ARBA MINCH WATER TECHNOLOGY INSTITUTE

FACULITY OF WATER SUPPLY AND ENVIROMENTAL ENGINEERING

WATER SUPPLY AND URBAN DRAINAGE (CEng - 3171)

Pre-requisites: **Engineering Hydrology**

Section – G3-Civil Engineering

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Course outline

Objective of course:

☛ *To familiarize the students with the design of water supply systems, demand projection, design of storm water drainage, and identification of water supply sources.*

Course Contents

1. Introduction

2. Methods of Forecasting Population

3. Water Demands

4.Sources of Water

4.1. Types of water source

4.2. Source Selection Criteria

5.Collection and Distribution of Water

5.1 Intakes

5.2 Service Reservoirs

5.3 Methods of Distribution

5.4 Layout of distribution System

5.5.Pipes Used in Water Distribution Systems

5.5.1. Pipe Materials

5.5.2. Determination of Pipe Sizes

5.5.3. Energy Losses in Pipes

5.5.4. Pipe Appurtenances

5.6. Design of the Distribution System

5.6.1. Analysis of Water distribution Systems

6. Urban drainage systems

- ✓ Introduction
- ✓ Types, Sources and Quantity of Waste Water
- ✓ Hydraulic Design of Sewers and Storm Water Drains
- ✓ Sewer materials
- ✓ Sewer Appurtenances
- ✓ Septic Tanks

7. **Water Supply Project Preparation**

1.Introduction

What is Water supply engineering?

- It's the branch of civil engineering deals with the development ,designing, planning and construction of elements of water supply system.
- A water supply system is a system for the collection, transmission, treatment, storage and distribution of water from source to consumers, for example, homes, commercial establishments, industry, irrigation facilities and public agencies for water related activities (fire-fighting, street flushing).

Type of Water Supply system

a. Continuous

- In this system, there is continuous water supply (for 24 hours). This is possible where adequate quantity of water is available.
- The major advantage of such system is that due to continuous water supply, water remains fresh and rusting of pipes will be low. However, losses of water will be more in case of any leakage.

b. Intermittent

- In such system, supply of water is either done in whole village/town for fixed hours or supply of water is divided into zones and each zone is supplied with water for fixed hours in a day or as per specified day.
- Such system is followed when there is low water availability, however, in certain cases, wastage of water is more due to tendency of community for storing higher amount of water than required.
- In such system, pipelines are likely to rust faster due to wetting and drying. However, maintenance can be easily done during no-supply hours.

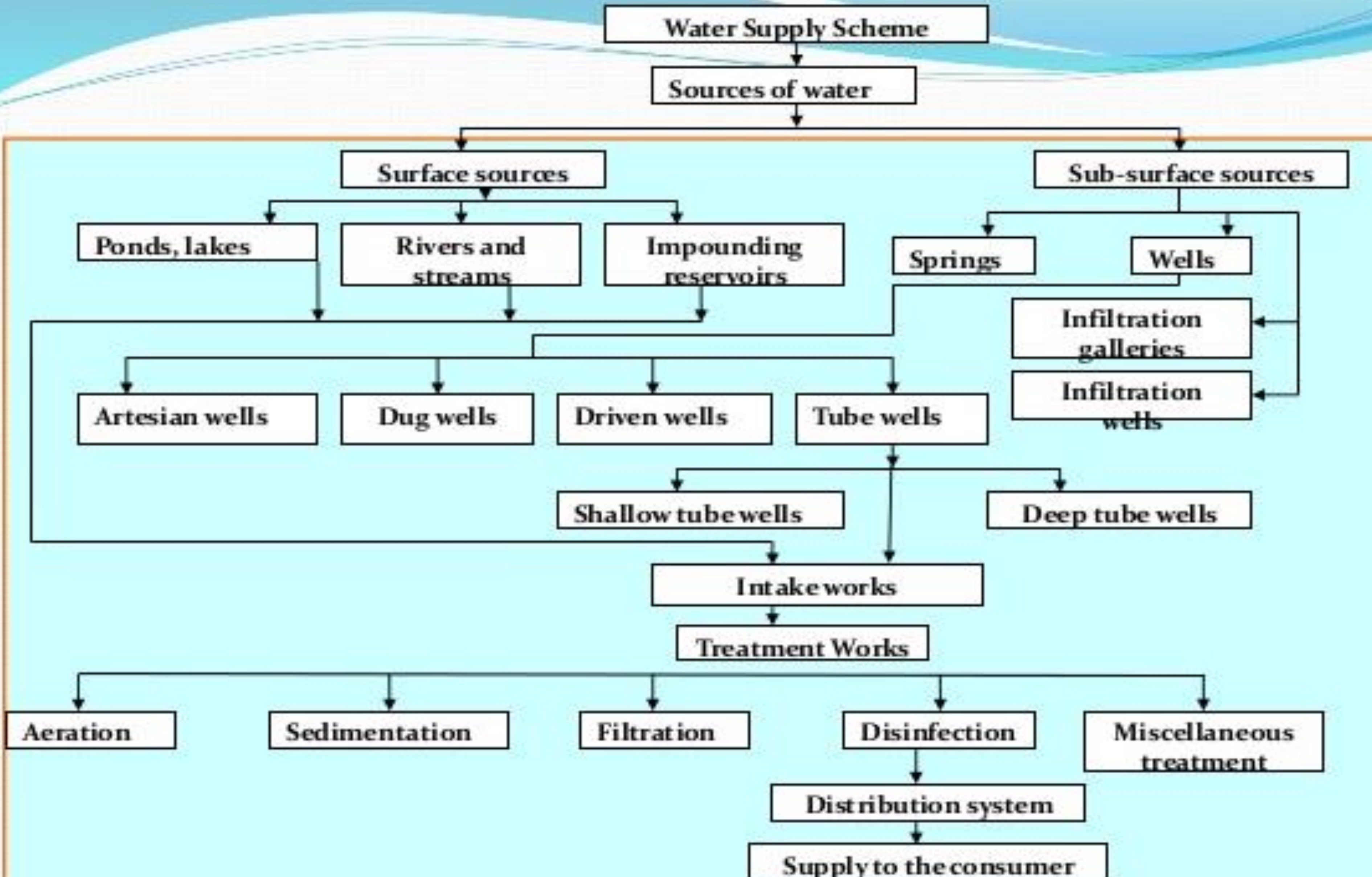


Fig 1 Schematic diagram of typical water supply scheme

Protected water supply means the supply of water that is treated to remove the impurities and made safe to public health. Water may be polluted by physical and bacterial agents.

Components of a protected water supply scheme

☛ Source of water

☛ Intake structure

☛ Treatment plant

☛ Distribution System

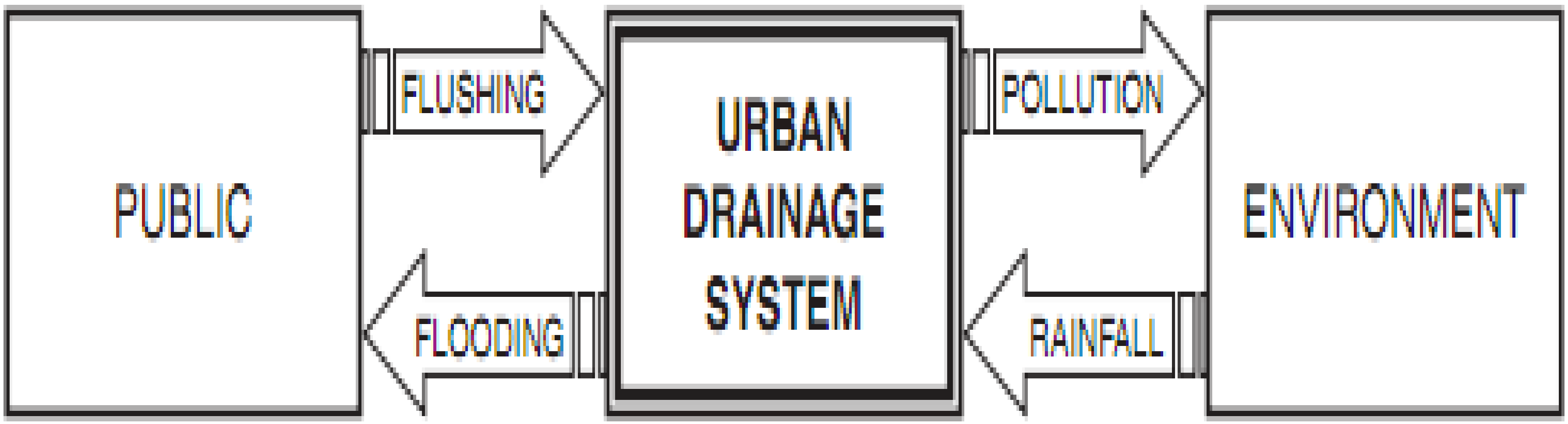
The objective of protected water supply scheme are

- ☛ To supply save and whole some water to consumers
- ☛ To supply water in sufficient quantities
- ☛ To supply water at convenient points and timings
- ☛ To supply water at reasonable cost to the users
- ☛ To encourage personal and house hold cleanliness of users

What is urban drainage?

- Drainage is the process of water or waste liquids flowing away from somewhere into the ground or down pipes. In the case of urban drainage, "somewhere" is the urban environment, mainly constituted of roads, houses and green spaces.
- Urban drainage may be used to describe the process of collecting and transporting wastewater, rainwater/storm water or a combination of both.
- Drainage systems are needed in developed urban areas because of the interaction between human activity and the natural water cycle.
- This inter-action has two main forms: the abstraction of water from the natural cycle to provide a water supply for human life, and the covering of land with impermeable surfaces that divert rainwater away from the local natural system of drainage. These two types of interaction give rise to two types of water that require drainage.

- The first type, wastewater, is water that has been supplied to support life, maintain a standard of living and satisfy the needs of industry.
- The second type of water requiring drainage, storm water, is rainwater (or water resulting from any form of precipitation) that has fallen on a built-up area.
- Urban drainage systems handle these two types of water with the aim of minimizing the problems caused to human life and the environment



2.Population Forecasting

2.Population forecasting

- ☛Design Period
- ☛Population forecasting
- ☛ Methods of population forecasting
- ☛Factors affecting population growth

Design Period

<u>Design Period: -</u> is the no of years for which the designs of the water works have been done. It should neither be too short or too long, mostly water works are designed for design period of 22 – 30 years.

Factor, which should be kept in view while fixing the design period:

- ☛ Fund
- ☛ The life of the material used in project (pipes, structural materials)
- ☛ Anticipated expansion of the town
- ☛ The rate of interest on the loan taken

Population forecasting

☛After the design period is fixed the next step is to determine the population in various periods,

Factors affecting changes in population are:

☛increase due to births

☛decrease due to deaths

☛increase/ decrease due to migration

☛increase due to annexation.

Methods of population forecasting

1. Arithmetical increase method
2. Geometrical increase method
3. Incremental increase method
4. Decreasing rate method
5. Logistic Curve method
6. Simple graphical method
7. Master plan method
8. CSA method

i. Arithmetical increase method

- This method is based on the assumption that the population is increasing at a constant rate. The rate of change of population with time is constant. The population after 'n' decades can be determined by the formula.

$$P_n = P_o + n*C$$

where

$P_n \longrightarrow$ population at indicates in the future

$P_o \longrightarrow$ population at present

$n \longrightarrow$ No. of decades

$C \longrightarrow$ Constant determined by the average of increase of 'n' decades

- **This method is generally applicable to a large and old city.**

Example-1 Predict the population for the year 2021,2031,and 2041 from the following population data using arithmetical increase method.

Year	1961	1971	1981	1991	2001	2011
Population	858548	1015672	1201553	1691538	2077820	2585862

Solution

$$P_n = P_o + n*C$$

n=1,2,3

Po=2,585,862

C=?

P2021=?   P2031=?   P2041=?

Year	Population	Increase in population	
1961	858545		
1971	1015672	C1=P@1971-P@1961	157127
1981	1201553	C2=P@1981-P@1971	185881
1991	1691538	C3=P@1991-P@1981	489985
2001	2077820	C4=P@2001-P@1991	386282
2011	2585862	C5=P@2011-P@2001	508042
Average Increase		**(C)=(C1+C2+C3+C4+C5)/5**	**345463**
		P=Po + n*C, Po=2585862,C=345,463	
2021	n=1	P2021=2585862+1*345,463	**2931325**
2031	n=2	P2031=2585862+2*345,463	**3276789**
2041	n=3	P2041=2585862+3*345,463	**3622252**

ii. Geometrical increase method.

- This method is based on the assumption the percentage increase in population from decade to decade remains constant. If the present population is p and average percentage growth is IG, the population at the end of n decade will be:

$$Pn=Po*(1+IG)^n$$

where

Pn ⟶ population at indicates in the future

P o⟶ population at present

IG ⟶Geometric mean(%)

n ⟶ No. of decades

- **This method is mostly applicable for growing towns and cities having vast scope of expansion.**

Example 2: Forecast the population of example 1 by using geometrical increase method.

Solution

$Pn=Po*(1+IG)^n$

$n=1,2,3$

$Po=2,585,862$

$IG=?$

$P2021=?$

$P2031=?$

$P2041=?$

Year	Population	Increase in population		Percentage increase in popn	
1961	858545				
1971	1015672	C1=	157127	IG1=(C1/P@1961)*100	**18.30**
1981	1201553	C2=	185881	IG2=(C2/P@1971)*100	**18.30**
1991	1691538	C3=	489985	IG3=(C3/P@1981)*100	**40.78**
2001	2077820	C4=	386282	IG4=(C4/P@1991)*100	**22.84**
2011	2585862	C5=	508042	IG5=(C5/P@2001)*100	**24.45**
	C		**345463**	**IG**	**25%**

Pn=Po*(1+IG/100)^n ,Po=2585862, IG=25%			
2021	n=1	P2021=2585862*(1+25/100)^1	**3232328**
2031	n=2	P2031=2585862*(1+25/100)^2	**4040409**
2041	n=3	P2041=2585862*(1+25/100)^3	**5050512**

iii. INCREMENTAL INCREASE METHOD

- This method is modification of arithmetical increase method and it is **suitable for an average size town under normal condition** where the growth rate is found to be in increasing order.
- While adopting this method the increase in increment is considered for calculating future population.
- The incremental increase is determined for each decade from the past population and the average value is added to the present population along with the average rate of increase.
- Hence, population after nth decade is

$$Pn = P + n*C + \{n*(n+1)/2\}*Y$$

Where, Pn = Population after nth decade C = Average increase Y = Incremental increase

Example 3: Forecast the population of example 1 by using **Incremental Increase Method.**

Solution

$$P_n = P_o + n*C + \{n*(n+1)/2\}*Y$$

n=1,2,3

Po=2,585,862

Y=?

P2021=?

P2031=?

P2041=?

Year	Population	Increase in population		Increamental Increase Population	
1961	858545				
1971	1015672	C1=P@1971-P@1961	157127		
1981	1201553	C2=P@1981-P@1971	185881	Y1 =C2-C1	**28754**
1991	1691538	C3=P@1991-P@1981	489985	Y2 =C3-C2	**304104**
2001	2077820	C4=P@2001-P@1991	386282	Y3 =C4-C3	**-103703**
2011	2585862	C5=P@2011-P@2001	508042	Y4 =C5-C4	**121760**
Average Increase		**(C)=(C1+C2+C3+C4+C5)/5**	**345463**	Y=(Y1+Y2+Y3+Y4)/4	**87729**
P=Po+C*n+(n*(n+1)/2)*Y) ,Po=258862,C=345463,Y=87729					
2021	n=1	P2021=2585862+1*C+(1*(1+1)/2)*87729)		**3019054**	
2031	n=2	P2031=2585862+2*C+(2*(2+1)/2)*87729)		**3539975**	
2041	n=3	P2041=2585862+3*C+(3*(3+1)/2)*87729)		**4148625**	

iv Decreasing rate method

- In this method, the average decrease in the percentage increase is worked out and is then subtracted from the latest percentage increase for each successive decades
- This method is applicable to **average size cities growing under normal condition.**

Pn=Po*(1+NG/100)^n

NG=Net Percentage increase in population

Example 4: Forecast the population of example 1 by using Decreasing rate method.

Solution

Pn=Po*(1+NG/100)^n

n=1,2,3

Po=2,585,862

NG=?

P2021=?

P2031=?

P2041=?

Year	Population	Increase in population		Percentage increase in popn		*Decreasing in the Percentage Increase*		*Net Percentage increase in population*	
1961	858545								
1971	1015672	C1=	157127	IG1=(C1/P@1961)*100	**18.30**				
1981	1201553	C2=	185881	IG2=(C2/P@1971)*100	**18.30**	IG1-IG2	**0.00**		
1991	1691538	C3=	489985	IG3=(C3/P@1981)*100	**40.78**	IG2-IG3	**-22.48**	NG@2021=24.45(IG5)-(-1.54)	**26**
2001	2077820	C4=	386282	IG4=(C4/P@1991)*100	**22.84**	IG3-IG4	**17.94**	NG@2031=26-(-1.54)	**27.53**
2011	2585862	C5=	508042	IG5=(C5/P@2001)*100	**24.45**	IG4-IG5	**-1.61**	NG@2041=27.5-(-1.54)	**29.06**
C			**345463**	IG	25%	*Average %*	**-1.54**		
				Pn=Po*(1+NG/100)^n					
2021	NG=26	Po=2585862	**3258186**						
2031	NG=27.53	Po=3258186	**4155165**						
2041	NG=29.06	Po=4155165	**5295567**						

V GRAPHICAL METHOD

- ☛In this method the census populations of cities already developed under similar conditions are plotted.
- ☛ The curve of past population of the city under consideration is plotted on the same graph.
- ☛The curve is extended carefully by comparing with the population curve of some similar cities having the similar condition of growth.
- ☛The advantage of this method is that the future population can be predicted from the present population even in the absent of some of the past census report.
- ☛The use of this method is explained by a suitable example given below.

Example-5 : Let the population of a new city X be given for decades 1970, 1980, 1990 and 2000 were 32,000; 38,000; 43,000 and 50,000, respectively. The cities A, B, C and D were developed in similar conditions as that of city X. It is required to estimate the population of the city X in the years 2010 and 2020. The population of cities A, B, C and D of different decades were given below:

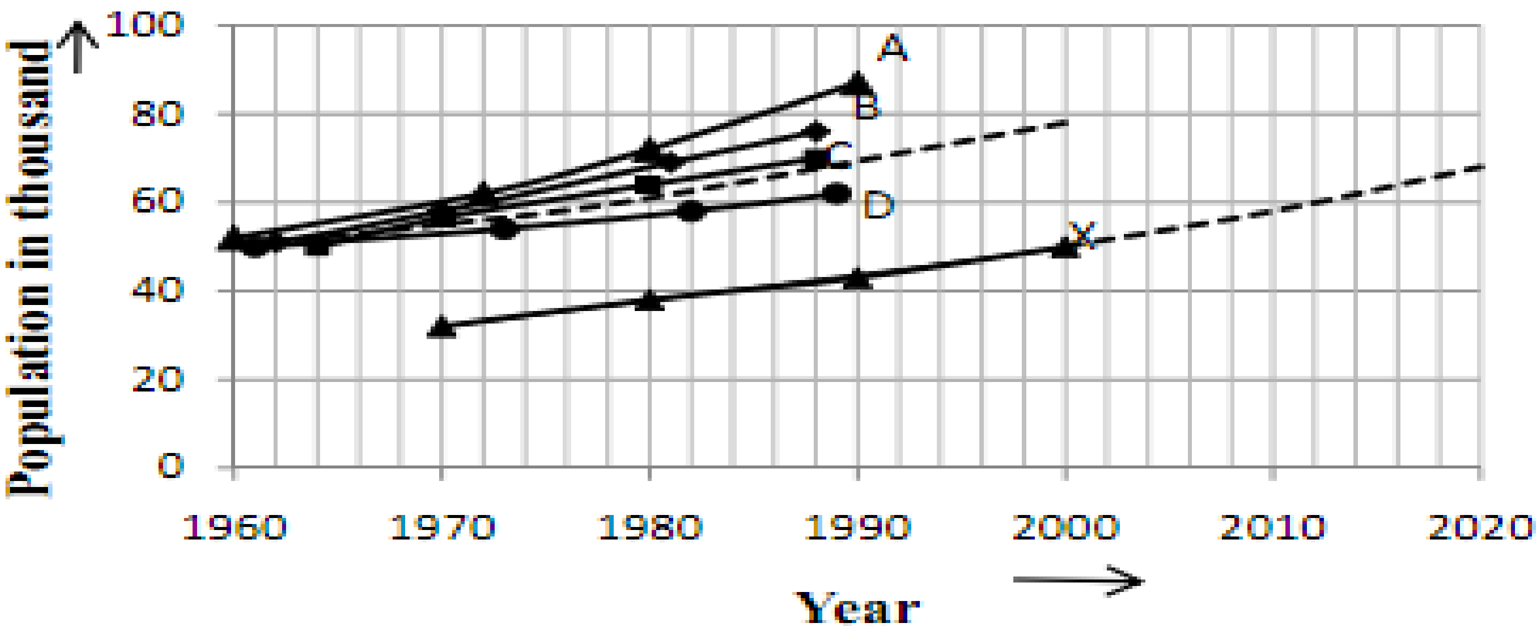
(i) City A was 50,000; 62,000; 72,000 and 87,000 in 1960, 1972, 1980 and 1990, respectively.

(ii) City B was 50,000; 58,000; 69,000 and 76,000 in 1962, 1970, 1981 and 1988, respectively.

(iii) City C was 50,000; 56,500; 64,000 and 70,000 in 1964, 1970, 1980 and 1988, respectively.

(iv) City D was 50,000; 54,000; 58,000 and 62,000 in 1961, 1973, 1982 and 1989, respectively.

- Population curves for the cities A, B, C, D and X were plotted. Then an average mean curve is also plotted by dotted line as shown in the figure.
- The population curve X is extended beyond 50,000 matching with the dotted mean curve. From the curve the populations obtained for city X are 58,000 and 68,000 in year 2010 and 2020.



Population curve

vi Logistic curve method

- This method is used when the growth rate of population due to births, deaths and migrations takes place under normal situation and it is not subjected to any extraordinary changes like epidemic, war, earth quake or any natural disaster etc.
- it is assumed that the plot of population versus time, under normal condition gives S shaped curve. A logistic projection can be based on the equation:

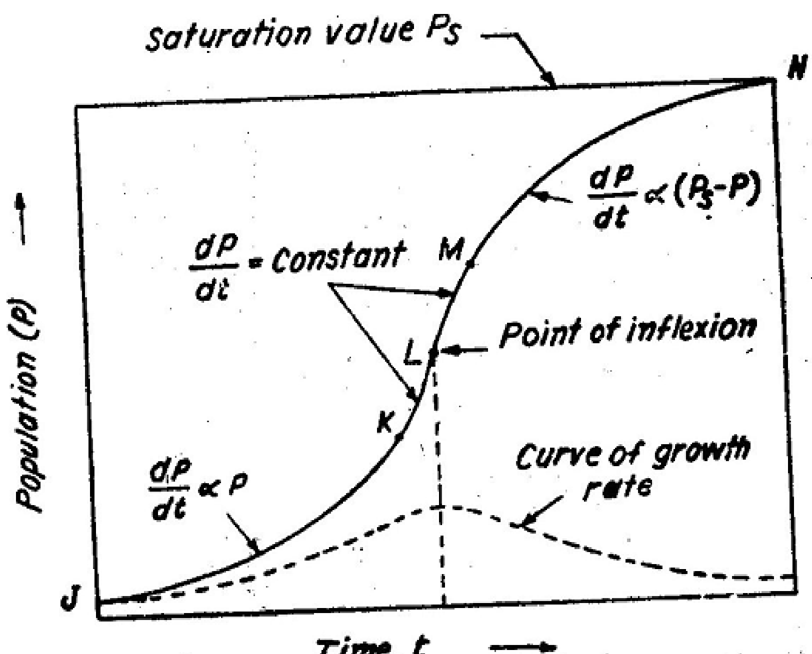
$$P = \frac{P_s}{1 + m \log e^{-1}(nt)}$$

Where Ps = Saturation population

Po = Population at starting point

P = Population at any time t from the starting point

$$m = \frac{p_s - p_o}{p_o}$$



- Taking three points from the range of census population data at equal time intervals (t_1, P_1), (t_2, P_2) and (t_3, P_3)

Where $t_2 = t_1 + \Delta t$, $t_3 = t_2 + \Delta t$

$$p_s = \frac{2p_o p_1 p_2 - {p_1}^2 (p_0 + p_2)}{p_0 p_2 - {p_1}^2}$$

$$n = \frac{2.3}{t} log10 \left(\frac{p_o (p_s - p_1}{p_1 (p_s - p_o} \right)$$

Example-6 The population of a city in three consecutive years i.e. 1991, 2001 and 2011 is 80,000; 250,000 and 480,000, respectively. Determine

(a) The saturation population,

(b) The equation of logistic curve,

(c) The expected population in 2021and 2031.

Solution

It is given that

$P0 = 80{,}000$	$to = 0$
$P1 = 250{,}000$	$t1 = 10$ years
$P2 = 480{,}000$	$t2 = 20$ years

The saturation population can be calculated by using equation

$$P_s = \frac{2P_0P_1P_2 - P_1^2(P_0 + P_2)}{P_0P_2 - P_1^2}$$

$$= \frac{2 \times 80{,}000 \times 2{,}50{,}000 \times 4{,}80{,}000 - 2{,}50{,}000 \times 2{,}50{,}000 \times (80{,}000 + 4{,}80{,}000)}{80{,}000 \times 4{,}80{,}000 - 2{,}50{,}000 \times 2{,}50{,}000}$$

$$= 655{,}602$$

We have $m = \frac{Ps - P0}{P0} = \frac{655{,}602 - 80{,}000}{80{,}000} = 7.195$

$$n = \frac{2.3}{t_1} \log_{10} \frac{P_0(P_s - P_1)}{P_1(P_s - P_0)}$$

$$= \frac{2.3}{10} \log_{10} \left[\frac{80{,}000(655{,}602 - 2{,}50{,}000)}{250{,}000(655{,}602 - 80{,}000)}\right]$$

Population in 2021

$$P = \frac{P_s}{1 + m\log_e^{-1}(n.t)}$$

$$= \frac{6{,}55{,}602}{1 + 7.195 \times \log_e^{-1}(-0.1488 \times 30)}$$

$$= \frac{6{,}55{,}602}{1 + 7.195 \times 0.0117} = 605{,}436$$

vii Method used by Ethiopians statistic Authority

$$P_n = P_o * e^{kn}$$

Where, P_n = population at n decades or years

P_o = initial population

n = decade or year

k = growth rate in percentage

Factors affecting population growth

- **Economic factors** : - New industries, discovery of oil or minerals.
- **Development programmes**: - of national importance such as river valley project.
- **Social facilities**: -Educational, medical and recreational facilities.
- **Communication links**: - Connectivity of the town with other cities and mandies (Markets) of agro products

- **Tourism**: - Tourists facilities, religious places and historical buildings.
- **Community life**: - Living habits, social customs and general education in the community.
- **Unforeseen factors**: - Earthquakes, Floods, Epidemics, Frequent famines etc.

Assignment-1

1) Explain different methods of population forecasting.
2) The population of a town as per the senses records are given below.

   Find out the population in the year 2021,2026,2031 and 2035 by

   (a) arithmetical (b) geometric (c) incremental increase methods

   (d) Decreasing rate method

Year	1971	1981	1991	2001	2011
Population	84000	115000	160000	205000	250000

3) In three consecutive decades the population of a town is 40,000; 100,000 and 130,000 Determine:

   (a) Saturation population;

   (b) Equation for logistic curve;

   (c)Expected population in next decade.

3.Water Demands

- ☛Introduction
- ☛Various Types Of Water Demands
- ☛Per Capita Demand
- ☛Factors Affecting Water Demand
- ☛ Variations In Demand

Introduction

- The estimated water supply coverage for Ethiopia is 85% for rural and 75% for urban and the country's water supply coverage 83% according to WASH MDG progress report.
- Access to water-supply services is defined as the availability of at least 20 liters per person per day from an "improved" source within 1 kilometer of the user's dwelling.
- In the design of any water work projects it is necessary to estimate the amount of water that is required. This is called water demand
- This water demand estimation involves:
  - ✓ The per capita water consumption
  - ✓ The determination of people who will be served
  - ✓ Analysis of the factors that may operate to affect consumption
- per capita water demand in liters per capita per day (l/c/d), is the amount of water (l, m3,..) consumed by one person in one day.
- Total consumption of a town will be:

  = **per capita water demand * population * some factors**

VARIOUS TYPES OF WATER DEMANDS

- While designing the water supply scheme for a town or city, it is necessary to determine the total quantity of a water required for various purposes by the city. There are different types of water demand. The demand for various purposes is divided under the following categories:
  - Domestic
  - Commercial & institutional
  - Public
  - Industrial
  - Fire fighting
  - Loss and waste

- **Domestic Demand:-** is the water required for;
  - ✓Drinking, Cooking, Bathing (shower),Hand and face washing, Household sanitation purpose, Washing clothes, floors, etc. Private gardening, Domestic animals, Private Vehicles washing.
- **Commercial and institutional:-** is the water required for ;
  - ✓School, hospital, health center, government office service religious institutions and other public facilities is classified as institutional water demand.
  - ✓Whereas the water reached for restaurant, shopping centers, hotels, local drinks and other commercial purpose is classified as commercial water demand.
  - ✓This consumption includes water used for commercial buildings & commercial centers including stores, hotels, shopping centers, cinema houses, restaurants, bars, airports &bus stations etc.
- **Public Use : -** is the Quantity of water required for ;
  - ✓public utility purposes such as for washing and sprinkling on roads, cleaning of sewers, watering of public parks, gardens, public fountains etc. comes under public demand.
  - ✓To meet the water demand for public use, provision of 5% of the total consumption is made designing the water works for a city.

- **Industrial demand: -** is the water required by factories.
  - ✓The amount of water is depend upon the type of industry in the city. Paper mills, textile mills, breweries, sugar mills, steel mills, leather mills, fertilizer mills, etc.
- **Fire fighting:-** this is water required for fighting fire breakout.
- **Losses and Waste/Un-accounted for water is due to:-**

All the water, which goes in the distribution, pipes does not reach the consumers. The following are the reasons

- ✓Losses due to defective pipe joints, cracked and broken pipes, faulty valves and fittings.
- ✓Losses due to, consumers keep open their taps of public taps even when they are not using the water and allow the continuous wastage of water
- ✓ Losses due to unauthorized and illegal connections
- ✓While estimating the total quantity of water of a town; allowance of 15%-20% of total quantity of water is made to compensate for losses, thefts and wastage of water

Per Capita Demand

- In community, water is used for various purposes as described above.
- For the purpose of estimation of total requirements of water, the demand is calculated on an average basic, which is expressed as so many litres/capita/day.
- If Q is the total quantity of water required by a town per year in litres, and the population of the town is P. the per capita demand will be.

$$\text{Per capital demand} = \frac{Q}{P*360} \text{ Litres/day}$$

- The per capita demand of the town depends on various factors and will be according to the living standard of the public and the number and type of the commercial places in the town etc.

Factors Affecting Water Demand

- The following are the main factors affecting for capita demand of the city or town.

a) **Climatic conditions :** The quantity of water required in hotter and dry places is more than cold countries because of the use of air coolers, air conditioners, sprinkling of water in lawns, gardens, courtyards, washing of rooms, more washing of clothes and bathing etc.

b) **Size of community :** Water demand is more with increase of size of town because more water is required in street washing, running of sewers, maintenance of parks and gardens.

c) **Living standard of the people :** The per capita demand of the town increases with the standard of living of the people because of the use of air conditioners, room coolers, maintenance of lawns, use of flush, latrines and automatic home appliances etc.

d) **Industrial and commercial activities :** As the quantity of water required in certain industries is much more than domestic demand, their presence in the town will enormously increase per capita demand of the town. As a matter of the fact the water required by the industries has no direct link with the population of the town.

e) **Pressure in the distribution system:** The rate of water consumption increase in the pressure of the building and even with the required pressure at the farthest point, the consumption of water will automatically increase. This increase in the quantity is firstly due to use of water freely by the people as compared when they get it scarcely and more water loss due to leakage, wastage and thefts etc.

f) **System of sanitation:** Per capita demand of the towns having water carriage system will be more than the town where this system is not being used.

g) **Cost of water:** The cost of water directly affects its demand. If the cost of water is more, less quantity of water will be used by the people as compared when the cost is low.

VARIATIONS IN DEMAND

- The water demand varies from season to season, day to day, even hour to hour.
  - ✓ **Seasonal variation:** high in hot season and less in cold season.
  - ✓ **Daily variation:** high during holidays, Saturday, Sundays.
  - ✓ **Hourly variation:** high in morning, launch time, evening.

Water demand factors

Average Water Demand

- ✓ The average day demand is taken to be the sum of the demands of domestic, non domestic and Non Revenue Water (NRW)).
- ✓ The average water demand represents the daily demand of the town averaged over the year

Maximum Day

- ✓ is the highest demand of any one 24-hour period over any specified year.
- ✓ The daily water demand changes with the season and days of the week.
- ✓ The ratio of the **maximum daily consumption** to the **mean daily consumption** is called the maximum day factor and usually varies between 1.0 and 1.3.
- ✓ The maximum day demand is used for calculations such as source pumping requirements.

Peak Hour

- ✓ It is the highest demand of any one-hour over the peak day.
- ✓ Such an event is likely to happen during morning hours when most people use water for bathing, washing utensils and cooking. Moreover, it could also occur at the end of the day when people need water for the same purpose after working hours.
- ✓ The peak hour water demand is greatly influenced by the size of the town, mode of service used and social activity pattern.
- ✓ The ratio of the **peak hour demand to the maximum day** demand is called peak hour factor

Population	Maximum Day Factor	Peak Hour Factor	Remarks
0 to 20,000	1.30	2.00	Small Towns
20,001 to 50,000	1.25	1.9	Moderate Towns
50,001 to 100,000	1.20	1.8	Large Towns
100,001 and above	1.20	1.6	Large Towns

Source: Design Criteria by MoWR

Estimation of Total Water Demand For A Town Or City

Adjustment for climate

- In order to account for changes in climate which affect the water demand, the values of the average per capita domestic water demand established above should be factored for climatic changes using the Ministry of Water Resources' climate factors guidelines shown in the table below.

Group	Mean annual Precipitation (mm)	Factor
A	600 or less	1.10
B	601 – 900	1.05
C	901 or more	1.00

Adjustment due to socio-economic conditions

- To accommodate the changes due to the potential for growth, the following recommended socio-economic factors are used.
- These factors are applied for all the domestic demand values above.

Group	Description	Factor
A	Towns enjoying high living standards and with very high potential development	1.10
B	Towns having a very high potential for development but lower living standards at present	1.05
C	Towns under normal Ethiopian conditions	1.00
D	Advanced rural towns	0.95

☛Theses figures are taken from the cost effective design report of the Ministry of Water Resources.

☛These factors are applied on the Average Day Demand to get the Maximum Day Demand and Peak Hourly Demand.

Population	Maximum Day Factor	Peak Hour Factor	Remarks
0 to 20,000	1.30	2.00	Small Towns
20,001 to 50,000	1.25	1.9	Moderate Towns
50,001 to 100,000	1.20	1.8	Large Towns
100,001 and above	1.20	1.6	Large Towns

Source: Design Criteria by MoWR

Descriptions	2010	2012	2015	2017	2020	2022	2027
Population	**42,201**	**46,203**	**52,929**	**57,706**	**65,690**	**71,275**	**87,404**
Connection Profiles (%)							
HC	8.00%	10.00%	12.00%	13.50%	15.00%	16.00%	18.00%
YCO	40%	45%	50%	55.00%	60%	65%	70%
YCS	22%	19.00%	18.00%	16.50%	15.00%	10.00%	5.00%
PT	20.00%	18.00%	17%	13.50%	10.00%	9%	7.00%
Un-Served	10%	8.00%	3%	1.50%	0%	0%	0%
Population Served							
HC	3,376	4,620	6,351	7,790	9,854	11,404	15,733
YCO	16,880	20,791	26,465	31,738	39,414	46,329	61,183
YCS	9,284	8,779	9,527	9,521	9,854	7,128	4,370
PT	8,440	8,317	8,998	7,790	6,569	6,415	6,118
Unserved	4,220	3,696	1,588	866	0	0	0
Demand by Connection Profile in l/c/d							
HC	35	40	50	55	60	65	7
YCO	20	22	25	26	27	28	3
YCS	20	20	23	24	25	25	2
PT	18	20	22	23	24	25	2
Demand in m^3/day under each connection profile							
HC	118.16	184.81	317.57	428.47	591.21	741.26	1,101.3
YCO	337.61	457.41	661.61	825.20	1,064.18	1,297.21	1,835.4
YCS	185.68	175.57	219.13	228.52	246.34	178.19	118.00
PT	151.92	166.33	197.95	179.18	157.66	160.37	165.19
Total Domestic Demand in m^3/day			**1,396.2**	**1,661.3**	**2,059.3**	**2,377.0**	

Exercise

1) What are the domestic needs of water?
2) Mention the reasons for losses and wastage of water.
3) Name the different types of seasonal variations.
4) Explain the various types of water demands.
5) Discuss the various factors affecting the per capita demand.

4. SOURCE WATER SUPPLY

TYPES OF WATER SOURCES

Sources of water supply schemes can conveniently be classified as:

1) **Rain:** harvesting rain water from different catchment

2) **Surface water:** Rivers, Lakes, Pond, Sea water, Impounding reservoirs, Wastewater reclamation

3) **Underground sources**

Springs

- **Depression springs**
- **Contact springs**
- **Artesian springs**

Wells

- **Shallow wells**
- **Deep wells**
- **Infiltration galleries**
- **Infiltration wells**

Surface water

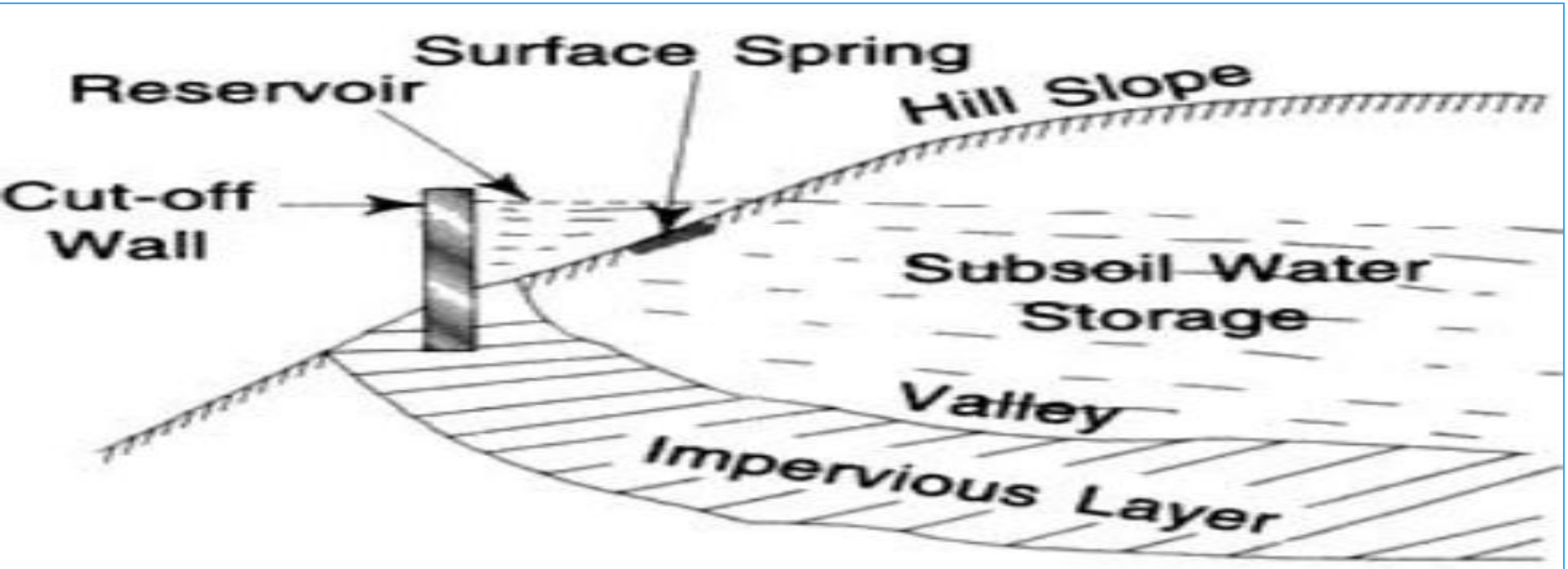
- ✓ **River or steam :** It is formed by the runoff in the mountain & hill areas. Some rivers are perennial (water available through out the year) and some are non perennial (water available in raining season only).
- ✓ Perennial River should always be selected for the scheme. Incase of non perennial rivers, the weir or low dam may be constructed to form a storage reservoir. The streams are suitable for small water supply scheme.
- ✓ **Pond or Lake :** It is natural or artificial depressions where surface runoff is collected in rainy season.
- ✓ **Impounding reservoirs:** Are artificial lakes formed by the construction of dams across a valley. This sources is always preferred for large water supply projects.
- ✓ **Wastewater reclamation:** Sewage or other waste water may be used as source of water for cooling, flushing water closets (WCS), watering lawns, parks, etc. for fire fighting and for certain industrial purposes after giving the necessary treatment to suit the nature of the use.

Underground sources

1. **Springs:** When the under ground water reappears at the ground surface by under ground pressure.

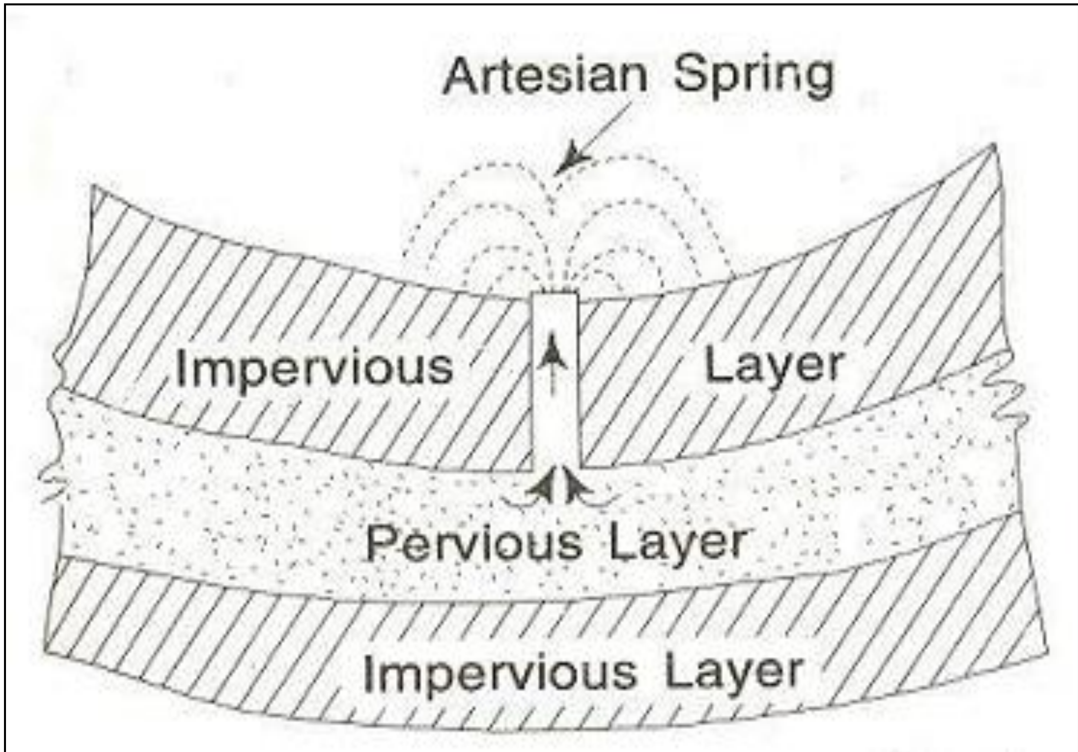
   i. **Depression spring:** is a spring formed when the ground surface intersects with the water table.

   ii. **Contact spring:** When due to an obstruction ground water is collected in the form of a reservoir and forces the water to overflow at the surfaces



iii. **Artesian spring:** is a spring that results from the release of water under pressure from confined water bearing formation either through a fault or fissure reaching the ground surface.

- It is also known as fracture spring



2. Wells:

- Are artificial holes or pits vertically excavated for bringing ground water to the surface.

i. **Shallow wells:** may be large diameter hand dug wells (diameter 1-4m) and depth <20m. Or machine drilled wells of small diameter (diameter 8-60cm) and depth <60m.

ii. **Deep wells**: are most large, deep, high-capacity wells constructed by drilling rig.

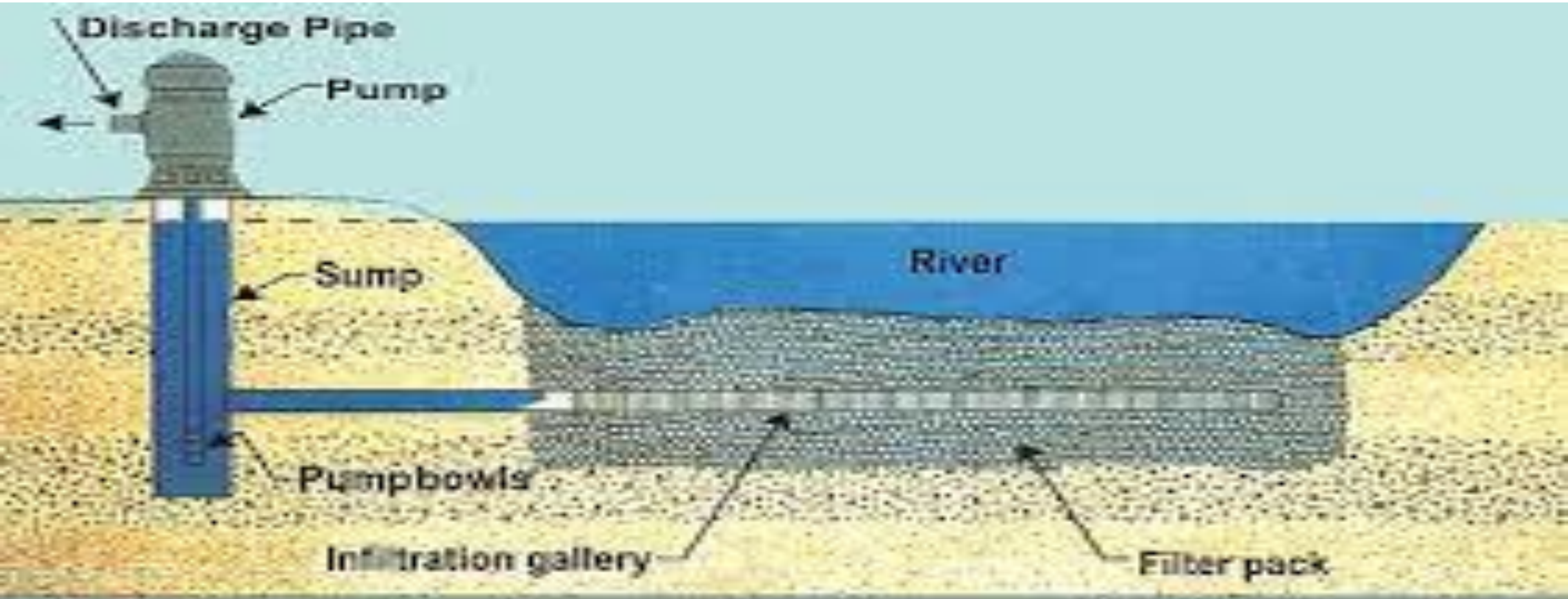
- Construction can be accomplished by cable tool method or rotary method.
- Drilling rigs are capable of drilling wells 8 to 60cm in diameter and depth < 600m.

iii. **Infiltration Gallery**: is a horizontal or nearly horizontal tunnel which is constructed through water bearing strata.

- It is sometimes referred as horizontal well.


MASSENZA

iv. Infiltration wells: are shallow wells constructed under the beds of rivers. They are suitable where there are deposits of sand and porous material at least 3m deep in river bed.



Cross section of pump placed in sump of infiltration gallery.

Source selection criteria's

- The choice of water supply source depends on the following factories:

A. Location: The sources of water should be as near as to the town as possible.

B. Quantity of water: the source of water should have sufficient quantity of water to meet up all the water demand through out the design period.

C. Quality of water: The quality of water should be good which can be easily and cheaply treated.

D. Cost: The cost of the units of the water supply schemes should be minimum.

Source selection criteria's:-

A. Surface water sources

- ✓ Safe water yield during the drought years
- ✓ Urbanization and land development in the watershed
- ✓ Proposed impoundments on tributaries
- ✓ Water quality
- ✓ Assessment of reliability
- ✓ Requirements for construction of water supply system components
- ✓ Economics of the project
- ✓ Environmental impacts of the project
- ✓ Water rights

B. Ground water sources

- ✓ Aquifer characteristics (depth, geology)
- ✓ Safe aquifer yield
- ✓ Permissible drawdown
- ✓ Water quality
- ✓ Source of contamination(gasoline, oil, chemicals)
- ✓ Saltwater intrusion(areas near to seas or oceans)
- ✓ Type and extent of recharge area
- ✓ Rate of recharge

5. COLLECTION AND DISTRIBUTION OF WATER

5.1. Intakes

5.2. Service Reservoirs

5.3. Methods of Distribution

5.4. Layout of distribution system

5.5. Pipes Used in Water Distribution Systems

- ✓ Pipe Materials
- ✓ Determination of Pipe Sizes
- ✓ Energy Losses in Pipes
- ✓ Pipe Appurtenances

5.6. Design of the Distribution System

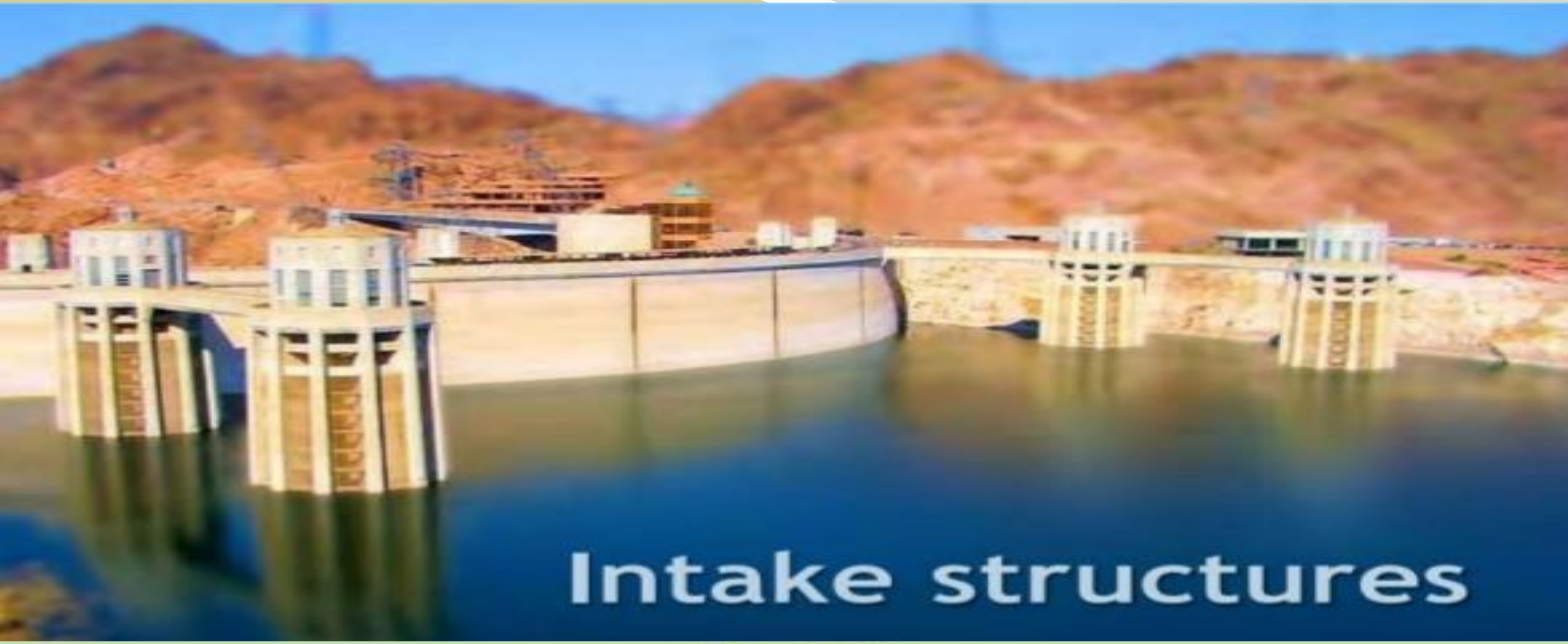
- ✓ Analysis of Water Distribution Systems

Introduction

- **Intake** is devices or structures in a surface water source to draw water from this source and then discharge in to an intake conduit through which it will flow in to the water works system.
- **An intake consists of:**

1) The opening, strainer, or grating through which the water enters, and
2) The conduit conveying the water, usually by gravity, to a well or sump.

- Intake structures are used for collecting water from the surface sources such as river, lake, and reservoir and conveying it further to the water treatment plant.
- The basic function of the intake structure is to help in safely withdrawing water from the source over predetermined pool levels and then to discharge this water into the withdrawal conduit (normally called intake conduit), through which it flows up to water treatment plant



Intake

The following must be considered in designing and locating intakes:

a. **The source of supply,** whether impounding reservoirs, lakes, or rivers (including the possibility of wide fluctuation in water level).

b. **The character of the intake surroundings.**
   - ✓ Depth of water
   - ✓ Character of bottom
   - ✓ Navigation requirement
   - ✓ The effect of currents, floods and storms up on the structure and in scouring the bottom.

c. **The location with respect to sources of pollution; and**

d. **The prevalence of floating materials such as ice, and vegetation**

Types of Intakes

- There are different types of intakes, such as reservoir intakes, river intakes, canal intake, and Lake Intake

Reservoir intake

- It is desirable to locate about a meter from the surface to get good quality water

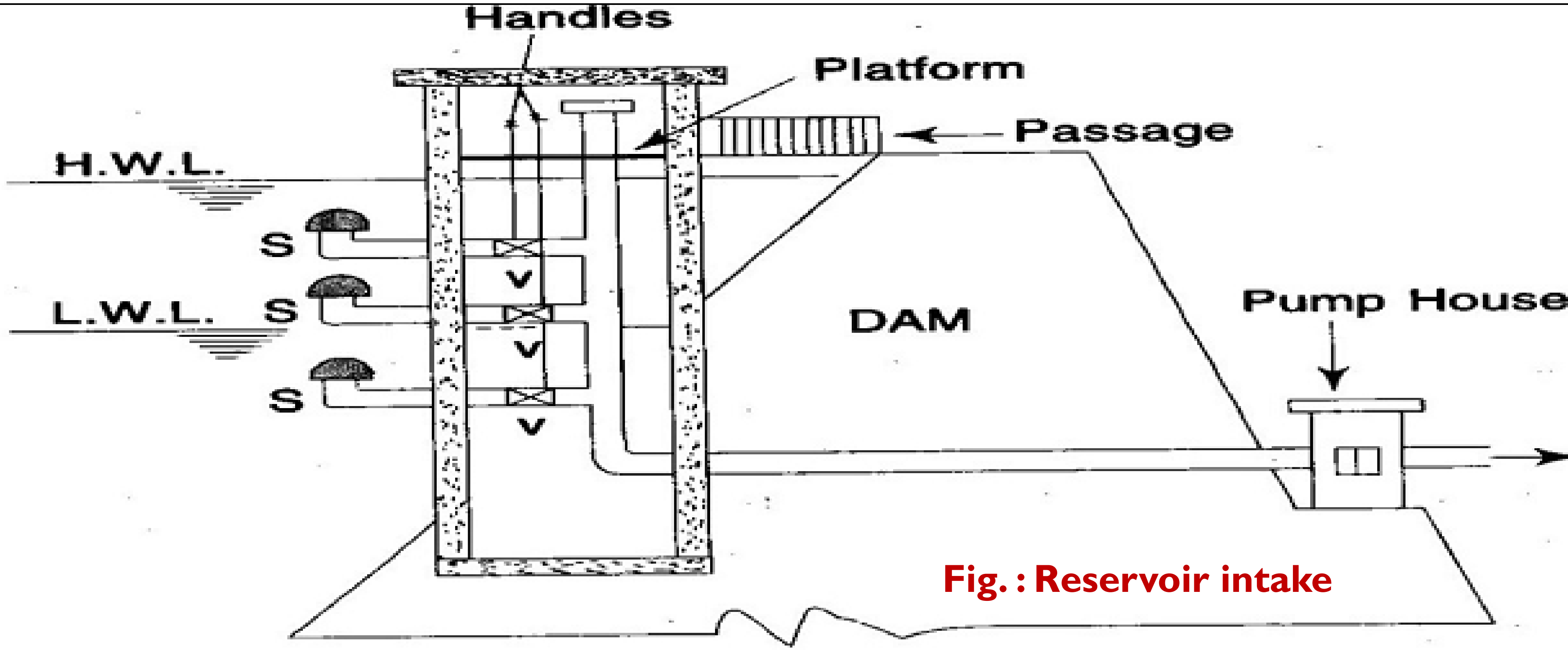


Fig. : Reservoir intake

River Intakes

- River intake is located inside the river so as to get adequate supply in all seasons.
- Water is drawn from the upstream side of the river, where it is comparatively of better quality.
- River intakes are especially likely to need screens to exclude large floating matter which might injure pumps.

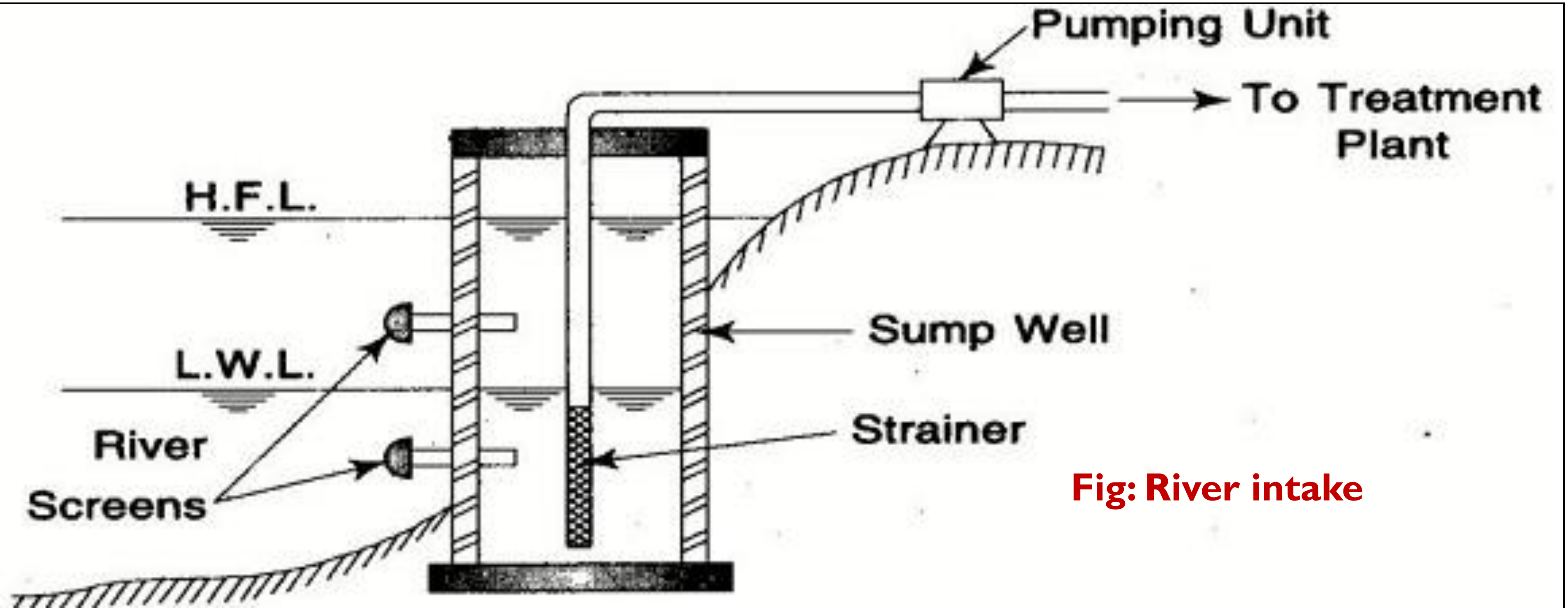


Fig: River intake

Lake Intake

- A submersible rectangular chamber is constructed at the bed of the lake below the low water level, so as to draw water in dry season also.
- The top cover of the chamber consists of several holes having gratings on it to prevent the entry of debris, aquatic life, weeds etc. to the chamber.
- A bell mouthed pipe is provided in the chamber which contain screen at the top.

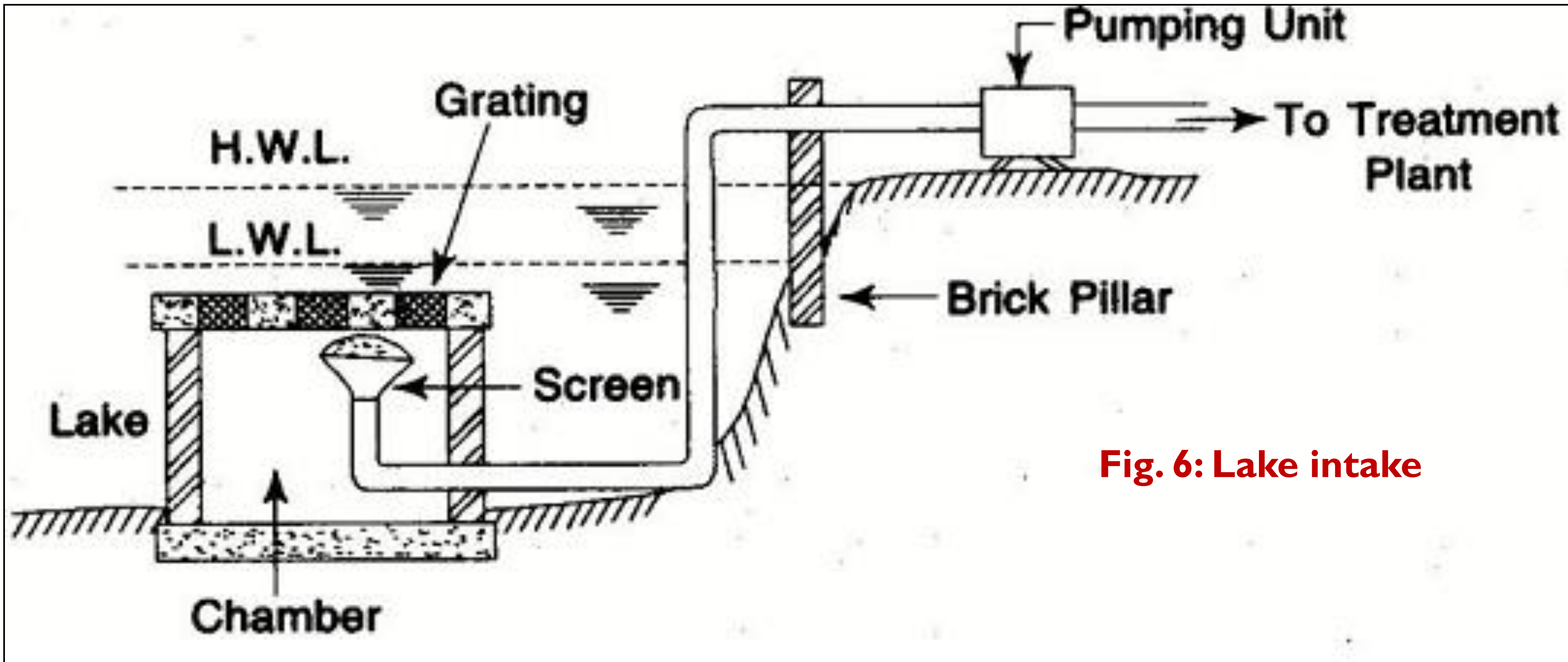


Fig. 6: Lake intake

Canal intake

- It is a very simple structure constructed on the bank of the canal.
- The well may be circular or rectangular and it is constructed with masonry work.
- It has an opening on its side provided with screen.

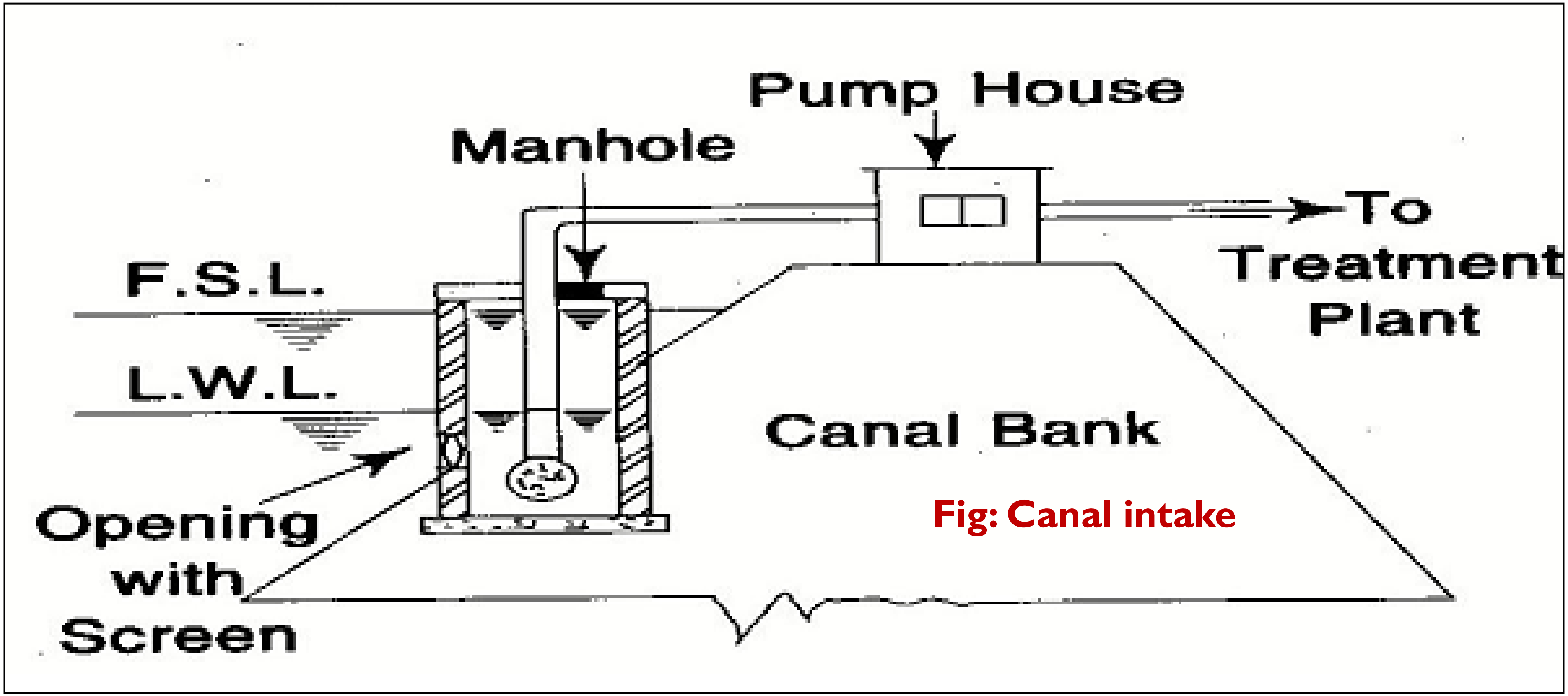


Fig: Canal intake

Exercise

1) What are points should be kept in mind while selecting a site for intake structure?
2) Explain any one of intake structure with neat sketch

5.2 Distribution Reservoirs

- Distribution reservoirs,also called service reservoirs, are the storage reservoir, which store the treated water for supplying water during emergencies(such as during fires, repairs, etc)and also to help in absorbing the hourly fluctuations in the normal water demand.

Functions:

- To balance the fluctuating demand from the distribution system.
- Provide a supply during a failure or shutdown of treatment plant, pumps or trunk main
- To give a suitable pressure for the distribution system and reduce pressure fluctuations there in.
- To provide a reserve of water to meet fire and other emergency demands.

Position and Elevation of Reservoirs

- The elevation at which it is desirable to position a service reservoir depends up on both the distance of the reservoir from the distribution area and the elevation of the highest building to be supplied.
- If the distribution area varies widely in elevation it may be necessary to use two more service reservoirs at different levels, so that the lower area do not receive an unduly high pressure.
- Pressure control valves are some times installed in inlet mains from service reservoirs in order to reduce the pressure to low laying zones, or to limit increase of pressure at night to reduce leakage.
- In making a decision to install pressure control devices it should be borne in mind that if the device fails to operate, which it will do if the equipment is not properly maintained, then the downstream mains will be subjected to a sudden increase of pressure and may burst.

Types of Service Reservoirs

Types: Classification of based on:

- position of the tank:

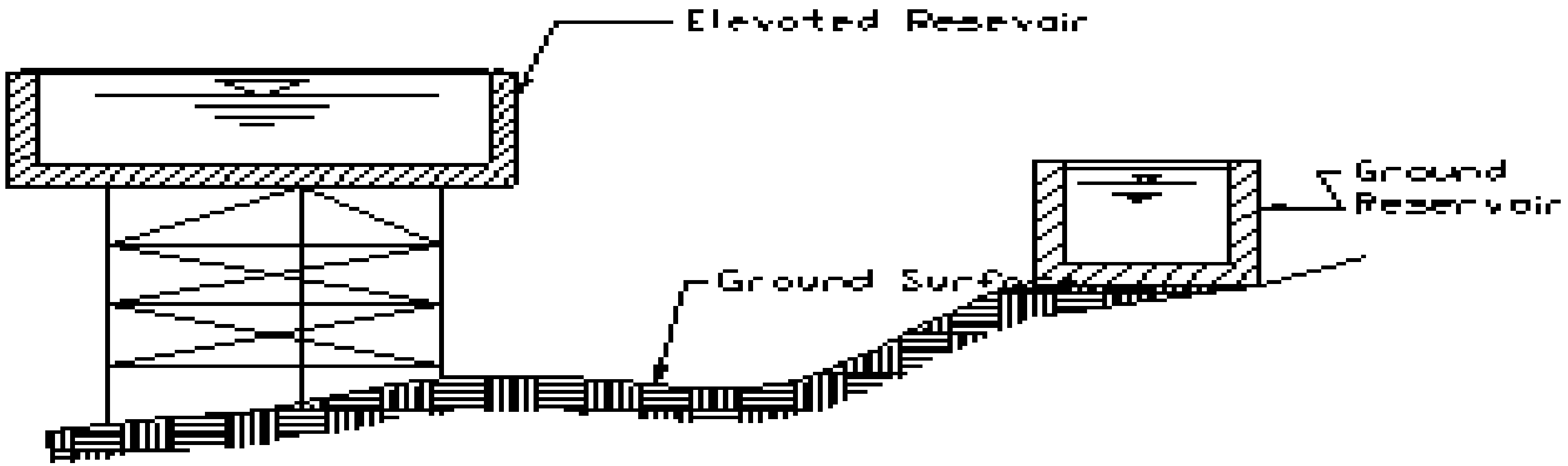
  Surface storage, Elevated storage, Underground tanks

- materials of construction

  RCC, Masonry, Masonry-concrete (sandwich), Plastic, Steel tanks

- shape of the tank:

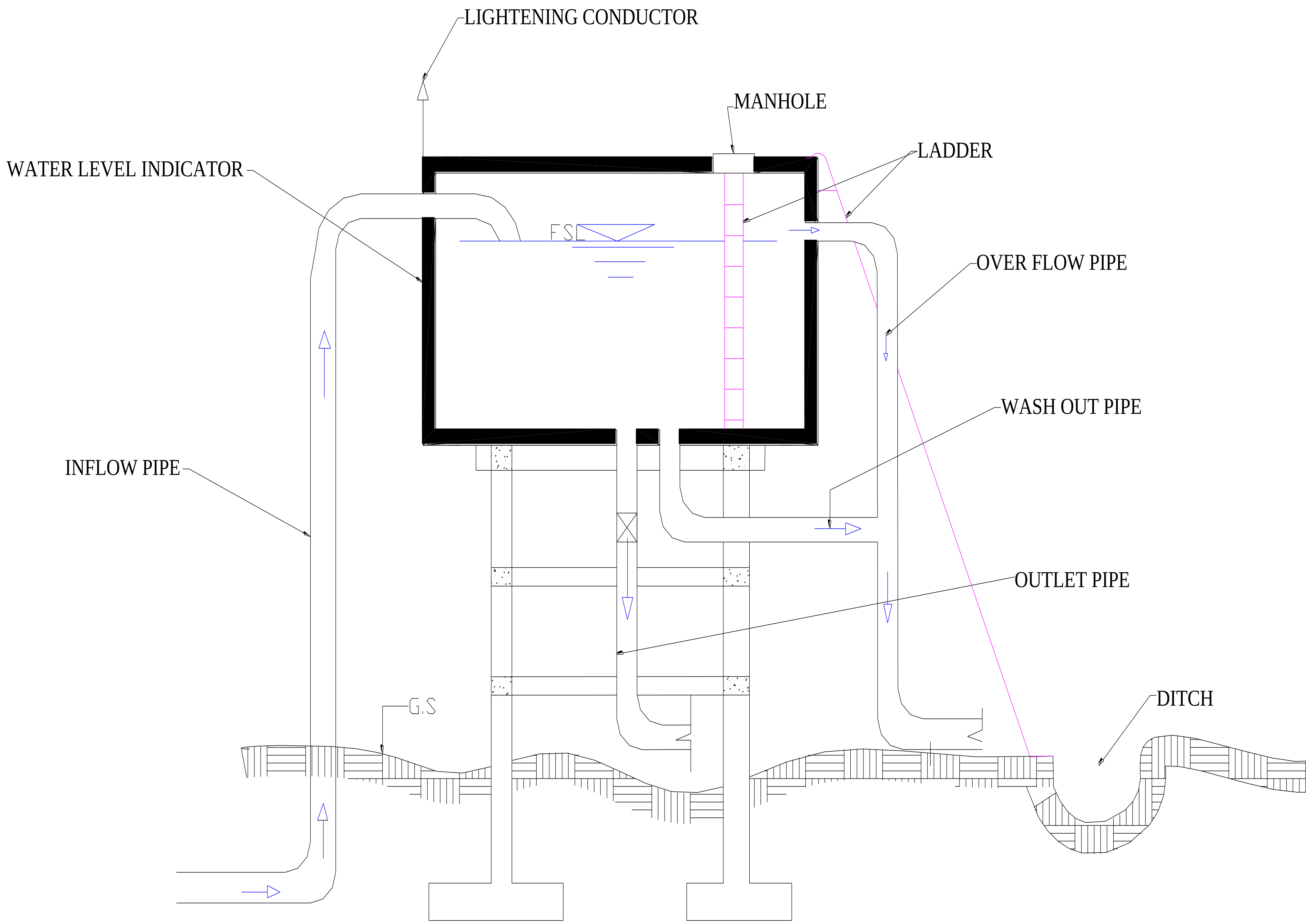
  Circular, Rectangular tanks



Distribution reservoirs

Position and Accessories of Service Reservoirs

- ☛ **Inlet Pipe:** for entry of water
- ☛ **Ladder:** to reach the top of the reservoir and then to the bottom, for inspection and cleaning.
- ☛ **Man Holes:** for providing entry to the inside of the reservoir, for inspection and cleaning.
- ☛ **Outlet Pipe:** for the exit of water
- ☛ **Out Flow Pipe:** for the exit of water above full supply level.
- ☛ **Vent Pipe:** for free circulation of air
- ☛ **Wash Out Pipe:** for removing water after cleaning the reservoir.
- ☛ **Water Level Indicator:** to know the level of water inside the reservoir from out side


LIGHTENING CONDUCTOR
MANHOLE
LADDER
WATER LEVEL INDICATOR
FSL
OVER FLOW PIPE
WASH OUT PIPE
INFLOW PIPE
OUTLET PIPE
G.S
DITCH

Design Capacity of Service Reservoirs

- The three major components of service storage are:
  1. Equalizing or operating storage
  2. Fire reserve
  3. Break down/Emergency reserve

Equalizing or operating capacity = max. surplus + max. deficit

- determined by finding out max. cumulative surplus during the stage when pumping rate is higher than water consumption rate and adding to this max. cumulative deficit which occurs during the period when the pumping rate is lower than the demand rate of water.
- This done by distributing daily demand over 24hrs, so that hourly demand is obtained.
- Max. surplus and max. deficit can be calculated using two methods

1. Analytical method 2. Mass- curve method

For Break down Reserve:-

- This is the amount of storage during the break down of pumps.
- From 2 – 3 hrs pumping capacity is provided against this storage.

For Fire reserve: -

- This is storage required for fighting a fire out break.
- In practice 2 – 5 lit/cap is normally provided for fire storage.

Depth and Shape of Service Reservoirs

- There is an economical depth of service reservoir for any given site.

Size (m3)	Depth of water (m)
Up to 3500	2.5 to 3.5
3500 to 15,000	3.5 to 5.0
Over 15,000	5.0 to 7.0

shape

- Circular reservoir is geometrically the most economical shape, giving the least amount of walling for a given volume and depth.
- But, It is unsuitable for division in to two compartments.
- circular shape frequently does not permit best use of available land,
- Problems of design will arise if it is to be partially buried in sloping ground

A rectangular reservoir with a length to width ratio 1.2 to 1.5:

- Usually proves most economical when division walls are incorporated
- Floors and roof should be sloped to not flatter than 1:250 for drainage
- It is good practice to set the over flow weir slightly higher, say by 50mm, than the top water level at which the supply is cut off by a ball valve or an electrode.

Examples

1. small town with a design population of 1600 is to be supplied water at 150liters per capita per day. The demand of water during different periods is given in the following table:

Determine the capacity of a service reservoir if pumping is done 24 hours at constant rate.

Time (hr)	0-3	3-6	6-9	9 -12	12 - 15	15- 18	18 -21	21- 24
Demand(1000liters)	20	25	30	50	35	30	25	25

Solution:

Rate of Water supply = 150l/c/d

Total water demand = demand * population

= 150*1600 = 240,000liters

Rate of pumping = 240,000/24 = 10,000lit/hr = 30,000lit/3hr

Time	Pumping	Demand	Cum.supply	Cum.demand	surplus	Deficit
0-3	30,000	20,000	30,000	20,000	10,000	
3-6	30,000	25,000	60,000	45,000	15,000	
6-9	30,000	30,000	90,000	75,000	15,000	
9-12	30,000	50,000	120,000	125,000		5000
12-15	30,000	35,000	150,000	160,000		10,000
15-18	30,000	30,000	180,000	190,000		10,000
18-21	30,000	25,000	210,000	215,000		5000
21-24	30,000	25,000	240,000	240,000		

Maximum cumulative surplus = 15,000 liters

Maximum cumulative deficit = 10,000 liters

Balancing storage = 15000 + 10000 = 25,000lit = $25m^3$

Let's take 3hr pumping rate

ii) For Break down storage = 3* 10,000 lit/hr = 30,000 lit **Let's take 5 lit/cap**

iii) For Fire reserve = 5 lit/cap * 1600 = 8,000 lit

Total capacity of reservoirs = 25,000 + 30,000 + 8,000

=63000lit =63m3

If the reservoir is circular with depth, h = 3.0 m,

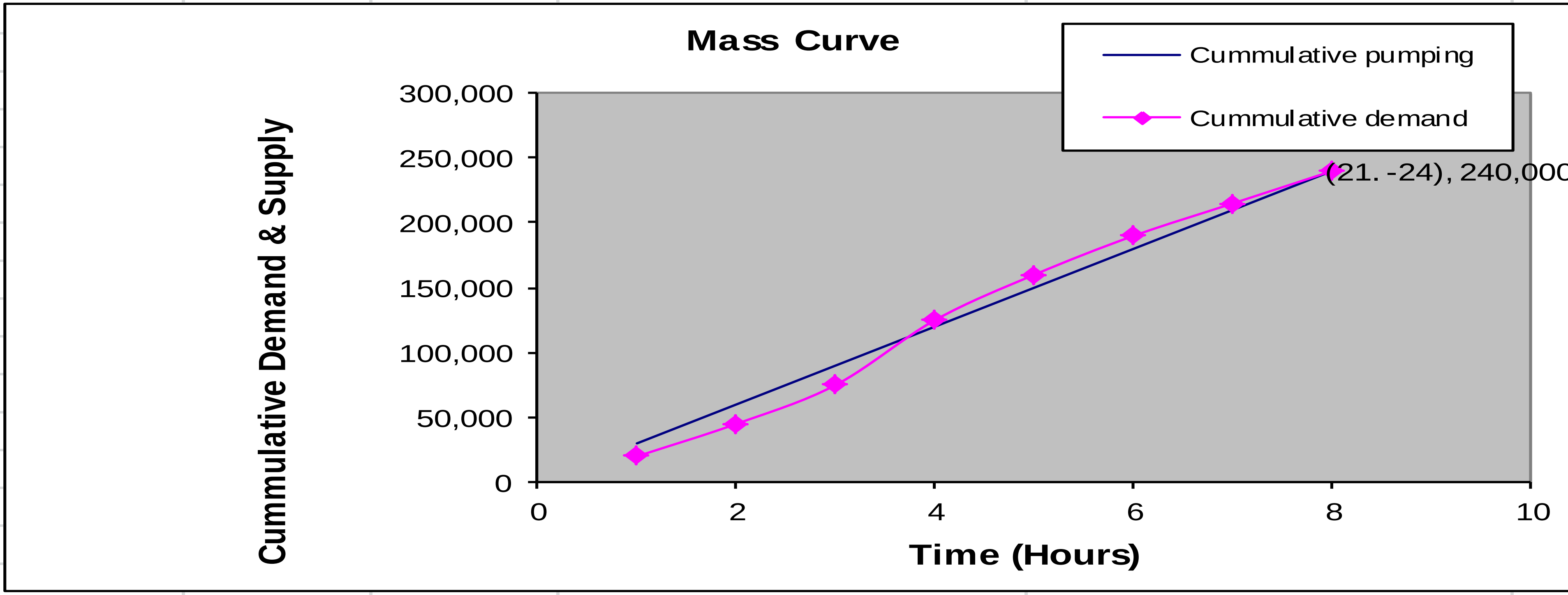
V=A*h

63m3=A*3m A=21m2

A=∏*d2/4 d=5.17m=5.2m

Graphically

Time	Pumping	Demand	Cummulative pumping	Cummulative demand
0-3	30,000	20,000	30,000	20,000
3. - 6.	30,000	25,000	60,000	45,000
6. - 9	30,000	30,000	90,000	75,000
9. - 12	30,000	50,000	120,000	125,000
12. - 15	30,000	35,000	150,000	160,000
15. -18	30,000	30,000	180,000	190,000
18. - 21	30,000	25,000	210,000	215,000
21. -24	30,000	25,000	240,000	240,000



Example 2. If in example -1 pumping is done for:

Case 1: Eight hours from 8 hrs to 16 hrs

Case 2: Eight hrs from 4 to 8 hrs and again 16 to 20 hrs.

Solution

Total water demand = 240,000 lit/day

Rate of pumping = 240,000 lit/8h = 30,000 l/h = 90,000 lit/3hrs

Case 1

A) Analytical Method

Time	Pumping	Demand	Cum. supply	Cum. Demand	Surplus	Deficit
0-3	0	20000	0	20000		20000
3-6	0	25000	0	45000		45000
6-8	0	20000	0	65000		65000
8-9	30000	10000	30000	75000		45000
9 -12	90000	50000	120000	125000		5000
12 -15	90000	35000	210000	160000	50000	
15 - 16	30000	10000	240000	170000	70000	
16 -18	0	20000	240000	190000	50000	
18-21	0	25000	240000	215000	25000	
21 -24	0	25000	240000	240000	0	0
	Maximum cumulative surplus =			70000		
	Maximum cumulative Deficit =			65000		
	Balancing Storage , S =			135000	litres	
				135	m3	

Let's take 3hr pumping rate

ii) For Break down storage = 3* 30,000 lit/hr = 90,000 lit

Let's take 4 lit/cap

iii) For Fire reserve = 4 lit/cap * 1600 = 6400 lit

Total capacity of reservoirs = 135,000 + 90,000 + 6,400

= 231,400lit

Determine the diameter of reservoir?

Case 2

A) Analytical Method

Time	Pumping	Demand	Surplus	Deficit	Cumulative	Cummulative Demand	Cummulative supply
						For Graphical Method	
0-3	0	20000		20000	-20000	20000	0
3-4.	0	8333		8333	-28333	28333	0
4 - 6	60000	16667	43333		15000	45000	60000
6 - 8	60000	20000	40000		55000	65000	120000
8 - 9	0	10000		10000	45000	75000	120000
9-12	0	50000		50000	-5000	125000	120000
12 - 15	0	35000		35000	-40000	160000	120000
15 -16	0	10000		10000	-50000	170000	120000
16-18	60000	20000	40000		-10000	190000	180000
18 - 20	60000	16667	43333		33333	206667	240000
20 - 21	0	8333		8333	25000	215000	240000
21 - 24	0	25000		25000	0	240000	240000

Maximum cumulative surplus =	55000	
Maximum cumulativeDeficit =	-50000	
Balancing Storage , S =	105000	litres

Self-test

A town with a population of **2000** is to be supplied water at **125** lpcd. The demand of water during different periods of the day is given as: Determine the capacity of a service reservoir if pumping is done ten hrs from **5 to 8** hrs,**13 to 17** and **19 to 22** hrs.

Time(hr)	0-3	3-6	6-9	9-12	12-15	15-18	18-21	21-24
Demand as% of total daily demand	6.125	8.7	13.5	24.525	17	12.6	7.7	9.85

Assignment-2

1.Water supply is provided through an overhead tank to a population of 60,000 living in town at a daily basis of 80 litres per capita water demand. The water is pumped into this tank at uniform rate for 16 hours of a day i.e. from 4AM to 12AM and from 2PM to 10PM.

The demand pattern during 24 hours of the day is as follows: Determine the required capacity of overhead tank to fulfill the balancing reserve.

Time(hr)	0-8	8-10	10-16	16-20	20-24
Demand Pattern (%age)	5	15	40	30	10

2. A water supply system is proposed to be designed for a small town which has a maximum daily demand of 515 m3/d. Estimate storage requirement if pumping is done for 10 hrs. only (from 5 to10 and 17 to 22). Use the following demand variation data.

Time (hr)	0 - 4	4 - 8	8 - 12	12 - 16	16 - 20	20 - 24
Demand as % of total daily demand	6.7	9.2	20.8	28.3	25	10

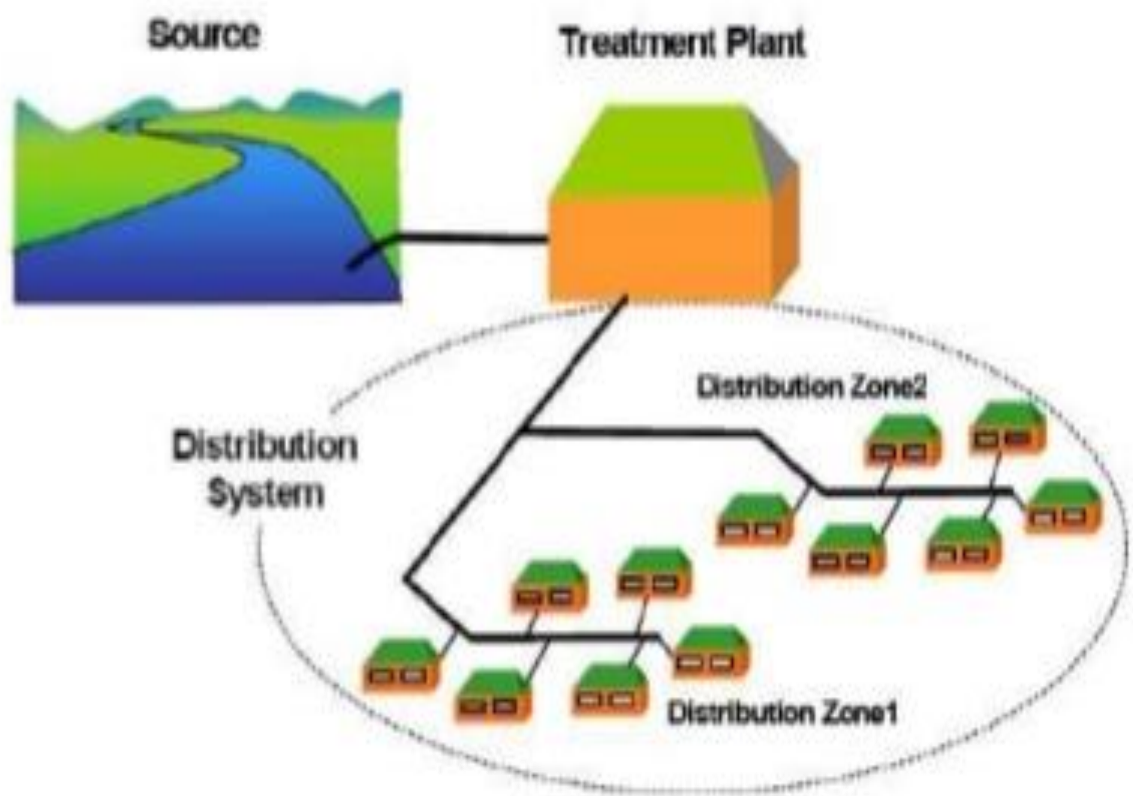
5.3 Distribution system of water

- Methods of Distribution System
- Layout of distribution System
- Pipe Materials and pipe corrosion
- Determination of Pipe Sizes
- Energy Losses in Pipes
- Pipe Appurtenances

INTRODUCTION

- "**<u>Transmission line"</u>** means any pipeline conveying raw or treated water from a well field or remote storage facility to a treatment plant and/or distribution storage tank.
- **<u>"Distribution line",</u>** including individual customer connections and distribution mains, means those lines conveying water to customers and fire protection systems from a common source.
- It is composed of pumps, pipe lines, ground or elevated storage reservoirs, valves and other appurtenances.

WATER DISTRIBUTION SYSTEM



Requirement of a Distribution System:

- The should convey the treated water up to consumers with the same degree of purity.
- The system should be economical and easy to maintain and operate.
- The diameter of pipes should be designed to meet the fire demand.
- It should safe against any future pollution. As per as possible should not be laid below sewer lines.
- Water should be supplied without interruption even when repairs are undertaken.
- The system should be so designed that the supply should meet peak hourly demand.

Methods of Distribution System

- Depending upon the level of the source of water and the city, topography of the area, and other local considerations.
- For efficient distribution it is required that water should reach to every consumers with required flow rate. Therefore, some pressure in pipe lines is necessary. Which force the water to reach at every place.
- Water can be transported from the source to the treatment plant, if any, and the distribution system, and eventually reach consumers through one of the following methods.

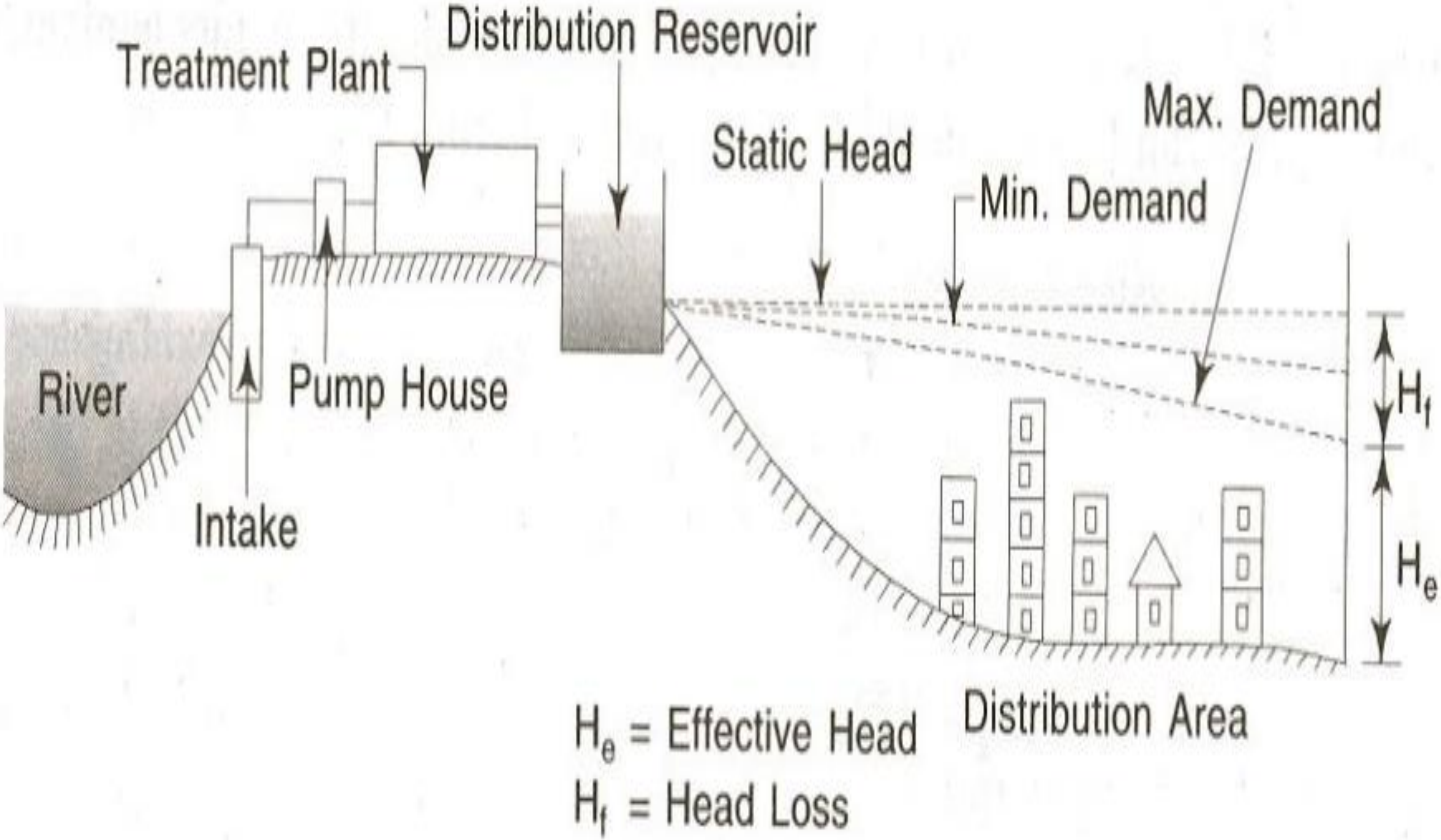
  i. **Gravity system**

  ii. **Pumping system**

  iii. **Dual system**

i) Gravity System

- In this system, the water flows under the force of gravity from the distribution reservoir to the distribution area.
- This system is suitable when the source of water treatment plant and the distribution reservoir are situated at a high level than the distribution area.
- action of gravity without any pumping
- most economical and reliable

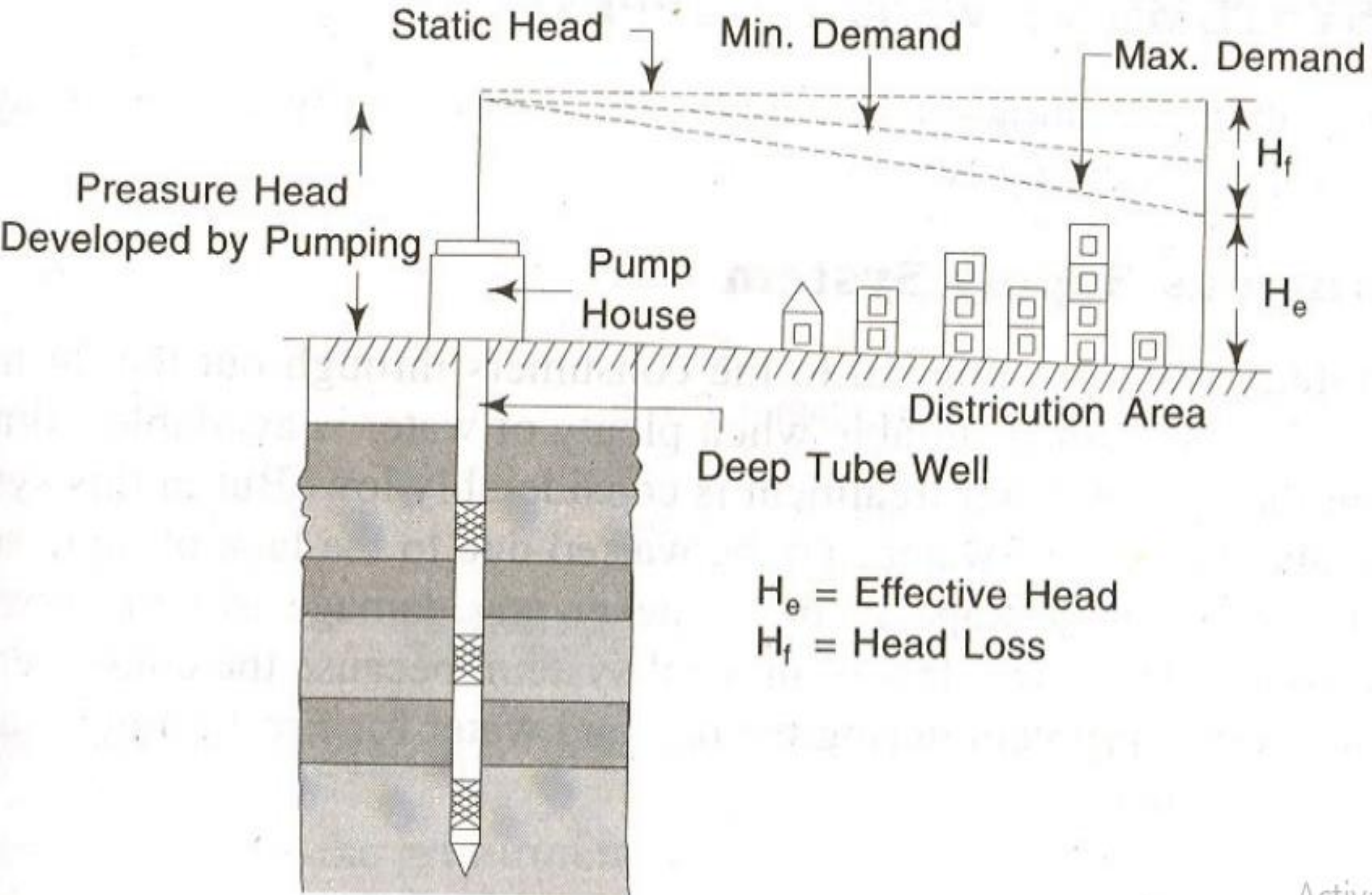


Distribution Reservoir
Treatment Plant
Max. Demand
Static Head
Min. Demand
River
Pump House
Intake
H_f
H_e
Distribution Area
H_e = Effective Head
H_f = Head Loss

ii. Pumping System

In this system, the water is directly pumped in the main.

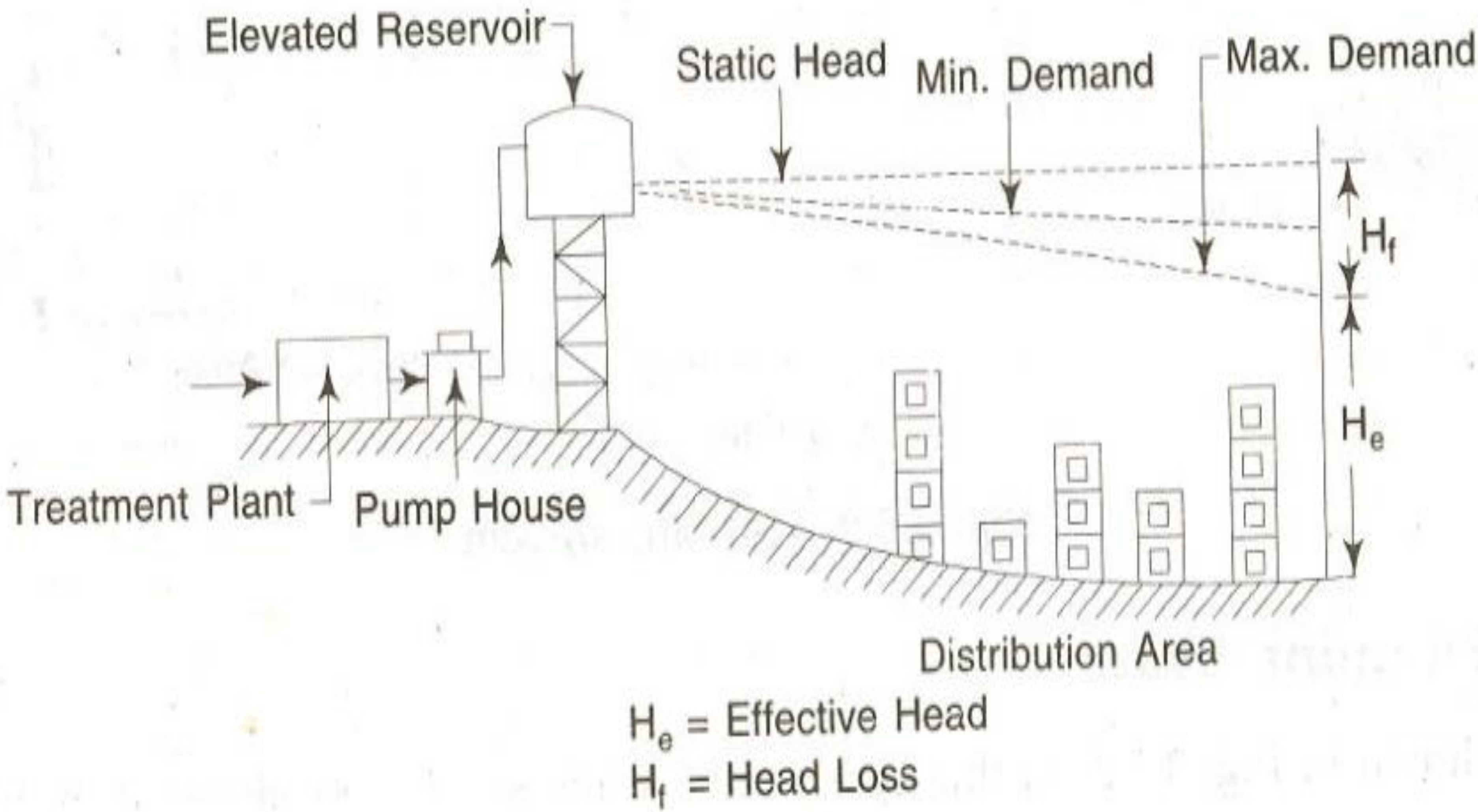
- treated water is directly pumped into the distribution mains without storing
- Since the pumps have to work at different rates in a day, the maintenance cost will increases.
- Disadvantageous (power failure) $\longrightarrow$ no reserve flow



Static Head
Min. Demand
Max. Demand
Preasure Head
Developed by Pumping
Pump
House
H_f
H_e
Districution Area
Deep Tube Well
H_e = Effective Head
H_f = Head Loss

iii. Dual System

- In this system, the pumping and gravity both systems are utilized simultaneously when required.
- Normally, the pumps are operated at a constant speed to meet the average demand of water. So, during the period of low demand, the excess water is stored in an elevated reservoir. During the period of peak demand, the water is supplied by pumping and from the elevated reservoir simultaneously.
- This system is fairly reliable b/c in the case of failure of pumping the water supply can be continued for some period from the reservoir.


Elevated Reservoir
Static Head
Min. Demand
Max. Demand
H_f
H_e
Treatment Plant
Pump House
Distribution Area
H_e = Effective Head
H_f = Head Loss

Layout of distribution systems

- layout of distribution pipes generally follows the road pattern
- Generally in practice there are four different system of distribution which is used.
- Depending upon their layout of direction of supply, they are classified as follows:
  1. Dead End or Tree system
  2. Grid Iron system
  3. Circular or Ring system
  4. Radial system

i) Dead end or Tree System

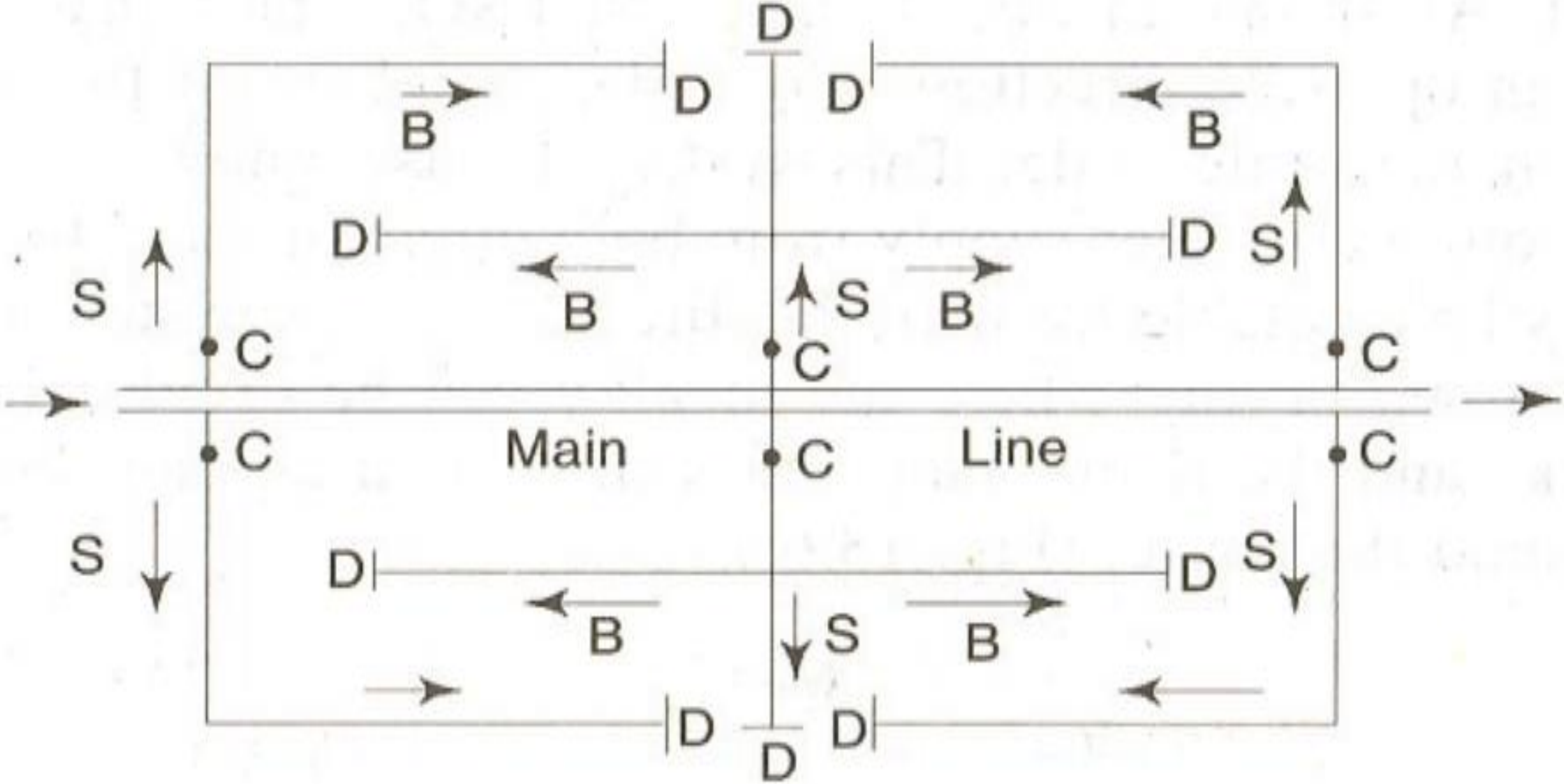
- In this system, a main line is taken from the reservoir along the main road.
- The sub-mains are taken suitably from the main line.
- Cut-off values are provided at the entry of sub-mains. From the sub mains, the branch lines are taken from which service connections are give to consumer through the ferrule.
- The end of the sub-mains and branch lines are stopped by scour values which are known as dead – ends.
- Due to the dead – ends, there is no free circulation of water and the water remains stagnant within the pipe line.
- This system is suitable for regular developing town or city.

ADVANTAGES

- ☛Discharge and pressure at any point in the distribution system is calculated easily
- ☛ The valves required in this system of layout are comparatively less in number.
- ☛ The diameter of pipes used are smaller and hence the system is cheap and economical
- ☛ The laying of water pipes is used are simple.

DISADVANTAGES

- ☛ There is stagment water at dead ends of pipes causing contamination.
- ☛ During repairs of pipes or valves at any point the entire down stream end are deprived of supply
- ☛The water available for fire fighting will be limited in quantity



S = Sub-main Line
C = Cut-off
B = Branch Line
D = Dead End

ii) Grid – Iron System

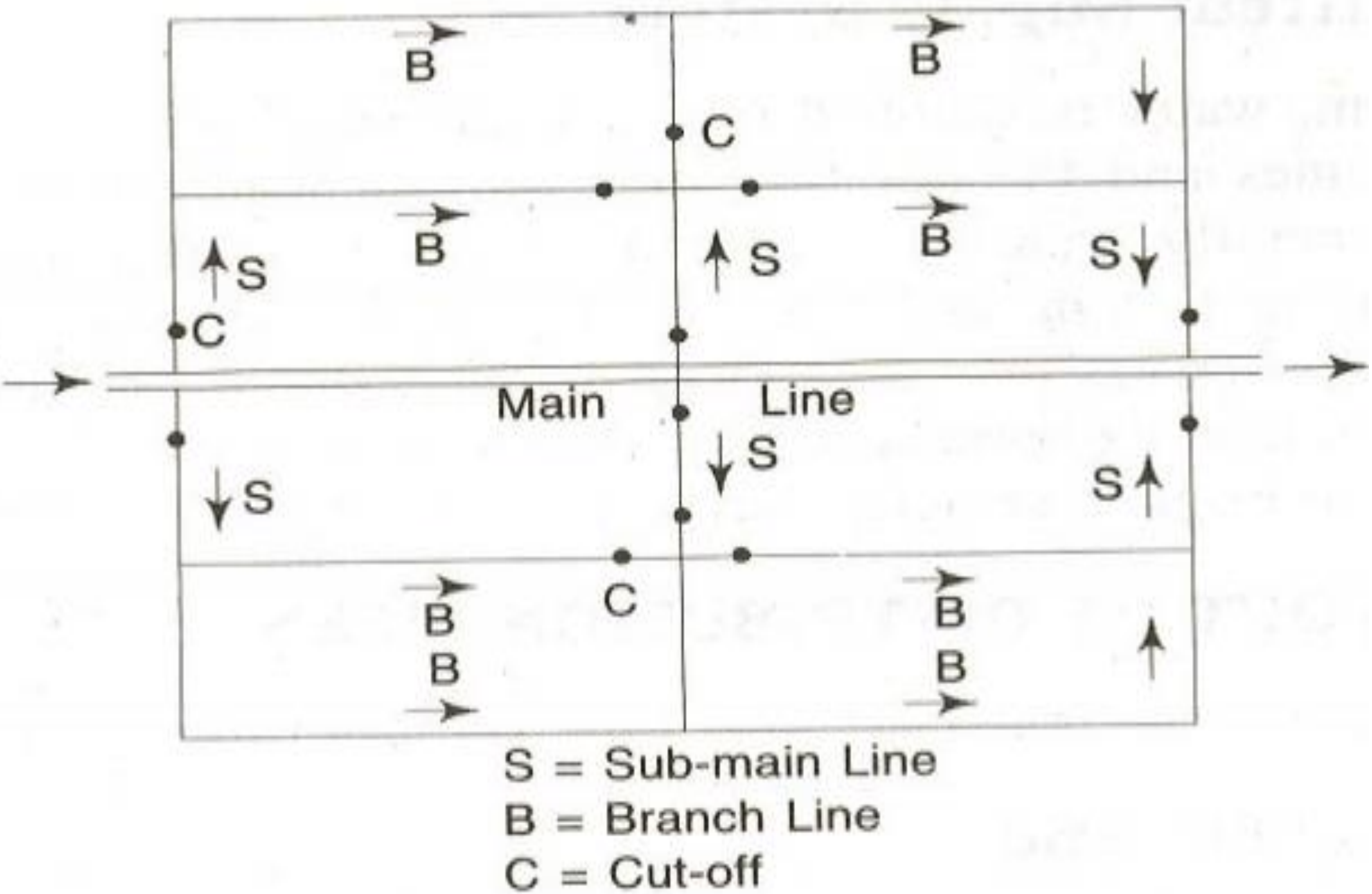
- In this system, the main line, the sub-main lines, and the branch lines are interconnected. So, there is free circulation of water through the pipe lines.
- Cut-off values are provided at each junction point so that the repair works may be conducted at a particular area without disturbing the whole area.
- In this system the length of the pipe as too long, and hence it is very costly.
- It is suitable for town or city having rectangular lay out of roads.

ADVANTAGES

- In the case of repairs a very small portion of distribution are a will be affected
- Every point receives supply from two directions and with higher pressure
- Additional water from the other branches are available for fire fighting
- There is free circulation of water and hence it is not liable for pollution due to stagnation.

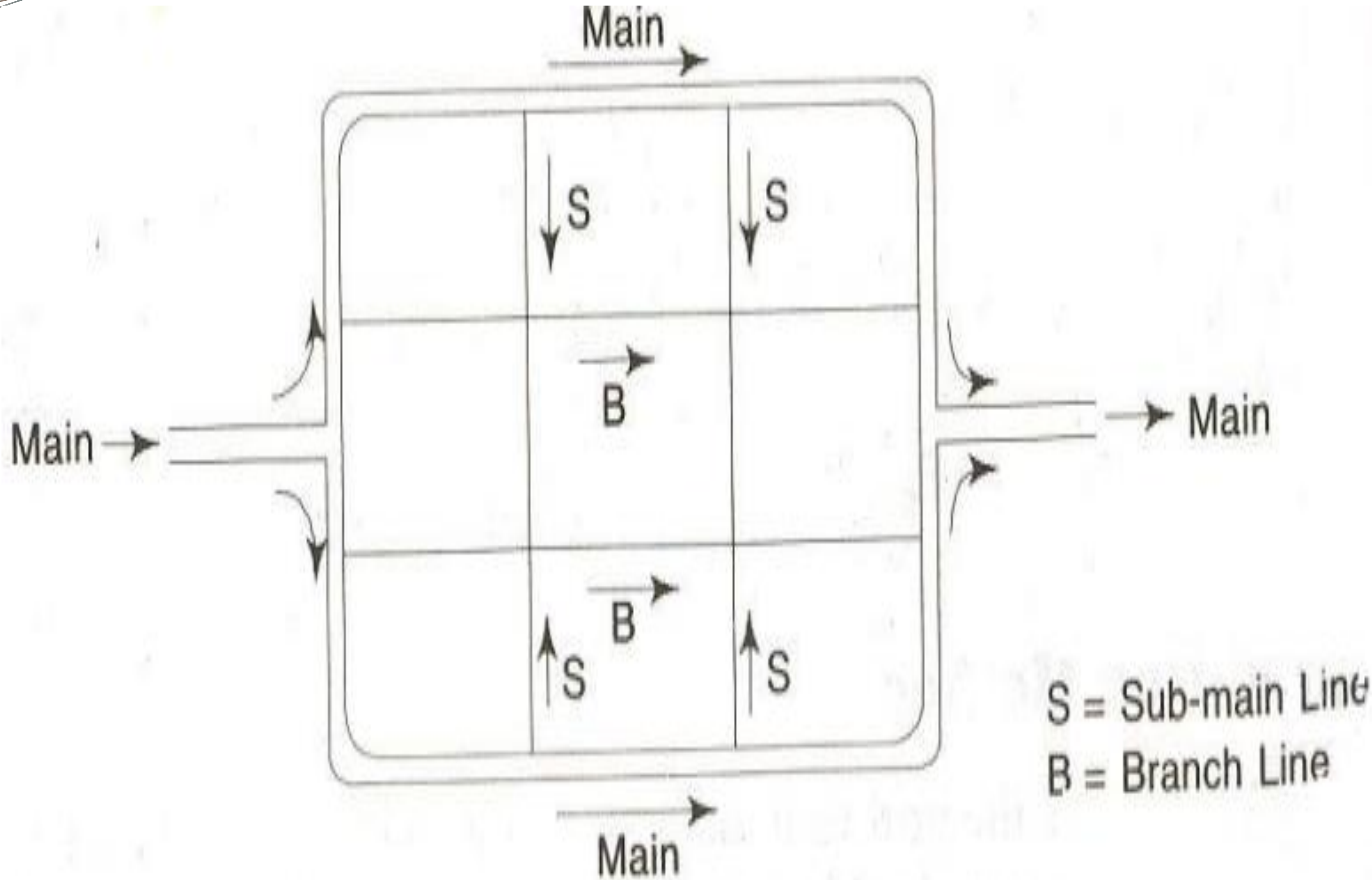
DISADVANTAGES

- More length of pipes and number of valves are needed and hence there is increased cost of construction
- Calculation of sizes of pipes and working out pressures at various points in the distribution system is laborious , complicated and difficult.


B
B
C
B
B
S
S
S
C
Main
Line
S
S
S
C
B
B
B
B
S = Sub-main Line
B = Branch Line
C = Cut-off

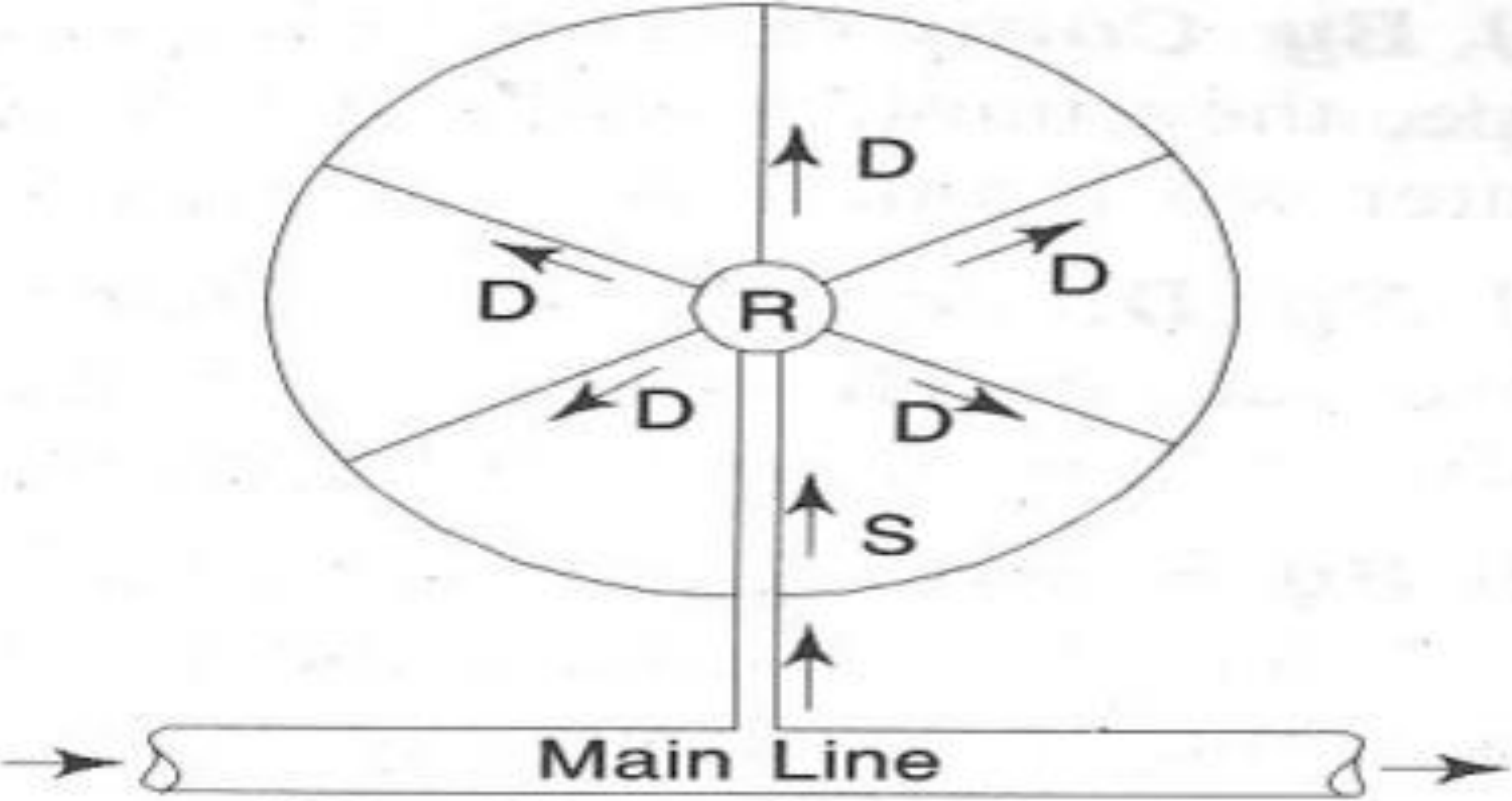
iii) Circular or Ring System

- ☛In this system, the main water line is divided in to two parts; in two direction left and right.
- ☛In inlet side, the left and the right water main on the outlet side.
- ☛It is suitable for well planned town or city where the locality can be divided in to square or circular blocks and the main water line can be laid around the sides of the squire or around the circle.


Main
S
S
B
Main
Main
B
S
S
Main
S = Sub-main Line
B = Branch Line

iv) Radial System

- In this system, the town or city is divided in to various circular or square zones and distribution reservoirs are placed at the center of each zone.
- The distributor lines are laid radially from reservoir towards the periphery of each zone.
- It is sailable when the town or city can oriented with radial roads and streets
- In this system, the water from the main reservoir is allowed to flow through the main pipe and sub-main pipe and get collected at distribution reservoir of each zone.
- The water is supplied to consumers through the distributor pipe lines.



R = Central Reservoir
S = Sub-main
D = Distributor

5.5. Pipes Used in Water Distribution Systems

- ✓ Pipe Materials
- ✓ Determination of Pipe Sizes
- ✓ Energy Losses in Pipes
- ✓ Pipe Appurtenances

- The types of pipes used for distributing water include:

1. Cast iron pipe
2. Galvanized Iron Pipes
3. Steel pipe
4. Concrete pipe
5. Plastic pipe
6. Asbestos cement pipe
7. Copper pipe
8. Lead pipe

Pipe Materials

- ☛ Pipes convey raw water from the source to the treatment plants in the distribution system.
- ☛ Water is under pressure always and hence the pipe material and the fixture should withstand stresses due to the internal pressure, vacuum pressure, when the pipes are empty, water hammer when the values are closed and temperature stresses.

REQUIREMENTS OF PIPE MATERIAL

- ☛ It should be capable of with standing internal and external pressures
- ☛ It should have facility of easy joints
- ☛ It should be available in all sizes, transport and errection should be easy.
- ☛ It should be durable
- ☛ It should not react with water to alter its quality
- ☛ Cost of pipes should be less
- ☛ Frictional head loss should be minimum
- ☛ The damaged units should be replaced easily.

Cast iron pipes

Advantages

- The cost is moderate
- The pipes are easily joined
- The pipes are not subjected to corrosion
- The pipes are strong and durable
- Service connections can be made easily

Disadvantage

- The breakage of this pipe is large
- Carrying capacity decreases with increase in life
- The pipes become heavy and uneconomical when their sizes increase

Galvanized Iron Pipes

Advantages

- The pipes are cheap
- Light in weight and easy to handle and transport
- Easy to join

Disadvantage

- These pipes are liable to incrustation (due to deposition of some materials inside part of pipe)
- Can be easily affected by acidic or alkaline water
- Short useful life

Cement Concrete Pipes

Advantages

- The inside surfaces of the pipes can be made smooth
- The maintenance cost is very low
- Under normal conditions the pipes are durable
- The pipes can be cast in place(in site)
- Due to high weight (heaviness) the pipes can resist force of buoyancy when placed under water even when they are empty
- Pipes can resist normal traffic loads when placed below roads
- There is no danger of rusting and incrustation

Disadvantage

- The pipes are difficult to transp
- If no reinforcement is provided it cannot resist high pressure
- The pipes are likely to crack during transport and handling
- The repair of these pipes are difficult
- These pipes are affected by acids, alkaline, and salty waters
- These pipes are likely to cause leakage due to porosity

Plastic Pipes

Advantages

- The pipes are cheap
- The pipes are flexible and possess low hydraulic resistance (less friction)
- They are free from corrosion
- The pipes are light in weight and it is easy to bend, join and install them
- The pipes up to certain sizes are available in coils and therefore it becomes easy to transport

Disadvantage

- The coefficient of expansion for plastics is high, the pipes are less resistant to heat
- Some types of plastics may impart taste to the water

Asbestos cement pipes

Advantages

- The inside surface of pipe is smooth
- The joining of pipes is very good and flexible
- The pipes are ant-corrosive and cheap in cost
- Light in weight to handle and transport

Disadvantage

- The pipes are brittle
- The pipes are not durable
- The pipes are not laid in exposed places
- The pipes can be used only for very low pressure

Determination of Pipe Sizes

- The size of the pipe is determined by considering the discharge through the pipe and permissible velocity of the flow in the pipe.

Q = A*V

Where, Q = discharge (m3/s)

V = permissible velocity (0.6 to 1.50m/s)

A = Cross sectional area of pipe (m2)

Or using

2. Darcy –Weisbach formula; $h_f = \frac{fLv^2}{2gD}$
3. Hazen-Williams formula; $Q = 0.278CD^{2.63}S^{0.54}, S = \frac{h_f}{L}$
4. Manning's Formula; $Q = \frac{AR^{2/3}S^{1/2}}{n}$

Where C = Hazen-William constant, coefficient that depends on the material and age of the pipe

D = pipe diameter

L = pipe length

S = slope

n = manning's coefficient

R = hydraulic mean radius

g = gravitational acceleration

Hf = the loss of head

f = friction factor (which is related to the relative roughness of the pipe material & the fluid flow characteristics)

Table - Values of C for the Hazen-Williams formula

Pipe Material	C
Asbestos Cement	140
Cast Iron ▪ Cement lined ▪ New, unlined ▪ 5years-old, unlined ▪ 20 years old, unlined	 130 – 150 130 120 100
Concrete	130
Copper	130 - 140
Plastic	140 -150
New welded Steel	120
New riveted Steel	100

The water supply pipes sizes available are given in the following table:

Metric sizes (mm)	10	20	25	30	40	50	60	80	100	150	200	250	300	350	375	400	450	500	525	600
English (In)	1/2	3/4	1	11/4	11/4	2	21/2	3	4	6	8	10	12	14	15	16	18	20	21	24

Metric sizes (mm)	675	750	900	950	1050
English (In)	27	30	36	38	42

Example

Given

- Total population of a town = 80,000
- Average daily consumption of water = 150liters/capita/day
- If the flow velocity of an outlet pipe from intake is 1.5 m/s, determine the diameter of the outlet pipe.

Solution

Total flow, Q = Demand* Population = 150*80,000 = 12x10^6lit/day

= 0.1389m3/s

Required pipe area, $A = \frac{Q}{v} = \frac{0.1389}{1.5} = 0.0926m2$

$$D = \sqrt{\frac{4 * A}{\pi}} = \sqrt{\frac{4 * 0.0926}{\pi}} = 0.343m$$

But the pipe size available on the market is 300mm & 350mm, then take D = 350mm

Example

- A town has a population of 100,000 persons. It is to be supplied with water from a reservoir situated at a distance of 6.44km. It is stipulated that one-half of the daily supply of 140lit/capita should be delivered in 6 hours. If the loss of head is estimated to be 15m, calculate the size of pipe. Assume f = 0.04.

Solution

Total daily supply $= 100{,}000 * 140 liter/d$= 14000000l/d=14000l/d

- Since half of this quantity is required in 6 hours

Maximum flow =$\frac{14000}{2*6*60*60} = 0.324 m3/s$

Using Darcy –Weisbach formula; $h_f = \frac{fLv^2}{2gD} = \frac{fLQ^2}{12.1D^5}$

D = 0.683m =683mm

Exercise:

From the above e.g. what is the size of pipe line ($L = 1000m$) should be used to supply 100l/s so that the head loss does not exceed 10m. Use all the three formula, $C = 100$, $n = 0.013$, $f = 0.035$, find also the velocity.

Energy Losses in Pipes

When a liquid is flowing in pipe , it loses energy or head due to friction of wall , change of cross section or obstruction in the flow . All such losses are expressed in terms of velocity head.

The following are losses which occur in a flowing fluid.

1. Loss of head due to friction
2. Loss of head due to sudden enlargement
3. Loss of head due to sudden contraction
4. Loss of head due to bends
5. Loss of head at entrance
6. Loss of head at exit.

Energy Losses in Pipes

- Energy loss (head loss) in pipes can be found by one of the following formulas:
  i. Darcy-Weisbach formula $$h_f = \frac{fLv^2}{2gD}$$
  ii. Hazen-Williams formula $$Q = 0.278CD^{2.63}S^{0.54}, S = \frac{h_f}{L}$$
  iii. Manning's Formula $$\mathrm{Q} = \frac{AR^{2/3}S^{1/2}}{n}, S = \frac{h_f}{L}$$
- Nomographs shown in fig – solve the equation for C = 100.
- Given any two of the parameters (Q, D, hf or V), the remaining can be determined from the intersections along a straight line drawn across the nomograph.

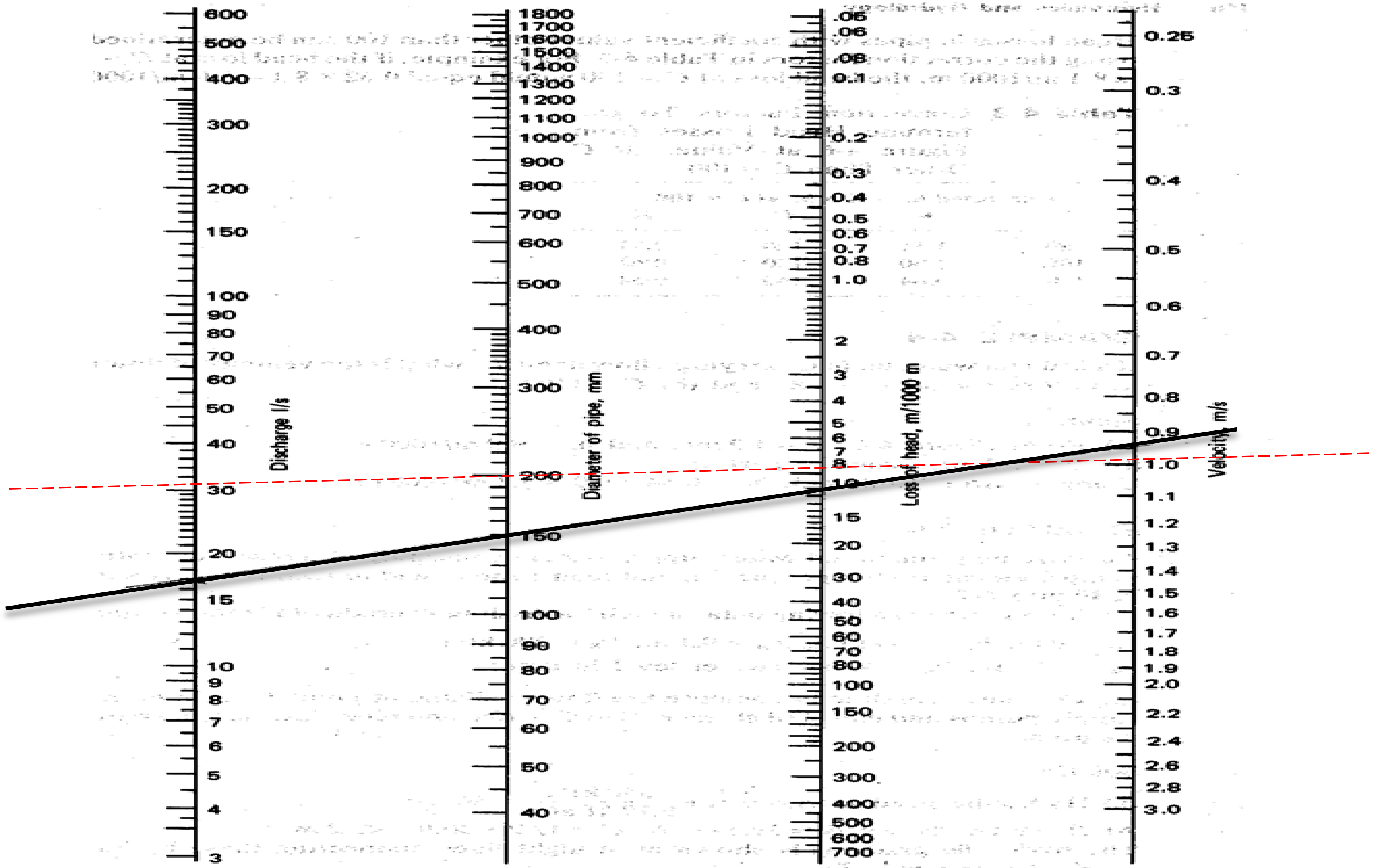


Figure 4-6 Nomograph for Hazen Williams Formula, Eq. 4-7, based on $C = 100$.

Example-1

Find the loss of the head due to friction in a pipe of 1000mm diameter and 2.0 km long. The velocity of water in the pipe is 2m/sec. Take coeff. of friction as 0.005

Solution:

Diameter of pipe, d = 1000mm = 10m

Length of pipe , l = 2.0 km = 2000m

Velocity of water, v = 2m/sec

Coeff of friction , f = 0.005

Loss of head, hf = ?

$$h_f = \frac{fLV^2}{2gD}$$

$$Hf = \frac{0.005*2000*2^\wedge 2}{2*9081*10} =$$

Exercises

1. Calculate the head loss in diameter 200mm pipe, Q = 30l/sec, f = 0.035, L= 1500m [Ans, hf = 12.20m]
2. For Q = 30l/s, D = 200mm,C = 100,L = 1500,Find head loss, hf.

Solution

From nomograph, hf = 12.0m,

i.e. hf/L = 8/1000, (8/1000)*1500 =12m

Using the formula, hf = 12.30m,$Q = 0.278CD^{2.63}S^{0.54}, S = \frac{h_f}{L}$

3. For Q = 30l/s, D = 200mm,n = 0.013,L = 1500, find head loss

From Nomograph, hf/L = 0.00825 = 0.00825*1500 = 12.38m

Pipe Appurtenances

- **Valves:** Used to isolate pipe line sections for test, inspection, cleaning and repair
- **Gate valves.** Are installed in every main and sub-main to isolate a portion of the network system during a repair.
- **Check-valves** (Non-Return valves): used to prevent reversal of flow when a pump is shut down
- **Air-Relief Valves.** In long pipes lines air will accumulate in the high points (summits) of the line and may interfere with the flow. It is necessary, therefore, to place air relief valves at those points where trouble is expected.
- **Pressure regulating valves.** These valves automatically reduce pressure on the d/s side to any desired magnitude and are used on lines entering low areas of a city, with out such reductions pressures would be too high.
- **Sluice Gates.** Are vertically sliding valves which are used to open or close openings in to walls.
    - **Fire hydrants:** It is used on mains to provide a connection for fire hazards to fire fighting
    - **Water meters:** to measure consumed water, and the consumer charged accordingly to the amount of water consumed

Exercise:

1) What are the requirements of a distribution system?
2) List out the different types of layouts of city water distribution system.
3) Discuss the methods of distribution of water supply.
4) What are advantages and disadvantages of different types of pipe material used in the water supply distribution system?
5) What are the requirements of pipe material?
6) Mention any four appurtenances used in water distribution system and explain their functions.

5.6. Design of the Distribution System

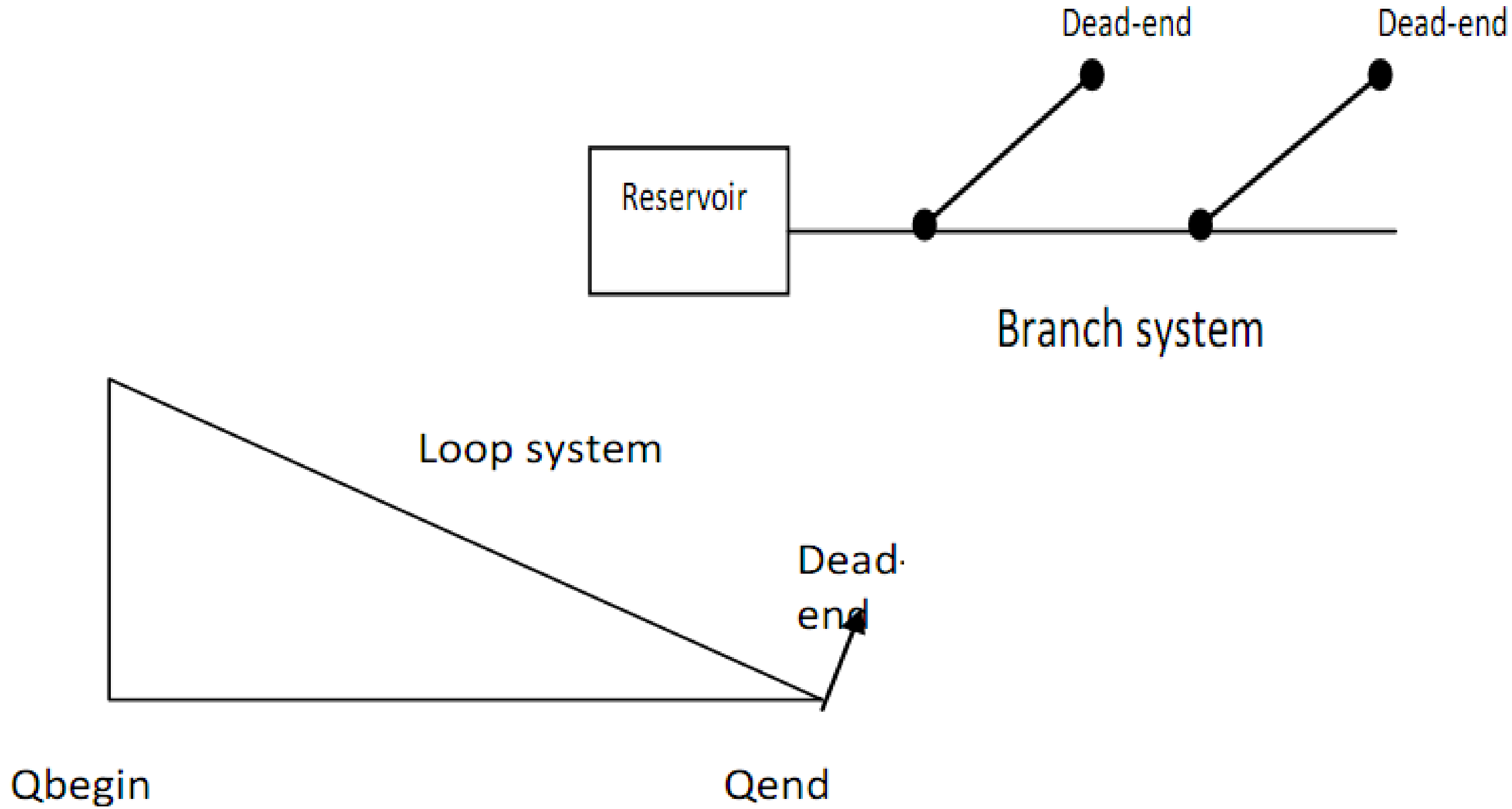
5.6.1. Hydraulic Analysis of Distribution Systems

Most commonly methods used are:

a) Dead-end method
b) Equivalent pipe method
c) Hardy-Cross method
d) Computer *Software(Epanet, Water* CAD and Water GEMs

A) Dead-End Method

- Determine the locations of "dead-ends" providing that water will be distributed in the shortest way. At the dead-end points there will be no flow distribution.



- While doing the design First of all the diameters of the pipes are assumed,
- Then the terminal pressure heads at the end of each pipe section are determined
- The determination of friction loss (hL) can be determined by using Hazen – William formula.

$$Q = 0.278 * C_H * D^{2.63} * S^{0.54}$$

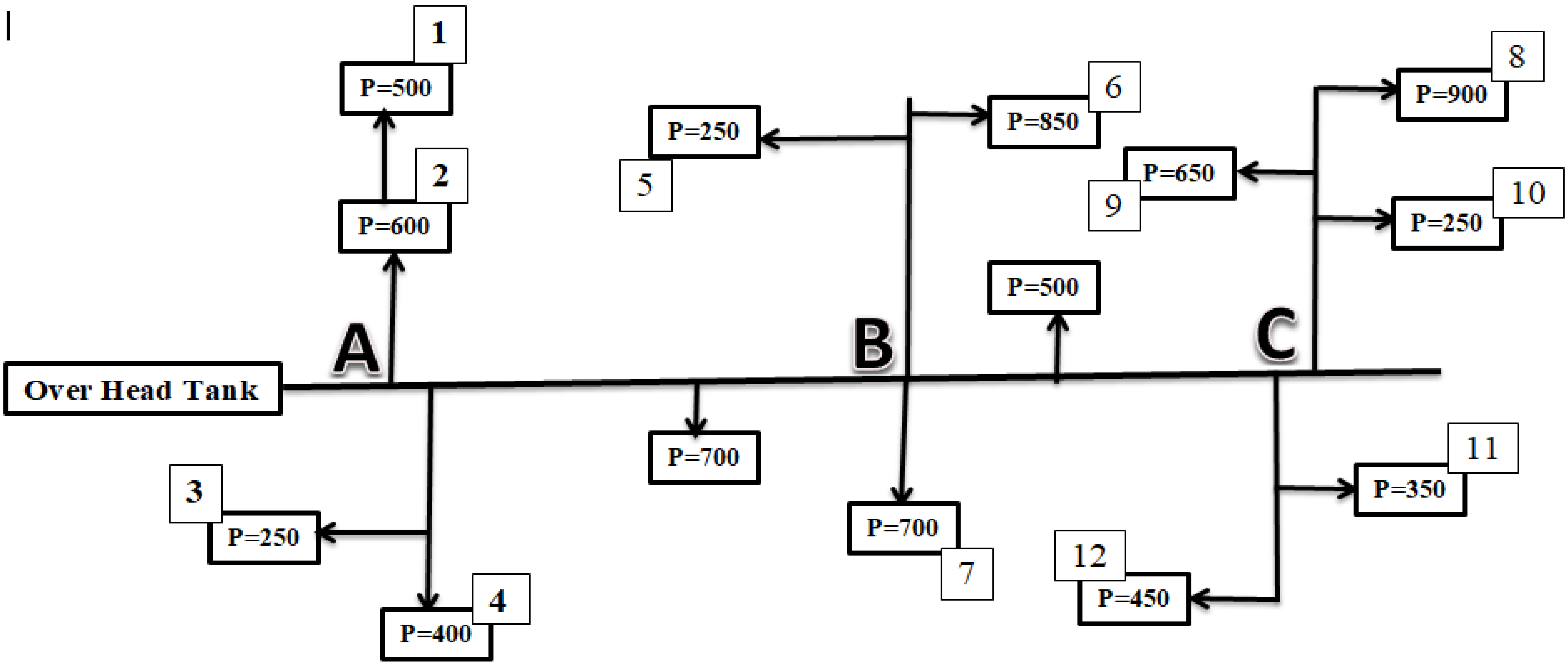
$$Q = 0.278 * C_H * D^{2.63} (H_L/L)^{0.54}$$

- To reduce the tedious calculation, the Hazen-Williams nomogram is used to determine HL=S*L in the nomograph CH=100 because usually in practice C.I. pipes of standard diameter are used.
- The distribution mains are designed for the max.hourly demand of the max.requirment day.

Example: Water supply line is to be laid in a town developed in haphazard way. The following figure shows the various zones of the town and the population in each zone. The pipe lines are to be laid in dead-end type. The average requirement of the town is 175 lit/cap/day. Design of the distribution pipes AB and BC with the following Data.

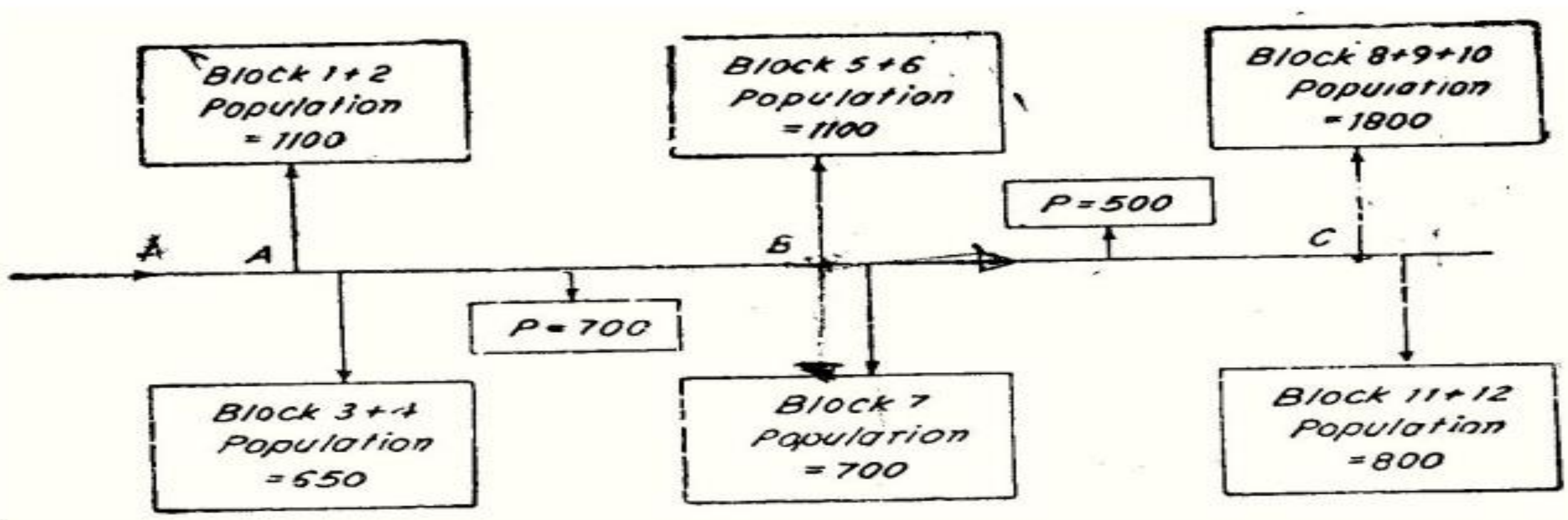
- (i) Take the population for design from the figure
- (ii) the R.L of the bottom of the service reservoir is 185.5m at A=168m, at B=154m and at C=146m
- (iii) the R.L of the pipe points on the main at C=146m

- (iv) The length of the pipe AB= 700m and BC=550m.
- (v) The distribution system should be designed for max. demand 3 times the average demand.
- (vi)The min pressure head to be maintained at any point in the distribution system should be 15m.



Solution

1 Pipe section	population 2 Previous	3 Local	4 Total	Max. demand 5 (l/s)	6 Assumed dia. (mm)	Head loss 7 Rate per 1000 (R)	8 Length (m) (L)	9 Actual (RxL)	10 Hydraulic level worked out from the last point (m)	11 Ground level(m)	12 Net head available at the indicated point (m)
BC	2600	500	3,100	18.83	150	15.5	550	8.52	171– 8.52 = 162.98=C	146=C	16.93=162.8 -146
AB	3100+1800= 4900	700+1750= 2450	7,350	44.66	200	20	700	14	185.5– 14 = 171.5=B 185.5-0=185.5=A	154=B 168=A	17.5=171.5-154 185.5-168=17.5



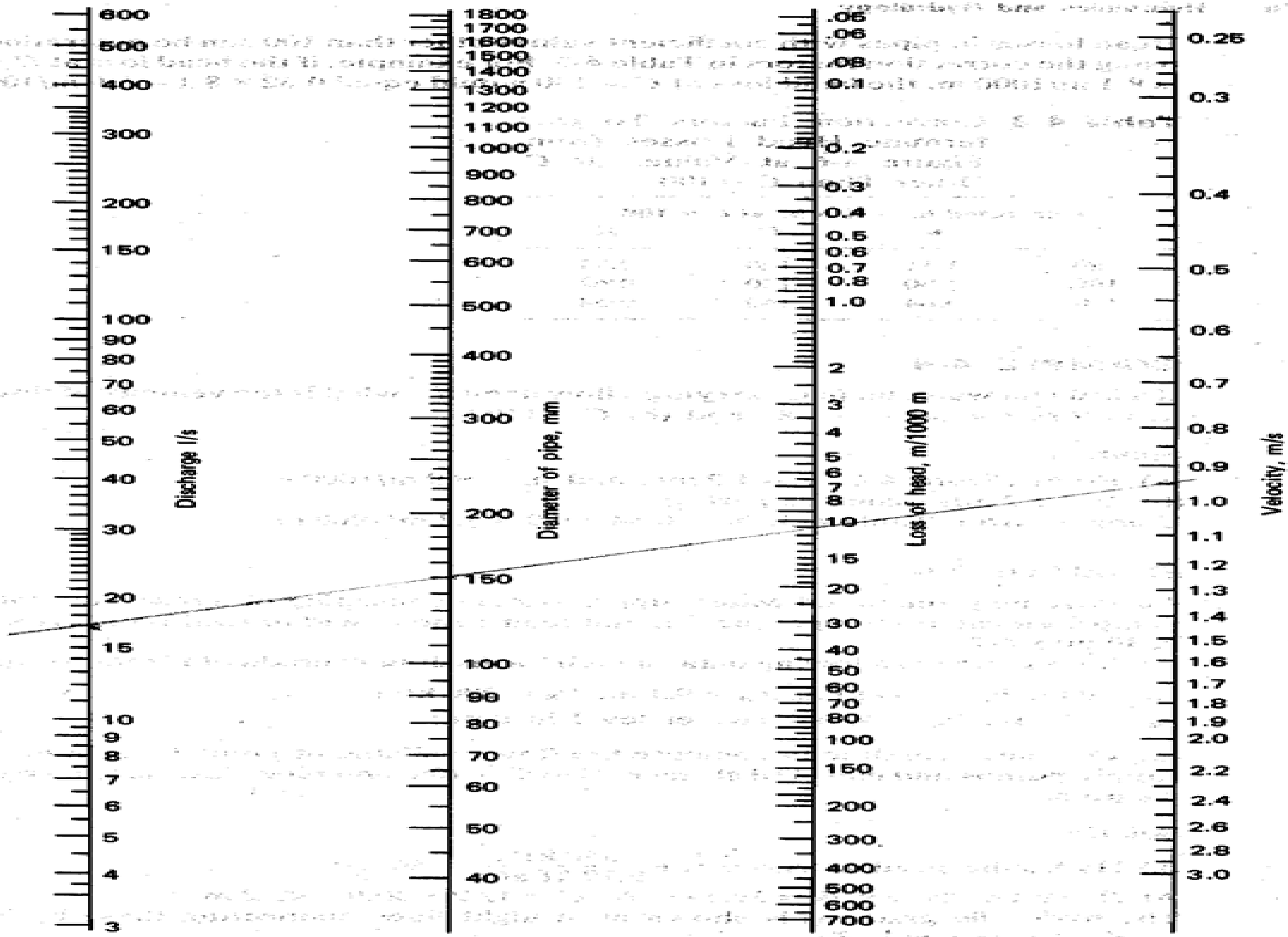


Figure 4-6 Nomograph for Hazen Williams Formula, Eq. 4-7, based on $C = 100$.

Average water demand = Population * Per capital demand

Max. Demand=peak factor*Average water demand

Col(4) = Col (2) + Col (3)

$$\text{Col}(5) = \frac{3*175*\text{Col}(4)}{24*60*60}$$

Col(7)= Read from the hazen -Williams nomograph by taking Q(col (5)) and D(col (6))

$$\text{Col}(9) = \frac{\text{col}(7)*\text{col}(8)}{1000}$$

Col (10) = start form point A,

A, Hydraulic level A = R.L of service reservoir =185.5

>> >> B = R.L of pt. A – Col (9) = 185.5- 14 = 171.5

>> >> C = H.L of pt. B – Col (9) = 171.5 – 8.52 = 162.98

Col (12) = col (10) – col (11

(b) Equivalent pipe method

In the water supply networks, the pipe link between two nodes may consist of a single uniform pipe size (diameter) or a combination of pipes in series or in parallel. As shown in Fig. 2.17a, the discharge Q flows from node A to B through a pipe of uniform diameter D and length L. The head loss in the pipe can simply be calculated using Darcy–Weisbach equation (2.3b) rewritten considering $h_L = h_f$ as:

$$h_L = \frac{8fLQ^2}{\pi^2 gD^5}.$$

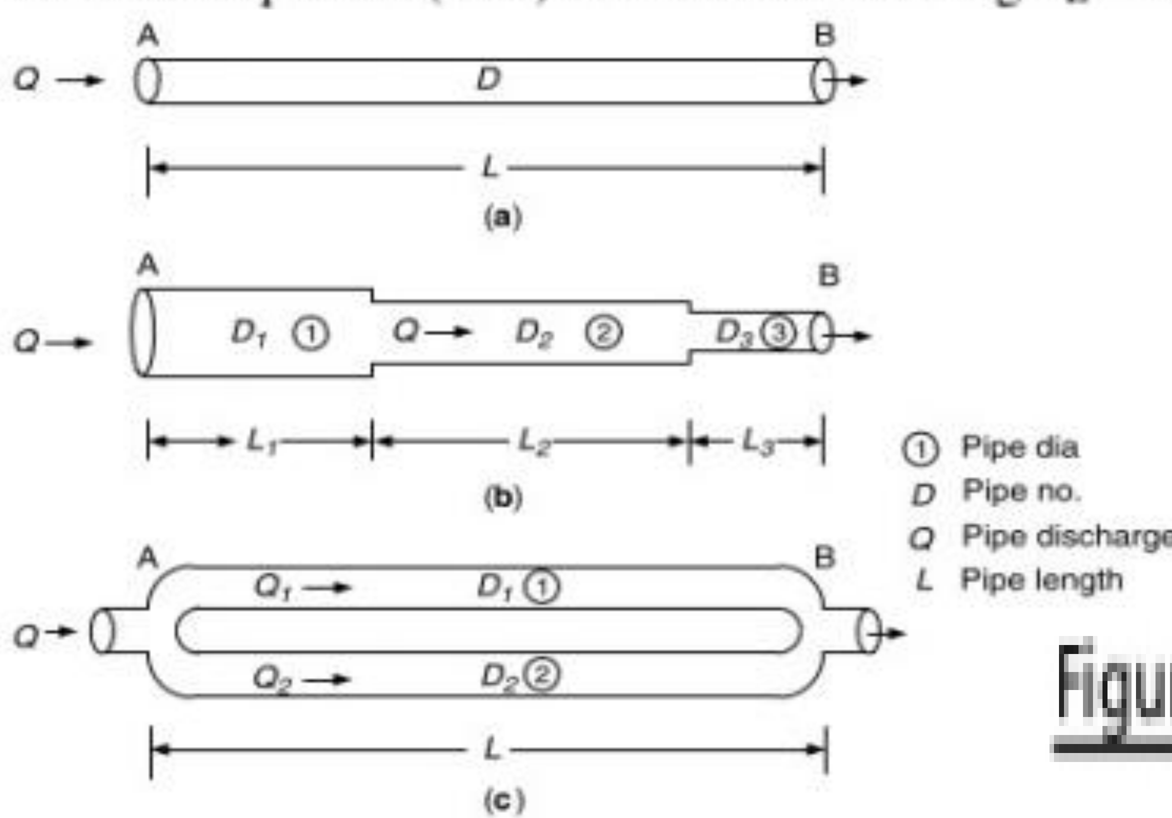


Figure 2.17. Pipe arrangements.

1) Flow Through Pipes in Series

In case of a pipeline made up of different lengths of different diameters as shown in Fig. 2.17b, the following head loss and flow conditions should be satisfied:

$$h_L = h_{L1} + h_{L2} + h_{L3} + \cdots$$

$$Q = Q_1 = Q_2 = Q_3 = \cdots.$$

Using the Darcy–Weisbach equation with constant friction factor f, and neglecting minor losses, the head loss in N pipes in series can be calculated as:

$$h_L = \sum_{i=1}^{N} \frac{8fL_iQ^2}{\pi^2 g D_i^5}.$$

Denoting equivalent pipe diameter as D_e, the head loss can be rewritten as:

$$h_L = \frac{8fQ^2}{\pi^2 g D_e^5} \sum_{i=1}^{N} L_i.$$

Equating these two equations of head loss, one gets

$$D_e = \left(\frac{\sum_{i=1}^{N} L_i}{\sum_{i=1}^{N} \frac{L_i}{D_i^5}} \right)^{0.2}.$$

Using Nomograph

Step1: - Assume any flow rate, Q. The flow rate should generally selected within the range of flows on the hazen William nomograph.

Step2:- using nomograph line up Q and D for each section of the original. Series pipeline, read slope and compute HL=S*L for each section. Where L= the Length of the Section

Step 3:- Add up the head losses for all section in the series.

To determine Total head loss (HT) for the assumed discharge Q in the pipe line

Step 4:- compute an overall hydraulic gradients S'= Hl/Ĺ Where Ĺ = is the specified total length of the equivalent pipe

Step 5:- Enter the nomograph again with the assumed value of Q and Computed value S', read D of the equivalent pipe

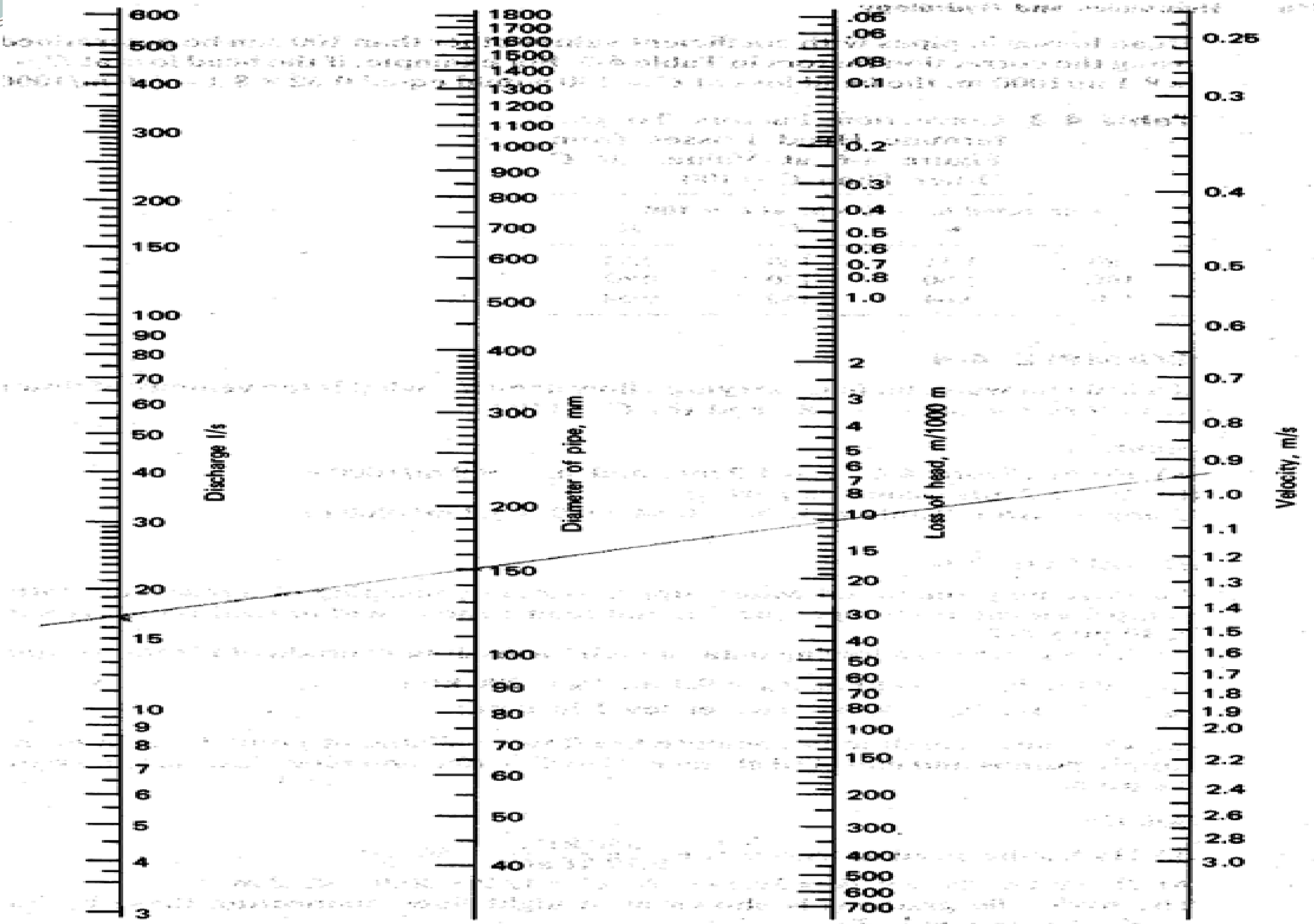
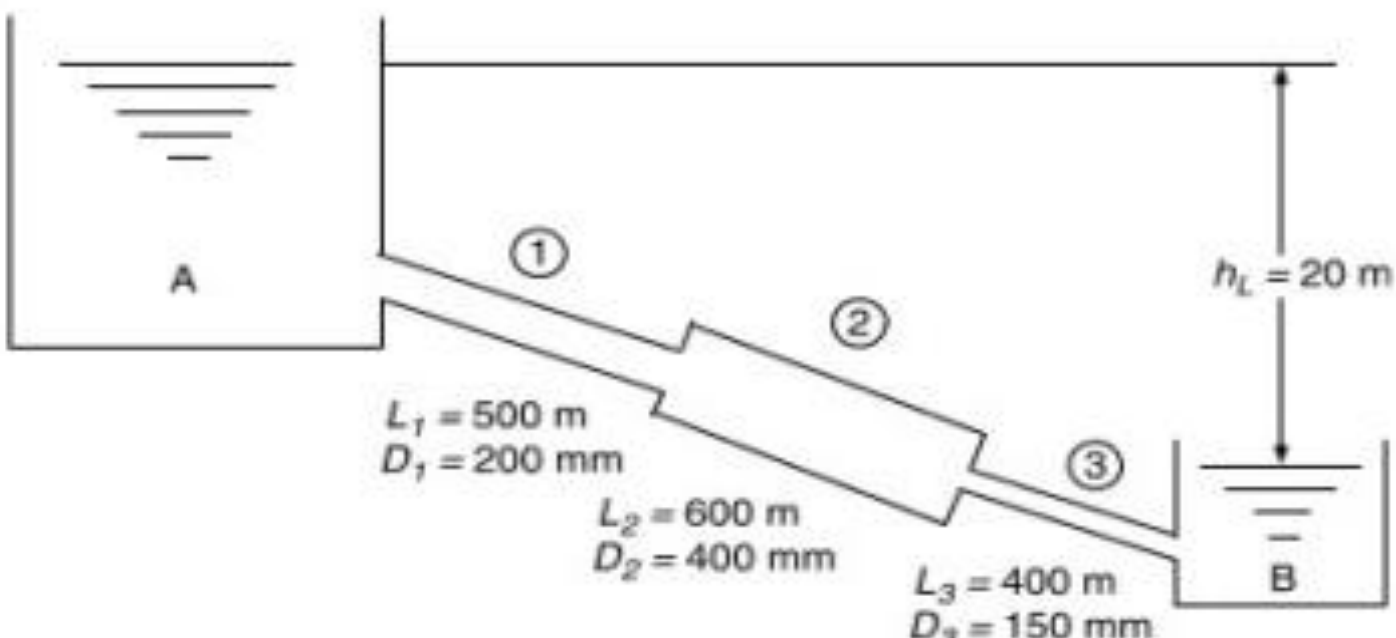


Figure 4-6 Nomograph for Hazen Williams Formula, Eq. 4-7, based on $C = 100$.

Example-1

An arrangement of three pipe in series between tank A and B is shown in figure. Determine the equivalent diameter and the corresponding flow using Darcy-weisbach equation and nomograph. .Assume Darcy-weisbach' friction factor f=0.02 and neglect entry any exist (minor)losses.



Substituting values,

$$D_e = \left(\frac{500 + 600 + 400}{\frac{500}{0.2^5} + \frac{600}{0.4^5} + \frac{400}{0.15^5}} \right)^{0.2} = 0.185\,\text{m}$$

and

$$K_e = \left[\frac{8fL_e}{\pi^2 g D_e^5} \right] = \frac{8 \times 0.02 \times 1500}{3.14^2 \times 9.81 \times 0.185^5} = 11{,}450.49\ \text{s}^2/\text{m}^5.$$

where $L_e = \Sigma L_i$ and K_e a pipe constant.

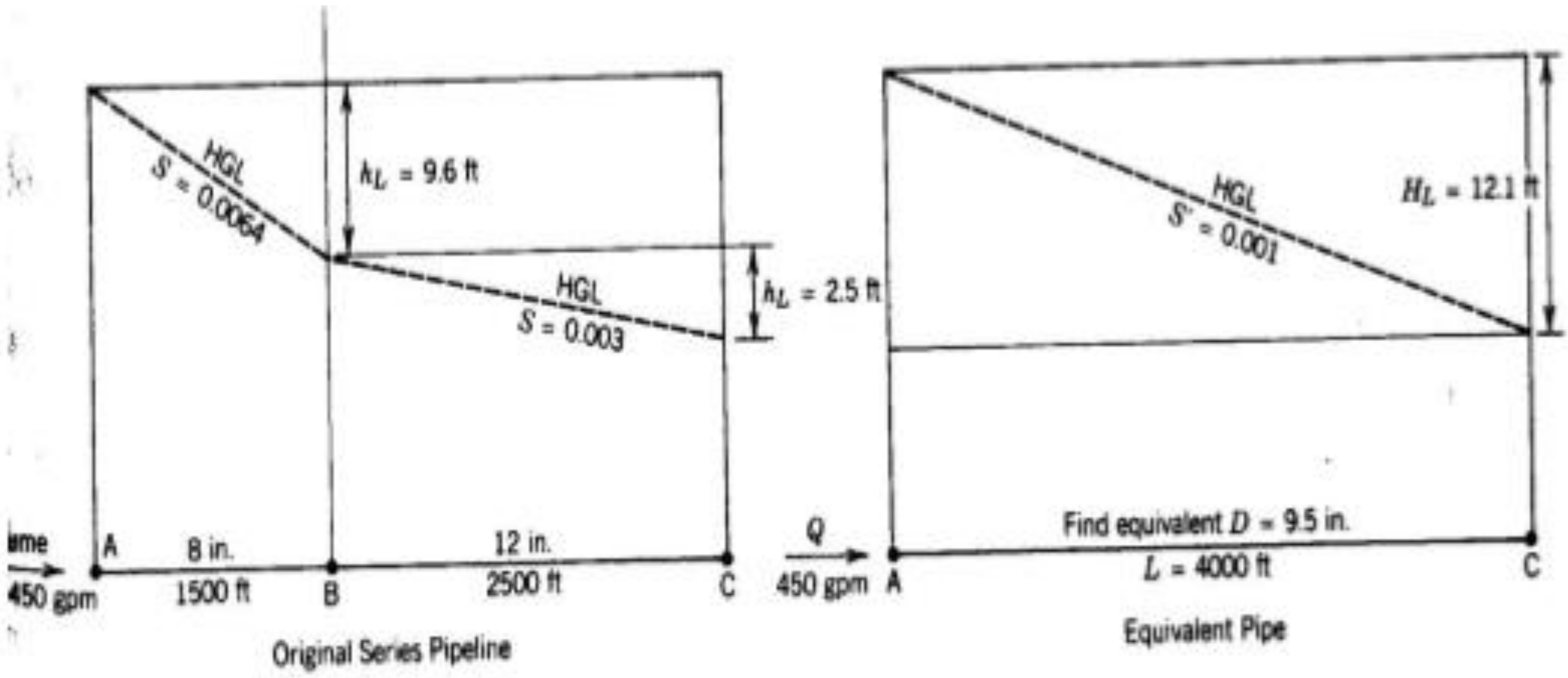
The discharge in pipe can be calculated:

$$Q = \left[\frac{h_L}{K_e} \right]^{0.2} = \left[\frac{20}{11{,}385.64} \right]^{0.2} = 0.042\,\text{m}^3/\text{s}.$$

The calculated equivalent pipe size 0.185 m is not a commercially available pipe diameter and thus has to be manufactured specially. If this pipe is replaced by a commercially available nearest pipe size of 0.2 m, the pipe discharge should be recalculated for revised diameter.

Example (2)

A series pipe line is given in the figure. It consists of 1500 ft of 8 in dia pipe from pt.A to B and 2500 ft 12 in diameter pipe from pt B to pt C. Determine the equivalent diameter of a single 4000 ft. long pipe line.

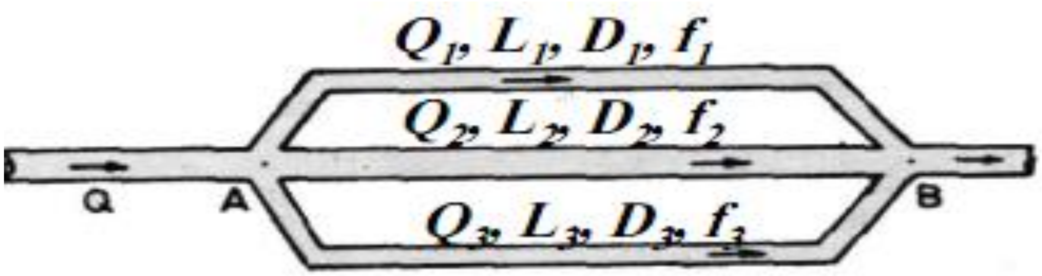


Solution

- Step 1:- Assume Q=25 l/s
- Step 2:- For section AB, using nomograph for Q= 25 l/s & D=8in S=0.0055
- Compute hl=S x L= 0.0055*1500 ft= 8.25ft
- For Section BC,using nomograph for Q==25l/s & D=12in S=0.0007
- Compute hl = S x L= 0.0007*2500ft=1.75ft
- Step 3:- Compute the total Head Loss Hl =8.25+1.75 ft= 10ft
- Step 4:- Compute the overall hydraulic gradient from A to C as S'= Hl/Ĺ = 10/4000 =0.0025
- Step 5:- By using nomograph, for Q=25 l/s & S=0.0025 D=9.6 in
- An equivalent pipe w/c has a dia. of 9.6 in and length of 4000ft.

2) Flow Through Parallel Pipes

If a main pipe divides into two or more branches and again join together downstream to form a single pipe, then the branched pipes are said to be connected in parallel(compound pipes). Points A and B are called nodes.



Discharge: $\boxed{Q = Q_1 + Q_2 + Q_3 = \sum_{i=1}^{3} Q_i}$

Head loss: the head loss for each branch is the same

$$\boxed{h_L = h_{f1} = h_{f2} = h_{f3}}$$

The pressure head at nodes A and B remains constant, meaning thereby that head loss in all the parallel pipes will be the same.

Using the Darcy–Weisbach equation and neglecting minor losses, the discharge Q_i in pipe i can be calculated as

$$Q_i = \pi D_i^2 \left(\frac{g D_i h_L}{8 f L_i} \right)^{0.5}.$$

Thus for N pipes in parallel,

$$Q = \pi \sum_{i=1}^{N} D_i^2 \left(\frac{g D_i h_L}{8 f L_i} \right)^{0.5}.$$

The discharge Q flowing in the equivalent pipe is

$$Q = \pi D_e^2 \left(\frac{g D_e h_L}{8 f L} \right)^{0.5},$$

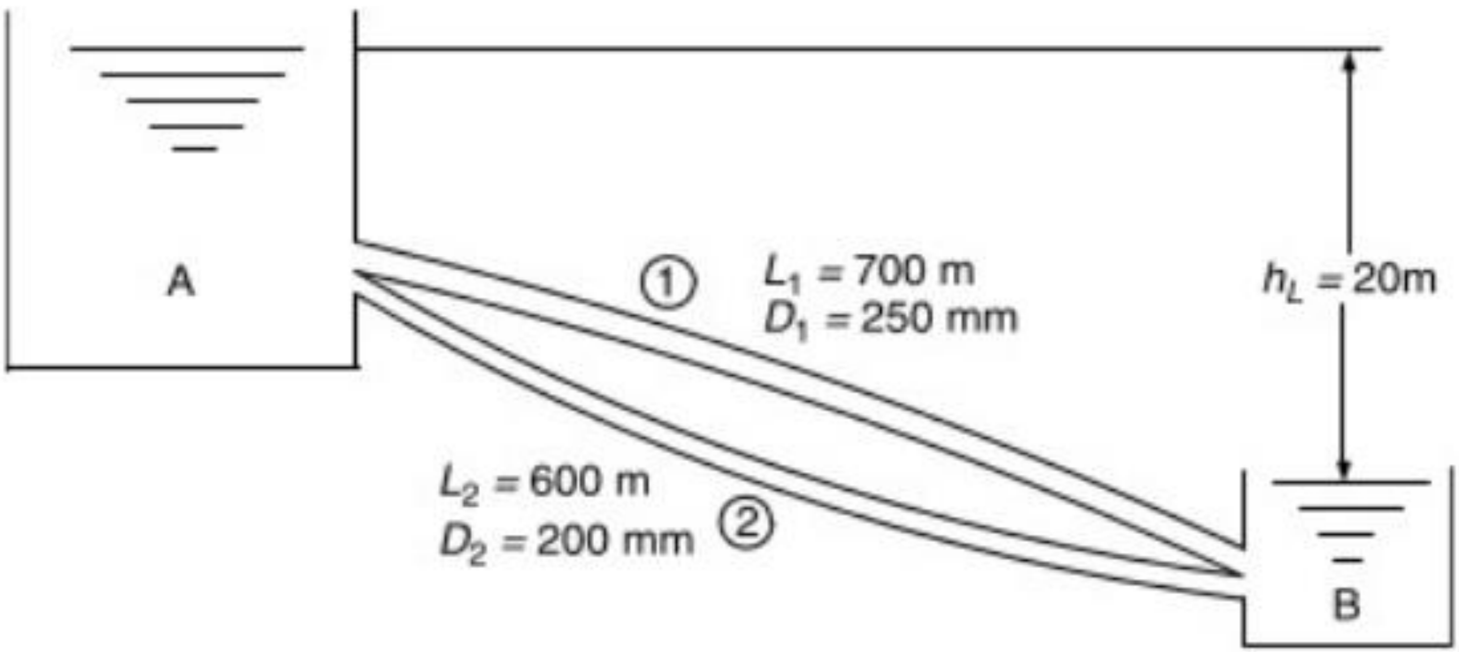
where L is the length of the equivalent pipe. This length may be different than any of the pipe lengths L_1, L_2, L_3, and so forth. Equating these two equations of discharge

$$D_e = \left[\sum_{i=1}^{N} \left(\frac{L}{L_i} \right)^{0.5} D_i^{2.5} \right]^{0.4}.$$

Using Nomograph

- ❖ Pipes in Parallel Procedures for determining an equivalent Pipe to replace the pipe w/c have parallel connection
- ☛ Step1:- Assume a total head loss, Hl across the loop.
- ☛ Step2:- Compute S= Hl /L for each Branches
- ☛ Step3:- Enter the Hazen Williams nomograph with D & S for each Branches, and Determine Q.
- ☛ Step4:- Compute the total flow rate at the Junction
- ☛ Step5:- Determine an overall hydraulic gradient S'=assumed Hl/ Ĺ Where Ĺ is the specified length equivalent pipe
- ☛ Step6:- using the nomograph with Q & S determines the equivalent D.

Example-1 For a given parallel pipe arrangement in Fig. calculate equivalent pipe diameter and corresponding flow. Assume Darcy–Weisbach's friction factor $f =$ 0.02 and neglect entry and exit (minor) losses. Length of equivalent pipe can be assumed as 500 m.



Solution. The equivalent pipe D_e can be calculated using Eq. (2.30):

$$D_e = \left[\sum_{i=1}^{N} \left(\frac{L}{L_i}\right)^{0.5} D_i^{2.5}\right]^{0.4}.$$

Substituting values in the above equation:

$$D_e = \left[\left(\frac{500}{700}\right)^{0.5} 0.25^{2.5} + \left(\frac{500}{600}\right)^{0.5} 0.20^{2.5}\right]^{0.4} = 0.283 \approx 0.28\,\text{m}.$$

Similarly, the discharge Q flowing in the equivalent pipe is

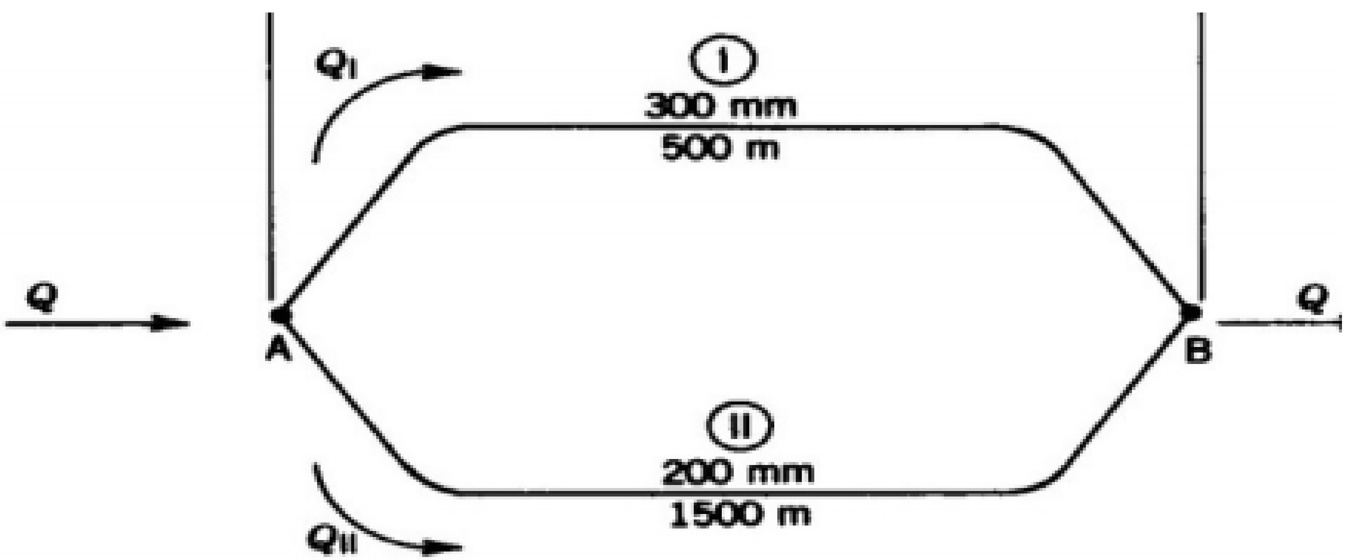
$$Q = \pi D_e^2 \left(\frac{g D_e h_L}{8 f L}\right)^{0.5}.$$

Substituting values in the above equation

$$Q = 3.14 \times 0.28 \times 0.28 \left(\frac{9.81 \times 0.28 \times 20}{8 \times 0.02 \times 500}\right)^{0.5} = 0.204\,\text{m}^3/\text{s}.$$

The calculated equivalent pipe size 0.28 m is not a commercially available pipe diameter and thus has to be manufactured specially. If this pipe is replaced by a commercially available nearest pipe size of 0.3 m, the pipe discharge should be recalculated for revised diameter.

Example 2 Two Pipelines are connected in parallel from junction A to Junction B as shown below. Branch AIB consists of 500m of 300mm diameters pipe, and branch AIIB consists of 1500m of 200mm pipe. Determine the equivalent diameter of a single 500m long pipeline from A to B that could replace the given loop.



Original Parallel System

Solution

- Step 1:- Assume Hl= 10m
- Step 2:- For Branch AIB, S=10/500= 0.02 For Branch AIIB, S= 10/1500 = 0.0067
- Step 3:- For Branch AIB, using Hazen-William's
- monograph with D=300mm & S=0.02 read Q1 =143L/S For branch AIIB, from monograph, with D=200mm and S=0.0067, read Q11=27L/S
- Step 4:- The total flow rate into Junction A, Q=Q1+Q11 = 143+27 = 170L/S
- Step 5:- The overall hydraulic gradients S==10/500 = 0.02 Step 6:- From monograph, with Q=170L/S and s=0.02, read D= 320mm
- An equivalent pipe for the problem w/c have D=320 mm with L=500m.

C) Hardy- crosses method

- This method is applicable to closed-loop pipe networks.
- The outflows from the system are assumed to occur at the nodes (NODE: end of each pipe section).
- This assumption results in uniform flow in the pipelines.
- The Hardy-Cross analysis is based on the principles that
- 1 At each junction, the total inflow must be equal to total outflow.

$$\sum Q_{inflow} = \sum Q_{outflow} \qquad \text{(flow continuity criterion)}$$

- 2. Head balance criterion: algebraic sum of the head losses around any closed-loop is zero.

$$\sum HL_{clockwise\ direction} = \sum HL_{counter\ clockwise\ direction}$$

3. Flows in a clockwise direction are considered to be positive (+) and flows in CCW direction are considered to be negative(-)

4. Head losses from CW flows are considered to be positive (+), Head losses from CCW flows are negative (-).

- For a given pipe system, with known junction outflows, the Hardy-Cross method is an iterative procedure based on initially estimated flows in pipes.
- Estimated pipe flows are corrected with iteration until head losses in the clockwise direction and in the counter clockwise direction are equal within each loop.
- The corrections applied to assumed flow rate are determining from the following formula,

$$\Delta Q = -\frac{\Sigma h_L}{1.85 \times \Sigma(h_L/Q)}$$

here ΔQ = flow correction

Σh_L = sum of head losses

$\Sigma h_L/Q$ = sum of h_L/Q ratios for each pipeline in a loop

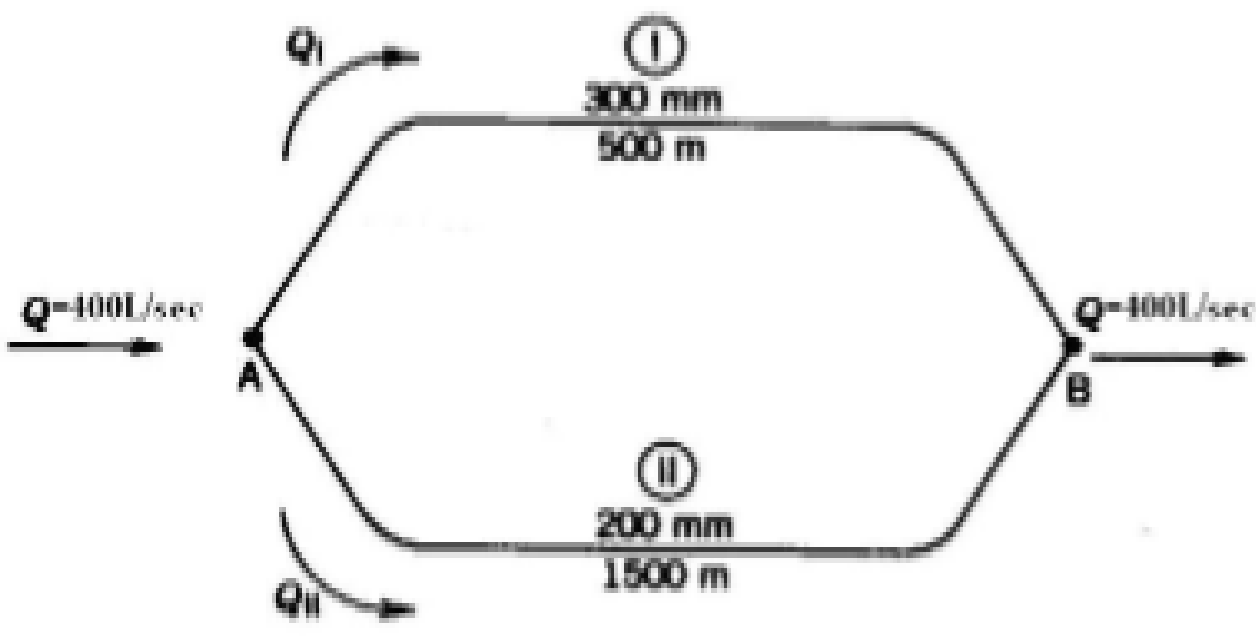
Procedures

- Step 1:- For each pipe in the network, assume a flow rate and flow direction of the flow going into a pipe junction must equal to the total flow going out of that junction.
- Step 2:- Using nomograph, designing S and hL =S*L for each pipe in the loop .Also, compute hL/Q for each pipe.
- Step 3:- Compute ∑ hL and ∑ hL/Q.The hL/Q terms are always positive
- Step 4:- Compute ΔQ and add ΔQ to the flow in each pipe of that loop.

- Step 5:- Repeat step 2 through 4 for an adjacent loop in the network. If one the pipe in the first loop is also part of the adjacent, use the previously corrected flow in the common pipe but the algebraic sign of the flows and head losses in common pipe change depending on w/c loop is being evaluated.
- Step 6:- Alternately repeat step2 through 4 for each loop in the network until the corrections obtained become sufficiently small.

Example-1

1. Consider the single loop of parallel pipes shown below. If a flow of 400 L/S enters the loop at junction A, what will be the flow rate Q1 in branch AIB and QII in branch AIIB? Use Hazen – Williams formula (C= 100) . If the pressure Head at point A is 45m, find the pressures at points A & B. Assume all pipe junctions are at the same elevation.



Solution

From the principle of continuity (the flow entering into a junction must equal to out from that junction)

- I.e. Q1+ QII =400L/S
- Let's assume Q1=300 L/S

$$Q = 0.278\, CD^{2.63} S^{0.54}, S = \frac{h_f}{L}$$

1 st trial

Pipe	Dia(mm)	L(m)	Q(l/s)	S(m/m)	hl(m)	hl/Q	∑hl	∑hl/Q	k	ΔQ=-∑hl/1.85*(∑hl/Q)
AIB	300	500	300	0.08	40	0.13333	-74	1.2733	1.85	31
AIIB	200	1500	-100	-0.076	-114	1.14	-74	1.2733	1.85	31

2nd trial

Pipe	Dia(mm)	L(m)	Q(l/s)	S(m/m)	hl(m)	hl/Q	∑hl	∑hl/Q	k	ΔQ=-∑hl/1.85*(∑hl/Q)
AIB	300	500	331	0.096	48	0.15	-12	1.01	1.85	6.39
AIIB	200	1500	-69	-0.04	-60	0.87	-12	1.01	1.85	6.39

3rd trial

Pipe	Dia(mm)	L(m)	Q(l/s)	S(m/m)	hl(m)	hl/Q	∑hl	∑hl/Q	k	ΔQ=-∑hl/1.85*(∑hl/Q)
AIB	300	500	337	0.099	49.5	0.15	1.5	0.91	1.85	-1
AIIB	200	1500	-63	-0.032	-48	0.76	1.5	0.91	1.85	-1

4th trial

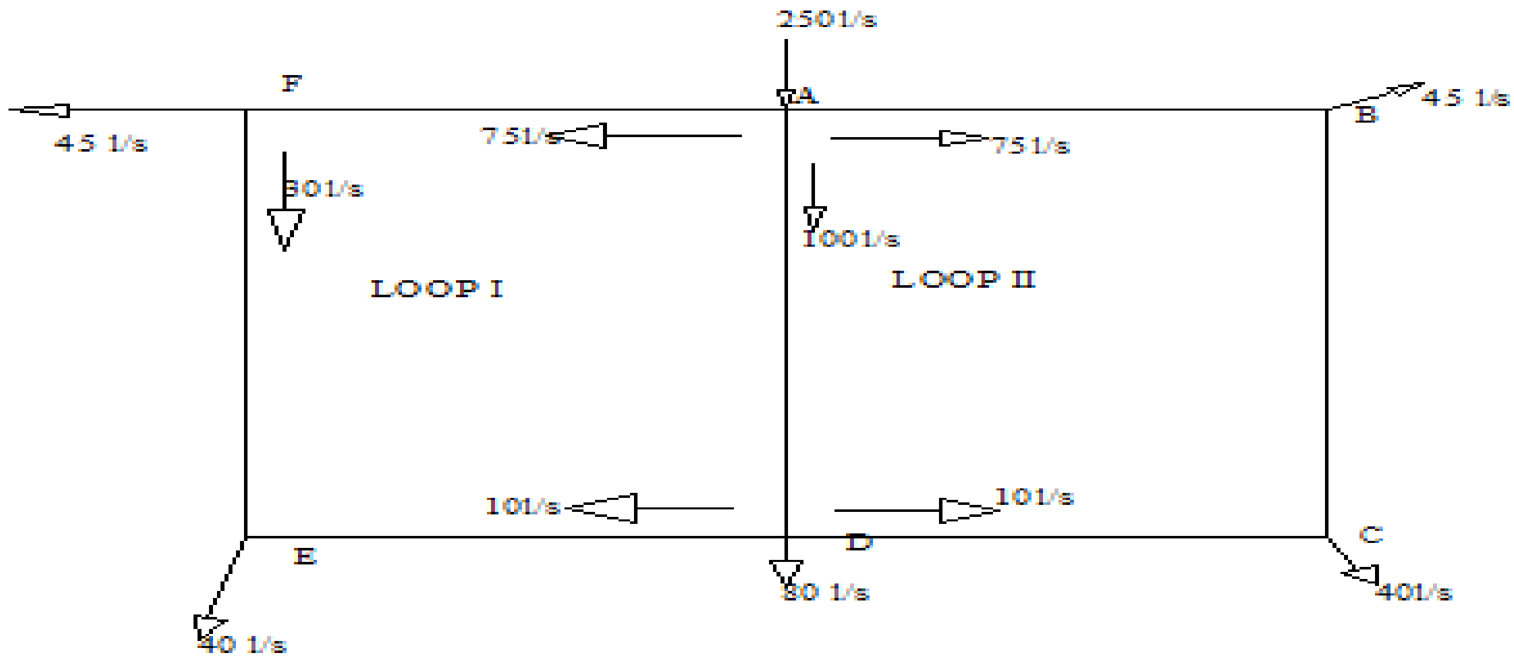
Pipe	Dia(mm)	L(m)	Q(l/s)	S(m/m)	hl(m)	hl/Q	∑hl	∑hl/Q	k	ΔQ=-∑hl/1.85*(∑hl/Q)
AIB	300	500	336	0.099	49.5	0.15	0	0.92	1.85	0
AIIB	200	1500	-64	-0.033	-49.5	0.77	0	0.92	1.85	0

Point A $P=\gamma * h = 45*10 =$ 450

point B H=(45+49.5) 94.5*10 =945

Example-2

Find the flow distribution in the gravity supply system through the following pipe network shown below. Use Hazen – Williams formula (C= 100) . If the pressure at point A is 490.5 KPa, find the pressures at points B & C. Assume all pipe junctions are at the same elevation.



1 st trial for loop-I

Pipe	Dia(mm)	L(m)	Q(l/s)	S(m/m)	hl(m)	hl/Q	Σhl	Σhl/Q	k	ΔQ=-∑hl/1.85*(∑hl/Q)	Q1II=Q1+ΔQ
AD	250	1000	100	0.0255	25.48	0.2548	-3.93	1.78742	1.85	1.189	97.43
DE	150	2000	10	0.0043	8.63	0.8628	-3.93	1.78742	1.85	1.189	11.19
EF	200	1000	-30	-0.0081	-8.13	0.2709	-3.93	1.78742	1.85	1.189	-28.81
FA	250	2000	-75	-0.0150	-29.92	0.3989	-3.93	1.78742	1.85	1.189	-73.81

1 st trial for loop-II

Pipe	Dia(mm)	L(m)	Q(l/s)	S(m/m)	hl(m)	hl/Q	Σhl	Σhl/Q	k	ΔQ=-∑hl/1.85*(∑hl/Q)	Q1II=Q1+ΔQ
AB	250	2000	75	0.0150	29.916	0.399	-49.60	7.14103	1.85	3.755	78.75
BC	200	1000	30	0.0081	8.127	0.271	-49.60	7.14103	1.85	3.755	33.75
CD	100	2000	-10	-0.0311	-62.164	6.216	-49.60	7.14103	1.85	3.755	-6.25
DA	250	1000	-100	-0.0255	-25.483	0.255	-49.60	7.14103	1.85	3.755	-96.25

2nd trial for loop-I

Pipe	Dia(mm)	L(m)	Q(l/s)	S(m/m)	hl(m)	hl/Q	Σhl	Σhl/Q	k	ΔQ=-∑hl/1.85*(∑hl/Q)	Q2I=Q1+ΔQ
AD	250	1000	97.435	0.0243	24.29	0.2492	-1.68	1.85394	1.85	0.488	97.14
DE	150	2000	11.189	0.0053	10.62	0.9495	-1.68	1.85394	1.85	0.488	11.68
EF	200	1000	-28.811	-0.0075	-7.54	0.2617	-1.68	1.85394	1.85	0.488	-28.32
FA	250	2000	-73.811	-0.0145	-29.04	0.3935	-1.68	1.85394	1.85	0.488	-73.32

2nd trial for loop-II

Pipe	Dia(mm)	L(m)	Q(l/s)	S(m/m)	hl(m)	hl/Q	Σhl	Σhl/Q	k	ΔQ=-Σhl/1.85*(Σhl/Q)	Q2II=Q1+ΔQ
AB	250	2000	78.755	0.0164	32.749	0.416	-7.42	5.12736	1.85	0.783	79.54
BC	200	1000	33.755	0.0101	10.111	0.300	-7.42	5.12736	1.85	0.783	34.54
CD	100	2000	-6.2453	-0.0130	-25.997	4.163	-7.42	5.12736	1.85	0.783	-5.46
DA	250	1000	-97.435	-0.0243	-24.285	0.249	-7.42	5.12736	1.85	0.783	-96.65

3rd trial for loop-I

Pipe	Dia(mm)	L(m)	Q(l/s)	S(m/m)	hl(m)	hl/Q	Σhl	Σhl/Q	k	ΔQ=-Σhl/1.85*(Σhl/Q)	Q3I=Q1+ΔQ
AD	250	1000	97.14	0.0241	24.15	0.2486	-0.35	1.88249	1.85	0.099	97.18
DE	150	2000	11.68	0.0057	11.50	0.9847	-0.35	1.88249	1.85	0.099	11.78
EF	200	1000	-28.32	-0.0073	-7.31	0.2579	-0.35	1.88249	1.85	0.099	-28.22
FA	250	2000	-73.32	-0.0143	-28.69	0.3913	-0.35	1.88249	1.85	0.099	-73.22

3rd trial for loop-II

Pipe	Dia(mm)	L(m)	Q(l/s)	S(m/m)	hl(m)	hl/Q	Σhl	Σhl/Q	k	ΔQ=-Σhl/1.85*(Σhl/Q)	Q3II=Q1+ΔQ
AB	250	2000	79.54	0.0167	33.354	0.419	-0.54	4.68745	1.85	0.062	79.60
BC	200	1000	34.54	0.0105	10.549	0.305	-0.54	4.68745	1.85	0.062	34.60
CD	100	2000	-5.46	-0.0101	-20.289	3.714	-0.54	4.68745	1.85	0.062	-5.40
DA	250	1000	-97.14	-0.0241	-24.150	0.249	-0.54	4.68745	1.85	0.062	-97.08

Pipe	Dia(mm)	L(m)	Q(l/s)	S(m/m)	hl(m)	hl/Q	Σhl	Σhl/Q	k	ΔQ=-∑hl/1.85*(∑hl/Q)	Q4I=Q1+ΔQ
AD	250	1000	97.178	0.0242	24.17	0.2487	-0.03	1.88848	1.85	0.008	97.18
DE	150	2000	11.777	0.0058	11.68	0.9918	-0.03	1.88848	1.85	0.008	11.78
EF	200	1000	-28.223	-0.0073	-7.26	0.2572	-0.03	1.88848	1.85	0.008	-28.22
FA	250	2000	-73.223	-0.0143	-28.62	0.3908	-0.03	1.88848	1.85	0.008	-73.22

4th trial for loop-II

Pipe	Dia(mm)	L(m)	Q(l/s)	S(m/m)	hl(m)	hl/Q	Σhl	Σhl/Q	k	ΔQ=-∑hl/1.85*(∑hl/Q)	Q4II=Q1+ΔQ
AB	250	2000	79.60	0.0167	33.402	0.420	-0.05	4.65249	1.85	0.005	79.60
BC	200	1000	34.60	0.0106	10.584	0.306	-0.05	4.65249	1.85	0.005	34.60
CD	100	2000	-5.40	-0.0099	-19.866	3.678	-0.05	4.65249	1.85	0.005	-5.40
DA	250	1000	-97.178	-0.0242	-24.167	0.249	-0.05	4.65249	1.85	0.005	-97.17

5th trial for loop-I

Pipe	Dia(mm)	L(m)	Q(l/s)	S(m/m)	hl(m)	hl/Q	Σhl	Σhl/Q	k	ΔQ=-∑hl/1.85*(∑hl/Q)	Q5I=Q1+ΔQ
AD	250	1000	97.180	0.0242	24.17	0.2487	0.00	1.88895	1.85	0.001	97.18
DE	150	2000	11.785	0.0058	11.69	0.9924	0.00	1.88895	1.85	0.001	11.79
EF	200	1000	-28.215	-0.0073	-7.25	0.2571	0.00	1.88895	1.85	0.001	-28.21
FA	250	2000	-73.215	-0.0143	-28.61	0.3908	0.00	1.88895	1.85	0.001	-73.21

5th trial for loop-II

Pipe	Dia(mm)	L(m)	Q(l/s)	S(m/m)	hl(m)	hl/Q	Σhl	Σhl/Q	k	ΔQ=-Σhl/1.85*(Σhl/Q)	Q5II=Q1+ΔQ
AB	250	2000	79.60	0.0167	33.406	0.420	0.00	4.64938	1.85	0.000	79.60
BC	200	1000	34.60	0.0106	10.587	0.306	0.00	4.64938	1.85	0.000	34.60
CD	100	2000	-5.40	-0.0099	-19.829	3.675	0.00	4.64938	1.85	0.000	-5.40
DA	250	1000	-97.18	-0.0242	-24.168	0.249	0.00	4.64938	1.85	0.000	-97.18

Therefore, the flow distribution will be

Pipe	Dia(mm)	L(m)	Q(l/s)	S	hl(m)
AD	250	1000	97.18	0.0242	24.17
DE	150	2000	11.78	0.0058	11.69
EF	200	1000	-28.22	-0.0073	-7.25
FA	250	2000	-73.22	-0.0143	-28.61
AB	250	2000	79.60	0.0167	33.41
BC	200	1000	34.60	0.0106	10.59
CD	100	2000	-5.40	-0.0099	-19.83
DA	250	1000	-97.18	-0.0242	-24.17

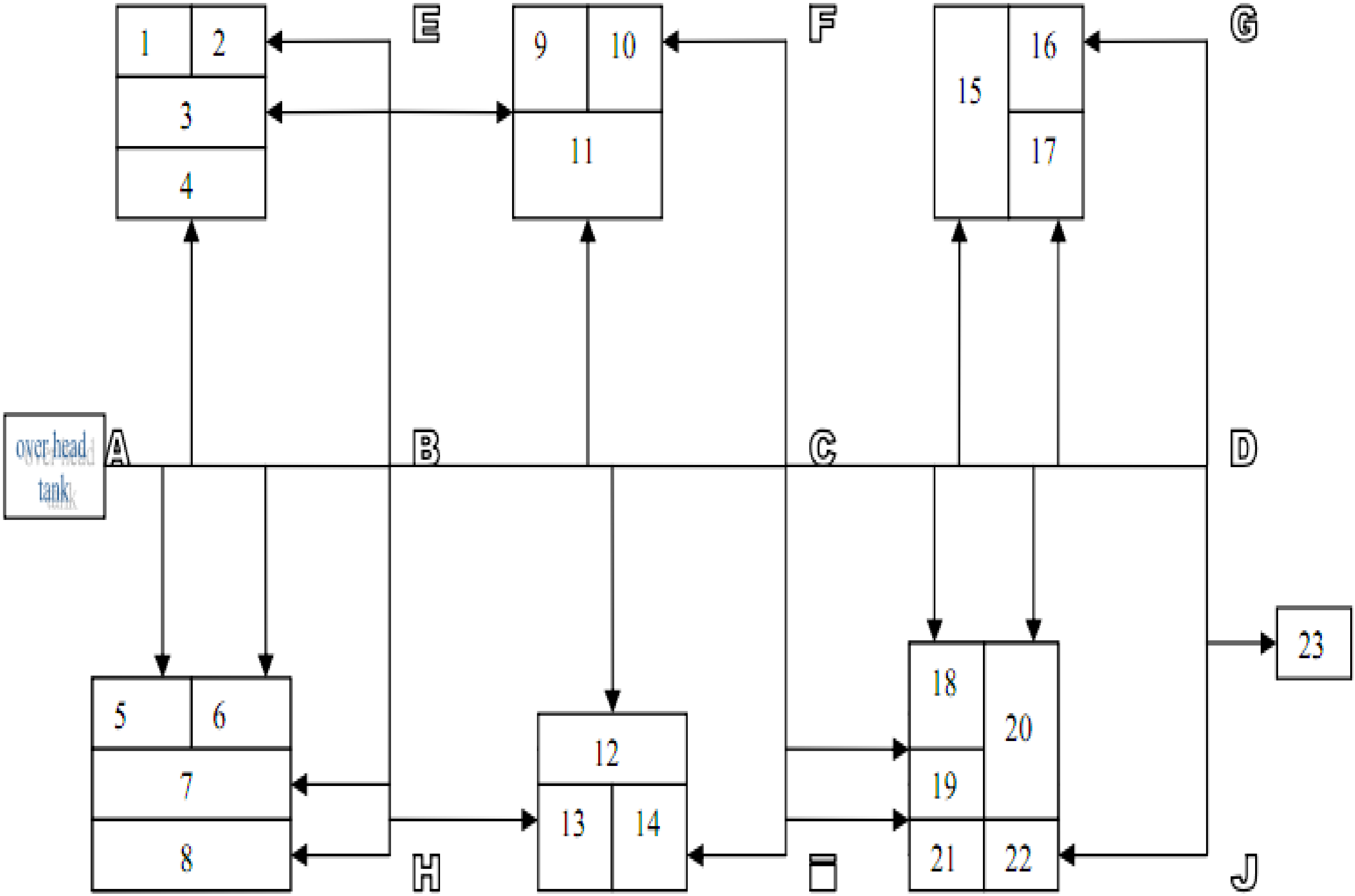
$$P=\gamma * h$$

Point	Head(m)	Pressur(kpa)
A	50	490.5
B	83.41	818.2
C	93.99	922.1
D	74.17	727.6
E	85.86	842.3
F	78.61	771.2

Assignment-Three

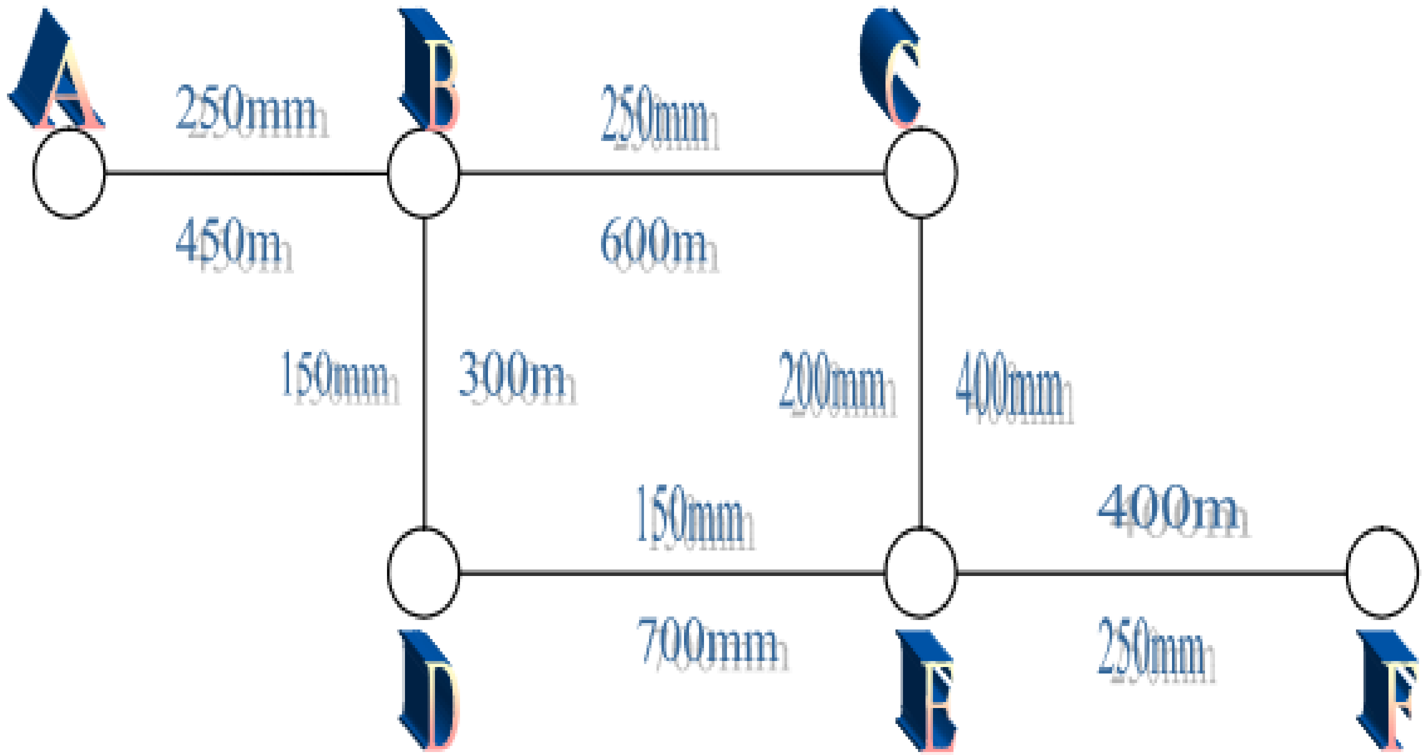
1) A typical layout of pipes in dead end pattern is given below. The rate of supply is 180 lit/cap/day and the min. pressure head of the water is 15m. The reduced levels of the bottom of the storage tanks and points A, B, C & D are 150, 130, 129, 131 & 128m. Design suitable sizes of pipes AB, BC&CD, lengths are 300m, 400m and 500m respectively.

Block no	popn	block no	popn	block no	popn	block no	popn
1	200	7	400	13	600	19	500
2	500	8	400	14	600	20	100
3	600	9	300	15	700	21	700
4	600	10	500	16	600	22	200
5	700	11	500	17	600	23	500
6	700	12	700	18	800		



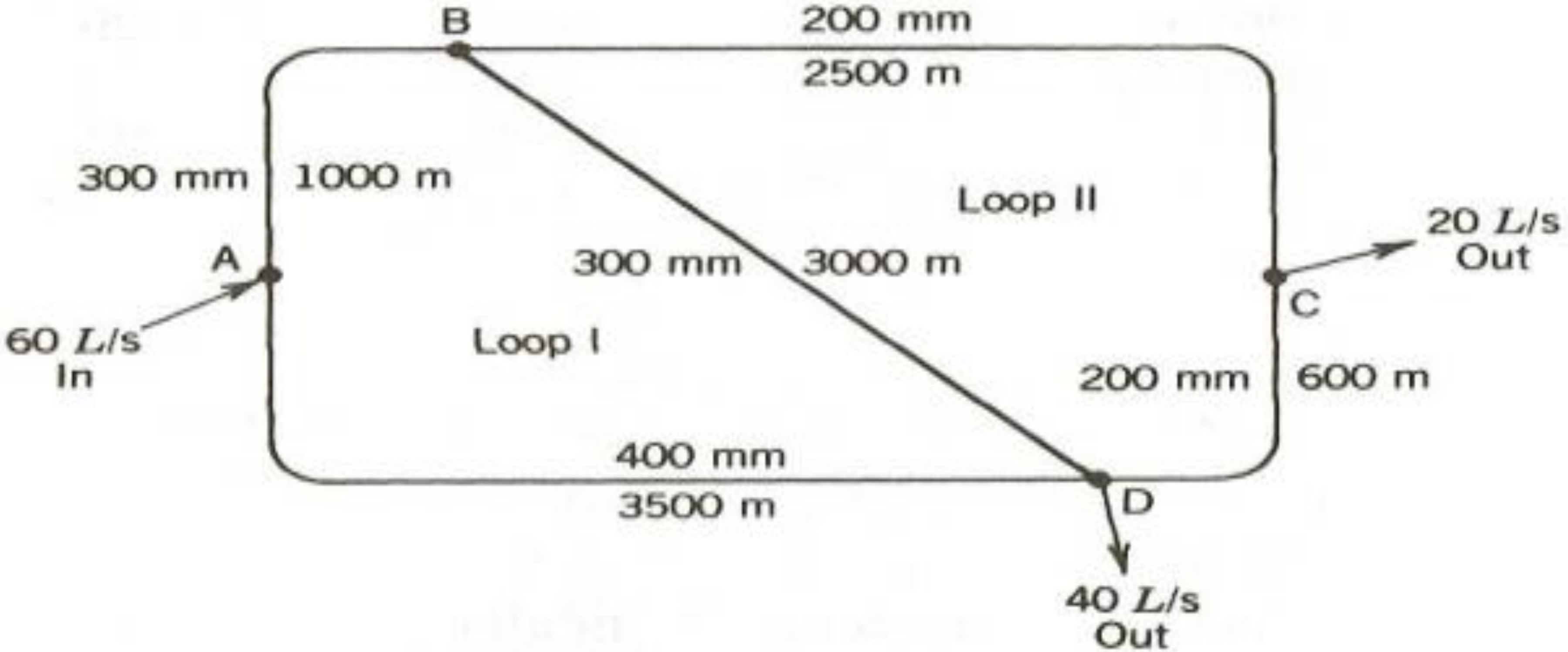
1
2
3
4
E
9
10
11
F
15
16
17
G
over head tank
A
B
C
D
5
6
7
8
H
12
13
14
18
19
20
21
22
J
23

2) Find a single pipe; 2000m long, w/c is hydraulically equivalent to the lines given below.



3) A water distribution system has been skeletonzed and reduced to the two-loop network even below. A flow rate of distribution in the gravity supply system is pumped into the network at point A, and two major water withdrawal points, C and D, discharge 20 l/s and 40 l/s, respectively. Find the flow through the following pipe network shown below. Use Hazen – Williams formula (C= 100) . If the pressure at point A is 420 KPa, find the pressures at points B, C & D. Assume all pipe junctions are at the same elevation.

Pipe	Diameter, mm	Length, m
AB	300	1000
BC	200	2500
BD	300	3000
AD	400	3500
DC	200	600



Urban drainage systems

- ✓ Introduction
- ✓ Types, Sources and Quantity of Waste Water
- ✓ Hydraulic Design of Sewers and Storm Water Drains
- ✓ Sewer materials
- ✓ Sewer Appurtenances
- ✓ Septic Tanks

Introduction

- ☛Urban drainage systems deal with both wastewater and storm water.
- ☛**What is wastewater? It** Is all liquid waste generated from different sources including run-off.
- ☛**Storm water (surface runoff)** is the second major urban flow of concern to the drainage engineer.

Two main categories:

1. Sanitary wastewater/dry weather flow
2. Storm water/wet weather flow

Some important terms

- **Sewers** – underground pipes that carry sewage to the point of discharge and disposal
- **Soil pipe** – pipes used to carry waste from water closet and urinals
- **Sewerage system** – entire system used for collection/carrying/disposal of sewage by water carriage system
- **Refuse** – all waste /liquid, solid and gas
- **Garbage** – dry refuse
- **Rubbish** – sundry combustible (paper/furniture – office/residence)
- **Sullage (gray water)**– liquid discharge (bath-rooms, kitchens, washing places, wash basins etc.)
- **Sewage** – sullage and foul discharges from water closet, hospitals… (sewage and trade waste / closed conduits

TYPES AND SOURCES OF WASTEWATER

Sanitary wastewater/dry weather flow:

- The sum total of domestic and industrial wastewater.
- Modern approach name-sewage
- Industrial sewage consists of liquid wastes originating from the industrial processes of various industries
- Domestic sewage consists of liquid wastes originating from urinals, latrines, bath-rooms, kitchen sinks, wash basins, etc. of the residential, commercial or institutional buildings.
- This sewage is generally extremely foul, because of the presence of human excreta in it.
- Ground water filtration also contributes significant amount of wastewater to sewage in the sewer.

Storm water/wet weather flow:

- The run-off resulting from the rain storms.
- modern approach is to call it storm drainage or simply drainage, so as to differentiate it from sewage, which is much fouler as compared to drainage, and requires treatment before disposal

Estimating Quantity of Dry-whether flow

- The sewage discharge which has to pass through a sewer must be estimated as correctly as possible;
    - otherwise the sewers may either prove to be inadequate, resulting in their overflow, or
    - may prove to be of too much of size, resulting in unnecessary wasteful investments.
- The quantity of sewage affected by the following factors:

1. Rate of water supply
2. Population growth
3. Types of area to be served (industrial, commercial, residential,…)
4. Infiltration/exfiltration of ground water to sewers

Generally,

$$Q_{DWF} = 75\% - 80\%\ of\ total\ water\ supply$$

- The remaining 20%-25% is quantity of water which do not join the sewer system. They includes:
    - The water used for drinking
    - Water used for clothe washing-evaporated during drying
    - Water used for sprinkling and gardening of roads, parks gardens and others.

Variation in rate of sanitary sewage

- Rate of sewage flow is not constant
- It varies hour to hour, day to day and season to season.
- Therefore, maximum and minimum flow should be considered during design of sewer pipes, it should have the capacity to carry max. flow and there should be no deposition in the sewers at min. flow.

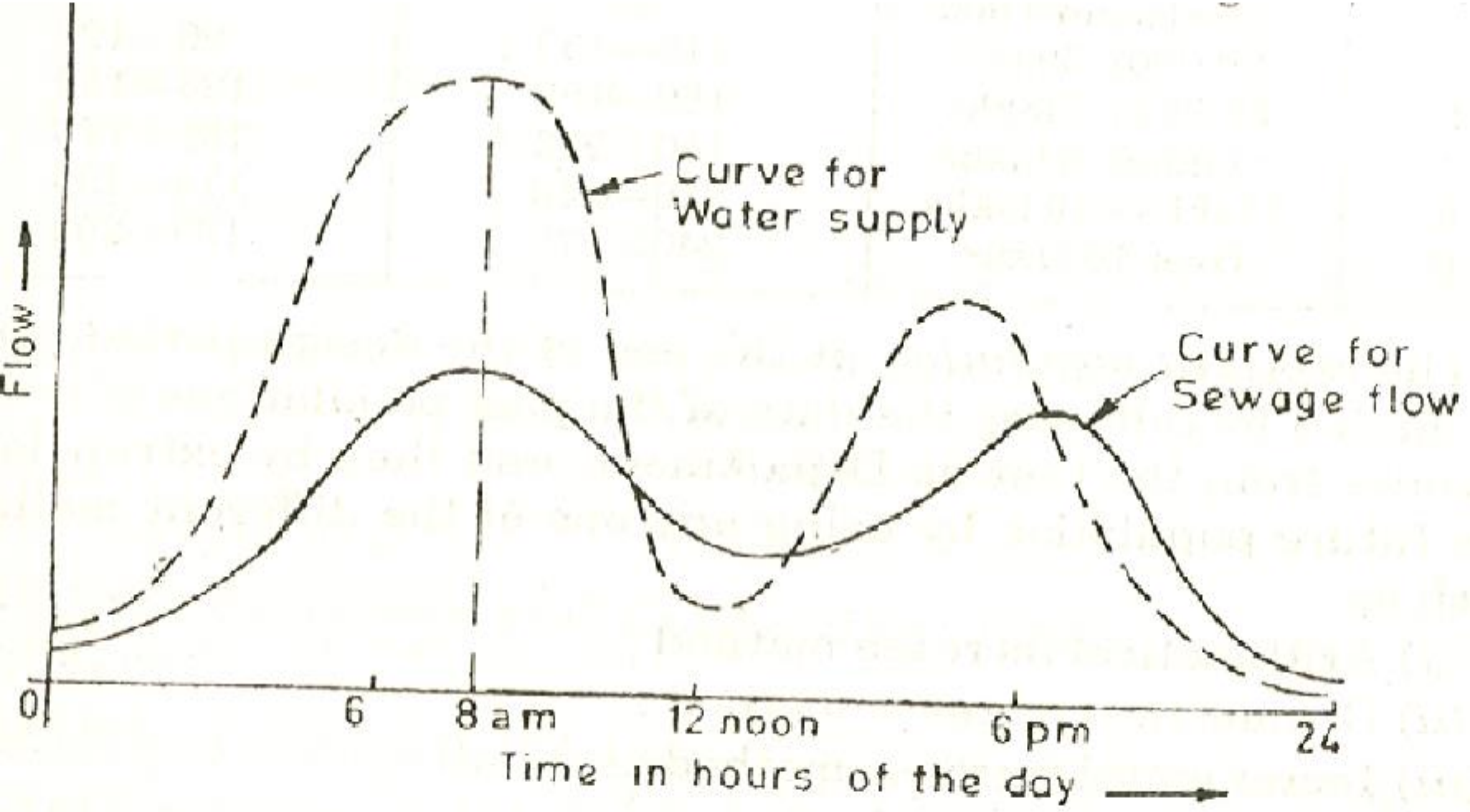


Fig. Hourly variation of sewage flow compared to that of water supply

Estimating the Peak storm water Discharge

- ✓ The quantity of storm water is affected by:
  i. Catchment area
  ii. Slope of the catchment area
  iii. Permeability of the ground
  iv. Intensity of the rainfall
  v. Duration of the rainfall
  vi. Compactness of catchment area
  vii. climatic conditions
  viii. Extent of vegetation
  ix. others

There are two methods to estimate quantity of drainage:

1. **Rational method**

$$Q = \frac{K.i.A}{36}$$

Where

Q = peak rate of runoff in cumecs (m^3/s).

K = coefficient of runoff

A = the catchment area in hectares

i = Critical Rainfall intensity of the design frequency i.e. the rainfall intensity during the critical rainfall duration equal to time of concentration in cm/hr.

- *Coefficient of runoff (K)*: the impervious factor of runoff, and the ratio of runoff to precipitation.
- The value of K increases as the imperviousness of the area increases.
    - In other words the quantity of run-off is high.

Fig. Impervious catchment

- If a given catchment area consists of various types of surfaces for which different runoff coefficient applicable, then the average runoff coefficient for whole area calculated as:

$$K_{avg} = \frac{A_1K_1 \ + \ A_2K_2 \ + \ \ldots\ldots + \ A_nK_n}{A_1 + A_2 + \cdots \ldots\ldots An}$$

Where, A_1, A_2,….A_n = areas of different surface of the catchment area.

K_1, K_2,…K_n = corresponding runoff coefficient for different surface area

Table 1: Values of Run-off coefficient (K) for various Surfaces

S.No	Type of surface	Value of K
1	Water tight roof surface	0.7 - 0.95
2	Asphalt pavement in good order	0.85- 0.90
3	Stone, brick, wood-block pavement with cemented joints	0.75 -0.85
4	same as above with un cemented joints	0.5 -0.7
5	Water bond macadam roads	0.25 -0.6
6	Gravel roads and walks	0.15 - 0.3
7	unpaved streets and vacant lands	0.1 - 0.3
8	Parks, Lawns, gardens, meadows, etc	0.05 - 0.25
9	Wooden lands	0.01 - 0.20

Intensity of Rainfall (i): is the rate at which it is falling.

- Certain empirical equations have been suggested for determining rainfall intensity, as given below:

- For places where heavy and frequent rains occur, and gives an intensity for 5 years frequency.

$$i = \frac{343}{T + 18}$$

Where:

T = concentration time (minutes) = the inlet time or overland flow time or time of equilibrium (T_i) plus the channel flow time or gutter flow time (T_r).

- For rains having frequency of 10 years, the equation suggested is:

$$i = \frac{38}{\sqrt{T}}$$

- For rains having frequency of 1 year, the equation suggested is:

$$i = \frac{15}{T^{0.62}}$$

- U.S. ministry of health formula

$i = \frac{25.4a}{t+b}$, where a and b are constant

  - If duration of storm is 5-20minutes then, a=30 and b=10
  - If duration of storm is 20-200minutes then, a=40and b=20

- And others……

2. Empirical formula

- For the design of drains having larger catchments (say above 400 hectares or so),
- Various empirical formulas have been suggested by various investigators; based on local conditions only, and can be adopted only when certain specific requirements are specified.

- Burkli-Ziegler formula, $Q_p = \frac{1}{455} k' p A \sqrt{\frac{S}{A}}$

Where Q_p = the peak runoff in cumecs

K' = the runoff coefficient depending up on the permeability of the surface and having an average value of 0.7 in cumecs.

P = the maximum rain fall intensity over the entire area and is usually taken as 2.5- 7.5cm/hr.

A= the drainage area in hectares

S_0 = the slope of the ground surface in meters per thousand meters

Dickens's formula

$$Q_p = CM^{3/4}$$

M = Catchment area in sq. Km , C = a constant

Examples

1. The drainage area of one sector of a town is 12 hectares. The classification of the surface of this area is as follows:

Percent of total Surface area	Type of surface	Coefficient of run-off
20%	Hard pavement	0.85
20%	Roof surface	0.8
15%	unpaved street	0.2
30%	Garden and Lawn	0.2
15%	Wooded areas	0.15

If the time of concentration for the area is 30 minutes, find the maximum runoff.

Use $i = \frac{900}{t+60}$ to calculate rainfall intensity.

Where, i is the rainfall intensity, generally expressed in mm/hr and t is the concentration time in minutes.

Solution:

Using rational formula, we have

$$\boldsymbol{Q} = \frac{\mathrm{K.\,i.\,A}}{36}$$

First average **K** value = **0.4425**

Then **i** value, $i = \frac{900}{t+60} = \frac{900}{30+60} = 10mm/hr$ = **1cm/hr**

$Q = \frac{0.4425 * 1cm/hr * 12hectares}{36}$ = **0.1475m³/s**

Exercise

2. A certain district of a city has a projected population of 50,000 residing over an area of 100hectares. Find the design discharge for the sewer line for the following data:
   - Rate of water supply = 150l/c/d
   - Average coefficient of run-off for entire area is 0.30
   - Time of concentration is 50minutes

The sewer line is to be designed for a flow equivalent to the WWF plus twice the DWF. Assume 75% of water supply reaches in sewer as wastewater.

Use $\frac{25.4a}{t+b}$ to calculate rainfall intensity, t is minutes.

WASTEWATER COLLECTION

- Transporting waste water from point of generation to its treatment places and ultimately to its disposal site.
- Depending upon the type of waste, two systems may be employed for its collection, conveyance and disposal:

a. ***Conservancy system:***

- is an old system
- Wastes (night soil, garbage etc.) are collected separately in vessels or deposited in pools or pits and then removed periodically at least once in 24hours.
- In this system, the waste products are generally buried underground, which may sometimes pollute the city's water supplies.

b) *Water carriage system.*

- In this system, the waste water is carried with the help of water through underground pipes (sewers).
- The water carriage system is more hygienic, because in this system, the society's wastes have not to be collected and carried in buckets or carts, as is required to be done in the conservancy system.
- Can be divided into the following types:
  1) Separate system
  2) Combined system
  3) Partially separate system

Separate system

- provides two separate systems of sewers, one for the conveyance of foul sewage, and the other for rain water.

Separate sewerage system

Advantages

- Size of sewers are small
- Sewage load on treatment unit is small
- River or stream waters are not polluted
- Storm water can be discharged into streams or rivers without any treatment
- In case pumping of sewage is required it is economical

Disadvantages

- Sewers being small, difficult to clean
- Likely to choke frequently
- Maintenance cost is more because of two sewers
- Storm water sewer used only in rainy season – can be dumping place of garbage in dry period and get choked
- In busy lanes laying of two sewers is difficult

Combined system

- provides only one sewer to carry both the foul sewage as well as the rain water.

Partially separate system

- Single sewer laid but in small rain period it is used for both but as amount of storm water is increased storm water is collected by open channel and drained to river or stream

Combined sewerage system

Advantages

- Large sewer size; easy to clean and less probability of choking
- Single set needed thus economical
- Sewage strength will be diluted hence nuisance level reduced
- Reasonable maintenance cost
- House plumbing can be done easily only one set of pipes will be required

Disadvantages

- Difficult to handle and transport large size sewers
- Load on treatment plants unnecessarily increased due to storm water
- Overflow of sewers during heavy rain may create unhygienic condition
- In dry season it becomes difficult to maintain proper flow as the sewers are large
- Unnecessary pollution of storm water

Advantages

- Size of sewer not very large since it takes only part of storm water
- Combines advantages of the two systems (Separate/Combine)
- No silting problem
- Problem of disposing of storm water from residence is simplified

Disadvantages

- Flow velocity may be low during dry period
- Storm water increases load on treatment plants and pumps
- There are possibilities of over-flow

HYDRAULIC DESIGNS OF SEWERS

- Sewer is under-ground conduit or drain through which sewage is carried to a point of discharge or disposal.
- *Main sewer or trunk sewer* is a sewer that receives waste water from many tributary branches and sewers, serving as an outlet for large territory.
- *Branch sewer or sub main:* receives sewage from a relatively small area,
- *Lateral sewer*: collects sewage directly from the houses.

Hydraulic Formulas for determining Flow Velocities in Sewers and Drains

I. Chezy's Formula $V = c\sqrt{rs}$

Where:

V= velocity of flow in the channel in m/sec

r = hydraulic mean radius of channel, i.e. hydraulic mean depth of channel

= a/p

a = is the area of channel and p is the wetted perimeter of the channel

S = hydraulic gradient, equal to the ground slope for uniform flows,

C = a constant, called Chezy's constant

II. **Manning's Formula**: this formula was evolved by Manning in 1890,

$$V = \frac{1}{n} r^{2/3} s^{1/2}$$

Where: n is rugosity/manning's coefficient

III. **Crimp and Burge's formula:** commonly used in England.

$$V = 88.5\, r^{2/3} s^{1/2}$$

IV. **William-Hazen's Formula**: used for flows under pressure for designing water supply pipes, and is seldom used for designing sewers. It states that

$$V = 0.85 C_H\, r^{0.63} s^{0.54}$$

Where: C_H is hazen coefficient

Effect of Maximum ‘and Minimum Velocities in Sewers

- The flow velocities in the sewers should be such that neither the suspended materials in sewage get silted up nor the sewage pipe material gets scoured out.
- What if the velocity of flow is less than the minimum allowed?
- What if the velocity of flow is greater than the maximum allowed?

Minimum Velocities

- silting of sewers
- the minimum velocity which will even scour the deposited particle of a given size is called self-cleansing velocity.

$$V_s = \frac{1}{n} . r^{\frac{1}{6}} \sqrt{K . d'(G-1)}$$

n = is manning coefficient; the usual value for sewer pipes is 0.013

G = is specific gravity

r = is hydraulic mean radius

K = is an important characteristic of the sediment

d' = For single grains, the volume per unit area (i.e. t) becomes a function of the diameter of the grain d'

The minimum velocity generated in the sewers will

a) ensure the adequate transportation of the suspended solids,

b) Keep the sewer size under control;

c) Preventing the sewage from getting stale and decomposition by moving it faster, thereby preventing evolution of foul gases.

Maximum Velocities/non-scouring velocity

- smooth interior surface of a sewer pipe gets scoured
  - This wear and tear of the sewer pipes will:
  - Reduce their life spans
  - Reduce their carrying capacities.

- This non-scouring velocity will mainly depend upon the material of the sewer

Hydraulic Characteristics of Circular Sewer Sections Running Full or Partial

- The circular section is most widely adopted for sewer pipes.
- They may, however, sometimes be of
  - 'egg shape' or
  - 'horse shoe shape' or '
  - rectangular shape'.
- The circular sewers may run full or may run partially full

When they run full, their hydraulic properties will be

1. Depth = Diameter, D
2. Area of cross-section

$$A = \frac{\pi}{4} D^2 \frac{\theta}{360} = \frac{\pi}{4} D^2,$$ Where D is the diameter of the pipe

3) **Wetted perimeter**

$$P = \pi D$$

4) **Hydraulic mean depth**

$$R = \frac{A}{P}$$

$$R = \frac{\frac{\pi}{4}D^2}{\pi D} = \frac{D}{4}$$

5) **Velocity**

$$\mathrm{V} = \frac{1}{N}R^{2/3}S^{1/2}$$

5) **Discharge when pipe is running full**

$$Q = V.A$$

Examples

1. Design an outfall circular sewer of the separate system for a town with a population of 100,000 persons with water supply at 180litre/head/day. The sewer can be laid at a slope of 10 in 10,000 with n=0.012. A self-cleansing velocity of 0.75m/sec. is to be developed. The dry weather flow may be taken as 1/3 of maximum discharge.

Solution:

- Population = 100,000
- Average rate of water supply = 180 liters/person/day
- 1st: calculate quantity of sewage, Q, assume 80% of water supply released as waste water,

Q = 100,000 x 180 liters/day * 0.8 = 14,400,000liters/day

Q = **0.1664m^3/s**

Q_{max} = 3 * 0.1664 = **0.4992m^3/s**

Let us design the sewer as running full at maximum discharge. Using Manning's formula, we have,

Q = A.V, but V = $\frac{1}{N} R^{\frac{2}{3}} S^{\frac{1}{2}}$

V = $\frac{1}{N} (\frac{A}{P})^{\frac{2}{3}} S^{\frac{1}{2}}$ but, A = $\pi D^2/4$ and P = πD

So, $Q = \frac{1}{N} \frac{\pi D^2}{4} (\frac{D}{4})^{\frac{2}{3}} S^{\frac{1}{2}}$

Assuming that the sewer is laid at the available slope of 10 in 10000, i.e. 1:1000

$$0.4992\text{m3/s} = \frac{1}{0.012}\frac{\pi D^2}{4}\left(\frac{D}{4}\right)^{\frac{2}{3}}\left(\frac{1}{1000}\right)^{\frac{1}{2}}$$

$$D^{\frac{8}{3}} = 0.758$$

$$\boldsymbol{D = 0.915}$$

Now, check for velocity of flow

$$V = \frac{Q}{A} = \frac{0.4992}{\pi\frac{D^2}{4}} = \frac{0.4992}{\frac{\pi 0.915^2}{4}} = \boldsymbol{0.76 m/s}$$

This is more than 0.75 m/sec, and hence satisfactory.

- Let us check for the velocity at min. flow $\frac{q}{Q} = \frac{1}{3} = \mathbf{0.3333}$

2. A combined sewer of a circular section is to be laid to serve a particular area. Calculate the size of this sewer from the following data:

- Area to be served = 120 hectares.
- Population = 1, 00,000
- Maximum permissible flow velocity = 3m/sec.
- Time of entry for storm water = 10minutes.
- Time of flow in channel = 20minutes.
- Per capita water supply = 250 liters / day / person.
- Coefficient 'of run-off for the area = 0.45.
- Hourly, Maximum rainfall for the area at the design frequency = 5 cm
- Assume any other data not given, if needed.

Solution:

Sewage Discharge (i.e. D.W.F.) Computations

$$Q_{\text{water supply}} = 250 * 100{,}000 l/day = 25*10^6 l/d = 0.289 m^3/s$$

Assuming that 80% of the water supplied appears as sewage,

$$Q_{\text{sewage}} = 0.8 * 0.289 = 0.23 m3/s$$

We assuming the maximum sewage discharge to be 3 times the average discharge,

$$Q_{\text{max. Sewage}} = 3 * 0.23 m3/s = 0.69 m3/s$$

Storm water discharge computations,

$$Q_{\text{storm}} = \frac{K\ i\ A}{36}$$

- Use, $i = p\left(\frac{2}{1+T_c}\right)$ to calculate intensity, Tc is conc. time in hour, p is max. Rainfall & i(cm/hr)
- But Tc = Time of entry + Time of flow = (10 + 20) minutes =30min. = **0.5hour**

$$i = 5\left(\frac{2}{1+0.5}\right) = 6.67cm/hr$$

Therefore, $Q_{storm} = \frac{0.45 * 6.67 * 120}{36} = \mathbf{10m^3/s}$

- ✓ The combined maximum discharge = $Q_{max.\ Sewage} + Q_{storm}$ = 0.69 + 10 **=10.69m³/s**
- ✓ Now assuming the sewer to be running full at the maximum velocity of 3 m/sec at the time of maximum flow, we have

$$A = \frac{Q}{V} = \frac{10.69}{3} = 3.56m^2$$

$$A = \frac{\pi D^2}{4}$$

$$D = \sqrt{\frac{4A}{\pi}} = \sqrt{\frac{4 * 3.56}{\pi}} = 2.13m$$

3. **Calculate the diameter and discharge of a circular sewer laid at a slope of 1 in 500 when running half full, and with a velocity of 2m/s. take N= 0.012 in Manning's formula.**

Solution: when sewer run half full, α =180°

From $d = \frac{D}{2}\left(1 - \cos\frac{\alpha}{2}\right)$

d = 0.5D

$$a = \frac{D2}{4}\left(\frac{\pi\alpha}{360} - \frac{\sin\alpha}{2}\right)$$

$$a = \pi \frac{D2}{8}$$

$$p = \pi D \frac{\alpha}{360^0} = \pi D \frac{1}{2}$$

$$r = a/p = D/2$$

$$v = \frac{1}{n} r^{2/3} s^{1/2} =$$

$$2 = \frac{1}{0.012} (\frac{D}{2})^{2/3} \frac{1}{500}^{1/2}$$

D=1.573m

*Q = a*v = 1.942m³/s*

Exercise

1. A town has a population of 100,000persons with a per capita water supply 200liters/day. Design a sewer running 0.7 times full at maximum discharge. Take a constant value of N =0.013 at all depth of flow. The sewer is to be laid at a slope of 1 in 500. take peaking factor 3.
2. A 60cm diameter sewer is to discharge $0.07m^3/s$ at a velocity as self-cleansing as a sewer flowing full at 0.85m/s. Find the depth and velocity of flow and the required slope. The uniform value of N = 0.015. (don't use table!)

3. A 40cm diameter sewer is to flow at 0.3m depth on a grade ensuring a degree of self-cleansing equivalent to that obtained at full depth at a velocity of 80cm/s. find

   i. The required grade
   ii. Associated velocity
   iii. The rate discharge in this depth

Given:

Manning's rugosity coefficient = 0.013

Proportionate area = 0.252

Proportionate HMD = 0.684

Sewer materials

Must be durable and strong to resist the abrasive and corrosive properties of the wastewater.

- Vitrified Clay Pipe (VCP)
- Polyvinyl Chloride Pipe (PVC)
- Ductile Iron Pipe (DIP)
- High-Density Polyethylene (HDPE)
- Reinforced Concrete Pipe (RCP)
- Truss Pipe

Sewer Appurtenances

Manholes: used to allow a means of access into a sewer system for inspection, repair and cleaning.

- placed at changes in direction, pipe size, grade and elevation, and at junctions, and at intervals of 90-120 m

Treet Inlets: an opening into sewer for entrance of storm runoff.

placed at intersections and at intervals of 20 to 100 m.

Catch Basins: inlets with a Basin, which allow debris to settle out.

septic tank

Design features of septic tank

- As septic tank is a settling digestion tank, its rational design is based on the following three functions. It is expected to perform:

  i. Sedimentation to remove the maximum possible amounts of suspended solids from the sewage

  ii. Digestion of settled solid resulting in a much reduced volume of dense, digested sludge, and

  iii. Storage of sludge and scum accumulating in between successive cleanings thereby preventing their escape

- Hence, the tank should be capable of storing the sewage flow during the detention period and additional volume of sludge for 6 months to 3 years depending on the periodicity of cleaning.

Detention period: A detention period of 24 to 48 hours based on the average daily flow of sewage can be sed.

Inlet and outlet baffles:

- The baffles or tees should extend of about 20cm above the top sewage line.
- Inlet should penetrate by about 30cm below the top sewage line and outlet should penetrate of about 40% of the depth of the sewage. (to prevent direct currents b/n inlet and outlet)
- The outlet invert level should be kept 5 to 7.5cm below the inlet invert level

Length to width ratio:

- Septic tanks are usually rectangular with their length at about 2 to 3 times the width (L = 2B to 3B)
- The width should not be less than 90cm
- The total depth of the tank generally ranges between 1.2 to 1.8m

Sludge withdrawal:

- The rate of accumulation of sludge has been recommended as 30l/c/year (0.082 l/c/day)
- Sludge is withdrawn from the septic tank either half yearly or yearly.
- For small domestic tanks, de-sludging may be done at least once in 6 months to 3 years.

Septic tank dimensions

Table 1: Minimal dimensions of a septic tank for isolated dwellings

Inside total height	1.5m
Height of the liquid	1.2m
Ratio L/W	2
Thickness of floor & ceiling	150mm
Thickness of the walls	200mm
Min. height between inlet and outlet pipes	75mm

Guidelines for installation

- For selecting a site to install the septic tank, these guidelines should be followed.
  - Stay at least 30m from drinking water sources, 15m from streams or ponds and 3m from water lines.
  - Slope drain fields away from houses, buildings and the water supply.
  - Keep drain fields unshaded and free from trees and shrubbery.
  - Allow sufficient space to enlarge the drain field if it should become necessary.
  - Keep septic tanks or drain fields uncovered by driveways
  - Locate septic tanks and drain fields away from drainage areas and waterways.
  - Never use an open flame or matches to inspect a septic tank. Sewer gases may explode violently.

Effluent disposal in septic tank

- The effluent coming out of the tank will be septic and malodourous.
- Final purification of the effluent and the removal/death of pathogen can be done by the following methods:
  - Soil absorption systems
  - Up flow filters
  - Biological filters
- Soil absorption systems:
  - Seepage pit or soak pit
  - Dispersion trenches

Effluent disposal in septic tank

- Soak pits or dispersion trenches can be adopted in all porous soils where:
  - percolation rate is below 30 minute per cm and
  - the depth of water table is 3m or more from the bottom level of the drain field.
- Percolation rate of soil or ground is the time in minutes required for seepage of water through that ground by 1cm.
- Incase, sufficient porous ground is not available,
  - the effluent of the septic tank shall be subjected to a secondary treatment:
    - biological filter
    - up-flow anaerobic filter

Effluent disposal in septic tank

- The total subsurface soil area required for the soak pits or dispersion trenches shall be worked out on the basis of:
  - the maximum allowable rate of effluent application which is given by the following empirical relation:

$$Q = \frac{130}{\sqrt{t}}$$

Where:

Q – maximum rate of effluent application in $l/d/m^2$ of leaching surface

t – standard percolation rate for the soil in minutes per cm

- In calculating the effective leaching area required,
  - only area of trench bottom in the case of dispersion trenches (0.5 to 1m deep and 0.3 to 0.9m wide) and
  - effective side wall area below the inlet level for soak pits should be taken into account
- The depth of a soak pit should be between 1.5 and 4m.

a. Disposal in Soak pits

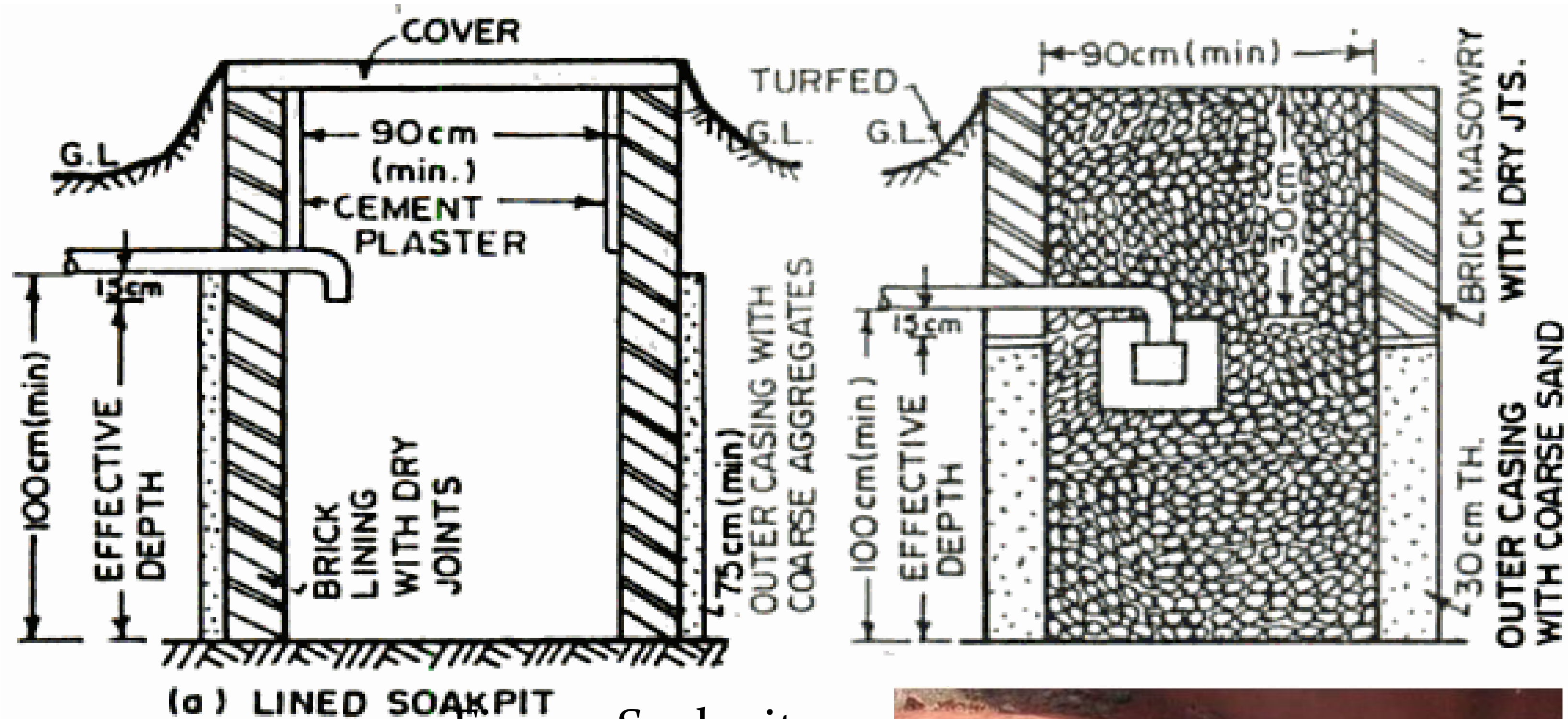


Figure: Soak pit

- As pre-treated wastewater and grey water percolates through the soil from the soak pit, small particles are filtered out by the soil matrix and organics are digested by micro-organisms.

b. Disposal in adsorption trenches

- The septic tank effluent is allowed to enter into a distribution box, from where it is uniformly distributed through an underground network of open jointed pipes into absorption trenches (dispersion trenches).
- The suspended organic matter present in the effluent will be absorbed in the absorption trenches, which are filled with gravel and well graded aggregate.
- The clearer water will seep down to the water table.
- Dispersion trenches are not recommended in areas where fibrous roots of trees or vegetation.
- Dispersion trenches may be adopted on soils having percolation rate note exceeding 30 minutes.

Example

a. Design a septic tank for the following data:

- Number of people = 100
- Sewage/capita/day = 120 liters
- De-sludging period = 1 years
- Length: Width = 3:1

b. What would be the size of its soak pit if the effluent from this septic tank is to be discharged in it. Assume percolation rate through the soak pit to be 1250l/m^3/d

Solution

Quantity of sewage produced per day

= 12000 liters/day

Assuming the detention period to be 24 hours = 1 day,

The quantity of sewage produced during the detention period, i.e. the capacity of the tank

$$C = 12000 * \frac{24}{24} = 12000\ liters$$

Assuming the rate of sludge deposit as 30 liters/capita/year and with the given 1 year period of cleaning,

The quantity of sludge deposited

$$= 30 * 100 * 1 = 3000\ litres$$

Total required capacity of the tank

$$= 12000 + 3000 = 15000\ liters = 15m^3$$

Assuming the depth of the tank as 1.5m [1.2, 1.8], the cross-sectional area of the tank

$$= \frac{15}{1.5} = 10m^2$$

Using L:W as 3:1

$$3B^2 = 10$$
$$B = \sqrt{3.33} = 1.8m$$
$$L = 3 * 1.8 = 5.5m$$

Therefore, the dimension of the tank will be

$$5.5m * 1.8m * (1.5 + 0.3)m$$

0.3m is for the free board.

- Hence, use a tank of size **5.5m*1.8m*1.8m**

Design of soak pit

The soak pit can be designed by assuming the percolating capacity of the media, as 1250 $l/m^3/d$

Sewage outflow = 12000l/d

Percolation rate = 1250$l/m^3/d$

Volume of filter media required for the soak pit

$$\frac{12000}{1250} = 9.6m^3$$

If the depth of the soak pit is taken as **2m** [1.5, 4]

The area of the soak pit required

$$= \frac{9.6}{2} = 4.8m^2$$

- Therefore, the diameter of the soak pit required is 2.47m = **2.5m**

Exercise

1. Estimate the size of a septic tank (length to width ratio of 2.25, liquid depth of 2m with 0.3m freeboard), de-sludging interval in years and the total trench area of the percolation field for a small colony of 300 people. Assume water supply of 100 l/c/d, wastewater flow at 80% of water consumption, sludge production of $0.04 m^3/c/y$ and the retention time of 3 days at start up. De-sludging is done when the tank is one-third full of sludge. A percolation test indicated an allowable hydraulic loading of $100 l/m^2/d$.
2. Design the absorption field system for the disposal of septic tank effluent for a population of 100 persons with sewage flow rate of 135 l/c/d. The percolation rate for the percolation test carried out at the site of the absorption field may be taken as 3 minutes.

$$Q = \frac{130}{\sqrt{t}} \quad l/d/m^2$$