1 Introduction

1.1 Importance of Water

Five most crucial for existence of human life are air, water, food, heat and light, of which next to air, water is the most important requisite for human life to exit. The availability of a water supply adequate in terms of both quantity and quality is essential to human existence. Water is perhaps the most important nutrient in our diets. In fact, a human adult needs to drink approximately 2 liters (8 glasses) of water every day to replenish the water that is lost from the body through the skin, respiratory tract, and urine. But some water sources cannot safely be used to meet our requirement for drinking water. In fact, 99.7% of the Earth's water supply is not usable by humans. This unusable water includes saltwater, ice, and water vapor in the atmosphere. Only freshwater, which is contained in rivers, lakes, and underground sources, can be used for human consumption. Furthermore, many freshwater sources are not suitable for humans to drink. Early humans could judge water quality only through the physical senses of light, taste, and smell. Its effects on human health and well being may be vital. World health organization has observes that 80% of communicable diseases that are transmitted through water. Many serious diseases, like cholera, gastroenteritis, typhoid, amoebia, diarrhoea, polio, hepatitis (Jaundice), Leptospirosis, Dracontiasis etc. are caused by drinking water that contains parasitic microorganisms. Water containing large amounts of industrial waste or agricultural chemicals (e.g., pesticides) can also be toxic and unfit for drinking. Hence, humans have a great need for a reliable source of clean freshwater for drinking.

In addition to the water needed for drinking, humans use much larger amounts of clean freshwater in other applications. These other uses of freshwater include household use (e.g., culinary and cleaning), industry, agriculture (e.g., irrigation), and recreation. Hence, the quality of the freshwater supply is important for virtually every aspect of our lives.

In response to this need for reliable supplies of clean and usable freshwater, governments at all levels have formed organizations and passed legislation to monitor, treat, and protect our water supplies.

1.2 Definition of types of water

1.2.1 Pure and impure water

Pure water contains two parts of hydrogen and one part of oxygen. Pure water (H_2O) is colorless, tasteless and odorless. Absolutely pure water is not found in nature. Even the rain water which is absolutely pure at the instant it is formed becomes impure because as it passes through the atmosphere it dissolves certain gases, traces of minerals, dusts and various other substances. Hence, pure water normally could be available only in laboratory. It is not actually suitable for drinking purpose due to absence of vital minerals required for the human growth.

Impure water contains substances such as minerals, suspended and dissolved materials etc. In addition to H_2O other minerals like Ca, Mg, fluoride, iodine, iron etc. is impure water. Impure water is used for drinking purposes but substances should be within acceptable limit.

1.2.2 Potable and wholesome water

Water fit for drinking purpose or safe enough to be consumed by humans or used with low risk of immediate or long term harm is termed as potable water. Potable water is suitable for drinking purpose having pleasant taste and useable for domestic purpose.

Water containing the minerals in small quantities at requisite levels and free from harmful impurities is wholesome water. Wholesome water is neither chemically pure nor contain excess minerals harmful to human health but contains useful or beneficial to human health.

Requirements of wholesome water

1. It should be free from microorganisms, radioactive substance, dissolved gas, salts, heavy metals etc.
2. It should be colourless and sparkling
3. It should be tasty, odour free and cool
4. It should be free from objectionable matter
5. It should not carrode pipes
6. It should have dissolved oxygen and free from carbonic acid so that it may remain fresh

1.2.3 Polluted and contaminated water

Water containing excess amount of impurities such as minerals, salts, gasses, microorganisms etc. is called polluted water. It is not fit for drinking purpose. It may contain various types of impurities (Physical, chemical and bacteriological).

Containing something in water as harmful substances such as bacteria, virus, protozoa, worms, heavy metals etc. that cause diseases is termed as contaminated water.

1.3 Historical development of water supply system

Evidence of activity concerned with human health and water supply has been found in civilizations throughout human history. The human search for potable water supplies must have begun in prehistoric times. Thousands of years passed before our more recent ancestors learned to build cities and enjoy the convenience of water piped to the home and drains for water-carried wastes.

The earliest human settlements were developed near the river bank as availability of plentiful water. As civilization progressed, population increased and community expanded, small community transformed into the village, village into town, and town into city. Over the years the enormous demands being placed on water supply due to rapid increment in population. The water sources also gradually become polluted due to discharge of large amount of waste into the sources. The ground water table has ongoing gradual depletion due to inadequate recharge as infra structure developed by people and massive deforestation, drought as of climate change, resulting low flow in surface water. The expansion of the community further from the water source made water collection difficulty and time consuming. Hence a planed scheme with free from pollution potable water to the consumers premises become necessitated. The provision of water making within easy access to consumers is water supply system or water supply scheme.

In Nepal, peoples use to fetching water traditionally from hiti, kuwa, dhunge dhara, pandhero, streams, rivers in hilly areas and dug wells and tube wells in terai. The construction of well and 'Dhunge Dhara' was started in 'Lichhibi kaal', in the 5th to 7th centuries and progress found very much encouraging in 'Malla kaal'.

History of piped water supply system development in Nepal dates back to 1895 A.D., when the first Bir Dhara system (1891-1893) was commissioned. The system also led to establishment of Pani Goshowara Adda and it provided limited private and community standpipes in few selected parts of Kathmandu. The water service were then gradually extended to few other prominent places like Amalekhgunj, Birgunj, Palpa and Jajarkot . The sector received a fair priority in the First Five Year Development Plan, which started in 1956, but the sector activities were placed under the Department of Irrigation for a long while until the Department of Water Supply and Sewerage (DWSS) was formally established in 1972. DWSS is providing a nationwide service through its 5 regional, 48 divisional and 22 Sub-divisional offices spread throughout the country.

The existing national water supply coverage of 37% in 1990 has been expanded to over 80% as of today. The supply of potable water has been challenging for water supply authorities in developing countries like Nepal. Prior to the year 2005, our country Nepal was following the water quality standard of World Health Organization (WHO). Afterwards, Nepal's own water quality standard 'National Drinking Water Quality Standard' (NDWQS) has came into effective.

In Nepal a variety of multi and bilateral organizations, NGOs, INGOs, and Civil Society organizations support and implement water supply schemes and provide services. Major organizations working in water supply sector are; Department of Water Supply and Sewerage (DWSS), Nepal Water Supply Corporation (NWSC), Kathmandu Upatekya Khanepani Limited (KUKL), Rural Water Supply and Sanitation Fund Development Board (RWSSFDB), Water supply and Sanitation User's Committee (WSUC), Department of Local Infracture Development and Agriculture Roads (DoLIDAR).

The interim constitution of Nepal has defined access to water as a fundamental right to its citizens and to support this, the country has set a target to provide all Nepalese with access to basic water supply and sanitation services by 2017 A D.

1.4 Objectives of water supply system

The primary objective of water supply system is to supply water for consumer from the best available source which will ensure water of good physical quality, free from unpleasant taste or odour and containing nothing which might be detrimental to health.

1. To provide safe wholesome water to the consumers in adequate quantity efficiently at low cost.
2. To make water available within easy access to public.
3. To make adequate provisions for emergencies like fire fighting, festivals, meeting etc.

4. To save time in fetching water.
5. To supply water rich in reliability, quality, quantity in effective and efficient way.

1.5 Schematic diagram of typical water supply system

Typical schematic diagram of w/s scheme for urban area is shown in figure 1.1. The components are collection works includes intake and pump, transmission main, purification works, reservoir and distribution systems. Pump may be required in case of lifting due to elevated supply area than the source. Water purification works may also optional as depends upon source water quality.

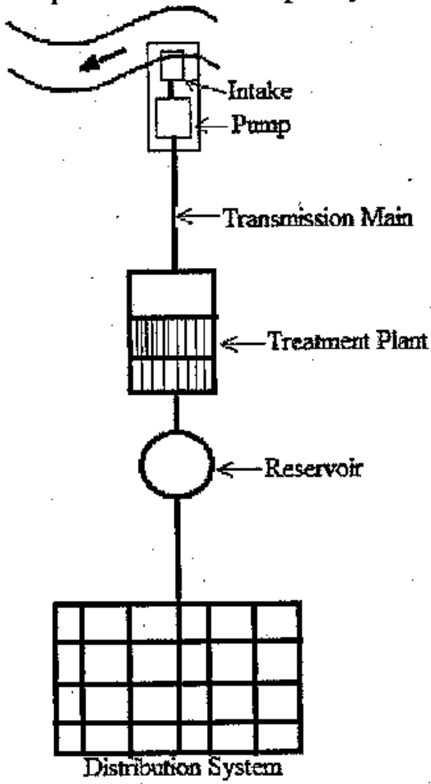
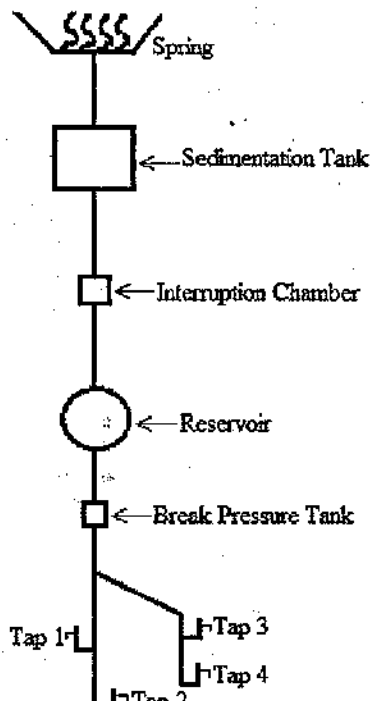


Figure 1-1 Schematic Diagram of a Typical Urban Water Supply System

The population density of Nepal in rural areas is low and houses are scattered but also affordability of consumers for private connection is low so that in rural area water supply service is provided by public taps. Typical schematic diagram of w/s scheme for hilly rural area is shown in figure 1.2. In hilly rural area common source is spring and generally water quality of spring is not necessary to treat. Hence water treatment is not provided in hilly areas also power system is avoided. Sometimes stream may be used in rural areas as source so sedimentation and filtration is used but sedimentation may be omitted or auxiliary. Break pressure tank (BPT) also known as pressure releasing tank is provided in distribution lines to prevent pipes from bursting due to excess pressure as greater elevation difference in rural areas and for the same purpose interruption chamber (IC) is provided in transmission mains.



Distribution System

Figure 1-2 Schematic Diagram of a Typical Rural Water Supply System

In terai either tube well or dug well is used as source for rural water supply system. Dug well may be used by private households or small community where as tube wells are used by big community. The water from the well is lifted to elevated reservoir and provided to consumers through public tap stand. Typical schematic diagram of w/s for terai is shown as in figure 1.3.

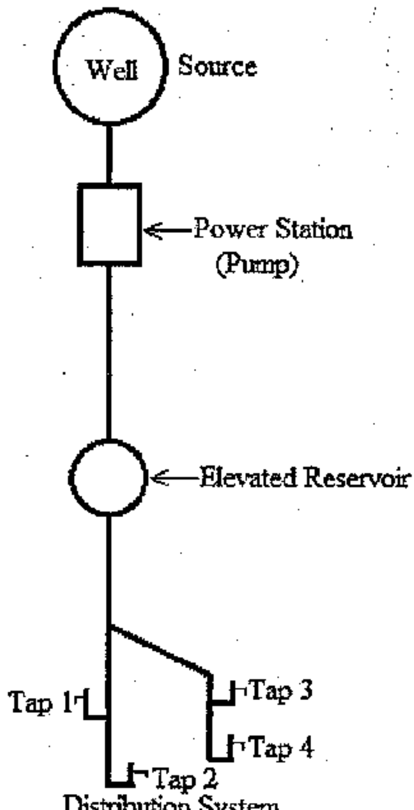


Figure 1-3 Schematic Diagram of a Typical Terai Water Supply System

1.6 Components of water supply system

Water supply system consists of; (i) collection, (ii) transmission, (iii) purification and (iv) distribution work as shown in figure 1-4.

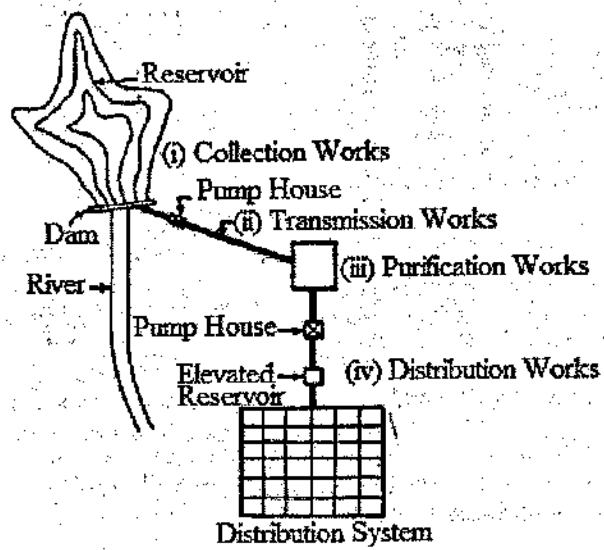


Figure 1-4 Water Supply Systems

The various components having own specific functions can perform a public water supply scheme are as indicated below;

1. Source of water
2. Intake for collecting water
3. Pump
4. Collection chamber
5. Transmission main
6. Interruption chamber
7. Treatment/purification works
8. Reservoir
9. Control valves
10. Distribution system
11. BPT
12. Public stand post

1. **Source of water**

   Primary source of water is precipitation which joins to the earth in the form of rain, snow, hail, etc. Rainfall is the most important source as it occurs is retained in surface depression, carried away as surface runoff in natural channel in the form of stream or river and some portion percolates into the ground further infiltrated to natural ground water reservoir. This portion of precipitation that may be utilize for water supply as source of water is available partly at the ground surface and partly below the ground surface.

2. **Intake for collecting water**

   These are device or a structure constructed at the source of water to draw off water which conveys to treatment plant through a conduit. Various appurtenances such as strainer, operating valve, pump, housing, conduit etc. are provided for operation and maintenance.

3. **Pump**

   Pump is a lifting device commonly required to lift water from source which is operated by the help of energy. Avoiding the use of this device save the operation and maintenance cost of scheme but is essential when area to be served is located at a higher elevation than the source

4. **Collection chamber**

   When demand of water do not met by a single source it may be required. Additions to collection of water from two or more sources this prevent the backflow of water one source to another.

5. **Transmission mains**

   Water conveyance from the source to treatment plant is entertained by pipe as known as transmission main. Water from these pipes is not given to consumers though generally pipe lay over the ground. Design of transmission pipe is considered for average flow.

6. **Interruption chamber**

These are the chamber provided in the transmission lines to prevent from bursting pipes due excessive pressure. Hence function of these chambers is to release high pressure or convert into atmospheric pressure forming the new static water level.

7. **Treatment or purification works**

Raw water may contain various impurities. The purpose of water treatment is to remove those impurities which are objectionable either from taste and odour aspect or public health aspect. The aim of water treatment is to produce and maintain water that is hygienically safe, aesthetically attractive and palatable, in an economic manner.

8. **Reservoir**

Reservoir is used to reserve water. Depending on the purpose of use, it can be clear water reservoir, balancing reservoir and service or distribution reservoir. Clear water reservoir is used for storing treated water, balancing reservoir for equalization or to address fluctuation of demand where as service or distribution reservoir for equalizes the hourly fluctuations and stores the water for break down reserve and fire reserve as for fire fighting.

9. **Control valves**

These are essential appurtenances provided in the pipelines. There are various valves used for different purposes like to control and regulate the flow of water, releasing valve, air relief valve, wash out valves etc.

10. Distribution system

It is a pipe network laying to deliver water to the consumers premises from the distribution or service reservoir. Distribution system is designed for the peak flow. The method of laying distribution system is guided by the road network of the city.

11. Break pressure tank (BPT)

These are the tank or a chamber facilitated in the rural water supply distribution system to overcome the failure by burst of pipes due to excessive pressure. The function of the BPT is releasing high pressure into atmospheric pressure.

12. Public stand post

These are the last component of water supply system from where consumes collect water to meet their household demand in rural area. If people cannot afford private connections in rural area and as of scattered houses in the area a stand post serves to 8 to 10 households.

Problems

1. Define the following terms:
   Pure & impure water;
   Potable and wholesome water; and
   Polluted & contaminated water.

2. Enlist objectives of water supply system focusing for rural water supply in Nepal. Draw its schematic diagram mentioning components.
3. Draw a typical layout of a water supply scheme and describe briefly of each component.
4. Discuss importance and necessity of water supply system.
5. What do you understand by water supply system? Describe historical development of water supply in Nepal.
6. Draw a schematic diagram of typical urban and rural water supply systems and briefly describe the function of each component.
7. With a neat sketch, discuss briefly about major components of water supply system.
8. Discuss about the importance of water and necessity of water supply schemes in the present day community.
9. Enlist the objectives of water supply system & briefly explain each of them in relation to rural & urban water supply systems.

2 Sources of Water

2.1 Classification of sources of water

Water obtained through precipitation is retained in surface depressions, carried away as surface runoff in natural streams or rivers and percolates into the ground and joins the groundwater. This portion of precipitation which may be utilized for water supply obtained partly at the ground surface and partly below the ground surface. Various sources of water available for water supply may be broadly classified into following categories as shown in figure 2-1.

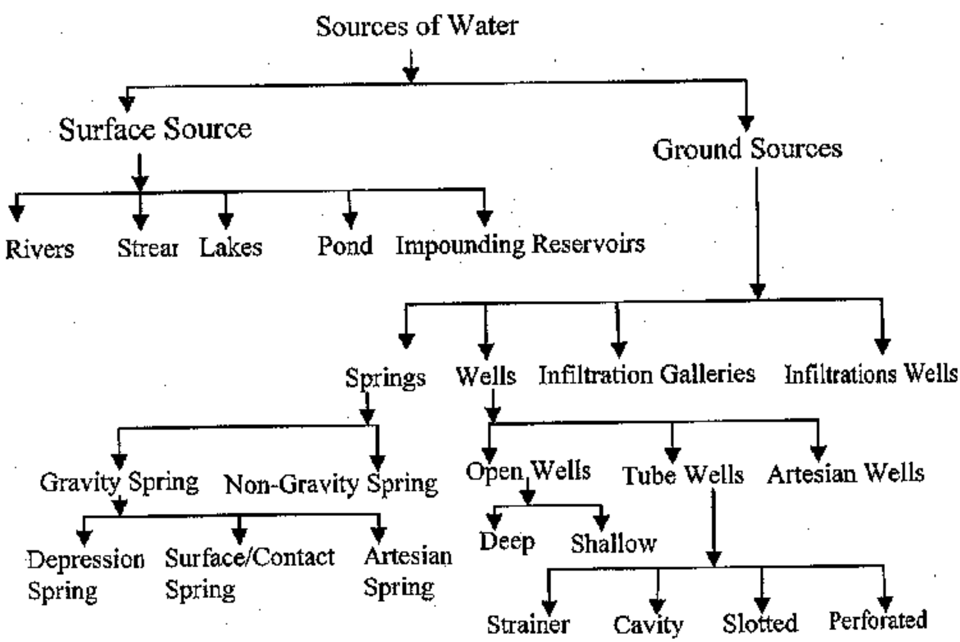


Figure 2-2 Classification of Sources of water

Apart from ground and surface source rain water harvesting is came in practiced nowadays that may require huge storage but it saves treatment cost. While waste water reclamation is practiced in developed counties which requires huge cost for installation and operation and maintenance.

2.2 Surface source of water

Water available at the ground/earth surface is termed as surface source. The water quality and quantity of surface source depends on rainfall patter; climatic and geological factors. The quality of water varies all the time from its origin until it reaches to oceans. During storms surface water carries numerous impurities as organic and suspended matters. Hence surface water during rainfall picks of various impurities which increase dose of coagulant, chlorine dose and burden in filter units at treatment plants. Various forms of surface sources of water are of the following.

1. Lake and ponds
2. Stream or rivers
3. Storage/Impounded reservoirs

2.2.1 Ponds

It is natural/artificial depression filled with water. Main source of water of pond is the rainfall. It contains lots of impurities so it is not useful for water supply purpose. The quantity of water in ponds is very less and used for washing purpose, animal bathing etc.

2.2.2 Lake

A large natural depression or hollow formed in earth's surface, which gets filled with water is a lake. Lake is mostly found in mountainous region. The *quantity* of water available in lake is generally very large, though it depends upon its size, catchment/drainage area, annual rainfall, porosity of ground and geological formation. The *quality* of lake water mainly depends upon the characteristics of the catchment. The water in a lake would be relatively of good quality if it is located in the uninhabited upland hilly areas though high degree of treatment of water may be required if lake is small and contain still water because that may have plenty of algae, weed and other vegetation growth imparting bad smell, taste and colour to the water.

2.2.3 Stream

A stream is a natural channel which carries surface runoff received by it from its small catchment. Little quantity of water available in stream but may have more in rainy period. Streams may be perennial or non- perennial. Quality of water is good in high altitude except the water from the first runoff but sometimes it contains impurities and can be used after some treatment.

2.2.4 River

A river is a natural channel which carries surface runoff received by it from its catchment or drainage area. The rivers may be perennial or non- perennial. Perennial rivers are those in which water is available throughout the year. Such rivers are fed by rains during the rainy season and by melting of snow during the summer season. On the other hand non-perennial rivers are those in which water is not available throughout the year. River water quality close to point of origin in the mountains is fairly good but as the river approaches plains the quality of its water deteriorates considerably, because it picks up lot of suspended matter, clay, silt etc. and becomes muddy in appearance so it always requires treatment.

2.2.5 Storage/Impounding reservoir

An artificial lake constructing by a dam or barrier across the river, which can store the excess water that flows in the river during the period of high flow for use during the periods of low flow. The quality of water in a storage reservoir mainly depends on the quality of the river water and needs to be properly analyzed and treated before supplying to the consumers.

Construction of impounding reservoir is not feasible under the following conditions.

- When average annual flow in lower than average demand
- When rate of flow of river in dry season is more than that of demand.

Example: *Following data gives the monthly inflows during the critical low flow period at the site of a proposed dam across a river. Determine analytically the storage capacity required of impounded reservoir to maintain a constant draft of 6202 million liters of water per month.*

Solution:

Demand = 6202 MLD =6202×10^6 liter/month

Month	Inflow, m^3/s	Month	Inflow, m^3/s
January	2.97	July	2.00
February	1.99	August	3.00
March	1.00	September	4.00
April	0.00	October	5.00
May	0.51	November	4.00
June	1.00	December	2.80

$1\ m^3/sec = 1000 \times 60 \times 60 \times 24 = 86.4 \times 10^6$ lit/day

Month	No. of days in each month	Inflow		Demand ($\times 10^6$ lit)	Surplus ($\times 10^6$ lit)	Deficit ($\times 10^6$ lit)
		Rate (m^3/sec)	Volume = q× d×86.4 ($\times 10^6$ lit)			
Jan	31	2.97	7954.848	6202	1752.848	-
Feb	28	1.99	4814.208	6202	-	1387.792
Mar	31	1.00	2678.400	6202	-	3523.600
Apr	30	0.00	0	6202	-	6202.000
May	31	0.51	1365.984	6202	-	4836.016
Jun	30	1.00	2592.000	6202	-	3610.000
Jul	31	2.00	5356.800	6202	-	845.2.000
Aug	31	3.00	8035.200	6202	1833.200	-
Sep	30	4.00	10368.00	6202	4166.000	-
Oct	31	5.00	13392.00	6202	7190.000	-
Nov	30	4.00	10368.00	6202	4166.000	-
Dec	31	2.80	7499.520	6202	1297.520	-
				Total	20405.52	20404.60

Storage capacity required = Total deficit = 20404.608 ML
Total deficit is smaller than surplus hence project is feasible.

2.3 Ground/sub-surface

Water exist below the ground surface is termed as ground source. Ground water acquires its chemical characteristics from surface water that *percolates into the ground. The quality of groundwater is generally good due to natural filtration as it percolates then infiltrate* to deeper strata. As ground water is not exposed to atmosphere it may be free from direct contamination and pollution from runoff. Various forms of ground sources of water are of the following forms.

1. Springs
    a) Gravity (Depression, surface, artesian)
    b) Non-gravity
2. Infiltration galleries
3. Infiltration wells

4. Wells and tube wells
   a) Open/Dug/Draw well (Shallow and deep)
   b) Tube well (Shallow and deep)
      (Strainer type, Cavity type, Slotted type, perforated type)
   c) Artesian well

2.3.1 Confined and unconfined aquifers

An aquifer may be defined as a geological formation that contains sufficient permeable material which permits storage as well as transmission of water through it under ordinary conditions. Terms commonly used to represent an aquifer are groundwater reservoir and water bearing formation.

A *confined* aquifer is the one in which groundwater is confined under pressure greater than atmospheric pressure by overlaying relatively impermeable strata. It is also known as artesian or pressure aquifer. An *unconfined* aquifer is one in which water table forms the upper surface of the zone of saturation.

Aquiclude: An impermeable body of rock or stratum of sediment that acts as a barrier to the flow of water. Clay layer is an example of aquiclude.

Aquifuge: An impermeable body of rock which contains no interconnected openings or interstices and therefore neither absorbs nor transmits water. Example of aquifuge is granite bed.

2.3.2 Springs

Whenever an aquifer or an underground channel reaches the ground surface such as a valley or a side of a cliff, water starts flowing naturally. This natural flow is known as a *spring*. A spring is an outflow of groundwater automatically to the ground surface due to geological formation. Spring is generally found in hilly areas. The quality of spring water is very good which does not require any treatment. Spring may be classified into;

i) those resulting from gravitational forces (Gravity Spring) and

ii) those resulting from non-gravitational forces.

Types of gravity springs

a) **Depression spring**

These springs formed due to overflowing of the water-table, where the ground surface intersects the water-table as shown in figure 2-2. The flow is variable with rise or fall of water-table and hence in order to meet with fluctuations a trench may be constructed.

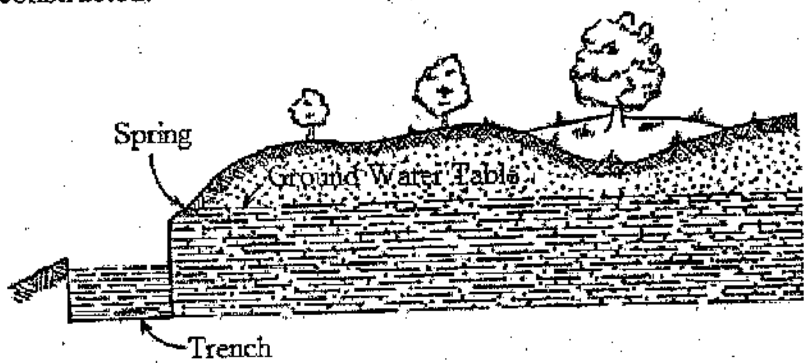


Figure 2-2 Depression Spring

b) **Contact/Surface springs**

These springs are created by a permeable water bearing formation over laying a less permeable or impermeable formation that intersects the ground surface as shown in figure 2-2.

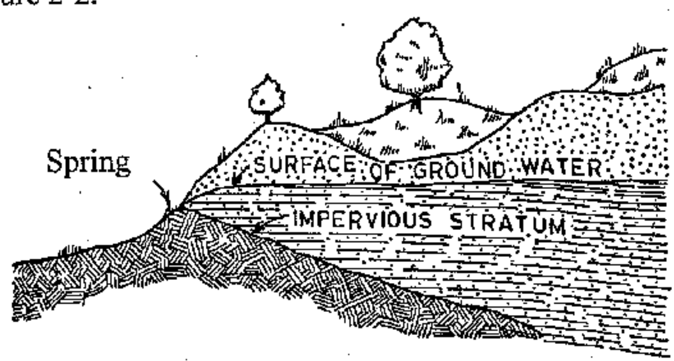


Figure 2-3 Contact Spring

c) **Artesian Spring**

Release of ground water under pressure from confined aquifer or through an opening in the confining bed as shown in figure 2-4 is artesian spring.

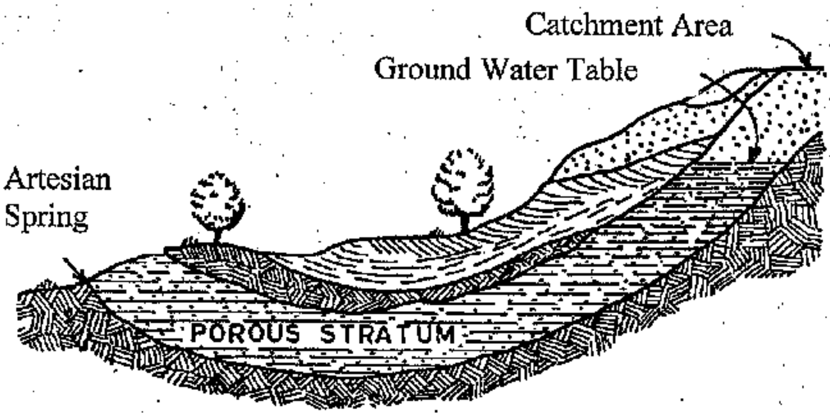


Figure 2-4 Artesian Spring

Non-gravity springs

Non-gravity springs include volcanic springs and fissure springs. The volcanic springs associated with volcanic rocks and fissure spring result from fractures extending to great depths in the earth crust shown in figure 2-5. These are usually thermal springs and discharge highly mineralized and often contains sulpher. However, the water obtained from some of hot springs is found to be useful for the cure of certain skin disease.

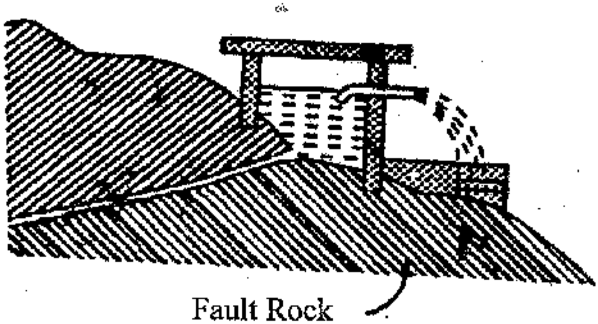


Figure 2- 5 springs due to fault in rock

2.3.3 Infiltration galleries

A horizontal or nearly horizontal tunnel having permeable boundaries for tapping underground water near rivers, lakes or streams are called 'Infiltration galleries' as shown in figure 2-6. Infiltration galleries are constructed by cut and cover method usually rectangular in cross-section and also sometimes known as horizontal well. Infiltration galleries may be constructed with dry brick masonry wall or porous concrete blocks with weep holes and RCC slab roof or an arch roof. The yield from the galleries may be as much as 1.5×10^4 lit/day/m length of infiltration galleries.

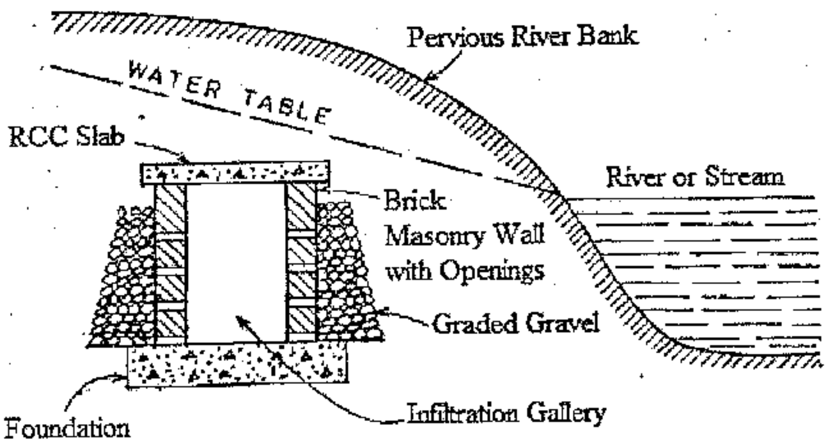


Figure 2-6 Infiltration galleries

2.3.4 Infiltration wells

A shallow well constructed in series along the banks of a river as shown in figure 2-7 to collect the water seeping through the banks of the river is called infiltration wells. These wells are constructed of brick masonry with open joints and RCC slab in the top with manhole for the purpose of infiltration. Various infiltration wells are constructed by porous pipes to a collecting sump well known as jack well. The water from the jack well is pumped to the treatment plant and supplied to the consumers after treatment.

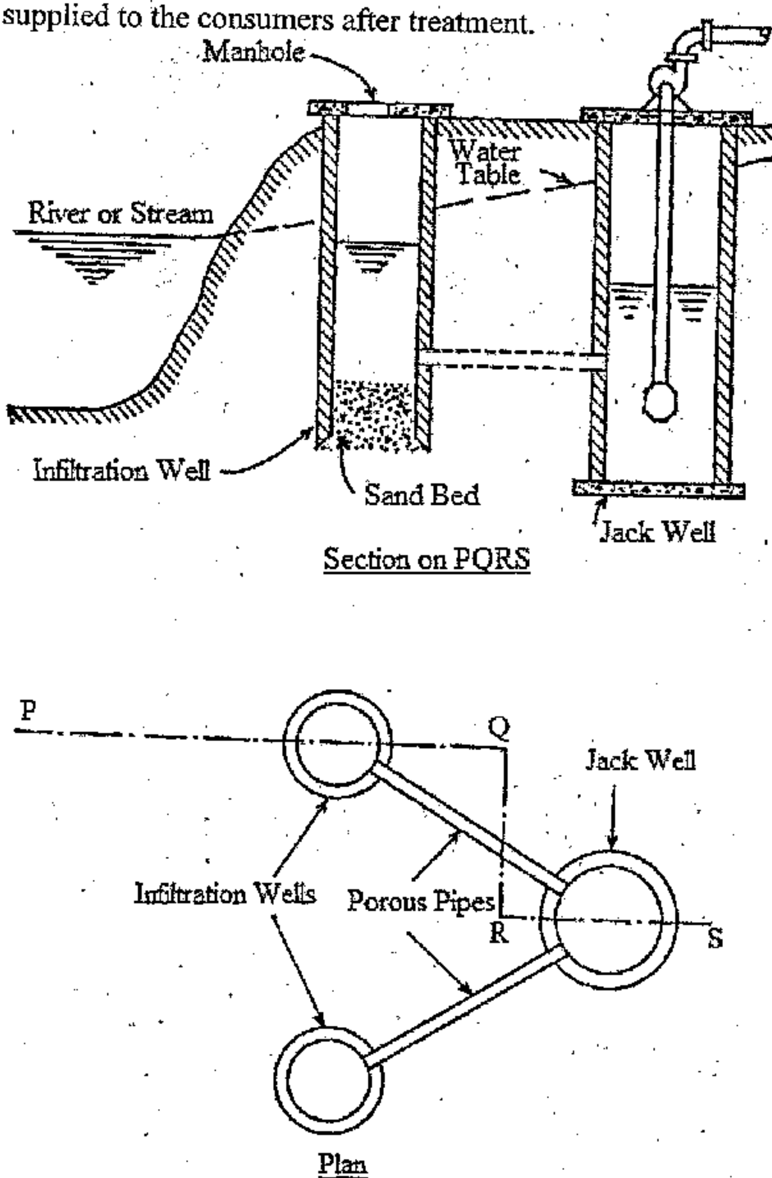


Figure 2-7 Infiltration wells

2.3.5 Wells

A well is a hole, usually vertical, excavated in the ground to take ground water. Water wells may be classified as;

1) Open/Dug/Draw wells
2) Tube wells

1) Open wells

Open wells are comparatively of large diameters but low yields and are not very deep. These wells may have diameter of 1 to 10 m usually and depths generally range of 2 to 20 m.

The walls of an open well may be built of brick or stone masonry or precast rings. Open wells may be further classified as shallow and deep.

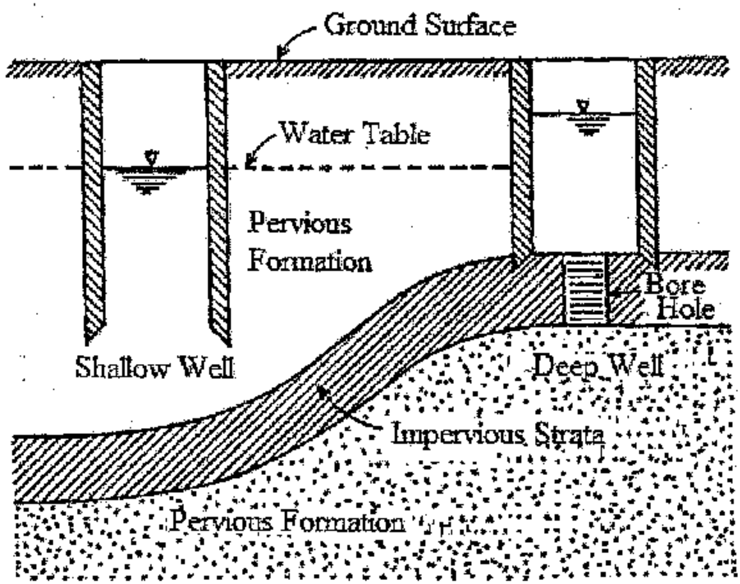


Figure 2-8 Shallow and Deep Well

a) Shallow open wells

Shallow open wells are those which rest in the top water bearing strata and draw their supplies from the surroundings. There is always risk of contamination because water is obtained from the top most pervious layer. Shallow open and shallow deep open well has shown in figure 2-8.

b) **Deep open wells**

Deep open wells are those which rest on impervious strata and draw their supplies from pervious formation lying below the impervious strata. The quantity of water is greater in this well than in shallow wells. Generally it has better quality but may contain dissolved impurities.

Tube wells

A tube well is a long pipe sunk into the ground intercepting one or more water bearing strata. As compared to open wells the diameter of tube wells are much less. Depth of tube wells up to 30 m is shallow tube wells and up to 600 m are deep tube wells. The tube wells may also classify as;

a) Strainer type tube well
b) Cavity type tube well
c) Slotted type tube well
d) Perforated type tube well

a) **Strainer type tube well**

It is the most common and widely used tube well as shown in figure 2-9. The pipe introduced into the ground is a assembly of strainer pipes and blind pipes which are alternately placed. The length of the strainer and the blind pipe are so adjusted that the strainer pipe rest against water bearing strata and the blind pipes rest against impervious strata. For construction of this tube well, boring is done by lowering a casing pipe of 5 to 10 cm larger than diameter of well pipe and record of strata during boring then blind and perforated pipes are jointed as per result than inserted into casing pipe. The gravel packing is done during removal of casing pipe.

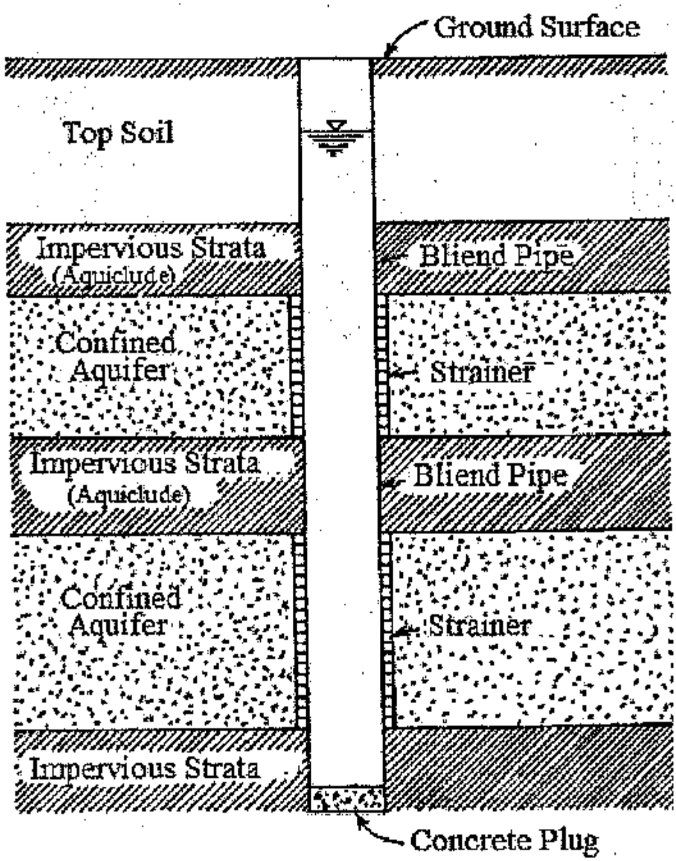


Figure 2-9 Strainer type tube well

b) Cavity type tube well

A cavity type tube well as shown in figure 2-10 consists of a pipe sunk into the ground and resting on the bottom of a strong strata. In the initial stage of pumping fine sand comes out with water and consequently a cavity is formed at the bottom. Though in the beginning sandy water is obtained from a cavity type tube well but with the passage of time clear water is obtain.

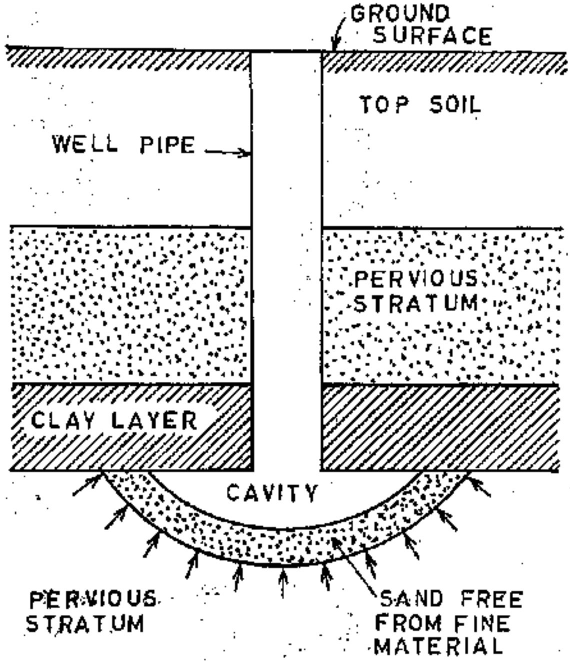


Figure 2-10 Cavity type tube well

c) Slotted type tube well

It consists of a pipe which is slotted for part of its length at one end and the rest of length is plain pipe. The mixture of gravel and coarse sand placed around the pipe to prevent the fine sand entering the well pipe. The slotted portion of pipe is surrounded by mixture of gravel and coarse sand called shrouding which is filled between casing pipe and slotted pipe during withdraw of casing. Thus the diameter of the casing pipe is kept more than the diameter of the well pipe. For example for a well pipe of 150 mm diameter casing pipe about 400 mm diameter is required. Slotted type tube well has shown in figure 2-11.

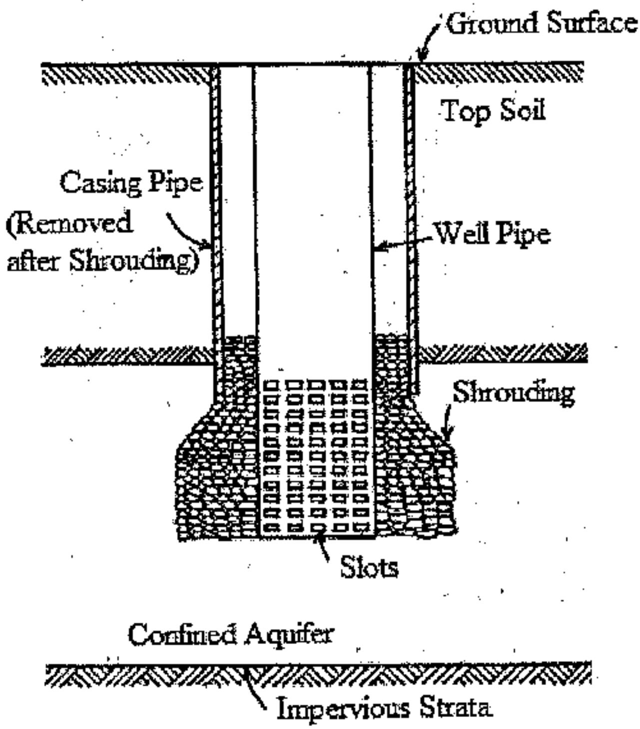


Figure 2-11 Slotted type tube well

d) Perforated type tube well

Perforated type tube well is shallow depth tube well and used for short duration such as in construction site. Pipes having covered by jute ropes act as strainer and which prevent the fine particles entering the well pipe (see figure 2-12).

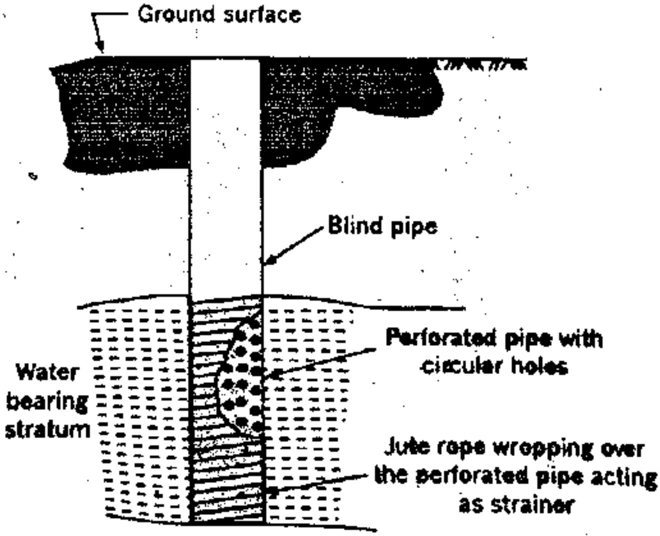


Figure 2-12 : Perforated type tube well

2.3.6 Artesian wells

An artesian well is a pump less water source from where ground water, retained in confined aquifer, flows automatically under pressure as shown in figure 2-13. Mostly they are found in the valley portion of the hills where aquifers on the both sides are inclined towards valley. The HGL passes much above the mouth of well, which causes flow under pressure.

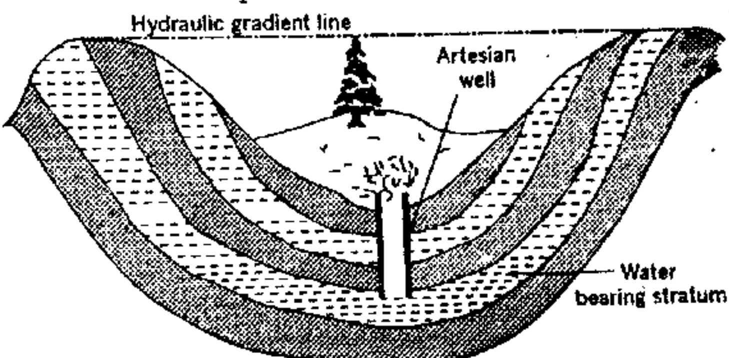


Figure 2-13 Artesian Well

Table 2-1 Surface Water Verses Groundwater

Parameters	Surface Water	Groundwater
Understanding	Easily seen & observed	Invisible, mysterious, complex.
Where Found	Streams, rivers, lakes etc.	Everywhere beneath the surface in layers of sand, gravel, clay or cracked rock. Rarely forms underground "streams" or "lakes"
Uses	**Drinking water**, food (fish, trapping, rice etc), transportation, bird/animal habitat, power, aesthetics and spiritual health.	Drinking water, energy, maintaining flow in surface water courses.
Flow Direction	Downhill	Usually from high to low elevations.
Flow Rate	Fast (meter/second)	Slow (meter per year)
Quantity (yield)	Easy to assess. Supply problem rare.	Drilling and pumping. Lots of water in sand and gravel or heavily fractured rock. Low yields in silt and clay or unfractured rock.
Quality	Low dissolved (soft, low iron). High organics. Temperature Changes Mud, clay, algae	High dissolved (hard, high iron) Low organics Constant, cool temperature No suspended solids
Consistency	Changes with seasons	Constant over time
Safety	Variable bacteria/virus counts	Safe because of filtration and natural purification processes
Treatment	Continuous chlorination	Initial chlorination
Cost	High	Low
Contamination Risk	Easily contaminated	Not easily contaminated.
Contamination Remediation	Natural breakdown by sun, air, mixing etc.	Clean-up difficult or impossible; May take many decades.

2.3.7 Selection of water source

The water source should selected considering various factors such as reliablity, sustainable and safe, free from water right problem, quality, quantity, location cost, etc. that will give adequate quantity of water with good quality requre less treatment at affordable cost to consumers. The following factors are generally considered while selecting a source of water supply for a particular town or city.

1. Quality of water
2. Quantity of water
3. Location
4. Cost of w/s project

1. **Quantity of water:** The source should be able to supply enough quantity of water to meet various demand of city during the entire design period. Water availability in source may be fluctuating with seasons so quantity of water that tapped in dry period should meet the water demand. Safe yield of source should be adequate to meet desired demand of water throughout the year.

2. **Quality of water:** The source should have safe wholesome, free from pollution of any kind and other undesirable impurities. The impurities present in the water should be as less as possible and these should be removed easily and cheaply.

3. **Location**: Source of water should be located near to the community as far as possible and it should be situated in elevated area. This will reduce length of pipe and water from the source would flow by gravity and hence cost of project reduced. Pumping in system increase operation and maintenance hence better to minimize the use as a component. If both ground and surface both source available near the area to served obviously it is preferred to surface source because use of ground water may has adverse impact in environment also

lowering of water table may effects fertility of soil, subsidence of land etc. in case of use of ground source provision of ground water recharge may require.

4. **Cost of water supply project:** As far as possible overall cost of the w/s project should be minimized so that water can be supplied to consumers at affordable price. Cost of water supply project mainly depends on the location of source, elevation of source, water quality available in the source, topography of the area, distance between source and community etc.

Problems

1. Write shorts note on
   a) Infiltration galleries
   b) Impounding reservoir
   c) Artesian well
   d) Tube well

2. Describe the selection criteria of a good source for drinking of water supply project.

3. Under what circumstances springs are formed? With the help of neat sketches describe various types of springs.

4. What are the various sources used in water in water supply schemes? Discuss their advantages and disadvantages.

5. What are the various sources of water used in water supply scheme? Discuss their merits and demerits from quality & quantity point of views.

6. Describe the possible water sources for public water supply system in (a) hilly areas and (b) terai areas of Nepal.

7. Under what circumstances springs are formed? Describe the various types of springs with the help of neat sketches.

8. Differentiate between shallow & deep wells. Which of them would you prefer for use in public water supply scheme and why?

9. Write short notes on:
   a) Infiltration well
   b) Artesian well
   c) Tube well
   d) Infiltration galleries.

10. Compare the water characteristics surface water verses ground water.

11. Following data gives the monthly inflows during the critical low flow period at the site of a proposed dam across a river. Determine analytically the storage capacity required of impounded reservoir to maintain a constant draft of 2500 million liters of water per month.

Month	Inflow(m^3/s)
January	0.04
February	0.45
March	0.62
April	0.93
May	1.33
June	1.78
July	1.89
August	3.17
September	3.35
October	1.10
November	0.07
December	0.06

3 Quantity of Water

Water supply project design involves the determination of water demand or quantity of water required daily for various purposes, which can be known after the determination of population and per capita demand or rate of demand. Population considered for design purpose is design year population and has to be calculated so that a water supply project is designed to meet present water demand as well as the future reasonable period is considered that it has to serve known as design period. After fixing a suitable design period; design year population can be known by using any suitable population forecasting method with the help of census data. Once knowing the design year water requirement reliable source is selected as per selection criteria. Hence, estimation of water demand or the quantity of water required for a community it is required to know following factors.

- Per capita demand / rate of demand
- Base period and design period
- population

3.1 Per capita demand of water / Rate of demand

It is defined as average quantity of water consumption per day for various purposes including all demands of water for a person. Also, it can be defined as *average quantity of water required a single person per day.* Let 'Q' be the total quantity of water required per year in liters by city or town having population (P), and per capita demand or rate of water demand (q) usually expressed in lpcd is given by the following expression.

$$\text{Per capita demand (q)} = \frac{Q}{P \times 365} \text{ lpcd}$$

3.2 Design and base periods

Design period

A water supply project is planned to meet the present demand of the city or town as well as the demand for a reasonable future period or number of years is considered is called as design period .
The number of years in future for which the project is designed is called design period. It should be realistic i.e. neither long (financial overburden) nor short (uneconomical). The **selection basis** of the design period is as follows.

1. **Availability of funds**: If only limited fund is available then a shorter design period will have to be considered and vice-versa.
2. **Availability of water**: Design period is controlled by the water available in source so that water should be sufficiently available for that design period.
3. **Population growth rate**: Growth of population is a major factor that of area to be served in fixing design period. In our context Nepal design period of rural water supply project is fixed as per population growth rate i.e. if higher the growth rate less design period and vice-versa.
   - $r \geq 2$, design period is 15 yrs.
   - $r \leq 2$, design period is 20 yrs
4. **Economic development**: Economic development of the area to be served is also a factor which governs to select design period. If economic development is rapid design period considered should be less.
5. **Life of the pipes and construction materials**: Design period should not be greater than components useful life.
6. **Rate of interest of loan**: If more the rate of interest lesser will be the design period and vice-versa.

Typical design periods

Usually in rural areas 15 to 20 years is taken as typical design period where as up to 30 years is taken as a typical design period in urban areas as shown in figure 3-1.

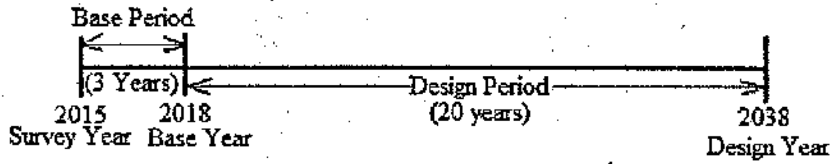


Figure 3-1 Typical Base and Design period

Survey year

This is the year in which data are collected for a start of the project of supplying water.

Base period

A water supply project takes time to conduct survey, analysis, design and construction of the components of water supply system. The time required for survey, design and construction before implementation is called base period. Base period is generally taken as 2 to 3 years but it is taken as 2 years in Nepal.

Base year

This is the year in which water is actually supplied to the consumers (implementation).

Design year

This the year considered for which water supply system is designed i.e. service of the water supply system can fulfill required demand. Rehabilitation or strengthening of the components may be required to get service from the project after design year.

Design Year = Survey Year + Base Period + Design Period

3.3 Types of water demand

The water required for various purposes as domestic, livestock, institutional and commercial, fire fighting, industrial, municipal or public, losses or wastage demand are the types of water demand. These various demand is not essential to take in account to calculate total water demand of city or community or town so that fire fighting, municipal, industrial demand may be excluded in rural areas where as livestock demand is excluded in urban area.

3.3.1 Domestic demand

This includes the water which is required for use in private residence for drinking, cooking, and bathing, washing of cloths, lawn watering and gardening and sanitary purpose. The amount of domestic water demand depends on the living conditions of the consumers, climatic condition, habit, social status etc. Domestic water demand is about 40 to 60 % of total demand of the entire scheme.

For design purpose in Nepal,

- For urban or fully plumed house – 112 lpcd
- For semi-urban or partially plumbed house –65 lpcd (No sewerage system)
- For rural area with public tap stand – 45 lpcd (Sometimes 25 lpcd)

3.3.2 Livestock demand

The quantity of water required for domestic animals is called livestock demand. It is considered only in rural water supply. In practice, up to 20% of domestic demand is taken as livestock demand. Livestock demand is not considered more than 20% of domestic demand so that domestic animals utilize natural source like stream, lake, ponds water at the time of grazing.

- For big animals – 45 lit/animal/day (Cow/ buffalo/horse)
- Medium animals – 20 lit/animal/day (Goat, dog, rabbit etc)
- Small animals – 20 lit/100 bird/ day (Birds, chicken, duck etc.)

3.3.3 Commercial/ Institutional demand

It includes the demand for commercial establishments and institutions like universities, school, office building, warehouse, stores, hotels, hospitals theaters, clubs etc. The commercial and institutional demand is considered in both the rural and urban water supply. The commercial and institutional water demand in Nepal is generally taken as shown in Table 3-1.

Table 3-1 Commercial and Institutional Demands

S. No.	Type	Demand
1.	Hospitals/Health post/Clinics	
	i) With bed	500 liter/bed/day
	ii) Without bed	2500liter/hospital/day
2.	School	
	i) Boarders	65 liter/pupil/day
	ii) Day scholar	10 liter/pupil/day
3.	Hotel	
	i) With bed	200 liter/bed/day
	ii) Without bed	500 – 1000 liter/day
4.	Restaurants/Tea stall	500 – 1000 liter/day
5.	Office	500 – 1000 liter/office/day

3.3.4 Public/Municipal demand

Water required for public or municipal utility such as washing and sprinkling on road, flushing sewers, watering public parks etc. is municipal or public demand. A provision of 5 to 10% of the total demand is taken as this demand. This demand is only considered in urban water supply system.

3.3.5 Industrial demand

Industrial area should be located far from the city though it may locate in periphery of city which may be vital in calculating water demand. Normally 20-25% of total demand is taken for industrial demand. It is considered only in urban area and depends upon the type and size of industry.

3.3.6 Fire fighting demand

During outbreak of fire, the water is used for firefighting is called fire demand. This demand is not fixed so it is difficult to calculate demand. Different empirical formula can be used to determine fire demand but it cannot directly used for Nepalese context. This demand is considered in urban water supply system.

i) National Board of fire underwriter's formula

$$Q = 4637\sqrt{p}\left(1 - 0.01\sqrt{p}\right)$$

Where, Q = quantity in lit/min

p = population in thousands

ii) Freeman's formula

$$Q = 1136\left(\frac{p}{5} + 10\right)$$

where, Q= quantity in lit/min

p= population in thousands

iii) Kuilching's formula

$$Q = 3182\sqrt{p}$$

where, Q = quantity in lit/min

p = population in thousands

iv) Buston's formula

$$Q = 5663\sqrt{p}$$

where, Q = quantity in lit/min

p = population in thousands

v) Indian water supply manual and treatment formula

$$Q = 100\sqrt{p}$$

where, Q = quantity in m^3/day

p = population in thousands

Guide lines published by department of water supply and sanitation (DWSS) recommend this formula to determine fire demand in Nepal. Maximum fire demand should not be more than 1 lpcd.

3.3.7 Loss and wastage

Loss and wastage may be termed as unaccounted for water which includes water due to faulty valves and fittings, poor distribution system, defective pipes, unauthorized connections, tap open etc. Lost and wasted being uncertain it cannot be predicted precisely so generally it is taken as 15 to 20% of total demand.

3.3.8 Total water demand

The sum of all water demands is total water demand as given below.

$$TD = DD + LD + ID + ID + PD + FD + LD$$

Where,

TD	=	Total water demand
DD	=	Domestic water demand
LD	=	livestock demand
ID	=	institutional and commercial demand
ID	=	Industrial demand
PD	=	Public/municipal demand
FD	=	fire demand
LD	=	Losses and wastage demand

3.4 Variations in demand of water/ rate of demand

The rate of demand of water represents the average consumption or demand of water per capita/head per day. Rate of demand does not remain constant but varies with the season or month of the year, with the days of week, and with the hours of the day. These variations in the rate of demand of water are termed as,

i) Seasonal/monthly variations
ii) Daily variations
iii) Hourly variations

i) Seasonal variations

The rate of demand of water varies considerably from season to season. In summer water demands usually 30 to 40% above the annual average rate of flow of water, because more water is required for drinking, bathing, washing etc. In winter the average rate of demand is about 20% lower than the annual average rate of demand of water because of less requirement of water.

$Q_{seasonal} = 1.3 \times Q_{average}$ (In India)

$Q_{seasonal} = 1 \times Q_{average}$ (In Nepal)

ii) Daily variations

Due to change in the day to day climatic conditions, or due to the day being a holiday or some festivals day the rate of demand of water varies from day to day and called daily variations.

$Q_{daily} = 1.8 \times Q_{average}$ (In India)

$Q_{daily} = 1 \times Q_{average}$ (In Nepal)

iii) Hourly variations

Maximum demand of water usually occurs in the morning and in the early morning hours the demand of water is at its maximum and also during noon. Hence, demand also varies even hour to hour called hourly variation. A typical graph showing hourly variation in the rate of demand is shown in figure 3-2.

$Q_{Hourly} = 1.5 \times Q_{average}$ (In India)

$Q_{Hourly} = 2 \text{ to } 4 \times Q_{average}$ (In urban area of Nepal)

$Q_{Hourly} = 3 \times Q_{average}$ (In rural area of Nepal)

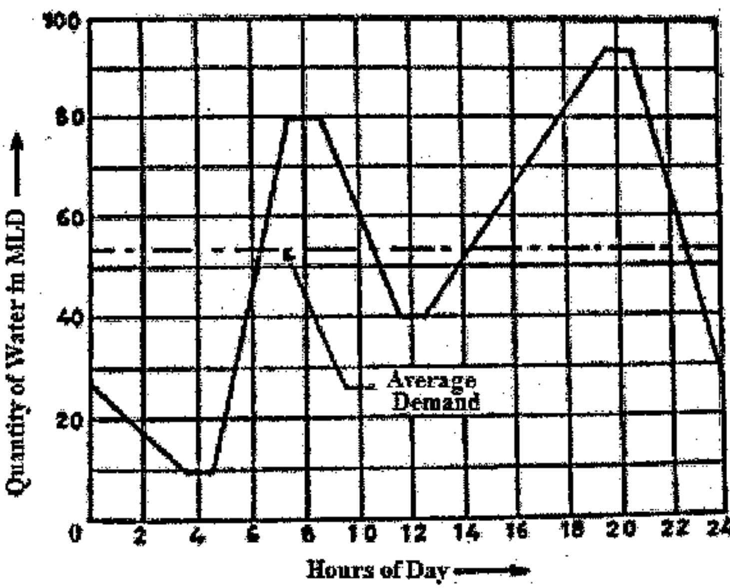


Figure 3-2 Hourly Variations in Water Demand

3.5 Peak factor

Peak factor is the ratio of maximum or peak demand of water to annual average demand of water as shown by the expression given below.

$$\text{Peak factor (pf)} = \frac{Q_{\text{peak/maximum}}}{Q_{\text{avgerage}}}$$

For transmission mains peak factor of 1 is adopted i.e. transmission mains are designed for maximum daily demand.

$Q_{seasonal} = Q_{average} \times PF_{seasonal}$

$Q_{daily} = Q_{average} \times PF_{seasonal} \times PF_{Daily}$

$Q_{daily} = Q_{average}$

The peak factor for distribution system of 2 to 4 is adopted for continuous system where as 4 to 6 is adopted in intermittent system i.e. distribution system is designed for maximum hourly demand.

$Q_{Hourly} = Q_{average} \times PF_{seasonal} \times PF_{Daily} \times PF_{Hourly}$

$Q_{Hourly} = 2$ to 4 (Where, $PF_s=1$, and $PF_D=1$)

3.6 Factors affecting demand of water

The variation in the rate of demand of water is due to several factors which must be carefully studied and analyzed before fixing the rate of demand of water for particular city. The various factors affecting demand of water are as follows;

1. **Climatic conditions**: The quantity of water requirement in the area having hot climate is obviously more comparatively area having cold climate area due to bathing and washing frequencies, air conditioning etc.

2. **Age and size of the city**: Bigger the city demand will be more and vice-versa. New city may have more demand than the old city due to construction works such as building construction, infra structure development etc.

3. **System of supply:** There may be two types of system of supply either throughout the day (continuous system) or limited period (intermittent system). Water demand is less in intermittent system due many problems faced in this system like limited water, storage requirement, alert at the period of supply hour etc.

4. **Cost of water**: Cost is a major factor which affect rate of demand because if cost of water is cheap there will be more consumption and if high cost lesser water may be consumed.

5. **Sewerage system:** If sewerage system is provided in the community water required for flushing and cleaning so demand will be increased.

6. **Pressure in supply**: Pressure in distribution system having adequate might be increase in demand where as low pressure results decrease in demand. Higher the pressure losses of water may be more.

7. **Method of charging**: To be sustain a water supply system charging is essential. Various tariff systems may be adopted in water supply system for charging. Such as monthly basis or metering system which affect the demand of water.

8. **Quality of water**: Water demand will be more if supplied water quality is good due to feeling safe to use water and use it liberally by consumers. While poor water quality supply may reduce in consumption.

9. **Living standard or status of consumers**: The higher the living standers of consumers will be greater the water demand because consumer can afford the luxury such as air conditioning, water fall, lawns and gardening, cleaning etc.

10. **Other socio-economic factors**
    - Public versus private tap stand
    - Wealthy versus survival (subsistence)

- Distance of tap stand
- Habits of people
- Urban versus rural

3.7 Population forecasting method

After fixing design period it is necessary to forecast or estimate the design year population. The present population of a city or ton may be obtained from the census records which are conducted by the government at an interval of 10 years. The future population of a town or city at the design year may be predicted on the basis of the census data for a number of preceding census year. However, in order to predict the future populations, as correctly as possible it is necessary to know the factors affecting the population growth, which are as, economic factors, development project, social facilities, death and birth rates, unforeseen factors, migration rates etc.

i) Arithmetical increase method

In this method it is assumed that the future increase in population is at a constant rate. This constant rate of increase in population in future is taken as the average increase in population per decade during a number of past successive decades. This method gives lower values and is adopted for large cities reached their maximum development.

$$\frac{d_p}{d_t} = c$$

$$p_1 = p_0 + c = p_0 + 1c$$

$$p_2 = p_1 + c = p_0 + 1c + c = p_0 + 2c$$

$$p_3 = p_2 + c = p_0 + 2c + c = p_0 + 3c$$

Generalizing, $p_n = p_0 + nc$

ii) Geometrical Increase Method

This method is based on assumption that % increase in population per unit remains constant for each time and also known as uniform growth method. This method usually gives higher results as compared to arithmetical increase method. This method is suitable for rapidly growing or young city.

$$p_1 = p_0 + r\% \text{ of } p_0 = p_0\left(1+\frac{r}{100}\right) = p_0\left(1+\frac{r}{100}\right)^1$$

$$p_2 = p_1 + r\% \text{ of } p_1 = p_1\left(1+\frac{r}{100}\right) = p_0\left(1+\frac{r}{100}\right)^2$$

$$p_3 = p_2 + r\% \text{ of } p_2 = p_2\left(1+\frac{r}{100}\right) = p_0\left(1+\frac{r}{100}\right)^3$$

Generalizing,

$$p_n = p_0\left(1+\frac{r}{100}\right)^n$$

iii) Incremental increase method

This method is improvements over the two methods arithmetical and geometrical. From previous census data the actual increase, then the increment in increase in each of those time unit and average of incremental increase per time unit 'i' is calculated. The population in the next time unit is calculated by adding the present population the average increase 'c' plus the net incremental increase for that time units and so on. The result given by this method is somewhere between the result given by arithmetic and geometric method.

$$p_1 = p_0 + (c + i) = p_0 + 1c + 1\left(\frac{1+1}{2}\right)i$$

$$p_2 = p_1 + (c + 2i) = (p_0 + c + i) + (c + 2i)$$
$$= p_0 + 2c + 3i$$
$$= p_0 + 2c + 2\left(\frac{2+1}{2}\right)i$$

$$p_3 = p_2 + (c + 3i) = (p_0 + 2c + 3i) + (c + 3i)$$
$$= p_0 + 3c + 6i$$
$$= p_0 + 3c + 3\left(\frac{3+1}{2}\right)i$$

Generlizing,

$$p_n = p_0 + nc + n\left(\frac{n+1}{2}\right)i$$

iv) Decreasing rate growth method

If we consider complete growth of a very old city we can see that the early growth takes place at highly increasing but latter the growth is at a decreasing rate. Hence this method is similar to the geometrical increase method except the instead of constant value of increase in population per time unit, the increase in population per unit time is adopted for each further time units. In this method the average decrease in % increment per time unit 'd' is calculated after calculating increase and then % increase in population.

Find net % increase in n^{th} time unit by,

$$I_n = I_{n-1} - d \quad \text{or,} \quad I_n = I_0 - nd$$

Then population at n^{th} time unit is,

$$p_n = \left[p_{n-1}\left(1+\frac{I_n}{100}\right)\right]$$

Generalizing,

$$P_n = P_0 \prod_{j=1}^{n}\left[1+\frac{I_0 - jd}{100}\right]$$

Where,

I_0 = % increase in population in last known yr % time unit

I_{n-1} = % increase in population in $(n-1)^{th}$ time unit

P_{n-1} = Number of just before population n^{th} time

d = Average decrease in % increment % time unit

P_1 = Number of population at time t_1

P_2 = Number of population at time t_2

P_n = Number of population at time n^{th} time unit

P_0 = Number of present population

C = Average increase in population

r = Average of the % increase

i = Average incremental increase

(Note: Unit may be year or decade)

Example: 3.1

Determine the population of the town in the year 2026 by a) Arithmetical, b) Geometrical, c) Incremental increase and d) Decreased rate of growth method.

Year AD	1961	1971	1981	1991	2001	2011
Population	18000	27000	38000	51000	66000	83000

Solution:

$$n = \frac{2026 - 2011}{10} = 1.5$$

Present or last known population (P_o) = 83000

Year AD	Population	Increase in population	% increase in population	Incremental increase	Decrease in % increase
1961	18000	-	-	-	-
1971	27000	9000	50.00	-	-
1981	38000	11000	40.74	2000	9.26
2091	51000	13000	34.21	2000	6.53
2001	66000	15000	29.41	2000	4.80
2011	83000	17000	25.75	2000	3.66
Total		65000/5	180.11/5	8000/4	24.25/4
Average		C =1300	r = 36.022	i = +2000	d = +6.0625

i) Arithmetic Method

$$P_n = P_o + nC$$
$$P_{1.5} = 83{,}000 + 1.5 \times 1300 = 1{,}31{,}672 \text{ nos.}$$

ii) Geometric Method

$$p_n = p_0\left(1 + \frac{r}{100}\right)^n$$

$$p_n = 83000\left(1 + \frac{36.022}{100}\right)^{1.5}$$

$P_{1.5} = 1,31,672$ nos.

iii) Incremental method

$$p_n = p_0 + nc + n\left(\frac{n+1}{2}\right)i$$

$$P_{1.5} = 83000 + 1.5 \times 1300 + 1.5\left(\frac{1.5+1}{2}\right) \times 2000$$

$P_{1.5} = 1,35,422$ nos.

iv) Decreased rate growth method

$$P_n = P_0 \prod_{j=I}^{n}\left[1 + \frac{I_0 - jd}{100}\right]$$

$$P_n = 83000\left(1 + \frac{25.75 - 6.0625}{100}\right)\left(1 + \frac{25.75 - 1.5 \times 6.0625}{100}\right)$$

$P_{1.5} = 1,15,887$ nos.

Example: 3.2

Estimate the population of a town for design year 2031 AD by any three methods and calculate the design quantity of water in lit/day. The census data are as follows.

Year AD	1971	1981	1991	2001
Population	40000	45000	55000	62000

Also consider fire demand and losses and wastage.

Solution:

$$n = \frac{2031 - 2001}{10} = 3$$

Present or last known population (P_o) = 62000

Year AD	Population	Increase in population	% increase in population	Incremental increase
1971	40000	-	-	-
1981	45000	5000	12.50	-
2091	55000	10000	22.22	5000
2001	62000	7000	12.73	-3000
Total		22000/3	47.45/3	2000/2
Average		C =7333	r = 15.82	i = +1000

i) Arithmetic Method

$P_n = P_o + nC$

$P_3 = 62{,}000 + 3 \times 7333 = 83999$ nos.

ii) Geometric Method

$$p_n = p_0\left(1 + \frac{r}{100}\right)^n$$

$$p_n = 62000\left(1 + \frac{15.822}{100}\right)^3$$

$P_3 = 96326$ nos.

iii) Incremental method

$$p_n = p_0 + nc + n\left(\frac{n+1}{2}\right)i$$

$$p_{1.5} = 62000 + 3 \times 7333 + 3\left(\frac{3+1}{2}\right) \times 1000$$

$P_3 = 89999$ nos.

Taking design year (2031 AD) population as forecasted by geometrical method = 96326 nos.

Assume per capita water demand for the town as 112 lpcd.

a) Domestic demand for town = population × per capita demand
= 96326 nos. × 112 lpcd
= 10788512 lit/day

b) Assume 1 lpcd of water for fire fighting
= 96326 nos. × 1 lpcd
= 96326 lit/day

c) Assume losses demand = 15 % of supplied water

Therefore, Total water demand

= Domestic demand + Fire demand + Losses demand

TD = 10788512 + 96326 + 0.15 TD

0.85 TD = 10884838

Total water demand = 12805692 lit/day

Example 3.3

In rural village, the survey is carried out in the year 2072 BS and the following data is obtained:

Population	= 5320 nos
Annual population growth rate	= 1.7 %
No. of cows	= 4030 nos
No. of goats	= 1520 nos
No. of chickens	= 5500 nos
No. of students	= 200 boarders & 1020 day scholars
No of VDC offices	= 5
No. of Tea shops	= 3

If the base year is taken as 2075 BS and the design period is 20 years, calculate the total water demand of the village for the service year.

Solution:

Survey year = 2072,

Total water demand in Design year (2095) = ?

For design year, n = Base period + design period

= 3+20 = 23 years

Design year population by geometrical forecasting method;

$$p_n = p_0\left(1+\frac{r}{100}\right)^n$$

$$p_{23} = 5320\left(1+\frac{1.7}{100}\right)^{23} = 7840 \text{ nos.}$$

Calculation of water demand;

a. Domestic demand = 7840 @ 45 lpcd = 352800 lit/day

b. Livestock demand

For Cows	= 4030 nos @ 45 lit/animal/day	= 181350 lit/day
For goats	= 1520 nos @ 20 lit/goat/day	= 30400 lit/day
For chickens	= 5500 nos @ 0.20 lit/bird/day	= 1100 lit/day
	Total livestock demand	= 212850 lit/day

Comparing livestock demand with 20% of domestic demand;

20% of domestic demand = 70560 lit/day.

Total livestock demand is greater than 20% of domestic demand so take livestock demand = 70560 lit/day

c. Commercial and Institutional demand

For day scholars students	= 1020 nos @ 10 lpcd	= 10200 lit/day
For boarders	= 200 nos @ 65 lpcd	= 13000 lit/day
For VDC office	= 5 @ 500 lit/office/day	= 2500 lit/day
For Tea shop	= 3 @ 500 lit/tea stall/day	= 1500 lit/day
	Total commercial & institutional demand	= 27200 lit/day

[Note: Students may be forecasted as per population growth rate]

Total water demand for design year = DD + LD + ID

= 352800+ 70560+27200 = **4, 50,560** lit/day

Example 3.4

Safe yield of a proposed spring is 5 lps and per capita water demand is 65 lpcd. Calculate the current population that can be taken under the scheme if design period is 20 years and population growth rate is 1.7% per annum.

Solution:

Safe yield of proposed spring (Q) = $5 \times 60 \times 60 \times 24$

= 432000 lit/day

$$\text{Per capita demand} = \frac{Q}{P_{20}}$$

$$P_{20} = \frac{432000 \text{ lit/day}}{65 \text{ lpcd}} = 6647 \text{ nos}$$

Let P_o be the current population for the scheme;

$$P_{20} \text{ for the scheme} = p_0\left(1+\frac{1.7}{100}\right)^{20}$$

The current population (P_o) that can take under the scheme is 4744 nos.

Example 3.5

In a rural village, the survey is carried out in the year 2072 BS and the following data is obtained:

Population = 5000 nos
Annual population growth rate = 1.5 %
Annual growth rate for students = 1%
No. of cows = 50 nos
No. of goats = 250 nos
No. of chickens = 2000 nos
No. of schools = 2 with overall 350 day scholars students
No of VDC offices = 2

No. of Tea shops = 3

If the base year is taken as 2075 BS and the design period is 20 years, calculate the total water demand of the village for the service year.

Solution:

Survey year = 2072,

Total water demand in Design year (2095) = ?

For design year, n = Base period + design period

= 3+20 = 23 years

Design year population by geometrical forecasting method;

$$p_n = p_0\left(1+\frac{r}{100}\right)^n$$

$$p_{23} = 5000\left(1+\frac{1.5}{100}\right)^{23} = 7042 \text{ nos.}$$

Similarly, Students population (P_{23}) $= 350\left(1+\dfrac{1}{100}\right)^{23} = 440$ nos.

Calculation of water demand;

a. Domestic demand = 7042 @ 45 lpcd = 316890 lit/day

b. Livestock demand

For Cows = 50 nos @ 45 lit/animal/day = 2250 lit/day

For goats = 250 nos @ 20 lit/goat/day = 5000 lit/day

For chickens = 2000 nos @ 0.20 lit/bird/day = 400 lit/day

Total livestock demand = 7650 lit/day

Comparing livestock demand with 20% of domestic demand;

20% of domestic demand = 63378 lit/day.

Total livestock demand is less than 20% of domestic demand so livestock demand = 7650 lit/day

c. Commercial and Institutional demand

For day scholars students = 440 nos @ 10 lpcd = 4400 lit/day

For VDC office = 2 @ 500 lit/office/day = 1000 lit/day

For Tea shop = 3 @ 500 lit/tea stall/day = 1500 lit/day

Total commercial & institutional demand = 6900 lit/day

Total water demand for design year = DD + LD + ID

= 31689000+ 7650+6900 = **331440** lit/day

Example 3.6

Calculate the water demand for the year 2025 for a rural area of Nepal. Use Geometrical method for population forecasting. Census population is:

Year	1961	1971	1981	1991	2001
Population	8500	10050	14000	18400	22800

Take 1 school with 80 boarders and 300 day scholars. Consider livestock demand also.

Solution:

$$n = \frac{2025 - 2001}{10} = 2.4$$

Present (2001) or last known population (P_o) = 228000

Year AD	Population	Increase in population	% increase in population
1961	8500	-	-
1971	10050	1550	18.24
1981	14000	3950	39.30
2091	18400	4400	31.43
2001	22800	4400	23.91
Total			112.88/4
Average			r = 28.22

Using geometrical method for population forecasting.

$$p_n = p_0\left(1+\frac{r}{100}\right)^n$$

$$p_n = 22800\left(1+\frac{28.22}{100}\right)^{2.4}$$

$$P_3 = 41403 \text{ nos.}$$

Design year (2025) population as forecasted by geometrical method = 41403 nos.

Assume students growth takes place same as population so average percentage increase in student per decade = 28.22 %.

$$\text{Boarders,}\quad p_{2.4} = 80\left(1+\frac{28.22}{100}\right)^{2.4} = 145 \text{ nos.}$$

$$\text{Day schoors,}\quad p_{2.4} = 300\left(1+\frac{28.22}{100}\right)^{2.4} = 545 \text{ nos.}$$

Assume per capita water demand for the rural village as 45 lpcd.

a) Domestic demand = population × per capita demand
= 41403 nos. × 45 lpcd
= 1863135 lit/day

b) Assume livestock demand as 20% of domestic demand
= 0.2 × 1863135
= 372627 lit/day

c) Institutional or commercial demand

For Boarders	=145 nos. @ 65 lpcd	= 9425 lit/day
For day scholars	= 545 nos. @ 10 lpcd	= 5450 lit/day

Therefore, design year water demand
= Domestic demand + livestock demand + institutional demand
= 1863135 + 372627 + 14875
= 2250637 lit/day

Problems

1. What is peak factor? Discuss the various factors affecting per capita demand. Describe the reasons of daily variation of water.
2. Define per capita demand of water. Describe the various types of water demands that should be considered in the urban water supply scheme.
3. What are the major factor governing the selection of design period in water supply scheme?
4. Estimate the population of the Damauli city in the year 2088 BS by four analytical methods. The census population of the city is as follows.

Year	2068	2058	2048	2038	2028
Population	65,500	57,000	47,000	37,000	29,000

[Ans: AM=83750, GM=97963, IIM=84250, DRGM=77193]

5. The population of a city obtained from census report is as given below.

Census year	Population
1968	20,000
1978	22,000
1988	25,000
1998	27,500
2008	34,100
2018	41,500
2028	47,050
2038	54,500
2048	61,000

Estimate the population of the city for the yr 2078 by Arithmetical, Geometrical, Incremental and decreasing growth method.

[Ans: AM=76375, GM=92919, IIM=80233, DRGM=86699]

6. The present population of the Kumveshwor city is 74,850 which was 68,000 ten years ago, 57,500 twenty yrs ago, 42,800 thirty yrs ago, 25,800 fourty years ago, 17,500 fifty years ago. Estimate the population of the city after the next 28 years by geometrical increase method. Also estimate the daily demand of that year.
7. The survey was carried out in 1998 to design a water supply project. The following data were collected during survey, Survey year population = 6580 (b) Total water demand= 60 lpcd (c) Design year = 2018 Estimate the total demand in liters per day at 5 years interval starting from survey year. Assume annual population growth rate as 1%.
8. Calculate total daily water demand of Kharibot VDC for a scheme of design period of 20 years and population growth is 2.2% per year with the help of the following data:
   Population = 9050, Cows = 680, Horses = 40, Chicken = 2500, school = 1 (with 450 day scholar students), Offices = 2 & Health post = 1
9. Determine the daily water demand for the coming year 2020 AD for the Liwang VDC as per following statistical data of 2000 AD.

   No. of households (HHs) = 500
   Average population in HH = 6 Number.
   Annual growth rate = 2.2%
   One health post without bed
   Triratna campus with maximum Capacity 500 and other staff 90 (all day scholars)
   6 numbers of schools with altogether 700 students & staffs (day scholars)
   5000 chickens and 160 animals (Cows, buffalos)
   Other office with altogether 250 personal capacity.

10. Calculate the design year total water demand for a village in Tanahun district. The following data were recorded during the survey year 2065 BS.

Base period	= 3 Yrs
Design period	= 20 Yrs
Survey year population	= 80000
Annual population growth rate	= 1.3%
Office	= 1
Tea shop	= 2
School having students 202 day scholars	
Number of cows	= 30
Number of goats	= 48
Number of chickens	= 225

11. Calculate the water demand for the year 2025 for a rural area of Nepal. Use Geometrical method for population forecasting. Census population is:

Year	1961	1971	1981	1991	2001
Population	8500	10050	14000	18400	22800

Take 1 school with 80 boarders and 300 day scholars. Consider livestock demand also.

12. The average increase in the population of a town per decade over a period of 6 decades was 4100 and the average percentage increase was 12%. If the population at the end of the sixth decade was 220000, estimate the population two decades later by the Arithmetical and Geometrical increase methods.

13. If the discharge of a spring source is 3.9 lps. Calculate the number of present population if it can serve for the design period of 15 years. The population growth over 15 years is 40%.

14. The survey data collected for a water supply scheme in a village of Nepal is given below:

Survey year	= 2013
Base period	= 3 years
Design period	= 15 years
Population	= 250
No. of cows	= 200
No. of goats	= 500
No. of chicken	= 5000
Annual population growth rate	= 1.5%
No. of day scholar in school	= 100
No. of boarders in school	= 10
No. of health post	= 1
No. of tea shop	= 1
VDC office	= 1

Calculate total water demand for design year.

4 Quality of Water

Pure water cannot found in nature even rain water which is pure at the instant of originate contains gases, dust and other substances as it passes through atmosphere. Ultimate or primary source of water is rain water as it comes to earth surface which is already impure passing through atmosphere further picks up organic and suspended matter as of surface run off and portion of rain water which percolates into the ground as ground water dissolves several salts and minerals, organic and inorganic matter while infiltrating into subsurface strata. The water thus tapped from surface and ground source may contain undesirable and excess substances as termed as impurities which may be classified as follows.

4.1 Impurities in water, their classification and effects

Various impurities may be present in water which is classified by two methods as follows.

i) Classification on the basis of properties or characteristics of impurities into,
    a) Physical impurities
    b) Chemical impurities
    c) Bacteriological impurities

a) Physical impurities:

Presence of the physical impurities in water affects the physical characteristics such as colour, odour, taste and turbidity. Colour, odour and taste in water is due to presence of organic matter, minerals, microorganisms etc. Turbidity in water is chiefly due to suspended matters where as colloidal also liable for turbidity.

b) **Chemical impurities**

Presence of chemical impurities in water affect the chemical characteristics of water such as pH, solids, salts of minerals, hardness, alkalinity, chloride, nitrogen etc. Presence of chemical impurities may cause various types diseases.

c) **Bacteriological impurities**

Presence of the bacteriological impurities affects the bacteriological characteristics of water such as pathogenic (Salmonella), non-pathogenic (E-coli) microorganisms. Bacteriological impurities present in water causes diseases to humans.

ii) Classification on the basis of state of presence/occurrences into;

a) Suspended impurities
b) Dissolved impurities
c) Colloidal impurities

a) **Suspended impurities**

These impurities are dispersion of solid particles resulting turbidity in water. Suspended impurities include silt, clay, algae, fungi, organic and inorganic matters, mineral matter etc. These impurities remain in suspension due to same specific gravity as that of water. Suspended impurities are macroscopic and cause turbidity in water. The concentration of suspended matter in water is measured by turbidity. Suspended impurities can be removed by settling or by filtration.

b) **Dissolved impurities**

Water is a very good solvent and can dissolve all the salts to which it comes in contact. The dissolved impurities may contain organic compounds inorganic salts, gases etc. The amount of dissolved solids is normally expressed in ppm. Salt of calcium and magnesium in water causes bad taste, hardness, alkalinity etc. Iron oxide and manganese when dissolved cause odor, taste, red or black or brown color, produce stain's on cloth in laundries and plumbing fixtures in buildings. Gases likes O_2 and CO_2 causes corrosiveness and H_2S causes smell or rotten egg. Dissolved impurities is in liquid having only one phase so such impurities can be remove only by phase change such as precipitation, adsorption, distillation.

c) **Colloidal impurities**

These impurities are so small that these cannot be removed by ordinary filter and are not visible to the naked eye. These are dispersion of particles in water with electrically charged and remain in continuous motion and do not settle due to same charge. These colloidal impurities are generally associated with organic matter containing bacteria and are the chief source of epidemics. Color in water is normally due to colloidal impurities. Their quantity is determined by color tests. Their quantity is determined by color tests. The size of colloidal impurities is between 10^{-3} mm to 10^{-6} mm. Colloidal can be remove from water by coagulation and sedimentation than filtration.

Table 4-1 Impurities in water, their Causes and Effects

Type	Cause	Effects
Suspended Impurities	Bacteria	Some cause disease
	Algal and Protozoa	Colour, odour, taste and turbidity
	Clay and silt	Turbidity
Dissolved impurities	(a) Salts of calcium and Magnesium	
	Bicarbonate	Hardness & alkalinity
	Carbonate	Hardness & alkalinity
	Sulphide	Hardness
	Chloride	Hardness
	(b) Salts of Sodium	
	Bicarbonate	Softening & alkalinity
	Carbonate	Softening & alkalinity
	Sulphide	Dental flurosis or mottled enamel
	Chloride	Taste
	(c) Metals and Compounds	
	Iron oxide	Taste, red colour, hardness and corrosiveness
	Manganese	Black or brown colour
	Lead	Cumulative poisoning
	Arsenic	Toxicity
	Barium	Toxic effect on heart and nerves
	Cadmium	Toxic and illness
	Cyanide	Fatal
	Boron	Affects central nervous system
	Selenium	Highly toxic to animals and fish
	Silver	Discoloration of skin
	Nitrate	Blue baby disease
	(d) Gases	
	Oxygen	Corrosiveness
	Carbon dioxide	acidity and corrosiveness
	Hydrogen Sulphide	Strong odour, acidity and corrosiveness
Suspended Organic impurities	Vegetable	Colour, taste and acidity
	Animal (dead)	Harmful disease germs and alkalinity
Dissolved organic impurities	Vegetable	Produce bacteria
	Animal (dead)	Pollution of water and disease germs

4.2 Hardness and alkalinity

Hardness is that characteristic of water which prevents the formation of sufficient lather or foam with soap. The hardness of water is caused by presence of bicarbonates, sulphates, chlorides and nitrates of calcium and magnesium. The excess amount of hardness in water may be unfit for use of water because it interferes during use of soap, in boilers, in dyeing industry by modifying colour, life of pipe due to corrosion, taste of food etc.

4.2.1 Types of hardness

There are two types of hardness:

a) **Temporary or Carbonate hardness (CH)**: The hardness caused by the presence of bicarbonates of calcium and magnesium is known as carbonate hardness. The carbonate hardness is also known as temporary hardness and it can be removed by boiling the water or by adding lime to the water.

b) **Permanent or Non-carbonate hardness (NCH):** The hardness caused by the presence of sulphates, chlorides and nitrates of calcium and magnesium is known as non-carbonate hardness. The non-carbonate hardness is also known as permanent and it cannot be removed by simple boiling the water. Hardness is expressed in terms of equivalent amount of $CaCo_3$ as;

Hardness due to M^{++} as $CaCo_3$,

$$= \text{hardness as } M^{++} \text{ (mg/lit)} \times \frac{\text{eq. wt. of } CaCo_3}{\text{eq. wt. of } M^{++}}$$

(The equivalent weight of $CaCO_3$ = 50, Ca^{++} = 20, Mg^{++} = 12.2, Sr^{++} =43.8, CO_3^{--}=30, HCO_3^{-} = 61)

4.2.2 Types of alkalinity

Alkalinity is caused by hydroxides, carbonates and bicarbonates but most natural alkalinity is due to bicarbonates. Alkalinity caused by hydroxides is called hydroxide alkalinity or caustic alkalinity, caused by carbonate is carbonate alkalinity and caused by bicarbonate is called bicarbonate alkalinity. Some but not all the compounds causing alkalinity also causes hardness. Bicarbonate alkalinity is chief form of alkalinity so that it formed in considerable amount from the action of carbon dioxide upon the basic material in the soil. Normally carbonate and hydroxide alkalinities may be present with bicarbonate alkalinity or hydroxide alkalinity. These bicarbonate and hydroxide alkalinity do not exist together in water. So, total alkalinity (TA) is sum of carbonate alkalinity (CA) and bicarbonate alkalinity (BCA).

$$\text{Alkalinity due to } B^- \text{ as } CaCO_3 = \text{alkalinity as } B^- \times \frac{\text{eq. wt. of } CaCO_3}{\text{eq. wt. of } B^-}$$

Alkalinity as CO_3 in (mg/lit)	= CA in mg/lit
Alkalinity as HCO_3^-	= BA in mg/lit
Total alkalinity (A)	= BA+CA

4.2.3 Relation between hardness and alkalinity

If total hardness (TH) is greater than total alkalinity (A), then carbonate hardness (CH) is equal to total alkalinity (A) and non carbonate hardness (NCH) is equal to total hardness minus carbonate hardness as given by following expression;

If, $TH > A$

then, $CH = A$

$$NCH = TH - CH = TH - A$$

If total hardness (TH) is less or equal to total alkalinity (A), then carbonate hardness (CH) is equal to total alkalinity (A) and non carbonate hardness (NCH) is equal to zero as given by following expression;

If, $TH \leq A$

then, $CH = TH$

$NCH = 0$

4.3 Living organisms in water

The water available at the source may contain various types of living organisms. Some of the organisms are water borne and remain in water due to their natural habit. Some of the living organisms such as bacteria, viruses and protozoa are infectious to human and are responsible for the serious outbreak of fatal water-borne diseases. The following are the main living organism of water;

4.3.1 Algae

These are simple photosynthetic unicellular aquatic plants. They grow abundant in nature in places of excreta disposal. Sometimes these grow in numbers and cover the surface of a body of water. They cause turbidity and apparent colour in water. They, sometimes, cause trouble by clogging filters. Growth of algae could be controlled by adding cupper sulphate or chlorine in water.

4.3.2 Bacteria

Bacteria are single-cell microorganisms, usually colourless, and are the lowest form of life capable of synthesizing protoplasm from the surrounding environment. They are prokaryotic, unicellular, and either free-living in soil or water or parasites of plants or animals. The bacteria usually found in water range from 1 to 4 microns in length. Reproduction of bacteria is normally by cell division of fusion. The original cell divides into two equal parts and each part of the cell develops to full size bacteria. Almost bacteria get developed in the intestine of warm-blooded animal. These come out through the excreta. In a day single person passes out 10^9 to 10^{11} bacteria. A well fit person gives off non pathogenic bacteria and sick (infected) person gives a pathogenic bacteria. These bacteria are bacilli type and are aerobic or facultative in nature. Coli form bacteria is a bacteria which forms gas in lactose at 30-35 ^{0}C within 48 hours. It is also called an indicator bacteria weather the water is contaminated or not. These coli form bacteria are of two types;

i) Escherichia coli (E-Coli): a species of rod-shaped, facultative anaerobic bacteria in the large intestine of humans and other animals, sometimes pathogenic.
ii) Aerobaticaerogenes: From soil, vegetable

Chlorine is effective to kill bacteria but when chlorine is used as a drinking water disinfectant it must be removed first before consumption as chlorine has adverse health effects.

4.3.3 Viruses

These are the smallest biological structures known to contain all the genetic information necessary for their own reproduction. They can be defined as a group of infectious agents which are smaller than ordinary bacteria. Water borne viral pathogens are known to cause:

Polio : It causes poliomyelitis to children.
Coxsackic : It affects our throat i.e. in breathing.
Hepatitis : It causes a type of jandis.
Echo : it causes meningitis, diarrhea
(Enteric Cytopathogenic Human Orphan)

4.3.4 Worms/Helminths

The life cycles of worms often involve two or more animal hosts, one of which can be human and water contamination may result from human or animal waste that contains worms. These are microscopic as well as macroscopic and can enter directly to the human body through skin or on drinking of water. They may be parasitic as well as free living. Hook worms, tape worm etc are parasite. They are classified as;

a) Nematodes or round worms
b) Rotifers
c) Flat worm
   i) Tape worms (Certodes)
   ii) Flukes (Trematodes)

Worms cause schistosomiasis.

4.4 Water related disease

To be fit and healthy, also to survive, requirement of water is essential nutrient to every human being in some form and the body disposes wastes as excreta or feces and urine. The improper disposals of human waste may contaminate water and unsafe for drinking. Safe water is essential to control of many diseases. These are the disease carried by water not necessity by drinking. Generally water related diseases are classified into four categories.

4.4.1 Water Borne Disease

Waterborne diseases are caused by pathogenic microorganisms that most commonly are transmitted through contaminated water. Infection commonly results during bathing, washing, drinking, in the preparation of food, or the consumption of food thus infected. Disease due to consumption of water containing impurities is called water borne disease. Water borne disease is also known as water quality disease. This is due to; presence of chemicals e.g. iron, lead, arsenic etc. and presence of microorganism like bacteria, virus, worms etc potentially water borne disease include the classical infections, notably.

Cholera: It is transferred due to bacteria known as vibrio comma (cholerae) through faeces of cholera suffered person. The victim vomits and passes extremely watery stools.

Typhoid fever: Transformed by means of bacteria 'Salmonella' but also include a wide range of other disease such as infectious hepatitis, diarrhea and dysentery etc. It is characterized by continued fever, headache, slow heart beat and malaise.

Diarrhea: It is an intestinal disorder characterized by abnormal frequency and fluidity of fecal evacuations. The most common cause is an infection of the intestines due to either a virus, bacteria, parasite, or a condition known as gastroenteritis. These infections are often acquired from food or water that has been contaminated by stool (feces), or directly from another person who is infected.

Dysentery: It is an infectious disease marked by inflammation and ulceration of the lower part of the bowels, with diarrhea that becomes mucous and hemorrhagic.

4.4.2 Water washed diseases

For the transmission of these diseases we need not necessarily to drink water. It gate transmitted due to lack of cleanness, poor sanitation and education. It depends on the quantity of water used, rather than the quality. Bacterial skin sepsis, scabies, and fungal infections of the skin are extremely prevent in many hot climates, while eye infections such as trachoma is also common and may lead to blindness. These infections are related to poor hygiene and it is to be anticipated that they will be reduced by increasing the volume of water used for personal hygiene.

Scabies: It is a contagious skin disease occurring especially in sheep and cattle and also in humans, caused by the itch mite, Sarcoptes scabiei, which burrows under the skin.

Trachoma: It is a chronic contagious disease of the eye characterized by inflammation of the conjunctiva and cornea and the formation of scar tissue, caused by infection with the virus-like bacterium Chlamydia trachomatis.

Bacillary Dysentery (Shigellosis): It is an acute intestinal infection caused by a bacterium of the genus Shigella. Dysenteriae is common among children and characterized by fever, abdominal pain, and diarrhea.

4.4.3 Water based disease

A water based disease is one in which pathogens spends a part of its life cycle in water like snail or other aquatic animal. These diseases are also known as water contact disease. All these disease are due to infection by parasitic worms (helminths) which depend on aquatic intermediate hosts to complete their life cycles. The degree of sickness depends upon the number of adult worms which are infecting the patient and the importance of disease must be measured in terms of the intensity of infection as well as the number of people infected. An example is schistosomiasis (snail fever) in which water polluted by excreta, contain aquatic snails and another water based disease is guinea worm.

Schistosomiasis: It is an infection caused by parasitic flukes of the genus Schistosoma, occurring commonly in eastern Asia and in tropical regions and transmitted to humans through feces-contaminated fresh water or snails: symptoms commonly include pain, anemia, and malfunction of the infected organ.

4.4.4 Water vector disease

Water vector diseases spread by insects which either breed in n water or bite near water. The transmission of these disease is complex so that it involve at least three living things; a host, a parasite and a carrier or vector (insect, fly or mosquito). These diseases are also known as water site insect carried diseases. Malaria, yellow fever, dengue, onchocerciasis (river blindness) are example of water vector disease.

Malaria: An infectious disease characterized by cycles of chills, fever, and sweating, caused by a protozoan of the genus Plasmodium in red blood cells, which is transmitted to humans by the bite of an infected female anopheles mosquito.

Onchocerciasis (river blindness): A disease caused by infestation with filarial worms of the genus Onchocerca. Volvulus and characterized by nodular swellings on the skin and lesions of the eyes. Transmitted by black flies, the disease occurs in tropical regions of Africa and Central America. It is also called river blindness.

Dengue: It is an infectious, eruptive fever of warm climates, usually epidemic, characterized especially by severe pains in the joints and muscles. This is transmitted to humans by virus through mosquito bites.

4.4.5 Transmission route and preventive measures

Transmission of water related disease is related with WATSAN (Water supply and Sanitation) due to inadequate and improper disposal of waste and unsafe drinking water. Most of water related disease is spread by pathogens present in human excreta as shown in figure 4-1 Most of the people get infected when the contaminated material enters their mouth. Other possible modes of transmission include:

- Dirty contaminated hands, clothes, cooking vessels, mugs, etc.
- Uncovered food and drinking water
- Contaminated water
- The practice of defecating in the open
- Via flies

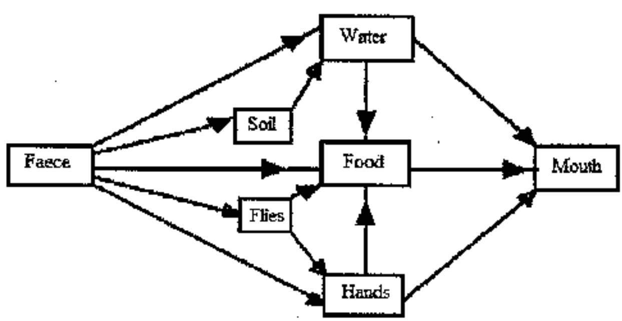


Figure 4-1 Fecal Oral transmission Routes

4.4.6 Preventive measures

For the suitable preventative measures it is essential to identify the transmission mechanisms. Certain barriers or interventions should be employed in the transmission route to interrupt transmission of diseases. In order to prevent infectious water borne diseases, it is important to take necessary precautions. The quality of water should be improved at the source itself. In certain areas, the quality of water supply might be of question. In such cases, it is necessary to disinfect the water before use. Water that is used for all purposes like drinking, cooking, and brushing of teeth should be disinfected properly. The common household ways to avoid water borne diseases by disinfection include:

(i) Vigorously boiling water for one minute can kill most microorganisms.
(ii) Common household items such as chlorine bleach, tincture of iodine, and iodine tablets can be used to disinfect water.

Another important measure that should be taken to avoid the spreading of pathological microorganisms is the interruption of routes of transmission such as protecting food from flies, chlorination of water, and maintaining proper sanitation, etc. It is vital to change ensure proper hygiene in order to avoid waterborne diseases.

- Drink only filtered water.
- Wash hands properly before eating.
- Eat cooked, warm foods.
- Keep your fingernails short and clean.
- Use of proper toilets for defecation.
- Wash food before cooking and cook food at high temperature so as to kill harmful bacteria.
- Avoid flies by disposing animal and organic wastes properly.

- Ensure to take proper care in disposing of infant and toddler feces.
- Avoid consuming foods, fruit juices, and milkshakes from roadside vendors.
- Always keep foods and beverages closed.
- Avoid drinking water at parks and other such recreational places. It is best to buy bottled water or carry your own water.
- Washing hands is the most important method of prevention of waterborne diseases. One should wash hands before preparing food and before eating. Likewise, it is necessary to wash hands after using the toilet, changing diapers, after using handkerchief, after changing clothes or beddings soiled with feces, after caring people with water borne illness, and after playing with pets and animals.

4.5 Examination of water

The examination of water is also known as water analysis which is done in systematic manure to identify the water quality or the presence of impurities in water. The water analysis or examination is conducted for the various purposes as listed below.

- to ascertain the quality of water & quantity of various impurities.
- to verify the treated water quality is as per standard or not.
- to identify the dose of chlorine, coagulant etc in treatment plants
- to prescribe the degree of treatment for the required water quality

Sampling techniques of water

1. Fully cleaned bottles or buckets or jarricans can be used for physical test but container should be greater than capacity of 2 liter for biological and chemical test.
2. Container should be rinse before sampling.

3. Sample should be well shake before test so certain air in container should be remain.
4. Tap should be let opened for two to three minutes to avoid stagnant water during the sample collection from distribution lines.
5. When sample is taken from well with hand pump, pump water for about 5 minutes then collect it into the clean and sterilize container. If there is no hand pump, take the well water directly from well.
6. When sample collection is from river, reservoir, lake, water sample should be taken from below 40 to 50 cm below the surface and it should not be so far or near draw-off site.
7. Sample should be tested within 1 hour of sample collection in case of microbiological test, if it is not possible sample should be kept in ice-chest or in cooler till 24 hours.

4.5.1 Physical Examination of water

a) Temperature

Temperature of water is determined by ordinary thermometers. Temperature of surface water is equivalent to the atmosphere where as that of ground water may be slightly more or less than that of atmospheric temperature. Temperature of water has an effect on the physical properties of water such as density, viscosity, surface tension, saturation value of gases dissolved in water, biological activity. For public water supply it should be between 10 ^{0}C to 15.6 ^{0}C. Temperature greater than 25 ^{0}C is undesirable and above 35 ^{0}C unfit for public water supply.

b) Colour

Pure water is colorless, but water in nature is often closed by foreign substance. It is measured by the ability of the solution to absorb light. Water having partly color due to suspended matter is said to have apparent colour. Colour contributed by dissolved solids that remain after removal of suspended matter is known as true colour. While true colour in water is due to dissolved materials only. The organic compounds casing true colour may exert a chlorine demand thereby seriously reduce the effectiveness of chlorine as a disinfectant. Colour is measured by tintometer. The intensity of colour is measured on platinum-cobalt scale. One milligram of potassium plus half milligram of metallic cobalt dissolved in 1 liter of distilled water is one true colour unit. Coloured water is aesthetically objectionable for drinking purpose. For drinking water colour should not be greater than 5 ppm in platinum cobalt scale. Greater than 5 is tolerable but rejected greater than that 25 ppm in platinum cobalt scale.

1 platinum cobalt scale = 1 ppm = 1 mg/lit

c) **Turbidity**

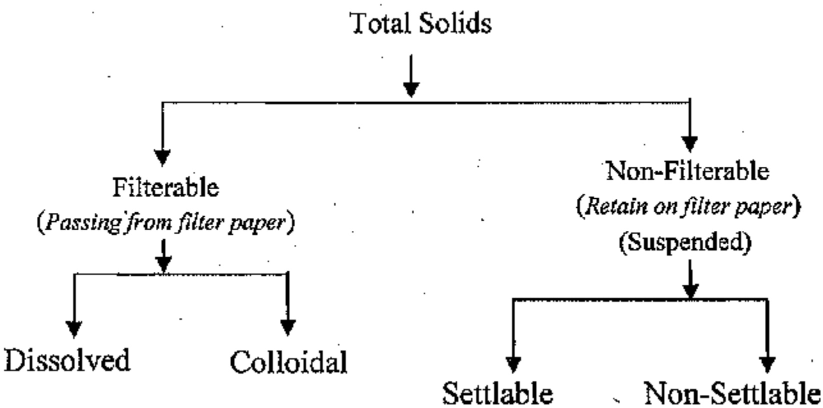
Turbidity is measure of the extent to which light is either absorbed or scattered by suspended material in water. Absorption and scattering are influenced by both size and surface characteristics of the suspended material; turbidity is not a direct quantitative measurement of suspended solids. For example, one small pebble in a glass of water would produce virtually no turbidity. If this pebble gets crushed into thousands of particles of colloidal size, a measureable turbidity would result. Turbidity is measured photometrically by determining the percentage of light of a given intensity that is either absorbed or scattered. The original measuring apparatus called a jakson turbidity meter, was based on light absorption and employed a long tube and standardized candle. The glass tube was calibrated with readings one JTU being equal to the turbidity produced by 1 mg Sio_2, in 1 liter of distilled water. Turbidity of 5 ppm is accepted and rejected if it is greater than 10 NTU.

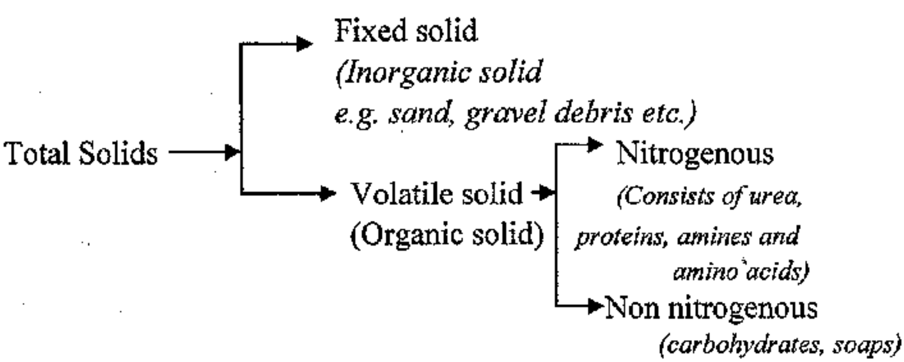
4.5.2 Chemical examination of water

The chemical examination of water involves the tests which are undertaken to determine the chemical impurities and the corresponding chemical characteristics of water.

Total solid

The solids present in water may be either dissolved or suspended solids and sum of these suspended and dissolved solids is total solids. The solids present in water is generally expressed in ppm or mg/lit.





The water sample when kept into oven for evaporation at 103°C- 105 °C for 24 hrs obtained residue is total solids. Further residue obtained if ignited in muffle furnace at 600°C for 15-20 minutes, volatile solids escape out and obtained solid is fixed or inorganic solid.

pH

It is the symbol for the logarithm of the reciprocal of hydrogen ion concentration in gram atoms per liter, used to express the acidity or alkalinity of a solution on a scale of 0 to 14, where less than 7 represents acidity, 7 neutrality, and more than 7 alkalinity. It is determined by electrometric and colorimetric method. The value of pH drinking water as per National drinking water quality standards (NDWQS) of Nepal is 6.5 to 8.5.

4.5.3 Biological examination of water

Pathogenic bacteria are difficult to detect in water because of their presence in small numbers. Even in polluted water their presence infrequently or at irregular intervals. Hence analysis of water for all the known pathogens would be very time consuming and expensive proposition. Test for specific pathogens are usually made only when there are reasons to suspect that particular organism is present. At other times, the purity of water is checked using **indicator organism**. An indicator organism is one whose presence presumes that contamination has occurred and suggests the nature and extent of the contaminant.

Generally both harmful and harmless bacteria occur together in water and difficult to detect pathogenic bacteria because their presence may be irregular and less in numbers even in polluted water. Non-pathogenic bacteria (E-coli) are largely found in the large intestine of human and animals and execrated with their faeces. The evidence of presence of pathogenic bacteria is indirectly obtained by testing those coli forms are called indicator organisms. The absence of E-coli almost justifies the assumption that the faecal polluted water is free from pathogen.

Escherichia coli (E-coli) are the coliform bacteria which inhabit the intestine of human beings and animals and are thus excerted in large amount with their faeces. As such the water which has been contaminated with sewage will contain E-coli bacteria. For detecting the presence of bacteria of coliform group and measuring their concentration in water the following methods are adopted.

a) **Total count or agar plate count test**

It is also known as ***standard plate count***. In this method 1 ml of water sample is diluted in 99 ml of sterilized/distilled water and diluted 1 ml water is mixed with 10 ml of agar gelatin (a culture medium used to cultivate bacteria) and incubated at 37 ^{0}C for 24 hrs or 48 hrs at 20 ^{0}C. The bacterial colonies which are formed, are then counted and the results are computed per 100ml. For drinking water the total count should not exceed 1 per 100 ml.

b) **Multiple tube fermentation technique/E-coli test**

This test is divided into the following these parts;

i) Presumptive test
ii) Conform test
iii) Completed test

i) **Presumptive test**

The presumptive test is based on ***the ability of coliform group to ferment the lactose broth and producing gas***.

Procedure: (1) Definite amount of diluted samples of water are taken in multiples of 10, such as 0.1 ml, 1.0 ml, 10 ml etc. (2) The water sample is placed in standard fermentation tubes containing *lactose broth* which is incubated at temperature of 37 ^{0}C for a period of 48 hrs. If gas is seen in the tube after this period, it indicates the presence of E-coli group and result of test is positive. If no gas is seen, it indicates the absence of E-coli group and the result of text is treated as –ve.

ii) **Conformed test**

The conformed test consists of **growing cultures of coliform bacteria as media which suppress the growth of other organism**. The gas produced in presumptive test does not conform the presence of bacteria of coliform group because these may be other bacteria present which also ferment lactose. So a portion of water from presumptive test is taken and placed in another fermentation tube containing *brilliant green lactose bile* as culture medium. It is again kept in incubator at 37 ^{0}C for 48 hrs, the evolution of gas in these tubes would conform the presence of the organisms of the coliform group and vice-versa. Colonies of bacteria indicates the presence of E-coli and completed test is necessary.

iii) **Completed test**

This test is based on the **ability of the culture grown in the conformed test to again ferment the lactose broth**. Colonies of bacteria grown in conformed test are kept into lactose broth fermentation tubes and agar tubes. The tubes are kept for incubation at 37 ^{0}C for 24 hrs to 48 hrs. If gas seen in tubes, it indicates the presence of E-coli group and the result of the test is treated as positive and further detailed tests are carried out to detect the type of bacteria present in water. Again the absence of gas indicates negative result and water is safe for drinking.

c) **Membrane filter technique**

The bacteria present in water are retained on the membrane having microscopic pores. The membrane with the bacteria is then put in contact with a suitable nutrient (M-Endo's medium) which inhabits the growth of bacteria other than the coliform group. It is than placed in an incubator at 37 ^{0}C for a period of 20 hrs. The bacteria of coliform group if present in water are developed into visible colonies which can be counted with the help of microscope.

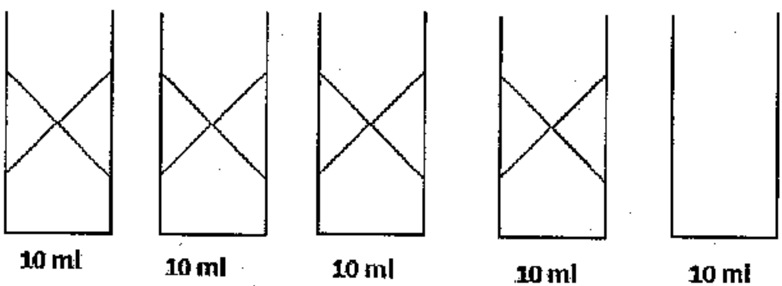
$$\text{coliform colony/100 ml} = \frac{\text{colony counted}}{\text{ml of sample}} \times 100$$

Most probable number index or MPN

MPN is defined as bacterial density which is based on the laws of probabilities and statistics and hence it provides a more rational approach for indicating the concentration of E-coli in water from the multiple tube fermentation technique.

$$\text{MPN per 100 ml of sample} = \frac{100 \times \text{nos. of positive test tubes}}{\sqrt{\text{ml in all portions} \times \text{ml in negative portions}}}$$

Example:



Number of positive portion = 1
Total volume in all tubes = 10 x 5 = 50 ml
Total volume in –ve portion = 10 x 4 = 40 ml

$$\text{MPN} = \frac{100 \times 1}{\sqrt{50 \times 40}} = 2.24 \text{ nos. per 100 ml sample}$$

4.6 Water quality standard for drinking purpose

Water available in source may contain many harmful constituents i.e. various impurities at various concentrations. Presence of higher concentration of impurities in drinking water cause disease but presence of some minerals in water may be beneficial to human beings which should be within a limit. The maximum concentration limit of impurities in water at which it is not harmful to human health is termed as water quality standards. Drinking water quality standards describes the quality parameters set for drinking water. Every human on this planet needs drinking water to survive and that water may contain many harmful constituents, there are no universally recognized and accepted international standards for drinking water. Where drinking water quality standards do exist, most are expressed as guidelines or targets rather than requirements, and very few water standards have any legal basis or, are subject to enforcement. National drinking water quality standards (NDWQS) of Nepal came into in the effective since 2005 which is shown in Table 4-1.

Table 4-1 Nepal's Drinking Water Quality Standards

Parameter	Unit	Maximum Concentration Limits
Turbidity	NTU	5 (10)**
pH		6.5-8.5*
Color	TCU	5 (15)**
Taste & Odor		Would not be objectionable
Total Dissolved Solids	mg/l	1000
Electrical Conductivity	µc/cm	1500
Iron	mg/l	0.3 (3)**
Manganese	mg/l	0.2
Arsenic	mg/l	0.05
Cadmium	mg/l	0.003
Chromium	mg/l	0.05
Cyanide	mg/l	0.07
Fluoride	mg/l	0.5-1.5*
Lead	mg/l	0.01
Ammonia	mg/l	1.5
Chloride	mg/l	250
Sulphate	mg/l	250
Nitrate	mg/l	50
Copper	mg/l	1
Total Hardness	mg/l	500
Calcium	mg/l	200
Zinc	mg/l	3
Mercury	mg/l	0.001
Aluminum	mg/l	0.2
Residual Chlorine	mg/l	0.1-0.2*
E-Coli	MPN/100ml	0
Total Coli form	MPN/100ml	95 % in sample

Note: * These standards indicate the maximum and minimum limits.
** Figures in parenthesis are upper range of the standards recommended.

Example1:

The analysis of water from a well showed following results in mg/lit;

$Ca = 65$, $Mg = 51$, $Na = 101.5$, $K = 21.5$,

$HCO_3 = 248$, $SO_4 = 221.8$, $Cl = 79.2$

Find the total hardness, carbonate hardness and non-carbonate hardness.

Solution:

Hardness due to M^{++} as $CaCo_3$,

$$= \text{hardness as } M^{++} \text{ (mg/lit)} \times \frac{\text{eq. wt. of } CaCo_3}{\text{eq. wt. of } M^{++}} \text{ (mg/lit)}$$

$$\text{Total hardness} = \left[65 \times \frac{50}{20} + 51 \frac{50}{12.2}\right] \text{ mg/lit as } CaCO_3$$

$$= 371.52 \text{ mg/lit as } CaCO_3$$

$$\text{Alkalinity due to } B^- \text{ as } CaCO_3 = \text{alkalinity as } B^- \times \frac{\text{eq. wt. of } CaCO_3}{\text{eq. wt. of } B^-}$$

$$= 248 \times \frac{50}{61} = 203.28\ mg/lit$$

In this case,

$\underline{TH > A}$

$CH = A \quad = 203.28$ mg/lit as $CaCo_3$

$NCH = TH - CH \quad = 168$ mg/lit

Example: 2

The analysis of a sample of water shows the following results in mg/lit:

$Ca = 7$, $Mg = 12$, $Na = 20$, $K = 30$,

$HCO_3 = 68$, $So_4 = 7$, $Cl = 40$, $No_3 = 12$

The concentration of Sr (Stroncium) is eq. to hardness of 2.52 mg/lit as $CaCo_3$ and the carbonate alkalinity in this water is zero. Calculate the total hardness and NCH in mg/lit as $CaCO_3$.

Solution: $\quad \text{Total hardness} = \dfrac{Ca^{++} \times 50}{20} + \dfrac{Mg^{++} \times 50}{12.2} + 2.52$

$$= \frac{7 \times 50}{20} + \frac{12 \times 50}{12.2} + 2.52$$

$$= 69.2 \text{ mg/lit as } CaCO_3.$$

Bicarbonate alkalinity = $68 \times \frac{50}{61}$ = 55.74 mg/lit as $CaCO_3$.

In this case,

$\underline{TH \quad > A}$

CH = A = 55.74 mg/lit as $CaCO_3$

NCH = TH – CH = 69.2-55.74

= 13.46 mg/lit as $CaCO_3$

Example 3:

The total hardness value obtained from the complete analysis of a water sample is found to be 150 mg/lit. The analysis further showed that the concentrations of all the three principal cations causing hardness are numerically the same. If value of carbonate hardness is 77 mg/lit, calculate;

i) the value of NCH

ii) the concentrations of principal cations; and

ii) the value of total alkalinity in mg/lit.

Solution:

Total Hardness =

Carbonate hardness + Non Carbonate Hardness

NCH = 150 – 77 = 73 mg/lit

Let, P is the concentration of principle cations.

Now, $$TH = \frac{P \times 50}{20} + \frac{P \times 50}{12.2} + \frac{P \times 50}{43.8}$$

Principle concentrations (P) = 19.38 mg/lit

In this case NCH is greater than zero so total hardness is greater than alkalinity.

When, TH $> A$

CH $= A$ $= 77$ mg/lit as $CaCO_3$

NCH $= TH - CH$

Example: 4

The hardness of a water sample was found to be 300 mg/lit as CaCO3. The hardness was found due to Ca and Mg ions are equal in water. The analysis showed the concentration of HCO_3, was 150 mg/lit.

i) the concentration of Ca and Mg

ii) the value of alkalinity of water.

ii) the CH and NCH; and

Solution:

i) Let, P is the concentration of principle cations.

Hardness due to M^{++} as $CaCo_3$,

$$= \text{hardness as } M^{++} \text{ (mg/lit)} \times \frac{\text{eq. wt. of } CaCo_3}{\text{eq. wt. of } M^{++}} \text{(mg/lit)}$$

$$\text{Total hardness} = \left[P \times \frac{50}{20} + P \times \frac{50}{12.2}\right] \quad \text{mg/lit as } CaCO_3$$

$$= 6.6\,P$$

$$300 = 6.6\,P$$

Therefore, $P = 45.45$ mg/lit

ii) Bicarbonate alkalinity

$$\text{Bicarbonate alkalinity} = 150 \times \frac{50}{61} = 122.95 \text{ mg/lit as } CaCO_3$$

Total alkalinity = CA + BA = 0 + 122.95 = 122.95 mg/lit

iii) Here, Total hardness > Total alkalinity

Carbonate hardness = Total alkalinity = 122.95 mg/lit

Total Hardness =

Carbonate hardness + Non Carbonate Hardness

NCH = 300 – 122.95 = 177.05 mg/lit as $CaCO_3$

Example: 4

If 400 ml of water with a pH 6 is mixed with 700 ml of water with a pH of 8, what will be the resultant pH of the mixture?

Given,

$V_a = 400$ ml, $(pH)_a = 6$

$V_b = 700$ ml, $(pH)_b = 8$

Total volume of solution = 1100 ml

$[H^+] = \text{antilog}_{10}[-pH]$, $pH = \log_{10}\dfrac{1}{[H^+]}$

$[H^+]_a = 10^{-6}$ moles/lit, $[H^+]_b = 10^{-8}$ moles/lit

Conc. of $[H^+]_a$ in 400 ml $= \dfrac{10^{-6}\times 400}{1100} = 3.64\times 10^{-7}\ \textit{moles/lit}$

Conc. of $[H^+]_b$ in 700 ml $= \dfrac{10^{-8}\times 700}{1100} = 6.36\times 10^{-9}\ \textit{moles/lit}$

Concentration mix $= 3.64 \times 10^{-7} + 6.36 \times 10^{-9}$

$= 3.7036 \times 10^{-7}$ moles/lit

pH $= 6.43$

Problems

1. Mention the various impurities in water which should be taken into account in deciding the portability of water. Describe various types of water related diseases.
2. Write the common impurities mostly found in natural water. And briefly explain their effects on water quality.
3. What are indicators organisms? Describe in detail about the membrane tube fermentation technique for the determination of E-coli in laboratory.
4. What are E-coli? Are they harmful to human beings? Why is their presence tested in water for drinking purpose? Describe the membrane filtration technique method.
5. Describe different types of water borne diseases, their transmission mechanism and preventative measures in brief.
6. Describe fecal-oral transmission route of diseases with a neat schematic diagram.
7. What is turbidity? Describe the various tests conducted for physical examination of water.
8. What do you understand by MPN? Describe the multiple tube fermentation method of determining coliforms in the laboratory.
9. There are three samples A, B and C of water having pH values of 4.4, 5.4 and 6.4 respectively. Calculate how many times sample A is acidic than sample C. [100 times]
10. The total hardness value obtained from the complete analysis of a water sample is found to be 330 mg/lit. The analysis further showed that the concentrations of the calcium and magnesium cations causing hardness were found equal. If value of carbonate hardness is 90 mg/lit, calculate; i) the concentrations of calcium and magnesium, ii) the value of total alkalinity in mg/lit.
11. The total hardness value obtained from the complete analysis of a water sample is found to be 117 mg/lit. The analysis further showed that the concentrations of all the three principal cations causing hardness are numerically the same. If value of carbonate hardness is 57 mg/lit, calculate;
    i) the value of NCH
    ii) the concentrations of principal cations; and
    ii) the value of total alkalinity in mg/lit.

5 Intakes

5.1 Definition

Intake is a device or a structure installed in the water source to permit the withdrawal of water and discharge it into an intake conduct then to the treatment plant. It consists of openings, grating or strainers, valve, operating devices, pump, a structure or housing to support intake conduct, an intake conduct etc.

The main function of the intakes works is to collect water from the water source and then discharge water so collected, by means of pumps or directly to the treatment plant.

The basic function of intake is;

- to ensure required water
- to reduce sediment entry
- to check trash and debris entry along with water entering
- to prevent entry of ice
- to secure entry of water with minimum disturbance

5.2 Site selection of an intake

1. Location of intake site should be as far as possible near the treatment plant so that cost of conveying would be less.
2. Water quality available in intake site should be high which will reduce treatment cost. There should not any disposal point of wastewater in upstream of intake.
3. Intake site should not locate near navigation channel due to chances of pollution of water.

4. Intake site should be located so as to ensure supply in worst condition. Intake to fetch water from deeper portion of the river and penstock may be kept two or more to take water in dry season.
5. Intake site should be located such that sufficient future extension and additions.
6. The intake site should be easily accessible even during flood.
7. Intake site should not locate in meandering. It should locate on the concave or outer bank so that water available in all times.
8. Intake site should be located in geologically stable and free from possibilities of erosion, silting, scouring and heavy current.
9. The intake site should be well connected by good approach road.
10 In the selection of intake site the natural cause such as seasonal variations, winds, currents, climate etc should be studied to ensure sustainability of intake works.

General preventive measures that should be considered during design of intake structures

Proper design of intake is required for efficient and effective work. Following are the factors which are to be considered in the design of intakes;

a) Factor of safety
b) Foundations
c) Protections of sites
d) Screens and strainers
e) Self weight
f) Size and number of inlets

a) **Factor of safety:** The intake structure should be designed with sufficient factor of safety so that it can effectively resists the external forces due to heavy waves and currents, ice pressures, impact of floating objects etc.

b) **Foundation:** The design depth of foundation of intake should be sufficient so that intake could be prevented from possible damage by the current of water.

c) **Protection of sides:** During flood boulders may enter to intake and may be damaged so its sides should be protected by a cluster of piles.

d) **Screens and strainers:** To avoid the entry of floating matters and fish in intake channel screens and strainers are provided. If screenings allow in conduit that may clog or damage the pumps, valves etc. and interfere during treatment works.

e) **Self weight:** The intake should be of adequate self-weight so that the chances of its floating or washing by the upthrust of water may be minimized. It is essential to construct the intake structures with masonry work and broken stones should be filled in the bottom to grant additional safety.

f) **Size and numbers of inlets:** Water pool level may vary season to season so adequate size and numbers of inlets should be provided to drawing water in dry season and during flood.

5.3 Classification of intakes

For the various surface source of water, different types of intakes are used. The various types of intakes which are commonly used are classified as below in figure 5-1.

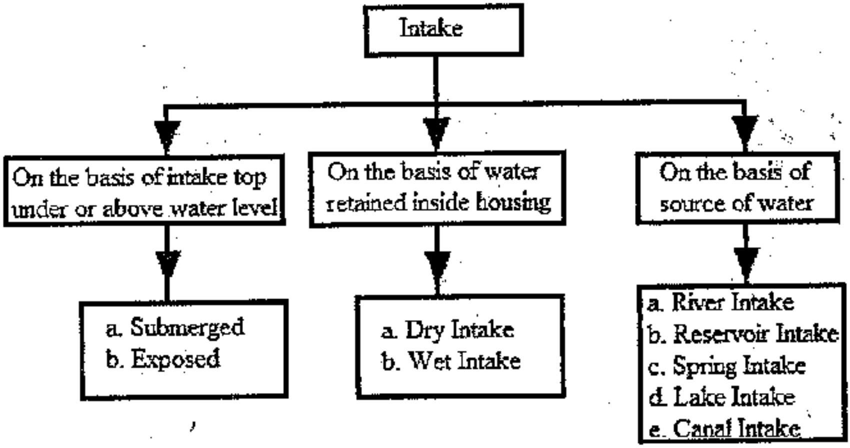


Figure 5-1 Classification of Intake

Submerged intake: Intake constructed entirely under the pool is submerged intake.

Exposed intake: When an intake is constructed with showing housing or intake tower above pool is exposed intake.

Wet Intake: In wet intake water is allows in intake tower or control room at get (valve) closed condition, entry port is inside housing.

Dry Intake: In dry intake if gets (valves) are closed there is no water inside intake tower or entry port is directly passed into the convey pipes but operation valves are used.

5.4.1 River Intakes

An intake tower constructed at the bank of the river to acquire water is river intake. It consists of masonry or RCC, intake tower (housing) which is provided with several inlets (3 common) called penstock as shown in figure 5-2. These penstocks are positioned at different levels to permit the river water for minimum flow, average flow and maximum flow, sometimes only two penstocks are provided. In entry port screen is provided to prevent the entry of debris. To control or regulate the flow valves are provided in the penstock which can be operated from control room. In control room pump is installed at the top. If the intake tower is filled with water during get or valve closed condition it is wet intake. In dry intake there is no water in intake tower during get or valve closed condition.

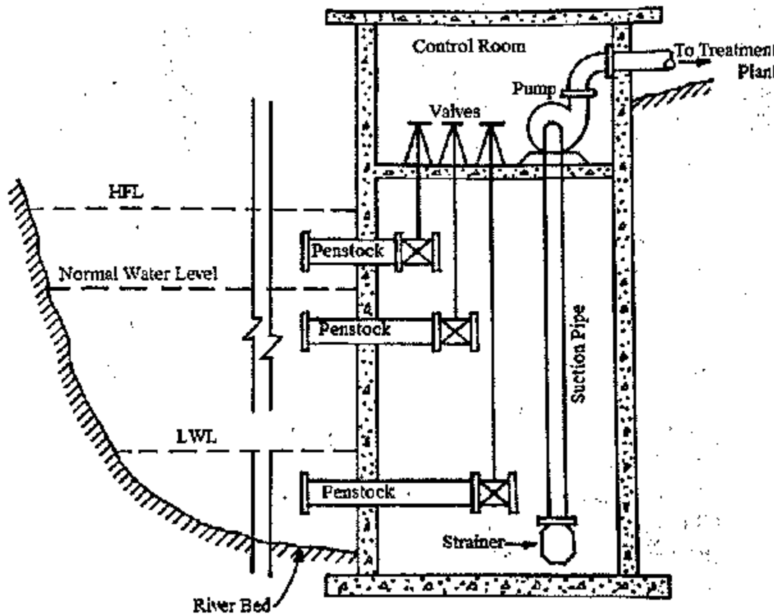


Figure 5-2 River Intake

Figure 5-2 is a typical wet river intake so that the water is always filled in the sump well of the intake tower as wet for all the time. Wet intake can be modified to dry by connecting penstocks to the suction pipe of the pump directly and hence water will not allowed to sump well.

In case of unstable river bed, the intake tower may be founded slightly offset from the river bed as shown in figure 5-3. In this type of river intake, a pipe from submerged intake deliver water to jack well then water is lifted and delivered to treatment plant through transmission mains.

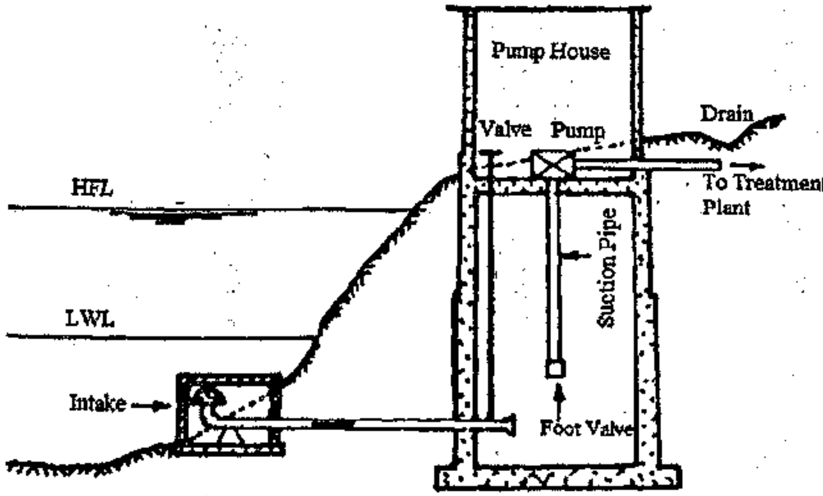


Figure 5-3 River Intake at Unstable Bank

5.4.2 Reservoir Intake

Water available in the river may not sufficient to meet demand in dry season in such case a dam or weir across the river is constructed to meet dry demand called impended reservoir. When intake tower is constructed in such case is called reservoir intake. There are two types of reservoir intake (Earthen and gravity dams) commonly used. Earthen dam consists of an intake tower constructed on the upstream toe at dam from where intake can draw sufficient quantity of water even in the worst condition shown in figure 5-4. Penstocks are installed in different levels through which water is withdrawn. There is provision of hemispherical screen in the entry of these penstocks to prevent the entry of floating matters. In the penstock valves are provided to control and regulate the flow of water. This intake is a dry intake because there is no water inside intake tower. For inspection and cleaning inside housing ladder or foot bridge is provided from control room.

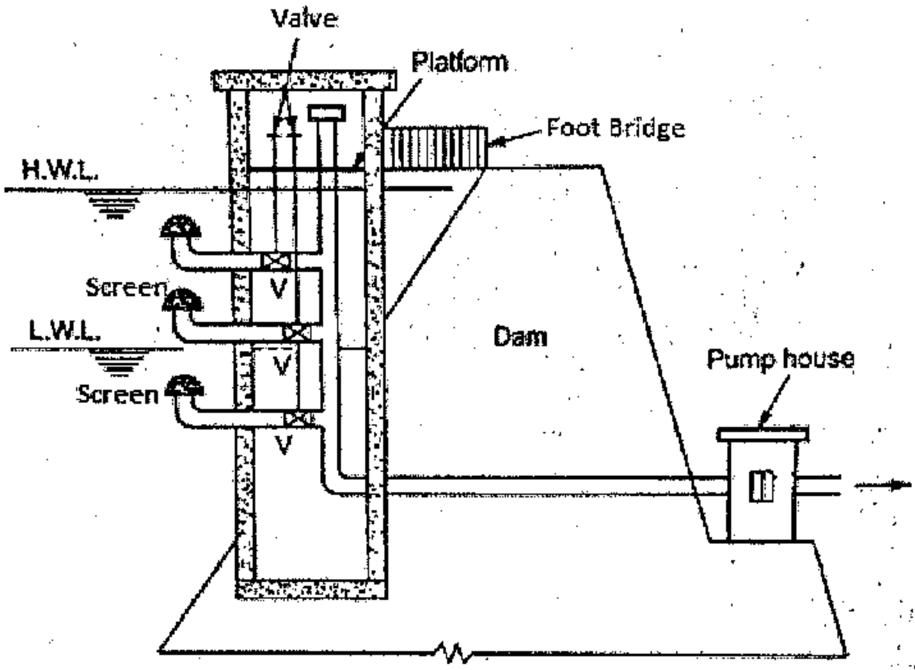


Figure 5-4 Reservoir Intake for Earthen Dam

In gravity dam, it has two alternatives forms of intake works as having single port and multiple ports. In Figure 5-5 an intake with entry of water is through a single port which has a trash rack structure to check the entry of debris and other floating matters. These are made in the form of semi polygonal grid of iron or steel bars. In order to control flow generally slide gate or sometimes valves are used which may be housed in the body of the dam itself.

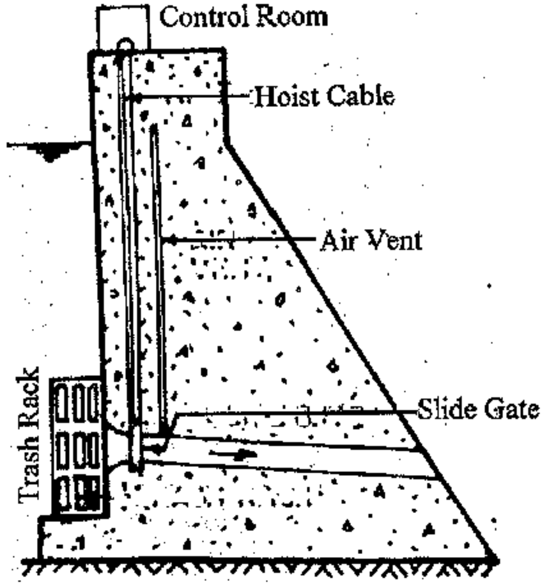


Figure 5-5 Reservoir Intake with Single Port (Gravity Dam)

In figure 5-6 an intake well is provided in the main body of the dam. Water enters the well through inlet ports located at different levels and provided with screened opening.

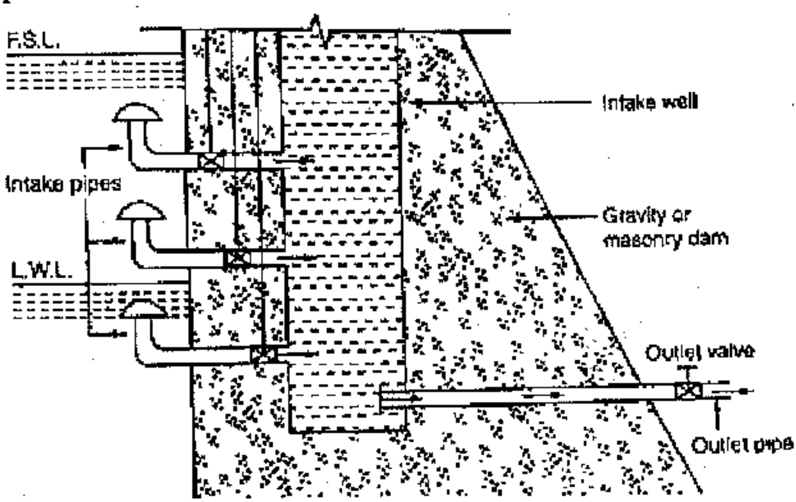


Figure 5-6 Reservoir Intake with Multiple Ports (Gravity Dam)

5.4.3 Spring Intake

A spring is a place on the earth's surface where groundwater emerges naturally. An intake constructed at the spring source to draw off water is called spring intake. Springs are generally found on hill slopes due to geological formation as impervious layer outcrops. Generally spring water does not contain suspended impurities and harmful bacteria. It may be used for small rural water supply scheme in Nepal.

Springs are susceptible to contamination by surface water, especially during rainstorms. Hence, U-shaped surface drainage diversion ditch or an earth berm at least 15 meter uphill from the spring to divert any surface runoff away from the spring has to be constructed. An area has to be fence at least 30 meter in all directions around the spring box to prevent contamination by livestock and people who are unaware of the spring's location. To maintain discharge plantation may be done in the periphery of spring source. Plan and section of a spring has shown in figure 5-6.

General requirements for selecting the location of the spring intake in order to get good quality of water.

- It should be as close to source as possible.
- It should be above populated or farming (agricultural) areas.
- It should be above foot path, cattle watering and washing places.
- It should be easy to drain off surface runoff during rain.
- It should not be easy accessible to people and livestock.
- It should not allow water logging near the intake.

Factors that should be take in account or considered while constructing the spring intake.

- To prevent from the creation of backup pressure, the collection chamber needs to be constructed away from the source by providing head of about 4 to 5 meter of free flow to occur from the intake

- Stone soiling below the floor should be avoided to prevent leakage.
- Heavy intake structure should be avoided to prevent from settlement.
- Adequate space in valve box should be provided so that repair and maintenance work could be performed easily.
- Union should be provided to avoid complex problem during replacement of gate valve at the time of repair.
- Over excavation of impervious layer at the base of the outlet of spring may lose flow so to avoid such problem especial care required at the time of excavation.
- Inlet pipe should be covered with stone soiling and upstream of intake with impervious material to prevent entry of suspended particles.
- It is essential to restrict access of animals in habitation of intake at least 30 m to avoid contamination.
- Surface runoff which occurs after rain should be easily drained off so that provision of drain in periphery of habitation of intake should be facilitated to prevent from pollution.
- Spring of low yield less than 0.05 lps should not be tapped for gravity flow schemes.

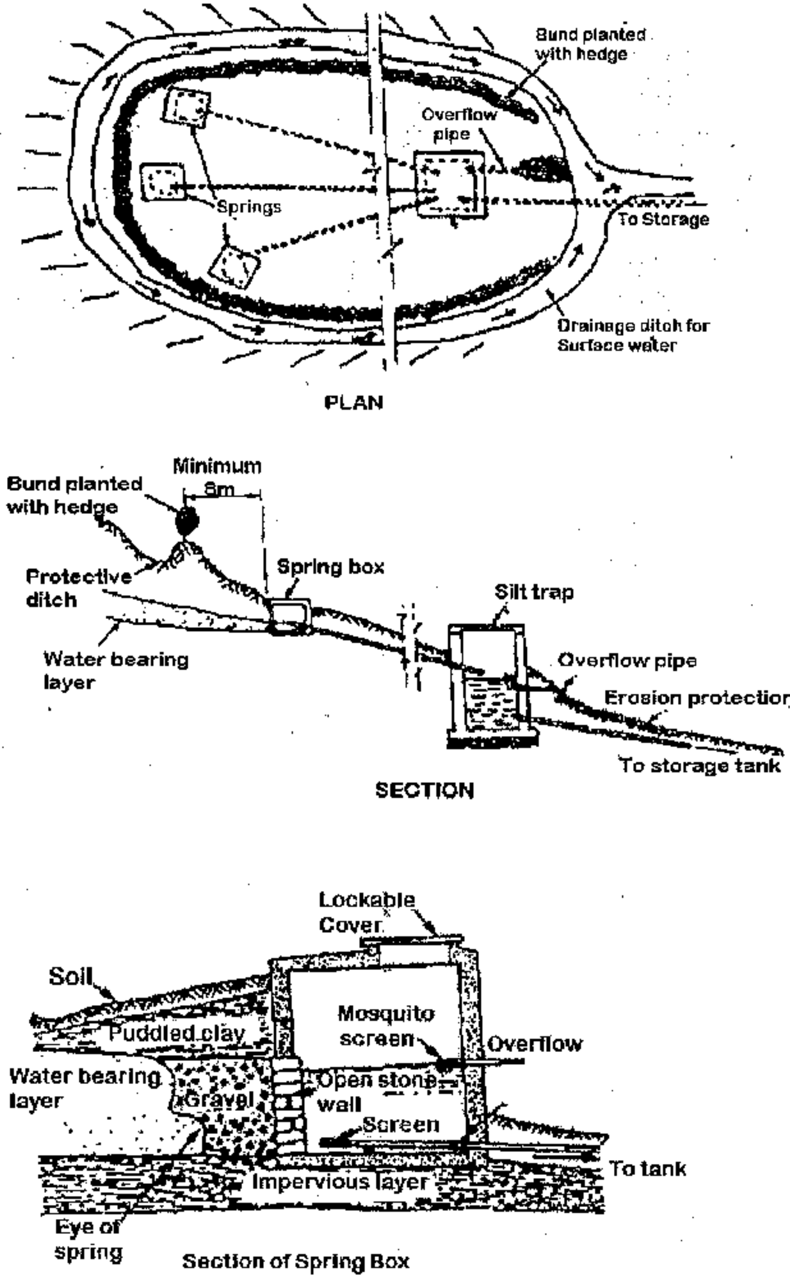


Figure 5-7 Spring Intake

Problems

1. What are the factors that should be considered during selection of an intake? With neat sketches, describe the construction of a spring intake.
2. What are the factors that should be considered during selection of an intake site? Describe the characteristics of wet reservoir intake with a neat sketch.
3. Define intake. Describe the construction of a spring intake generally used in hilly areas of Nepal.
4. Describe a dry river intake with a neat sketch.
5. When reservoir intake is constructed? Describe a typical reservoir intake.

6 Treatment of Water

Water treatment is a process of making the water for the intended purpose by removing or reducing the impurities present in water. The removal of harmful constituents from water or adjusting the concentration of impurities at a desired level within standard that do not harm to human health is called water purification or water treatment. Hence during water treatment the impurities present in water are not essential to make nil but to a level which will not harmful for intended use as per the standards. The degree of treatment depends upon quality of raw water and desired standard. The quality of water available in source is analyzed to know characteristics of impurities prior to the selection of the water treatment process.

Raw water may contain various impurities. The purpose of water treatment is to remove those impurities which are objectionable either from taste and odour aspect or public health aspect. The aim of water treatment is to produce and maintain water that is hygienically safe, aesthetically attractive and palatable, in an economic manner.

6.1 Objectives of water treatment

The primary objective of water treatment for public supply is to take water from the best available source and to subject is to processing which will ensure water of good physical quality, free from unpleasant taste or odour and containing nothing which might be detrimental to human health.

1. To make water as per requisite standard.
2. To reduce the impurities to an acceptable range.
3. To make water potable.
4. To kill disease causing organism (bacteria).
5. To minimize the corrosive nature of water that affect pipe.

6.2 Process of water treatment

There are various water treatment processes. Choice and arrangement of process may depend on the following factors;

- Raw water quality that to be treated.
- Desired water quality standard.
- Source (Either ground or surface).
- Types of impurities and concentration.
- Availability of funds.
- Purpose of supply.

The most widely used treatment processes are mentioned below.

1. **Screening**: Purpose of screening is to remove large suspended bulky floating matters like tree branches, leaves, bushes etc. from water.
2. **Plain sedimentation**: Sedimentation is the process of separating course or heavier suspended particles such as silt, sand etc. from water. Sedimentation removes turbidity from water as well as colour, odour, taste in some extent.
3. **Sedimentation with coagulation**: Sedimentation with coagulation process is most popular in urban water treatment process and its location is before rapid sand filter. This is to remove electrically charged finer particles especially colloidal.
4. **Filtration**: Water treatment may be synonymous to filtration to everyone. Hence filtration is the most important stage in water purification. Simply filtration is the process of passing water through a thick porous media. Especially filtration is to remove bacteria, finer particles and colloidal.
5. **Disinfection**: Disinfection is the process of killing pathogenic microorganisms present in water. In order to disinfect water various methods may be adopted but chlorination is common in

developing countries like Nepal where as ozone and UV rays treatment in developed countries.

6. **Aeration:** This is the process which is to remove colour, odour, and taste from water. Especially dissolved gases such as CO_2, H_2S, NH_3, and CH_4 etc. can be release to atmosphere, and Fe and Mn also precipitated. In order to remove nitrate, nitride, ammonia from water may be presence in ground water; bio-filtration has already came in practiced.

7. **Softening:** Ground water may contain hardness in water which is removed by softening.

8. **De-chlorination:** To remove excess chlorine.

9. **Miscellaneous:** To remove Iron, Manganese and other harmful matter.

Figure 6-1 shows a typical schematic layout of water treatment plant for treating surface water.

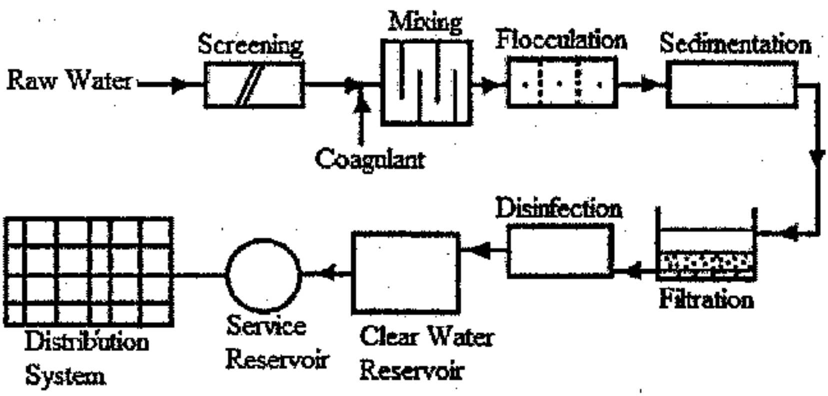


Figure 6-1 Schematic Diagram of Water Treatment Plant

Table 6-1 summarizes various water treatment processes and the impurity removal by corresponding process. Actually in water treatment process location and requirement of each unit depends on water quality and desired standard.

Table 6-1 Treatment Process and Impurity Removal

Treatment Process	Impurity Removal
Screening	Large suspended and floating matters
Plain Sedimentation	Coarse and heavy suspended impurities
Sedimentation with coagulation	Fine suspended and colloidal
Filtration	Very fine suspend and colloidal particle, microorganisms
Disinfection	Pathogenic microorganisms
Softening	Hardness of water
Aeration	Colour, odour, taste, dissolves gases, iron and manganese etc.
Miscellaneous treatment	Iron, manganese, dissolved gases, and other harmful constituents

6.3 Screening

The process followed by passing water through screen to remove large suspended matters like sticks, branches of tree, leaves, dead animal body, debris etc is called screening. Screening is accomplished by means of screens, having opening of uniform size, circular or rectangular in shape. The screening element is compromised of parallel bars, rods, wires, grating, wire mesh or perforated plate. When composed of parallel bars or rods, it is called *rack or bar screen* and when made from wire mesh, perforated pipes etc. it is called *screen*. Screen may be further divided as coarse, medium and fine screen depending upon their size and opening.

Bar rack or coarse screen

In the beginning of water purification works bar screen (generally 25 mm size) having clear openings about 50 mm or more (generally 75-150 mm) used to trap large size bulky floating matters like wood, rags, paper etc that may contain surface water.

Coarse screen is used to remove relatively larger type of floating matters and made of circular or rectangular bars of horizontal spacing of 75 to 150 mm placed vertically or at 30 to 80° inclined with vertical (see figure 6-2). The trapped matters are removed either manually or mechanically.

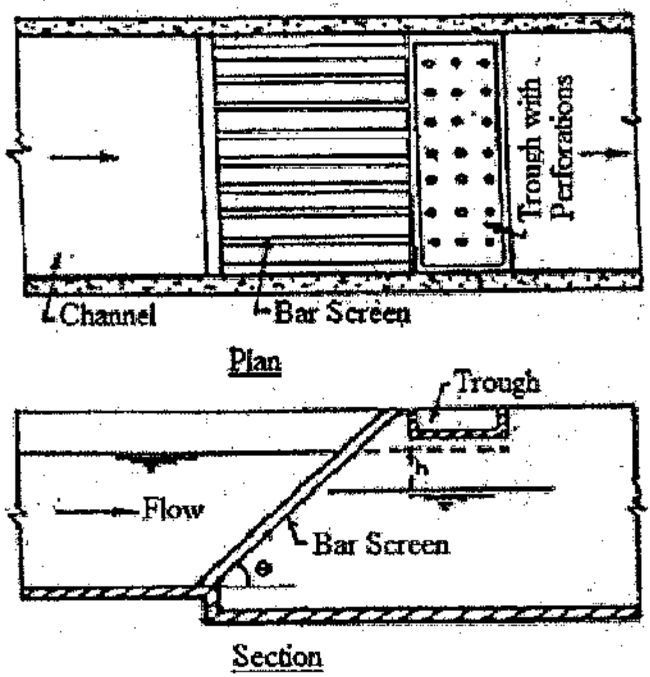


Figure 6-2 Bar or course screen

Sometimes medium screen having clear openings are about 20 to 50 mm may be used to trap medium size screenings and placed at the inlet of pumps. The maximum allowable head loss to a clogged racks and screens is 0.8 m.

Perforated or fine screen

Fine screen is used to remove small suspended matters generally this screen is not used because it gets clogged quickly and needs frequent cleaning. It may have mechanically cleaning device and made of perforated plate of diameter of 0.6 cm perforations. This fine screen may be used to relatively clean water at the time of no other treatment entertained except it. A rotating disc type fine screen is cleaned by a cone brush. Provisions of such type of screen may be at surface water intakes, sometimes alone, sometimes following a bar rack.

6.4.1 Purpose of settling

The most common form of sedimentation follows coagulation and flocculation and precedes filtration. This type of sedimentation requires chemical addition (in the coagulation/flocculation step) and removes the resulting floc from the water. Sedimentation at this stage in the treatment process should remove 90% of the suspended particles from the water, including bacteria. The purpose of sedimentation here is to decrease the concentration of suspended particles in the water, reducing the load on the filters.

- To remove coarse dispersed phase.
- To remove coagulated and flocculated impurities.
- To remove precipitated impurities after chemical treatment.
- To reduce extra burden to subsequent process.
- To reduce the dose of chemicals in the subsequent processes

Plain Sedimentation

Sedimentation can also occur as part of the pretreatment process, where it is known as presedimentation. Presedimentation is commonly known as plain sedimentation because the process depends merely on gravity and includes no coagulation and flocculation. Without coagulation/flocculation, plain sedimentation can remove only coarse suspended matter (such as grit) which will settle rapidly out of the water without the addition of chemicals.

Sedimentation (Hydraulic subsidence)

The terms sedimentation and clarification are commonly used interchangeably with regard to preparation of potable water. Sedimentation is the process in which solid matter separated (physically) by the action of gravity from water. In water mainly two types of suspended matters inorganic (sp. gr. 2.65) and organic (sp. gr. 1 to 1.4), these suspended particles held in suspension because of turbulence in flowing water. However, when water is retained in a tank it is brought to rest and there bring no turbulence and the suspended particles settle down and get deposited at the bottom of the tank or basin.

Discrete particle

Particles which do not alter shape, size and weight during rising and settling in any fluid is called discrete particle.

Theory

All the particles having more specific gravity than the liquid (water) will move vertically downward due to gravitational force.

Settling or hydraulic subsidence of particle in a sedimentation tank is affected by velocity of water, size and shape of particles, specific gravity of particles, viscosity of water, surface overflow rate, detention period, inlet and outlet structures, effective depth of settling zone.

6.4.2 Settling velocity of discrete particle

When a discrete particle is placed in quiescent fluid, it will accelerate until the frictional resistance (called drag force F_D) of the fluid equals the impelling force F_i (or driving force) acting on the particle. At this stage, the particle attains a uniform or terminal velocity and settles down with this constant velocity known as settling velocity.

Impelling/driving force at the uniform velocity is equals to the effective weight of the particle in fluid.

$$F_i = (\gamma_s - \gamma)V = (\rho_s - \rho)gV$$

$$\therefore F_i = \frac{\pi d^3}{6}(\rho_s - \rho)g \quad ......(I)$$

Force of gravity = $\gamma_s V$

Bouyant force = γV

Volume of spherical velocity $(V) = \frac{\pi d^3}{6}$

(Self weight = actual weight – force of buoyancy)

Where, γ = unit weight

ρ = mass density of fluid

ρ_s = mass density of particle

Acceleration due to gravity (g) = 9.81 m/sec^2

Newton's law for frictional resistance or drag force (viscous friction) is given by,

(Drag force depends upon; Dynamic viscosity, Mass density of the fluid Shape and size of particle)

$$F_D = C_D \frac{\rho A V_s^2}{2} = C_D \frac{\rho \pi d^2 v_s^2}{8} \quad ....(II)$$

Projected area of a spherical particle $(A) = \frac{\pi d^2}{4}$

Equating two forces (I) & (II)

$$V_s = \sqrt{\frac{4gd(S-1)}{3C_D}} \quad ....(III)$$

$$\left[\because \textit{specific gravity of the particle}(S) = \frac{\rho_s}{\rho}\right]$$

Vs is settling velocity of particle

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

$$Re = \frac{\rho v d}{\mu}$$

Equation III is called **Hazen's equation** and applicable for particle diameter 0.1 to 1 and Re 1 to 1000 and settling is **transition.**

For small particles less than 0.1 mm and Re ≤ 1 is **laminar** settling. Value of drag F_D acting along on a small spherical particle settling in a viscous fluid neglecting the inertia force is given by,

$F_D = 3\pi\mu v_s d$ ....(IV)

Equating (II) & (IV)

$$C_D = \frac{24}{\text{Re}}$$

Putting value of C_D in equation (III)

$v_s = \frac{g}{18\upsilon}(S-1)d^2$ ....called **Stoke's** equation

Table 6-1 Kinematic viscosity of water

Temperature (°C)	0	5	10	15	20	25	30
Kinematic Viscosity (υ) (Centistokes)	1.792	1.519	1.308	1.141	1.007	0.897	0.804

Centistokes equivalent to mm^2/sec

Temperature effect on settlement

Kinematic viscosity depends on temperature and settling of particle is directly affected by the temperature. Settling velocity of a particle will be increased with increase in temperature so that kinematic viscosity decreases in the increase in temperature. Stoke's equation can be expressed by introducing temperature instate of kinematic viscosity as indicated below.

$$v_s = 418(S-1)d^2\left(\frac{3T+70}{100}\right)$$

Where,

v_s is settling velocity of particle in mm/sec

d is diameter of particle in mm

T is temperature of water in °C

S is specific gravity of particle

Again, for particles of diameter greater than 1 mm and Re 1000 to 10000 the nature of settling of particle is **turbulent** and settling is turbulent settling.

The drag coefficient for the settling $C_D = 0.4$, putting the vale in equations (III) we get,

$V_s = \sqrt{3.33gd(S-1)}$ ....called **Newton's** equation.

Table 6-2 Equations for Settling Velocity of Discrete Particles

S. No.	Law and Equation	Applicable range of	
		Re	Particle Size in mm
1	Stoke's (Laminar) $v_s = \frac{g}{18\,\upsilon}(S-1)d^2$	Up to 1	Up to 0.1
2	Hazen's (Transition) $V_s = \sqrt{\frac{4gd(S-1)}{3C_D}}$ $C_D = \frac{24}{Re} + \frac{3}{\sqrt{Re}} + 0.34$	>1 to 1000	>0.1 to 1
3	Newton's (Turbulent) $V_s = \sqrt{3.33gd(S-1)}$	$>10^3$ to 10^4	Greater than 1

6.4.3 Ideal sedimentation tank

When time taken by water particle to move from the inlet to the outlet is equal to time taken by a suspended particle to settle down in a tank is ideal condition of the sedimentation tank. An ideal sedimentation tank (Figure 6-3) is a tank which aims of equal velocity at all points lying in each vertical in settling zone at the design. Ideal case of sedimentation tank could not found in real or practically. The *ideal rectangular horizontal flow sedimentation tank* is considered divided into four zones.

Inlet zone: in which momentum is dissipated and flow is established in a uniform forward direction

Settling zone: where quiescent settling is assumed to occur as the water flows towards the outlet

Outlet zone: in which the flow converges upwards to the decanting weirs or launders

Sludge zone: where settled material collects and is moved towards sludge hoppers for withdrawal. It is assumed that once a particle reaches the sludge zone it is effectively removed from the flow.

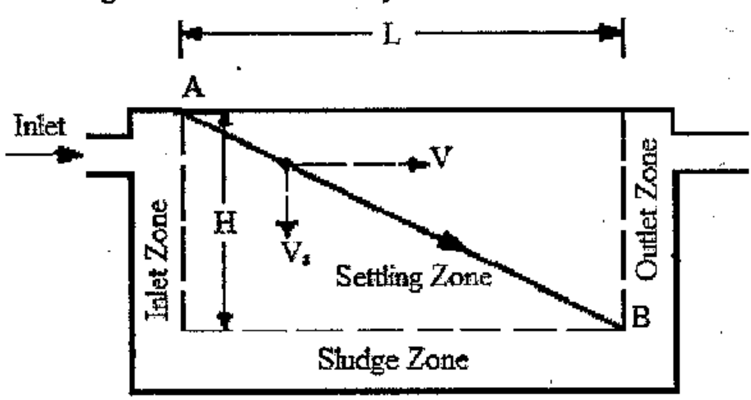


Figure 6-3 Ideal Sedimentation Tank

Design of horizontal flow sedimentation tank

Design of horizontal flow tank is based on following assumptions;

- The particles in the settling zone exactly settle in the same way as they settle in a quiescent tank of equal depth.
- In all parts of settling zone, flow of water is steady and velocity is uniform for a time equal to detention period.
- All the sediment entering is uniformly distributed as at all points of the vertical cross section at the inlet zone.
- Particles that reaches to sludge zone is removed and do not rise.

Relation between settling velocity of a particle and SOR

Consider a tank of length L, breadth B, height H and it is assumed that sediment is uniformly distributed and water enters to the tank at a uniform velocity V as shown is figure 6-4.

If 'Q' is the discharge entering in the tank, the velocity is given by;

$$V = \frac{Q}{BH}$$

As all the discrete particles is moving with the flowing water and also tending to settle down. It possesses a horizontal velocity V and settling velocity v_s in the vertical downward direction. It is assumed that particles whose paths of travel are above the line AC will pass through tank or these particles will not settle in the tank. If 't' is detention period of the tank then;

$$\text{detention time (t)} = \frac{L}{V} = \frac{H}{v_s}$$

$$v_s = V \times \frac{H}{L} = \frac{Q}{BH} \times \frac{H}{L} = \frac{Q}{BL}$$

$$v_s = \frac{Q}{BL} = \frac{Q}{A_s}$$

Discharge per unit plan area of a sedimentation tank is known as surface overflow rate or surface loading rate or hydraulic loading rate and expressed in terms of $m^3/m^2/day$.

Consideration of the assumed criteria for the settling of the particles indicates that all the particles having settling velocity v_s equal or greater than SOR will be settle or removed.

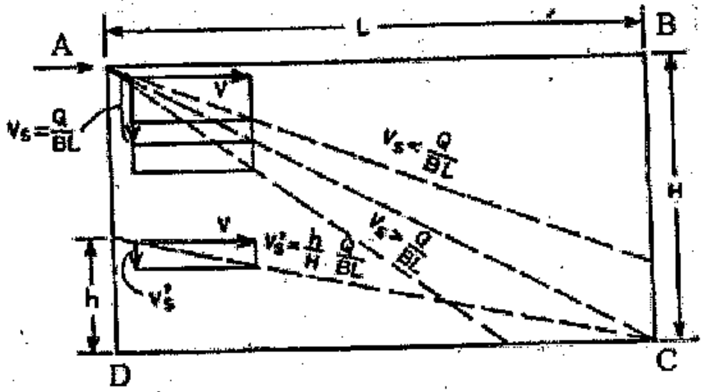


Figure 6-4 Settling Zone of Sedimentation Tank

Now if a smaller particle having settling velocity $v_s' < \dfrac{Q}{BL}$ enters the tank at point A then it will not settle in the tank. However, if this smaller particle enters the tank at some other level h as shown in figure 6-4 then;

$$\frac{V}{v_s'} = \frac{L}{h} \quad \text{or, } v_s' = \frac{h}{H}V \qquad \left(V = \frac{Q}{BH}\right)$$

$$\text{or, } \quad v_s' = \frac{h}{H}\left(\frac{Q}{BL}\right)$$

$$\therefore \eta = \frac{v_s'}{Q/BL} = \frac{x}{x_o} = \frac{h}{H}$$

$$\%\ \text{settled} = \frac{v_s}{\text{overflow rate}} \times 100$$

Out of x_0 particles a particular size present in water x particle settle down and are removed i.e. the ratio (x/x_0) represents the removal efficiency of a sedimentation tank for the particles of same size.

6.4.4 Types of sedimentation tank

The sedimentation tanks are generally made of reinforced cement concrete and may be rectangular or circular in shape depending upon the method of operation the sedimentation tanks are of the following two types,

1) Fill and draw type sedimentation tank
2) Continuous flow type sedimentation tank

1) Fill and draw type sedimentation tank

The fill and draw type sedimentation tanks are also known as quiescent type or intermittent type sedimentation tank. The raw water is filled in the tank and it is retained until the particles in suspension settled down and get deposited at the bottom of the tank. The clear water is then drawn off from the tank and the tank is cleaned of silt deposited in it. After cleaning, the tank is again filled with raw water and the same operation related.

- detention period normally 24 hrs
- minimum three tanks will be required
- fill and draw type sedimentation tanks are not being used these days.

Disadvantages

i) Labor and supervision
ii) Loss of head
iii) Number of tank required
iv) Wastage of time

2) Continuous flow type sedimentation tank

The water to be treated is continuously admitted into the tank and allowed to flow slowly in the tank during which the particles in suspension settled down and clear water flows out continuously from the tank. These tank works on the principle that by reducing the velocity of flow of water can be made to settle down.

Generally continuous flow type sedimentation tank are rectangular or circular (sometimes square) in shape. These tanks may be classified on the basis of flow direction of water in the tank as; (a) Horizontal flow tanks, (b) Vertical flow tanks.

(a) Horizontal flow tanks

As flow in these tanks is substantially horizontal it may be further classified into; (i) Rectangular tanks with longitudinal flow and (ii) Circular tanks with radial flow.

i) Rectangular sedimentation tank (longitudinal flow)

The raw water enters the tank through an inlet provided on the side of the tank and after flowing slowly in the horizontal direction in the tank, it passes out through an outlet provided on the opposite side of the tank. The inlet and outlet are provided generally at the top edge of the tank. Figure 6-5 (a) and (b) shows a rectangular tank without baffles and are facilitated with mechanically removal of sludge called mechanized tank. This may be with and without baffles.

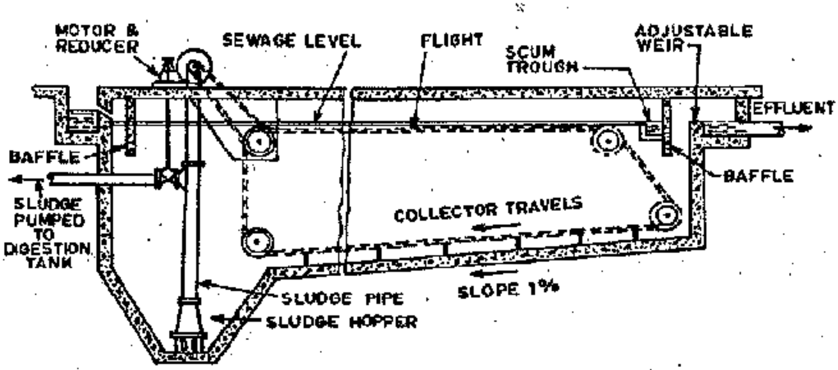


Figure 6-5 (a) Rectangular Continuous flow settling tank

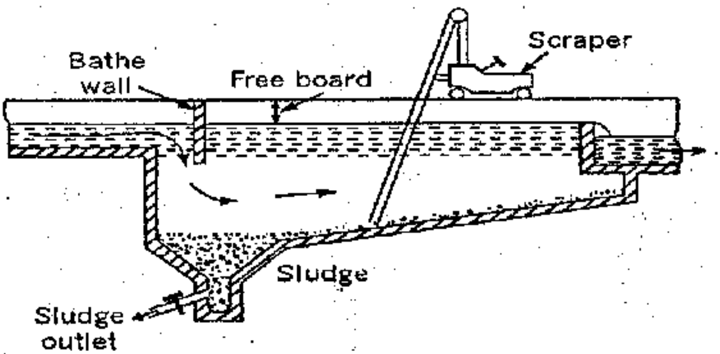


Figure 6-5 (b) Typical Mechanized Sedimentation tank

Commonly sedimentation tank are provided with baffles along the length so that flow length in tank can be prolonged and overcome the problem of short circuiting. Since baffles induced longer path of travel it also prevent the turbulence of water as shown in figure 6-6.

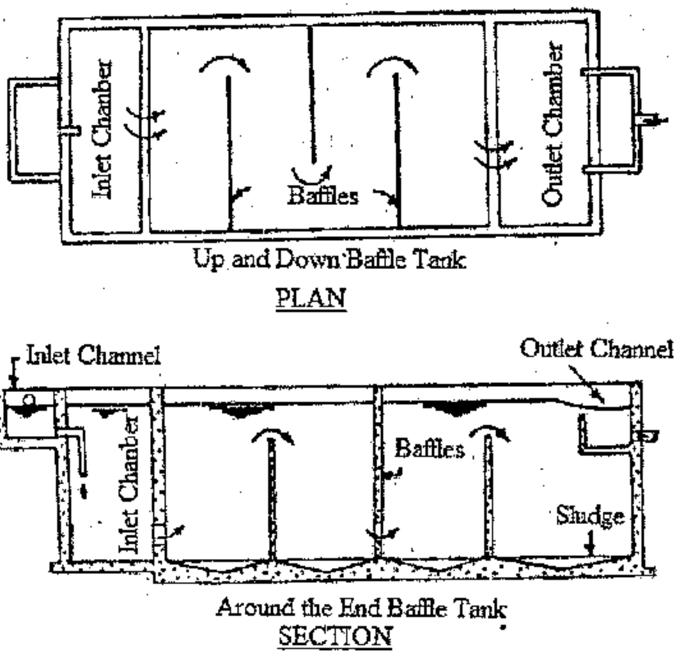
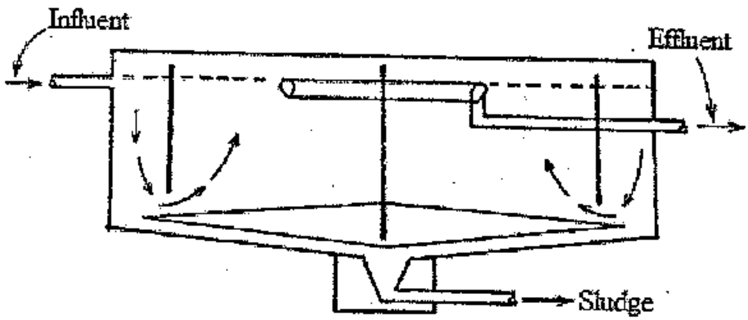


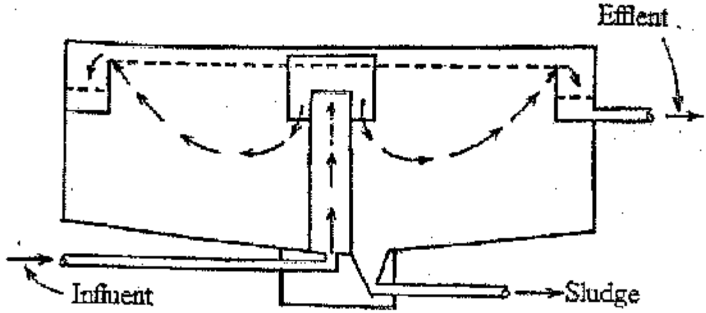
Figure 6-6 Rectangular Sedimentation Tank with Baffles

ii) Circular sedimentation tank with radial flow

These tanks with radial flow are of the two types as (a) Circular tank with central feed (b) Circular tank with peripheral feed. Figure 6-7 shows the schematic diagrams of these two types of circular tanks. In tank with central feed, water enters the tank at the center and passes through its periphery. In the tank with peripheral feed raw water enters through two or three vertical silts. There is a rotating arm in the tank which allows the water to move along the periphery of the tank. Water while moving at very low velocity allows it suspended particles settle in the tank. Central feed circular tank is common in practice.



Circular Tank with Peripherl Feed



Circular Tank with Central Feed

Figure 6-7 Circular Sedimentation Tanks with Radial Flow

Hopper bottom tank with Vertical flow

In this tank water enters through the centrally placed inlet pipe and is deflected downwards by the action of a deflector box. In hopper bottom tank water travels vertically downwards. Settled particles in the bottom (Sludge) of hopper tank are removed by suction through pipe operated by pump as shown figure 6-8.

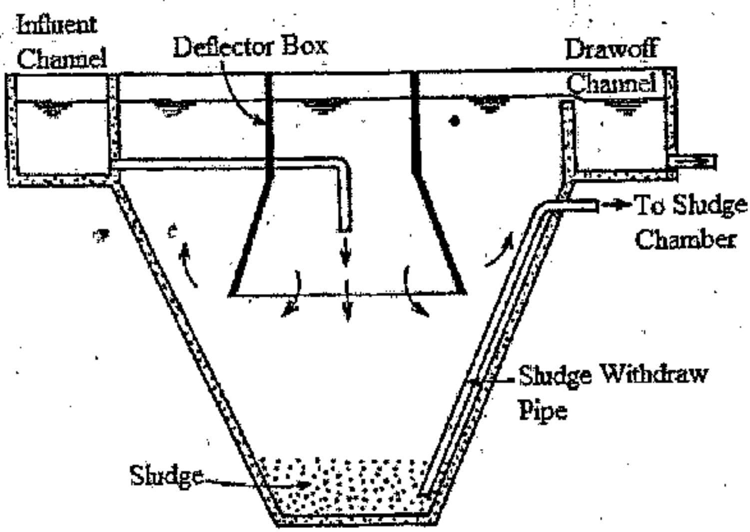


Figure 6-8 Hopper Bottom Tank

Dorr Clarifier

Dorr clarifier is a common type central feed continuous flow type sedimentation tank as shown in figure 6-9. In this tank water enters continuously through a vertical inlet pipe at the center of the tank and emanates from multiple ports of influent air diffuser provided at the top of the inlet pipes so that water flows radially outwards in all direction equally. A circular baffle is provided to reduce velocity of incoming water. The water passes through peripheral weir over which it reaches to effluent channel then effluent pipe. Settled particles

(Sludge) in the bottom of the tank is continuously removed by sludge removal mechanism.

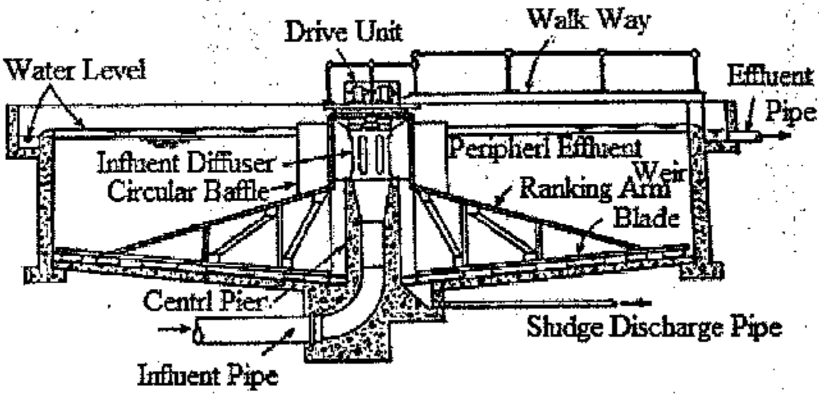


Figure 6-9 Dorr Clarifier

Configuration of sedimentation tank

In order to remove high percentage of suspended materials in a continuously flow type sedimentation tank proper design is essential. So that various design considerations are need to adopt sequentially as described below.

1. Velocity of flow
2. Detention period
3. Flowing through period
4. Relation between surface overflow rate and settling velocity of particles
5. Tank dimension
6. Inlet and outlet structure

1. Flow velocity

The flow velocity in sedimentation tank should be such that maximum settling of suspended particles is cause in the tank. It should remain uniform throughout the tank and should not be greater than 300 mm/min (generally range 150 -300 mm/min).

It is essential that once the particle settled it should not be scoured or lifted up by the velocity of flow of water over the bed. Scouring or displacement velocity (V_d) for the particles of size d. Camp's expression for the displacement velocity,

$$V_d = \sqrt{\frac{8\beta g(s-1)d}{f}}$$

Where,

β = a dimensionless constant which represents the characteristic of the sediments or a solid particles present in the sewage. The value of β may be taken as 0.04 for initiating scour of clean grit and 0.8 for full removal of sticky grit;

f = Darcy Weisbach friction factor, the common value of which may be taken as 0.03;

S = Specific gravity of sediments/solids flowing in the sewage. Its value range from 2.65 for inorganic sediments to 1.2 for organic sediments;

g = acceleration due to gravity 9.81 m/sec^2

Expression for fine and light solids in terms of v_s,

The flow velocity in water should be less than scouring or displacement velocity in water. The scouring velocity can be expressed in terms of settling velocity s following.

$$V_d = V_s\left(\frac{8}{f}\right)^{\frac{1}{2}}$$

(When, f = 0.025)

$$v_d \approx 18\ v_s$$

Practically, $v_d = 10\ v_s$

Surface area (As) = B x L

Cross sectional area (a) = B x H

$$\frac{A}{a}=\frac{L}{H}=\frac{v_d \times t_d}{v_s \times t_o}=\left(\frac{t_d}{t_o}\right)\times\left(\frac{8}{f}\right)^{1/2}$$

t_d= time taken by water particle to move from inlet to the outlet

t_o= time taken by suspended particle settle down

For an ideal tank $t_d = t_o$,

More practical relation generally used,

$$\frac{v_d}{v_s}=\frac{A}{a}=\frac{L}{H}=10$$

2. Detention period

The detention period of a sedimentation tank is the theoretical time water is detained in it. In case of a continuous flow type sedimentation tank the detention period may be considered as the theoretical time taken by a particle of water to pass from the entry to the exit of the tank. Let C be the capacity of a sedimentation tank, and Q be the discharge or rate of flow of water through the tank, than the detention period t of the tank is given as;

$$\text{detention period} \quad (t)=\frac{C}{Q} \quad =\frac{LBH}{Q}$$

3. Flowing through period

It is the actual time taken by the water to pass through a sedimentation tank. Theoretically it is same as detention period, but it may vary so that the incoming flow is seldom uniform. This variation or if the water particles leaves before intended time (theoretical detention period) flowing water is short circuited. The ratio of flowing through period and detention period is called displacement efficiency. A well designed tank should have a flowing through period of at least 30% of the detention period.

4. **Surface overflow rate and settling velocity of particles**

Discharge per unit plan area of a sedimentation tank is known as surface overflow rate or surface loading rate or hydraulic loading rate and expressed in terms of m^3/m^2/day.

The increase in surface or plan area of a sedimentation tank will increase in the settling or removal efficiency of the tank. The settling of the particles indicates that all the particles having settling velocity v_s equal or greater than SOR will be settle or removed. Commonly adopted vale of SOR for plain sedimentation tank is 15 to 30 m^3/m^2/day.

5. **Tank dimension**

Common shape of sedimentation tank is either rectangular or circular. Range of effective depth of a sedimentation tank is 2.5 to 4 m. The provision of free board and sludge zone in sedimentation tank range is of 0.5 to 1 m. For rectangular sedimentation tank maximum width or breadth is 12 m with length to width ratio of 3 to 5 where as length up to 100 m can be taken but commonly do not exceed 30 m. In circular sedimentation tank diameter may be considered less than 60 m but generally is of less than 30 m.

6. **Inlet and outlet structure**

The structure through which water enters to the sedimentation tank is inlet or influent structure and through which water leaves is effluent or outlet structure. To enhance the performance of sedimentation different arrangement of inlet and outlet structures are provided. Purpose of inlet structure is to minimize large-scale turbulence and create uniform flow for initiate longitudinal or radial flow. Outlet or effluent structure consists of weir, notches or orifices, effluent trough or launder and outlet pipe. The discharge per unit length of weir is weir loading. Weir loading up to 288 m^3/day/m is normally adopted.

Inlet structure: Inlets shall be designed to distribute the water equally and at uniform velocities. A baffle should be constructed across the basin close to the inlet and should project several feet below the water surface to dissipate inlet velocities and provide uniform flow. Inlet structures are shown in figure 6-10.

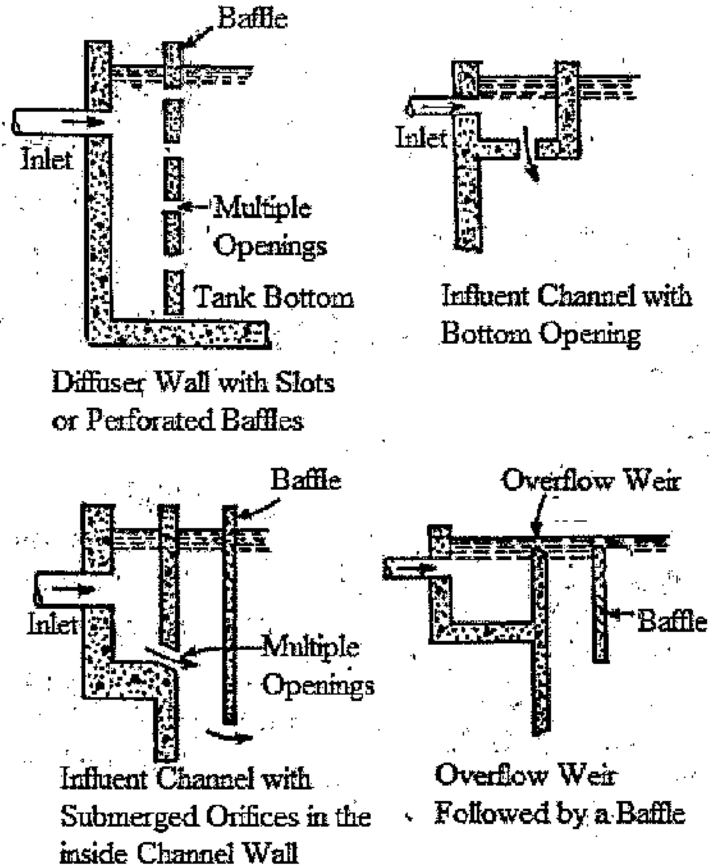
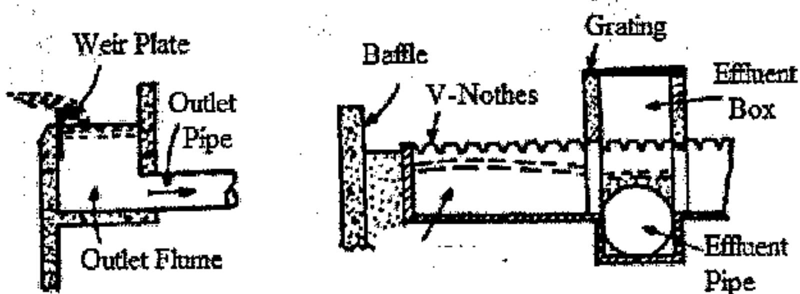


Figure 6-10 Inlets for Sedimentation Tank

Outlet structures: Outlet weirs or submerged orifices shall be designed to maintain velocities suitable for settling in the basin and to minimize short-circuiting. Weirs shall be adjustable, and at least equivalent in length to the perimeter of the tank. However, peripheral weirs are not acceptable as they tend to cause excessive short-circuiting. Typical sections of outlet structures are shown in figure 6-11.



Outlet with Rectangular Weir

Outlet Consistig of V-Notches, Effluent Launder, Effluent Box and an Effluent Pipe

Figure 6-11 Outlets for Sedimentation Tanks

Summarized sedimentation tank design criteria

1. Detention period (t) = 4 to 8 hrs (Plain Sedimentation)

   1 to 4 hrs for Sedimentation tanks using with coagulants *(1 to 4 hrs for Plain sedimentation in Nepal)*

2. Velocity of flow (V_h) = Not greater than 300 mm/min

3. Surface Overflow Rate (SOR) = 15 to 30 m^3/m^2/day for (Plain sedimentation)

   = 30 to 40 m^3/m^2/day for (Sedimentation with coagulation)

4. Tank dimension:

   L: B=3 to 5:1

   Length (L) = 30 m (common) maximum 100 m

   Breadth (B) = Maximum 12 m

   Effective depth ($d_{eff.}$) = 2.5 to 4 m (3 m common)

   Free board (FB) = 0.5 to 1 m

   Sludge zone = 0.5 to 1 m

5. For circular

   Diameter maximum 60 m (generally less than 30m)

   Slopes: Rectangular 1% towards inlet and 8% circular

   For circular tank with a bottom of 1 vertical to 12 horizontal

   $C = d^2 (0.011d + 0.785H)$

   $$t = \frac{H}{SOR}$$

For rectangular; $t = \frac{LBH}{SOR}$

6.5 Sedimentation with coagulation

Virtually all surface water sources contain perceptible turbidity and also water may contains very fine suspended particles of clay and silt and light colloidal matter, which can't be arrested by plain sedimentation within a reasonable detention period. The phenomenon settling down and removal of such finer suspended and colloidal matter can be achieve by chemically assisted sedimentation known as sedimentation with coagulation. In this method certain chemicals (coagulants) are added to raw water and mixed thoroughly causes an insoluble, gelatinous, flocculent precipitate called *floc*. The floc become heavier and gets settled. The process of adding a coagulant to raw water and mixing it thoroughly is known as coagulation and the process of forming of floc is known as flocculation. Suspended particles of clay and silt and colloidal matter present in water having negatively charged and added chemical (coagulant) is positively charged. Hence, as per ionic theory due to having opposite charge become attract each other and floc is formed, which is further settled and removed.

In sedimentation with coagulation suspended particles and colloidal matter removal is more complete and rapid removal and hence instead of plain sedimentation, the sedimentation with coagulation is almost universally adopted in all the major treatment plants.

If raw water contains suspended solid greater than 50 ppm, the sedimentation with coagulation is invariably used and it is used as a preliminary step in the preparation of water for filtration.

6.5.1 Chemicals used as coagulants

1. Aluminum sulphate (or alum)
2. Iron salts
3. Chlorinated copperas
4. Sodium aluminate

Chemicals to be added in water or dose of coagulants depend upon the following factors.

- Turbidity of water
- Its colour
- pH value of water
- Time of settlement and
- Temperature of water

The amount of the coagulant may also increase with lower temperature because of slower reaction and floc formation.

1. Aluminium sulphate (or alum) [$Al_2(So_4)_3.18H_2o$]

Alum (or filter alum) is most widely used chemical for the coagulation of water because as indicated below there are several advantages of using alum as coagulant. In order that alum may react to form a floc, it is necessary that the water to which alum is added shall contain alkalinity. When raw water has present of bicarbonate alkalinity reaction involved as follows;

$$[Al_2(So_4)_3.18H_2o]+3Ca(HCO_3)_2=2Al(OH)_3\downarrow+3CaSO_4+6CO_2\uparrow+18H_2O$$

The aluminium hydroxide formed is insoluble in water and it behaves as a floc and thus helps sedimentation. CO_2 leads to corrosion and $CaSO_4$ imparts some permanent hardness.

It has been indicated that for the functioning of alum as a coagulant it is necessary that raw water contains some alkalinity. When raw water contains little or no alkalinity then either lime or soda ash may be added to the raw water.

Alum conjunction with lime;

$$Al_2(SO_4)_3.18H_2O+3Ca(OH)_2=2Al(OH)_3\downarrow+3CaSO_4+18H_2O$$

Alum conjunction with soda ash;

$$Al_2(SO_4)_3.18H_2O+3Na_2CO_3+3H_2O=2Al(OH)_3\downarrow+3Na_2SO_4+3CO_2\uparrow+18H_2O$$

The usual dose of coagulant (alum) may vary from 5 mg/lit for relatively clear water to 30 mg/lit for highly turbid water. The average dosage for normal water is about 14 mg/lit. However, dosage of coagulant may determined by jar test.

Advantage of using alum as coagulant

1. It forms an excellent and stronger floc which is better than that formed by any other coagulant.
2. The floc formed is stable, which is not broken easily.
3. It is relatively cheap, easily available in market and also removes colour, odour and taste from water.

4. It produces crystal clear water and does not require any skilled supervision for handling.

Disadvantages of using alum as coagulant

1. If raw water does not contain sufficient alkalinity, then lime or soda ash will have to add for the pH adjustment because it can function only when pH range is 6.5 to 6.8.
2. Calcium sulphate which imparts some permanent hardness and free CO_2 may leads to corrosion.
3. It is difficult to manage sludge produced because it is not suitable for low laying lands.

2. Iron salts

The various salts used as coagulants includes *ferrous sulphate, ferric chloride and ferric sulphate*. The iron salts are used as coagulation in conjunction with lime.

A. Ferrous sulphate ($FeSO_4.7H_2O$)

Ferrous sulphate used as coagulant in water with conjunction to lime is also known as *copperas*.

a) When ferrous sulphate is added first and water contains bicarbonate alkalinity present in water;

$FeSO_4.7H_2O+Ca(HCO_3)_2=Fe(HCO_3)_2+CaSO_4+7H_2O$

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

b) when lime is added first,

$FeSO_4.7H_2O+Ca(OH)_2=Fe(OH)_2+CaSO_4+7H_2O$

The ferrous hydroxide formed further oxidised by DO in water as

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

The ferric hydrooxide $Fe(OH)_3$ forms a gelatinous floc.

When ferrous sulphate and lime is used as coagulant in water it works effectively in the pH range 8.5 and above.

B. Ferric chloride [Fecl₃]

It may be used as a coagulant either in conjunction with lime or without lime.

without lime;

$FeCl_3+3H_2O=Fe(OH)_3+3H^++3Cl^-$

with lime;

$2FeCl_3+3Ca(OH)_2=2Fe\,(OH)_3\downarrow+3CaCl_2$

The effective range of pH value for coagulation with ferric chloride is 3.5 to 6.5 and above 8.5.

C. Ferric sulphate [$Fe_2(SO_4)_3$]

Ferric sulphate is used as coagulant in conjunction with lime.

$Fe_2(SO_4)_3+3Ca(OH)_2=2Fe(OH)_3+3CaSO_4$

The effective range of pH value for coagulation with ferric sulphate is 4 to 7 and above 9.

Advantages of using Iron salts as coagulant

1. It produces a faster forming, denser, quicker settling and stable floc than alum especially at lower temperature.
2. It works over a wider range of pH values than alum.
3. Ferric chloride and ferric sulphate can remove manganese at pH above 9.
4. Ferric chloride is quite effective in removal of tastes, odours and hydrogen sulphide.

3. Chlorinated copperas [$FeCl_3.Fe_2(SO_4)_3$]

A mixture of ferric chloride ($FeCl_3$) and ferric sulphate [$Fe_2(SO_4)_3$] is known as chlorinated copperas. It is prepared by adding 1 part chlorine to 7.8 part ferrous sulphate (or copperas).

$6[FeSO_4.7H_2O]+3Cl_2=2[FeCl_3.Fe_2(SO_4)_3+42H_2O$

When it is used as coagulant in water forms a tough floc which helps sedimentation. The effective working range of pH value is 6 to 9. This coagulant is especially effective in removal of colour of water. This is frequently used as coagulant for wastewater treatment.

4. Sodium aluminiate [$Na_2Al_2O_4$]

This may be used sometimes as a coagulant in water. It reacts with salts of calcium and magnesium present in raw water and forms the precipitates of calcium aluminate and magnesium aluminate.

$Na_2Al_2O_4 + Ca(HCO)_3 = CaAl_2O_4\downarrow + Na_2CO_3 + CO_2 + H_2O$

$Na_2Al_2O_4 + CaCl_2 = CaAl_2O_4\downarrow + 2NaCl$

$Na_2Al_2O_4 + CaSO_4 = CaAl_2O_4\downarrow + Na_2SO_4$

Similar chemical reactions take place with magnesium salts. This coagulant removes both carbonate and non-carbonate hardness of water. The effective working pH range is 6 to 8.5 and it is not necessary to adjust pH after coagulation because the pH value of water lies within range. This coagulant is costly and hence it is not adopted for treating water for large scale for public water supplies but adopted treating water to used for boilers.

Operations involved in sedimentation with coagulation;

In sedimentation with coagulation following operations are involved;

1. Feeding the coagulant
2. Mixing of coagulant
3. Flocculation
4. Sedimentation

1. Feeding the coagulant

The coagulant may fed to water either in powder form (dry feeding) or in solution form (wet feeding). The wet feeding requires more space for preparing solution but dosage of coagulant in wet feeding can be controlled as compared to dry feeding.

The selection basis between dry and wet feeding depends on;

1. Characteristics of coagulant and the convenience of its application.
2. Dosages of coagulant: If the dosage of coagulant to be used is high it can be fed by dry feeding and in case of small dosage feeding it must be fed in solution form.
3. Size of treatment plant: In large water treatment plants wet feeding is adopted and for smaller ones dry feeding is adopted.

2. Mixing of coagulant

The success of floc formation mainly depends on the mixing of coagulant with water. In order to formation of floc coagulant should be thoroughly and vigorously mixed with water which can be achieve by various mixing devices as indicated below;

a. Mixing basins with baffle walls
b. Mixing basins with mechanical means
c. Mixing channel
d. Hydraulic jump method
e. Compressed air
f. Centrifugal pumps

a. Mixing basins with baffle walls

In rectangular tanks or basins baffle walls are provided so that hindrance and disturbance created in the path of the flowing water which causes vigorous agitation of water and results thorough mixing of coagulant with water. Such basins are of two types as indicated below.

i) Horizontal or round the end type

The water with coagulant after entering to the tank or basin through an inlet provided at one end of the tank, flows horizontally for a short distance and due to the presence of baffle walls it takes a turn and moves further as shown by arrows, and finally it flows out through an outlet provided at the other end of the tank. Figure 6-12 shows a mixing basin with horizontal or round the end type.

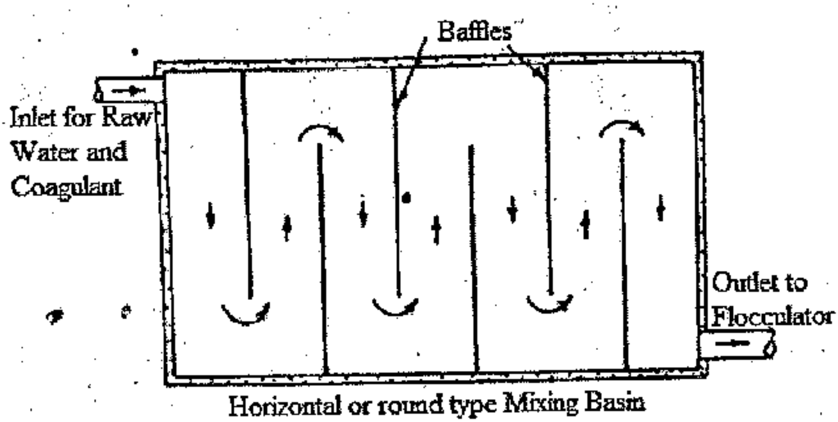


Figure 6-12 Mixing basins with baffles

ii) **Vertical or over and under type**: The water with coagulant entering to the tank through inlet provided at one end of the basin flows up and down, as shown by arrows due to the presence of vertical baffle walls projecting alternately from the roof and floor of the tank, and finally it flows out through an outlet provided at the other end of the tank. . Figure 6-13 shows a mixing basin with vertical or over and under type.

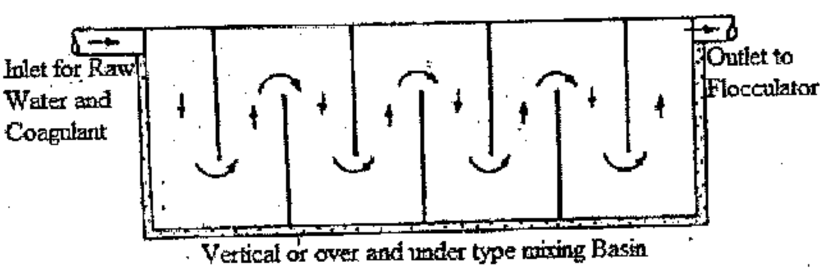


Figure 6-13 Mixing basins with baffles

b. Mixing basins with mechanical means

This is also known as flash mixture which is mechanically driven impeller or paddle as shown in figure 6-14. A flash mixture consists of a deep, circular or square tank provided with a propeller type impeller fixed at the lower end of a vertical shaft which is driven by an electric motor. Mixing basin or tank with baffle walls is usually used for small treatment plant where as mechanically driven paddles is used for large treatment plant. Detention period of 0.5 to 1 minute is provided and impeller is rotated at a speed more than 100 rpm.

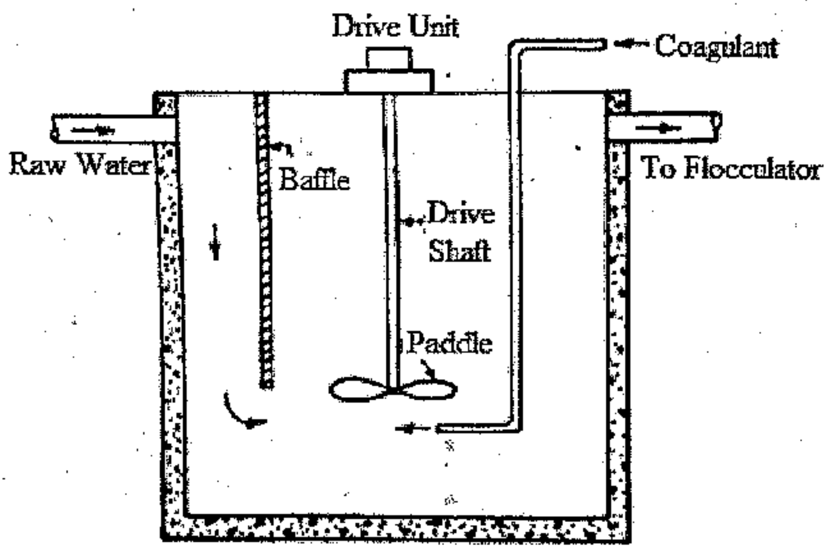


Figure 6-14 Mixing with Mechanical Means

c. Mixing with channel

Vertical baffles are provided in the channel so that projecting in an inclined position from both the sides of the channels. The water flowing through the channel strikes against these baffles and creates violent agitation which cause through mixing of raw water with the coagulant. As shown in figure 6-15, sometimes a flume may be entertained to develop hydraulic jump in the channel.

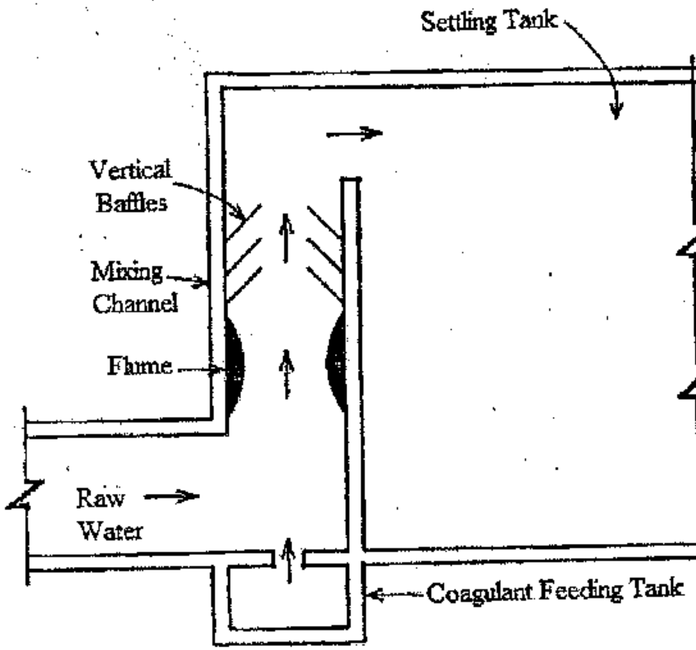


Figure 6-15 Mixing with Channel

d. Hydraulic jump

In such kind of device a flume is with considerable slope is adjusted in the channel to pass flow of a mixture of raw water and coagulant. Thus a hydraulic jump is created when water containing coagulant flows through the flumed sloping part of the channel.

e. Compressed air

In this method the water is fed with the coagulant into the basin. Diffused air is supplied from the bottom of the mixing basins or tanks. The compressed air while rising through water induces mixing of the coagulant with the raw water.

f. Centrifugal pumps

The mixing of coagulant with raw water could be done at the period of rising water in sedimentation tanks which is achieved by introducing coagulant in the suction pipe of the pump. The agitation generated during the passage of water through impeller of the pump cause mixing of coagulant with water. After such mixing either slight agitation is desirable or increase in detention period of sedimentation tank is required otherwise floc formation will take place slowly.

3. Flocculation

Following mixing device flocculator is installed for flocculation. In a flocculator slow stirring of water is brought about to permit build up and agglomeration of the floc particles. Commonly mechanical flocculator is in practice and same is described below.

Mechanical flocculator consists of basins provided with paddles for stirring of water hence also known as paddle flocculator. As per the direction of flow water in the basin, the mechanical flocculator may be longitudinal flow flocculator and vertical flow floculator.

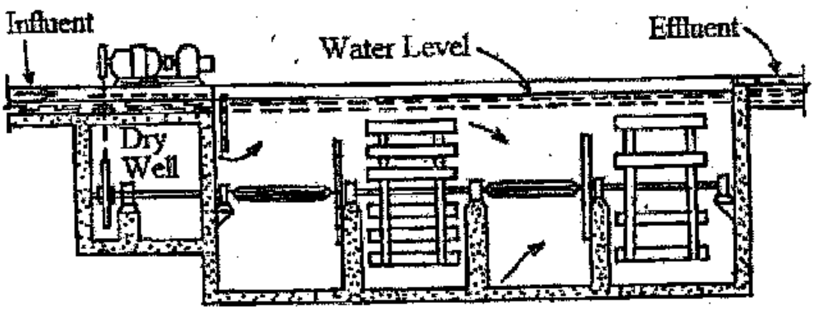


Figure 6-16 Longitudinal Flow Flocculator

A longitudinal flow flocculator as shown in figure 6-16 consists of a rectangular basin provided with paddles removing on a horizontal shaft and a vertical flow flocculator as shown in figure 6-17 consists of a circular tank provided with paddles removing on a vertical shaft. The paddles are rotated by an electric motor.

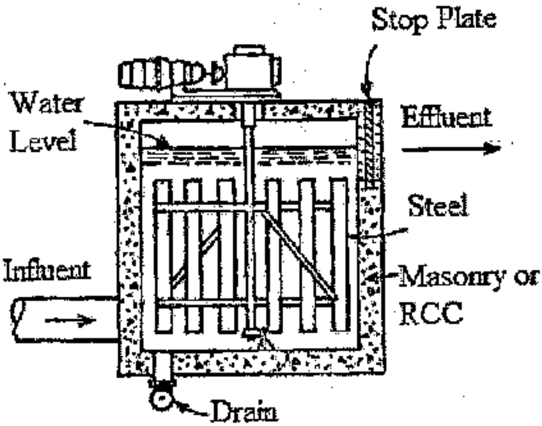


Figure 6-17 Vertical Flow Flocculator

4. Sedimentation

Following flocculation chamber a sedimentation basin (see figure 6-18) is installed to trap floc which is produced in flocculation chamber. This basin or tank is also known as clarifier or coagulation tank (or basin). In the sedimentation tank the flocculated water is retained for a sufficient period so that the floc along with the suspended particles of clay and silt and colloidal matter present in water will get settle down to the bottom of the tank.

It is possible to combine flocculation with sedimentation tank and such tank is known as coagulation-sedimentation tank. The detention period for the floc chamber is about 15 to 40 minutes and that for the sedimentation tank is about 3 to 4 hrs. The depth of floc chamber is usually about half that of sedimentation tank. The overflow rate for sedimentation tank is about 30 to 40 $m^3/m^2/day$.

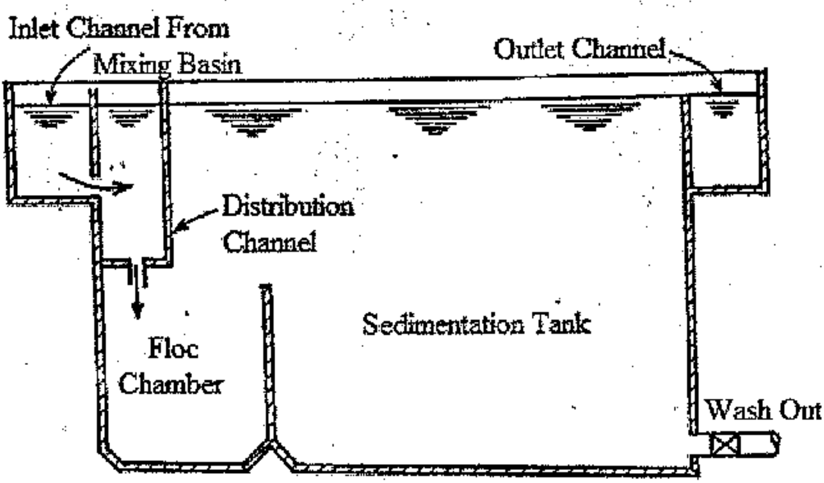


Figure 6-18 Coagulation Sedimentation Tank

Dorr clariflocculator

Dorr co. introduced a clariflocculator incooperating entire all four units (feeding, flash mixing, flocculation and clarification) in a single compact unit known as dorr clariflocculator as shown in figure 6-19.

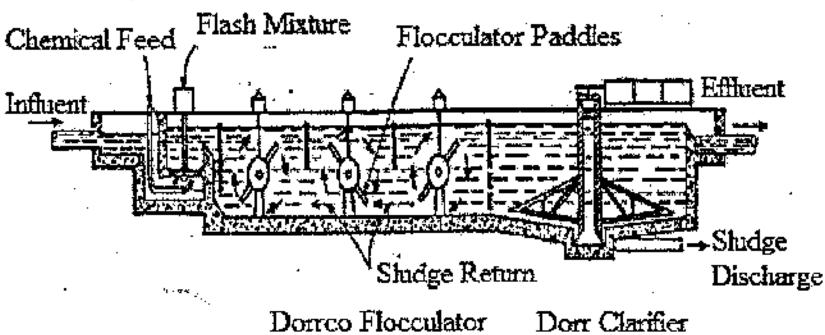


Figure 6-19 Dorr Clariflocculator

6.5.2 Jar Test

Water quality obtained in source does not remain same all the days. Especially in rainy days it picks up lots of suspended impurities and relatively clear water during non rainfall period. Hence quantity of chemicals added as coagulant for good floc formation in raw water may also varies daily in the range of 5 to 30 ppm. For effective removal of particles a suitable dose of coagulant is needed for floc formation as known as optimum dose of coagulant. Hence in order to determine optimum dose of coagulant jar test is undertaken in laboratory.

The optimum dosage of coagulant determined in laboratory for the water is as following.

1. Equal sample (1 liter commonly) of water is taken in six jars.
2. Record the initial turbidity, pH, temperature of water sample.
3. Add coagulant (Alum) in each jar with varying dose as of 5 mg, 10 mg, 15 mg, 20 mg, 25 mg and 30 mg.
4. Arrange the jars in the apparatus as shown in figure 6-20.
5. Then rapid mixing of chemicals in water is done at 100 rpm for 2 minutes and reduced for flocculation to 20 rpm for 10 to 15 minutes.
6. Then about 15 to 30 minutes is allow for floc settle.
7. Finally temperature, pH and turbidity are recorded and the jar with minimum turbidity that gives the best floc for the clear water. Hence the optimum dose of coagulant will be dose that added to jar which gives minimum turbidity.

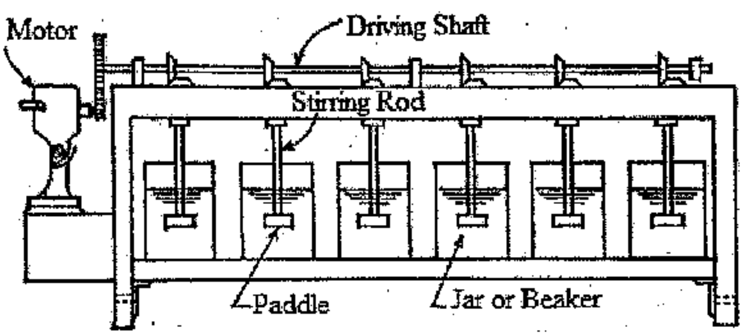


Figure 6-20 Jar Test Apparatus

6.6 Filtration

As practiced in modern water treatment plants, filtration is most often a polishing step to remove small flocs or precipitant particles not removed by settling of coagulated waters. Considerable amount of suspended particles of clay and silt and colloidal matter present in raw water will be removed in sedimentation with coagulation, but even after that it may contain certain impurities which need to remove before supplied to the consumers. Sedimentation with coagulation is not effective in removing finer floc, colour, dissolved minerals, and microorganisms or bacteria from water. So that the operation called filtration is adopted which can remove such impurities. The most commonly used filtration process involves passing the water through a stationary bed of granular medium.

Filtration is the process of passing water through thick layers of porous media which in most of the cases is a layer of filter media (sand) supported on a base material (gravel). Impurities in water are retained by the filter medium.

6.6.1 Theory/Principle of filtration

It has been noticed that during filtration various action take place which can be explained on the basis of the following actions;

1. Mechanical straining
2. Sedimentation and adsorption
3. Biological metabolism
4. Electrolytic action

1. **Mechanical straining**

   Suspended impurities having larger size than that the size of the interstices or voids between the sand grains cannot pass through these interstices, and are trap and removed by the action of mechanical straining. But it cannot be strained out or remove colloidal matter or bacteria because of small in size.

2. Sedimentation and adsorption

The interstices or voids between the sand grains act as minute sedimentation tanks in which suspended particles settle and removed.

Due to the physical attraction between the sand grain and the suspended particle and due to the presence of a gelatinous coating formed on the sand grains which is by already deposited colloidal matter and bacteria. This action can remove finer suspended particles, colloidal and bacteria.

3. Biological metabolism

Organic impurities such as algae, plankton which are caught in voids of sand grains is utilize by the bacteria as the food for their survival. Hence bacteria is responsible for convert them into harmless compounds. The harmless compounds so formed are deposited at the surface of the sand in the form of a layer which contains a zoological jelly called 'dirty skin' which further support in absorbing and straining out the impurities. Bacteria are not only responsible for break down the organic impurities and convert them into harmless compounds, but also destroy each other and maintain a balance of life in the filter.

4. Electrolytic action

Ionic theory states that, when two substances with opposite electric charges come in contact with each other, the electric charges are neutralized and in doing so, new chemical substances are formed. In filer, filter media also have charge of some polarity and particles of suspended and dissolved matter having opposite polarity when come into contact they neutralize each other and results in changing the chemical characteristics of water. After a interval of time filter media (sand) gets exhauster and it require to clean filter for regeneration of charges.

6.6.2 Types of filters

Filter are classified on the basis of either filtration rate as slow sand and rapid sand filter or the driving force to overcome the frictional resistance encountered by water flowing through filter as gravity and pressure. Thus combining these there are following three types of filter as;

1. Slow sand filters
2. Rapid sand filter
3. Pressure filter

6.6.2.1 **Slow sand filters**

These are the earliest type of gravity filter developed by James Simpson in 1829 in England. Filtration rate of slow sand filter is usually one–twentieth (or less) of the rate of rapid sand filter or pressure filter. These filters require large areas of land and correspondingly large quantity of filter media (sand) and base material (gravel). Cleaning of filter is done by surface scrapping which may involves a lot of labour. Slow sand filter is suitable when availability of land, labour, filter media are at low cost.

A distinguishing feature of slow sand filters is the presence of a thin layer, called the schmutzdecke, which forms on the surface of the sand bed and includes a large variety of biologically active microorganisms. It is a very simple and effective technique for purifying surface water. It will remove practically all of the turbidity from the water as well as most of the pathogens without the addition of chemicals. If turbidity of raw water is high then plain sedimentation would be require to reduce turbidity to some extent so that the filters are not unduly loaded.

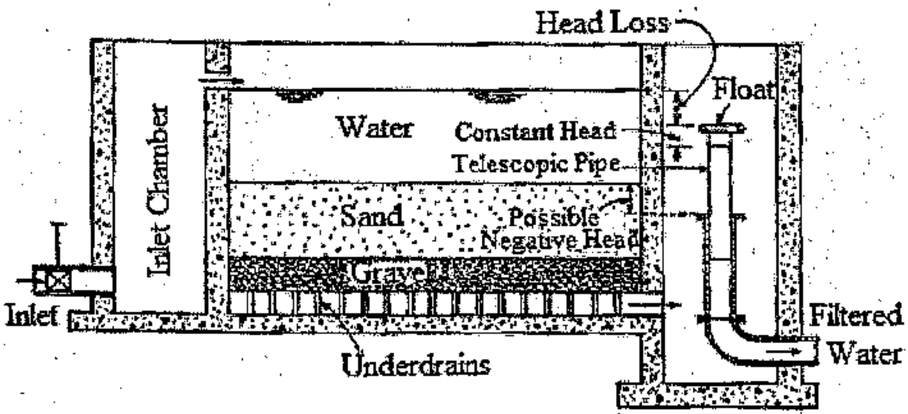


Figure 6-21 Slow Sand Filter

Figure 6-21 shows the section of a slow sand filter and slow sand filter consists of following parts;

a) Enclosure tank
b) Filter media
c) Base material
d) Underdrainage system
e) Appurtenances

a) Enclosure tank

An open watertight rectangular tank constructed of brick masonry or stone masonry or concrete. The tank having depth of 2.5 to 3.5 m, surface area 50 to 1000 m^2 or more depends upon the filtration rate which varies from 100 to 200 lit/hr/m^2. The floor of the tank is provided at the cross slope of 1 in 100 to 200 towards the central drain.

b) Filter media

It consists of sand layer of 90 to 110 cm thick with effective size of 0.25 to 0.35 mm (0.3 mm common) and uniformity coefficient C_u of 3 to 5. Finer the sand increases the removal efficiency of turbidity and bacteria but decrease filtration rate. The filter media (sand) should not contain Ca and Mg more than 2%.

c) **Base material**

The filter media is supported on the base material (gravel) of 30 to 75 cm thick bed. The gravel bed is graded and it laid in different layers each of 15 cm thick. A typical section of base material as indicated below.

	Thickness	Size
Top layer	15 cm	3 to 6 mm
Intermediate layers	15 cm	6 to 20 mm
	15 cm	20 to 40 mm
Bottom layer	15 cm	40 to 65 mm
Total	60 cm	

d) **Under drainage system**

Under drainage system supports the filter media and the base material and collects the filtered water and delivers it to the clean water reservoir. A central drain receives filter water from lateral drains. The lateral drains are placed at a distance of 2 to 3 m and ended about 50 to 80 cm from the walls of the tank as shown in figure 6-22.

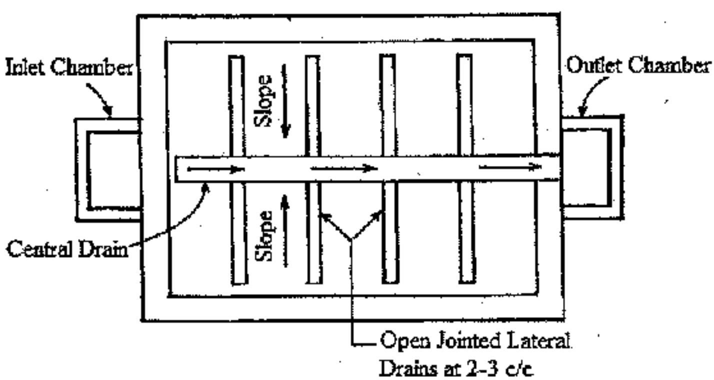


Figure 6-22 Under Drainage system of slow Sand Filter

e) **Appurtenances**

For efficient and proper functioning of filter certain devices are installed during construction known as appurtenances such as vertical air pipes, depth controlling devices, head loss measuring devices, flow regulator etc.

Filter bed requirement (Slow sand filter): As per Design guide lines for Rural Water Supply and Treatment Government of Nepal (DWSS) for slow sand filter. Design period 10 to 15 years, filter beds as indicated below.

- Minimum - 2 nos.
- Area upto 20 m^2 - 2 nos.
- Area 20 to 249 m^2 - 3 nos.
- Area 250 to 649 m^2 - 4 nos.
- Area 650 to 1200 m^2 - 5 nos.
- Area 1201 to 2000 m^2 -6 nos.

Working and cleaning of slow sand filter

Water from the sedimentation tank is permitted to enter the filter through inlet. The depth of water over the filter media is generally kept equal to the thickness of the layer of filter media. The water percolate through filter media and during this water gets purified. The purified water is collected by under drains and comes out from outlet from where it is taken to clear water storage reservoir.

Slow sand filter is operated up to 65 to 85% of the thickness of the sand bed. Cleaning of filter requires after period of 1 to 3 months. Top layer of filter media is scrapped or removed through depth of 15 to 30 mm (25 mm commonly). After cleaning the filter is operated at one-fifth of normal rate for 12 to 15 hrs then filtration rate kept one-third of normal rate for 3 to 4 days. After this period the filter is operated at its normal rate. Scrapping is done till thickness of filter media reaches to 60 cm. When this stage reached, filter media is replaced up to its original level.

Efficiency of slow sand filter

1. Bacteria removal efficiency of slow sand filter is quit efficient i.e. it is about 98 to 99%of bacterial load from raw water. It can be achieve up to 99.9% when pretreatment is done.
2. It can remove turbidity to the extent of about 50 ppm.
3. 20 to 25% of colour can be removed.
4. Colloidals cannot be removed efficiently.

Advantages of Slow Sand Filter

The cost of construction is low, and its simplicity of design and operation means that slow sand filters can be built and used with limited technical supervision. Little special pipe work, equipment, or instrumentation is needed, and the labour required for maintenance can be unskilled as the major labour requirement is in cleaning the beds, which can be done by hand. Imported materials and equipment is usually negligible and no chemicals are required. Likewise, power is not required if a gravity head is available, and there are no moving parts or requirements for compressed air or high-pressure water. Variations in raw water quality and temperature can be accommodated, provided turbidity does not become excessive, and overloading for short periods does no harm.

Compared to rapid sand filtration, there is a net savings of water as large quantities of backwash water are not required.

Disadvantages of Slow Sand Filter

Slow sand filtration units require large land areas for plants treating large flows (about five times that of rapid sand filtration plants). Clogging may occur if the source water is excessively turbid or if certain (filamentous) types of algae are present in the raw water. Pre-treatment with roughing filters or settling tanks may be necessary if such clogging occurs frequently. Also, toxic chemical contamination of the raw water may affect the biological surface layer (this could be a good indication of water source problems)

6.6.2.2 Rapid sand filter

Rapid sand filtration is a technique common in developed countries for treating large quantities of drinking water. It is a relatively sophisticated process usually requiring power-operated pumps for backwashing or cleaning the filter bed, and flow control of the filter outlet. A continuously operating filter will usually require backwashing about one to three days (commonly in 2 days) when raw water of relatively low turbidity is used. Pretreatment of the raw water, using chemical flocculation agents in combination with setting tanks is common where turbidity is high. Generally water is allowed from sedimentation with coagulation to the rapid sand gravity filter. Figure 6-23 shows the section of rapid sand filter consists of the following parts;

a) Enclosure tank
b) Filter media
c) Base material
d) Under drainage system
e) Appurtenances

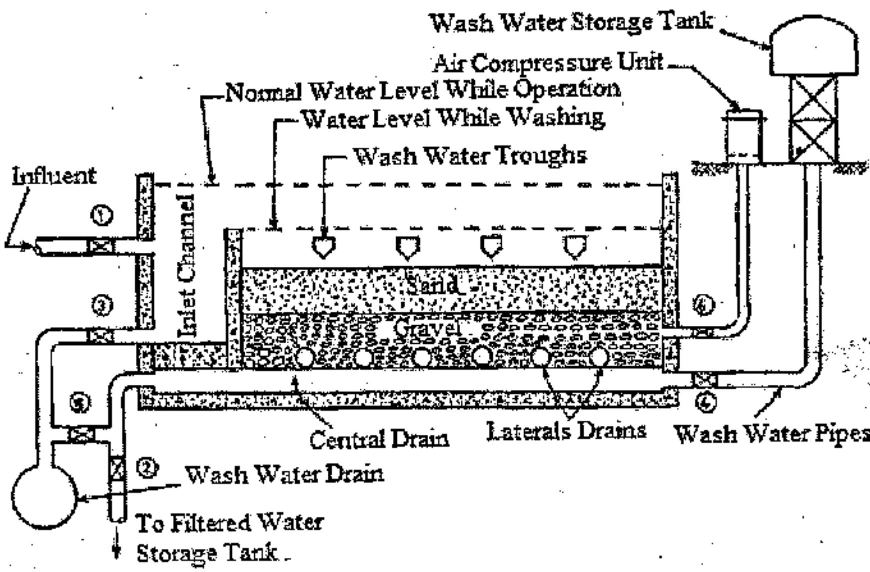


Figure 6-23 Rapid Sand Filter

a) **Enclosure tank**

Open and watertight rectangular tank constructed of brick or stone masonry or concrete. The depth of the enclosure tank is about 2.5 to 3.5. Surface area or plan area of the tank may vary from 10 to 50 m^2 and length to breadth ratio of the tank is normally kept 1.25 to 1.35. The filtration rate varies from 3000 to 6000 lit/hr/m^2. Figure 6-25 shows a view of rapid sand filter. A view of filter tank has shown in figure 6-25.

b) **Filter media**

It consists of sand bed or sand layer of 60 to 75 cm thick having effective size (D_{10}) 0.45 to 0.7 mm. Uniformity coefficient (Cu) of filter media varies from 1.3 to 1.7 and commonly 1.5.

c) **Base material**

The filter media is supported on the base material (gravel) of 45 to 60 cm thick bed. The gravel bed is graded and it laid in different layers each of 15 cm thick. A typical section;

	Thickness	**Size**
Top layer	15 cm	2 to 6 mm
Intermediate layers	15 cm	6 to 12 mm
	15 cm	12 to 20 mm
Bottom layer	15 cm	20 to 50 mm
Total	60 cm	

d) **Under drainage system**

Figure 6-24 shows the under drainage system of rapid sand filter. In rapid sand filter under drainage system acts two purposes; i) to collect filtered water and ii) to provide uniform distribution of backwash water. There are various under drainage systems for these filters out of which following two systems are commonly adopted are described below.

i) Perforated pipe system

ii) Pipe and strainer type

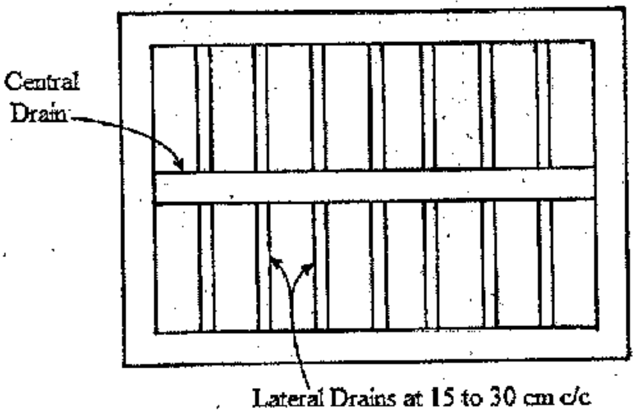


Figure 6-24 Plan of under Drainage Structure of Rapid Sand Filter

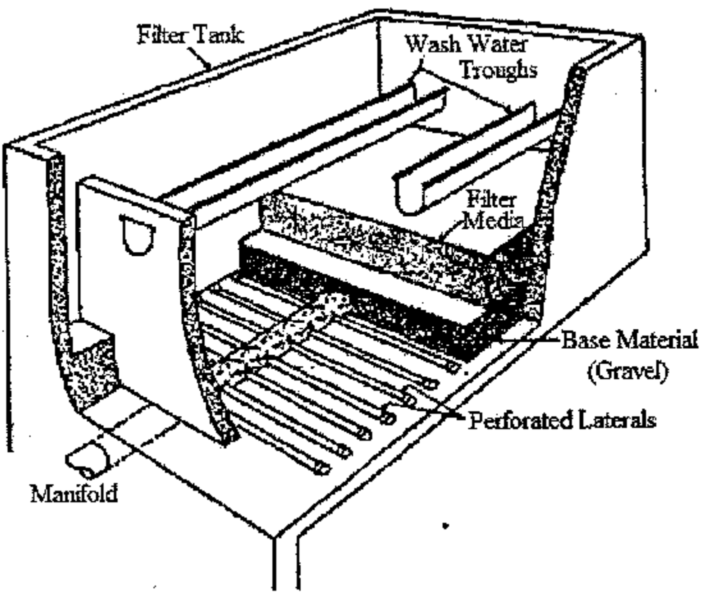


Figure 6-25 Rapid Sand Filter

i) **Perforated pipe system**

This system consists of a central drain or manifold to which a number of lateral drains are connected on either side as shown in figure 6-26. The lateral drains are provided at a spacing of 15 to 30 cm with perforations. This system is economical and simple in operation. However, more quantity of water in high velocity needs for the back washing of filter.

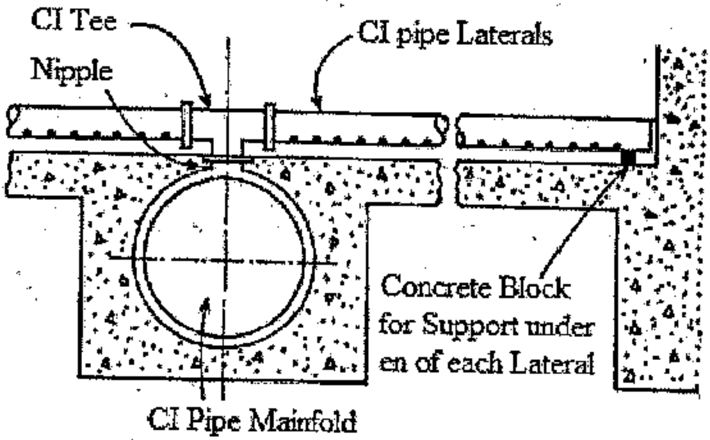


Figure 6-26 Perforated Pipe

ii) **Pipe and strainer system**

This system also consists of a central drain or manifold to which a number of lateral drains are connected on either side as in figure 6-27. Holes are drilled at the top of the laterals and each hole is provided with strainer. The strainers are either screwed or fixed on the top of the laterals drains. Generally, spacing of strainers placed at 15 to 30 cm.

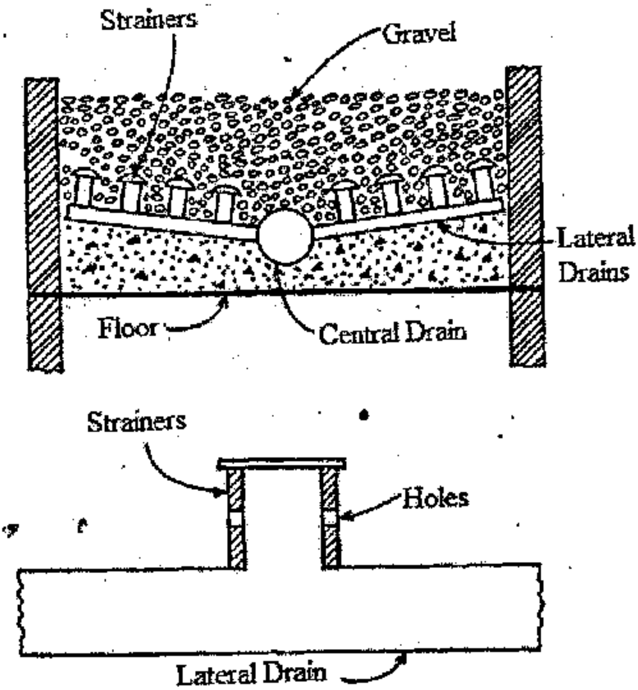


Figure 6-27 Pipe and Strainer of Rapid Sand Filter

e) **Appurtenances**

For efficient and proper functioning of filter certain devices are installed during construction known as appurtenances such as wash water trough, air compressor, valves, head loss measuring devices, flow regulator etc.

Working and cleaning of Rapid Sand Filter

At the normal operation stage of rapid sand filter only valves 1 and 2 are opened and rests of other valves are kept closed. Valve 1 (inlet) is opened to allow water from sedimentation with coagulation and valve 2 (outlet) is opened to deliver filtered water to the storage tank. Water level over the sand bed varies between 1 to 2 meter. Back washing is usually done when head loss is reached to between 2.5 to 3 m or filter cleaning is done at a interval of 1 to 3 days. Normally for back washing 10 to 15 minutes may require but for restart operation total time of about 30 minutes may required. For backwashing filter, first step is to close valve 1 and let to decrease water level to the edge of wash water troughs. Next step is to close valve 2, now open valve 6 to permit compressed air for about 2 to 3 minutes in upward direction surface scum will break-up and loosen the dirt. Close valve 6 and open valve 4 to permit wash water. Open valve 3 to discharge dirty wash water through wash water drain. Now close valve 4 an valve 3 and allow certain time to settle materials to the surface of sand. Now open valve 1 slightly and open valve 5 for few minutes to discharge filtered water to wash water drain. Finally close valve 5 and open valve 2. Now filter can be operated at normal filtration rate.

Efficiency of rapid sand filter

1. It is excepted that bacteria removal efficiency is about 80 to 90%
2. Turbidity of water can remove the extent of 35 to 40 ppm.
3. These filters are highly efficient in colour removal i.e. below 3 on cobalt scale.

Advantages of Rapid Sand Filters

The advantages of this technology are that it is a proven technology, effective in removing suspended solids, and that it requires a minimal land area for construction and operation compared to slow sand filters.

Disadvantages

Rapid sand filters have high capital and operation costs, which may be increased further if there is a need for pretreatment of the raw water. The technology uses energy for pumping, and requires a relatively high degree of training for the plant operator.

6.6.2.3 Pressure filter

Pressure filter is also a kind of rapid sand filter consists of closed steel cylindrical tanks and through which water to be purified passes under pressure of 3 to 7 kg/cm^2. Pressure filter may be horizontal or vertical type as shown in figure 6-28 (a) and (b). Specification like filter media, base material used to pressure filter are the same as those provided in rapid sand filter. Also water is allowed to filter from sedimentation with coagulation.

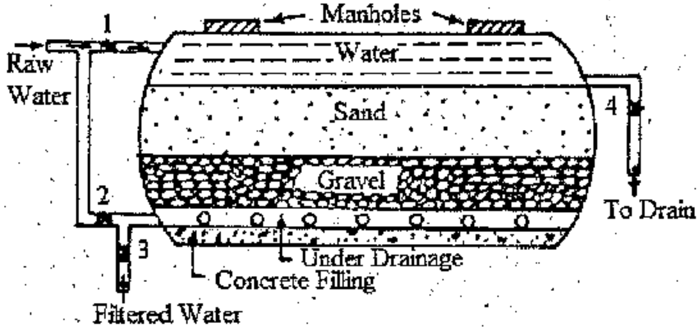


Figure 6-28 (a) Pressure Filter of Horizontal type

Back washing of filter is facilitated with automatic pressure control system and operated at a fixed interval of time. Filtration rate of pressure filter is 6000 to 15000 lit/hr/m^2 of filter area. Water treated from pressure filter is not suitable for public water supply due to high cost and inefficiency of filter. However, pressure filter can be installed for small colonies, industrial plants, swimming pools etc.

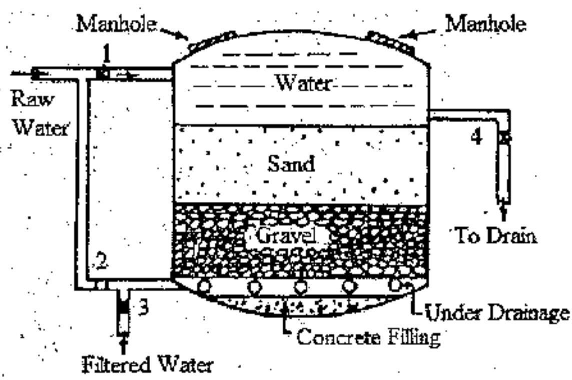


Figure 6-28(b) Pressure Filter of Vertical type

Table 6-3 Comparison of Slow and Rapid sand filter

Item	Slow Sand Filter	Rapid Sand Filter
Area requirement	50 to 1000 m^2	10 to 50 m^2
Filtration rate	100 to 200 lit/hr/m^2	3000 to 6000 lit/hr/m^2
Depth of enclose tank	2.5 to 4 m	2.5 to 3.5 m
L:B ratio	2:1	1.25 to 1.35:1
Filter media	Sand layer 90 to 110 cm, effective size (D_{10}) 0.25 to 0.35 Cu 3 to 5	Sand layer 60 to 75 cm, effective size (D_{10}) 0.45 to 0.75 Cu 1.3 to 1.7
Base material	Depth 30 to 75 cm, effective size (D_{10}) 3 to 65 mm	Sand layer 60 to 90 cm, effective size (D_{10}) 2 to 50
Function of under drains	Only collect filtered water	Collect filtered water and distribution of backwashed water
Method of cleaning	Surface scrapping (20 to 30 mm)	Backwashing
Period of cleaning	1 to 3 months	1 to 3 days
Pretreatment	Sedimentation	Sedimentation with coagulation
Supplementary treatment	Chlorination	Chlorination (essential)
Suitability	For small town if land value is cheap	For big cities
Amount of wash water	0.2 to 0.6 % of filtered water	2 to 5 % of filtered water
Loss of head	15 cm initially and up to 1 m	30cm initially and up to 2 m
Efficiency	Efficient in bacteria and suspended impurities removal	Efficient removal of colour, odour, taste and turbidity, but less efficient in bacteria removal.

6.7 Disinfection

Boiling water may be effective as a method of disinfection, but it is not practical for large quantities. Sunlight can also act as a natural method of disinfection, but it is difficult to control and manage. Thus, chemical disinfection, principally using chlorine products, is practiced extensively. Disinfection also may be achieved with ultraviolet (UV) light, and is best suited to individual household applications, although larger-scale units are being used in some countries. However, UV disinfection requires a power supply is required and does not provide the residual level of disinfection in the water as chlorine does.

The use of chlorine has become practically universal for the disinfection of water. As practiced in water purification, disinfection refers to operations aimed at killing, or rendering harmless pathogenic microorganisms. The substances or agent used for disinfection termed as disinfectant. Another term of disinfection is sterilization, and is the complete destruction of all living matter. The purpose of disinfection might be the reduction of water born diseases.

Criteria for a good disinfectant:

1. It should have a fast rate of kill and should be persistent enough to prevent regrowth and recontamination of microorganisms in the distribution system.
2. It must be toxic to microorganisms at concentrations well below the toxic thresholds to humans.
3. It should be easily available at reasonable cost.
4. It should be safe to handle, and its method of application should be simple.

Methods of disinfection:

A variety of disinfection methods may be used in special circumstances. Chemical agents such as the halogen group (iodine, bromine), oxidants potassium permanganate, metals (copper, silver) and physical means like gamma wave or ultraviolet light, electrocution, heating or boiling. Out of various methods of disinfection, chlorination is most commonly adopted in least developed country like Nepal. So that chlorine can remain in residual form to persist against recontamination. However, a variety of other methods can be used to disinfect water. The Table 6-3 below summarizes disinfection methods and its uses.

Table 6-4 Disinfection Method, Processes and Uses

Disinfection Method	**Disinfection Process (Advantages & Disadvantages)**	**Uses**
Chlorine	chemical reaction with pathogens a small dose kills bacteria rapidly; residual can be maintained in some cases; chlorination can cause the formation of trihalomethanes	widespread use to disinfect water; also used in color, taste, and odor removal, improving coagulation, and killing algae.
Iodine	chemical reaction with pathogens good disinfectanthigh cost; harmful to pregnant women	emergency treatment of water supplies; disinfecting small, non-permanent water supplies
Bromine	chemical reaction with pathogens handling difficulties; residuals hard to obtain; supply is limited	very limited use, primarily for treating swimming pool water
Bases (sodium hydroxide and lime)	chemical reaction with pathogens bitter taste in the water; handling difficulties	sterilize water pipes
Ozone	chemical reaction with pathogens good disinfectant; better virucide than chlorine; oxidizes iron, manganese, sulfide, and organics; removes color, odor, and taste high cost; lack of residual; storage difficulties; maintenance requirements; safety problems; unpredictable disinfection; no track record	disinfection; treating iron and manganese, helping flocculation, removing algae, oxidizing organics, removing color, treating tastes and odors
Ultraviolet	UV light causes biological changes which kill the pathogenslack of dangerous by-productslack of measurable residual; cost of operation; turbidity interferes with disinfection	small or local systems and industrial applications
Ultrasonic	sound waves destroy pathogens by vibration very expensive	
Heat	boiling water for about five minutes will destroy essentially all microorganisms simple, requires little equipment very energy intensive; expensive	Individuals may boil their water for household quantities of water when quality of water is questionable

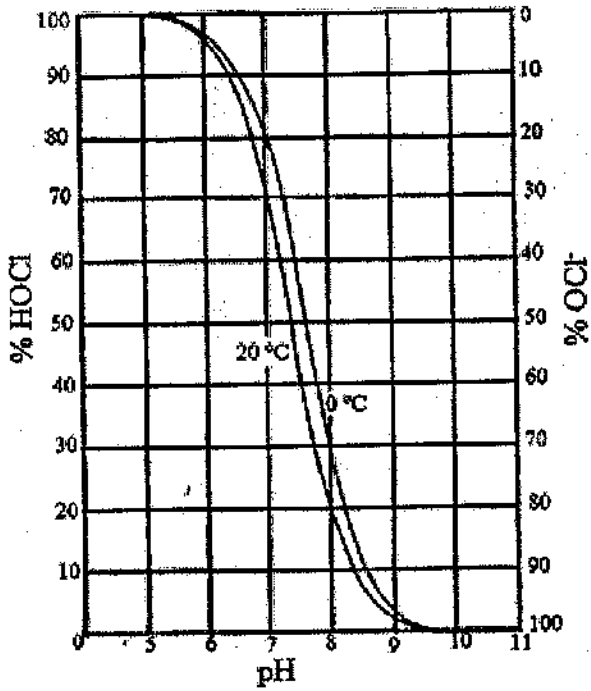


Figure 6-29 Distribution of HOCl and OCl^- as a Function of pH

Chlorination *(Theory)*

When chlorine is added to water, the following reaction take place;

$Cl_2 + H_2O \leftrightarrow HOCl + H^+ + Cl^-$ (Hydrolysis)

The hypochlorous acid (HOCl) further dissociates into hydrogen ions (H^+) and hypochlorite ions (OCl^-) as indicated below.

$HOCl \leftrightarrow H^+ + OCl^-$ (Ionization)

As per the enzymatic hypothesis, the hypochlorous acid (HOCl) and hypochlorite ions (OCl^-) penetrate into the cell wall and react the enzymes and protoplasm resulting the end of life of the microorganism.

Hypochlorous acid (HOCl) is the more effective disinfectant, which is 80 to 100 times destructive than hypochlorite ions (OCl^-). The sum of HOCl and OCl^- is called free chlorine residual. At pH value 5.5 and below, chlorine exists in molecular form, if pH is between 5 to 9.5 HOCl and OCl^- is formed. HOCl is generally formed at the pH value of 5 to 7. Chlorine water is unstable and may be decompose rapidly during expose to sunlight. Distribution of HOCl and OCl- as a function of pH has shown in figure 6-29.

Chlorine demand

The amount of chlorine consumed in the oxidation of organic and inorganic matters present in water is known as chlorine demand of water. After the fulfillment of chlorine demand applied chlorine in water remains as residual chlorine. *Chlorine demand is equal to the difference between the amount of chlorine added to water and the amount of residual chlorine after contact period.*

Residual Chlorine

Amount of chlorine remaining in water after chlorine demands satisfied is residual chlorine. The amount of unreacted chlorine left in water as residual chlorine in distribution line will acts as safeguard against recontamination.

Dosage of chlorine

The dose of chlorine is the required amount of chlorine to be added which leaves about 0.2 mg/lit after contact period of 10 minutes.

Chlorine dosage = Chlorine demand + Residual chlorine

Contact Period

Microorganisms present in water do not die instantaneously at the chlorine applied. Contact period is the time required to destroy pathogens life after the application of chlorine. Time commonly assumed to kill bacteria after application of chlorine in water is about 10 minutes.

6.7.1 Types (Forms of application) of chlorine

1. Bleaching powder or hypochlorite
2. Chloramines
3. Chlorine gas or liquid chlorine
4. Chlorine dioxide

1. Bleaching powder or hypochlorite

Calcium hypochlorite $Ca(OCl)_2$ or bleaching powder is a chlorinated lime. When it is added to water dissociates into calcium Ca^{++} and hypochlorite OCl^- ions as;

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

$$H^+ + OCl^- \leftrightarrow HOCl \qquad \text{(hypo-chlorination)}$$

Commercially available bleaching powder contains approximately 30 to 35 percent of available chlorine. Chlorine is unstable compound which goes on losing its content when exposed to atmosphere so requires careful storing. Bleaching powder is not used for large public water supply schemes but can be adopted for small colonies, swimming pools etc. Common dosage of bleaching powder may be about 0.5 to 2.5 kg per million liters of water.

2. Chloramines

Chloramines can be formed by first adding a small quantity of ammonia (1 part) to the water, then adding chlorine (4.5 parts). The reactions involved chlorine with ammonia as;

$$NH_3 + HOCl \rightarrow NH_2Cl \text{ (monochloramine)} + H_2O$$

$$NH_2Cl + HOCl \rightarrow NHCl_2 \text{ (dichloramine)} + H_2O$$

$$NHCl_2 + HOCl \rightarrow NCl_3 \text{ (nitrogen trichloride)} + H_2O$$

These reactions are dependent on several factors, the most important of which are pH, temperature, and reactant quantities.

Since combined residuals are less effective as a disinfectant, contact period in excess of 30 min is often required.

3. Chlorine gas or liquid chlorine

Chlorine gas may be use in two ways (i) directly to the point of application to the water supply with a pressure of 7 kg/cm^2 (ii) it may be applied in a solution form (dissolved in small quantity of water) to the point of application to the water supply. When chlorine gas is applied under pressure may be less expensive but it is less satisfactory because of poor diffusion of chlorine in water. At low temperatures crystalline hydrates of chlorine formed and there may be possibility of chocking of pipes also undissolved chlorine gas may corrode valve and pipes. As such only second method is often adopted.

4. Chlorine dioxide

ClO_2 has many of the same properties as ozone. A strong oxidant which forms neither chloroforms nor chloramines, it is particularly effective in oxidizing phenolic compounds. Its application widely has been in wastewater disinfection and has limited use in potable water treatment for oxidizing iron and manganese and removal of taste and odour compounds. Its possible reduction to chlorate (substance toxic to humans) makes questionable its use in potable water.

$$2NaClO_2 + Cl_2 \leftrightarrow 2NaCl + 2Cl_2\uparrow$$

6.7.2 Forms of chlorination

Depending upon the step of purification at which chlorine is added to water and also upon the desired result of application of chlorine, which may be of the following forms;

1. Simple or plain chlorination
2. Pre-chlorination

3. Post chlorination
4. Multiple or double chlorination
5. Break point chlorination
6. Super chlorination
7. De-chlorination

1. Simple or plain chlorination

In simple or plain chlorination only chlorine is added in raw water and no other treatment is given and supplied to the consumers. When raw water is relatively clear having low turbidity (not exceeding 10 NTU) plain chlorination is resorted with usual dosage of 0.5 to 1 ppm.

2. Pre-chlorination

In pre-chlorination chlorine is applied to raw water initially before any treatment. Chlorine is added before filtration sometimes may be before sedimentation. Dosage of chlorine should be such that residual chlorine of 0.1 to 0.5 ppm at the time of entering to filter. Pre-chlorination may assist or reduce burden to other unit by; controlling against growth of algae in sedimentation tank, reducing bacterial load to filter, interval of cleaning period of filter may prolong, coagulant dosage may reduce in sedimentation with coagulation, eliminating taste and odour from water.

3. Post chlorination

It is the application of chlorine to water after all treatments i.e. chlorine is added to water as it leaves filters and before enters to distribution system. It is useful for protections against contamination in the distribution system and dosage is adjusted about 0.1 to 0.2 ppm.

4. **Multiple or double chlorination**

When raw water is highly contaminated and may contains large number of bacteria, chlorine is added at two or more points in the purification process. Pre-chlorination is adopted once before sedimentation and post chlorination is in which chlorine is added after filtration and before distribution system.

5. **Break point chlorination**

Chlorine and chlorine compounds by virtue of their oxidizing power first react with inorganic materials present in water before any disinfection accomplished. The chlorine then performs disinfecting bacteria in water. Subsequently chlorine oxidized the organic matter present in water.

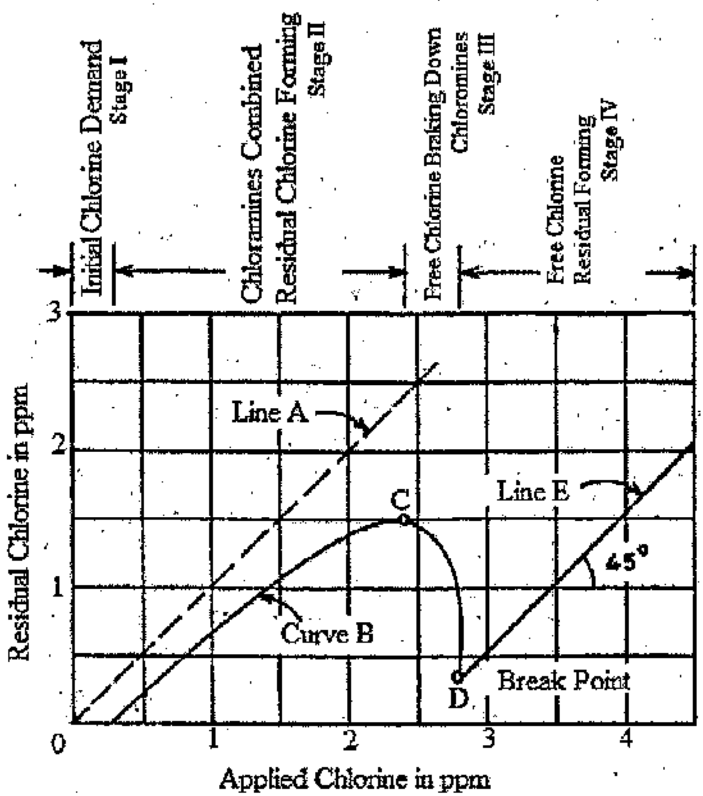


Figure 6-30 Break Point Chlorination

Hence, as chlorine added to water two actions take place;

i) kills bacteria present in water, thus disinfection is accomplished, and

ii) oxidized the organic matter present in water.

When chlorine is added to water and if water has no chlorine demand applied chlorine appear as residual chlorine hence relation between applied and residual chlorine will be as indicated by line A as shown in figure 6-30, having a slope of 45° as shown in figure. However, water generally has some chlorine demand which the relation of applied chlorine and residual chlorine will be indicated by curve B. As the chlorine added first it performs the function of destroying bacteria and also readily reacts with oxidiziable substances such as iron, manganese, nitrites, sulphides and organic matter that may present in water. During this entire applied chlorine is utilized for killing bacteria and oxidizing substances that may present in water and there is no residual chlorine indicated as stage I. After meeting immediate demand, the chlorine reacts with compounds such as ammonia, proteins, amino acids and phenols that may present in water to form chloramines and chloroderivatives which constitutes the combined available chlorine. At this II stage relation between applied chlorine and residual chlorine is represented by curve B and combined available chlorine also recorded as residual chlorine. At this stage increase in applied chlorine also increase residual chlorine and curves B further rises till point C where the amount of residual chlorine is recorded maximum. Further application of chlorine, there is sudden decrease in the residual chlorine due to the fact that the added chlorine break downs chloramines by changing them to nitrogen compounds, thus reducing the residual chlorine and also lots of applied chlorine is utilized in oxidation of organic matter present in water. This is stage III and is accomplished by bad smell. This stage is represented by curve CD. At the point D taste and bad smell suddenly disappears and oxidation of organic matter is also complete. Point D on the curve is known as break point because any chlorine that is added to the water beyond this point breaks through the water and appears as residual chlorine. *The break point chlorination of water may be defined as the point on the applied-residual chlorine curve at which all, or nearly all, the residual chlorine is free chlorine.*

The application of chlorine in water, with chlorine dosage equal to or slightly greater than that at of break point occurs is break point chlorination.

Hence, chlorine applied beyond point D results in an increase in residual chlorine as represented by line E the slope of which will be 45° so that the entire applied chlorine will appear as residual chlorine.

Advantages of break point chlorination

1. It can remove manganese, taste, odour.
2. It will have adequate bactericidal effect.
3. It contains desired chlorine residual.
4. It can complete the oxidation of ammonia and other compounds.

6. Super chlorination

In super chlorination chlorine is applied to water beyond the stage of break point chlorination. Normally dosage of super chlorination may be 2 to 3 mg/lit or 0.5 to 2 ppm after break point chlorination. Super chlorination is adopted during the epidemic in a certain area to control water born disease.

7. De-chlorination

It is the process of removing excess chlorine from water. The excessive chlorine is to be removed from the water to avoid chlorine taste before distribution to consumers. Which may be attain by aeration of water or adding sodium thiosulphate, sodium metabisulphate, sodium sulphite, ammonia, sulpherdioxde etc to water.

6.7.3 Factors affecting efficiency of chlorination

1. Turbidity
2. Presence of metallic compound
3. Ammonia compounds
4. pH vale of water
5. Temperature
6. Time of contact
7. Type, condition and concentration of microorganisms.

1. Turbidity

Turbidity of the water influences disinfection primarily through influencing the chlorine demand. Turbid water tends to contain particles which react with chlorine, reducing the concentration of chlorine residual which is formed.

2. Presence of metallic compound

Metallic compounds such as iron and manganese utilizes more chlorine for oxidation so that to increase effectiveness of chlorination metallic compound should be removed.

3. Ammonia compounds

The free chlorine can react with compounds such as ammonia, proteins, amino acids and phenol to form chloramines and chloroderivatives which obeys combined chlorine path and which is less effective in disinfection.

4. pH of water

Hypochlorous acid (HOCL) is formed effectively as shown is figure 6-29 in the pH range of 5 to 7 and which penetrate into the enzyme system of bacteria and killed them. When pH value of water is less than 5 and greater than 10 generally chlorine remains in molecular form. Hence pH is a major factor that directly affects the bactericidal removal efficiency.

5. Temperature of water

Decrease in temperature of water results substantial decrease in killing power of both free and combined chlorine. If temperature lowered, free available is decreased and efficiency also decreased.

6. Time of contact

Time period after application of chlorine required to kill pathogens should be adequate for effective disinfection. Generally for disinfection by free chlorine a contact period of 30 minutes is required while it takes twice of this for combined chlorine path.

7. Type, condition and concentration of microorganisms

Bacteria and virus are the microorganisms that they are commonly occurs in water. The enteric pathogenic bacteria are less resistant to chlorine than E-coli bacteria and viruses are more resistant hence dosage and contact time differ with type, condition and concentration of microorganisms.

6.8 Softening

The removal of hardness from water is known as water softening. It is the process of removing Ca^{2+} and Mg^{2+} from the water. There are two types of hardness;

(i) Temporary hardness

(ii) Permanent hardness

6.8.1 Removal of temporary hardness

Temporary hardness of water can be removed by the following methods:

i) By boiling (Heating)

It can be only used in household level not in public water supply.

$$Ca(HCO_3)_2 \xrightarrow{\Delta} CaCo_3\downarrow + CO_2\uparrow + H_2O$$

$$Mg(HCO_3)_2 \xrightarrow{\Delta} MgCo_3\downarrow + CO_2\uparrow + H_2O$$

The insoluble precipitates $MgCO_3$ and $CaCO_3$ are removed by sedimentation.

ii) Addition of lime

When lime is added to water following reaction takes place;

$$Ca(HCO_3)_2 + Ca(OH)_2 \rightarrow 2CaCo_3\downarrow + 2H_2O$$

$$Mg(HCO_3)_2 + Ca(OH)_2 \rightarrow MgCo_3\downarrow + CaCO_3\downarrow + 2H_2O$$

The insoluble precipitates $MgCO_3$ and $CaCO_3$ are removed by sedimentation.

6.8.2 Removal of permanent hardness

Permanent hardness of water can be removed by;

1. Lime-soda process
2. Zeolite process or base exchange
3. Demineralization (Deionization)process

1. Lime-soda process

In this method lime $Ca(OH)_2$ and sodium carbonate Na_2CO_3 (Soda ash) are added to water. Lime and sodium carbonate can be added simultaneously or separately to water. Generally half hour to one hour is given as contact time.

$$CO_2 + Ca(OH)_2 \rightarrow CaCO_3\downarrow + H_2O$$

$$Ca(HCO_3)_2 + Ca(OH)_2 \rightarrow 2CaCO_3\downarrow + 2H_2O$$

$$Mg(HCO_3)_2 + Ca(OH)_2 \rightarrow MgCo_3 + CaCO_3\downarrow + 2H_2O$$

$$MgCO_3 + Ca(OH)_2 \rightarrow Mg(OH)_2\downarrow + CaSO_4$$

$$CaSO_4 + NaCO_3 \rightarrow CaCO_3\downarrow + Na_2SO_4$$

$$MgCl_2 + Ca(OH)_2 \rightarrow Mg(OH)_2\downarrow + CaCl_2$$

$$CaCl_2 + Na_2Co_3 \rightarrow CaCo_3\downarrow + 2NaCl$$

$$MgCl_2 + Na_2Co_3 \rightarrow MgCO_3 + 2NaCl$$

The compounds calcium carbonate and magnesium hydroxide are insoluble in water and can be removed by clarifier.

2. Zeolite process or Base Exchange

In this process no chemicals are added to water but hard water is passed through a bed of ion- exchange material or zeolite, which has a property of interchanging base or ion. Artificially granular substance zeolite also called permutit, common permutit is sodium aluminum silicate ($SiO_2.Al_2O_3.Na_2O$) which is manufactured from feldspar, kaolin and soda.

$$2SiO_2Al_2O_3Na_2O + Ca(HCO_3)_2 \rightarrow 2SiO_2Al_2O_3CaO + 2NaHCO_3$$

$$2SiO_2Al_2O_3Na_2O + CaSO_4 \rightarrow 2SiO_2Al_2O_3CaO + Na_2SO_4$$

$$2SiO_2Al_2O_3Na_2O + CaCl_2 \rightarrow 2SiO_2Al_2O_3CaO + 2NaCl$$

$$2SiO_2Al_2O_3Na_2O + Mg(HCO_3)_2 \rightarrow 2SiO_2Al_2O_3MgO + 2NaHCO_3$$

$$2SiO_2Al_2O_3Na_2O + MgSo_4 \rightarrow 2SiO_2Al_2O_3MgO + 2Na_2SO_4$$

$$2SiO_2Al_2O_3Na_2O + MgCl_2 \rightarrow 2SiO_2Al_2O_3MgO + 2NaCl$$

If we indicate permutit as Na_2Z (Z is anionic exchanger) then,

$$Na_2Z + {}^{Ca}_{Mg}\begin{bmatrix} (HCO_3)_2 \\ SO_4 \\ Cl_2 \end{bmatrix} \rightarrow \begin{bmatrix} Ca \\ \\ Mg \end{bmatrix} Z + \begin{bmatrix} 2NaHCO_3 \\ Na_2SO_4 \\ 2NaCl \end{bmatrix}$$

Calcium and magnesium present in water is replaced by sodium and water. Sodium salts which are formed are soluble in water and do not impart hardness hence water get softened. The product $SiO_2Al_2O_3CaO$ and $SiO_2Al_2O_3MgO$ remains in zeolite. A typical zeolite water softener is shown in Figure 6-31;

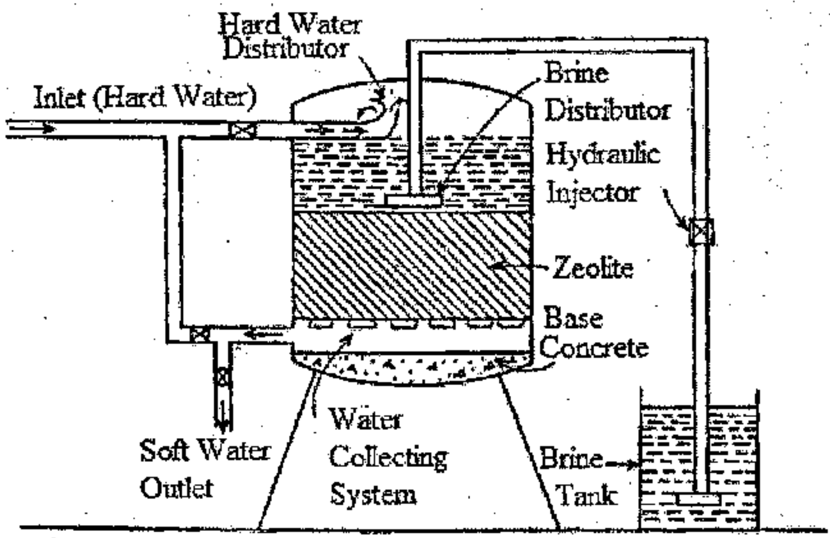


Figure 6-31 Typical Softeners

Regeneration of zeolite

Due to continuous use of zeolite the sodium present in it is exhausted and it requires to be regenerated to make it effective for removal of hardness. Passing a solution of salt through zeolite it can be regenerated. Reactions take place during regeneration as;

$$2SiO_2Al_2O_3CaO+2NaCl \rightarrow 2SiO_2Al_2O_3Na_2O+CaCl_2$$

$$2SiO_2Al_2O_3MgO+2Nacl \rightarrow 2SiO_2Al_2O_3Na_2O+MgCl_2$$

The above reactions indicate that sodium of salt solution replaces calcium and magnesium of the exhausted zeolite.

It can be written as;

$$\begin{bmatrix} Ca \\ Mg \end{bmatrix} Z + 2NaCl \rightarrow Na_2Z + \begin{bmatrix} Ca \\ Mg \end{bmatrix} Cl_2$$

Regeneration of hydrogen exchanger

Due to continuous use of hydrogen exchanger its hydrogen content is exhausted and it requires to be regenerated to make it effective for removal of hardness. Passing from a solution of sulphuric acid or hydrochloric acid of suitable strength through hydrogen exchanger it can be regenerated. Reaction take place during regeneration as;

$CaR+H_2SO_4 \rightarrow H_2R+CaSO_4$

$CaR+2HCl \rightarrow H_2R+CaCl_2$

$MgR+H_2SO_4 \rightarrow H_2R+MgSO_4$

$MgR+2HCl \rightarrow H_2R+MgCl_2$

$Na_2R+H_2SO_4 \rightarrow H_2R+Na_2SO_4$

$Na_2R+2HCl \rightarrow H_2R+2NaCl$

This can be represented as;

$$\begin{bmatrix} Ca \\ Mg \\ Na_2 \end{bmatrix} R + \begin{matrix} H_2SO_4 \\ \\ HCl \end{matrix} \rightarrow H_2R + \begin{bmatrix} Ca \\ Mg \\ 2Na \end{bmatrix} \begin{bmatrix} SO_4 \\ \\ Cl/Cl_2 \end{bmatrix}$$

3. Demineralization (Deionization) process

This process is similar to zeolite process. However, in demineralization process ions viz. calcium, magnesium etc. are exchanged for hydrogen ions and substances. This process is costlier and often adopted in industrial purposes. The substances are as;

1. Acidic resin (removes alkali substances) like zero karb, organolite, catex etc
2. Base resin (removes acidic substances)

If we indicated resins as H_2R (H means hydrogen and R means organic part of the substance then,

$$H_2R + \begin{matrix} Ca \\ Mg \end{matrix} \begin{bmatrix} (HCO_3)_2 \\ SO_4 \\ Cl_2 \end{bmatrix} \rightarrow \begin{bmatrix} Ca \\ \\ Mg \end{bmatrix} R + \begin{bmatrix} 2H_2O+2CO_2 \\ H_2SO_4 \\ 2HCl \end{bmatrix}$$

6.9 Miscellaneous treatment

Impurities imparts impotability of water could not be separated or removed or reduced by only these processes as coagulation, sedimentation, filtration, disinfection, softening. Hence further certain methods of water treatment may be required for specific purpose include as;

1. Removal of dissolve gases, colour, odour and taste
2. Removal of iron and manganese

6.9.1 Aeration

Aeration is the process of bringing water in intimate contact with air. Since water is brought in intimate contact with air gas transfer between water and air takes place i.e. both degasification and oxidation takes place by aeration. Aeration often used to treat groundwater

Aeration of water serves following purposes.

1. It removes taste and odour from water that produces due to organic decomposition.
2. To absorbed oxygen from atmosphere thereby oxygen deficiency of water may be eliminated, and also freshness is imparted to water.
3. Aeration may also acts as aeromix process for mixing chemicals in water.
4. To remove undesirable gases like carbon dioxide, hydrogen sulphide and volatile substances (like humic acids, phenols etc.) imparting taste and odour to water are easily expelled, and
5. To precipitate iron and manganese present in water by converting soluble to insoluble state.
6. Microorganisms may be destroyed in some extent due to agitation of water.

6.9.1.1 Method of aeration

Methods that are adopted for aeration may be are as following;

1. Free fall or gravity aerator
    a) Cascade aerator
    b) Inclined apron or riffle plate aerators
    c) Tray aerator
    d) Gravel packed bed aerator
    e) Trickling bed aerator
2. Spray aerator
3. Diffuse air aerator
4. Mechanical means

1. a) Cascade aerator

It consists of a series of waterfalls that drop into small pool as shown in figure 6-32. In this case water is exposed to the atmosphere in thin sheets as it cascades down each step. Each step in a cascade is usually 0.3 m in height and as many as 10 steps may be employed. Cascade may be arranged longitudinally like stair steps or may be arranged in a circle, with the steps extending concentrically outward from top to bottom. These are the simplest free fall aerators.

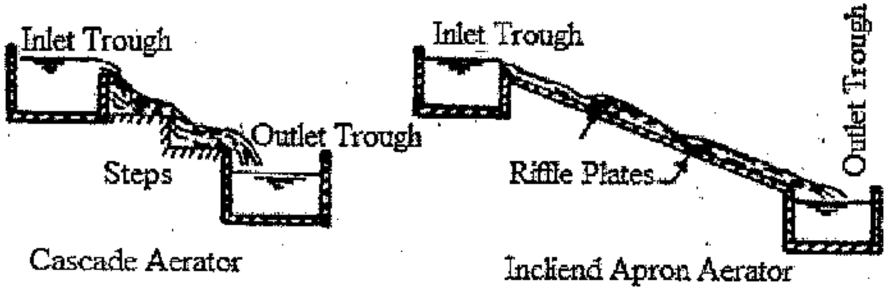


Figure 6-32 Free fall or Gravity Areator

b) Inclined apron or riffle plate aerators

In this type of aerator, water is allowed to fall along an inclined/apron and the breaking up to the sheet of water will cause agitation of water and consequent aeration. Figure 6-32 shows the inclined apron or riffle plate aerators.

c) Tray aerator

These are similar in nature to cascade aerators in that the water is lifted and allowed to fall to a lower elevation. Instead of being intercepted in pools, tray aerators intercept the flow with solid surfaces over which the water must pass in its downward journey. The solid surface may be a series of redwood salt trays which break the flow of the water or a series of porous-bottom trays containing stones, ceramic spheres, or other porous packing. At the top ventilator is provided to escape gases to the atmosphere. Figure 6-33 shows a tray aerator.

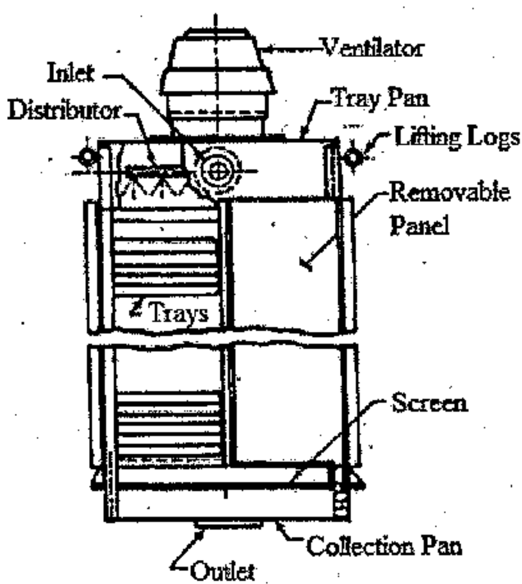


Figure 6-33 Salt Tray Areator

d) Gravel packed bed aerator

In this aerator water is allowed to fall through beds of coke, lime-stone, or anthracite and air is blown from bottom. This method is believed to have more efficient CO_2 removal rather than the other methods of aeration. Figure 6-34 shows a gravel packed bed aerator.

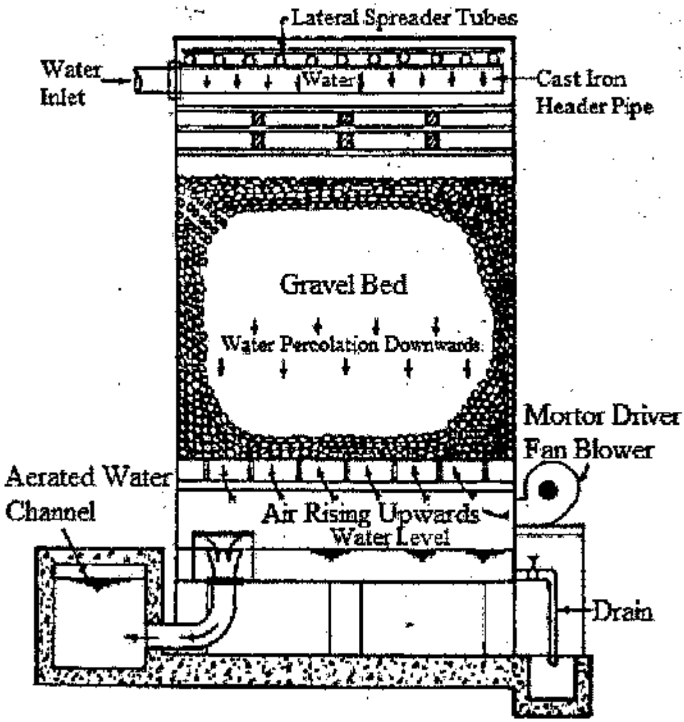


Figure 6-34 Gravel Bed Areator

e) Trickling Bed

This method is the form of gravel bed aerator and commonly used as aeration to treat ground water. Three or four trays having perforated bottom filled with coke, slag or stone are used. Lifted water is allowed from top and at downward journey consequent aeration. Figure 6-35 shows the trickling bed aerator.

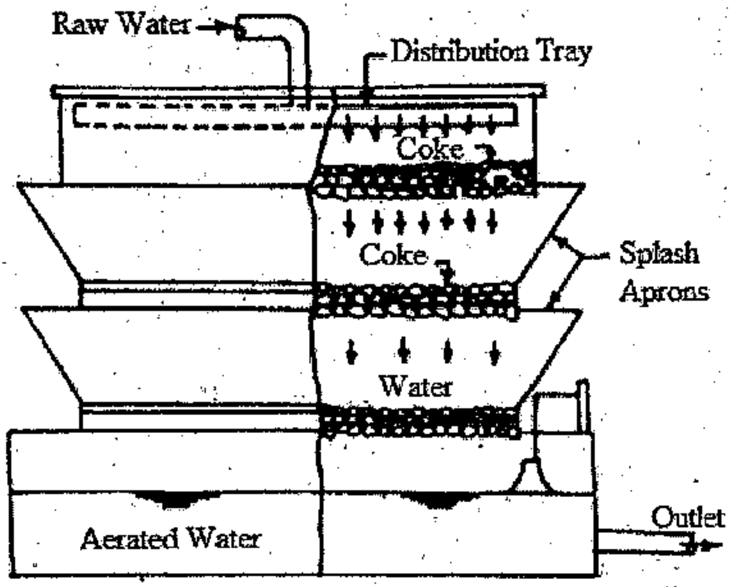


Figure 6-35 Trickling Bed Aerator

2. Spray aerator

In this method of aeration water is sprayed through nozzles upwards into atmosphere and broken up into either a mist of droplets. The installation consists of fixed nozzles on a pipe grid and trays for collecting the sprayed water. It requires considerable pressure head about 0.75 to 1.5 kg/cm^2. Spray aerator is quite efficient to remove CO_2 (70 to 90%) and H_2S about 90 to 99%.

3. Diffuse air aerator

It consists of a tank or basin facilitated with perforated pipe at bottom through which compressed air is blown. The air bubbles emerging from the perforations rise up from the bottom of the tank or basin consequent aeration. Water retention in the tank may be about 15 min and basin depth of 3 to 5 m is provided.

4. **Mechanical means**

   In this method of aeration a tank or basin with propeller connected to motor which is rotated in a suitable rpm consequent mixes air to water and aeration of water can be achieve.

6.9.2 Removal of colour, odour and taste

Presence of impurities in water like organic matter, vegetable matter, industrial waste, domestic wastewater, minerals, microorganisms etc. imparts colour, odour and taste in water. Such impurities which can be removed by various methods as;

1. **Aeration**

   Aeration is a unit process in which air and water are brought into intimate contact.

2. **Activated carbon process**

   Activated carbon is formed by heating a carbonaceous material like coke, charcoal, paper mill waste, saw dust, lignite etc. in a closed vessel at a high temperature. The activation of the carbonaceous materials removes the hydrocarbons which might interfere the adsorption of organic matter and activation is done by passing air, steam, CO_2, chlorine or flue gases. This process removes organic contaminants from water by the process of adsorption.

3. **Adding copper sulphate**

   Copper sulphate is available in crystal form or powder form which is readily soluble in water. To remove colour, odour, taste, bacteria, some extent of aquatic weeds and to control the growth of algae cupper sulphate can be added to water either in the distribution pipes or in open reservoirs. The common dose of copper sulphate is about 0.3 to 0.65 mg/lit.

6.9.3 Removal of iron and manganese

It is found that all water weather surface or ground water contain some traces of iron and also manganese, usually in small amounts, may company iron in water. Presence of their content in water exceeds 0.3 mg/lit become objectionable. Iron and manganese can be removed from water by aeration, adding lime and passing over manganese zeolite.

Iron and manganese can be precipitate present in water to certain extent by aeration and during aeration the following reaction take place;

Iron

$$4Fe+O_2+10H_2O \rightarrow 4Fe(OH)_3\downarrow+8H$$

$$Fe(HCO_3)_2+2H_2O \rightarrow FeO+2CO_2+3H_2O$$

$$4FeO+O_2 \rightarrow 2Fe_2O_3$$

$$Fe_2O_3+O_2+3H_2O \rightarrow 2Fe(OH)_3\downarrow$$

Manganese

$$6Mn+3O_2+6H_2O \rightarrow 6MnO_2\downarrow+12H$$

$Fe(OH)_3$ and MnO_2 are insoluble precipitate which can be removed by sedimentation or clarification.

Example: 6.1

Find the settling velocity of a particle in water under the given condition; diameter of particle = 0.06 mm, specific gravity of particle = 2.65, temperature of water =25 ^{0}C, kinematic viscosity at 25 ^{0}C = 0.897 centistokes.

Solution:

Using Stoke's equation;

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

$$v_s = \frac{9810\ \text{mm/sec}^2}{18\times 0.897\ \text{mm}^2\text{/sec}}\times(2.65-1)\times 0.06^2$$

Therefore; $v_s = 3.609$ mm/sec

Check:

Reynolds number

$$Re = \frac{\rho v d}{\mu} = \frac{vd}{\upsilon}$$

$$= \frac{3.609\times 0.06}{0.897}$$

$$= 0.241 < 1$$

Hence Stoke's law is applicable.

Alternatively the problem can also solved by using equation;

$$v_s = 418\,(S-1)\,d^2\left(\frac{3T+70}{100}\right)$$

$$= 418\times(2.65-1)\times 0.06^2\times\left(\frac{3\times 25+70}{100}\right) = 3.60\ \text{mm/sec}$$

Example: 6.2

Find the settling velocity of a particle in water under the given condition; diameter of particle = 0.6 mm, specific gravity of particle = 2.65, temperature of water =25 ^{0}C, kinematic viscosity at 25 ^{0}C = 0.897 centistokes.

Solution:

Using Stoke's equation;

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

$$v_s = \frac{9810\text{ mm/sec}^2}{18 \times 0.897\text{ mm}^2\text{/sec}} \times (2.65-1) \times 0.6^2$$

Therefore; $v_s = 360.9$ mm/sec

Check:

Reynolds number

$$Re = \frac{\rho v d}{\mu} = \frac{vd}{\upsilon}$$

$$= \frac{360.9 \times 0.6}{0.897}$$

$$= 241 > 1$$

Hence Stoke's law is not applicable.

As the particle size given is greater than 0.1 mm Stoke's equation will not be applicable but it only guide the value of Re to assumed. The settling velocity will have to be determined by Hazen's equation.

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

$$C_D = \frac{24}{Re} + \frac{3}{\sqrt{Re}} + 0.34 \qquad \left[Re = \frac{\rho v d}{\mu} = \frac{vd}{\upsilon} \right]$$

In this case C_D depends on Reynolds number R_e, which in turn depends on settling velocity, which is to be determined. As such a trial solution will have to be adopted as indicated below.

Assumed Re	C_D	Vs mm/sec	Calculated Re
241	0.632	143.14	95.74
95.74	0.897	120.15	80.36
80.36	0.973	115.36	77.16
77.16	0.992	114.25	76.42
76.42	0.997	113.96	76.22
76.22	0.998	113.90	76.18
76.18	0.998	**113.90**	76.18

Therefore the settling velocity (V_s) = 113.90 mm/sec

Example: 6.3

Find the settling velocity of a particle in water under the given condition; diameter of particle = 6 mm, specific gravity of particle = 2.65, temperature of water =25 ^{0}C, kinematic viscosity at 25 ^{0}C = 0.897 centistokes.

Solution:

Using Newton's equation;

$$V_s = \sqrt{3.33gd(S-1)}$$

$$V_s = \sqrt{3.33 \times 9810 \times 6 \times (2.65-1)}$$

Therefore; v_s = 568.68 mm/sec

Check:

Reynolds number

$$Re = \frac{\rho v d}{\mu} = \frac{vd}{\upsilon}$$

$$= \frac{568.68 \times 6}{0.897}$$

$= 3804$ which is between 10^3 to 10^4

Hence Newton's law is applicable.

Example: 6.4

In a continuous flow settling tank 3 m deep and 60 m long, what flow velocity of water would be required for effective removal of 0.025 mm particles at 25 ^{0}C. The specific gravity of particles is 2.65 and kinematic viscosity at 25 ^{0}C = 0.897 centistokes.

Solution:

Since the diameter of the particle is less than 0.1 mm, Stoke's law will be applicable.

Using Stoke's equation;

$$V_s = \frac{g}{18\,\upsilon}(S-1)\,d^2$$

$$V_s = \frac{9810 \text{ mm/sec}^2}{18 \times 0.897 \text{ mm}^2\text{/sec}} \times (2.65-1) \times 0.025^2$$

Therefore; $V_s = 0.626$ mm/sec

Reynolds number

$$Re = \frac{\rho v d}{\mu} = \frac{vd}{\upsilon} \qquad = \frac{0.626 \times 0.025}{0.897}$$

$= 0.017 < 1$ Hence Stoke's law is applicable.

Now using relation,

$$\frac{V}{V_s} = \frac{L}{H}$$

$$\text{or,}\ \ V = V_s \times \frac{L}{H}$$

$L = 60$ m, and $H = 3$ m

$$\text{or,}\ \ V = 0.626 \times \frac{60}{3}$$
$$= 12.52 \text{ mm/sec}$$

Thus in order to ensure effective removal of particles finer than 0.025 mm size the flow velocity in the settling tank should not be more than 12.52 mm/sec.

Example: 6.5

Find the diameter of the particles with specific gravity 1.4 removed in a tank having a surface area of 250 m^2 and treating 10 MLD of water. The temperature of water is 22 ^{0}C.

Solution:

$$\text{Surface overflow rate} = \frac{Q}{A_s} = \frac{10 \times 10^6 \text{ lit/day}}{250 \text{ m}^2}$$
$$= 40 \text{ m}^3/\text{m}^2/\text{day}$$

In a tank, when settling velocity of a particle is greater than SOR; all particles will be removed and when V_s is equals to SOR particles will drop at the end of the tank.

Thus minimum settling velocity required for the particles to settle should be equal to SOR = 40 m^3/m^2/day.

$$= \frac{40 \times 1000}{24 \times 60 \times 60} \text{ mm/sec}$$
$$= 0.463 \text{ mm/sec}$$

Now,

$$v_s = 418\,(S-1)\,d^2\left(\frac{3T+70}{100}\right)$$

$$0.463 = 418\times(1.4-1)\times d^2\times\left(\frac{3\times 22+70}{100}\right)$$

$$d = 0.045\ \text{mm}$$

Example: 6.6

A sedimentation tank is designed for an overflow rate of 5000 liters per m^2 per hour. What percentage of particles of diameter (a) 0.05 mm and (b) 0.025 mm will be removed in this tank? Temperature of water is 20 ^{0}C and specific gravity of particles is 2.65.

Solution:

Surface overflow rate = 5000 litres/m^2/hr

$$= \frac{5000}{1000}\ m^3/m^2/hr$$

$$= \frac{5\times 1000}{60\times 60}\ \text{mm/sec}$$

$$= 1.39\ \text{mm/sec}$$

(a) The settling velocity of particle of diameter 0.05 mm

$$v_s = 418\,(S-1)\,d^2\left(\frac{3T+70}{100}\right)$$

$$v_s = 418\times(2.65-1)\times 0.05^2\times\left(\frac{3\times 20+70}{100}\right)$$

$$v_s = 2.242\ \text{mm/sec}$$

Now,

$$\%\ \text{settled} = \frac{v_s}{\text{overflow rate}}\times 100$$

$$= \frac{2.242}{1.39} \times 100$$

$$= 162\ \% > 100\ \%$$

Hence all the particles of diameter 0.05 mm will settle down.

(b) The settling velocity of particle of diameter 0.025 mm

$$v_s = 418\,(S-1)\,d^2 \left(\frac{3T+70}{100}\right)$$

$$v_s = 418 \times (2.65-1) \times 0.025^2 \times \left(\frac{3 \times 20 + 70}{100}\right)$$

$$v_s = 0.56 \text{ mm/sec}$$

Now,

$$\%\ \text{settled} = \frac{v_s}{\text{overflow rate}} \times 100$$

$$= \frac{0.56}{1.39} \times 100$$

$$= 40.28\ \%$$

Hence 40.28 % of the particles of diameter 0.05 mm will settle down.

Example: 6.7

A rectangular sedimentation tank is to treat 10 MLD of water. A detention basin of width to length ratio of one third is proposed to trap all particles larger than 0.04 mm in size. Assuming a specific gravity of 2.65 for the particles and temperature of water is 20 ^{0}C, compute the basin dimensions. If the depth of the tank is 3.5 m, calculate detention period.

Solution:

Flow (Q) = 10 MLD = 10×10^6 lit/day

$$= 0.11574 \text{ m}^3/\text{sec}$$

The settling velocity of particle of diameter 0.04 mm

$$v_s = 418\,(S-1)\,d^2\left(\frac{3T+70}{100}\right)$$

$$v_s = 418 \times (2.65-1) \times 0.04^2 \times \left(\frac{3\times 20+70}{100}\right)$$

$$v_s = 1.4345 \text{ mm/sec}$$

In a tank, when settling velocity of a particle is greater than SOR; all particles will be removed and when V_s is equals to SOR particles will drop at the end of the tank.

Thus minimum settling velocity required for the particles to settle should be equal to SOR.

Therefore; v_s = SOR = 1.4345 mm/sec

$$\text{SOR} = \frac{Q}{BL}$$

$$\text{or, } 1.4345 \times 10^{-3} \text{ m/sec} = \frac{0.11574 \text{ m}^3/\text{sec}}{BL}$$

$$\therefore \quad BL = 80.678$$

$$\frac{B}{L} = \frac{1}{3}$$

Breadth (B) = 5.2 m

Length (L) = 15.6 m

Height (H) = 3.5 m

Therefore volume (LBH) of a rectangular basin = 283.92 m^3

$$\text{Detention period (t)} = \frac{LBH}{Q} = \frac{283.92 \text{ m}^3}{0.11574 \text{ m}^3/\text{sec}}$$

$$= 40.88 \text{ minutes} = 0.68 \text{ hours}$$

Example: 6.8

An old tank having dimension of 12m x 5m x 3m is available in a village. It is proposed to use as a settling tank. At least 95 % of particles having diameter of 0.025 mm, specific gravity 2.65 is expected to remove on that tank at 20 ^{0}C. What will be an overflow rate on using that tank? Dose tank dimension is enough to remove 99 % of particles having diameter 0.04 mm at same conditions?

Solution:

The settling velocity of particle of diameter 0.025 mm

$$v_s = 418\,(s_s - 1)\,d^2 \left(\frac{3T+70}{100}\right)$$

Now,

$$v_s = 418 \times (2.65-1) \times 0.025^2 \times \left(\frac{3 \times 20 + 70}{100}\right)$$

$$v_s = 0.56 \text{ mm/sec}$$

$$\% \text{ settled} = \frac{v_s}{\text{overflow rate}} \times 100$$

$$\text{or,} \quad 95 = \frac{0.56}{\text{overflow rate}} \times 100$$

Therefore overflow rate = 0.589 mm/sec = 50.96 m^3/m^2/day

Again,

The settling velocity of particle of diameter 0.04 mm

$$v_s = 418\,(S-1)\,d^2 \left(\frac{3T+70}{100}\right)$$

$$v_s = 418 \times (2.65-1) \times 0.04^2 \times \left(\frac{3 \times 20 + 70}{100}\right)$$

$$v_s = 1.4345 \text{ mm/sec}$$

Now,

$$\% \text{ settled} = \frac{v_s}{\text{overflow rate}} \times 100$$

$$\text{or,} \quad \% \text{ settled} = \frac{1.434}{0.589} \times 100$$

$$= 243\% > 100\%$$

Hence the tank dimension is enough to remove 99% of particles having diameter 0.04 mm at same conditions.

Example: 6.9

In a continuous flow settling tank 30 m long and 3 m deep, what detention time would you recommend for effective removal of 0.02 mm particles at 25 °C? Assume specific gravity of particles = 2.65. Also determine the percentage of 0.01 mm particles removed in the same tank at 10 °C.

Solution:

The settling velocity of particle of diameter 0.02 mm

$$v_s = 418\,(S-1)\,d^2 \left(\frac{3T+70}{100}\right)$$

$$v_s = 418 \times (2.65-1) \times 0.02^2 \times \left(\frac{3 \times 20 + 70}{100}\right)$$

$$v_s = 0.4 \text{ mm/sec}$$

In a tank, when settling velocity of a particle is greater than SOR; all particles will be removed and when V_s is equals to SOR particles will drop at the end of the tank.

Thus minimum settling velocity required for the particles to settle should be equal to SOR.

Therefore; $v_s = SOR = 34.56\ m^3/m^2/day$

Now,

$$\text{detention period (t)} = \frac{d}{v_s} = \frac{3000 \text{ mm}}{0.4 \text{ mm/sec}} = 2.083 \text{ hours}$$

Again,

The settling velocity of particle of diameter 0.01 mm

$$v_s = 418\,(S-1)\,d^2\left(\frac{3T+70}{100}\right)$$

$$v_s = 418 \times (2.65-1) \times 0.01^2 \times \left(\frac{3 \times 20 + 70}{100}\right)$$

$$v_s = 0.06897 \text{ mm/sec}$$

$$\text{or,} \quad \%\text{ settled} = \frac{0.06897}{0.4} \times 100$$

$$= 17.24\%$$

Example: 6.10

Determine the settling velocity of spherical particles of size 0.05 mm. The specific gravity is 2.65 and kinematic viscosity of water 20 °C is 1.01 mm^2/sec. If the specific gravity of another particle of identical size is 0.9, check at what rate the particle will rise or settle.

Solution:

Since the diameter of the particle is less than 0.1 mm, Stoke's law will be applicable.

Using Stoke's equation;

$$V_s = \frac{g}{18\,\upsilon}(S-1)\,d^2$$

$$V_s = \frac{9810\ \text{mm/sec}^2}{18 \times 1.01\ \text{mm}^2/\text{sec}} \times (2.65 - 1) \times 0.05^2$$

Therefore; $V_s = 0.223$ mm/sec

Check:

Reynolds number

$$Re = \frac{\rho v d}{\mu} = \frac{vd}{\upsilon}$$

$$= \frac{0.223 \times 0.05}{1.01}$$

$= 0.11 < 1$ Hence Stoke's law is applicable.

For another particle of identical size having specific gravity (S) = 0.9

Using Stoke's equation;

$$V_s = \frac{g}{18\ \upsilon}(S - 1)\, d^2$$

$$V_s = \frac{9810\ \text{mm/sec}^2}{18 \times 1.01\ \text{mm}^2/\text{sec}} \times (0.9 - 1) \times 0.05^2$$

Therefore; $V_s = -1.349$ mm/sec

Since the velocity is negative the particle will rise at a velocity of 0.1349 mm/sec

Example: 6.9

Compute the dimensions of a continuous flow rectangular settling tank to treat 2 MLD of water. Assume detention period 4 hrs.

Solution:

Flow (Q) = 2×10^6 lit/day = 2000 m^3/day

Capacity of tank (C) = Q × t

$$= \frac{2000 \times 4}{24} \ \text{m}^3$$

$$= 333.34 \text{ m}^3$$

$$L \times B \times H = 333.34 \text{ m}^3 \text{ ---------(i)}$$

Assuming, SOR= 15 $m^3/m^2/day$

$$\text{SOR} = \frac{Q}{BL} = 15 \text{ m}^3/\text{m}^2/\text{day}$$

$$BL = \frac{2000 \text{ m}^3/\text{day}}{15 \text{ m}^3/\text{m}^2/\text{day}} = 133.34 \text{ m}^2$$

Putting the value of BL in equation (i)

$$\therefore \quad H = \frac{333.34}{133.34} = 2.5 \text{ m}$$

Take, $L : B = 5:1$

$$5B^2 = 133.34$$

$$B = 5.16 \approx 5.2 \text{ m}, \qquad L = 26 \text{ m}$$

Check:

$$\text{Horizontal velocity } (V_h) = \frac{Q}{BH}$$

$$= \frac{Q}{BH} = \frac{2000 \text{ m}^3/\text{day}}{2.5 \text{ m} \times 5.2 \text{ m}}$$

$$= 106.83 \text{ mm/min} < 300 \text{ mm/min}$$

Hence OK

Example: 6.10

Design rectangular and circular sedimentation tank for a town to purify the water of a rate of 8 MLD. Assume velocity of flow as 150 mm/min and detention period as 5 hrs.

Solution:

Horizontal velocity (V_h) = 150 mm/min

Detention period (t) = 300 min

Flow (Q) = 8×10^6 lit/day = 8000 m^3/day

Capacity of tank (C) = $Q \times t$

$$= \frac{2000 \times 5}{24} \ m^3$$

$$= 1666.67 \ m^3$$

Length of the rectangular basin (L) = $V_h \times t$

$$= 150 \text{ mm/min} \times 300 \text{ min}$$

$$= 45 \text{ m}$$

Assuming effective depth of basin = 3.5 m

$L \times B \times H$ =1666.67 m^3

45 m $\times$ B $\times$ 3.5 m = 1666.67 m^3

B = 10.58 m $\approx$ 10.6 m

Check:

$$\text{SOR} = \frac{Q}{BL} = \frac{8000 \text{ m}^3/\text{day}}{45 \text{m} \times 10.6 \text{m}} = 16.77 \text{ m}^3/\text{m}^2/\text{day}$$

$$\frac{L}{B} = \frac{45}{10.6} = 4.2$$

Hence OK.

Providing free board 0.5 m and sludge depth 0.5 m. Overall depth of tank = 4.5 m

Therefore size of basin equals to **45m × 10.6 m× 4.5m**

For Circular;

Taking effective depth = 3.5 m

$$C= d^2 (0.011d+0.785H)$$

$$1666.67 = d^2 (0.011d+0.785 \times 3.5)$$

$$d = 23.54 \text{ m} \approx 23.6 \text{ m}$$

Check:

$$SOR = \frac{H}{t} = \frac{3.5\,\text{m}}{5\,\text{hours}} = 16.8 \text{ m}^3/\text{m}^2/\text{day}$$

Providing free board 0.5 m and sludge depth 0.5 m. Overall **depth of tank = 4.5 m and diameter 23.6 m.**

Example: 6.11

Calculate the dimension of a continuous flow rectangular basin for population of 25000 with a per capita demand of 100 lpcd. Assume detention period as 4 hrs.

Solution:

Average quantity of water to be treated

$$(Q) = 25000 \times 100 \text{ lpcd}$$

Design discharge (Q) = 2500000 lit/day

$$= 2500 \text{ m}^3/\text{day}$$

Detention period (t) = 4 hours

Capacity of tank (C) = Q × t

$$= \frac{2500 \times 4}{24} \quad \text{m}^3$$

$$= 416.67 \text{ m}^3$$

$$L \times B \times H = 416.67 \text{ m}^3 \text{ ---------(i)}$$

Assuming, SOR= 15 m^3/m^2/day

$$SOR = \frac{Q}{BL} = 15 \text{ m}^3/\text{m}^2/\text{day}$$

$$BL = \frac{2500 \text{ m}^3/\text{day}}{15 \text{ m}^3/\text{m}^2/\text{day}} = 166.67 \text{ m}^2$$

Putting the value of BL in equation (i)

$$\therefore \quad H = \frac{466.67}{166.67} = 2.8 \text{ m}$$

Take, $L: B = 3:1$ (Range L/B = 3 to 5:1)

$$3B^2 = 166.67$$

$$B = 7.45 \approx 7.5 \text{ m}, \quad L = 22.5 \text{ m}$$

Check:

$$\text{Horizontal velocity } (V_h) = \frac{Q}{BH}$$

$$= \frac{Q}{BH} = \frac{2500 \text{ m}^3/\text{day}}{2.8 \text{ m} \times 7.5 \text{ m}}$$

$$= 171.43 \text{ mm/min} < 300 \text{ mm/min}$$

Hence OK

Providing free board 0.5 m and sludge depth 0.5 m. Overall depth of tank = 3.8 m

Therefore size of basin equals to **22.5 m × 7.5 m× 3.8 m**

Example: 6.12

Determine amount of bleaching powder required annually in a water treatment plant treating 10 MLD of water if 0.3 ppm of chlorine dosage is required. Available bleaching powder contains 27% of chlorine.

Solution:

Flow (Q) = 10×10^6 lit/day = 10000 m^3/day

Dosage = 0.3 mg/lit

Chlorine required = 0.3×10^{-6} kg/lit $\times 10 \times 10^6$ lit/day

= 3 kg/day

$$\text{Bleaching powder required} = \frac{3}{0.27} \text{ kg/day}$$

= 11.11 kg/day

Bleaching powder required annually = 4.05 tones

Example: 6.13

Chlorine usage in the treatment of 25 MLD of water is 9 kg/day. The residual chlorine after 10 minutes contact is 0.2 mg/lit. Calculate the dosage in mg/lit and chlorine demand of water.

Solution:

Flow of water (Q) = 25×10^6 lit/day = 25000 m^3/day

Quantity of chlorine used = 9 kg/day = 9×10^6 mg/day

$$\text{Dosage or Chlorine used} = \frac{9 \times 10^6 \text{ mg/day}}{25 \times 10^6 \text{ lit/day}} = 0.36 \text{ mg/lit}$$

Chlorine demand = Dosage – Residual chlorine

= (0.36 – 0.2) mg/lit = 0.16 mg/lit

Example: 6.14

Design slow sand filter beds from the following data:

Population to be served = 60, 000 nos

Average rate of demand = 160 lpcd

Rate of filtration = 150 liters/hr/m^2

L: B = 2:1

Assume design discharge as twice the average flow and;

Assume that one unit will be kept standby.

Solution:

Average quantity of water to be treated

$$= 60000 \times 160 \text{ lpcd}$$
$$= 9600000 \text{ lit/day}$$

Design discharge (Q) = 2×9600000 lit/day

$$= 19.2 \times 10^6 \text{ lit/day}$$

Rate of filtration = 150 liters/hr/m^2

$$= 24 \times 150 = 3600 \text{ liters/day/m}^2$$

$$\text{Surface area required} = \frac{Q}{\text{Rate of filtration}} = \frac{19.2 \times 106 \text{ liter/day}}{3600 \text{ litre/day/m}^2}$$
$$= 5333.34 \text{ m}^2$$

Assuming 5 filter units;

$$\text{Area for each filter bed } (a_s) = \frac{5333.34\text{m}^2}{5}$$
$$= 1066.67\text{m}^2$$

$$A_s = L \times B = 2B^2 = 1066.66 \text{ m}^2$$

$$B = \sqrt{\frac{1066.66}{2}}$$

$B = 23.09$ m ≈ 23.10 m

$L = 46.20$ m

Breadth (B) = 23.10 m, Length (L) = 46.20 m

Providing Free board = 0.5 m, water depth = 1 m, depth filter media = 1 m, depth of base material = 0.6 m, depth of under drain pipe = 0.2 m, Therefore overall depth of filter = 3.30 m

Providing one filter unit standby, required filter units = 6 with dimension of 46.20 m × 23.10 m × 3.30 m.

Example: 6.15

Average water consumption rate is 150 lpcd in an urban area. Design a slow sand filter for a community having the population of 10000 at the base year 2068.

Solution:

Assume annual population growth rate 1.7% and design period = 15 years.

Using geometrical method for forecasting population of design year 2083;

$$p_n = p_0\left(1+\frac{r}{100}\right)^n = 10000\left(1+\frac{1.7}{100}\right)^{15} = 12877 \text{ nos.}$$

Average quantity of water to be treated

$$= 12877 \text{ nos.} \times 150 \text{ lpcd}$$
$$= 1931550 \text{ lit/day}$$

Let design discharge is equal to average quantity water (Q)

$$= 1931550 \text{ lit/day}$$

Assuming Rate of filtration = 150 liters/hr/m^2

$$= 24 \times 150 = 3600 \text{ liters/day/m}^2$$

$$\text{Surface area required} = = \frac{Q}{\text{Rate of filtration}} = \frac{1931550 \text{ liter/day}}{3600 \text{ litre/day/m}^2}$$
$$= 536.54 \text{ m}^2$$

Assuming 3 filter units;

Area for each filter bed (a_s) = $\dfrac{536.54 \text{ m}^2}{3}$

$$= 178.84 \text{ m}^2$$

$A_s = L \times B = 2B^2 = 178.84 \text{ m}^2$

$$B = \sqrt{\frac{178.84}{2}}$$

$B = 9.456 \text{ m} \approx 9.5 \text{ m}$

$L = 19 \text{ m}$

Breadth (B) = 9.5 m, Length (L) = 19 m

Providing Free board = 0.5 m, water depth = 1 m, depth filter media = 1 m, depth of base material = 0.6 m, depth of under drain pipe = 0.2 m, Therefore overall depth of filter = 3.30 m

Providing one filter unit standby, required filter units = 4 with dimension of 19 m $\times$ 9.5 m $\times$ 3.30 m.

Example: 6.16

Calculate the dimension of a set of rapid sand filter for treating water required for a population of 0.1 million with an average rate of demand 200 lpcd.

Solution:

Average quantity of water to be treated

$$= 0.1 \times 10^6 \times 200 \text{ lpcd}$$

$$= 20 \times 10^6 \text{ lit/day}$$

Assuming that 3% of filtered water is required for filter backwashing and time required for backwashing = 30 minutes.

Discharge (Q) = 20000000 lit/day

$$= 20 \times 10^6 \text{ lit/day}$$
$$= 833333.34 \text{ lit/hr}$$

Design discharge (Q) = $\frac{833333.34}{(1 - 0.03)} \times \frac{1}{(24 - 0.5)} = 36557.72$ lit/hr

Assume rate of filtration = 4000 liters/hr/m^2

$$= 24 \times 4000 = 96000 \text{ liters/day/m}^2$$

Surface area required = $\frac{Q}{\text{Rate of filtration}}$

$$= \frac{877385.39 \text{ lit/day}}{96000 \text{ lit/day/m}^2}$$

$$= 9.13 \text{ m}^2$$

Minimum Surface area (A_s) = 10 m^2

Assuming,

L: B = 1.3:1

$A_s = L \times B$

B = 2.77 m ≈ 2.8 m

L = 3.64 m ≈ 3.65 m

Breadth (B) = 2.8 m, Length (L) = 3.65 m

Provide, Free board = 0.5 m, Water depth = 1.6 m, Filter media = 0.6 m, Base material = 0.6 m, Under drain pipe depth = 0.2 m Therefore overall depth = 3.5 m

Providing one filter unit standby, required filter units = 2 with dimension of 3.65 m × 2.8 m × 3.5 m

Problems

1. What do you understand by treatment of water? Why it is necessary? Give outline of various processes adopted for treatment of water.
2. Enlist and discuss in brief, various treatment processes adopted collectively for treating water supplies drawn from Perennial River.
3. What is meant by treatment of water and why it is necessary? What do you mean by flocculation?
4. What do you understand by sedimentation with coagulation? Explain the different processes involved in sedimentation with coagulation.
5. Working from fundamentals, derive Stoke's law for settlement of discrete particles in water. How do you modify the law, taking into account the temperature effect?
6. What do you understand by coagulation and flocculation? Why are they necessary? Describe various types of coagulants commonly used with chemical reactions involved.
7. State the theory of filtration and briefly describe them in context of water treatment process.
8. Describe, with the help of a neat sketch, a slow sand filter. Explain its working and cleaning.
9. Differentiate between slow sand and rapid sand filter.
10. What are the merits and demerits of slow sand filter as compared with rapid sand filter.
11. With neat sketches describe the construction of rapid sand filter.
12. What is break point chlorination? How can you obtain the break point? Describe.
13. Explain in brief the theory of disinfection by chlorine.
14. Why chlorination is required in drinking water supply? Explain the various forms of chlorination.
15. Explain break point chlorination in water treatment. Describe the factors affecting chlorination.

16. Explain the break point chlorination in relation to water supply system. Explain significance of residual disinfectant.

17. Give major requirements of a disinfectant. Describe in brief about the methods of aeration.

18. What are the purposes of aeration? Describe various methods of aeration.

19. What do you mean by aeration of water? Why it is required? Describe briefly about various methods used for aeration of water.

20. Explain the methods of removal of iron and manganese from water. Why their removal necessary?

21. Design a rectangular sedimentation tank for a town to purify the water at a rate of 8×10^6 liters per day. Assume velocity of flow as 150 mm/minute and detention period as 5 hrs.

22. Determine the surface area of a sedimentation tank for a design flow of 0.5 m^3/s, using the design surface overflow rate as 35 $m^3/day/m^2$. Find the depth of the tank for a detention period of 2 hrs. Assume suitable data if necessary.

23. 2 MLD of water is passing through a sedimentation tank which is 6 m wide, 15 m long and having a water depth of 3 m. find the detention time of the tank, compute overflow rate and average velocity through the tank?

24. Find the settling velocity of spherical silica particle of specific gravity 2.6, in water at 20 °C, if the diameter of the particle is 0.2 mm. Take kinematic viscosity at 20 °C =1.007 centistokes.

25. A settling tank is designed for an overflow rate of 72 $m^3/m^2/day$. What percentage of particles of diameter 0.025 mm with 2.65 specific gravity will be removed at water temperature 20 °C.

26. Find the diameter of the spherical particles with specific gravity of 1.4 removed in the tank having surface area of 300 m^2 and treating 20 MLD of water. Assume kinematic viscosity of 1.01 centistokes at 20 °C.

27. In a continuous flow settling tank 60 m long and 3 m deep, what flow velocity of water would you recommend for effective

removal of 0.025 mm particles at 25 °C. Take specific gravity of particle = 2.65 and kinematic viscosity 0.897 mm^2/sec.

28. Determine the dimension of a continuous flow rectangular settling tank for a population of 30000 wath a daily per capita water demand of 120 lpcd. Assume detention period as 4 hrs.
29. Design a plain sedimentation tank for treating 5 million liters of water per day. Take a detention period of 4 hours and assume a depth of 3.5 meters.
30. A settling tank is designed for and overflow rate of 4000 liters per m^2 hour. What percentage of particles 0.05 mm, will be removed in this tank at 20 °C? Assume specific gravity as 2.65.
31. Design a plain sedimentation tank for treating 4 MLD of water, assume other data suitably.
32. Design a slow sand filter for a population of 20000 with an average rate of water demand as 150 lpcd. Assume other necessary data suitably.
33. Average water consumption rate is 110 lpcd in an urban area. Design a rapid sand filtration unit for a community having the population of 5550. Assume necessary data suitably.
34. Average water consumption rate is 45 lpcd in the village. Design a filtration unit for a community having the population of 2500 at the base year 2058. Assume necessary data suitably.
35. Calculate the quantity of bleaching power required per year for disinfecting 5 million liters per day. The dosage of chlorine has to be 0.3 mg/lit and the bleaching powder contains 35% of available chlorine.
36. Chlorine used to treat m^3/day of water is 8 kg/day. The residual chlorine after 10 minutes contact is 0.2 mg/lit. Calculate the dosage of chlorine in mg//lit and chlorine demand of water.

7 Reservoir and Distribution

The water which has been purified has to be supplied to the consumers. The supply of water to the consumers is accomplished through a well planed pipe networks including; reservoirs for storing treated water, stabilizing pressures, fire hydrants, pumps, valves, service connections, water meter etc called distribution system. Aim of distribution system is to supply safe and wholesome water to consumers at adequate pressure in sufficient quantity and also minimizing loss of water. Water distribution works may cover 40 to 70 % of the total outlay on the water supply system.

Requirements of a good distribution system

A good distribution system should have the following requirements;

1. It should be capable to deliver treated water to consumers in adequate quantity at required pressure.
2. Water quality should not be degraded at time of supply in distribution lines i.e. should maintain the degree of purity.
3. It should be efficient and easy to operate and maintain.
4. It should be watertight with having minimum loss of water.
5. It should be safe against bursting of pipe due to possible excess pressure.
6. It should be capable to meet for emergencies like fire fighting.

7.1 System of Supply

Treated water may be supplied to the public by the following two systems;

a) Continuous system of supply
b) Intermittent system of supply

a) Continuous system of supply

This system is the most ideal system of supply of water which should be adopted as far as possible. In this system water is supplied to the public for all 24 hours of day. This system has following advantages and disadvantages;

Advantages: In this system water is available throughout the day hence it is not require to private storage tank and also water is available at the time fire fighting. There is no chance of sediment stagnant in the pipe due to continuous supply of water hence fresh water is always available.

Disadvantages: Considerable wastage of water may occur if there is leakage. If the public do not realize the significance of treated water wastage and losses problem may arises, such problem may be avoided by metering system with reasonable tariff system. At the period of repair and maintenance supply may be interrupted.

b) Intermittent system of supply

In this system of supply, water is supplied to the consumers for the period of fixed hours of the day only. The supply hour is fixed normally morning and evening but time may be changed to suit the seasons of the year. In this system water to be served area is divided into several zones and the supply hour of each zone are so adjusted that required pressure is maintained in each zone. When water availability is not sufficient to meet demand this system is useful and also the available pressure poor.

Drawback of the intermittent system

1. It requires private storage tank in individual houses. If sufficient water could not store there may be insanitary condition at the non supply periods.
2. Inconvenience to consumers because people have to remain alert to collect water at supply period.

3. If fire breaks out during non-supply hours, there may be great inconvenience that water could not be available for fire fighting.
4. During non-supply hour water taps may be left open unknowingly or due to negligence, which may lead wastage at the supply period.
5. This system requires large number of valves for its functioning and greater size of pipe required to meet full day supply in short period.
6. Due to negative pressure in supply line at the period of non-supply may induce suction through leaking joints causes pollution of water.
7. Extra staff will be required to operate and maintain because system require number of valves for its works to distribute water in rotation in different zones.

7.2 Clear water reservoirs

Clear water reservoir is used to store treated water before distribution to consumer. Storage capacity of clear water reservoir must be 14 to 16 hours average daily flow. Generally such reservoirs are constructed half below ground and half above the ground level depending on site conditions and also in two or more compartments to enable repairs or cleaning. The system will be more reliable if larger the capacity of clear water reservoir so that the supply of water can be maintained for long period at the period of repair and maintenance of systems components.

Advantages of intermittent system

1. At the breakdown of system or in emergencies private storage of water can be utilized for few days to meet immediate needs.
2. Repair and maintenance works can be done in non supply period without inconvenience to consumer.
3. If there is leakage pipeline wastage of water will be less.

7.3 Service/distribution reservoir

Service reservoir or distribution reservoirs are used in distribution system to provide storage;

- to meet fluctuations in demand of water
- to provided storage for fire fighting
- to manage emergencies like breakdown, repairs, etc.
- to stabilize pressure in distribution system

These reservoirs should be located near to area to be served so that water can be supplied to consumers in short duration with minimum loss of head in the pipes. Besides, it will save cost of the pipelines so that transmission mains are designed for average flow and distribution lines for peak flow.
If provision of breakdown and fire reserve is not take into account then these reservoir also act as balancing reservoir.

7.3.1 Purpose and construction

Various purpose served by balancing reservoir as;

1. In case of lifting, reservoir provision assist run the pumps at uniform rate.
2. Provision of reservoir helps in distribution line to economize the pipe size.
3. Fluctuations in hourly demand of water could be meeting by such storage.
4. These reservoir can address problem rises due to breakdown of pumps, bursting mains, interruption in power supply etc
5. These reservoir help in over burden reduction and overall reduction in the treatment units, pumps, pipes etc.
6. Pressure can be maintained by provision of these reservoirs.

Service or distribution reservoir may be constructed of stone masonry, brick masonry, plain cement concrete, reinforced or pre-stressed concrete or steel. Such reservoir is always covered to avoid contamination and prevent algal growth. For the efficient functioning and maintain there is provision of mosquito proof ventilation, manhole for inspection and cleaning, access ladder, scour valve, overflow valve, water level indicator etc.

7.3.2 Types of service reservoirs

According to ground condition service reservoirs are classified as;

1. Surface reservoirs
2. Elevated reservoirs
3. Standpipes

1. Surface reservoirs

These reservoirs are rectangular or circular in shape and constructed in the earth surface (at the ground level) or below ground level known as ground reservoirs or non-elevated reservoirs. Treated water from this reservoir is lifted to elevated reservoir to supply consumers and also may be supplied directly if surface reservoirs located at higher elevations. A section of a typical surface reservoir in two compartments as shown in figure 7-1.

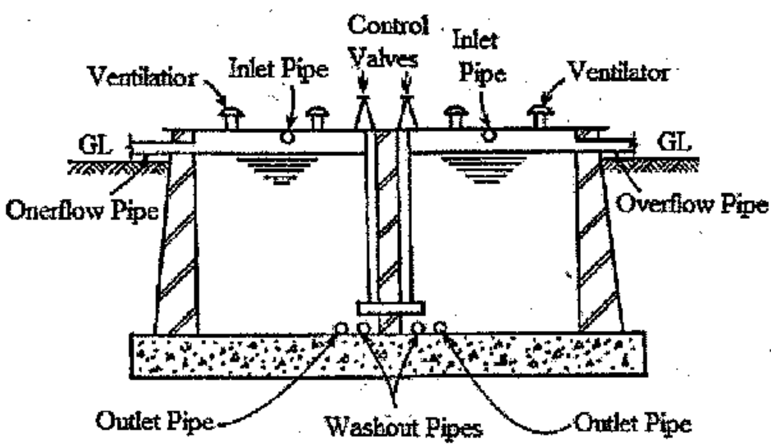


Figure 7-1 Surface Reservoir

2. Elevated reservoirs

These reservoirs may be rectangular, circular or elliptical in shape, spherical, egg shaped which are constructed at an elevation from ground so that are also called overhead tanks. The assorted accessories provided for elevated reservoir are manhole, inlet, outlet, overflow, ladder, ventilation, scour valve, water level indicator, lightening conductor etc. Figure 7-2 shows a section of a rectangular elevated reservoir.

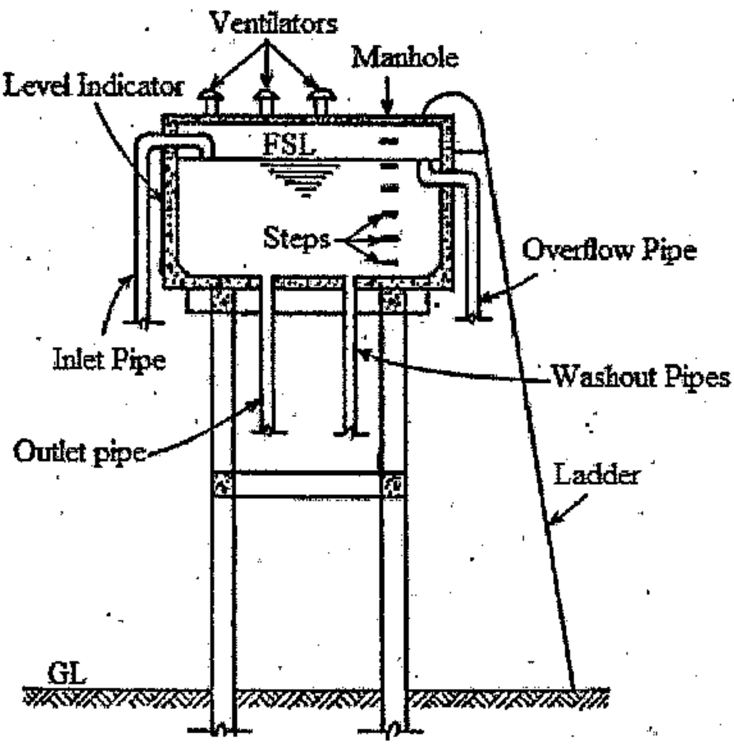


Figure 7-2 Elevated Reservoir

A RCC tank known as Intz tank shown in figure 7-3 is more common and widely adopted these days.

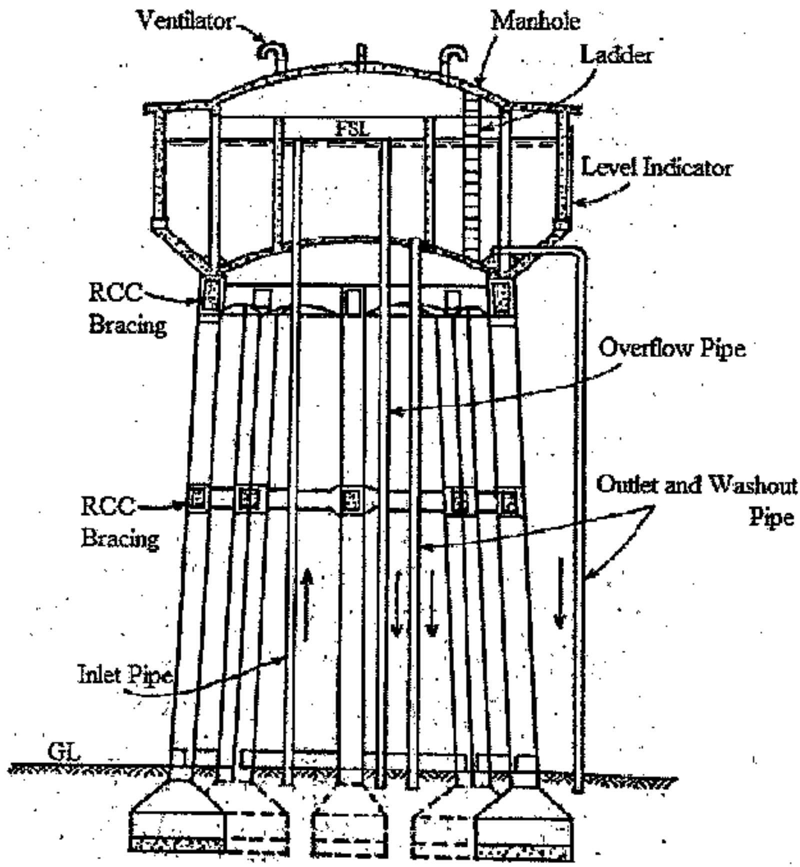


Figure 7-3 Intz Tank

3. Standpipes

A vertical cylindrical tank resting just above ground (overhead) up to 15 to 30 m height and diameter 10 to 15 m which are made of steel or RCC standpipes. Purpose of such tank is to increase pressure in distribution system by creating extra storage in the tank above elevation required to give the necessary pressure for distribution of water. The volume of water stored in the tank above entrance of the outlet pipe can only be used and only serves as a support for the useful storage and is termed as supporting storage. However, entire storage can be utilized. As reservoir tank facilitated with accessories standpipes are also provided with inlet, outlet, overflow, scour valve, manhole for inspection etc. Figure 7-4 shows a standpipe.

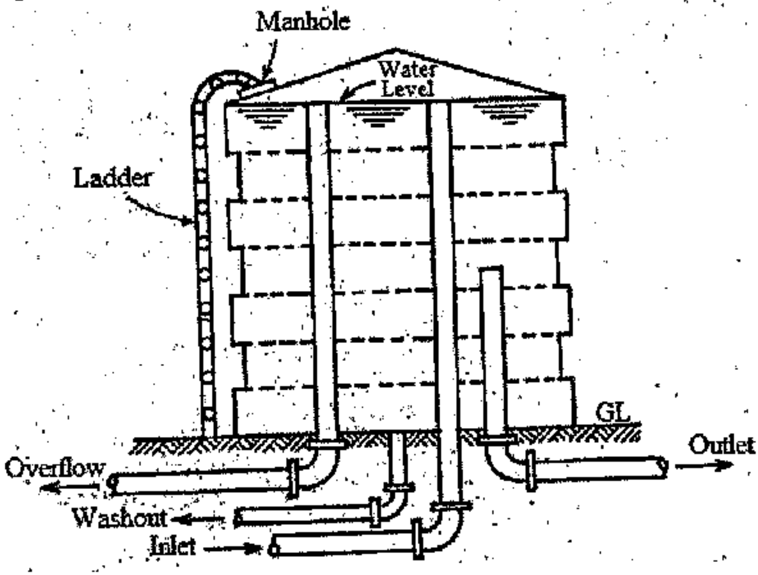


Figure 7-4 Standpipe

7.4 Storage capacity of service reservoirs

The storage capacity of distribution/service reservoir is based on various requirements as follows;

1. Balancing reserve (equalizing/operating):
2. Breakdown storage or emergency storage
3. Fire storage

The total of the above three categories, determines the capacity of the service reservoir.

1. **Balancing reserve (equalizing/operating storage):**

   This provision of storage is to meet the fluctuating demand of water with a constant rate of pumping of water into the reservoir.

The quantity of water required being stored in the reservoir for balancing or equalizing variable demand of water against the constant rate of pumping is balancing storage or reserve and can be determined by two methods.

a) Hydrograph method
b) Mass curve method

a) Hydrograph method

In this method hourly demands of water for a typical maximum day are plotted against the respective hours of the day and a hydrograph is obtained. For a uniform 24 hours pumping, the pumping rate will be equal to the mean hourly demand of water and the same is plotted by line AB. The storage required for distribution reservoir is then obtained by determining the area between the curve QRST and the line AB. This area when covered into volume units gives the storage required for balancing reservoir. Figure 7-5 shows the hydrograph for determining storage required for distribution reservoir.

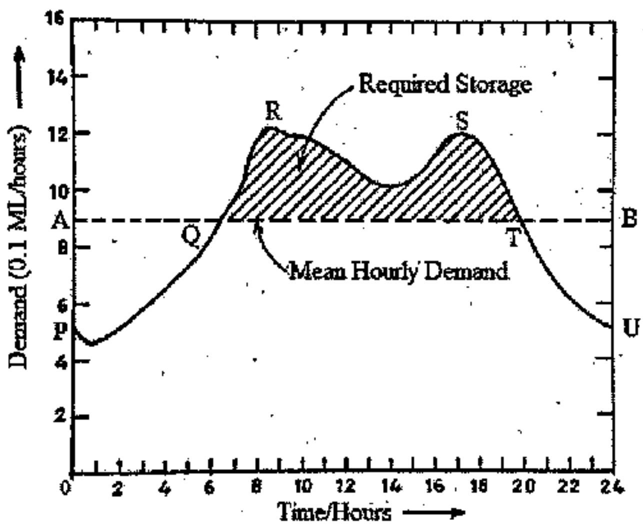


Figure 7-5 Hydrograph

Analytical method is more convenient for determining the storage reservoir capacity for a distribution system.

Procedure

1. First calculate hourly demand and cumulative hourly supply for 24 hrs.
2. Find the hourly excess of demand (deficit) and excess of supply (surplus).
3. Obtain the maximum cumulative surplus and maximum cumulative deficit.

Reservoir capacity

Case 1. If TS > TD, RC= MCS+MCD-TS+TD

Case 2. If TS ≤ TD, RC=MCS+MCD

Where,

TS = Total Supply (Inflow) quantity of water for 24 hours

TD = Total Demand (Outflow) quantity of water for 24 hours

RC = Reservoir Capacity

MCS = Maximum cumulative surplus

MCD = Maximum cumulative deficit

b) Mass curve method

In this method a mass curve of demand is drawn which is a plot of cumulative demand of water against time. Figure 7-6 shows mass curve for determining storage required for distribution reservoir when pumping is done for 24 hours.

Procedure

1. Values of cumulative demand of water at successive hours are obtained and the same are plotted against the corresponding hours to obtain mass curve of demand. A mass curve continuously rises.

2. Draw tangents through lowest point E and highest point F. Find the two ordinates (Deficit and surplus) between demand and supply curves and calculate $S = A + B$.

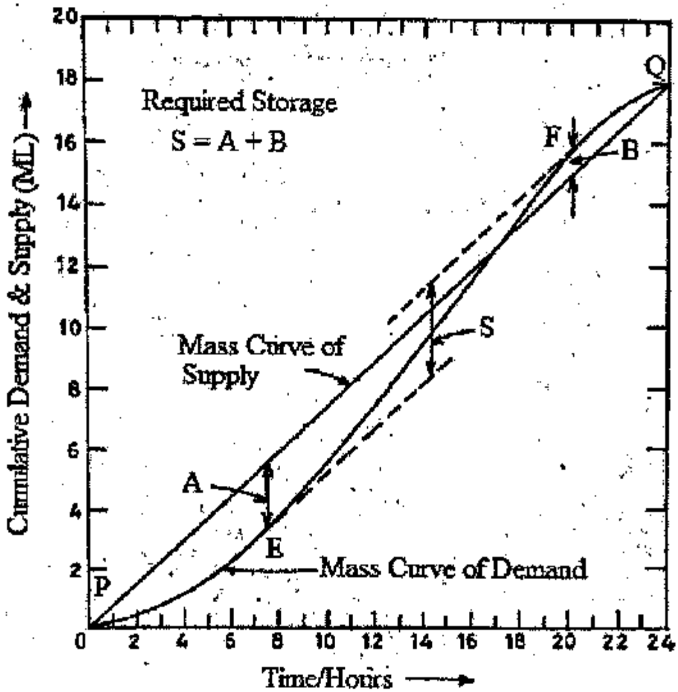


Figure 7-6 Mass Curve

2. Breakdown storage or emergency storage

The break down storage requirement may be due to failure of pumps, power supply, repair time etc. It is difficult to assess the magnitude of the storage to meet this requirement hence 25% of total capacity of reservoir or about 1.5 to 2 times the average hourly supply may be considered as breakdown storage. Breakdown storage is not considered in our country Nepal.

3. Fire storage/reserve

The water stored in the service reservoir to provide water for firefighting purpose is fire reserve. Water required for fire storage can be determined by following expression.

$$FR = \{F - P\} \times T$$

Where,

FR = Fire reserve (Liter)

F = Fire demand (lit/minute)

P = Reserve for pumping capacity (lit/minute)

T = Duration of fire (minutes)

Water Consumption Pattern

Water consumption pattern refers to the variation of water consumption with respect to time in a day. Consumption pattern depends on various factors such as type of the area, climatic conditions, socio economic factors, consumer's habits and standard, geographic location etc. Consumption pattern differ in rural and urban area. Consumption pattern recommended by DWSS guideline for rural areas for Nepal is shown in Table 7-1.

Table 7-1 Water Consumption Pattern

Time	Water consumption %
05:00-07:00	25
7:00-12:00	35
12:00-17:00	20
17:00-19:00	20
19:00-05:00	0

7.5 Layout of distribution system

Depending upon the layout of various pipes of distribution systems are as follows;

1. Dead-end or tree system
2. Grid-iron system or Reticulation system or Interlaced system
3. Circular or ring system
4. Radial system

1. Dead-end or tree or Branched system

In this system, one main pipe lines laid through the center of the area to be served and from both sides of the main pipe line sub-main take off. The sub-main divide into several branches from which service connections are given to the public. There is no any cross connections between sub-mains and branches and hence there may be number of dead ends in this system. This system is suitable for the cities or town which is growing haphazard manner without planning like our city Kathmandu. A typical layout of tree system has shown in figure 7-7.

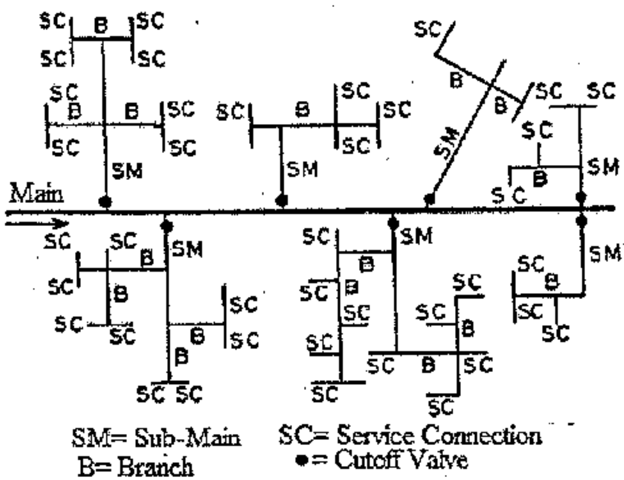


Figure 7-7 Dead End or Branched System

This system has various **advantages** as;

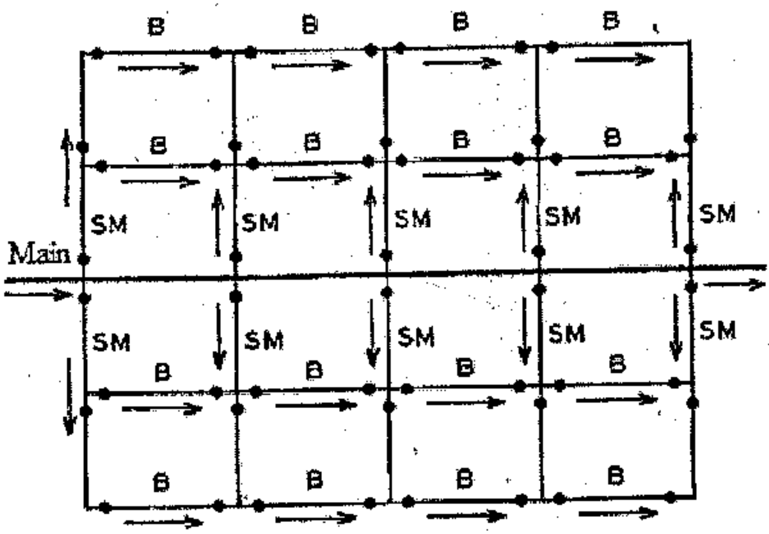
1. Discharge and pressure at any point can be calculated accurately hence design may be simple and easy.
2. Pipe diameters are to be designed for population likely to be served which become cheep and economical.
3. Pipe lying is simple and less cut off valves required.

Disadvantages

1. There is great inconvenience to consumers beyond the repair or damage due to large portion of distribution area may be affected.
2. Due to number of dead ends accumulation of sediment stagnation may lead water quality degradation. In order to remove deposited sediment provision of scour valve may essential which measure costly and large quantity of water required to thrown to waste.
3. This system is less effective to maintain or distribute pressure in remote parts.
4. Water availability for firefighting may be low because there is no chance of increase of supply by diverting from any other side.

2. Grid-iron or Reticulation or Interlaced system

In this system, main pipeline runs through the centre of the area to be served and from both sides of the main pipe line sub-mains take off in perpendicular direction as shown in figure 7-8. Layout of mains, sub-mains, and branches are interconnected with each other. This system is suitable for planed rectangular cities or grid iron.



SM= Sub-Main B= Branch • = Cutoff Valve

Figure 7-8 Dead Grid Iron System

This system has various **advantages** as;

1. There is no sediment stagnation problem due to free circulation of water and no chances of pollution due to sediment deposition.
2. In this system, minimum head loss occurs due to interconnections of mains, sub-main, and branches.
3. In case of damage or repair in any section, small area may be affected.
4. For the fire fighting water could be available by diverting the supplies from other sections.

Disadvantages

1. This system requires a large number of cutoff valves and longer lengths of pipe.
2. Discharge and pressure calculation at any point may be tedious and time consuming and become complex design.

3. Circular or ring system

In this system, main pipeline laid on the periphery to form a closed ring, either small rectangular or circular blocks, around the area to be served. The submains take off from the main pipe lines and run on the interior of the area. In this system water can be supplied to any point from at least two directions. This system is suitable for well planed city having planed road networks. This system possesses the same merits and demerits as grid iron system. However pipe length is much longer and water availability for firefighting is large in quantity.

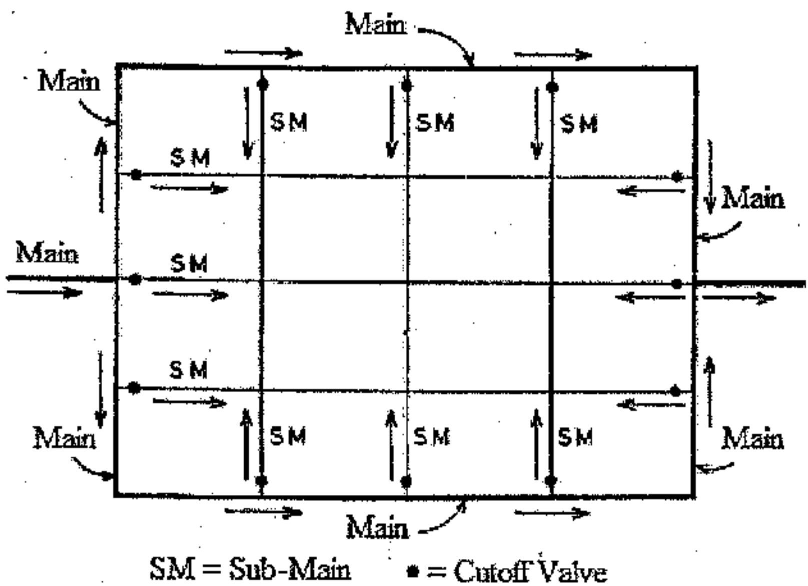
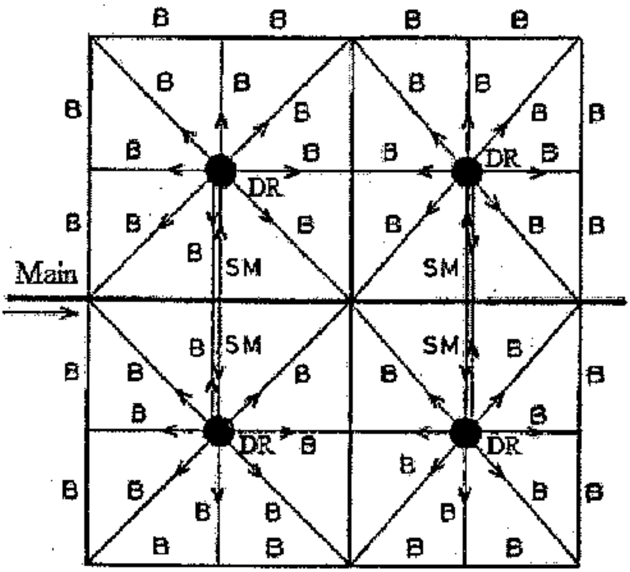


Figure 7-9 Ring System

4. Radial system

In this system the entire area to be served is divided into a number of small zones and in the centre of each zone a distribution reservoir is provided and from which water flow radially towards the outer periphery. Water from the main line is lifted into the distribution reservoir. This system ensures high pressure in distribution and it gives quick and efficient water distribution. This system is preferable for the cities having roads laid out radially. This system possesses the same merits and demerits as grid iron system. However it requires more reservoirs and water availability for firefighting is large in quantity.



DR = Distribution Reservoir
SM.= Sub-Main
B = Branch

Figure 7-10 Radial System

7.6 Design of distribution system

Distribution system design involves hydraulic and structural design of pipe. Basically in hydraulic design size of pipe and required residual pressure in pipelines is calculated. Structural design include determination of thickness of pipe, international pressure, external pressure, thermal stress, thrust blocks, flexural strength etc.

7.6.1 Pipe hydraulics

Calculation of size of pipe and velocity in pipelines continuity and Bernoulli's equation are used.

Continuity equation

This is the equation of mathematical expression for the principle of conservation of mass flow.

$$Q = AV = \text{constant}$$

At a constant discharge; increase in size velocity will be decreased and decrease in pipe size velocity and head loss will be increased. In pipe design velocity should be maintained such that it should be neither silting nor scouring.

Bernoulli' s concept

Bernoulli's concept is the mathematical expression for the principle of conservation of energy. It states for an ideal, incompressible, steady, irrotational flow; total energy per unit weight at any point of flow is constant, if no energy added and taken out from the mass. Using Bernoulli's equation in pipe design, the total energy at exist section is found by subtracting head loss from the total energy at inlet section.
Mathematically,

$$Z_1 + \frac{P_1}{\gamma} + \frac{V_1^2}{2g} = Z_2 + \frac{P_2}{\gamma} + \frac{V_2^2}{2g} + h_L$$

Where, Z= Potential head

$\frac{P}{\gamma}$ = Pressure head

$\frac{V_1^2}{2g}$ =Velocity head

h_L= head loss

Head losses in pipes

When water flows through pipes it losses energy due to fittings, change of section, wall friction, resistance or friction in the flow result the loss of head. The following are the head loss which occurs in flow of liquid.

a) Major loss
b) Minor loss

a) Major loss

The major head loss is due to frictional resistance of the pipe and can be calculated by using equations; Darcy Weisbach or Hazen William's or Manning's equation.

i) **Hazen William's equation**: This is common equation in pipe design.

Velocity (V) = $0.849\ CR^{0.63}\ S^{0.54}$

Where, C is roughness coefficient

(Value of C; New CI=130, GI=70, HDPE= 140, Old CI= 100)

Head loss (h_f) = SL

$$\therefore \quad h_f = \frac{10.68\ LQ^{1.852}}{d^{4.87} C^{1.852}}$$

ii) **Darcy Weisbach equation**: This equation is used to determine the major head loss in the pipes due to friction.

$$h_f = \frac{flV^2}{2gd} \qquad \left[V = \frac{4Q}{\pi d^2}\right]$$

$$\therefore \quad h_f = \frac{flQ^2}{12.1d^5}$$

Where, value frictional factor (f) = 0.02 to 0.075

iii) **Manning's equation**: This equation is common for open channels flow whereas it can be used in pipe flow.

$$V = \frac{1}{n} R^{2/3} S^{1/2}$$

$$h_f = SL$$

$$\therefore \quad h_f = \frac{10.294 n^2 L Q^2}{d^{\frac{16}{3}}}$$

b) Minor loss

Loss of energy due to either magnitude or the change of direction in pipe is minor loss. If the length of pipe is large the loss of energy due to friction is cooperatively high than minor losses so it is called minor loss and can be neglected.

7.6.2 Design criteria of distribution system

There are some criterions that should be satisfied at the design of distribution system of water.

1. **Discharge**

   Pipe design in distribution system should be for the peak or maximum discharge. Generally for continuous system design flow is considered as of 2 to 4 times average demand.

2. **Residual pressure**

   Pressure in distribution line is desired for flow of water to the consumers' overhead tank and maintains distribution equally to all consumers building situated at different elevation. In rural area of Nepal residual pressure to be maintained should be more than 0.5 kg/cm^2. The residual pressure required for private connections is 1.5 kg/cm^2.

3. **Size of pipe**

   Pipe size should be adopted as commercially available size. The minimum size considered in pipe design is 20 mm. Commercially available size in market are; 15, 20, 25, 32, 40, 50, 65, 80,100, 125, 150, 175, 200, 250, 300, 350, 400, 450, 500, 600, 700, 800, 900, 1000, 1200, 1400, 1600, 1800, 2000, 2200, 2400, 2600, 2800, 3000 mm. It is recommended adopting on higher side of calculated size.

4. **Velocity**

   In pipe flow velocity should be neither silting nor scouring and should be maintained self cleansing velocity so that sediment deposition problem in pipeline do not occur. Recommended velocity for treated water supply is 0.3 to 3 m/sec. For unpurified water supply minimum velocity of 0.6 m/sec may be considered.

7.6.3 Design steps involved in water supply distribution system

1. **Population survey and preparation of contour maps**

   Population to be served needs to be surveyed and design year population is calculated with suitable forecasting method. Topography of the area between treatment plant and distribution arêa prepared with details showing roads, parks, electric lines, telephone lines, existing water supply lines is prepared and studied.

2. **Tentative layout**

   Tentative layout of various components like reservoirs, valves, hydrants, mains, sub-mains, position or location is to be marked.

3. **Calculation of discharge**

   Based on the density of population, distribution zones and fire demand discharge is computed. Average quantity of discharge is calculated from population and per capita demand and in distribution line is design for peak discharge and generally it is considered as 2 to 4 times of average discharge.

4. **Computation of pipe diameters**

   From the contour map reduced level of difference point can be known. Adding the residual head in elevation difference; allowable head loss in pipe can be known. Once maximum allowable head loss obtained, using head loss equation diameter can be obtained to deliver required discharge.

5. **Computation of residual pressure and velocity**

   Computation of residual pressure head at any point of distribution lines can be known by computing pressure available in the upstream points, reduced level of that point, and actual head loss. In the context of Nepal minimum residual pressure head is 0.5 kg/cm^2. For calculation of head loss; Manning's equation is common in open channel where as Hazen William's in pipe flow. Once diameter of pipe adopted velocity is computed for design discharge.

7.6.4 Design of pipe networks

There are two main methods of laying of pipe networks. These are branched and looped system.

Branched system

This system is the tree or dead end system. To design the system following steps should be followed.

1. Calculate population to be served by each section of design year.
2. Calculate design discharge for each section with the help of per capita demand, population and peak factor.
3. Assume size of pipe for each section.
4. Calculate head loss for each section using head loss equation.
5. Check residual pressure and velocity within permissible value. Adopt the size if pressure and velocity within allowable limits otherwise repeat the process after changing size of pipe.

Looped system

Looped system consists of pipe loops; loop pipe design is complex system. *It may consist of pipe in series or in parallel. Pipe in loop may be analyzed and designed by various methods. Following methods* mostly used to analysis of a loop system.

1. Equivalent pipe method
2. Hardy cross method

1. Equivalent pipe method:

This method is based on the principle that; loops are replaced by single equivalent pipe. Equivalent pipe that can deliver equal discharge losing equal head by given pipe system that has to replace. Equivalent pipe method is based on following assumptions.

1. Head loss through pipes in series are additive
2. Head loss through parallel pipes are equal

Consider a loop of pipe network as shown in figure below. In portion ABC pipe AB and BC and portion ADC pipe AD and DC are in series so head loss in pipes are additive whereas pipe ABC and ADC are in parallel so head loss occurred is equal.

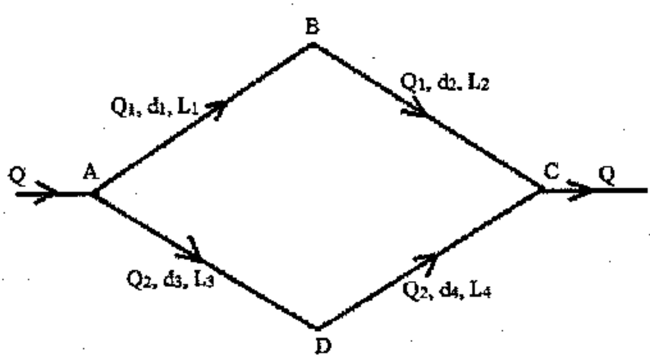


Figure 7-11 Pipe Network

Pipe in series

Let us consider ABC portion with AB and BC pipes in series in figure 7-11 and the equivalent pipe of length L_e and diameter d_e. So, head is additive and given by,

$(h_f)_{AC} = (h_f)_{AB} + (h_f)_{BC}$

Using Hazen William's equation

$$\frac{10.68\, L_e Q_1^{1.852}}{d_e^{4.87}\; C^{1.852}} = \frac{10.68\, L_1 Q_1^{1.852}}{d_1^{4.87}\; C^{1.852}} + \frac{10.68\, L_2 Q_1^{1.852}}{d_2^{4.87}\; C^{1.852}}$$

$$\text{or, } \frac{L_e}{d_e^{4.87}} = \frac{L_1}{d_1^{4.87}} + \frac{L_2}{d_2^{4.87}} \qquad \left[\therefore \quad h_f = \frac{10.68\, LQ^{1.852}}{d^{4.87} C^{1.852}} \right]$$

$$\text{or, } L_e = d_e^{4.87} \left[\frac{L_1}{d_1^{4.87}} + \frac{L_2}{d_2^{4.87}} \right]$$

Generalizing for pipes in series;

$$\text{or, } L_e = d_e^{4.87} \left[\frac{L_1}{d_1^{4.87}} + \frac{L_2}{d_2^{4.87}} + \ldots\ldots \right]$$

Pipes in parallel

Consider ABC and ADC pipes in parallel in figure 7-11 and let the equivalent pipe of length L_e and diameter d_e. Then head loss occurred in parallel pipes is equal and given by;

$(h_f)_{AC} = (h_f)_{AB} + (h_f)_{BC} = (h_f)_{AD} + (h_f)_{DC}$

Using Hazen William's equation

$$\frac{10.68\, L_1 Q_1^{1.852}}{d_1^{4.87}\; C^{1.852}} + \frac{10.68\, L_2 Q_1^{1.852}}{d_2^{4.87}\; C^{1.852}} = \frac{10.68\, L_3 Q_2^{1.852}}{d_3^{4.87}\; C^{1.852}} + \frac{10.68\, L_4 Q_2^{1.852}}{d_4^{4.87}\; C^{1.852}}$$

$$\left[\frac{Q_1}{Q_2}\right] = \left(\frac{\frac{L_3}{d_3^{4.87}} + \frac{L_4}{d_4^{4.87}}}{\frac{L_1}{d_1^{4.87}} + \frac{L_2}{d_2^{4.87}}}\right)^{\frac{1}{1.852}} = K(\text{Say})$$

Therefore; $Q_1 = Q_2 K$

Again, $Q = Q_1 + Q_2 = Q_2 (K+1)$

If the length and diameter of equivalent pipe causing equal head loss between A and C are L_e and d_e.

$(h_f)_{AC} = (h_f)_{AD} + (h_f)_{DC}$

Using Hazen William's equation

$$\frac{10.68\, L_e Q^{1.852}}{d_e^{4.87}\, C^{1.852}} = \frac{10.68\, L_3 Q_2^{1.852}}{d_3^{4.87}\, C^{1.852}} + \frac{10.68\, L_4 Q_2^{1.852}}{d_4^{4.87} C^{1.852}}$$

$$\text{or, } \frac{L_e}{d_e^{4.87}} (K+1)^{1.852} = \left[\frac{L_3}{d_3^{4.87}} + \frac{L_4}{d_4^{4.87}}\right]$$

$$\text{or, } L_e = \frac{d_e^{4.87}}{\left(\frac{Q_1}{Q_2} + 1\right)^{1.852}} \left[\frac{L_3}{d_3^{4.87}} + \frac{L_4}{d_4^{4.87}}\right]$$

7.6.5 Hardy Cross Method

It is a method of successive approximations involves a controlled trial and error process. This method is based on following three laws.

1. In each pipe of the network there is relationship between the head loss in the pipe and discharge through it i.e. $h_f = r\, Q^n$ where r and n are constants.
2. At each junction the algebraic sum of the quantities of water entering and leaving the junction is zero i.e. $\sum Q = 0$.
3. In each loop head loss due to flow in clockwise and anticlockwise must be equal to zero i.e. $\sum h_f = 0$.

The analysis of pipe network by Hardy Cross method can be done by any one of the following method.

a. Balancing head by correcting assumed flows

b. Balancing flow by correcting assumed heads

a) Balancing head by correcting assumed flows

This is the common Hardy Cross method used when the flow of water entering and leaving in the network is known. In this method loss of head in the loop is balanced by the correction of assumed flows.

Let, Q is correct flow and Q_o is assumed flow in the pipe. Then,

$$Q = Q_o + \Delta Q$$

Where, ΔQ is flow correction

If h_f is head loss in pipe under reference;

$h_f = r\,Q^n$ where r and n is constant

Now, $h_f = r\,(Q_o + \Delta Q)^n$

$$h_f = r(Q_o^n + nQ_o^{n-i}\Delta Q + \ldots + \Delta Q^n)$$

If ΔQ is small compared to Q_o all terms of the series after the second one (higher power) may be neglected.

Now,

$$h_f = r(Q_o^n + nQ_o^{n-i}\Delta Q)$$

For a loop pipe, $\sum h_f = 0$

$$h_f = 0 = r(Q_o^n + nQ_o^{n-i}\Delta Q)$$

$$\Delta Q = -\frac{\Sigma r Q_o^n}{\sum rnQ_o^{n-1}} = -\frac{\sum h_f}{n\sum(h_f/Q_o)}$$

Value of n =2 for Manning's and Darcy Weisbach equation. For Hazen William's equation n= 1.852

Value of r is;

$$= \frac{10.68\,LQ^{1.852}}{d^{4.87}C^{1.852}} \quad \text{For Hazen William's}$$

$$= \frac{flQ^2}{12.1d^5} \quad \text{For Darcy Weisbach}$$

$$= \frac{10.294n^2LQ^2}{d^{\frac{16}{3}}} \quad \text{For Manning's}$$

Steps involved in this method

1. In a junction, assume suitable Q_o in each pipe so that $\sum Q = 0$ (inflow positive and outflow negative)
2. Compute h_f in each pipe considering $h_f = r\,Q^n$ (assume h_f positive for clockwise Q_o and negative for anticlockwise Q_o in the loop)
3. If $\sum h_f = 0$, assumed flow (Q_o) will be ok. If not, calculate flow correction by,

$$\Delta Q = -\frac{\Sigma rQ_o^n}{\sum rnQ_o^{n-1}} = -\frac{\sum h_f}{n\sum(h_f/Q_o)}$$

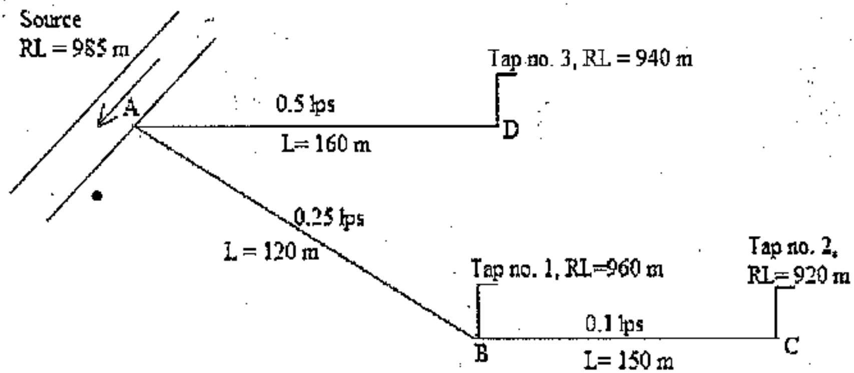
4. For common pipes in the two loops correction from both loops is required.
5. Repeat trials with corrected flow till ΔQ become negligible.

b) Balancing flow by correcting assumed heads

This is the modification of original Hardy Cross method and useful in case of unknown discharge and having several inlets in the pipe network. In this method, flows at each junction are corrected for assumed heads at the junction and the corresponding head loss in pipes.

Example: 7.1

Design pipelines AB, BC and AD for the following pipe network. A minimum pressure of 1 kg/cm^2 is required at the tap. Take Hazen William constant C = 100.



Solution:

$$\text{Residual head (H)} = \frac{P}{\gamma} = \frac{1\ \text{kg/cm}^2}{10000\ \text{N/m}^3} = \frac{1 \times 10 \times 10000\ \text{N/m}^2}{10000\ \text{N/m}^3} = 10\ \text{m}$$

Pipe AB

Maximum allowable head loss $h_{f(AB)} = 985 - (960+10)$

$= 15$ m

Length (L) = 120 m, flow (Q) = 0.25×10^{-3} m^3/sec

C = 100,

Using Hazen William's head loss equation;

$$h_f = \frac{10.68\ LQ^{1.852}}{d^{4.87} C^{1.852}}$$

$$\text{or, } 15\ \text{m} = \frac{10.68 \times 120 \times \left(0.25 \times 10^{-3}\right)^{1.852}}{d^{4.87} \times (100)^{1.852}}$$

or, d = 18.46 mm

Adopting commercially available pipe size 20 mm

Actual head loss in pipe AB

$$h_f = \frac{10.68\ LQ^{1.852}}{d^{4.87}C^{1.852}}$$

$$\text{or, } h_f = \frac{10.68 \times 120 \times \left(0.25 \times 10^{-3}\right)^{1.852}}{0.02^{4.87} \times (100)^{1.852}}$$

$$= 10.156\ \text{m}$$

Residual head available at B = 985 - (960+10.156) =14.844 m which is greater than 10 m. Hence OK.

HGL at point B = 985 m – 10.156 m

= 974.844 m

Check:

$$\text{Velocity (v)} = \frac{4Q}{\pi d^2} = \frac{0.25 \times 10^{-3} \times 4}{\pi (0.02)^2} = 0.795\ \text{m/sec}$$

(0.3 m/sec- 3 m/sec) OK

<u>Pipe BC</u>

Maximum allowable head loss $h_{f(BC)}$ = 974.844 – (920+10)

= 44.844 m

Length (L) = 150 m, flow (Q) = 0.1×10^{-3} m^3/sec

C = 100,

Using Hazen William's head loss equation;

$$h_f = \frac{10.68\ LQ^{1.852}}{d^{4.87}C^{1.852}}$$

$$\text{or, } 44.844\ \text{m} = \frac{10.68 \times 150 \times \left(0.1 \times 10^{-3}\right)^{1.852}}{d^{4.87} \times (100)^{1.852}}$$

or, d = 10.89 mm

Adopting commercially available pipe size 15 mm

(Note: It is recommended to use minimum pipe size 20 mm while less than 20 mm pipe is rarely used)

Actual head loss in pipe AB

$$h_f = \frac{10.68\,LQ^{1.852}}{d^{4.87}C^{1.852}}$$

$$\text{or, } h_f = \frac{10.68 \times 150 \times \left(0.1 \times 10^{-3}\right)^{1.852}}{0.015^{4.87} \times (100)^{1.852}}$$

$$= 9.44 \text{ m}$$

Residual head available at point B

$$= 974.844 \text{ m} - (920+9.44)$$

$$= 45.40 \text{ m} > 10 \text{ m OK.}$$

Check:

$$\text{Velocity (v)} = \frac{4Q}{\pi d^2} = \frac{0.1 \times 10^{-3} \times 4}{\pi (0.015)^2} = 0.565 \text{ m/sec}$$

Pipe size can be reducing to achieve self cleansing velocity but residual head should be maintained.

(0.3 m/sec- 3 m/sec) OK

<u>Pipe AD</u>

Maximum allowable head loss $h_{f(AD)} = 985 - (940+10)$

$$= 35 \text{ m}$$

Length (L) = 160 m, flow (Q) = 0. 5 × 10^{-3} m^3/sec

C = 100,

Using Hazen William's head loss equation;

$$h_f = \frac{10.68\,LQ^{1.852}}{d^{4.87}C^{1.852}}$$

$$\text{or, } 35\text{ m} = \frac{10.68 \times 160 \times \left(0.5 \times 10^{-3}\right)^{1.852}}{d^{4.87} \times (100)^{1.852}}$$

$$\text{or, } d = 21.42\text{ mm}$$

Adopting commercially available pipe size 25 mm

Actual head loss in pipe AB

$$h_f = \frac{10.68\,LQ^{1.852}}{d^{4.87}C^{1.852}}$$

$$\text{or, } h_f = \frac{10.68 \times 160 \times \left(0.5 \times 10^{-3}\right)^{1.852}}{0.025^{4.87} \times (100)^{1.852}}$$

$$= 16.48\text{ m}$$

Head available at B = 985 - (940+16.48) = 28.52m which is greater than 10 m. Hence OK.

HGL at point B = 985 m – 16.48 m

$$= 968.52\text{ m}$$

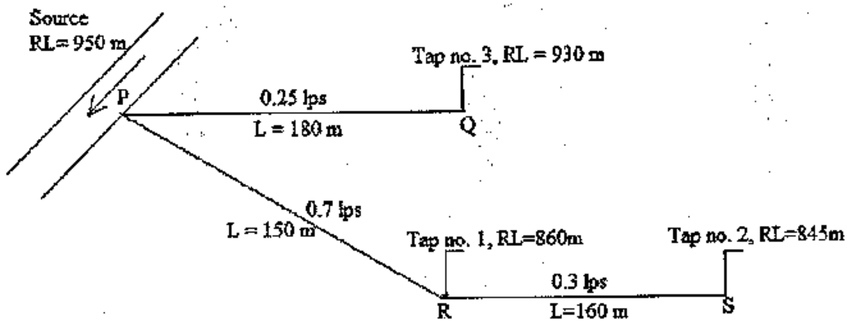
Check:

$$\text{Velocity } (v) = \frac{4Q}{\pi d^2} = \frac{0.5 \times 10^{-3} \times 4}{\pi (0.025)^2} = 1.018\text{ m/sec}$$

(0.3 m/sec- 3 m/sec) OK

Example: 7.2

Design pipelines PQ, PR and RS for the following pipe network. A minimum pressure of 1 kg/cm^2 is required at the tap. Take Hazen William constant C = 110.



Solution:

Residual head (H) = $\dfrac{P}{\gamma} = \dfrac{1\ \text{kg/cm}^2}{10000\ \text{N/m}^3} = \dfrac{1\times 10\times 10000\ \text{N/m}^2}{10000\ \text{N/m}^3} = 10\ \text{m}$

<u>Pipe PR</u>

Maximum allowable head loss $h_{f(PR)} = 950 - (860+10)$

$= 80$ m

Length (L) = 150 m, flow (Q) = 0.7×10^{-3} m^3/sec

C = 110,

Using Hazen William's head loss equation;

$$h_f = \frac{10.68\, LQ^{1.852}}{d^{4.87} C^{1.852}}$$

$$\text{or, } 80\ \text{m} = \frac{10.68 \times 150 \times \left(0.7\times 10^{-3}\right)^{1.852}}{d^{4.87} \times (110)^{1.852}}$$

or, d = 19.55 mm

Adopting commercially available pipe size 20 mm

Actual head loss in pipe PR

$$h_f = \frac{10.68\ LQ^{1.852}}{d^{4.87}C^{1.852}}$$

$$\text{or, } h_f = \frac{10.68 \times 150 \times \left(0.7 \times 10^{-3}\right)^{1.852}}{0.02^{4.87} \times (110)^{1.852}}$$

$$= 71.63 \text{ m}$$

Residual head available at R = 950 - (860+71.63) =18.37 m which is greater than 10 m. Hence OK.

HGL at point B = 950 m – 71.633 m

= 878.367 m

Check:

$$\text{Velocity (v)} = \frac{4Q}{\pi d^2} = \frac{0.7 \times 10^{-3} \times 4}{\pi (0.02)^2} = 2.228 \text{ m/sec}$$

(0.3 m/sec- 3 m/sec) OK

Pipe RS

Maximum allowable head loss $h_{f(RS)}$ = 878.367 – (845+10)

= 23.367 m

Length (L) = 160 m, flow (Q) = 0.3×10^{-3} m^3/sec

C = 110,

Using Hazen William's head loss equation;

$$h_f = \frac{10.68\ LQ^{1.852}}{d^{4.87}C^{1.852}}$$

$$\text{or, } 23.367 \text{ m} = \frac{10.68 \times 160 \times \left(0.3 \times 10^{-3}\right)^{1.852}}{d^{4.87} \times (110)^{1.852}}$$

or, d = 18.48 mm

Adopting commercially available pipe size 20 mm

Actual head loss in pipe RS

$$h_f = \frac{10.68\ LQ^{1.852}}{d^{4.87}C^{1.852}}$$

$$\text{or, } h_f = \frac{10.68 \times 160 \times \left(0.3 \times 10^{-3}\right)^{1.852}}{0.02^{4.87} \times (110)^{1.852}}$$

$$= 15.9 \text{ m}$$

Residual head available at point S

$$= 878.367 \text{ m} - (845 + 15.90)$$

$$= 17.467 \text{ m} > 10 \text{ m OK.}$$

Check:

$$\text{Velocity (v)} = \frac{4Q}{\pi d^2} = \frac{0.3 \times 10^{-3} \times 4}{\pi (0.02)^2} = 0.955 \text{ m/sec}$$

(0.3 m/sec- 3 m/sec) OK

Pipe PQ

Maximum allowable head loss $h_{f(PQ)} = 950 - (930+10)$

$= 10$ m

Length (L) = 180 m, flow (Q) = 0.25×10^{-3} m^3/sec

C = 110,

Using Hazen William's head loss equation;

$$h_f = \frac{10.68\ LQ^{1.852}}{d^{4.87}C^{1.852}}$$

$$\text{or, } 10 \text{ m} = \frac{10.68 \times 180 \times \left(0.25 \times 10^{-3}\right)^{1.852}}{d^{4.87} \times (110)^{1.852}}$$

or, d = 21.03 mm

Adopting commercially available pipe size 25 mm

Actual head loss in pipe PQ

$$h_f = \frac{10.68\ LQ^{1.852}}{d^{4.87}C^{1.852}}$$

$$\text{or, } h_f = \frac{10.68 \times 180 \times \left(0.25 \times 10^{-3}\right)^{1.852}}{0.025^{4.87} \times (110)^{1.852}}$$

$$= 4.3 \text{ m}$$

Residual head available at Q = 950 - (930+4.3) = 15.7 m which is greater than 10 m. Hence OK.

HGL at point B = 950 m – 4.3 m

= 45.70 m

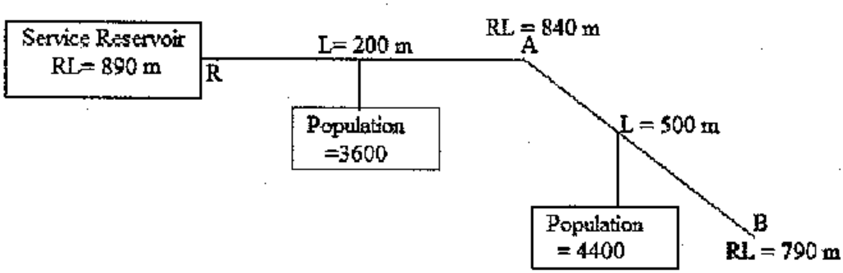
Check:

$$\text{Velocity (v)} = \frac{4Q}{\pi d^2} = \frac{0.25 \times 10^{-3} \times 4}{\pi (0.025)^2} = 0.52 \text{ m/sec}$$

(0.3 m/sec- 3 m/sec) OK

Example: 7.3

Design pipelines RA and AB for the water distribution network shown below.



Take per capita demand of water as 200 lpcd. Assume peak factor = 3 and Hazen William's constant C = 100. The residual head at any point in the distribution system should not be less than 15 m. Check velocity in the pipe also.

Solution:

Pipe RA

Average quantity of flow = (3600+4400) × 200 lpcd

= 1600000 lit/day

Design discharge (Q) = peak factor × average flow

= 3 × 1600000 lit/day

= 0.056 m^3/sec

Maximum allowable head loss $h_{f(RA)}$ = 890 – (840+15)

= 35 m

Length (L) = 200 m,

C = 100

Using Hazen William's head loss equation;

$$h_f = \frac{10.68\,LQ^{1.852}}{d^{4.87}C^{1.852}}$$

$$\text{or, } 35\text{ m} = \frac{10.68 \times 200 \times (0.056)^{1.852}}{d^{4.87} \times (100)^{1.852}}$$

or, d = 134.49 mm

Adopting commercially available pipe size 150 mm

Check:

$$\text{Velocity (v)} = \frac{4Q}{\pi d^2} = \frac{0.056 \times 4}{\pi (0.15)^2} = 3.17 \text{ m/sec}$$

Limiting velocity is greater than limit so size of pipe should be increased.

Taking commercial size 175 mm

$$\text{Velocity (v)} = \frac{4Q}{\pi d^2} = \frac{0.056 \times 4}{\pi (0.175)^2} = 2.328 \text{ m/sec}$$

(0.3 m/sec- 3 m/sec) OK

Actual head loss in pipe RA

$$h_f = \frac{10.68\ LQ^{1.852}}{d^{4.87} C^{1.852}}$$

$$\text{or, } h_f = \frac{10.68 \times 200 \times (0.056)^{1.852}}{0.175^{4.87} \times (100)^{1.852}}$$

$$= 9.855 \text{ m}$$

Residual head available at A = 890 - (840+9.855)

= 40.145 m which is greater

than 10 m. Hence OK.

HGL at point B = 890 m – 9.855 m

= 880.145 m

Pipe RS

Average quantity of flow = 4400 × 200 lpcd

= 880000 lit/day

Design discharge (Q) = peak factor × average flow

= 3 × 880000 lit/day

= 0.0306 m^3/sec

Maximum allowable head loss $h_{f(RS)}$ = 880.145 – (790+15)

= 75.145 m

Length (L) = 500 m,

C = 100,

Using Hazen William's head loss equation;

$$h_f = \frac{10.68\ LQ^{1.852}}{d^{4.87}C^{1.852}}$$

$$\text{or, } 75.145\ \text{m} = \frac{10.68 \times 500 \times (0.0306)^{1.852}}{d^{4.87} \times (100)^{1.852}}$$

or, d = 110.60 mm

Adopting commercially available pipe size 125 mm

Actual head loss in pipe RS

$$h_f = \frac{10.68\ LQ^{1.852}}{d^{4.87}C^{1.852}}$$

$$\text{or, } h_f = \frac{10.68 \times 500 \times (0.0306)^{1.852}}{0.125^{4.87} \times (100)^{1.852}}$$

$$= 41.41\ \text{m}$$

Residual head available at point S

$$= 880.145\ \text{m} - (790+41.41)$$

$$= 40.735\ \text{m} > 10\ \text{m OK.}$$

Check:

$$\text{Velocity (v)} = \frac{4Q}{\pi d^2} = \frac{0.0306 \times 10^{-3} \times 4}{\pi (0.125)^2} = 2.5\ \text{m/sec}$$

(0.3 m/sec- 3 m/sec) OK

Example: 7.4

Case I: Supply and demand both continuous

The water demand of a city is 320 MLD. The water demand is to meet from the river flowing under gravity to the reservoir. The water is supplied to the consumers from the reservoir by continuous system. Calculate the capacity of balancing reservoir for the consumption pattern as shown below.

Time	05-07	07-12	12-17	17-19	19-05
Water consumption (%)	25	30	15	20	10

Solution:
In this case; assuming water tapped from the source is equal total daily demand.

Total water demand (Q_d) = 320 ML/day

Total supply of water (Q_s) = 320 ML/day

= 13.333 ML/hr

Time (hr)	Duration (hr)	Water Cons. (%)	Demand (ML)	Cum. Demand (ML)	Supply (ML)	Cum. Supply (ML)	Cum. Surplus (ML)	Cum. Deficit (ML)
5-7	2	25	80	80	26.666	26.666	-	53.334
7-12	5	30	96	176	66.666	93.332	-	82.668
12-17	5	15	48	224	66.667	159.999	-	64.001
17-19	2	20	64	288	26.667	186.666	-	**101.334**
19-5	10	10	32	320	133.334	320	-	-

In this case;

Total demand (TD) = 320 ML

Total supply (TS) = 320 ML

TS = TD,

Maximum cumulative surplus (MCS) = 0

Maximum cumulative deficit (MCD) = 101.334 ML

Reservoir Capacity = MCS+MCD

= 0 + 101.334

= 101.334 ML

Case II: Supply continuous and demand intermittent

The water demand of a city is 320 MLD. The water demand is to meet from the river flowing under gravity to the reservoir. The water is supplied to the consumers from the reservoir by intermittent system supplying water from 5 to 7 hours in the morning and 16 to 18 hours in the evening. Calculate the capacity of balancing reservoir.

Assuming water tapped from the source is equal total daily demand.

Total water demand (Q_d) = 320 ML/day = 80 ML/hr

Total supply of water (Q_s) = 320 ML/day = 13.34 ML/hr

Time (hr)	Duration (hrs) In	Duration (hrs) Out	Demand (ML)	Cum. Demand (ML)	Supply (ML)	Cum. Supply (ML)	Cum. Surplus (ML)	Cum. Deficit (ML)
5-7	2	2	160	160	26.666	26.666	-	133.334
7-16	9	-	-	160	120	146.666	-	13.334
16-18	2	2	160	320	26.667	173.333	-	146.667
18-5	11	-	-	320	146.667	320	-	-

In this case;

Total demand (TD) = 320 ML

Total supply (TS) = 320 ML

TS = TD,

Maximum cumulative surplus (MCS) = 0

Maximum cumulative deficit (MCD) = 146.667 ML

Reservoir Capacity = MCS+MCD

= 0 + 146.667

= 146.667 ML

Case III: Supply intermittent and demand continuous

The water demand of a city is 320 MLD. The water demand is to meet through pumping from a tubewell. The pumping period is 4 to 10 hours in the morning and 16 to 22 hours in the evening. The water is supplied to the consumers from the reservoir by continuous system. Calculate the capacity of balancing reservoir for the consumption pattern as shown below.

Time	05-07	07-12	12-17	17-19	19-05
Water consumption (%)	25	30	15	20	10

Solution:

Assuming water tapped from the source is equal total daily demand.

Total water demand (Q_d) = 320 ML/day

Total supply of water (Q_s) = 320 /12 = 26.666 ML/hr

Time (hr)	Hours		Water Cons. (%)	Demand (ML)	Cum. Demand (ML)	Supply (ML)	Cum. Supply (ML)	Cum. Surplus (ML)	Cum. Deficit (ML)
	In	Out							
5-7	2	2	25	80	80	53.333	53.333	-	26.667
7-12	3	5	30	96	176	80	133.333	-	42.667
12-17	1	2	15	48	224	26.666	160	-	64.00
17-19	2	5	20	64	288	53.333	213.333	-	74.667
19-5	4	10	10	32	320	106.667	320	-	-

In this case;

Total demand (TD) = 320 ML

Total supply (TS) = 320 ML

TS = TD,

Maximum cumulative surplus (MCS) = 0

Maximum cumulative deficit (MCD) = 74.667 ML

Reservoir Capacity = MCS+MCD

$$= 0 + 74.667$$

$$= 74.667 \text{ ML}$$

Case IV: Supply and demand both intermittent

The water demand of a city is 320 MLD. The water demand should be meet through pumping from a tubewell. The pumping period is 4 to 10 hours in the morning and 16 to 22 hours in the evening. The water is supplied to the consumers from the reservoir by intermittent system supplying water from 5 to 7 hours in the morning and 16 to 18 hours in the evening. Calculate the capacity of balancing reservoir.

Solution:

Assuming water tapped from the source is equal total daily demand.

Total water demand (Q_d) = 320 ML/day = 320/4 = 80 ML/hr

Total supply of water (Q_s) = 320 ML/day = 320/12 ML/hr

= 26.666 ML/hr

Time (hr)	Duration (hr) In	Duration (hr) Out	Demand (ML)	Cum. Demand (ML)	Supply (ML)	Cum. Supply (ML)	Cum. Surplus (ML)	Cum. Deficit (ML)
5-7	2	2	160	160	53.333	53.333	-	106.667
7-16	3	-	-	160	80	133.333	-	26.667
16-18	2	2	160	320	53.334	186.667	-	133.333
18-5	5	-	-	320	133.333	320	-	-

In ths case;

Total demand (TD) = 320 ML

Total supply (TS) = 320 ML

TS = TD,

Maximum cumulative surplus (MCS) = 0

Maximum cumulative deficit (MCD) = 133.333 ML

Reservoir Capacity = MCS+MCD

= 0 + 133.333

= 133.333 ML

Example: 7.5

A village has design year population of 600 with 65 lpcd per capita demand. The demand is to be fulfilled by spring sources with safe yield 0.5 lps. The consumption pattern in % of a day is as below.

Time	05-07	07-12	12-17	17-19	19-05
Water consumption (%)	30	30	15	20	5

Is balancing reservoir necessary? Calculate capacity of balancing reservoir if needed.

Solution:

Average quantity of total water demand (Q_d)

$$= \text{population} \times \text{per capita demand}$$

$$= 600 \times 65 \text{ lpcd} = 39000 \text{ lit/day}$$

Safe yield of source = 0.5 lps = 43200 lit/day = 1625 lit/hr

In this case; water tapped from the source is more than total daily demand.

Time (hr)	Dur. (hr)	Water Cons. (%)	Demand (lit)	Cum. Demand (lit)	Supply (lit)	Cum. Supply (lit)	Cum. Surplus (lit)	Cum. Deficit (lit)
5-7	2	30	11700	11700	3600	3600	-	8100
7-12	5	30	11700	23400	9000	12600	-	10800
12-17	5	15	5850	29250	9000	21600	-	7650
17-19	2	20	7800	37050	3600	25200	-	11850
19-5	10	5	1950	39000	18000	43200	4200	-

There is surplus and deficit which should be stored for balancing hence reservoir is necessary.

In this case;

Total demand (TD) = 39000 lit

Total supply (TS) = 43200 lit

TS > TD,

Maximum cumulative surplus (MCS) = 4200 lit

Maximum cumulative deficit (MCD) = 11850 lit

Reservoir Capacity = MCS+MCD-TS+TD

= 4200 + 11850 - 43200 + 39000

= 11850 lit

Example: 7.6

For the water supply of a town with daily requirement of 0.25 MLD, it is proposed to construct a distribution reservoir. The consumption pattern is as follows:

Time	07-08	08-17	17-18:30	18:30-07
Water consumption (%)	30	35	30	5

The pumping is to be done at a constant rate of 0.032 ML/hr for 8 hrs (08:00 to 16:00). Determine the required capacity of balancing reservoir by analytical method.

Solution:

Total water demand (Q_d) = 0.25 ML/day

Total supply of water (Q_s) = 0.256 ML/day = 0.032 ML/hr

In this case; water tapped from the source is more than total daily demand.

Time (hr)	Hours		Water Cons. (%)	Demand (ML)	Cum. Demand (ML)	Supply (ML)	Cum. Supply (ML)	Cum. Surplus (ML)	Cum. Deficit (ML)
	In	Out							
07-08	1	-	30	0.0750	0.0750	-	0	0.0750	-
08-17	9	8	35	0.0875	0.1625	0.256	0.256	-	0.0935
17-18:30	1.5	-	30	0.0750	0.2375	-	0.256	-	0.0185
18:30-7	12.5	-	5	0.0125	0.2500	-	0.256	-	0.0060

There is surplus and deficit which should be stored for balancing hence reservoir is necessary.

In this case;

Total demand (TD) = 0.25 ML

Total supply (TS) = 0.256 ML

TS > TD,

Maximum cumulative surplus (MCS) = 0.0750 ML

Maximum cumulative deficit (MCD) = 0.0935 ML

Reservoir Capacity = MCS+MCD-TS+TD

= 0.0750 + 0.0935 – 0.256 + 0.25

= 0.1625 ML

Example: 7.5

A village in mid-western development region of Nepal has a design year population of 500. Per capita demand recommended for that particular village is 65 liters per day. The demand is to be met by a continuous system of supply from a spring source with safe yield of 0.5 lps. The consumption pattern is:

Time	Water consumption %
05:00-07:00	25
7:00-12:00	35
12:00-17:00	20
17:00-19:00	20
19:00-05:00	0

Solution:

Average quantity of total water demand (Q_d)

$$= \text{population} \times \text{per capita demand}$$

$$= 500 \times 65 \text{ lpcd} = 32500 \text{ lit/day}$$

Safe yield of source = 0.5 lps = 43200 lit/day = 1625 lit/hr

In this case; water tapped from the source is more than total daily demand.

Time (hr)	Dur. (hr)	Water Cons. (%)	Demand (lit)	Cum. Demand (lit)	Supply (lit)	Cum. Supply (lit)	Cum. Surplus (lit)	Cum. Deficit (lit)
5-7	2	25	8125	8125	3600	3600	-	4525
7-12	5	35	11375	19500	9000	12600	-	6900
12-17	5	20	6500	26000	9000	21600	-	4400
17-19	2	20	6500	32500	3600	25200	-	**7300**
19-5	10	0	0	32500	18000	43200	10700	-

There is surplus and deficit which should be stored for balancing hence reservoir is necessary.

In this case;

Total demand (TD) = 32500 lit

Total supply (TS) = 43200 lit

TS > TD,

Maximum cumulative surplus (MCS) = 7300 lit

Maximum cumulative deficit (MCD) = 10700 lit

Reservoir Capacity = MCS+MCD-TS+TD

$$= 7300 + 10700 - 43200 + 32500$$

$$= 7300 \text{ lit}$$

Example: 7.6

A town with a population with 1.2 million is to be supplied with water daily at 45 lpcd. Water has to be stored also for fire demand keeping at least 1% of total demand. The variation in demand is as follows;

Time	Water consumption %
05:00-07:00	25
7:00-12:00	35
12:00-17:00	20
17:00-19:00	20
19:00-05:00	0

Determine analytically the balancing reservoir capacity assuming pumping to be done at a uniform rate and the period of pumping is 5:00 AM to 10:00 AM and 5:00 PM to 8:00 PM in two shifts.

Solution:

Average quantity of total water demand (Q_d)

$$= \text{population} \times \text{per capita demand}$$

$$= 1.2 \times 10^6 \times 45 \text{ lpcd} = 54 \text{ ML/day}$$

Pumping hours = 8 hrs

Total quantity of water supplied (Q_s) = 54 ML/day

$$= \frac{54}{8} = 6.75 \text{ lit/hr}$$

In this case; water tapped from the source is more than total daily demand. ,

Time (hr)	Hours		Water Cons. (%)	Demand (ML)	Cum. Demand (ML)	Supply (ML)	Cum. Supply (ML)	Cum. Surplus (ML)	Cum. Deficit (ML)
	In	Out							
05-07	2	2	25	13.5	13.5	13.5	13.5	-	-
07-12	3	5	35	18.9	32.4	20.25	33.75	1.35	-
12-17	-	5	20	10.8	43.2	-	33.75	-	9.45
17-19	3	2	20	10.8	54	20.25	54	-	-
19-05	-	10	0	0	54	-	54	-	-

In this case;

Total demand (TD) = 54 ML

Total supply (TS) = 54 ML

TS = TD,

Maximum cumulative surplus (MCS) = 1.35 ML

Maximum cumulative deficit (MCD) = 9.45 ML

Reservoir Capacity = MCS+MCD

$$= 1.35 + 9.45$$

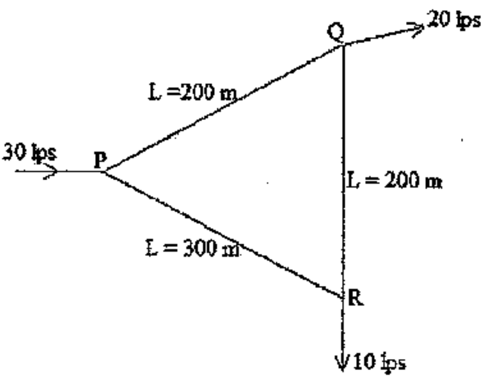
$$= 10.8 \text{ ML}$$

Fire fighting reserve 1% of total demand= 0.54 ML

Therefore reservoir capacity = 10.8 + 0.54 = 11.34 ML

Example: 7.7

Design pipelines PQ, QR and PR for the water distribution network given below. Assume Hazen William constant C = 100 for the entire pipe.



Assume RL of points P, Q, and R is same. The pressure available at P is 15 kg/cm^2 and minimum pressure at Q and R is 1.5 kg/cm^2.

Solution:

Pressure head (H) at point P;

At point Q and R minimum pressure head;

$$= \frac{P}{\gamma} = \frac{15 \text{ kg/cm}^2}{10000 \text{ N/m}^3} = \frac{15 \times 10 \times 10000 \text{ N/m}^2}{10000 \text{ N/m}^3} = 150 \text{ m}$$

$$= \frac{P}{\gamma} = \frac{1.5 \text{ kg/cm}^2}{10000 \text{ N/m}^3} = \frac{1.5 \times 10 \times 10000 \text{ N/m}^2}{10000 \text{ N/m}^3} = 15 \text{ m}$$

Assume size of pipes PQ = 100 mm, QR = 50 mm, and PR= 80 mm. Let us assume Flow in pipe PQ (Q_{PQ}) = Q m^3/sec, Q_{QR}=Q-0.02 m^3/sec, and $Q_{PR} = 0.03 - Q$ m^3/sec.

For the loop, $hf_{PQ} + hf_{QR} = hf_{PR}$

Using Hazen William's equation

$$\frac{10.68\times200\times Q^{1.852}}{0.1^{4.87}\ C^{1.852}}+\frac{10.68\times200\times(Q-0.02)^{1.852}}{0.05^{4.87}\ C^{1.852}} = \frac{10.68\times300\times(0.03-Q)^{1.852}}{0.08^{4.87}\,C^{1.852}}$$

$$\text{or,}\quad \frac{200\times Q^{1.852}}{0.1^{4.87}}+\frac{200\times(Q-0.02)^{1.852}}{0.05^{4.87}}=\frac{300\times(0.03-Q)^{1.852}}{0.08^{4.87}}$$

Solving;

$Q = 0.0206$ m^3/sec = 20.6 lps

Q_{PQ} = Q m^3/sec = 20.6 lps,

Q_{QR}=Q-0.02 m^3/sec = 0.6 lps and,

$Q_{PR} = 0.03 - Q$ m^3/sec = 9.4 lps

Head loss in pipe PQ

$$h_f = \frac{10.68\ LQ^{1.852}}{d^{4.87}C^{1.852}}$$

$$\text{or, } h_f = \frac{10.68\times200\times(0.0206)^{1.852}}{0.1^{4.87}\times(100)^{1.852}} = 23.60\,\text{m}$$

Pressure at Q = Pressure at P – h_f in PQ

= 150 – 23.6 =126.4 m = 12.4 kg/cm^2

Check:

$$\text{Velocity (v)} = \frac{4Q}{\pi d^2} = \frac{0.0206 \times 4}{\pi (0.1)^2} = 2.62 \text{ m/sec}$$

Head loss in pipe PR

$$h_f = \frac{10.68\, LQ^{1.852}}{d^{4.87} C^{1.852}}$$

$$\text{or, } h_f = \frac{10.68 \times 300 \times (0.0094)^{1.852}}{0.08^{4.87} \times (100)^{1.852}} = 24.54\,\text{m}$$

Pressure at R = Pressure at P – h_f in PR

= 150 – 24.54 =125.46 m = 12.546 kg/cm^2

Check:

$$\text{Velocity (v)} = \frac{4Q}{\pi d^2} = \frac{0.0094 \times 4}{\pi (0.08)^2} = 1.87 \text{ m/sec}$$

Head loss in pipe QR

$$h_f = \frac{10.68\, LQ^{1.852}}{d^{4.87} C^{1.852}}$$

$$\text{or, } h_f = \frac{10.68 \times 200 \times (0.0006)^{1.852}}{0.05^{4.87} \times (100)^{1.852}} = 0.99 \approx 1\,\text{m}$$

Check:

$$\text{Velocity (v)} = \frac{4Q}{\pi d^2} = \frac{0.0006 \times 4}{\pi (0.05)^2} = 0.306 \text{ m/sec}$$

(For the treated water neither silting nor scouring velocity should be between 0.3 to 3 m/sec)

Problems

1. List out the requirements of a good distribution system.
2. List the various systems of layout of a water distribution system. Describe gridiron system and its advantages and disadvantages with a neat sketch.
3. Describe the layout of distribution system with their advantages and disadvantages for water supply system.
4. Compare the continuous and intermittent system of water supply.
5. What are the general considerations to be observed in the planning of distribution system? Under what condition would you recommend the use of intermittent system of water supply?
6. What do you understand by the term balancing reserve, breakdown reserve and fire reserve?
7. State the factors you would take into consideration and the procedure you would follow in designing a distribution system for the water of a city.
8. List the various systems of layout of a water distribution system. Describe gridiron system and its advantages and disadvantages with a neat sketch.
9. The water demand of a town is 0.96 MLD. The water demand is to meet through pumping from a tubewell. The tentative pumping period is 4:00 to 10:00 hours in the morning and 16:00 to 22:00 hours in the evening. The water is supplied to the consumers from the reservoir by intermittent system supplying water from 5:00 to 7:00 hours in the morning and 16:00 to 18:00 hours in the evening. Calculate the capacity of balancing reservoir. Assume necessary data suitably.
10. A village has a design year population of 1500 and per capita water demand of 45 lpcd. The safe yield of a spring source is 0.9 lps. The system is continuous system and the consumption pattern is as follows.

Time	Consumption, %
05:00-07:00	25
07:00-12:00	35
12:00-17:00	20
17:00-19:00	20
19:00-05:00	0

Is a balancing storage tank necessary? If necessary calculate its capacity and justify your answer.

11. A village has a design year water demand 15 m^3/day. The demand is to meet by continuous system of supply from spring source with safe yield of 0.2 lps. The consumption pattern is as follows.

Time	Consumption, %
05:00-07:00	25
07:00-12:00	35
12:00-17:00	20
17:00-19:00	20
19:00-05:00	0

Is a balancing storage tank necessary? If necessary calculate its capacity and justify your answer.

12. A newly established town with a population of 1.5 million is to be supplied with water daily at 80 liters per head. The variation in demand is as follows.

Time	Consumption, %
05:00-10:00	45
10:00-14:00	10
14:00-18:00	25
18:00-22:00	15
22:00-05:00	5

Determine analytically the balancing reservoir capacity assuming pumping to be done at uniform rate and the period of pumping is 05:00 to 18:00 hours. Neglect fire demand. (Ans: 32 ML)

13. A city with a population of 1.2 million has a continuous water supply system. The average water demand of the city is 250 lpcd. The water is supplied to the city through a reservoir. The total water supply of 200 lpcd is supplies as follows.

Time : Hour	05-11	11-15	15-21	21-24	24-05
Supply, lpcd	25	30	15	20	10

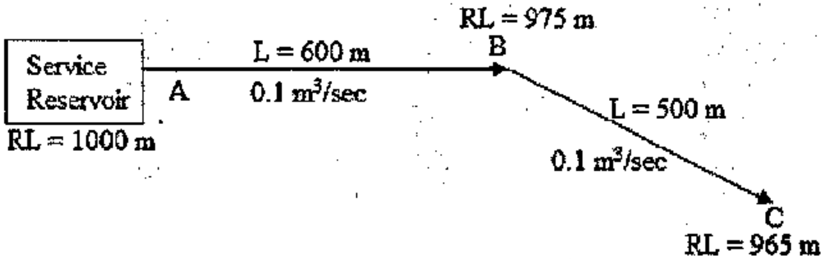
Water is pumped from the tube well to the reservoir at a uniform rate of 12.5×10^6 liter/hour for all the 24 hours. Determine the capacity of reservoir.

14. A village has a design year water demand 90 m^3/day. The demand is to meet by continuous system of supply. The water tapped from source is 1.4 lps. The consumption pattern is as follows.

Time	Consumption, %
05:00-07:00	25
07:00-12:00	35
12:00-17:00	20
17:00-19:00	20
19:00-05:00	0

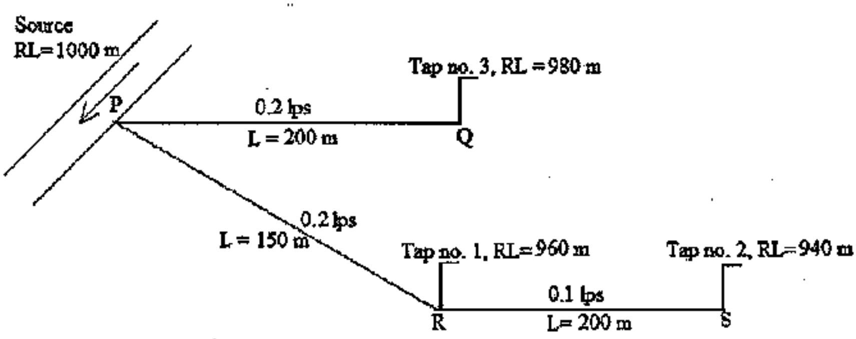
Is a balancing storage tank necessary? If necessary calculate its capacity and justify your answer.

15. A layout of rural water distribution system is shown below. Design pipes AB and BC. Assume f = 0.04 and minimum pressure in pipe line should be 1.5 kg/cm^2.

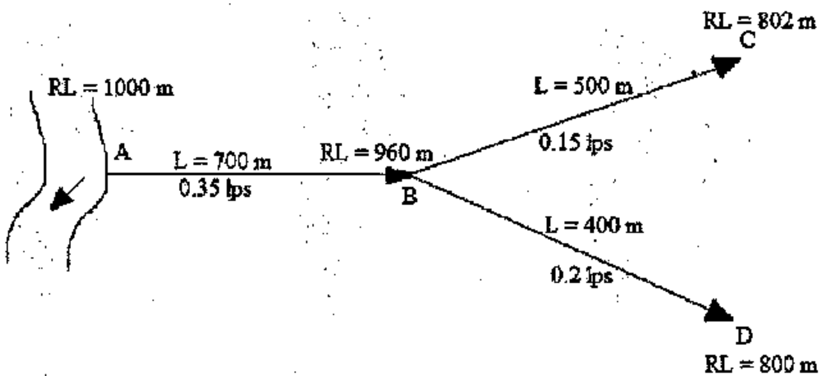


16. Design suitable size of pipes AB and BC having 300 and 500 meters length respectively. Assume Hazen Williams coefficient C as 100 and maximum demand as 3 times the average demand. The average water consumption is 75 lpcd and the population is distributed within the two blocks of 825 in AB section and 455 in BC section. The elevated storage tank is fixed at point A above 4 meter height tower. The RL of points A, B, and C are 2150, 2130 and 2120 meters respectively. The minimum pressure head of water is to be 5 m. Check velocity in the pipes also.

17. Design pipelines PR, RS and PQ for the following pipe network. A minimum pressure of 5 m is required at the tap. Take Hazen William constant C = 150.

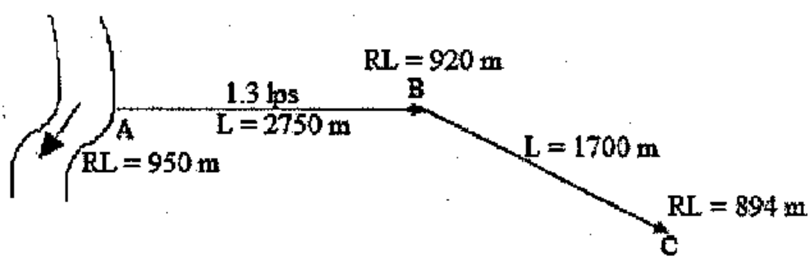


18. A layout of rural water distribution system is shown below.

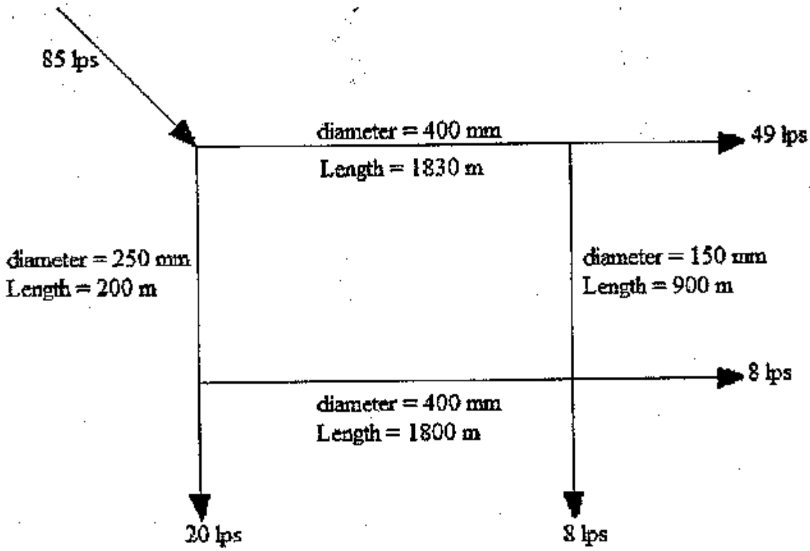


Design pipes AB, BC, and BD. Assume Hazen William's coefficient C = 100 and minimum pressure to be maintained at all points of pipeline is 1 kg/cm^2.

19. A layout of water distribution is shown in figure below. Design pipelines AB and BC considering Hazen William's Coefficient 100. Minimum pressure required at B and C is 1 kg/cm^2 of water.



20. Calculate the head losses and the corrected flows in the various points of the distribution network in the figure below after one correction. The diameter and lengths of the pipes used are given against each pipe. Use Hardy- Cross method with Hazen William's formula. Take Hazen William's Coefficient = 1.



8 Conveyance of Water

The water withdrawn from the source is conveyed or transported to the treatment plant and after purification it is supplied to the consumers. If the source is at the higher elevation water can be transmit and distribute through conduit under gravity is called gravity conduit. In gravity conduit water flows under action of gravity and free surface is developed in which water surface will be atmospheric pressure. In such case HGL (hydraulic grade line) will coincide with the water surface. The gravity conduits include open channels, flumes aqueducts. However, in pressure conduits water flows under pressure above the atmospheric pressure. These conduits can therefore follow the natural ground surface and can be taken up and down hills as per the topography of the area. HGL in pressure conduit generally lies above the conduit. In pressure conduits when HGL falls below conduits then negative pressure will develop which should be avoided to prevent from the cavitation of pipe. A pressure conduits when taken along a hill may rise above the HGL it is called a siphon.

8.1 Pipe Materials

Pipes are circular conduits which can carry liquid in pressure are of various sizes and different materials. In case of drinking water conveyance and distribution circular pipes are mostly used. Pipes are of different materials as GI, DI, CI, WI, steel, cement concrete, HDPE, PPR, PVC etc. Selection of pipe materials depends upon fund availability, type of water to be conveyed and its life span and durability, corrosive properties, resistant to temperature stresses, ability to resist pressure, repair and maintenance cost etc.

8.1.1 Requirement of good pipe material

1. It should have adequate structural strength so that able to resist pressure induce internally and externally, compressive and tensile

stresses, temperature stress, load due to overlaying soil, impact shock.

2. It should be impervious, corrosion resistant, durable and long lasting.
3. It should be cost effectiveness and available in the market at reasonable cost.
4. As far as possible Darcy frictional coefficient should be low and Hazen William's constant high so that hydraulic efficiency maximizes.
5. Water may contains inorganic particles especially in transmission mains like sand, silt etc., which at high velocity of flow along invert of the pipe and the erosion of material may take place due to abrasion, hence it should possess enough resistance to abrasion.
6. It should be light in weight so that easy to transport, handle and laying.
7. It should be easy to join, flexibility of design, easy to repair and maintenance.
8. It should be environment friendly

8.1.2 Types of pipe materials

The pipes are usually classified according to the materials which are as follows;

1. Cast iron pipes
2. Galvanized iron pipe
3. Steel pipe
4. PVC pipe
5. PPR pipe
6. DI pipe
7. Concrete pipe

1. Cast iron pipe (CI)

These pipes are widely used for the conveyance of water in water supply scheme. These pipes are highly corrosion resistant and possess other desirable properties like durability, easy to make joint, long life, strong and can resist maximum pressure likely to develop. Cast iron pipe are expensive and heavy hence difficult to transport.

CI pipes are available in 2.5 to 5.5 m length with various diameters. According to thickness CI pipes are classified as LA, A and B class and can resist pressure of 10, 12.5 and 16 kg/cm^2 respectively.

Advantages of CI pipe

1. Cost of these pipes is moderate.
2. The pipes are easy to join.
3. The pipes are highly resistant to corrosion.
4. The pipes are strong and durable
5. Service connections can be easily made
6. Usual life is about 100 years

Disadvantage of CI pipes

1. CI pipes are heavier so that difficult in handling and transport.
2. The carrying capacity of these pipes decreases with the increase in life of pipes due to tuberculation.
3. These pipes generate metallic taste in the water due to the iron leaching into the water from the rusting of the pipe.
4. These pipes are brittle.
5. Since CI pipes are heavier use of larger size become uneconomical.

2. Galvanized iron pipe (GI pipe)

These are the mild steel or wrought iron pipes, provided with a protective coating of zinc on the both inner and outer surface.

These pipes are commonly used in house plumbing or after service connection. For water pipe fittings 12 mm to 25 mm diameter pipes are used and available in a length of 7 m. These pipes are cheap, light, easy to join, easy to transport and handling. The life span of these pipes is around 20 years. GI pipes may get corrode by acidic or alkaline or otherwise activated waters also liable to insulation. In water if chlorine is added as disinfectant, an increase in corrosion of iron materials can be expected.

Advantages of GI pipes

1. These pipes are cheap.
2. Light in weight and easy to handle.
3. The pipes are easy to join.
4. These pipes can be easily cut and threaded.

Disadvantages of GI pipes

1. The pipes are affected by acidic or alkaline waters.
2. The pipes are less durable.
3. Hydraulic efficiency decrease with time as decreases smoothness.

3. Steel pipe

These are fabricated by rolling the mild steel plates to proper diameter and can jointed by riveting or welding. These pipes are strong, cheaper, light in weight, can resist high pressure up to 400 m. The welded steel pipes are made up to 2.4 m in diameter and up to 12 m length. Theses pipe are costlier, liable to corrosion, and can't resist pressure due to external load during vacuum inside.

Advantages of Steel pipes

1. Numbers of joint are less because these are available in long lengths.
2. The pipes are cheap in first cost.

3. The pipes are durable and strong enough to resist high internal water pressure.
4. The pipes are flexible to some extent and they can therefore laid on curves.
5. Transportation is easy because of light weight.

Disadvantages of Steel pipes

1. Maintenance cost is high.
2. The pipes are likely to be rusted by acidic or alkaline water.
3. The pipes require more time for repairs during breakdown and hence not suitable for distribution pipes.
4. The pipes may deform in shape under combined action of external forces.

4. Polyvinyl chloride (PVC) pipe

PVC pipes are made of combination of plastic and vinyl. These plastic pipes are highly rigid, easy to join, strong in resisting pressure, light in weight hence transportable, can resist acids, alkalis, salts and organic chemicals, cheaper so commonly used in Nepal. This pipe requires support closer due to flexibility so that can break or crack if miss-handled. PVC pipes can resists temperature up to 60 °C. A minimal skills and tools are required to install PVC pipes. As they do not rust, rot, or wear over a time, these pipes are commonly used in water systems.

Advantages of PVC pipes

1. Pipes are cheap.
2. The pipes are durable.
3. The pipes are flexible.
4. The pipes are free from corrosion.
5. The pipes are good electric insulators.
6. The pipes are light in weight and it can easy to mould any shape.

Disadvantages of PVC pipes

1. The co-efficient of expansion for plastic is high.
2. It is difficult to obtain the plastic pipes of uniform composition.
3. The pipes are less resistance to heat.
4. Some types of plastic impart taste to the water.

5. Polypropylene random copolymer (PPR) pipe

Polypropylene random copolymers are thermoplastic resins produced through the polymerization of propylene, with ethylene links introduced in the polymer chain. Homopolymer, random copolymer, and block copolymer are the types of polypropylene. These pipes are highly resistant to temperature and impact load. These pipes can resist temperature up to 70 °C so it can be used for hot water supply. Nowadays PPR pipes and fittings are popular in plumbing and water supply plants due to easy in joint, perfect seal tight system, no calcification problem, durable and long life expectancy, non-deforming, no reaction with salts and acids, eco friendly, recyclable, good chemical resistance etc.

Advantages of PPR pipes

1. These pipes perform good hydraulic efficiency.
2. Light in weight and easy to handle.
3. The pipes possess high resistivity against heat.
4. These pipes are eco friendly and durable.
5. It performs good resistivity to chemicals.
6. Calcification problem does not arise in these pipes.
7. Life expectancy of these pipes is of more than 50 years.

Disadvantages of PPR pipes

1. Joining and repairing of PPR pipes is possible by use of a fusion –welding tool.

2. As PPR pipes are plastic product exposed to direct sunlight may drying out the oil content present in all plastics.

6. Ductile iron (DI) pipe

These pipes are made of ductile iron commonly used for potable water transmission and distribution. Typically, the pipe is manufactured using centrifugal casting in metal or resin lined molds. Protective internal linings and external coatings are done to ductile iron pipes to overcome corrosion problems. Hence these pipes are highly corrosion resistant and long life (100 yrs).

Advantages of DI pipes

1. Comparatively DI pipes possess greater ductility and impact resistance then CI pipes.
2. Lighter than CI pipes so that easy to handle and transport.
3. The pipes are easy to join and simple also can accommodate some angular deflection.
4. These pipes are of more strength than CI pipes.

Disadvantages of DI pipes

1. The pipes require internal and external lining or protection.
2. Corrosion may take place as equal in CI pipes.
3. The polyethylene wrappings may cause damaged.

7. Concrete pipe

Cement concrete pipe pipes may be either plain cement or reinforced cement concrete. PCC pipes can be used up to 15 m head where as RCC pipes can be used up to head of 60 m and for higher head pre-stressed concrete can be used. These pipes are also called on situ pipe so that they may be casted in site also. Pipes of reinforced pipe are also known as Hume pipes. These pipes are non-corrosive and longer life. The maintenance cost of cement concrete pipe is less and joints are very simple. However, concrete pipes are inconvenient as they are heavy, less resistance to withstand impact and shock.

Advantages of concrete pipes

1. There are pipes are most durable with usual life of about 75 years.
2. The pipes can cast at site work and thus there is reduction in transport charges.
3. Maintenance cost is less.
4. Inside surface of pipe can made smooth.
5. No danger of rusting.

Disadvantages of concrete pipes

1. Transportation is difficult.
2. Repair work is difficult.
3. Initial cost is high.
4. These pipes are affected by acids, alkalies and salty waters.

8.2 Pipe joints

For the ease in handling, transporting and placing in position of pipe, these are manufactured in suitable length hence pipe should be jointed. A joint may require at bent to change direction of pipe or to connect pipe for continuation of pipeline which device is called pipe joint. These pipe joint should be watertight, strong, durable, and economical.

8.2.1 Types of pipe joint

In water supply system a network of long pipelines is required for the conveyance and distribution of water. Pipe are not manufactured in a single length as they may be long as 200 m coils of HDPE and other pipes are commonly 2 to 6 m. Hence these pipes are required to join together of smaller length. Pipe joints are of various types which depend on pipe materials, resistivity of pressure, durability, water tightness, site conditions etc. Pipe joint commonly used in pipeline are as follows,

1. Socket and spigot (bell and spigot joint)
2. Flanged
3. Expansion
4. Collar
5. Screwed and socket

8.2.1.1 Socket and spigot joint

This joint as shown in figure 8-1 is commonly used for CI pipes as well as DI pipes. In this joint spigot of one end of the pipe is slipped in socket or bell end of other pipe and jute or hemp yarn is wrapped around spigot tightly up to 50 mm depth and a gasket or joint runner is clamped in place of round joint to fit tightly. By the help of chalking tool molten lead poured into the V shape opening left on the top by clamped joint runner. Space between hemp yarn and clamped runner is filled with molten lead. Now runner is removed after hardening of lead then tightened by chalking tool and hammer. Lead about 3.5 to 4 kg may required for up to the diameter 150 mm and for diameter 120 cm pipe 40 to 45 kg per joint. It takes high cost but makes joint perfect.

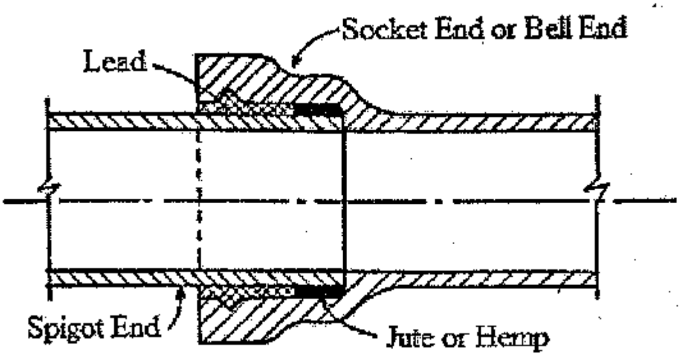


Figure 8-1 Socket and Spigot Joint

8.2.1.2 Flanged joint

This joint is commonly used for CI pipes, steel pipes and GI pipes. This joint as shown in figure 8-2 may be used for temporary works so that it can easily assemble and dissemble. The flanges of both pipes brought together and placing gasket in-between it is water tightened by screw or welding. This joint is suitable for pumping station filter plant, laboratories and boiler house but not used in place having vibration and deflection.

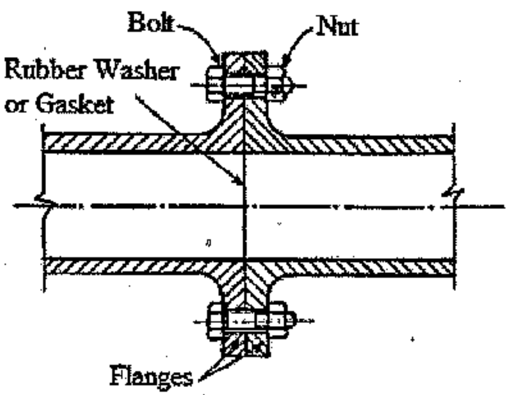


Figure 8-2 Flanged Joint

8.2.1.3 Expansion joint

This joint can bear temperature stress which results expansion and contraction. Function of this joint is to maintain water-tightness of the joint due to stress produced by temperature variations. Socket end is flanged with cast iron follower ring which can be freely slide on the spigot end and a rubber gasket is tightly pressed between angular space of spigot and socket by means of bolt as shown in figure 8-3. Water-tightness is maintained by rubber gasket during slight movement of socket end in forward and backward directions due to temperature stress.

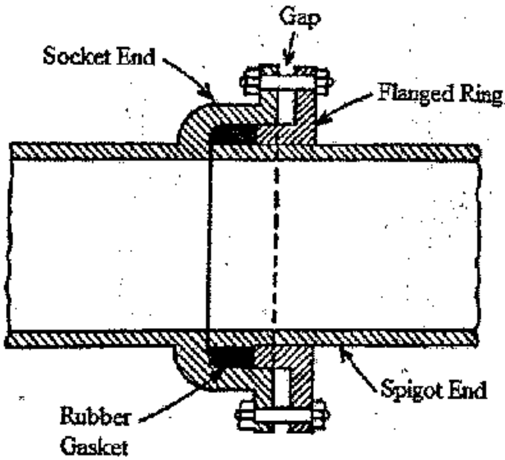


Figure 8-3 Expansion Joint

8.2.1.4 Collar joint

This joint is commonly used for cement concrete pipes; both reinforced and pre-stressed concrete pipes with plain ends. Tow ends of pipes are brought in same level with rubber gasket in-between, collar is placed with lap on both pipes. Cement mortar (1:1) is filled between the space of collar and pipe. (See figure 8-4)

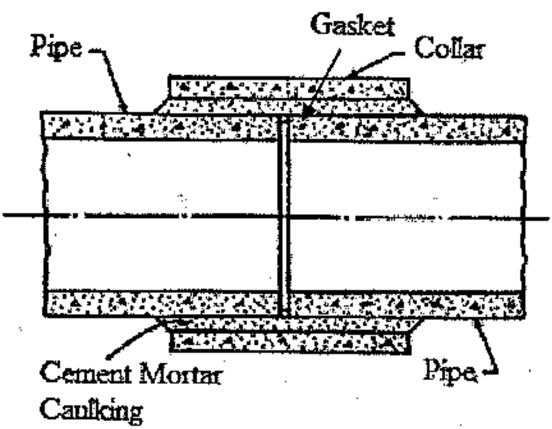


Figure 8-4 Collar Joint

8.2.1.5 Screwed and socket

Screwed and socket joint as shown in figure 8-5 is commonly used for GI pipes and also for small diameter steel pipes. Ends of the both pipes have screw threads on the outer surface in which socket are screwed to make the joint.

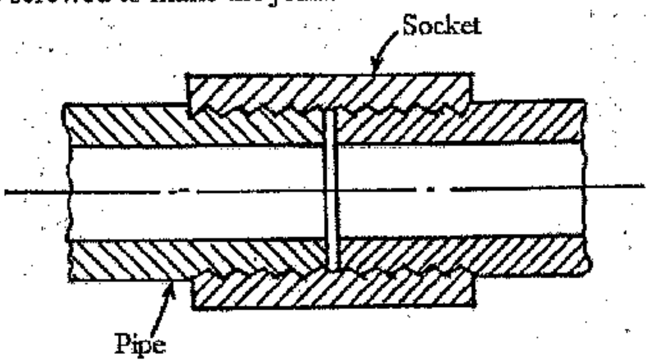


Figure 8-5 Screwed and Socket Joint

8.3 Laying of pipes

1. *Preparation of detailed maps showing alignment of pipelines*: Survey is conducted to prepare detailed map showing alignment of pipelines from source to treatment plant and distribution network and also map should show the position of roads, sewer lines, existing water pipes, telephone lines, electric lines etc.

2. *Locating the proposed alignment of the pipe line on the ground:* The alignment of pipeline is marked by pegging in 30 m interval on straight and 7.5 to 15 m apart in curves.

3. *Pipe laying with respect to ground level:* Pipe laying is of transmission main may be above ground level but support at a suitable interval may be required to prevent from settlement where as distribution network are laid below the ground surface.

4. *Excavation and preparation of trench:* Excavation may be done mechanically or manually as per specification but it is commonly

done manually in Nepal. Trench should excavated by adding 30 to 45 cm in external diameter of pipe. More than 90 cm clearance is provided above crown but 15 to 20 cm more excavation is done both directions (Horizontal and Vertical) at the place of joint. The bottom of the trench carefully prepared so that the pipe can be laid true to line and gradient and also there would be adequate protection against possible settlement.

5. *Dewatering of trench:* If GWT coincide at the trench level dewatering may be required which could be done by pumping or gravity method.

6. *Lowering of pipes into the trench:* Pipe should be transported carefully to site and lowered by stacked on one side (one side construction material and earth material in other side). Protective coating and pipe end should not be damaged.

7. *Joining pipe:* Proper types of joint should be selected and joined. Valves should be fitted at proper places along the pipeline.

8. *Testing of pipes:* Leakage test should be done at a suitable section of about 500 m length. Water is pumped twice the normal operation pressure from one end and next end is closed for 24 hrs. Allowable leakage may be known by following formula;

$$Q = ND\frac{\sqrt{P}}{3.3}$$

where, Q = allowable leakage cm^3/hr,

N = No of joints to be tested,

D = Diameter in mm

P = Average test pressure Kg/cm^2.

9. *Disinfection and back filling of trench*: Disinfection is done by adding chlorine 50 ppm for 12 hrs. Backfilling is done by parent material with suitable compaction method up to 15 cm above GL.

Problems

1. Why pipe joints are required? Describe socket and spigot joint with a neat sketch.
2. Describe briefly the process of pipe laying and jointing.
3. Mention different pipe materials and describe about GI pipe with its advantages and disadvantages.
4. Describe the factors that should be considered while selecting pipe materials for water supply schemes. Give comparative merits and demerits of CI and steel pipes.
5. Write short notes on:
   i) Expansion Joint
   ii) Spigot and Socket Joint
   iii) Flanged Joint
   iv) Concrete Pipe
   v) Pipe Joints

9 Valves and Fittings

For the efficiently functioning and to maintain the water supply scheme various devices may require controlling the flow, maintaining pressure, preventing from back flow, to release air from pipelines etc. These devices are called valves. Similarly, in the pipe network system various devices are used to connect pipe are fittings.

9.1 Valves

Valves are provided in the pipelines for various purposes. Main purpose of providing valve is to control and regulate the flow of water. The various types of valves commonly used are as follows;

1. Sluice or gate or cutoff valve
2. Reflux (check valve)
3. Safety or pressure relief valve
4. Air relief valve
5. Drain or scour or blow off valve

9.1.1 Sluice or gate or cutoff valve

These are the commonly used valve to regulate the flow of water through pipelines. These valves as shown in figure 9-1 are extensively used in the distribution system to shut off the supplies. It consists of a disc or circular gate parallel sided or wedge shaped in cross-section and having a nut which slot in with the thread of an operating spindle. The disc or circular gate, by raising or lowering flow can be regulate or control. These valves may be provided in every junction and in a suitable interval of about 150 to 300 m in straight portion. These valves are operated by rotating the spindle in clockwise to close and anticlockwise to open.

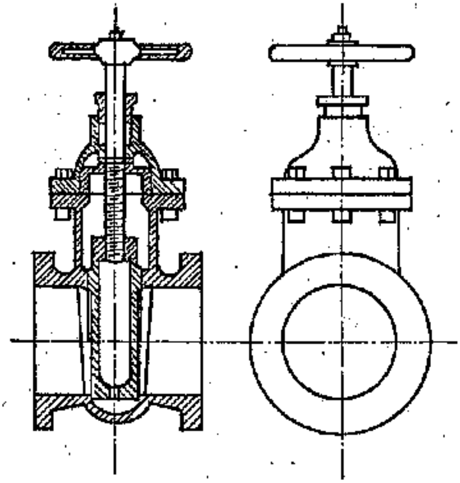


Figure 9-1 Sluice Valve

9.1.2 Reflux (check valve)

This non-return valve allows water to flow in one direction only and the flow in reverse direction automatically stopped. It consists of a disc hinged at its top edge provided at the one end in such a way that it opens when flow is forward and closes if water tends to flow in reverse direction. This valve is invariably placed in a pumping main. Figure 9-2 shows the check valve of horizontal type.

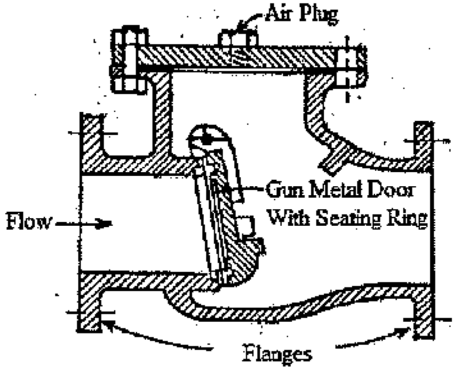


Figure 9-2 Reflux Valve

9.1.3 Safety or pressure relief valve

It consists of a disc controlled by spring which can be adjusted to desired pressure. It is provided to release the excessive pressure from the pipeline and protect the pipeline against possible danger of bursting due to excessive pressure. When the pressure in pipelines exceeds the desired pressure, the disc is forced to be lifted up and certain amount of water flows out from the cross pipe thereby releasing the pressure in the pipeline. This valve is also called automatic cutoff valve as shown in figure 9-3.

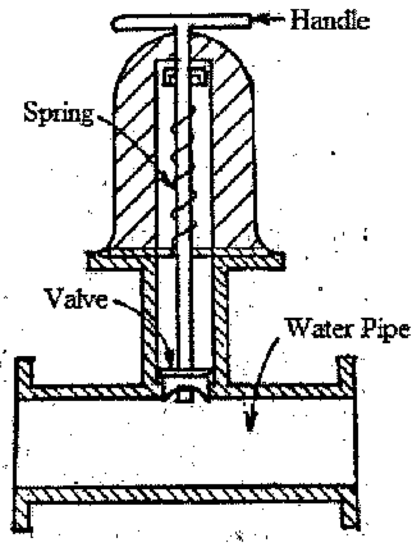


Figure 9-3 Pressure Relief Valve

9.1.4 Air relief or air valve

In a pipeline air may enter or entrained air get trapped which may be accumulated in summit or high points of pipeline and may be serious blockade to flow of water. It consists of a CI chamber, float, lever and poppet valve is held in closed position. The chamber is connected to the bolted on the pipe top opening in the crown. A float mass and a lever in it are adjusted and when water is under pressure it lowers and poppet valve is opened result release of pressure. (see figure 9-4)

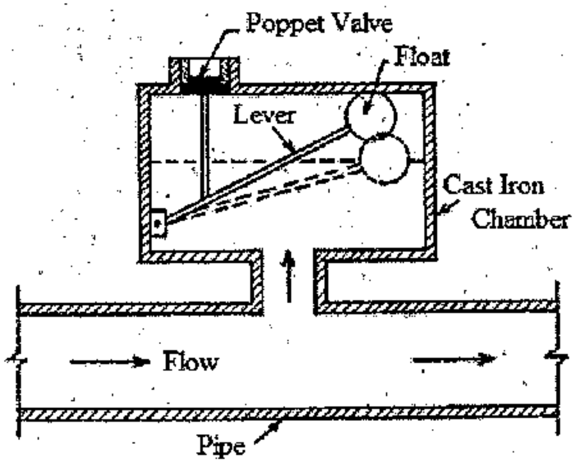


Figure 9-4 Air Valve

9.1.5 Drain or scour valve

These are similar to blow off valves also known as wash out valves. They are ordinary sluice valves operated by hand. They are located at the depressions and dead ends to remove the accumulated silt and sand. After the complete removal of silt; the value is to be closed.

9.2 Fittings

Pipe fittings are used in the pipelines for the various purposes as for connect pipe sections or make a joint, change direction of pipelines, metering, closing or sealing of pipe etc different fittings are used which are as follows;

1. **Fittings for pipe**

   Various fittings commonly used in pipe network are shown in figure 9-5 such as unions, caps, plugs, flanges, nipples, crosses, tees, elbows, bend etc.

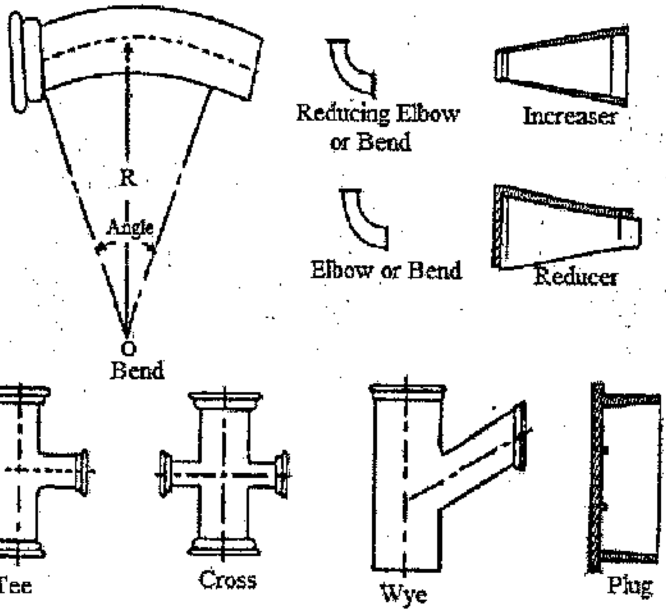


Figure 9-5 Pipe Fittings

2. **Stop cocks and water taps**

   Stop cock is a valve fitted at the end of communication pipe (*communication pipe is owned and managed by the water supply authority*) just outside the property boundary to isolate the house from the water supply at the period of maintenance of system inside the house. The stop clocks are particularly sluice or gate valve of small size. Temporary disconnections are made at the stopcock while permanent disconnections are made at ferrule. Figure 9-6 shows a stop Cock.

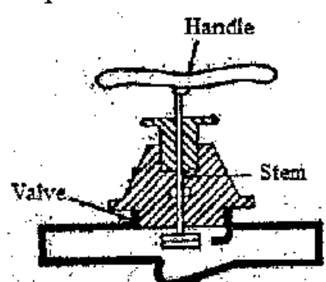


Figure 9-6 Stop Cock

Water taps are also known as bib cocks or faucets. These are attached at the end of water pipe line in wash basins, kichens, bathrooms etc. from which the consumers obtinined water. Water taps are operated by handle as shown in figure 9-7.

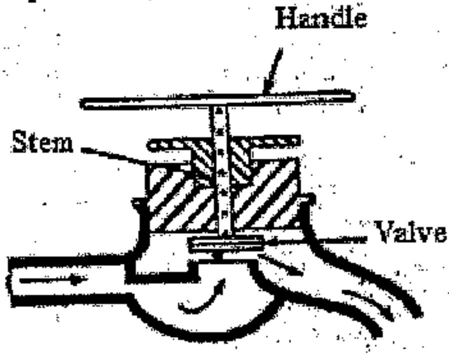


Figure 9-7 Water Tap

3. **Water meters**

These are the devices which are installed on the pipes to measure the quantity of water flowing at a particular point along the pipe. The readings obtained from the meters help in working out the quantity of water supplied and thus the consumers can be charged accordingly. The water meters are usually installed to supply water to consumers metering prevents the wastage of purified water. A rotary type water meter is shown in figure 9-8.

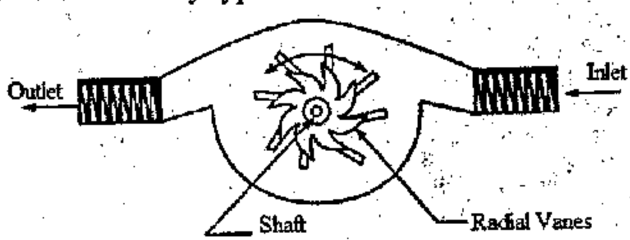


Figure 9-8 Rotary Water Meter

9.3 Break pressure tank (BPT)

BPT is a small tank or chamber provided in order to break the excessive hydrostatic pressure in the pipeline. In this tank the water freely discharges and hydrostatic pressure is reduced to zero thereby establishing a new static water level hence these tanks may also known as pressure releasing tank. A simple BPT has shown in figure 9-9 and these are generally provided in gravity flow water supply system to prevent the pipeline against possible danger of bursting due to excessive pressure. This tank may be circular or rectangular in shape and made of masonry, RCC or Ferro cement. It is closed chamber with provision of different valves as inlet, outlet, overflow, float (to prevent from overflow) etc. Reservoir may also act as BPT.

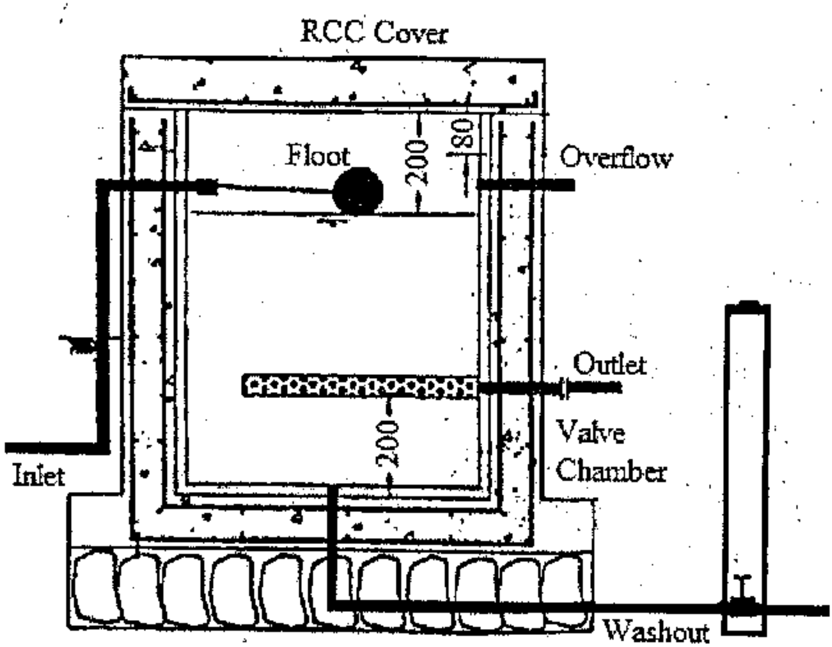


Figure 9-9 Break Pressure Tank

When level difference (head) exceeds then that of capacity to withstand pressure by used pipe in gravity flow system a BPT is provided. The strategic provision of BPT in pipelines can avoid or minimize the use of high pressure rating pipes and optimizes the system cost. HDPE pipes with pressure ratings of 4 kg/cm^2, 6 kg/cm^2 and 10 kg/cm^2 are widely used to convey the water in gravity flow water supply system and when pressure exceeds 10 kg/cm^2 GI pipes are used.

9.4 Public stand post

Public stand post as shown in figure 9-10 is most frequently used component of water supply schemes from where people collect water to meet their household demand in rural area.

Public stand post should be located at a suitable place as convenient for washing, laundering and filling water, aesthetically pleasant, clean and inviting and also generated wastewater should be easily drained off.

Generally walking distance for fetching water should not be more than 50 m (80 m in exceptional case) in vertical and 200 m (250 m in exceptional case) in horizontal. A stand post is designed to serve 8 to 10 households. A public stand post should serve maximum of 100 users.

The minimum discharge in a standpost should not be less than of 0.1 lps and maximum discharge is limited to 0.25 lps, for adequate provision it should be of 0.15 lps. When flow in a public stand post is 0.1 lps, 2.5 minute is required to fill a vessel of 15 liters. A tap stand will adequately serve a population of about 100 persons at an average per capita demand of 45 liters/day with peak factor of 3.

Minimum residual head to be maintained in our context of Nepal is of 5 m. Residual head in a public stand post is desired to be of 15 m. Absolute maximum residual head is 56 m.

Construction of a tap stand includes several components: a post supporting a 15 mm mild steel riser pipe from the pipeline up to a bibcock or faucet which should discharge at least 0.1 lps; a stand on which to place a bucket; a apron to collect spillage; and a gutter and drainage to a soak way in order to prevent the breeding of flies and mosquitoes and to keep the area clean. The height of the faucet should be 1300 mm in average above the apron. Normally faucet is not protruding more than 300 mm.

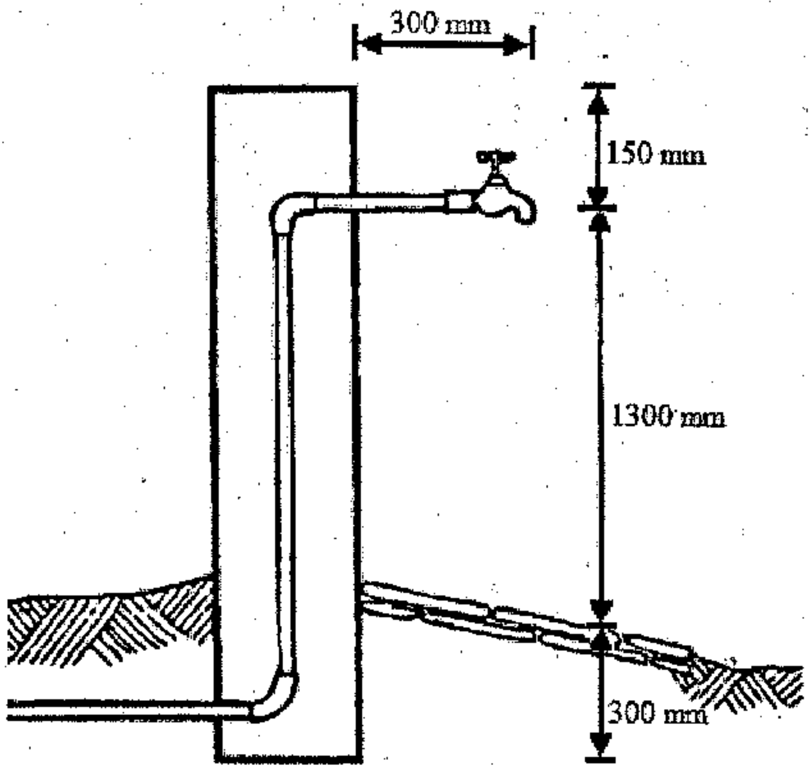


Figure 9-10 Public Tap Stand

9.5 Maintenance of water supply system

In order to get efficient service up to design year it is essential to have timely repair and maintenance. Afterwards design year of the scheme there may require rehabilitation or re-strengthening of the project or scheme. Proper protection may also require preventing from corrosion, failure, bursting, damaged of components.

General requirements of operation and maintenance is; to preparation of plan, to manage spare parts and tools, to trained technicians, maintain standard and to prevent from possible risk of damage of other components.

Methods of maintenance of water supply system

1. **Preventive maintenance**

   Especially in rainy season sediment accumulated in source, in pipelines should be removed on time so that possible risk could be minimized. Drainage facilities, plantation, protective walls may be done or construct to protect from further damage, such works are preventative maintenance.

2. **Regular maintenance**

   For the protection of components of water supply cleaning and replacing of devices is regularly done in a suitable interval of time is called regular maintenance.

3. **Emergency maintenance**

   Natural disasters like flood, landslide may damage components of water supply like reservoir, pipelines, valves, source etc. At that time immediate maintenance may be required for proper functioning of water supply project called emergency maintenance.

Problems

1. With neat sketches, describe purpose and construction of a break pressure tank.
2. Enlist the requirements of the public stand post along with its importance in rural area.
3. Why pressure relief valves are necessary? Describe with a neat sketch.
4. Describe the functions of the following with neat sketches.
   a) Pressure Release Valve
   b) Air Valve
   c) Check Valve
   d) Sluice Valve