NETWORK ANALYSIS OF WATER DISTRIBUTION SYSTEM OF DHAKA CITY

A Thesis

by

MD. DELWAR HOSSAIN

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Approved as to style and content by:

(Dr. M. Feroze Ahmed) Professor, Dept. of Civil Engineering, BUET, Dhaka.	Chairman
(Dr. J.R. Choudhury) Professor, Dept. of Civil Engineering, and Dean, Faculty of Civil Engg. BUET, Dhaka.	Member
(Dr. Shamim-Z. Bosunia) Professor and Head, Dept. of Civil Engineering, BUET, Dhaka.	Member
(Mr. S.A.N.M. Wahed) Chief Engineer, Dhaka WASA, Dhaka.	Member (External)

October, 1985

ABSTRACT

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

Using the model, existing water distribution network of Dhaka city has been analyzed and a network has been designed, which will be capable of supplying water to the dwellers of Dhaka city in 1990. It has been observed that most of the existing pipes in the water distribution network of Dhaka city have adequate water carrying capacity in comparison to their flow and a large number of pipes have little flow and can only be serving as service pipes.

The primary mains connecting the three one million gallon elevated water tanks have also been redesigned considering that the excess water required for increased population would be supplied from surface water source. In this case the existing water supply pipes need a major change in size.

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CHAPTER 1

INTRODUCTION

1.1 General

Water is absolutely essential to all life, both animal and plant. It is difficult to imagine any clean and healthy environment without water. Invariably, the progress of sanitation throughout the world has closely been associated with the availability of water. The larger the quantity and the better the quality of water, the more rapid and extensive has been the improvement of public health.

In our houses, whether in the city or in the village, water is essential for cleanliness and health. Man uses water not only for drinking purpose but also for bathing, washing, laundering, heating and air conditioning, for agricultural, industrial and other recreational purposes. Though all these needs are important, water for human consumption and sanitation is considered to be of greater social and economical importance, since the health of the people influences all other activities.

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.

1.2 Need for the Study

The impact of urbanization is felt more intensely in major cities of the country. Specially service facilities of these cities 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. Dhaka city 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 September 1984 estimate, the Dhaka Water and Sewerage Authority (WASA) is capable of supplying only 91 million gallons of water per day for the population of 3.2 millions. It is estimated that only 0.6 million of city population are enjoying adequate supply of piped water at present and rest of the people are experiencing inadequate supply of water(2).

The situation is likely to deteriorate further due to various economic and social reasons associated with unplanned growth of the city. 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 pointssand reduce losses and wastages in the system. In the context of water supply problems in the city of Dhaka, it is essential to analyze the existing water supply network of the city to identify the deficiencies and to suggest improvements in the existing system. The study is expected to outline a rational distribution network suitable for the rapid growing city of Dhaka.

1.3 Objective of the Study

The above discussion of the study essentially focuses on the greater importance of analysis and design of a rational water distribution system for Dhaka city. The water supply problem of Dhaka city sets the following specific set of objectives:

i) To study the present WASA water supply system and to collect data of existing distribution networks in Dhaka city.

ii) To develop a suitable computer programme for the analysis and design of the water distribution network systems.

iii) To analyze the present distribution network by using the programme and to develop some guidelines for the improvement of the existing system.

iv) To design a system capable of supplying adequate quantity of water with sufficient pressure at all service points for the projected population in 1990.

v) Finally, to design the primary network, when the excess water required in 1990 is supplied from a surface water source and the excess flow is regulated by existing primary reservoirs.

CHAPTER 2

LITERATURE REVIEW

2.1 History

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[3] 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 Jerusalam.

The water for the city of Rome was brought from the surrounding hills in aqueducts totalling about 385 miles in length. Among these aqueducts - the Appia, Marcia, claudia and Anionova[9] 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 date can be assigned to their construction but they probably go back to the times of the kings of Judah, 600 to 900 B.C. The 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 16th 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(3) was completed by the Nawab of Dhaka (Sir Nawab Abdul Ghani) in 1876. The water works in Calcutta, Bombay, Madras and Poona were completed in the years 1870, 1875, 1880 and 1890 respectively.

2.2 Water Distribution System

2.2.1 General

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 layout of distribution systems 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

2.2.2 Tree or Branch or Dead End System[3]

There is a simple main which goes on diminishing in size. The small pipe takes off from the main known as branch as shown in Fig. 2.1. This system is suitable for irregular growing towns.

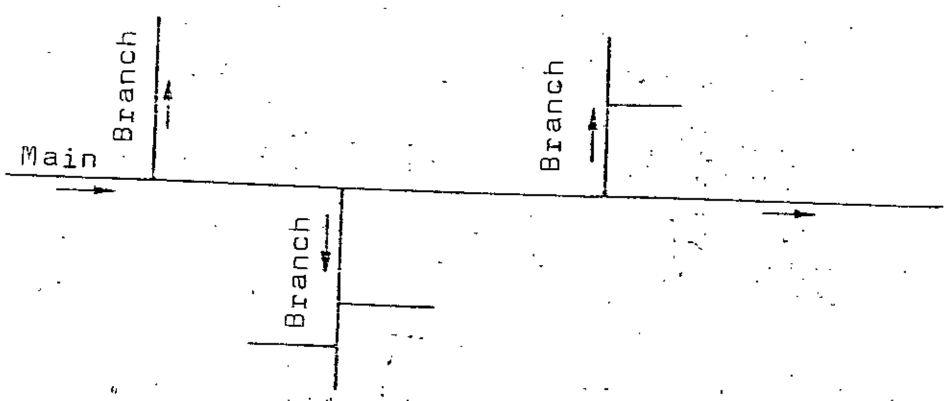

Fig. 2.1 Tree or branch or dead-end system.

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.

2.2.3 Grid Iron System[12]

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 usefull for a city laid out on a rectangular plan (Fig. 2.2). The connections

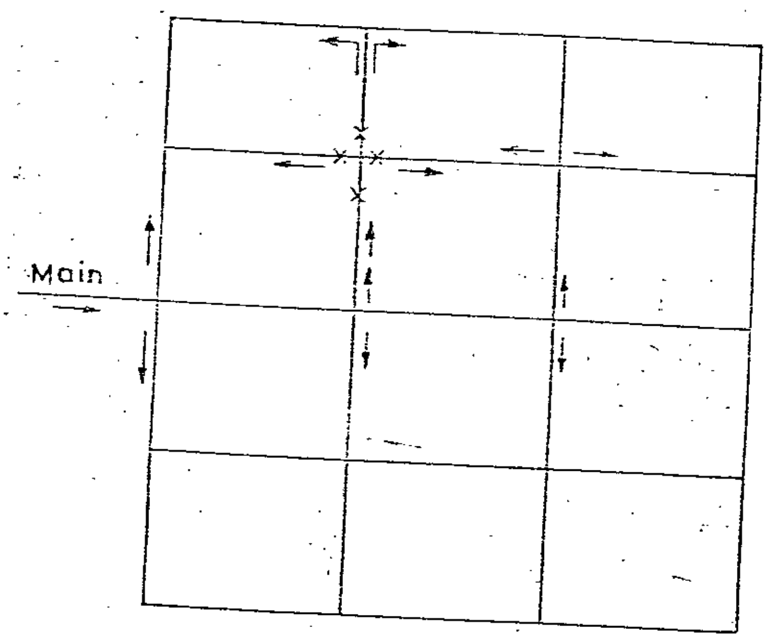

Fig. 2.2 Grid iron system.

of the dead end producing a grid iron pattern, with mains running on main roads in one direction or in perpendicular directions and submains also running alike on minor roads and streets.

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 submain. Disadvantage is that a large number of valves are to be provided.

2.2.4 Circle or Ring System[3]

This consists of dividing the entire district into circular or rectangular blocks and then laying the mains along the peripheral roads with submains branching out from the mains and running on the inner roads and streets as shown in Fig. 2.3. Water can be supplied to any point from atleast two directions.

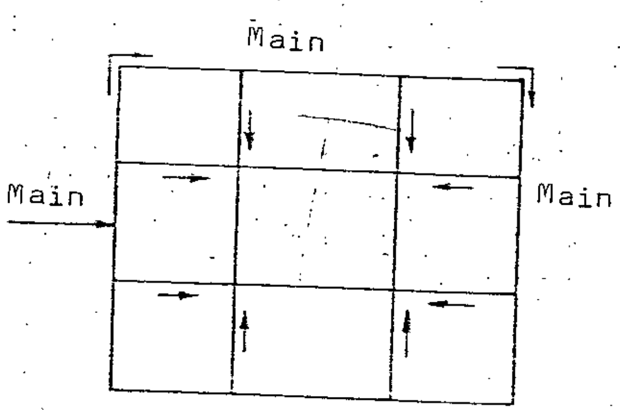

Fig. 2.3 Ring system.

2.2.5 Radial System(3)

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 Fig. 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.

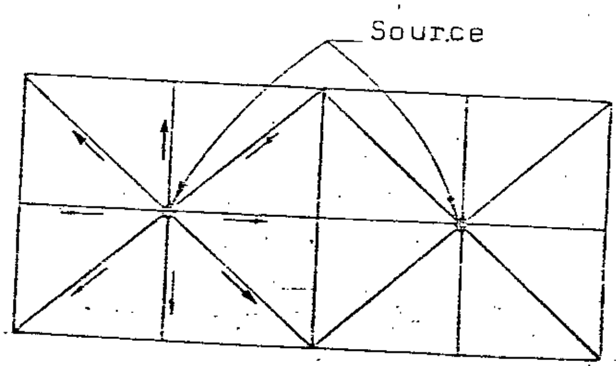

Fig. 2.4 Radial system.

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

2.3 Methods of Network Analysis of Water Distribution System

2.3.1 Equivalent Pipe Method[3]

This method is useful in rendering a complex network of pipes into an equivalent pipe system giving the same discharge and loss of head as in the complex system. For purposes of analysis the entire network of pipes is considered to be arranged in two categories (i) pipes in series and (ii) pipes in parallel.

Pipes in series - pipes carry arbitrarity chosen-values of discharge Q_1 flowing through branches AB and BD and Q_2 flowing through AC and CD (shown in Fig. 2.5). It is assumed that the loss of head for pipes in series is additive. Knowing the discharge (say Q_1) and the diameters of pipe lines AB and BD through which it flows, it is possible to determine the loss of head H_1 in their total length (AB + BD). A single length of equivalent pipe AD of known diameter can be selected to give the same values of discharge Q_1 and loss of head H_1.

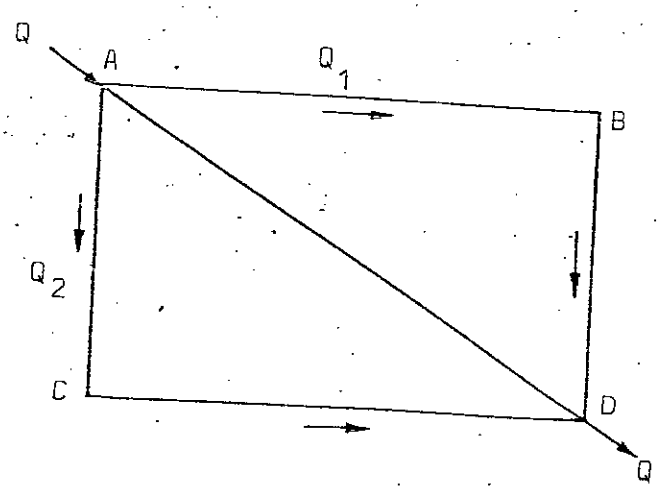

Fig. 2.5 Equivalent pipe method.

Pipes in parallel - In this case, it is assumed that the loss of head through pipes in parallel i.e ABD and ACD (shown in Fig. 2.5) is the same. If a certain loss of head (say H_1) is now assumed to occur in either arm length ABD and ACD, flow through the arms can be worked out and added together to see that the total flow corresponds to the original flow Q. The size and length of a single pipe line can then be calculated to give the same discharge and loss of head.

2.3.2 Electrical Analyzer Method[12]

This method is used in U.S.A. to design the pipe networks for big cities. This method being very costly can only be employed for big cities. In this method electrical resistors are used. The electrical resistors are designed as per equation $H = KQ^{1.85}$. The corresponding electrical equation becomes $V = k_e I^{1.85}$. In this equation V is the voltage drop and I is the current and k_e is the nonlinear co-efficient for the resistor whose value is suited to the pipe coefficient for the selected voltage head loss and the average water flow scale ratio. When inputs and drawoffs of current are made proportional to the water flowing into and out of the system, the loss of head becomes proportional to the measured voltage drop. This may be used to determine the necessary improvements in the existing system.

2.3.3 Hardy-Cross Method[3]

Pipe network problems in water distribution systems are usually solved by methods of successive approximation since any analytical solution requires the use of many simultaneous equations, some of which are nonlinear. It is convenient to express head loss as a function of discharge, i.e,

$$H = KQ^x \tag{2.1}$$

in which H is the head loss in the pipe, Q is the discharge, K is the constant depending upon length, diameter and roughness of the pipe as well as the fluid properties, and x is the exponent.

The solutions for pipe network problems suggested by Hardy-Cross requires that the flow in each pipe be assumed so that the principle of continuity is satisfied at each junction. A correction to the assumed flow is computed successively for each closed loop in the network until the correction is reduced to an acceptable magnitude. If Q_a is the assumed flow and Q is the true flow in a pipe, then the correction is $Q-Q_a$ and

$$Q = Q_a + \Delta \tag{2.2}$$

Expressing head loss by Eq. 2.1 the condition that the head loss around any closed loop be zero gives

$$\Sigma K(Q_a + \Delta)^x = 0 \qquad 2.3$$

Expanding this summation

$$\Sigma KQ_a^x + \Sigma xK\Delta Q_a^{x-1} + \Sigma \frac{x-1}{2} xK\Delta^2 Q_a^{x-2} + \ldots = 0 \qquad 2.4$$

If Δ is small compared to Q, the third and all succeeding terms of the expansion may be neglected. Hence

$$\Sigma KQ_a^x + \Delta\Sigma xKQ_a^{x-1} = 0 \qquad 2.5$$

where, Δ has been removed from the summation since it is the same for all pipes of the loop. Solving for Δ gives

$$\Delta = - \frac{\Sigma KQ_a^x}{\Sigma xKQ_a^{x-1}} \qquad 2.6$$

or $$\Delta = - \frac{\Sigma H}{x\Sigma H/Q} \qquad 2.7$$

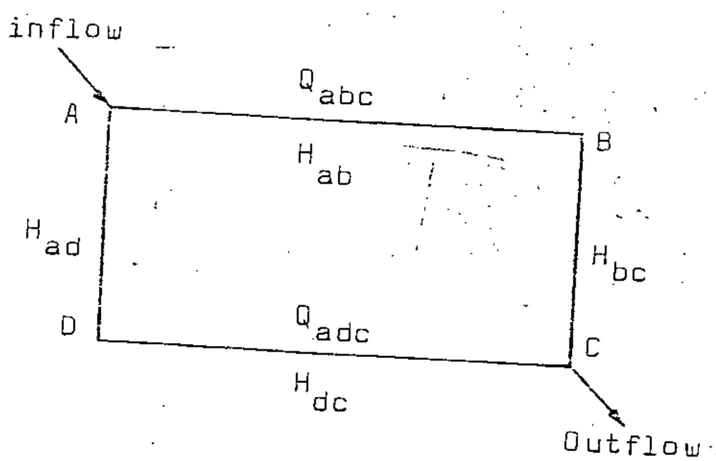

Fig. 2.6 Hardy-Cross method.

Considering a simple loop shown in Fig. 2.6 the arrow heads showing the assumed direction of flow. Two conditions must be satisfied.

i) Inflow at the junction must be equal to outflow at the junction.

ii) Head losses due to flow in the clockwise direction (in pipes ab and bc) must be equal to head losses in the counterclockwise direction (in pipes ad and dc). At the junction C, the flow correction Δ can be computed by

$$\Delta = \frac{(H_{ab} + H_{bc}) - (H_{ad} + H_{dc})}{x(H_{ab} + H_{bc})/Q_{abc} + (H_{ad} + H_{dc})/Q_{adc}} \qquad 2.8$$

where, Δ = flow correction in gpm or cusec.

Q_{abc} = assumed flow in the clockwise direction in pipes ab and bc in gpm or cusec.

Q_{adc} = assumed flow in the counterclockwise direction in pipes ad and dc in gpm or cusec

H_{ab} = head loss in the pipe-ab in ft

H_{bc} = head loss in the pipe bc in ft

H_{ad} = head loss in the pipe ad in ft

H_{dc} = head loss in the pipe dc in ft

x = exponent = 1.85

2.3.4 Newton-Raphson Method (31)

In any complex network of pipe lines, the following three laws are applicable:

i) At each junction, the algebraic sum of inflows and outflows is zero (node countinuity equations).

ii) In any closed path or loop, the algebraic sum of head losses in the elements is zero (loop continuity equations).

iii) In each pipe or element, the head loss is related to the flow by a unique functional relationship (resistance equation).

In a network of NJ nodes and NL loops, the node continuity equations may be written as

$$F_j = Q_{ji} + C_j = 0; \; j = 1,2,\ldots. \; NJ \tag{2.9}$$

Where Q_{ji} is the discharge from node i to node j; ($Q_{ji} = 0$ when no pipe connects i and j, and C_j is the consumption at node; (C is positive when it is an input to the node). The most commonly used friction formula in the water supply systems is the Hazen-Williams formula which may be written as

$$Q = 6.2\times10^{-4} \, C_{HW} \, D^{2.63} \left(\frac{\Delta H}{L}\right)^{0.54} \tag{2.10}$$

in which Q = discharge in cfs, C_{HW} = the Hazen-Williams coefficient of the pipe; D = pipe diameter in inch;

ΔH = head loss along the pipe, in ft; and L = pipe length, in ft

Eq. (2.10) is used in the form

$$Q_{ji} = \frac{H_i - H_j}{R_{ij}^{0.54} \left| H_i - H_j \right|^{0.46}} \tag{2.11}$$

in which R_{ij}, the resistance of the pipe connecting nodes i and j, is given by

$$R_{ij} = \frac{850260\ L_{ij}}{C_{HW_{ij}}^{1.85}\ D_{ij}^{4.87}} \tag{2.12}$$

Eq. (2.11) is written in a form which guarantees a constant sign convention for discharge (i.e $Q_{ji} > 0$ means flow from i to j). Eq. (2.11) can now be used to write eqn. (2.9) (The continuity equation) in terms of heads and consumptions at nodes and pipe resistances, as follows

$$F_j = \sum_{i=1}^{NJ} \frac{H_i - H_j}{R_{ij}^{0.54} \left| H_i - H_j \right|^{0.46}} + C_j = 0 \quad j=1,2,\ldots NJ \tag{2.13}$$

Since there are NJ simultaneous equations one can solve for NJ unknowns. They may be heads, consumptions or resistances. As the equationssare nonlinear, the solution is achieved by successive iterations.

The Newton-Raphson method may be conveniently illustrated for the one dimensional case shown in Fig. 2.7 as follows. The value X_o is sought, such that

$$f(x)/x = X_o = f(x_o) = 0 \qquad 2.14$$

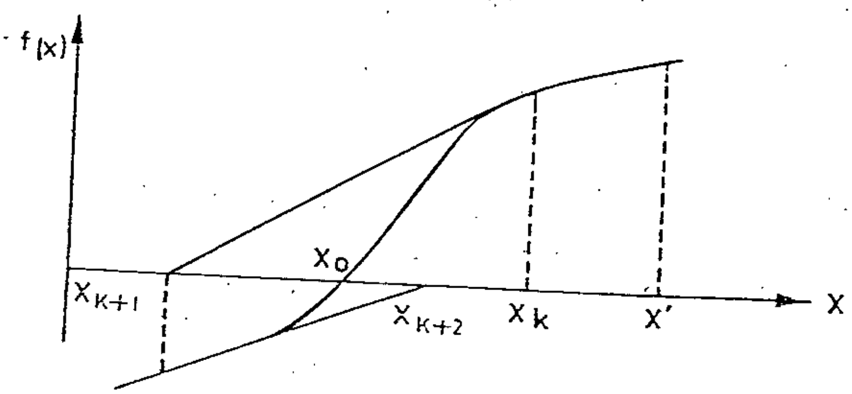

Fig. 2.7 Illustration of the Newton-Raphson method for the one-dimensional case.

At the kth iteration the approximation for x_o is denoted by X_k. The next approximation is given by

$$X_{k+1} = X_k + \Delta X_k = X_k - \frac{f(x_k)}{df(x_k)/dx} \qquad 2.15$$

in which $df(X_k)/dx$ is the derivative of $f(x)$ evaluated at X_k. The equation for the kth improvement ΔX_k, can then be written

$$f(x) + \frac{df}{dx} \Delta x = 0 \qquad 2.16$$

in which it is understood that both $f(x)$ and its derivative are evaluated using the present value of x. The set of NJ nonlinear algebraic equations with NJ unknowns may be written as

$$fj(x_1, x_2, \dots X_n) = 0; \; j = 1,2, \dots NJ \tag{2.17}$$

If NJ unknown $(x_1, x_2, \dots x_{nj})$ to be solved for, the set of improvements $(\Delta x_1, \dots \Delta X_{NJ})$. Expanding eqn. (2.16) by Taylor series and ignoring second order terms we get the general iteration formula for the Newton-Raphson method

$$fj(x_1, x_2, \dots X_{nj}) + \sum_{i=1}^{NJ} \frac{\delta fj}{\delta xi} \cdot X_i = 0$$

$$j = 1, \dots NJ \tag{2.18}$$

After solving equations corrections are then added algebraically to the present values of unknowns solution. A check is then made to determine if all equations given by eqn. (2.13) are satisfied within some specific error criterion (The error criterion representssthe maximummallowable unbalanced discharge at any node). If in checking the present solution, the error criterion is not met at any of the nodes, a new iteration is begun.

2.4 Previous Works

2.4.1 Hardy-Cross Method

Dillingham[11] developed a computer programme named PAWDS by using Hardy-Cross method that contains the following features.

i) The input data is in free format, which means that each of the input quantities does not have to be in specific columns of the data cards, i.e. position on the card is unimportant.

ii) Extensive error checks are performed on the input data.

iii) The method of solution gives good results without requiring excessive computer time.

iv) An initial assumption of head loss or flow distribution is not required as input data.

v) The printed output is self explanatory with good notation.

Hoag and Weinberg(17) developed a programme for the solution of network problems by an electronic digital computer using the basic iterative procedure originally developed by Hardy-Cross. The developed method was more accurate and less costly than the other available analytic methods for a large class of network problems at that time. Programme flow chart for electronic digital computer is given in Fig. 2.8.

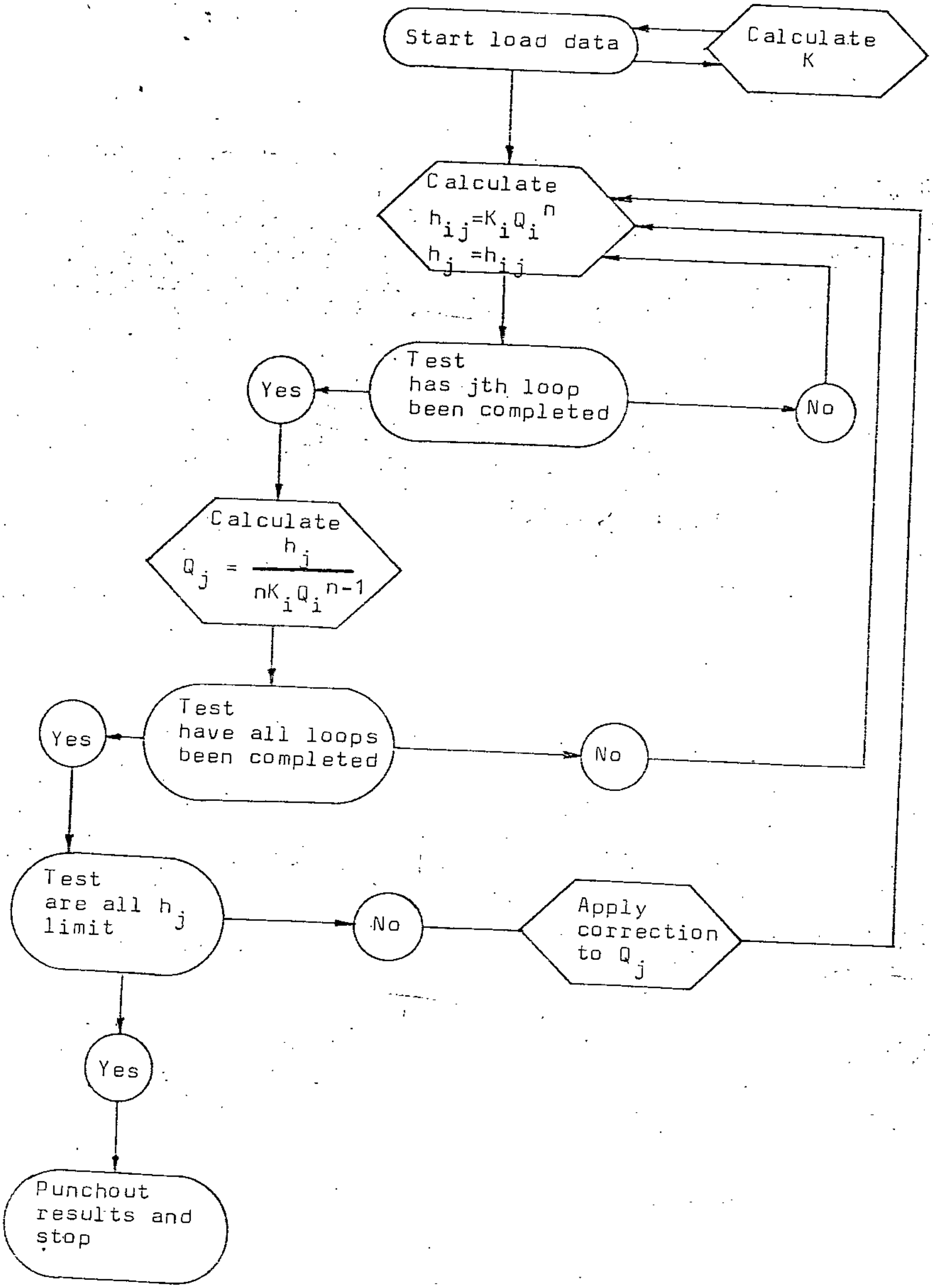

Fig. 2.8 Flow chart for the solution of network by digital computer(17).

Adams[1] outlined various methods available at that time for the theoretical analysis of complex distribution systems and the means by which such analysis could be applied to the practical design for an existing network. To reduce hand calculation he developed a computer programme and analysed by using an electronic digital computer. He also described a method of compiling the data which are supplied to the computer, the method of computation, the methods of interpreting and utilizing the results obtained, an example to use the method and a comparison between the digital computer and the McIlroy analogue as a means of analysing flow networks. The design of the complex distribution systems which are a feature of most urban areas can never be an exact science because of the number of unknown factors involved. The exact loading of the existing system can never be fully simulated assumed roughness factors are only approximate; and the pattern of future demands is even more uncertain. In these circumstances Adams[1] stressed on the electronic computer method combined with field test (availability of pressure contours and flow rates for all parts of the distribution network) that appeared to offer a complete, practicable solution to the whole problem of distribution analysis.

2.4.2 Graph Theoretic Method

Kesvan and Chandrashekar[21] developed Graph Theoretic models for the analysis of nonlinear pipe networks. Some of the differences between the procedure of Graph Theoretic and conventional methods are:

i) Graph Theoretic methods contain topological information in the continuity equations (both loop and node) together with the component characteristics to derive the minimum set of independent equations in a systematic manner.

ii) Once the equations are formulated in Graph Theoretic method, any suitable numerical method for solution can be chosen.

iii) The Graph Theoretic formulation procedure is highly computer worthy. Several programs have been written for the analysis of physical systems related to different fields. One such program, called 'SYSTEM' was developed for the analysis of pipe networks.

Kesavan and Chandrasheker(21) also compared the Graph Theoretic method (SYSTEM) and the Hardy-Cross method by solving a network having same parameters. Number of iterations required in case of Hardy-Gross and SYSTEM were 202 and 60 respectively.

2.4.3 Linear Theory Method

Wood and Charles(35) developed 'Linear Theory' to analyze hydraulic network and found that the use of linear network theory modified to account for the nonlinear head-loss can be readily applied to solve for flow distribution in hydraulic network, convergence to the final result is very rapid. The method does not require initial estimates of flowrate and is directly applicable to both closed and open types of network. They also found that the type of analysis is suitable for digital computer programming.

2.4.4 Electric Analyzer Method

Mcpherson and Radziul[28] discussed the design experiences the McIlroy Network Analyzer and compared it with digital and analogue computers. They concluded that the McIlroy Analyzer was tailor-made for water department network distribution design and is heartily recommended for use in medium and large water departments. The more prevalent digital computers will probably become increasingly adapted to network design by private consultants and will probably be extensively used as soon as general instructions are made available. If a simple pipe simulating component for analogue computer is developed, water hammer characteristics of networks can then be determined with little difficulty and vastly reduced approximation.

2.4.5 Newton-Raphson Method

Martin and Peter[24] established a procedure for solving with the help of an automatic digital computer, the equations which govern the distribution of pressure within a water mains network. The procedure was formulated by means of Newtons method for nonlinear algebraic equations. Before Martin and Peters, Newton's method had been programmed successfully for an electronic digital computer[1] to solve Hardy-Cross method, but the programme was not well organized to facilitate with field adjustment.

Martin and Peter[24] suggested a criterion for adjusting estimates of unknown pipe resistances to provide solution in agreement with measurement.

McCormick[25] studied Newton-Raphson method of water distribution network analysis and found many branches having negligible head differences in their end nodes (i.e. $|H_i - H_j| = 0$) resulted a very small corrections those would tend to produce convergence to the wrong points. To overcome this McCormick considered zero flow (i.e $Q_{ij} = 0$) for those branches and found no difficulty in employing this technique. McCormick[25] further found that applying corrections to individual nodes in a sequence defined by the magnitudes of the continuity errors was more efficient than applying corrections in a sequence defined by the way in which the nodes were numbered, which is directly correcting all variables as one iteration.

Newfville and Hester[26] studied on the Newton-Raphson method for hydraulic network analysis and found that convergency depended on initial assumption of the value and did not converge in some situations. To avoid divergence they suggested that initial estimates would require to be within 5% of the final solution.

Epp and Fowler(13) developed an efficient complete programme to solve the problem of steady state flows in water networks. The features of the program include (a) the use of Newton's method for solving a system of simultaneous nonlinear equations; (b) a loop oriented network to reduce the number of equations to be solved; (c) automatic loop numbering that produces a banded symmetric matrix with consequent reduction in computer memory requirements; (d) the requirement of a minimum input data; (e) an automatic method of reducing core storage requirements and (f) a method of estimating initial flows that lead to fast convergence. In addition, some new ideas have been introduced in the overall solution to the network problem, one of which, namely the algorithm for minimizing the band width of the matrix of coefficients.

Shamir and Howard(31) generalized a steady state solution of a water distribution network by Newton-Raphson method to solve directly for combinations of unknowns which include heads, consumptions and element resistances, using Hazen-Williams' equation, the Newton-Raphson technique finds a new set of improvements and are computed from the first term of a Taylor expansion and the solution was obtained for each iteration by using the Gauss-Jordan elimination procedure.

Shamir and Howard(31) found the technique of solution as the best one to establish relative heads in comparison to known head of any node and it was very much easy to find out consumption in any node or resistance of any network branch.

2.4.6 Study on Different Features of Distribution System

Kanga(19) studied on large distribution systems in the developing countries where towns and cities are growing in size. He tried to solve with the help of theoretical models for studying the effect on cost of adopting different types of designs and the behaviour of the systems subjcted to conditions for which they were not actually designed.

Kanga(19) concluded in his study that tree and branch systems are cheaper than networks and seem to be the obvious choice for adoption by growing towns and cities in developing countries. Over a period of time tree and branch system is converted into networks as the level of economy rises and decisions taken to match the needs as growth takes place. He further suggested that the best course to adopt in a new town is to prepare a design to meet the ultimate needs using a tree and branch pattern and laying the pipes accordingly as the streets are laid out. Later, as development progresses if further strengthening is required it can be achieved in the process of filling in the crosslinks while establishing the ultimate looped network. In that way infrastructure expenditure on constantly remodelling systems with modified boundaries will be avoided, and at the same time by virtue of planning being ahead of demand, the consumers dependent on the system will be better protected against changing conditions as the city grows. The expenditure on link lines which only improve the operational flexibility of the system can also be postponed to the time when the town has grown sufficiently to be able to afford that facility.

Bhave and Lam(6) developed a procedure for obtaining the optimal geometrical layout for a branching network consisting of several source and demand nodes. Initially they considered 3 node system consisting of one source and two demand nodes and showed that the optimal layout is generally the one in which the demands of the two demand nodes are jointly transported to a junction point through a link and then to the two demand nodes through two separate links. The junction point generally lies within the triangle formed by the three nodes, but may also coincide with one of the given nodes. The procedure was then extended to multiple node networks which can be applied to water supply systems for large urban areas in which the several service reservoirs and are supplied from one or more sources.

Featherstone and Jumaily(14) developed a method for the optimal least cost design of new water distribution pipe networks. The design procedure based on either the head balance or Quantity balance method of analysis of flows and pressure distributions in the network of initially assumed pipe sizes. These initial diameters individually are successively and systematically adjusted until the global cost of the network is a minimum. After each adjustment the network is reanalyzed by Darey-Weisbach and Colebrook White equations. The method contains no constraints such as assumed fixed pressure heads or pipe flows and may be applied to either closed loop or open networks of any size.

Kumar[22] used a logarithmic velocity profile in conjunction with a formulation for the origin of the profile to study the nature of wall roughness and the influence of roughness elements on turbulent flow through circular pipes with part smooth, part rough walls and used experimental data on velocity distribution and frictional head loss to derive expressions for surface characteristics of pipes with uniform and non-uniform wall roughness those are helpful in the hydraulic design of pipes in the transition zone. The conclusionsddrawn by Kumar in his studies were as follows:

The hypothetical origin of a logarithmic velocity profile over a surface in the transition zone may be formulated to include viscous effects and the influence of roughness elements. The effect of roughness is specified in terms of equivalent sand roughness projection length and a scale factor. The formulation has been found useful in the interpretation of the nature of wall roughness and in the hydraulic design of pipes in the transitional zone.

The relationship between scale factor and wall roughness Reynolds number, which may be derived from experimental observations of velocity distributions and/or frictional head loss, provides a numerical description of the surface characteristics.

The scale factor increases with increasing discharge and approaches unity at the end of transition for pipes with uniform sand grain roughness and for some pipes used in engineering practice. The scale factor decreases with increasing discharge and approaches unity for pipes with nonuniform roughness and for some pipes used in engineering practices.

Karmeli, Gadish and Meyers[20] formulated a linear programming model for the problem of selecting optimal pipe diameters and power requirements for a water distribution network, which is only suitable for branched network and based on the assumption that the configuration of the pipes connecting the delivery points to the water source is decided upon a priority and having only one source. The model is suitable both for the case in which the water pressure at the source is to be selected and also the case where the pressure is given. For the numerical solution of the problem, any standard linear programming code may be used.

Tong, Connor and Stearns[34] developed a method of sizing mains in water distribution networks directly, rather than selecting pipe sizes by trial and error, which includes an iterative procedure similar to that in the Hardy-Cross method i.e. the method successively adjusts assumed flows in the network to balance relative pipe resistances rather than head losses. In that method equivalent length (Le) of 8 in diameter pipe is calculated by the following equation.

$$Le = L\left(\frac{100}{C}\right)^{1.85}\left(\frac{0.667}{D}\right)^{4.86} \qquad 2.19$$

Le is established as plus (+) and minus (-) for clockwise and counter clockwise direction of the pipe in a loop respectively.

The correction factor is solved for every loop as

$$Q = \frac{L_e}{1.85\left(\frac{Le}{Q}\right)} \qquad 2.20$$

The hydraulic resistance of water main increases after the mains have been in service for some time due to growths or deposits upon the internal surfaces. Colebrook and White(10) developed a formula which gives the relation between the age of a pipe and its carrying capacity, which may be written as

$$\frac{Q}{Q_o} = \frac{1}{P_o}\log\left(\frac{T\alpha}{3.7d} + 10^{-P_o}\right) \qquad 2.21$$

where Q denote the discharge, $P_o = C_o/2{:}8g$ (where C_o is the initial Chezy coefficient) and α is the average rate of growth of roughness.

Colebrook and White computed the value of α from the results of experimental observation by means of the equation

$$\alpha = \frac{3.7d}{f} \left(10^{-P} - 10^{-P_o}\right) \qquad 2.22$$

where $P = C/2\sqrt{8g}$ and C denotes the final Chezy coefficient, d is the diameter of the pipe.

Lam and Wolla(23) formulated a system of node equation based upon the theory of linear graph and can be generated byymeans of a digital computer. In their formulation the system of node equations was generated only once at the beginning of an analysis process and the system of equations was in such a form that minimal time was required to evaluate it during an iterative solution process. They also used different headloss relationships, in addition to the Hazen-Williams formula in the computer program and a subroutine that describes the head-flow relationships which can be modified accordingly.

McPherson(27) generalized network head loss characteristics demonstrated under the limitation of proportional loading. The use of proportional loads is an assumption usually incorporated in network design analyses. He analyzed two examples and found that with proportional loads the percentage distribution of flows in individual pipes in a balanced network was constant irrespective of the magnitude of the total demand.

Gagnon and Jacoby(15) described the simplified approach of computer simulation of water distribution networks for implicit loop method with automatic search technique. They found the implicit loop method for analyzing water distribution networks, designs, and operating conditions and was also an effective method for reducing the number of independent variables and well suited to digital computations.

Graves and Branscome[16] discussed how to utilize digital computers for pipe line network solution and compared with analogue computers and concluded that the solution of water distribution system problems may be performed with the aid of the electronic digital computer with equal or greater facility than McIlroy analogue type computer. The digital computer has the additional advantage of being very useful in the solution of many other problems.

2.4.7 Summary of Previous Works

The oldest method for systematic solution of distribution networks, and the one still most commonly used, is the Hardy-Cross method. This method is well suited for solution by hand, and is easily adopted for machine computation. Computer programme written to perform the Hardy-Cross analysis are described by Hoag and Weinberg[17], Graves and Branscome[16], Adams[1], Dillingham[11] and others.

Electronic network analyzers are discussed by McPherson[27] McPherson and Radziul[28], Graves and Branscome[16], Adams[1] and others. A summary of methods and techniques were presented by McPherson[27].

Bhave and Lam[6] developed a procedure for optimal geometric layout, whereas Featherstone and Jamaily[14], Tong, Corner and Stearns[34]; Karmeli, Gadish and Meyers[20] and others developed methods for sizing mains in water distribution networks.

Martin and Peter[24], McCormick[25], Epp and Fowler[13], Shamir and Howard[31] and others used the Newton-Raphson method in a computer programme to solve for the unknowns of the network and found that the Newton-Raphson technique converges rapidly from a reasonable assumption.

CHAPTER 3

WATER SUPPLY SYSTEM IN DHAKA CITY

3.1 The Water Authority

The Dhaka water supply is managed by Dhaka Water Supply and Sewerage Authority (WASA), created in 1963 as a semi-autonomous government agency. It was enpowered to provide potable water supply and waste water disposal service to greater Dhaka[30]. The authority falls under the aegis of the ministry of Local Government Rural Development and Cooperatives (LGRDC).

The Dhaka WASA is headed by a Chairman, who serves as Chief Executive of the Authority. He is assisted by an Engineering department, a secretariat and a commercial division. The total number of employees of WASA is around 2105 including professional, clerical and operational personnel[18].

The area under Dhaka WASA's jurisdiction has been proposed to be extended to include the towns of Narayangonj, Demra, Tongi, Joydevpur and a part of Savar. Some of these twons are supplied with water by the Directorate of Public Health Engineering (DPHE)[32]. But at present the water supply network of the WASA is functioning within the Dhaka Municipal Corporation limit only. Dhaka city has an extensive water supply system with a primary network mains.

3.2 Source of Water

Ground water is the main source of water supply in Dhaka. At présent (October 1984) Dhaka WASA operates 106 deep tubewells. The underground water is drawn by these deep tubewells and pumped to the consumers. These tubewells are producing 87 million gallon per day (MGD) of water which is about ninety five percent of the total. The average depth of these tubewells varies from 250 to 475 feet, but most of them are over 350 feet deep. The capacity of a tubewell is about 40 thousand gallons per hour.

The only surface water treatment plant at Chandnighat on Buriganga river, was set up in 1876. After treatment, water is pumped to the high level reservoirs. This plant is known as "The Dhaka Water Works". At present the production of this plant is about 4 million gallons per day which is about five percent of WASA's production. This production is mainly used for the old part of the city.

3.3 Water Distribution Mains

Dhaka city has water distribution system consisting of 570 miles mains. Diameter of water mains varies from four to eighteen inches. The distribution system may be divided into primary, secondary and tertiary water mains.

The primary mains bring water from the sources within 2000 ft of any point of the city. The diameter of these mains is eighteen inches. The four inches diameter and larger diameter pipes are designated as secondary mains, and in general any mains less than four inches diameter is classified as tertiary mains. But it has been observed that there is no distinct differences between secondary and tertiary mains, with respect to the function and relative position.

3.4 Overhead Storage Reservoirs

There are about thirtyfive elevated reservoirs in Dhaka city with an aggregate capacity of 6.9 million gallons; however six of these are out of order. At present Dhaka WASA has twenty nine overhead tanks, three with a capacity of 1 million gallons each at Fakirapool, Mohakhali and Lalmatia known as primary distribution reservoirs, and the rest of reservoirs are with a capacity varying from 15000 to 200000 gallons designated as secondary reservoirs. There are tertiary reservoirs with a capacity of 250 to 400 gallons each known as roof tanks.

3.5 Service Connection and Street Hydrants

According to WASA's records there are 79516 service connections (including 14300 unrecorded or illegal connections) serving about 3.2 million population of Dhaka city. Out of these there are about 75753 domestic connections, 1007 for industrial 1656 for commercial and the rest for the government and community services. It is expected that all house connections will be metered by the year 1992[18]. Street hydrants are mainly installed for those people who are not provided or unable to take separate service connections. At present there are about 1225 street hydrants serving 0.67 million persons; these are mainly located in old Dhaka. It is also mentioned here that this figure will be reduced to zero by the year 2010[18].

Besides these, the emergency water carriers which are used to meet the emergency demand of water anywhere in the city. There are about ten water carriers engaged for this purpose. The carrying capacity of these carriers varies from 1000 to 1800 gallons[2].

3.6 Demand and Supply of Water in Dhaka City

Bangladesh is the eighth most populous country of the world, with one of the lowest degrees of urbanization. According to the census 1981, Bangladesh had an urban population of eleven percent of the total population[4]. Dhaka is the largest city of the country having population of 3.4 million, which constitutes about twenty eight percent of the total urban population. Urban population of Dhaka is predicted to be around 9.0 million by the year 2000[29].

There is an existing deficiency of water supply to meet the requirements of population. At the rate of forty gallons per capita per day about 128 MGD of water is needed for the population of Dhaka city, whereas WASA is capable of supplying only 91 MGD for 3.2 million population. It can meet only 71% of the total requirements. Out of the total production, 65% is supplied to the new part of the city and 35% to the old part of the city. 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. In 1980, about 3.5 million gallons of the daily industrial demand was concentrated in the densely built up areas, and this figure could increase to about 6.0 MGD by the year 2010[30]. Other institutional demand depends on different service facilities such as school, hospital, religious institutions, etc. In addition to these, there are numerous demand for parks street gardening, recreations etc. Water production and growth of population with percapita consumptions in Dhaka city are shown in Fig. 3.1.

The increasing demand of water supply in Dhaka city is due to the rapid growth of population. In 1990 the population of greater Dhaka is likely to rise to 6.4 million and

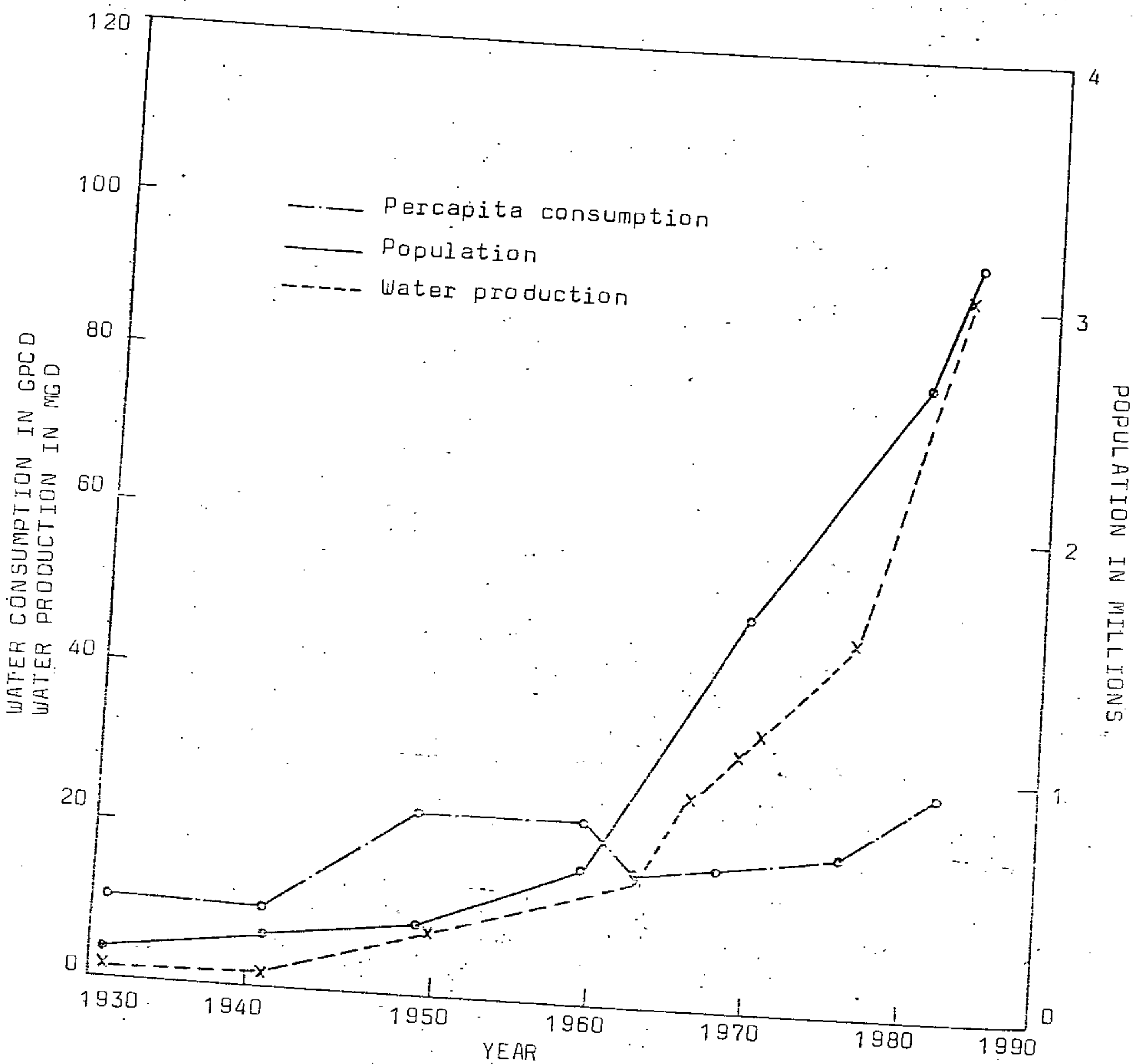

Fig. 3.1 Water production, growth of population and percapita consumption of water in Dhaka city. (Source: Dhaka WASA, 1984).

by the year 2010 it may be 10.20 million[30]. The demand for water supply by that time will be 270 MGD and 670 MGD respectively. With ever increasing number of city's population the demand for water has increased to such an extent which WASA can hardly cope with at present[30]. With the implementation of the Demra water treatment plant it is hoped that the crisis will be minimized.

3.7 Water Supply by Other Organizations

The water supply system of Dhaka city is divided into public and private institutional supply. Public supply means organized water supply system. The Dhaka water and sewerage authority is the only public water supply organization in the city of Dhaka. Private institutional water supply means privately arranged water supply which is made by different institutions, organizations, commercial or industrial firms etc. Such private institutions are Bangladesh University of Engineering and Technology, Dhaka University, Bangladesh Railway, Different Government agencies and many other industrial and commercial firms. Table 3.1 shows the details of public and private water supply in Dhaka city.

Besides these, there are different sources of water supply in Dhaka city, viz. tubewells, wells, ponds and rivers, which also play a significant role in order to meet a part of water demand in Dhaka city.

Table 3.1 Public and private water supply system: Distribution of production (18)

Public water supply		Production in MGD
Dhaka WASA no. of tubewell = 106		87
1 surface treatment plant (Chandnighat)		4
Total		91
Private and insitutional water supply		
No. of well	Institution	Production MGD
3	BUET	0.9
8	Dhaka University	1.75
6	Bangladesh Railway	2.00
7	Different Govt. agencies	2.75
16	Commerce and industries	6.60
40		14.0
Public and private water supply = 105.0 MGD		

3.8 Leakage and Wastage of Piped Water

In Dhaka city water supply is not only inadequate but also irregular. Moreover the flow of water is slow and a huge quantity of water is wasted through street taps, leaky pipes, defective pipe fittings and overflow of roof tanks. At present about forty percent of water production is lost as leakage, wastage and as uncounted water. By increasing the installations of meters and reducing the leakage, wastage of water could be reduced to about twenty percent WASA hopes to be do so by the year 2010 (30). And it is hoped that it will aggrandize the water supply in future.

3.9 Future Development Programme of Dhaka WASA

Dhaka WASA has taken a long term development plan for water supply facilities. The jurisdiction area would be a group of urban centres including urban Dhaka, Narayanganj, Tongi, Joydevpur, Saver and Jinjira. This plan will meet the demand for water supply facilities of greater Dhaka through the year 2010 A.D. and are summarized in Table 3.2.

Table 3.2 Summary of existing facility of Dhaka WASA and proposed future expansion in greater Dhaka (Source: Dhaka WASA)

Name of elements	Facilities of Dhaka WASA existing in 1985	Proposed future expansion in greater Dhaka		
		1986-1990	1991-1995	1996-2000
1. Water purification plant	1	1		
2. Deep tube well	110+3	80	21	20
3. Water mains (miles)	616.4	277.0	262.0	224.0
4. Reservoir (1 MG.each)	3	5	5	4
5. Water service connection	87487	30000	20000	20000

The proposed capacity of the water purification plant at Demra is 100 MGD and there will be a provision in the plant for increasing the capacity upto 150 MGD after 2000 A.D.

But dueeto shortage of fund, the project is being delayed. In the second project from July, 1979 to June, 1985 Dhaka WASA has sunk 45 DTWs and has construsted 174 miles of major water main and also has connected 34000 water service connections. Moreover as an interim measure, Dhaka WASA will sink 25 more DTW from 1986 to 1989, which may help to overcome the worst water crisis in Dhaka city before implementation of the costly project of surface water treatment plant at Demra[18].

CHAPTER 4

ANALYTICAL APPROACH

4.1 Introduction

A simplified method of solution for water distribution networks is presented in this chapter. The method is a modification of Newton-Raphson methodearlier proposed by Shamir and Howard[31]. Obtaining the solution, as defined herein, consists of finding the values of the specified unknowns which satisfy the following physical laws of the network:

i) preservation of mass continuity at each node;

and ii) that for each element there is a known relationship between discharge and energy gradient.

The Hazen-Williams equations most commonly used for water distribution studies, was selected as the law relation pipe discharge to energy loss. Other equivalent equations can be selected if desired.

The present work takes full advantage of the Newton-Raphson method to solve directly for combinations of unknowns which may include heads, consumption and element resistances.

The main advantages of the method are that the number of iterations are fewer than Hardy-Cross technique and any types of water network (i.e branched or closed network) can be analyzed. Moreover errors converge quadratically by Newton-Raphson method.

4.2 Solution of Pipeline Networks

Considering the network having NJ nodes, NJ unknowns can be solved by Eqn. (2.9). Say, the set of unknown heads is denoted by $\bar{H}$, the set of unknown consumptions by $\bar{C}$, and the set of unknown resistances by $\bar{R}$, Eqn. (2.18) can now be used for NJ simultaneous equation for the corrections are

$$F_j(\bar{H},\bar{R},\bar{C}) + \sum_{R_{ij}\varepsilon\bar{R}} \frac{\delta F_j}{\delta R_{ij}} \Delta R_{ij} + \sum_{H_i \varepsilon \bar{H}} \frac{\delta F_j}{\delta H_i} \Delta H_i$$

$$+ \sum_{C_i \varepsilon \bar{C}} \delta_{ij} \Delta C_i = 0 \qquad 4.1$$

in which $H_i \varepsilon \bar{H}$ signifies 'H_i is in the set $\bar{H}$' (i.e H_i is an unknown), and δ_{ij} is the kroneker delta ($\delta_{ij} = 1$, when $i = j$ and $\delta_{ij} = 0$ otherwise).

The partial derivatives in eqn. (4.1) are given by

$$\frac{F_j}{H_i} = \frac{0.54}{R_{ij}^{0.54} \left| H_j - H_i \right|^{0.46}} = \frac{F_i}{H_i} \qquad 4.2$$

$$\frac{F_j}{H_j} = - \sum_{i \neq j} \frac{F_j}{H_i} \qquad 4.3$$

$$\text{and} \quad \frac{F_j}{R_{ij}} = \frac{0.54 \left| H_j - H_i \right|^{0.54}}{R_{ij}^{\;1.54}} = \frac{F_i}{R_{ji}} \qquad 4.4$$

Eqn. (4.4) is a modified form which was proposed earlier by Shamir and Howard[31].

Eqn. (4.1) can be written inmatrix form as follows

$$\begin{bmatrix} \frac{F_1}{R_{pq}} & \cdots & \frac{F_1}{H_s} & \cdots & \frac{F_1}{C_t} \\ \cdot & & \cdot & & \cdot \\ \cdot & & \cdot & & \cdot \\ \frac{F_n}{R_{pq}} & \cdots & \frac{F_n}{H_s} & \cdots & \frac{F_n}{C_t} \end{bmatrix} \begin{bmatrix} R_{pq} \\ \vdots \\ H_s \\ \vdots \\ C_t \end{bmatrix} = \begin{bmatrix} -F_1 \\ \cdot \\ \vdots \\ -F_n \end{bmatrix} \qquad 4.5$$

The matrix of derivatives on the left side is called the Jacobian of the set of equations. The numerical values in this matrix and on the right side vary from one iteration to the next, as they are computed at each iteration with a new set of values for the unknowns. The solution of eqn. (4.5) was readily obtained for each iteration by using the Gauss elimination procedure.

To illustrate the structure of eqn. (4.5), it is written for the network shown in Fig. 4.1, an example network, with NJ = 5, NL = 7.

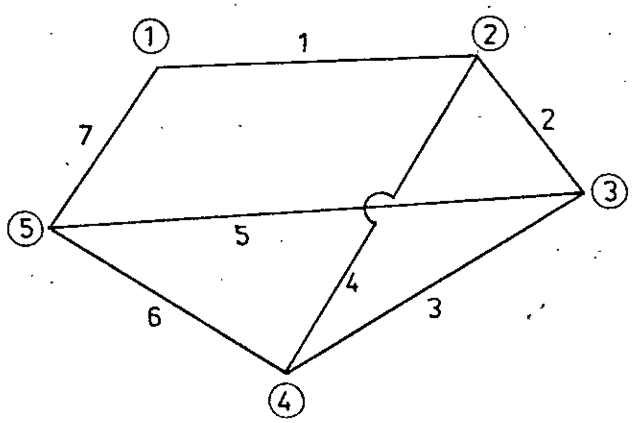

Fig. 4.1 Example network .

The unknowns for this example are

$$\bar{H} = (H_1, H_3) \quad 4.6$$

$$\bar{C} = (C_3, C_4) \quad 4.7$$

and $\bar{R} = (R_{24}) = R_4$ 4.8

and eqn. (4.5) takes the form

$$\begin{bmatrix} 0 & \frac{F_1}{H_1} & 0 & 0 & 0 \\ \frac{F_2}{R_4} & \frac{F_2}{H_1} & \frac{F_2}{H_3} & 0 & 0 \\ 0 & 0 & \frac{F_3}{H_3} & 1 & 0 \\ \frac{F_4}{R_4} & 0 & \frac{F_4}{H_3} & 0 & 1 \\ 0 & \frac{F_5}{H_1} & \frac{F_5}{H_3} & 0 & 0 \end{bmatrix} \begin{bmatrix} R_4 \\ H_1 \\ H_3 \\ C_3 \\ C_4 \end{bmatrix} = \begin{bmatrix} -F_1 \\ -F_2 \\ -F_3 \\ -F_4 \\ -F_5 \end{bmatrix} \qquad 4.9$$

in which the derivatives are given by eqns. (4.2), (4.3) and (4.4). Since the continuity equations involve only differences between heads, it is necessary to specify atleast one head to establish a datum. If the problem is to solve for all the heads in the network, all the consumption and resistance are given. Then it is necessary to fix one head, considering a resistance as unknown.

4.3 Computer Programme

A computer programme, based on the theory (sec. 2.3.4 and 4.2) is written in FORTRAN IV, which may be run on both the IBM 370-115 and IBM 4331 computers installed at the Computer Center, BUET. Total real memory of IBM 370-115 and IBM 4331 are 256 KB and 2 MB respectively. The virtual memory assigned to a particular partition may vary, however the programme can be run using 576 KB virtual memory on IBM 370-115 and 2188 KB on IBM 4331-Ko_2 system.

The Computer programme consists of a main programme and a subroutine. The name of the main programme and the subroutine are 'MAIN' and 'RESIS' respectively. A flow chart of the computer programme is given in Appendix-B.

The following are the functions of the main programme and the subroutine.

Main Programme

The main programme reads the necessary input data for the analysis of water distribution network. The input data required for the computations are number of node, number of pipe, Hazen-Williams coefficient, node to node relation, designation of nodes connecting each node, starting and end node number of pipes, pipe number, pipe length and pipe diameters. Heads of each node are given for initial iteration.

The main programme computes pipe resistances in each pipe. In any iteration, the main programme also compute netflows in each node, form a banded coefficient matrix and solve for unknowns. Unknowns are used as corrections of nodes head. Also there is a provision to check whether the netflow in each node is less, equal or greater than a given allowable value. And finally prints the results of computations, when netflow in any node does not exceed given allowable limit.

The main programme calls subroutine RESIS, in computing netflows and formation of coefficient matrix. Memory requirements for analysis and design of the water distribution network of Dhaka city are 227 KB and 237 KB respectively. To analyze the network of water distribution system of Dhaka city, time require per iteration on IBM 370-115 and IBM 4331 computers are about 15 minutes and 1 minute 30 seconds respectively.

Subroutine RESIS

This subroutine is mainly used to reduce the memory storage of the computer programme. By using starting and end node number, corresponding pipe numbers are selected to use pipe resistances in the computation of netflows and in matrix formation.

4.4 Analysis of Water Distribution System

4.4.1 General

The network of water distribution system of Dhaka city includes 761 number of pipes having diameter 4 inch or more and 536 nodes that were shown in Fig. 4.1. The network was analyzed by Newton-Raphson method and Hazen-Williams coefficient (C_{HW}) was assumed as 120. Water connections to houses are usually made at intermediate points between two subsequent nodes (shown in Fig. 4.2) but for the analysis it was assumed that all water was consumed from nodes only. Though difference in elevations of most parts of Dhaka city varies within 3 feet, the network was considered to be laid in a horizontal plane.

In the Computer programme number of pipes, number of nodes, Hazen-Williams coefficient, node consumptions, node to node relations along with length, diameter, starting node number and end node number of pipes were supplied as data. To calculate the flow in each pipe by Hazen-Williams equation, heads were assumed in each node initially. After calculating netflow (i.e. inflow-outflow) of nodes, A Jacobian of the 536 equations was formulated by using eqns. (4.2), (4.3) and (4.4). The Jacobian was a non-symmetric banded matrix having band width 71. The linear equations formulated by Jacobian as left side matrix and netflows with negative sign as rightside column matrix were solved for unknowns, by Gauss elimination method for non-symmetric banded systems[8].

4.4.2 Phase 1: Analysis of Existing Network

In this phase existing network of Dhaka WASA water supply system was analyzed. Dhaka WASA operates 106 tubewells but 103 tubewells were considered in the analysis (excluding two

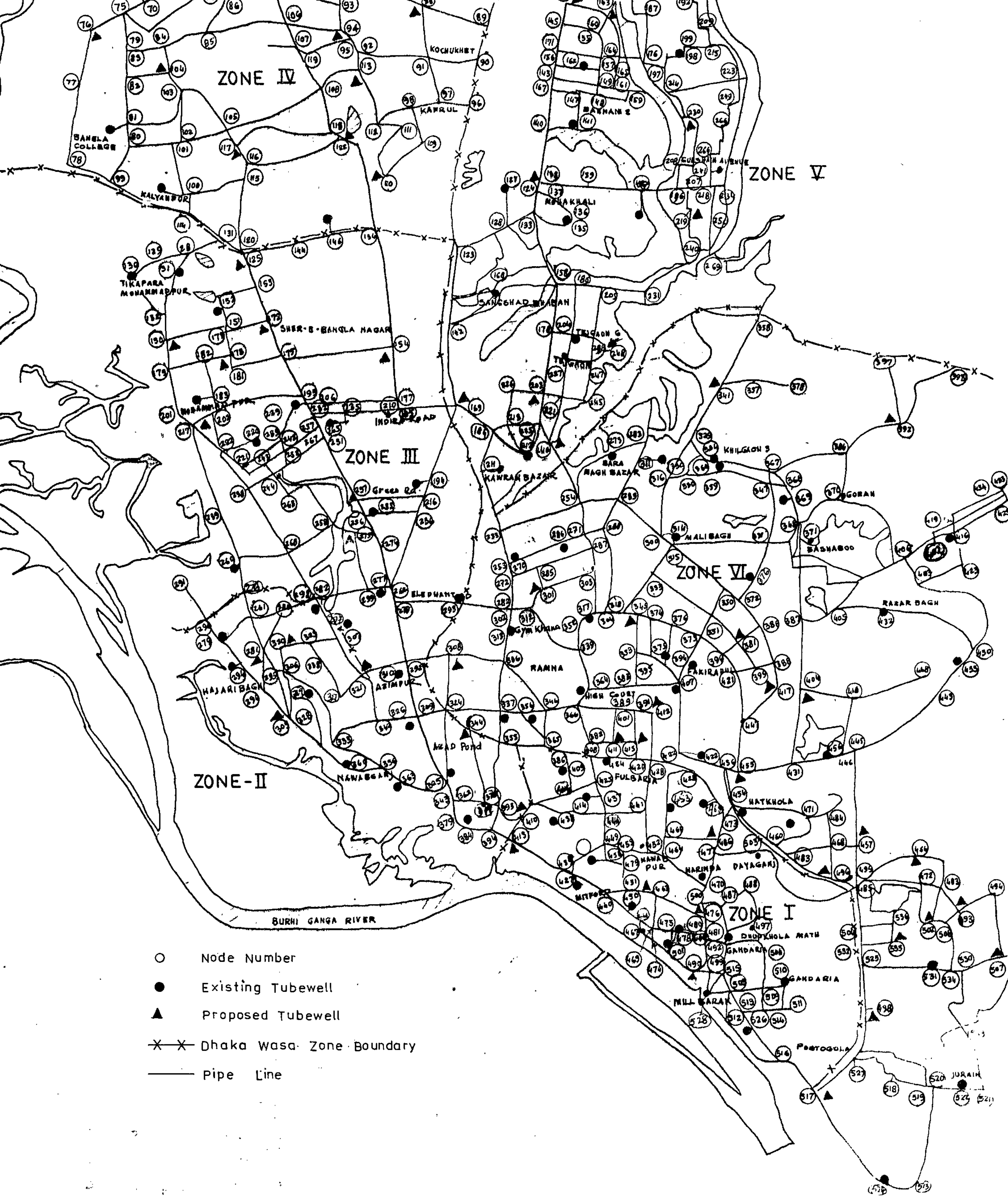

Fig. 4.2 Network of Dhaka WASA water supply system (Source: Dhaka WASA, 1984).

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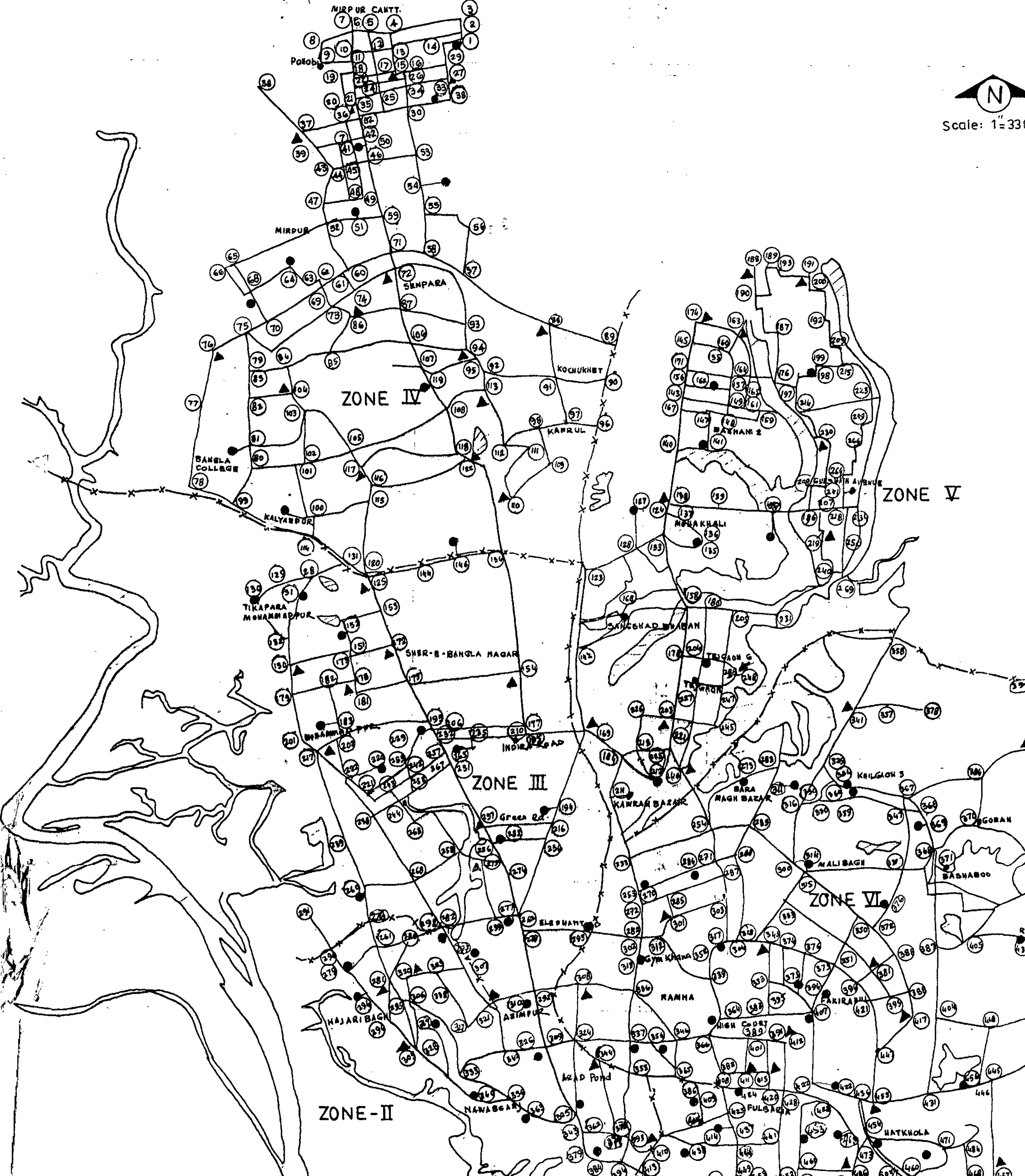
N
Scale: 1"=33
MIRPUR CANTT.
Pallabi
MIRPUR
SENPARA
KOCHUKHET
ZONE IV
KAFRUL
BANGLA COLLEGE
KALYANPUR
TIKAPARA MOHAMMADPUR
SHER-E-BANGLA NAGAR
INDIRA ROAD
ZONE III
Green Rd.
ELEPHANT
GULSHAN AVENUE
ZONE V
MOHAKHALI
SANGSHAD BHABAN
TEJGAON
KAWRAN BAZAR
BARA MAGH BAZAR
KHILGAON
GORAN
MALIBAGH
BASHABOO
ZONE VI
RAMNA
HIGH COURT
FAKIRAPUL
AZIMPUR
HAJARIBAGH
AZAD Pond
FULBARIA
NAWABGANJ
HATKHOLA
ZONE-II

tubewells in Uttara and one in Baridhara). The network was solved considering one head of a node (node number 1) as fixed and one diameter of a pipe (pipe number 73, connected by node number 1 and node number 2) as unknown along with other unknown node heads. To analyze the existing network, 46 iterations were required to balance the flow of all nodes.

4.4.3 Phase 2: Design of the Network Based on Ground Water

In this phase, the network was designed for the distribution of water, which would fulfil the demand of Dhaka city in 1990. It was expected that population of Dhaka city in the existing water supply area would be about 4 millions. At the rate of 40 GPCD, total water demand in 1990 would be 160 MGD. Assuming the production capacity of existing tubewell would decrease by about 20 percent, the network was designed by proposing 64 new DTWs to fulfil the water demand in 1990. Location and discharge capacity of DTWs were shown in Fig. 4.2 and in Appendix-B respectively. The network was designed considering the velocity of flow between 2 and 5 fps in pipes. Computer programme was modified to design the network. A provision to check the velocity of flow in the pipes and accordingly to adjust pipe diameter was incorporated in the program. A total of sixty seven iterations was required to obtain the sizes of the pipes in the network.

4.4.4 Phase 3: Design Based on Surface Water

In this phase, the main network consisting of large diameter pipes i.e pipes connecting primary reservoir system (described in Chapter 3 and shown in Fig. 4.3) was analysed. In this case, total water proposed to be supplied by DTWs (proposed in Phase 2) is now assumed to be collected from surface water source and supplied through these three primary reservoirs. Assuming 54.8, 54.0 and 54.0 cfs through Fakirapool, Mohakhali and Lalmatia primary water reservoirs respectively, the pipes were designed considering velocity of flow between 2 and 5 fps and diameters of pipes were changed accordingly.

4.4.5 Phase 4: Design Based on Both Surface and Ground Water

In this phase, the network described in third phase of study was redesigned considering both surface and ground water as sources of water supply to meet the additional water demand in 1990. It is expected that additional 25 DTWs having capacity two cfs could be sunk in the periphery of Dhaka city, without interferring existing DTWs. The rest demand of water those were described in the third phase of study, would be obtained from surface water source and supplied by three primary reservoirs. In this case, 37.95, 37.4 and 37.4 cfs water would flow through Fakirapool, Mohakhali and Lalmatia primary reservoirs respectively.

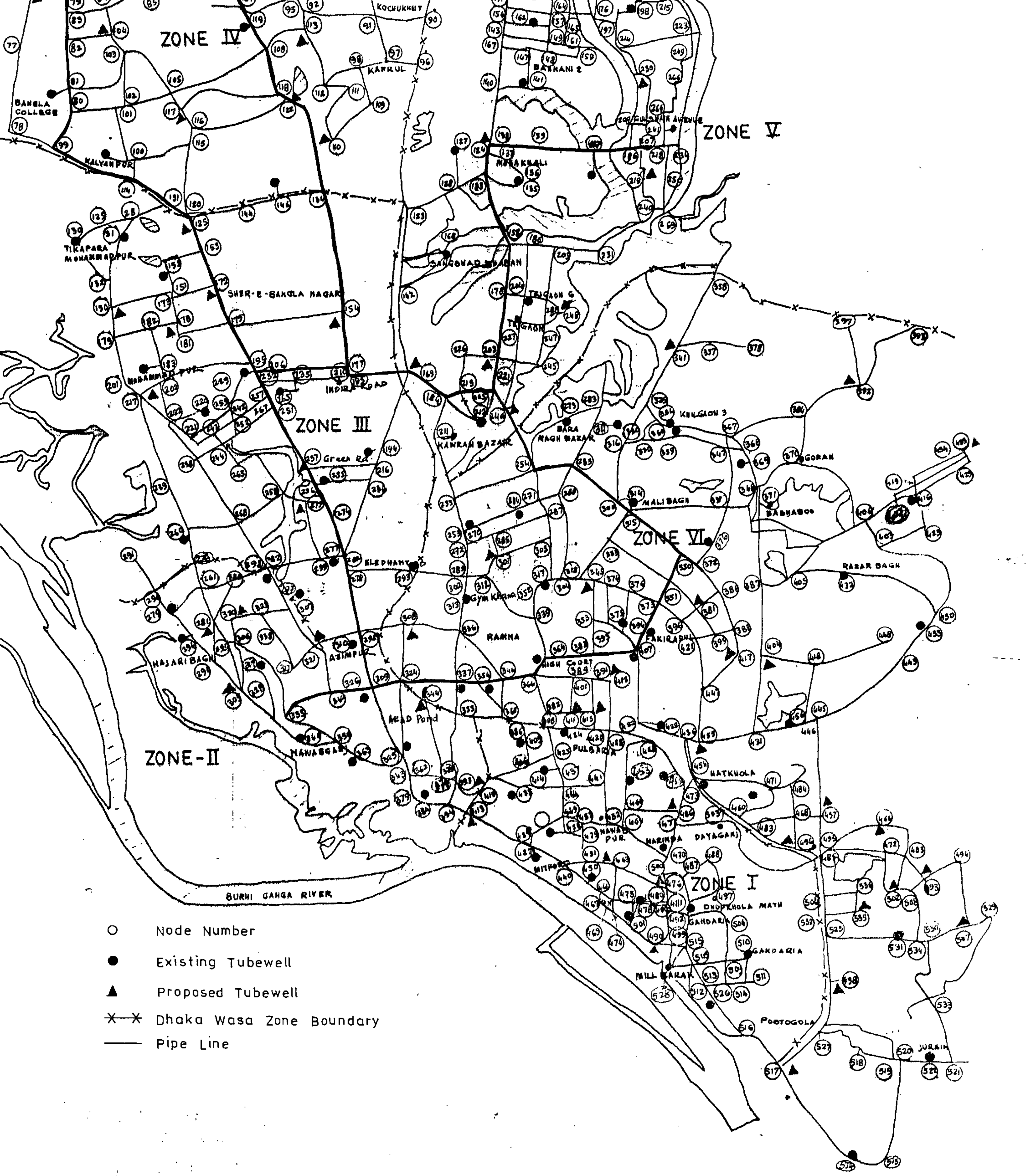

Fig. 4.3 Network of Dhaka WASA water supply system showing primary network by thick line (Source: Dhaka WASA, 1984).

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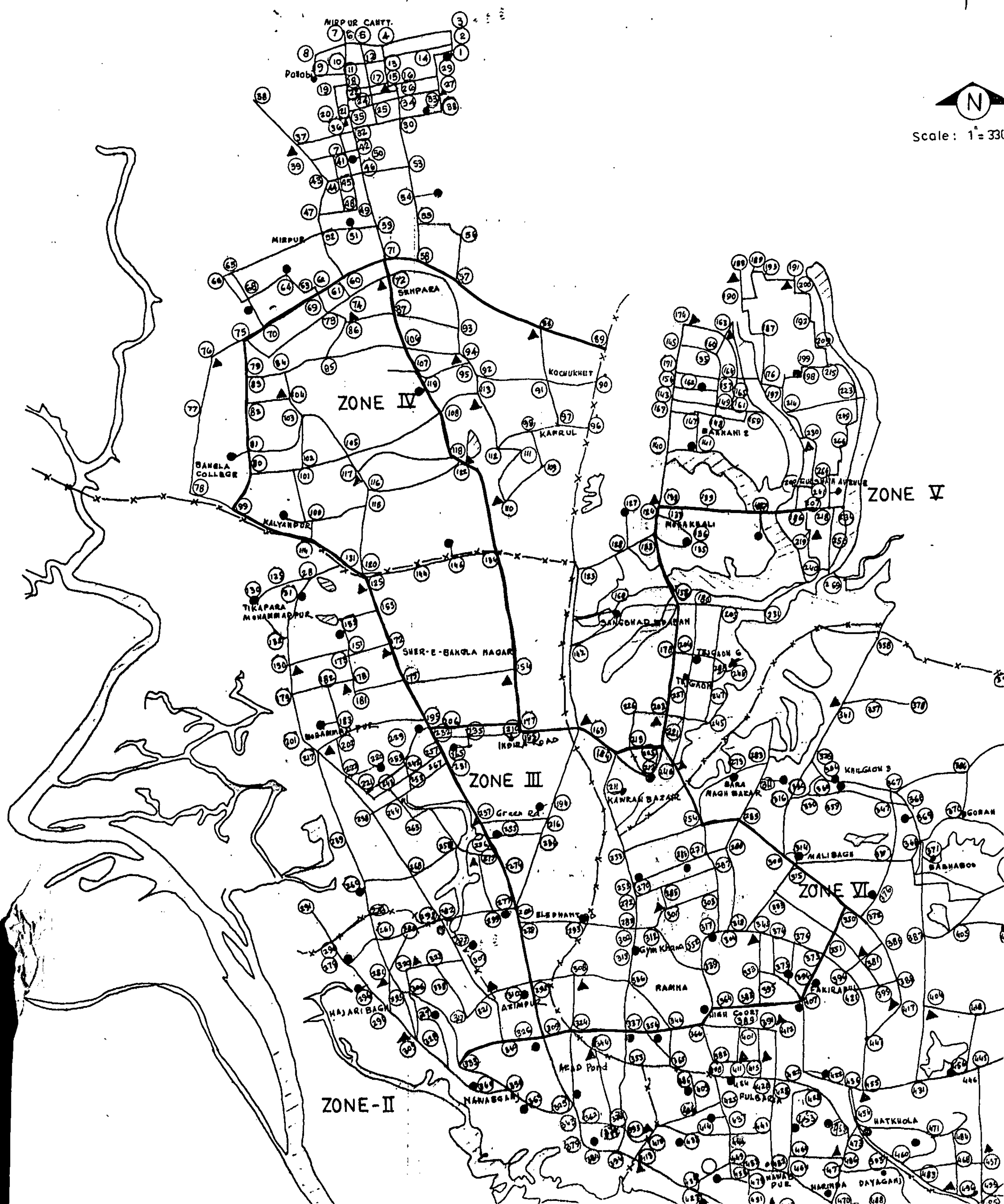
N
Scale: 1" = 330
MIRPUR CANTT.
Pallabi
MIRPUR
SENPARA
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KAFRUL
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KALYANPUR
TIKAPARA MOHAMMADPUR
SHER-E-BANGLA NAGAR
MOHAMMADPUR
INDIRA ROAD
ZONE III
Green Rd.
ELEPHANT RD
Gym Khana
RAMNA
AZIMPUR
HAJARIBAGH
Azad Pond
NAWABGANJ
ZONE-II
MOHAKHALI
SANGSHAD BHABAN
TEJGAON
KAWRAN BAZAR
BARA MAGH BAZAR
KHILGAON
MALIBAGH
GORAN
BASABOO
ZONE V
ZONE VI
GULSHAN AVENUE
BARKANI 2
FAKIRAPUL
HIGH COURT
FULBARIA
HATKHOLA
NAWABPUR
NARINDA
DAYAGANJ
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CHAPTER 5

RESULTS AND DISCUSSIONS

In this study the method used to analyze water network system is Newton-Raphson method. Wood and Charles summarized the comparison made by Liu[35] that number of iterations required to analyze a 58-pipe network by Hardy-Cross and Newton-Raphson method were 635 and 24 respectively. The Hardy-Cross method adjustments each loop independently providing no direct interaction between the basic network equations. The Newton-Raphson method adjusts all loops simultaneously and thereby, greatly improves the convergence to the solution. Martin and Peter[24], McCormick[25], Epp and Fowler[13], Shamir and Howard[31] and others used the Newton Raphson method to solve for unknowns of the network and found, the Newton-Raphson technique converges rapidly provided a reasonable initial assumption is made. In this method netflow (i.e. error) in any node of a network converges quadratically[33] hence in a few iterations the flow in a network can be balanced. A typical pattern of convergence has been shown in (Fig. 5.1). Moreover any type of network, whether closed or branced can be solved by Newton-Raphson Method. Though the calculations of the method are difficult, the author utilized the full advantages of the use of IBM 4331 Computer of BUET Computer Centre.

The most commonly used friction formulae in the water supply systems is the Hazen-Williams formulae[31]. For smooth

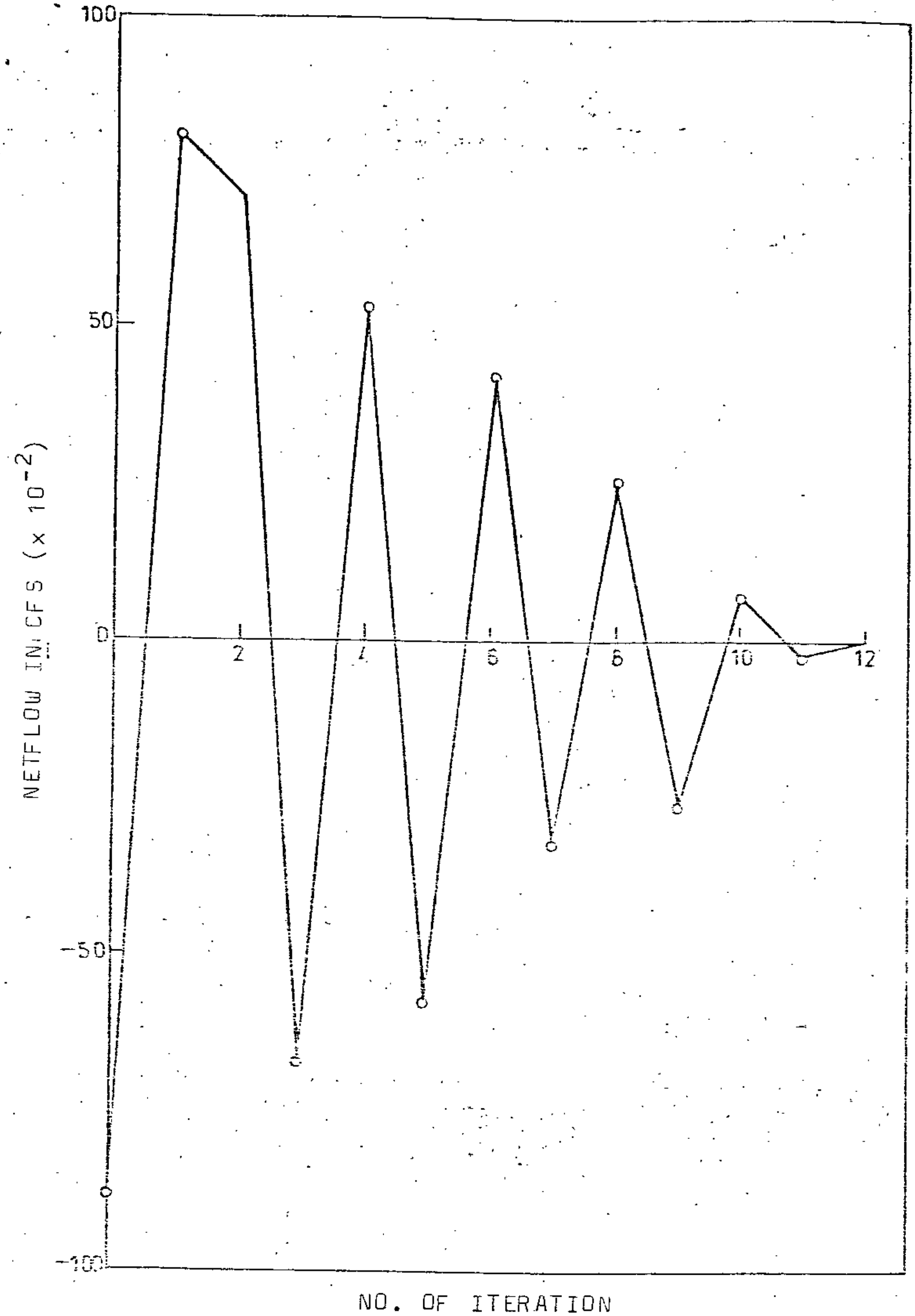

Fig. 5.1 Convergence of netflow in a node (Node no. 2).

and most rough pipes Hazen-William coefficients (C_{HW}) are 140 and 100 respectively[9]. Most of the recently constructed pipe lines by Dhaka WASA have considerably smoother PVC pipes. Again many pipe lines of Dhaka WASA are quite old. Moreover due to some of the valves not being operated, velocities of water in some of the pipes are very low. Growth of algae, and deposition of insoluble salts, iron oxides and suspended and colloidal particles inside the pipes in nonsupply hours may make the pipe semirough. The author, therefore found it more logical to use the Hazen-Williams friction formulae with C_{HW} as 120 for network analysis.

A simplified method of solution for water distribution system has been presented in Chapter 4, which is a modification of Newton-Raphson method earlier proposed by Shamir and Howard[31]. Shamir and Howard[31] presented eqn. (4.4) as

$$\frac{F_j}{R_{ij}} = \frac{-0.54\ (H_i - H_j)}{R_{ij}^{\ 1.54}\ |H_i - H_j|^{0.46}} \qquad 5.1$$

But, using this equation the author found that more than one identical equations had been formed in the analysis of a network that made the solution of equation not feasible when computed both by hand and by computer programming. By observing eqn.(2.12) it is found that flow in a pipe depends on head difference of the connecting nodes of that pipe. Also, for any direction of flow, magnitude of resistance is always positive. Moreover, from computation, it is found that eqn. (4.4) gives the general solution instead of eqn. (5.1), which was simplified and used by Shamir and Howard[31].

Population of Dhaka city was about 2.66 million in 1981[4] and approximately present (Oct. 1984) estimate is about 3.4 million of which Dhaka WASA serves water for 3.2 million through a network of distribution system shown in Fig. 4.1 and private agencies and other institutions serve for the rest[18]. Population in Dhaka city increases mostly in periphery. Rate of population increase in the existing Dhaka WASA water supply area is expected not to be changed significantly in future. For this reason the author estimates the rate of growth initially as 6.5 percent upto 1984 and an average of 3.5 percent for the next years upto 1990. Because growth of population approaches towards the saturation point in the existing water distribution area. The expected population of Dhaka city in the existing water supply area would be about 4.0 million in 1990.

A survey was conducted by the author to assess water consumption rate of some resisential areas. The results of survey are shown in Appendix-A. It appears from the survey that the consumption rate is as high as 70 GPCD in some high class residential areas like Gulshan, Dhanmondi etc. Water consumption in some slum areas are around 15 to 20 GPCD[2]. In this study an average rate of water consumption has been assumed as 40 GPCD.

The total water supplied by Dhaka WASA was about 91 MGD (October 1984), 71 percent of total water demand in the city. Analysis of present network are shown in Appendix-B. The analysis was performed assuming the network to be in a horizontal plane as the variation in elevation in most parts of Dhaka city is within 2 to 3 feet. Heads at different nodes were calculated with respect to a fixed head. In the analysis, head of node 1, considered as 81 feet (35 psi) fixed head and relative heads of other nodes were shown in Appendix-B. Existing flow in pipes were shown in Appendix-B. Diameter of pipe 1 become 4 inch after last iteration. From the analyzed results it was visualized that a large number of pipes conveying very small quantity of water relative to their pipe size and 90 pipes conveying less than 0.1 cfs (i.e negligible flow) flow. The later pipes can be considered as dummy pipes (or excess pipes) in the network mains. These dummy pipes are being used as service pipes in the distribution system.

In the year of 1990, for 4.0 million expected population, at the rate of 40.0 GPCD, total requirement would be 160.0 MGD. In the second phase of the study, the author assumed that capacity of the existing tubewells would decrease by twenty percent and the balance would be supplied by 64 proposed new tubewells. The location of proposed DTWs are shown in Fig. 4.1 and capacity of proposed DTWs are shown in Appendix-B.

Velocity of flow in a water supply pipe should be 3 - 4 fps[3]. In designing the network, the author considered the permissible velocity between 2 and 5 fps. Velocity of flow may be reduced because of flow fluctuations. Head at different nodes with respect to node number 1, flow in pipes and designed diameter of pipes were shown in Appendix-B.

Layout of Dhaka city water distribution system is a combination of radial, grid iron and dead end systems. Source of water in Dhaka city is mostly ground water (i.e Deep tube-well). Around most of the tubewells, upto some distant, layout of pipes are almost radial. In many areas in the periphery of Dhaka city, where houses are constructed in a hapazard way, layout of water distribution system generally follows dead end one. Other than these, layout of water distribution system are grid iron.

Kanga[19] studied on large distribution systems and found that tree and branch system are cheaper than any other networks in growing cities in developing countries. With the development of the area, if further strengthening is required, the cross-links are filled, establishing the ultimate loop network. Some parts of Dhaka city have gradually developed and are still developing in periphery areas. Initially water is supplied through dead end system, which ultimately become grid iron when the areas are sufficiently developed. Though it concides with the findings of Kanga[19], due to improper design, water distributions in Dhaka city are not evenly distributed. It is found from the analysis that a large number of pipes conveying very small quantity of water relative to their pipe sizes.

It has been observed that out of 185.3 miles analyzed pipe lines, 125.93 miles pipe would require reduced diameter in the network system. It has also been observed that 9.48 miles pipes would be used as dummy, having no utility in the distribution system and is indicated by 0.0 inches diameter as shown in Appendix-B. As mentioned earlier, water connection to houses are usually made at intermediate points between two subsequent nodes but for the analysis and design, it was assumed that water was consumed from nodes only. The later pipes (i.e. dummy pipes) may serve as service pipes in water distribution system. Due to improper design these larger diameter pipes have little contribution in the distribution system but have significant influence on the increase in cost of water distribution system. A properly designed distribution system could save money and prevent incrustation of the pipes in the network.

The base of the aquifer of Dhaka is at a depth of about 450-500 feet. Generally this layer has coarser material with the increase in depth and contains medium and coarse grained sand with gravel. The deeper portions tend to be the most productive parts of the aquifer. Below 500 feet the aquifer contains sediments of finer material (mostly silt & clay) upto a depth of 1,500 feet. The observed aquifer storage coefficients and transmissibility of the aquifer are $1.5 \times 10^{-4} - 1.9 \times 10^{-4}$ and 50,000 - 180,000 gallons/ ft. respectively.

Continuous ground water withdrawal in the past 20-30 years has reduced ground water levels in the Dhaka metropolitan area to elevation below local confining layers. The principal source of the ground water is the percolation of abundant rainfall, which occurs yearly during the monsoon season of April to September. The precipitation enters the aquifers by vertical seepage through the overlying confining layer and the river system. The confining layer in Dhaka city area is about 20-30 ft thick and has poor water transmission characteristics.

Ground water withdrawal for Dhaka has gradually increased since the first use of ground water in 1949. In response to this withdrawal, the piezometric surface has been gradually depressed. By comparing the increase in production and decrease in the static water line it appears that the two are inversely proportional, as shown in Fig. 5.2 and Fig. 5.3. The Fig. 5.2 shows that the yearly withdrawal of water is not fully recharged and causing a gradual lowering of water table as shown in

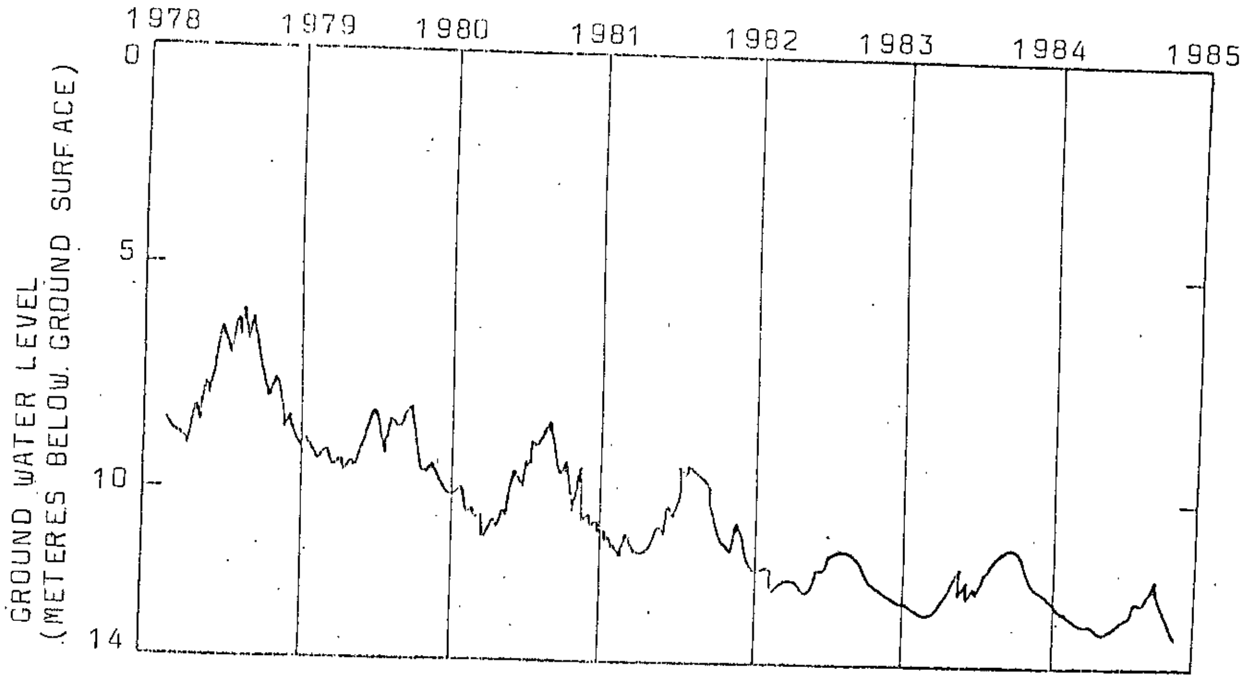

Fig. 5.2 Hydrograph of observation well DH 103,
Tejgaon, Dhaka
(Source: Master Plan Organization, 1985).

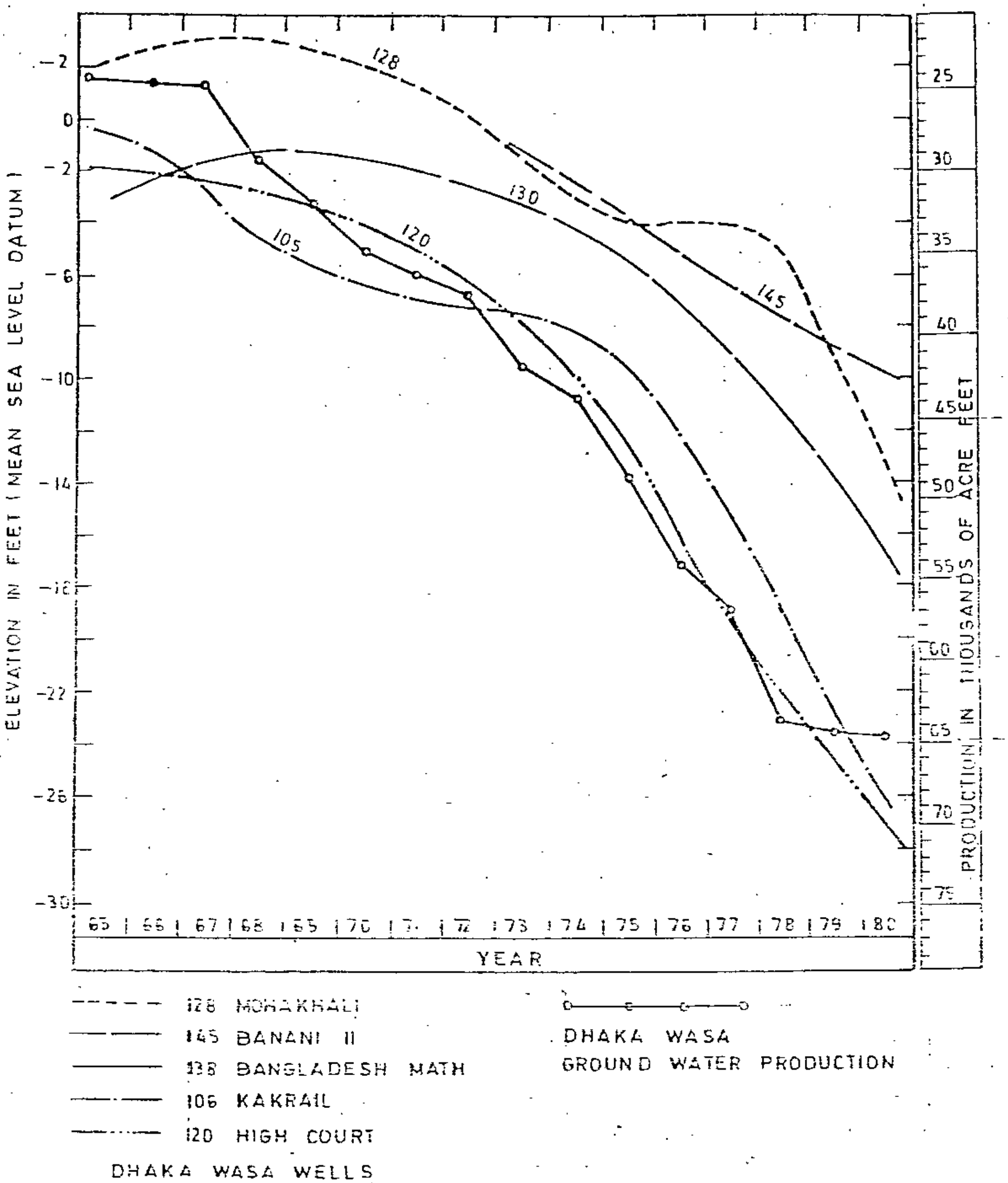

Fig. 5.3 Static groundwater level decline and Dhaka's WASA ground water production (Source: Feasibility report, Dhaka WASA, 1981).

Fig. 5.3. It may be noted that in Fig. 5.3 the production scale has been inverted for ease of comparing the variables.

Dhaka WASA began installing water wells in 1949. From 1949-1959, WASA wells were drilled and constructed to depths between 200-350 feet, using 70-100 feet sloted brass screens. The housing pipe was 10-12 inches in diameter and about 70 feet deep. Between 1959-1960 the wells were increased to 240-250 feet in depth using 100-150 feet of slotted brass screens. The housing pipes during that period increased in depth to 90-110 feet and to 14 inches in diameter. From 1969-1974, WASA wells were drilled progressively deeper, ranging generally from 330-460 feet using 170 feet of slotted stainless steel screen. The pump housing increased in depth to 120 feet and the diameter to 16 inches. From 1974 to the present days the general well depth has increased slightly and the housing pipe depth has increased to 160 feet. It is expected that the present depth of well construction of 390-490 feet uses the maximum thickness of the principal aquifer(30).

After numerous ~~tests~~ and observations, projected drawdown conditions are expected to be 80-90m, 50m and 30-35m when water will be extracted from ground water source to fulfil 85, 60 and 35 percent of the projected average daily demand for Dhaka metropolitan areas in the year 2010 respectively.

At the beginning ground water production cost was lower than that of surface water but, as regional extraction rises, ground water levels fall and ground water production cost is increasing rapidly. Moreover, a large increase in ground water extraction would cause an immediate lowering of the ground water level in the Dhaka city. Interference of wells reduce the yield of wells and greatly reduces the effectiveness of existing wells, which are not designed for extreme lifts or extreme regional drawdown. Also the numerous shallow hand pump wells currently used in periphery of Dhaka city would probably be useless. Conversely, the greater the level of surface water production, the lower is the unit cost.

Transports of ground water (distributed source) require lesser cost than a centre source (Treatment plant). However, a combination of sources is less expensive than an all ground water system in a high demand.

From the above discussion, it is clear that water in ground water source is limited and cannot fulfil the long term needs of Dhaka city; therefore it is not wise to sink all proposed DTWs, because of interference of wells become crucial and further ground water extraction and distribution cost become larger than water supply from surface water sources. Moreover there would be a problem of Environmental disaster in the city. Hence it is required to take immediate step to collect water from surface water sources.

Deep tubewells are proposed for the sake of design of the network and just to identify the design deficiencies of the existing network by the computer programme developed in this work. Instead of establishing all proposed tubewells it will be better to establish some overhead reservoirs as center of water supply system, assuming that water from surface water source will pass through these reservoirs.

Meanwhile, to compensate water demand in the city Dhaka WASA has taken a measure to establish a treatment plant at Demra to purify surface water. The project is in the final stage of approval. The route of transportation and the mode of distribution of the treated water have not yet been designed. The author assumes that for the better uniformity in distribution of surface water, most of the required treated water would pass through primary water reservoirs system and the author considers these reservoirs as sources to design the network connected by primary large diameter pipes with these reservoirs (shown in Fig. 4.2). Designed diameters and flow in pipes are shown in Table 5.1.

In the third and fourth phases of the study, 22.88 miles main pipe network has been taken in the design. It was found that in the third phase of the study, to supply additional water demand in 1990 through primary reservoirs, the diameter of 3.45 miles pipe remained unchanged and 0.72, 1.90, 0.19, 2.03, 3.34 and 7.14 miles of pipe having diameter 18 inches would have to be increased to 42, 36, 30, 27, 24 and 21 inches in diameter respectively. Among the rest of the network 2.0 miles

Table 5.1 Design of primary distribution system based on partially surface water source

Pipe joining node numbers	Pipe length ft	Existing pipe diameter inch	Phase-3 Designed diameter inches	Flow* cfs	Phase-r Designed diameter inch	Flow* cfs
1	2	3	4	5	6	7
394-407	793.0	18.0	36.0	36.23	30.0	25.54
373-394	1124.0	18.0	27.0	-18.57	24.0	-12.43
350-373	1850.0	18.0	18.0	-03.07	15.0	-1.69
300-350	2975.0	18.0	21.0	11.93	18.0	8.70
254-300	3472.0	18.0	24.0	15.93	21.0	11.48
225-254	3306.0	18.0	27.0	19.93	24.0	14.25
184-225	1653.0	18.0	21.0	-10.0	18.0	-6.54
184-337	110580.0	12.0	15.0	5.56	12.0	3.36
309-337	3140.0	18.0	4.0	0.21	4.0	0.20
262-309	4960.0	18.0	24.0	14.21	18.0	9.90
257-262	6282.0	18.0	27.0	20.21	24.0	14.06
206-257	992.0	18.0	30.0	24.21	24.0	16.83

Table 5.1 (Contd..)

1	2	3	4	5	6	7
177-206	3390.0	18.0	24.0	-13.53	18.0	-8.53
177-184	4300.0	18.0	18.0	- 3.21	15.0	-2.28
71-177	18515.0	18.0	21.0	-11.34	18.0	-7.07
71-75	5950.0	18.0	18.0	- 3.96	18.0	-3.53
75-99	6120.0	18.0	18.0	-7.96	18.0	-6.30-
99-120	5950.0	18.0	21.0	-11.96	18.0	-9.07
120-206	5950.0	18.0	24.0	-15.96	21.0	-11.84
337-407	7275.0	18.0	21.0	-11.23	18.0	- 8.22
158-225	5125.0	18.0	36.0	33.0	30.0	23.22
138-158	4130.0	18.0	36.0	36.0	30.0	24.96
138-166	3800.0	18.0	42.0	-44.0	36.0	-30.51
166-196	1325.0	18.0	21.0	10.0	18.0	6.934
71-89	7865.0	18.0	18.0	3.0	12.0	2.08

* +ve flow indicates flow from lower node number to higher nodes number.

of 12 inches pipe lines would have to be changed to 15 inches in diameter and 2.09 miles 18 inches, diameter pipe lines would require smaller diameters.

In the fourth phase of the study, to supply required water estimated in Chapter 4 through three primary reservoirs, it may be seen in Table 5.1 that about 56.8 percent of the length of network remains unchanged and the rest 0.72, 1.92, 2.22, 1.78, 1.16, 1.49 and 0.60 miles pipe diameter have to be changed to 36, 30, 24, 21, 15, 12 and 4 inches respectively. It should be mentioned that these design considerations in third and fourth phase of study have been done by assuming approximately equal quantity of water flow through each primary reservoir and node consumptions have been estimated according to the demand of areas on population basis.

Detention time of each primary water reservoir is 49 minutes and 1 hour 12 minutes in cases of the third and fourth phases respectively. Utilizing the storage capacity of the reservoirs, non-pumping period can only be maintained for 49 minutes and 1 hour 12 minutes in cases of the third and fourth phases of the study respectively. In case of the fourth phase of the study, utilizing storage capacity of all existing primary and secondary water reservoirs, nonpumping period may be increased to 2 hours 45 minutes.

CHAPTER 6

CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusion

A numerical model has been developed based on the model proposed by Shamir and Howard to analyze and design water distribution system of any type - grid iron, radial, circular dead end etc. The model has been used for the analysis and design of the water distribution system of Dhaka city.

The conclusions drawn from this study are summarized below:

i) The analysis of existing network of water distribution system of Dhaka city shows that a large number of pipes is conveying very small quantity of water relative to their pipe sizes and some pipes are conveying less than 0.1 cfs of water.

ii) The design of the water distribution system for 1990 based on ground water reveals that out of the existing 185.3 miles of pipe line, the diameter of only a few needs to be increased; 125 miles pipe would require to reduce their diameter for effective conveyance of water in 1990 and diameter of 9.48 miles pipe becomes less than 4.0 inches conveying water with flow velocity less than 2 ft/sec, which is not desirable. These pipes are mainly used as service pipes.

iii) From available data, it appears that ground water source cannot fulfil the future water supply needs of Dhaka city. It is essential to take immediate steps, to establish a surface water purification plant to meet the additional water requirements for growing population.

iv) Diameter of most primary mains is required to be increased when additional water from surface water sources is feed in the distribution system through presently available three primary reservoirs.

v) It is possible to establish 25 deep tubewells in the periphery of Dhaka city. Primary network has also been designed considering that the additional water require in 1990 would be supplemented by surface water treatment and pass through the primary reservoirs. In this case 56 percent pipes of primary network remains unchanged and the remaining pipes are required to be increased or reduced in size for effective conveyance of water.

6.2 Recommendations for Further Studies

Several avenues of additional study have been opened up by this work and they are summarized below:

i) The study have been done considering steady-state condition. The analysis can be extended for unsteady flow.

ii) The study should be extended for designing the network by keeping a constant head required for domestic water supply at different nodes.

iii) Present configuration of network may be changed and the study may be extended to design an economic water distribution system for the city of Dhaka.

iv) The study should be extended for the design of network of water distribution system, which will fulfil the water demand of greater Dhaka beyond 2000 A.D.

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APPENDIX-A

PERCAPITA WATER CONSUMPTION

Water Consumption of Typical Middle Class Household
Source: Questionnaire Survey

No. of observation	Site	Population	Total water consumption gallons/month	Per capita consumption GPCD
1	Azimpur	6	9000	50.0
2	Bijoynagar	11	13200	40.0
3	Dhanmondi	7	15540	74.0
4	Farmgate	11	9000	27.0
5	Gandaria	8	9600	40.0
6	Gulshan	5	10000	67.0
7	Kalabagan	40	27000	22.5
8	Lalmatia	17	24000	47.0
9	Mohammadpur-I	45	29465	22.0
10	Mohammadpur-II	55	39800	24.0
11	Mohammadpur-III	12	7500	21.0
12	Monipuripara			30-44
13	Noyatola	60	80000	44.5
14	Rampura	16	23550	49.0

APPENDIX-B

(Flow chart and listing of computer programme along with results are presented in volume 2)