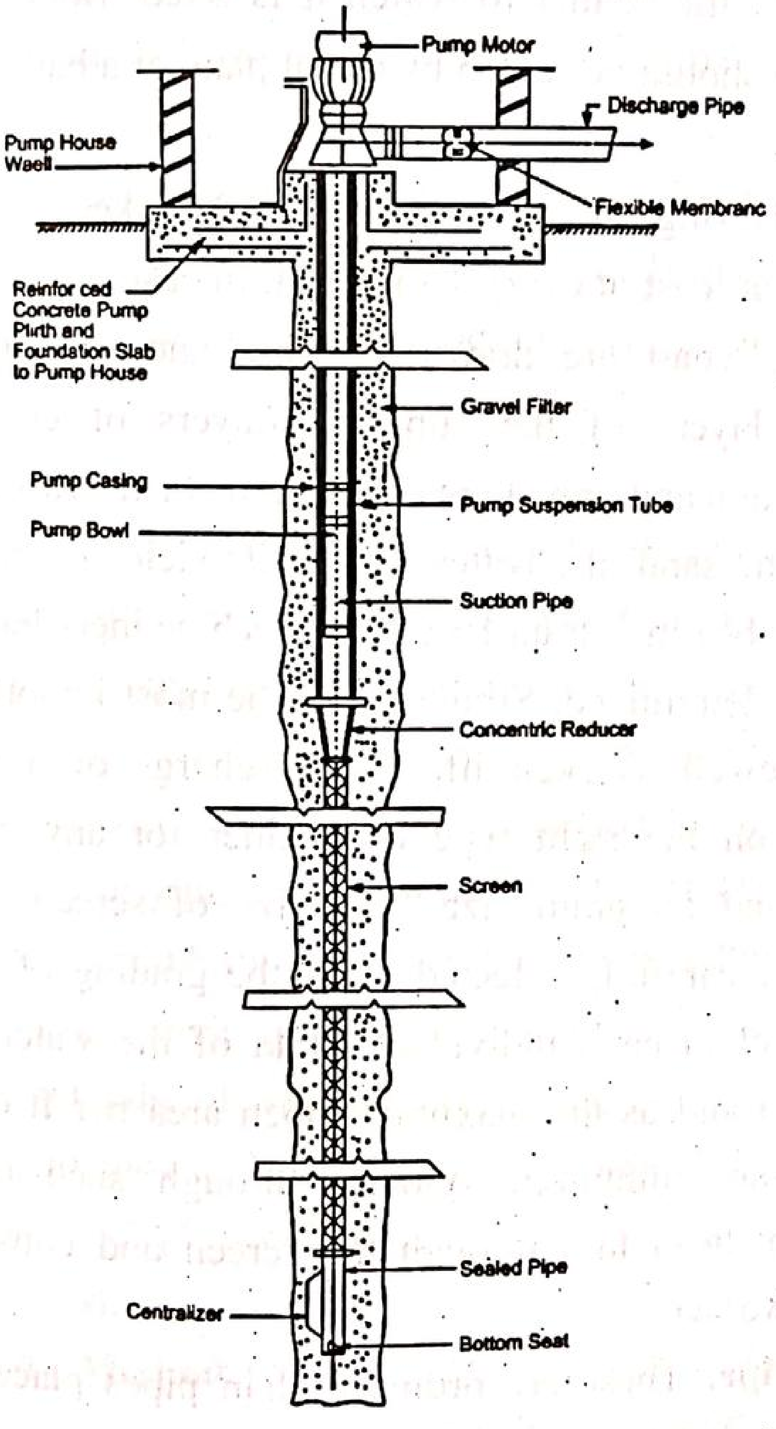


Fig. 3.12 Deep Tubewell

Sump pipe: The object of this pipe is to provide a space where sand and other heavy particles entering the well during pumping may settle down. Thickness of the pipe should be $^{1}/_{4}''$ minimum and its diameter is equal to the diameter of the stainer to which it is fixed. Bottom of the sump pipe should be sealed by a seal plate or a bail plug.

Stainer: During boring, soil samples are taken from every layer and at least at every 10 ft. (5 ft. in case of test boring) and strata chart are drawn showing and describing the different layers of the sub-soil. Layers of coarse and medium sand and gravel are considered water bearing strata. Coarser the sand, the better will be its yield. As certaining the water bearing strata levels at which stainers have to be fixed are determined. Strainer play the most important part in a tubewell as such life and discharge of a tubewell depends on the right type of strainer for any particular aquifer and its grain size. The size of screen openings should be carefully selected to fit the grading of the sand and gravel in each individual strata of the water bearing formation and as the maximum open area per ft of screen will ensure minimum velocity through such openings. minimum head loss through the screen and consequently less drawdown.

Blind Pipe: These are ordinary plain pipes placed in the non-water bearing strata. These are placed opposite to strata of fine sand and clayey soil. The thickness of blind pipes is $^1/_4''$. The pipes and strainers are connected by means of thread or sometimes welded.

Housing Pipe : The larger diameter plain pipe on top in which the pump bowels are housed is known as bousing pipe. The internal diameter of housing pipe depends upon the adopted pump and may vary between 10″ to 15″. The thickness of housing pipes is $^1/_4''$ inch. The housing pipe and the blind pipe are joined together by means of a tapered reducing socket.

Methods of Tubewell Sinking : Cities using ground water usually depend upon deep wells. These wells have the advantage of tapping deep and extensive aquifers. Such deep water is likely to be of good sanitary quality unless it is contaminated by seepage into the aquifer from caverns or fissures in overlying rocks. All deep wells are drilled wells. The successful sinking of deep drilled wells requires special training, experience, tools, and equipment. Among the various methods of sinking wells may be included (1) Standard method, (2) California stovepipe method. (3) Jetting method (4) Core-drill method, and (5) Hydraulic rotary method.

Standard Method : The standard method involves percussion drilling in which a drill is alternately raised and dropped in the descending borehole. It is suited to the drilling of wells in any material from soft clay to the hardest rock. Modifications of the standard method include the "pile-tool" method in which a wooden rods are used instead of rope and the "hollow rod" method in which a hollow rod replaces the rope to support the boring bit. Water is pumped into the top of the well between the casing and the hollow rod. The water leaves at a relatively high velocity through holes in the bit and rises through the hollow rod carrying the cuttings with it and permitting continuous advance of the well without the withdrawal of the bit to remove cuttings.

California Stovepipe Method: First used in California, this method is used in unconsolidated alluvial deposits. It consists of pushing by means of hydraulic jacks, steel cylinders with wall thickness of 0.1 to 0.16 inch and 2 to 4 ft long into the earth. Cylinders of two diameters are used, one size just slipping within the other, the joints of the outer cylinder falling midway between those of inner cylinder. The inner and outer casings are locked together by denting them with a pick, and successive length of pipes are added and sunk. As sinking progresses, the casing is kept even or ahead of excavation. the meterial inside the casing being removed by use of a bailer or clamshell bucket. The California stovepipe method has been used on wells 6 to 36 inch in diameter and upto 200 ft deep.

Jetting Method : Jetting is accomplished by means of a drill pipe having a nozzle or a drill bit on the lower end. Water is pumped down through the drill pipe and escapes through the drill bit, which is raised, lowered and turned slowly. The stream of water loosens material and the water rising through the casing lifts the finely divided cuttings from the well. This method of drilling is suitable in soft, unconsolidated alluvial deposits, depths upto 400 to 500 ft. In suitable materials, a well can be sunk by this method faster than by any other method.

Core-Drill Method : Core drills consist of a hollow but armed with diamond or steel teeth on the annualar circumference of the bit. The hollow bit is attached to a drilling rod and rotated, water being used to remove the cuttings.

Rotary Drilling Method : In rotary drilling, a cutting bit is attached to a hollow drill rod rotated rapidly by an engine driven rotary table. Either water or a suspension of colloidal clay is pumped down the drill pipe, flows through openings in the bit, and transports the loosened materials to the surface. The clay suspensions are designed to reduce loss of drilling fluid into permeable formations, lubricate the rotating drill pipe, bind the wall against caving, and suspend the cuttings.

Installation of Tubewells : After the boring of the well has been completed to the desired depth, the blind pipes and strainers are to be loward and fixed to the position. The bore hole is always to be drilled a little more than the installed depth of a well, so as to make allowance of materials which may cave in before shrouding of the well. Strainers are to be located opposite water bearing strata to draw water from them. This is done by assembling on the ground together with the whole length of the strainers and plain pipes exactly in the same order and length in which they are to be lowered in the bore. Every pipe, socket and strainer is numbered serially. There is a length of about four to five feet of plain pipe at the bottom with a cap to close the bottom end. The cap has an eye on the inside with which a hook lowered from the top, inside the tube, can be made to contact whenever necessary. If a hook tied to a wire rope is lowered into a tubewell from ground level it can easily be made to contact this eye at its bottom end, and the whole tubewell can thus be lifted with the wire rope and kept suspended by it. This method can be used for extracting a newly lowered tubewell where necessary.

The strainers and plain pipes are lowered inside the boring tube one by one as required starting from the bottom end and in the same order in which they have been assembled at ground level. This is done by fixing a pair of clamps at the top of the end piece and then lowering it inside the boring tube till it rests on it. The next piece is then screwed or welded on top of the bottom piece and a second set of clamp is fixed at the top end of the upper piece. The upper clamp is tied to the wire rope going round the pulley at the appex of a tripod. The other end-of this rope is coiled round a crab winch. The ropea is tightened up till it takes the whole weight of the pipes to which the clamp is fitted and raises these a few inches. This lifts the lower clamp from the top of the boring tube and enable it to be released. The lower clamp is then removed and wire rope is gradually blackened and the tubes lowered into the bore till the upper clamp rests on top of the boring tube. The wire rope is now removed and the next pipe is screwed on top of the lowered pipe and the whole operation repeated as before. Thus, one by one the whole series of pipes and strainers are lowered inside the boring tubes to the correct level. The whole finished length of these pipes and strainers is now kept suspended from the top and the boring tubes are jacked up and extracted one by one. Before the boring tubes are extracted, the shrouding materials are dropped and the bore hole is filled. These shrouding materials grip the strainers and pipes on the outside and help to hold in position. Before starting lowering of strainers and pipes inside the boring tubes, the latter are raised to such a level that their bottom end is only slightly below the proposed level of the bottom of the finished tubewell, so that the bottom end of the tubewell will rest on the shrouding materials, when lowered in position.

The small piece of plain pipe is kept at the bottom end of the tubewell so that any heavy particles of sand setting inside the tubewell during development of the well may get collected in these space and may not choke the bottom end of the strainer tubes thus cutting out useful length of water bearing stratum and strainer. This small piece of plain pipe is known as "sump pipe".

Care should be taken that the entire blind pipe and strainer length is installed straight and vertical and concentric to pemit the filling in of gravel filter of uniform thickness and the installation of the pump in such a manner that it will operate satisfactorily and without damage. No deviation in the vertical greater than one inch in 100 feet will be tolerated for the length of the pump housing casing.

Joining of well pipes : The blind pipes and strainers can be connected by means of threads, or by welding. For welded connection a splice ring shall cover the joint and be welded on either side to the adjacent pipes.

Gravel Shrouding : The annular space between the well pipes and the wall of the bore hole shall be filled up immediately after installation of the well pipes, with a mixed gravel filter suited to the subsoil. The gravel shrouding shall extend from the seal plate up to at least 30 to 50 ft above the top of the upper strainer. The remaining space may be filled up with sandy materials up to about 3 ft underneath the bottom of pump base plate.

The gravel must be of good quality, clean, hard and round without clay particles and washed if required. The gradation of filter gravel must suit the formation of the subsoil but it is limited in general to grain sizes 3 to 7 mm and 1 to 3 mm. Where no casing pipe is used in boring, gravel should be placed by means of gravel fill pipes 3 inch diameter reaching down to the bottom of the borehole and the fill pipe liftd gradually as the height of shrouding increase. The filer hall be filled sufficiently dense so that no dangerous settlement occur during pumping test. Again where the casing pipe is kept in position till installation of well pipes, filling in the space and compaction occur automatically during extraction of the casing pipe.

Development of Tubewell : After placing of the gravel filter, the well shall be test-pumped and developed by pumping in such a way that the filter become permanently estabilised. Pumping shall be continued until the water clear. While pumping continues the finer particles get washed through strainer into the tube. The tube being of small bore in comparison to the quantity of water passing through it, the velocity of water up the tube is sufficient to carry the finer particles with it, keeping the inside of the tube and the gravel filter free from siltine. The subsoil surrounding the gravel filter tube get freed from its finer particles and therefore has a porosity higher than the undistrubed subsoil. Thus the surroundings of the strainer is freed from smaller particles of sand and coarser material loosely packed. The coarse materials finally arrange themselves around according to the size of grains, the largest being next to the strainer than the next larger and so on.

Eccentricity: In all cases where turbine pumps are installed, it is of great importance that the finished well and the bore itself should be as nearly vertical (i.e in plumb), as possible. Turbine pumps generally revolve with a speed between 1,500 and 3,000 revolutions per minute, and it can easily be imgined how damaging the effect of eccentricity can be in such cases on all the moving part of the pump and motor.

The stress and strain on all bearings, shafts, bushings, etc., are so greatly increased due to the eccentricity that all the care and patience exercised in securing a truly vertical bore is amply rewarded by the ease and smoothness attained in the installation and working of the pump later on. In spite of exercising reasonable care in boring, the finished well is likely to get slightly out of plump partly due to eccentricity in the bore itself and partly due to variation in the verticality of the top pipe when gravel filter is placed.

The inside diameter of the top casing pipe is always kept slightly larger than the outside diameter of the pump bowls, so that there is a certain gap between the outer face of the pump bowls and the inner surface of the casing pipe. So long as the eccentricity does not exceed the clearance allowed in this gap, advantage can be taken of this in reducing eccentricity. The eccentricity will be neutralised if the pump is installed not centrally in the casing pipe, but in such a way that one edge of the lowest pump bowl is almost touching pipe on the side to which it is inclined. But if the eccentricity exceedss the clearance allowed by this annular gap (i.e., the difference between the inside diameter of the casing pipe and the outside diameter of the pump bowl) it cannot be eliminated without setting right the eccentric casing pipe. The eccentricity greater than one inch, in 100 ft will not be allowed for the pump housing casing.

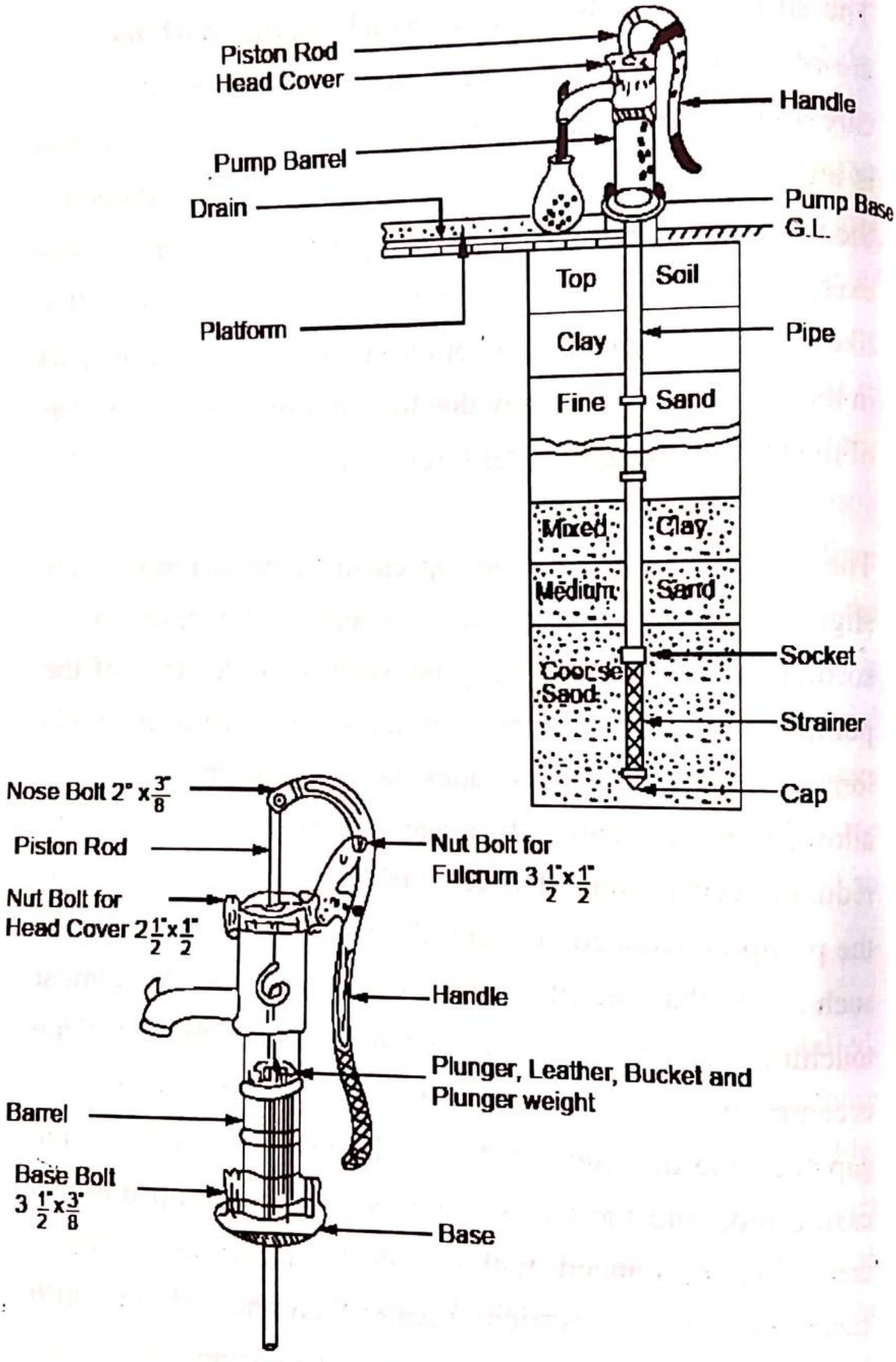


Fig. 3.13 Hand Pump Tubewell

Handpump Tubewell : In rural areas of Bangladesh, about 90% of ground water is being lifted by handpump tubewells. Even in urban areas, handpump tubewells are used where water supply facilities could not be extended. The handpump tube wells are also very useful at the time of electric power failure.

For this reason handpump tubewells are installed at certain strategic locations of cities so that water can be made available under any circumstances.

Generally, the component parts of a handpump tubewell are pipes, screens, sand trap, handpump, etc. A typical handpump tubewell is shown in Fig 3.13. Size of well pipes varies from $1^1/_4''$ -1 $^1/_2''$ inch. Pipes of various materials such as mild steel (M.S)[4], galvanized iron (G.I.), polyvinyl chloride (PVC), etc. are used in handpump tubewells. But G.I. pipes are universally used for small diameter tubewells due to its protection against corrosion and ease in sinking PVC pipes are excellent for corrosion resistant but due to their weak joints and low tensile strength, there are many construction difficulties.

Screens : The screen is a perforated portion of a well pipe through which ground water enters into the well from the aquifer. The following types and sizes of screens are generally used in Bangladesh : (a) Brass Screen : These screens are made of $1^1/_4''$ to$1^1/_2''$ diameter M.S. or G.I. pipes covered with 60 to 80 mesh copper wire net and brass perforated sheet. These screens are usually 6 ft long. (b) PVC. Screens : These screens have slot sizes of 0.008 to 0.012 inch, technically known as 8 slotted and 12 slotted screens. These slotted screen is so called when a pipe is bodily slotted, (c) Other types of screens are brass screen and stainless-steel screen.

Sand trap : A sand trap is an extension of a blank pipe of about 4 to 6 ft long fixed at the bottom end' of the Filter. The open end of the blank pipe is sealed with a cap. The purpose of the sand trap is to receive the incoming sand which settle ultimately in the trap and thus save the strainer from, blocking.

Hand pump : The function of a handpump is to tap water from the well. Handpumps are classified as No. 4, 5 and 6 according to the size (in inch) of the barrel. Generally, No. 6 hand pumps are widely used in Bangladesh. The capacity of a hand pump No. 6 is about 1/8 gallon per stroke. The component part of a handpump are a barrel, a base, a head cover, a handle, a piston rod, a plunger, a valve weight, bolts and nuts, a bucket and a seat valve made of leather or plastic.

Methods of Sinking : There are mainly two methods : (1) Sludger Method and (2) Water Jetting Method.

Sludger method (also known as Dheki Method) is a primitive one but less costly. In this method the well pipe is pushed into the ground by manual labour with the help of a bamboo staging. This method is adopted when the depth of wells does not exceed 150 ft. In water jetting method, a powerful force pump is used to pump the water into the Jetting pipe to loosen the soil and force it out from the top through the annular space between the jetting pipe and the casing. This method is costly and efficient and generally used when the depth of the well exceeds 150 ft.

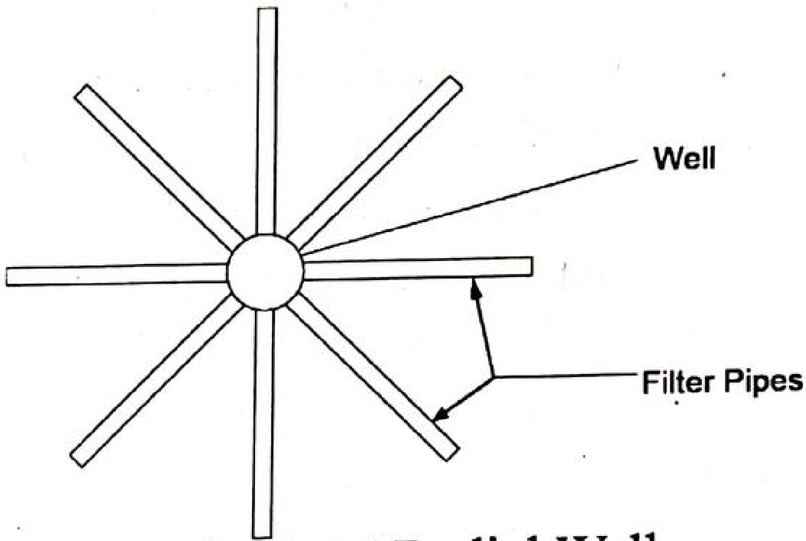


Fig. 3.14 Radial Well

3.15 Radial Wells :

Radial wells have found increasing use for water supplies. A typical radial wells system is shown in Figt 3.14 Screens are laid radially from the caisson (main well) into the aquifer. The number, length and spacing of screens are dictated by local conditions. The average length of screens is about 200 ft and the resulting well has a much larger discharge than would be possible with the conventional vertical wells.

Radial wells have been installed adjacent to streams where they serve to increase percolation from the stream to the ground water. These types of wells are also installed in coastal area where there is salinity problem.

3.16 Well Sanitation :

An important advantage of ground water as a source of municipal supply is its comparative freedom from bacterial pollution. But polluted surface water may enter the ground water around the top of the well casing or through the annular space between the casing and the wall of the hole. A suitable seal should be provided at the top of casing, and the space around the casing may be grouted. Surface water should not be permitted to collect arouud the top of the well. Abandoned wells should be sealed with clay, concerete or other filler to avoid contamination of the aquifer. Even though a well is built for irrigation or industrial use, sanitary precautions are advisable to avoid contamination of adjacent well.

A step in the construction of a well for domestic use is chloriantion to eliminate any contamination introduced during construction. Chlorination is accomplished by filling the well with a solution of chlorine (50 mw/l) and allowing it to stand for 2 hours. It is then rinsed out with fresh water pumped in at the bottom of the well.

3.17 Maintenance of Wells :

A properly constructed well requires little maintenance unless it is pumped at excessive rates. Excessive pumpes may result in movement of the fine materials in the aquifer, with the possibility of clogging near the screen. Sand entering the well as a result of high entrance velocity may damage the pump. Highly mineralized waters can cause an incrustation on the well screen by increased of dissolved minerals. The decrease in pressure caused by increased velocity near the screen may reduce the ability of the water to hold dissolved salts, particularly calcium carbonate in solution. Thus incrustation may be accelerated by high entrance velocity as a result of inadequate screen area or excessive pumping. Incrustation can sometimes be relieved by surging the well with a plunger and causing alternation flow back and forth through the screen. Severe cases will have to be treated with hydrochloric acid which is to be allowed to stand in the well of several hours. Bacterial clogging of wells is sometimes corrected by chlorination. After treatment the well should be pumped vigorously to remove-loosened materials.

Very little can be done about corrosion of screen or casing. Leakage resulting from a corroded casing may sometimes be checked by grouting around the casing. If the well is in corks; a damaged casing can sometimes be withdrawn and replaced but if the well is in unconsolied material, it may have to be abandoned.

3.18 Well Log :

The well log is a record of the underground materials penetrated as various depths and of the diameter and other characteristics of the well including size, depth, capacity, etc, It is very essential for safe operation and maintenance of the well.

3.19 Location of Groundwater Supplies :

Because of the cost to-well drilling, it is desirable to have some assurance that a well will reach a satisfactory aquifer. It may be possible to predict the depth, and productivity of an aquifer from conditions in other wells in the vicinity. Large projects will justify a more elaborate exploration, by a competent geologist. Sub-surface exploration is often done with small diameter tests holes from which sample of soils and rocks-may be obtained and tested permeability and specific yield, pumping, tests also be conducted on these test wells to determime the transmissibility and storage constant for the aquifer. Sometimes seismic, acoustic and radioactive methods are employed to determine the characteristics of the aquifer.

3.21 Yield of Ground water :

Actually safe yield cannot to defined to truely practical and general terms. The location of wells with respect to areas of recharge and discharge, the character of the aquifer, the potential sources of pollution, and many other factors involved in estimates of the maximum feasible withdrawal from an aquifer. Determination of safe yield is a complex problem of hydrology, geology hydraulics and economics for which each aquifer requires solution. The safe yield from an aquifer is equal to the annual recharge less the unavoidable natural discharge.

Thus, safe Yield = P–R–E–G. (3.16)

where P and E are the annual precipitation and evapo-transpiration from the area tributary to the aquifer. R is the mean annual runoff from the aquifer area and G is the mean annual surface discharge from the aquifer.

3.22 Artificial Recharge of Gronnd water :

Water may be forced into the ground for the following purposes : (1) to dispose of waste water, (2) to replenish overpumped ground water aquifers, (3) to-form a fresh water barrier against intrusion of salt water into heavily pumped ground water reservoirs along the coast and (4) to compliment or replace surface storage by ground storage in increasing the safe yield of surface supplies. Recharge may be accomplished by induced infiltration, spreading and recharge well.

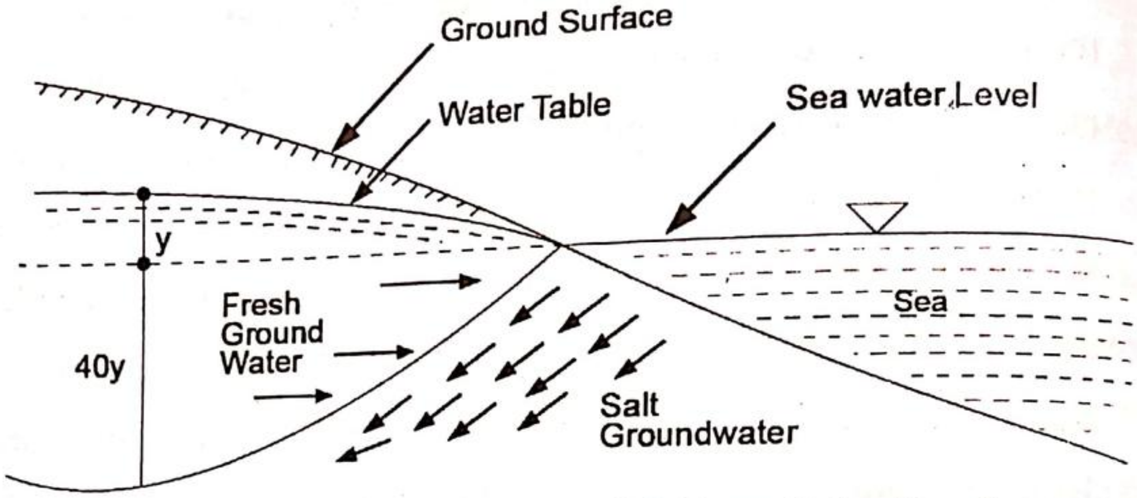


Fig. 3.15 Natural Equilibrium (Idealized)

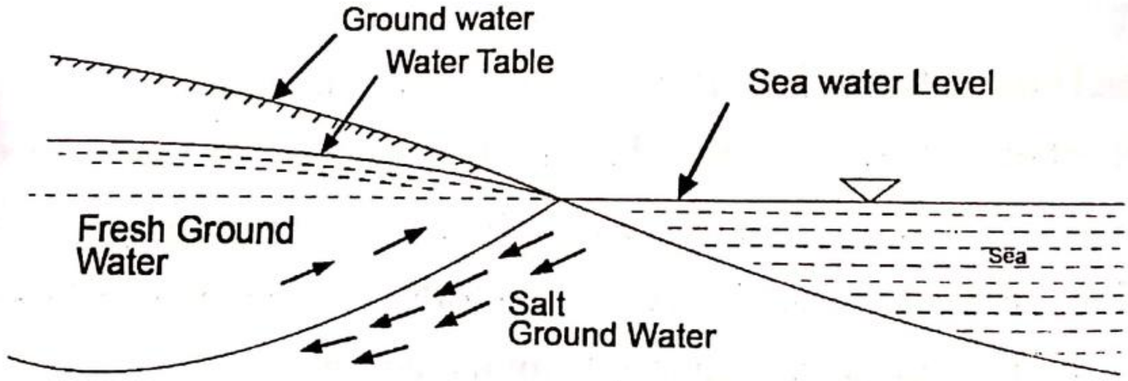


Fig. 3.16 Detail of Interface

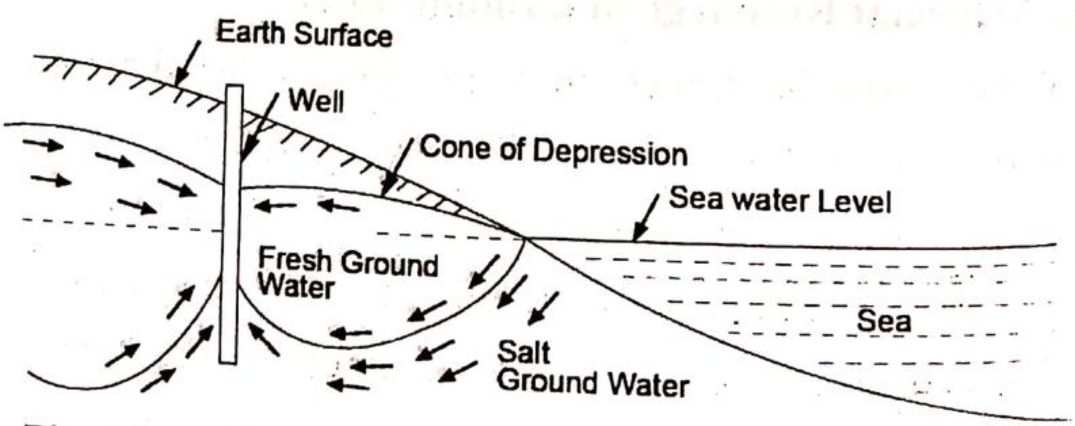


Fig. 3.17 Effect of pumping Fresh and Salt water Intrusion phenomena near a coastline

3.23 Salt Water Intrusion in Coastal Areas :

Fresh water is found in contact with salt water in coastal areas. A natural equilibrium between fresh and salt ground water develops along coastlines. The specific gravity of sea water is about 1.025 and the fresh water floats on the sea water. Hydrostatic equilibrium would require a fresh water column about 1.025 times as high as a salt water column. i.e. lft. of fresh water would exist above sea-level for each 40 ft below sea level as shown in Fig. 3.15. Conditions of hydrostatic equlibriutn do not occur, however, because of the hydraulic gradient imposed by the sloping water table. Magnification of the interfaces near the sea-level shows that the fresh water is flowing out of the fresh water-aquifer through a seepage face and across a portion of the sea bottom into the sea (Fig. 3.16). Thus the true shape of the interface is governed by hydrodynamic balance of fresh water and salt water. For most conditions, however, the 1/40 ratio rule may be applied without introducing serious error; If the water table lowered hy pumping, equilibrium is distrubed, and an inverted cone of salt water runs under the well (Fig. 3.17;. For equilibrium, the salt water rises approximately 40 ft. for each foot of drawdown in the fresh water. This severely limits the pumping rates of wells-along coastal areas.

Radial wells which operate with a small drawdown and need not extend below sea level can be used to avoid pumping salt water. This technique is widely used for water supplies on islands where the fresh water lens is surrounded by salt water. Overdraft of the fresh groundwater may reduce the gradient and permit the salt water to advance inland. Recharge wells along the coast have been tried as means of maintaining adequate fresh water to avoid salt water intrusion. Salt water intrusion presents an alarming problem to the coastal areas of Bangladesh.

QUESTIONS

1. Critically examine the merits and demerits of ground water over surface water as a source of municipal water supply. (BUET, 63, 69)
2. Discuss briefly the occurrence of ground water.
3. Write explanatory notes on the following :

   (a) Aquifer, (b) Aquiclude, (c) Connate water, (d) Artesian aquifer, (e) Springs, (f) Piezometric water level, (g) Hot springs, and (h) Geysers.
4. Explain briefly the geology of ground water. (BUET, 64,70)
5. Discuss briefly the ground water hydrology, (BUET, 66, 72)
6. Explain briefly the physical laws of ground water flow, and give a mathematical analysis of the "Theory of flow" into an ordinary well in the light of above laws. (BUET, 62, 68, 72)
7. State the hydraulics of ground water flow into an artesian well and deduce a mathematical expression for the yield of such an well having given, well diameter, 12 inch and the radius of circle of influence, 650 ft.

   What would be the effect on the yield of this well if a similar well is located at a distance of 300 ft and both the wells are pumped at the same time ? (BUET, 63)
8. With an ordinary well of 1 ft. diameter, having a depth of 100 ft. below the level of water table, the depth of water when the well is pumped is 80 ft. The coefficient of permeability is 923 gallons per sq ft. per day and the radius of circle of

influence, 1,000ft. What is the probable flow into the well at this rate of pumping in gpm ? (BUET, 72) Ans. 970 gpm.

9. For extensive exploration of the extent of underground water in a locality what study of the subsoil will you make to evaluate the same ? (BUET, 64, 69)

10. An 18 inch diameter ordinary well is being pumped at a rate of 350 gpm with a drawdown of 30 ft. The static depth of water in the well is 200 ft. During pumping the depth of water in a similar well, not being pumped, at 3 distance of 24 ft is 185 ft. At what rate could water be pumped from the two wells, if both the wells were being pumped together with a drawdown in each well of 30 ft? (BUET, 65) Ans. 474 gpm

11. A 12 inch diameter ordinary well produces 50 gpm when the drawdown is 6 ft. This well penetrates an aquifer 105 ft. thick. Find the flow from this well for a drawdown of 6 ft. if its diameter were : (a) 9 inch, {b)10 inch. Assume that the radius of the cone ot depression is 2500 ft in all cases. (BUET, 70)

12. Name and explain common features of ground water collection works. (BUET, 64p. 68, 72)

13. Discuss briefly the various methods of well construction. Which of the methods do you think best for rocky strata and why ? (BUET, 65, 72)

14. Write short notes on the following :

(a) Well Sanitation and Maintenance, (b) Well log; (c) Safe Yield from an Aquifer, (d) Recharging of wells, and

(e) Salt water intrusion in wells in coastal areas. .

15. Ground water is a vital source of water supply in Bangladesh. Discuss. (BUET, 67, 70, 72, 73)

4

SURFACE WATER COLLECTION AND TRANSPORTATION

4.1 Introduction :

Surface water is collected from rivers. lakes. reservoir, canals. ponds and seas. The collection system is a set of engineering works designed to convey water from a source to a distribution system via treatment plant and includes intakes, suction pipes, delivery pipes and pumping stations.

4.2 Intakes :

The intake is a device placed in a surface water source to permit the withdrawal of water from the source and then discharge it into intake pipe through which it will flow into the water-works system. There are mainly two types of intakes :(1) River intakes, and (2) Lake and reservoir intakes.

River Intakes : Understandably. river intakes are constructed well upstream from points of discharge of sewage and industrial wastes. Optional location will take advantage of deep water a stable bottom,and favourable water quality, all with proper reference to protection against floods, debris and river traffic. Where the river bed shifts or depth of flow varies greatly, intake pumps may be mounted on carriages that are moved up and down on the river bank to stay within desired suction lift as flows the and fall. A typical river intake is shown in the Fig. 4.1.

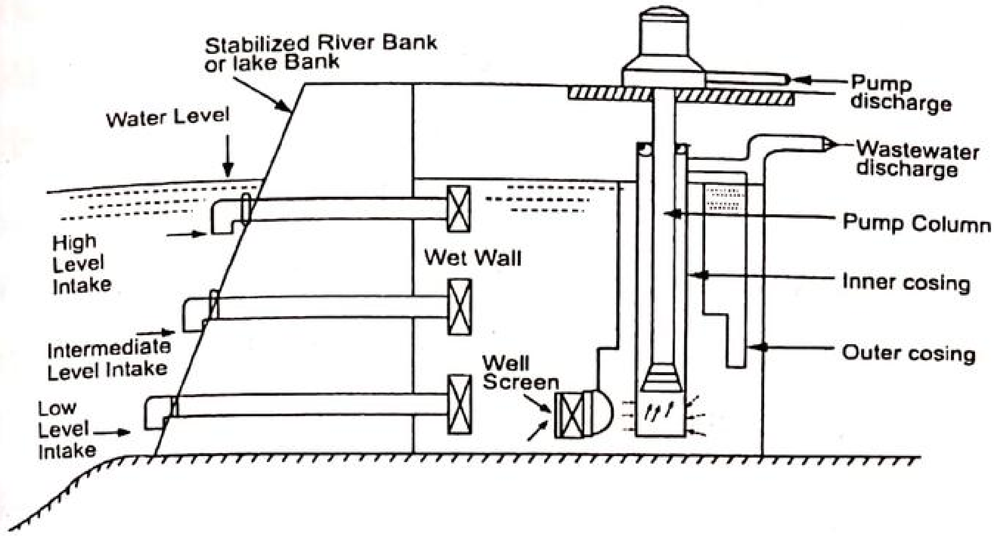


Fig. 4.1 River or Lake intake with vertical pump

Lake and Reservoir Intakes : Lake intakes are sited with due reference to sources of pollution, prevailing winds and surface currents. Reservoir intakes resemble lack intakes but generally lie closer to bank in the deepest part of the reservoir They are often incorporated into the impounding structure itself (Fig 4.2).

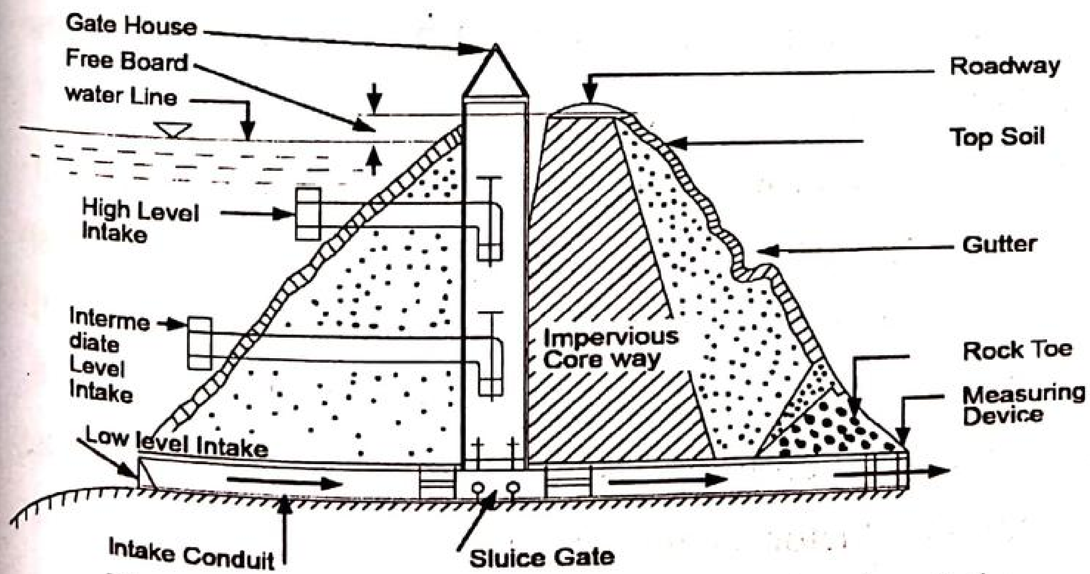


Fig. 4.2 Dam and intake tower for an impounded Surface water Supply.

Intake Velocities and Depths : Intake entrance should lie 10 to 15 ft below the water surface but 4 to 6 ft above the river, lake or reservoir floor to keep bottom sediments out of intakes and entrance velocities are kept down to 3 or 4 inch per sec, At such low velocities vegetation debris and other materials are not entrained in the flowing water, fish and other aquatic lives are well able to escape from the intake cureent. Gratings or screens of 2 to 8 mesh to an inch are provided at the intake-entrance.

Intake Pipe and Pumping Station : Intakes are connected to the banks of rivers or to the shores of lakes and reservoirs (1) by pipe lines (often laid with flexible joints) or (2) by tunnels blasted through rock beneath the floor. The pipe lines are generally laid in a trench on the river bank or on the lake or reservoir floor and covered after completion. Pipe passing through the foundation of dam are subjected to heavy loads and to stresses caused by consolidation of the foundation.

Intake pipe are designed to operate at self cleansing velocities, 3 to 4 fps. Flow may be by gravity or by suction. Pump wells are generally located on shore or banks. Suction lift including friction should not exceed 15 to 20 ft. Accordingly, pump wells are often quite deep. The determining factor is the elevation of water level in the river, lake or reservoir in times of drought.

Intake Design Considerations : The following are important considerations for designing an intake :

(1) Selection of a particular type for the given source.

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

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

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

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

(6) Installation of intake valves or port holes at 2 or 3 different levels to get the best available quality of water, eliminating seasonal fluctuation of water levels.

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

(8) Assurance of the safety of the intake structure, provision future extension and installation of standby units of pumps.

4.3 Transportation of Water :

The arrangements for the transportation of water from the source of supply to the treatment plants and subsequent distribution to the consumers form an important part of the water-works system. The source of supply usually being at some distance away from towns and cities, it is necessary to construct structures for the transportation of water. These structures are know as pipes or conduits. There are two general classes of pipes; (1) Pressure pipes in which the water flows under hydraulic pressure, and (2) Gravity pipes (open channels) in which the water flows by gravity.

A pressure pipe is also defined as a pipe flowing full. Such pipes are often less costly than open channels (canals and flumes / because they can generally follow a shorter route. If water is scare, pressure pipes may be used to avoid loss of water by seepage and evaporation which generally occurs in open channels. Pressure pipes are preferable for public water supplies because of the reduced opportunity for pollution.

The open channel may take the form of a canal, flume. tunnel, aqueduct or partly filled pipe. Open channels are charcterized by a free water surface, in contrast to pressure pipes, which always full.

4.4 Pressure pipes:

The desirable qualities of pressure pipes are as follows:

(1) They should be made of durable materials so that no leakage develop causing wastage of water.

(2) They should be strong and of sufficient thickness to withstand both internal and external stresses.

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

(4) The pipe materials should not impart any physical or chemical effects to water.

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

(6) Low initial cost and maximum service period of pipes are desirable.

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

(8) Tte pipe sections should possess good hydraulic properties.

Materials for Pressure Pipes: The principal pipe materials are steel, cast iron, concrete, wood, asbestos-cement, and vitrified clay, PVC, etc. Relative economy plays a large part in the selection of pipe materials but availability of skilled labour for construction and accessibility of the site may be influencing factors.

Steel Pipe : Steel pipe has been used in all sizes upto more than 20. ft. in diameter. Steel pipe in sizes of 1 to 12 inch in diameter is often a continuous tube formed by drawing over a mandrel. In sizes under 42 inch, steel pipes are often made of long, narrow steel plates which are bent to shape and welded or riveted along a spiral joint. This type of pipe has considerable flexural strength. Larger sizes are built on the job by welding or riveting steel plates. Steel bands or stiffening ringe are sometimes provided on larger steel pipes to aid resisting bursting, pressures. The working stress for steel is usually taken as 16,000 psi. Steel pipes are made much thinner than cast iron pipes because the material is stronger, is more uniform and can be more easily inspected and the defects are more readily located.

Buried steel pipes are not usual provided with expansion joints since they are not subject to large temperature changes. Pipes exposed to the atmosphere may, however, require expansion joints to minimise temperature stresses.

In the range of sizes encountered in engineering practice, steel; pipe sizes vary by 3 inch increments from 12 to 30 inch diameter and by 6 inch increments from 30 to 72 inch in diameter. The internal diameter of steel pipe depends on the wall thickness, The life of any pipe material depends very much on the conditions to which it is exposed, but properly protected steel pipe should have a life of at least 40 years under ordinary conditions. Protection is commonly provided both internally and externally. The internal coating is applied centrifugaly and the external coating is painted or sprayed on at the same time the internal lining is placed.

Cast-iron Pipe : Cast-iron pipe is widely used for city water supplies because of its high resistance to corrosion and consequent long life. Under normal conditions, castiron pipe can be expected to last 100 years. Standard pipe sizes are 2, 3, 4, 6, 8, 9, 12, 15, 18, 21, 24,27, 30, 36,42, 48, 54, 60,66,72 inch in diameter. The usual length of a pipe section is 12 ft but lengths upto 20 ft can be obtained. Cast-iron pipe is made in several thikness classes for various pressures upto a maximum of. 350 psi. Its main advantages are durability, resistant to corrosion, ease of laying, joining and admitting of junctions. Cast-iron pipes are usually dipped in a bituminous compound for protection against corrosion and to improve their hydraulic qualities; larger size are provided with a lining of cement mortar.

Concrete Pipe : Reinforced cement concrete pressure pipe is generally used for large water supply systems. Such pipe has the advantage of good hydraulic properties and the pipe is resistant to tuberculation and corrosion. When the pipe is properly made and laid, the leakage is small.

Precast concrete pipe is available in sizes upto 72 inch in diameter and sizes upto 180 inch have been reported to have made on special order. All concrete pipes are reinforced except in sizes under 12 inch in diameter. The reinforcement may take the form of spirally wound wire or elliptical hoops. In large pipes the reinforcement usually consists of two cylindrical cages. Because of the better control in its manufaeture, precast pipe is usually of higher quality and need not to be so thick as cast-in-situ pipe of the same size. Because of the need to move plant and forms over long distances, cast-in-situ pipe is relatively expensive and is normally used only for pipe sizes not available in precast form or where transportation difficulties make use of precast pipes impossible. Concrete pipes should at least last 40 to 60 yrs under average conditions. Alkaline water may cause rapid deterioration of thin concrete sections. Concrete pipes carrying vaste water may be subject to sulphide corrosion and may be short lived unless proper precautions are taken.

Nowadays, prestressed concrete pressure pipes are used in water supply systems. This pipe withstands veiy high pressure and is advantageous because it requires less steel and weighs less. But it is costlier because special high strength steel and techniques are required in its manufacture.

Asbestos cement Pipe : Asbestos-cement pipe is made from-asbestos, silica, and cement converted under pressure to a dense, homogeneous material possessing considerable strength. The asbestos fibre is thoroughly mixed with the cement and serves a reinforcement. Asbestos-cement pipes are available in diameters from 3 inch to 36 inch in 10 to 15 ft lengths. The pipe is manufactured in three different grades intended for internal pressures of 100, 150 and 250 psi. Asbestos-cement pipe is assembled by means of a special coupling which consists of a pipe sleeve and two rubber rings which are compressed between the pipe and the interior of the sleeve. The joint is as resistant to corrosion as the pipe itself and is flexible enough to permit as much as 12 deflection in laying pipe around curves. Asbestos-cement pipe is light in weight and can be assembled without skilled labour. It can be joined to cast-iron pipe with lead or sulphur-base compounds. It is easily cut and tapped, drilled and treated for service connections. The hydraulic efficiency of this pipe is high. It is highly resistant to tuberculation, incrustation, and corrosion. The pipe is easily damaged by excavation tools and does not have much strength in bending.

Vitrifird-clay-Pipe: Clay pipe is not often used as pressure pipe but widely used in sewerage and drainage systems for flow at partial depth. The main advantage of clay pipe is that it is virtually free from corrosion, has a long life and its smooth surface provides high hydraulic efficiency. Use of clay pipe under pressure is usually prevented by its low strength in tension and the difficulty of securing water tight joints.

Clay pipe is most commonly made in 3 to 5 ft lengths. Inside diameters vary by 2 inch increaments from 4 to 12 inch diameters, greater than 36 inch is rarely used.

PVC (Poly vinyl Chloride) Pipe: PVC pipe is manufactured from poly vinyl choloride, resin and some stabilizer. It is manufactured for wide range of dimensions and pressures. These are specified by the manufacturer and have to be strictly followed. The pipe material does not have any detrimental effects on the water flowing through it. These pipes are very light and easy to handle.

They are highly resistant to tuberculation, incrustation and corrosion, but not resistant to temperature exceeding 150° C. Because of their adaptability, use and serviceability, PVC pipes are nowadays preferred to steel, cast iron, or concrete both in water supply and sewerage systems.

G. I. (Galvanized Iron) Pipe : G. I. pipes are generally used for service connections from distribution branch lines. They offer great facilities for fitting and fixing branches. bends, for reduction in sizes, for fitting and fixing taps. cocks, etc. Pipe diameter varies from 1/4 inch to 12 inch.

larger diameter pipes up to 18 inch are available but they are very costly. For acidic water, these pipes may cause lead poisoning.

Miscellaneous Pipes : Other materials used for pipe include-copper, wrought iron, plastics, asphaltic fibre and lead. Copper and wrought iron are used for small diameter pressure pipes. Copper is quite expensive but may be advantageous in situations, where corrosion is likely to occur. Plastic pipe is corrosion free and light in weight. Its low strength, however, does not permit it to be used in large sizes. It is mainly used for washing purposes. Asphaltic fibre pipe is sometimes used as house connections to sewers. Lead pipe is used as pressure pipe in house plumbing.

4.5 Corrosion of Metal Pipes :

Metal pipes are subject to corrosion. Corrosion is a phenomenon by which metals and their alloys are attacked by the environment consisting of chemicals. There are mainly two types of corrosion in pressuse water pipes. (1) External corrosion and (2) Internal corrosion. External corrosion is caused by external agents like biological action. oxygen, etc., and the internal corrosion is generally attributed primarily to the nature of water which flows through pipes.

The chemical attack of an environment upon a metal results in the oxidation of the metal and the formation of corrosive products. usually the oxids, hydroxides, carbonates, sulphides, etc. In, most case, corrosion product is insoluble in the environment and forms a separate phase on or adjacent to the metal. Hence, corrosion may be defined simply as the process by which the metals and their alloys are destroyed by chemical or electrochemical means. So, according to reaction with metals, corrosion is of two types; (1) Chemical corrosion, and (2) Electro-chemical corrosion (Electrolysis). In its simplest from, chemical corrosion occurs when iron enters solution as positive ions and combines with the negative ions of water to form ferrous hydroxide. If the water contains oxygen, the ferrous hydroxide is oxidised to ferric hydroxide, an insoluble, red-brown precipitate. The initial rust-coating which form on the pipe tends to protect it from further corrosion, but the coating is not impermeable and some corrosion usually continues. Water, with a large amount of dissolved carbon dioxide is an active corrosive agent. Corrosion of metal pipes results in the formation of tubercles of ferric hydroxide on the inside of the pipe. This deposit (known as tuberculation) decreases the pipe area and increases the pipe roughness, thus greatly-reducing the hydraulic carrying capacity. Cast-iron pipes of smaller diameter have had their capacities reduced as much as 50 percent in 5 year by tubercualtion.

Corrosion of metal pipes may result from electrolysis. Electrolysis is often caused by the galvanic action resulting when two dissimilar metals are immersed in water. The rate of electrolysis depends on the dissimilarity of the two metals as indicated by their relative position in the electrochemical series. A metal which is high in the series is dissolved and deposited on the other metal. This type of corrosion may occur in water supply systems between pipes and fittings of different metals or between the pipe metal and the impurities in the pipe metal. Pipes laid in soil that has a high electrical conductivity are principally vulnerable to electrolysis.

The following are some of the important causes of corrosion of metal pressure pipes :

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

(2) **Influence of Acids and Alkalies :** Acidity or alkalinity of water passing through pipes will help vigorously to corrode pipes.

(3) Influence of Sulphur Compounds: The infleunce of sulphur compounds on metal pipes is harmful. It has been reported that the presence of sulphide particles raised the proportions causing rust from 22 to 90 per cent. The effect of sulphide is almost due to the liberation of hydrogen sulphide which accelerates the attack of acids on the pipe metal.

(4) Biological Action: Soil contains various types of bacteria both aerobic and anaerobic. Certain anaerobic bacteria are capable of rendering the oxygen present in sulphates, nitrates and carbonates available for the free oxygen and thereby corrosion will proceed pace. The most important sulphate reducing bacteia (Vibrio desulphuricans; which can cause serious attack on buried pipes when three conditions are satisfied : (a) Absence of oxygen as in many clayey soils (b) Presence of proper food (organic matter) and other environmental conditions needed for the growth of bacteria and (c) Presence of large amount of sulphates. These conditions occur in many calyey soils and lead to intense corrosion product is black iron sulphide. If subsequently the conditions become aerobic the iron sulphide is oxidised to ferric sulphate, which accelerates the corrosion by acting as an oxygen carrier, acid is said to be liberated in the oxidation process giving pH values as low as 3-5-which may also stimulate the attack. Sulphate reducing bacteria cause corrosion of ferrous metals. They work in absence of oxygen to react with sulphates and with organic compounds containing sulphur in the soil to produce hydrogen sulphide. Bacteria do not attack the pipe directly. The hydrogen sulphide combines with irons to form compounds of sulphur iron or it combines with water to form sulphurous or sulphuric acid which corrodes the pipe materials.

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

(6) Temperature: The increase in temperature accelerate the rate of corrosion. The rate ot corrosion in water pipes may be increased three or four fold by raising the temperature from 60 to 150°F.

(7) Velocity of Flowing Water: As the velocity of the water in the pipe increases from linear to turbulence type, the rate of corrosion is sharply increased.

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

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

Large pipes are usually protected by non-metallic coatings, such as bituminous coatings or cement linings. Numerous commercial bituminous compounds are available for both hot and cold application.

4.6 Scale Formation In Pressure Pipes :

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

The impurities which are mainly responsible for scale formation in water pipes may be classified under two heads: (1) Dissolved mineral matter, and (2) Dissolved gases.

Dissolved mineral matter include the hardness producing substances, i.e., carbonates, bicarbonates, sulphates and chlorides of calcium and magnesium, and silica. Dissolved gases include carbon dioxide, oxygen, nitrogen, hydrogen sulphide and methane.

Control of Scale Formation : To control scale formation in pressure pipes, water is softened. The chief objective of water softening is to remove dissolved mineral compounds which constitute the hardness and which deposit scales in water pipes, boilers and hot water heating system, cause serious difficulties in many processes including textile finishing, dyeing, canning, paper making, cold drinks preparation, tanning and others.

The following are the effective processes by which scale forming minerals and gases are removed from water : (1) Lime-Soda process, (2) Zeolite process, (3) Phosphate process, and (4) Lime process. All these processes are dicussed in chapter 8 (water purification).

4.7 Forces Acting on Pipes :

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

1. Internal force; due to static head
2. Internal forces due to water hammer
3. Forces at bends and changes in Cross-section
4. Forces due to temperature changes
5. External forces in the form of backfill, traffic and own weights.

Internal Forces dus to Static Head : Internal forces due to static head create hoop stress (transverse stress or circumferential stress and longitudinal stress.

Hoop stress, $S_h = pd/2t$. (4.1)

where S_h = hoop stress per linear length in inch of the pipe.

p = intersity of static pressure in psi=wh, in which

h is the static head and w is the unit wt of water.

d = pipe diameter in inch.

t = thickness of the pipe shell in inch.

Longitudinal (tensile) stress, $S_t = pd^2/[4t(d+t,]$ (4.2)

$= pd/4t$ (approximately) (4.3)

Water Hammer : One of the most damaging factors to a water piping system is water hammer action. In addition to its effect on the piping system, water hammer causes banging noises in the system that are very disagreeable to occupants in the building. Water hammer occurs when a colum of water flowing thtough a pipe line and discharging at an open outlet, is suddenly stopped by closing the outlet. Since flowing water has force, tremendous pressures result at the point of closure and pressure surges move along the pipe. The manner in which water hammer occurs is illustrated in Fig. 4.3.

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

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

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

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

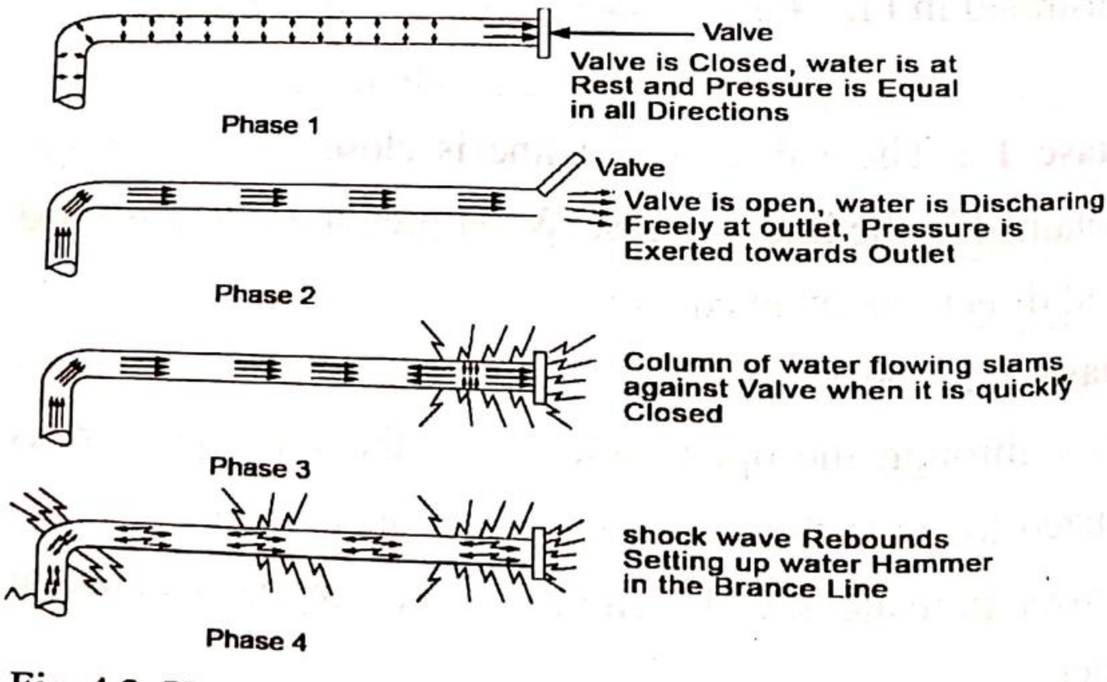


Fig. 4.3 How water hammer can Develope in a Pipe Line

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

Causes of Water Hammer: Not all the noises heard in the piping system can be attributed to water hammer. Loose faucet or valve washers can cause a pounding or chattering in the piping Improperly supported and secured piping can created noises as the flowing water causes the piping to vibrate and thus rattle, against steel members. This noise is easily transmitted through the piping system, undersized water piping with excessive pressures will produce shrill sounds. Then of course, there is certain, equipment, as pumps, that will produce noises unless the Pumps are insolated and the piping connections are equipped with flexible piping connectors.

As for water hammer, it is generally impossible to predict just where in the piping system that water hammer will occur. There may be a small diameter branch line in the system which by the nature of its length and the fixture it supplies should produce water hammer, yet it does not. On the other hand a larger diameter line which should not cause water hammer often will cause such noise.

One of the basic causes of water hammer is quick closing of valves. These include any valve or faucet that is closed rapidly. If the same valve were closed slowly, then the flowing water would have the chance to stabilize without producing the shock waves of water hammer. However, since these valves must Junction in the manner intended, other means must be employed to minimize the violent action of water hammer.

Other factors that contribute to the positive occurrence of water hammer are excessive water pressuers, inadequate piping sizes and water piping that is improperly installed.

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

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

Air is the most effective medium for absorbing the shock wave cuased by water hammer for (1) water is noncompressible; (2) air can be compressed to considerable pressure when the water compresses the air, it also fills the void offered by the displaced air. Because water has this flexible means to expend its force, the shock wave that would otherwise result, is quickly absorbed. The manner in which air serves to eliminate water hammer is shown in

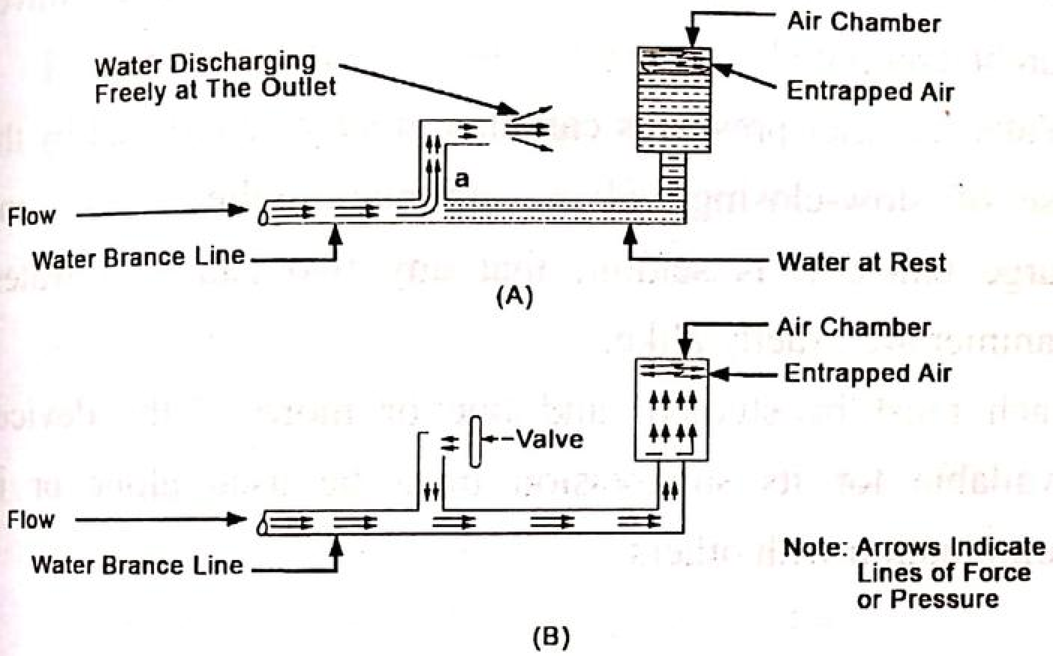


Fig. 4.4 How Air Chambers cushion the initial shock wave generated by water hammer

The Fig 4.4 (A) shows water flowing in a branch line and discharging at an open outlet. Water in the line from a to b is at rest with very little pressure exerted on the air that is contained to the chamber. The Fig. 4.4 (B) illustrates the condition when the following water is suddenly stopped by the valve. The shock wave generated rebounds but is absorbed by the air in the chamber. Thus by absorbing the initial shock wave, the water pressure is stabilized and the occasion of water hammer has been removed. Both details are diagrammetric and have been used only for example in showing how the phenomenon of water hammer can be properly controlled.

The extent of water pressure in a given line and the required sizes of air chambers to absorb the resultant shock waves can be computed.

Water hammer pressures can also greatly be reduced by the use of slow-closing valves, automatic relief values and surge tanks. It is seldom that any two cause of water-hammer are exactly alike.

Each must be studied, and one or more of the devices available for its suppression must be used alone or in combination with others.

Forces at bends and changes in cross-section : A change in direction or magnitude of flow velocity is accompained by a change in the momentum of water. The force required to produce this change: in momentum covers from pressure variation within the water and from the forces transmitted to the water from the pipe walls. For a pipe bend of uniform section.

Longitudinal force = $s\ (\pi d)\ t$ ....(4,4) where is the unit stress, d is the diameter of the pipe and t the wall thickness of the pipe.

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

Forces due to temperature changes: Longitudinal stress of considerable mangitude may develop in pipes exposed to large changes in temperature. The change in length δ of a pipe length L when subjected to a temperature change ΔT is.

$\delta = \alpha L. \Delta T$ .... (4.5)

when α is the coefficient of thermal expansion of the pipe material. If this change in length is prevened, longitudinal stresses will develop. From the principles of mechanics of materials it is known that in the elastic range,

$\delta = E \in = E\delta/L$ .... (4.6)

where α is the unit strain (elongation per unit length) and E is the modulus of elasticity and S is the resulting unit stress.

Combining Eqs. 4.5 and 4.6 gives

$\sigma = E\alpha\Delta\Gamma$ .... (4.7)

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

External Forces : An unsupported pipe acts as a beam with loads resulting from the weight of the pipe, weight of water in the pipe, and any other supperimposed loads. The stresses resulting from beam action, generally termed as flexural stresses, may be determined by the usual methods of analysis applied to beams. A pipe is a fairly efficient beam section, and stresses resulting from beam action alone are usually negligible except for long spans or when there are large superimposed loads. A rigorous analysis of the combined stresses resulting from internal pressure; external loads, temperature changes and beam action involves application of the principles of elasticity.

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

Rigid pipes (concrete cast iron and vitrified clay) cannot deform materially without cracking. On the other hand, flexible steel pipe can deform considerably witthout structural damage. Pipes are usually constructed in ditches or trenches which are excavated in natural soil and then covered by refilling the trench up to the original ground level. Trench conditions of pipe installation are shown diagrammatically in Fig. 4.5.

The vertical load to which a pipe is subjected, when so constructed, is the resultant of two major forces. The first of these is the weight of the prism of soil within the trench and above the top of. the pipe, and the second is the friction or shearing forces generated between the prism of soil in the trench and the sides of the trench. The backfill soil has a tendency to settle downwards in relation to the undisturbed soil in which the trench is excavated. The downward movement or tendency for movement induces upward, shearing forces which support a part of the weight of the backfill. The resultant load on the horizontal plane at the top of the pipe and within the width of the trench is equal to the weight of the backfill minus these upward shearing forces, as shown in the Fig. 4.6.

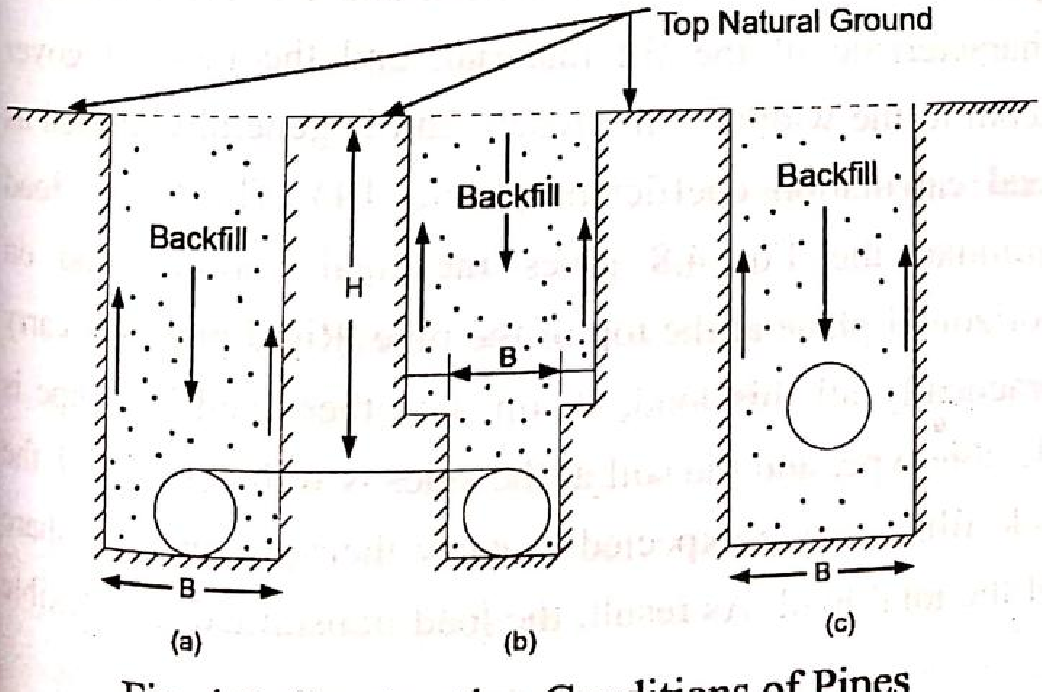


Fig. 4.5 Construction Conditions of Pipes

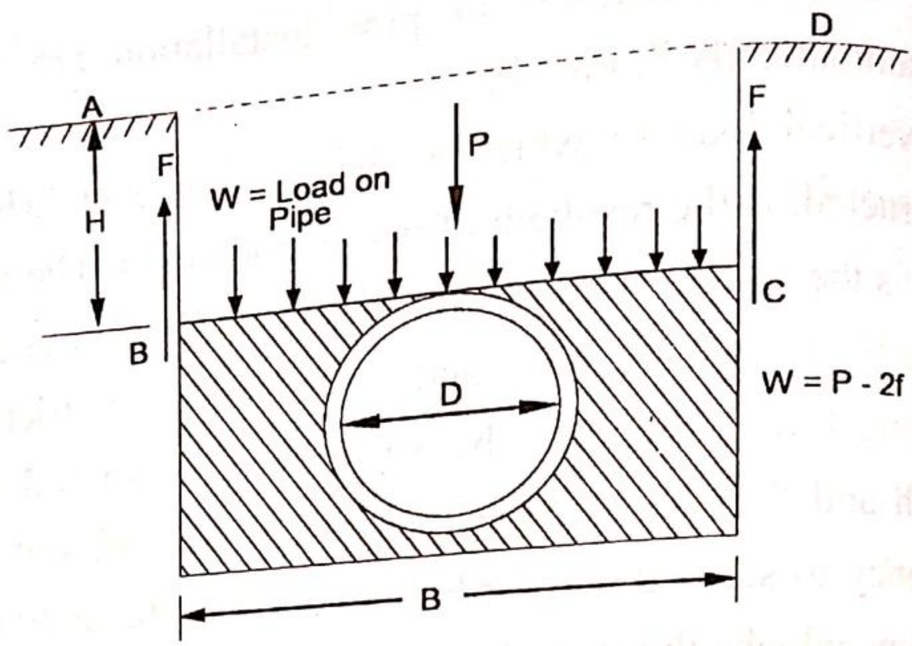


Fig. 4.6 Load Producing Forces

According to Anson Marston, (Iowa, USA) for rigid pipes in narrow trenches, the load w in pound per foot of pipe has been found to be

$$w = c\gamma B^2 \quad \text{.............................} \quad (4.8)$$

where B is the trench width at the top of the pipe. y is the specific weight of the fill material, and c is the coefficient characteristic of the fill material, and the ratio of cover depth to the width of the trench, and is generally termed as load calculation coefficient (Table 4.1). The trench load formula, the Eq. 4.8 gives the total vartical load on horizontal plane at the top of the pipe. Rigid pipe will carry practically all this load. If, on the other hand, the pipe is flexible type, and the soil at the sides is well compacted, the side fills may be expected to carry their proportional share of the total load. As result, the load transmitted to a flexible pipe is less than that for a rigid one. The empirical formula for the load on a buried flexible pipe in a narrow trench is

$$W = c\gamma BD \quad \text{..................... (4.9)}$$

where D is the outside diameter of the pipe.

If a pipe is placed on undisturbed ground and covered with fill (as a high -way culvert), the fill adjacent to the pipe is deeper than that over the pipe and can, therefore, still a greater distance (Fig. 4.5c). Under these conditions, generally referred to as embankment or broad-fill conditions, a portion of the weight of the adjacent prisms of fill is transfered to the central prism by shear, and the load on the pipe is greater than for trench conditions. The equation for the load on a buried pipe under embankment conditions is

$$w = c_p \gamma D^2 \quad \text{....................... (4. 10)}$$

values of c_p depend on the type of the pipe and the characteristics of the foundation and backfill. Typical values for c_p are given in Table 4.2.

Critical examinations of the Eq. 4.8. and 4.10 indicate the important influence which the width of the trench exerts on the load. It is seen that the width of the trench at the elevation of the top of the pipe is the controlling factor. Consequently, the width of the trench should be kept of an absolute minimum consistent with the provision of sufficient working space at the sides of the pipe. To this end, the engineer computing the load, the engineer supervising construction and the contractor actually installing the structure should see eye to eye with respect to the design criteria.

The load on a pipe is also influenced directly by the unit waight of the backfill materials. This value may very widely for different soils from a minimum of about 100 lb/cu ft to a maximum of about 135. Ib/cu ft. The average maximum unit weight of the soil which will constitute the backfill over the pipe may be determined by actual density measurement in advance of the structural design of the pipe.

Table 4.1 Values of the coefficient of Eqs. 4.8 and 4 9

Fill Material	Sand and gravel	saturated top soil	Clay	Saturated Clay
Specific weight lb/cu ft.	100	100	120	130
Cover depth / Trench width =H/B	Values of c			
1.0	0.84	0.86	0.88	0.90
2.0	1.45	1.50	1.55	1.62
3.0	1.90	2.00	2.10	2.20
4.0	2.22	2.33	2.49	2.65
5.0	2.45	2.60	2.78	3.04
6.0	2.60	2.70	3.04	3.33
7.0	2.75	2.95	3.23	3.57
8.0	2.80	3.03	3.37	3.76
9.0	2.88	3.11	3.48	3.92
10.0	2.92	3.17	3.56	4.04
12.0	2.97	3.24	3.68	4.22
14.0	3.00	3.28	3.75	4 34

Table 4.2 Values of the coefficient cp for Eq. 4.10

Cover depth / pipe diameter =H/D	Rigid pipe, Unyielding base noncohesive backfill	Flexible pipe average conditions
1.0	1.2	1.1
2.0	2.0	2.6
3.0	4.0	4.0
4.0	6.7	5.4
6.0	11.0	8.2
8.0	16.0	11.0

Loads of Pipes due to Superimposed Loads. Two types of superimposed loads are commonly encountered in the structural design of pipes. They are (a) concentrated loads, and (b) distributed loads.

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

$$w_{sc} = c_1 p_{sc} 1/L \quad \text{.........................} \quad (4.11)$$

in which w_{SC} is superimposad concentrated load on the pipe in pounds per foot length, p_{SC} is the superimposed concentrated loads in pounds, 1 is the impect factor. c_1 is the load coefficient which is a function of D/2H and L/2H shown in the Table 4.3, where H is the height from the top of the pipe to ground surface in ft., D is the diameter of the pipe in ft, and L is ths effective length of the pipe in ft (Fig. 4.7).

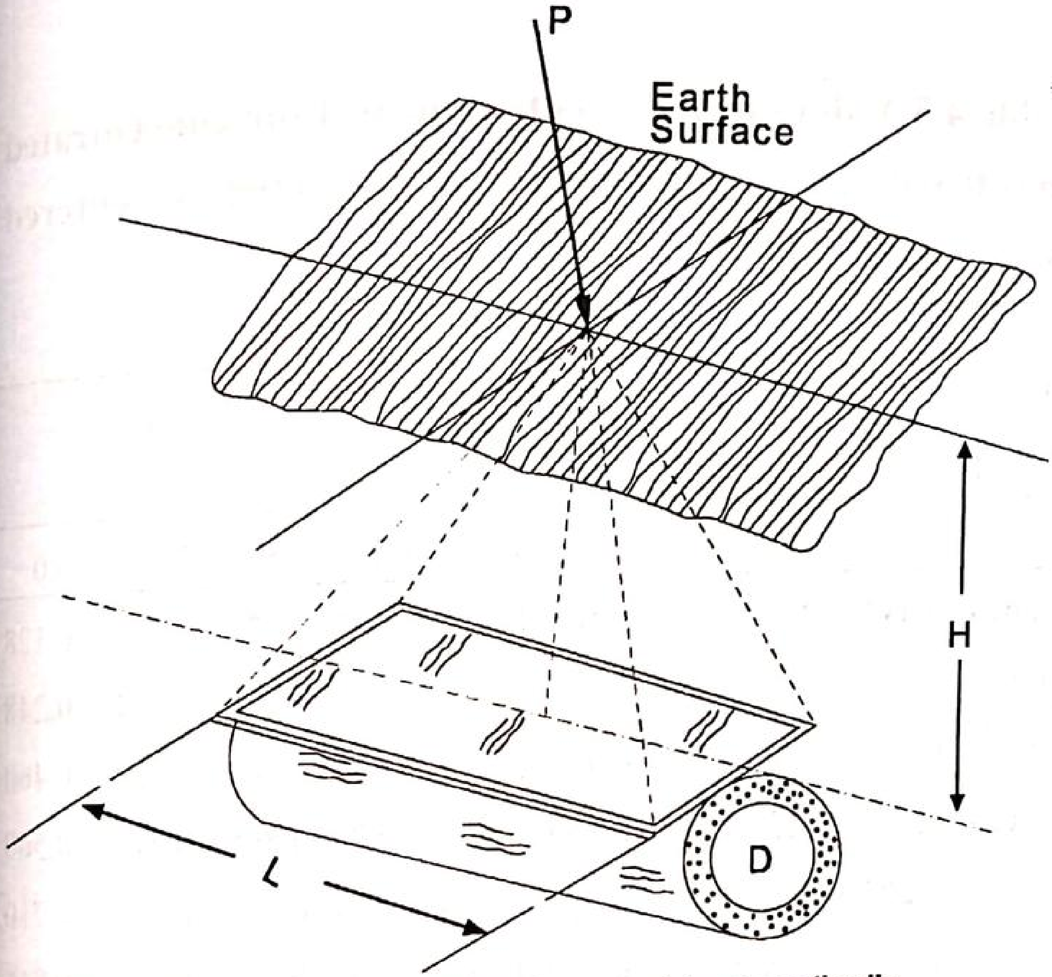


Fig. 4.7 Concentrated Superimposed load vertically centered over the pipe

In case of a distributed superimposed load as shown in the Fig. 4.8, the expression for load on the pipe is

$$w_{sd} = c_1 pD \quad \text{.............................} \quad (0.12)$$

where w_{sd} is the superimposed distributed load on the pipe in pounds per foot length, p is the intensity of distributed load in lb/sq ft, and. c_1 is the load coefficient which is a function of N/2H and M/2H from the Table 4.3 : M and N are the length and width respectively in ft of the area over which the distributed load acts.

Table 4.3 Values of load coefficient (c_1) for concentrated and-distributed superimposed loads vertically centered over pipes.

D/2H or L/2H	M/2H or L/2H									
	0.1	0.2	0.4	0.6	0.8	1.0	1.2	1.5	2.0	5.0
0.1	0.019	0.037	0.053	0.089	0.103	0 112	0.117	0.121	0.124	0.128
0.2	0037	0.072	0.109	0.252	0.202	0.219	0.229	0.238	0.244	0.248
0.4	0067	0.131	0.190	0.320	0.373	0.405	0.425	0.440	0.454	0.460
0.6	0.089	0.174	0.252	0.428	0.499	0.541	0.572	0.595	0.613	0.560
0.8	0.103	0.202	0.292	0.499	0.581	0.639	0.674	0.703	0.725	0.710
1.0	0.112	0.219	0.318	0.544	0.639	0.701	0.740	0.773	0.800	0.816
1.2	0.117	0.229	0.333	0.572	0.674	0.740	0.783	0.820	0.849	0.868
1.5	0.121	0.238	0.345	0.596	0.703	0.774	0.820	0.861	0.894	0916
2.0	0.124	0.244	0.355	0.613	0.725	0.800	0.849	0.896	0.930	0.956.

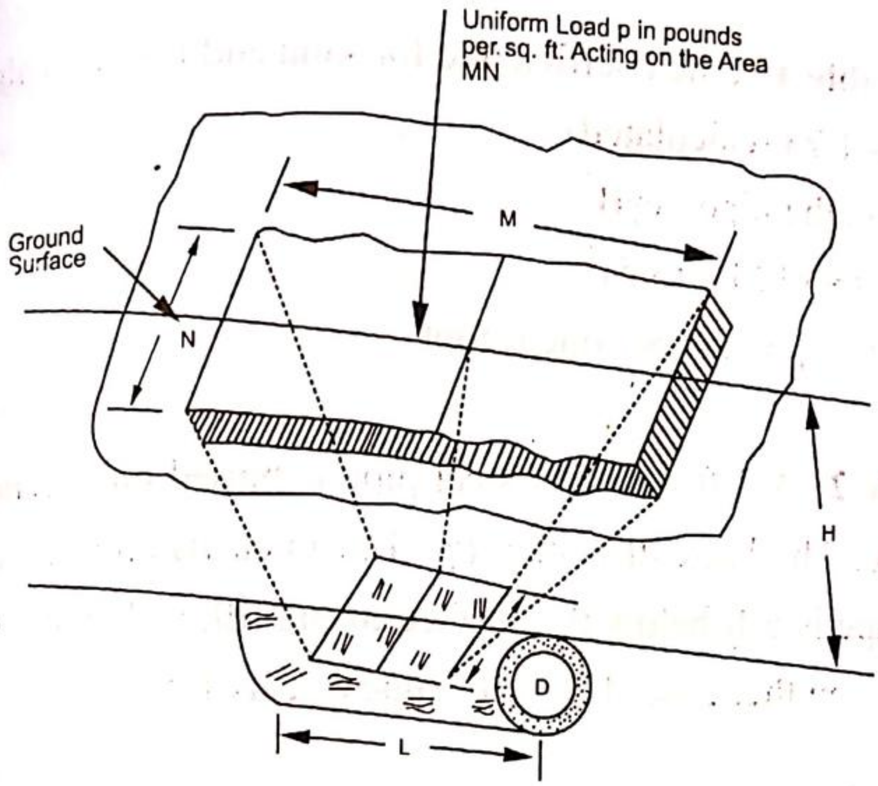


Fig. 4.8 Distributed Superimposed Load Vertically Centered Over Pipe

Traffic vehiclas which cause loads on the pipes produce mostly dynamic loads. Suggested impact factors for various kinds of traffic are as follows.

Highways	1.50
Railways	1.75
Airfields	1.00

Example 1 : What is the probable maximum load on a pipe laid in a trench that is 3.5ft wide if the depth of the fill above the top of the pipe is 9 ft and the filling material is wet sand? (γ= 120 Ib/cu ft.)

Solution : Here H = 9 ft. B = 3.5 ft. H/B . 9.5/3.5=2.57.

From Table 4.1, the coefficient c for sand and for this value of H/B is 1.73 (calculated).

$\therefore$ Load on the pipe= $c\gamma B^2$

$= 1.73 \times 120 \times (3.5)^2$

= 2540 pounds per linear foot.

Example 2: A 3 ft diameter steel pipe is buried on a trench 4 ft wide. The backfill is clay (γ =120 lb/cu ft) and the top of the pipe is 6 ft below the surface of the fill. Calculate the total load on ther pipe. Take the value of c as 1.2.

Solution :

Load on the pipe =$c\gamma BD = 1.2 \times 120 \times 4 \times 3$

= 1730 lbs per linear foot.

Example 3 : An 8 ft, diameter rigid concrete pipe rests on including ground and is covered with land (γ= 100 lb/cu ft) to depth of 6 ft. Calculate the pressure exerted by the fill material on the pipe. Take the value of c_p as 0.9.

Solution :

Pressure exerted by the fill $c_p = \gamma D^2 = 0.9 \times 100 \times (8)^2$

= 5760 lb/ft.

4.8 Strength of Pipes :

Because of the complex nature of the combined stress in pipes, the stresses are rarely analyzed in detail except for large and important pipe lines. Structurally, pressure pipes must resist the following forces singly or in combination.

(1) Internal pressure equal to the full head of water to which the pipe can be subjected.

(2)Unbalanced pressures at bends contractions and closures.

(3) Water hammer or increased internal pressure caused by sudden reduction in the velocity of water by rapid closing of a gate or valve or shutdown of a pump for example.

(4) External load in the form of backfill, traffic and their own weights between external supports.

(5) Temperature induced expansion and contraction.

According to ASTM standards the pipes are tested for crushing strangth by two methods (1) Sand Bearing Test (2) Three-edge beering Test. Strengths from sand bearing tests will be 50 percent more than those for three edge bearing test. Table 4.4 presents ASTM standards for crushing strengths of various types, of pipes in three edge bearing test.

Table 4.4 Crushing strength of Clay and Concrete Pipes by the three-edge Bearing Test :

(All strengths in pounds per linear foot)

Internal Diameter in inch	Clay	Plain Concrete	Reinforced Concrete			
			Class -I	Class-II	Class-III	Class IV
4	1,000	1,500	15,000			
6	1,200	1,600				
8	1,200	1,800				
10	1,400	2,400				
12	1,500	2,500	1,500	2,000	3,000	3,750
15	1,750	2,700	1,875	2,500	3,750	4,700
18	2,000	3,300	2,250	3.000	4,500	5,600
21	2,200	3.700	2,750	3,500	5,200	6,500
24	2,400	4,000	3,000	4,000	6,000	7,500
27	2,750		3,300	4,500	6,700	8,400
30	3,200		3,750	5,000	7,500	9,400
36	4,000		4,500	5,000	9,000	11,250
42			5,250	7,000	18,500	13,200
48			6,000	8,000	12,000	15,000
60			7,500	10,000	15,000	18,000
72			9,000	12,000	18,000	22,000

Field Supporting Strength - load factor × three edge bearing strength. (4.13)

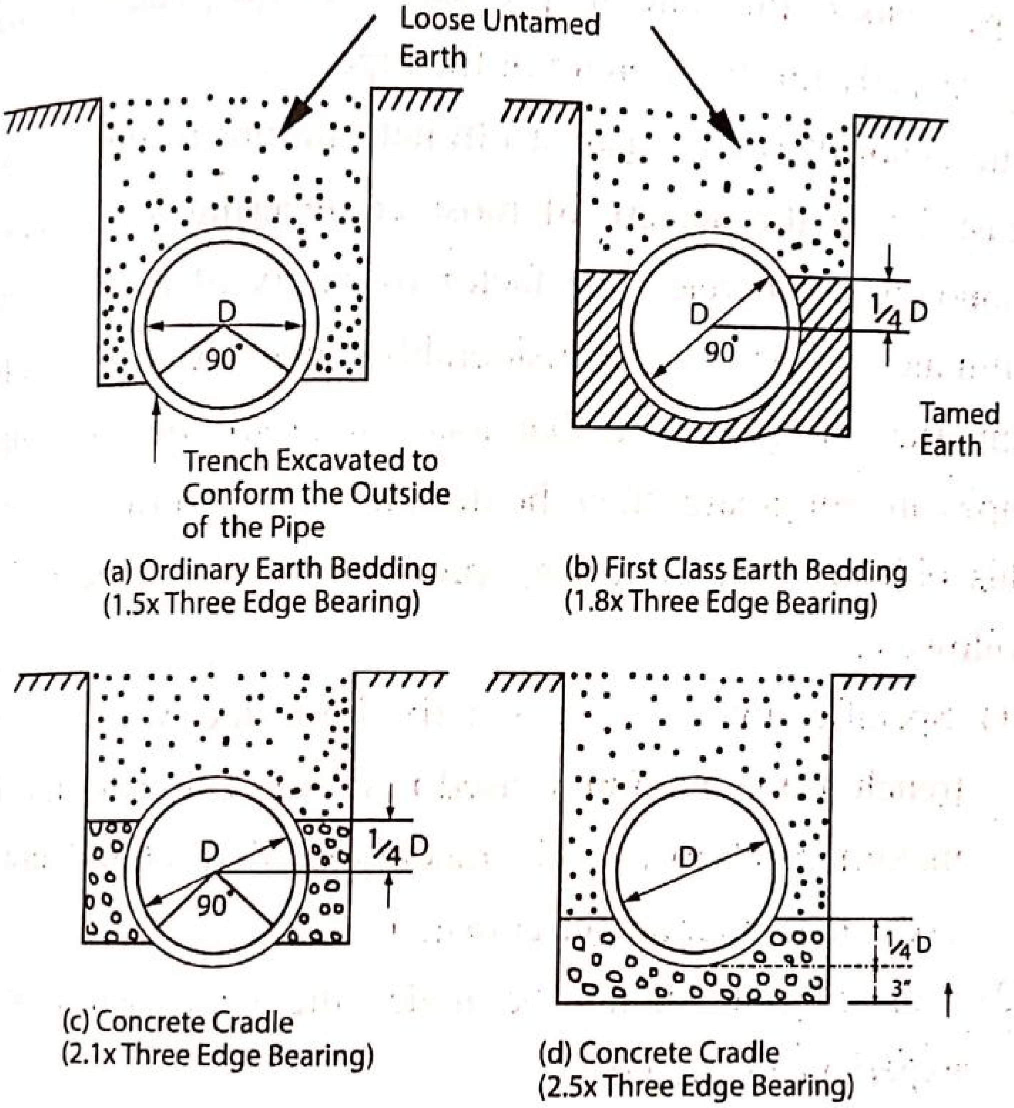


Fig. 4.9 Some Methods of Laying Pipes

The Fig. 4.9 shows several types of bending and indicates the strength ratio, i.e., the factors by which the three edge bearing strength is multiplied to find the effective strength in the field. In general, all buried pipes should be placed on a bed which has been rounded to fit the pipe and the backfill should be carefully placed and thoroughly tamed around and above the top of the pipe. This is specially true of flexible pipes since the lateral pressure of the backfill adds appreciably for the strength of the pipe.

The factor of safety against ultimate failure in generally at least 2.5 in the design, of most engineering structures of monolythic concrete. The factor of safety of pipes against ultimate collapse is considerably less. It is therefore important to guarantee that loads imposed on the buried pipes are not greater than the design loads. In order to attain this objective the following procedures should be strictly followed :

(1) Specifications should strictly limit the width of the trench to the maximum used in design calculations. The maximum allowable clearance specified should not be exceeded under any circumstances.

(2) Construction should be under ihe supervision of an experienced engineer.

(3) Pipe testing should be done under the supervision of a skilled technician in a testing laboratory and close liaison should be maintained between the laboratory and the field engineer.

(4) The field engineer and the design engineer should-confer time to time so that any deviation from the design specifications is immediately corrected or compensated for.

4.9 Pipe Joints :

The pipes are required to joint together pipes which are available in smaller lengths, say 6, 10, 12, 15 and 20 ft. only. The requisites of a jointing material are (1) imperviousness, (2) elasticity, (3) strength, (4) durability, (5) adhesiveness, (6) availability, (7) workability, and (8) economy.

There are various types of joints of which the powered joint, spigot and socket joint, flanged joint, screwed and socketed joint are important.

4.10 Pipe Laying :

Operation involved in the laying of pipe-frames include the following steps :

(1) **Preparation of detailed maps of roads and streets :** Showing position of curbs, gutters, other unground service lines-sewers, existing water pipe (if there is any), gas pipes, telephone and electric conduits.

(2) **Locating the proposed aliganment on the ground :** The trench line is marked by driving centrally stakes 100 ft apart on straight reaches and 25 to 50 ft apsrt on carves.

(3) **Excavating Trenches :** With width sufficient to allow The pipes to be properly laid and jointed, and with depth sufficient to give adequate protection to the pipes

against impact of traffic and other factors. Width is usually, kept 12″ to 18″ more than the outside diameter of the pipe and depth such as to give a ground cover of about 3 ft. from the top of the barrel of the pipe.

(4) Preparation of the bottom of the trench excavated : the bottom of the trench should be carefully prepared so that the barrel of the pipe can bedded true to line and gradient for its entire length on a firm surface. In many cases, a bed of concrete 6″ thick would provide a hard and even surface and adequate protection against possible settlement. Joint-holes should be left in the bed at suitable intervals to assist in the jointing of pipes where necessary.

(5) Lowering of pipes into the trench : Pipes stacked on either side of the trench after transported to the site should be gently and carefully lowered into the trench so as not to damage thin outer protective coatings or their ends. Before lowering, pipes should be wiped clean to remove any dirt or foreign matter sticking to them.

(6) Laying of Pipes : Pipes are seldom laid with a flat slope parallel to the hydraulic gradient. This is to avoid any air lock troubles. Every pipe length should, therefore, be laid either with a continuous size to high points or cotinuous fall to low points. Pipe laying should proceed in an uphill direction to facilitate joint-making.

(7) **Jointing Pipes :** It should conform to the operation and specification of pipe jointing.

(8) **Anchoring of Pipes :** At all bends, tees, valves and other branch connections, it should be necessary to provide thrust blocks of concrete to transmit the hydraulic thrust and distribute it over a wider area of the ground. Where the hydraulic thrust is upwards as in case of pipes on sloping grounds, anchor Mocks of concrete would be required to be provided at regular intervals and pipes should be firmly secured to them with steel straps.

(9) **Back Filling or Refilling the Trench with the Excavated material:** The material surrounding the pipes must be soft and laid preferably in layers of 6″ to 12″ thickness, well rammed so as to resist subsequent movement of the pipes. The remaining upper portion of the trench may be refilled as before with the excavated material and the top brought flush with the road level or a little projecting above it for later consolidation by the traffic.

(10) **Pipe Testing :** After laying and jointing and before backfilling the pipe is required to be tested under pressure. The test consists of filling the pipe-line with water expelling all air from within, allowing it to stand full for some time and then applying the test pressure of about 70 psi. The pressure is applied by means of a

manually operated test pump fitted with a pressure gauge. The test is generally carried out in sections as the pipe-laying proceeds. The open end of the pipe is closed for testing by fitting a suitable watertight plug.

QUESTIONS

1. What is an intake ? Mention the points that are to be taken into consideration in deciding the location and design-of an intake for the water supply of a large city, the source being apperennial river. Draw a neat and detailed sketch of the intake and describe it fully. (BUET.1964, '67 ; AMIB, '69)
2. Name the common types of pipes that are used for conveyance of water. What are the desirable qualities you will look for these pipes ? (BUET 1963. 72).
3. Name common types of pressure pipes and briefly discus their merits and demerits? (BUET, '62, '68, 72)
4. Why scales are formed in pressure pipes? Do you think that the scale formation in pressure pipes is harmful 7 Justify your answer. (BUET, 1964).
5 Why is corrosion of great significance in connection water supply pipes? What factors induce it? are employed to minimise it? (BUET, 1965)
6. What is water-hammer? Explain the phenomenon water-hammer by suitable sketches? How can you reduce water-hammer effect in water-works practices? (BUET, 1963, '68,'72).
7. Give an analysis of the external forces acting on buried pressure pipes.

8. A pipe 2 ft. dia. is laid in a trench 4 ft. wide depth of fill above the top of the pipe is 10 ft. What is the load on the pipe if the fill is (a) saturated clay and (b) wet sand? Assume standard values of data not supplied. (BUET, 66)
9. A pipe of 36 inch dia. laid in a trench 5 ft. wide the bottom is covered with 22 ft. of saturated top soil. What is the load on the pipe? Assume reasona we values of data not given. (BUET, 1972).
10. Describe briefly the procedure of laying underground water pipes in city areas and mention the safety precaution you will take to safeguard the traffic and workmen from an untoward accident. (BUET,'67, '71).

5

PUMPS AND PUMPING MACHINERY

5.1 Purposes :

Pumps and pumping machinery serve the following purposes in water supply systems :

(1) Lifting water from the source so that the water will flow into the mains by gravity.

(2) Boosting water from low service to high service areas, to separate the fire supplies, and to the upper floors of multistoried buildings.

(3) Transporting water through treatment plant, draining component settling tanks, filter beds and other treatment units, withdrawing deposited solids,supplying water (especially pressure water) to operating equipment and pumping chemical solutions to treatment units

(4) Lifting water to the overhead water tanks or reservoirs to flow by gravity to the distribution systems.

5.2 Types of Pumps :

Pumps may be classified according to the service for which they intended in water-works, the power by which they are driven, or the mechanical principles on which their operation is based. The following is a brief description of each type :

(i) Pumps on the Basis of Type of Service : These may be (a) deep well pumps, (b) low lift pumps, (c) high lift pumps, (d) booster pumps, (e) fire service pumps, and (f) stand-by pumps.

(a) Deep Well Pumps: These operate in tube-wells and pump water into service reservoirs or directly into the distribution system.

(b) Low Lift Pumps : These operate for small heads such as at treatment plants for pumping water from one unit to another and also for pumping water from river source or reservoir source to the treatment plant.

(c) High Lift Pumps : These operate under large heads for pumping water from clear water-reservoir to the elevated water tanks or directly into the distribution system.

(d) Booster Pumps : These pumps are used to increase pressures in parts of the distribution system where adequate pressure cannot be obtained either because of greater elevation or excessive loss of head in the distribution pipes. These are also used to supply water in the upper stories of tall buildings.

(e) Fire-Service Pumps : These are used to build up pressure to the extent required for effective fire-fighting in factories and multi-storied buildings.

(f) Stand-by Pumps : These are used for large pumping installations where auxiliary forms of power are also available. In case of temporary power (electric power) failure. the stand-by units can be driven on steam, diesel, etc.

(2) Pumps on the basis of Power used to Drive Them : These pumps may be classed as (a) steam pumps. (b) gasoline pumps, (2) diesel pumps, and (d) electric pumps.

(a) Steam Pumps : These are used in large pumping plants where prime considerations are (i) production of power at low cost. (ii) durability of service, and (iii) flexibility in operation.

(b) Gasoline Pumps : These are seldom used because of high cost in continuous operation. They are, however, suitable for stand-by service and are efficient for moderate heads.

(c) Diesel Pumps: These are reliable and economical for pump drives but not commonly used because of lower speeds. These are suitable for use only in small capacity water-works and as stand-by units.

(d) Electric Pumps: These pumps are generally used in all modern water-works. Advantages include freedom from smoke and dust, quiet operation, economical supervision, and economy of floor space for pumps and motors. Disadvantages include frequent power failure and necessity of providing stand-by units.

(3) Pumps on the basis of Mechanical Principles of operation : The common types are (a) displacement pumps, (b) centrifugal pumps, and (c) airlift pumps.

(a) Displacement Pumps : These pumps work on the principle of mechanically inducing vacuum in a chamber thereby drawing in a vacuum of water which is then mechanically displaced and forced out of the chamber. **These are of two types :** (i) Reciprocating pumps, and (ii) Rotary pumps.

In reciprocating pumps, a plunger or piston is operated so as to draw water into a closed chamber and to expect it into the pressure mains. The rotary pumps has cams or gears that revolve in a Close-fitting case and force the water around and out of each revolution, but this type is seldom used in water-works.

(b) Centrifugal Pumps : These pumps employ the principles of centrifugal force to impart energy to the water. Water entering into the pump-casing is revolved by a wheel called impeller which discharges it in a direction at right angles to its original direction of flow. In doing so, the kinetic energy of water is converted into static or pressure-head. Centrifugal pumps are extensively used in all types of modern water supply systems.

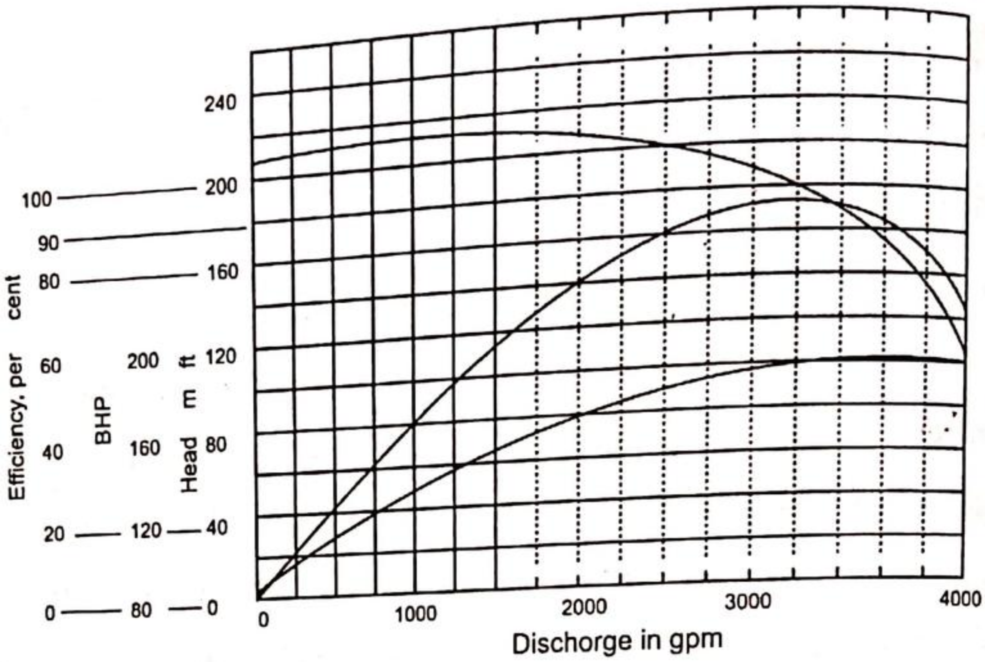
The centrifugal pumps may be classified as follows :

(i) According to the system of rotation as right-handed or left-handed.

(ii) According to the manner of conversion of kinetic energy into static head as volume pumps, diffuser pumps, turbine pumps, and axial flow pumps.

(iii) According to the number of stages as single-stage, two stage or multiple-stage pumps etc, depending upon the number of stages of pressure developed by impeller. Each stage of the pressure head is added together by leading the discharge from one impeller into the suction of another and this when added to the initial pressure at the inlet, results in an increased discharge-pressure of the centrifugal pump.

Characteristics of Centrifugal Pumps: The centrifugal pumps is essentially a high-speed pump, normally operating at speeds more than 1,000 rpm; in some cases, efficient speeds of 3,500 rpm are possible. The discharge is uniform and consequently the power requirements also are uniform/Because of high speed of the pump, its direct connection to an electric motor is possible.



Dischorge in gpm

Fig. 5.1 Operating Curves for Centrifugal Pumps

The discharge of centrifugal pump is fixed by the design and by the speed. In Fig. 5.1, are shown typical operating curves for a centrifugal pump operating at a constant speed under various cónditions. The pump was designed to deliner 3,000 rpm against a 185 ft. head with an efficiency of 85 percent. It is observed from the carves in Fig. 5.1 that (a) the discharge of the pump increases with the decrease in head, maximum discharge being limited by a certain minimum head below which the pump will not operate. The head increases to a maximum at zero-discharge with the discharge valve being closed. This condition is called shot-off head and should not be permitted to last long: otherwise pressure will abnormally rise above the design head, (b)

Maximum efficiency is obtained with moderately low head and high discharge and decreases with lesser or greater discharge a fact greatly utilized in rating pumps, (c) The power required from the prime mover to drive the pump increases with increase in discharge, this being maximum at the shut-off head, but as the discharge valve is closed under this condition, the prime mover will not be over-loaded. However, this centrifugal pumps are much easier to shaft than other types of pump.

Advantages of Centrifugal Pumps : The centrifugal pumps has a wide range of usefulness in water supply systems, as it is relatively cheap, compact and simple, and is adaptable in various kinds of power.

(4) Pumps according to the position of pump-shaft : Pumps may also classified as horizontal shaft and a vertical shaft pumps. Horizontal shaft pumps have such characteristics as larger required head-room; less of corrosion and abrasion and higher efficiency. Vertical shaft pumps are commonly used as deep-well pumps and have such characteristics as lesser floor-space requirement, positive suction, easy priming, higher discharge head, higher initial cost and difficulty in maintenance.

Well Pumps : These include deep well turbine pumps, air lift pumps and submersible pumps.

Deep Well Turbine Pumps : A deep well turbine pump operates in the same way as a centrifugal pump but the channel into which the water is discharged by the impeller is of uniform cross-section. While somewhat higher efficiencies are obtained with a turbine pump than with ordinary centrifugal pump, the initial cost is higher and the cost of maintenance is greater for the turbine pump. The turbine pump is particularly adapted to deep well pumping. In Fig. 5.2 is shown a common type of deep well turbine pump.

Air Lift Pumps : The air lift pump is particularly adapted for use in wells that are drilled through is at a considerable depth below the surface of the ground. In Fig. 5.3 are shown the main parts of an air lift pump. Compressed air from an air pipe is admitted into the education pipe at its lower end through a foot-piece or air diffuser. The mixture of air and water so created has a lower specific gravity than that of water alone and thus rises to the surface. With the continued supply of air, the column of water in the education pipe is forced upwards ultimately discharging from an outlet of the top. The effectiveness of this pump is dependent upon a factor called percentage submergence which is the ratio $\frac{D}{H+D} \times 100$; where H being the effective lift of the pump and D, the depth of submergence, and H+D representing the effective length of the education pipe. Its value should be at least 25 per cent for the pump to operate at all and 70 per cent to operate of best efficiency.

The advantages of the air lift pump are (a) freedom from submerged moving parts maxing them useful to handle corrosive or gritty materials, (b) case of operation and maintenance the compression unit being located on the ground surface, (c) increased yield caused by forcing greater air pressure into the pumps, and (d) its suitability for installation in crooked holes where other types of pumps are least suitable.

Disadvantages are (a) greater depth of submergence necessiating wells to be made deeper, (b) low efficiency, 20 to 45 per cent, and (c) little flexibility in muting variations in demands.

Submersible Pumps : They differ from the usual deep well pumps in respect that the motor here is below the turbine bowls of the

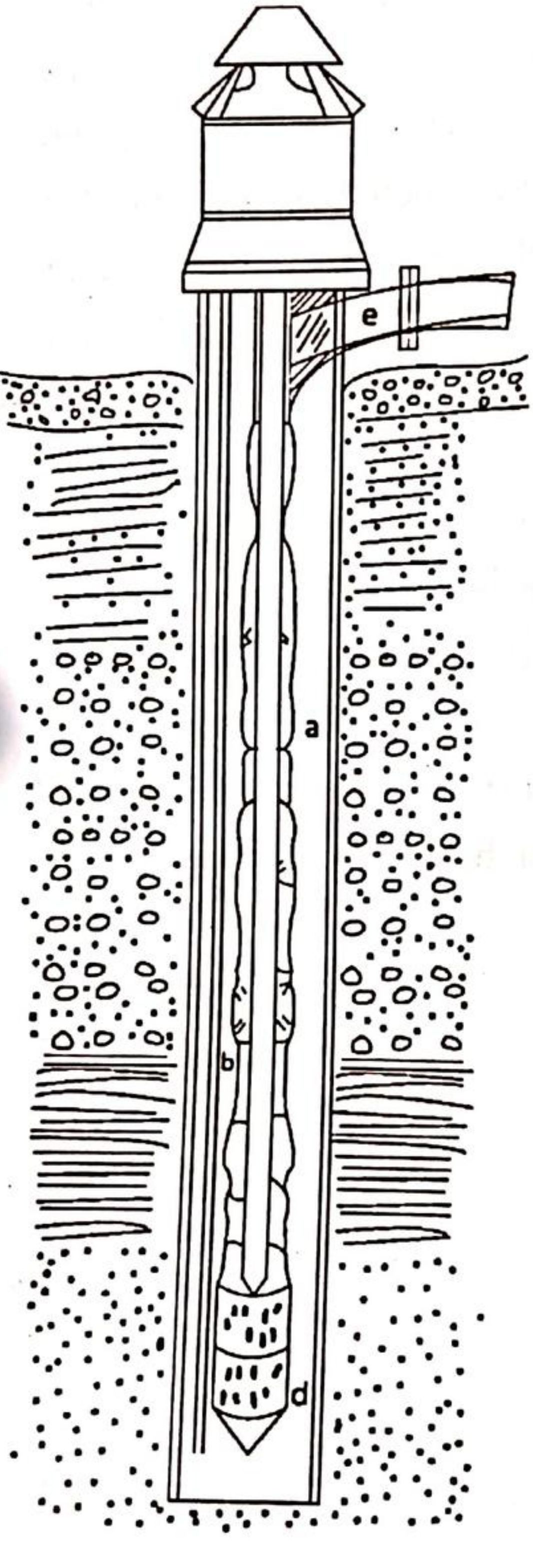


Fig. 5.2 Turbine Pump

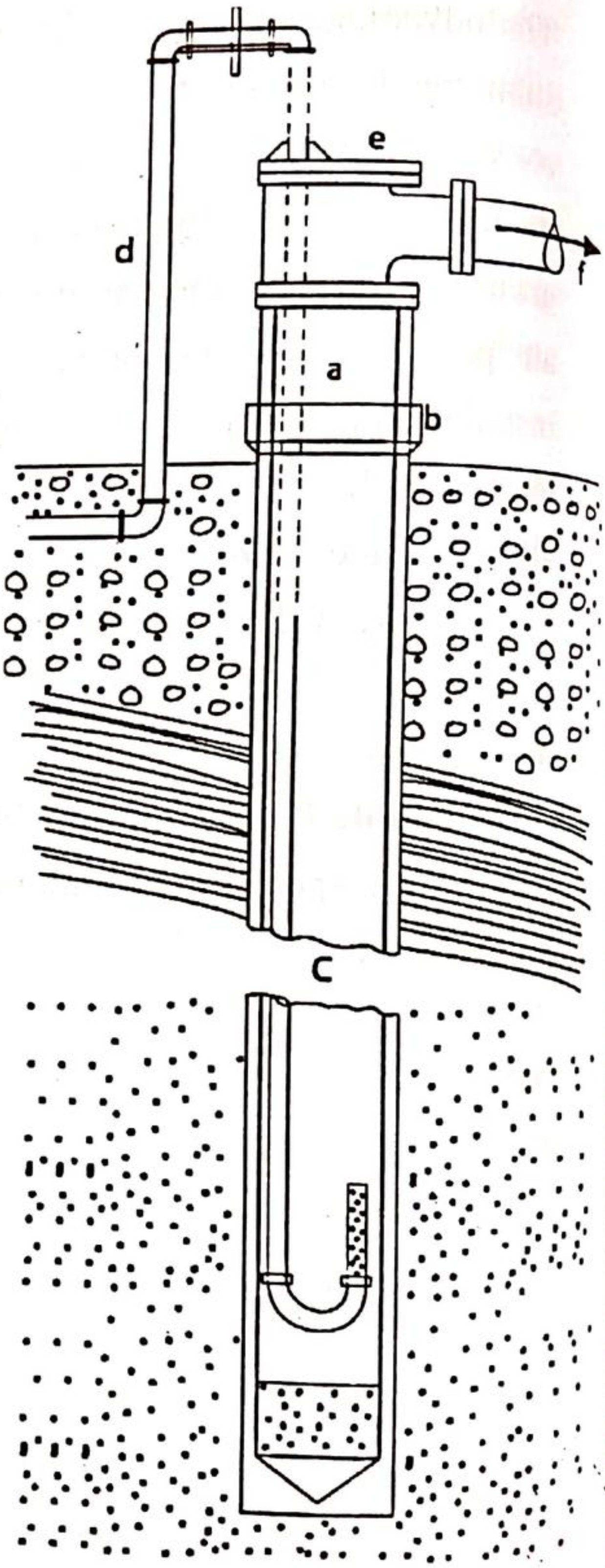


Fig. 5.3 Air Lift Pump

pump and submerged at all times in the well. The water pumped may be prevented from coming into contact with the electrical parts or the motor bearings by enclosing in an oil-filled case with a murcury seal where the shaft passes through the top, though modern practice is to allow the windings to be surrounded by the well water which is considered to act as a coolant and bearing lubricant Motors (usually squirrel cage induction type) used with these pumps are designed for long service without attention should a motor failure occur, the entire pump-must be lifted from the well, a disadvantage in extremely deep-well.

Submersible pumps are extremely useful both for shallow and deep wells and especially where the well is poorly alonged or crooked. They are easy to instal and can be made of smaller diameter. Thin efficiency, however, is. comparatively less.

5.3 General Considerations For Pumping Installations :

Suction Lift: The theoretical maximum suction lift for pumps is approximately 34 ft.; but, because of various losses, such as friction losses in the pipe and strainer, and loss due to velocity head, this limit can never be reached in practical works. The elevation of any pump above the source of water supply should not exceed 22ft. and for a centrifugal pump, the practical limit is about 15 ft. The suction pipe should be short, straight and of ample diameter. Strainers, when used should have a clear waterway area that is at least $1^1/_2$ times the cross, sectional area of the suction pipe.

Power for Pumps : The cost of pumping is mainly the cost of power to operate the pump, so that in areas of limited economic means the use of power is of utmost importance. Stream engines, the gasoline engines, diesel engines, and electric motors are commonly used to derive pumps. Electricity is to be preferred if it is available at resonable cost. Electric, motors are reasonably low in original cost and are cheap to operate.

Stand-by or Reserve Power : No matter what type of power employed, it is desirable to provide some extra pump units for use in case of breakdown or when repairs to the regular pumps are necessary.

Size of Units: A water-work pumping station is not operated at full capacity all the time. This factor has considerable tearing on the sizes of the pumps that are installed in any particular station. As previously explained. the efficiency of pumping engines varies with the amount of load. For example, the full load efficiency of the pump may be 80 per cent, the half-load efficiency, not greater than 75 per cent and the quarter-load efficiency not greater than 50 per cent. Consequently, the complete pump installation should be so designed that some of the pump units may be operated of the full capacity at all times, regardless of the variation in demand. When the demand is increased, additional pumps may be started, and when the demand is decreased, some pumps may be stopped.

Total Lift of Pump : The total lift of pump or the total head against which the pump must operate, is the algebraic some of the following quantities: The discharge lift or the vertical distance from the centre of the pump to the level to which the water is to be raised; the suction lift or the vertical distance from the level of the source of water supply to the centre of the pump; the friction head and the minor losses in the discharge and suction pipes, the velocity head.

The suction lift may be positive or negative, depending upon whether the pump is above or below the level of the source of water supply. Thus, if the pump is set 6 ft. above the surface of the source the suction lift of 6 ft. is positive and is to be added to the discharge lift in determining the total lift. But if the pump is 6 ft. below the surface of the source, the suction lift of 6 ft. is negative and is to be deducted from the discharge lift.

5.4 Horsepower Required :

When the volume of discharge and the total lift or head of the pumps are known, the theoretical horsepower required may be found by the formula.

$$P = \frac{HQ}{3.960} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (5.1)$$

in which P= theoretical horsepower required to operate the pump (W.H.P. = Water Horse Power):

H = total lift or head of the pump:

Q = volume of water to be pumped. gpm.

If the head to be pumped against is given in psi, the formula becomes.

$$P = \frac{QP}{1.715} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (5.2)$$

in which P = intensity of pressure, psi.

The actual horsepower required depends on the efficiency of the pump and may be found by relation.

$$P_1 = \frac{P}{E} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (5.3)$$

in which P_1 = actual horsepower required to operate the pump (B.H. P = Break Horse Power)

E = efficiency of the pump.

PROBLEM 1 : It is required to pump water at the rate of 6.750 gpm from a reservoir whose surface is at an elevation of 180 ft. to a tank whose bottom is at an elevation of 372 ft. The pump is placed at an elevation σf 192 ft., the diameter of the suction pipe is 30 inch, the length of the pipe from the pump to the tank is 290ft. and the estimated size of this pipe is 24 inch. The sum of tbe minor head losses in the suction and discharge pipe may be taken as 1.5 ft. If the maximum depth of water in the tank is to be 25 ft., what is the required horsepower of a pump for which the overall efficiency is 67 per cent ?

Assume head loss due to friction in 290 ft.+1.5 ft.

Neglect all other head losses.

Solution :

Elevation of water surface in the tank = 372 + 25=397 ft.

Discharge lift or the vertical distance from the centre of the pump to that surface =397 -192 =205 ft.

Suction lift, or the vertical distance from the water surface in the reservoir to the centre of the pump = 192-180 = 12 ft.

Since that pump is above the water surface this lift *a* positive.

Total head H = 205+12+1.5+1.5 = 220 ft.

$$P = \frac{HQ}{3{,}960} = \frac{220 \times 6{,}750}{3{,}960} = 375\,(\text{W.H.P})$$

$$P_1 = \frac{375}{0.67} = 560(BHP)Ans$$

PROBLEM 2 : Design a suitable set of pumping unit to deliver 4,50,000 gph from an intake well of a river bank to the treatment plant. Total length of rising main from the intake well to the treatment plant is 800ft. and the static head is 60 ft-Design also the cast iron main.

Assume: Velocity of water = 12 fps

friction factor = 0.0075

efficiency = 70%

Solation.

$Q = 4{,}50{,}000 \div 60 = 7{,}500$ gpm

again $Q = \dfrac{4{,}50{,}000}{60 \times 60 \times 6.24} = 20$ cfs

Cross-sectional area $= \dfrac{20}{12} = 1.667$ sq. ft.

$\dfrac{\pi d^2}{4} = 1.667$ (d = diameter of pipe)

$d = \sqrt{\dfrac{1.667 \times 4}{\pi}} = 1.5$ ft $= 18$ inch

Frictional head loss h_f

$= \dfrac{4flv^2}{2gd} = \dfrac{4 \times 0.0075 \times 800 \times (12)^2}{2 \times 32.2 \times 1.5} = 36\text{ft.}$

Velocity head, $hv = v^2/2g = (12)^2/2 \times 32.2 = 2.24$ft.

Total head $H = h_S + h_f + h_v = 60 + 36 + 2.24 = 98.24$ ft

$P = \dfrac{HQ}{3{,}960} = \dfrac{98.24 \times 7{,}500}{3{,}960} = 188$ (WHP)

$P_1 = \dfrac{180}{0.70} = 265$ (BHP) Ans.

PROBLEM 3 : Water is supplied from an impounding reservoir 30 miles away to a service reservoir near the town. A cast iron main is to be designed to supply 425 mgd. Loss of head due to friction in the pipe is estimated to be 300 ft. All other head looses are neglected. What size cast iron pipe would you use?

Assume f = 0.0075

Solution :

$$h_f = \frac{4 \times 0.0075 \times (30 \times 5280) \times v^2}{2 \times 32.2 \times d} = 300$$

$$\frac{v^2}{d} = 4.06 \quad \therefore V^2 = 4.06 \times d \text{....1)}$$

Again. Q = 425 mgd = 425×1.547 = 787 cfs

[1 mgd = 1.547 cfs]

$$Q = av = \frac{\pi d^2}{4} \times v \quad \therefore \; v = \frac{4Q}{\pi d}$$

$$\therefore \; v^2 = \frac{16Q^2}{(\pi d)^2}$$

Substituting the value of v^2 from Eq. 2 to Eq. 1.

$$\therefore \; \frac{16Q^2}{\pi^2 d^5} = 4.07$$

$$d^5 = \frac{16 \times (787)^2}{\pi^2 \times 4.07^2} = 246{,}800$$

$\therefore$ d=11.98 $\simeq$ 12ft. Ans.

Problem 4 : Design a pumping unit capable of lifting 5 mgd of water from an intake well to the treatment plant against a static head of 60 ft ; length of suction main is 120 ft and that of rising main is 400 ft. The pump will work in two shifts of eight hours each.

Assume : velocity of flow = 6 fps

friction factor $= 0.01$

efficiency $= 75\%$

Solution :

Total length of pips = 120+400 = 520 ft.

$$\text{Discharge, } Q = 5\text{mgd} = \frac{5\times10^6}{6.24} = 8\times10^5 \text{ cu ft/day}$$

Since total pumping time is 16 hrs/day,

$$\text{Pumping capacity} = \frac{8\times10^5}{16} = 50{,}000 \text{ cuft/hr.}$$

$$= \frac{50{,}000}{60\times60} = 13.9 \text{ cfs}$$

$$Q = \frac{\pi d^2}{4} = 13.9$$

$$\therefore d = \sqrt{\frac{4\times13.9}{\pi\times6}} = 1.7 \simeq 21.75\text{ft} = 21 \text{ inch}$$

$$h_f = \frac{4\times0.01\times520\times6^2}{2\times32.2\times1.75} = 6.7\,ft$$

$$h_v = \frac{6^2}{2\times32.2} = 0.56\,ft$$

$$H = h_s+h_f+h_v = 60+6.7+0.56 = 67.26\text{ft}$$

$$Q = \frac{50{,}00{,}000}{16\times10} = 5{,}220\,gpm$$

$$P = \frac{HQ}{3{,}960} = \frac{67.26\times5{,}220}{3{,}960} = 88.5 \text{ (WHP)}$$

$$P_1 = \frac{P}{E} = \frac{88.5}{0.75} = 118 \text{ (BHP) Ans.}$$

Problem 5 : Design the transmission main and the pumping unit from the following data :

Water supply rate = 40 gpcd

Estimated population=85,000

Ground R. L. = at the pum house = 102.50 ft.

Treatment plant R. L. = 193.00 ft.

Velocity through pipes = 8 fps

Pumping time = 10 hrs. daily

Total length of pipe = 3,500 ft.

Friction factor = 0.01

Efficiency = 65%

Solution :

Total water required = 40×85,000 =34,00,000 gpd

$$= \frac{34{,}00{,}000}{6.24} = 5.45\times10^5 \text{ cuft/day}$$

$$\text{Pumping rate} = \frac{5.45\times10^5}{10} = 5.45\times10^5 \text{ cuft/day}$$

$$= \frac{5.45\times10^5}{60\times60} = 15.15 \text{ cfs}$$

$$Q = \frac{\pi d^2}{4} = \times v = 15.15$$

$$\therefore d = \sqrt{\frac{4\times15.15}{\pi\times8}} = 1.56 \text{ ft.}$$

Use a 21 inch diameter pipe d = 1.75 ft.

Static head = 193.00 -102.50 = 90.50

$$\text{Friction head} = \frac{4\times0.01\times3.00\times8^2}{2\times32.2\times1.75} = 80.0 \text{ ft}$$

$$\text{Velocity head} = \frac{v8^{2.}}{2\times32.2} \quad 1.0 \text{ ft}$$

Total head, H = 90.5+80.0+1.0=171.5ft

Discharge, $Q = \dfrac{34,00,000}{10 \times 60} = 5,667$ gpm

$\therefore P = \dfrac{HQ}{3,960} = \dfrac{171.5 \times 5667}{3,960} = 246$ (WHP)

$\therefore P_1 = \dfrac{P}{E} = \dfrac{246}{0.65} = 380$ (BHP) Ans.

QUESTIONS

1. State the purposes or pumps and pumping machinery in water supply systems.

   Water supply to a small town with ultimate population of one lakh (1,00,000) supplied with 60 gped has to be arranged from a river flowing nearby. Design the economical section of the rising main and the necessary pumping unit from the following data;

Static head	60 ft
Total length of pipe	520 ft.
Coefficient of friction	0.01
Velocity of water in the pipe	6 ft/sec
Pump efficiency	70%

   The pumps will work in two shifts of six hours each in a day, (BUET, 1972)

2. Write the characteristics of centrifugal pumps. Design a suitable pumping unit and the size of the transmission main delivering water from a source 500 yds. away to a treatment plant or a small town having the design. Population of 2.5 millions supplied with 50 gpcd against a frictional head of 70ft. The pump will operate only 8 hrs, in a day. Take pump efficiency =65% and f= 0.01. Neglect all other head losses. (BUET, '71).

3. State the general considerations for pumping installation. What is the theoretical horsepower required for a pump to

raise 1800 gpm of water against a total head of 150 ft. including all losses ?

Which type of pump do you suggest and why? (BUET, '68)

4. What is the total lift of a pump ?

Design a pumping unit to transmit water from a source to a treatment plant of a snail town Having ultimate oppulation of 80,000 supplied with 50 gped of water.

Given : R. L. of the ground at pump hours = 98.20 ft.

R. L. of the entry site ground to a treatment plant = 154.60ft.

Length of pipe line = 2000ft.

Velocity of water through the pipe = 8 fps.

Friction factor = 0.0075

Pump efficiency = 65% (BUET, '67)

5. A multistoried building requires 15,000 gpd of water. The water will be supplied by a 3 inch diameter well. Design a suitable pumping unit from the following data : Suction head =12 ft. Delivery head= 180 ft. Size of the suction and delivery pipe = 2 inch Velocity of water through pipes = 6 ft/sec. Friction factor = 0.01 Assume reasonable values of data not supplied. (BUET, 1972)

6. Write explanatory notes on :

Deep well turbine Pump, Air Lift Pump, Submersible pump. Suction lift.

6

HYDRAULICS OF FLOW

6.1 Flow in Gravity Pipes :

In planning a water-supply system, it is occasionally necessary to design a gravity conduit, or open channel, to transport the water from the source of supply to the distribution system or treatment plant. The velocity of flow in a gravity conduit is usually computed either by applying Kutter's and Chezy's formula. The results are very nearly the same by both methods, but Manning's formula is more convenient for general use.

Chezy's formula for velocity is

$$V = C\sqrt{rs} \quad \text{............................} \quad (6.1)$$

in which V = velocity, in feet per second ;

C = a coefficient, which depends on roughness of surface, hydraulic radius, and slope of conduit ;

r = hydraulic radius, in feet ; .

s = slope of conduit.

The value of C in Chezy's formula is computed by Kutter's formula, which may be written in the following form :

$$C=\frac{\frac{1.81}{n}+41.7+\frac{0.00281}{s}}{1+\frac{n}{\sqrt{r}}\left(41.7+\frac{0.00281}{s}\right)} \quad (6.2)$$

in which C, s, and r have the same meanings as in formula 6.1.

n = coefficient of roughness of surface.

Values of n for various meterials are as follows : 0.011 for smoothly-lined steel pipe, extremely smooth concrete, 0.012 for new cast-iron pipe; 0.013 for good vitrified pipe, unlined riveted steel pipe, or cast-iron pipe 15 to 20 years old ; 0.014 for ordinary concrete-lined channels; and 0.015 for concrete pipe. When unlined riveted steel pipe is used, the effective diameter is considered to be the clear diameter between rivet heads.

Manning's formula of velocity is

$$V=\frac{1.49}{n}r^{2}-\frac{1}{s2} \quad \text{.................................} \quad (6.3)$$

6.2 Flow in Simple Pressure Pipes :

In designing a water distribution system, the general formulas for the flow of water in pipes are used to compute the size of a pipe necessary to supply a required amount of water under a given pressure, and to determine the amount of water that may be expected from a pipe of a given size in which the pressure is known. Generally, the amount of water required in any particular locality or district is known only approximately, and often the anticipated demand is bssis on estimates of conditions 20 or more years in the future. Under such circumstances and, indeed in most computations in water supply, extreme accuracy is not warranted. However, the original data must be carefully compiled, and care in the selection of assumed values is essential.

In all formulas relating to the flow of water in pipes, an important quantity is the static head, or simply the head, which is the difference in elevation between the surface of the source of supply and the point of discharge. Then this pipe discharges into a body of water, the surface of that body is considered and not the end of the pipe. If, for example, the elevation of the surface of the water in a certain reservoir is 370 feet, and this reservoir is connected by a pipe with another reservoir in which the surface is at an elevation of 230 feet, the head is 370 - 230 = 140 feet.

In engineering work, elevations are referred to some fixed level surface, called a datum, which is usually assumed to be below the entire work to be executed. Near the coast, the datum generally selected is sea level. If work is done below sea level, either the elevations of points below that surface are indicated by writing, a minus sign before each value, or else the datum is changed so that all points will be above it.

6.3 Flow in Long Pipes:

There are various types of formulas, charts, and tables for determining the flow of water in long pipes. For ths solution of problems, use will be made of the monographic chart shown in Fig, 6.1. This chart applies to circular pipes, that is, to pipes of circular cross-section, and is based on the Hazen-Williams formula, which may be written in the following form :

$$V = 1.318C\, r^{0.63} s^{0.54} \quad \text{.................................} \quad (6.4)$$

in which V=velocity of flow, in feet per second ;

C = a coefficient, which depends on the type and condition of the conduit:

r = hydraulic radius in feet ;

s = slope of hydraulic gradient, or loss of head in feet per foot of length.

For a circular pipe that is flowing full, as is the case for a pressure conduit, the hydraulic radius is one-quarter of the diameter d and the Hazen-Williams formula becomes

$$V=0.55Cd^{0.63}s^{0.54} \quad \text{........................} \quad (6,5)$$

Values of C for various types and condtions of pipe are as follows 140 for smoothly-lined steel pipe; for very smooth concrete, and for smooth new cast-iron pipe, 130 for ordinary cast-iron pipe in good condition, 120 for cast iron pipe 5 years old and for welded steel pipe ; 110 for cast iron pipe in service about 10 years and for new unlined riveted steel pipe ; 100 for cast-iron pipe 15 to 20 years old ; and 95 for unlined riveted steel pipe in service.

6.4 Use of Chart for Circulars Pipes :

The chart shown in Fig. 6.1 may be used for solving all problem of flow in circular pipes that are flowing full and under pressure and for which it may be assumed that the only cause of loss of head is simple friction. The chart has four vertical lines, the graduations along which are, respectively, for discharge, dia-meter of pipe, loss of head, and velocity. Two sets of graduations are given for discharge, one set being for cubic feet per second and the other set for gallons per minute. The loss of head given on the chart is in feet per 1,000 feet of pipe; it is equal to 1,000 times the slope s of the hydraulic gradient. The chart applies directly to pipes for which C in the Hazen-Williams formula is 100. However, it can be readily used also for any other value of C by applying a correction.

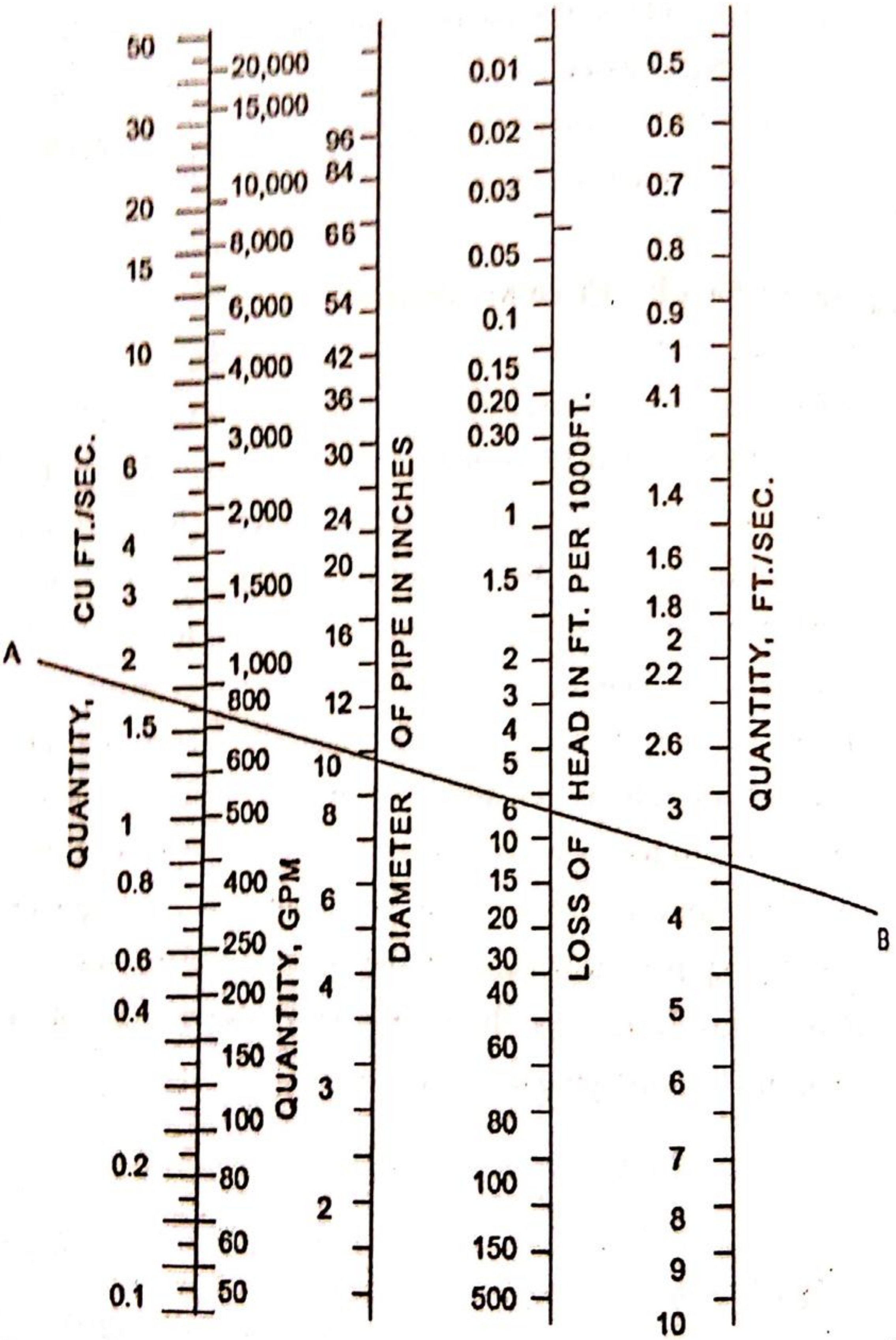


Fig. 6.1 Nomograph for Hazen --Williams Formula in which

C=100.

When any two of the four quantities shown on the chart known for a certain pipe, the corresponding values of the other two quantities for C = 100 can be found directly from the chart by placing a straightedge through the given values on the proper two vertical lines and observing where the edge intersects each of the other two vertical lines. For example, if the discharge from a 10 inch pipe for which C is 100 is to be 825 gallons per minute, the velocity and the loss of head may be found as follows : A straight edge is placed on the chart so as to pass through the value representing 10 inches on the vertical line for diameter of pipe and also through the value representing 825 gallons per minute on the line for discharge, as indicated by line AB on the chart, Since the edge then intersects the vertical line for velocity at a point representing 3.37, the velocity is 3.37 feet per second. Likewise, the loss of head is found to be 7 feet per 1,000 feet of length.

The velocities or discharges for two pipes having the same diameter and the same slope are directly proportional to the values of the coefficient C of the pipes. Therefore, in order to use the chart when the value of C is not 100, it is merely necessary to apply a correction to the velociy or discharge. If it is desired to determine the velocity or discharge when the diameter, the slope, and the value of C are given, the velocity or discharge for a value of C 100 is first obtained from the chart, and that velocity or discharge is multiplied by the ratio of the given value of C to 100. On the other hand, if the velocity or discharge is given and it is required to find the diameter for a given slope or the slope for a given diameter, it is first necessary to determine the velocity or discharge that would apply if the value of C were 100, by multiplying the given velocity or discharge by the ratio of 100 to the given value of C. Then, this modified velocity or discharge is used in setting the straightedge on the chart as if the given value of C were 100.

Example 1: What is the discharge, in cubic feet per second, from a 12-inch pipe for which C = 20, if the loss of head per 1,000 feet is 8 feet ?

Solution: The discharge corresponding to the given diameter and loss of head and to a value of 100 for C is found first. A straight edge passing through 12 on the diameter line and 8 on the loss of-

TABLE 6.1
VELOCITY AND LOSS HEAD IN SMALL PIPES
(Based on Hazen Williams formula in which C=100)

	½ Inch Pipe		¾ Inch Pipe		1 Inch Pipe		$1^1/_2$ Inch Pipe		2 Inch Pipe	
Discharge Gallons per Minute	Velocity Feet per Second	Loss of Head Feet per 100 Feet	Velocity Feet per Second	Loss of Head Feet per 100 Feet	Velocity Feet per Second	Loss of Head Feet per 100 Feet	Velocity Feet per Second	Loss of Head Feet per 100 Feet	Velocity Feet per Second	Loss of Head Feet per 100 Feet
1	*1.05*	*2.1*								
3	*3.16*	*15.8*	*1.80*	*4.1*	*1.12*	*1.26*				
5	*5.26*	*41*	*3.01*	*10.5*	*1.86*	*3.25*	*0.79*	*0.40*	*1.02*	*0.50*
10	*10.52*	*147*	*6.02*	*38*	*3.72*	*11.7*	*1.57*	*1.43*	*1.53*	*1.06*
15			*9.02*	*80*	*5.58*	*25*	*2.36*	*3.03*	*2.04*	*1.82*
20			*12.03*	*136*	*7.44*	*42*	*3.15*	*5.2*	*2.55*	*2.73*
25					*9.30*	*64*	*3.94*	*7.8*	*3.06*	*3.84*
30					*11.15*	*89*	*4.72*	*11.0*	*4.08*	*6.6*
40					*14.88*	*152*	*6.30*	*18.8*	*5.11*	*9.9*
50							*7.87*	*28*	*6.13*	*13.9*
60							*9.44*	*40*	*8.17*	*24*
80							*12.59*	*68*	*10.21*	*36*
100							*125.74*	*102*	*12.25*	*50*
120									*14.30*	*67*
140									*16.34*	*86*
160										

head line will intersect the discharge line at 3.2 c.f.s. Hence, the required discharge is

$$32\times\frac{120}{100}=3.8cfs$$

Example 2: A. 10-inch pips for which C is 130 is discharging 825 gallons, per minute What is the loss of head per 1,000 feet of pipe ?

Solution : The first step is to determine the modified discharge, or the discharge that would correspond to a value of C of 100 this-is

$$825\times\frac{100}{130}=635\ \text{gpm}$$

A straightedge through 10 on the diameter line and 635 on the discharge line intersects the loss-of-head line at 4.2 ft. per 1,000 ft.

6.5 Influence of age on flow in Pipes :

The capacity of a metal pipe is considerably reduced after a number of years of service. Tests-on the capacity of a 30 inch cast-iron pipe for which the slope of the hydraulic gradient was 0.00224 showed the following results. The original capacity of the pipe was 15 million gallons per day; at the end of 10 years, the capacity was about 13 mgd; at the end of 20 years, it was about 11.4 mgd; and at the end of 30 years.it was about 10.2 mgd.

6.6 Loss due to organic Growths and other Losses:

Pipe capacity may also be materially reduced by organic growths. It is difficult to predict losses due to organic growths. Studies of existing lines carrying similar water are necessary as a guide in design and in preventive treatment of the water.

Valves, bends, and other fittings and sudden enlargements or contractions cause losses of head. If a valve is partly closed, the loss of head is increased.

6.7 Flow in Service Pipes :

The chart in Fig. 6.1 is not suitable for the solution of problems on flow in service pipes, because their diameters are usually too small to permit the use of that chart. In Table 6.1 are given values of the velocity and the loss of head corresponding to various discharges, in gallons per minute, from pipes with diameters of ½, $^3/_4$, 1, $1^1/_2$ and 2 inches. The values in this table are based on the Hazen-Williams formula in which C=100. There is almost no corrosion in lead, lead-lined, brass, or copper service pipes, and the age of such pipes is not an important consideration in design.

QUESTIONS

1. Explain briefly the hydraulics of flow in simple gravity pipes (BUET, '61)
2. Find (a) the velocity in fps, and (b) the discharge in cfs for a 30 inch cast-iron pipe 15 years old, when the loss of head is 4ft. per 1,000 ft. of length. (BUET,'68)

   Ans. (a) 5 fps, (b) 24,5 cfs,
3. Critically discuss the hydraulics of flow in long pressure pipes (BUET, '70)
4. An 8 inch pipe, for which C=130, is discharging 500 gpm. What is the loss head per 1,000 ft. of the pipe? (BUBT, '66)

   Ans. 5 2 ft. 5
5. Write short notes on :

   Head, Influence of age on flow in pressure pipes, Manning's Roughness coefficient.
6. A Service pipe $1^1/_2$ inch diameter and 40 ft. long is discharging 50 gpm. Determine (a) the velocity and (b) the total head loss in the pipe. (BUET, 1964)

   Ans : (a) 7.86 fps.

   (b) 11.40 ft

7
WATER QUALITY

7.1 Impurities Present in Water :

To slake man's thirst and; other needs, water must be pure. But pure water is not found in nature. There are various impurities present in natural water. The impurities present in water are classed into four different groups: (I) Impurities of Mineral Origin, (2) Impurities of Organic origin, (3) Living Impurities, and (4) Radioactive Impurities. The Table 7.1 shows the various types of impurities present in water.

TABLE 7.1 IMPURITIES PRESENT IN WATER

1	2	3	4
In Suspension : Silt, clay and colloids	Decomposable organic matter in sewage, industrial wastes, plants, leaves and organic colouring matter	Bacteria, algae, Protozoa, fungi and other living organisms,	Radioactive substances like Sodium, uranium, cobalt, etc.
In Solution : Carbonates, bicarbonates, sulphates and sulphides of potassium, sodium, calcium, and magnesium, hydroxides of iron. etc.	organic matter, organic acids, etc.		
In pseodo solution : Silica, alumina, iron oxides, etc,	Colloidal decompossable organic wastes such as animal secietions, etc.		
Dissolved gases			
oxygen, carbon-dioxide, nitrogen, ammonia, hydrogen etc.	Methane, hydroxide sulphide, etc.		

The Table 7.2 shows the classification of impurities of raw-waters according to behaviour in conventional water-works treatment plants.

Table 7.2 Classification of Impurities In Raw-waters According To Behaviour In Conventional Water-Works Treatment Plants

Class-A Impurity cannot be reliably removed.	Class-B Impurity cannot be reliably removed to within acceptable limits	Class-C Impurity interferes with treatment processes.
Chloride Phenolic Fluoride substance, Sulphate Petroleum Nitrate hydrocarbons Arsenic Dyestuffs Lead Synthetic Copper organic Zinc wastes, Barium Radioactive Selenium substances, Chromium synthetic Cadmium detergents Calcium Cyanides Magnesium Alkali metals	Bacteria, Inorganic suspended matter, Dissolved natural organic matter (colour). Dissolved natural organic matter (colourless) Iron Manganese Calcium Magnesium Free carbon dioxide	Ammonia Phosphates nutrients planktonic algae, etc.

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7.2 Effects of Impurities : The table 7.3 shows the effects of impurities present in water.

Table 7.3 Effects of Impurities

Name of the Impurities		Effects
Algae, Protozoa, Fungi,		Diseases
Clay and colloids.		Diseases; odour, colour, turbidity
		Turbidity
Salts of calcium and magnesium	Carbonates	Hardness, alkalinity, taste corrosiveness, scale formation.
	bicarbonates	Hardness, alkalinity, taste, corrosiveness, scale formation.
	sulphates	Hardness taste
	chlorides	Hardness, corrosiveness, taste.
Salts of sodium	Carbonates	Alkalinity,
	bicarbonates	Alkalinity,
	sulphates	Foaming and scaling
	fluorides	Tooth-decay
	Chlorides	Taste
Iron Oxides		Taste, red water hardness, corrosiveness.
Manganese		Black or brown water.
Vegetable dyes		Colour, acidity
Gases	Oxygen	Corrosive to metals
	Carbon dioxide	Acidity, corrosive to metals
	Hydrogen sulphide	Odour, acidity, corrosive to metals
	Nitrogen	Child-disease, algal growth
Radioactive Impurities		Diseases

7.3 Living Organisms Present in Water : The following is a brief description of the organisms present in water.

(1) Bacteria : Bacteria are the basic plant unite being the simplest form of plant life. Many people find it difficult to believe that bacteria are plants and not animals. This erroneous impression is derived from the fact that plant life is associated with photosynthesis and the presence of the green colour pigment, chlorophyll. Most bacteria, especially the ones commonly found in nature, do not contain chlorophyll and are colourless. Bacteria are classed as plants because of their structure and method of food intake. They are single-cell organisms which utilize soluble food. Each cell is an independent organism capable of carrying out all the necessary functions of life.

Bacteria are found everywhere in nature, in water, in soil and in air. Most of the bacteria are found in water and soil. Bacteria come into three shapes : (i) Rods, (ii) Spheres, and (c) Spirals. These three bacterial shapes are shown in Fig.7.1. The technical names for these shapes are bacillus for rod, coccus for sphere, and spirillum for spiral.

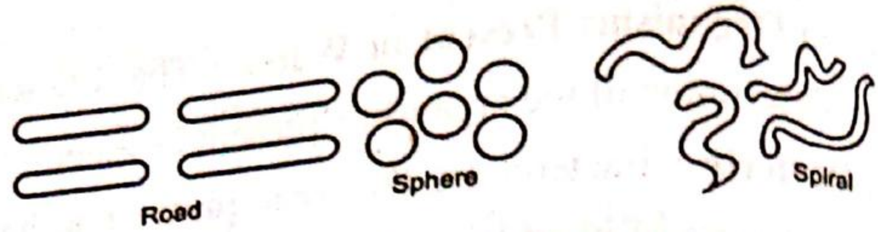
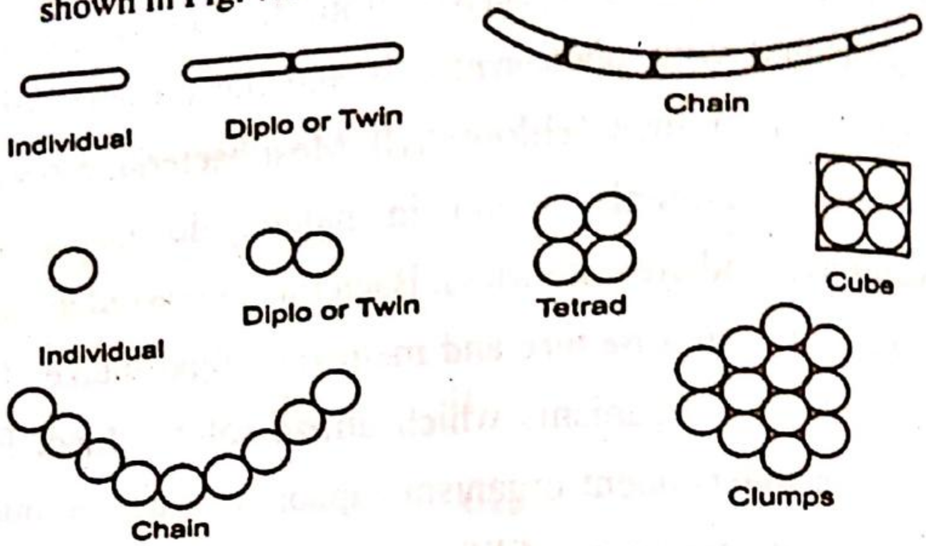


Fig. 7.1 Three Common Shapes of Bacteria

The forms that these three shapes of bactaria take vary, as shown in Fig. 7.2



The rod is the most common bacterial form and can be- observed in three distinct groups : (a) individual cells, (b) diplo-or twin cells, and (c) Chains of cells.

The spheres have the most different groups : (a) individual, (b) diplo. (c) tetrad, (d) cube, (e) chain, and (f) clumps.

The spiral form occurs primarily as individual cells and diplos. The diplo form is merely the intermediate form during growth and division.

The size of the individual cells vary over rather wide limits. The cell size changes with time during growth and death. The limits of the cell sizes range from 0.3 micron to 50 microns. The limit of the common bacteria are from 0.5

micron to 3.0 micron The average rod is from 0.5 to 1.0 micron wide and 1.5 to 8.0 microns long. The average sphere is from 0.5 to 1.0 micron in diameter, while the average spiral cell is from 0.5 to 5 microns wide and 6 to 15 microns long.

According to metabolism (metabolism determines the bacteria's ability to grow in any environment), bacteria are divided into two distinct groups (a) Autotrophic Bacteria and (b) Heterotrophic Bacteria.

Autotropaic Bacteria : The Autotrophic Bacteria are the most complex group of bacteria from a bio-chemical standpoint. These bacteria have the ability to make all the complex chemical structures within the bacterial cell from the basic inorganic chemicals in water. The autotrophic bacteria have not been studied extensively because of their limited importance in water supply engineering.

Heterotrophic Bacteria: The heterotrophic bacteria are the most important group of bacteria. They require organic compounds which supply their carbon and energy. The heterotrophic bacteria can be divided into three groups based on their relationship with oxygen in energy reaction. The heterotrophs which utilize free oxygen are known as aerobes. The heterotrophs which oxidize organic matter in complete absence of dissolved oxygen are known as anaerobes. There is a group of bacteria which utilize free oxygen when it is present but which can also carry on metabolism in the absence of free oxygen. This latter group of bacteria is known as facultative. The facultative bacteria have sometimes been designated as facultative aerobes or facultative anaerobes.

Pathogenic Bacteria: Pathogenic bacteria (disease-producing bacteria, commonly termed as pathogens) present in water include Bacillus Typhi, Salmonella Para-Typhe, Shigella Dysenteriae and Vibrio Comma.

(2) Fungi: Fungi are similar to bacteria. In fact, to be technical, the bacteria are actually fungi, fission fungi. The importance of bacteria has caused them to be singled out and studied separately from the fungi. This fact has caused common usage of the term fungi to mean all fungi except bacteria. Two other terms which require consideration are mold and yeast. Mold is synonymous in common usage with fungi. Yeasts are a part of the fungi but like the bacteria, their importance has caused a separation from fungi similar to bacteria.

By definition, fungi include the nonphotosynthetic plants and except bacteria, they are multicellular. The absence of photosynthetic pigments (chlorophyll) requires the fungi to utilize organic matters as their source of carbon and energy. It is this property of metabolism of organic matter that makes both the fungi and bacteria importaht to the public health engineers.

Fungi and aerobic organisms reproduce by spore-formation. Fortunately, there are few fungi which attack man, although there are large number of fungi which are pathogenic to plants. Growth of fungi in higher animals can occur only on the body surfaces in the blood stream or in the lungs spere there is an ample supply of dissolved oxygen.

(3) Algae : The third group of microscopic plants are the algae. The algae differ from the fungi and bacteria in their ability to carry out photosynthesis. The algae can utilize the energy in light and do not have to depend upon the oxidation of matter to survive. In fact, the algae evolve oxygen during their growth in presence of sunlight.

The evolution of oxygen by algae and their production of taste producing oils have made the algae of extreme interest to the public health engineers. The oxygen production by algae is very helpful in maintaining the oxygen reserve of water resources. On the other hand, the excess growth and death of some algae in water produces taste, odour and colour, sometimes, algae serve as the dwelling place of pathogens.

There is no clear cut definition of algae which can satisfy every one. The simplest definition of algae is that it includes all microscopic plants carrying out true photosynthesis. Algae are autotrophic organisms.

(4) Viruses : The viruses are the smallest plants known at present time. They are intracellular parasites and are highly specific in their reactions, some with plants and some with animals, viruses are present both in air and water. Both types are pathogenic to man, animals and plants. The Table 7.4 shows the diseases caused by various types of viruses.

Table 7.4 Virus-Diseases

Type of viruses	**Types of Disease caused**	**Mode of Transmission**
Dermotropic	Various pox-like diseases (small pox, chicken pox and eye-infection)	Close contact, water, air, food and milk
Neurotropic	Pollimyelitis	Water-borne
Pneumotropic	Influenza, colds	Nasal and oral discharges, water, air, food and milk
Viscerotropic	Cancer	Water, food and milk.
Neoplastic	Eye diseases (Leukemia)	Close-contact, air and water.
Enteric	Diarrhoea, nauses," vomiting, Polio-like diseases and infectious hepatitis (Jaundice)	Water, food and milk

(5) Protozoa and other Higher Animals: Of equal importance with the microscopic plants are the microscopic animals. For most part of the microscopic animals are scavengers which tend to clean the excess bacteria from water and waste water.

In streams and lakes the lower animals form an important link in the gross aquatic biological cycle. The microscopic animals eat the microscopic plants and in turn are eaten by higher animals. **(a) Protozoa:** It is not possible to define protozoa easily. In fact, it has been considered by some as impossible to define. For our purposes, it suffices to say that the protozoa are single-celled animals which reproduce by binary fission. Most of the protozoa metabolize solid food materials and have more complex digestive systems than the microscopic plants. Bacteria are the main food for the protozoa.

Very few protozoa are pathogenic to man. The most common pathogenic protozoa of public health significance are Endamoeba Histolytica, Trypanosoma and Plasmodium.

(b) Rotifers: As the animal progresses to higher forms, it goes from a single-cell animal to a multicell animal. The rotifer is the simplest of the multicell animals. The rotifer is a strict aerobe and is normally found only when the water contains at least several milligrams per litre dissolved oxygen. Bacteria form the chief source of food for the rotifers, but they can ingest any small organic particles. Because of their metabolic habits, rotifers are found only in water of low organic content. They act as good indicators of low polluted waters.

(c) Crustaceans: Normally, crustacians bring to mind the hard-shelled animals such as the crayfish and the lobster. There are also microscopic crustaceans. The rigid shell structure is the chief characteristics of these multicellular microorganisms. The crustaceans are strict aerobes which feed on bacteria and algae. They are important as a source of food for fish. The metabolic complexity of the crustaceans limits their growth at relatively stable streams and lakes.

(d) Worms and Larvae : Worms and larvae are normal inhabitants in organic muds and biological slimes. Two common organisms usually found in water are the worm Tubifix and the midge fly larvae, chironomidae,

7.4 Water-Borne Diseases:

The important water-borne diseases are typhoid and paratyphoid fevers, amoebic and bacillary dysenteries, cholera, gastroenterities, small pox, schistosomiasis, tuberculosis, infectious hepatities (Jaundice), poliomyelitis, tularemia and anthrax.

Common modes of transmission other than through drinking water are (1) through fishes harvested from sewage polluted waters (typhoid, para-typhoid, bacillary dysentery, and infectious hepatitis (2) through vegetables and fruits contained by feces, sewage or sewage sludge (typhoid, paratyphoid, the dysenteries, cholera, small pox, infectious hepatitis); (3) through exposure to soil contaminated by human excreta (parasitic worms and hook-worms); (4) through all manner of food contaminated by flies, and other vermins that fed also upon human fecal matter (typhoid, paratyphoid, the dysenteries, cholera, small pox and infectious hepatitis) ; (5) through milk and milk products contained by utensils that have been washed in polluted water (cholera, paratyphoid, bacillary dysentery, and small pox) ; (6) through bathing or other exposure to polluted waters (schistosomiasis and other skin diseases).

Other chains of infectious link (1) tuberculosis to the milk of cows infected by drinking from sewage polluted streams running through pastures below tuberculosis sanatorium; (2) anthrax from water contaminated by tannery waste; (3) skin diseases from industrial waste polluted waters.

Typhoid and paratyphoid fevers, the dysenteries, cholera, gastroenteritis are transmitted by the intestinal and urinary discharge of sick persons to waters. Gastroenteritis is a diarrhoeal disorder. It resembles food poisoning and apparently some of the organisms that are involved in food poisoning are sometimes water-borne after having reached water by means of bowel discharges.

Infections hepatitis, also known as epidemic hapatitis and epidemic jaundice is caused by a virus which has been responsible for several water-borne epidemics but may also be transmitted by direct personal contact, food and milk.

Polyomyelitis, also a virus disease, is found in the faces of infected persons and in sewage. Hence it is supposed that the virus may occour in drinking water.

Schistosomiasis organisms are animal parasites which pass part of their life cycle in certain species of aquatic snails. After leaving the snail they are swimming forms known as cercariae, which may be drunk or may enter the skin of persons who wade or swim in water.

The Table 7.5 shows the bacteria and viruses responsible for various types of waterborne Diseases

Table 7.5 Water-borne Diseases

Name of Diseases	Organisms Responsible
1. Typhoid Fever	Bacillus typhi } Bacteria
2. Paratyphoid Fever	Eberthela typhosa } Bacteria
3. Amoebic Dysentery	Salmonella paratyphi (Bacteria)
4. Bacillary Dysentery	Endamoeba hystolytica (Protozoa)
5. Cholera	Shigella dysenteric (Bacteria)
6. Small-pox and chicken pox	Vibrio comma (Bacteria)
7. Gastroenteristis	Dermotropic viruses
8. Poliotmyelitis	Enteric viruses
9. Schistosomiasis	Neurotropic viruses
10. Infectious Hepatitis (Jaundice)	Cercarnie (animal parasites)
11. Anthrax	Enteric viruses
12. Tularemia	Bacillus anthracis (Bacteria)
13. Influenza, Colds	Spirochetes (Bacteria)
14. Pneumonia, Broncho-pneumonia	Pneumotropic viruses. Diplo Coccus pneumonia (Bacteria)
15. Diphtheria	Corynebacterium diphtheria (Bacteria)
16. Cancer	Viscerotropic viruses
17. Polio-like disease	Enteric Viruses
18. Urinary Inflamation	Escherichia coli (Bacteria)
19. Whooping cough	Hemophilus pertussis (Bacteria)
20. Tuberculosis	Mycobacterium tuberculosis (Bacteria)
21. Leprosy	Mycobacterium Leprae

Waterborne Poisons: A variety of poisons may conceivably find their way into public water supplies, among them (1) toxic subtances leached from mineral formation such as fluorapatitis; (2) phytotoxins produced by specific algae; (3) heavy metals dissolved from water-works structures principally metallic pipes and improperly manufactured plastic pipes, or added as water treatment chemicals; (4) poisonous compound contained in industrial and household wastes emptied into water courses; (5) radioactive substances in fallout and from the nuclear energy industry and (6) pesticides reaching water courses from chemical dusts and sprays applied to crops and to land and water surfaces for the control of agricultural blights, nuisanee insects vectors of human and animal diseases, water weed and the like.

Fluorine, selenium, arsenic, and boron are examples of natural mineral contaminants. Except for fluorine, their concentration in drinking water is seldom significant However, the mottling of tooth enamel is in fact caused by excessive concentrations of fluorides in water. By contrast, small amounts of fluoride are ingested with safety and lower the prevalence of dental caries. Large amounts of nitrates may cause an illness (methemoglo-binemia on nitrate cyanosis) of infants. Nitrates, moreover, may possibly be responsible, along with other goitrogenic substances for the widespread occurrence of goiter in districts with heavily polluted wells. High nitrates are also dangerous to crops, man and animals. Sudden death of cattle after drinking water supporting luxuriant growths of blue-green algae.

Contaminants acquired within distribution systems by corrosion of metals often evoke consumer complaints ; they rarely cause poisoning. Soft water high in carbon dioxide are quite corrosive. Among the metals easily corroded by such water is lead, a cumulative posion. However, this metal is no longer used in modern water works, and plumbism (lead poisoning) is disappearing as a waterborne disease. Copper, zinc, and iron impart a metallic taste. They may also discolour the water itself (zinc and iron), of fixtures (copper and iron) and fabrics (iron).

Ingestion of large concentrations of copper produces nausea this may preclude poisoning.

Chemicals added to water for its coagulation or disinfection, for the destruction of algae and water weeds, or for control of corrosion should obviously be harmless in themselves or in the concentrations used. Indeed, it should be an axiom of public health engineers not to apply new chemicals to water until evidence of their safety has become incontroversible.

Industrial, commercial, and agricultural operations can release a variety of toxic contaminants to water. Except for agricultural chemicals, their control is not difficult, but their presence is sometimes unsuspected.

Fallout from the testing of nuclear weapons and escape of low-energy radioactive wastes into water course call for the routine monitoring of water supplies for radioactivity; also of receiving water below processing and power plants.

Of increasing concern are the toxic substances added to the environment for the control of agricultural and other pests. Chlorinated hydrocarbons, such as DDT, and organic phosphates are examples. Because of the ways in which the chemicals are disseminated, residual amounts are bound to reach water courses and their plant and animal populations including food and game fish. Only rarely will they accumulate in concentrations high enough to cause acute poisoning in man, but they may be responsible for chronic poisoning during lifetime exposure. Indeed, man's increasing span of life subjects him to incidious and cumulative poisons in proportionate degree.

7.6 Palatability of Water :

To be palatable, water must be significantly free from turbidity, colour, taste and odour, of moderate temperature in summer and winter, and well aerated. At least four human perceptious respond to these qualities : (1) shows of sight (colour and turbidity), (2) taste (odour), smell (odour), and (4) touch (temperature).

Turbidity : The term turbid is applied to waters containing suspended matter that interferes with the passage of light through the water or in which visual depth is restricted. The turbidity may be caused by a wide variety of suspended materials which range in size from colloidal to coarse dispersions, depending upon the degree of turbulence. In lake or other waters existing under relatively quiescent conditions, most of the turbidity will be due to colloidal and extremely fine dispersions. In rivers under flood conditions, most of the turbidity will be due to relatively coarse dispersions. Turbidity is an important consideration in public water supplies for three major reasons.

Aesthetic : Consumers of public water supplies expect and have a right to demand turbidity-free water. Laymen are aware that domestic sewage is highly turbid. Any turbidity in the drinking water is automatically associated with possible sewage pollution and the health hazards occasioned by it. This fear has a sound basis historically, as any one knows who is familiar with the water borne epidemics.

Filterability : Filteration of water is rendered more difficult and costly when turbidity increases. The use of slow sand filters has become impractical in most areas because high turbidity shortens filter runs and increases cleaning cost. Satisfactory operation of rapid sand filters depend upon effective removal of turbidity by chemical coagulation before the water is admitted to the filters. Failure to do so result in short filter runs and production of an inferior quality water.

Disinfection : Disinfection of public water-supplies is usually accomplished by means of chlorine or ozone. To be effective, there must be contact between the agent and the organisms that the disinfectant is to kill.

In turbid waters, most of the harmful organisms are exposed to the action of the disinfectant. However, in cases where turbidity is caused by sewage solids, many of the pathogenic organisms may be encased in the particles and protected from the disinfectant. For this and aesthetic reasons, the USPHS has placed a limit of 10 units of turbidity as the maximum amount allowable in public water supplies.

Standard Unit of Turbidity : Because of the wide variety of materials that cause turbidity in natural waters, it has been necessary to use an arbitrary standard. The standard chosen was

1 mg SiO_2/1=1 unit of turbidity

and the silica used must meet certain specifications as to particle-size.

Application of Turbidity Data : Turbidity measurements are of particular importance in the field of water supply.

They have limited use in the field of sewage and industrial waste treatment.

Knowledge of the turbidity variation in raw-water supplies is of prime importance to the water supply engineer. He uses it in conjunction with other information to determine whether a supply requires special treatment by chemical coagulation and filtration before it may be used for a public water supply.

Water supplies obtained from rivers usually require chemical coagulation because of, high turbidity. Turbidity measurements are used to determine the effectiveness of the treatment produced with different chemicals and the dosages needed. Thus they aid in selection of the most effective and economical chemical to use. Such information is necessary to design facilities for feeding the chemicals and for their storage.

Turbidity measurements help to gauge the amount of chemicals needed from day to day in the operation of treatment works. Measurement of turbidity in settled water prior to filtration is of aid in controlling chemical dosages so as to prevent excessive loading of rapid sand filters. Finally, turbidity measurements of the filtered water are needed to check on faulty filter operation.

Colour : Many surface waters, particularly those emanating from swampy areas, are often coloured to the extent that they are not acceptable for domestic or some industrial uses without treatment to remove the colour. The colouring material results from contact of the water with organic debris, such as leaves, needles, and wood, all in various stages of decomposition. It consists of vegetable extracts of a considerable variety. Tannins, acid, and oximates, from the decomposition of lignin, are considered to be the principal colour bodies. Iron is sometimes present as ferric humate and produces a colour of high potency.

Natural colour exists in water primarily as negatively charged colloidal particles. Because of this fact, its removal can usually be readily accomplished by coagulation with the aid of a salt havirg a trivalent metallic-ion, such as aluminum or iron.

Surface waters may appear highly coloured, because of coloured suspended matter, when in reality they are not. Rivers which drain areas of red clay soils become highly coloured during times of flood. Colour caused, by suspended matter is referred to as apparent colour and is differentiated from colour due to vegetable or organic extracts that are colloidal and which is .called true colour. In water analysis it is important to differentiate between "apparent"' and "true' colour.

Surface waters may become coloured by pollution with highly coloured waste waters. Notable among these are wastes from dyeing operations in the textile industry and from pulping operations in the paper industry. Dye wastes may impart colours of wide variety that are readily recognized and traced. The pulping of wood produces considerable amounts of waste liquors containing lignin derivative's and other materials in dissolved form. The lignin derivatives are highly coloured and quite resistant to biological attach. Much of this material is disposed of into natural watercourses adding colour which persists for great distances. Considerable research is currently under way to find an economical way of removing colour from pulp-mill wastes. The USPHS recommends that waters intended for human use should not have a colour exceeding 20 units.

Standard Unit of Colour : Natural colour like turbidity, is due to a wide variety of substances, and it has been necessary to adopt an arbitrary standard for its measurement. This standard is employed directly and indirectly in the maasurement of colour.

Many samples require pretreatment to remove suspended matter before true colour can be determined.

Waters containing natural colour are yellow-brownish in appearance. Through experience, it has been found that solutions of potassium chloroplatinate ($K_2,PtCl_6$) tinted with small amounts of cobalt chloride yield colours that are very much like the natural colours. The shading of the colour can be varied to match natural hues very closely be increasing or decreasing the amount of cobalt chloride.

The colour produced by 1 mg/L of platinum (as K_2PtCl_6) is taken as the standard unit of colour.

Interpretation and Application of Colour Data : The colour of surface waters utilized for domestic supplies is of major concern for reasous mentioned above. Many industrial processes also require the use of colour free water. Removal of colour is an expensive matter when capital investment and operating costs are considered. Therefore the water supply engineer, when developing or looking for new supplies, is always searching for a suitable supply with a colour low enough so that chemical treatment will not be required. His "prospecting" may or may not be successful. If it is, he will use colour data as one of the parameters to satisfy his client fiat expensive chemical treatment is not necessary. If it is not succeseful, he will use colour data along with other information to prove that expensive chemical coagulation and sand filtration are needed to produce an acceptable supply.

Before a chemical treatment plant is designed, research should be conducted to ascertain the best chemicals to use and amounts required. In dealing with coloured waters. colour determinations serve as the basis of the decisions.

Such data must be obtained for proper selection of chemical feeding machinery and the design of storage space.

Once operation of the treatment facilities has begun, colour determinations on the raw and finished waters serve to govern dosages of chemicals used, to ensure economical operation and to produce a low-colour water that is well within the accepted limits.

Tastes and Odour : The words taste and odour are often used loosely and interchangeably. Actually, there are but four tastes : sour, salt sweet, and bitter-strictly confined in their perception to the taste buds of the tongue. Odours appear to be without limit in number and are known to change in quality as the concentration of the odourous compounds, or the intensity of their small, is varied. However, careful screening of odours suggests that there may be certain fundamental odours from which all odours could be compounded. The smallest number in any classification is four: sweet or fragrant, sour or acid, burnt or empty-reumatie, and goaty or caprylic.

Tastes and odours are associated with (1) decaying organic matter, (2) living algae and other microscopic organisms containing essential oils and other odourous compounds. (3) iron and manganese and other metallic products of corrosion, (4) industrial wastes, particularly phenolic substances, (5) disinfecting chlorine and its substitute compounds, and (6) biologically nondegradable synthetic organics.

Generally speaking, tastes or odours should not be sufficinetly intense to impress themselves upon the user without his knowingly searching for them. Chlorination so important to water safety often produces or accentuates odours and tastes. Fortunately, however, these can be destroyed by strong oxidants such as chlorine dioxide and ozone and by chlorine itself (superchlorination directed into the break point reaction of followed by chemical dechlorination). Many of them can be removed by absorption on to activated carbon. Other can be kept from developing by adding ammonia in advance of chlorine to form chloramines. The prevention of algal odours and tastes through the judicious destruction of incipient growths with copper sulphate or other copper Compounds is also important.

Temperature : The most desirable range of temperatures for a public water supply is between 40 and 50°F. Natural waters are seldom found below 40°F. As the temperature rises above 50°F, the water becomes less palatable and less suited to certain uses. Temperatures above 80°F are undesirable, and above 90° to 95°F the water is unfit for a public supply. Records of the temperature of the raw water are desirable in the operation of a treatment plant because the character of the load changes with temperature. and more or less coagulant may be required : Temperature observations at the source in a deep lake or river may guide the operator in securing water filora the most desirable depth or location.

7.7 Other mealth connected water quality parameters :

pH (Hydrogen-lon-Concentration) : pH is a term used rather Universally to express the intensity of the acid or alkaline condition of a solution. More exactly, it is a way of expressing the hydrogen-ion concentration. It is of importance in practically every phase of public health engineering practice. In the field of water supplies, it is a factor that must be considered in chemical coagulation, disinfection, water softening, and corrosion control. In sewage and industrial waste treatment employing biological processes, pH must be controlled within a range favourable to the particular organisms involved. Chemical processes used to coagulate sewage or such as cyonide ion require that the pH be controlled within rather narrow limits. For these reasons and because of the fundamental ralationships that exist between pH, acidity, and alkalinity, it is very important to understand the theoretical as well as the practical aspects of pH.

The concept of pH evolved from a series of developments that led to a fuller understanding of acids and bases. Acids and bases were originally distinguished by their difference in taste and later by the manner in which they affected certain materials that came to be known as indicators. With the discovery of hydrogen by Cavendish in 1766, it soon became apparent that all acids contained the element hydrogen. Chemists soon found that neutralization reactions between acids and bases always produced water. From this and other related information, it was concluded that bases contained hydroxyl groups.

In 1887, Arrhenius announced his theory of ionization. Since that time acids have been considered to be substances that dissociate to yield hydrogen ions, and bases have been considered to be substances that dissociate to yield hydroxyl ions. According to the concepts of Arrhenius strong acids and bases are highly ionized and weak acids and bases are poorly ionized in aqueous solution. Proof of these claims had to await the development of suitable devices for the measurement of hydrogen-ion concentration.

Measurement of Hydrogen-ion Concentration : The hydrogen electrode was found to be a very suitable device for measuring hydrogen-ion concentration. With its use, it was found that pure water dissociates to yield a concentration of hydrogen-ions equal to 10^{-7} gm/l.

$$H_2O \rightleftharpoons H^+ + OH^- \quad \text{...} \quad (7.1)$$

Since water dissociates to produce one hydroxyl-ion for each hydrogen-ion, it is obvious that 10^{-7}gm of hydroxylion is produced simultaneously. By substitution into the mass action equation, we obtain.

$$\frac{[H+][OH^-]}{[H_2O]} = K \quad \text{.............................} \quad (7.2)$$

But, since the concentration of water is so extremely large and is diminished so very little by the slight degree of ionization, it may be considered as constant and Eq. 7.2 may be written.

$$[H^+]\,'[OH^-]=Kw \quad \text{..................................} \quad (7.3)$$

and for pure water at about 25°C

$$[H^+]\,[OH^-]=10^{-7}\times10^{7}=10^{-14} \quad \text{....................} \quad (7.4)$$

This is known as the ion product or ionization constant for water.

When an acid is added to water, it ionizes in the water and the hydrogen-ion concentration increases; consequently the hydroxy lion concentration must decrease in conformity with the ionization constant. For example, if acid is added to increase the H^+ to 10^{-1}, the OH^- must decrease to 10^{-13}.

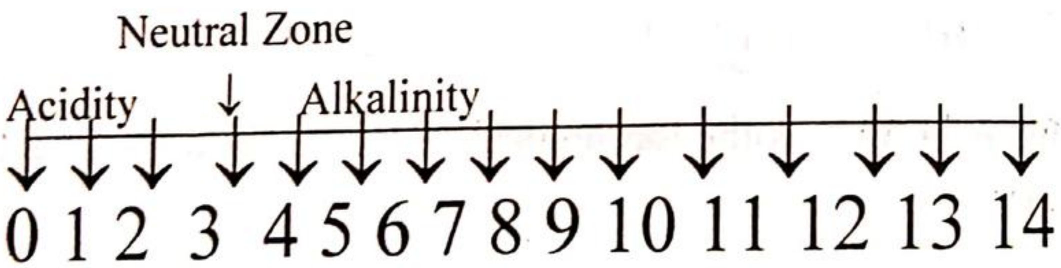
$$10^{-1}\times10^{13}=10^{-14}$$

Likewise if a base is added to water to increase the OH^- to 10^{-5} the H^+ decreases to 10^{-11}. It is important to remember that the OH^- or the H^+ can never be reduced to zero, no matter how acidic or basic the solution may be.

The pH Concept: Expression of hydrogen-ion concentrations in terms of molar concentrations is rather cumbersome. In order to overcome this difficultly, Sorenson (1909) proposed to express such value in terms of their negative logarithms and designated such, values as $p^{H}+$. His symbol has been superseded by the simple designation pH.

The term may be represented by pH= $-\log [H^+]$ or pH= $\log \left(\frac{1}{H}\right)$

and the pH scale is usually represented as ranging from OH to 14.. with pH 7 representing absolute neutrality.



pH scale

Acid conditions increase as pH values decrease and alkaline conditions increases as the pH values increase.

Practical Applications of pH : The pH of natural waters is largely dependent on the CO_2 equilibrium and lies between 7.0 and 8.5. In general, lower pH indicates too great an acidity for use as a beverage. The pH of distilled water may be 7.0 or lower because of the solution in it of carbon dioxide. The significance of pH is therefore of doubtful value in a sanitary water analysis.

The reduction of the amount of alum required for coagulation has proved a valuable return from application of knowledge of pH in water treatment. Some additional advantages of such knowledge, enumerated are(l) the prevention of the passage of alum through filters and the prevention of its after precipitation in distribution systems (2) prevention of corrosive action; (3) control of small plant and animal life through ability to control the optimum living conditions; (4) possible reduction in size of coagulating basins due to better fluctuation and (5) possible-increase in the efficiency of bacterial removal.

The control of pH it of such importance in water treatment and in industrial processes that automatic devices for its control are in wide use in the field. These devices control the dosage of chemicals automatically to maintain any predetermined value of pH.

Specific Conductance : The specific conductance of water is the reciprocal of the resistance in ohms of a column of the water 1 cm. long and having a cross section of 1 sq. cm. at a specified temperature, usually 25°C. It is commonly reported in mhos. The specific conductance is used as a measure of the quality of the water, particularly in chemical and minerological industries, in the study of polluted waters, and in some treatment processes, and in the study of industrial wastes and also in the study of salinity problems.

In general, excepting in plains and deserb regions, the specific conductance of inland fresh waters supporting a good fish fauna lies between 150 and 500×10^{-6} mhos at 25°C.

The specific conductance of pure water at 25°C is in the neighbourhood of 5.5×10^{-8} mhos ; for purest rainwater at 17.6°C, 128×10^{-6}, and for sea water at 25°C, about 500×10^{-6} mhos.

Acidity, Alkalinity, and Salinity : Acidity is generally considered to be undesirable in the source of a public water supply because natural waters are normally alkaline. Low acidity is nor, however, detremental to potability. No standard limitations of acidity or alkalinity are placed on potable water. Both characteristics are expressed in terms of equivalent weight of calcium carbonate, which is not a measure of their reacting abilities. This is measured by the concentration of hydrogen or hydroxyl ions present, usually expressed as pH.

Alkalinity represents the content of carbonates, bicarbonates, hydroxides, and, occasionally, silicates and phosphates present in water. Alkalinity is commonly found in natural waters in the form of carbonate of soda and as bicarbonate of calcium and of magnesium. Caustic alkalinity, caused by hydroxides, is an undesirable characteristic. It is seldom found in natural waters.

Salinity represents the presence of neutral salts such as the chlorides and sulphates of calcium, magnesium, sodium, and potassium.

Aluminium : Compounds of aluminium found in natural waters and in domestic water supplies resulting from the use of aluminium cooking utensils have no pathological significance. Water supplies properly treated with aluminium compounds, such as aluminium sulphate, contain less aluminium than the untreated water because of the insolubility of the aluminium compounds formed and their removal by sedimentation and filtration.

Arsenic: Despite the general use of arsenical compounds in the spraying of crops, arsenic is rarely found in natural waters. There is, however, a recorded instance of a fatality resulting from arsenic in a water supply. The USPHS'standards limit the presence of arsenic to 0.05 ppm, and the British limit is set at 0.2 ppm.

Boron: Less than 30 ppm is tolerable in potable water. Some boron is desirable for the support of plant life. Few, if any, public water supplices in our country are known to contain more than about 1.5 ppm.

Calcium, Magnesium, and Sodium salts : Although the presence of these substances in most waters has no sanitary significance, they may be so detrimental to domestic and industrial uses as to tender the water unfit as a public supply.

Compounds of calcium, which are most commonly found dissolved mineral substances in natural waters, are supplied from three principal sources : limestone, gypsum, and calcium chloride. Limestone is soluble in water to the extent of 13 ppm. If carbon dioxide is present the solubility of the calcium carbonate may increase to 1,000 ppm. Gypsum ($CaSO_4$) is soluble to the extent of 2,000 ppm and calcium chloride ($CaCl$) is so highly soluble as to render water unit as a beverage long before saturation is reachad. Magnesium salts have a pronounced laxative effect on persons unaccustomed to them. Standards of the USPHS limit magnesium to 125 ppm. Magnesium compounds are found in concentrations upto 5 to 20 per cent of the amount of calcium. Sodium is found most commonly in deep ground waters, or at higher underground elevations near the seacoast. It is usually in the form of sodium chloride.

Carbon Dioxide: Carbon dioxide in water may be result of the decomposition of organic matter or of the metabolism of some organisms. In this action dissolved oxygen is converted to carbon dioxide. Hence in general, as carbon dioxide increases. dissolved oxygen decreases. A high carbon dioxide content may, therefore, indicate biological activities ; and high dissolved oxygen may indicate a high degree of purity of the water. This is not always true, however, since a few organisms secret oxygen, or ground water high in carbon dioxide may enter the lake or river from submerged springs, thus increasing the carbon dioxide content of the water without affecting its sanitary quality.

Free carbon dioxide may be found dissolved in ground waters up to 50 ppm, although the concentration is normally much less than this. Its normal concentration in surface water in the neighbourhood of 0.1 to 2.0 ppm.

Carbon dioxide is desirable in water at a concentration known as the carbonate balance, will be discussed in chapters. At this concentration it should impart a desirable taste and it should affect the solubility of carbonates so that a light film may be laid down on metallic surfaces to protect them from corrosion.

Chlorides : The presence of chlorides in a concentration higher than is found in natural waters in a region is an indication of pollution. Generally, an excess of chlorides is a danger sign. Since soluble chlorides are generally unaffected by biological processes they can be reduced mainly by dilution. Although USPHS standards limit chlorides to 250 ppm, higher concentrations have no pathological effects and concentrations upto 700 ppm impart no taste that can generally be detected.

Chlorins : Dissolved free chlorine is not found in natural water. Its presence in treated water results from disinfection with chlorine and the leaving of a residual for the sake of safety against the existence of pathogenic bacteria. The customary residual is in the neighbourhood of 0.1 to 0.2 ppm. Residuals up to 2 ppm are successfully carried but may result in widespread complaints of unpleasant a taste. Chlorine must not be confused with chlorides, for they are not the same.

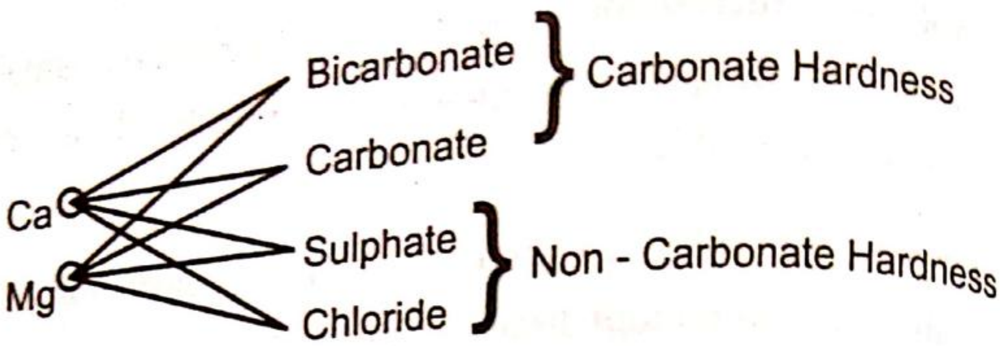
Chromium : The presence of chromium indicates pollution by industrial wastes since compounds of the metal are not round in natural waters. The USPHS standards permit a maximum of 0.05 ppm of chromium in an approved water supply.

Copper : Copper is not present in significant quantities in natural water. Its use as copper sulphate in this control of microscopic organisms and its presence in food tend to pollute water supplies. The presence of copper is therefore an indication of pollution. However, ingest ion of more than 100mg per day is necessary to cause physiological reactions, and the USPHS standards permit a maximum concentration of 3.0 ppm of copper. Such a limit places no-inhibition on normal doses of copper sulphate used in the control of microscopic organisms. Extremely small quantities of copper are sometimes considered to be hygienically desirable.

Fluorine : Fluorides are found in natural Waters and are desirable at a minimum limit of about 0.6 to 1.5 ppm to prevent dental caries, and a maximum of about 3.0 ppm to prevent mottling of the enamel of the teeth of infants. The maximum limit permitted under the USPHS standards is 1.5 ppm.

Dissolved Gases : The solubility of gases in natural water is a function of the temperature of the water and the pressure and concentration of the gas in the atmosphere. The laws of the solubility of gases in water control the concentration of such gases as oxygen, nitrogen and carbon dioxide found in acceptable water.

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



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

Table 7.6 Hardness of Water

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

The U.S Geological survey has classified waters of various degrees of hardness, as shown in Table 7.6. The total hardness of natural waters used as sources of public water supplies ranges from slight below 10 ppm to about 1,800 ppm. Such amounts of hardness has no sanitary significance, but above 100 to 200 ppm the usefulness of the water for domestic and industrial purposes is limited.

Principal Bad Effects of Hardness :

(1) Enough consumption of soap.

(2) Clogs skin, discoloures porcelain, stains and shortens fabrics, toughens and discolours vegetables.

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

(4) Form scales in boilers, resulting in great heat transfer losses and danger of boiler failure

Hydrogen Sulphide : Hydrogen sulphide is objectionable in a public water supply above a concentration of about lmg/l because of the-typical "rotten-egg" odour. Other objections include the tarnishing of metals and its corrosive qualities.

Iodine: Iodines are rarely found in natural waters in quantities sufficient to render the water unfit for public use. A concentration upto 1.0 ppm is considered desirable to prevent goiter. At one time the application of iodides to the public water supply was advocated but dependence may now be placed on the use of iodized table salt.

Iron: Iron is objectionable in a public water supply because it causes stains on plumbing fixtures and on clothing and textiles in the laundry, it may cause tastes and odour and it offers difficulties in manufacturing processes. Sulphate of iron causes acidity and corrodes ferrous metal and brass. Bronze and lead-lined pipes are required to carry waters heavy in iron sulphate.

Organisms whose life processes depend on compounds of iron may cause taste and odour and may create what is known as "red water". Such organisms are sometimes known as "iron bacteria". Compounds of iron found in ground waters are usually in the form of ferrous carbonate $Fe(HCO_3)_2$ which is soluble only in the absence of oxygen; in the presence of oxygen it is quickly oxidized to ferric hydroxide, $Fe(OH)_3$ which is isoluble. The presence of iron is common in ground water because of the wide distribution of hematite in nature and its solubility in water containing carbonic acid. The sulphates of iron frequently result from contact of the water with coal and with iron ores. The quantity of iron in ground water may increase after development of a well owing to the accumulation of organic matter which, on decomposition, will produce acid that dissolves an increasing amount of iron.

Most ground waters, even in iron-bearing regions, contain less than 5 ppm of iron, some falling into the range between 5 and 15 ppm, with a maximum of 40 to 50 ppm for acid waters. The USPHS standards for potable water limit the total amount of iron and manganese, taken together, to a maximum of 0.3 ppm.

Lead : The presence of lead is undesirable in water because of its tendency to accumulate in the body, resulting in plumbium. Standards for potable water limit its concentration to 0.1 ppm. Its presence in natural waters is unusual, but it may be found in water supplies that have come in contact with lead containers such as lead pipes, lead-lined tanks and lead paints.

Manganese : Manganese behaves so much like iron in natural waters that it is sometimes difficult to distinguish between them. Manganese is not so widely distributed as iron in nature so that it is less frequently encountered. The total content of iron and manganese in potable water is limited to 0.3 ppm.

Methane : Methane is a colourless and odourless gas, soluble in water. It is sometimes found in ground water in sufficient concentration to be released ; on reaching atmospheric pressure, to mix with air and create an explosive mixture containing 5 to 15 per cent of methane. Where the presence of methane is below a concentrations of 1.4 ppm, at a temperature of 57°F, sufficient methane cannot escape from the water to form a 5 per cent mixture with the surrounding air. Since the gas is lighter than air it can be excluded from a building by closing the top of the well casing and leading a vent pipe from the inside of the casing to the outer air. The vent pipe should be at least 2 inch in diameter and it should be protected against clogging by frost, birds, and other causes.

Nitrogen, Nitrates, and Nitrites : Nitrogen is an inert gas making up about 80 per cent of the atmosphere. It has no sanitary significance in water. Compounds of nitrogen are reported in a sanitary water analysis as total organic nitrogen, albuminoid ammonia, free ammonia, nitrites, and nitrates. The presence of nitrogenous compounds indicates only the presence of organic matter' in the water. Either free or albuminoid ammonia in water is generally considered evidence of pollution. The presence of nitrogenous compounds in a sanitary analysis requires explanation; it signifies nothing otherwise. Nitrogenous compounds in water that has been treated serve as "food for after growths in distribution pipes and reservoirs and are considered to be undesirable.

The presence of nitrites and nitrates indicates an organic contact sufficiently remote to permit of some oxidizing action on organic matter. Nitrate in water is commonly reported in terms of the nitrogen equivalent, but in mineral analyses results are reported in terms of the acid radical

(NO_3) and as such, in ground waters it may have sanitary significance as a cause of methemoglobinemia.

The total nitrogen in natural waters is seldom sufficient to require its removal by purification processes. The total nitrogen in polluted waters may however be high.

Oxygen: The word "oxygen" is used in reporting chemical analyses as dissolved oxygen, and also in the terms biochemical oxygen demand, oxygen consumed permanganate, and oxygen consumed-dichromate. The tests are unrelated and their insults signify different things. Only the dissolved oxygen test indicates oxygen content of the water. Surface waters of a satisfactory quality should be saturated with dissolved oxygen. The concentration of dissolved oxygen in water exposed to air at various temperature is shown in Table 7.7.

Table 7.7 Solubility of Oxygen in Water

Temperature		Dissolved oxygen mg/l	Temperature		Dissolved oxygen mg/l
°C	°F		°C	°F	
0	32.0	14.62	16	60.8	9.95
1	33.8	14.23	17	62.6	9.74
2	35.6	13.84	18	64.4	9.54
3	37.4	13.48	19	66.2	9.35
4	39.2	13.13	20	68.0	9.17
5	41.0	12.80	21	69.8	8.99
6	42.8	12.48	22	71.6	8.83
7	44.6.	12.17	23	73.4	8.68
8	46.4	11.87	24	75.2	8.52
9	48.2	11.59	25	77.0	8.38
10	50-0	11.33	26	78.8	8.22
11	51.8	11.08	27	80.6	8.07
12	53.6	10.83	28	82.4	7.92
13	55.4	10.60	29	84.2	7.77
14	57.2	10.37	30	85.0	7.63
15	59:0	10.15			

Undersaturation or supersaturation in surface water Indicates pollution. Underground waters deficient in dissolved oxygen may be satisfactory as a public water supply because of the exhaustion of the dissolved oxygen by reactions with dissolved minerals. Oxygen is a highly active gas and its presence in water may contribute to its corrosiveness.

Oxygen consumed-permanganate is a measure of organic matter, particularly carbonaceous matter, this is chemically consumed in the test on digesting the sample of water with potassium permanganate in a bath of boiling water. In general a water of good quality sould be low in oxygen consumed, Oxygen consumed-dichromate, sometimes known as chemical oxygen demands is not a "standard" test. It is performed by digesting the water with oxidizing agents other than potassium permanganate, particularly with potassium bichromate. It too indicates the presence of undesirable organic matter.

Biochemical oxygen demand : Usually abbreviated to BOD, is a measure of the amount of oxygen demand by living organisms in the water to oxidize organic matter present as food for the organisms. BOD is a characteristic, not a constituent, of the water. Unpolluted water should have less than 5 ppm of BOD. Higher amounts are danger signals demanding investigation of the cause before the water is pronounced potable. The test is seldom used in the determination of the quality of a water supply because inorganic pollution may be present when the BOD is low.

Phenol : Phenol in water supplies result from pollution by industrial wastes. They are undesirable in concentrations above 0.001 ppm because of the taste produced in combination with chlorine. At such low concentrations phenols have no hygienic significance.

Phosphate : Phosphates are not significant in natural waters, but the use of polyphosphates in this prevention of corrosion may result in their presence in a public water supply. Generally speaking the concentration resulting from such treatment has no hygienic or aesthetic significance but unfortunately, like compounds of nitrogen, phosphates may serve as food for after growths in distribution systems.

Selenium : Selenium, like arsenic, is found in water supplies principally as a result of the sparying of fruits and vegetables. It is usually present in insignificant concentrations, but waters-with more than about 0.05 ppm of selenium would not be used for a public water supply.

Silica : Silica is present in many natural waters in concentrations between 2-60 ppm, and to a maximum of over 100 ppm. It has no sanitary significance at these concentrations but it is objectionable because of the scale formed in steam boilers.

Sulphate: Sulphates are of hygienic significance because of their laxative effects. The concentration is limited therefore to higher concentrations which are in use in some communities; the higher concentration being disturbing only to persons not accustomed to them.

Tin: Compounds of tin in the concentrations normally encountered in natural waters, are of no sanitary significance; cause no pathological conditions, are harmless aesthetically, and are of no special consideration in most industrial processes.

Zinc : Extremely small concentrations of zinc are sometime thought to be hygienically desirable, but concentrations greater than 15 ppm are undesirable.

7.7 Radioactivity in Water Supplies :

One of the phases of human environment most susceptible to radiation contamination is water which provides a direct and potent avenue for the introduction of radioactive materials into the human body. Thus, the rapid increase of radioactivity in water due to natural sources, uncontrolled disposal of radioactive wastes, and nuclear failout has created a potentially major public health problem.

Sources of Radioactivity in Water : There are two main sources of radioactivity that can contaminate waters :

(1) Natural Radioactivity and (2) Man-made radioactivity.

(1) Natural Radioactirify : (a) Radioactive substances like Uranaium, Thorium, Actinium, Radium, etc. are present in the earth in all sections of the world and they contribute radioactivity to water which comes in contact with them.

(b) The cosmic rays produce radioactivity and disseminate it into the atmosphere. From the atmosphere it can be carried down to the earth's surface and to the body's of water through rainfall. Cosmic rays also produce radioactivity directly on earth, for example, Carbon—14.

(2) Man-Made Radioactivity :

(a) With the rapid development of industrial uses of nuclear fission, particularly for the generation of power, waste products are formed and they are discharged unavoidably into watercourses with resulting radioactive pollution of water.

(b) Mining operations to obtain radioactive minerals for use in the application of nuclear reactors to industrial and other uses will produce wastes. These contribute very markedly to the radioactive pollution of water.

(c) The increasing use of radioisotopes in research, in medical science and in industry presents a rapidly increasing problem of disposal of these substances after use. These may at times be discharged inadvertently into sewers and thus find their way into receiving streams or they may be deposited on land and still find their way into water supplies, both surface and ground sources.

(d) Atomic explosions resulting from the test of nuclear weapons will discharge into atmosphere large amounts of radioactive particles. Todate this has been the major source of environmental contamination. These form clouds from which radioactive particles will gradually settle and fall to the earth's-surface. This phenomenon is usually designated as fallout. Also, rain will wash out of the atmosphere radioactive materials and will discharge them on the surface of the earth from where they will ultimately find their, way into surface watercourses or wil be carried down into the ground and appear in ground water source. This type of carriage of radioactivity is usually called rainout.

(e) Water may be contaminated from the accidents of nuclear power ships or by the radioactive wastes produced by them. Thus, there are now numerous avenues through which water may be unexpectedly contaminated with radioactive materials. With the expanding peace-time uses of nuclear energy, many new and presently unanticipated sources of radioactive pollution of water will develop.

An important phase in the protection of water supplies from radioactive contamination is the establishment in the radiation back ground level of the stream or other sources. This investigation provides information on the radioactivity arising from natural sources and permits evoluation of the extent of contamination from users of radioisotopes or from nuclear reactors installed on the watershed. Also, the radiation background level must be known in order to judge the suitability of the stream as a disposal medium and to determine the treatment needed for wastes to be discharged into it.

7.8 Standards of Water Quality :

The setting of standard of quality to which all acceptable water supplies must conform been attempted by the USPHS (United States Public Health Service), and WHO (World Health Organization) for water sources, drinking waters and waters used-for other ,purposes like industry, agriculture, etc.

Standard For Water Sources : Water Supplies should be drawn from the best available source. Table 7.8 shows the standards of quality for water sources.

Tabla 7.8 **Standard of Quality 7.8 for Water Sources.**

1.	Physical Quality Colour	Maximum Allowable Limits In ppm or mng/1 300
2.	Compounds affecting the potability of water Total dissolved solids	1500

	Iron Manganese Copper Zinc Magnesium phus sodium sulphate Alky1-benzyl-sulphonates (ABS)	50 5 1.5 1.5 1000 0.5
3.	Components hazardous to health Nitrate as NO_3 Fluoride	45 15
4.	Toxic Substances Phenolic substances Arsenic Cadmium Chromium Cyanide Lead Selenium	 0.002 0.05 0.01 0.05 0.2 0.05 0.01
5.	Chemical Indicators of Pollution Chemical Oxygen Demand (COD) Total nitrogen exclusive of NO_3 Ammonia (NH_3) Carbon chloroform extract (CCE) Oil and grease	 10 1 0.5 0.5 1
6.	Biochemical Indicator of Pollution Biochemical Oxjgen Demand (BOD)	6
7.	Bacteriological Standards	MPN/100 ml

		(Coliform Bacteria)
	(a) Bacterial quality applicable to disinfection treatment only	0-50
	(b) Bacterial quality requiring conventional methods of treatment /coagulation, filtration and disinfection)	50-5,000
	(c) Heavy pollution requiring extensive type of treatment	5,000-50,000
	(d) Very heavy pollution, requiting special treatments Greater than	50,000

7.10 Drinking Water Standards : The Table 7.9 shows the standards of physical and chemical quality of drinking water:

Table 7.9 Drinking Water Standards

Quality	Maximum allowable limits in ppm or mg/l		
	USPHS	WHO	European (WHO)
Physical	1962	1963	1964
Total dissolved solids	500	500	250
Turbidity	10	5	5
Colour	20	15	10
Temperature	50°F	50°F	50°F
Taste and odour : Water should be completely free from taste and odour			
Chemical			
Arsenic, As	0.05	0.05	0.2

Cyanide. Cn	0.01	0.01	0.01
Lead. Pb	0.05	0.05	0.1
Barium. Ba	1.0	1.0	1.5
Selenium. Se	0.01	0.01	0.05
Chromium,Cr (hexavalent)	0.05	0.05	0.05
Cadmium. Cd	0.05	0.05	0.05
Silver. Ag	0.05	0.05	0.05
Detergents (ABS)	0.5	0.5	0.5
Chloride. Cl	250	200	350
Copper. Cu	1.0	1.0	1.0
Carbon-chloroform extract	0.2	0.2	0.2
Iron, Fe	0.25	0.25	0.1
Manganese. Mn	0.05	0.05	0.1
Iron and Manganese	0.3	0.3	0.2
Nitrate. NO_3	45	45	50
Phenols	0.001	0.001	0.001
Suiphate. SO_4	250	200	250
Zinc, Zn	5	5	5
Calcium, Ca	80	75	75
Magnesium, Mg	80	75	75
Total Hardness	150	100	100
Total Alkalinity	120	100	100
Fluoride	0.6	0.5	0.5
pH	7 to 8	7 to 8	7 to 8

7.11 Examination of Water :

Water is examined to evaluate its treatability, treatment effectiveness and quality.

Water to be used for a public supply must be potable (drinkable), i.e., not contain pollution. Pollution can be defined as the presence of any foreign substance (organic, inorganic, radiological, or biological; which tends to degrade the water quality and constitute a Hazard or impair the usefulness of the water. Routine analysis of developed water sources are made usually to determine acceptability of the water for domestic and industrial uses. Results of these analyses also indicate the kind of corrective treatment which should be considered for specific applications of the water, the complete analysis of a potential water source should include a sanitary survey and physical, chemical and biological analysis.

Methods of collection and analysis must be standardized if results obtained by different laboratories are to be comparable. In the United States, Standard Methods for the Examination of water and waste water has been published jointly by the American Public Health Association, the American Water Works Association, and the Water Pollution Control Federation. These methods have also been accepted by the American Chemical Society. No attempt will be made here to describe or explain the various analysis procedures. The reader can acquaint himself with the techniques involved by reading Standard Methods. The objective here is to explain the reasons for conducting the different exaninanions and to indicate the significance in the light of accepted standards.

Sanitary Survey: Sanitary surveys of water include (l) surveys of the conditions under which water is processed and (2) observations and examinations of certain properties of the water in its natural or artificial field environment.

Surveys are made of receiving streams for gross qualities such as unsightly floating mater, sludge banks, growths of sewage fungi and other biological indicators of pollution. Physical properties such as temperature are evaluated in the field. Fixing of chemical constituents for example carbon dioxide and dissolved oxygen is also carried out at the site.

Sanitary surveys are made not only of the source of a water supply but of ihe water supply system as well. These surveys uncover environmental conditions. that may affect the potability and treatability of the water being considered. Increasing pollution problems point up the need for additional attention to the quality of source waters. Abatement and control of pollution at the source-will significantly aid in producing a high-quality water. A periodic survey of water-distribution systems is necessary to control physical-defects or other health hazards in the system.

Sampling : Because the environmental engineer is often called upon to collect samples or advise on sampling techniques, a knowledge of correct procedures is important.

A 2-liter portion is adequate for most physical and chemical analyses. Larger volumes may be required for special determinations. Separate portions should be collected for chemical and bacteriological examination since the methods of collection and handing are quite different. The shorter the time interval between collection of a sample and its analysis, the more reliable the results. Immediate field analysis is required for certain constituents and physical characteristics to assure dependable results since changes in composition of the sample may occur in transit to the laboratory. It is difficult to state exactly how much time can be allowed between the collection of a sample and its analysis. A potable-water specimen may ordinarily be held for a much longer period than a raw-sewage sample. The following maximum limits are suggested in Standard Methods as reasonable for physical and chemical analyses :

Unpolluted waters ... ... 72 hr

Slightly polluted waters ... ... 48 hr

Polluted waters ... ... 12 hr

The time and place of sampling and analysis should be recorded on the laboratory report. If the portions have been preserve if by an additive or deviations made from the procedure outlined in Standard Methods, these facts should be recorded on the report.

Certain cautions such as iron and copper are subject to loss by absorption on, or ion exchange with, the walls of glass bottles. Also, pH and carbon dioxide are subject to change in transit. These changes in pH-alkalinity-carbon dioxide balance can cause calcium carbonate to be, precipitated and underestimates of total hardness and calcium may result.

The micrdbiologic activity of a sample can produce changes in the nitrate-nitrite-ammonia balance in biochemical oxygen demand (BOD). Colour, odour, and turbidity may change significantly between time of sampling and time of laboratory analysis.

It is impossible to prescribe absolute rules for the prevention of all changes might take place in a sample bottle. The sample should be collected and stored consistent with the character of the laboratory examinations to be made. Often much time and trouble can be saved if the person collecting the samples and the laboratory analyst will confer in advance on the best technique for collecting and storing the samples.

Representative Samples : Care must be taken to obtain a specimen that is truly representative of existing conditions. The sample bottle should be rinsed two or three times with the water to be sampled prior to filling. No general recommendations can be made as to the number or places of sampling, since details of collection vary considerably with local conditions Care should be exercised in identifying every sample bottle, preferably by attaching an appropriately inscribed tag of label. The record needed includes all information pertinent to the purpose of the sample, such as the, name of the collector, date, hour, and exact location of the source.

Prior to collecting samples from water-distribution' system, pipes sbould be flushed for a sufficient period of time to insure that the sample represents the supply. Samples from wells should be taken only after the well has been pumped for a sufficient period of time to reach equilibrium if the test is to represent the ground water that feeds the well.

Samples collected from a stream may vary with depth, stream flow and distance from shore. An "integrated" portion fron top to bottom in the middle of the stream is generally representative of stream flow. A grab sample (a single sample); can be collected at middepth in the middle of the stream.

Because the quality of water in lakes and reservoirs is subject to considerable variation, the choice of location and depth of sampling depends upon local conditions and the purpose of the investigation. Lakes and reservoirs -are affected by rainfall, runoff, wind, and seasonal stratification. Composite specimens are prepared from a series of grab samples of from an automatic sampler. A series of grab samples might be taken every 20 min. and an amount proportional to the flow rate at the sampling time is placed in the composite container. For example, 1 ml may be taken for each gallon per minute of flow.

It is better to obtain too large a sample rather than one too small as the analyst may wish to make check determinations or run additional tests. Special preservation methods are necessary for portions that are not to be analyzed immediately. Cooling is the most common technique, but special chemicals are added for certain tests. Instructions regarding sampling procedures for specific tests are given in Standard Methods.

Expression of Results : Analytical results may be expressed either as milligrams per litre (mg/l) or as parts per million (ppm). Assuming that 1 litre of water or average weighs 1 kg, the number of milligrams per litre is equivalent to the number of parts per million. For industrial wastes having a specific gravity different from water, the specific gravity should be given due consideration when milligrams per litre are used. When the concentrations is less than 1 mg/L, it is convenient to express the result in micrograms per litre (mg/L) or parts per billion (ppb). Where the concentration is greater than 10,000 mg/L, the results may be expressed in percent (1 percent if equivalent fo 10,000 mg/l).

Results expressed in mg/1 may be converted to ppm by the formula ppm by weight = $\frac{mg/l}{S}$ .... (7.1)

where S = specific gravity.

The Results of analyses of polluted waters or evaluations of plant operations are expressed on a weighted basis which includes the concentration and volume rate of flow. Rate of flow is expressed in either cubic feet per second (cfs) or million gallons per day. (mgd). These are based on biochemical oxygen demand (BOD). The units are calculated as follows :

Pounds per 24 hours (lb/24 hr)=(mg/l)×mgd×8-34 (7-2)

or lb/24 hr = (mg/l) ×cfs × 5.39 (7-3)

Population equivalents: (mg/l of 5 day.

BOD)×mgd×$\frac{8.34}{0.17}$ (7-4)

Test results for colour and turbidity are recorded in units of colour and turbidity. Hydrogen-ion concentration is expressed in terms of pH value. Bacteriological results are stated in terms of the plate count per milli-litre or the most probable number, (MPN) of coliform bacteria per hundred milli-litre.

Standard Tests : Many of the analyses employed in the examination of samples of water and wastewater are identical, but the information sought is used for different purposes. The usual objective of a water analysis is to determine the acceptability of the water for its intended use and as a guide in treatment.

Waters are classified as potable or polluted, safe or unsafe. pure or impure, hard or soft, corrosive or uncorrosive, sweet or sour.

Standard Methods describes the following tests which are performed on waters and waste waters.

Physical and Chemical Examination :

Temperature, turbdity, colour, taste, and odour, total solids, dissolved solids, and suspended solids.

Hardness, Acidity, alkalinity (Phenolphthalein and total). pH value, carbon dioxide (free and total, bicarbonate ion) carbonate ion, and hydroxide.

Silica, copper, lead, aluminum, iron, chromium, manganese. and zinc.

Magnesium, calcium, sodium, and potassium.

Nitrogen, ammonia, albuminoid, organic, nitrite and nitrate.

Chloride, iodide, and fluoride.

Phosphate, orthophosphate, total phosphate, and polyphosphate.

Sulphate, sulphide, and sulphite.

Arsenic, boron, cyanide, selenium, cadmium, and silver.

Alkyl benzene sulfonate (surfactants), phenols.

Chlorine (free available), monochloramine, dichloramine, nitrogen trichloride, and chlorine demand.

Tannin and lignin

Oxygen dissolved, biochemical demand, chemical demand and ozone

Oil and grease

Biological Examination :

Plate count, Coliform group, group density, and differentiation of coliform. Enterococcus group.

Examination and enumeration of microscopic organisms

Bacteriological Examination : Most bacteria in water are derived from contact with air, soil, living or decaying plants or animals, mineral sources, and fecal excrement. Some of the most common types of bacteria which may be present in water include the coliform group, fecal streptococcus, fluorescent bacteria, chromogenic bacteria, proteus group, spore-producing rods, and archromobacter. Many of these bacteria are without sanitary significance because they die rapidly in water, come from unknown sources, are widely distributed in air or soil, or have no known or suspected association with pathogenic organisms.

A variety of procedures have been used in the last 100 years to measure the bacteriological quality of water. These, biological tests and procedures include the following :

1. The total plate count on gelatin at 20°C and on agar at 37°C.
2. The specific identification of pathogenic bacteria.
3. Use of the coliform group as a sewage pollution indicator.
4. Checking the fecal streptococcus group to indicate fecal pollution.
5. Examining Clostridium perfringens as an indicator of pollution.
6. Employing miscellaneous indicators including bacteriophage tests, serological methods, and identification of other specific bacteria and virus.

The plate count is not required in defining a safe standard for potable waters by the "1962 Drinking Water Standards". It is useful however as a routine quality control test in the various water treatment processes and as a method for estimating the sanitary conditions of basins, filters, distribution systems, or equipment. The specific identification of pathogenic bacteria as pollution indicators requires extremely large samples and a wide variety of media and methods to exclude all the various pathogens which could be present. Neglecting the expense, the time required for this test procedure would greatly reduce the usefulness of the results. Such a group of test procedures is not applicable to the frequent routine sampling of water supplies.

The coliform group is considered a reliable indicator of the adequacy of treatment for bacterial pathogens. The 1962 Drinking Water Standards reaffirms this standard and includes all of the aerobic and facultative anaerobic, gramnegative, nonspore-forming, rod-shaped bacilli which ferment lactose with gas production within 48 hr at 35°C in the coliform group.

This coliform grouping includes organisms that differ in biochemical and scrologic characteristics and in their natural sources and habitats. Escherichia coli is characteristically an inhabitant of the intestines of man and animals, Aerobacter cloacae are frequently found in various types of vegetation and as deposits in the distribution system. The intermediate-aerogenes-cloacae (IAC) subgroups are found in fecal discharges but normally in smaller numbers than *E.coli*. All of the coliform group except *E.coli* are commonly found in soil and in waters polluted in the past.

Organisms of the IAC group tend to survive longer in water than to E. coli and are more resistant to chlorination. The relative survival times of the coliform subgroups may be used to distinguish recent from less recent pollution. Waters that have been newly polluted will show an *E.coli* density greater than the IAC density. Polluted waters that have not lately received fecal material will tend to have a higher IAC group density than *E.coli*. There is considerable confusion and controversy regarding both the advantages and disadvantages claimed for *E.coli* as an indicator of recent pollution.

The fecal streptococci are characteristic of fecal pollution and are consistently present in the feces of all warm-blooded animals. They do not multiply in water as sometimes occurs with the coliform group, therefore the presence of fecal streptococci in water indicates fecal pollution with a density equal to or less than that originally present. Improved methods and media are needed for the routine analysis of the streptococcai group which is not recognized as an official test procedure in the United States, although Standard Methods lists a tentative procedure for enterococcus, The fecal streptococci tests have application in stream-pollution investigation.

Clostridiam perfringens is widely distributed over the earth's surface and uniformly present in the intestinal tract of warmblooded animals. The spores have a long survival time in water and are resistant to chemical treatment and natural purification. The presence of spores in deep wells indicates a direct connection between the surface and the underground water source. Methods of isolation and dification are unsatisfactory for routine use. There is no test for Clostridiutn perfringens in Standard Methods.

Numerous other biological indicators of pollution have been suggested by various invastigators for example. bacteriophage tests. Phage types in water cannot be interpreted with our current knowledge and therefore bare little use as evidence of pollution. Agglutination tests have been propounded for the sero-logical separation of the coliform group. Difficulty has been encountered in duplication of these test.

Bacteriological Standards of Drinking Water (USPHS) :

The number of bacteriological samples collected from representative points throughout the distribution system must be in keeping with the size of the population at risk. Typical minimum monthly numbers are prescribed as follows :

Population served (thousands)	12	10	50	100	900	2000	4500
Number of monthly samples	2	12	50	95	300	400	500

Official evaluations of bacteriological quality are bassed upon quantitative tests for the presence of numbers of the coliform group of organisms performed by health agencies or their designated representatives, govermental laboratories, water works authorities, or approved commercial laboratories.

Depending upon method of examination and number of monthly samples, the following specification must be met in accordance with serial dilation, most probable number (MPN), and membrane filter (MF) techniques :

1. When 10- ml standard portion are examined, and more than 10% should show the presence of coliform bacteria. Water showing their presence in three or more 10-ml portions of standard sample will not be acceptable if it is noted in the following :

(a) two consecutive sample.

(b) more than one sample when less than 20 are examined.

(c) more than 5% of the samples when 20 or more are examined.

When coliforms occur in three or more of the 10-ml portions of a single standard sample of water, daily samples drawn from the same sampling point must be examined until at least two consecutive samples show that the water is of satisfactory quality.

2. When 100-ml standard portions are examined, not more than 60% should show the presence of coliform bacteria. Wate showing their presence in all five of the 100-ml portions of a standard sample will not be acceptable if it is noted in the following .

(a) two consecutive samples

(b) more than one sample when less than five are examined

(c) more than 20% of the samples when five or more are examined.

When conforms occur in all five of the 100-ml portions of a single standard sample of water, daily samples drawn from the same sampling point must be examined until atleast two consecutive samples show the water is of satisfactory quality.

3. When the membrane filter technique is applied, the arithmetic mean coliform density of all standard samples must not exceed 1 per 100-ml. Moreover, coliform colonies per standard sample must not exceed 3 per 50-ml, 4 per 100-ml, 7 per 200-ml, or 13 per 500-ml in the following :

(a) two consecutive samples

(b) more than one standard sample when less than 20 are examined.

(c) more rhan 5% of the standard samples when 20 or more are examined.

When coliform colonies in a single standard sample exceed these values, daily samples drawn from the same sampling point must be examined until the results obtained from at least two consecutive samples show that the water is of satisfactory quality.

QUESTIONS

1. Name the various types of impurities present in natural water and write down their effects. (BUET, 64, 69)
2. Name the different types of living organisms present in water and comment about their significance in public water supplies. (BUET 63, 68, 70)
3. Write an explanatory notes on waterborne diseases. (BUET, 69)
4. Name the important waterborne poisons and their health-connected effects. (BUET, 62, 63, 69)
5. What is meant by palatability of water ? How water can be made palatable ? (BUBT, 67, 71)
6. What are the objectives of examination of water ? Discuss the surveys, sampling and analysis is to achieve Ihose objectives. (BUET, 66, 70 72)
7. What are the physical, chemical, biochemical, and biological tests usually carried out for determining the potability of water ? What is the significance of' pH value in relation to some of those tests ? (BUET, 70, 7A 73)
8. What do you understand by pH value of water? Determine mathematically the pH value of a natural water. What is pH scale ?
9. Show the relationship between acidity, alkalinity and pH value. Why the pH value is very important in public water supplies ? (BUET 66, 70)

10. Name the tests that are generally carried out on water for the following :
(a) Safety and wholesomeness. (b) Palatability. (c) Economic usefulness. (d) Water treatment processes and (c) Functional tests. (BUET, 68, 72)
11. Why certain standards are adopted for maintaining water quality for various purposes? Write in a tabular form the standards of water quality for water sources. (BUE, 71, 73)
12. Write down the international standards (physical, chemical, and bacteriological) for drinking waters.
13. Write abort notes on any five of the following :
(a) Bacteria, (b) Virus, (c) Pathogenic organisms, (d) Radioactivity in water supplies, (e) pH value of water, if) Chemical Indicators of pollution.

8

WATER PURIFICATION

8.1 Objectives:

Safe drinking water is essential to the health and welfare of a community, and many water supply systems must have some means of water purification. The main objectives of water purification are to make water potable, i.e. to make wafer (1) safe to drink (2) pleasant to taste and (3) suitable for domestic uses.

8.2 Methods of Water Purification :

Various methods are used to make water potable and attractive to the consumer. The method selected depends mostly on the character of the raw water. The following are the various methods of water purification :

1. Plain Sedimentation.
2. Sedimentation with coagulation.
3. Filtration.
4. Disinfection.
5. Softening.
6. Aeration.
7. Activated Carbon Application.
8. Fluoridation,
9. Recarbonization.
10. Demineralization.
11. Desalinization.

Objectives :

To make the water :

(1) Safe to drink

(a) Removal and destruction of pathogenic organisms — 1, 2, 3, 4

(b) Removal of excess fluoride — 2, 3

(c) Addition of fluoride — 8

(2) Pleasant to Taste

(a) Removal of objectionable gases (control of tastes and odours) — 4, 6, 7

(b) Destruction of algae — 4

(c) Removal of algae oils, products of decomposing organic waste matter and industrial wastes — 2, 4, 6, 7

(3) Suitable for domestic used :

(a) Removal of turbidity — 1, 2, 3

(b) Removal of Colour — 3, 6

(c) Removal of iron and manganese — 2, 3, 6

(d) Removal of hardness — 5

(e) Removal of objectionable minerals — 10,11

(f) Removal of objectionable salts — 11

8.3 Selection of Water Purification Method :

The type of treatment required depends on the physical, chemical, and biological characteristics of the water. Water from deep wells, for example, is usually free of pathogenic bacteria and no purification is necessary. Most well waters are hard and softening, together with removal of iron and manganese, may be desirable. The most turbid water will generally requires no more than plain sedimentation for clarification. If there is possibility of pollution, chlorination is also advisable.

The relatively high turbidity of river waters will usually require facilities for chemical sedimentation and filtration. The turbidity of river water varies considerably throughout the year. It may be quite high during floods but low at other times. Some plants treating river water may have the facilities for adding coagulant but use them only during floods. Storage in reservoirs will reduce the need for sedimentation. Many countries have also taken steps to reduce erosion in watersheds tributary to their water source in order to minimize the requirements for clarification. Most surface waters are subject to contamination, and disinfection is usually essential.

Flow diagrams for two typical water-treatment plants are shown in-Fig. 8.1.

8.4 Plain Sedimentation :

This is the process of causing heavier solid particles in suspension both organic and inorganic to settle by retaining water in a tank or basin. Many of the impurities suspended in water have specific gravity greater than one and are held in suspension by virtue of the turbulence or currents maintained in the water, When these currents are retarded, the suspended matter generally settle to the bottom of the body of water. This is the basic principle involved in the plain sedimentation process.

The particles having specific gravity greater than one tend to move downwards in water by force of gravity with acceleration until the frictional resistance of water or the "drag" approaches the impelling force due to gravity. Thereafter the particles travel with a constant vertical velocity called the settling velocity. The settling velocity of the particles depends upon the (1) horizontal flow velocity of water, (2) Shape and size of the particles, (3) Specic gravity of the particle, (4) Viscosity of water, (5) Density of water. and (6) Temperature of water. and is computed usually from the Stoke's Law which is in the following form :

$$V = \frac{g}{18}(S-1)\frac{d^2}{\gamma} \quad \text{.....................} \quad (8.1)$$

in which V= Settling Velocity in cm/sec.

$g = 981$ cm/sec^2

S = Specific gravity of particles

d = diameter of settling particles in cm

γ = Kinematic Viscosity in centi-stokes.

It may be noted that Stoke's Law holds good only for particles of 0.1 cm diameter and of Reynold's Number 1 or less. In this range, Viscous forces are larger in relation to inertia. For particles of greater diameter and higher Reynold's Number, friction or drag becomes more prominent and where diameter of particles increases above 1 cm and the Reynold's Number above 2,000. Newton's Law for frictional resistance or drag applies, i.e.,

$$V = \sqrt{\frac{4g}{3C_D}(S-1)d} \quad \text{.......................} \quad (8.2)$$

in which C_d = Newton's Coefficifnt of drag and other letters have the same meaning as explained before.

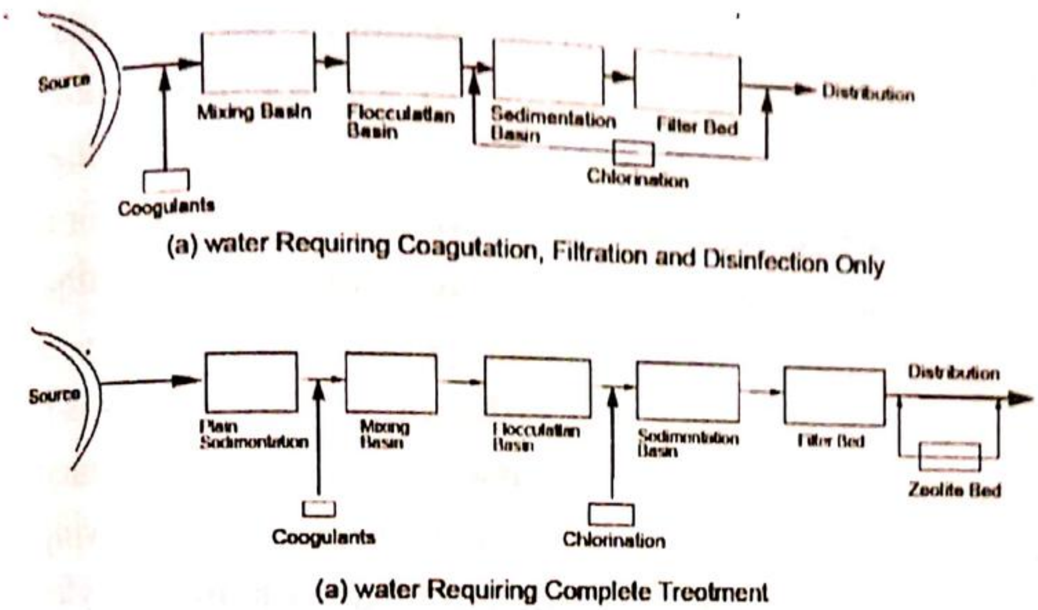


Fig. 8.1 Flow Diagrams for Two Typical water Purification Plant

Detention Period : The detention period of a settling basin in the theoretical time water is detained in it. It is given by the ratio of the volume of the basin to the volumetric rate of flow through the basin. It may vary in practice from a portion of an hour to many days, but since the greater part of the total removal occurs during the first portion of the period and since the water is to be later subjected to further treatment, basins are designed usually for a detention period of 3 to 4 hours.

Flowing-Through Period : This is the actual time of flow or the average time required for a small amount of water to pass through the basin at the given rate of flow.

Theoretically, the detention period and the flowing through period are of one and the same measure. Actually, they will vary as the displacement of the water in the tank by the incoming flow is seldom uniform because of any of the reasons as (1) difference of temperature, (2) particular position of baffle walls, (3) manner of withdrawing outflow from the tank. Asa result, part of the water reaches the outlet more rapidly than remainder and part stays in the tank for a longer period of time. The percentage ratio of the flowing through period to the detention period is, therefore, a measure of the efficiency of distribution of flow in the basin or the efficiency of the basin itself. For a well-designed basin, this is seldom taken as less than 30 per cent. Flowing through period is practically determined with the help of dyes, chemicals such as sodium chloride, radioactive isotopes, etc.

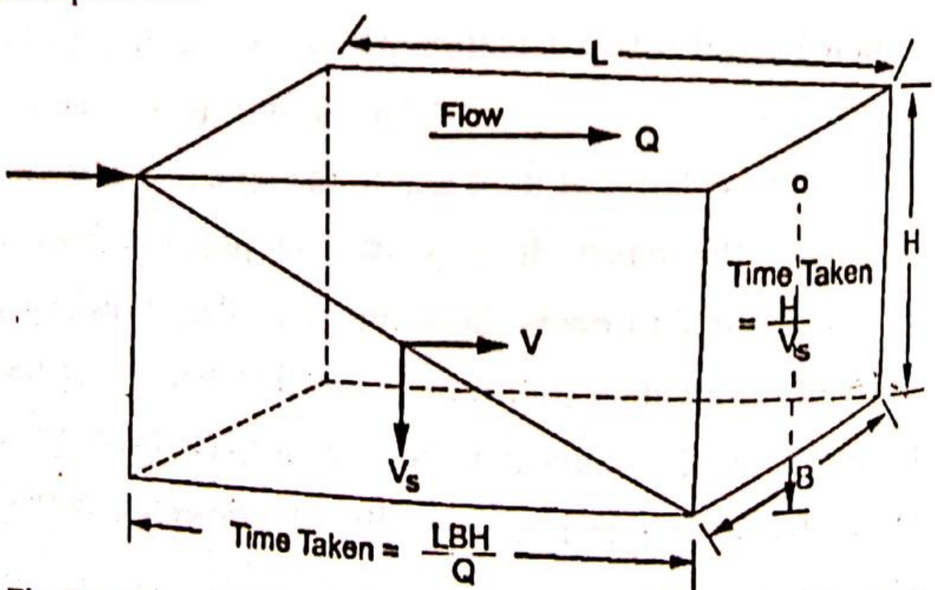


Fig. 8.2 Theoretical Flow Through a Rectangular Sedimentation Basin

THEORY ; In Fig. 8.2

L = length of the basin

H = depth of the basin

B = width of the basin

V = horizontal velocity of flow of water in the basin

V = settling velocity of the particle in the basin

Q = quantity of water flowing through the basin

then $V = \dfrac{Q}{BH}$ and

$$\text{time of horizontal flow} = \frac{L}{V} = \frac{L}{\frac{Q}{BH}} = \frac{LBH}{Q}$$

$$\text{Time of falling or vertical flow} = \frac{H}{V_s} = \text{Detention period.}$$

Now, for the particle to reach the bottom before water leaves the tank time of fall= time of horizontal flow

i.e, $\dfrac{H}{V_s} = \dfrac{IBH}{Q}$

or $V_S = \dfrac{Q}{BL} = \dfrac{Q}{A}$ where A=cross-sectional area of the basin.

This is the limiting velocity of fall to enable to reach the bottom of the tank, meaning thereby that all particles with velocity greater than $\dfrac{Q}{A}$ will reach the bottom before reaching the outlet from the basin and thus removed, whereas all particles with velocity less than will not reach the bottom and only be removed in the ratio of their velocity to $\frac{Q}{A}$ i.e., if their velocity is only half of $\frac{Q}{A}$ then only half the particles falling at this velocity will reach the bottom. For example, if A=100 sq. ft. and Q=1 cfs. then $\frac{Q}{A} = \frac{1}{100} = 0.01$ fps. so that all particles which fall with a velocity of 0.01 fps will be removed, whereas only 50% of those having a velocity of $\frac{0.01}{2} = 0.005$ fps shall be removed and 25% of those having a velocity of $\frac{0.01}{4} = 0.0025$ fps shall be removed. Thus, $\frac{Q}{A}$ or the over flow rate for any basin is a measure of the effective removal of particles in this basin.

This equation, $V_s = \frac{Q}{A} =$ will also indicate that with a given Q the greater the surface area of the tank, the smaller will be V_s, the smaller the particles removed and without consideration of the depth, the basin could be designed efficiently-evidently an absurd conclusion. Actually, velocity is not so simple as given by the above equation and depends upon a number of factors as specific gravity, size and shape of particles as well as temperature and kinematic viscosity. The fact, however, goes beyond saying that the overflow rate has been an important bearing in the design of sedimentation tanks

A typical rectangular sedimentation tank is shown in Fig. 8.3.

Example 1 : One million gallons of water per day (1 mgd) passes through a sedimentation tank which is 20 ft. wide, 50 ft. long, and 10 ft. deep, (a) Find the detention time for this basin, (b) What is the average velocity of flow through the basin? (e) If the suspended solids content of the water averages 40 ppm, what weight of dry solids will be deposited every 24 hours assuming 75% removal in the basin, (d) What is the over flow rate?

Solution : (a) Detention time = $\dfrac{\text{Volume of tank}}{\text{Flow per unit time}}$

$$= \frac{20\times50\times10}{1\times106}\times7.48\times24$$

= 1.8 hrs.

(b) Velocity, $V_S = \dfrac{Q}{A} = \dfrac{1\times10^6}{20\times10\times7.48\times60\times60\times24}$

= 0.0077 fps.

(c) Totat solids deposited $= \dfrac{40\times10^6}{10_6}\times8.34\times0.75$

= 250 lb/day

(d) Over flow rate = $\dfrac{Q}{BL} = \dfrac{1\times10^6}{50\times20} = 1000 gpd/sft.$

Example 2 : A rectangular sedimentation tank is to treat 4,00,00 gpd of raw water. The detention period is to be 4 hours, 'the velocity of flow 3 inch per minute and the depth of water and sediment is 14 ft. If an allowance of 4 ft. for sediment is made, what should be the length and width of the sedimentation tank?

Solution :

Velocity of flow = 3 inch/min–0.25 fpm

Total length of the tank =240 × 0.25×60 ft.

Volume of water to be treated in

$$4 \text{ hours} = \frac{4,00,000 \times 4}{24} = 66,700 \text{ gallons}$$

$$= 8,920 \text{ cft.}$$

Cross-sectional area of the tank

= 8,920/60 = 148.7 sft.

Effective depth of the tank- 14-4 =10 ft.

Width of the tank= 148.7/10= 14.87= 15 ft.

Circular Sedimentation Tank: Sometimes, circular sedimentation tanks are used, as shown in Fig. 8.4. The diameter of the circular tank depends on the overflow rate, volume and depth. As for circular tanks, equipment is made in certain standard sizes, generally, the tank bottom is cone-shaped with a slope of about 1 inch vertical in 1ft. horizontal. With these conditions as basin the volume of circular sedimention tank is found by the formula

$$V=D^2 (0.011D+0.786H)$$

in which V = volume of circular tank in cft

D = diameter of the tank in ft

H = vertical depth at wall in ft.

The value of H (generally 10 to 14 ft) is assumed and the equation can be solved for D if the volume V is known.

The following factors generally govern the design of sedimentation tanks, both circular and rectangular : (1) No. of mixing basins, (2) length, (3) width, (4) effective, depth, (5) velocity of flow, (6) inlet arrangements, (7) intermediate arrangements. (8) outlet arrangements, (9) detention time, (10) sludge storage volume, (11) method of sludge removal, and (12) cover if any.

Examples 3 : A circular sedimentation tank with standard mechanical sludge removal equipment is to handle 7,50,000 gpd of raw water.

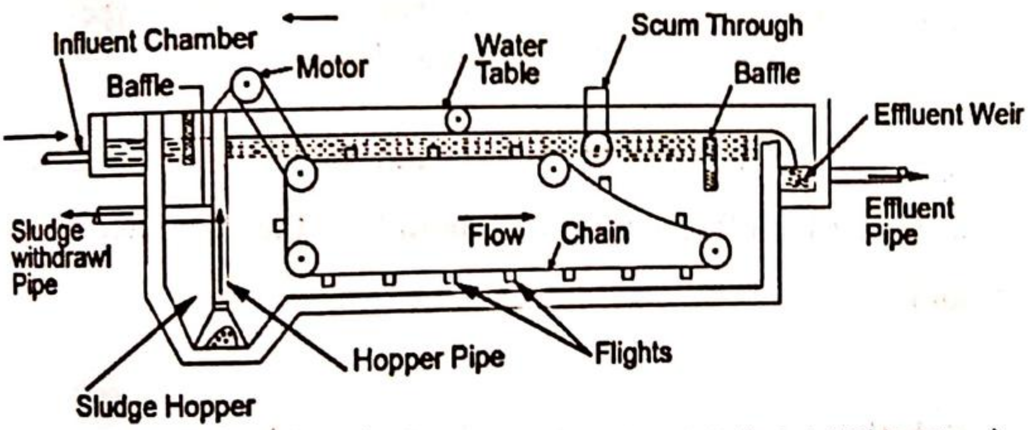


Fig. 8.3 (a) Rectangular Sedimentation Tank (Horizontal Flow Type)

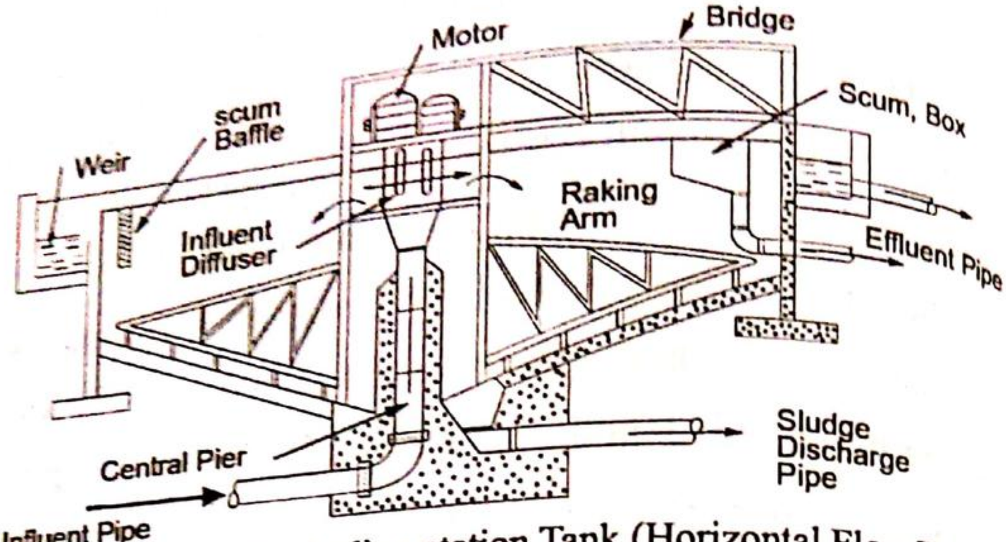


Fig. 8.4 (a) Circular Sedimentation Tank (Horizontal Flow Type)

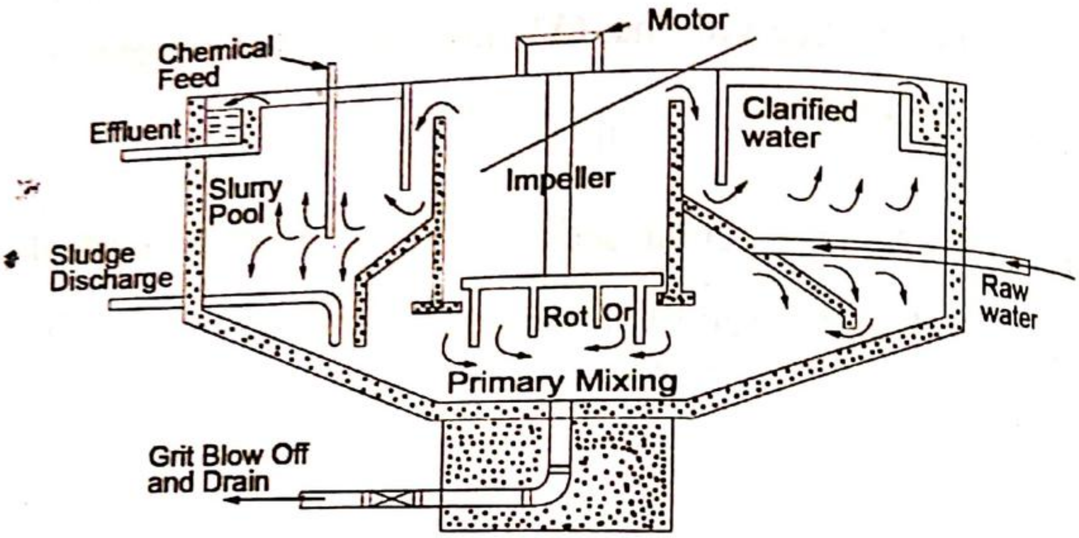


Fig. 8.4 (b) Circular Sedimentation Tank (Vertical Flow Type)
Types of Sedimentation Tanks

If the detention period is 4 hours and the depth of the tank is to be 10 ft. (effecrive) What should be the diameter of the tank ?

Solution ; Volume of water to be bandled by the

$$\text{tank} = \frac{7{,}50{,}000 \times 4}{24} = 1{,}25{,}000 \text{ gallons}$$

$$= 16{,}710 \text{ cu ft.}$$

Using the equation, $V = D^2 (0.011D + 0.785H)$

$16.710 = D^2 (0.011D + 0.785 \times 10.)$

By trial and error method,

$D = 44.67$ ft. say 45 ft.

8.5 Sedimentation with Coagulation :

The removal of very fine light colloidal impurities from water is difficult to achieve in practice by the aforesaid process of plain sedimentation. This can be greatly expedited by the addition to water of certain chemical compounds which where thoroughly mixed form wooly masses of focculent precipitate enmeshing the suspended particles become heavier and finally settle out. These substances are called coagulants and their process of reaction is coagulation. Strictly speaking, Coagulation is the name given to the actual combining action whereas flocculation is the term to indicate the process of building up of the larger and heavier particles of floc which will settle.

The coagulants commonly used are : alum or aluminum sulphate, copperas or iron sulphate and lime. Other less commonly used coagulants are sodium aluminate, ferric choride, ferric sulphate, chlorinated copper as and activated silica, etc. The characteristics of some common coagulants are shown in Table 8. 1

Chemical Reactions : Typical chemical reactions involved are :

(1) $Al_3\ (SO_4)_3 + 3Ca(HCO_3)_2 \rightarrow Al_2(OH)_6 + 3CaSO_4 + 6CO_2$

$\downarrow$

(Floc)

For waters having low natural alkalinity, it would be necessary to add alkalies are lime or soda ash in order that the coagulant remains active and is completely precipitated out during the chmical action.

(2) $FeSO_4 + Ca(HCO_3)_2 \rightarrow Fe(HCO_3)_2 + CaSO_4$

The bicarbonate of iron is change to iron hydroxide by the addition of caustic lime. Thus,

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

The reaction continues with the oxidation of the ferrous hydroxide by the oxygen present in the water to ferric hydroxide $4Fe(OH)_2 + O_2 + H_2O \rightarrow 2Fe(OH)_3$

$\downarrow$

(Floc)

Table 8.1 Some Common Chemicals Used in Water Purification

Substance	Weight lb percuf t	Troublesome characteris-tics	Properties	pH range for best coagula-tion.	Normal dosage, grains 'per gal
1	2	3	4	5	6
Aluminum sulphate(Al_2)(SO_4)$_3$18 H_2O) alum	39	Dusty	Acid, and corrosive	5.5-8.0	0.3-0.5
Ferrous sulphate ($FeS0_4.7H_20$)$_2$copperas	46	Cakes at temp above 20° in moist air stains	Acid and corrosive	8.5-11.0	0.3-30
Calcium oxide (CaO)$_2$ quick-lime	65	Dusty,slakes on standing in air	Alkaline and incrustant	5.5-8.5	0.2-1.5
Calcium hydroxide ($Ca(OH)_2$), hydrated lime	50	Dusty	Alkaline and incrustant	7.5-9.0	0.3- 2.0
Sodium aluminate (Na^2 Al_2-0_4)	58	Hygrosco pic	Alkaline	7.5-9.0	0.2-2.0
Sodium carbonate (Na_2 CO_3)$_2$ soda ash		Dusty	Alkaline	6.5-9.5	0.2-2.5
Ferric chloride ($FeCl_3$)	63	Acid, corro-sive, staining liquid	Acid and corrosive	5.0-11.0	0.5-3.0
Ferric sulphate $Fe_2(S0_4)_3$ Ferrisul	70	Stains	Acid and corrosive	5.0-11.0	0.4-3.0
Sodium silicate $Na_2 SiO_3$ -Na_2O+SiO_4	66	Alkaline and sticky	Mild	6.0-10.0	0.3-3.5

Thus the gelatinous precipitates or floes produced in the two cases are aluminum hydroxide and ferric hydroxide respectively. The coagulation is considered to be taking place in two distinct stages :

First stage is marked by the neutralization of electric charges. As the coagulant dissolves, the dissociate, positively charged particles of turbidity. Colloidal clay and colour and thereby coalesce or combine into a single mass. For this a rapid and thorough mixing is desirable.

Second Stage : The initially formed particles are molecular in size. If now the water is gently stirred, the particles flocculate and grow bigger in size i.e., anglomerate. By this anglomeration, the larger colloidal particles of turbidity and colour are enveloped and the floe becomes heavier and is carried dawn to be ultimately removed by sedimentation. Flocculation is brought about by slow stirring. In this process, the floe also absorbs suspended particles from the large surface with which it is in contact.

Factors Influencing Coagulation : Many factors influence the coagulation of waters. Among them, the following are important. (1) Kind of coagulant, (2) quantity of coagulant, (3) characteristics of water (suspended matter, pH and temperature) and (4) time of mixing, flocculation and coagulation.

Dosage of Coagulants : This depends upon a number of factors such as (1) turbidity of water, (2) Colour of water, (3) pH value of water, (4) time of settlement and (5) temperature of water. The amount per litre varies in practice from 0.03 to 0.13 gram or more of alum. It is found that greater the turbidity of water, more coagulant is required. The amount of coagulant also increases with lower temperatures because of slow reaction and floe formation. The control of pH value is important from the point of view of maintaining the character of the floes. Thus, for aluminum hydroxide floe, if the pH value is lower than the optimum, the insoluble floes become soluble and the floes disappear. If on the other hand, the pH value is more than the optimum, the aluminum hydroxide ionizes into an aluminate which is both negative in electrioal charge and soluble in water. It is found that the optimum pH lies in the range of 5.0 to 6.5. For proper control of the pH value, it may sometimes be necessary to lower or raise the pH ; the former can be achieved by adding into water sulphuric acid, or hydrochloric acid or carbon dioxide while the later by the addition of caustic soda, lime or soda ash.

For removal of colour, colour floc is required to be formed first before subsequent removal through filtration. With most coloured waters, this floe is formed with pH value below 6.5. Alum is quite effective in removing colour.

Coagulation is usually found necessary for waters having turbidity greater than 40 ppm.

Jar Test : This is a laboratory method to determine the optimum dosage of a particular coagulant which is required to be added to the raw water for coagulation and subsequent sedimentation in a treatment plant. Since alum is the coagulant commonly used at the water treatment plant, tests are run to find out its optimum dosage.

The Jar Test apparatus consists of a rotary device having six number vertical stirring rods provided with pedals at lower ends and called multiple stirrer, with stirring water and alum contetns

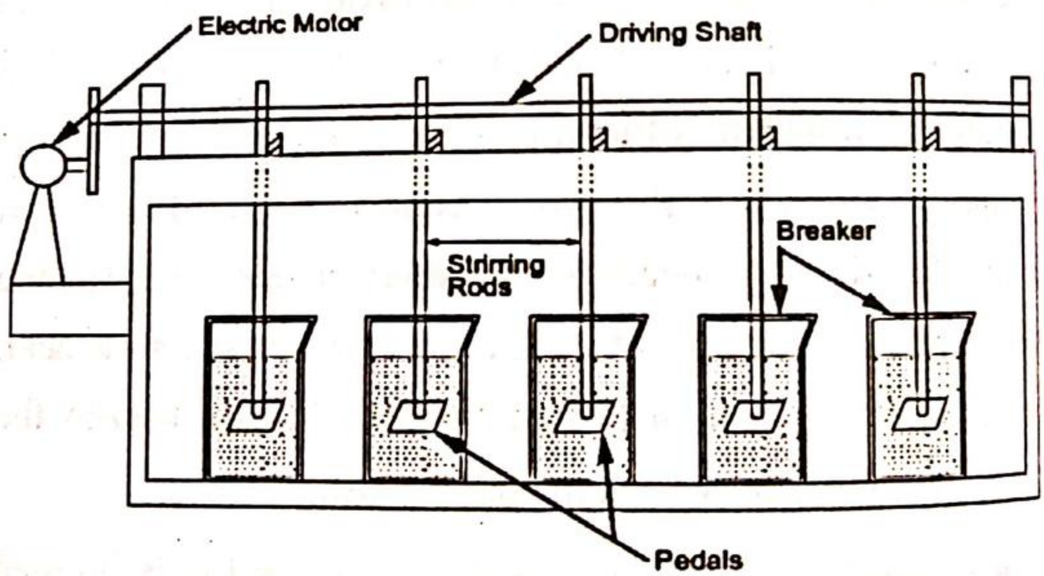


Fig. 8.5 Jar Test Apparatus

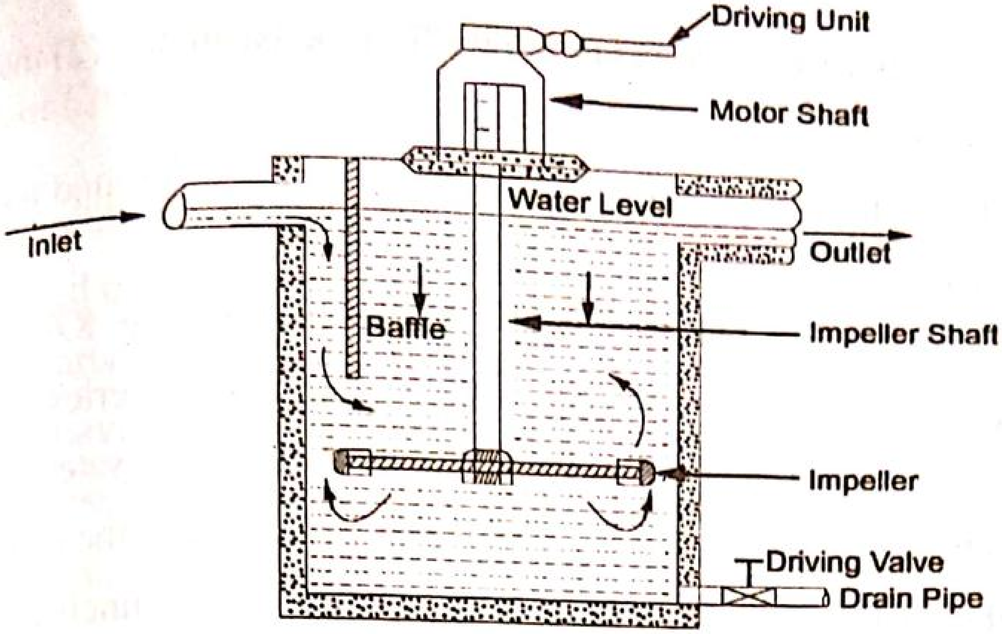


Fig. 8.8 Flesh Mixer

placed in six number beakers or jars each of 1 litre capacity (Fig. 8.5) The multiple stirrer is turned through a horizontal shaft which is geared directly to an electric motor. The motor is provided with speed-reducing gears which enable the pedals to revolve at different speeds as required in the first and second stages of the coagulation process.

When different amounts of alum solution are added to each jar containing the same amount of water sample and the constants stirred rapidly at first for 2-3 minutes and slowly later for obout 30 minutes and the liquid finally allowed to stand so that floe may form, the smallest dosage of alum that produces good floc is taken as the optimum dosage for the particular water.

Mixing Basins : Tanks in which water and the coagulants are mixed together are called mixing basins. Mixing basins are usually of two types (1) the baffle type basin and (2) the mechanically agitated type basins.

Baffle Type Basins : These are of two types : (a) round the end baffle, and (b) over and underflow baffles.

In the case of basins with round the end baffles (Fig. 8.6), a ractangular tank is divided into a continuous series of channels by means of baffles which cau'se the water to which the coagulant has been added to flow around the ends of the baffles through numerous channels. The channels are made narrow so that the velocity of flow will be sufficient to prevent the formation of deposits. In the case of with over and under baffles (Fig.8.7) the coagulant-mixed water flows over and under the baffles through the channels.

Design of Mixing Basings : In the design of baffle type mixing basins, following points are to be noted. They are principally based on conventional practice.

(1) Velocity of flow in the channels between baffles = 0.5 to 1.0 ft/sec.

(2) Mixing period or detention period = 20 to 50 mins.

(3) Distance between baffles = 18 inches.

(4) Required depth of channel for baffles of the ground the end type may be calculated from.

$$\text{Depth of channel} = \frac{\text{cross-sectional area of each channel}}{\text{Distance between baffles}}$$

(5) Required width of the channel for baffles of over and under type may be calculated from

$$\text{Width of Channel} = \frac{\text{Cross-sectional area of each channel}}{\text{Distance between baffles}}$$

Example : A mixing basin with a round the end baffles is to treat 3 mgd of raw water. The basin is to be divided into two similar parts by a longitudinal partition wall so that each half will have a clear width of 10 ft. What should be (a) the depth of the basin, (b) the number of channels, and overall inside length of the tank ?

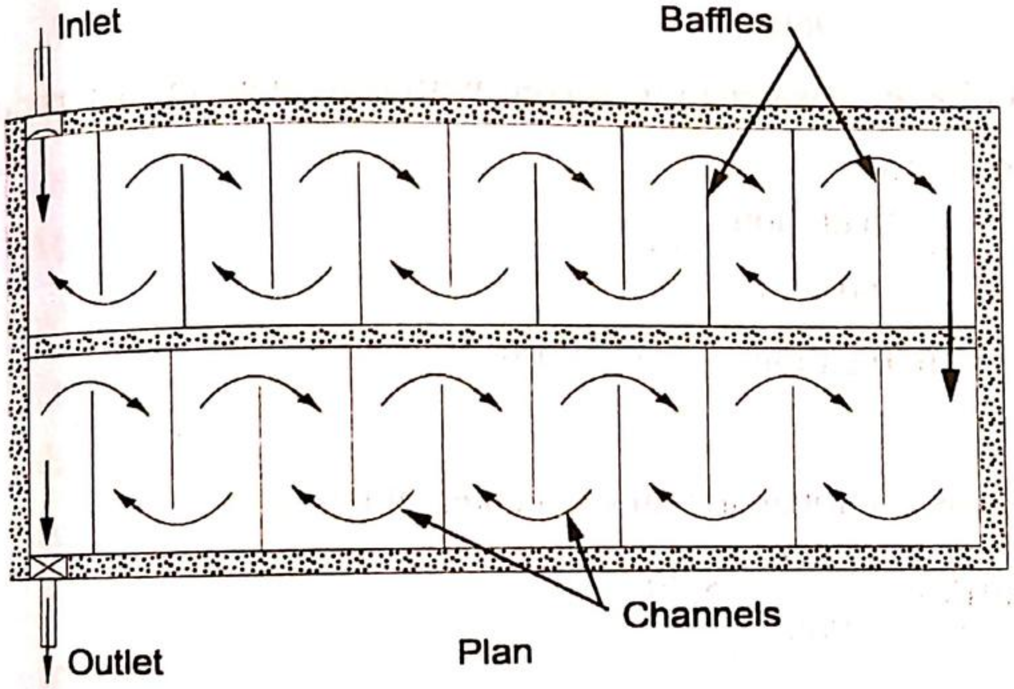


Fig. 8.6 Mixing Basin with round the end baffles

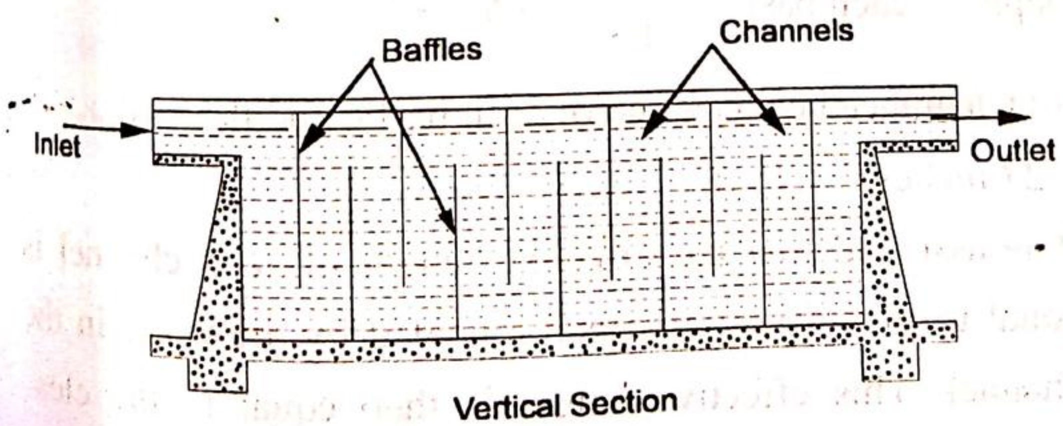


Fig. 8.7 Mixing Basin with over and under baffles

Assume: Distance between baffles = 18 inch

Mixing period = 20 mins.

Velocity of flow = 0.8 ft/sec

Clearance between end of each baffle and wall = 1.5×distance between baffles.

Solution :

(a) Velocity of flow + 0.8 ft/sec = 48 ft./min.

Total distance the water will travel = 20×48

= 960ft.

Volume of water to be mixed with coagulant in each period of 20 mins

$$= \frac{30.00.000}{60 \times 25} \times 20$$

= 41.670 gallons = 5.570 cu ft.

Cross-sectional area of each channel between

$$\text{baffles} = \frac{5.570}{960} = 5.79 \text{ sq. ft.}$$

$$\text{Depth of each basin} = \frac{5.70}{1.5} = 3.8 = 4\,ft.$$

(b) Clearance between end of each baffle and the wall = 27 inches = 2.25 ft.

It is assumed that the effective length of each channel is equal to the average distance travelled by the water in the channel. This effective length is then equal to the clea width of either half of the basin minus half the clearance at each wall or the clear width of the half of the basin minus the full clearance at one wall. Since the width of each half of the basin is limited to 10 ft in this case, the effective length of each channel is 10–2.25 = 7.75 ft.

No. of channels = 960/7.75 =124

(c) No. of channels at each half of the basin = 124/2=62.

No. of baffles at each half of the basin = 61

clear length of the basin (exclusive of the thickness of baffles) = 1.5 × 62 =93ft.

If the thickness of each baffle is 3 inch, the total length of the basin = 93 + 0.25 × 61 = 108.25 ft.

Mechanically Agitated Mixing Basins: These are basins in which Water is subjected to a violent agitation when the coagulant is first added to water and to a gentle action when the flock is forming. The first of these operations is usually achieved in mixers, called Flash Mixers and the second one in Flocculator.

In a Flash Mixer (Fig.8.8), the rapid mixing is caused in a rectangular chamber, by the revolutions of a propeller fixed to a shaft and driven by an electric motor. The period of detention may be minute (usually less than a minute).

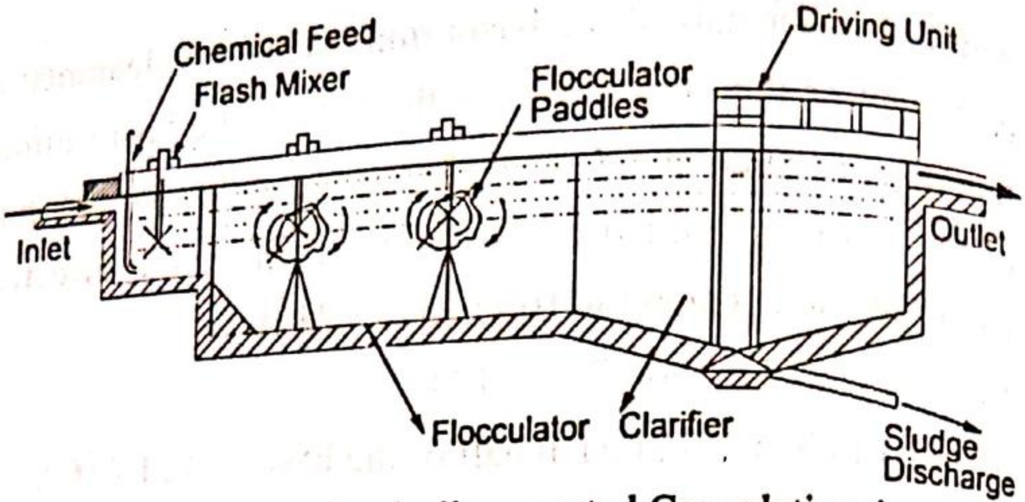


Fig. 8.9 Dorr Mechanically operated Coagulating Apparatus

In a Flocculator the slow stirring is mechanically brought about inside a circular tank equipped with paddles revolving on a vertical shaft. The paddles operate 2 to 3 revolutions per minute. Time allowed for flocculation varies from 30 to 60 minutes. In practice,. a combination coagulant feed, flast mix, flocculator and a clarifier (Fig.8.9) is quite commonly used. In the clarifier, small scrabers-are attached to radial arms moving towards a sludge pump from where sludge can be continuously removed.

The mechanical operated mixing basins have such advantages over the baffle type basins as (a) reduction in the amount of coagulant (as much as 40%) due to more thorough mixing, (b) negligible loss of head which in case of baffle type basins is higher,, about 1.75 ft, (c) greater flexibility of operation and (d) lesser cost in installation.

8.6 Filtration:

Filtration is a process of water purification in which water from a sedimentation tank is allowed to pass through, a bed a filtering media, usually sand and gravel, and the filtrate is collected at the bottom through an underdrain system. Essentially, a filter consists of a bed of sand and gravel to remove suspended solids from water, with devices to maintain a uniform rate of flow through the bed and with provision for periodical washing of accumulated solids from the filter media. The filter media are very efficient in retaining finer and colloidal particles of clay and silt. It also aids in removing colour, odour, turbidity, iron and manganese.

Types of Filters : There are two general types of sand filters commonly used in water purification. These are as follows :

1. Slow Sand Filters - usually gravity type

2. Rapid Sand Filters-both gravity type and pressure type. In gravity type filters, the water passes through the bed by the action of gravity alone. This type of filters are commonly used in municipal water supplies. Filter medium, is mostly sand.

In pressure filters, hydraulic pressure from a supply line forces the water through the bed. The pressure filters are mainly used in small installations like swimming pools, industries, etc. The filter media are sand, anthracites. or diatomaceous earth.

Slow sand filter was first developed and used in Great Britain early in the nineteenth century and later a number of plants of this type were constructed in the USA (from 1890 to 1910). The slow sand filter is also known as the British Filter. The rapid sand filter, also known as the American Mechanical Filter was developed in the USA during the period 1900–1910. Because of its greater adaptability and advantages over the slow sand filter, it has replaced largely the slow sand filter nowadays.

The general features of both slow sand filter and rapid sand filter are summarized in the Table 8.2

Table : 8.2 General Features of Conventional slow sand filter and rapid sand filter.

Features considered	Slow Sand Filter	Rapid Sand Filters
1. Rate of filtra tion	2 to 5 gals/sqft of filter area per hour	120 to 250 gals/sq ft of filler area per hour
2. Size of bed	Large, $\frac{1}{2}$ acre	Smal 1/100 to 1/10 acre
3. Depth of bed	12 inch of gravel, 42 inch of sand, usually reduced to not less than 24 inch by scraping	18 inch of gravel, 30 inch of sand, or less : not reduced by washing
4. Size of sand	Effective size : 0.25 to 0.35 mm ; uniformity coeffi- cient : 2 to 3	Effective size : 0.35 to 0.5 mm. and higher : unifoimity coefficient : 1.8 or lower, depending on the underdrainage system.

5. Size of gravel	$\frac{3''}{8}$ *to* $\frac{3''}{4}$	$^1/_{12}$" to $2^1/_2$"
6. Grain size distribution of sand in filter	Unstratified	Stratified with smallest or lightest grains at top and coarsest or heaviest at bottom
7. Underdrainage system	Split tile laterals laid in coarse stone and discharging into tile or concrete main drains	(a) Perforated pipe laterals discharging into pipe mains : (b) Porous plates above inlet box ; (c) Porous blocks with included channels-
8. Loss of head	2.5 inch initially to 4 ft finally	4 to 6 inch initially to 8 or 10 ft finally
9. Length of run between clearings	60 to 90 days	12 10 72 hrs.
10. Penetration of suspended matter	Superficial	Deep
Features considered	Slow Sand Filters	Rapid Sand Filters

11. Mathod of clearning	(a) Scraping off surface layer of sand, washing and storing cleaned sand for per iodic resanding of bed (b) Washing surface sand in place by was-her travelling over sand bed	Dislodging and removing suspended matter by upward flow or backwash-ing, which fluidizes the bed.
12. Amount of wash water required in cleaning sand	0.2 to 0.6% of water filtered	1 to 6% of water filtered
13. Pretreatment of water	Generally none	Coagulation, flocc-ulation and sedim-entation are required
14. Post filter treat-ment of water	Required, generally chlorination	Required, generally chlorination
15. Suitability	Not suitable for water Having turbidity more than 50 ppm with	Suitable for all types of turbid, coloured or tasty waters

	colour and taste	
16. Supervision	Skilled supervision is not necessary	Skilled supervision is necessary at every step
17. Bacteria removal efficiency	95 to 98%	85 to 95%
18. Efficiency to remove colour, odour, turbidity, iron and man-ganese.	80 to 85%	75 to 95%
19. Cost of cons-truction	Relatively low	Relatively high
20. Cost of operation	Low	High
21. Depreciation cost	Low	High

Theory of Filtration through Sand-bed: Filtration through a bed of sand aids in removing bacteria, finely divided suspended and colloidal matter. These phenomena are explained on the basis of the following four actions :

1. Mechanical Straining
2. Sedimentation and absorption
3. Bacterial action or Biological metabolism
4. Electrolytic Action

(1) Mechanical Straining : The straining action offered by the sand-bed removes the particles of suspended matter that are too large to pass through the interstices between the sand grains. As the small particles move through the pores in sand, they come in contact with sand surfaces and adhere. This process is aided by curved flow, paths around the grains, which by centrifugal force, throw the particles against the sand grain surfaces. The suspended particles within the pores, eventually increase in volume and adhere to sand surface; they also establish training action. This process cannot remove colloidal matter or bacteria too small to be strained out.

(2) Sedimentation and Absorption : This process accounts for the removal of colloids, small particles of suspended matter and bacteria. The interstices between the sand grains act as minute sedimentaticn basins in which the suspended particles settle upon the sides of said grains. These particles adhere to the grains because of the physical attraction between the two panicles of matter and became of the presence of a gelationous coatings farmed on the sand grains by previously deposited bacteria and colloidal matter.

(3) **Biological Metabolism :** It is the growth and life processes of living cells. This, together with electrolytic actions, causes chemical changes that occur in a water filter. This the grains of and in the top layer of the media. Adhesion of bacteria on this coating forms a zoological film or jelly around each sand grain in which the biological activities are carried out. This film is termed as "Schmutzdecke" or "Dirty Skin" and is wholly responsible for the detention of bacteria on the sand layer. The process is very prominent in slow sand filter.

(4) **Electrolytic action :** This action is explained by ionic theory. It states that when two ions with opposite electric charges are brought into contact the electrical charges are neutralized or so altered as to form different chemical substances. The electrical charges on the sand filter, and on the ionized matter in raw-water, react to alter the chemical constituents of filtered water. Ultimately the electrical charge on the filter is exhausted and the filter must be cleaned to renew these charges.

The Difference of Action between the two types of filters may now be explained.

In a slow sand filter, the action takes place principally at the surface of the sand-bed, though it is also continued for some distance below. The surface gets coated with a skin or layer formed due to the bacterial action of finely suspended matter, plankton and other organic matter present in raw water, with the algae, bacteria etc., previously coated on the surface of the sand-bed by the raw water itself. The layer is called schmutzdecke. The successful operation of a slow sand filter is principally dependent upon the existence of this layer. Below this layer and upto 12 inch or so it depth are present other bacterial zones. Here the actions involved are to completely oxidize the organic matter, destroy most of the bacteria present and let only simple and unobjectionable inorganic salts to pass through the filter bed into the effluent.

In the case of a rapid sand filter there is no such schnautzdecke acting as a surface straining-mat. The straining action in this case proceeds throughout the depth of the filter sand rather than, on the surface or to a small distance below this. Other actions involving complex biological and chemical changes as water passes through the sand-bed are similar to those in a slow-sand filter. It is believed that sedimentation and absorption are largely responsible in causing the suspended load to settle on the sand grains. It is then acted upon by the previously coated material and gets converted. The existence of the former i.e., the previously-coated material is important in this case. This material is largely fed by the pre-treatment or coagulation of water. That means that it is necessary for waters. to be coagulated before they are allowed to pass through the rapid sand filters. Other considerations for successful operation of these filters are proper thickness of sand-bed and size of the sand grains.

Filter Sand : The selected filter sand should be free from clay, loam, vegetable or organic matter. It should also be uniform and of proper size. If the sand is too fine, it tends to quickly clog, causing a greater loss of head in the filter and if it is too coarse, it will permit suspended solids and bacteria to pass through the voids between the sand grains. It is usual, therefore, to classify the filter sand by such characteristics as the effective size, the uniformity coefficient and the per cent size.

Effective size of the sand is defined as the sieve size in mm. which permits 10 percent of the sand, by weight to pass, or in other words, as the size of the grain that is larger than 10 per cent by weight of all the particles comprising the sand. This expression would merely indicate the minimum size of 90 per cent by weight of the sand. This does not give any information about toe degree of variation in the sizes of the particles or about the sizes of the largest and smallest grains. It is found that considerable variation in individual grain size adversely affects the efficiency of the filter.

Uniformity coefficient of sand is an expression of the degree of variation. This may be defined as the ratio between the sieve size, that will pass 60 per cent by weight to the effective size, or in other words, as the ratio of the particle size which is coarser than 60 per cent by weight of the sand to the effective size of sand. Thus, if sand has an effective size, of 0.50 mm and 60 per cent of sand passes a 0.80 mm sieve, the uniformity coefficient

$$= \frac{0.80}{0.50} = 1.60$$

Recently, with certain special sands, it is found that the expression per cent size is more suitable than effective size. Per cent size may be defined as the size of the grain that has the given per cent, by weight, of materials finer in size. On this basis, sands, 1, 10, 60 and 90 per cent sizes are specified (Table 8.3). Thus a per cent size of 10 means that 10 per cent of the sand is smaller than the grain size given.

Table 8.3 Per Cent Size Distribution of Filter Sand Grains

Per cent Size	Grain size, mm					
	Fine		Medium		Coarse	
	Min.	Max.	Min.	Max.	Min.	Max.
1	0.26	0.32	0.34	0.39	0.41	0.45
10	0.35	0.45	0.45	0.55	0.55	0.65
60	0.53	0.75	0.68	0.91	0.83	1.08
90	0.93	1.50	1.19	1.80	1.46	2.00

Depth and Grading of Sand Bed : The depth should be sufficient from the point of view of safe bacteria removal and for ensuring a uniform rate of filtration. The depth recommended in use should lie between 60 cm. to 90 cm (24 inch to 36 inch).

For a slow sand filter, the sand-bed is graded having top layer of fine sand underlying bottom layer of coarse sand. The effective size usually lies between 0.25mm. and 0.35 mm. and uniformity coefficient 2 to 3. For a rapid sand filter, depth of sand-bed is about the same i.e., 60cm. to 75 cm., but it is uniformly graded throughout with coarse sand having effective size lying between 0.35 mm. and 0.5 mm. and uniformity coefficient preferably not exceeding 1.8. The uniform grading increases void spaces, makes bottom and top of the filter bed equally effective, resulting in an increased rate of filtration.

Filter Gravel: This has, strictly speaking, no function in the actual purification of water. In a slow sand filter, it simply supports the sand-bed whereas in a rapid sand filter, this also serves to distribute the wash water evenly throughout the sand. It is usually placed in five or six layers totalling 12 to 18 inch or 30 to 45 cm. with the finest layer on top. The gravel used should be hard, rounded and durable.

Filterability of Water : The ability of water to be filtered is termed as its filterability. It is expressed quantitatively as the volume of water per unit head loss when passed at a strndard rate through a unit area of a standard filter. This is also termed as filterability index.

The knowledge of the value of this index is very useful in treating a raw water before filtering since higher the value of index the greater the volume of water that will be filtered between cleanings of the filter. Observations of the filterability index are also useful in the control of various filtration devices in a treatment plant.

Slow Sand Filter : This consists of a water tight tank 8-12 ft. in depth, having a sand-bed 24 to 48 inch thick, supported on a

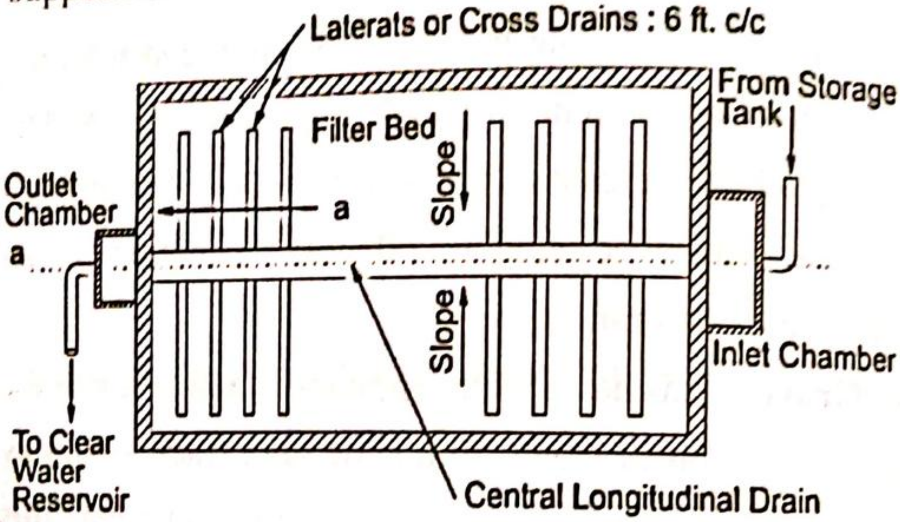


Fig. 8.10 Plan of Slow Sand Filter

bed of gravel 9-12 inch thick laid in 5-6 layers, beneath which the underdrainage system is laid over a concrete bed sloping towards a central longitudinal drain. The underdrainage system consists of open-jointed drains of baked clay or concrete pipe in lengths of 12-15 inch with a maximum spacing of laterals of 6ft. (Fig. 8.10)

Operation: The raw water is led gently on the filter bed. and percolating downwards passes through the underdrains into an outlet chamber (Fig. 8.10). The outlet chamber is provided with a regulating arrangement. consisting of a telescopic pipe and an adjustable wire-plate in order to keep the rate of filtration constant. It is also equipped with a loss of head gauge operated with a float arrangement to measure the loss of head i.e., the difference in the water level in the filter and in the outlet chamber. The outlet is so arranged as to prevent the possibility, of negative pressures and resulting filter damage.

Cleaning of Filter-beds : For a freshly cleaned filter. the loss of head is 4 inch 6 inch. After some use. the filter gets clogged necessitating an increase in the filtration head in order to keep the rate of filtration constant. This goes on till the maximum permissible limit of 24-36 inch has been reached. The filter is now taken out of service. About 5-10 inch of the sand surface is care, fully scraped off. The filter is then returned into service but the effluent is not turned into supply for 24-36 hours unless water is absolutely purified as determined by tests. The process of scraping of sand surface every time the filter is clogged is repeated till the sand-bed has been sufficiently thinned down to prevent efficient filtering. At this stage, the bed is topped up with clean new sand to its original level. Cleaning by scraping will normally be required once every 2 or 3 months, provided raw water is of a suitable character.

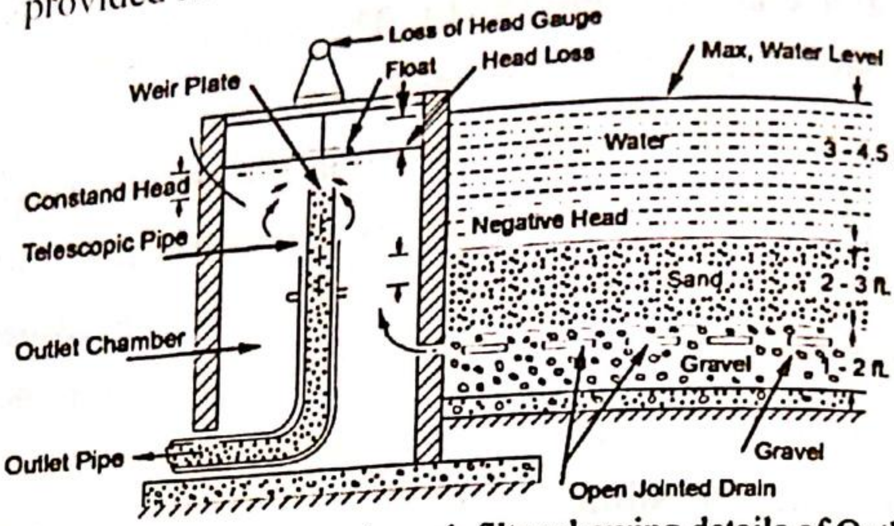


Fig. 8.11 Section a---a through filter showing details of Outlet Chamber

Characteristics : The essential characteristics of a slow sand filter are (a) Rate of filtration is low-2 to 5 gal. per sq. ft. per hour, (b) The bacterial efficiency is high-as much as 95-98 per cent, (c) Cleaning of sand is by scraping and removal. Other secondary characteristics are (d) high first cost and (e) unsuitability for waters having turbidity greater than 50 ppm. average turbidity being generally less than 50 ppm., (f) not very effective in the removal of colloidal matter.

Rapid Sand Filter (Gravity Type)

Construction : It consists of an open water-tight tank 9-12 ft. deep of masonry or concrete with a concrete floor, having coarse sand 30-36 inch thick laid on the top, with a layer of graded gravel 18 inch thick supporting below. The gravel is underlain by an underdrainage system consisting of a cast iron central longitudinal conduit or manifold with strainers mounted on top and pipes of smaller diameter called laterals branching off at right angles to the manifold. The laterals are fixed 6-8 inch centres and carry perforations on sides and bottom. about 5 to 8 ft. of water depth may be allowed on the filter bed.

Essential Characteristics : The essential characteristics of a rapid sand filter plant are (1) Careful pretreatment of the water to be filtered, (2) High rate of filtration and (3) Washing the filter beds by reversing the flow of the filtered water :

The flow diagram of a water treatment plant based on the principle of the rapid sand filter plant is shown in the Fig. 8.12

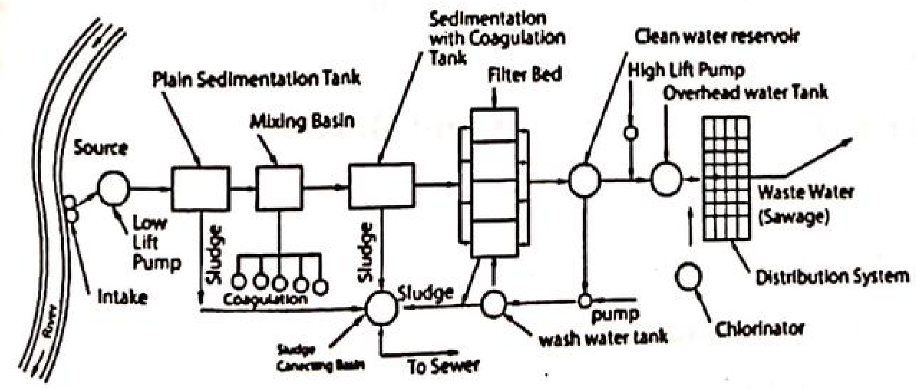


Fig. 8.12 Diagrammatic Sketch (flow diagram) of a typical water Supply System based on Rapid Sand Filter

The following are the various units of a rapid sand filter plant :

Principal Units (Structures) :

1. Plain Sedimentation basin or tank
2. Mixing and Flocculation Basins.
3. Coagulation-Sedimentation Basin
4. Filter Building
5. Filtered Water Basin
6. Overhead Wash Water Tank
7. Pump Building

Auxiliary Units:

1. Raw-Water Intake Line
2. Raw-Water Pump
3. Mechanical Mixing and Flocculation Devices
4. Pipe Gallery
5. Wash Water Pump
6. Dirty Water Drain
7. Service pump
8. Service Main

Other Essential Equipments and Devices :

1. Chemical Feeders
2. Filter Operating Gallery
3. Loss of Head Gauges
4. Rate of Flow Controllers
5. Chlorinators for Disinfection

Principal Parts of the Filter :

1. Inlet Pipe
2. Filter Bed with Sand and Gravel
3. Washwater Troughs
4. Underdrain System

Pipe Gallery

1. Settled Water Conduit (to filters)
2. Filtered Water Pipe (to filtered water basin)
3. Wash Water Pipes
4. Re-Wash Line (Filter to Waste)
5. Dirty Water Drain
6. Loss of Head Device
7. Rate of Flow Controllers
8. Appropriate Valves

The diagram of the rapid sand filter bed is shown in the Fig. 8.13.

The following is a brief description of the important parts of the rapid sand filter :

Enclosure Tank : This is usually a rectangular water tight box of reinforced cement concrete, generally 8'×12' to 16'×24' in sizes.

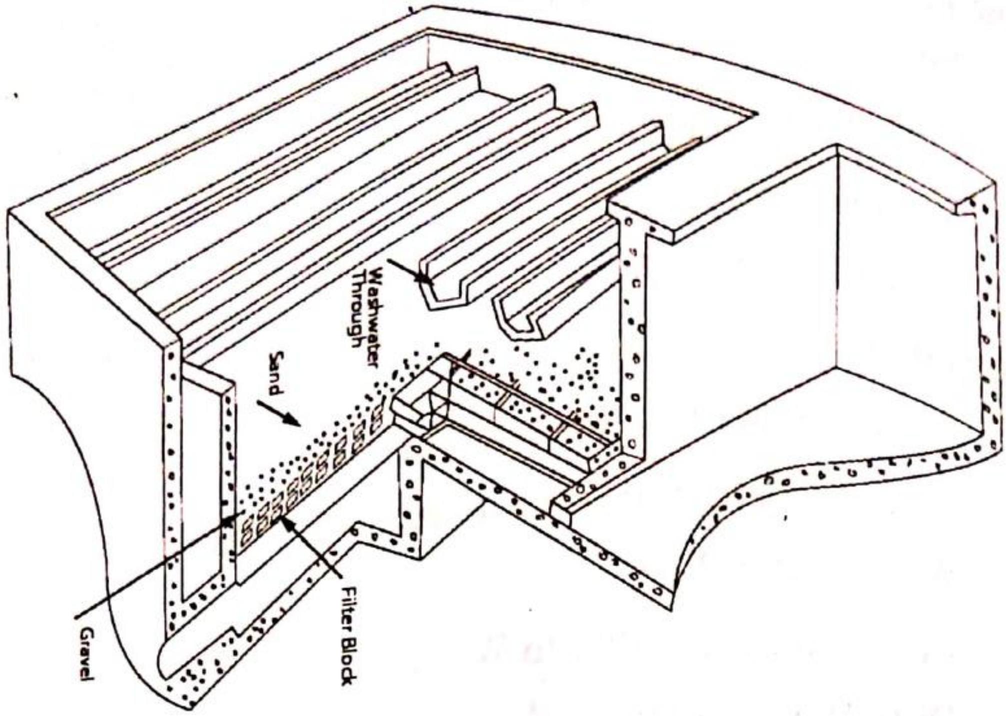


Fig. 8.13 Perspective View of a Rapid Sand Filter Bed

Filter Media : Coarse sand is usually used. The effective size should be 0.45 mm or more : uniformity coefficient, 1.8 or lower depending on the underdrainage system. The depth of sand varies between 24 to 48 inch. The coarse sand grains give less surface area and much void which gives high rate of filteration. Sometimes crushed anthracite coal, crushed magnetite and garnet sands are used as filter media.

Filter Gravel : Gravel is placed between the sand and the underdrainage system (1) to prevent sand from entering the underdrains and (2) to aid in uniform distribution of wash water. The filter gravel usually 10 to 24 inch thick, also support the overlying filter sand. The gravel size varies between 1/12 inch to 3 inch. The gravels should be carefully packed in the bed. The Fig. 8.14 shows the asymmetrical and symmetrical sequences of gravel below a sand bed. Because the behaviour of fine gravel and coarse sand in a stratified bed is much the same. The arrangement of supporting gravel is shown in the Fig. 8.14 (a) is by no means as stable as the design engineer may want it to be. For this reason, some engineers have introduced symmetrically from coarse to fine to coarse again. At the same-gravel interface, coarse sand slips into the interstices of the upper half, but there is no harm in this. The supporting gravel layer maintains itself no matter what the rate of wash-water rise is within reason.

Underdrainage Systems : Underdrainage systems of rapid sand filters perform two primary functions : (1) they collect the filtrate and send it on its may to a pure water reservoir, and (2) they distribute wash water to the bed during scouring operations. Because the rate of wash is many times that of filtration, the hydraulics of underdrains is governed by upflow requirements. If these are met, downflow distribution should be satisfactory. A secondary responsibility of underdrainage systems is to withdraw and waste chemical solutions added to filter beds from time to time, (1) to break-up, loosen and remove incrustants accumulated, on filter grains and (2) to break up mudballs formed and built up near the cleavage plane between supporting grains and filtering and expanding grains.

Two types of underdrainage systems are in common use in rapid sand filters : (1) pips grids and (2) filter floors or false bottoms.

Pipe Grids : In their simplest from, pipe grids comprise a main, called a manifold and perforated laterals (Fig. 8.15). Perforations are normally drilled into the laterals in a single raw of orifices directed downward at angles of 45° on either side of the vertical. To protect them against corrosion, pipe grids are normally lined with cement or bituminous and well coated on the outside,

Filter Floors : Filter floors, also called false bottoms or false floors are intended to replace pipe grides and serve two functions (1) support the filter bed, and (2) create a single, box-like waterway beneath the filter as the dispenser of washwater and the collector of filtered water (Fig. 8.16). The floor, depending upon its thickness, is perforated by short tubes or orifices of such dimension as to introduce the controlling loss of head that will make for even distribution of washwater. The openings must be relatively small and closely spaced, and their jets must be broken up by discharging from an effective nozzle or into a suitable depth of gravel. In some designs, inverted square pyramids are cast

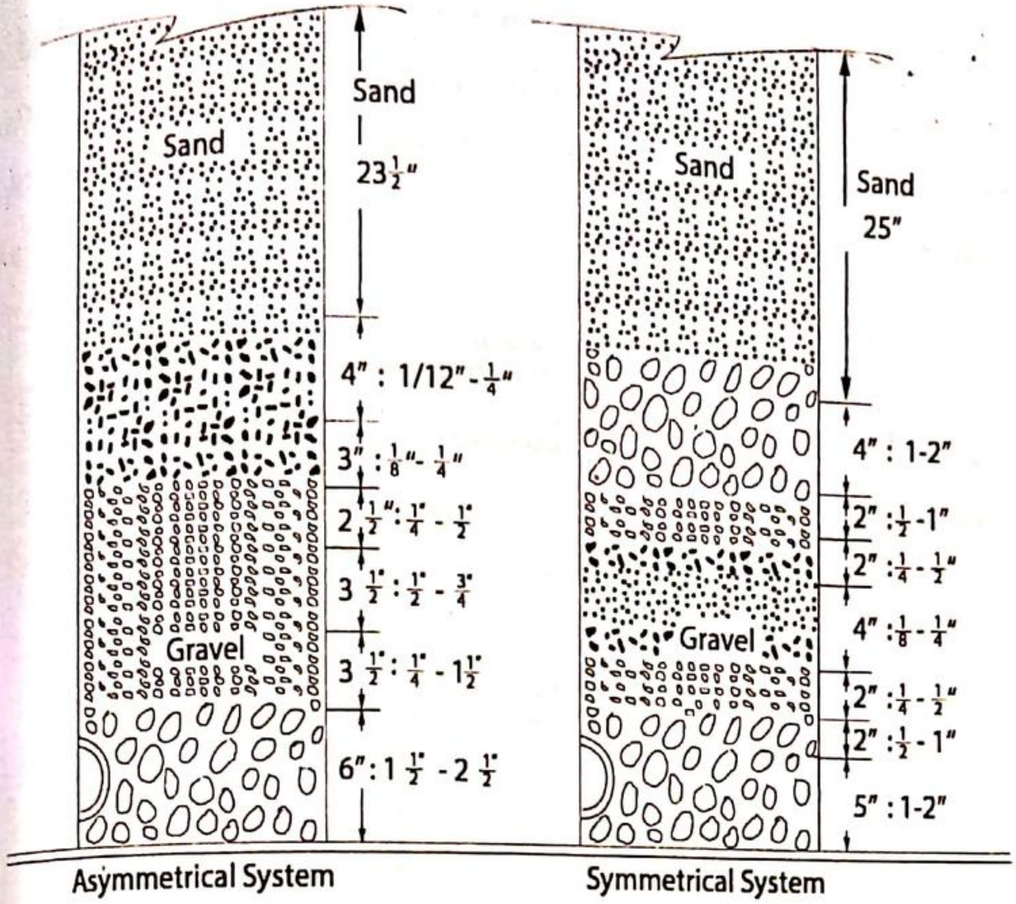


Fig. 8.14 Filter Beds

(Fig. 8.16c) into the false floor. Large and small spheres within these geometric depressions force the rising jets to spread out. In other designs, a checkerboard of porous plates is supported on bolts or beams that are anchored to the true floor or bottom of the filter box. Channeled porous blocks that create continuous waterways and are set directly on the true bottom may take the place of porous plates. Graded gravel is not required. The porous introduces the required controlling loss of head. Necessary calculations are best supported by model testing, because the hydraulic balance on the underdrainage systems is delicate and easily upset by differential clogging of the porous plates and the filter bed. Iron and lime are common clogging precipitates. Others are the result of after precipitation of coagulants and suspended matter. These

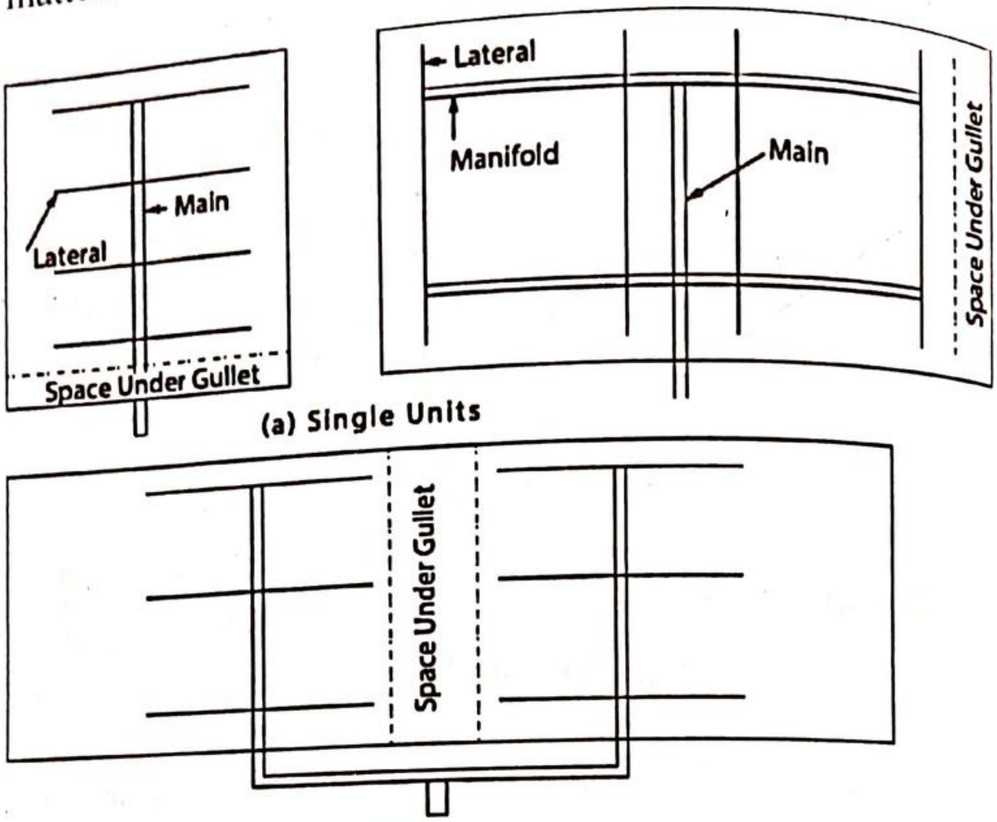


Fig. 8.15 Typical Arrangements of Perforated Pipe Underdrains in Rapid Sand Filter

reach the false floor from poorly cleanned or otherwise poorly operated beds. Slime growths are troublesome on occasion. However, it is generally possible to restore the porosity of clogged plates and blocks with acids or alkali washes.

Wash Water Troughs : Wash Water troughs serve to collect and carry to a main gutter the dirty water resulting from washing the filter. Troughs may be square, V-shaped, or semicircular. They should be set at such an elevation that the overflow lip will be at, or somewhat above, the top of the sand rise ; otherwise, sand may be washed out of the filter. A desirable height of the bottom of the trough above the top of the sand during operation (No

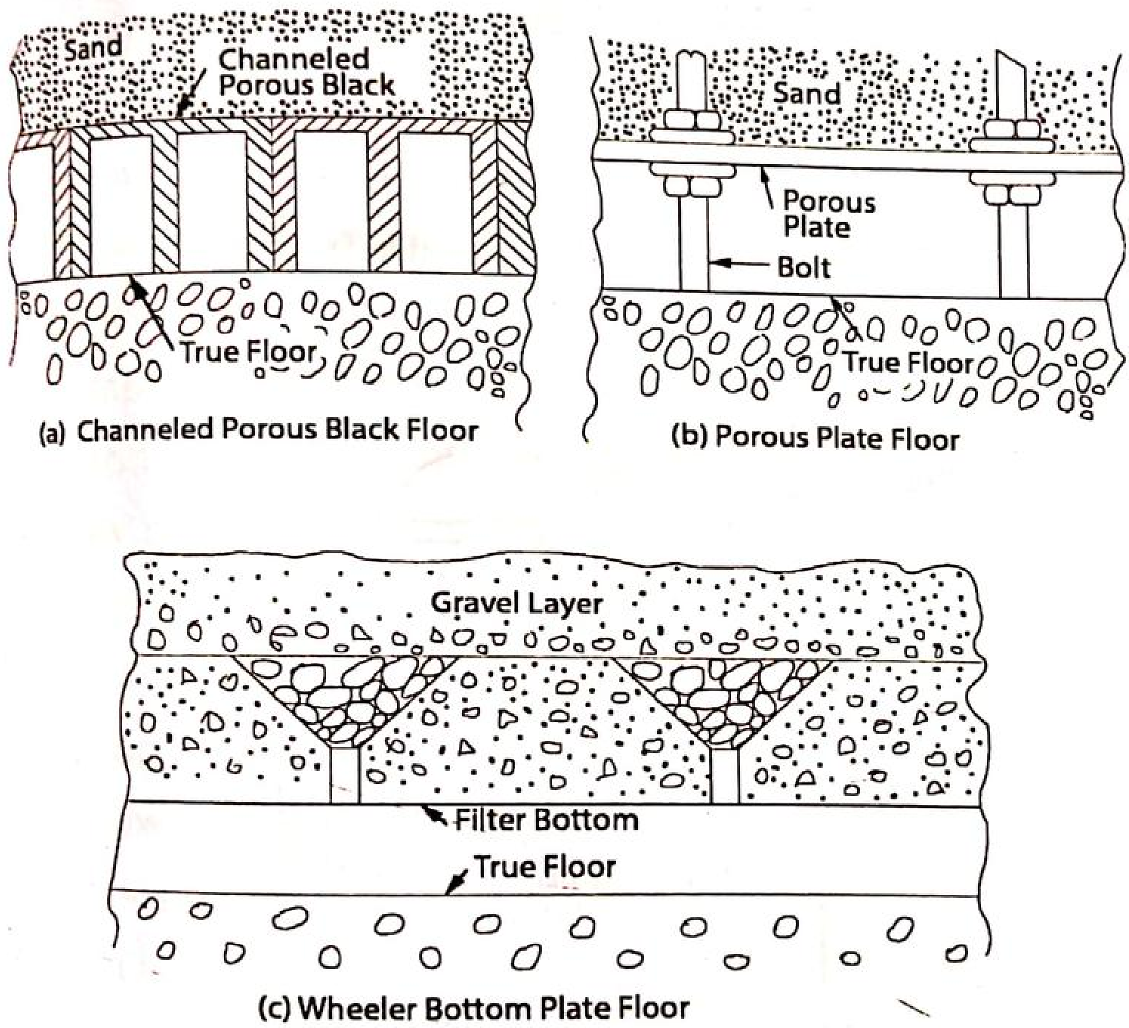


Fig. 8.16 Various Types of Filter Floors

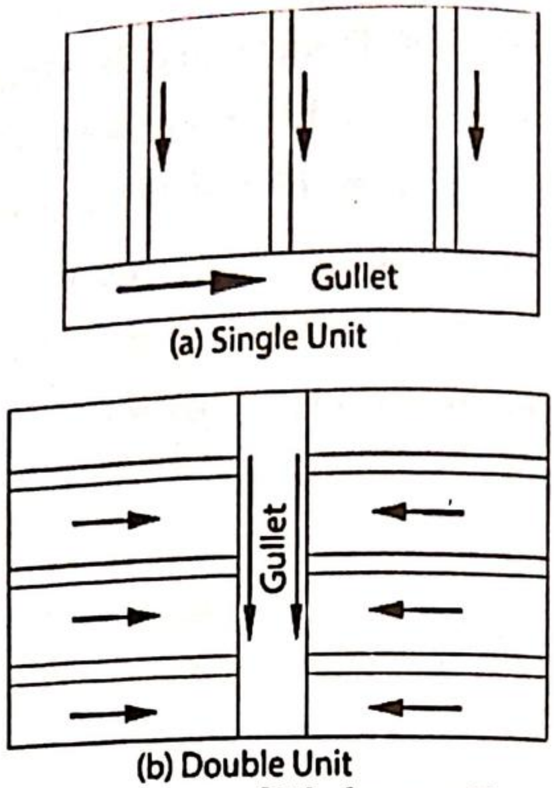


Fig. 8.17 Arrangement of Washwater Troughs

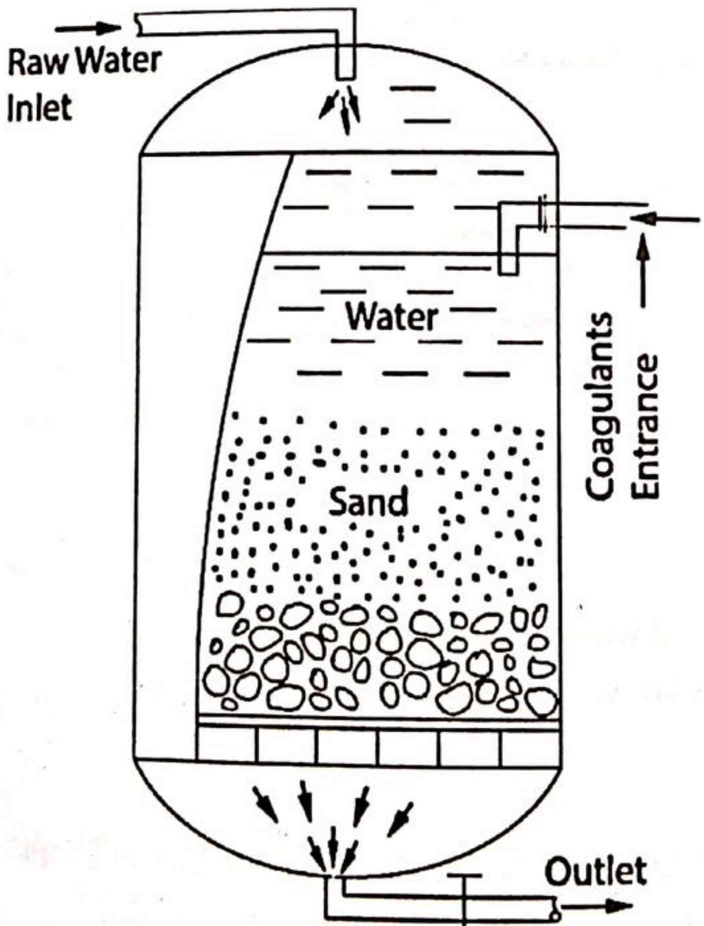


Fig. 8.19 (a) Pressure Filter (Vertical Type)

during washing) is approximately one-half the depth of the sand. Thus, for a sand bed depth of 28 inch, the bottoms of the troughs should clear the sand by about 14 inch. Flat bottom troughs with clearance above the sand less than about one-half the depth of the sand bed cause eddies during washing, and sand boils and other operating problems may result.

Troughs are spaced 5 to 7 ft apart. This spacing limits horizontal water travel to 2.5 to 3.0 ft and the water surface during washing is, therefore, maintained as nearly as possible at a uniform level. In the Fig. 8.17 are shown the various arrangements for wash-water troughs can be computed only approximately. Because of the overflow over both lips for the entire length of the trough, the surface of flow in the trough is not level and flow is not normal. Sizes have been standardized by manufacturers to correspond to the size of filter and the rate of washing. The flat-bottom rectangular troughs may be designed approximately by the following formula suggested by Miller and Ellms :

Q = 1.72 by 3/2

Where Q = total water received by the trough in gpm ;

b = width of the trough in inches :

y=depth of water at the supper and of the trough in inches.

An additional 2 inch of depth should be provided to allow for freeboard.

Example : A flat-bottom trough is to receive wash water from a section of the filter which is 6 ft wide and 8 ft long. The wash water rate is 15 gpm per sq. ft. If the water is to have a depth of 10 inch at the ' upper end of the trough. What should be the dimensions of the rectangular trough?

Solution : Q= 6×8×15= 720 gpm and y = 10 inch.

$Q = 1.72by^{3/2}$, $720 = 1.72 \times b \times 10^{3/2}$

$\therefore$ b= 15inch (width)

Depth of the trough=10+2= 12 inch (2" freeboard)

Example 2 : A V-bottom trough (with sides vertical) is to receive the wash water from a section of a filter bed 6.5 ft. wide and 8 ft long. The wash water rate is 15 gpm per sq. ft. Determine the dimensions of the trough. Assume the depth of water in the trough as 12 inch (effective).

Solution: Q = 6.5×9 × 15=875 gpm

$Q = 1.72by^{3/2}$. $875 = 1.72b\,(12)^{2/3}$

$\therefore$ b= 12 inch

If the V-bottom is a triangle with an elevation of inch it will have an area equivalent to a rectangle whose depth is 1.5 inch. With 2 inch freeboard, total depth will be 12+(3-1.5)+2= 15.5 inch and the vertical sides will be 12.5 inch.

Washing Process : Washing consists of passing filtered water upward through the bed at such a velocity that it causes the sand bed to expand until its thickness is 30 to 50 per cent greater than during operation. The sand grains move through the rising water, rub against each other and are cleaned of dirts (mud). A bed is usually washed when the gauge provided for the purpose shows that the friction head through the bed has reached 6 to ft or whatever amount the operator deems the advisable limit. The standard rate of application of wash water has been 15gpm per sq. ft. of surface area. This will produce a wash water rise of 24 inch permin. and a sand expansion 30 to 50 per cent with the smaller effective sizes. The larger sizes recently used require higher application rates although it appears that less than 30 per cent expansion will be satisfactory, particularly with surface wash. Some recent design engineers provide for a 48 inch rise, which will result in expansions of 50 per cent for sand of 0.55 mm effective size and 30 per cent for sand of 0.65 mm size. Most popular rise rate are 27 or 30 inch with surface wash when sands have effective sizes of 0.50 mm or more. The amount of water required for washing varies from 1 to 5 per cent of the total amount filtered with 2 per cent as an average figure. The period between washes will depend upon the character of the water. At times beds only operate 72 hr. without clogging, while at other times the run may be less than 24 hr. Therefore, length of run varies between 12 to 72 hr.

Wash water at most plants is supplied by gravity from an elevated shortage tank at the plant. The tank is placed to give a head of 30 to 40 ft. above the wash water troughs when the tank is full and holds enough water to wash two filters for about 5 min each. Laboratory observations showed that approximately 0.1 ft. of head is lost per ft depth of gravel when washing at a vertical rise of 12 inch per min. and that the head loss is in direct proportion to other thicknesses and rise rates. The total loss of head during washing will also include that in the underdrain system and in the piping system from the wash water tank to the filter. Actual washing requires 5 min. and the bed will remain out of operation for 10 min for resettlement of the sand.

Example : A filter bed has sand depth of 2.5 ft. and gravel depth of 1.5 ft. If is to be washed at the rate of 15 gpm per sq. ft or a rise rate of 2.0ft. per min. It is assumed that the specific gravity of sand is 2.65, its porosity is 40 per cent, and all of its is lifted or supported by the rising water. Calculate the total head loss in the filter bed.

Solution :

Head loss in the sand$=(2.65-1.0)\times(1-0.40)\times2.5=2.48$ ft.

Head loss in the gravel$=0.1\times2.0\times1.5=0.3$ ft.

Total head loss $=2.48+0.3=2.78$ ft.

Filter Effluent Box : The filtered water after being collected in the central drain is led into a small effluent box outside the filter unit tank. The water from the box next flows into a common filter water channel and then to storage reservoir.

Operating Difficulties : Rapid sand filters are subject to a veriety of difficulties; negative head, cracking of the bed, formation of mud balls, air binding, jetting and sand boils, sand leakage into the underdrainage system. These operating difficulties may become so great that the filter media and supporting gravel may have to be removed every two or three years, cleaned, Declassed and replaced in proper order. Troubles of this kind usually result from poor plant design and operation, particularly inadequate or disruptive backwashing.

Badly clogged filters can be restored to usefulness (1) by ejecting the sand and cleaning it in a sand washer, (2) by agitating the expanded bed by hand with the help of long-timed rakes (3) by directing boss streams into the expanded bed, and (4) by adding a detergent such as a 2 to 5% solution of caustic soda, draining it off through the filter to-waste connection, and washing the bed clean.

Negative Head and Air Binding : Negative head at any point in a filter is equal to the intensity of vacuum at that point and is usually a maximum at the point where the layer enters the under drainage system. When a filter is clean, there is some slight loss of head, say about 0.5 to 1 ft. in the sand, gravel and underdrains. As clogging occurs, the friction losses increase greatly, mostly however, in the top few inches of the filter sand. When the loss in the top layers becomes greater than the head of water above the sand the column of water below acts as a draft tube and a partial vacuum results. This condition is known as negative head and, when excessive allows air to escape from solution in the water and lodge in the sand. This is known as air binding and it may interfere considerably with filtration. Also, a mass of air may, at the beginning of the backwash, escape before the whole surface of the sand is broken. This will allow high local velocity of the wash water and may displace the gravel. The Fig. 8.18 shows a practical example how the phenomenon of negative head occurs. The point a is 11ft below the water level of the filter and that the total loss of head is 9 ft. The three tubes with their water levels at A, B and C show the pressure heads at the water level at b. which is 0.5 ft below the sand surface and at a. which is in 'the effluent respectively. With the total fictional head loss of 9 ft. it is reasonable to assume that 8.5 ft. loss occurs in the top 0.5 ft. of sand. By applying Bernoulli's equation between c and b, with datum at a, by equating pressure head and potential head at b plus friction, head to the total head (neglecting velocity head) the following equation is obtained in which P_b is the pressure'in pounds per square inch at b, and w is the weight of a column of water 1 ft high and 1 sq inch in cross-section. At *b* therefore, there is a partial vacuum or negative head of 3.5 ft.

The use of negative head has the advantages, in design, in permitting shallow filter boxes and, in operation, of permitting longer filter runs, provided no other difficulties arise.

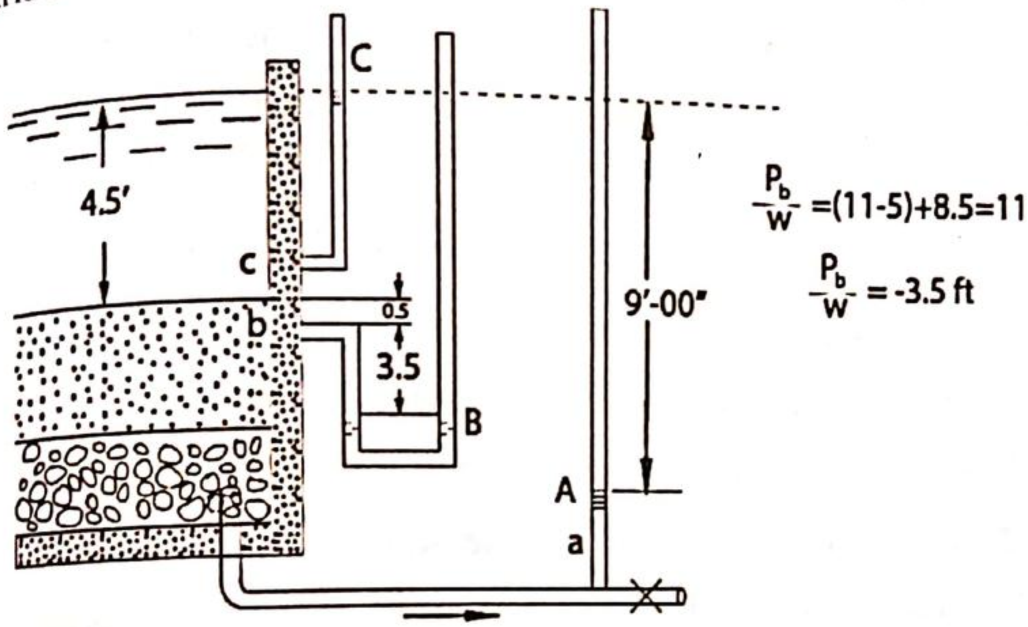


Fig. 8.18 Negative Head Phenomenon

Mud Balls : Mud balls consist of grains of sand, and of material carried over from the coagulating basin. Because of their lighter specific gravity, balls are found most densely collected at or near the top of the filter; but when of specific gravity equal to or heavier than that of the sand, they may be distributed throughout the sand and, in some cases, where the rising swirls and velocities of the wash water are least violent, thus inducing unevenness in the rise of wash water and aggravating the inequality of the filter wash. Large caked lumps of sand are more commonly found attached to the gravel or to the side walls of the filter.

It is believed that the cause of mud ball formation is insufficient; washing of the sand which permits gelatinous material to adhere to the surface of the sand grains. The effect is noticed particularly in fine sands, less than 0.4 mm indiameter; and is aggravated by the presence of microorganisms, manganese and other substances present in water.

Methods of removing mud balls in the filter after they have formed include (1) dipping them off the surface with strainers while wash water it running through the filter at a slow rate; (2) breaking them up with rakes and subsequently off the particles (3) washing the sand in place to break up the mud balls with high velocity surface wash; (4) washing the filter with a soluiion of some chemicals such as caustic soda for alum deposits and organic material, sulpher dioxide for dissolving iron, manganese and alum, hydrochloric acid or carbonic acid for calcium carbonate deposits' chlorine for biological growths, copper sulphate for algae, etc and (5) removing, clcarning and replacing the sand.

Cracking of Filter : Because the resistance to flow is least along the walls of a filter, less head is lost there and resulting pressure differentials within the bed establish inward as well as downward flows; filter grains are pushed away from walls; water short-circuits through insulating shrinkage cracks and fills with dirty matter which penetrate deeply into the bed. even into the gravel. The cracking of filter impaires both the washing of the filter and' the efficiency of filtration. This phenomenon is remedied by the same methods. as the formation of mud balls is overcome.

Jetting and Sand Boils: Even in small difference in porosity and permeability of sand and gravel cause the first flush of backwash water to flow paths of least resistance and break through to the surface at scattered points. Within the zone of flow, the clogged and compressed sand is fluidized, back pressure is reduced. flow is increased, and water is jetted from the gravel into the sand. If jetting becomes severe, the sand boils up like quicksand, and gravel as well as sand is lifted to the surface. If, in defence against these happenings, backwash valves are opened slowly, the bed is given an opportunity to disintegrate from the surface downward. Surface wash is sometimes helpful to remove these difficulties.

Sand Leakage : Sometimes, sand leaks into the underdrainage systems if gravel layers and sand are not properly sized and placed and if smallest gravel particles are displaced during backwashing.

Loss of Head in Filter Operation : The frictional resistance to the passage of water through the filtering media and the under drainage systems causes a loss of head in the operation of the filter. The loss of head is equal to the vertical distance between the surface of the water on the filter and the elevation of the hydraulic grade line at the filter outlet. This elevation can be determined by placing a piezometer in the effluent pipe and observing the water level in the piezometric tube. The loss of head immediately after washing should not be more than 4 to 6 inch. The maximum permissible head loss is 8 to 10 ft.

Operation of a Rapid Sand Filter Plant : The operation of a rapid sand filter plant for a municipal water supply demands careful consideration of the following :

(1) Careful pretreatment of the raw water.

(2) Routine observations that should be recorded in the control of operation of the filter plant may include (a) the quantity of water treated daily, (b) length of run of each filter between washing (c) percentage of wash water used for each filter, (d) quantity of chemicals used and rates of application for all purposes, (e) alkalinity, turbidity, colour, temperature and solid contents in the raw, settled, filtered and chlorinated water, and (f) presumptive tests for coliforms in the raw, settled, filtered and chlorinated water.

(3) Conditions that must be guarded against in the operation include (a; formation of mud balls, (b) occurrence of negative head and air binding, (c) cracking of filters, (d)

jetting and sand boils. (e) sand leakage, (f) overloading, (g) shortening of filter run.

Design of Rapid Sand Filter Plant : The dimensioning of filters and their appuartenances depends on the incoming water quality, filter type, process and hydraulic loading, method and intensity of cleaning, nature, size and depth of filtering materials and the desired quality of filtered water.

Performance objectives to be reconciled in the design are terminal head loss, standards of effluent quality (for instance, turbidity) and length of filter run.

Finally, the design demands excercise of judgement based of an investigation of local conditions and features of (be plant layout which include (a) economy in construction and operation, (b) future extensions, (c) soil conditions, foundations and structural problems, (d) flexibility in operation, (e) compactness and convenience to minimise materials required, minimise head losses and simplify operation, (f) utilization of topography to minimise excavation and backfilling, and to make proper use of gravity in operation.

The size of the filter is determined from the required capacity of the plant. The number of units is generally determined by the following empirical equation.

$$N=2.7\sqrt{Q}$$

in which N is the number of units and Q is the plant capacity in mgd.

The filter beds are usually rectangular in size, having the ratio of length and breadth as 3 : 2.

Example 1 : A rapid sand filterplant is to be designed for a capacity of 6 mgd. What should be the number and size of the filter units? What should be the percentage of filtered water required to wash the filter beds? What should be the capacity of wash water tank?

Assume: (1) Rate of filtration : 2 gprn per sq. ft.

(2) Rate of washing : 15 gpm per sq. ft.

(3) Length of filter run : 24 hrs including 5 min for washing the filter bed and 10 min for resettlement of sands.

Solution :

(a) The plant will operate only 23 hr and 45 min.

Filtration rate = 2×60×23.75=2850 gallons per day per sq ft.

Filter area required = (6,000,000/2,850)=2,100 sq ft.

No. of units. $N=2.7\sqrt{Q}=2.7\sqrt{6}=7$

Area of each units =2100/7 = 300 sq ft.

Size of each unit = 20' × 15'

(b) Total wash water required=15×2100×5= 157,500 gallons per day

Percentage of filtered water required

=(157,500/6,000,000) ×100 = 2.625%

(c) Capacity of wash water tank-1,57,500 gallons.

Example 2 : A filter bed has an area of 360 sq ft. If the washing for 5 min at the rate of 24 inch per min is contemplated, how much wash water will be required?

Solution : 24 inch rise of water is attained by applying the wash water at the rate of 15 gpm per sq ft of filter area. Wash water requirement-15 × 360×5=27,000 gallons.

Example 3 : A rapid sand filter operating at 2 gpm per sq ft needs washing after 24 hr of operation. The filter has an area of 350 sq ft and it needs washing at the rate of 15 gpm per sq ft for 5 min. The time required for resettlement of sand is 10 minutes.

What per cent of the water that is filtered will be required for wash water.

Solution : Filtration rate = 2×60 ×23.75=2,850 gpd ped sq ft. Capacity of the plant=2,850×350=997,5000 gpd. Wash water requirement=15×5×350=26,250 gpd percent of filtered water for washing the filter=(26,250/997,500) × 100=2.63

Efficiency of Rapid Sand Filter: With proper pretreatment of the water, rapid sand filters are applicable for treatment of any surface water. Rapid sand filters are very efficient in removing bacteria, colour, odour, turbidity, iron and manganese. The following are the percentage removal of the above mentioned water quality.

F-23

Water Quality	Percentage Removal
Bacteria	85 to 95
Turbidity	80 to 95
Colour	75 to 90
Odour	70 to 90
Iron and manganese	60 to 85

Pressure Filters : Filter is essentially a rapid sand filter contained in an air tight container and through which water passes under pressure. The Fig.8.19 shows the section of typical pressure filters. The sand bed is usually 18 to 24 inch thick with effective size and uniformly coefficients of the sand following those of the gravity rapid band filter. Gravel layers follow the same practice as in gravity rapid sand filters and the underdrams are either pipe grids or false bottoms. Washing is accomplished through reversing the flow by manipulating valves in the piping.

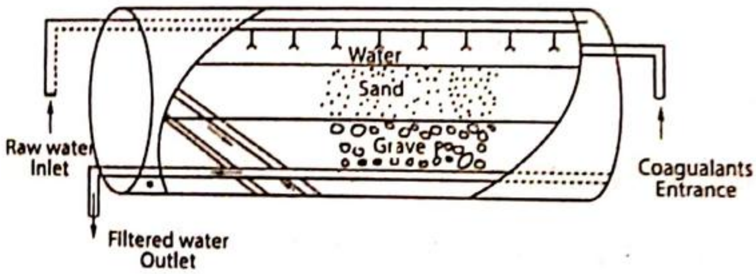


Fig. 8.19 (b) Pressure Filter (Horizontal Type)

A loss of head gauge indicates when washing is needed.

Both vertical and hrizontal units are 6 to 12 ft. in diameters and 8 to 20 ft long pressure filters are used principally for swimming pools, industries and for small installations for public water supply. The principal objection to their use for public water supplies is the difficulty in providing adequate space for coagulation and sedimentation. The water is given-only a small does of coagulant before it reaches the filter. It is however, impractical to use a pressure filter where the water is highly turbid. Rates of operation for pressure filters depend on the quality of water being filtered. Filtration rates generally range from 2 to 5 gpm per sq ft of filter area. The swimming pool rate is usually 3 gpm per sq ft.

Example : A pressure filter operates at the rate of 2.5 gpm per sq ft. If the diameter is 12 ft. how much water will be filtered during an 8 hr shift?

Solution: Surface area of the filter $=\{\pi(12,^2/_4\}=113$sqft.

Amount of water filtered $= 2.5\times113\times 60\times8$

$= 132,600$ gallons.

8.6 Disinfection of water : The production of water of satisfactory quality is often dependent upon the elimination or destruction of two groups of living organisms: (1) the pathogenic microorganisms which may infect man through his use of polluted water and (2) the algae and other aquatic growths which may render water aesthetically unfit for human consumption. The purpose of disinfecting water supplies is to kill pathogenic organums and thus prevent the spread of water-borne diseases. Most pathogenic bacteria and many other harmful microorganisms are destroyed or removed from water in varying degrees by most of the conventional treatment processes. The destruction and removal is brought about in several ways : (1) physical removal through coagulation, sedimentation and filtration (2) natural die-away of the organisms in an unfavourable environment during storage ; and (3) destruction by chemicals introduced for treatment purposes other than disinfection.

Although the number of microorganisms in polluted waters is reduced by treatment processes and natural purification, the term disinfection is used in practice to describe treatment processes that have as their sole objective of killing the pathogenic organisms. Strictly defined, disinfection is the destruction of all pathogenic organisms while sterilization is the total destruction and removal of all microorganisms. These two terms are similar but quite different. Disinfection is usually brought about by heating ultraviolet irradiation, chlorination, radioactive substances and the like. These methods (except chlorination) and materials can be used to sterilize water, but it is usually impractical to do so from an economic point of view. Disinfection by bleaching powder is for more practical.

Bleaching powder contains about 33% chlorine when fresh. Chlorine has an immediate and disastrous effect on most forms of organisms and is an ideal disinfectant. When chlorine is added to water, it reacts according to the following equation.

Cl_2 +	$H_2O \rightleftharpoons$	$HOCl$ +	HCl
clorine	Water	Hypochlorous acid	Hydrochloric acid

The hypochlorous acid, HOCl is very unstable and ionizes or dissociates into hydrogen ions (H^+) and hypochlorite ions OCl^-) in another reversible equation of the following nature.

$HOCl \rightleftharpoons$	H^+ +	OCl^-
Hypochlorous acid	Hydrogenions	Hypochlorite ions

It is the hypochlorous acid and hypochlorite ions which accomplish the task of disinfection. The chlorine existing in water as hypochlorous acid, hypochlorite ions and molecular chlorine is defined as free available chlorine.

If ammonia (NH^3) is present in water, other compounds formed are monochloramine (NH_2Cl), dichloramine ($NHCl_2$) and nitrogen trichloride (NCl_3) according to the following reaction.

$NH_3 + HOCl \rightarrow H_2O + NH_2Cl$

$NH_2Cl + HOCl \rightarrow H_2O + NHCl_2$

$NHCl_2 + HOCl \rightarrow H_2O + NCl_3$

The chlorine present in water in chemical combination with ammonia or other nitrogenous compounds is known as the combined available chlorine.

These resulting chlorine compounds either in the form of free or combined available chlorine interfere with certain enzymes in tie bacterial cell-wall forming a toxic chlorocompouhd thus destroying the bacteria completely. This is the generally accepted theory explaining the action.

Another theory describes the destruction of bacteria to the liberation of nascent oxygen ($HOC1 \rightarrow HCl + O$) which oxidizes the organisms. This theory is considered to be inadequate and obsolete on the grounds that the quantity of nascent oxygen produced is too less. for the purpose.

The effect of chlorine as a disinfectant is principally dependent upon the period of contact and the concentration of chlorine in water. The killing power or power to disinfect is regarded proportional to the product of contact period and the chlorine concentration. Besides, the pH value of water, the water temperature and the presence of residual chlorine in the form of free available or combined available chlorine have a definite effect on the effectiveness of chlorine. Thus, it is found that in the pH range of 7.0–9.5. a minimum of 0.05 mg/L, free available chlorine can be effective with water temperature between 5°–22°C. Disinfection proceeds very slowly at lower temperature and with pH valve above 8.5. Also whereas free available chlorine is effective in concentration of 0.05 mg/L for a contact period of 10-20 minutes, the combined available chlorine, to be comparatively effective within the same reaction period, requires a higher concentration upto about 1.8 mg/l.

This shows that the combined available chlorine is not generally as effective as free available chlorine.

To test residual chlorine in water, 1 ml of orthotolsidine solution is added to 100 ml of the sample. If a yellow colour results, the sample contains a chlorine residual. The deeper the yellow, 'the greater the residual. If chloramines are formed, the orthotolidine test will then show a residual and this is the combined available residual. If additional chlorine is added, not only are the other compounds oxidized but the chloramines also. The orthotolidine residual will then show the hypochlorous acid, hypochlorite ions, and molecular chlorine if any and this is known as the free available chlorine residual

Chlorine Demand : The chlorine demand of a water is the difference between the amount of chlorine added and the amount of cholorine present as a residual either free or combined, after some designated period.

Application : In the application of chlorine to water certain important characteristics of chlorine should be understood. Chlorine is a greenish yellow gas, $2^1/_2$ times heavier than air. When compressed, the gas liquefies. The liquid chlorine can be stored in steel cylinders which can be transported. At 21°C, the pressure required to maintain the liquid form is 7.6 kg./cm^2.

The pressure inside the cylinder however increases with the temperature. When liquid chlorine is drawn from the cylinder, it changes into gas and temperature inside the cylinder path. Chlorine may be applied by any of the following methods.

(1) As dry chlorine gas drawn from the liquied chlorine cylinder, it is applied directly to the water supply through submerged diffusers. This method is unsatisfactory because of improper diffusion and resulting corrosion in pipes.

(2) As chlorine solution, as in a solution feed calorimator, formed by mixing chlorine gas with a small quantity of water. The chlorine solution is applied to the water supply by means of a water injector through a discharge line. This method is very commonly used in works practice.

(3) In powder form as hypochlorites, bleaching powder Ca(OCl) or sodium bypochlorite (NaOCl). The available chlorine varies from 25 to 33 percent for bleaching powder to 70 percent high strength hypochlorites. Bleaching powder is not as stable as high strength hypochlorites. It also loses strength of long storage or exposure.

Hypochlorites are applied to water as a solution by means of hypochlorite-feeding apparatus. This may comprise of a constant-head orifice feeder or diaphragm pumps equipped so as to deliver the desired amount of solution of specified strength. This constant head in the former case is maintained by a wooden float at the orifice which may be large enough to prevent clogging. In general, equipment used for chlorination must be reliable and capable of operation within narrow limits of accuracy. To little chlorine is ineffective, too much may cause tastes and odours. Operation of chlorinator should be automatic so that the amount of chlorine fed to water should be proportional to the volume of flow and the chlorine demand of water.

Chlorine Dosage : The amount of chlorine required to be added to the water supply can be determined in the laboratory by adding varying dosages of chlorine to equal proportions of the waste? sample and finding the amount of residual after a contact period of 10-20 minutes.

Example : It is required to disinfect 5,00,000 gpd of water with 0.3 mg/l of chlorine. If bleaching powder is used (which coatains. $33^1/_3$ percent of available chlorine), how many pounds of bleaching powder are needed to treat the daily flow of water ?

Solution : Available chlorine in the bleaching powder is $33^1/_3\%$

$\therefore$ 1 lb of chlorine is available in 3 lbs of bleaching powder.

Since 1 mg/l of chlorine = 8.34 lbs of chlorine per million gallons of water.

$\therefore$ Amount of bleaching powder required per million gallon $= 3 \times 8.34 = 25\text{-}02$ lbs.

$\therefore$ Amount of bleaching powder required for a flow of 500,000 gpd and a chlorine dosage of 0.3 mg/l.

$$= 25.02 \times \frac{500,000}{1,000,000} \times 0.3 = 3.75 \text{ lbs}$$

Minimum chlorine Residuals for drinking water at 20°C

pH Valve	6-7	7-8	8-9	9-10
Free available chlorine, mg/l after 10 mins	0.2	0.2	0.4	0.8
Combined available chlorine, mg/l after 60 mins	1.0	1.3	1.5	1.8

Special Methods : Chlorine is generally applied after all other treatments have been given to the water supply. This may be termed as post chlorination and is the standard treatment at all waterworks. There are, however, other special methods of chlorination to be used depending upon the particular purposes to be gained.

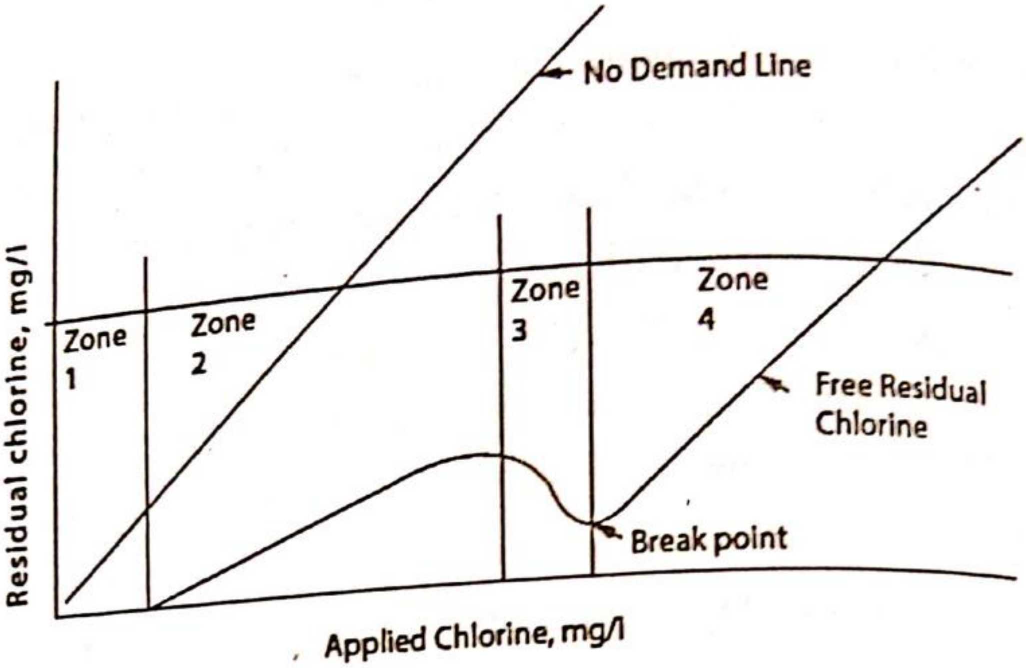
(a) Pre-chlorination is the application of chlorine preceding filtration, either added into the suction pipes of raw water pumps or to the water as it enters the mixing basin. Pre-chlorination reduces bacterial load filters resulting in increased filter runs, and oxidizes excessive organic matter thus removing taste and odour.

(b) Double chlorination is the application of chlorine at two points in the treatment process. It is essentially pre-chlorination with an added treatment to the final effluent from the filters. Advantages claimed with this method of chlorination are (i) decrease in the load on filters, (ii) greater removal of bacteria higher bacterial efficiency (iii) greater factor of safety due to maintenance of two chlorinating plants and (iv) control of algae and slimy growths in coagulating basins and filter.

(c) Super chlorination is the application to water of an excess amount of chlorine. The dose may vary from less than 1 ppm, to 2 ppm. The method is effective in destroying high concentrations of tastes and odours in water. Bacteria removal is also high. However, it becomes necessary to move resulting tastes due to excess chlorine by the use of de-chlorinating agents like activated carbon, sodium thiosulphate etc.

(d) Break point chlorination also termed free -residual chlorination involves the addition of sufficient chlorine so as to oxidize all the organic matter, reducing substances and free ammonia in raw water leaving behind mainly free available chlorine which possesses strong disinfecting action against pathogens.

It is observed that when the dosage of applied chlorine to water high in ammonia or organic matter is gradually increased, the reactions are marked by the appearance of following four zones (Fig. 8.20).



Applied Chlorine, mg/l

Fig. 8.20 Break point chlorination.

Zone -1 Destruction of chlorine by reducing compounds.

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

Zone-3 Destruction of chloro-organic compounds and chloramine.

Zone - 4 Formation of free available chlorine.

The addition of chlorine at the break (or dip) is termed break point chlorination. This indicates the point at which free residuals begin to appear. Usually, all tastes, odour? chlorinous and others disappear at this point resulting in appearance of waters free from bad tastes and odours. Further, because of the highly persistent and powerful disinfection possessed by free available chlorine, any type of pathogenic organisms present in. water is destroyed making disinfection highly efficient.

Other Methods of Disinfection:

(1) Ozonization : The effectiveness of ozone in the disinfection of water lies in its high oxidizing power. Ozone is an unstable isotope of oxygen containing 3 atoms of oxygen (O_3), which while changing to the stable molecular form (O_2), releases nasent oxygen (O). The nascent oxygen reduce organic matter present in water without the production of objectionable tastes and odours as with chlorine. The ozone dose is 2 to 3 ppm. to give a trace to 0.1 ppm residual after 10 minutes contact.

Ozonization is regarded as a "natural" means of disinfecting water and is particularly useful in disinfecting waters containing bacterial spores. It is, however, costly to manufacture, has very little residuals present and is not quite suitable for highly turbid waters.

(2) Ultra-violet Rays : Ultra-violet Rays is an effective method for disinfecting of clear water. Use is made of the invisible light rays beyond the violet of spectrum which are very powerful in, killing all types of bacteria, casts and spores. The rays are generated by passing electric current through mercury-vapour lamp enclosed, in a quartz bulb. Water requiring disinfection is pasted over the lamp. The effective penetration of the rays in water is only fat a depth of 30 cm or so.

The process has the advantages of causing no taste, no odour in the water and presenting no danger of overdose. The disadvantages are high cost and absence of any residual action. Its use is restricted normally to small installations like swimming pools.

(3) Excess Lime : Excess lime involves the application of sufficient lime for the combined objectives of softening and disinfecting of water. Coliform reduction may be as high as 99 percent. Dose to be given is between 10 and 20 ppm. It frequently becomes necessary to remove the excess lime after the process through recarbonation.

(4) Silver : When immersed in water 'actuated' silver has been observed to exert an inhabiting action on bacterial life. Tubes of silver electrodes contained in hollow cylinders, allowing water to flow from outside to inside, are energized. Dosage is 0.05 to 0.10 ppm, and the period of contact about 3 hours. The process is called Electro-Katadyn Process. It is claimed that this process is effective in destroying the bacterial spores and algae present in comparatively clear waters. The method, however, has not so far been used on a large scale because of high cost.

(5) Iodine and Bromine : Iodine and bromine also possess disinfecting power. Their use is normally restricted to small water supplies such as army camps and swimming pools. Water which is muddy or coloured needs to be filtered before disinfection. Iodine and bromine are cheaply available in the form of pellets. Dosage is about 8–10 ppm. For heavily polluted water, it may be doubled. Only objection is the resulting medicinal taste.

(6) Potassium Permanganate : Potassium permanganate has been frequently used in individual water supply for disinfection. Its action is principally based on its oxidizing capacity on organic matter. Dosage is 0.5 gram per litre. Though effective against cholera vibrio, it has not been found to be effective against other disease germs. It also produces coating on glass or porcelain vessels which is difficult to be easily cleared without scouring. Potassium permanganate is neither regarded satisfactory nor recommended for water disinfection.

8.7 Aeration :

This is the process of bringing waters into intimate contact with air with the object of driving out objcetionable dissolved gases and oxidizing other soluble compounds present in the ground waters or in stagnant waters of pools and reservoirs.

Aeration ia effected in many ways (i) by causing the water to flow over weirs and waterfalls called Cascade aerators. (ii) by dropping water through perforated plates. (iii) by forcing it throught pray nozzles. (iv) by filtering through perforated trays, coke beds, and (v) through special devices which aspirate air by diffusion porous plate. The spray nozzle is the most effective aerator. Aeration is effective in removing 75 percent of the odours. Removal of carbon dioxide is equally high.

8.8 Taste and Odour Control :

Tastes and odours in water supplies may be caused due to the presence in water of any of the following :

(i) Decaying organic matter resulting from algae and other micro-organisms, (ii) industrial wastes such as phenols. (iii) chlorophenol compounds resulting from combination of residual chlorime with phenol, (iv) dissolved gases like H_2S, CO_2. (v) excess amount of chlorine.

Methods for the control of tastes and odours, therefore, amount to treating the aforesaid causes of trouble and may be listed a below :

(a) Copper sulphate treatment, (b) Ammonia-chlorine process. (c) Use of activated carbon, (d) Use of chlorine dioxide, (e) Aeration, (f) Prechlorination, (g) Superchlorination followed by dechlorination and (h) Ozonization.

Methods (e) to (h) have already been described, others shall now be dealt with.

Copper Sulphate Treatment is chiefly used for the control of algae which is destroyed within a few days following treatment copper sulphate is applied in a single dose usually use than about 3 ppm. An excessive dose often results in destroying fish-life along with other micro-organisms present in water. Copper sulphate is generally applied by dragging jacks of crystals through water with the help of boats or by the spraying action produced by dropping pulverized crystals on rotating discs mounted in a boat. Copper sulphate is also effective in killing many types of aquatic weeds.

Ammonia Chlorine Process consists of applying ammonia with chlorine which results in the formation of chloramines. The compounds have the effect of prolonging the disinfecting action of chlorine. As a result of this extended action, quantity of chlorine used is reduced. This process is also helpful in removing chloro-phenol tastes. As the chloramines do not react with the organic matter, full effect of chlorine as a disinfectant may be utilized in. the presence of organic matter. In the application of this process., ammonia is added first, thoroughly mixed with water before chlorine-is introduced. Usual ratio of ammonia to chlorine is 1 : 4.

Activated Carbon is probably the most important method for the control of taste odour. Activated carbon is obtained by heating carbonaceous material like lignite, sawdust, paper mill waste etc., in closed retorts and then activating, by passing air or steam for the removal of hydrocarbons which may otherwise-interfere with the adsorption of organic matter. Activated carbon is very porous and has the property of removing many of the dissolved impurities in water, H_2S, iron and manganese through the adsorptive action of carbon atoms. It also acts as a dechlorinating agent to remove phenolic tastes and colour.

Activated carbon is available both in the granular as well as-powered from. The letter form is most commonly used. Dosage: varies 2 to 20 ppm.

Chlorine Dioxide is produced by injecting a solution of sodiums chloride into a chlorine solution as it leaves the chlorinator. The: resulting reaction is :

$$2NaClO_2 + Cl_2 = 2NaCl + 2C1O_2,$$

(Chlorine dioxide)

The chlorine dioxide produced is a powerful oxidizing agent and is quite effective in the removal of tastes aad odours caused by phenols and other algal growth. The dose required for effective taste and odour control varies, as 0.5 to 1.5 ppm.

8.9. Iron mod Manganese Removal : Iron is present in water either as ferrous bicarbonate or ferrous sulphate. Manganese is often associated with the iron present. Iron and manganese when present in amounts greater than 0.3 ppm. are objectional because of (i) unpleasant taste and odour due to decay crenothrix organism present in iron-rich water (ii) colouring of water, standing paper, fabrics producing rust spots and (iii) deposits of iron precipitations in pipes.

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

(a) Aeration is an effective method in precipitating out iron when present as ferrous bicarbonate and subsequent removal through the processes at sedimentation and filtration. If, however, iron is present as ferrous sulphate, it would be necessary to add lime.

(b) Base Exchange Process : The principle involved in the process is discussed in Art. 8.10.

The zeolite bed is made of manganese zeolite obtained by treating the base exchange material with manganese sulphate and potassium permanganate. The iron and manganese of raw water are oxidized to insoluble hydrated oxides and later removed by filtration. Potassium permanganate is used for regeneration process. It is necessary that the raw water would not have been aerated prior to entering the base exchanger otherwise the already precipitated iron and manganese may clog the zeolite bed.

(c) Chlorination either alone or in combination. with aeration also sometimes employed.

Water Softening Processes : Hardness or the soap destroying power of water has already been discussed in chapter 7. We have also seen the causes of hardness and the deleterious effects produced. We shall now discuss methods used for softening of water. There are three general methods used for water-softening, (i) Lime process (ii) Lime and soda ash process and (iii) Base-Exchange Process. The broad difference between the first two methods and the last method lies in that whereas we remove the hardness compounds in the former case, we only change the compounds in the latter case. The result is that the total solids or impurities are reduced in the first two methods but are not reduced in the last methods.

Lime Process reduces only the carbonate hardness. The principle involved is to neutralize the CO_2 with milk of lime i.e. Ca $(OH)_2$, forming normal carbonates which precipitate out when present in excess and are removed by settlement and filtration. The process is also known as the Clark Process. Chemical reactions involved are :

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

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

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

Lime and Soda ash Process : Lime has no effect on sulphates of calcium and magnesium, which are responsible for causing most of the non-carbonate hardness found in natural waters. However by the use of soda ash (sodium carbonate), the non-carbonate hardness can be removed. The reactions are :

(i) $CaSO_4,+ Na_2CO_3 =CaCO_3+Na_2SO_4$

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

$CaSO_4+Na_2CO_3=CaCO_3 +Na_2SO_4$

Most of the insoluble compounds, which precipitate out in the above-mentioned methods, will settle out if the treated water is passed through a settling, basin. As, however, some of the insoluble material tends to remain in a finely-divided state and may be found to be later depositing on the filter sand or in the distribution mains service pipes and metres, this difficulty may be overcome by treating the partially-clarified water from settling basin with carbon dioxide, before it reaches the filters. This process called recarbonation, has the effect of converting any unprecipitated carbonates back into the soluble carbonate form.

Base Exchange Process : In this process hard water is passed through a bed of zeolit sand (complex silicates of aluminium and sodium) whereby it exchanges its Ca and Mg for the sodium (Na) in the zeolite until Na becomes exhausted. The sodium is then restored by regenerating the zeolite with a solution of common salt (NaCl) resulting in the reversal of ionic reactions viz, Ca and Mg taking the place of Na and being washed out, chlorides while the Na remains in the zeolite so that it is again ready to act as a softening agent. Chemical reactions are :

For softening :

$$\underset{\text{Hard water}}{\begin{bmatrix} CaSO_4 \\ MgSO_4 \\ MgCO_3 \\ CaCO_3 \\ CaCl_2 \\ \text{etc.} \end{bmatrix}} + \underset{\text{Zeolite}}{Na_2Al(SiO_3)_2} = \begin{bmatrix} Ca \\ Mg \end{bmatrix} Al(SiO_3)_2 + \begin{bmatrix} Na_2CO_3 \\ Na_2SO_4 \\ 2NaCl \end{bmatrix}$$

Hard water Zeolite

For regeneration :

$$\begin{bmatrix} Ca \\ Mg \end{bmatrix} Al(SiO_3)_2 + 2NaCl = \underset{\text{Zeolite}}{Na_2Al\,(SiO_3)_2} + \begin{bmatrix} CaCl_2 \\ MgCl_2 \end{bmatrix}$$

Zeolite

Zeolites are artificial products in granular form with size varying between 0.5 mm. to 0.25 mm. in diameter. Its trade names are Permutit, Verdite etc. The zeolite softeness is very much similar to the rapid gravity filter with the difference that the zeolite layer is thicker 1.2 to 1.8 m. and the flow of water may be either upwards or downwards. The treated effluent can show water of zero hardness.

The zeolite or the base-exchange process of water softening is applicable only to clear water since the presence of turbity causes elogging of the zeolite material. Its use is, therefore, limited to ground waters.

8.11 Swimming Pool-Water Purification :

It is often found necessary to recirculate used water of swimming pools for treatment purposes, as this arrangement is more economical than the removal of water time and again, for use.

The most satisfactory method of treatment involves forced circulation of water through pressure filters, to be followed by super-chlorination and dechlorination, while maintaining a certain amount of chlorine residual. The dose may be as much as 3 to 4 ppm to leave a residual of 0.5 ppm.

The process of filtration is greatly assisted by adding to water a coagulant of the type and in dose sufficient to combat the degree of pollution involved. Alum (Aluminium sulphate) is the usual coagulant used. Since a heavy dose of alum is likely to result in the lowering of pH value or in the acidification of water, giving water such unpleasant characteristics as odour and causing irritation to nose and throat, it would be necessary to correct it by adding soda ash or sodium bicarbonate.

For keeping the water in good condition, especially in the case of indoor swimming pools, it is better if auxiliary method of disinfection such as aeration could also be provided as discussed, for methods of aeration. In order that pumps operate satisfactorily in recirculating water and no small solid particles may clog the pump-impellers, it would be advisable to have some physical treatment, involving mechanical straining of water prior to filtration. This may consist of a 'hair-filter' having grilles.

Sometimes to prevent the growth of algae, especially in the case of open air swimming pools, sulphate in small doses is also added.

8.12 Fluoridation :

Though fluorine in excess of about 1.5 mg/l is undesirable and in higher amounts injures the teeth of children, it has been found that a fluorine content of about 1 mg/l prevents dental caries in children. In many cases, such a fluorine content has reduced tooth decay by 50 to 65 per cent.

Consequently where the water is deficient in fluorine, a great many communities add this chemical to the water. Several fluorine compounds are available for water treatment, namely, sodium fluoride, hydrofluoric acid, hydroflue silicic acid and sodium silicoxide. Of these, the most widely used one sodium, fluoride, which is usually fed as a solution by utilizing chemical feed equipment, and also sodium silicofluoride. Usually the solute is fed under pressure. Control is established by accurate feeding and by frequent tests to determine the fluorine concentration in the water. Equipment used for applying fluorine is normally capable of maintaining a fluorine content within about 0.05 mg/L of the desired concentration. It appears that water has no fluorine demand and that all fluorine added is indicated by the Standard Tests.

8.13 Defluoridation :

A water that contains more than 1.5 mg/l of fluorine is generally considered objectionable. Various methods are employed for removal, but all of them are difficult. Among the methods that have been successful are (1) bringing the water into contact with tricalcium phosphate $Ca(PO_3)_2$ (2) softening the water by the use of lime and soda ash with the addition of magnesium, if lacking, to accomplish removal of fluorine by the use of magnesium oxide or hydroxide, (3) cation exchange, and (4) the use of hydroxy apatite, $3ca_3(PO_4)_2 Ca(OH)_3$

8.14 Recarbonation :

The caustic alkalinity in water softened by the excess-lime method can be removed by the application of carbon dioxide to the water. Recarbonation stabilizes the $CaCO_3$ content and reduces the tendency of colloidal carbonates of calcium and hydroxides of magnesium to precipitate on the grains of the filter sand and in the pipes of the distribution system.

The reaction by which calcium carbonate and carbon dioxide, form calcium bicarbonate may be written as follows :

$CaCO_3$	+	CO_2	+	$H_2O \rightarrow$	$Ca(HCO_3)_2$
Calcium	+	Carbon	+		Calcium
Carbonate		dioxide		Water	bicarbonate.

8.15 Demineralization :

By "demineralization" is meant the removal of those dissolved mineral constituents which, for one reason, or another, cause water to be unsatisfactory for either domestic or industrial consumption.

(1) Hard water, objectionable because, of scale formation in boilers, heating system, resulting in great heat transfer losses and danger of boiler failure is the first and most

important example of necessity for eleminating undesirable mineral constituents.

(2) The second and really the only one example of practical and worth-while demineralization is the removal of iron and manganese when present in qualities sufficient to cause trouble from taste, odours staining and discolouration. **Use of demineralized Water :** (1) Manufacture of Soft drinks. (2) In the high pressure boilers. (3) In the pharmaceutical works. (4) Many other industrial plants like textile, paper manufacture, etc,

Processes of demineralization

Hydrogen Zeolite-available under the name Zeo-karb catex and organolite. They are sometimes called carbonaceous zeolites because they are made from coal and lignite. Hydrogen zeolite exchange all cations for hydrogen.

HYDROGEN EXCHANGE

Raw Water	Hydrogen Exchange bed	Treated water	Exhausted Exchange bed

$$\begin{array}{l} Ca \\ Mg \\ NO_2 \\ Fe \end{array} \begin{bmatrix} (HCO_3)_2 \\ Cl_2 \\ SO_4 \end{bmatrix} + H_2x = H_2 \begin{bmatrix} CO_3 \\ SO_4 \\ Cl_2 \end{bmatrix} + \begin{bmatrix} CO \\ Mg \\ NO_2 \\ Fe \end{bmatrix} x$$

REGENERATION

Exhausted exchange bed	Sulphuric acid	Hydrogen exchange bed	Waste water

$$\begin{bmatrix} Ca \\ Mg \\ NO_2 \\ Fe \end{bmatrix} x \quad + H_2SO_4 \quad = \quad H_2 \times \quad + \begin{bmatrix} Ca \\ Mg \\ NO_2 \\ Fe \end{bmatrix} SO_4$$

Where X Stands for Zeolite

Advantage : The advantage of hydrogen zeolite is that it contains no silica, hence treated water does not have it silica contents increased which it specially desirable in boiler water

(2) Strong Base Anion Exchange : The water treated by using hydrogen zeolite contains carbonic acid (H_2CO_3), Sulphuric acid H_2SO_4, and hydrochloric acid (HC1).

The acid can be removed by passing water through a strong have anion exchange material, known as Di-Acidite, Anex etc. The anion exchange bed removes sulphates, chlorides, nitrates, carbonates and leaves the water practically demineralized.

Bed can be regenerated by using Na_2CO_3, or caustic Soda (Na_2OH).

STRONG BASE ANION EXCHANGE

Cation exchange Anion exchange Treated Exasusted bed water exchange bed

$$H_2\begin{bmatrix} SO_4 \\ Cl_2 \\ (NO_3)_2 \\ SiO_3 \\ CO_2 \end{bmatrix} + A(OH)_2 = H_2O + A\begin{bmatrix} SO_4 \\ Cl_2 \\ (NO_3)_2 \\ SiO_3 \\ CO_3 \end{bmatrix}$$

REGENERATION

Exhausted exchange bed	Caustic Soda	Regenerated exchange bed	Waste water

$$A\begin{bmatrix} SO_4 \\ Cl_2 \\ (NO_3)_2 \\ SiO_3 \\ CO_2 \end{bmatrix} + 2NaOH = A(OH)_2 + NO_2\begin{bmatrix} SO_4 \\ Cl_2 \\ (NO_3)_2 \\ SiO_3 \\ CO_3 \end{bmatrix}$$

Where A stands for exchange material.

It is demanded that water treated by the above two methods first through the hydrogen zeolite and second, through the anion exchanger, can be compared with distilled water in quality.

8.16 Desalinization :

Since 1955 there has been a rapidly increasing interest the development of processes that will convert saline water to fresh water. This interest has been stimulated by the realization that freshwater supplies will soon be inadequate in many areas.

Open water has a dissolved-salts content of about 35,000 mg/l. There are several large desalinization plants that are removing salts from sea water. Many groundwater supplies are blackish (dissolved salts 1000 to 3000 mg/l and are much too salty for consumption. The salts can be removed, but the cost is great. There are many processes for removing salts from the water. Some of them are discussed in the following paragraphs.

Distillation : Distillation of seawater has been practiced for many years. Recent research has been aimed at the development of improved evaporators that would have minimum difficulty with scale formation. Various types of vapor-compression and multiple effect flash systems show promise. Solar stills have been used successfully in areas having a high proportion of sunlight throughout the year. With a solar still the energy costs are nil, but the cost of investment is high per unit of producting.

Freezing : In the freezing process the temperature of the seawater is gradually lowered untill ice crystals are formed. These are free of salt and can be separated from the brine.

Demineralization : Salts can be removed from water through use of ion changers. The process is similar to zeolite softening except that sodium is removed by hydrogen-cation exchanges. The process is prohibitively expensive for use of seawatcr, however, it is well adapted for use on waters with salt content of less than 1000mg/l.

Electrodialysis : By this method ions are removed by an electrochemical process whercin in they diffuse under the action of an electric potential through membranes that are selectively permeable to different types of ions. In the process there is about 1 gal of water wasted for each gallon produced. Cost of salt removal by electrodialysis is proportional approximately to the amount of salt in the water. Because of extremely high cost, electrodialysis is unsuitable for use with seawater. It is likely that the first widespread application of desalinization will beakish water. Ocean water, when converted to fresh water, generally has to be pumped and transported considerable distances to the place of use, thus further adding to the cost of such water.

8.17 Removal of Colour :

Colour may be removed, as other colloids are removed, by absorption and by chemical precipitation, possibly followed by sand filtration. No one method will remove all colour. Other methods include bleaching by sunlight in open reservoirs or by the use of chlorine. Coagulation with alum, followed by filtration is probably the most widely used procedure. About 1 grain of alum per gallon of water should remove 10 ppm of colour under favourable circumstances. In the removal of colour by chemical coagulants the same principles apply as in the addition of coagulants for other purposes.

8.18 Removal of Radioactivity :

In planning treatment of water contaminated with radioactive materials, four things must be known (1) the initial concentration of the radioactive materials, (2) the composition of the main constituents, which requires a radiochemical analysis of water, (3) the MPC (Maximum Permissible Concentration) established for the radioisotopes in question, and (4) the remaining or residual activity. The MPC has an important influence on the type of treatment, while the residual activity permits determination of the efficiency of the treatment process selected.

Studies have been made of the effectiveness of various methods of water treatment in the removal of radioactive materials. Conventional coagulation has produced removals in excess of 90 percent for most cationic radionuclides with a valence of 3 or more, for p-32 as phosphate, for 1-131 as iodine when silver nitrate was added, and for Pu. The elements Sr and Cs were not removed. Filtration will generally remove only those radioactive materials associated with suspended solids. The lime soda process of softening has been effective in removing most radionuclide. Excess calcium aids precipitation of strontium, and high removals have been reported.

Cation-exchange resins were found effective for removing cationic radionuclides, while anion.exchange resins were effective for removing anionic radionuclides, either in separate columns or in mixed beds.

Modified techniques have also been used successfully. Strontium and other radionuclides which form insoluble phosphates can be removed effectively by phosphate coagulation. In removing strontium, careful control of the pH and of the phosphate-calcium ratio is necessary. An electrolytic cell of effective for removing soluble constituents, but colloids had to be removed prior to treat-ment. Except for 1-131 and Cs-137, Ba-137, 90 precent removal of radioactive contaminants was obtained by using iron as metallic dust. Distillations is the most effective method of radionuclide removal, but it is expensive.

QUESTIONS

1. What are the objectives of water purification and how can you achieve them? What causes colour, odour and tastes in water and how can you remove them? (BUET,64, 68, 70)
2. What is potable water? How can you make water potable? What are the causes of presence of colour, turbidity, phenolic compounds, lead and acidity in water supplies ? Name the processes by which you can remove them individually and also mention about their permissible concentration in drinking water. (BUET, 62, 67, 70, 72, AMIE, 67)
3. Discuss in brief the principles involved in the design of sedimentation tank. (BUET, 65. 73)
4. "In a sedimentation tank, area and the overflow rate rather than the detention period should govern the design". Co-mment on the statement. (AMIE, 72)
5. Critically examine the factors that affect the water treatment by the process of sedimentation with coagulation. (BUET, 70, 72)
6. A mixing basin is treat 6 mgd of raw-water. The basin is to be divided into two similar parts by a longitudinal partition wall of 6 inches thick so that each half will have a clear width of 10 ft. Also, the distance between baffles is 2 ft, the mixing period is 20 minutes, the average velocity of flow is to be 0.8 fps : and thickness of each baffle is 5th of the spacing of baffles : If a free-board of 6 inches in allowed

with the depth, what should be (i) the total depth of the basin, (ii) the number of channels in each half, and (iii) the over all inside length of the basin. (BUET, 66)

7. A circular sedimentation tank is to crest 0.92 million gals, of raw water per day. The average spiral velocity of flow is 0.25 fpm. and the detention period is 3.25 hours. If the radius of the tank is 1.5 times the effective depth calculate the diameter and total depth of the tank. An allowance of 3 ft is to be taken for sludge deposit. Also give a neat proportionate sectional elevation of the tank with all necessary details. (BUET, 67)

8. Give a brief description of "The theory of filteration" in a sand bed. (BUET, 64, 67, 70, 72, 73, AMIE, 65, 12).

9. A city water treatment plant has a design capacity of 8 mgd. Design rapid sand filter bed units and calculate the capacity of wash water tank. Assume standard values of data not supplied. (BUET, 70).

10. Make a neat diagramatic sketch of a water treatment plant based on rapid sand filters and discuss functions of each units. (BUET, 68, 72, AMIE, 64, 69)

11. (a) What is meant by "filtration of water"? Discuss briefly the operation difficulties of a rapid sand filter plant.

(b) A rapid sand filter bed having as area of 700 sq. ft. operating at 2 gals, per sq.ft. per minute, needs washing at 15 gals/sft/min. for 5 minutes in a day. Three troughs are placed in each unit and the depth of the trough should not exceed I5 inches including a free-board of 2 inches. Calculate (a) the dimension of the flat-bottomed wash water trough and fb) the percentage of filtered water required for washing the filter beds. (BUET, 66).

12. (a) What are the essential characteristics of a rapid sand filter plant? Explain briefly the theory of filtration through & sand-bed.

(b) A rapid sand filter plant is to treat 1.44 mgd of raw water. If the allowable rate of filteration is 2 gals/sft/min. and wash water-requirement is 15 gakl/sft/min.

(i) What would be the capacity of the wash water tank ? (ii) What would be the size of the flat bottom wash water trough ? The depth of the trough should not exceed 14 inches including a free-board of 2 inches. Assuming, standard value of date missing.

13. What are the characteristics of a Rapid sand filter? Make a comparative study of the merits and demerits of Rapid sand and Slow sand filters.

14. A rapid sand filter plant is to be designed for a city with a prospective population of 200,000 to be supplied with 45 gpcd. Determine.

(i) No. and size of filter units.

(ii) Capacity of overhead wash water tank which will be able to wash two units at a time. (iii) Leading dimensions of a flatbottomed wash-water trough when wash water does not travel more than 3'-0" laterally. (BUET, 65).

15. (a) What is coagulation-sedimentation ? How does it differ from plain sedimentation ? Briefly explain the influence of each factor that affect sedimentation with coagulation.

(b) A rectanguhr sedimentation tank is to treat 960,000 gals, of water per day. Given :

Depth of tank just before cleaning	: 10
Settling velocity of particles	: 0.50 fpm.
Length : Breadth	: 3:2

Determine :

(a) Dimensions of the tank

(b) Overflow rate in gph. (BUET.70).

16. (a) What would be the dimension of rapid sand filter unit having a capacity of 1 mgd if it is to operate at 2 gal/sft/min?

(b) With a wash-water rise of 24 in per min, compute the rate of wash-water application and the total amount of wash-water used in a 5 min. wash. (AMIE, 67).

17. (a) If the rate of operation of filter is 2.5 gpm per sq ft of filter surface how large an area will be required to filter the water for a population of 12,500 using 120 gpcd ?

(b) What will be the washwater requirements for this filter if the entire area t& washed at one time ? (AMIE, 72).

18. (a) Explain the processes of disinfection of water. What is break-point chlorination? Explain fully.

(b) How much chlorine, in pounds per hour, will be required to treat 10 rtgd of water with 0.4 mg/L of chlorine? (BUET, 68).

19. How many pounds of bleaching powder with 30 per cent of available chlorine will be required to treat 4 million gal of water with a dosage of 0.5 mg/L ? (AMIE, 66).

20. With a flow a 72,000 gpd, and a chlorine doages of 0.4 mg/L, how much hypochlorite with an available chlorine content of 70 per cent will be required in a day ? (BUET, 72).

21. (a) State the criteria commonly employed in the design of plain sedimentation tanks used at water treatment plants.

(b) What should be the size of a rectangular sedimentation tank to treat 1 ragd, with 2 hour detention and overflow rate less than 20,000 gal. per day per sq. ft. of the surface area. (BUET, 65).

22. "Theoretically, depth is not a criteria in designing a sedimentation tank for the removal of granular solids." Justify the statement. (AMIE, 66, BUET, 73).

23. (a) What is the action of coagulants added to raw water ?

(b) What is flocculation ? What are the common aids used to make the process more efficient? Explain the reactions involved.

(c) What are the factors affecting good coagulation ?

(d) What effect has the depth of a sedimentation tank on its efficiency ?

24. State the object of adding 'alum' to water before filtration. When is ferrous sulphate preferred to alum ? Mention the function of a sedimentation tank and explain the various points to be remembered in its design. (AMIE, 69).

25. With the help of a sketch of the essential features describe the arrangement and working of a rapid sand filter of the gravity type.

26. (a) Why must be the rate of filtration controlled and how is it done?

(b) What are the usual rates of filtration allowed in rapid and slow sand filters respectively ? Why are these different rates to be used ?

(c) What is air binding ? What are its effects ?

(d) Why is the uuderdrain system necessary in sand filters? (BUET, 1964).

27. (a) Explaine what is meant by sterilization of water. Describe the various methods of application of chlorine for sterilization of water and explain one method in detail.

(b) Explain fully-Break point Chlorination. (BUET, 62, 72).

28. Write a brief note on "the treatment of swimming pool water". (BUET, 72)

29. Write short notes on ;

(i) Taste and odour control, (ii) Iron and manganese removed, (iii) Water softening, (iv) Algae control, (v) Negative head in filters, (vi) Double filtration, (vii) Chlorine Demand.

9
DISTRIBUTION SYSTEM

9.1 Definition :

The distribution system is that part of the water works which receives the water from the pumping station or from conduits by the gravity flow and delivers it throughout the district to be served.

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

9.2. General Requirements :

A distribution system should satisfy the following general requirements :

(a) The distribution system should, be such as to furnish water in adequate quantities and pressures to all parts of the district served. This would include all demands such as water supply for domestic and industrial use and for fire-fighting purposes.

(b) The distribution system should be thoroughly reliable. This would involve (i) interconnecting all the water mains and controlling flow through shuice valves located at suitable points, so as to ensure an uninterrupted supply of water to all other sections when one of the sections has to be cut out of service following breakdown and consequent repair; (ii) protecting the following supply mains, valves and other appartenances. The main would normally be required to be laid with a sufficient cover of about 3 ft under roads and streets, so that thay arc not open to any damage because of any hazard of passing traffic. At such points as stream crossings, rail-roads etc., the pipes need to be amply protected by carrying: them on concrete trestles or similar supports.

(c) The distribution system should be economical in its design, lay-out and construction; this being considered to be the costliest part of the water supply scheme. It is estimated that the ratio of the cost of the distribution system to the ovarall cost in case of a large scheme is 50 to 75 per cent and for a small scheme as much as 90 per cent.

9.3 Classification:

A distribution system is classified depending upon the method of distribution involved as (1) Gravity system, (2) System with direct pumping and (3) System with pumping and storage.

Gravity System : A gravity svstem is adopted where the source of supply such as a lake or an impounding reservoir, is at a sufficient elevation with respect to the city in order to produce adequate pressures for fire and domestic service. This method, evidently, is the safest and most reliable.

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

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

9.4 Methods of Supply :

Water may be supplied to the consumer either intermittently i.e., for a few fixed hours of the day say 5 A.M. to 11 A.M. and 3 P.M. to 9 P.M., or continuously i.e., for all the twenty-four hours of the day. A continuous method of supply is always better than the intermittent method because of the following reasons.

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

(2) The unused water of storage tanks is most likely to be thrown out to be replaced, during the supply hours, by fresh supply of water. Evidently, this is a wasteful use of water. Also, where the supply is not metered, there is a tendency on the part of consumers to leave the taps open for all hours, resulting in additional wastage of water. The receptacles so left under public hydrants and faucets may remain overflowing, witout being attended to, for a long time.

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

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

9.5 Pressure Requirements in Water Distribution System :

In designing water-distribution systems pressure requirements for ordinary use, and for fire fighting must be considered. In residential districts fire pressures of 60 psi at the hydrant are recommended. In commercial districts minimum pressure of 75 psi is tolerable, but higher pressures must be provided in districts with tall buildings. The American Water Works Association recommends a normal static pressures of 60 to 75 psi is throughout a system. Many cities use fire-department motor pumpers to develop the necessary fire pressure so that normal operating pressure can be less than that quoted above. The maintenance of high pressure in mains means increased pumping costs and usually also increased leakage. Some large cities have installed dual systems in business districts, a low-pressure system for ordinary use and a high-pressure system (150 to 300 psi) for fire fighting only. Other cities use standby pumps to raise the pressure in the entire system whenever a fire occurs.

Faucet pressures of 5 psi are satisfactory for most domestic needs. Assuming a maximum pressure loss of 5 psi in the meter the main about 5 ft below ground level, a total pressure of about 35 psi in the main is adequate for residential districts with one and two storey houses. Allowing about 5 psi for additional storeys, a pressure of 75 psi should be satisfactory for buildings upto 10 storey in height. Many cities require owners of tall buildings to install booster pumps in order to avoid the need for very high pressures in the mains.

9.6 Distribution System Components:

Pipes, gates and hydrants are the basic elements of distribution systems. Their dimensioning and spacing rest upon experience normally precise enough in its minimum standards to permit roughing in all but tbe main 'arteries and feeders. Common standards include the following :

Pipes

Smallest pipes in grid iron	6 inch
Smallest branching pipes (dead ends)	8 inch
Largest spacing of 6 in, grid (8 inch pipe used beyond this value)	600 ft
Smallest pipes in high-value district	8 inch
Smallest pipes on principal streets in central district	12 inch
Largest spacing of supply mains for feeders	2,000 ft

Gates

Largest spacing on long branches	800 ft
Largest spacing in high-value district	500 ft

Hydrants

Areas protected by hydrants	50,000-70,000 sqft each

Largest spacing when fire flow exceeds 5,000 gpm 200ft
Largest spacing when fire flow is low as 1,000 gpm 300 ft

9.7 Distribution Reservoirs (TANKS) :

Distribution reservoirs are used to provide storage to meet fluctuations in use, to provide fire storage, and to stabilize pressure in the distribution system.

The reservoir should be located as close to the center of use as possible. The water level in the 'reservoir must be high enough to permit gravity flow at staisfactory pressures to the system which it serves. In large cities several distribution reservoirs may be located at strategic points throughout the city. Water is usually pumped into a distribution reservoir when the demand is low and withdrawn by gravity flow during periods of high demand. The required capacity of a distribution reservoir is established by the use of characteristics of the district which it serves.

Types of Reservoirs : The storage reservoirs are commonly built up in four different types: (1) R.C.C. tank on R.C.C. staging *(2)* Steel tank on brick ower, (3) steel tank on steel staging, (4) Prestressed steel tanks on steel staging.

The R.C.C. tanks are commonly used in our country. The reservoirs may be of various shapes like square, rectangular, cylindrical, cylindrical with conical base (Inze type) lindrical with hemispherical base, etc. Cylindrical type is most econmical.

Accessories: The following are the main accessories of an overhead reservoir: (1) Inlet and outlet pipes filled with bell-mouth. (2) Overflow and wash-out pipes combined with valve control. (3) Ladder and manhole for cleaning and inspection. (4) Water level indicator. (5) Ventilators and lightning arresters.

A. Typical Intze type R.C.C. cylindrical overhead water tank is shown in Fig. 9.1,

Most Economic Dimensions of a Cylindrical Water lank

Dimensions of tank may be determined by applying the simple principle of calculus, Maxima and minima. Since the tank is cylindrical, minimum surface area that will be required to contain a constant volume of water by the condition of maxima and minima.

(See Fig. 9.2)

Assuming floor thickness is equal to that of the wall.

$A = A_1 + A_2$, where A_1 = Area of the base

A_2 = Area of the shell-surface

A = Total Area

$A = \pi r^2 + 2\pi rh$

V = Volume of the tank

$= \pi r^2 h$

$\therefore h = \dfrac{v}{\pi r^2}$

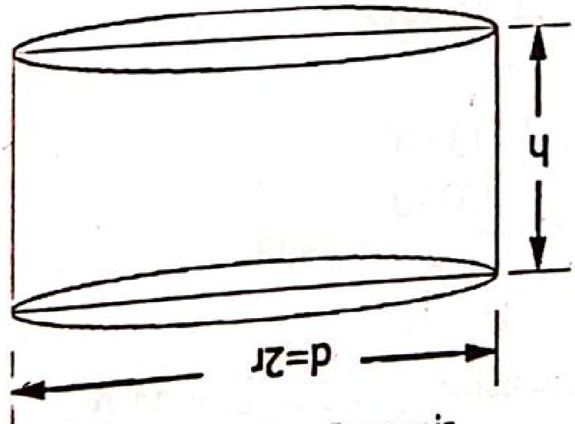


Fig. 9.2 Water Reservoir

$$A = \pi r^2 + 2\pi r \frac{v}{\pi r^2}$$

$$= \pi r^2 + \frac{2v}{r}$$

Differentiating with respect to r

$$\frac{dA}{dr} = 2\pi r - \frac{2V}{r^2}$$

From the condition of Maxima and Minima : $\frac{dA}{dr} = 0$

$$\therefore\ 2\pi r = \frac{2V}{2} = O$$

$$\text{or, } 2\pi r = \frac{2V}{r^2}$$

$$\text{or, } 2\pi r = \frac{2\pi r^2 h}{r^2}$$

or, r = h

Hence, for economical dimension of a cylindrical tank radius of the tank must be equal to the height of the tank.

Example : Calculate the economic dimension of a cylindrical: water tank for one lakh gallons.

Solution : capacity of the tank $= \frac{1,00,000}{7.48} = 13,370$ cuft.

$V = \pi r^2 h = 13,370$

$\therefore$ r = h.

$\therefore\ \pi r^3 = 13370$

$$r^3 = \frac{13370}{\pi} = 4258$$

r = 16.5 ft.

Dimensions : Diameter = 33 ft.

Height = 16.5 ft.

Example : In constructing an elevated circular R.C.C. tank if the cost per sft of the shell is 1.5 times to that of the floor. what be the most economical dimensions of the tank : Assume thickness of the floor thickness of the shell.

Let C = cost, and A = surface area.

$C \propto A$ constant

$C=K(A)=K[\pi r^2+1.5(2\pi rh)]$

$C=K(\pi r^2+3.\pi rh)$, V=Volume of the tank

$C=K(\pi r^2+3.\frac{v}{\pi r^2})$, $V=\pi r^2 h$

$=K(\pi r^2+\frac{3w}{r^2})$, $h=\frac{v}{\pi r^2}$

Differentiating cost with respect to radius

$$\frac{dc}{dr}=K(2\pi r=\frac{3v}{r^2})$$

for minimum condition:

$$\frac{dc}{dr}=0,\ \text{Hence, } K(2\pi r=\frac{3v}{r^2})=0$$

or, $2\pi r^3=3v$

or, $2\pi r^2=3.\pi r^2 h$

or, $2r=3h$

or, $r=1.5h$

9.8 Pipe Systems : The pipe system comprises of the following four units (See Fig. 9.3) :-

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(i) The supply main, (ii) the sub-mains, (iii) minors distribution and (iv) valves.

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

The sub-mains are the secondary feeders connected to either side may be placed at about 1000 ft apart and should be of sufficient size to discharge domestic supply and fire flow.

The minor distributors or branches make up the grid irons of pipes and supply water to the fire hydrants and service pipes of the residences and other buildings. For fire service, minimum

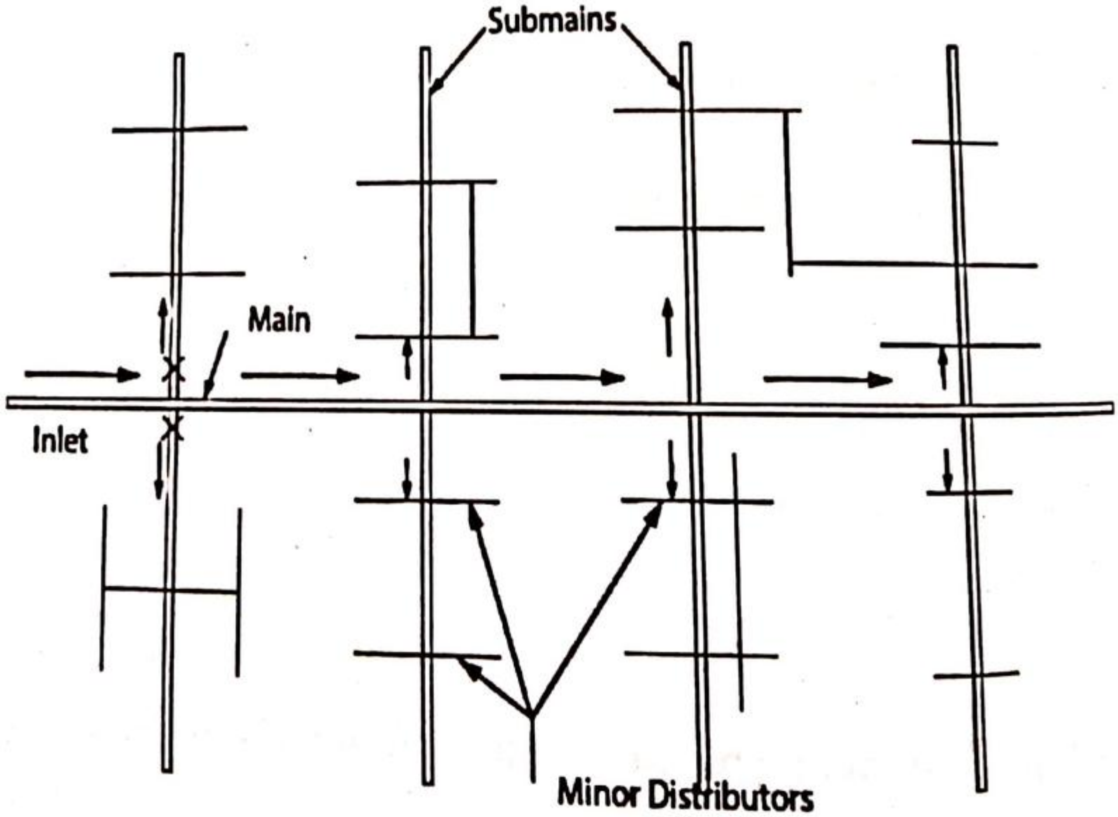

Fig. 9.3 Components of the Pipe System

diameter of pipe should be 6 inch and for domestic service alone 4 inch and less.

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

9.9. Layout of Distribution System:

There are four different systems of distribution depending upon the methods of layout of the pipe-system. These are (1) Dead End (2) Grid Iron System (3) Circle or Ring System, (4) Radial System.

Dead End System comprises of a supply main starting from the service reservoir and laid along the main road, with submains running at right angles to it in both directions and laid along other roads joining the main road. Across the sub-mains run the minor distributors or branches, laid along streets and connecting buildings and houses (Fig. 9.3).

This system is suitable to old towns and 'cities which have been irregularly developed having no definite pattern of roads and streets. Its advantages are (i) its relative cheapness and (ii) easy determinations of discharge and pressure at any point in the system. By suitably locating valves, water supply can be so regulated that by closing any valve say at A, a section of the system can be cut out for repairs without affecting the rest.

Its disadvantages are (i) each pipe has a dead and where the water becomes stagnant and sediment accumulates, requiring the provision of blow-off or drain valves to remove the same. (ii) A large district is to be cut out when repairs have to be made to an important pipe.

Grid Iron System is an improvement over the Dead End System. caused by connecting the ends of the various mains so as to eliminate the dead ends. The water then circulates freely throughout the system. Such a system is very useful for a city laid out on a. rectangular plan (Fig. 9.4); the connections of the dead end producing a grid iron pattern, with mains tunning on main reads in one direction or in perpendicular direction ard submains also running alike on minor roads and streets.

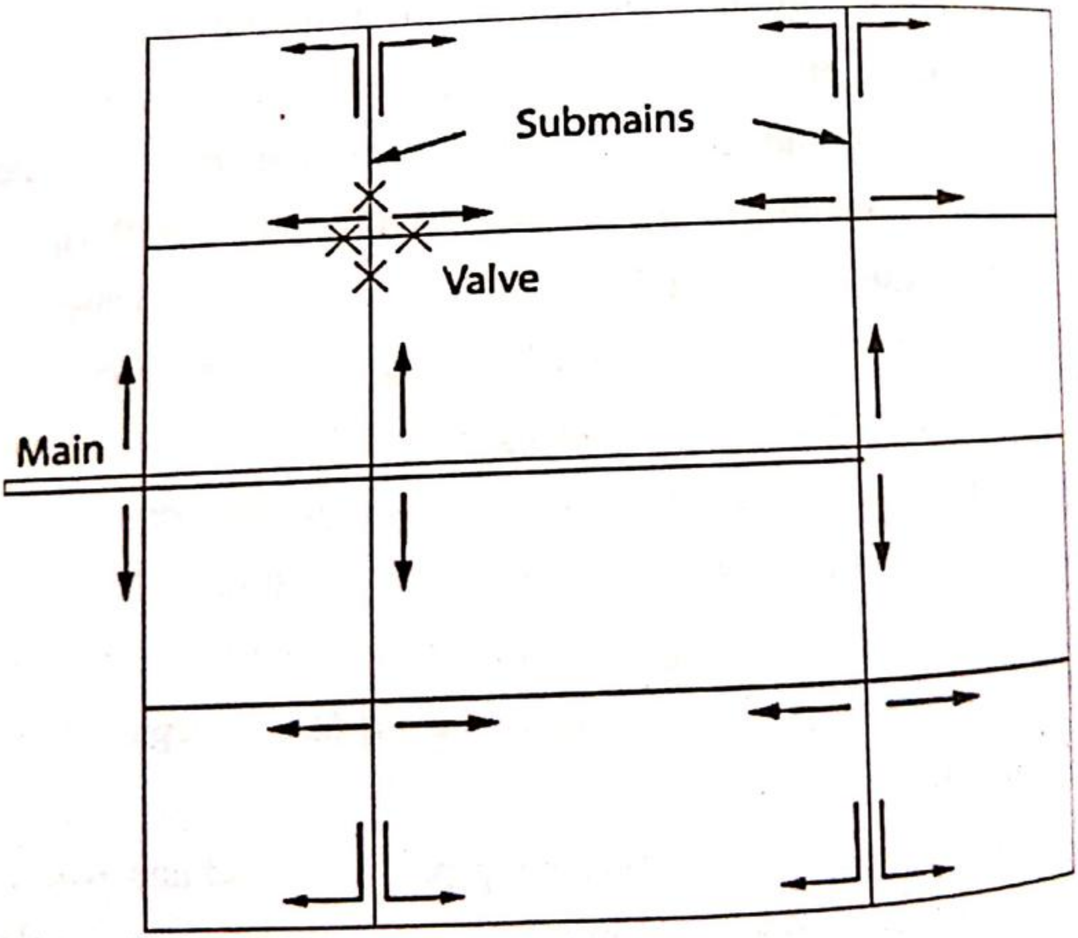

Fig. 9.4 Grid Iron System

Advantages to be gained with this system are (i) avoidance of any stagnation due to water circulating continually, and (ii) absence of the discontinuity of water supply anywhere in the system in the event of any repair-work to a main or submain, water being easily available from another main or submain. Disadvantage is the provision of a very large number of valves. At every junction of two roads, four valves are required (see at B, Fig. 9.4). The system is, therefore, costlier.

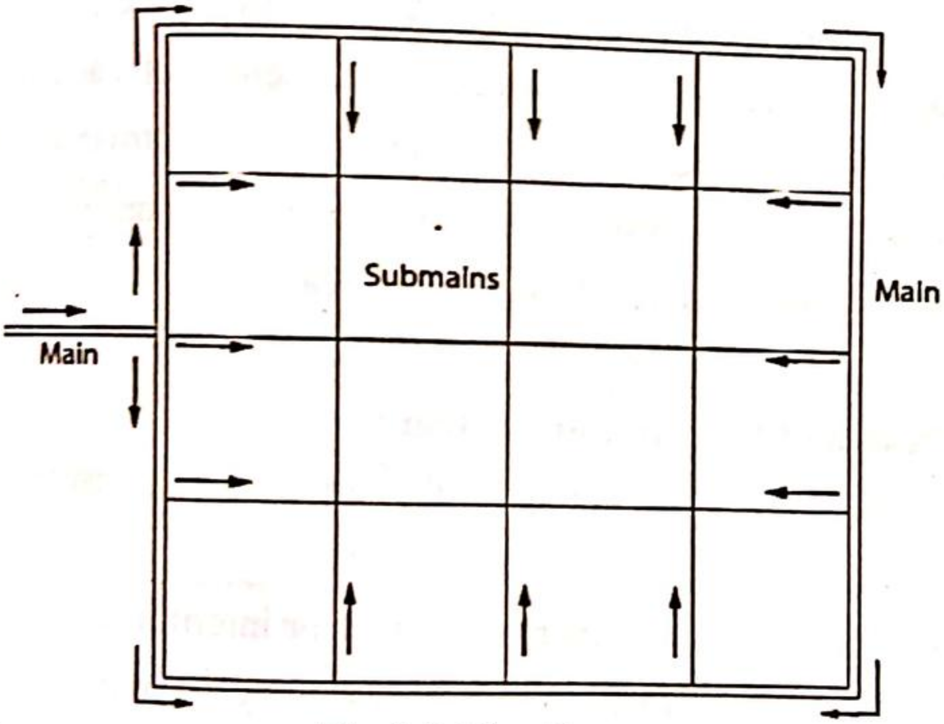

Fig. 9.5 Ring System

Circle or Ring System (Fig. 9.5) : This consists of cutting the entire district into circular or rectangular blocks and then laying the mains all along the peripheral roads with submaim branching out from the mains and running on the inner roads and streets. Thus this system also follows the Grid-Iron pattern with thet difference that the flow pattern is now similar in character to the of the Dead End System. That makes the determination of discharge or size of pipe easier. Also, water can be supplied to any point from at least two directions. This shows that this system possesses the advantages of both of the previous systems.

Radial System (Fig. 9.6) : This system is the reverse instead of from it. The entire district is divided into a number of distribulion zones AEKH, EBKH etc., and a distribution reservoir is placed in the centre of each zone. The supply pipes are laid radially away towards the periphery. This system is most advantageous with the "direct-indirect system" for obvious reasons.

9.10. Design of Distribution System :

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

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

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

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

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

(e) The flow-formulae used in the design have been discussed in Chapter 6.

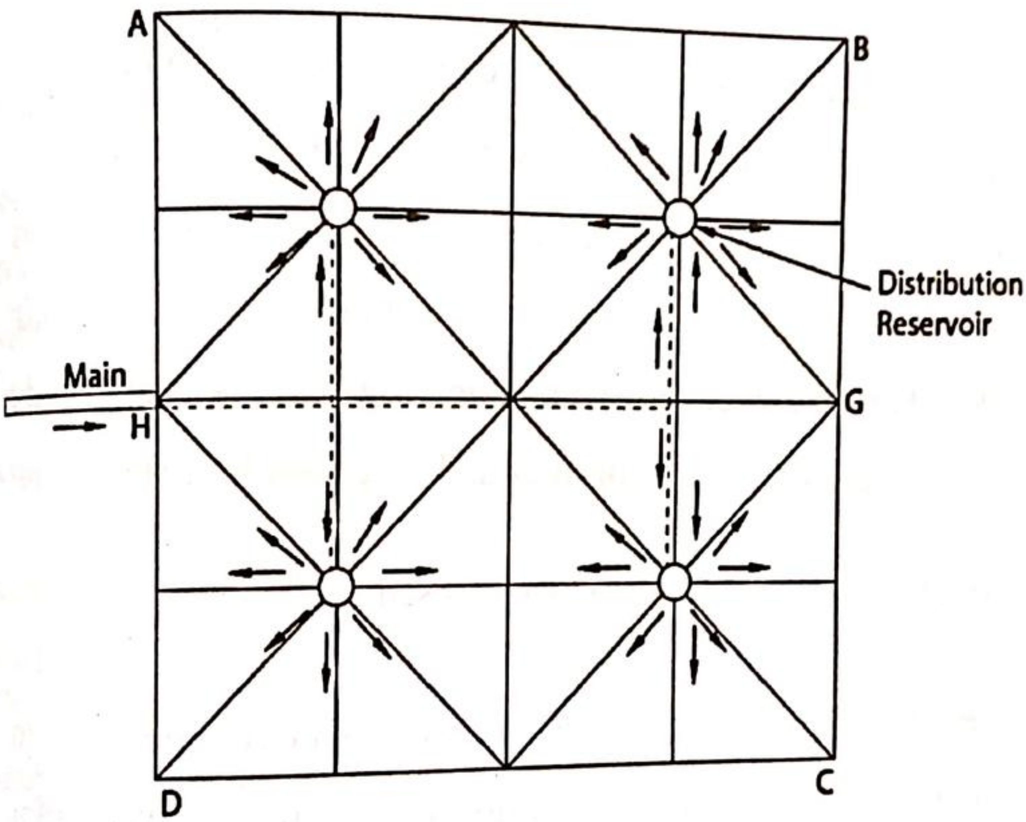

Fig. 9.6 Radial System

The principle involved in the design is to assume the pipe size-and then to work out the terminal pressure heads which could be made available at the end of each pipe section after allowing for the loss of pressure-head in the pipe section when discharging the peak flow. The peak or maximum flow in the pipes is taken as 3 time the average daily flow. Factors causing loss of pressure-head include size of pipe. rate of flow, and friction. Usually losses due to friction the pipes are considered.

The available pressure heads as calculated are checked upto see if they correspond to the permissible residual pressure-heads. If not, the pipe size is changed and the system reinvestigated until satisfactory conditions are obtained.

The design-procedure may now be outlined as below :

(a) Prepare a contoured plan of the city or town, locating: on it the positions of districts or distribution zones with their population, service reservoirs, pumping stations, main roads and streets, existing main-lines and other similar features. A small , scale ($say \frac{1}{10,000}$) may be used

(b) Prepare detailed map of each district, showing in addition to the aforesaid information for the particular district, location of all principal and minor streets. The tentative alignment of all mains, sub-mains and branches as well as position of valves and other appartenances should be marked. Probable population tc be served by each section of pipe-line should be indicated. Choose a bigger scale (say $(say \frac{1}{2.000})$.

(c) Estimate the rate of demand for all purposes including the fire-demand and determine the quantity flowing in each section of the pipe length. This gives the average daily flow in the pipe. The maximum flow will be 3 times of this.

(d) Assume pipe sizes. The velocity of flow varies 3-4 ft/sec.

(e) Find loss of head due to friction in the pipe-length. Use in made of the Hazen-Williams formula because of the ease with which calculation can be made due to the availability of Nomograph (Fig. 6.1). Usual value of C taken is 100. Alternatively the friction-flow formula viz,

$h_f = \frac{4fLv^2}{2gd}$ may also be usen (f = 0.01)

(f) Determine the available terminal pressure-heads. Starting, from the service reservoir of the pumping station where the total pressure-head is known, the pressure-head at the end of any line would be determined by allowing for the fritional loss of head and any rise or fall due to slope of the pipe line and the ground levels.

(g) In case of difference between the availale terminal pressure-head and the permissible pressure-head, revise the assumed pipe-size.

9.11 Analysis of Distribution System:

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

Equivalent Pipe Method : This method is useful in rendering a complex network of pipes into an equivalent pipe system giving the same discharge and loss of head as in the complex system.

For purposes of analysis, the entire network of pipes is considered to be split up into two portions : (i) pipes in series and (ii) pipes in parallel.

Pipes in Series : Pipes carry arbitrarily chosen values of discharge Q_1, flowing through branches AB and BD and Q_2, flowing throuth AC and CD (See Fig. 9.7) It is assumed that the loss of head for pipes in series is additive.

Knowing discharge (say Q_1) and diameters of pipe-lines AB and BD through which it flows, it is possible to determine the loss of head H, in their total length (AB+BD) Here, use is made of the nomograph as discussed in Fig. 6.1. A single length of equivalent pipe AD of

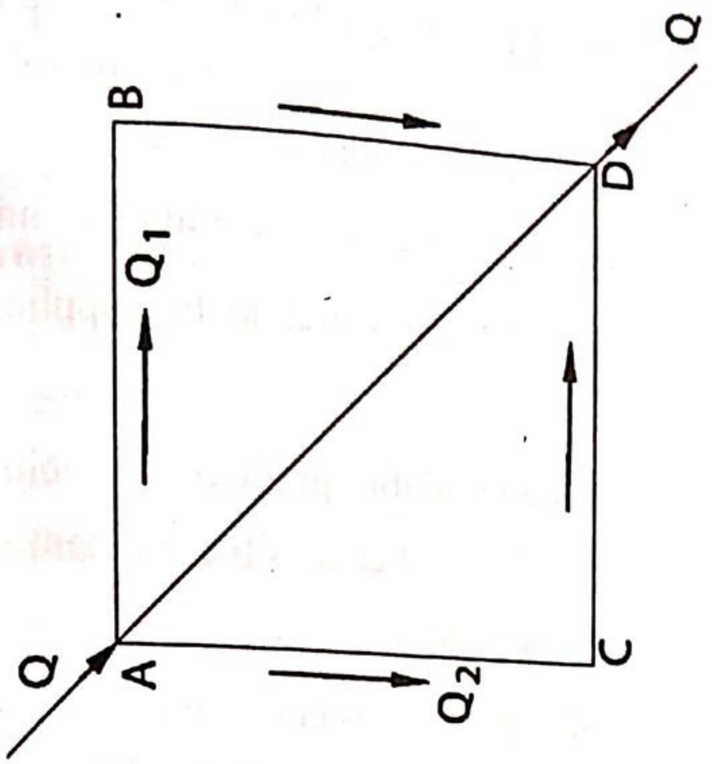

Fig. 9.7 Equivalent Pipe

known diameter can then be selected to give the same values of discharge Q_1 and loss of head H_1.

Pipes in Parallel : In this head through pipes in parallel i.e., ABD and ACD is the same.

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

Method of Sections : This is an approximate method but gives a qucik check and is simple to follow.

The method may be described in the following steps :

(i) Cut the network of pipes by a number of sections aa, bb, cc, at right angles to the assumed direction of flow (see Fig. 9.8). Consider a proper sequence of pipes and character of district served (residential or commercial).

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

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

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

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

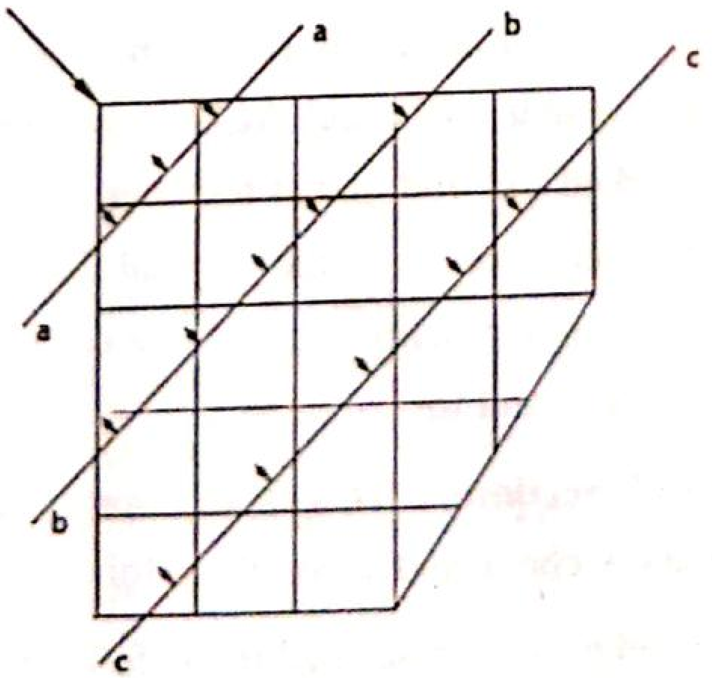

Fig. 9.8 Method of Sections

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

Hary-Cross Method of Analysis of Flow in a Pipe Network : City water distribution systems are composed of inter connected pipes in branches and loops, and the flow to given outlet may come from several pipes. The determination of the probable flow in each pipe of a network requires complicated and tedious trial and error solution.

In any pipe network two conditions must be satisfied ; (1) the flow entering a junction must equal the flow leaving it, and (2) the algebraic sum of the pressure drops (head losses) around any closed loop must be zero. The first condition is a statement of the lew of continuity. The second condition states that there can be no discontinuity in pressure, i.e., the pressure drop (head loss) through any route between two junctions in a loop must be the same

Pipe network problems in water distribution systems are usually solved by methods of successive approximation since any analytical solution requires the use of many simultaneous equations, some of which are nonlinear. It is convenient to express head loss as a function of discharge. i.e,

$H = KQ^x$ (9.1)

in which H is the head loss in the pipe, Q is the discharge, K is the constant depending upon length, diameter and roughness of the pipe as well as the fluid properties, and x is the exponent. On the basis of the Manning equation the Value of x would be 2, while for the Hazen-Williams equation, x=1.85. The Darcy-Wesbach formula gives values of x varying from 1.75 for smooth pipes to 2 for rough pipes. In pipe net-work, the value of x is usually taken as 1.85.

Hardy Cross (USA) developed a method of successive approximations in which the circuits are balanced, distribution of flow is determind and the two conditions of flow are satisfied. The-solutions for pipe network problems suggested by Hardy cros requires that the flow in each pipe be assumed so that the principle of continuity is satisfied at each junction. A correction to the assumed flow is computed successively for each pipe loop in the network until the correction is reduced to an acceptable magnitude. If Q_a is the assumed flow and Q is the true flow in a pipe, then the correction is $Q-Q_a$ and $Q=Q_a+\Delta$ (9.2)

Expressing head loss by Eq. 9. 1 the condition that the head loss around any closed loop be zero gives

$\Sigma K(Q_a+\Delta)^x = 0$

Expanding this Summation

$$\Sigma KQ_a^{x} + \Sigma xK\ \Delta Q_a^{x-1} + \frac{x-1}{2}\Sigma xK\ \Delta^2 Q_a^{x-2} + \ldots = 0 \qquad (9.4)$$

If Δ is small compared to Q the third and all succeeding terms of the expansion may be neglected. Hence,

$$\Sigma KO_a^{x} + \Delta\Sigma xKQ_a^{x-1} = 0 \qquad (9.5)$$

where Δ has been removed from the summation since it is the same for all pipes of the loop. Solving for Δ gives

$$\Delta = -\frac{\Sigma KQ_a^{x}}{\Sigma xKQ_a^{x-1}} \qquad (9.6)$$

$$\text{or } \Delta = -\frac{\Sigma H}{\Sigma xKQ_a} \qquad (9.7)$$

Inflow

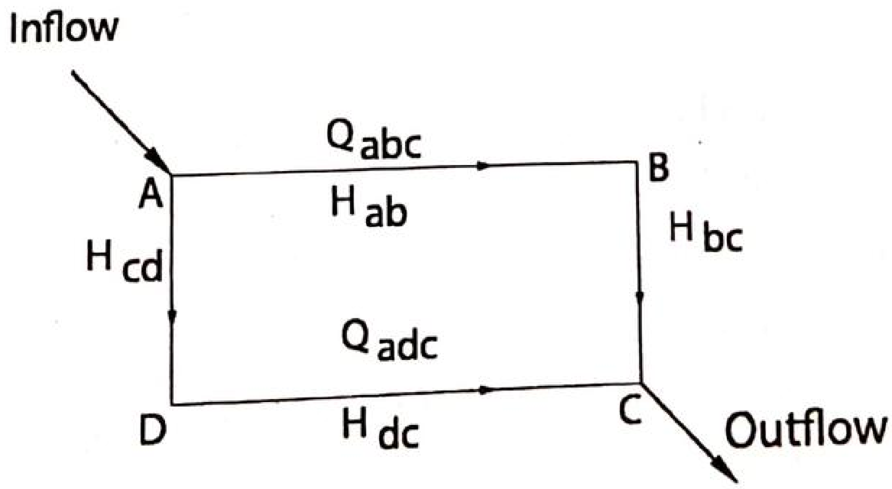

Fig. 9.9

Consider the simple loop shown in Fig. 9.9, the arrow heads showing the assumed direction of flow. Two conditions must be satisfied.

(1) Inflow at ths junction A must equal outflow at the junction

(2) Head losses due to flow in the clockwise direction (in pipes ab and bc) must equal head losses in the counterclockwise direction (in pipes ad and dc) at the junction C. The flow correction A can be computed by

$$\Delta = \frac{(H_{ab} + H_{bc}) - (H_{ad} + H_{dc})}{\times[(H_{ab} + H_{bc} / Q_{abc} + (H_{ad} +_{dc}) / Q_{abc})]} \qquad 9.8$$

where Δ = flow correction in gpm or cusec.

Q_{abc} = assumed flow in the clockwise direction in pipes ab and bc in gpm or cusec.

Q_{adc} = assumed flow in the counterclockwise direction in pipes ad and dc in gpm or cusec.

H_{ab} = head loss in the pipe ab in ft.

H_{bc} = head loss in the pipe bc is ft.

H_{ad} = head loss in the pipe ad in ft.

H_{dc} = head loss in the pipe dc in ft

x = exponent =1.85

In practice, the Hardy Cross method is usually applied by the following procedure :

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

(b) In each loop determine the head loss, Hand H/Q for each pipe.

(c) With due attention to sign, compute the total head loss around each circuit.

(d) Compute, without regard to sign, for the same circuit the sum : ΣH/Q.

(e) Δ, the correction, is computed for each loop by the Eq.. 9.7. The minus sign can be disregarded since the correction so obtained is made by inspection.

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

(g) With adjusted flows, repeat the procedure for the second approximation. The procedure is continued untile he desired accuracy is attained.

Electrical Network Method : An electrical analogy is sometimes used to solve complex pipe-network problems. Ohm's law is not applicable to turbulent flow in pipes since it makes dischage (current) a function of the first power of the potential gradient. Special tubes, called fluistors, have been developed which cause voltage to vary with the 1.85 power of the current. With these tubes in the circuit, the pressure (voltage) and flow (current) distribution can be determined by measuring voltage and current at the desired junctions.

Computer Method : Another method of solving a pipe-network problem is to use a computer. Programming takes time and care, but once set up, there is great flexibility. The effect of changing a pipe size, for example, can be readily determined by simply replacing the old data card for that pipe with a new one. The speed of the computer and the accuracy that can be obtained are the outstanding advantages of this approach.

Example: Given Pipe network shown in Fig. 9.10 with inflow of 3500 gpm at the junction A and outflows as follows : junction C-700 gpm, junction E-2,100 gpm, and junction K-700 gpm. Use Williams and Hazen formula, C= 120 and determine the rate of flow in each pipe when the pipe network is porperly balanced.

SOLUTION

First Trail denoted by (1)

Loop abed

Pipe	H	H/Q
ab	10.0	0.0050
be	6.0	0.0067
	16.0	
ad	15.0	0.0100
de	3.7	0.0051
	18.7	0.0270

$$\Delta Q = \frac{16.0-18.7}{1.85\times0.0270} = -54.0$$

Loop bcfe

Pipe	H	H/Q
bc	3.5	0.0032
cf	3.8	0.0051
fe	0.3	0.0030
	7.6	
be	6.0	0.0067
	6.0	0.0224

$$\Delta Q = \frac{7.6-6.0}{1.85\times0.0224} = +39.0$$

Loop dehg

Pipe	H	H/Q
de	3.7	0.0053
	3.7	
dg	4.6	0.0058
gh	4.6	0.0058
he	1.3	0.0071
	10.5	0.0232

$$\Delta Q = \frac{3.7-10.5}{1.85\times0.0232} = -182.0$$

Loop efkh

Pipe	H	H/Q
fk	2.3	0.0077
he	1.3	0.0033
	3.5	
fe	0.3	0.0030
hk	3.9	0.0098
	4.2	0.0238

$$\Delta Q = \frac{3.5-4.2}{[illegible]} = -14.0$$

SECOND TRAIL Denoted by (2)

Loop abed

Pipe	H	H/Q
ab	11.2	0.0055
be	7.0	0.0071
	18.2	
ad	14.0	0.009
de	5.0	0.0060
	19.0	0.0283

$$\Delta Q = \frac{18.2-19.0}{1.85\times0.0283} = -15.0$$

Loop bcfe

Pipe	H	H/Q
bc	3.3	0.0031
cf	3.3	0.0092
fe	0.1	0.0021
	6.7	
be	7.0	0.0071
	7.0	0.0215

$$\Delta Q = \frac{6.7-7.0}{1.85\times0.0215} = -8.0$$

Loop dehg

Pipe	H	H/Q
de	5.0	0.0060
	5.0	
dg	3.0	0.0049
gh	3.0	0.0049
he	0.50	0.0022
	6.5	0.0180

$$\Delta Q = \frac{5.0-6.5}{1.85\times0.0180} = -45.0$$

Loop efkh

Pipe	H	H/Q
fk	2.6	0.0083
he	0.50	0.0022
	3.10	
fe	0.10	0.0021
hk	1.50	0.0091
	1.60	0.0217

$$\Delta Q = \frac{3.10-1.60}{1.85\times[illegible]} = -12.0$$

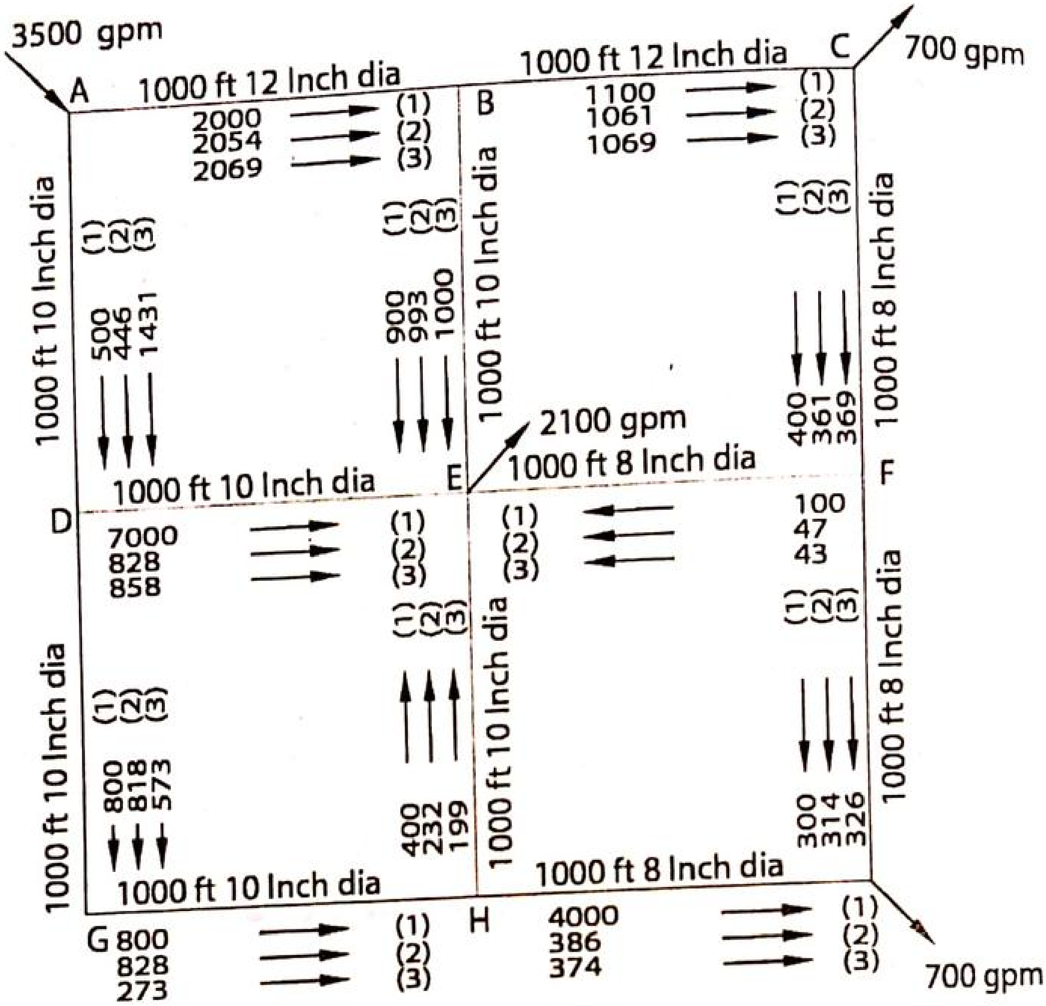

Fig. 9.10

Third trial denoted by (3): Detailed calculations have not been shown but the results have been shown on the Fig. 9.10.

After third trial it is seen that the rate of flow in various pipes of ihe network has been approximately balanced.

9.12 Appurtenances in the Distribution System :

The various appurtenances commonly used in a distribution system are:-

(1) Sluice valves or shut-off valves. (2) Check valves or non return valves. (3) Air valves. (4) Drain valves, or Scour valves. (5) Hydrants and (6) Meters. They are described in brief as follows

Sluice Valves : These are used to control the flow of water through pipe-lines and are freequently used in mains and submains to isolate certain sections enabling repairs to be carried out therein without affecting supplies in the rest of the sections. These may be placed at intervals of about 500 800 ft. and invariably provided af every point of intersection.

A vertical section through a screw-down sluice valve, commonly used is shown in Fig. 9.11. It consists of a wedge-shaped gate which is raised or lowered on grooves with gunmetal faces through a spindle by turning a hand wheel or by turning the cast iron cap with a wrench, thereby opening or closing the passage of water through the pipe on which it is fixed. The direction of rotation for closing the valve is usually clockwise.

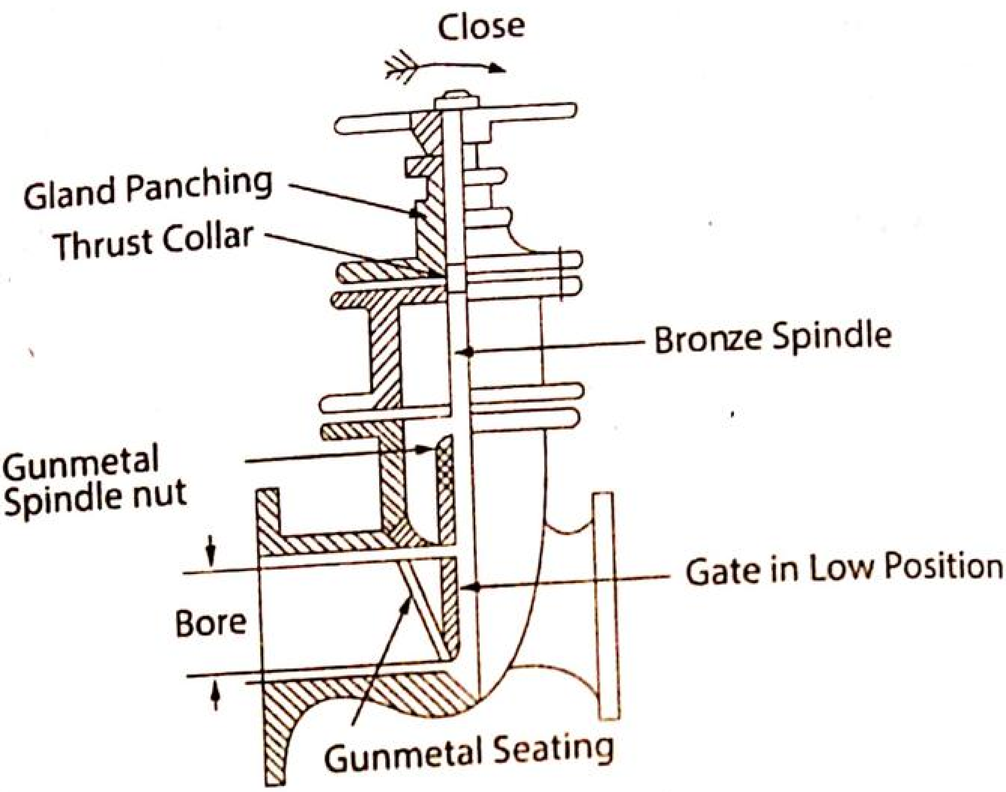

Fig. 9.11 Scrow-down Sluice Valve

Stop Cock: Stop cock is another type of screw down valve only in smaller sizes in the case of a bib tap. The body of the valve is so cast that the water must pass through an orifice which is normally arranged in the horizontal plane. A plug or a 'jumper' can then be forced down on to this orifice by a screwed handle, shutting off the water flow as shown in Fig 9.12. Stop cocks extensively used in case of service pipes for sizes up to 2 inch.

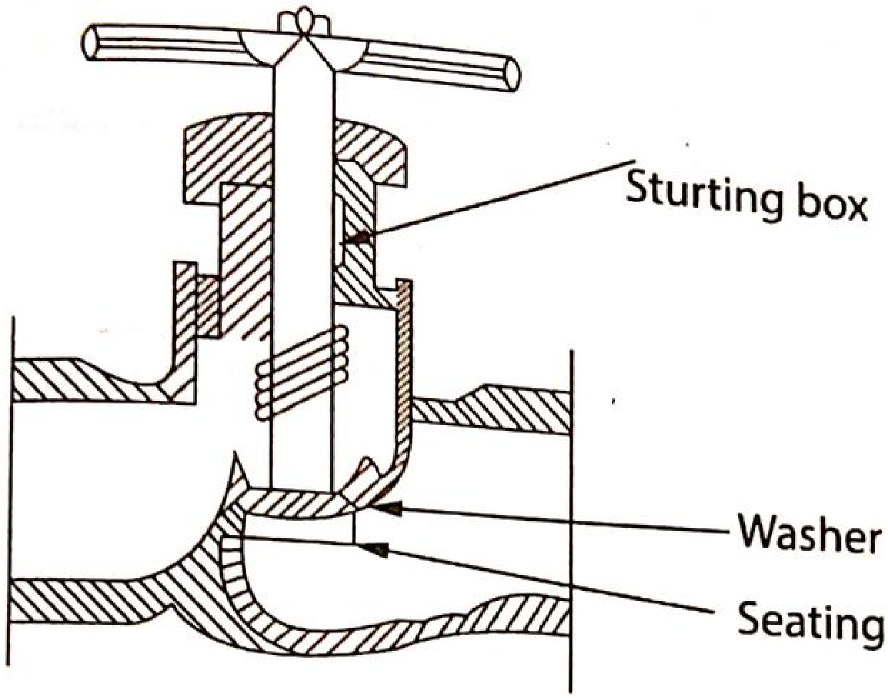

Fig. 9.12 Screw-down Stop Cock

Check Valves : Also called reflex valve or nonreturn valve, as valve is an automatic type of valve which allows the water to flow through in one direction and prevents it from flowing back.

It consists of a flat disc or door within the pipe line, pivoted so that it is forced open when flow of water is in one direction, and forced shut against a gunmetal seating when flow tries to be in the reserve direction (see Fig. 9.13). It will be observed that the seating is normally arranged slightly out of perpendicular. This enables the disc to close automatically by gravity when there is no flow, with the valve fixed in a horizontal pipe.

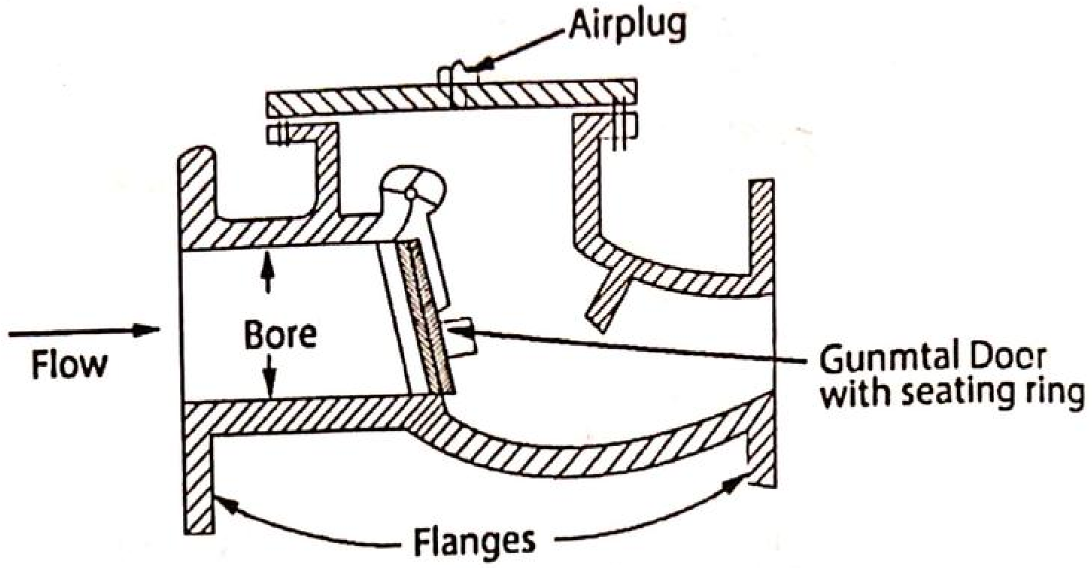

Fig. 9.13 Check Valve

It is necessary for the check to be placed in every pipe line through which water is forced by a pump, so that if the pump stops, water may not run back to the pump, start it in a reverse directions, and thereby damage motor connections of the pump.

Air Valves : The water flowing through a pipe always contains some air which tends to accumulate at high points in the pipe. A pipe is seldom laid parallel to the hydraulic gradient and has invariably some fall or rise in it. As such, air will accumulate at those points in the pipe-line where a rise is suddenly followed by a fall or at every change of gradient as when a very steep rise is followed by a lesser steep me of the pipe-line. Air-accumulation results in a serious blockade to the flow that may materially diminish the area of the pipe available flow and thus reduce the effective flow through the pipe.

It is, therefore, usual for pipe lines to be provided with valves having apertures or orifices, for releasing the accumulated air. Depending on the size of the orifice, an air valve may be called a large-orifice air valve designed to release large quantities of air when a pipe line is being filled with water or a small-orifice air valve for releasing small quantities of air at summit points during continuous operation.

An air valve essentially consists of a cast iron chamber with a floating ball of special composition and a circular orifice in the

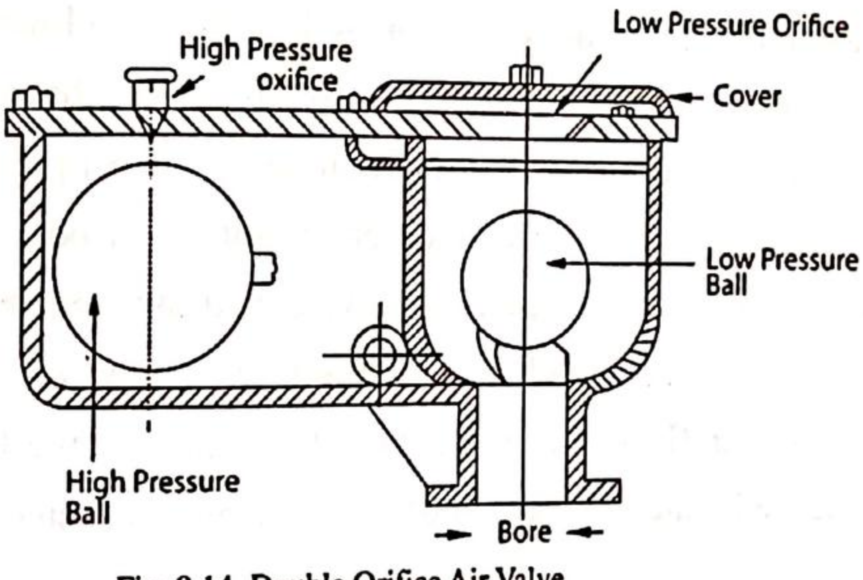

Fig. 9.14 Double Orifice Air Valve

top portion of the chamber. The chamber is connected through a short pipe-length into an opening in the soft of a pipe. Air from the pipe accumulates on the top of the ball floating on the water surface in the chamber and is released by the circular orifice but only to the extent that no water escapes along with the air. This is ensured by the ball rising and closing the orifice, the moment water starts being released along with air.

A double orifice air-valve (See Fig. 9.14) is made up of two cast iron chambers interconnected and provided with separate balls of different compositions and orifices designed to operate for the different conditions of high pressure and low pressure of flow in the pipe line.

Drain Valves, also termed blow-offs are provided at all dead ends in the distribution system as well as at all depressions in the mains. These are valves that are opened occasionally to remove silt or sediment by allowing the water to run out until it is clear. A wash out i.e., branch pipe takes off from the invert of the main at its lowest point leads the discharge to some convenient point of disposal viz. a ditch or soak way. Alternatively, a hydrant-branch i.e., a branch pipe connected to a hydrant, may also be used to wash out or flush the main. For this purpose, the hydrant should be located at the end of the distribution system.

9.13. Waste Detection and Prevention :

Detection of the wastage of water, wilful or otherwise, is an important function of the water-works management. It is esitmated that the wastage of water in an unmetered supply may be as much as 40 per cent of the entire water consumption. This colossal waste represents a severe drain on the resources of the local authority managing the waterworks and has, therefore, to be thoroughly investigated and prevented.

The means of detecting waste or 'unaccounted-for' water may be classified as (1) Waste water surveys, (2) Location of leaks and (3) Waste detecting meters.

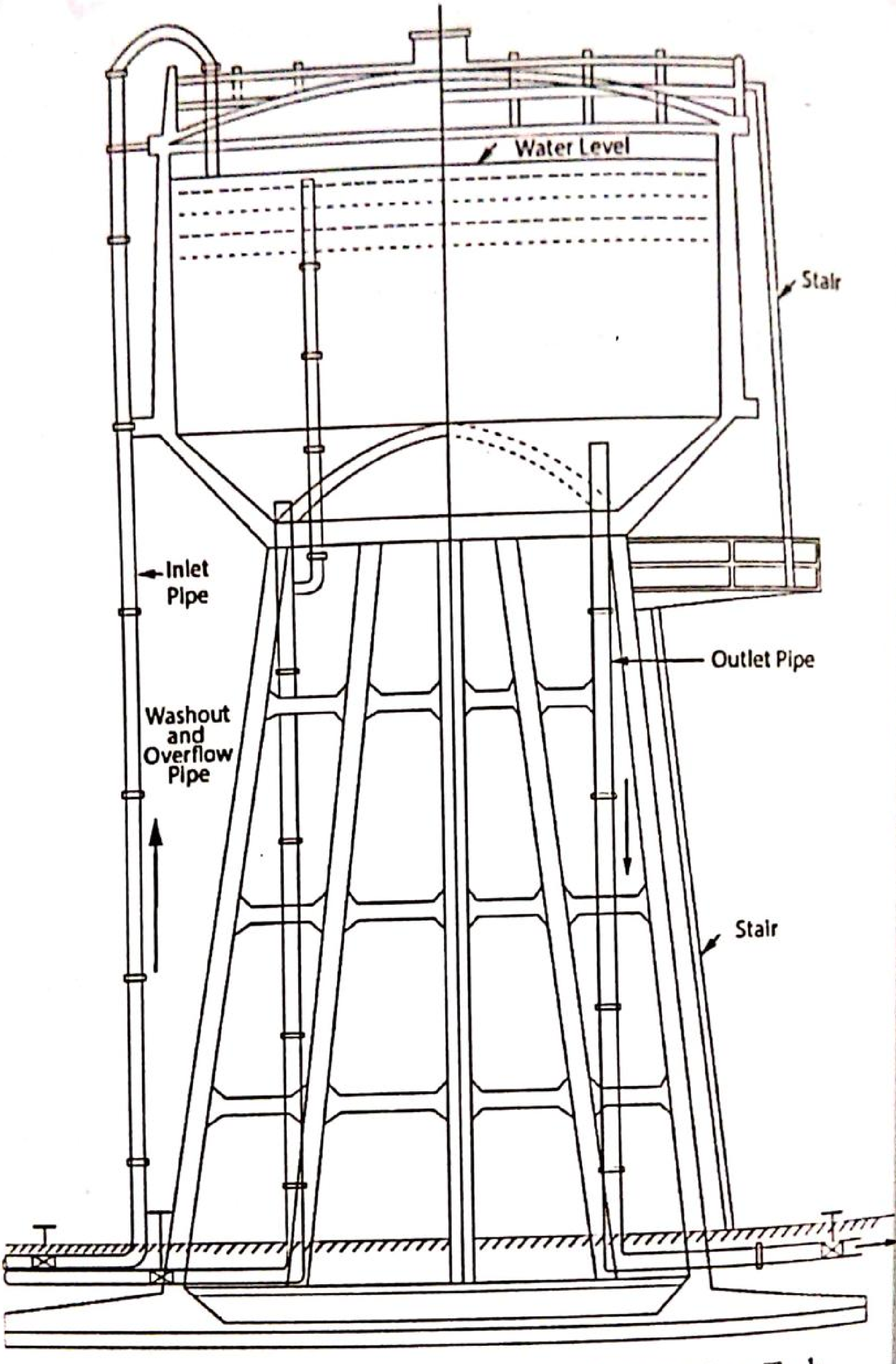

Fig. 9.2 Intze Type Overhead Cylindrical R.C.C Water Tank

QUESTIONS

1. Discuss various methods of distributing water and discuss the advantages and disavantages of each. (BUET, 63)
2. What are the methods available for supplying water to the consumers ? Which one do you think to be preferable and why 1 (AMIE, 64);
3. Describe the various lay-outs of distribution net-work in a water supply system and state their advantages and disadvantages. (AMIE, 66)
4. (a) What points are to be kept in view in the design of distribution system ?

   (b) What are the different methods of analysing a given distribution system ? Explain any one in detail. (BUET, 67)
5. (a) What is a service reservoir ? Give its importance in a distribution system. Draw a neat sketch of an elevated tank (Inize type) and show on it all of its component parts and appurtenances.

   (b) How is the capacity of a distribution reservoir determined ? (BUET, 63, 68).
6. In a water supply system, it is observed that water could flow from the source by gravity to all parts of the distribution system. Do you think a service reservoir is still necessary. If so, why?

   Uuder what conditions should water be supplied to the distribution system by direct pumping ? (AMIE, 68)

7. Name the various fittings used in a water distribution system. What is the function of a small orifice air valve ? Sketch a double air valve.

8. An elevated water tank of 80,000 gals. capacity has to be designed on a staging of 45 ft. above ground level. What important date you would look for to make an economical design of the same ?

   If concrete work per sft in the floor and wall of shell costs Tk. 12.00 and Tk. 18.00 respectively, what would be the most economical dimensions for the tank shell ? (BUET, 62)

9. Describe with the help of illustrative sketches the procedures adopted for "Flow balancing" in a network of watermains by applying the Hardy-cross method. (AMIE, 68, BUET, 64,69,72)

10. What is the purpose of a overhead water tank and what are the considerations for the most economic design of an overhead water tank ?

    Calculate the most economic dimensions for an overhead water tank ol capacity 0.20 million gallons. (BUET, 67)

11. An elevated water tower of 150,000 gals. Capacity has to be constructed over soil having a load bearing capacity of 1.50 tsf, and over a staging.of 65 ft. above G.L., Calculate .the economical dimensions of the tank shell. (BUET, 69)

12. What are the basic assumptions made in the Hardy-Cross method of computing flow-distribution in a network of pipes ? Give an analysis of the method.

How does, this method of computation compare with that of an Electrical grid system? Justify your views. -(BUET, 71)

10

INDUSTRIAL WATER SUPPLY

10.1 Introduction :

Industry uses large volumes of water in its manufucturing processes and in supporting operations. Indeed, the production of foodstuffs, metals, chemicals, and other basic commodities calls for a tonnage of water that far exceeds the combined tonnage of other raw materials. Finishing operations generally draw appreciable quantities of water fabrication and assembly are satisfied with relatively small amounts. Not much of the water entering industrial works actually becomes part of the manufactured product, and only a small fraction is otherwise consumed or lost by evaporation. The larger fraction is employed nonconsumptively. It becomes spent water and may contain many pollutants.

10.2 Water Requirements by Various Industries :

The water needs of industry are varied as well as large. The water requirement for some selected industries have already been shown in Table 2.4.

10.3 Water Quality Requirements:

Water use varies greatly from plant to plant. Quality required is apt to be fairly uniform in all plants making a specific product, but it will differ materially from industry to industry. In many cases the quality requirements for industrial water supplies are much more exacting than are those lor supplies for municipalities. Furthermore, deviations from these requirements may result in loss of production by the plant, whereas the domestic water consumer will accept, though complainingly, occasional colour taste or odour and other lapses in quality. In terms of finished water characteristics, permissible limits for some industries are as shown in Table 10.1.

Table 10.1 Water Quality for Industrial Processes

(All units in ppm or mg/1)

Industry	Tur-bidity	Col-our	Hard-ness	Alka-linity	Iron and Mng	Total Solids
Food products Baked goods	10	10	20 50	20-40	0.1	100-200
Beer	10	10	25-'60	75-150	02	100-300
Channed goods	10	10	25-75	40-80	0.1	100-200
Confectionery	5	5	25 60	30-50	0.2	100
Ice cream	5	5	25-60	30-50	0.2	100
Laundering	10	5	50-100	50-100	0.2	300
Manufactured Products					0.2	300
Leather	20	10-100	50 135	135	0.4	200
Paper	5	5	50	50	0.1	200
Paper pulp	15-50	10-200	100-180	50	0.1-1.0	200-300
Plastics	2	2		50	0.25	200

Textiles, dyeing	5	5.20	2	40	0.25	100
Textiles, general	5	20	2	40	0.25 U	100

Boiler Feed-Waters : Modern high-pressure boilers must be supplied with feed water of high purity. As the water evaporates the concentration of impurities in the liquid phase rise, dissolved and entrained impurities accumulate, heat transfer deteriorates and boiler tubes are overheated, porous sludges, crystalline scales, and other coatings appear on the boiler metal, rising concentrations of specific soluble impurities cause metal corrosion and embrittlement and foaming produces a wet steam and other carry-over of liquid water. The water in low-pressure boilers may be kept in satisfactory condition by blowdown alone, i.e., by the water supply.

Table: 10.2 Composition of Boiler Feed, water

(maximum limits in mg/l established by the American Boiler Manufactures Association)

Boiler Pressure, psiq	Total Solids	Alkal-inity	Suspende Solids	Silica
200-300	3500	700	300	125
301-450	3000	600	250	90
451-600	2500	500	150	50
601-750	2000	400	100	35

751-900	1500	300	60	20
901-1000	1 250	250	40	8
1001-1500	1000	200	20	2.5
1501-2000	750	150	10	1.0
over 2000	500	100	5	0.5

regular discharge of accumulated impurities. However, as pressures have been increased to as much as 2000 psig, special control measures have bad to be introduced.

The most important boiler precipitates consist of silica iron, sush ended matter, and oil in addition to calcium and magnesium as bicarbonates and sulfates. The principal factors in corrosion are (1) DO, CO_2, low pH, and deficient alkalinity, too, can promote, corrosion. The American Boiler Manufacturers Association has set the upper limits for solids, alkalinity, and silicate reported in Table 10.2. As there shown, quality requirements become more stringent as operating pressures go up. The limits on total dissolved solids are meaningful parameters of purity because all hardness is removed. The limits on silica permit concentrations of 0.02 to 0.03 ppm of silica in the steam.

Boiler waters are kept to specification by one or more of the following techniques: (1) external treatment for the removal of impurities from the raw water, (2) internal treatment for the conditioning of water within the boiler, (3) blowdown for the removal of concentrates and sludges, (4) treatment condensates. Although some impurities escape in the steam. their removal from the system does not contribute much to water quality maintenance. Moreover, steam condensates are usually returned to the system as feed water or make-up water.

Upto pressures of about 300 psig, boilers operate economically with minimal external treatment. Feed waters can vary widely in quality, internal treatment being relied upon to condition the sludge and prevent the accumulation of objectionable deposits. As a rule, boilers can be operated at pressures up to 1500 psig only if heat-transfer rates and thermal efficiencies are kept high. Deposits are minimized by producing uniformly good feed water, i.e., water low in hardness and total solids and free from suspended matter. Internal treatment comprises the addition of chelating agents and substances that control residual hardness and keep metal surfaces clean. For boiler pressures at and above 2000 psig, feed waters must be more or less completely determined and stand in little or no need for internal treatment other than oxygen scavenging and pH adjustment.

Cooling Water : Water is the common coolant of the process industries. Temperatures encountered range from the fractionation of liquid air through the chilling or refrigeration of food products to the quenching of red-heat steel. Water has a high heat capacity, is almost universally available, and is easy to transport and apply. Where the same stream, provides cooling waters to sequent manu-factories, its waters may become too hot for further use. The resulting thermal pollution may also harm fish and other useful aquatic life. Hence some states have placed legal restrictions on this form-of pollution too.

Urban supplies provide much coolirng water. For reasons already states, however, large consumers are quite likely to turn to supplies of their own for cooling waters. Only food industries may continue to bank on the implied warranty and assured purity of municipal sources, few industries isolate their water systems. To obtain reasonable insurance rates, most of them must construct independent fire supplies in any case. To this purpose, they are prepared to pump water from nearby streams, lakes, or tidal estuaries. Highly polluted waters can term as cooling waters without purification, but they must not be cross-connected to clean and safe process and drinking water systems. When costal industries draw cooling waters from estuaries or the sea, the expense of corrosion control is usually offset by the low cost of the water. Although seasonally lower temperatures make groundwaters excellent coolants during the summer, falling water tables in many industrial areas of the world warn against such use. Groundwater recharge offers only a partial solution of this problem. Here and there, waste water has been purchased for use as an industrial collant.

Especially in recirculating systems must cooling water be treated to inhibit corrosion, scale formation, sediment deposition, and microbial growths. Pretreatment provides protection during the start-up of new or newly cleaned systems. Standard chemical processes often serve this purpose, but specific methods of treatment may have to be elaborated.

In cooling towers corrosion may extend over the entire metal surface, appear as localized piling, or be induced galvanically by contact of dissimilar metals. The most important water properties affecting corrosion are oxygen and other dissolved gases, dissolved or suspended solids, pH, velocity, temperature, and microbic growths. The water itself may be treated, its pH adjusted the closed recirculation system deaerate, and chemical inhibitors supplied. Anodtic inhibitors, such as chromates and phosphates protect by suppressing anodic reactions, they can be effective, but too small a dosage may cause localized pitting. Cathodic inhibitors such as bicarbonates and polyphosphates, retard cathodic reactions. General inhibitors soluble oils, amines, quanternary ammonium salts, and other organics, for instance protect metals by covering their surfaces with protective films.

Cooling-system deposits are classified as foulants, sediments, or scales. Dirt, silt, precipitates, microbial masses, general debris, and corrosion products are the common foulants of one through and open-recirculation systems. In closed systems only the corrosion products themselves arc normally significant. Mechanical cleaning by filters and oil traps offers an obvious although only partially effective remedy. Foulants can be dispersed, dissolved, and suspended by the addition of organic reagents. This makes it possible to remove them ultimately in the bleed stream. Scale may be quite troublesome, but it can be held in check by chemical processes similar to those effective inboiler feed water control. Polyphosphates and organic compounds are the common cooling-water reagents. Microbial growths can put a cooling-water system out of commission. The growths are usually simy and stringy, but they occur also as loose floes composed of bacteria, fungi, and algae. In addition to plugging water conduits, the growths may form scales accelerate the corrosion of metals, and attack wood in cooling towers. Routine addition of oxidizing chemicals, such as chlorine and bromine, is the common control measure for all types of cooling systems. For surer kill, toxicants such as chlorophenates, cooper salts, and quaternary amines may be added as well. To improve their effectiveness these substances are often formulated with penetrants, dispersants, and other special-duty substances.

Cooling water should not contain hardness more than 50 ppm.

Irrigation Waters : The quality of irrigation waters is interest in relation (1) to resource developments in which available waters are exploited in parallel for agricultural and municipal purposes, (2) to schemes in which the waters made available for urban use are derived -wholly or in part from the underflow of irrigated fields, and (3) to - wastewater disposal by irrigating agricultural areas either by direct discharge from the drainage system or by diversion of sewage polluted receiving waters. Narrower in its implications yet of broad concern, is the quality of municipal waters applied to the parks, lawns, and gardens of the community. The properties of irrigation waters must be excellently or well-suited to the watering of most plants under most conditions :

Irrigation waters should, not contain more than the limit the following substances: Sodium-5 mg/l, boron 0.5 mg/l chlorides 5.3. mg/l, Sulphates10 mg/l, total salts-700 mg/l, and specific conductivity-1000 micrornhos.

Excessive salinity at root zone level leads in succession to leaf burn, leaf drop, twig dieback, and plant destruction. Salinity effects are largely osmotic. High sodium exchange breaks down soil structures, seals pores, and interferes with drainage, in extreme cases of soil breakdown the pH of the soil tray rise to the level of alkali soils. Although traces of boron are essential for plant growth, high concentrations are injurious. Chlorides are generally more injurious than sulfates, because chlorides are relatively more soluable and more toxic to some plants and because sulfates are precipitated as calcium sulfate.

Swimming Pool Waters : The sanitary quality of water in swimming pools is determined by certain bacteriological, chemical, and physical tests. The, bacterial quality of swimming pool water is indicated by the bacteria count and coliform test. Not more than 15 per cent of the samples covering any considerable period of time shall contain more than 200 bacteria per ml or shall show positive (confirmed) test in any of five 10 ml portions of water at times when the pool is in use. Swimmning pools must be kept clean at all times. Sediment, fungi, algae, and visible dirt should not be permitted to accumulate on the bottom, sidewalls, or surrounding walks.

Because swimming pools are normally disinfected with chlorine, all samples must be dechlorinated at the time of collection. This is done by adding suitable amounts of sodium thiosulphate to the sample bottles. A free chlorine residual of at least 0.4 ppm must be maintained throughout the pool. The alkalinity of the water shall be at least 50 ppm, and a pH greater than 7.2 but less than 8.2 shall be maintained. Pool waters should be highly transparent so as to keep divers from colliding with bathers swimming under water. A black dish 6 inch in diameter placed on a white field at the deepest point of the pool should be clearly visible from the side decks of the pool. To avoid rapid multiplication of saprophytic bacteria., pool temperatures should not be raised above 78° F. Ionized silver is not recommended as a disinfectant.

Because they do not persist in water. ozone and ultraviolet light need to be supplemented by chlorine.

10.4 Treatment of Water for Industries :

The methods used for treatment of municipal waters (see Chapter 8) are also used for the treatment of industrial waters. The choice of a particular method or combination of methods depends entirely upon the type of industry.

QUESTIONS

1. Explain briefly the amount of water required by the following industries : (a) Tannery, (b) Textile, (c) Paper pulp manufacturing (d) Food processing.
2. Supply the water quality standard for the following : (a) Boiler, (b) Irrigation, (c) Cooling, (d) Swimming pool.

11

PLANNING & DESIGN OF A WATER PURIFICATION PLANT

11.1 Introduction:

Planning can be defined as the orderly consideration of a project from the original statement of purpose through the evaluation of alternatives to the final decision on a course of action. It includes all of the work associated with the design of a project except the detailed engineering of the structures. It is the basis for the decision to proceed with (or to abandon) a proposed project and is clearly a most important aspect of the total engineering for the project. Because each water supply project is unique in its physical and economic setting, it is impossible to describe a simple process which will inevitably lead to the best decision. There is no substitute for "engineering judgment" in the selection of the method of approach water supply project planning even though each individual step toward the final decision should be supported by quantitative analysis rather than estimates or judgment.

11.2 Steps in Planning:

The various steps to be followed in planning a water supply system have already been stated in Chapter- 1.

11.3 Design Considerations :

The design of a water purification plant for any particular case in affected by the quality and the quantity of the water that is to be treated, the amount of attention and skill that the operation of the plant will receive, the amount of money available for the construction of the plant, and various other factors such as location and climatic conditions. After the type of plant that seems best for the given conditions has been determined, the plant may be designed in a logical manner, each unit being considered in its proper sequence. However, each step in the design may involve the consideration of several possible shapes of unit or types of apparatus. Circular and rectangular tanks are compared in the preliminary design for both economy and esthetic apperance. Even after the general shape has been determined, it is common practice not to recommend a particular type or piece of apparatus but to design the unit so that any one of two or three mechanical devices, controls, or feeders may be used; The limits of capacity and other desired characteristics are specified, and the manufacturer is left to fit his standard apparatus to the requirements of the engineer. Some engineers, however, show details of particular equipment and specify that the, equipment shown or other equipment of equal merit should be provided.

The most economical size of any unit is determined by considering the cost of the various items of equipment and of construction bearing in mind that costs of construction increase rapidly with the depth below ground of such units as mixing basins or sedimentation basins.

11.4 Design Procedure :

The procedure for determining the size of the various units of a plan of the type shown in Fig. 11.1 is as illustrated in the following example :

A water purification plant is to be designed for treating 500,000 gallons of raw water in 8 hr. of actual operation. Coagulant (Aluminium sulphate) will be applied to the raw water as it enters the plant, and the mixing, which is to be accomplished by mechanical means, will require 30 min. The required sedimentation period is 4 hr with a limiting velocity of not more than 0.4 fpm, and the filtration rate is 2 gpm per sq. ft. of filter surface. Wash water is to be supplied at the rate of 15 gpm persq ft. The filters will be washed only when the loss In head in a filter bed reaches a certain maximum permissible-limit, and this provision will require a storage of about 4 percent of the total water filtered in a day. The clear-water reservoir must be capable of storing a volume of water equal to normal consumption for at least 3 hr.

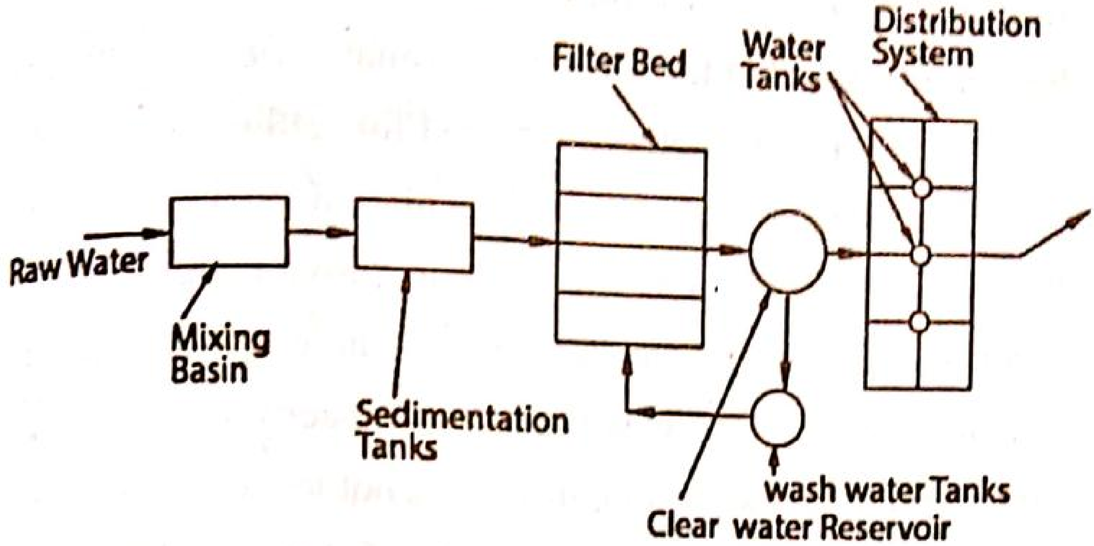

Fig. 11.1 Flow Diagram of the proposed Treatment Plant

Solution :

Mixing Basin : The total flow through the plant is to be 500,000 gal in 8 hr., therefore, the flow per minute will be 500,000/ (8×60)= 1050 gal = 140 cu ft. Since the mixing period is to be 30 min, the capacity of the mixing tank must be $140\times30=4200$ cu ft. In order to prevent sedimentation in, the mixing tank, the velocity of flow should preferably be some what greater than 1 fpm. If this minimum value were used, the length of tank would be 30 ft and the required cross sectional area would be 4200/30, or 140 sq ft, which could be supplied by a section 10 ft deep and 14 ft wide. Because it is desireable to have a velocity somewhat greater than 1 fpm, the width of the tank may be reduced so that the depth and width are equl and each is 10 ft. Then the length of tank will be 4200/(10 × 10) = 42 ft. and the velocity of flow will be 42/30 = 1.4 fpm. which is satisfactory.

Sedimentation Tank : Since the estimated flow is 140 cfm. or 8400 cft. and the detention period for settling is 4 hr.. the required capacity of the sedimentation tanks must be 8400×4 = 33.600 cu ft. In order to provide flexibility of control. there must be at least two sedimentation tanks. in which case each basin will have a capacity of 33.600/2= 16,800 cu ft. Since the rate of flow is not to exceed 0.4 fpm. the maximum permissible length of each tank would be 60 × 4 × 0.4 = 96ft. If a length of 90ft is considered the velocity of flow will be 90/240=0.375 fpm which is satisfactory. Then, the cross-sectional area will be 16,8000/90. or 187 sq. ft. A width of 16 ft and a depth of 12 ft will give a cross-sectional area of 192 sq ft, which is slightly greater than the required area and is satisfactory. The capacity of each tank will then be 90 ×16 × 12 = 17,280 ft.

Filters : Since the flow is 1050 gpm and the rate of filtration is 2 gpm per sq ft of filter area. the required filter area is 1050/2=525 sq ft. Every filter plant should have at least three filtering units. and if three units are provided in this plant. the minimum area of each unit will be 525/3 = 75 sq ft. For. the purpose of design. it will be assumed that a filter unit may be washed, drained. and returned to service in 15 min., which is about 3 per cent of the 8 hr run. Since not more than one filter unit will be washed in any one day. it may be assumed that all three units will be out of service only 3/3=1 per cent of the time, and the surface area ot the filters should be alout one per cent greater than that required for full time operation. Therefore, 175 sq ft represents 99 per cent of the required area of each filter unit, and this area is 175/0.99= 177 sq ft. It each unit is made 18 by 10 ft, its area will be 180 sq ft, which is slightly more than is required.

The ratio of the width of a filter unit to its leangth, as well as the area, of the unit, is influenced by the size of the building in which the fillers to be housed. When there are four or more units, they are generally placed in pairs on either side of the pipe gallery. With three small units, it is usually best to place them side by side. However, if the need for a fourth units in the near future is anticipated it may be preferable to place two units on one side of the pipe gallery and one unit on the other side and to leave a space for the fourth unit with piping in place. In the design under consideration the three units will be placed side by side and the pipe gallery and the operating floor above it will be placed to front of them, in this case the mixing tank is within the building, but in some plants only the chemical feeder and the mixing control units are placed inside the building and the mixing tank is placed outside. The length of the building, inside to inside of brick wall, is equal to the length of the mixing tank, or 42 ft. If the channel at the end of the filter units is assumed to be 18 inch wide and the wall between the filter units and that channel is assumed to be 12 inch thick the length of the filter units is thus increased to 20 ft 5 inch. Then the floor space on the operating floor and also on the pips gallery floor below would be $21\frac{1}{2}$ by 30 ft.

Not more than half of the space on the operating floor will be needed for operating controls and valves. Hence. about 300 sq ft on that floor could be used for the office and other required rooms.

For the purpose of the design. it may be assumed that the vertical distance from the operation floor to the surface of the water in the filter units during operation is 12 inch. from that surface to the top of the wash water gutters is 39 inch from the top of the gutters to the top of the sand is 24 inch. from the top of the sand to the top of the gravel is 30 inch. and from the top of the gravel to the center line of the drains is 15 inch. Also it may be assumed that the half-thickness of the drains. is 6 inch and that the floor of the halp gallery is 24 inch. below the bottom of the filter. If it is assumed that the elevations of the operating floor is 80.00 ths elevations at the various levels are as follows :

Level	Elevation (ft)
Operating floor	80.00
Surface of water	79.00
Top of wash-water gutters	75.75
Top of sand'	73.75
Top of gravel	71.25
Centre line of underdrains	70.00
Bottom of filter underdrains	69.50
Floor of pipe gallery	67.50

The influent pipe from the sedimentation tank should be such size as to give a velocity between 1 and 2 fps with a very small loss in head. The required flow is 1050 gpm. According to the Hazen Williams formula, either a 16 inch pipe or an 18 inch pipe will satisfy these conditions, the velocity being 1.7 fps in the 16 inch pipe and 1.3 fps in the 18 inch pip. The 16 inch pipe will be used.

Each filters requires 1050/3=350 gpm. The smallest pipe that can supply this amount at a velocity of less than 2 fps is a 10 inch pipe, and that size will be used for the short pipes leading to the individual filter units.

The washing of a filter requires 15 gpm per sq ft for surface area, or 15×180=2700 gpm. The wash water should not enter the filter with a velocity greater than 10 fps, and the pressure at the strainers should be about 15 psi, which is equivale to about 35 ft of head. Thus, the wash-water tank, in which the low-water level is customarily 40 ft about the filter, will provide satisfactory pressure if the total friction loss does not exceed about 5 ft. A diameter of 12 inch will satisfy these requirements for pipes from the wash-water tank, but to provide for indeterminate losses and the possibility of future high velocity wash by auxiliary pumping it may be desirable to install 16 inch pipes.

Since the waste-water pipe must remove the wash water at the rate at which the water centers the filter, the diameter of this pipe will be made equal to that of the wash-water pipe, or 16 inch.

The pipe that carries filtered water. from each filter unit also serves to carry the wash water to that tank the size of this pipe has already been establishid as 16 inch. A short distance in front of the filter unit. this pipe is connected to the pipe leading to the clear-water well. The velocity of flow in this line should be between 3 and 4 fps. Each filter produces 350 gpm filtered water. and this may be carried in a 6 inch pipe at a velocity of slightly less than 4 fps.

The quantity of wash water to be pumped is 4 per cent of the total quantity treated. or $0.04\times500.000= 20.000$ gal in 8 hr. This amount of water may be pumped from the clear-water well to the wash-water tank continuously throughout the 8 hour-day at the rate of about 40 gpm. or the pump may be operated at the higher rate for a shorter period of time. For the size of plant under consideration. it may be assumed that a pump capable of handling 100 gpm at maximum efficiency will be most satisfactory.

The loss in head due to friction will while pumping 100 gpm will be about 5 ft per 100 ft pipe for a 3 inch pipe 1.2 per 100 ft. of pipe for a 4 inch pipe. Since there is but little difference between the costs of the two sizes, it will be more economical to use the 4 inch size for the pipe and thus reduce the cost of pumping.

The effluent pipe from the clear water will to the distribution system must carry the tolal flow less the amount that is pumped to the wash-water tank, or $1050-0.04\times1050=$

1010 gpm. For a desired velocity between 3 and 4 fps a 12 inch pipe will be required.

Wash-Water-Tank: The wash-water tank must have a capacity of at east 0.04×500,000= 20,000-gal tank or, preferably, a 25,000-gal tank may be used. Since the low water level of the wash water in the tank is to be 4. ft above the water surface in the filters, the tank will have to be mounted on supports. An elevated tank of standard type will be provided.

Filtering media : The gravel course in the filter will be made up of five layers, each 3 inch. The diameters of the articles in the various layers will be as follows :

Layers	Particle Diameter (inch)
Bottom	$1\frac{1}{2}$-$2\frac{1}{2}$
Second	$\frac{3}{4}$-$1\frac{1}{2}$
Third	$\frac{1}{2}$-$1\frac{1}{2}$
Fourth	$\frac{1}{4}$-$\frac{1}{2}$
Top	$\frac{1}{8}$-$\frac{1}{4}$

The sand bed will consist of 30 inch of sand having an effective size of 0.45 mm and a uniformity coefficient of 1.50.

Wash-Water Troughs : Each filter unit contain two wash-water troughs, which will be parallel to the 18ft dimension of the unit. In order to provide for approximately equal flow, the center lines of the unit. In order to provide for approximately equal flow, the center lines of the troughs will be located 5 ft apart and 2 ft 6 inch from the walls of the unit. Since the rise of tne wash water is to be 2 fpm, the lips of the trough should be 24 inch above the top of the sand bed. The standard size of trough for a flow of 2700/2 = 1350 gpm is 15 inch wide, and the standard slope of the bottom is 1 in 36. However, a trough with a level bottom may be used. With a width of 15 inch, the depth required. If a V-bottom trough is desired, the same cross-sectional area should be provided. **Misccillaneous equipment:** For proper operation of the plant, the following equipments with proper specifications are required : (1) flow control devices (2) chemical feeders, (3) clorinators, etc.

11.5 Design considerations of Swimming Pools

The plan and elevation of a recirculating type of Swimming pool are shown in Fig. 11.2. The following are the various phases of Swimming pool design :

Site Selection and Layout : (1) Accessible to users, space for parking, recreation, picnicking, bath-house and purification equipment. (2) Adequate and satisfactory water supply. (3) Adequate sanitary sewer or separate disposal system. (4) Pool drainge and waste water disposal proper. (5) Site drains, gently sloping, 100 years or more from roads, factories, and wooded areas. (6) Pool area enclosed with high wall or fence.

Size of Pool : (1) Provision for diving, swimming, and non-swimming areas are to be provided. A minimum length of 60 feet and water depth of at least 6 feet at the deep end is suggested for public pools. A depth of 7 feet is required for a 1 to 3 ft diving board, 8 feet for a 3 to 5 foot board, 9 feet for a 5 to 8 foot board and 10 feet for a 8 to 10 foot board.

(2) About 10 square feet of deck space per bather and about 3 feet wide strip around pool are to be provided. Floor drain for each 100 square feet is to be supplied. Deck space is to be supplied.

Source of Fresh Water : (1) Municipal source if available. (2) Existing stream or lake if clean to fill pool. (3) Development of an well if necessary.

Recirculation System : (1) Design to replace water in 6 to 8 hours. For a private pool 12 hours may be acceptable. (2) Provision for inlets on four sides, not more than 20 feet apart or 10 feet from side walls, directional inlets with gate valve or similar control. Design of pool drain as the filtered water inlet and overflow gutters as outlet. (3) Elimination dead areas. (4) Pool drain as permits pool to be emptied in 4 hours or less. No direct connection to sewer. (5) Outlet opening or grating area is at least 4 times drain pipe. Use of multiple outlets if pool is more than 20 feet wide. (6) Design for head loss of 5 to 7 pounds per square inch in sand filter and 30 to 50 pounds in diatomite filter. (7) Selection of correct pump for type of filter and head loss in recirculation piping.

Accessories : (1) Water heater with automatic thermal control for indoor pool and so the outdoor pools. (2) Locate water heater if provided. on water flowing from filters to pool. Use cold water for filter washing. Temperature of water entering pool not greater than 110°F. (3) Hair and lint catcher. Provide spare on bypass. with valves. Area of stiainer openings at least 10 times area of main recirculation line (4) Vacuum cleaner connected to portable pump or recirculating pump. with proper connections in pool sides at least 8 inches below the pool water surface. (5) Space heater and ventilation for indoor pools. (6) Residual chlorine and pH testing kits. Chlorine range 0.3 to 3.0. pH range 7.2 to 8.8 (7) Spare parts for chlorinator. including ammonia bottle and gas mask where gas machine used.

QUESTIONS

1. Discuss briefly the various steps that are adopted for designing a municipal water supply system.
2. Explain step by step the procedure to be followed in designing a water treatment plant.
3. Design a water treatment plant for 3 mgd capacity. The treatment plant should include a plain sedimentation tank, a mixing basin, a sedimentation coagulation tank, a rapid sand filter bed, a filtered water reservoir, a wash water tank and an overhead.cylindrical water tank.

   Assume the standard values of data you need to design the plant.
4. State the design considerations of a recirculating type Swimming pool. Also explain briefly its working principle.

APPENDIX-I

CONVERSION FACTORS

Length

Inch (in.)=2.54 centimetres (cm) = 25.4 millimeters (mm)

Foot (ft)=12 in. = 30.48 cm=0.3048 metre (m)

Yard (yd)=3 ft=36 in. =91.44 cm=0.9144 m

Mile=5280 ft= 1.009 kilometres (km)

Meter (m) = 100 cm= 39.37 in. =3.28 ft

Kilometre (km) = 1000 m= 0.621 mile

Area

Square inch (sq. in.) = 6.45 square centimetres (cm^2)

Square foot (sq. ft) = 144 sq in. = 929 cm^2 = 0.0929 sq.m.

Acre = 43,560 sq. ft=4047 square metre (m^2) = 0.4047 hectare

Square metre (m^2) =10.764 sq. ft

Square mile (sq. mile) = 640 acres=2.59 square kilometres (km^2) = 259 hectares

Weight

Grain = 0.0648 gram (g)

Ounce (oz) = 28.35 g

Pound (lb) = 453.59 g = 0.454 kilogram (kg)=7000 grains.

Gram (g)= 15.4324 grains

Kilogram (kg)= 2.2046 lb. =35,274 oz. = 0.0197 cwt.

Stone = 6.350 kg

Hundredweight (cwt). British =112 lb.=50.802 kg

Ton. British (long ton). 1.016 metric ton = 2,240 pounds (lbs)

Short ton. USA. 0.907 metric ton =2000 pounds (lbs)

Metric ton (g)=1000 kg=0.984 long ton=1.102 short ton=2204.6 lb.

Volume and liquid capacity

Cubic inch (cu.ft)=16.387 cubic centimetres (cc) or mililitres (mL)

Cubic foot (cu. ft)-1728 cu. in. = 28.317 litres (L)

Cubic foot =7.48 US gal. = 6.229 Imp. gal.

Cubic yard (cu. yd) = 27 cu. ft.

Fluid ounce = 29.57 mL or cc

Pint (pt). British = 0.5682 L

Pint. USA = 0.4732L = 16 fluid ounces =28.875 cu. in.

Quart (qt). British = 1.136 1 = 2 pt.

Quart. USA=0.9461 = 2 pt.

Gallon. Imperial (British) /Imp. gal.L=4.546=1.201 US. gal.

Gallon. USA (US gal.)= 3.785 L = 231 cu. in.

Millilitre (mL) = 0:061 cu-in.

Litre (L) = 1000 mL= 1.0567 liquid quarts (US =61.025 cu. in.

Litre = 0.22 Imp. gal.=0.264 US gal. = 0.03531 cu. ft.

Cubic metre (m^2)=10001=35.31 cu. ft =264.2 US gal. = 220 Imp. gal. Acre-foot = 1233 m^3

Pressure

Pound per square inch (psi.) 0.0703 kc/cm^2=0.9678 atmosphere (atm)

Pound per squre inch = 0.70m of water =2.31 ft of water

Atmosphere (atm.) =10.342 m of water= 33.929 ft of water.

Kilogram per square centimetre (kg/cm^2)=14.22 p.s.i. =0.9678 (atm.)

Velocity

Foot per second =0.305 metre per second

Metre second = 3.28 feet per second

Rates of flow

Imp. gal. per minute =0.07572 litre per second

Imp. gal. per minute =0.27261 m^2 per hour

Imp. gal. per minute = 1.201 US gal. per minute

US gal. per minute (gpm) = 0.0625 litre per second

US gal. per minute =0.225 m per hour

Million US gal. per 24 hours (mgd) - 52.6 litres per second

Cubic foot per minute =0.4719 litres per second

Cubic foot per second 0.007851 litre per second 0.544 mgd (Imp)=0.646 mgd (US)

Litre per second = 19.00 Imp. gal. per minute

Litre per second = 15.48 US gal. per minute

Litre per second=2.119 cu. ft per minute

Litre per second =0.0228 mgd (US)

mgd =1.547 cuec (US) = 1.55 cusec (British)

Filtration rate

Million US gal. per acre per day (mgad) =$0.935m^2/m^2$/day

M^2/m^2/day=1.069 mgad (US)

Viscosity

1 ft unit $\left(\frac{lb.sec}{ft^2}\right)$	$= 479$ Poise $\left(\frac{dyne.sec}{cm^2}\right)$
1 Poise	$= \frac{2,083}{1,000}$ $ft./unit.$

Kinematic Viscosity

1 ft. unit (ft. /sec.)	= 929 Stokes (cm^2 /sec.)
1 Stoke	$= \frac{1.076}{1.004}$ $ft./unit.$

Temperature

1^oF	$= \frac{5}{9}(^oF-32)]^oC.$
1^oC	$= \frac{2}{5}(^oC+32)]^oF.$

Heat

1 BTU	= 0.262 k cal.
1 BTU	= .076 m. kg.
1 k cal	= 3.968 BTU
1 m. kg.	$= \frac{9.29}{1,000}$ BTU

Miscellaneous data

Horsepower (h.p.)- 33,000 foot-pounds per minute

Horsepower = 0.746 kilowatt- 1.0139 cheval vapour (CV)

Kilowatt=1.34 h.p.=0.736 kilowatt = 0.986 b.p.

Cheval vapour=0.736 kilowatt=0.986 h.p,

One litre of water weighs one kilogram at 4^oC

One cubic foot of water weighs 62.35 pounds.

One US of water weighs 8.33 pounds.

One ft of water= 3.4332 psi

One psi	= 2.3057ft of water
One ppm	= One mg/L =8.34 lb per million gallon
One cusec	= 646.300 gallons per day
e	= 2.71828
$\log_e$	= 0.43429
$\log_{10}$	= 2.3026

APPENDIX-II

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THE END

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