HYDRAULIC DESIGN OF WATER DISTRIBUTION USING EPANET SOFTWARE

Submitted in partial fulfillment of the Degree of

Bachelor of Technology

विद्या तत्व ज्योतिसम:

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Candidate's Declaration

We hereby certify that the work which is being presented in this report, in partial fulfillment of the requirement for the award of bachelor of technology degree, submitted in the department of civil engineering, Jaypee University of Information Technology, Waknaghat,Solan,is an authentic record of our own work carried out from July 2012 to May 2013 under the guidance of Dr.AshishRohilla

We have not submitted the matter embodied in this report for the award of any other degree.

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CERTIFICATE

This is to certify that the work entitled, submitted by SiddharthYadav (091607), IshaanGoel (091614), YashYadav (091654) and Tanuj Jain (091639) in partial fulfillment for the award of degree or Bachelor of Technology in Civil Engineering of Japee University of information Technology, Waknaghat has been carried out under my supervision. This work has not been submitted partially or wholly to any other University or Institute for the award of this or any other degree or diploma.

Ashish Kumar

Dr.AshishRohilla

ACKNOWLEDGEMENT

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We wish to gratefully acknowledge the library staff and several authors of various text books.

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ABSTRACT

Water is essential to all life –human, animal and vegetation. It is very important that there is distribution of sufficient amount of water to all of them. The Water supplied has to be clean and potable to be used for drinking purpose. A good water distribution network supplies water in sufficient quantity according to demand with minimum losses and optimum velocity. The design of a water distribution network should be such that there is economy and the sizes of the pipes should be in accordance with the design parameters. The main aim of this project is to solve a water distribution network using both traditional method and using software.

Hardy Cross method has been used for solving the network manually while EPANET, software designed by USEPA is used to solve the same network. An attempt has been made to characterize the hydraulic parameters and find a tally between the two results.

Lengthy calculations and manual errors in computing the network parameters stress the need to solve such complex network using program efficient software's such as EPANET.

CHAPTER 1: INTRODUCTION

After water has been properly treated and made safe and wholesome, it has to be supplied to consumer at their home. The water has therefor to be taken from the treatment plant to the roads and street in the city, and finally to the individual houses. This function of carrying water from treatment plant to the individual house is accomplished through a well-planned distribution system.

A distribution network thus consists of pipe lines of various sizes for carrying the water to the street. Valves for controlling the flow in the pipes, hydrants for providing connection with the water mains for releasing water during fire, meters for measuring discharge, pumps for lifting the water, service reservoirs for storing the treated water etc. The collection of all these elements in a network is called water distribution network.

The hydraulic design of the distribution network consists of evaluating the head loss, pipe sizes, flow rate and velocity in each pipe of the network that will be available at various points in the network.

Several methods are available for evaluating the hydraulic parameters of the network. Each methods is based on the local topographical conditions and orientation of roads and hence the likely layout of distribution network. The choice of layout distribution network also determines the labor put in its mathematical analysis with the traditional method. The use of models, therefor, comes handy in quicker analysis and better interpretation of network principles. The present project was undertaken to apply the principles of water distribution network design using both the traditional method and learning the use of software in analyzing the same network as well.

1.1 OBJECTIVES

The objectives of the present work have been the following:

1. To analyze the water distribution network using most common method of analysisviz., Hardy Cross Method

2. To solve the same network using elaborate and powerful software, USEPA EPANET for finding out various hydraulic parameters of network including flow, head loss, friction in each pipe of the network.
3. To compare the results obtained by the above two methods.

1.2 METHODOLOGY

The methodology adopted to fulfill the above objectives is described in the following

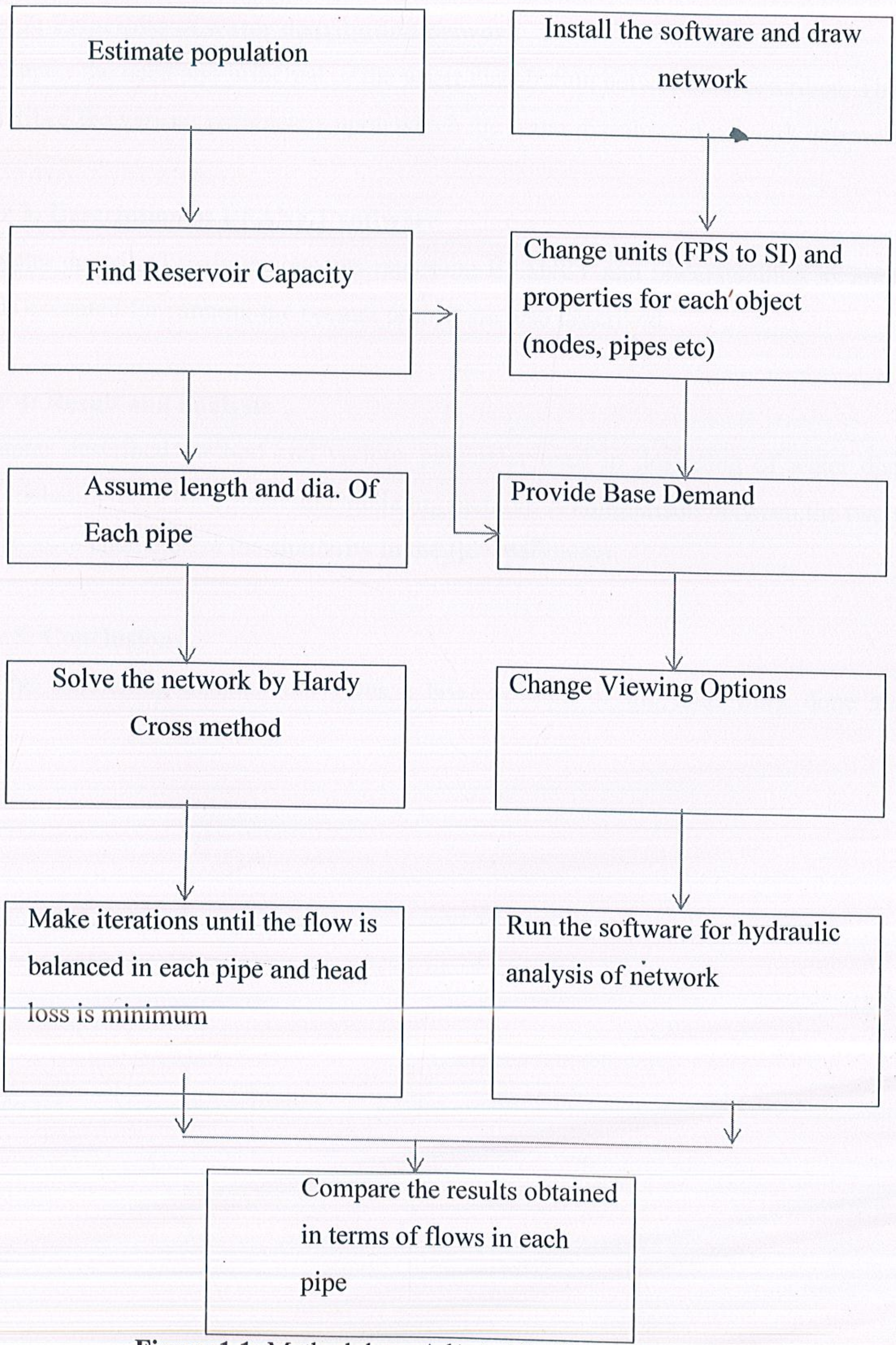


Figure 1.1: Methodology Adopted for Network Analysis

1.3 ORGANIZATION OF REPORT

Chapter 1: Introduction

The present chapter deals with the overview of the objective of the project and the procedure that would be followed from here-on.

Chapter 2: Principles of water distribution network

In this chapter the need and principal of the water distribution network are described. This chapter also described the various parameters upon which the water distribution network depends.

Chapter 3: Description of EPANET software

This chapter described various concepts regarding EPANET and understanding its applications. The steps executed for running the present project are also described.

Chapter 4: Result and analysis

This chapter described method and various parameter used in designing of water distribution network including both traditional and modeling method. A comparison between the two methods is made so as to characterize the similarity in the flow parameter.

Chapter 5: Conclusions

This is the concluding chapter and gives a brief account of the total work done and result obtained.

CHAPTER 2: PRINCIPLES OF WATRE DISTRIBUTION NETWORK DESIGN

The principle of water distribution network include considerations of different elements of design necessary for characterizing the water distribution network. The following section discusses few of these elements.

2.1 RQUIREMENTS OF WATER DISTRIBUTION NETWORK

The various requirements for proper functioning of a distribution system are:

1. It should be capable of supplying water at all the intended places within the city with a reasonably sufficient pressure head.
2. It should be capable of supplying the requisite amount of water for Fire Fighting during such needs.
3. It should be cheap with the least capital construction cost. The economy and the cost of installing the distribution system is a very important factor, because the distribution system is the most costly item in the entire water supply scheme. So much so, that it gobbles up about 70% of the total cost of the scheme.
4. It should be simple and easy to operate and repair, thereby keeping the RMO cost and troubles to the minimum.
5. It should be safe against any future pollution of water.
6. It should be safe as not to cause the failure of the pipe lines by bursting etc.
7. It should be fairly water tight, as to keep the ''losses due to leakage'' to the minimum.

2.2 PLANNING CONSIDRATIONS

The water in a treatment plant before distribution is made fit as per the desired water quality standards. These standards are different for different use the treated water will be put to viz. domestic, industrial use. Therefore, while designing the water distribution network, it becomes pertinent design in a fashion such that the standard of this quality is to be maintained when the treated water is flowing through the

network. For this reasons, the following general consideration of the planning of distribution system should be observed in its design.

2.2.1 Circulation of Water

The layout of distribution system should be such that there is free circulation of water and that the number of dear ends is unavoidable, the hydrant will be provided to act as washout.

2.2.2 Construction and Design

The design should be such that ample water is all time at desired pressure in all portion of network. The minimum residual pressures at ferrule point for direct supply to single- storied and two storied three-stored buildings should be respectively 7m, and 17m. If the available pressure in the pipe is low, it has to be boosted up with the help of pump in case of fire occurrence.

2.2.3 Contamination by sewage

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

2.2.4 Earth Cushioning

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

2.2.5 Economy

The layout and design of distribution system should be economical {within budget}.

2.2.6 Fire Demand

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

2.2.7 Gradients

It is not necessary to lay mains at constant gradients. But the gradients of main should in general follow the contour of ground.

2.2.8 Leakage

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

2.2.9 Repair

The distribution system should be so laid as to permit easy repair.

2.2.10 Safety from pollution

The lay out should be such that it does not contribute to the pollution of water flowing in it.

2.2.11 Sanitation

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

2.2.12 Unsafe cross connection

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

2.3 METHODS OF DISTRIBUTION

Depending on the topography of the country, any one of the following three methods may be adopted for the distribution of water.

2.3.1 Gravity System

In this system water is conveyed through pipes by gravity only. The gravity system is the most reliable method of distribution. But it is useful only when the source of water supply is situated at higher level than that of distribution area. In case of fire, pumps can be used for creating high pressure. As shown in figure 2.1

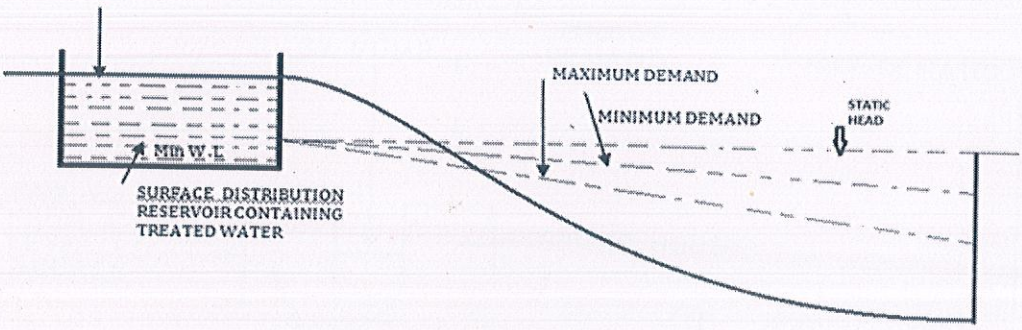


Figure 2.1: Gravitational distribution System

2.3.2 Gravity and Pumping System Combined

In this the treated water is pumped and stored in an elevated distribution reservoir as shown in the Figure 2.2. The excess water during low consumption remains in the elevated reservoir and it is supplied during peak period. The pumps are usually worked at constant rate and this rate of pumping is so adjusted that the excess quantity of water stored in reservoir during low consumption is nearly equal to the extra water demand during peak period.

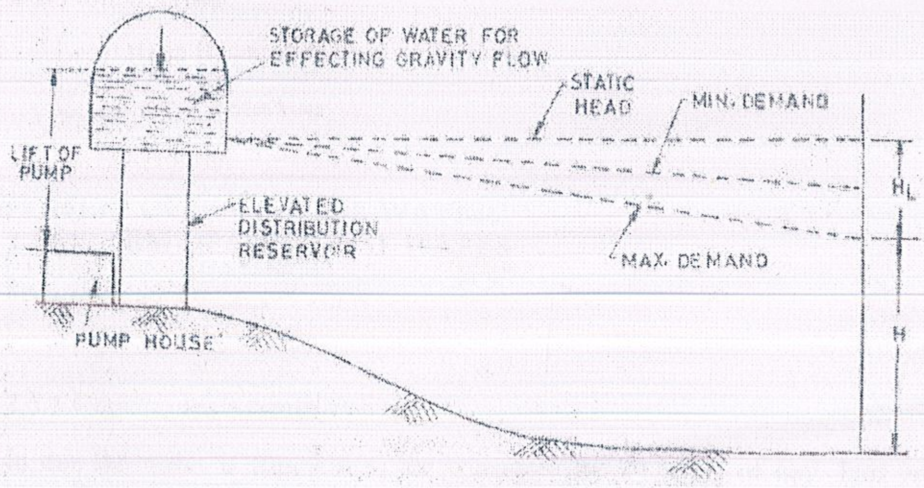


Figure 2.2: Combined Gravity and Pumping System for water distribution

2.3.3. Pumping System

In this system, the water is directly pumped in to the mains leading to consumers. The number of pumps required in this system will depend on demand of water. In case of fire the pressure can be developed easily by operating fire pumps of high capacity (fig. 2.3).

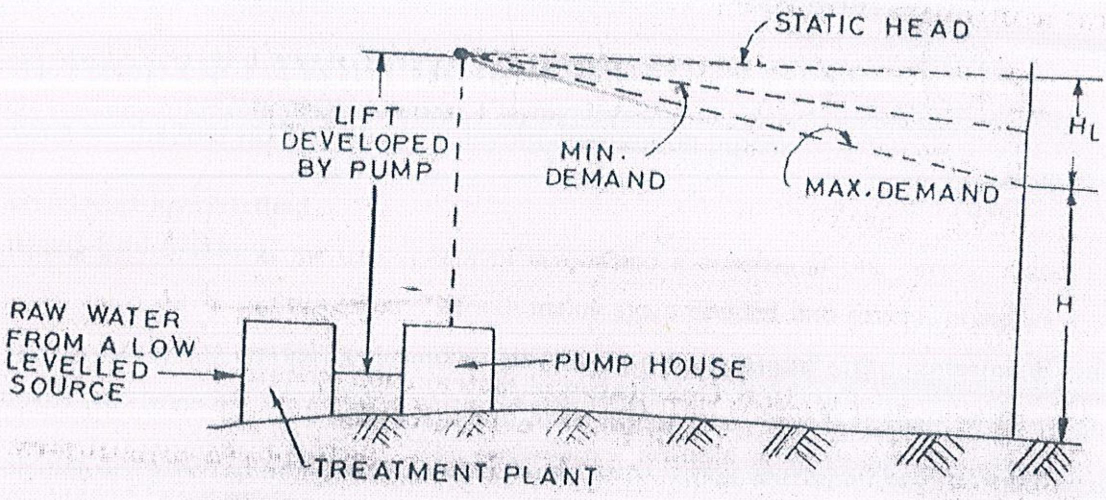


Figure 2.3: Pumping system for water distribution

2.4 SERVICE RESERVOIRS

The service or distribution reservoirs are generally provided in the distribution system to store the clear treated water before it is distributed to the consumers. These may be constructed of brick masonry, cement concrete etc.

It is of three types:

1. Surface Reservoir
2. Elevated Reservoir
3. Stand Pipes

2.5 SYSTEM OF SUPPLY OF WATER

Following are the two systems:

2.5.1 Continuous System:

In this the water is supplied to the consumer for 24 hours of day. This is the most ideal system of supply and it should be adopted as far as possible. The only disadvantage of this is that considerable wastage of water occurs.

2.5.2 Intermittent System

In this the water is supplied during certain fixed hours of the day. The usual period is about one to four hours in the morning and about the same period in the afternoon. The timing of water supply may be changed to suit the season of year and it is more or less a matter of convenience only.

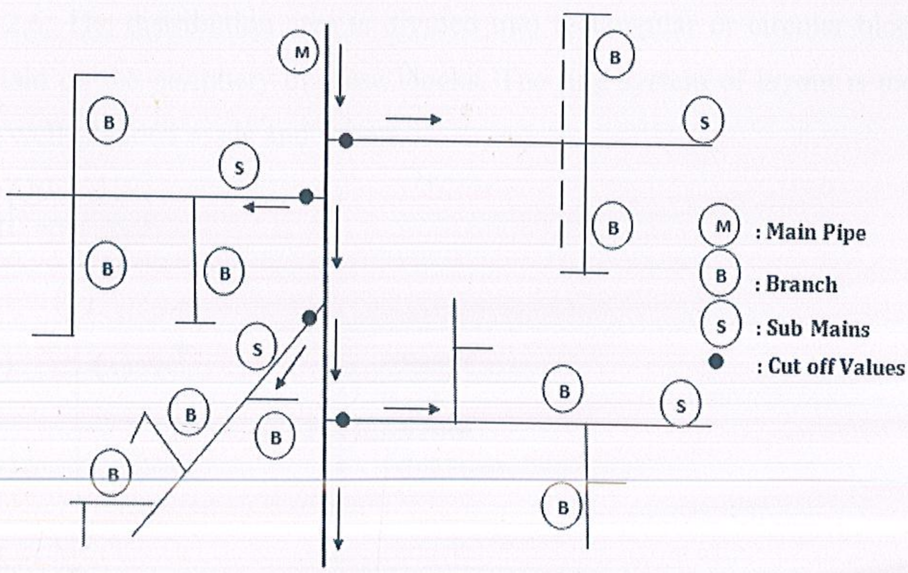
2.6 METHOD OF LAYOUT OF DISTRIBUTION PIPES

Following are the four main methods of laying distribution pipes:

2.6.1 Dead-End Method

This is also known as the three system of layout and it consists of one supply main from which sub-mains are taken. The sub mains again divided into several branches lines from which service connections are given to the consumers the diameter of mains, and branches are suitably designed (fig. 2.4). This system is used in location which is expanding irregularly. The water pipes are laid at random without any planning of future roads.

Dead- End method:-



2.6.2 Grid-Iron Method

This is also known as the interlaced system or reticulation system. The mains, sub-main and branches are interconnected as shown in Figure 2.5. This layout is more suitable for town having well-planned roads and streets as in Chandigarh.

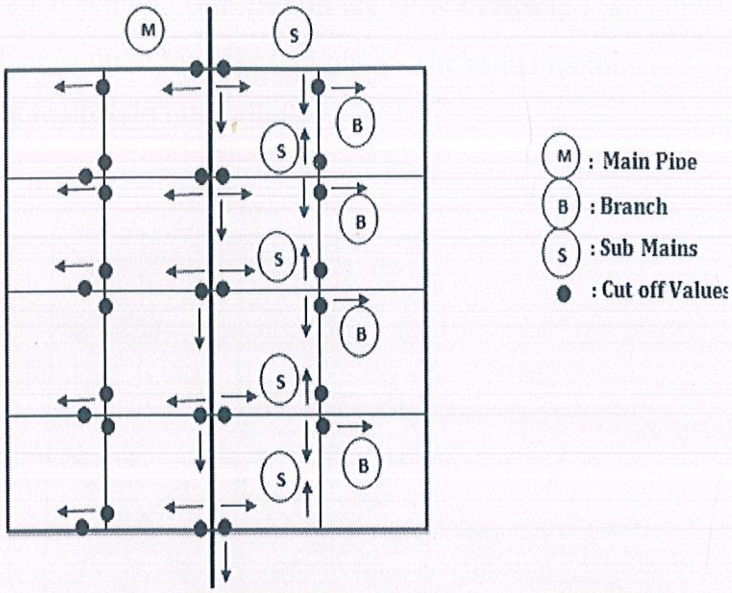


Figure 2.5 Gird method of water distribution

2.6.3 Circular Method (Ring Method)

This is also known as the ring system and a ring of mains is formed around the distribution area as shown in figure 2.6. The distribution area is divided into rectangular or circular blocks and the water mains are laid on the periphery of these blocks. The ring system of layout is most suitable for towns having well-planned roads and streets.

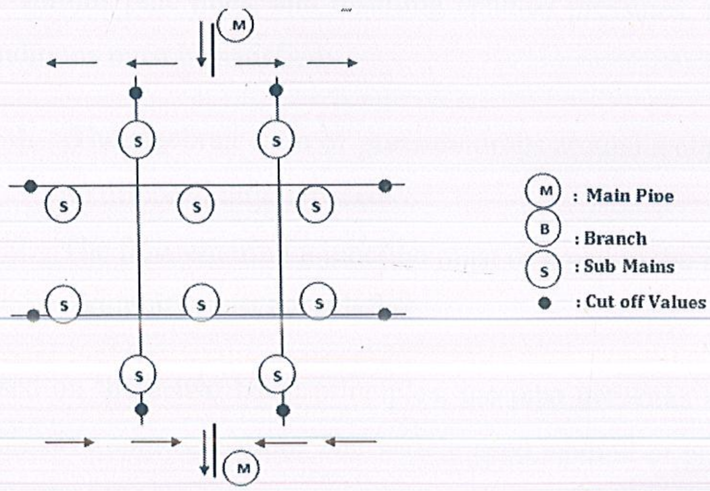


Figure 2.6 Circular method of water distribution

2.6.4 Radial Method

This method of lay out is just reverse of the ring method. In this system the water is taken from mains and pumped into the distribution reservoirs which are situated at centers' of different zones as shown in Figure 2.7. The water is then supplied through radially laid pipes. The radial method is most suitable fir towns having roads laid out radially.

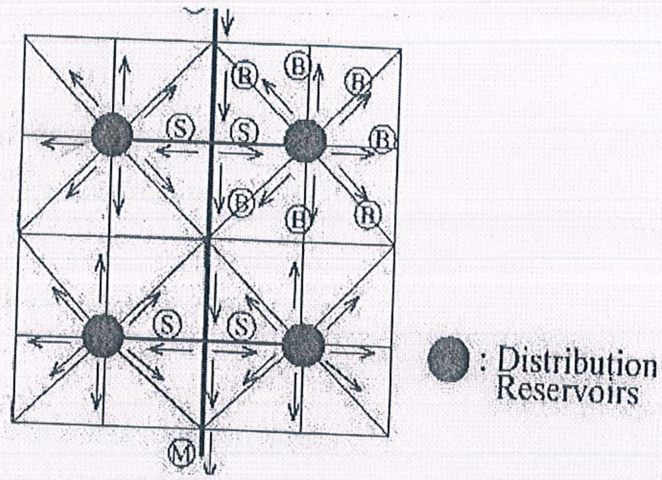


Figure 2.7: Radial method of Water distribution

2.7 ANALYSIS OF NETWORK

Analysis of water distribution system includes determining quantities of flow and head losses in the various pipe lines, and resulting residual pressures. In any pipe network, the following two conditions must be satisfied:

1. The algebraic sum of pressure drops around a closed loop must be zero, i.e. these can be no discontinuity in pressure.
2. The flow entering a junction must be equal to the flow leaving that junction; i.e. the law of continuity must be satisfied.

Based on these two basic principles, the pipe networks are generally solved by the methods of successive approximation. The widely used method of pipe network analysis is the Hardy-Cross method.

2.7.1 Hardy-Cross Method

This method consists of assuming a distribution of flow in the network in such a way that the principle of continuity is satisfied at each junction. A correction to these assumed flows is then computed successively for each pipe loop in the network, until the correction is reduced to an acceptable magnitude.

If Qa is the assumed flow and Q is the actual flow in the pipe, then the correction & is given by

$\&=Q-Q_a$; or

$Q= Q_a+\&$ (1)

Now, expressing the head loos **(H1)**as

$H_1 =K, Q_x$ (2)

We have the head loos in a pipe

$=k.(Q_a+\&)^x$(3)

$=k.\ [\ Q_a^x +x.Q_a^{x-1} +$.................................negligible terms] (4)

$=k.\ [\ Q_a^x +x.Q_a^{x-1}\&$ (5)

Now, around a closed loop, the summation of head losses must be zero.

$\sum k[Q_a^x + x.Q_a^x\text{-}1\ \&]=0$ (6)

Or $\sum k.Q_a^x =-\sum(KxQ_a^x\text{-}1).\delta$

(7)

Since, &is the same for all the pipes of the considered loop, it can be taken out of the summation. Therefore,

$\sum k.Q_a^x=-\&\sum(KxQ_a^x\text{-}1$ (8)

δ =- (∑k.Q_a^x)/(∑(KQ_a^(x-1) (9)

Since δ is given the same sign (direction) in all pipes of the loop, denominator of the above equation is taken as the absolute sum of the individual items in the summation. Hence,

$$\delta = -(\sum K.Q_a^x)/(\sum x.KQ_a^{(x-1)}) \quad (10)$$

Or

$$\delta = -(\sum H_l)/(x.\sum H_l/Q_a) \quad (11)$$

Where H is the head loss for assumed Flow $Q_{i.}$

The number in the above equation is the algebraic sum of the hard losses in the various pipes of the closed loop computed with assumed flow. Since the direction and magnitude of flow in these pipes is already assumed, their respective head losses with due regard to sign can be easily calculated after assuming their diameters. The absolute sum of respective $\mathbf{KQ_a}^{x-1}$ or H_L/Qiis then calculated. Finally the value of δ is found out for each loop, and the assumed flows are corrected. Repeated adjustments are made until the desired accuracy is obtained.

The value of x in Hardy- Cross method is assumed to be constant (i.e. 1.85 for Hazen- William's formula, and 2 for Darcy- Weisbach formula)

2.7.2 Equivalent Pipe Method

This method is sometimes used as an aid in solving large networks of pipes, in which it becomes convenient to: first of all, replace the different small loops by single equivalent pipes having the same discharging capacities and causing the same head loss.

In this method, circuit can be reduced in to a single equivalent pipe by using the following two principles of hydraulics

1. The loss of head caused by a given flow of water through the pipes connected in series is additive;
2. The quantity of discharge flowing though the different pipes connected in parallel will be such as to cause equal head loss through each pipe.

CHAPTER 3: DESCRIPTION OF ''EPANET'' SOFTWARE

EPANET is a computer program developed by USEPA. It is freely downloadable software from site www.epa.gav. The software performs extended period simulation of hydraulic and water quality behavior within pressurized pipe networks. A network consists, nodes, pumps, vaives and storage tanks or reservoirs. EPANET tracks the flow of each pipe, the pressure at each node, the height of water in each tank, and the concentration of a chemical species throughout the network during a simulation period comprised of multiple time steps.

In addition to chemical species, water age and source tracing can also be simulated. EPANET's hydraulic simulation model computers junction heads and link flows for a fixed set of reservoir levels, and water demands over a succession of points in time. From one time step to the next reservoir levels and junction demands are updated according to their prescribed time patterns while tank levels are updated using the current flow solution. The solution for heads and flows at a particular point in time involves solving simultaneously the conservation of flow equation for each junction and the head loss relationship across each link in the network. This process, known as ''hydraulically balancing'' the network requires using an iterative technique to solve the nonlinear equations involved.

The hydraulic head lost by water flowing in a pipe due to friction with the pipes wall can be computed by using one of three different formulas:

1. Hazen- Williams formula
2. Darcy- Weisbach formula
3. Chezy- Manning formula

The Hazen- Williams formula is the most commonly used head loss formula. It cannot be used for liquids other then water. The Darcy-Wasatch is the most theoretically Correct.
It applies over all flow regimes and to all liquid. The Chewy-Manning formula is more commonly used for open channel flow.

3.1 EPANET'S WORKSPACE

It consists of the user interface elements: a Menu Bar, two Toolbars, a status Bar, in the Network Map window, a Browser window, and a Property Editor window. These elements are shown in the figure 3.1 below.

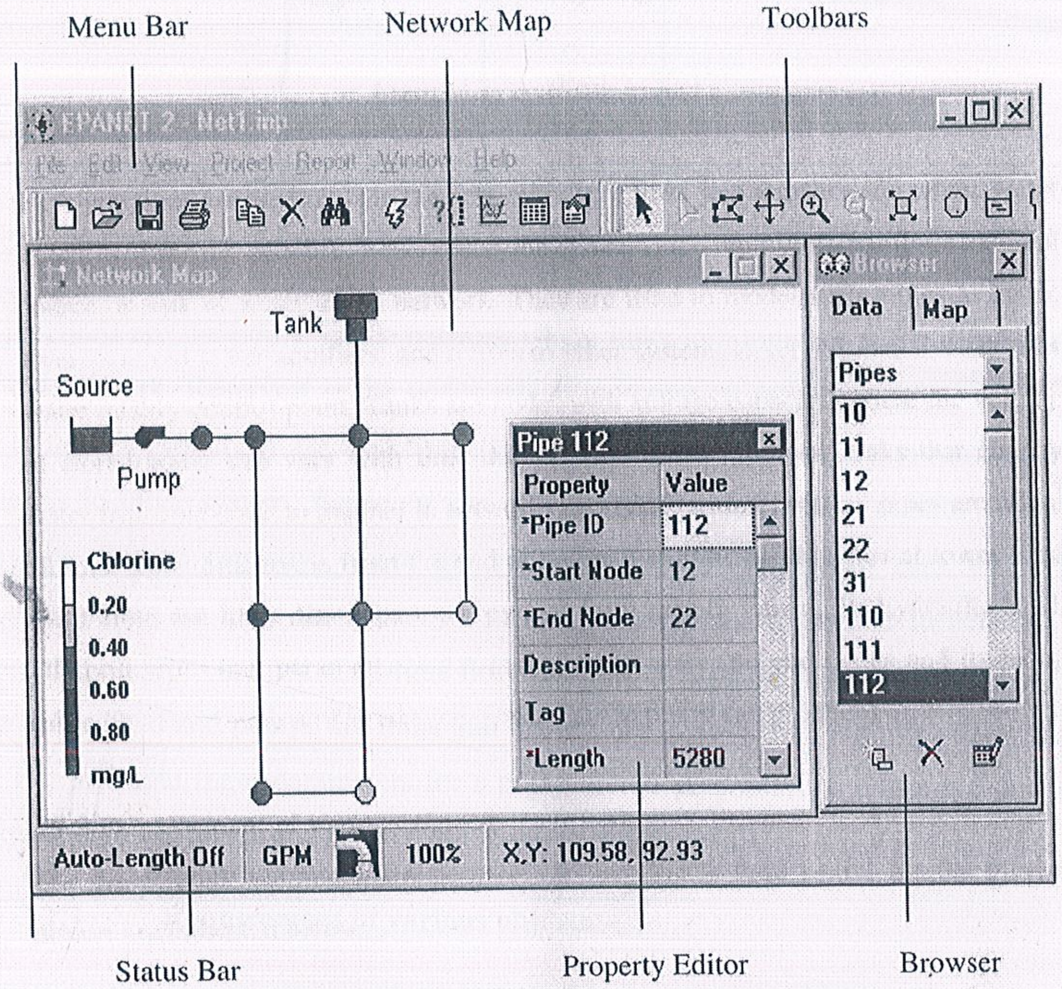


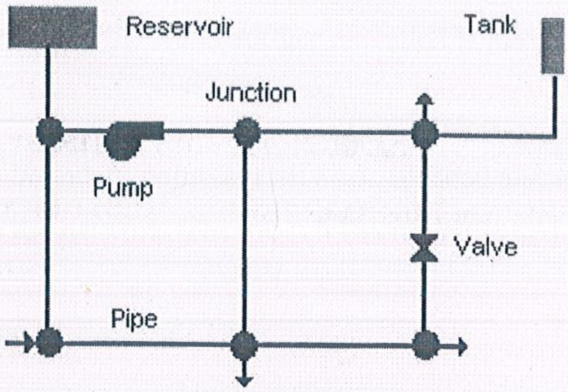
Figure 3.1: Working window of EPANET

3.2 WORKING IN SOFTWARE

3.2.1 Representation of network

EPANET models a water distribution system as a collection of links connected to nodes as shown in 3.2. The link represents pipes, pumps and control valves. The nodes represent junction, tank and reservoirs. The figure below illustrates how the object can be connected to one another to form a network.

Figure 3.2 Network Model



Here junctions are the points in the network where links join together and where water enters or leave the network. Reservoir is the nodes which represent an infinite external source or sink of water to the network. They are used to model such things as lakes, rivers, ground water aquifer's, and tie-ins to other system. Reservoir can also serve as water quality control point. Tanks are nodes with storage capacity, where the volume of stored water can vary with time during simulation. Pipes are links that convey water from one point to another in network. EPANET assumes that all pipes are full at all-time flow direction is from the end at higher hydraulic head to that at lower head and pumps are links that impart energy to a fluid thereby raising its hydraulic head. The principal input parameters for a pump are its start and end nodes and its pump curve (the combination of heads and flows that the pump can produce).

3.2.2 Requirement of various objects

The basic input/ output Requirement of various objects fund useful for the present project are indicated below:

Junction

The basic input data required for junctions are:

1. Elevation above some reference (usually mean sea level)
2. Water demand (rate of withdrawal from the network)
3. Initial water quality.

The output result computed for junctions are:

1. Hydraulic head (internal energy per unit weight of fluid)
2. Pressure
3. Water quality.

Reservoir

The primary input properties for a reservoir are:

1. Hydraulic head (equal to the water surface elevation if the reservoir is not under pressure)
2. Initial quality

Because a reservoir is a boundary point to a network, its head and water quality cannot be affected by what happens within the network. Therefore it has no computed output properties.

Pipes

The principal hydraulic input parameters for pipes are:

1. Start and end nodes
2. Diameter
3. Length
4. Roughness coefficient (for determining head loss)
5. Status (open, closed, or contains a check valve).

Computed outputs for pipes include:

1. Flow rate
2. Velocity
3. Head loss
4. Darcy-Weisbach friction factor
5. Average reaction rate (over the pipe length)
6. Average water quality (over the pipe length)

Pump

The principal input parameters for a pump are:

1. Start and end nodes
2. Pump curve

The principal output parameters are:

1. Flow
2. Head gain
3.

3.3 STEPS EXECUTED

3.3.1 Drawing the network

We are ready to being drawing out network by making use of our mouse and the buttons contained on the Map Toolbar as shown below.

Figure 3.3: Toolbar in EPANET Workspace

To add the reservoir, click the Reservoir button . Then click the mouse on the map at the location of the (somewhere to the left to the left of the map).

To add the junction nodes, click the junction button the map at the location of nodes.

Next we will add the pipes, click the pipe button on the toolbar. Then click the mouse on node 2 on the map and the click on another node.

Finally we will add the pump. Click the pump button click on the nodes in between-which pump is required.

3.3.2 Setting the Object properties

As objects are added to a project they are assigned a default set of properties. To change the value of an specific property for an object one must select the object into the property Editor (figure 3.3). There are several different ways to do this. if the Editor is already visible then you can simply click on the object or select it from the Data page of the Browser. If the Editor is not visible then you can make it appear by one of the following actions:

1. Double-click the object on the map.
2. Right –click on the object and selected Properties from the pop-up menu that appears.
3. Select the object from the Data page of the Browser window and then click the Browser's Edit button

The property Editor (shown at the left) is used to edit the properties of network nodes, links, labels, and analysis options. It is invoked when one of these objects is selected (either on the Network Map or in the Data Browser) and double-clicked or the Browser Edit button is clicked. The following figure help explain how to use the Editor.

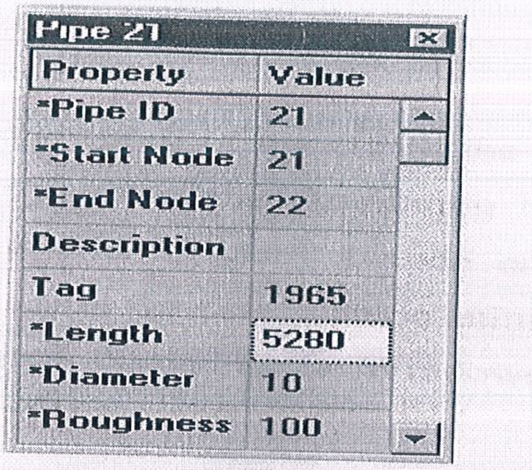


Figure 3.4: Property Editor

3.3.3 Running the software

To run the analysis selects **Project>> Run** Analysis or click the run button on the Standard Toolbar.

3.3.4 Viewing the Result

The different ways in which the result of an analysis as well as the basic network input data can be viewed include the following ways: different map views, table, and special reports.

Viewing Result on the Map

There are several ways in which database values and result of a simulation can be viewed directly on the Network Map:

1. For the current settings on the Map Browser, the nodes and links of the map will be clouded according to the color coding used in the Map Legends. The

map'scoloring will updated as a new time period is selected in the browser.

2. When the flyover map labeling program preference is selected. Moving the mouse over any node or link will display its ID label and the value of the current viewing parameter for that node or link in a hint- style box.
3. ID labels and viewing parameters values can be displayed next to all nodes and/or links by selecting the appropriate options on the Notation page of the map options dialing form.
4. Nodes or links meeting a specific criterion can be identified by submitting a map query.
5. You can animate the display of result on the network map either forward or backward in time by using animation buttons on the map browser.

   Animation is only available when a node or link viewing parameter is a computed value (e.g., link flow rate can be animated but diameter cannot).computed value can be printed, copied to the Windows metafile.
6. The map can be printed, copied to the windows clipboard, or saved as a DXF file or windows metafile.

Viewing Results with a Table

EPANET allows to view selected project data and analysis results in a tabular format:

1. A network table list properties and result for all nodes or links at a specific period of time.
2. A time series table lists properties and result for a specific node or link in all time period.

Tables can be printed, copied to the Windows clipboard, or saved to file.

To create table refer to figure 3.1: select **View >> Table** or click on the standard toolbar, use the table options dialog box, that appears, to select type of table and the quantities to display in each column.

CHAPTER 4: RESULT AND ANALYSIS

With a view to analyze a distribution network, a hypothetical residential sector of a city is assumed. Suitableassumptions have been made in estimating the number of heads, number of plots and thus the population to be served. The network as analyses is shown in Figure 4.1 below.

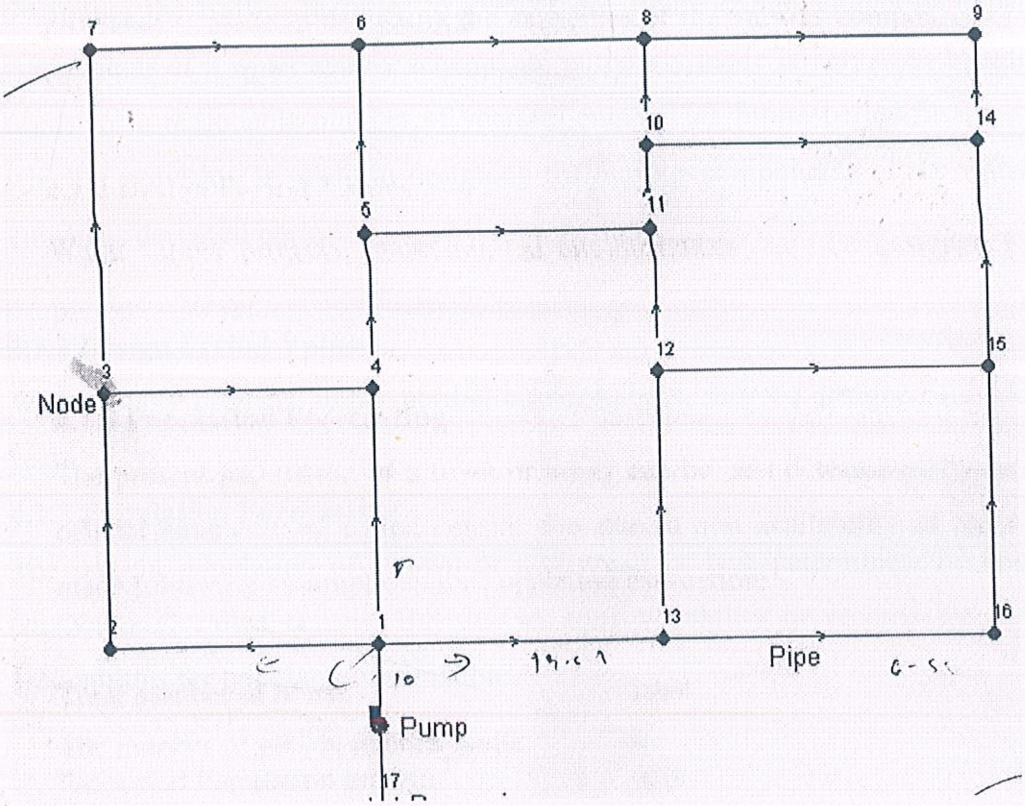


Figure 4.1: Sample Network diagram

4.1 DESIGN PERIOD AND POPULATION PORECASTING

4.1.1 Design Period

A water supply scheme includes huge and costly structures which cannot be replaced or increased in their capacities, easily and conveniently, For example, the water mains includes the distributing pipes are laid underground, and cannot be replaced or added easily, without digging the road or disrupting the traffic. In order to avoid these future complications or expansions, the various components of a water supply scheme are purposely made larger, so as to satisfy the community needs for a reasonable number of years to come. This future period or the number of years for which a provision is made in designing the various components of the water supply scheme is known as designed period.

4.1.2 Design Period Values

Water supply projects, under normal circumstances, may be designed for a design period of 30 years.

4.1.3 Population Forecasting

The present population of a town or a city can be best determinate by conducting an official enumerating, called census, But due to non availability or records we have made following assumptions for population estimation:

Total number of house	1994
The number of person in each house	4
Estimated Population growth	40%
Total estimated population	7976+3190

=11166 heads

4.2 STORAGE CAPACITY OF DISTRIBUTION RESERVOIR

4.2.1 Distribution Reservoir

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

The total storage capacity of a distribution reservoir is the summation of:

1. Balancing storage (or equalizing or operating storage): the main and primary function of a distribution reservoir is to meet the Fluctuating demand with a constant rate of supply from the treatment plant. The quantity of water required to be stored in the reservoir for equalizing or balancing this variable demand against the constant supply is known as the balancing reservoir. This balancing storage can be worked out by utilizing the hydrographs or inflow and outflow, either by mass curve method or by using an analytical tubular solution.

2. Breakdown storage: the Breakdown storage or often called emergency storage is the storage preserved in order to tide over the emergencies posed by the failure of pumps the electricity or any other mechanism driving the pumps. A value of about 25% of total storage capacity of the reservoir, or 1.5 to 2 times of the average hourly supply, may be considered as enough provision for accounting this storage, under all normal circumstances.

3. Fire storage: The third component of the total reservoir storage is the fire storage. This provision takes care of the requirement of water for extinguishing fires.

To determine the balancing storage, the following two methods are used:-

1. Mass Curve Method ; A mass diagram is the plot of accumulated inflow (supply) or outflow (demand) versus time given in Figure 4.1. The mass curve of supply is therefore, first of all, and is superimposed by the demand curve.The amount of balancing storage can then be easily determined by adding the maximum ordinates between the demand and supply lines. To construct such diagrams for a particular water supply project. We have to proceed as follows:

1. From the past records, determine the hourly demand for all 24 hours for typical days (maximum, average and minimum).
2. Calculate and plot the cumulative demand against time, and thus plot the mass curve of demand
3. Draw the cumulative supply also against time.

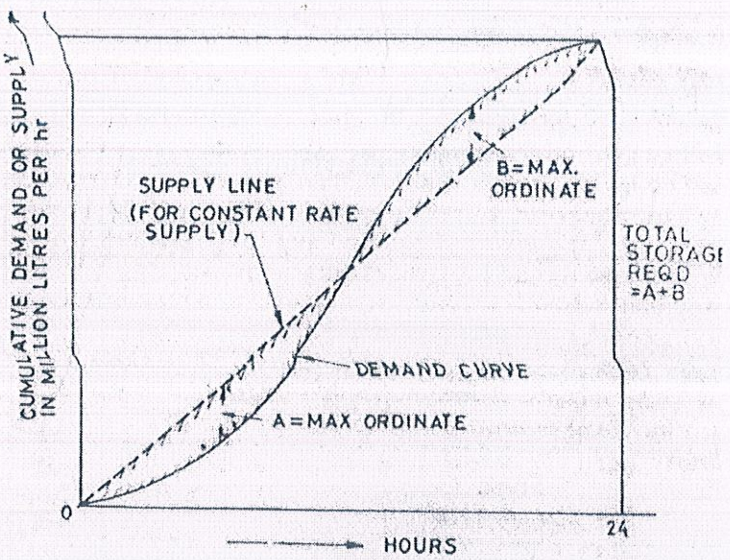


Figure 4.2: Mass Diagram for 24 hours pumping

1. Analytical Method: in this method, the cumulative hourly demand and cumulative hourly are tabulated for all the 24 hours, The hourly excess of demand as well as the hourly excess of supply are then worked out. The summation of maximum of the excess of the excess of demand and the maximum of excess of supply will give us the required storage capacity.

Here we will find out balancing storage capacity of reservoir using above mentioned analytical solution.

4.3 DESIGN PARAMETERS

4.3.1 Storage Capacity of Reservoir

The following are the assumption made while estimating the storage capacity of the reservoir for continuous water supply,

Assumed supply	200 LPCD
Assumed population growth after 30 years	40% current population

Fire demand 4 LPCD

Breakdown storage 25% of total storage

The variation in hourly demand considered for analysis is computed in table 4.1

Shown below .

TABLE 4.1: The probable hourly variation in the rate of demand

Period of day (in hrs)	***Percentage of Average Hourly flow* Expected**	**Period of Day in (in hrs)**	**Percentage of *Average* Hourly flow Expected**
0-1	15	12-13	100
1-2	*15*	13-14	80
2-3	15	14-15	60
3-4	*20*	15-16	110
4-5	*25*	16-17	150
5-6	*40*	17-18	180
6-7	*80*	18-19	180
7-8	*120*	19-20	160
8-9	*180*	20-21	140
9-10	*220*	21-22	80
10-11	*220*	22-23	45
11-12	*150*	23-24	15

From above analysis, the total demand of water supply is found as 11166*200=2.23 **MLD** with Average hourly demand as 2.23/24=**0.0929 ML**

The analytical solution is carried out as shown in table 4.2

TABLE 4.2 Determination of Demand and supply

Period (in hours)	Cumulative Demand (ML)	Constant pumping rate (in ML/hr)	Cumulative Pumping (IN ML)	Excess of Demand (ML)	Excess of supply (IN ML)
1	.0139	.0929	.0929		.079
2	.0278	.0929	.1858		.157
3	.0481	.0929	.2787		.2369
4	.0743	.0929	.3716		.2973

5	.1161	.0929	.4645		.3484
6	.2229	.0929	.5574		.3345
7	.0929	.0929	.6503		.1303
8	.892	.0929	.7432	.1488	
9	1.505	.0929	.8361	.6689	
10	2.044	.0929	.929	1.115	
11	2.248	.0929	1.0219	1.2261	
12	1.672	.0929	1.1148	.557	
13	1.207	.0929	1.2077	0	0
14	1.04	.0929	1.3006		.2606
15	.836	.0929	1.3935		.5574
16	1.635	.0929	1.4864	.1486	
17	2.369	.0929	1.5793	.79	
18	3.011	.0929	1.6722	1.338	
19	3.177	.0929	1.7651	1.412	
20	2.973	.0929	1.858	1.115	
21	2.731	.0929	1.9509	.781	
22	1.635	.0929	2.0438		1.8803
23	.962	.0929	2.1367		1.175
24	.334	.0929	2.2296		**1.895**

From Table 4.2 we can conclude the following: Maximum excess demand is **1.412 ML,** Maximum excess of supply is 1.895ML, and Total balancing storage as worked out is thus 1.412+1.895 or **3.307 ML**

4.3.2 Design of pipes

The selection of a particular type of material for a depends mainly upon the relative economy, the pressure likely to come and the working pressure , the maximum permissible sizes and capacities, availability of materials and labor of their construction etc.

Cast iron pipes are widely used for city water supplies. They are sufficiently resistant to corrosion and may last as 100 years or so. So for our analysis we take cast iron pipes.

Length of pipes

The following assumption is made on the basis of lengths of plots to determine the lengths of pipes of our sample water distribution network using scale 0.5 cm= 10 m.

Table 4.3 given the lengths of various pipes of our distribution network.

Diameter of pipes

Pipe diameter depends upon the flow velocity. This can be estimated using standard values and the present analysis taken from Gang, 2008 given as in Table 4.4.

TABLE 4.3: length of pipes of sample Distribution Network

Pipe	Length (cm)	Length (m)
1-2	28.5	570
2-3	14	280
3-4	21.5	430
3-7	22	440
7-6	14.5	290
6-5	18.5	370
5-4	4.5	90
4-1	7.5	150
1-13	20	400
6-8	15.5	310
5-11	15.5	310
13-12	11	220
12-11	2.5	50
11-10	8.5	170
10-8	10	200
8-9	23	460
10-14	23	460
9-14	10	200
14-15	11.5	230
15-16	7	140
12-15	23	460
13-16	23.5	470

TABLE 4.4: Optimum Velocities in pipes of Different Sizes

Pipe Dia (in cm)	Approximate Velocity (in m/sec)
10	0.9
15	1.2
25	1.5
40	1.8

On the basic of above mentioned velocities the diameter of various pipes assumed are as shown in Table 4.5below:

TABLE 4.5: Diameter of Various Pipes Selected

Pipe No	Diameter (m)	Pipe no.	Diameter (m)
1-2	.15	9-14	.15
2-3	.15	8-10	.15
3-4	.15	10-11	.10
4-1	.10	13-1	.15
3-7	.15	11-12	.10
7-6	.15	12-13	.15
6-5	.15	16-13	.15
5-4	.10	12-15	.15
11-5	.15	15-16	.10
6-8	.15	10-14	.15
8-9	.15	14-15	.15

4.4 METHOD OF ANALYSIS

The pipe networks are generally solved by the methods of successive approximation, because any direct solution is not possible, as the same will involve carious equations to be solved simultaneously and many of which are non-linear.

4.4.1 Hardy Cross method of Iterations

The procedure suggested by Hardy and Cross requires that the flow in each pipe is assumed by designer in such a way that the principle of continuity is satisfied at each junction. As explained in chapter 2, the following tables give the values of k and corrected discharge in each pipe.

TABLE 4.6:Calculation of k Values for different pipes in the loop

Pipe	**Length (m)**	**Diameter (m)**	**$K=L/470d^{4.87}$**
1-2	570	0.15	12666.66
2-3	280	0.15	6140
3-4	430	0.15	9429.82
4-1	150	0.10	23659.30
3-7	440	0.15	9777.77
7-6	290	0.15	6444.44
6-5	370	0.15	8222.22
5-4	90	0.10	14195.58
11-5	310	0.15	6888.88
6-8	310	0.15	6888.88
8-10	200	0.15	4444.44
10-11	170	0.10	26813.88
13-1	400	0.15	8888.88
11-12	50	0.10	7886.43
12-13	220	0.15	4888.88
16-13	470	0.15	10444.44
12-15	460	0.15	10222.22
15-16	140	0.10	22082.01
10-14	460	0.15	10222.22
14-15	230	0.15	5111.11
8-9	460	0.15	10222.22
9-14	200	0.15	4444.44

TABLE 4.7: Corrected Discharge-First Iteration

Pipes	Assumed Flow		K value	H_L=K.Q_a^{185}	[H_1/Q_a]	Corrected Flow ($Q_{a=}Q_a$)
	l/s	$m_{/s}^3$				
Loop 1234						
1-2	16.62	0.17	12666.66	6.74	396.47	15.48
2-3	16.62	0.17	6140.35	3.27	192.35	15.48
3-4	-6.70	.006	9429.82	-0.731	121.87	.43
4-1	-12.61	-.012	23659.30	-6.61	550.83	-13.502
Loop 4-3-7-6-5						
4-3	6.70	.006	9429.82	.731	121.87	-0.43
3-7	23.32	.023	9777.77	9.11	396.08	15.05
7-6	23.22	.023	6444.44	6.00	260.86	14.95
6-5	-2.62	-0.002	8222.22	-0.084	42	-14.61
5-4	-5.46	-0.005	14195.58	-0.785	157	-13.93
Loop 11-5-6-8-10						
11-5	-2.84	-0.0028	6888.88	-0.070	35	0.678
5-6	2.62	0.002	8222.22	.083	41.5	14.61
6-8	25.84	0.0025	6888.88	0.105	42	29.5
8-10	-5.47	-0.005	4444.44	-0.245	49	7.86
10-11	-9.09	-0.009	26813.88	-4.40	488.88	-3.57
Loop 13-1-4-5-11-12						
13-1	-19.83	-0.0198	8888088	-6.27	316.66	-19.62
1-4	12.16	0.0121	23659.30	6.71	554.54	13.50
4-5	5.46	0.005	14195.58	0.785	157	13.93
511	2.84	.002	6888.88	0.070	35	-0.678
11-12	-6.25	-0.006	7886.4	-0.61	101.66	-4.248
12-13	-11.75	-0.011	4888.88	-1.16	105.45	-13.378
Loop 14-10-8-9						
14-10	-3.62	-0.003	10222.22-	-0.219	73	-11.43
10-8	5.47	0.005	4444.44	0.245	49	-7.86
8-9	31.31	0.031	10222.22	16.54	533.54	21.7
9-14	-17.2	-0.0172	4444.4	-2.41	140.11	-26.81
Loop 15-12-11-10-14						
15-12	-5.5	-0.005	10222.22	-0.565	113	-9.13

12-11	6.25	0.006	7886.4	0.611	101.83	4.248
11-10	9.09	0.009	26813.88	4.40	488.88	3.57
10-14	3.62	0.003	10222.22	0.219	73	11.43
14-15	-13.58	-0.013	5111.11	-1.65	126.92	-15.38
Loop 16-13-12-15						
16-13	-8.08	-0.008	10444.44	-1.38	172.5	-6.25
13-12	11.75	0.011	4888.88	1.163	105.72	13.378
12-15	5.5	0.005	10222.22	0.565	113	9.13
15-16	-8.08	0.008	22082.1	-2.915	364.37	-6.25

TABLE 4.8 Corrected Discharges Second Iteration

pipes	Assumed flow		K value	$H_L=K.Q_a^{185}$	$[H_1/Q_a]$	Corrected Flow ($Q_q=Q_a+$)
	l/s	M^3/s				
Loop 1234						
1-2	15.48	0.0154	12666.66	5.61	364.79	15.42
2-3	15.48	0.0154	6140.35	2.72	176.84	15.42
3-4	0.43	0.0004	9429.82	0.005	12.5	-0.471
4-1	-13.502	0.0135	23659.30	-8.224	609.23	-11.67
Loop 4-3-7-6-5						
4-3	-0.43	0.0004	9429.82	-0.005	12.5	0.471
3-7	15.05	0.0150	9777.77	4.13	275.37	15.9
7-6	14.95	0.0149	6444.44	2.70	181.58	15.8
6-5	-14.61	-0.0146	8222.22	-3.30	226.58	-5.18
5-4	-13.63	-0.0139	14195.58	-5.22	376.22	-11.2
Loop 11-5-6-8-10						
11-5	0.678	0.0006	6888.88	0.007	11.66	-6.022
5-6	14.61	0.0146	8222.22	3.30	226.68	5.18
6-8	29.56	0.0295	6888.88	10.20	346.042	20.98
8-10	7.86	0.007	4444.44	0.458	65.48	-0.72
10-11	-3.57	-0.003	26813.88	-0.576	192.27	-12.75
Loop 13-1-4-5-11-12						
13-1	-19.62	-0.01962	8888.88	-6.17	314.51	-21.5
1-4	13.502	0.01350	23659.30	8.22	609.23	11.67
4-5	13.93	0.01393	14195.58	5.22	375.39	11.2

5-11	-0.678	-0.000678	6888.88	-0.0094	13.95	6.022
11-12	-4.248	-0.00424	7886.4	-0.321	75.88	-6.428
12-13	-13.378	-0.01337	4888.88	-1.66	124.85	-14.67
Loop 14-10-8-9						
14-10	-11.43	-0.0114	10222.22	-2.60	227.98	-11.73
10-8	-7.86	-0.007	4444.44	-0.458	65.48	.72
8-9	21.7	0.0217	10222.22	8.55	394.036	21.7
9-14	-26.81	-0.0268	4444.4	-5.49	205.12	-26.81
Loop 15-12-11-10-14						
15-12	-9.13	-0.009	10222.22	-1.67	186.48	-8.246
12-11	4.248	0.004	7886.4	0.288	72.21	6.428
11-10	3.57	0.003	26813.88	0.576	192.27	12.75
10-14	11.43	0.0114	10222.22	2.61	229.10	11.73
14-15	-15.38	-0.01538	5111.11	-2.26	147.03	-15.08
Loop 16-13-12-15						
16-13	-6.25	-0.006	10444.44	-0.809	134.99	-6.834
13-12	13.378	0.0133	4888.88	1.65	124.30	14.67
12-15	9.13	0.009	10222.22	1.67	186.48	8.246
15-16	-6.25	-0.006	22082.01	-1.72	285.40	-6.834

TABLE 4.9:Corrected Discharges: Third Iteration

pipes	Assumed flow		K value	$H_{L=}$ $_{K.}\,Q_A^{1.85}$	[H_l/Q_a	Corrected Flow (Q_a=Q_a+)
	l/s	M^3/s				
Loop 1234						
1-2	15.42	0.01542	12666.66	5.63	365.20	14.40
2-3	15.42	0.01542	6140.35	2.72	177.03	14.40
3-4	-0.471	-0.00047	9429.82	-0.0066	14.014	0.681
4-1	11.67	-0.01167	23659.30	-6.28	538.28	-12.57
Loop 4-3-7-6-5						
4-3	0.471	0.00047	9429.82	0.0066	14.01	-0.681
3-7	15.9	0.0159	9777.77	4.60	289.35	13.73
7-6	15.9	0.0159	6444.44	2.99	189.68	13.73
6-5	-5.18	0.00518	8222.22	-0.485	93.79	-8.82
5-4	-11.2	0.0112	14195.58	-3.49	311.88	13.25
Loop 11-5-6-8-10						

11-5	-6.022	0.006022	6888.88	-0.537	89.31	-4.434
5-6	5.18	0.00518	8222.22	0.485	93.79	8.82
6-8	20.98	0.02098	6888.88	5.413	258.03	22.45
8-10	-0.72	0.00072	4444.44	-0.00682	9.47	0.97
10-11	-12.75	-0.01275	26813.88	-8.385	657.72	-7.92
Loop 13-1-4-5-11-12						
13-1	-21.5	-0.0215	8888.88	-7.30	339.95	-21.61
1-4	11.67	0.01167	23659.30	6.28	538.28	12.57
4-5	11.2	0.0112	14195.58	3.49	311.88	13.25
5-11	6.022	0.006022	68888.88	0.537	89.31	4.434
11-12	-6.428	-0.00642	7886.4	-0.693	107.96	-3.186
12-13	-14.67	-0.01467	4888.88	-1.98	135.10	-14.64
Loop 14-10-8-9						
14-10	-11.73	0-01173	10222.22	-2.73	233.58	8.59
10-8	0.72	0.00072	4444.44	0.0068	9.47	-0.97
8-9	21.7	0.0217	10222.22	8.55	394.03	21.48
9-14	-26.81	0.02681	4444.44	-5.49	205.05	-27.03
Loop 15-12-11-10-14						
15-12	-8.246	0-00824	10222.22	-1.42	173.12	-11.46
12-11	6.428	0.0064	7886.4	0.694	108.07	3.186
11-10	12.75	0.01275	26813.88	8.38	657.72	7.92
10-14	11.73	0.0117	10222.22	2.73	233.58	-8.59
14-15	-15.08	0.0150	5111.11	-2.18	144.59	-18.44
Loop 16-13-12-15						
16-13	-6.834	0.00683	10444.44	-1.03	150.78	-6.97
13-12	14.67	0.01467	4888.88	1.98	135.10	14.64
12-15	8.246	0.008246	10222.22	1.427	173.12	11.46
15-16	-6.834	0.006834	22082.01	-2.17	318.79	-6.97

Table 4.10: Corrected Discharges: Fourth Iteration

Pipes	Assumed Flow		K Value	$H_L = K.Q_a^{1.85}$	$\lvert H_l/Q_a \rvert$	Corrected Flow ($Q_a = Q_a + \Delta$)
	l/s	m^3/s				l/s
Loop 1234						
1-2	14.40	0.01440	12666.66	4.96	344.56	14.31
2-3	14.40	0.01440	6140.35	2.40	167.03	14.31

3-4	0.681	0.00068	9429.82	0.0130	19.17	0.454
4-1	-12.57	0.01257	23659.30	-7.20	573.37	-11.69
Loop 4-3-7-6-5						
4-3	-0.681	0.00068	9429.82	0.0130	19.17	0.454
3-7	13.73	0.01373	9777.77	3.506	255.42	13.87
7-6	13-73	0.01373	6444.44	2.311	168.34	13.87
6-5	-8.82	-0.00882	8222.22	-1.30	147.74	-6.575
5-4	-13.25	-0.01325	14195.58	-4.76	359.77	-12.14
Loop 11-5-6-8-10						
11-5	-40434	-0.0044	6888.88	-0.305	68.85	-5.57
5-6	8.82	0.0088	8222.22	1.30	147.74	6.575
6-8	22.45	0.0224	6888.88	6.136	273.32	20.35
8-10	0.97	0.00097	4444.44	0.0118	12.20	1.83
10-11	-7.92	0.00792	26813.88	-3.47	438.80	-11.85
Loop 13-1-4-5-11-12						
13-1	-21.61	-0.0216	8888.88	-7.37	341.43	-22.57
1-4	12.57	0.0125	23659.30	7.207	573.37	11.69
4-5	13.25	0.01325	14195.58	4.76	359.77	12.14
5-11	4.434	0.0044	6888.88	0.305	68.85	5.57
11-12	-3.186	0.00318	7886.4	-0.189	59.51	-5.97
12-13	-14.64	0.01464	4888.88	-1.97	134.87	-14.78
Loop 14-10-8-9						
14-10	8.59	0.00859	10222.22	1.53	179.24	3.8
10-8	-0.97	-0.00097	4444.44	-0.011	12.20	-1.83
8-9	21.48	0.02148	10222.22	8.39	390.63	18.52
9-14	-27.03	0.02703	4444.44	-5.58	206.48	-29.99
Loop 15-12-11-10						
15-12	-11.46	0.01146	10222.22	-2.62	229	-8.813
12-11	3.186	0.00318	7886.4	0.189	59.51	5.97
11-10	7.92	0.00792	26813.88	3.47	438.80	11.85
10-14	-8.59	0.00859	10222.22	-1.53	179.24	-3.8
14-15	-18.44	0.01844	5111.11	-3.16	171.55	-16.61
Loop 16-13-12-15						
16-13	-6.97	-0.00697	10444.44	-1.068	153.33	-7.787
13-12	14.64	0.0146	4888.88	1.97	134.87	14.78
12-15	11.46	0.0114	10222.22	2.62	229	8.813
15-16	-6.97	0.0069	22082.01	-2.25	324.18	-7.787

4.4.2 **Result from Software**

After successful run of the software the following table gives the flow as computed for the given network in four iterations. The table also highlights the computed flow values taken from Table 4.7to 4.10

Table 4.11: Comparison of Computed vs. Calculated flow value

loop	Pipe	Flow (lps)		Other hydraulic parameters		
		Software computed	Manually calculation	Velocity m/s	Friction factor	Hard loss (m/km)
1-2-	1-2	13.27	14.31	0.75	0.038	7.26
3-4-	2-3	13.27	14.31	0.75	0.038	7.26
	3-4	0.65	0.452	0.04	0.059	0.03
	4-1	-11.66	-11.69	1.49	0.036	41.19
3-4-	3-4	-0.65	-452	0.04	0.042	0.03
5-6-	4-5	-12.31	-12.14	11.57	0.036	45.54
7	5-6	-6.50	-6.575	0.37	0.042	1.94
	6-7	12.62	13.87	0.71	0.038	6.61
	7-3	12.62	13.87	0.71	0.038	6.61
5-6-	5-6	6.50	6.575	0.37	0.042	1.94
8-	6-8	19.12	20.35	1.08	0.036	14.28
10-	8-10	-1.11	-1.83	0.06	0.055	0.07
11	10-11	-9.34	-11.85	1.19	0.038	27.31
	11-5	-5.81	-5.57	0.33	0.043	1.57
1-4	1-4	11.66	11.69	1.49	0.037	41.19
5-	4-5	12.31	12.14	1.57	0.036	45.54
11-	5-11	5.81	5.57	0.33	0.043	1.57
12-	11-12	-3.35	-5.59	0.45	0.044	4.51
13	12.13	-15.20	-14.78	0.86	0.037	9.33
	13-1	-23.68	-22.57	1.34	0.035	21.21
10-	10-8	1.11	1.83	0.06	0.055	0.07
8-9	8-9	20.24	18.52	1.15	0.036	15.86
14	9-14	-28.37	-29.99	1.61	0.034	29.65
	14-10	-8.23	-3.8	0.47	0.041	3.00

12-	12-11	3.53	5.97	0.45	0.044	4.51
11-	11-10	9.34	11.85	1.19	0.038	27.31
10-	10-14	8.23	3.80	0.47	0.041	3.00
14-	14-15	-20.144	-16.61	1.14	0.036	15.72
15	15-12	-11.67	-8.813	0.66	0.039	5.72
13-	13-12	15.20	14.78	0.86	0.037	9.33
12-	12-15	11.67	8.813	0.66	0.039	5.72
15-	15-16	-8.48	-7.787	1.08	0.038	22.82
16	16-13	-8.48	-7.787	0.48	0.040	3.17

CHAPTER 5: CONCLUSIONS

In this report hydraulic analysis of a network is made. Hydraulic design of water distribution network involves number of iteration so as to reach flow to in each pipe with minimum head loss. Hardy cross method has been used to solve the network for minimum head loss in four iterations. Some of the basic requirement of the Hardy Cross method such as population to be served, total demand, water supply pipe design parameters were either computed based on hypothetical data or appropriately assumed.

EPANET software, US-EPA freeware software is used to solve similar network.
The running of model required setting of model parameters to suit the present requirements. Successive run of the model where compared with the obtained from manual labor. It was found that values were quite nearly the same, barring few exceptions. This may be due to improper characterization of object properties, especially pump characteristics, as per the requirement of the software.

Solving a network manually is cumbersome, prone to human error and time consuming, that grows with the complexity of the network. Hence, use of powerful and user friendly software like EPANET makes the analysis easy and computable; provided the data need of the model carefully determinate and executed.

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