ASSESMENT OF ADEQUACY OF THE WATER DISTRIBUTION NETWORK OF GAZIPUR POURASHAVA

A Thesis
by

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entitled **Assessment of Adequacy of the Water Distribution Network of Gazipur Pourashava** be accepted as fulfilling this part of the requirements for the degree of Master of Science in Engineering (Water Resources).

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ABSTRACT

Water distribution network of Gazipur Pourashava has been analyzed to determine the water demand and pressure at each node of the water distribution pipe network and the amount of flow in each pipe. The demand at each node was estimated using population data and per capita consumption rate. Amount of flow in pipes and pressure at various nodes were computed using EPANET2 software which is developed by US Environmental Protection Agency. In this computer programme number of pipes, number of nodes, Hazen-Williams coefficient, nodal demand, elevation of each node, node to node relations along with length, diameter, starting node number and end node number of pipes and pump capacity curve were supplied as input data. From the study the flow and pressure at each node as well as flow in each pipe were computed. Actual flow and actual pressure at each node and actual flow in each pipe was obtained by field survey. The amount of estimated water demand and actual supply and actual pressure and computed pressure in each node were compared. Similarly computed demand and actual supply in each pipe were compared. In this study it is observed that about 55% nodes have excess supply, 17% nodes meet required demand and 28% nodes experience deficient supply. Generally pressure in the distribution system under normal operating condition is very low. In this study it is also observed that about 11% nodes have sufficient pressure and the pressure of 89% nodes varies from 0.00 psi to 4.5 psi. The consumers nearer the pumping stations get more water and has tendency to waste water. Analysis of the water distribution network of Gazipur shows that the computed pressure is higher than the actual pressure measured in field.

Given pumping capacity and extent of the water distribution network there is scope for improvement of supply situation. If the wasteful use of water and leakage of the supply system could be controlled, then all of the consumers could get water according to their required demand. This will require adequate management of water supply.

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TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

CHAPTER 1

INTRODUCTION

1.1 General

The increasing obsolescence of many urban pipeline systems used for the transport of water, gas, and steam, raises serious questions concerning the appropriate actions for repair, replacement and rehabilitation of deteriorating pipe sections. A clear need is developing for more scientific approaches to the assessment of various performance dimensions of pipelines, the evaluation of reliability of provided services, the accurate measurement of risk factors involved, and the scheduling of capital improvement need.

As a result of the increase in break rates and loss of carrying capacity and the deterioration of water quality in aging water distribution infrastructure, many studies were conducted in order to analyze failure patterns and attempt to evaluate and predict the performance of water distribution systems.

The performance of water distribution system network can be measured by a number of interrelated factors. These are stated as follows: the overall cost of maintaining and operating the system, the quality of water and the serviceability of the system in terms of both quantity and pressure delivered and the structural integrity and safety of system operation and the reliability of water supply, as is relates to the probability of meeting required service levels (Karaa & Marks, 1990).

The assessment of water main conditions for performance evaluation purposes requires the combination of a number of data sources. Due to the fact that water distribution infrastructure is buried, visual inspection of different aspects of water main deterioration is difficult. While testing of some of the performance measures such as carrying capacity and pumping costs is feasible, the reliance on past maintenance history is still an important component of the assessment task.

1.2 Importance for the Study

The water requirement of a modern city is so great that a system capable of supplying a sufficient quantity of potable water is essential. Without a water distribution system water must be pumped and carried by hand. Pumping and carrying by hand are strenuous and unpleasant, which result in the use of minimum amount of water. This is not good for health and sanitation. Handling of water by hand is also expensive. For these reasons it is much better and cheaper to use a water system of some kind in conjunction with a distribution system.

The impact of urbanization is felt more intensely in major cities and secondary towns of Bangladesh. Especially service facilities of these cities and secondary towns could not be expanded to cope with the rapid population growth. As a result it was not possible to meet the minimum service facilities for the citizen. Gazipur Pourashava is, therefore, experiencing continuous deterioration of service facilities for the dwellers.

The most needed service facility such as water supply needs special attention due to its priority in daily life. According to November 2002 estimate, the Gazipur Pourashava is capable of supplying only 1.04 million gallons of water per day for the population of about 123500 persons (Census, 2001). It is estimated that only 21.44% Pourashava population is enjoying supply of piped water at present and rest of the population is deprived from supply water (MIR, 2002).

The situation is likely to deteriorate further due to various economic and social reasons. The present water supply problems related to distribution system include inadequate supply of water against demand, high rate of loss and wastage, and inadequate pressure at service points. A properly designed distribution network is the vital element in water supply systems. It ensures proper distribution of water to meet various demands with adequate pressure at all service points and reduces losses and wastages in the system.

1.3 Objective of the Study

In the context of water supply problems in the secondary town like Gazipur Pourashava, it is essential to analyze the existing water supply network of the town to identify the deficiencies and to suggest improvements in the existing system. Thus the overall objective is to evaluate the performance of the water distribution system of Gazipur Pourashava in terms of demand and supply. The specific objectives are as follows:

(1) To estimate water demand at various junction points of the water distribution pipe network based on population served,

(2) To determine the amount of flow in each pipe and pressure at each node, given the amount of groundwater pumped and head available at pumping points, and

(3) To assess the adequacy of the water supply system.

CHAPTER 2

LITERATURE REVIEW

2.1 Historical Development of Water Distribution System

Water supply has its history, archeology, literature, science and technology as ancient as human civilization and culture. Waterworks structures are found in excavation of prehistoric ruins. The remains of Lake Moeris in Egypt indicates its construction about 2000 B. C. It was the largest of the reservoirs of the Nile Valley which is believed to supply water for 20,000,000 people.

The water supply of towns in very early times was derived from large tanks excavated on minor drainage lines which collected and stored the rainfall in the wet season to provide a supply during the dry periods. Especially notable are the structures of water supply of Mohenjodaro, Babilonia, Rome and Jerusalem.

The water for the city of Rome was brought from the surrounding hills aqueducts totaling about 385 miles in length. Among these aqueducts the Appia, Marcia, Claudia and Anionova were 11, 62, 46.5 and 58.5 miles long and were built in 312 B.C, 144 B.C, 50 A.D and 52 A. D. respectively. All aqueducts were constructed along the hydraulic grade line in order to avoid the necessity for building pressure conditions.

The numerous conduits which supply water to ancient Jerusalem are very old, no exact data can be assigned to their construction but they probably go back to the times of the kings of Judah, 600 to 900 B. C. The conduits were rock-cut canals partly built in masonry.

Wells were used at antediluvian periods in Greece, Italy, India and China to utilize the underground water. London was perhaps the first modern city in the world, in which at the end of the 16^{th} century lead pipes were used for conveyance or distribution of water. After that for many years wood pipes bored out of logs came to be used. Cast iron pipe for conveyance of water was laid in Philadelphia in the United States in 1804 and in London in 1807.

In our country, water supply on modern lines is comparatively of recent origin. The first water works for the supply of water to Dhaka city was completed by the Nawab of Dhaka (Sir Nawab Abdul Ghani) in 1876. After this, the Government at Chandpur, Chittagong and other places constructed water-works. The water works in Calcutta, Bombay, Madras and Poona was completed in the years 1870, 1875, 1880 and 1890 respectively.

Schemes for the collection of groundwater through handpump tubewells for community water supplies in rural Bangladesh were taken as early as 1928. In the context of very high prevalence of diarrhoeal diseases in Bangladesh, groundwater being usually free from disease producing microorganism received priority as a source of water for water supply. Since 1928, about 3 to 4 million hand tubewells in Bangladesh have been sunk to provide drinking water to 97% of the rural population.

2.2 Water Treatment and Distribution System

Natural waters usually contain impurities, which require treatment to make the water suitable for domestic water supplies. The type and degree of treatment required is dependent on the quality of water. In case of most surface waters, the treatment processes may involve removal of turbidity, color, taste and odour, and removal and destruction of pathogenic (disease producing) microorganism. Groundwater is relatively free form disease-producing bacteria but rich in mineral substances and may require removal of iron, hardness, arsenic, fluoride etc. If the dissolved minerals in groundwaters are within acceptable limits, the water may be supplied without any treatment. The most common methods used for treatment include screening, sedimentation, and aeration treatment with chemicals, filtration, demineralization, and disinfection.

A distribution system is needed to deliver water to individual consumers. The piped water supplies require distribution network of pipes with storage reservoirs, pumping devices, standposts, valves, and other appurtenances. In unpiped water supplies, the source of water is to be distributed to make easily accessible to the consumers. The rural water supply based on manually operated tubewells does not require a distribution network but the tubewells are required to be distributed over the area in such a way that the distances from the households are reasonable and each tubewell serves an optimum number of households. In the location of community type treatment plants, accessibility and distances from the community are required to be taken into consideration.

2.3 Per Capita Water Consumption

Water is used for various domestic purposes, such as drinking, cooking and preparation of food, bathing, cleaning, washing, personal hygiene, watering of vegetables, gardens, watering of livestock, sanitation, loss and wastage.

The per capita water consumption is greatly influenced by various factors. These include population distribution, climatic condition, quality of water, pressure of water, water rates and metering, nature of supply, water source, availability of an alternative source and sanitation.

In the rural areas in Bangladesh, the water requirements for various purposes have been estimated as follows:

Drinking	:	2-3 lpcd
Washing cloths	:	8-10 lpcd
Washing utensils	:	6-8 lpcd
Cooking foods	:	3-5 lpcd
Bathing	:	14-20 lpcd
Others	:	9-14 lpcd

The water requirements in rural and urban areas of Bangladesh, which are used for planning and design of water supply systems, are given in Table 2.1.

Table 2.1: Water requirements in rural and urban areas in Bangladesh

Areas	Water consumption, lpcd
Rural areas	50
Upazila towns	100
Zila towns	120
City corporation	180

The water requirements data mentioned here may be used for planning and preliminary design and may serve as a guide for final design (Ahmad and Rahman, 2000). Studies of existing water supply system in a similar area and collection of primary data by field survey can provide accurate and useful water usage data for final design.

2.4 Types of Water Distribution Systems

A water works distribution system includes pipes, valves, hydrants and appurtenances for conveying water; reservoirs for storage, equalizing and distribution purposes; service pipes to the consumers, meters and all other parts of the conveying system after the water leaves the main pumping station or the main distribution reservoirs. The main purposes of the construction of water transmission and distribution pipelines are:

- to make water available in close proximity to the consumers;
- to supply water in adequate quantities according to the demand of the consumers;
- to supply water with adequate pressure;
- to regulate water supply as per requirement.

The layout of distribution system may be classified for convenience, as

(i) Tree or branch or dead end system

(ii) Grid iron system

(iii) Circle or ring system

(iv) Radial system

2.4.1 Dead End System

Dead end system, also known as tree or division system, consists of a simple main, which goes on diminishing, in size. The small pipe takes off from the main known as branch as shown in Figure 2.1. This system is suitable for irregular growing towns.

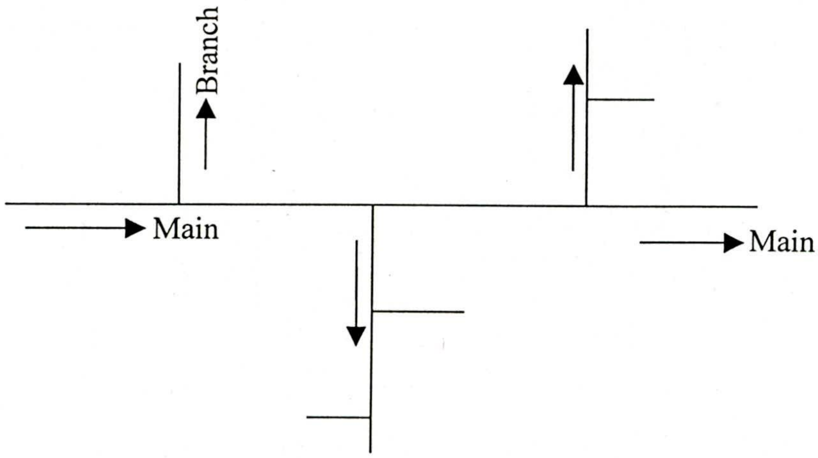


Figure 2.1: Tree or Branch or Dead end System

The pipes can be added as the town develops. There are many dead ends in the system, which cause stagnation of water. Also, in case any repair is to be done, the area beyond this point will go without water. However, the advantages are that this method will have lesser number of valves and pipe sizes are easy to calculate.

2.4.2 Grid Iron System

It is an improvement over the branch system, caused by connecting the ends of the various branched pipes so as to eliminate the dead ends. The water then circulates freely through the system. Such a system is very useful for a city laid out on a rectangular plan (Figure 2.2). The connections of the dead end producing a grid iron pattern, with mains running on main roads in one direction or in perpendicular directions and sub mains also running alike on minor roads and streets.

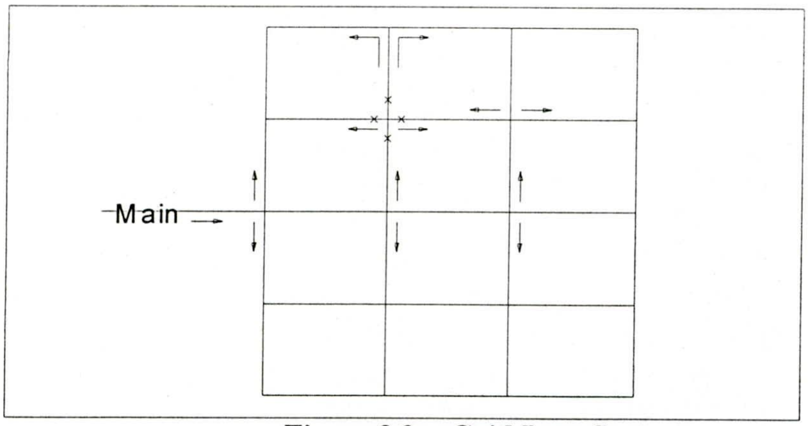


Figure 2.2: Grid Iron System

Advantages to be gained with this system are (i) avoidance of any stagnation due to continuous water circulation and (ii) absence of the discontinuity of water supply anywhere in the system in the event of any repair work to a main or sub-main. Disadvantage is that a large number of valves are to be provided.

2.4.3 Circle or Ring System

This consists of dividing the entire district into circular or rectangular blocks and laying the main along the peritoneal roads with sub-mains branching but from the mains and running on the inner roads and streets as shown in Figure 2.3. Water can be supplied to any point from at least two directions.

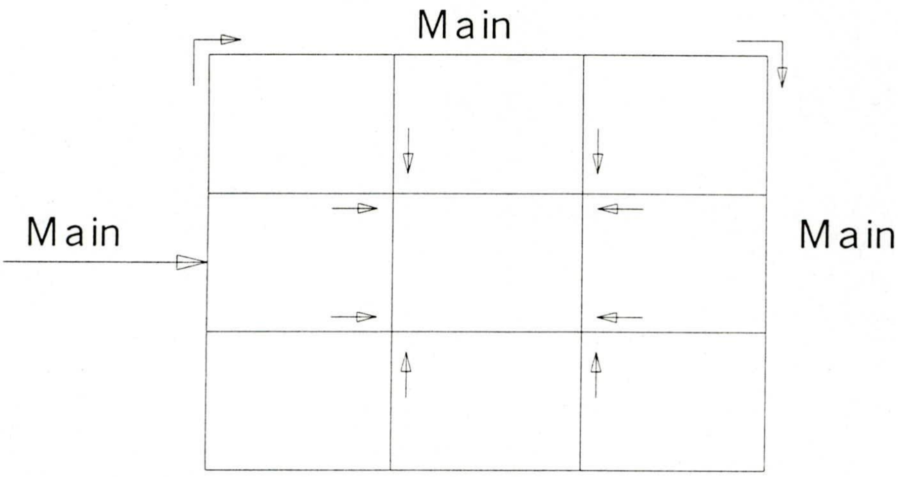


Figure 2.3: Circle or Ring System

2.4.4 Radial System

This system is the reverse of the ring system. The water is pumped into the distribution reservoirs situated in the middle of each zone as shown in Figure 2.4 and the supply pipes are laid radially ending towards the boundary of the area to be served. It provides quick service. The calculation of pipe size is easy. This system is suitable when the town has a radial road layout.

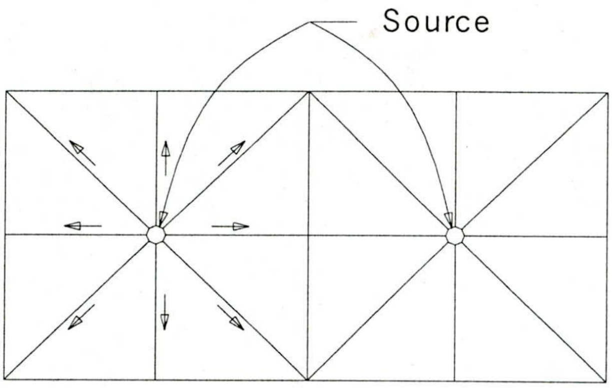


Figure 2.4: Radial System

As a matter of fact no city follows one system alone. A combination of several systems is often employed depending upon the local conditions.

2.5 Nonrevenue Water

Nonrevenue Water (NRW) which is the difference between the quantity of water entering the system and the sum of water which is measured but for some reasons not paid for (Chowdhury et al., 2002), i.e.

$$Q_{ufw} = Q_{prod.} - Q_{sold}$$

Nonrevenue water can be categorized into two major heads (DWASA, 1996b):

- Technical water losses, caused by technical defects in the distribution system, and
- Commercial/Administrative non-revenue water originated from deficient billing procedure.

Major sources contributions to technical losses are:

- Leaking pipes;
- Leaking service connection;
- Leaking operational fittings (valves, hydrants etc.);
- Leakage, seepage and losses from overhead tanks and pump stations.

Sources contributing to commercial nonrevenue water are:

- Inaccurate consumer database
- Un-metered connections;
- Illegal and illegally reconnected service connection;
- Bypass connections;
- Inaccurate meters;
- Reverse fixing of the meters;
- Inaccurate meters;
- Faulty reading and billing,
- Wrong consumer classification etc.
- Free water supply from water tanker etc.

Inaccurate consumer database originates from inaccurate consumer information leading to under billing and Commercial Nonrevenue Water.

Unmetered customers are charged with either a flat rate, or a construction rate or not at all (forgotten customer). However, water consumption habits conform mainly to those customers with a broken meter.

Illegal connection means those connections, which have no legal documents and are not recorded with the concerned revenue offices.

By-pass lines means the service connections taken bypassing the meters.

Inaccurate water meters refer mainly to under-registration of meters. This includes partially also such meters with a relatively high starting flow, which creates revenue losses especially during periods of minimal system pressure (e.g. starting flow (Q_{st})) for a standard ¾" meter should be normally 40 1/hr, if the actual starting flow of a meter is say 60 1/hr, then at each such service connection 20 1/hr are lost.

Broken/tampered water meters normally lead to an average or minimum billing which is generally lower than the actual consumption because the customer has no incentive to safe water as nobody gets an accurate consumption figure. Thus the customer tends to draw as much water as possible.

Wrong consumer classification is attributed to improper consumer category aiming at lower billing rates. For example commercial consumers are charged with residential rates.

Other issues contributing to administrative losses are reading errors on the water meter, intentionally or not, errors made during conversion from the meter reading to the water bill, loss of customer cards/books and subsequent non-reading of these meters, non enforcement of payment by customers.

2.6 Previous Works

The water distribution system needs to be designed in such a way that the system will meet the water demand at various nodes at required pressure head. But the present water supply systems in different urban areas in Bangladesh suffer from problems of inadequate supply, high rate of loss and wastage, and inadequate pressure at service points. Several evaluation studies have been performed in Bangladesh. Most of these relate to evaluation of Nonrevenue Water (NRW). After putting a water distribution in service, it is necessary to evaluate if a given system can meet the estimated demands at different supply points or nodes.

Some of the relevant studies published in the literature are summarized below:

Karaa and Marks (1990) proposed that the performance of water distribution network could be measured by the cost of maintaining and operating the system. Hydraulic condition can be evaluated through pressure testing and customer low pressures complain. Water loss conditions are monitored through leak detection programs.

Wagner et al. (1988a) developed an analytical methods for calculation of useful probabilistic reliability measures for water distribution systems. Measures of connectivity and reachability are fairly easy to calculate only for moderately sized, complex systems. Connectivity and reachability measures can be used to identify basic sources of unreliability in a system, such as lack of network interconnections or extremely unreliable links.

Wagner et al. (1988b) determine heads and flows throughout the system with no failure by solving the network by a simulation model. The simulation proceeds taking into account the randomly generated failure times of the pipes and pumps according to the specified failure time distribution. When a link fails, it is removed from the system. The new heads at the demand nodes in the reduced network are determined by solving it again. It is assumed that link failures leave the demands unchanged .The new heads at the demand nodes are used to judge how the system is performing.

Wood (1980) used hydraulic simulation model to determine pressure heads for the nodes throughout the water distribution system. After a certain number of iterations, the nodal or system reliability was computed.

Bao and Mays (1990) used a methodology to estimate the nodal system reliabilities of a distribution system accounting for uncertainty using Monte Carlo simulation.

Damelin et al. (1972) first proposed the use of reliability techniques to design a water distribution system. They measure the reliability of being able to meet demand, which is affected by the random failure of the delivery system.

Rossman (1993) developed EPANET2 hydraulic simulation model which computes junction heads and link flows for a fixed set of reservoir levels, tank levels, and water demand 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 based on 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.

In pilot study Haskoning & IWACO (1980) conducted under the feasibility study and master planning for Khulna water supply system, it was revealed that nonrevenue water amounted to 50-70% of the daily water supply.

LGED (1993) conducted a study on water distribution system leak detection in the district towns of Sylhet, Pabna, Kushtia, and Cox's Bazar. Non-revenue water estimated by the Water Supply and Sewerage Authorities of Dhaka and Chittagong (DWASA and CWASA) in different years have been summarized by Chowdhury et al. (1997, 1998) and Ahmed (2002).

In an effort to improve performance, Dhaka WASA (1996a) undertook such works as proper meter reading, billing, collection, meter installation/replacement, disconnection and reconnection activities etc. The work was done in Lalbag and Dhanmondi zones. The target and achievement were primarily intended to be based on revenue billed per period, collection achieved per period, the level of accounts receivable, the level of unaccounted for water in the zones and the number of new consumers connected. Determination of available water in the two zones was required to identify the level of nonrevenue (system losses) in the zones as used as to fix the target of achievement. Hydraulic analysis of the distribution system in the two pilot areas of Dhaka city was carried out to check whether amount of water supply was sufficient to meet the required demand and match with the available pressure found from the field observations, so that the necessary recommendation for further improvement of the system could be made.

Bari (1986) developed a computer model to simulate unsteady flow in water supply pipe networks. The model is based on the numerical solution of continuity and momentum equations. The method of characteristics has been used to integrate those non-linear and hyperbolic types of partial differential equations. The grid broken characteristic method has been tested in this study. This modified method has been used in order to overcome the restrictions on computational time step imposed by conventional method of characteristics. Finally, as a practical test, the model has been applied to study the existing as well as the proposed water supply system of Baridhara residential area.

Hossain (1985) developed a numerical model for the analysis and design of water distribution system of Dhaka City. The model is modified version of the earlier proposed by Shamir and Howard (1968). Application of Hazen-Williams equation for steady condition to a water distribution network results in a system of simultaneous non-linear equations, which has been linearized by Newton-Raphson method. To reduce computational time and computer storage, a banded matrix algorithm, based on Gaussian elimination has solved the linearized system of equations.

CHAPTER 3

METHODOLOGY

3.1 Study Area

Most of the studies related to urban water distribution in Bangladesh have been with large cities. For this study the Gazipur Porashava area has been chosen for assessment of the water distribution network. The study area as shown in Figure 3.1. The study area has selected considering data availability, proximity and size of the distribution system that can be handled within the scope of such study.

In GazipurPourashava area a small scale water distribution system was first developed in the early sixties. In district towns Department of Public Health Engineering (DPHE) generally develops water distribution system and operation and maintenance of these systems are done by Pourashava (municipalities). In 1981 DPHE took over the scheme and started the improvement and expansion of the water supply system under the 12 District Town Project with assistance of the Netherlands Government. Preliminary design and detailed design were completed in 1983 and 1986 respectively (Manual, 1990). After works the Local Government and Engineering Department took up project for expansion and rehabilitation of existing urban water distribution systems.

3.2 Steps of the Methodology

The steps of methodology can be stated as follows:

(1) The pipe network map for the area was collected from Gazipur Poursava.

(2) Pipe network diagram with pipe dimensions, locations of deep tube wells with discharge and head was collected from Pourashava.

(3) Number of population served by each of four pumps was estimated using available population data and household survey. For this an information sheet and a questionnaire were designed, tested in field and used for household survey.

(4) The number of junction points or nodes was counted from the pipe network diagram and average demand at each node was calculated by field survey data.

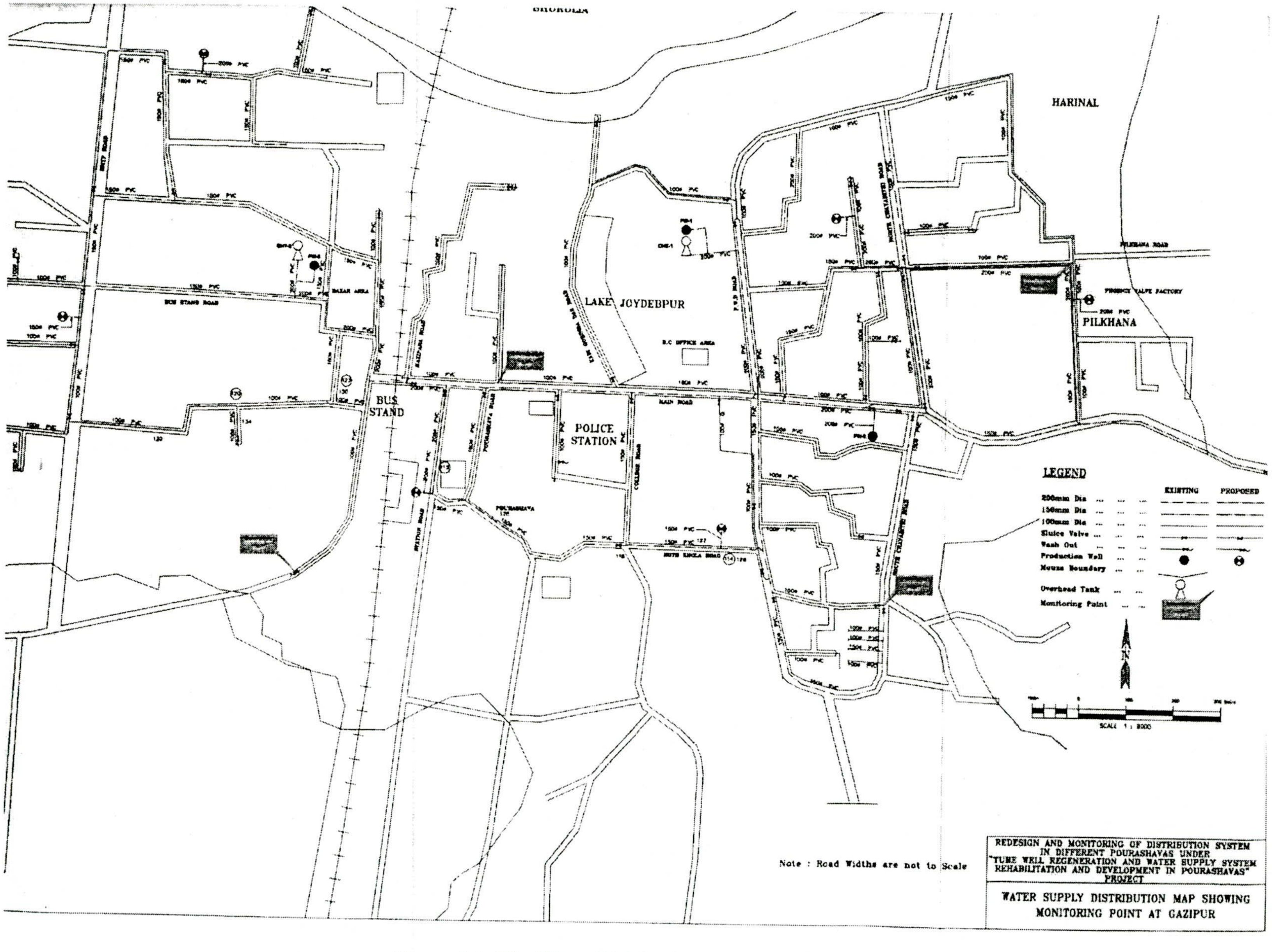


Figure 3.1 The Water Distribution Network of Gazipur Pourashava

(5) It is intended that EPANET2 used for the proposed distribution of flow in each pipe were obtained using this software given the discharge and head at the points of the deep tube well and the demand at each node.

(6) The amount of estimated water demand and actual supply and actual pressure and computed pressure in each node were compared. Similarly computed demand and actual supply in each pipe were compared to assess the adequacy of supply.

3.3 Description of EPANET2

EPANET2 is a computer program that performs extended period simulation of hydraulic and water quality behavior within pressurized pipe networks. A network consists of pipes, nodes (pipe junctions), pumps, valves and storage tanks or reservoirs. EPANET2 tracks the flow of water in 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 (Rossman, 1993).

EPANET2 is designed to be a research tool for improving our understanding of the movement and fate of drinking water constituents within distribution system. It can be used for many different kinds of applications in distribution system analysis. Sampling program design, hydraulic model calibration, chlorine residual analysis, and consumer exposure assessment are some examples. EPANET2 can help assess alternative management strategies for improving water quality throughout a system. In this study the EPANET2 is used to determine the amount and direction flow in link and pressure at a nodal points in the pipe network.

3.2.1 Physical Components of EPANET2

EPANET2 models a water distribution system as a collection of links connected to nodes. The links represents pipes, pumps and control valves. The nodes represent junctions, tanks and reservoirs.

Junctions

Junctions are points in the network where links join together and where water enters or leave the network. The basic input data required for junctions are:

- elevation above some reference (usually mean sea level)

- water demand (rate of withdrawal from the network)
- initial water quality.

The output results computed for junctions at all time periods of a simulation are:

- hydraulic head (internal energy per unit weight of fluid)
- pressure
- water quality

Junctions can also:

- have their demand vary with time
- have multiple categories of demands assigned to them
- have negative demands indicating that water is entering the network
- be water quality source where constituents enter the network
- contain emitters (or sprinklers) which make the outflow rate depend on the pressure.

Reservoirs

Reservoirs are nodes that represent an infinite external source or sink of water to the network. They are used to model such things as lakes, rivers, groundwater aquifers, and tie-ins to other systems. Reservoirs can also serve as water quality source points. The primary input properties for a reservoir are its hydraulic head (equal to the water surface elevation if the reservoirs is not under pressure) and its initial quality for water quality analysis. 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. However its head can be made to vary with time by assigning a time pattern to it.

Tanks

Tanks are nodes with storage capacity, where the volume of stored water can vary with time during a simulation. The primary input properties for tanks are:

- bottom elevation (where water level is zero)
- diameter (or shape if non-cylindrical)
- initial, minimum and maximum water levels
- initial water quality

The principal outputs computed over time are:

- hydraulic head (water surface elevation)
- water quality

Tanks are required to operate within their minimum and maximum levels. EPANET2 stops outflow if a tank is at its minimum level and stops inflow if it is at its maximum level. Tanks can also serve as water quality source points.

Pipes

Pipes are links that convey water from one point in the network to another. EPANET2 assumes that all pipes are full at all times. Flow directions is from the end at higher hydraulic head (internal energy per weight of water) to that at lower head. The principal hydraulic input parameters for pipes are:

- start and end nodes
- diameter
- length
- roughness coefficient (for determining head loss)
- status (open, closed, or contains a check valve).

The status parameter allows pipes to implicitly contain shutoff (gate) valves and check (non-return) valves (which allow flow in only one direction).
The water quality inputs for pipes consist of:

- bulk reaction coefficient
- wall reaction coefficient

Computed outputs for pipes include:

- flow rate
- velocity
- headloss
- Darcy-Weisbach friction factor
- average reaction rate (over the pipe length)
- average water quality (over the pipe length)

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

- Hazen-Williams formula
- Darcy-Weisbach formula
- Chezy-Manning formula

The Hazen-Williams formula is the most commonly used headloss formula in the US. It cannot be used for liquids other than water and was originally developed for turbulent flow only. The Darcy-Weisbach formula is the most theoretically correct. It applied over all flow regimes and to all liquids. The Chezy-Manning formula is more commonly used for open channel flow.

Pumps

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). In lieu of a pump curve, the pump could be represented as a constant energy device, one that supplies a constant amount of energy (horsepower or kilowatts) to the fluid for all combinations of flow and head. The principal output parameters are flow and head gain. Flow through a pump is unidirectional and EPANET2 will not allow a pump to operate outside the range of its pump curve.

Variable speed pumps can also be considered by specifying that their speed settings be changed under these same types of conditions. By definition, the original pump curve supplied to the program has a relative speed setting of 1. If the pump speed doubles, then the relative setting would be 2; if run at half speed, the relative setting is 0.5 and so on. Changing the pump speed shifts the position and shape of the pump curve.

As with pipes, pumps can be turned on and off at preset times or when certain conditions exist in the network. A pump's operation can also be described by assigning if a time pattern of relative speed settings. EPANET2 can also compute the energy consumption and cost of a pump. Each pump can be assigned an efficiency curve and schedule of energy prices. If these are not supplied then a set of global energy options will be used. Flow through a pump is unidirectional. If system conditions require more head than the pump can produce, EPANET2 shuts the pump off. If more than maximum flow is required, EPANET2 extrapolates the pump curve to the required flow, even if this produces a negative head. In both cases a warning message will be issued.

Pump Curve

A pump Curve represents the relationship between the head and flow rate that a pump can deliver at its nominal speed setting. Head is the head gain imparted to the water by the pump and is plotted on the vertical (Y) axis of the curve in feet (meters). Flow rate is plotted on the horizontal (X) axis in flow units. A valid pump curve must have decreasing head with increasing flow.

3.2.2 Hydraulic Considerations in EPANET 2

The method used in EPANET2 to solve the flow continuity and headloss equations that characterize the hydraulic state of the pipe network at a given point in time can be termed a hybrid node-loop approach. Todini and Pilati (1987) and later Salgado et al. (1988) call it the "Gradient Method".

Considering the network having N junction nodes and NF fixed grade nodes (tanks and reservoirs). Let the flow-headloss relation in a pipe between nodes i and j be given as:

$$H_i - H_j = h_{ij} = rQ_{ij}^n + mQ_{ij}^2 \qquad \textbf{(3.1)}$$

Where H = nodal head, h = headloss, r = resistance coefficient, Q = flow rate, n = flow exponent, and m = minor loss coefficient. The value of the resistance coefficient will depend on which friction headloss formula is being used (see below). For pumps, the headloss (negative of the head gain) can be represented by a power law of the form:

$$h_{ij} = -\omega^2 (h_0 - r(Q_{ij}/\omega)^2) \qquad \textbf{(3.2)}$$

Where h_o is the shutoff head for the pump, ω is a relative speed setting and r and n are the pump curve coefficients. The second set equation that must be satisfied is flow continuity around all nodes:

$$\sum_j Q_{ij} - D_i = 0 \qquad \text{for } i = 1, \ldots N. \qquad \textbf{(3.3)}$$

where D_i is the flow demand at node i and by conversion, flow into a node is positive. For a set of known heads at the fixed grade nodes, we seek a solution for all heads H_i and flows Q_{ij} that satisfy Eqs. (3.1) and (3.4).

The Gradient solution method begins with an initial estimate of flows in each pipe that may not necessarily satisfy flow continuity. At each iteration of the method, new nodal heads are found by solving the matrix equation:

$$AH = F \tag{3.4}$$

Where A = an (NxN) Jacobian matrix, H = an (Nx1) vector of unknown nodal heads, and F = an (Nx1) vector of right hand side terms.

The diagonal elements of the Jacobian matrix are:

$$A_{ij} = \sum_j p_{ij} \tag{3.5}$$

while the non-zero, off-diagonal terms are:

$$A_{ij} = -p_{ij} \tag{3.6}$$

Where P_{ij} is the inverse derivative of the headloss in the link between nodes i and j with respect to flow. For pipes,

$$p_{ij} = \frac{1}{nr|Q_{ij}|^{n-1} + 2m|Q_{ij}|} \tag{3.7}$$

while for pumps

$$P_{ij} = \frac{1}{n\omega^2 r(Q_{ij}/\omega)^{n-1}} \tag{3.8}$$

Each right hand side term consists of the net flow imbalance at a node plus a flow correction factor:

$$F_i = \left(\sum_j Q_{ij} - D_i\right) + \sum_j y_{ij} + \sum_f P_{if} H_f \tag{3.9}$$

where the last term applies to any links connecting node i to a fixed grade node f and the flow correction factor y_{ij} is:

$$y_{ij} = p_{ij}\left(r|Q_{ij}|^n + m|Q_{ij}|^2 \operatorname{sgn}(Q_{ij})\right) \tag{3.10}$$

for pipes and,

$$y_{ij} = -p_{ij}\,\omega^2\left(h_0 - r(Q_{ij}/\omega)^n\right) \tag{3.11}$$

for pumps, where sgn(x) is if $x > 0$ and -1 otherwise, (Q_{ij} is always positive for pumps.)

After new heads are computed by solving Eq. (3.4), new flows are found from:

$$Q_{ij}=Q_{ij}-\left(y_{ij}-p_{ij}\left(H_i-H_j\right)\right) \qquad \textbf{(3.12)}$$

If the sum of absolute flow changes relative to the total flow in all links is larger than some tolerance (e.g., 0.001), then Equations. (3.4) and (3.12) are solved once again. The flow update formula (3.12) always results in flow continuity around each node after the first iteration.

3.2.3 Hydraulic Simulation Process

A scheme of the algorithm used in EPANET2 to perform the hydraulic simulation is shown in Figure 3.2. After an initialization phase, Loop A simulates the network hydraulic behavior over an extended period of time, iterating for successive time steps for the duration of the simulation period. The most important and computationally expensive task is Loop B, in which the system of nonlinear equations is solved. A linear system of equation is solved by means of the Choleski factorization in each iteration of the loop, until convergence is achieved.

3.4 Calculation of Demand Discharge

For calculation of demand discharge a field survey was carried in Gazipur Pourashava area and the number of house connections were determined in each pipeline. According to Guidelines for the Monthly Management Information System for Twelve and Eighteen District Town Projects, the following assumptions were made for the calculation of demand discharge at nodal points (Guidelines, 1994):

- for house connection, water consumption/head/day = 120 liters
- for street hydrants, water consumption/head/day = 30 liters
- number of users per house connection = 10 persons
- number of users per street hydrants = 200 persons

3.5 Actual Pressure at Nodal Points and Actual Supply through Pipelines

For the determination of actual pressure at nodal points and actual supply through pipelines field surveys were carried on. By using pressure gauge pressures at nodal points were measured. Fixed amount of water was collected from one or more house connection(s) and the corresponding time was recorded and thereby flow rate was measured.

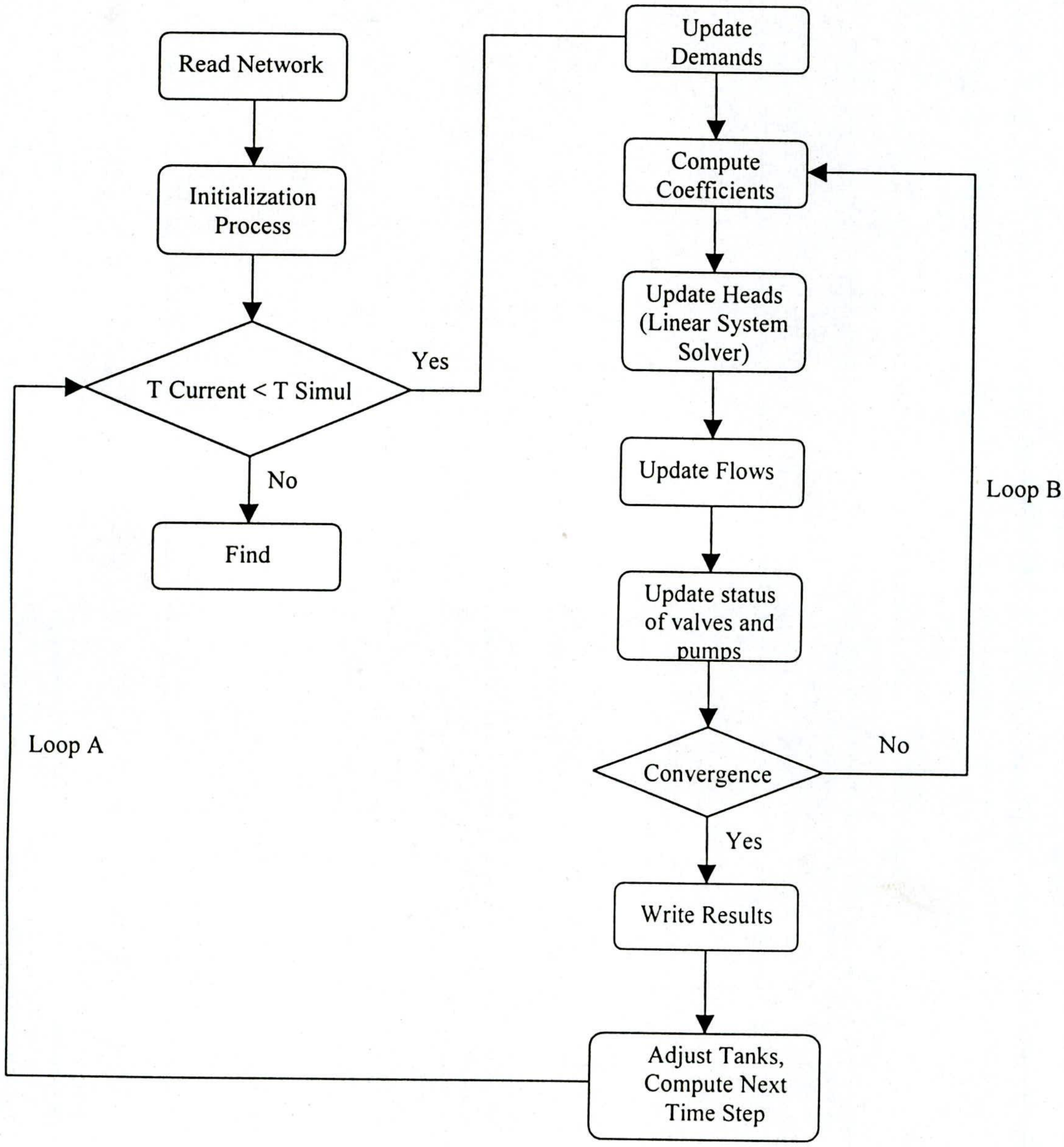


Figure 3.2 Flowchart for Hydraulic Simulation

Thereafter the total amount of water flowing through a link was calculated by multiplying the flow rate by the number of house connections from a particular pipeline.

3.6 Assessment of the Adequacy of Supply

EPANET2 software was used to compute junction heads and link flows for a fixed set of reservoir levels, tank levels and water demands. The values of junction heads and link flows obtained as an output from the EPANET2 software were compared with the field survey results of the same parameters and finally the adequacy of the supply system in terms of junction heads and link flows was assessed.

CHAPTER 4

DATA COLLECTION AND ANALYSIS

4.1 Data Need

For this study the following data are required:

i) The pipe network map with pipe dimensions, for the study area to determine the X and Y coordinates to plot network diagram by EPANET2 for analysis.
ii) Locations of deep tubewells with discharge and head.
iii) Number of population served by each pump was estimated using available population data and household survey.
iv) The number of junction points or nodes was counted from the pipe network diagram and average demand at each node was calculated by field survey data.
v) The water billed and revenue collection data were collected from Gazipur Pourashva.
vi) The water production and static water level below ground surface were also collected.
vii) The reduce levels of all nodal points were determined by field survey.

The pipe network diagram, pumping capacity, water billed and revenue collection data, static water level below ground surface etc. were collected from Gazipur Pourashava office.

4.2 Description of Gazipur Water Distribution System

The distribution system consists of a network, which can be divided into three zones . Each zone has a production well and can be operated as a separate system if the connections with the other zones (sluice valves) are closed. Zone 1 consists of the part of the town west of the railway. Zone 2 is the middle part, from the railway to the jail. Zone 3, the east part of the town, is the only zone without a storage reservoir. In zones 2 and 3 both old and new pipe lines exist. Some interconnections are made between the old and new lines.

The inventory of existing water supply system of Gazipur Pourashava is shown in Table 4.1. The Gazipur Pourashava water supply is not metering system. They collect water revenue per connection. The water charge per connection varies from 90/- to 160/- Tk depending on the diameter of house connection. The operating cost and revenue of Gazipur Pourashava are shown in Table 4.2. The service indicators of the water distribution system of Gazipur Pourashava are shown in Table 4.3. It is seen from Table 4.3 that about one fifth population of Pourashava consume the supply water. Many of the consumers who uses the supply water do not pay their monthly water tariffs in due time. The water billed and revenue collection are shown in Figure 4.1.

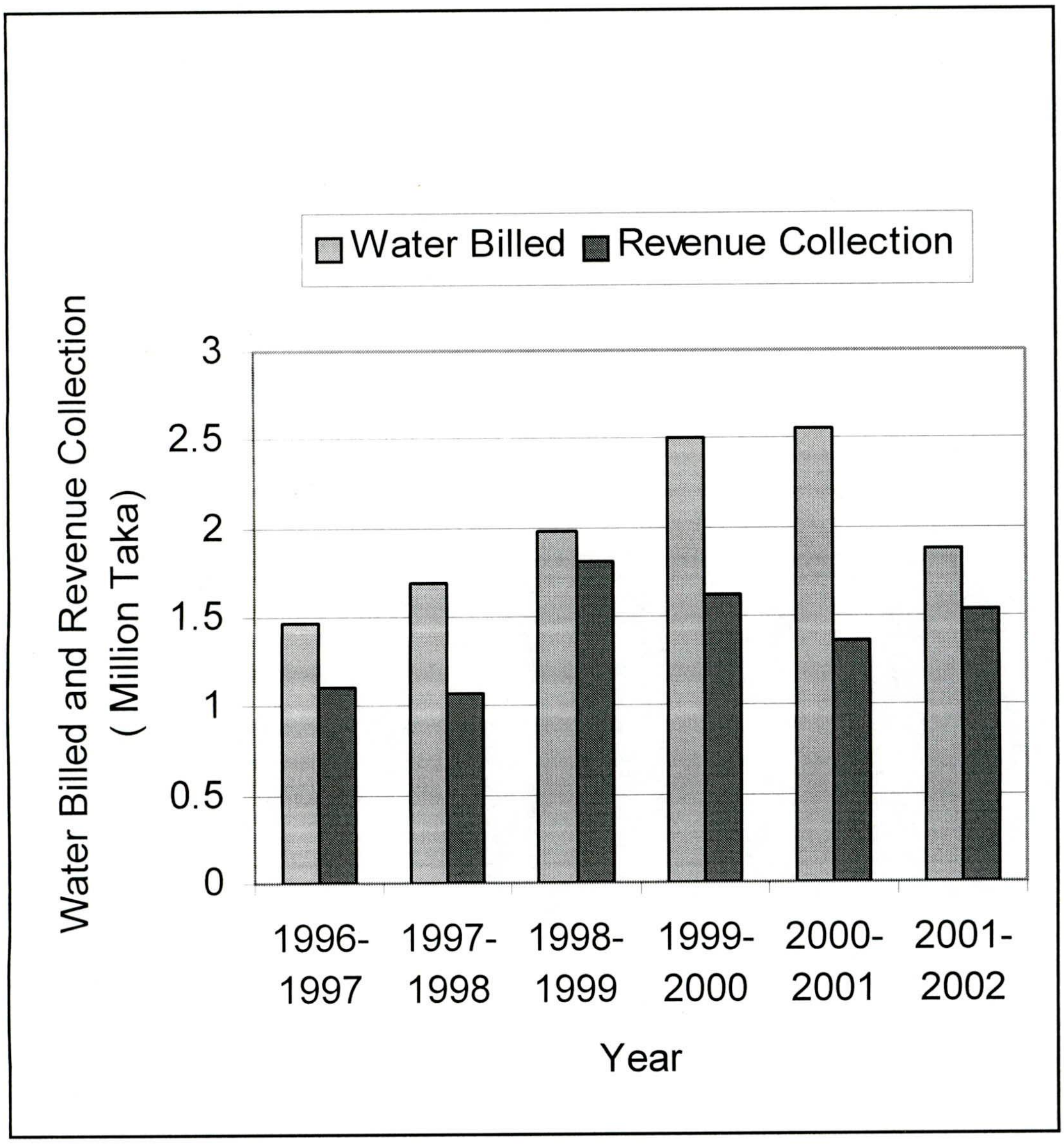


Figure 4.1: Water Billed and Revenue Collection

Table 4.1: Inventory of Existing Water Supply System of Gazipur

Sl. No.	Description	1997	1998	1999	2000	2001	2002
1	Holding no.	8500	9150	9950	10320	10756	12120
2	No. of Production Well	3	3	4	4	4	4
3	Overhead Tank	2	2	2	2	2	2
4	Length of pipeline	14.42	15.09	15.09	15.09	15.89	17.71
5	No of service connection	1524	1577	1649	1712	1752	1828
6	Stand post (operation)	35	35	35	35	35	30
7	*Running Tube well	580	580	591	590	590	589
8	Road (Pucca and semipucca in km)	180	195	202	210	217	217.58
9	Total storage capacity of overhead tank (m^3)	454	454	454	454	454	454
10	**PWSS staff	10	10	10	10	10	10
11	Daily water production (m^3)	3902	4742	3874	4186	4627	3936
13	Daily pumping hour	10	10	10	10	10	10

* Supplied by Department of Public Health Engineering
** Pourashava Water Supply System

Table 4.2: Operating Cost and Revenue Data[1]

Sl. No.	Description	1997	1998	1999	2000	2001	2002
1	Electric bill per month (Tk)	58372	60032	56155	65725	70042	64170
2	Staff salary per month (Tk)	25372	47282	41337	65732	40935	42668
3	Total operating cost(Tk)	92188	95152	90767	57159	185750	125762
4	Average monthly billed (Tk)	121763	140278	164674	209332	212915	156348
5	Average monthly collection(Tk)	92705	89918	150992	134686	114473	127676
6	Bank balance at the end of june(Tk)	197857	65693	144393	183325	583224	-
7	Consumers bill arrears at the end of june(Tk)	30716	32140	90140	51330	34960	344065
8	Other cost (Tk)	500	487	560	550	600	18924

[1]Management Information Report

Table 4.3: Service Indicator

Sl. No.	Description	1997	1998	1999	2000	2001	2002
1	Coverage of supply water(%)	15.60	15.98	16.38	16.85	17.29	21.44
2	Hand tube well coverage (%)	20.86	23.32	23.32	23.32	23.32	23.32
3	Water tariff in Tk per 1/2" size connection	60	60	60	60	60	90/-
4	No. of connection /km of pipe line	105	105	105	105	110	103

4.2.1 Pumping Capacity and Delivery Head

Ground water is the main source of water supply in Gazipur Pourashava. At present Gazipur Pourashava operates four deep tube wells. The underground water is drawn by these deep tube wells and pumped to the consumers. These tube wells are producing 1.04 mgd. The average depth of these tube wells varies from 400 to 455 feet (Manual, 1990). In Gazipur no water treatment plant is necessary, since the ground water is of sufficient quantity and acceptable quality. The discharge of each pump is measured by orifice meter once every three months. The measured discharge against delivery head are shown in Table 4.4. The well characteristics are shown in Table 4.5.

Table 4.4: The Measured Water Production against Delivery Head

Pump Location	Q (gpm)	Delivery Head (ft)
Market	532	157
Chayabithi	364	80
Rajbari	478	132
Bilashpur	508	82

4.2.2 Overhead Storage Reservoir

There are two elevated reservoirs in Gazipur Pourashavah having an aggregate capacity of 120000 gallons; however one of these two is not in used. The reservoirs are used to maintain constant pressure and to meet the peak demand.

Table 4.5: Well Characteristics Data

Well no.1	Location	Rajbari water works compound
	Year of installation	1967
	Year of commission	1995
	Well capacity (Q)	114.66 m^3 / hr
	Depth of the well	121.91 m
	Length of the housing pipe	36.57 m
	Length of the screen	24.38 m
Well no.2	Location	Chayabithi (Jorpukurpar)
	Year of installation	1985
	Year of commission	1998
	Well capacity (Q)	82.04 m^3 / hr
	Depth of the well	138.67 m
	Length of the housing pipe	43.27 m
	Length of the screen	30.48 m
Well no.3	Location	Market
	Year of installation	1996
	Year of commission	1996
	Well capacity (Q)	127.87m^3 / hr
	Depth of the well	132.62 m
	Length of the housing pipe	43.29 m
	Length of the screen	35.06 m
Well no.4	Location	North Bilashpur
	Year of installation	1999
	Year of commission	2000
	Well capacity (Q)	127.87 m^3 / hr
	Depth of the well	134.62 m
	Length of the housing pipe	36.58 m
	Length of the screen	30 m

4.2.3 Water Distribution Mains

Gazipur Pourashava has water distribution system consisting of 17.71km mains. Diameter of water mains varies from three to eight inches. The distribution system may be divided into primary and secondary water mains. The primary mains bring water from the sources. The diameter of these mains is six to eight inches.

The four and three inches diameter pipe is designated as secondary mains. The type and length of the pipe are shown in Table 4.6.

Table 4.6: Type and Length of the Pipe

Dia. of Pipe (inch)	Type and length of the pipe			
	PVC (km)	MS (km)	GI (km)	Total length (km)
8	1.83	-	-	1.83
6	6.02	-	-	6.02
4	10.47	0.65	0.01	11.12
3	0.5	0.25	-	0.75
			Total length:	17.71

4.2.4 Consumer Connections

Consumer connections enable consumers to get their portion of water out of the system through small diameter pipelines from the distribution mains. Private connections deliver the water to their house or yard. Street hydrants deliver water at a public place for people who cannot afford a private connection.

(a) Private Connections

Private connection can be in-house connection or yard connection. A connection consists of a CI pipe clamp at the main line, a gate valve, GI pipe to the house or yard and one or more supply points with taps.

(b) Street Hydrants

A street hydrant consists of a RCC platform, with good drainage facilities, a RCC pillar to support the GI supply pipe, the supply itself and a tap.

Table 4.7: Zone Wise Distribution of Street Hydrants

Description	Zone 1	Zone 2	Zone 3	Total
Number of Street Hydrants	5	22	14	41

4.2.5 Demand and Supply of Water in Gazipur Pourashava

According to the 2001 census, the population of Gazipur Pourashava is about 123500. Out of the total population 50000 live in the core area and rest in the fringe area.

There is an existing deficiency of water supply to meet the requirements of population. At the rate of 120 lit. per capita per day about 6 mgd of water needed for the population of Gazipur Pourashava, whereas the existing system is capable of supplying only 1.04 mgd for 50 thousand population. The water production against demand is shown in Figure 4.2. It can meet only 21.44% of the total requirements. 23.32% is coverage by hand tube well and rest of percent use water from own tubewell of the people (MIR, 2002). The demand of water supply varies in different sectors, i.e. domestic, industrial and institutional.

Domestic demand is determined by the extent of service connection. For example, a consumer who uses water from the street hydrant is less significant than one who has a tap within his house. Moreover, it varies in between multitap consumers, full service consumers and so on. The industrial demand depends on different types and function of the industry. Other institutional demand depends on different service facilities such as school, hospital, religious institutions, etc.

The increasing demand of water supply in Gazipur Pourashava is due to the rapid growth of population. The living cost of Dhaka city is more than Gazipur. So many serviceman and businessman live in Gazipur. Moreover, many important institutions of Bangladesh like Bangladesh Machine Tools Factory, Bangladesh Rice Research Institute, Bangladesh Agriculture Research Institute, Bangladesh Institute of Technology Dhaka, CERDI, Bangladesh Security Printing Corporation, Bangladesh Ordnance Factory etc. are within Gazipur Pourashava area. With ever increasing number of Pourashava`s population the demand for water has increased to such an extent which Pourashava Water Supply System (PWSS) can hardly cope with at present.

4.2.6 Leakage and Wastage of Piped Water

In Gazipur Pourashava water supply is not only inadequate but also irregular. Moreover, the flow of water is slow and huge quantity of water is wasted through house connection, street taps, leaky pipes, defective pipefitting and overflow of roof tanks.

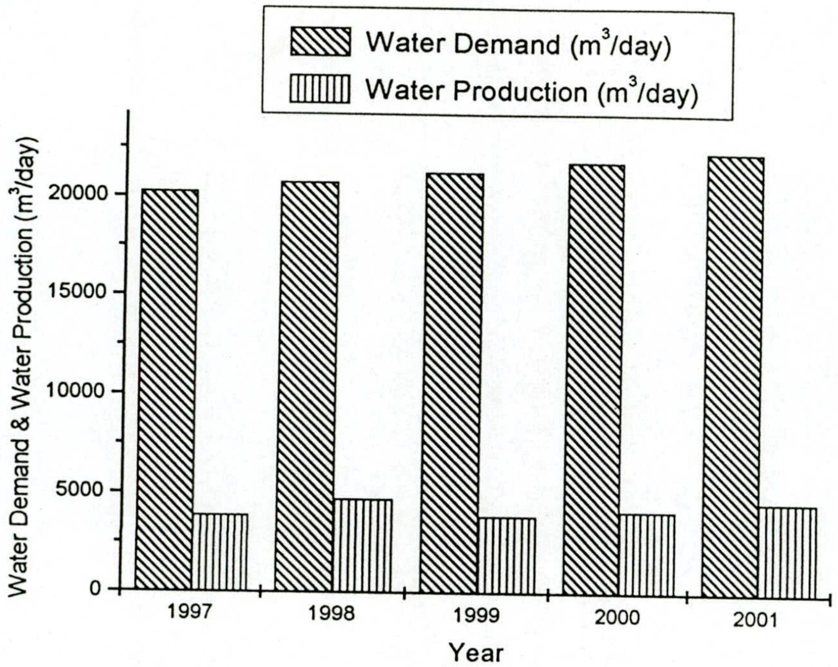


Fig.4.2: Water Production against Demand

4.2.7 Water Supply Schedule

Water is pumped into the system intermittently in 2 to 3 shifts. Table 4.8 shows the water supply schedule to different areas of Gazipur Pourashava.

Table 4.8: Water Supply Schedule from different Pumps

Pump	Area	Time	Total Supply Hour
Market and Bilashpur Pump	Shahapara Pourashava Chandona Lakshipura Bilashpur Market	6:10-8:30 11:30-2:00 4:25-5:35	6
Chayabithi Pump	Boruda North Chayabithi	6:00-7:45 12:00-1:45 6:00-9:30	$6\frac{1}{2}$
	South Chayabithi	7:45-10:30 1:45-5:00	6
	North Chayabithi	5:00-6:00 11:00-12:00 5:00-6:00	3
Rajbari Pump	Rathkhola College Road Lake Side Kazi Market Uttar Para	5:30-8:00 11:30-2:00 4:30-7:00	$7\frac{1}{2}$
	South Chayabithi	8:00-9:30 2:00-4:30	4

4.3 Analysis of the Water Distribution System

The network of water distribution system of Gazipur Pourashava includes 184 pipes having diameter varying from 3 to 8 inches and 162 nodes which are shown in Fig 4.3. The network was analyzed by EPANET2 software. This method consists of the application of the Newton-Raphson method and Hazen-Williams headloss formula. Water connections to houses are usually made at intermediate points between two subsequent nodes but for the analysis it was assumed that all water was consumed from nodes only.

In the computer programme number of pipes, number of nodes, Hazen-Williams coefficient, nodal demand, elevation of each node, node to node relations along with length, diameter, starting node number and end node number of pipes and pump capacity curve were supplied as input data. These are shown in APPENDIX A. The solution of the model requires the values of several parameters. These are assumed as follows:

- Hazen-Williams coefficient for 8 inch, 6 inch, 4 inch and 3 inch diameter of pipes are 120, 110, 100 and100 respectvily.
- specific gravity of water=1
- relative viscosity of water=1
- flow units in gpm
- maximum trials=100
- accuracy=0.001

Depending on the supply schedule the whole network is divided into five sub-areas. The analysis of each sub-area is described separately.

Water supply system of sub-area 1

The sub-area 1 is shown in Fig 4.4. The pump located near the market is connected to an overhead tank but for the pump at Bilaspur there is no overhead tank. For convenience of analysis the pressure at the second pump is converted to equivalent supply head. From these two pumps water is supplied by three shifts to Bilaspur, Chandana, Shahpara and market areas.

Water supply system of sub-area 2

The sub-area 2 is shown in Fig 4.5. The pump located at Rajbari is connected to an overhead tank. The over head tank is filled by the pump at non-supply period. The water from the overhead tank is released to meet the peak hourly demand. In this analysis only the pump is connected to the network system. Water is supplied from Rajbari pump to Rajbari, Lakeside, Rathkhola, Kazi market and south Chayabithi areas. Water is supplied from this pump by three shifts. In this analysis the ground water aquifer is considered as a reservoir.

Water supply system of sub-area 3

The sub-area 3 is shown in Fig 4.6. The pump located at Chayabithi supplies water directly to the system. From this pump water is supplied into the system by three shifts and cover only the north Chayabithi area. The ground water aquifer is considered as a reservoir.

Water supply system of sub-area 4

The sub-area 4 is shown in Fig 4.7. The Rajbari and Chayabithi pumps combinedly supply water into the system. From these two pumps the water is supplied by three shifts to south Chayabithi area. In both pumps the ground water aquifers are considered as a reservoir.

Water supply system of sub-area 5

The sub-area 5 is shown in Fig 4.8. The pump located at Chayabithi supplies water directly to the system. From this pump water is supplied into the system by three shifts and covered Boruda and rest of the north Chayabithi areas. In the analysis of this sub-area the ground water aquifer is considered as a reservoir.

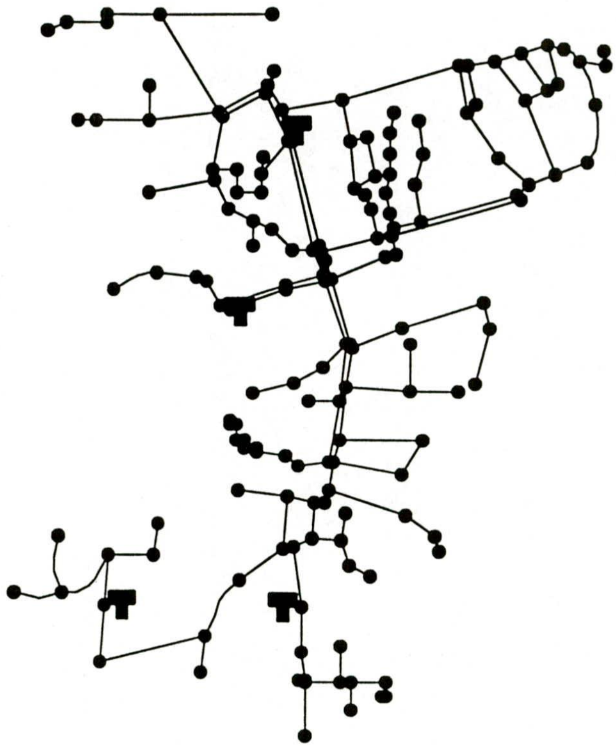


Figure 4.3 The Whole Water Distribution Network of Gazipur Pourashava

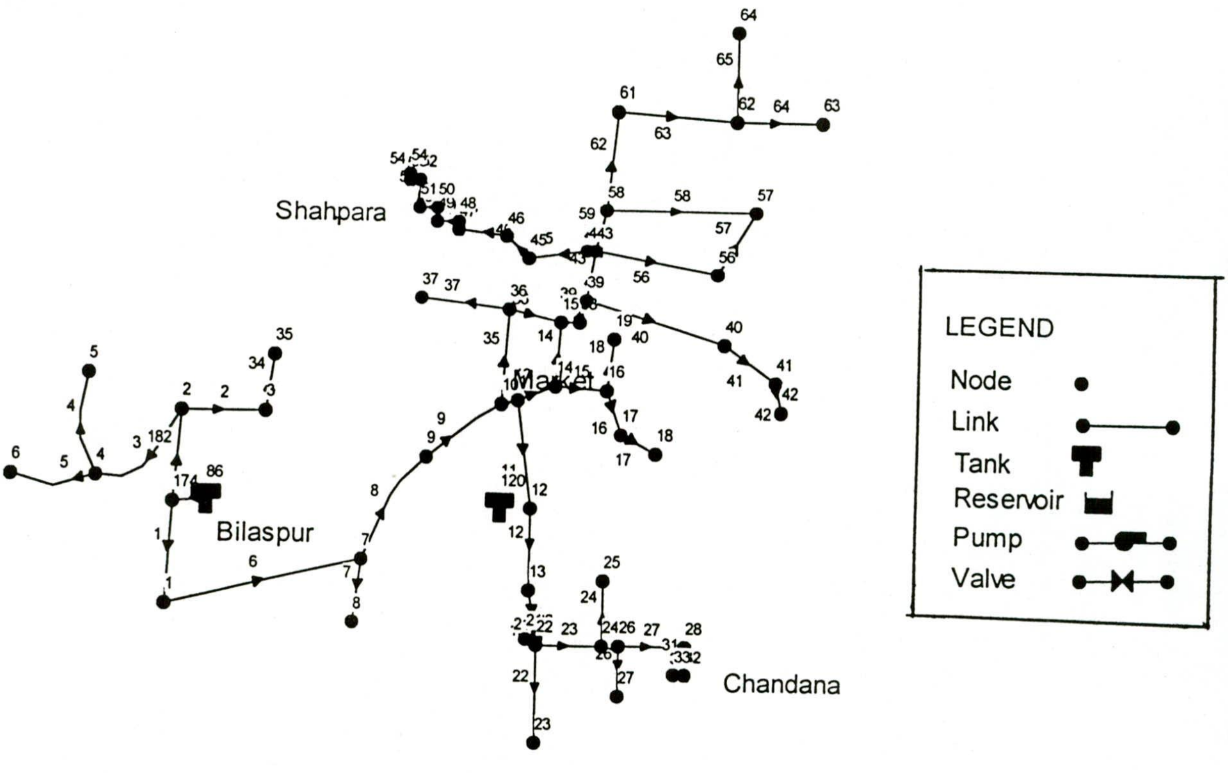


Figure 4.4 Water Supply from Bilaspur and Market Pump (Sub-Area 1)

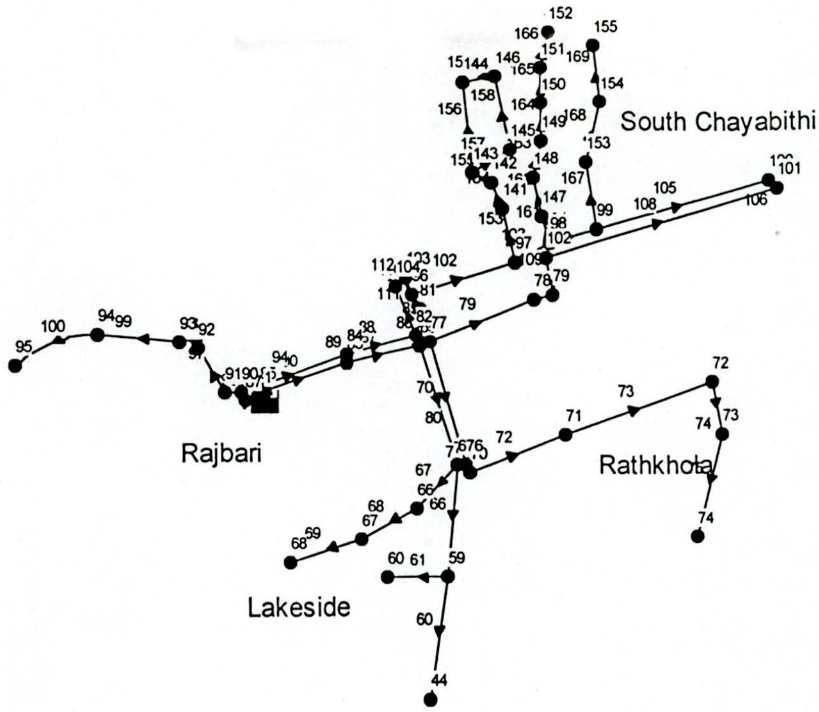


Figure 4.5 Water Supply from Rajbari Pump (Sub-Area 2)

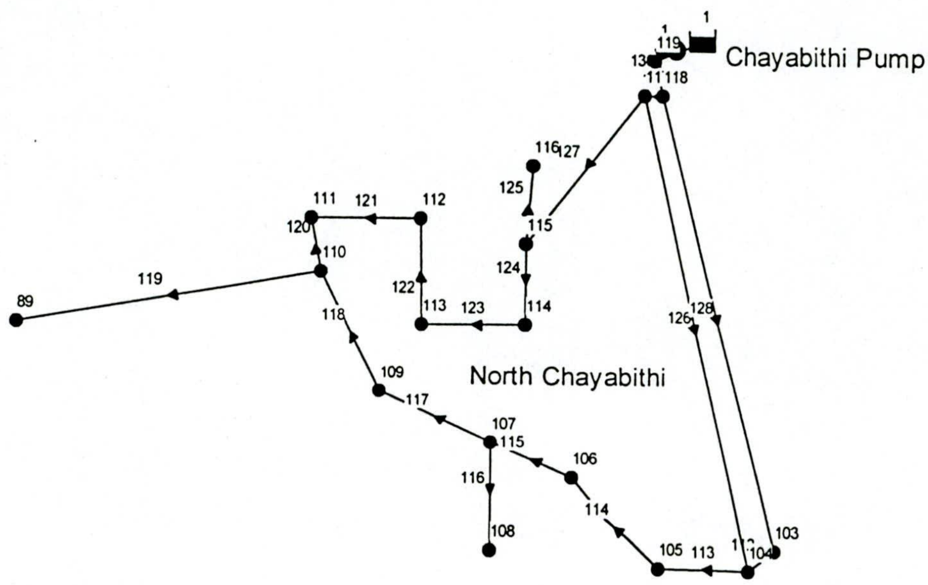


Figure 4.6 Water Supply from Chayabithi Pump (Sub-Area 3)

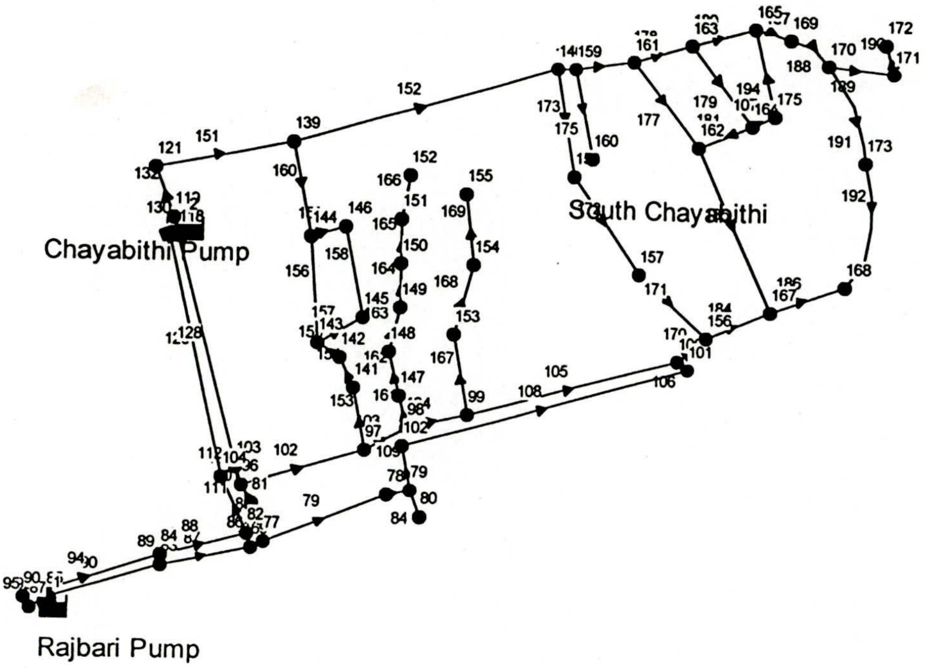


Figure 4.7 Water Supply from Rajbari and Chayabithi Pump (Sub-Area 4)

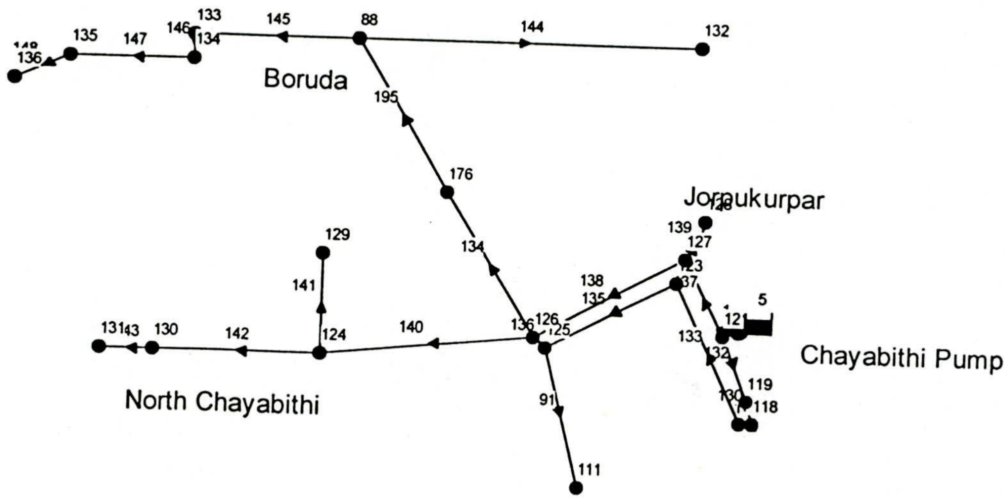


Figure 4.8 Water Supply from Chayabithi Pump (Sub-Area 5)

CHAPTER 5

RESULTS AND DISCUSSIONS

5.1 Results

In this study EPANET2 hydraulic simulation model has been used to compute junction heads and link or pipes flows for a fixed set of reservoir levels, tank levels and water demands.
The estimated and computed water supply and actual pressure and computed pressure at each node are shown in Table 5.1. The estimated and computed supply in each pipe is shown in Table 5.2. From Table 5.1, the amount of estimated water demand and actual supply actual pressure and computed pressure in each node could be compared. Similarly computed demand and actual supply in each pipe can be compared from Table 5.2.

Sub-Area 1

Comparison between estimated demand and actual supply:

It is observed from the result of sub-area 1 under Bilaspur and Market pumps that the consumers of the nearest nodal points of these pumps get excess amount of water which is about two to three times more than their required demand. The consumers served by nodes 33, 39, 40, 47, 52, 53, 54, 57, 61, 62, 63 and 64 get a deficient amount of water. In most of the pipes the actual flow is greater than the computed flow. But there is deficient flow in pipes 23, 25, 27, 31, 32, 52, 53, 54, 57 and 58.

Comparison of actual pressure and computed pressure:

The computed pressure at all nodes is greater than the actual pressure. The pressure variation at nodal points along the pipe length from Market pump to Chandana and Shahpara are shown in Fig 5.1 and Fig 5.2 respectively. It is shown in Fig 5.1 that at a distance from the pump the pressure at nodal points is zero. It is shown in Fig 5.2 that the pressure remains up to last point.

Sub-Area 2

Comparison between estimated demand and actual supply:

It is observed from the result of sub-area 2 under Rajbari pump that the consumers of the nearest nodal points of the pump get excess amount of water. There is a colony near the pump. The water is served to the colony from node 84 by three inch in diameter pipe connection into their underground storage reservoir. The consumers served by nodes 74, 101 and 146 get a deficient amount of water. In most of the pipes the actual flow is greater than the computed flow. But there is deficient flow in pipes 105, 106, 108, 162 and 154.

Comparison of actual pressure and computed pressure:

The computed pressure at all nodes is greater than the actual pressure. The pressure variation at nodal points along the pipe length from Rajbari pump to Rathkhola and South Chayabithi are shown in Fig 5.3 and Fig 5.4 respectively. In both cases the actual pressure is lower than the computed pressure.

Sub-Area 3

Comparison between estimated demand and actual supply:

It is observed from the result of sub-area 3 under Chayabithi pump that the consumers of the nearest nodal points of the pump get excess amount of water. The consumers served by nodes 89, 110, 111 and 112 get a deficient amount of water. In most of the pipes the actual flow is lower than the computed flow. But there is excess flow in pipes 116, 125, 129 and 130.

Comparison of actual pressure and computed pressure:

The computed pressure at all nodes is greater than the actual pressure. The pressure variation at nodal points along the pipe length from Chayabithi pump to north Chayabithi are shown in Fig5.5.

Sub-Area 4

Comparison between estimated demand and actual supply:

It is observed from the result of sub-area 4 under Rajbari and Chayabithi pumps that the consumers of the nearest nodal points of the pump get excess amount of water. The consumers served by nodes 143, 146,157,158, 163, 165, 169, 171, 172 and 173 get a deficient amount of water. Some of the consumers get supply water half or one third of their demand. There is no supply of water at nodes159, 160, 164, 167, 168,170 and 175. These are the severest water crisis nodes.

The deficient supply of water exists in pipes 152, 154, 155, 171, 172, 173, 174, 176, 178, 180, 187, 188, 189, 190 and 191. There is no flow in pipes 175,177, 181, 184, 185, 186,192 and 194.

Comparison of actual pressure and computed pressure:

The computed pressure at all nodes is greater than the actual pressure. The pressure variation at nodal points along the pipe length from Chayabithi pump to south Chayabithi is shown in Fig 5.6. At a distance from the pump the pressure head is not only zero and but also that below ground level.

Sub-Area 5

Comparison between estimated demand and actual supply:

It is observed from the result of sub-area 5 under Chayabithi pump that the consumers of the nearest nodal points of the pump get excess amount of water. The consumers served by nodes 124, 129, 130, 131, 132, 133, 134,135 and 136 get a deficient amount of water. In most of the pipes the actual flow is lower than the computed flow. But there is excess flow in pipes 131, 132, 133, 135, 136, 137, 138, 139 and 195. The ground level of node 176 is five feet below the nearest nodal points. So the consumers under this node get huge amount of water. On the contrary most of the consumers around node 88 do not get any water.

Comparison of actual pressure and computed pressure:

The computed pressure at all nodes is greater than the actual pressure. The pressure variation at nodal points along the pipe length from Chayabithi pump to Boruda is shown in Fig 5.7. At a distance from the pump the pressure is not only zero but also the hydraulic grade line lies below the ground level.

Table 5.1: Comparison between Estimated Demand and Actual Supply And Computed Pressure and Actual Pressure at Nodes

Node ID	Estimated Demand (gpm)	Actual Supply (gpm)	Computed Pressure		Actual Pressure(psi)
			ft	psi	
Junc 1	0.89	3.96	114.38	36.43	15
Junc 2	8.89	30.08	114.99	39.81	15
Junc 3	5.33	14.64	114.99	39.01	15
Junc 4	11.56	33.32	114.95	42.48	15
Junc 5	8.00	19.04	114.95	42.34	15
Junc 6	7.11	9.52	114.95	40.37	8.5
Junc 7	41.11	89.23	110.31	19.90	5
Junc 8	15.78	43.94	110.29	20.37	3.5
Junc 9	9.78	12.3	109.90	18.98	2.4
Junc 10	8.00	13.2	109.67	17.23	2.1
Junc 11	0.00	0.00	109.67	16.95	-
Junc 12	39.11	113.52	111.30	17.13	11
Junc 13	12.60	26.4	110.98	17.94	4.5
Junc 14	15.11	47.6	109.07	16.96	4.5
Junc 15	0.00	0.00	108.49	15.98	-
Junc 16	16.45	43.94	108.93	17.19	4.2
Junc 17	10.67	19.1	108.77	16.95	4.1
Junc 18	10.67	19.1	108.72	17.93	4
Junc 19	16.45	43.94	108.83	17.39	4
Junc 20	4.44	7.04	110.79	17.43	3.5
Junc 21	4.44	7.04	110.79	18.00	3.5
Junc 22	12.00	22.44	110.68	17.38	2.5
Junc 23	12.00	22.44	110.58	17.95	1.5
Junc 24	7.15	9.1	110.62	16.94	0.3
Junc 25	3.52	4.47	110.62	16.93	0.2
Junc 26	10.0	16.4	110.62	16.93	0.2
Junc 27	2.44	4.00	110.61	16.93	0.1
Junc 28	7.11	6.8	110.61	16.34	0.1
Junc 32	0.00	0.00	110.61	16.93	-
Junc 33	1.78	1.04	110.61	16.93	0.1
Junc 35	1.89	4.76	114.99	39.99	15.0
Junc 36	10.68	31.68	109.40	16.53	2.1
Junc 37	7.12	21.12	109.37	16.56	2.0
Junc 38	0.00	0.00	108.17	15.88	-
Junc 39	9.78	6.4	107.85	15.75	3.0
Junc 40	4.44	4.2	107.75	16.65	2.1
Junc 41	4.44	7.95	107.66	17.28	3
Junc 42	4.44	7.95	107.66	17.30	2.5
Junc 43	6.22	20.16	107.74	14.68	2.5
Junc 44	4.44	6.2	106.55	14.60	2.5

Table 5.1: Comparison between Estimated Demand and Actual Supply And Computed Pressure and Actual Pressure at Nodes (Contd.)

Node ID	Estimated Demand (gpm)	Actual Supply (gpm)	Computed Pressure		Actual Pressure(psi)
			ft	psi	
Junc 45	20.44	24.99	106.02	16.50	2.1
Junc 46	6.22	8.33	105.60	16.00	2
Junc 47	1.78	1.74	105.23	14.59	3
Junc 48	13.33	18.75	105.17	14.56	3
Junc 49	5.33	5.34	105.11	14.91	2.5
Junc 50	4.44	5.56	105.08	15.45	2
Junc 51	6.22	7.56	105.06	16.44	1.5
Junc 52	2.33	2.03	105.05	16.44	3.25
Junc 53	1.78	1.43	105.05	16.44	1.3
Junc 54	1.22	1.03	105.05	16.44	1.2
Junc 56	5.33	8.4	107.72	15.49	0.2
Junc 57	13.33	8.4	107.72	14.82	0.2
Junc 58	5.48	11.55	107.74	15.80	0.5
Junc 59	7.09	16.95	123.45	40.03	0.5
Junc 60	11.40	27.25	123.01	39.15	0.3
Junc 61	3.20	2.98	107.73	15.15	1.75
Junc 62	8.00	7.74	107.63	17.90	0.75
Junc 63	1.73	1.32	107.63	16.24	0.75
Junc 64	2.67	1.84	107.62	16.07	0.75
Junc 65	2.13	4.32	123.70	37.59	4.5
Junc 66	7.82	18.37	123.42	37.83	1.5
Junc 67	9.25	18.72	123.28	41.20	0.2
Junc 68	9.25	18.72	123.23	41.70	3.5
Junc 69	0.00	0.00	123.92	36.70	-
Junc 70	2.13	4.13	123.68	37.59	4.5
Junc 71	44.80	206.64	123.41	38.35	4.0
Junc 72	1.89	2.76	123.39	38.72	2.5
Junc 73	5.31	13.12	123.39	39.24	1.25
Junc 74	1.89	1.38	123.38	42.53	0.25
Junc 76	0.00	0.00	123.69	37.59	-
Junc 77	0.00	0.00	123.70	36.61	-
Junc 78	0.00	0.00	122.64	36.09	-
Junc 79	5.56	10.64	122.43	35.99	4.3
Junc 81	0.00	0.00	123.55	36.54	4.5
Junc 82	1.78	5.56	123.93	36.71	4.5
Junc 83	1.78	5.56	124.98	37.17	5.0
Junc 84	46.93	157.08	124.98	37.16	11.5
Junc 85	0.00	0.00	126.57	40.19	11.5
Junc 87	17.78	59.8	126.48	40.16	15
Junc 88	17.00	7.04	166.03	59.14	0.2

Table 5.1: Comparison between Estimated Demand and Actual Supply And Computed Pressure and Actual Pressure at Nodes (Contd.)

Node ID	Estimated Demand (gpm)	Actual Supply (gpm)	Computed Pressure		Actual Pressure(psi)
			ft	psi	
Junc 89	47.22	33.5	155.56	59.15	0.2
Junc 90	1.78	5.58	126.45	39.58	14
Junc 91	5.69	19.04	125.94	39.35	14.25
Junc 92	39.11	99.96	124.36	38.98	5
Junc 93	12.80	23.76	124.14	39.27	4.5
Junc 94	11.3	20.98	123.91	39.29	1
Junc 95	10.03	16.62	123.85	42.90	1
Junc 96	1.78	1.87	123.52	37.16	4.5
Junc 97	10.67	20.4	121.67	36.72	2.5
Junc 98	23.46	52.47	121.45	36.63	2.0
Junc 99	14.93	33.39	121.36	36.15	1
Junc 100	12.8	15.8	121.31	36.13	0.2
Junc 101	24.29	18.34	121.31	36.13	0.1
Junc 102	16.36	36.57	122.05	35.83	2.0
Junc 103	0.00	0.00	123.52	37.29	-
Junc 104	14.22	18.08	123.53	37.29	2.5
Junc 105	21.33	27.12	159.18	56.10	2.5
Junc 106	30.22	38.42	158.56	56.59	3.25
Junc 107	17.78	22.6	157.91	57.48	3.5
Junc 108	17.78	22.6	157.81	56.51	2
Junc 109	0.00	0.00	157.70	58.83	-
Junc 110	47.22	31.25	157.35	57.94	2
Junc 111	19.55	9.79	157.37	57.88	1.5
Junc 112	12.44	6.23	157.58	58.71	0.1
Junc 113	0.00	0.00	157.63	58.64	-
Junc 114	23.11	28.6	158.21	58.53	0.2
Junc 115	6.22	7.7	158.30	58.35	0.2
Junc 116	6.22	7.7	158.30	58.77	0.2
Junc 117	40.00	71.4	161.35	59.51	2.0
Junc 118	36.44	70.00	161.46	59.55	2.0
Junc 119	0.00	0.00	161.73	59.66	-
Junc 121	18.67	58.38	156.00	61.66	2.0
Junc 123	13.7	61.4	170.32	61.51	2.0
Junc 124	13.66	7.4	169.53	60.44	0.1
Junc 125	0.00	0.00	169.64	59.98	-
Junc 126	19.38	69.02	169.63	59.97	1.7
Junc 127	24.00	61.1	170.70	61.68	2.5
Junc 128	28.44	72.36	170.69	62.19	2.5
Junc 129	13.66	7.4	169.52	61.94	0.2
Junc 130	9.44	8.28	169.50	60.51	0.2
Junc 131	9.44	8.28	169.49	61.61	0.2
Junc 132	15.0	6.21	165.68	61.81	0.1
Junc 133	3.28	1.4	165.98	59.29	0.1

Table 5.1: Comparison between Estimated Demand and Actual Supply And Computed Pressure and Actual Pressure at Nodes (Contd.)

Node ID	Estimated Demand (gpm)	Actual Supply (gpm)	Computed Pressure		Actual Pressure(psi)
			ft	psi	
Junc 134	1.37	0.51	165.98	61.32	0.1
Junc 135	1.37	0.51	165.98	61.61	0.1
Junc 136	1.37	0.51	165.98	59.78	0.1
Junc 139	55.11	141.78	150.78	53.27	5.15
Junc 140	27.56	28.62	145.36	52.95	1.5
Junc 141	8.00	11.61	120.40	47.58	3.5
Junc 142	0.00	0.00	119.83	47.93	-
Junc 143	20.45	14.19	119.26	48.71	0.2
Junc 144	18.03	32.53	119.13	52.46	0.3
Junc 145	11.56	16.25	119.18	50.43	2.0
Junc 146	7.12	2.09	119.13	53.15	0.5
Junc 147	5.69	38.08	121.43	48.97	3.0
Junc 148	2.84	9.99	121.35	50.23	3.5
Junc 149	1.42	4.04	121.29	51.07	3.0
Junc 150	2.84	8.16	121.57	51.93	2.0
Junc 151	2.85	8.08	121.26	52.79	2.5
Junc 152	2.85	8.08	121.26	53.22	3.0
Junc 153	7.11	12.9	121.25	48.42	2.0
Junc 154	3.56	6.45	121.22	49.71	4.0
Junc 155	3.56	6.45	121.22	51.01	4.0
Junc 156	46.22	64.26	144.62	47.62	1.5
Junc 157	7.11	5.13	144.79	49.74	0.1
Junc 158	7.11	5.13	145.06	51.66	0.1
Junc 159	3.5	0.00	145.21	52.32	0.00
Junc 160	1.79	0.00	145.21	51.19	0.00
Junc 161	3.56	5.6	144.89	51.95	0.2
Junc 162	0.00	0.00	144.66	51.85	-
Junc 163	8.89	6.1	144.72	51.87	0.1
Junc 164	0.89	0.00	144.66	52.28	0.00
Junc 165	12.45	4.48	144.63	51.84	0.1
Junc 167	24.89	0.00	144.58	51.81	0.00
Junc 168	31.11	0.00	144.54	51.84	0.00
Junc 169	7.11	5.56	144.59	50.95	0.1
Junc 170	8.00	0.00	144.55	52.67	0.00
Junc 171	4.44	1.7	144.55	52.67	0.1
Junc 172	3.56	1.3	144.55	52.66	0.1
Junc 173	3.56	1.2	144.54	52.66	0.1
Junc 174	1.78	3.66	115.15	39.19	15.0
Junc 175	4.44	0.00	144.65	52.28	0.00
Junc 176	23.40	45.2	167.12	62.66	2.5

Table 5.2 Computed Flow and Actual Flow in Pipes

Pipe ID	Computed Flow (gpm)	Actual Flow (gpm)
Pipe 1	137.70	269.53
Pipe 2	7.22	19.4
Pipe 3	26.67	61.88
Pipe 4	8.00	19.04
Pipe 5	7.11	9.52
Pipe 6	136.81	265.57
Pipe 7	15.78	43.94
Pipe 8	79.92	132.4
Pipe 9	70.14	120.1
Pipe 10	3.60	5.91
Pipe 11	167.80	291.5
Pipe 12	73.77	114.07
Pipe 13	171.40	297.41
Pipe 14	102.07	123.73
Pipe 15	54.22	126.08
Pipe 16	21.33	38.2
Pipe 17	10.67	19.1
Pipe 18	16.45	43.94
Pipe 19	64.88	87.67
Pipe 20	4.44	7.04
Pipe 21	56.00	73.56
Pipe 22	12.00	22.44
Pipe 23	32.00	28.68
Pipe 24	3.52	4.47
Pipe 25	21.33	15.12
Pipe 26	2.44	4.00

Pipe ID	Computed Flow gpm)	Actual Flow (gpm)
Pipe 27	8.89	7.84
Pipe 31	1.78	1.04
Pipe 32	1.78	1.04
Pipe 34	1.89	4.76
Pipe 35	58.54	100.92
Pipe 36	40.74	48.12
Pipe 37	7.12	21.12
Pipe 38	142.81	171.85
Pipe 39	142.81	171.85
Pipe 40	13.32	20.1
Pipe 41	8.88	15.9
Pipe 42	4.44	7.95
Pipe 43	113.49	145.35
Pipe 44	67.53	82.96
Pipe 45	63.09	76.76
Pipe 46	42.65	51.77
Pipe 47	36.43	43.44
Pipe 48	34.65	41.7
Pipe 49	21.32	22.95
Pipe 50	15.99	17.61
Pipe 51	11.55	12.05
Pipe 52	5.33	4.49
Pipe 53	3.00	2.46
Pipe 54	1.22	1.03
Pipe 56	14.55	14.2
Pipe 57	9.22	5.8

Table 5.2 Computed Flow and Actual Flow in Pipes (Contd.)

Pipe ID	Computed Flow (gpm)	Actual Flow (gpm)
Pipe 58	4.11	2.6
Pipe 59	25.19	28.03
Pipe 60	4.44	6.2
Pipe 61	11.40	27.25
Pipe 62	15.60	13.88
Pipe 63	12.40	10.9
Pipe 64	1.73	1.32
Pipe 65	2.67	1.84
Pipe 66	22.93	50.4
Pipe 67	26.31	55.81
Pipe 68	18.49	37.44
Pipe 69	9.25	18.72
Pipe 70	88.33	239.6
Pipe 72	68.33	223.9
Pipe 73	9.09	17.26
Pipe 74	7.20	14.5
Pipe 75	1.89	1.38
Pipe 76	0.00	0.00
Pipe 77	70.46	228.03
Pipe 78	36.96	133.38
Pipe 79	44.03	58.19
Pipe 80	33.50	94.64
Pipe 81	194.32	374.71
Pipe 83	39.41	58.19
Pipe 85	132.89	221.88

Pipe ID	Computed Flow (gpm)	Actual Flow (gpm)
Pipe 86	13.62	30.65
Pipe 87	269.02	582.35
Pipe 88	43.21	81.23
Pipe 89	2.71	4.91
Pipe 90	273.52	592.82
Pipe 91	19.55	9.79
Pipe 92	185.91	415.76
Pipe 93	182.26	384.55
Pipe 94	42.27	76.32
Pipe 95	138.98	258.68
Pipe 96	78.93	182.36
Pipe 97	73.24	163.32
Pipe 98	34.13	63.36
Pipe 99	21.33	39.6
Pipe 100	10.03	16.62
Pipe 101	132.89	221.88
Pipe 102	142.03	236.71
Pipe 103	86.91	169.81
Pipe 104	44.97	69.99
Pipe 105	15.82	10.08
Pipe 106	3.02	2.28
Pipe 107	11.32	0.00
Pipe 108	21.27	16.06
Pipe 109	37.63	52.63
Pipe 110	13.59	16.70

Table 5.2 Computed Flow and Actual Flow in Pipes (Contd.)

Pipe ID	Computed Flow (gpm)	Actual Flow (gpm)
Pipe 111	27.81	50.58
Pipe 112	162	160.5
Pipe 113	179.93	169.02
Pipe 114	158.60	141.9
Pipe 115	128.38	103.48
Pipe 116	17.78	22.6
Pipe 117	85.43	58.28
Pipe 118	85.43	58.28
Pipe 119	47.11	33.5
Pipe 120	8.79	6.47
Pipe 121	28.34	16.26
Pipe 122	48.18	22.49
Pipe 123	48.18	22.49
Pipe 124	71.29	51.09
Pipe 125	6.22	7.7
Pipe 126	27.31	26.28
Pipe 127	83.73	66.49
Pipe 128	166.83	160.5
Pipe 129	151.05	164.17
Pipe 130	354.32	394.64
Pipe 131	280.68	519
Pipe 132	278.44	313.3
Pipe 133	33.66	113.25
Pipe 134	62.79	61.38
Pipe 135	33.66	51.85

Pipe ID	Computed Flow (gpm)	Actual Flow (gpm)
Pipe 136	33.66	42.06
Pipe 137	147.12	192.1
Pipe 138	94.68	119.7
Pipe 139	28.44	72.36
Pipe 140	46.17	31.36
Pipe 141	13.66	7.4
Pipe 142	18.87	16.56
Pipe 143	9.44	8.28
Pipe 144	15.00	6.21
Pipe 145	7.39	2.93
Pipe 146	4.11	1.53
Pipe147	2.74	1.02
Pipe 148	1.37	0.51
Pipe 151	259.77	254.92
Pipe 152	142.83	64.82
Pipe 153	44.45	46.5
Pipe 154	36.45	34.89
Pipe 155	36.45	34.89
Pipe 156	5.93	6.54
Pipe 157	10.07	14.16
Pipe 158	7.12	15.79
Pipe 159	1.49	2.09
Pipe 160	25.15	48.32
Pipe 161	18.48	66.43
Pipe 162	12.79	38.35

Table 5.2 Computed Flow and Actual Flow in Pipes (Contd.)

Pipe ID	Computed Flow (gpm)	Actual Flow (gpm)
Pipe 163	9.95	28.36
Pipe 164	8.53	24.32
Pipe 165	5.69	16.16
Pipe 166	2.85	8.08
Pipe 167	14.22	25.8
Pipe 168	7.11	12.9
Pipe 169	3.56	6.45
Pipe 170	63.56	64.36
Pipe 171	28.15	5.13
Pipe 172	42.37	5.13
Pipe 173	42.37	10.26
Pipe 174	72.90	25.94
Pipe 175	1.79	0.00
Pipe 176	71.11	25.94
Pipe 177	16.64	0.00
Pipe 178	50.91	20.34

Pipe ID	Computed Flow (gpm)	Actual Flow (gpm)
Pipe 179	6.75	0.00
Pipe 180	35.27	14.24
Pipe 181	5.46	0.00
Pipe 182	42.78	111.36
Pipe 184	45.49	0.00
Pipe 185	11.18	0.00
Pipe 186	31.78	0.00
Pipe187	29.70	9.76
Pipe 188	22.59	4.2
Pipe 189	11.70	3.0
Pipe 190	7.26	1.3
Pipe 191	2.89	1.2
Pipe 192	0.67	0.00
Pipe 194	6.88	0.00
Pipe 195	39.39	16.19

5.2 Discussions

From the analysis of results it is observed that water production from the pump is more than required demand. The computed demand according to house connections and measured supply from the pumps are shown in Table 5.3.

Table 5.3: Estimated Demand and Measured Supply for Different Sub-Areas

Sub Area	Estimated Demand (gpm)	Measured Supply from Pump (gpm)
Sub Area-1	450	1040
Sub Area-2	459	478
Sub Area-3	340	364
Sub Area-4	620	842
Sub Area-5	265	364

In spite of excess supply of water from different pumps many consumers do not get any water. The following are the main reasons for shortage of water, which are found from the field observation:

1) Loss of water due to overflow of reservoirs and collection pots at different household, which are exhibited in Figure 5.8.

2) Loss of water due to leakage and breakage of pipelines, which are exhibited in Figure 5.9.

3) Unused water discharge which is exhibited in Figure 5.10.

4) Waste of excess amount of water due to high pressure, which is exhibited in Figure 5.11.

5) Use of excess amount of water by consumers located near the pumps.

According to the pressure the supply area may be divided into three regions:

i) high pressure region,
ii) medium pressure region and
iii) very low or zero pressure region.

The consumers of high-pressure region waste excess amount of water. The consumers of medium pressure region collect water and use properly. The consumers, who live in the low-pressure region, get a deficient amount of water. The discharge rate of water is very slow which is shown in Fig 5.14. It is shown in Fig 5.15 that the many consumers do not get any water due to hydraulic grade line being below the ground surface. The consumers of that place try to get water by caving the soil. Some consumers try to draw water from main line by syphonic action. The people, who do not get any water from the supply system, sink deep-set hand tubewells to meet their required demand. But the people, who have not capability for sinking tubewell, collect water from other houses.

The actual pressure is lower than the computed pressure. The following may be the possible causes for rapidly dropping of actual pressure:

i) Illegal house connection.

ii) Using of pumps to draw water from the main line.

iii) Leakage in the pipelines.

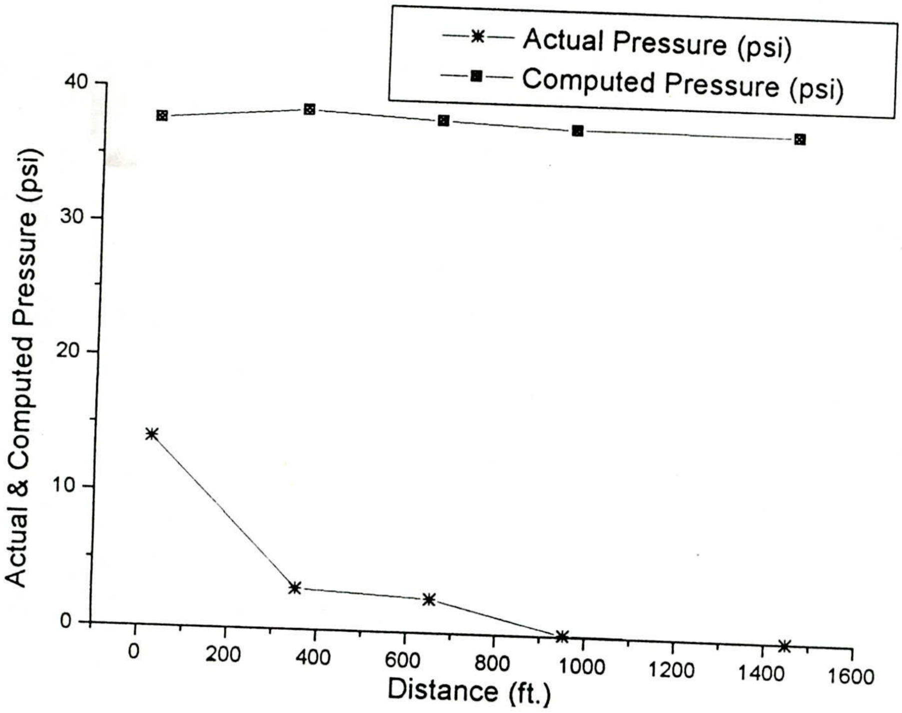


Figure 5.1: The pressure variation at nodal points along the pipe length (From Market pump to Chandana)

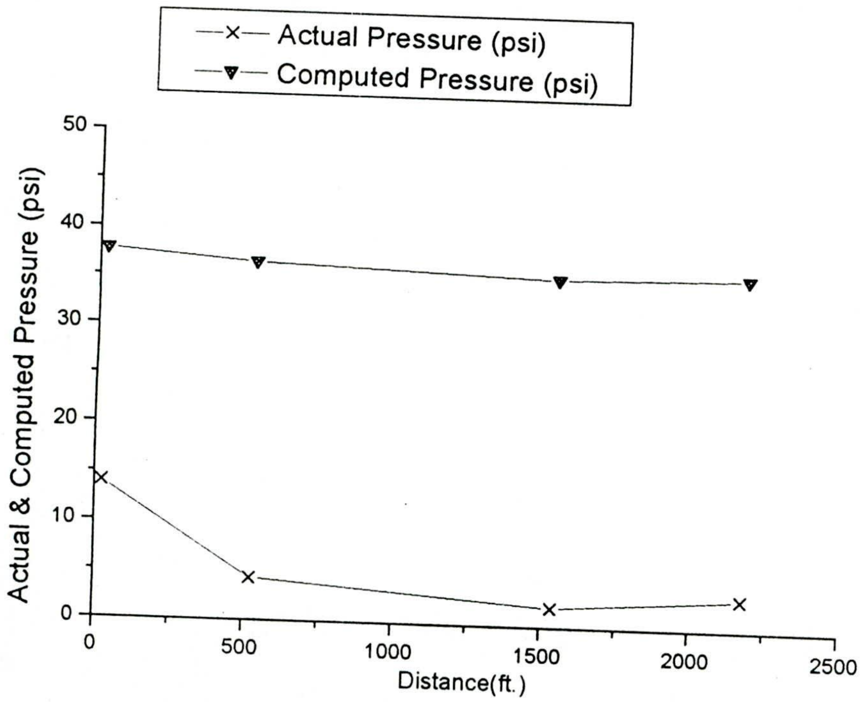


Figure 5.2: The pressure variation at nodal points along the pipe length (From Market pump to Shahpara)

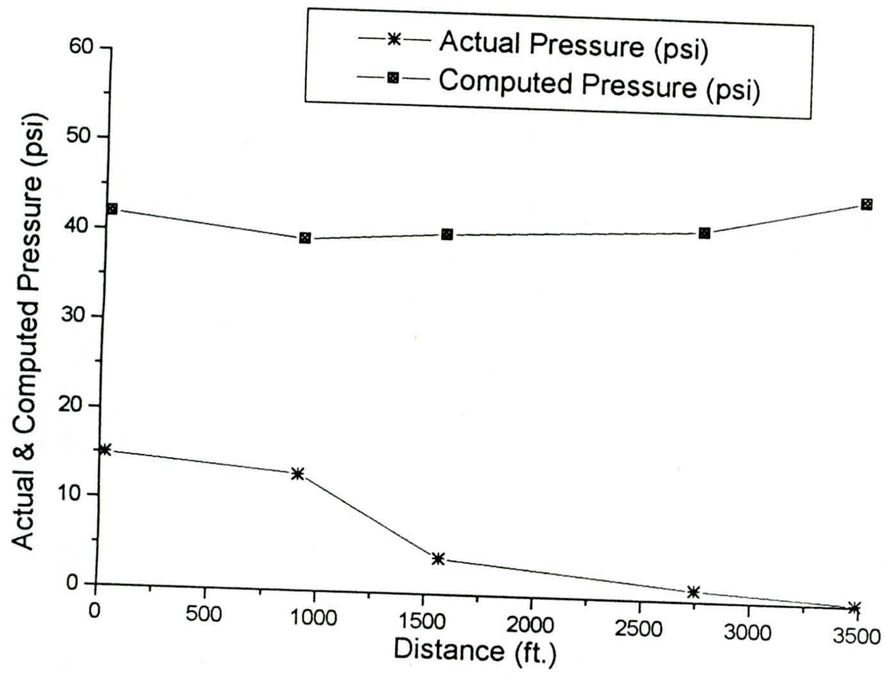


Figure 5.3: The pressure variation at nodal points along the pipe length (From Rajbari pump to Rathkhola)

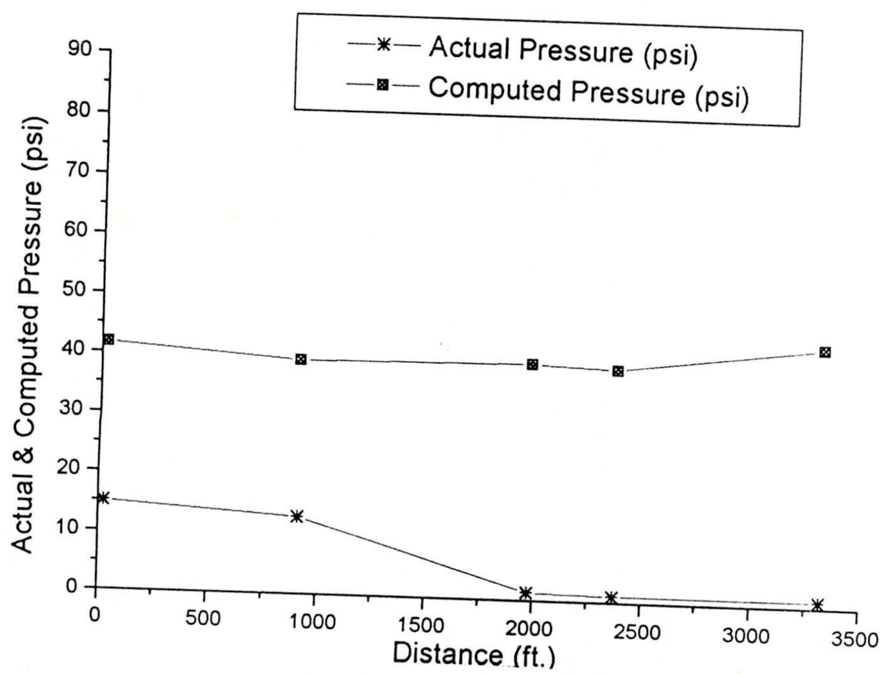


Figure 5.4: The pressure variation at nodal points along the pipe length (From Rajbari pump to South Chayabithi)

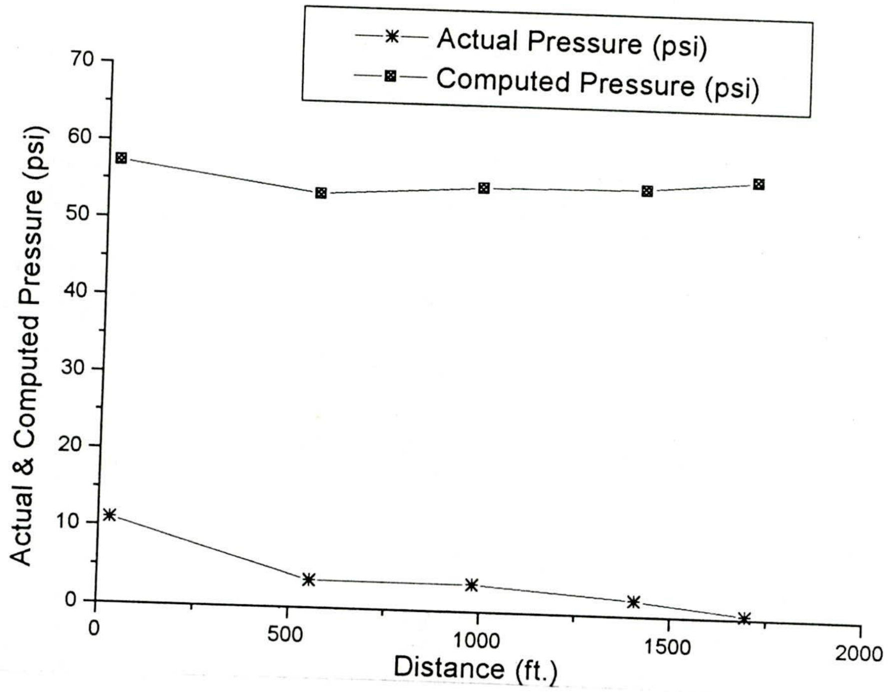


Figure 5.5: The pressure variation at nodal points along the pipe length (From Chayabithi pump to North Chayabithi)

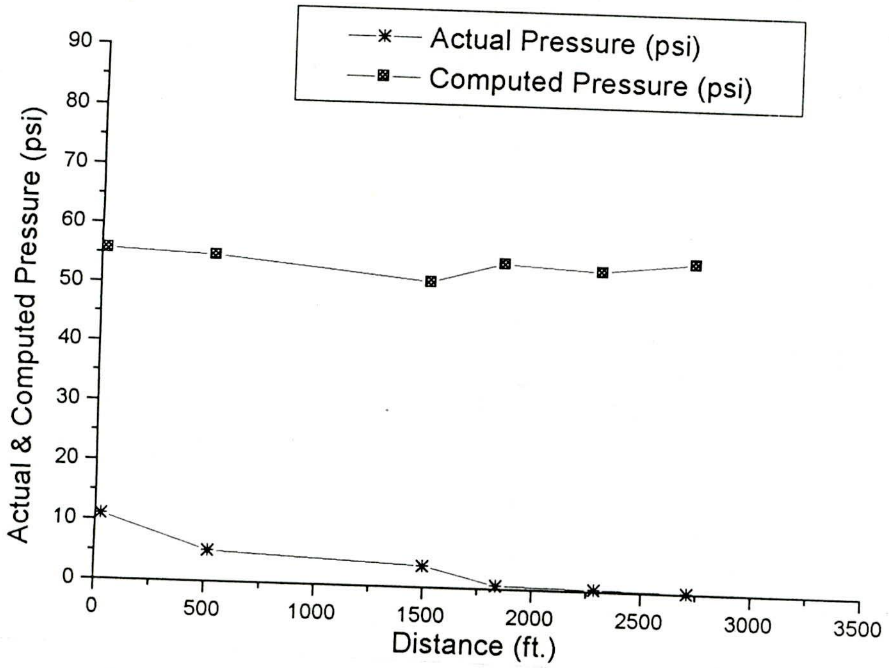


Figure 5.6: The pressure variation at nodal points along the pipe length (From Chayabithi pump to South Chayabithi)

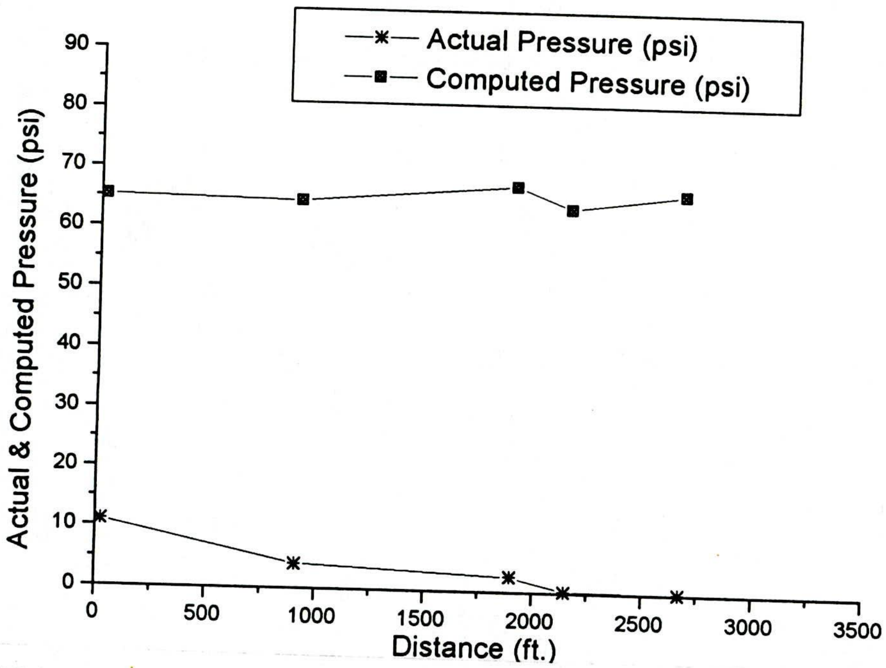
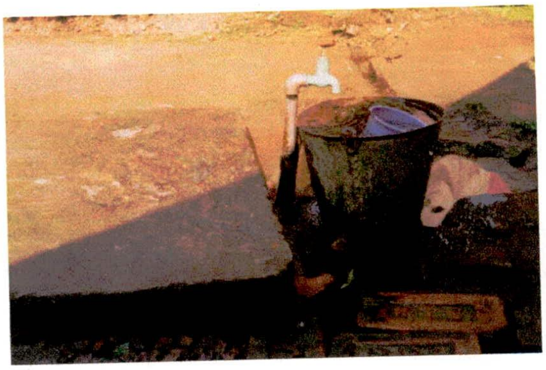
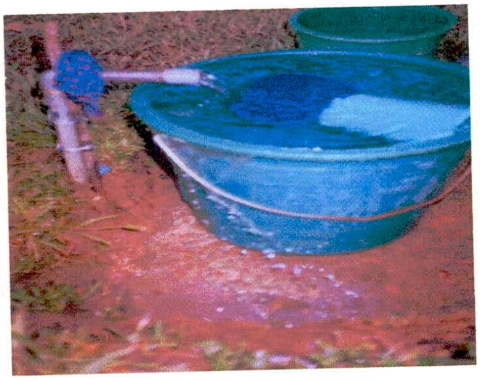


Figure 5.7: The pressure variation at nodal points along the pipe length (From Chayabithi pump to Boruda)

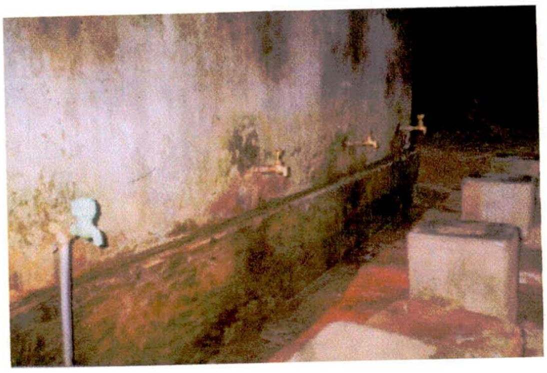


Overflow of bucket
(North Chayabithi, Date: 22-11-02)

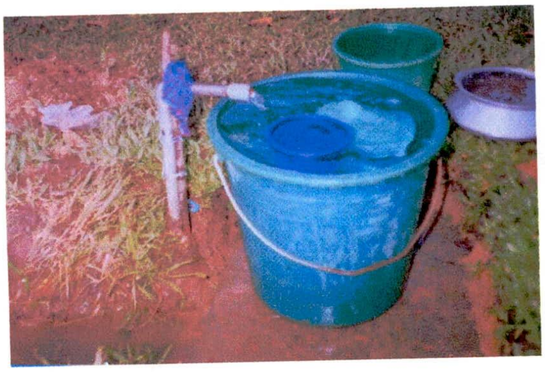


Overflow of bowl
(North Chayabithi, Date: 22-11-02)

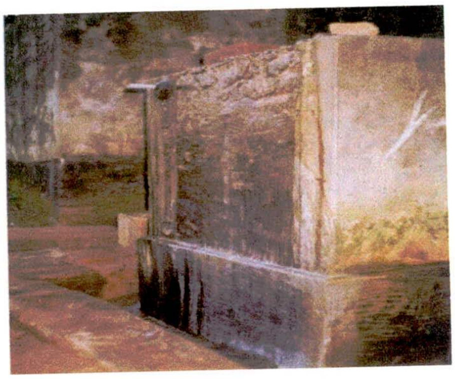
Overflow of bucket
(North Chayabithi, Date: 22-11-02)



Wastage of water
(Jorpukurpar, Date: 22-11-02)



Overflow of bucket
(North Chayabithi, Date: 22-11-02)



Overflow of storage tank
(Jorpukurpar, Date: 22-11-02)

Figure 5.8: Losses of water due to overflow of reservoirs and collection pots.

Leakage of pipelines
(South Chayabithi, Date: 22-11-02)

Leakage of pipeline
(South Chayabithi, Date: 22-11-02)

Breakage of pipelines
(South Chayabithi, Date: 22-11-02)

Leakage water overflow a road
(South Chayabithi, Date: 22-11-02)

Leakage of pipeline
(South Chayabithi, Date: 22-11-02)

Leakage of pipelines
(South Chayabithi, Date: 22-11-02)

Figure 5.9: Losses of water due to leakage and breakage of pipelines.

Figure 5.10: Wastage of water

(South Chayabithi, Date: 23-11-02)

Figure 5.11: Wastage of huge amount of water due to high pressure

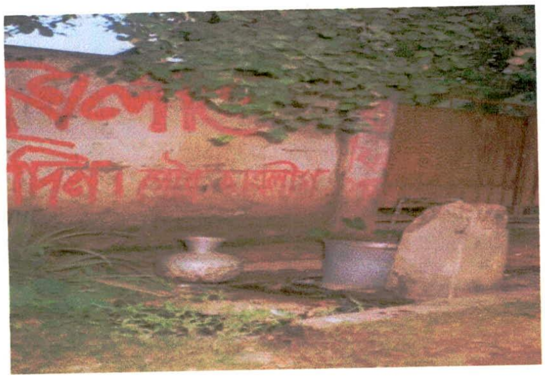
(Boruda, Date: 23-11-02)

Figure 5.12: Releasing airlock

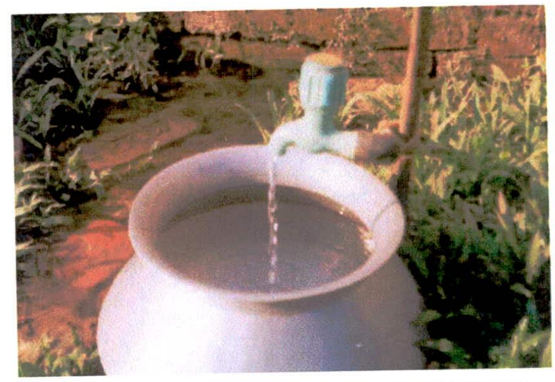
(South Chayabithi, Date: 23-11-02)

Figure 5.13: Air releasing pipe

(South Chayabithi, Date: 23-11-02)

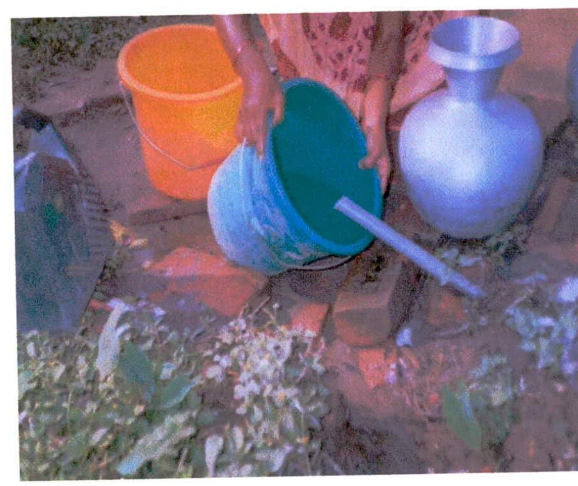


Low pressure at street hydrant
(South Chayabithi, Date: 23-11-02)

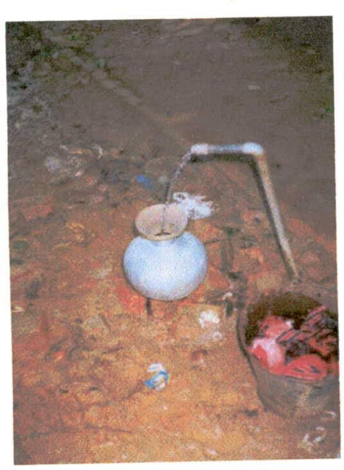


Low pressure at yard connection
(South Chayabithi, Date: 23-11-02)

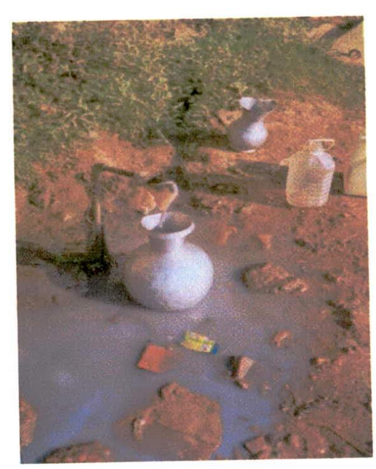
Low pressure at house connection
(North Chayabithi, Date: 23-11-02)



Low pressure at house connection
(North Chayabithi, Date: 23-11-02)



Low pressure at street hydrant
(South Chayabithi, Date: 23-11-02)



Low pressure at street hydrant
(South Chayabithi, Date: 23-11-02)

Figure 5.14: Water discharged at very low pressure.

Drawing water by psyphonic action
(Boruda, Date: 23-11-02)

Yard connection without water
(Boruda, Date: 23-11-02)

Placement of pot digging the soil
(South Chayabithi, Date: 23-11-02)

Street hydrant without water
(South Chayabithi, Date: 23-11-02)

Placement of pot digging the soil
(South Chayabithi, Date: 23-11-02)

Yard connection without water
(Boruda, Date: 23-11-02)

Figure 5.15: Hydraulic grade line below ground level.

CHAPTER 6

CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

From analysis of results of the water distribution network of Gazipur Pourashsva the following conclusions can be drawn:

i) The water supply system should have adequate supply of water. In this study it observed that about 55% nodes have excess supply, 17% nodes meet required demand and 28% nodes have deficient supply.

ii) The water supply system should have sufficient pressure. Faucet pressure of 5 psi is satisfactory for most domestic needs. Generally pressure in the distribution system under normal operating condition is very low. In this study it is observed that about 11% nodes have sufficient pressure and the pressure of 89% nodes varies from 0.00 psi to 4.5 psi.

iii) The consumer nearer the pumping stations gets more water and has tendency to waste water.

iv) It is seen from the analysis of the water distribution network of Gazipur Pourashava that the computed pressure is higher than the actual pressure measured in field.

v) If the wasteful use of water and leakage of the supply system could be controlled, then all of the consumers could get water according to their required demand.

Given pumping capacity and extent of the water distribution network, there is scope for improvement supply situation. This will require adequate management of water supply.

6.2 Recommendations

Recommendation to improve the supply situation:

Measures to increase deficient supply:

- reduction of wasteful use of water
- introduction of metering system in the house connection
- introduction of an intensive community information/motivation participation programme to convince consumers for avoidance of wasteful use of water
- repair of street hydrants as early as possible
- to prevent overflow of the storage reservoirs and collection pots in consumers houses

- by installing pressure reducing valve at supply points where water pressure in the main is high
- replacement of broken pipes, leaking joints, close street hydrants which are not used

Measures to control excess collection by households located nearer to the pumping points:

- disconnection of supply line for using excess amount of water
- water billing per unit volume of water

Recommendation for future studies:

- This study has been done considering steady-state condition without incorporating various types of valves, fire hydrants in the network analysis. In future studies variable demand of consumers at different nodes with respect to time incorporating various types of valves, fire hydrants, tanks in the network system may be considered.
- Using the EPANET2 water quality modeling capabilities one can study water quality in municipal water supply system, such as age of water throughout a supply system, loss of chlorine residuals and growth of disinfection byproducts.

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APPENDIX A

LIST OF INPUT DATA

Table A1: Elevation and Estimated Demand at Nodes

Node ID	Elevation ft	Estimated Demand(gpm)
Junc 1	24.42	0.89
Junc 2	22.41	8.89
Junc 3	24.26	5.33
Junc 4	16.21	11.56
Junc 5	16.53	8 .00
Junc 6	21.08	7.11
Junc 7	30.77	41.11
Junc 8	29.66	15.78
Junc 9	26.42	9.78
Junc 10	25.96	8 .00
Junc 11	25.57	0.00
Junc 12	24.34	39.11
Junc 13	22.13	12.60
Junc 14	25.08	15.11
Junc 15	26.94	0.00
Junc 16	24.40	16.45
Junc 17	24.80	10.67
Junc 18	22.48	10.67
Junc 19	23.83	16.45
Junc 20	23.10	4.44
Junc 21	21.79	4.44
Junc 22	23.11	12.00
Junc 23	21.70	12.00
Junc 24	24.08	7.15
Junc 25	24.08	3.52
Junc 26	24.08	10.00
Junc 27	24.08	2.44
Junc 28	25.44	7.11
Junc 32	24.08	0 .00
Junc 33	24.08	1.78
Junc 36	27.20	10.68
Junc 37	27.09	7.12
Junc 38	26.88	0.00
Junc 39	26.88	9.78
Junc 40	24.71	4.44
Junc 41	23.17	4.44
Junc 42	23.12	4.44
Junc 43	29.24	6.22
Junc 44	29.24	4.44
Junc 45	23.31	20.44
Junc 46	24.06	6.22
Junc 47	26.93	1.78
Junc 48	26.95	13.33
Junc 49	26.07	5.33
Junc 50	24.80	4.44
Junc 51	22.49	6.22
Junc 52	22.49	2.33
Junc 53	22.49	1.78
Junc 54	22.49	1.22
Junc 56	27.35	5.33
Junc 57	28.89	13.33
Junc 58	26.66	5.48
Junc 59	30.42	7.09
Junc 60	32.01	11.40
Junc 61	28.14	3.20
Junc 62	21.70	8 .00
Junc 63	25.52	1.73
Junc 64	25.92	2.67
Junc 65	36.28	2.13
Junc 66	35.47	7.82
Junc 67	27.54	9.25
Junc 68	26.34	9.25
Junc 69	38.56	0.00
Junc 70	36.28	2.13
Junc 71	34.24	44.80
Junc 72	33.38	1.89
Junc 73	32.16	5.31
Junc 74	24.57	1.89
Junc 76	36.28	0.00
Junc 77	38.56	0.00
Junc 78	38.56	0.00
Junc 79	38.56	5.56
Junc 81	38.56	0.00
Junc 82	38.56	1.78
Junc 83	38.56	1.78
Junc 84	38.56	46.93
Junc 85	33.59	0 .00
Junc 87	33.59	17.78
Junc 90	34.92	1.78
Junc 91	34.92	5.69
Junc 92	34.21	39.11
Junc 93	33.31	12.8
Junc 94	33.03	11.30
Junc 95	24.65	10.03
Junc 96	37.11	1.78
Junc 97	36.23	10.67
Junc 98	36.23	23.46
Junc 99	37.23	14.93
Junc 100	37.23	12.80
Junc 101	37.23	24.29

Table A1: Elevation and Estimated Demand at Nodes (Contd.)

Node ID	Elevation ft	Estimated Demand (gpm)
Junc 102	38.56	16.36
Junc 103	36.82	0.00
Junc 104	36.82	14.22
Junc 105	37.56	21.33
Junc 106	35.95	30.22
Junc 107	33.41	17.78
Junc 108	35.56	17.78
Junc 109	30.17	0.00
Junc 110	32.00	47.22
Junc 89	28.86	19.55
Junc 111	32.13	19.55
Junc 112	30.33	12.44
Junc 113	30.51	0.00
Junc 114	31.17	23.11
Junc 115	31.66	6.22
Junc 116	30.69	6.22
Junc 117	31.45	40.00
Junc 118	31.45	36.44
Junc 119	31.45	0.00
Junc 121	29.41	18.67
Junc 123	28.35	13.70
Junc 125	31.22	0.00
Junc 126	31.22	19.35
Junc 127	28.35	24.00
Junc 128	27.16	28.44
Junc 88	29.55	17.00
Junc 124	30.04	13.66
Junc 129	26.57	13.66
Junc 130	29.84	9.44
Junc 131	27.31	9.44
Junc 132	23.03	15.00
Junc 133	29.14	3.28
Junc 134	24.47	1.37
Junc 135	23.79	1.37
Junc 136	28.00	1.37
Junc 139	27.83	55.11
Junc 140	23.17	27.56

Node ID	Elevation ft	Estimated Demand (gpm)
Junc 141	35.00	8.00
Junc 142	34.00	0.00
Junc 143	32.00	20.45
Junc 144	29.62	18.03
Junc 145	28.00	11.56
Junc 146	28.00	7.12
Junc 147	32.00	5.69
Junc 148	29.00	2.84
Junc 149	27.00	1.42
Junc 150	25.00	2.84
Junc 151	23.00	2.85
Junc 152	22.00	2.85
Junc 153	33.00	7.11
Junc 154	30.00	3.56
Junc 155	27.00	3.56
Junc 156	34.71	46.22
Junc 157	30.00	7.11
Junc 158	25.84	7.11
Junc 159	24.46	3.50
Junc 160	27.07	1.79
Junc 161	25.00	3.56
Junc 162	25.00	0.00
Junc 163	25.00	8.89
Junc 165	25.00	12.45
Junc 164	24.00	0.89
Junc 167	25.00	24.89
Junc 168	24.89	31.11
Junc 169	27.00	7.11
Junc 170	23.00	8.00
Junc 171	23.00	4.44
Junc 172	23.00	3.56
Junc 173	23.00	3.56
Junc 174	24.00	1.78
Junc 35	22.00	1.89
Junc 175	24.00	4.44
Junc 176	24.55	23.40

Table A2: Pipe Dimensions and Roughness

Link ID	Pipe Length (ft)	Pipe Diameter (inch)	Hazen-Williams Roughness Coefficient
Pipe 1	250	6	110
Pipe 2	443	6	110
Pipe 4	394	6	110
Pipe 3	262	6	110
Pipe 5	131	6	110
Pipe 6	1345	6	110
Pipe 7	328	6	110
Pipe 8	360	6	110
Pipe 9	262	6	110
Pipe 10	98	6	110
Pipe 11	367	6	110
Pipe 12	328	6	110
Pipe 13	131	6	110
Pipe 14	328	6	110
Pipe 15	262	6	110
Pipe 16	230	4	100
Pipe 17	230	4	100
Pipe 18	230	4	100
Pipe 19	250	6	110
Pipe 20	50	6	110
Pipe 21	25	4	100
Pipe 22	450	4	100
Pipe 23	300	6	110
Pipe 24	300	4	100
Pipe 25	75	6	110
Pipe 26	230	4	100
Pipe 27	295	6	110
Pipe 31	125	6	110
Pipe 32	48	6	110
Pipe 35	426	6	110
Pipe 36	394	4	100
Pipe 37	328	4	100
Pipe 38	98	6	110
Pipe 39	98	6	110
Pipe 40	328	4	100
Pipe 41	656	4	100
Pipe 42	131	4	100
Pipe 43	197	8	120
Pipe 44	33	4	100
Pipe 45	295	4	100
Pipe 46	164	4	100

Table A2: Pipe Dimensions and Roughness (Contd.)

Link ID	Pipe Length (ft)	Pipe Diameter (inch)	Hazen-Williams Roughness Coefficient
Pipe 47	197	4	100
Pipe 48	33	4	100
Pipe 49	98	4	100
Pipe 50	66	4	100
Pipe 51	82	4	100
Pipe 52	131	4	100
Pipe 53	49	4	100
Pipe 54	33	4	100
Pipe 56	460	6	110
Pipe 57	295	6	110
Pipe 58	590	4	100
Pipe 59	262	8	120
Pipe 60	262	4	100
Pipe 61	500	3	100
Pipe 62	426	8	120
Pipe 63	394	4	100
Pipe 64	66	4	100
Pipe 65	394	4	100
Pipe 66	295	4	100
Pipe 67	262	4	100
Pipe 68	262	4	100
Pipe 69	394	4	100
Pipe 70	656	8	120
Pipe 71	262	8	120
Pipe 72	459	6	110
Pipe 73	722	6	110
Pipe 74	574	6	110
Pipe 75	164	6	110
Pipe 77	33	8	120
Pipe 78	33	6	110
Pipe 80	656	8	120
Pipe 81	33	6	110
Pipe 79	492	4	100
Pipe 83	98	4	100
Pipe 85	164	8	120
Pipe 86	33	4	100
Pipe 87	361	8	120
Pipe 88	361	4	100
Pipe 89	33	8	120
Pipe 90	525	8	120

Table A2: Pipe Dimensions and Roughness (Contd.)

Link ID	Pipe Length (ft)	Pipe Diameter (inch)	Hazen-Williams Roughness Coefficient
Pipe 92	65	8	120
Pipe 94	525	4	100
Pipe 95	33	8	120
Pipe 96	66	4	100
Pipe 97	230	4	100
Pipe 98	131	4	100
Pipe 99	328	4	100
Pipe 100	394	4	100
Pipe 101	33	8	120
Pipe 102	443	6	110
Pipe 103	164	6	110
Pipe 104	230	6	110
Pipe 105	820	6	110
Pipe 106	33	4	100
Pipe 108	1082	4	100
Pipe 109	197	4	100
Pipe 110	98	8	120
Pipe 111	295	4	100
Pipe 112	33	6	110
Pipe 113	262	6	110
Pipe 114	164	6	110
Pipe 115	262	6	110
Pipe 116	200	4	100
Pipe 117	164	6	110
Pipe 118	262	6	110
Pipe 119	590	4	100
Pipe 120	131	4	100
Pipe 121	209	4	100
Pipe 122	15	4	100
Pipe 123	197	4	100
Pipe 124	15	4	100
Pipe 125	25	4	100
Pipe 126	886	4	100
Pipe 127	360	4	100
Pipe 128	886	8	120
Pipe 129	33	6	110
Pipe 130	66	8	120
Pipe 132	197	8	120
Pipe 133	394	4	100
Pipe 135	426	4	100

Table A2: Pipe Dimensions and Roughness (Contd.)

Link ID	Pipe Length (ft)	Pipe Diameter (inch)	Hazen-Williams Roughness Coefficient
Pipe 136	33	4	100
Pipe 137	230	6	110
Pipe 138	492	6	110
Pipe 139	98	6	110
Pipe 134	987	4	100
Pipe 140	246	6	110
Pipe 141	286	6	110
Pipe 142	394	6	110
Pipe 143	459	6	110
Pipe 144	955	4	100
Pipe 145	456	4	100
Pipe 146	66	4	100
Pipe 147	335	4	100
Pipe 148	150	4	100
Pipe 151	492	6	110
Pipe 152	984	6	110
Pipe 153	230	4	100
Pipe 154	131	4	100
Pipe 155	131	4	100
Pipe 156	295	4	100
Pipe 157	131	4	100
Pipe 158	361	4	100
Pipe 159	337	4	100
Pipe 160	100	4	100
Pipe 161	30	4	100
Pipe 162	318	4	100
Pipe 163	315	4	100
Pipe 164	155	4	100
Pipe 165	255	4	100
Pipe 166	220	4	100
Pipe 167	328	4	100
Pipe 168	328	4	100
Pipe 169	295	4	100
Pipe 170	656	6	110
Pipe 171	328	6	110
Pipe 172	426	6	110
Pipe 173	394	6	110
Pipe 174	98	6	110
Pipe 175	302	4	100
Pipe 176	230	6	110

Table A2: Pipe Dimensions and Roughness (Contd.)

Link ID	Pipe Length (ft)	Pipe Diameter (inch)	Hazen-Williams Roughness Coefficient
Pipe 177	299	4	100
Pipe 178	230	6	110
Pipe 179	292	4	100
Pipe 180	230	6	110
Pipe 181	73	4	100
Pipe 184	295	6	110
Pipe 185	729	6	110
Pipe 186	328	6	110
Pipe 187	164	6	110
Pipe 188	197	6	110
Pipe 189	230	6	110
Pipe 190	98	6	110
Pipe 191	426	6	110
Pipe 192	492	6	110
Pipe 182	60	4	100
Pipe 34	200	4	100
Pipe 91	300	4	100
Pipe 93	30	6	110
Pipe 131	30	6	110
Pipe 183	25	8	120
Pipe 193	15	8	120
Pipe 107	50	4	100
Pipe 194	250	4	100