OPTIMAL DESIGN AND SIMULATION OF WATER DISTRIBUTION NETWORKS

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Submitted in Partial Fulfillment of the Requirement
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MASTER OF ENGINEERING
IN
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(with specialization in Environmental Engineering)

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CERTIFICATE

This is to certify that the project entitled **"Optimal Design and Simulation of Water Distribution Networks"**, which is being submitted by **Vivekanand Sharma,** is a bonafide record of student's own work carried by him under my guidance and supervision in partial fulfillment of requirement for the award of the Degree of **Master of Engineering in Civil Engineering (with specialization in Environmental Engineering),** Department of Civil and Environmental Engineering, Delhi College of Engineering, Delhi, University of Delhi.

The matter embodied in this project has not been submitted for the award of any other degree.

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ABSTRACT

Progress of urbanization demands more per capita water with improved performance and reliability of water distribution networks. Developing methodologies for the minimum cost design of network has been under investigation for last few decades. Considerations to performance and reliability of the system along with capital cost considerations in the design phase draws the guidelines for the overall improved management of water distribution network thus improving serviceability of the essential infrastructural assets involving large investments. Traditionally, in optimal design of water distribution networks, the objective is focused on the minimization of capital cost of network components. Choice of various design alternatives is governed with the capital cost criteria of network. In this study network's performance and reliability evaluation approach along with the capital cost of network, has been advocated in making choice for suitable alternative. Optimization of water distribution network is done through cost-head loss ratio technique and genetic algorithms. Optimal design solutions achieved by both the techniques are followed by performance evaluation and reliability analysis of the network in various hydraulic loading conditions. Investigations are carried out for both the solutions in order to investigate their costs as well as performance and hydraulic reliability. A detailed evaluative comparison is then presented to review the cost and performance and reliability of the design alternatives, thus making choice of the suitable alternative. This shows the good overall performance of the genetic algorithm optimization over cost-head loss ratio optimization for looped water distribution network.

Chapter-1

INTRODUCTION

1. INTRODUCTION

GENERAL

In water distribution system, water is taken from source or treatment plant to the roads and streets in the city and finally to the individual houses. This function is accomplished through water distribution network.

1.1 COMPONENTS OF WATER DISTRIBUTION NETWORKS

A water distribution network generally consists of pipe lines of various commercially available sizes for carrying water to the streets, flow control valves, pressure control valves, fire hydrants and flow meters, service connections to individual homes, pumps, distribution or service reservoir etc.

1.2 LAYOUTS OF WATER DISTRIBUTION NETWORKS

Generally Water distribution networks follow the layout of road networks. In general, four different types of pipe networks, any one of which, either singly or in combination, can be used for a particular place, depending upon the local conditions and orientation of roads. These systems are

1. Dead end system
2. Grid iron (Looped) system
3. Ring system and
4. Radial system

1.3 METHODS OF DISTRIBUTION

The main objective of a distribution network is to develop adequate pressure at various points of withdrawal. Depending upon the elevation of the source of water and that of area to be served, its topography and other local conditions and considerations, three methods of distribution exist

1. Gravity feed system
2. Pumping feed system
3. Combined gravity feed and pumping system

1.4 SYSTEMS OF SUPPLY

From system of supply point of view, there are two major systems of supply

1. Continuous supply system
2. Intermittent supply system

In continuous supply system, water is supplied continuously throughout the day and in intermittent supply system, water is supplied intermittently only for the peak periods during morning and evening.

1.5 ARRANGEMENT OF WATER

From arrangement of water to the service area point of view, there are three types of water supply arrangements

1. Centralized water supply system
2. De-centralized water supply system
3. Mixed water supply system

In centralized water supply system, water to the service area is supplied through central source of supply i.e., reservoir, lake, river after adequate treatment etc. This system is adopted when there is sufficiently large source of supply to serve the demand of entire service area. This system is enriched with various technological advantages such as uniformly treated water supply to the entire area, improved supervision and management of water supply system. It also escapes the consumers from possible adverse affects through ground water contamination.

In de-centralized water supply system, service area is divided in various sub-areas and water is supplied through de-central source of supply i.e., ground water wells etc. Generally this system is adopted when no such large source of water is available near service area. It also proves to be economical over centralized system.

In India generally mixed water supply arrangements are made. Service area is divided in sub areas and water is made available from centralized source, additional requirements of water are fulfilled through local tube wells in the sub areas, filling local elevated tank. Supply from these tanks to the area is under gravity.

1.6 ZONING OF WATER DISTRIBUTION SYSTEM

Zoning in the distribution system ensures equalization of supply of water throughout the area. The zoning depends upon

1. Population density
2. Type of locality
3. Topography of area
4. Facility for isolating for assessment of waste and leak detection

If there is an average elevation difference of 15 to 25 m between zones, then each zone should be served by a separate system. The neighboring zones may be interconnected to provide emergency supplies. The valves between zones should normally be kept close and not partially opened. The layout should be such that the difference in pressure between different areas of the same zone or same system does not exceed 3 to 5 m.

1.7 REQUIREMENTS OF GOOD WATER DISTRIBUTION SYSTEM

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

1. Should be capable of supplying water at all the intended nodes with a reasonably sufficient pressure head.
2. Should be capable of supplying fire fighting water demands.
3. Should be economical, requiring least capital and operation and maintenance costs.
4. Should be simple and easy to operate and repair.
5. Should be safe against any kind of pollution of water. This is achieved by keeping water pipe lines above and away from sewerage and drainage lines.
6. Should be safe against pipe bursting.
7. Should be fairly water tight to minimize the losses due to leakages.

1.8 DESIGN OF WATER DISTRIBUTION NETWORKS

Design of water distribution system is a broad term which covers designing a water supply scheme from a large city to a small locality. Various steps to be followed depend upon extent of the engineered problem. For a gravity feed water distribution network for sufficiently large area, design is a task in which layout of network is set to ensure availability of water to all the points of requirement i.e., nodes, and the sizes of pipes i.e., links are set to ensure availability of water at adequate pressure to all the nodes.

Apart from network layout and link size designs, design of water distribution network requires sizing, location and elevation of distribution reservoir. The ideal location is the central place in the distribution system. Where the system is fed by direct pumping as well as through local reservoirs, the location of reservoirs may be kept at the tail end of the system (Floating tank).

Various steps followed in the design of water distribution network for a service area are as follows

1. Setting up the layout of water distribution network to ensure availability of water to all the points of requirement i.e., nodes. Generally pipelines are installed below pavements so pipeline layout follows the road patterns.

2. Estimating the demands at each and every node, governed by population served by that node and per capita consumption and forecasting these demands for future socio-economic growth.

3. Finalizing design demands at nodes, governed by various factors such as seasonal, daily and hourly variations of demand at nodes, along with fire demands.

4. Skeletonization of the network. A model is developed in order to represent areal demands and layouts. Skeletonization is the process of selecting for inclusion in the model only the parts of the network that have a significant impact on the behavior of the system

5. When detailed, skeletonized model of the network is available, a design solution is achieved. The sizes of pipes i.e., links are set to ensure availability of water at the withdrawal points of the pipes, with the minimum allowable pressure at the time of maximum demand. The terminal pressure in all the pipes should, therefore, remain above the minimum allowable pressure chosen for the design of distribution network.

1.9 SIMULATION OF WATER DISTRIBUTION NETWORKS

In order to meet consumer expectations and regulatory requirements, water supplies are feeling a growing need to understand better the serviceability and performance of distribution networks. Simulation is a task that helps meet these goals. It predicts the dynamic hydraulic and water quality behavior within a water distribution system operating over an extended period of time.

Simulation 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. Simulation is to be a research tool for improving our understanding of the dynamic hydraulic and water quality water within distribution systems. It can be used for many different kinds of engineering and management applications in distribution systems analysis. Sampling program design, hydraulic model calibration, chemical residual analysis, and consumer exposure assessment are some examples. Simulation can help assess alternative management strategies for improving performance and serviceability throughout a system.

1.10 PROBLEMS ENCOUNTERED IN WATER DISTRIBUTION SYSTEMS

Various problems that are generally encountered in water distribution systems are

1. Pressure at nodes is too low or too high continuously.
2. Range of variation of pressure is too high, causing high pressure at times resulting in loss of water and low pressure at times resulting in poor level of service.

3. Insufficient available quantity of water at nodes resulting in water scarcity.
4. Water hammer problems.
5. Leakages in network causing water losses
6. Contamination through sewer line and drains
7. Degraded level of service caused by deteriorations in components over time
8. Reservoir problems

Considerations in the design phase can overcome first three of them. Rests are taken care by proper operation and maintenance tasks.

1.11 PERFORMANCE OF WATER DISTRIBUTION SYSTEM

Performance of water distribution system is its ability to meet its desired objectives in variable real life situations. Generally distribution networks are designed for maximum expected flows, making it uneconomical. If some compromises are made, it turns into degraded performance of the system. Hourly variations in water demands cause short term performance degradation and yearly variations in characteristics e.g., roughness of pipes cause long term performance degradation.

Performance parameters

1. Average pressure at nodes
2. Variation of pressure at nodes
3. Flow in pipes
4. Velocity in pipes
5. Unit headloss in pipes
6. Friction factor
7. Variation in roughness over years

1.12 OPTIMIZATION OF WATER DISTRIBUTION NETWORKS

Optimization of water distribution networks is a task of selecting the sizes of pipes in such a way to ensure quantity and pressure requirements to all points of the network, at the minimal possible cost. Design plays an important role in the water supply distribution network. An economical design of water distribution network would be the aim of any agency dealing with water supply distribution. As such a cost effective design of water supply network is desirable. Several methodologies are available for the design of water distribution network. Some of them are heuristic and provide solution which may not be optimal. There are also design methodologies which take cost into consideration.

Procedure for optimization covers

1. Model formulation for the network problem.
2. An objective function is developed considering cost of network.
3. All hydraulic constraints are supposed to get satisfied.
4. An algorithm is then followed to achieve the design solution.
5. When design solution i.e., pipe diameters are achieved, they are converted to commercially available sizes.

There are several methodologies available which already take commercially available pipe sizes into consideration. While adopting such methodologies, pipe sizes obtained are directly adopted.

1.13 NEED FOR OPTIMIZATION

Water supply schemes are essential national infrastructural assets involving large investments. The economy and the cost of installing the distribution system is a very important factor. Water distribution system is the most costly item in the entire water supply scheme. It even goes up to about 70% of the total cost of the scheme. For the best utilization of resources available, optimization of such assets involving large investments is today’s need.

1.14 REASONS FOR NOT ADOPTING OPTIMIZATION PRACTICES

In spite of being capable of saving large investments, optimization practices for water distribution systems are not popular and frequently adopted among water supply design engineers community. Possible reasons for this are

1. Lake of detailed and suitable methodology
2. Lake of suitability of method of optimization for various types of network problems
3. Fear of performance degradation
4. Fear of reliability degradation
5. Lake of awareness and popularity of the techniques available

1.15 OBJECTIVES OF STUDY

Objectives of present study are

1. To study different methods for optimization of water distributions networks.

2. Performance evaluation of water distribution networks.

3. Reliability evaluation of water distribution networks.

4. Comparative evaluation of optimization alternatives in terms of capital cost along with the performance and reliability of the network.

1.16 SCOPE AND EXTENT OF STUDY

In the present study, along with capital cost, consideration of various parameters governing the criteria for selection of best suitable design alternative is carried out for a two looped, gravity feed water distribution network[1] taken from literature. Optimized design solution is achieved through two techniques namely Cost-Headloss Ratio[2] and Genetic Algorithms[3] followed by detailed

1. Capital cost comparison of alternatives

2. Network performance comparison of alternatives in various loading conditions

3. Reliability comparison of alternatives

These alternatives are compared in terms of capital cost and performance and reliability of network. Investigations carried out in this manner conclude Genetic algorithm solution to be selected over Cost-Headloss Ratio solution for its overall improved performance and reliability, for looped, gravity feed water distribution network.

Chapter-2

LITERATURE REVIEW

2. LITERATURE REVIEW

GENERAL

Shamir[4] developed a methodology for optimal design and preparation of a water distribution system which is to operate under one or several loading conditions. Alperovits and Shamir[5] presented a method called linear gradient, by which the optimal design of a water distribution system can be obtained. The system is a pipeline network, which delivers known demands from sources to consumers. Kessler & Shamir[6] used the linear programming gradient (LPG) method as an extension of the method proposed by Alperovits & Shamir (1977). Later Fujiwara & Khang[7] used a two-phase decomposition method extending that of Alperovits & Shamir (1977) to non-linear modelling. Also Eiger et al.[8] used the same formulation as Kessler & Shamir (1989). Bhave[9] has given a method of computer optimization of single source networks. The design and optimization techniques are based on equivalent pipe concepts. Sonak[10] has presented a methodology based on linear programming for obtaining the global optimum tree solution for single source looped water distribution network subjected to a single loading pattern. Maidawar, et al[11] has reported in the last few decades on optimal design of branching as well as looped water distribution system.

Recently genetic algorithms (Goldberg[12], Michalewicz[13]) have been applied in the problem of pipe network optimization. Goldberg[14], Simpson & Goldberg[15], Dandy et al., Murphy et al.[16] and Savic & Walters[17] applied both simple genetic algorithm (SGA) and improved GA, with various enhancements based on the nature of the problem, and reported promising solutions for problems from literature.

In order to analyze the reliability of water distribution system, different approaches are presently being employed by different researchers and analysts. Mays[18] computed the reliability of water distribution system by treating the demand, pressure head, and pipe roughness as random variables. He performed hydraulic simulation and computed the pressure heads at the demand nodes, provided the demands are satisfied. Finally, he computed the nodal and system hydraulic reliabilities. Chengchao Xu and Goulter Ian C[19] developed a two stage methodology for assessment of reliability of water distribution networks. Tyagi and Haan[20] developed the Generic Expectation Functions (GEF) as a function of means and coefficient of variations of input random variables. They developed GEF for different probability distributions by considering a power function and taking higher order moments of it about the origin. They used GEF to calculate the probability of failure of storm sewer design by calculating the expectations of the input random variables. Ostfeld[21] developed a tailor-made reliability methodology for the reliability assessment of regional water distribution and applied it to the regional water distribution system. The methodology comprised of two interconnected stages: the analysis of storage/conveyance properties of the system and implementation of stochastic simulation through the use of the software "US Air Force Rapid Availability Prototyping for Testing Operational Readiness" (RAPTOR). Shinstine et al.[22] applied reliability models to large-scale municipal water distribution systems based on minimum cut-set method and examined the reliability levels that engineers implicitly design into their systems. Muhammad Al-Zahrani and Juned Laiq Syed.[23] carried out hydraulic reliability analysis on water distribution system.

2.1 REVIEW OF FUNDAMENTALS

This chapter will review the fundamental concepts and principles upon which the hydraulics of pipeline system is based. We will begin with the introduction to the fundamental equations that are the foundation of Design and analysis of Water Distribution Networks and subsequently optimization in this project.

THE FUNDAMENTAL PRINCIPLES

2.1.1 THE BASIC EQUATIONS

Mass Conservation Principle

Conservation of mass is the most basic principle. In general, the fluid density ρ may vary in response to change in the fluid temperature and/or pressure. For a fixed control volume V enclosed by surface S, a general statement of mass conservation is:

$$\frac{\partial}{\partial t}\int_V \rho \, dV + \int_S \rho \; \bar{v} \cdot \bar{n} \, dS = 0 \qquad (2.1)$$

in which $\bar{v}$ is a velocity at a point and $\bar{n}$ is an outer normal unit vector to the surface S, and t is time. The first term represents the accumulation of mass over time in the control volume; for steady flows it is zero. At a surface point the dot product $\bar{v}\,.\,\bar{n}$ gives the component of velocity which crosses the surface, so the second term computes the net outflow of fluid across the entire control surface. For steady incompressible flow of a liquid in a pipe, the conservation of mass is generally referred to as the continuity principle, or simply continuity, and it is written

$$Q = \int_A v \, dA = V_1 A_1 = V_2 A_2 \qquad (2.2)$$

in which Q is the volumetric discharge through a pipe cross section, which can also be written as the product of the mean velocity V and cross-sectional area A of the pipe.

Work-Energy Principle

The second, equally important, principle is the work-energy principle, sometimes called simply the energy principle. For the steady one-dimensional flow of a liquid in a pipe, per unit weight of fluid, the principle can be written between two sections or stations as

$$\frac{V_1^2}{2g} + \frac{\rho_1}{\gamma} + z_1 = \frac{V_2^2}{2g} + \frac{\rho_2}{\gamma} + z_2 + \sum h_{L_{1-2}} - h_m \qquad (2.3)$$

In this equation $V^2/2g$ is the velocity head or kinetic energy; ρ/γ is the pressure head or flow work, and z elevation head or potential energy, all per unit weight. The head loss term, or the accumulated energy loss per unit weight, Σh_L is the sum of individual head losses in the reach caused by frictional effects between sections 1 and 2. The last term h_m is the mechanical energy per unit weight added to flow by means of hydraulic machinery. A pump adds energy to the flow so h_m is then positive and called h_p, a turbine extracts energy from the flow so h_m would then be negative and called h_t.

Linear Momentum Principle

The last of major principles is Linear Momentum Principle, which is governed by the impulse-momentum equation

$$\frac{\partial}{\partial t}\int_V \rho\overline{v}\, dV + \int_S \overline{v}\,(\rho\overline{v}\cdot\overline{n})\, dS = \vec{F}_{net} = \vec{F}_S + \vec{F}_b \qquad (2.4)$$

in which the net force on the contents of the control volume, fluid and solid, which can be divided into surface forces and body forces, is equal to the rate of accumulation of momentum within the control volume plus the net flux of momentum through the surface of control volume.

In a steady state flow first term is zero. For steady, incompressible, one-dimensional flow through a pipe, the component momentum equation along the direction of flow is

$$\overrightarrow{F_{net}} = \rho Q(\vec{V_2} - \vec{V_1}) \qquad (2.5)$$

For two dimensional flow in the x-y plane, the components of this equation are

$$\sum F_X = (\rho Q V_X)_2 - (\rho Q V_X)_1 = (\rho A V_X)_2 - (\rho A V_X)_1 \qquad (2.5\text{ a})$$

$$\sum F_Y = (\rho Q V_Y)_2 - (\rho Q V_Y)_1 = (\rho A V_Y)_2 - (\rho A V_Y)_1 \qquad (2.5\text{ b})$$

2.1.2 ENERGY AND HYDRAULIC GRADE LINES PRINCIPLES

The Energy Grade Line or EGL, also called the Energy Line or simply EL, is a plot of the sum of the three terms in the work-energy equation, which is also the Bernoulli sum

$$EL = \frac{V^2}{2g} + \frac{\rho}{\gamma} + z \qquad (2.6)$$

Each term at the right hand side presents head. First term $V^2/2g$ is velocity head, Second term ρ/γ is pressure head and the last one z is the elevation (datum) head.

The Hydraulic Grade Line or HGL, is the sum of only the pressure head and elevation heads. The sum of these two terms is also called the Piezometric Head, which can be conveniently measured by a piezometer tube inserted flush into the side of the pipe.

It is important to note here that if a tube is inserted in opposite flow direction, along the pipe, it's tip gives EGL and if the same tube is inserted flush into the side of the pipe, it's tip gives HGL. Reason is that in first case velocity head $V^2/2g$ is converted into additional pressure head, thus causing the liquid to rise to the elevation of the EGL for that point in flow.

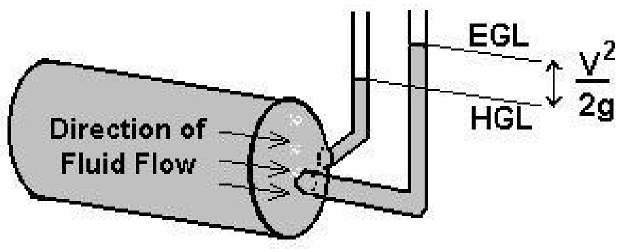


Fig. 2.1 Measurement of EGL and HGL with the help of a tube

2.2 HEAD LOSS FORMULAE

The head loss term $\sum h_{L_{1-2}}$ in equation 2.3 is responsible for representing accurately two kinds of real-fluid phenomena,

1. Head loss due to fluid shear at pipe wall, called pipe friction and
2. Additional head loss caused by local disruptions of the fluid stream

The head loss due to pipe friction is always present throughout the length of pipe. The local disruptions, called local losses, are caused by valves, pipe bends, and other such fittings. Local losses may also be called minor losses if their effect, individually and/or collectively, will not contribute significantly in the determination of the flow. Thus they are generally neglected in previous design stages.

2.2.1 DARCY-WEISBACH EQUATION

Fundamentally the most sound and versatile equation for frictional head loss in a pipe, the Darcy-Weisbach Equation is derived from general functional relation between the wall shear stress τ_W, the mean velocity V, pipe diameter D, fluid density ρ, and viscosity μ, and the equivalent sand-grain roughness e can be expressed as

$$h_f = f\frac{LV^2}{D2g} = f\frac{L}{D}\frac{Q^2}{2g\,A^2} \tag{2.7}$$

In this Eq. the friction factor f is a function of (1) pipe Reynolds number $R_e = \rho VD/\mu$ and (2) equivalent sand-grain roughness factor e/D. The functional behavior of f with R_e and e is fully displayed in Moody diagram shown below

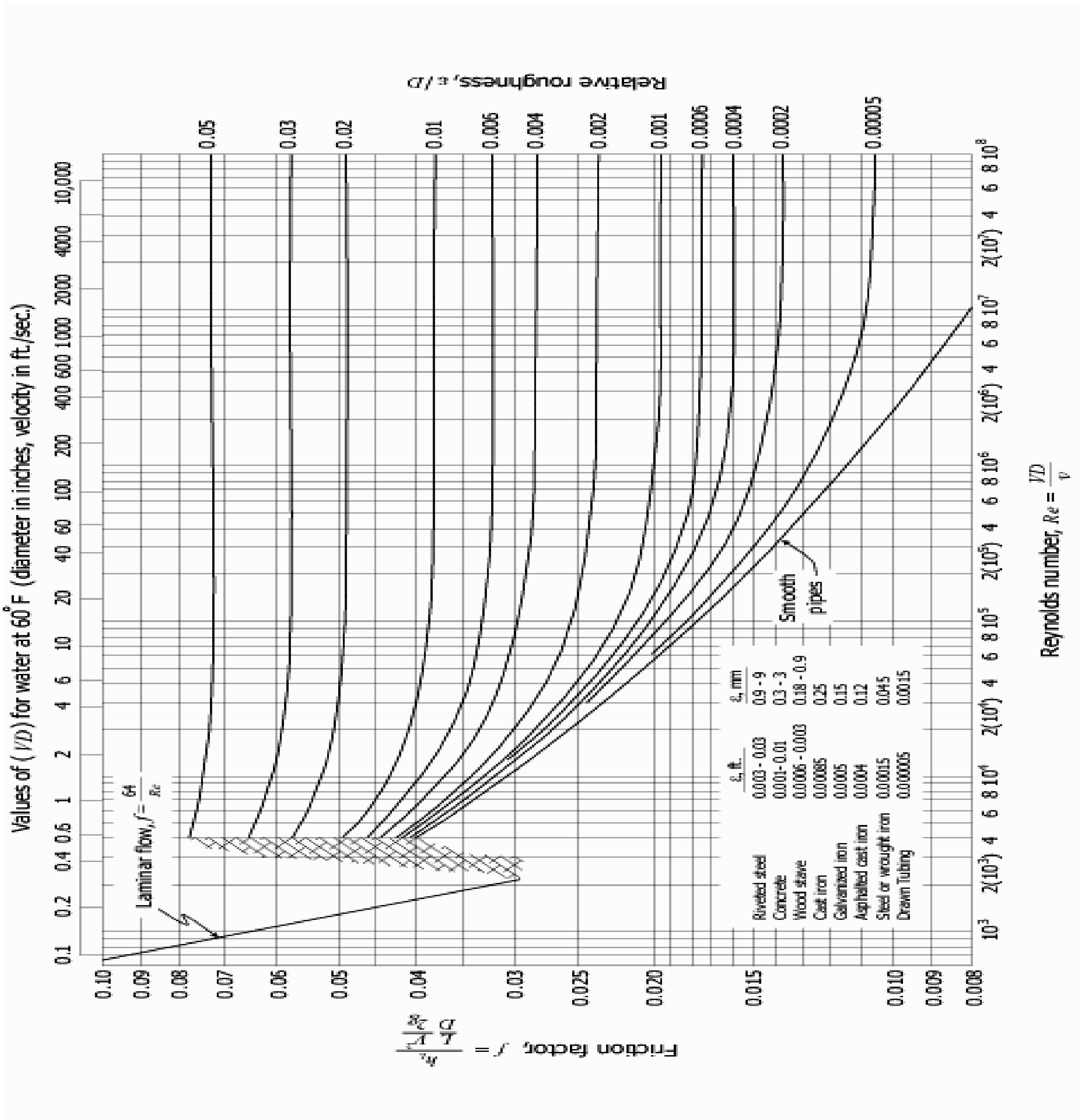


Fig. 2.2 Moody Diagram for calculation of 'f' From L. F. Moody, "Friction Factors for Pipe Flow," Trans. A.S.M.E., Vol. 66, 1944. Source[24]

In moody diagram, there are several zones that characterize different kinds of pipe flow. First we note that the plot is logarithmic along both axes.

1. Below the Reynolds number R_e= 2100 there is only one line, which can be derived solely from the laminar, viscous flow equations without experimental input; the resulting friction factor for laminar flows is $f=64/R_e$.
2. Because there is only one line in this region, we say all pipes are hydraulically smooth in laminar flow.
3. Then for Reynolds number up to about 4000 is a so-called "critical" zone in which the flow changes from laminar to weakly turbulent flow.
4. For still large Reynolds number there are three flow zones:

   (i) A dashed line borders the upper right portion of the plot. In this zone, called wholly rough flow or the region of complete turbulence for rough pipes, **f = F(e/D) and f ≠ F(R_e)**. Thus if pipe material is known, the value of f follows immediately.

   (ii) The lowest line is called the smooth-pipe line and is described by the empirical equation

$$\frac{1}{\sqrt{f}} = 2\log_{10}(R_e\sqrt{f}) - 0.8 \qquad (2.8)$$

This line continually slopes and never becomes horizontal, as in the wholly rough flow zone, so **f always depends on R_e**. Since the flow in PVC pipe is described by this line, It has become increasingly important in some fields in recent years.

(iii) Between zone (i) and zone (ii) is an important transitional band, called the turbulent transitional zone, in which **f depends on both R_e and e/D**. The Colebrook-White equation is used

$$\frac{1}{\sqrt{f}} = 1.14 - 2\log_{10}\left(\frac{e}{D} + \frac{9.35}{R_e\sqrt{f}}\right) \tag{2.9}$$

Table 2.1 shown below summarizes the D-W friction equation for various flow regions.

Table 2.1 D-W friction equation for various flow regions

Type of Flow	**Equation for f**	**Range**
Laminar	$f = 64/R_e$	$R_e < 2100$
Smooth Pipe	$\frac{1}{\sqrt{f}} = 2\log_{10}(R_e\sqrt{f}) - 0.8$	$R_e > 4000$ and $e/D \rightarrow 0$
Transitional Colebrook-White Eq.	$\frac{1}{\sqrt{f}} = 1.14 - 2\log_{10}\left(\frac{e}{D} + \frac{9.35}{R_e\sqrt{f}}\right)$	$R_e > 4000$
Wholly Rough	$\frac{1}{\sqrt{f}} = 1.14 - 2\log_{10}\left(\frac{e}{D}\right)$	$R_e > 4000$

Source[24]

For each pipe material either a single value or range of e/D values has been established. Table 2.2 presents common values for several materials.

Table 2.2 Pipe Roughnesses for various pipe materials

Material	**e, mm**
PVC, Drawn Tubing, Glass	0.0015
Commercial or Welded	0.045
Steel	0.12
Asphalted Cast Iron	0.15
Galvanized Iron	0.26
Cast Iron	0.3 - 3.0
Concrete	0.9 - 9.0
Riveted Steel	

Source[24]

2.2.2 SWAMEE-JAIN FORMULA

Much easier to solve than the iterative Colebrook-White formula, the formula developed by Indian researchers Swamee and Jain (1976) also approximates the Darcy-Weisbach friction factor. This equation is an explicit function of the Reynolds number and the relative roughness, and is accurate to within about one percent of the Colebrook-White equation over a range of

$4\times10^3 \leq R_e \leq 1\times10^8$

$1\times10^{-6} \leq \epsilon/D \leq 1\times10^{-2}$

Most importantly in majority of water distribution networks, R_e and ϵ/D fall in this range.

$$f = \frac{1.325}{\left[\ln\left(\frac{\varepsilon}{3.7D} + \frac{5.74}{R_e^{0.9}}\right)\right]^2} \qquad (2.10)$$

Because of its relative simplicity and reasonable accuracy, most water distribution system modeling analysts use the *Swamee-Jain formula* to compute the friction factor.

2.2.3 EMPIRICAL EQUATIONS

Hazen-Williams equation

Most widely used of empirical equations is the Hazen-Williams equation. To compute the discharge, the equation takes the form

$$Q = 0.849\ C_{HW}\ A\ R^{0.63}\ S^{0.54} \qquad \text{SI units} \qquad (2.11)$$

In which C_{HW} is the Hazen-Williams roughness coefficient, $S = h_f/L$ is the slope of the energy line, $R = A/P$ is the hydraulic radius, A is the cross-sectional area, and P is the wetted perimeter.

Manning Equation

Another empirical equation, which was originally and primarily developed for flow in open channels, is the Manning equation

$$Q = (1/n)\ A\ R^{2/3}\ S^{1/2} \qquad \text{SI Units} \qquad (2.12)$$

Where n is the pipe boundary roughness.
Table 2.3 shown below gives value for C_{HW} and n for some common pipe materials.

Table 2.3 Hazen-Williams and Manning Roughness

Pipe Material	C_{HW}	**Manning's n**
PVC	150	0.009
Very Smooth	140	0.010
Cement-lined Ductile Iron	140	0.012
New Cast Iron, Welded Steel	130	0.014
Wood, Concrete	120	0.016
Clay, New Riveted Steel	110	0.017
Old Cast Iron, Brick	100	0.020
Badly Corroded Cast Iron	80	0.035

Source[24]

A comparison of the Hazen-Williams and Manning equation with Darcy-Weisbach equation would show conclusively that the empirical equations are much more limited in their range of applicability. Each is applicable only to the turbulent flow of water.

2.2.4 EXPONENTIAL FORMULA

It will be advantageous to express the head loss in each pipe in a network by an exponential formula so one presentation of the theory covers all the cases, regardless the D-W equation, the H-W equation or the Manning equation is used. Head loss as a function of discharge can be written as

$$h_f = KQ^n \tag{2.13}$$

The values for K and n changes, depending upon whether the Darcy-Weisbach, Hazen-Williams or Manning equation is used.

Table 2.4 Values of K and n in exponential formula

Formula	**n**	**K**
Hazen-Williams	1.852	$\dfrac{C_K L}{C_{HW}^{1.852}\ D^{4.872}}$
Manning	2	$\dfrac{C_K n^2 L}{D^{5.33}}$

Source[24]

Table 2.5 Values of Coefficient C_K

Units of		**Hazen-Williams**	**Manning**
D	**L**	C_K	C_K
ft	ft	4.73	4.66
in	ft	8.53×10^5	2.65×10^6
m	m	10.67	10.29

Source[24]

Values of K and n for the Darcy-Weisbach equation can be approximated over a limited range on the Moody diagram by the following relations.

$$n = 2 - b$$

$$K = aL / 2gDA^2$$

Where

$$b = \ln(f_1/f_2) / \ln(Q_1/Q_2) \text{ and}$$

$$a = f_1 Q_1^{\,b} \tag{2.14}$$

f_1, f_2 are friction factors on Moody's diagram corresponding to the values of Q_1 and Q_2.

2.2.5 LOCAL AND MINOR LOSSES

A local loss is any energy loss, in addition to that of pipe friction alone, caused by some localized disruptions of the flow appurtenances, such as valves, bends and other fittings. These losses are usually computed from the equation

$h_L = K_L(V^2/2g)$ for fittings (2.15)

$h_L = K_L((V_1-V_2)^2/2g)$ for enlargements (2.16)

Table 2.6 Loss Coefficients for fittings Source[24]

Fitting	**K_L**
Globe valve, fully open	10.0
Angle valve, fully open	5.0
Butterfly valve, fully open	0.4
Gate valve, fully open 3/4 open 1/2 open 1/4 open	0.2 1.0 5.6 17.0
Check valve, swing type, fully open	2.3
Check valve, lift type, fully open	12.0
Check valve, ball type, fully open	70.0
Foot valve, fully open	15.0
Elbow, 45^0	0.4
Long radius elbow, 90^0	0.6
Medium radius elbow, 90^0	0.8
Short radius elbow, 90^0	0.9
Close return bend, 180^0	2.2
Pipe entrance, rounded, r/D < 0.16	0.1
Pipe entrance, square-edged	0.5
Pipe entrance, re-entrant	0.8

2.2.6 FLOW WITH PUMPS

The solution of water distribution network problems involving pumps normally requires date to be read from pump characteristics curves. Pump characteristic curve for head h_p can be expressed by a second-order polynomial.

$$h_p = aQ^2 + bQ + c \tag{2.17}$$

In which the coefficients a,b and c are determined by the use of three (h_p,Q) data pairs that bracket the expected range of operation of the pump. Solution is obtained by solving

$$[Q]\ [x] = [h_p] \tag{2.18}$$

where

$$[Q] = \begin{bmatrix} Q_1^2 & Q_1 & 1 \\ Q_2^2 & Q_2 & 1 \\ Q_3^2 & Q_3 & 1 \end{bmatrix}$$

$$[x] = \begin{bmatrix} a \\ b \\ c \end{bmatrix}$$

$$[h_p] = \begin{bmatrix} h_{p_1} \\ h_{p_2} \\ h_{p_3} \end{bmatrix}$$

2.3 BASIC RELATIONS BETWEEN NETWORK ELEMENTS

The two basic principles, upon which all network analysis is developed, are

1. The conservation of mass or continuity, principle, and
2. The work-energy principle, Including the Darcy-Weisbach or Hazen-Williams equation to define the relation between the head loss and the discharge in a pipe.

The equations that are developed from the continuity principle will be called Junction Continuity Equations, and those that are based on the work-energy principle will be called Energy Loop Equations. The number of these equations that constitutes a non-redundant system of equations is related directly to fundamental relations between the number of pipes, number of nodes and number of independent loops that occur in branched and looped pipe networks.

In defining these relations NP will denote the number of pipes in the network, NJ will denote the number of junctions in the network, and NL will denote the number of loops around which independent equations can be written. In defining junctions a supply *source* will not be numbered as a junction. A supply source is a point where the elevation of the energy line, or hydraulic grade line, is established; a junction, or node is a point where two or more pipes join. A node can exist at each end of a “dead end” pipe; this instance is an exception to the usual rule, where only one pipe is connected to a node. In a branched system there are by definition no loops, and thus $NL = 0$ for any branched system. In branched systems the number of nodes is always one larger than the number of pipes, or $NP = NJ - 1$ unless a reservoir is shown at the end of one pipe and this is not considered to be a junction. Then $NP = NJ$. (This situation actually occurs).

For a looped network the number of loops (around which independent energy equations can be written) is given by

$$NL = NP - NJ \tag{2.3.1}$$

If the network contains two or more supply sources, or

$$NL = NP - (NJ - 1) = NP - NJ + 1 \tag{2.3.2}$$

If the network contains fewer than two supply sources and the flow from the single source is determined by adding all of the other demands, then this source is shown as a negative demand and the source is caned a node. We note that this is the case in the small looped networks.

Equation 2.3.2 also applies to a branched system with $NL = NP - NJ + 1 = 0$, since a branched system can have at most one supply source. Actually, every pipe system must have at least one supply source, but sometimes the source is not shown since the discharge from this supply source is known, and the source is replaced by a negative demand, which is a flow coming into this junction, equal to the sum of the other demands. When this is done the elevation of the energy line (or HGL or pressure) must be specified at a node so the other HGL elevations can be determined. Energy loops that begin at one supply source and end at another are called pseudo loops, i.e. these loops do not close on themselves. The number of pseudo loops, which are numbered as art of NL, equal the number of supply sources minus one. In forming pseudo loops all Supply sources must be located at the end of a pseudo loop. It is generally possible to form more loops than are needed to produce a set of independent equations. As each new loop is formed, see that at least one pipe in the new loop is not a part of any prior loop; in this way the formation of redundant loops can usually be avoided.

2.4 EQUATION SYSTEMS FOR STEADY FLOW IN NETWORKS

Three different systems of equations can be developed for the solution of network analysis problems. These systems of equations are named after the variables that are regarded as the principal unknowns in that solution method. These systems of equations are called the

***Q*-equations** (when the discharges in the pipes of the network are the principal unknowns), the

***H*-equations** (when the HGL-elevations, also simply called the heads ***H*,** at the nodes are the principal unknowns), and the

Δ*Q*-equations (when corrective discharges, ΔQ, are the principal unknowns). Each of these three systems of equations will be studied separately.

2.4.1 SYSTEM OF *Q*-EQUATIONS

The analysis of flow in pipe networks is based on continuity and work-energy principles. To satisfy continuity, the volumetric discharge into a junction must equal the volumetric discharge from the junction. Thus at each of the *NJ* (or *NJ* - 1) junctions an equation of the form of Eq. 2.4.2 is obtained:

$$QJ_J - \sum Q_J = 0 \qquad (2.4.2)$$

In this equation *QJj* is the demand at the junction j, and each Q_i is the discharge in one of the pipes that join at junction *j*. These junction continuity equations are the first portion of the *Q*-equations. The work-energy principle provides additional equations which must be satisfied. These equations are obtained by summing head losses along both real and pseudo loops to produce independent equations. There are *NL* of these equations, and they are of the form of Eq. 2.4.4 or 2.4.5, depending upon whether the loop is a real loop or a pseudo loop, respectively, and they are the second portion of the Q-equations:

$$\sum h_P = 0 \qquad (2.4.4\ a)$$

$$\sum h_P = \Delta WS \qquad (2.4.5\ a)$$

When the head losses are expressed in terms of the exponential formula, then these equations take the forms

$$\Sigma K_i Q_i^n = 0 \qquad \text{for Real loops} \qquad (2.4.4\ b)$$

$$\Sigma K_i Q_i^n = \Delta WS \qquad \text{for Pseudo loops} \qquad (2.4.5\ b)$$

In which the summation includes the pipes that form the loop. If the direction of the flow should oppose the direction that was assumed when the energy loop equations were written, such that Q_i becomes negative, then there are two alternatives: One is to reverse the sign in front of this term, i.e., correct the direction of the flow. The second, which is generally preferred when writing a program to solve these equations, is to rewrite the equations as follows:

$$\Sigma K_i Q_i \mid Q_i \mid^{n-1} = 0 \qquad (2.4.4\ c)$$

$$\Sigma K_i Q_i \mid Q_i \mid^{n-1} = \Delta WS \qquad (2.4.5\ c)$$

2.4.2 SYSTEM OF *H*-EQUATIONS

If the elevation of the energy line or hydraulic grade line throughout a network is initially regarded as the primary set of unknown variables, then we develop and solve a system of *H*-equations. One H-equation is written at each junction (or at $NJ - 1$ junctions if fewer than two supply sources exist). Since looped pipe networks have fewer junctions than pipes, there will be fewer *H*-equations than *Q*-equations. Every equation in this smaller set is nonlinear, however, whereas the Junction continuity equations are linear in the system of *Q*-equations.

To develop the system of *H*-equations, it is begin by solving the exponential equation for the discharge in the form

$$Q_{ij} = (h_{f\,ij}/K_{ij})^{1/n_{ij}} = [(H_i - H_j)/K_{ij}]^{1/n_{ij}} \qquad (2.4.11\ a)$$

Here the frictional head loss has been replaced by the difference in HGL values between the upstream and downstream nodes. In addition, in this equation a double subscript notation has been introduced; the first subscript defines the upstream node of the pipe, and the second subscript defines the downstream node. Thus *Qij* and *Kij* denote the discharge and loss coefficient for the pipe from node *i* to node *j*.

An alternative way of writing Eq. 2.4.11(a) is

$$Q_k = (h_{f\,k}/K_k)^{1/n_k} = [(H_i - H_j)/K_k]^{1/n_k} \qquad (2.4.11\ b)$$

in which k is the pipe number.

Substituting Eq. 2.4.11 into the junction continuity equations, Eq. 2.4.2, yields

$$QJ_j - \Sigma\{[(H_i - H_j)/K_{ij}]^{1/n_{ij}}\}_{in} + QJ_j - \Sigma\{[(H_j - H_i)/K_{ij}]^{1/n_{ij}}\}_{out} = 0 \qquad (2.4.12\ a)$$

in which the summations are over all pipes that flow to and *from* junction *j*, respectively. If it is desired to automate the choice of sign, then Eq. 2.4.12 a can be written as

$$QJ_i - \Sigma\{[(H_i - H_j)/K_{ij}]\,|(H_i - H_j)/K_{ij}|^{1/n_{ij}-1}\}_{in}$$

$$+\ \Sigma\{[(H_j - H_i)/K_{ij}]\,|(H_j - H_i)/K_{ij}|^{1/n_{ij}-1}\}_{out} = 0 \qquad (2.4.12\ b)$$

2.4.3 SYSTEM OF ΔQ-EQUATIONS

The number of ΔQ-equations is normally about half the number of H-equations for a network. This reduction in number is not necessarily an advantage, since all of the equations are nonlinear and may contain many terms. These equations consider the loop corrective discharges or $\Delta Q's$ as the primary unknowns. These corrective discharges or ΔQ's will be determined from the energy equations that are written for NL loops in the network, and thus NL of these corrective discharge equations must be developed. To obtain these equations, discharge is replaced in each pipe of the network by an initial discharge, denoted by Qoi, plus the sum of all of the initially unknown corrective discharges that circulate though pipe i, or

$$Qi = Qoi + \Sigma \Delta Q_k \qquad (2.4.15)$$

In which, the summation includes all of the corrective discharges passing through pipe i. The initial discharges Qoi must satisfy all of the junction continuity equations. It is not difficult to establish the initial discharge in each pipe so that the junction continuity equations are satisfied. However, these initial discharges usually will not satisfy the energy equations that are written around the loops of the network.

Equation 2.4.15 is based on the fact that any adjustment can be added (accounting for sign) to the initially assumed flow in each pipe in a loop of the network without violating continuity at the junctions. It is very important to understand the validity of this decomposition; it may help to note that any ΔQ entering a junction as it flows around a loop must also leave that junction, and vice versa. Because of this fact, it is decided to establish NL energy loop equations around the NL loops of the network, in which each initial discharge plus the sum of corrective loop discharges $\Sigma \Delta Qk$ is used as the discharge. The junction continuity equations are satisfied by the initial discharges Qoi and are not a part of the system of equations. The corrective discharges can be chosen as positive if they circulate around a loop in either the clockwise or counterclockwise direction. It is necessary to be consistent within anyone loop, but the sign convention may change from loop to loop, if desired. A corrective discharge adds to the flow *Qoi* in pipe *i* if it is in the same direction as the pipe flow, and it subtracts fro the initial discharge if it IS m the opposite direction.

To summarize how the Q-equations are obtained, replace the *Q's* in the energy loop equations, Eqs. 2.4.4 and 2.4.5, by

$$Qi = Qoi \pm \Sigma\Delta Q_k \qquad (2.4.16)$$

Here the summation includes all corrective discharges which pass through pipe i, and the plus sign is used if the net corrective discharge and pipe flow are in the same direction; otherwise the minus sign is used before the summation. Thus Eqs. 2.4.4 and 2.4.5 become

$$\Sigma K_i \{Q_{oi} \pm \Sigma\Delta Q_k\}^{n_i} = 0 \qquad \text{For real loops} \qquad (2.4.17\text{ a})$$

and

$$\Sigma K_i \{Q_{oi} \pm \Sigma\Delta Q_k\}^{n_i} = \Delta WS \qquad \text{for pseudo loops} \qquad (2.4.18\text{ a})$$

To automate the choice of sign, these equations can be rewritten as

$$\Sigma K_i \{Q_{oi} \pm \Sigma\Delta Q_k\} \mid Q_{oi} \pm \Sigma\Delta Q_k \mid^{n_i - 1} = 0 \qquad \text{for real loops} \qquad (2.4.17\text{ b})$$

and

$$\Sigma K_i \{Q_{oi} \pm \Sigma\Delta Q_k\} \mid Q_{oi} \pm \Sigma\Delta Q_k \mid^{n_i - 1} = \Delta WS \qquad \text{for pseudo loops} \qquad (2.4.18\text{ b})$$

2.5 SOLVING THE NETWORK EQUATIONS - Newton method for large systems of equations

In previous sections, writing of systems of algebraic equations to describe the relations between the discharges, pressures, and other variables and parameters in a pipe network is explored. Many of the equations in these systems of equations are nonlinear. A good method for solving nonlinear equations is therefore needed. Numerous methods exist, but the Newton Method is the method of choice here. Its application to the solution of the Q-equations, the H-equations and the ΔQ-equations will be discussed- in this section. To treat the unknown discharges (when using the Q-equations), the unknown heads (when using the H-equations), and the unknown corrective loop discharges (when using the ΔQ equations) in a uniform way, the primary unknown variable in this section will be called the vector $\{x\}$.

The Newton iterative formula for solving a system of equations can be written as

$$\{x\}^{(m+1)} = \{x\}^{(m)} + [D]^{-1}\{F\}^{(m)} \qquad (2.5.32\ \text{a})$$

Here x is an entire column vector $\{x\}$ of unknowns, $\{F\}$ is an entire column vector of equations, and $[\boldsymbol{D}]^{-1}$ is the inverse of a matrix $[\boldsymbol{D}]$ which is the Jacobian. The Jacobian occurs in several applications in mathematics, and it represents the following matrix of derivatives:

$$[\boldsymbol{D}] = \begin{bmatrix} \frac{\partial \boldsymbol{F}_1}{\partial \boldsymbol{x}_1} & \frac{\partial \boldsymbol{F}_1}{\partial \boldsymbol{x}_2} & . & . & \frac{\partial \boldsymbol{F}_1}{\partial \boldsymbol{x}_n} \\ \frac{\partial \boldsymbol{F}_2}{\partial \boldsymbol{x}_1} & \frac{\partial \boldsymbol{F}_2}{\partial \boldsymbol{x}_2} & . & . & \frac{\partial \boldsymbol{F}_2}{\partial \boldsymbol{x}_n} \\ . & . & . & & . \\ . & . & & . & . \\ \frac{\partial \boldsymbol{F}_n}{\partial \boldsymbol{x}_1} & \frac{\partial \boldsymbol{F}_n}{\partial \boldsymbol{x}_2} & . & . & \frac{\partial \boldsymbol{F}_n}{\partial \boldsymbol{x}_n} \end{bmatrix} \qquad (2.5.34)$$

Likewise $\{x\}$ and $\{\boldsymbol{F}\}$ are actually

$$\{\boldsymbol{x}\} = \begin{Bmatrix} \boldsymbol{x}_1 \\ \boldsymbol{x}_2 \\ \cdot \\ \cdot \\ \boldsymbol{x}_n \end{Bmatrix} \qquad\qquad \{\boldsymbol{F}\} = \begin{Bmatrix} \boldsymbol{F}_1 \\ \boldsymbol{F}_2 \\ \cdot \\ \cdot \\ \boldsymbol{F}_n \end{Bmatrix} \qquad (2.5.34)$$

Equation 2.5.32a indicates that the Newton method solves a system of nonlinear equations by iteratively solving a system of linear equations because $[\boldsymbol{D}]^{-1}$ $\{\boldsymbol{F}\}$ represents the solution of the linear system of equations

$$[\boldsymbol{D}]\ \{\boldsymbol{z}\} = (\boldsymbol{F}) \qquad (2.5.32b)$$

That is, the vector that is subtracted from the current estimate of the unknown vector $\{x\}$ in Eq. 2.5.32a is the solution $\{z\}$ to the linear system of equations that is Eq. 2.5.32b. In practice we therefore see that the Newton method solves a system of equations by the iterative formula

$$\{\boldsymbol{x}\}^{(m\ +\ 1)} = \{\boldsymbol{x}\}^{(m)} - \{\boldsymbol{z}\} \qquad (2.5.32c)$$

where $\{z\}$ is the solution vector that is obtained by solving $[\boldsymbol{D}]\{\boldsymbol{z}\} = \{\boldsymbol{F}\}$. If the system should actually contain only linear equations, then the first iteration will produce the exact solution..

The development of Eq. 2.5.32 follows. We begin by using a multi-dimensional Taylor series expansion to evaluate the individual equations Fi in the neighborhood of an initial solution estimate that we call $\{\boldsymbol{x}\}$ which is presumed to be near the actual solution:

$$
\begin{aligned}
F_1^{(m+1)} &= F_1^{(m)} + \frac{\partial F_1}{\partial x_1}\Delta x_1 + \frac{\partial F_1}{\partial x_2}\Delta x_2 + \cdot\ \cdot + \ \frac{\partial F_1}{\partial x_n}\Delta x_n + O(\Delta x^2) = 0 \\
F_2^{(m+1)} &= F_2^{(m)} + \frac{\partial F_2}{\partial x_1}\Delta x_1 + \frac{\partial F_2}{\partial x_2}\Delta x_2 + \cdot\ \cdot + \ \frac{\partial F_2}{\partial x_n}\Delta x_n + O(\Delta x^2) = 0 \\
\cdot \quad & \quad \cdot \quad \cdot \quad \cdot \quad \cdot \quad \cdot \quad \cdot \\
\cdot \quad & \quad \cdot \quad \cdot \quad \cdot \quad \cdot \quad \cdot \quad \cdot \\
F_n^{(m+1)} &= F_n^{(m)} + \frac{\partial F_n}{\partial x_1}\Delta x_1 + \frac{\partial F_n}{\partial x_n}\Delta x_2 + \cdot\ \cdot + \ \frac{\partial F_n}{\partial x_n}\Delta x_n + O(\Delta x^2) = 0
\end{aligned}
\qquad (2.5.35)
$$

When we use matrix notation and make the substitution $\Delta x_i = x_i^{(m+1)} - x_i^{(m)}$, this system of equations becomes

$$
\begin{Bmatrix} F_1 \\ F_2 \\ \cdot \\ \cdot \\ F_n \end{Bmatrix}^{(m)} + \begin{bmatrix} \frac{\partial F_1}{\partial x_1} & \frac{\partial F_1}{\partial x_2} & \cdot & \cdot & \frac{\partial F_1}{\partial x_n} \\ \frac{\partial F_2}{\partial x_1} & \frac{\partial F_2}{\partial x_2} & \cdot & \cdot & \frac{\partial F_2}{\partial x_n} \\ \cdot & \cdot & \cdot & & \cdot \\ \cdot & \cdot & & \cdot & \cdot \\ \frac{\partial F_n}{\partial x_1} & \frac{\partial F_n}{\partial x_2} & \cdot & \cdot & \frac{\partial F_n}{\partial x_n} \end{bmatrix} \begin{Bmatrix} x_1^{(m+1)} - x_1^{(m)} \\ x_2^{(m+1)} - x_x^{(m)} \\ \cdot \\ \cdot \\ x_n^{(m+1)} - x_n^{(m)} \end{Bmatrix} = 0 \qquad (2.5.36)
$$

which can be written compactly as

$\{F\}(m) + [\boldsymbol{D}]^{(m)}(\{\boldsymbol{x}\}^{(m+1)} - \{\boldsymbol{x}\}^{(m)} = \{\boldsymbol{0}\}$

and solved for $(\boldsymbol{x}\}^{(m+1)}$ to produce Eq. 2.5.32a.

Chapter-3

OPTIMIZATION OF WATER DISTRIBUTION NETWORKS

3. OPTIMIZATION OF WATER DISTRIBUTION NETWORKS

GENERAL

Optimization, as it applies to water distribution system modeling, is the process of finding the best, or optimal, solution to a water distribution system problem.

Use of the model for design applications follows the process shown in Fig 3.1 (formulate alternatives, test alternatives, cost analysis, and make decision). Before this process can begin, however, a descriptive network model must be developed, or simulation, that predicts the behavior of the system under various conditions.

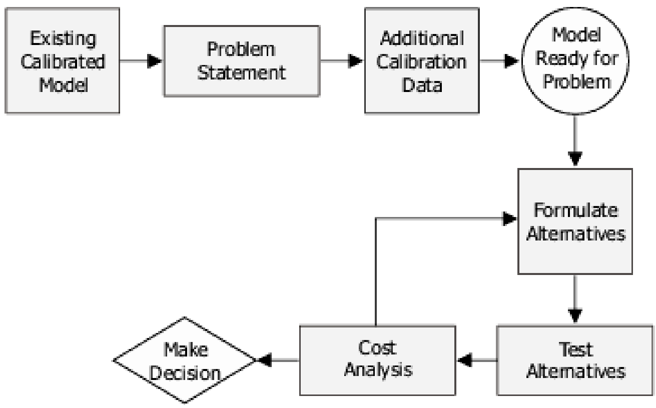


Fig 3.1 Overview of model application

The idea of rerunning a simulation model under all possible conditions to determine the most favorable alternatives is simple. However, the number of alternatives quickly becomes enormous, and the associated time and cost can become prohibitive. Optimization techniques provide a way to efficiently examine a broad range of alternative solutions by automatically altering the details of the system to generate new, improved solutions. A decision-making model that uses these automated techniques is called an optimization model.

3.1 OPTIMIZATION TERMINOLOGY

3.1.1 OBJECTIVE FUNCTION

When optimizing a system or process, it is important to quantify how good a particular solution is. A mathematical function called an objective function is used to measure system performance and indicate the degree to which the objectives are achieved. If multiple objectives exist, then there will be multiple objective functions.

The term optimization refers to mathematical techniques used to automatically adjust the details of the system in such a way as to achieve, for instance, the best possible system performance (that is, the best value of the objective functions), or the least-cost design that achieves a specified performance level. The best or most advantageous solution (or solutions in multiobjective analysis) is called the optimal solution.

3.1.2 DECISION VARIABLES

In order to improve the performance of a system, the parameters that can be changed must be known. These quantifiable parameters are called decision variables, and their respective values are to be determined. In WDN pipe-size optimization, the decision variables are the diameter for each of the pipes being considered. Any restrictions on the values that can be assigned to decision variables should be clearly stated in the optimization model. In the case of pipe sizes, each discrete pipe size available should be defined.

3.1.3 CONSTRAINTS

When judging systems and solutions, it is necessary to consider the limits or restrictions within which the system must operate. These limits are called constraints. If one's objective is to attain a minimum-cost solution, one must also consider the constraints on system performance and reliability. Constraints serve to define the decision space from which the objective function can take its values. The *decision space* is the set of all possible decision variables, and the *solution space* is the set of all possible solutions to the problem.

Constraints may be further classified as hard constraints, which may not be exceeded without failure or severe damage to the system, and soft constraints, which may be exceeded to a certain extent, although it is generally not desirable to do so. An example of a hard constraint in WDN is the maximum pressure that a pipe can withstand without jeopardizing the structural integrity of the system. A minimum pressure requirement for all water system nodes and a maximum permissible velocity for system pipes are possible soft constraints. Constraints may be applied explicitly to decision variables (for example, pipe sizes are discrete) or implicitly to other system parameters (for example, net head loss around a loop must be zero).

In evaluating systems and possible solutions, it is also important to understand how the various constraints interact. The limits for a given constraint often prevent one from obtaining a better value in another. For instance, a larger pipe size may aid in meeting required fire flows, but at the same time may be detrimental to water quality.

3.2 THE OPTIMAL DESIGN PROCESS

Figure 3.2 shows the basic steps in creating (formulating) an optimization model.

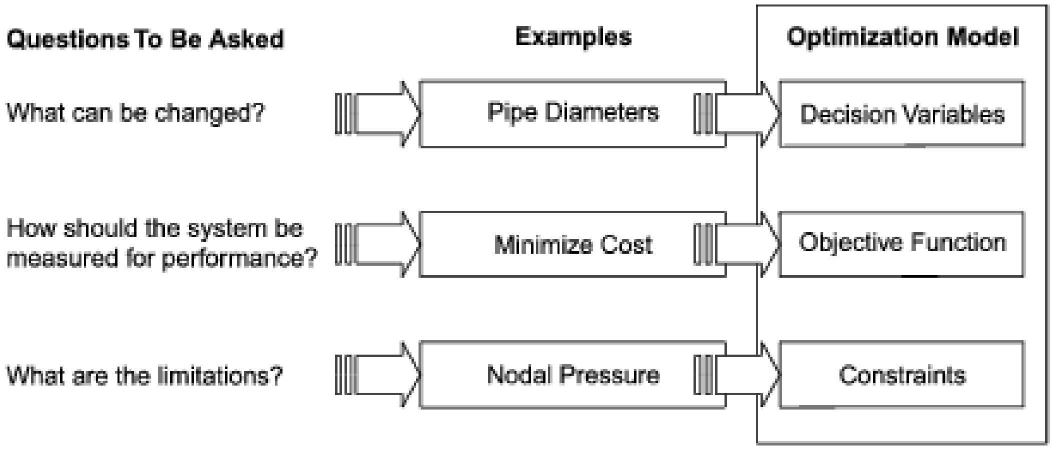


Fig 3.2 Optimization model formulation process

Based on Fig 3.2, it can be said that optimization involves the selection of a set of decision variables to describe the decision alternatives.

1. The selections of an objective or several objectives, expressed in terms of the decision variables that one seeks to optimize (that is, minimize or maximize).
2. The determination of a set of constraints (both hard and soft), expressed in terms of the decision variables that must be satisfied by any acceptable (feasible) solution.
3. The determination of a set of values for the decision variables so as to minimize (or maximize) the objective function, while satisfying all constraints.

3.3 COST HEAD LOSS RATIO METHOD FOR OPTIMIZATION

The cost head loss ratio method is based on continuous diameter approach. For minimum cost design of distribution network, a criterion termed as cost head loss ratio criterion must be satisfied at all nodes other than source nodes and critical nodes. The critical nodes are those nodes in which available hydraulic gradient level (HGL) is equal to the minimum required HGL.

The demand nodes are labeled 1, . . , Z, and pipes are labeled as 1, . . . , . . . z, where '0' is the source node. For any node j in a network

$$\sum_{\substack{\text{for all} \\ \text{supply link incidents} \\ \text{at node } j}} \left(\frac{mC}{h}\right)_{ij} = \sum_{\substack{\text{for all} \\ \text{distribution links from} \\ \text{the node } j}} \left(\frac{mC}{h}\right)_{jk} \qquad (3.1)$$

where m is the exponent of diameter in a typical cost diameter relationship, where cost and diameter are related as $C = kd^m$; C, cost of the link; h, head loss in link, where each link is the section of pipe between successive nodes; d, diameter of the pipe in each link; and ij and jk are supply and distribution link incident at node j. An iterative methodology is followed to satisfy the optimality criterion at all nodes. In the iterative methodology, HGL values are assumed initially at all demand nodes using critical path method. Using these HGL values, it is checked whether the cost head loss ratio criterion is satisfied at all nodes. If it is not satisfied, a correction is applied to the HGL values at all nodes. The correction term for all demand nodes is given by

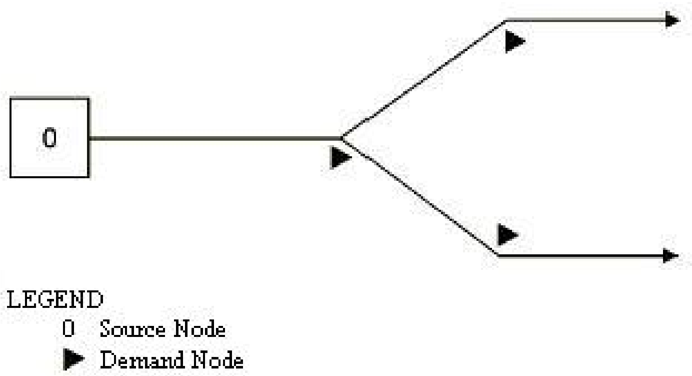


Fig. 3.3 Simple Branching System

$$\Delta Hj = -\frac{\sum(mC/h)_{ij}+\sum(mC/h)_{jk}}{\sum(mC/h^2)_{ij}+\sum(mC/h^2)_{jk}} \quad (3.2)$$

The detailed derivation of this expression is included later. The iterative procedure is terminated when

(i) HGL correction values become insignificantly small; or

(ii) When successive reduction in cost of the network become negligible.

3.3.1 CRITICAL PATH METHOD

The critical path method is not an optimization technique and is applied in the optimal design method discussed earlier to minimize the iterations. The HGL values obtained using this method serve as initial values of HGL to be input into the cost head loss ratio method. This method is devised here. The basic requirement of the system is that flow should take place in all parts of the system satisfying minimum head requirement and flow at each demand node, for the given HGL value at the source. The method designs a distribution network by providing uniform slope for all pipes on the critical path.

`

Consider a simple branching system 0-1-2-3 as shown in Figure 3.3 above. Available friction slope S_1 for path 0-1-2 is given by S_1 =

$(H_0 - H_2)/(L_1 + L_2)$ and HGL at the junction point '1' is $H_{11} = H_0 - S_1L_1$. Similarly, S_2 and $H_{12} = H_0 - S_2L_2$, are the slope and corresponding HGL at junction '1' for path 0-1-3. If S_1 and S_2 are equal, $H_{11} = H_{12}$ and both H_{11} and H_{12} will be greater than H_2 and H_3. Thus, the flow will take place in the entire system 0-1-2-3. However, in most cases S_1 and S_2 are unequal. In such a case let S_1 be smaller than S_2. It can be easily proved that H_{11} will always be greater than both H_2 and H_3 , while H_{12} may not necessarily be greater than H_2. From this it can be said that, if a distribution system is designed so that friction slopes are equal for the path having the least friction slopes, flow will take place in all parts of the system. A path having least friction slope is termed critical path and the corresponding least friction slope is termed as critical slope. Distribution node at the end of the critical path is termed critical node. Thus, the system should be designed so that various sections of the critical path will have equal slopes.

3.3.2 OPTIMIZATION MODEL DESCRIPTION

Headloss and Cost Functions

The general **head loss** relationship for a link can be expressed as

$$H_L = \frac{K_1 L Q^P}{D^r} \tag{3.1}$$

where H_L is headloss through a link; L, length of pipe; D, diameter of pipe; Q, discharge for the link; K_1, coefficient that depends upon the pipe material and diameter type of flow, and the units of other terms; p, r, exponents. The **capital cost** of the link can be expressed as

$$C = KLD^m \tag{3.2}$$

where C is the capital cost of a link.

Combining equations (3.1) and (3.2) gives

$$C = KK_1^{m/r}\ L^{1+m/r}\ Q^{pm/r}\ H_L^{-m/r} \tag{3.3}$$

Generally *pm/r* is almost always less than 1.

3.3.3 OPTIMIZATION MODEL FORMULATION

Consider a water distribution system consisting of one source node (labeled 0), Y demand nodes (demand can be zero, nodes labeled 1 to Y), X links (labeled 1 to X) and Z non-overlapping loops (labeled 1 to Z). The network cost optimization can be mathematically stated as follows.

The **objective function** is

Minimize

$$C_{TN} = \sum_{x=1}^{x} C_x \tag{3.4}$$

$$= \sum_{x=1}^{x} \left(KK_1^{m/r}\ L^{1+m/r}\ Q^{pm/r}\ H_L^{-m/r} \right)_x \tag{3.5}$$

in which C_{TN} denotes the cost of the network, and suffix x denotes a link x. The **constraints** are

1. The flow continuity constraints

Source node:

$$q_{0\,in} = \sum Q_{0\,out} + q_0 \tag{3.6}$$

Demand nodes:

$$\sum Q_{j\,in} = \sum Q_{j\,out} + q_j \tag{3.7}$$

j = 1,…,…,…Y

Where $q_{0\ in}$ is inflow at the source, $Q_{0\ out}$ is outflow along the links at the source.
$q_0 \geq 0$ demand at the source

$q_{j\ in}$ and $q_{j\ out}$ = inflow and out flow along the links at the j_{th} node.
q_j = demand at the j_{th} node

2. The loop head loss constraints

$$\sum_{loop\ z-Lx} H = \sum_{loop\ z} \left[\frac{K_1 L Q^P}{D^r} \right]_x = 0 \tag{3.8}$$

z = 1 …, …, …,Z

3. The node HGL constraints

Source $H_0 \geq H_0^{nat}$ (3.9)

Demand nodes: $H_j^{max} \geq H_j^{act} \geq H_j^{min}$ (3.10)

J=1 …, …, …, Y

4. The link availability constraints

D_x = one or any combination from

di; i=1 …, …, …, a *x=1 …, …, …,X* (3.11)

3.3.4 OPTIMIZATION THEORY

The equation (3.5) can be written as

$$C_{TN} = \sum_{x=1}^{x} C_x \sum_{x=1}^{x} \left(A H_L^{-m/r} \right)_x \tag{3.12}$$

In which

$$A = KK_1^{m/r} \ L^{1+m/r} \ Q^{pm/r} \tag{3.13}$$

3.3.5 OPTIMALITY CRITERIA

For a distribution network, the cost function, equation (3.12) can be shown to be a unimodal continuous convex function of H_L and thus of H_j. Therefore, suppressing initially the node HGL constraints, equation (3.10) for minimum CTN

$$\frac{\partial C_{TN}}{\partial H} = 0; \qquad j = 1 \ldots, \ldots, \ldots, Y_s \tag{3.14}$$

let

$ij,\ i = 1 \ldots, \ldots, \ldots, \ldots M$ be the supply links

and

$jk,\ k = 1 \ldots, \ldots, \ldots, \ldots N$ be the distribution links

For the node j, from equations (3.12) and (3.14)

$$\frac{\partial}{\partial H_j} \left[\sum_{i=1}^{M} C_{ij} + \sum_{k=1}^{N} C_{jk} \right] = 0$$

or

$$\frac{\partial}{\partial H_j}\left[\sum_{i=1}^{M} A_{ij}\left(H_i - H_j\right)^{-mij/r} + \sum_{k=1}^{N} A_{jk}\left(H_j - H_k\right)^{-mjk/r}\right] = 0 \qquad (3.15)$$

Differentiating with respect to H_j

$$\sum_{i=1}^{M} A_{ij}\left(-m_{ij}/r\right)\left(H_i - H_j\right)^{-(mij/r)^{-1}} + \sum_{k=1}^{N} A_{jk}\left(-m_{jk}/r\right)\left(H_j - H_k\right)^{-(mij/r)^{-1}} = 0$$

$$\therefore \sum_{i=1}^{M} \frac{\left(m_{ij}/r\right)A_{ij}\left(H_i - H_j\right)^{-mij/r}}{\left(H_i - H_j\right)} = \sum_{k=1}^{N} \frac{\left(m_{jk}/r\right)A_{jk}\left(H_j - H_k\right)^{-mjk/r}}{\left(H_j - H_k\right)}$$

Canceling r from the denominator in the numerator, and using equation (3.12), we get equation (3.17), and equation (3.18), which gives the cost-headloss-ratio criterion that should hold good for optimality.

$$\sum_{i=1}^{M} \frac{m_{ij}C_{ij}}{H_i - H_j} = \sum_{k=1}^{N} \frac{m_{jk}C_{jk}}{H_j - H_k} \qquad (3.16)$$

$$\sum_{i=1}^{M} \left(\frac{mC}{H_L}\right)_{ij} = \sum_{i=1}^{N} \left(\frac{mC}{H_L}\right)_{jk} \qquad (3.17)$$

Thus, for the entire network, from equation (3.14)

$$\sum\frac{m_{ij}C_{ij}}{H_i-H_j}=\sum\frac{m_{jk}C_{jk}}{H_j-H_k};\quad j=1\ldots,\ldots,\ldots,Y \tag{3.18}$$

$$\sum\left(\frac{mC}{H_L}\right)_{ij}=\sum\left(\frac{mC}{H_L}\right)_{jk};\qquad j=1\ldots,\ldots,\ldots,Y \tag{3.19}$$

Corrections to Node HGL Values

For corrections to the assumed node HGL values, equation (3.20)

$$\therefore\sum\left[\frac{m_{ij}C_{ij}}{\left(H_i-H_j\right)-\Delta H_j}\right]=\sum\left[\frac{m_{jk}C_{jk}}{\left(H_j-H_k\right)-\Delta H_j}\right] \tag{3.20}$$

The equation can be written as

$$\sum\left[\frac{m_{ij}C_{ij}}{\left(H_i-H_j\right)-\Delta H_j}\times\frac{\left(H_i-H_j\right)+\Delta H_j}{\left(H_i-H_j\right)+\Delta H_j}\right]$$

$$\sum\left[\frac{m_{jk}C_{jk}}{\left(H_j-H_k\right)+\Delta H_j}\times\frac{\left(H_j-H_k\right)-\Delta H_j}{\left(H_j-H_k\right)-\Delta H_j}\right]$$

and hence

$$\therefore\sum\frac{m_{ij}C_{ij}\left(H_i-H_j\right)}{\left(H_i-H_j\right)^2-\left(\Delta H_j\right)^2}+\sum\frac{m_{ij}C_{ij}\left(\Delta H_j\right)}{\left(H_i-H_j\right)^2-\left(\Delta H_j\right)^2}$$
$$=\sum\frac{m_{jk}C_{jk}\left(H_j-H_k\right)}{\left(H_j-H_k\right)^2-\left(\Delta H_j\right)^2}-\sum\frac{m_{jk}C_{jk}\left(\Delta H_j\right)}{\left(H_j-H_k\right)^2-\left(\Delta H_j\right)^2}$$

In the vicinity of optimal solution, ΔHj is small. Therefore, $(\Delta Hj)^2$ is further smaller, hence, negligible. Discarding it from the denominators,

$$\sum\left(\frac{mCH_L}{(H_L)^2}\right)_{ij}+\sum\left(\frac{mC\Delta H_j}{(H_L)^2}\right)_{ij}$$

$$=\sum\left(\frac{mCH_L}{(H_L)^2}\right)_{jk}-\sum\left(\frac{mC\Delta H_j}{(H_L)^2}\right)_{jk}$$

and hence

$$\therefore \Delta H_j\left[\left(\frac{mC}{(H_L)^2}\right)_{ij}+\left(\frac{mC}{(H_L)^2}\right)_{jk}\right]$$

$$=-\sum\left(\frac{mC}{H_L}\right)_{ij}+\sum\left(\frac{mC}{H_L}\right)_{jk} \qquad (3.21)$$

which gives equation (3.21).

Alternately the equation marked * can also be written as

$$\sum m_{ij}C_{ij}\left[\left(H_i-H_j\right)-\Delta H_j\right]^{-1}+\sum m_{jk}C_{jk}\left[\left(H_j-H_k\right)-\Delta H_j\right]^{-1}$$

Expanding this equation in Taylor's series, and neglecting second and higher order terms in ΔH_j and rearranging terms, equation (3.21) can be obtained.

3.3.6 COST FUNCTION MODEL

Pipe diameters against cost per m are given above. In order to calculate the total cost of the network a general relationship is used and it becomes,

$C = kLD^m$

where C, is capital cost of link; d, diameter of pipe, mm; m, exponent taken from graph; and k, link constant, it is also taken from graph.

3.4 GENETIC ALGORITHMS FOR OPTIMIZATION

Over many generations, natural populations evolve according to the principles first stated clearly by Charles Darwin. The main principles are those of preferential survival and reproduction of the fittest members of the population. Additional principles that characterize natural systems are the maintenance of a population with diverse members, the inheritance of genetic information from parents, and the occasional mutation of genes. Evolutionary Programs (EPs) are general artificial-evolution search methods based on natural selection and the aforementioned mechanisms of population genetics. This form of search evolves throughout generations, improving the features of potential solutions by means of biologically-inspired operations. Although they represent a crude simplification of natural evolution, EPs provide efficient and extremely robust search strategies.

Genetic algorithms are probably the best-known type of EP. Although called an algorithm, a genetic algorithm is actually an adaptive heuristic method. It has achieved fame among analysts and engineers for its ability to identify good solutions to previously intractable problems. During recent years, GAs have been applied to hydraulic network optimization (calibration, design, and pump scheduling) with increasing success. Commercial modeling software packages are now making this technology widely available and adoptable to engineering professionals. Optidesigner 1.0[25] is a software for optimization of water distribution networks. It is a GUI (Graphical User Interface), which uses genetic algorithms for the optimization of water distribution networks.

3.5 TEST PROBLEM DESCRIPTION

In this study, the test problem for optimization is a two-loop gravity flow water distribution network with 8 pipes, 7 nodes and one reservoir (fig 3.4). The test problem is obtained from the literature. All the pipes are 1000 m long and Hazen-Williams coefficient is assumed to be 130 for all the pipes. The minimum pressure requirement at nodes is 30m. There are 14 commercially available pipe diameters. [Table 3.2].

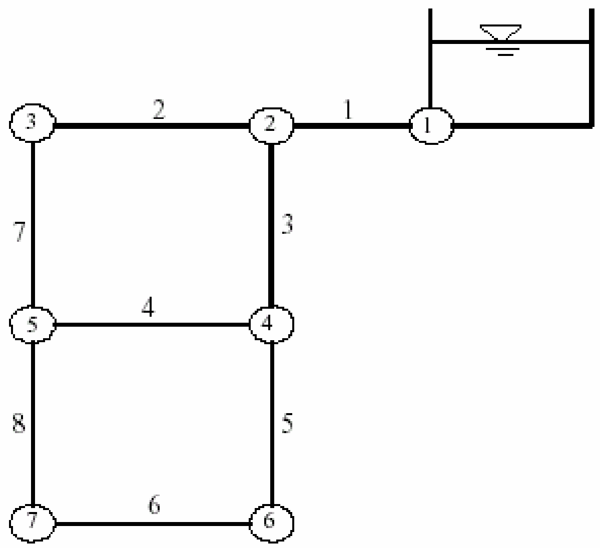


Fig 3.4 The two looped one reservoir problem

Table 3.1 Node data for the two-loop one reservoir network

Node No	Demand (m^3/hr)	Elevation (m)
1	-1,120	210.00
2	100	150.00
3	100	160.00
4	120	155.00
5	270	150.00
6	330	165.00
7	200	160.00

Table 3.2 Cost data for the two-loop one reservoir network

Diameter (in)	Cost (units)
1	2
2	5
3	8
4	11
6	16
8	23
10	32
12	50
14	60
16	90
18	130
20	170
22	300
24	550

3.6 METHODOLOGY ADOPTED FOR OPTIMIZATION

Presented herein is the methodology for the optimization of water distribution network. Our objective is to optimize the network for the minimal network capital cost. To achieve this objective, optimal design solutions for the test problem network are searched using two different techniques. First is Cost-Headloss Ratio technique as described above in sec 3.3, in which optimization is done through an iterative procedure. Second is Genetic Algorithm technique (described above), in this technique optimal design solution is searched through Genetic Algorithms using OPTIDESIGNER 1.0.[25] Capital cost of network for both the solutions are then estimated from pipe costs data as shown above in Table 3.2.

Chapter-4

PERFORMANCE EVALUATION AND RELIABILITY ANALYSIS

4. PERFORMANCE EVALUATION AND RELIABILITY ANALYSIS

GENERAL

In common engineering practice water distribution systems are designed using only heuristic criteria. Determining the optimal configuration and network parameters that can meet required hydraulic parameters are the result of optimization. The performance evaluation and probability of system failure and other reliability evaluations are rarely included in such optimization analyses. The performance and reliability of water distribution systems relies on examining the possibility of meeting desirable objectives under some predefined variable demand scenarios. In the absence of such practice, certain system elements are over designed, but performance and reliability of the entire system is still inadequate.

In the phase of planning and design of the optimal water distribution system configuration and selection of best suitable optimal design alternative, required performance and reliability of system should be included as an important parameter. The mutual comparison of different design alternatives with including performance and reliability as criteria, can lead the designer to an improved, serviceable and reliable solution.

Due to the fact that the poor level of service and failure of water distribution systems causes serious consequences in the social and economical environment, these characteristics have become a field of examination in the past few years. Presented here is an analysis of network performance and reliability and it's consideration in evaluating various design alternatives which helps in choosing best among all optimal design alternatives, in the design and planning phase of water distribution systems.

4.1 PROBLEM FORMULATION – NEED FOR PERFORMANCE AND RELIABILITY ANALYSIS

In past few decades, attention has been given to optimization of water distribution networks for minimizing capital cost of network. Several methodologies have been achieved and also improved further for optimal design of water distribution networks. Although capital cost comparisons for the solution alternatives obtained from various techniques have been carried out. Lake of further investigations for suitability of these methodologies for distribution networks discourages design engineers to adopt them with clarity and ease. A detailed investigations carried out on various design alternatives achieved by such methodologies is thus required. Investigations should consider various parameters governing the criteria for selection of best suitable design alternative.

4.2 PROPOSED SOLUTION

Along with capital cost, consideration of various parameters governing the criteria for selection of best suitable design alternative is proposed. It recommends the detailed evaluation of

1. Capital cost of network
2. Performance of network in various loading conditions
3. Reliability of network

Each alternative should be compared in terms of capital cost, followed by comparative evaluation of performance and reliability of network. Investigations carried out in this manner would draw the guidelines for selection of suitable, individual optimal design alternative.

4.3 GOALS OF PROPOSED METHODOLOGY

Goals of proposed methodology for performance and reliability analysis are to:

1. Encourage consideration of short term and long term performance and hydraulic reliability at node, along with capital cost of network, in the selection of best among various optimal design alternatives.

2. Combine simulator with pipe network design and analysis models.

3. Encourage greater use of optimization models followed by Simulator for the improved, serviceable, reliable, cost effective water distribution networks.

4. Provide a flexible methodology for

    - Analyzing existing networks

    - Optimal design of new water distribution networks

    - Operation and maintenance and better management of existing systems

4.4 PERFORMANCE EVALUATION OF WATER DISTRIBUTION NETWORK

Performance of water distribution system is its ability to meet its desired objectives in variable hydraulic loading situations. It is performed through Extended Period Simulation[26] of water distribution network under various loading situations.

4.4.1 PERFORMANCE OF WATER DISTRIBUTION NETWORK

There are basically two types of performances

1. Short term performance
2. Long term performance

Short term performance considers hourly variation in demand and evaluates the systems on daily performance basis. Whereas long term performance considers variation in pipe characteristics e.g., roughness of pipes on long term basis and evaluates the systems on yearly performance basis.

4.4.2 PERFORMANCE PARAMETERS

Various parameters that are considered in systems performance evaluation are

1. Variation of pressure at nodes
2. Variation of flow in pipes
3. Variation of velocity in pipes
4. Variation of unit headloss in pipes
5. Variation of friction factor in pipes
6. Variation in roughness in pipes over a long period

Among above mentioned parameters, consideration of first 5 parameters show systems short term performance and the last parameter shows systems long term performance of a water distribution network.

4.5 HYDRAULIC RELIABILITY EVALUATION

One of the reasons that reliability has not yet become a common phase in design practice is its complexity. Unlike other systems the water distribution system has the request of meeting the network hydraulic parameters, for network reliability analysis. So, the demand in some node will be satisfied (the node will fulfill its task) if it is physically connected to at least one source node and if pressure is in accordance with designed levels. This second probability can be defined as the probability of meeting hydraulic parameters, or hydraulic reliability for short. For consideration of mechanical reliability, it is assumed that all links are physically connected.

In this study hydraulic reliability of nodes are considered. It defines the probability that the network will meet specified hydraulic parameters (specified pressure at nodes) in variable demand scenarios.

4.5.1 SINGLE NODE RELIABILITY

Probability that the defined hydraulic parameters will be satisfied in a specified demand node. Mechanical reliability of the network is not being investigated here and it is assumed that all links have mechanical reliability equal to 1. These assumptions may be questioned and they may be the topic of future investigations, but presently they are standard assumptions defined in almost all reliability calculations.

4.5.2 RELIABILITY OF MEETING HYDRAULIC PARAMETERS

For water distribution systems connection to a source is not only sufficient, but a necessary condition to ensure that a given node is functional. That is why hydraulic calculation has to be included in determining hydraulic reliability. Hydraulic calculation has to be performed for each scenario.
Hydraulic failure probability at a node can be calculated as

$$P_J = \sum_{i=1}^{S} \left(P_{Hyd,\,J} \right) / \, S$$

Where S is the number of scenarios

If the chosen hydraulic parameter (pressure in nodes) is in specified boundary levels, the hydraulic failure probability is equal to 0 (P_{hyd} =0), and hydraulic reliability is equal to one. If any parameter at any node of the system is not in desired levels, the probability of meeting the hydraulic parameter is not fulfilled, and the hydraulic failure probability is equal to 1 (P_{hyd}=1).

$$P_S = \prod P_i = P_1 \times P_2 \times P_3 ...,...,...P_n$$

$$R_S = 1 - P_S$$

Based on this methodology hydraulic reliability is investigated for various design alternatives. Results of hydraulic reliability are then coupled with cost and performance criterion in order to evaluate and choose best design alternative among achieved optimal design alternatives.

Chapter-5

RESULTS AND DISCUSSIONS

5. RESULTS AND DISCUSSION

GENERAL

As explained above, test problem i.e., two looped, gravity feed water distribution network was optimized using two techniques namely Cost-Headloss Ratio method and Genetic Algorithm. [Details of test problem and cost of pipes are given above [sec. 3.5]. Optimal design solution details are given below. Their capital costs are also included here for comparison between these two, in terms of capital cost of the optimal design alternatives.

5.1 COST COMPARISON BETWEEN COST HEADLOSS RATIO AND GENETIC ALGORITHM OPTIMIZED NETWORK SOLUTIONS

Test problem [sec. 3.5] was optimized using cost headloss ratio [sec 3.3] and genetic algorithm [sec 3.4]. The achieved solutions are given in table 5.1 and 5.2. Their capital costs are also given in respective tables. Graphical representation of these comparison is given below in fig 5.1(a).

5.1.1 RESULTS OF COST COMPARISON

Cost Comparison made from table 5.1 and table 5.2 shows that capital cost for CHR solution comes 410000 whereas for GA it is 419000 units.

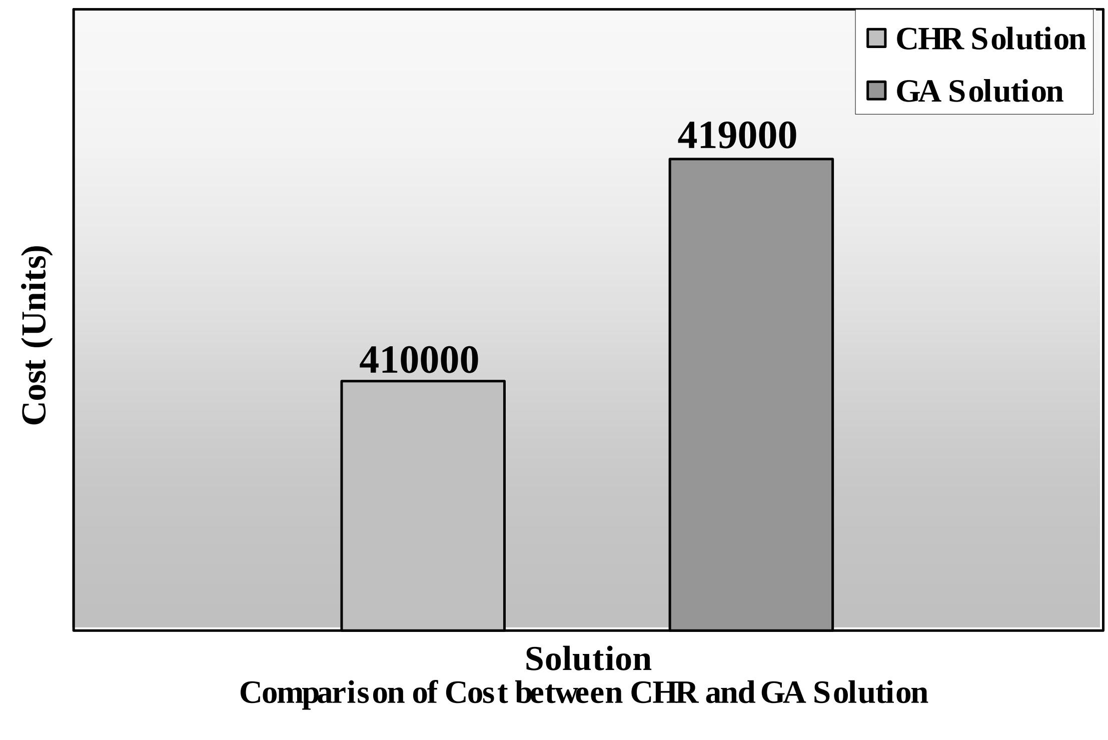
Table 5.1 Optimal Design Solution by Cost Headloss Ratio Method

Link	Start	End	Length	Diameter	Cost per m	Cost of Pipe
ID	**Node**	**Node**	**m**	**mm**	**Units**	**Units**
1	1	2	1000	457.2	130	130000
2	2	3	1000	254	32	32000
3	2	4	1000	406.4	90	90000
4	5	4	1000	25.4	2	2000
5	4	6	1000	406.4	90	90000
6	7	6	1000	254	32	32000
7	3	5	1000	254	32	32000
8	5	7	1000	25.4	2	2000
					Total Cost	**410000**

Table 5.2 Optimal Design Solution by Genetic Algorithm

Link	Start	End	Length	Diameter	Cost per m	Cost of Pipe
ID	**Node**	**Node**	**m**	**mm**	**Units**	**Units**
1	1	2	1000	457.2	130	130000
2	2	3	1000	254	32	32000
3	2	4	1000	406.4	90	90000
4	5	4	1000	101.6	11	11000
5	4	6	1000	406.4	90	90000
6	7	6	1000	254	32	32000
7	3	5	1000	254	32	32000
8	5	7	1000	25.4	2	2000
					Total Cost	**419000**

Fig 5.1(a) Cost Comparison between CHR and GA Optimized Network Solutions



5.1.2 DISCUSSION

Capital cost of network for Cost Headloss Ratio optimal design solution comes to 410,000 units whereas for GA optimal design solution, it is 419,000. In terms of initial cost of network, CHR solution proves to be more economic over GA solution.

Further investigation on performance and reliability of the network are still needed for choice of design alternatives, as other costs such as maintenance and replacement costs depend on overall performance of distribution system.

5.2 SHORT TERM PERFORMANCE COMPARISON BETWEEN COST HEADLOSS RATIO AND GENETIC ALGORITHM OPTIMIZED NETWORK SOLUTIONS

Short term performance was evaluated based on predefined parameters [sec. 4.4.2]. Hourly variation of pressure at nodes by both the solutions is shown in Fig 5.1 and Fig 5.2 for comparison. Hourly variation of flow in pipes is shown in Fig 5.3 and Fig 5.4. Hourly variation of velocity in pipes is shown in Fig 5.5 and Fig 5.6. Hourly variation of unit headloss in pipes is shown in Fig 5.7 and Fig 5.8. Hourly variation of friction factor in pipes is shown in Fig 5.9 and Fig 5.10.

5.2.1 RESULTS FOR SHORT-TERM VARIATIONS

Following were the results for short term performance comparison between cost head loss ratio and genetic algorithm optimized network solutions:

1. For pressure variation at particular nodes, as shown in fig 5.1 and 5.2, GA solution [fig 5.2] reduced the range of pressure variations at nodes over CHR solution [fig 5.1]. For node 5, variation of pressure was in the range of -25 m to 42.5 m for CHR solution, whereas same was in the range of -5 m to 42.5 m for GA solution. It showed the variation of 47.5 m, in terminal pressure at node 5, for GA solution against that of 67.5 m for CHR solution. Similar reductions in terminal pressure variation at nodes, as shown in same figures were found for node 3 and 7 also.

2. Minimum pressures at various nodes, as shown in fig 5.1 and 5.2, were found to be comparatively higher for GA solution [fig 5.2] over CHR solution [fig 5.1]. For CHR solution, as shown in fig 5.1, Node 6 had minimum terminal pressure in the order of -15 m in morning peak hours between 7 AM to 10 AM and even below -20 m in evening peak hours between 6 PM to 7 PM, whereas respective pressure orders for GA solutions, as shown in fig 5.2, were comparatively higher i.e., -5 m

and -15 m respectively. Similar increases in minimal terminal pressure at nodes, as shown in same figures were also found for other nodes.

3. Maximum flow velocity and range of variation of flow velocities, in pipes were found to be lower for GA solution, as shown in fig 5.6, over CHR solution, as shown in fig 5.5. Maximum velocity for pipe 2 was 3.5 m/s for CHR solution [fig 5.5], whereas same for GA solution [fig 5.6], was 3.125 m/s. Similar results were found for other pipes. As shown in same figures, variation of flow velocity for pipe 2 was in the range of 0.3 m/s to 3.5 m/s for CHR solution, whereas same for GA solution was in the range of 0.5 m/s to 3.125 m/s. It showed the variation of 525% for GA solution against that of 1066% for CHR solution. Similar results, from same figures, were also found for other pipes.

4. Unit head loss and the range of its variation in pipes were found to get reduced in case of GA solution over CHR solution. It was observed from fig. 5.7, 5.8, that for unit headloss in various pipes, GA solution [Fig 5.8] reduced the unit headloss and its range of variation in pipes over CHR solution [Fig 5.7]. Maximum headloss for pipe 4, as shown in fig 5.7, was 54 m/km for CHR solution, whereas same for GA solution, as shown in fig 5.8, was 42 m/km. Similar results were also found for other pipes. Variation of headloss for pipe 4 [fig 5.7] was in the range of 2 m/km to 54 m/km for CHR solution, whereas same for GA solution [fig 5.8], was in the range of 2 m/km to 42 m/km. It showed the variation of 2000% for GA solution against that of 2600% for CHR solution. Similar results were found for other pipes.

5. Friction factor and the range of its variation in pipes were found to get reduced in case of GA solution over CHR solution. It was observed from fig. 5.9 and 5.10, that for unit headloss and friction factor in various pipes, GA solution [Fig 5.10] reduced the unit headloss and friction and their range of variation in pipes over CHR solution [Fig

5.9]. Maximum friction factor for pipe 4, as shown in fig 5.9, was 0.039 for CHR solution, whereas same for GA solution, as shown in fig 5.10, was 0.028. Similar results were obtained for other pipes. Variation of friction factor for pipe 4, as shown in fig 5.9, was in the range of 0.03 to 0.039 for CHR solution, where same for GA solution, as shown in fig 5.10, was in the range of 0.022 to 0.028. It showed the variation of 27% for GA solution against that of 30% for CHR solution. Similar results were found for other pipes.

Fig 5.1 Hourly variation of pressure at nodes for CHR Solution

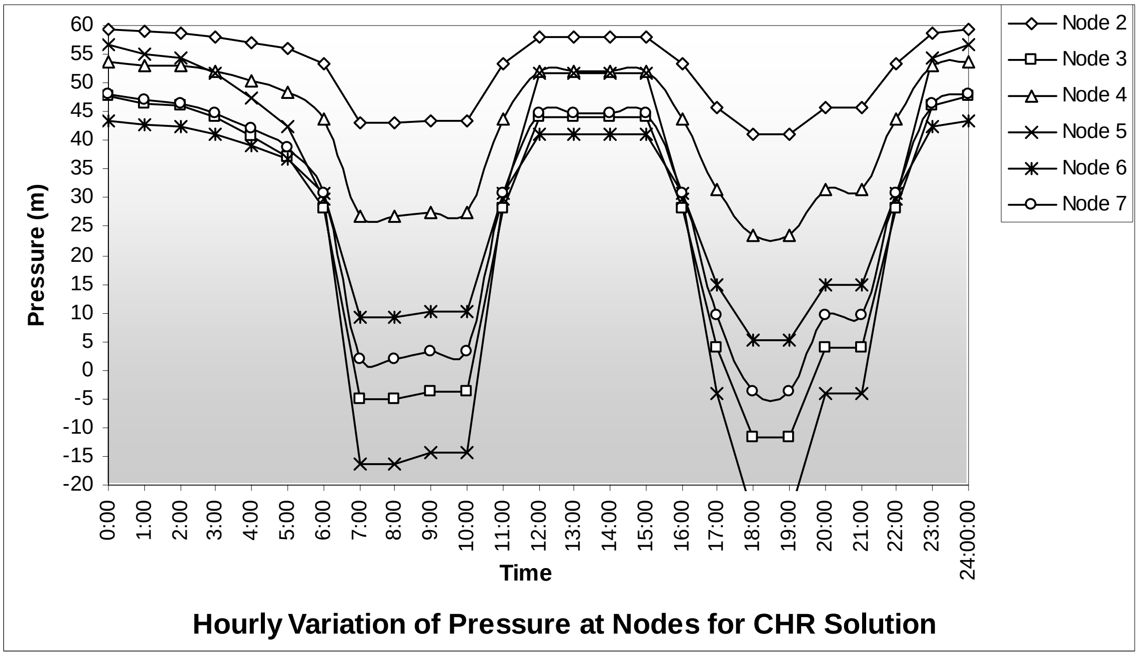


Fig 5.2 Hourly variation of pressure at nodes for GA Solution

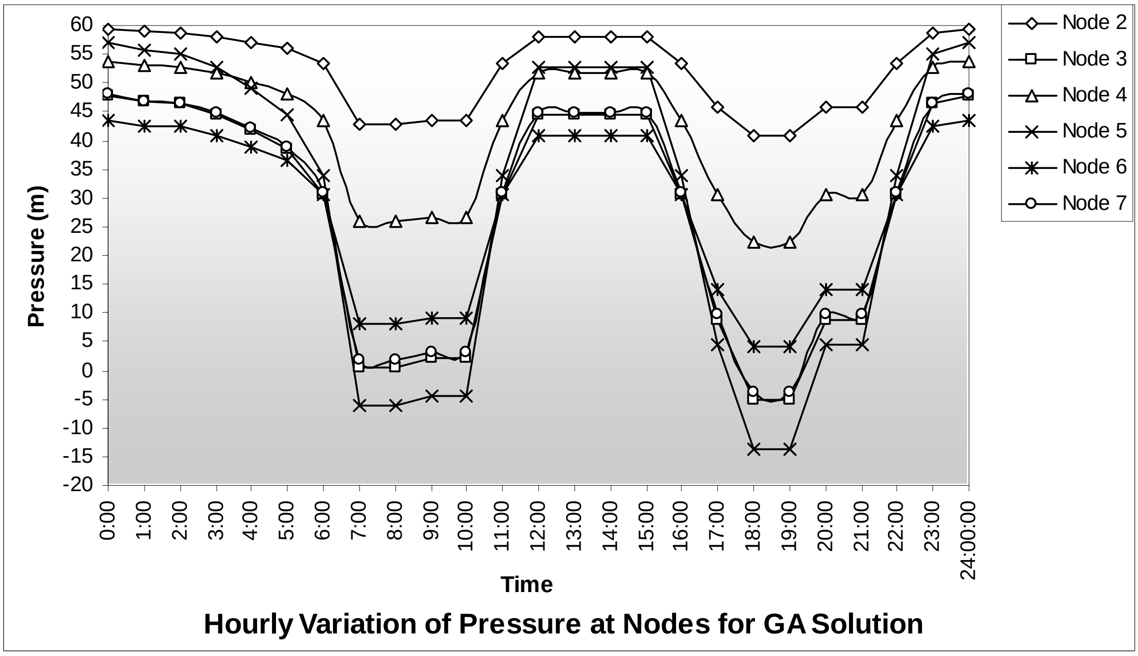
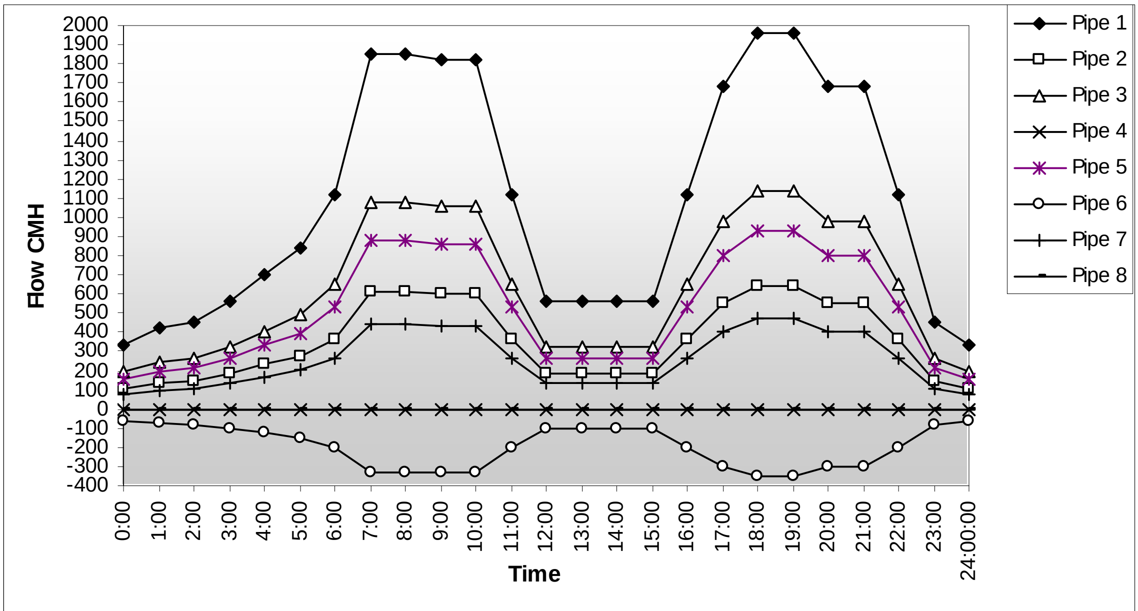
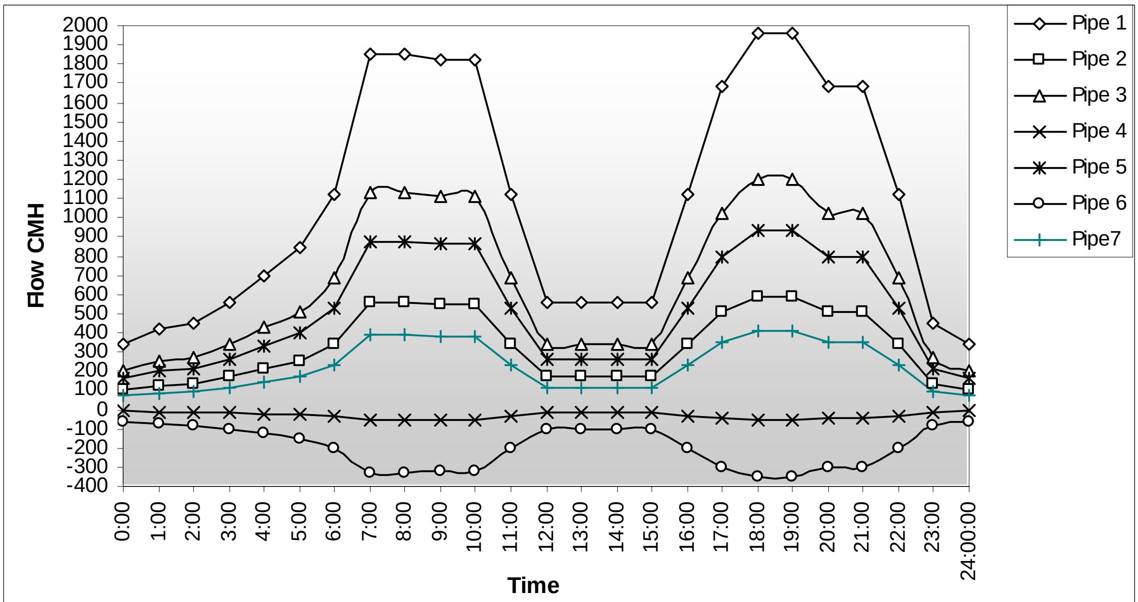


Fig 5.3 Hourly variation of flow in pipes for CHR Solution



Hourly Variation of Flow in Pipes for CHR Solution

Fig 5.4 Hourly variation of flow in pipes for GA Solution



Hourly Variation of Flow in Pipes for GA Solution

Fig 5.5 Hourly variation of velocity in pipes for CHR Solution

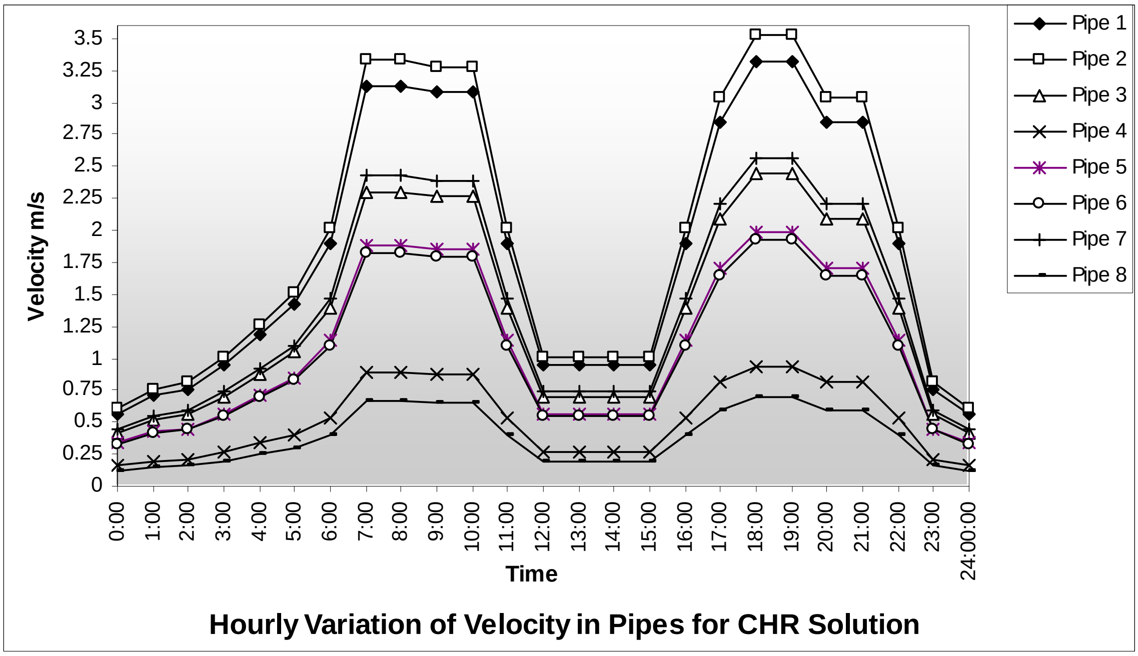


Fig 5.6 Hourly variation of velocity in pipes for GA Solution

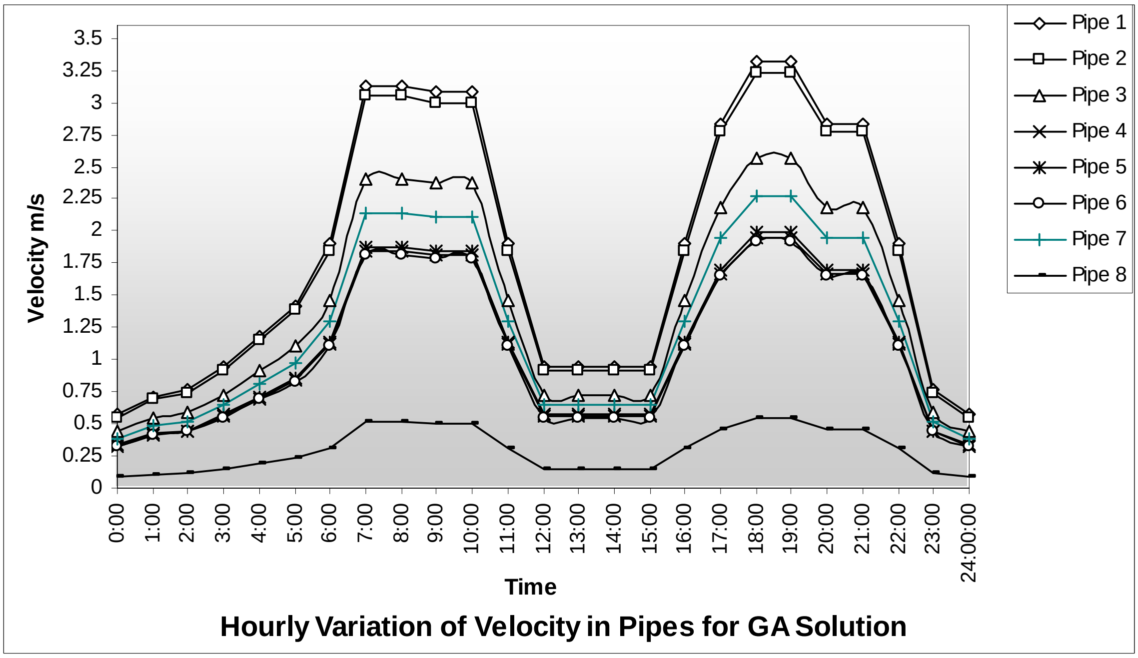


Fig 5.7 Hourly variation of unit head loss in pipes for CHR Solution

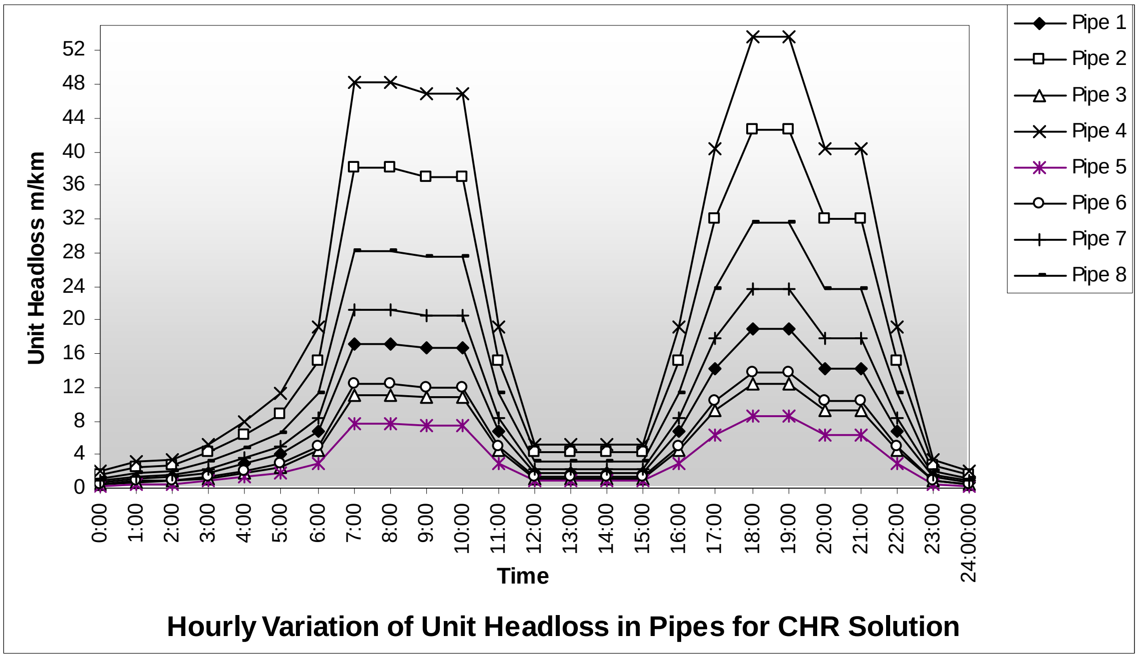


Fig 5.8 Hourly variation of unit head loss in pipes for GA Solution

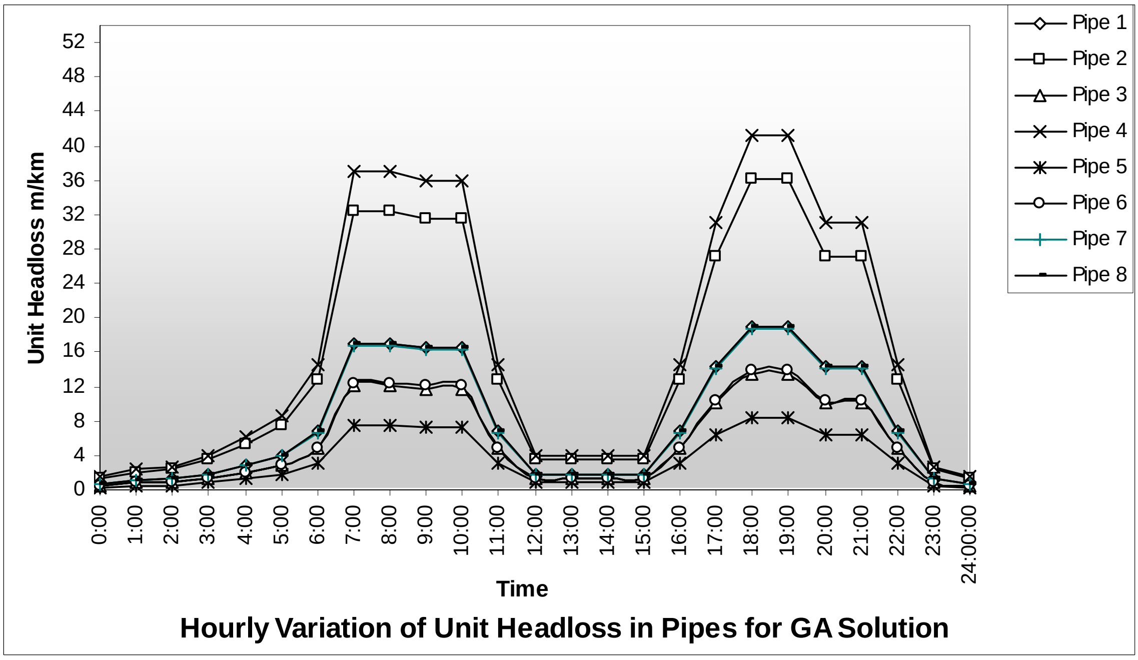


Fig 5.9 Hourly variation of friction factor in pipes for CHR Solution

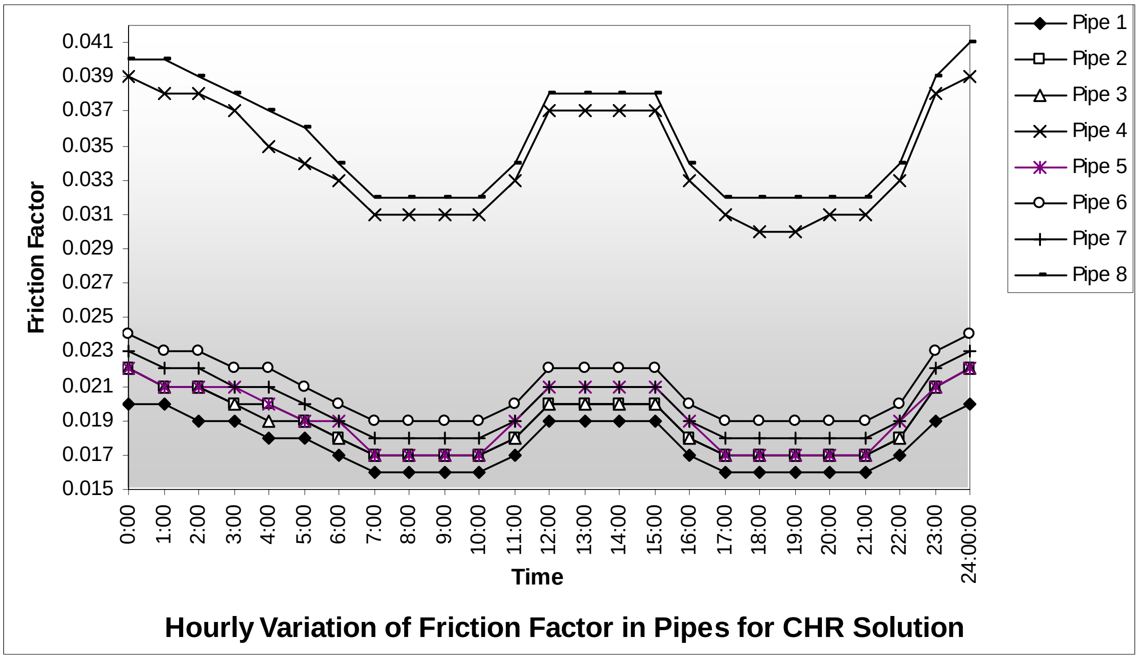
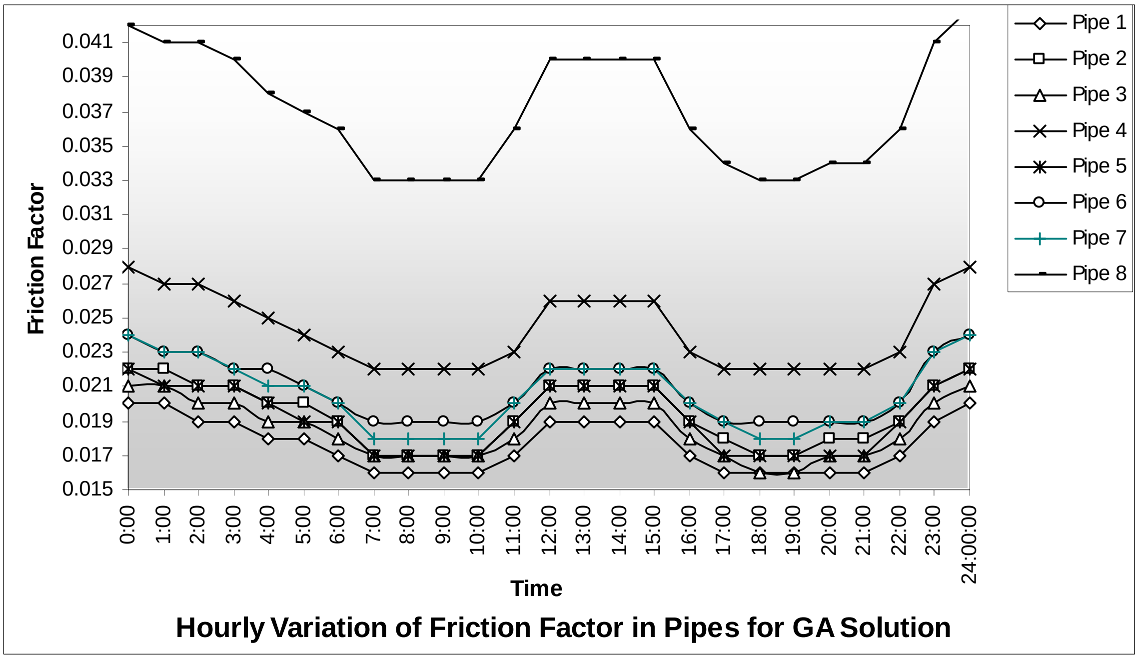


Fig 5.10 Hourly variation of friction factor in pipes for GA Solution



5.2.2 DISCUSSION

Short term performance i.e., system's performance for hourly demand variations, for GA solutions proved to be better over CHR solutions. It is clearly observed from the results obtained that

1. Reduction in the range of pressure variations at particular nodes in GA solution over CHR solution turns into better overall performance of network and improved and reliable services to the consumers.

2. Increase in minimum pressure at nodes for GA solutions over CHR solution turns into improved and reliable services to the consumers.

3. Reduction in maximum flow velocity and variation of flow velocities, in pipes is associated with better serviceability of the network. It is also associated with lesser pipe problems and other maintenance works required thus reducing maintenance expenses and overall cost reduction and better management of distribution system.

4. Reduction in unit headloss and friction factor in pipes and also their range of variation is associated with improved performance of the network. It is also associated with lesser pipe problems and other maintenance works required thus reducing maintenance expenses and overall cost reduction and better management of distribution system.

5.3 LONG TERM PERFORMANCE COMPARISON BETWEEN COST HEADLOSS RATIO AND GENETIC ALGORITHM OPTIMIZED NETWORK SOLUTIONS

Long term performance was evaluated based on predefined parameters [sec. 4.4.2]. Variation of terminal pressure on long term, i.e., yearly basis, at nodes by both the solutions is shown in Fig 5.11 to Fig 5.16 for comparison.

5.3.1 RESULTS FOR LONG-TERM VARIATIONS

After considering a long term variation in pipe roughness over years, performance analysis for degraded pipes, as shown in fig 5.11 to 5.16, shows that in case of GA solution, terminal pressure at nodes were found to be comparatively higher over CHR solution. For pressure at node 3 and 5, as shown in fig 5.12 and 5.14, the difference was found to be clearly distinguishable. Figures given below show the comparison of terminal pressure variation at various nodes.

Fig 5.11 Comparison of Pressure Variation at Node-2 on yearly basis

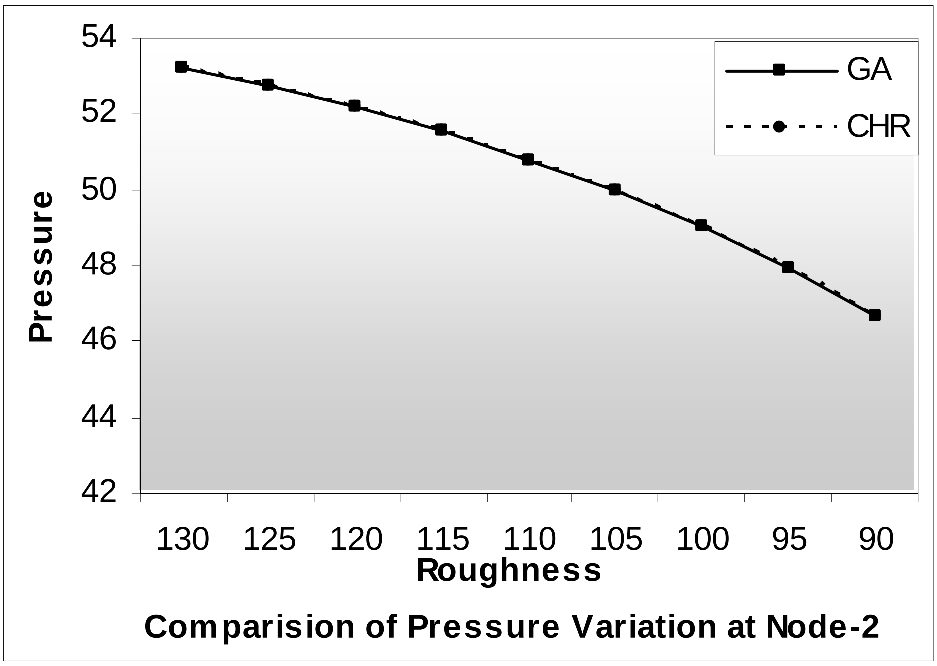


Fig 5.12 Comparison of Pressure Variation at Node-3 on yearly basis

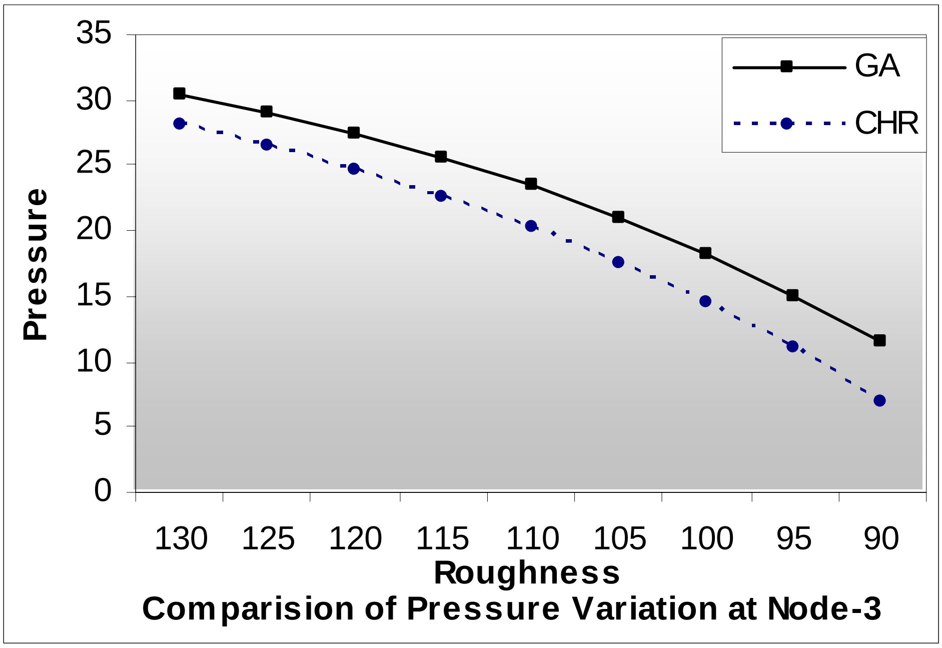


Fig 5.13 Comparison of Pressure Variation at Node-4 on yearly basis

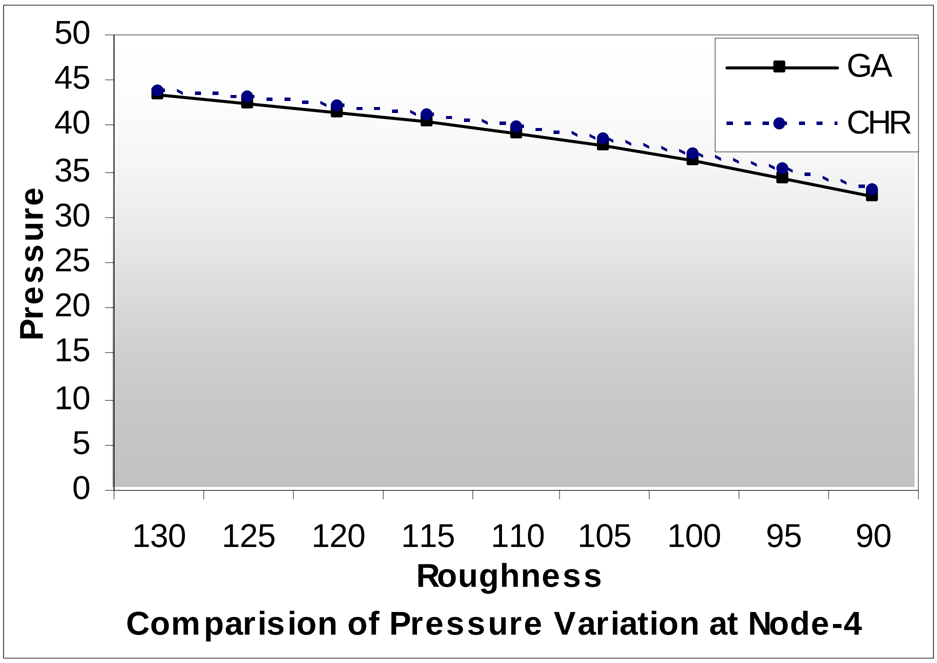


Fig 5.14 Comparison of Pressure Variation at Node-5 on yearly basis

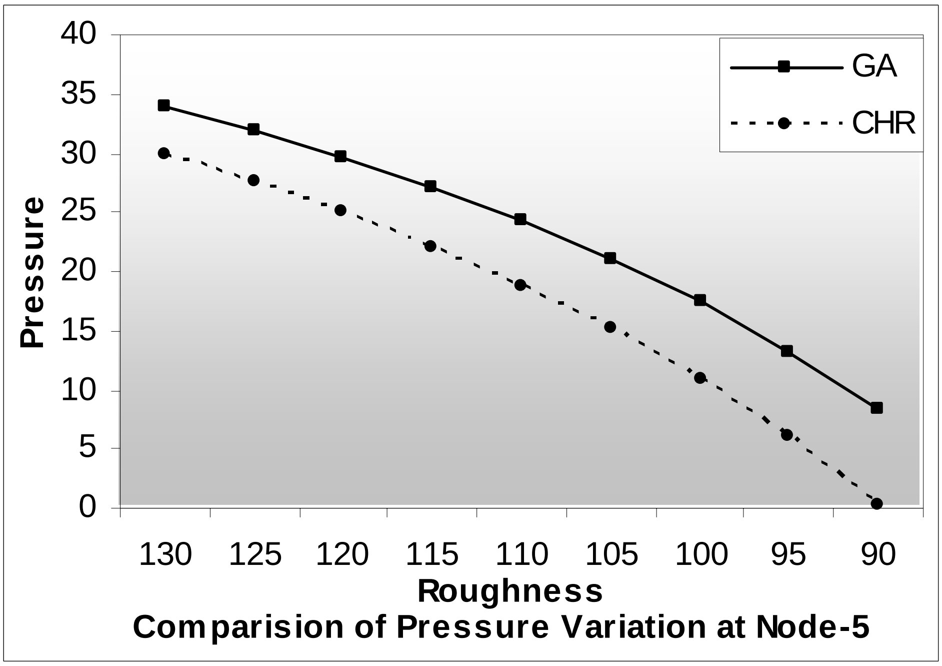


Fig 5.15 Comparison of Pressure Variation at Node-6 on yearly basis

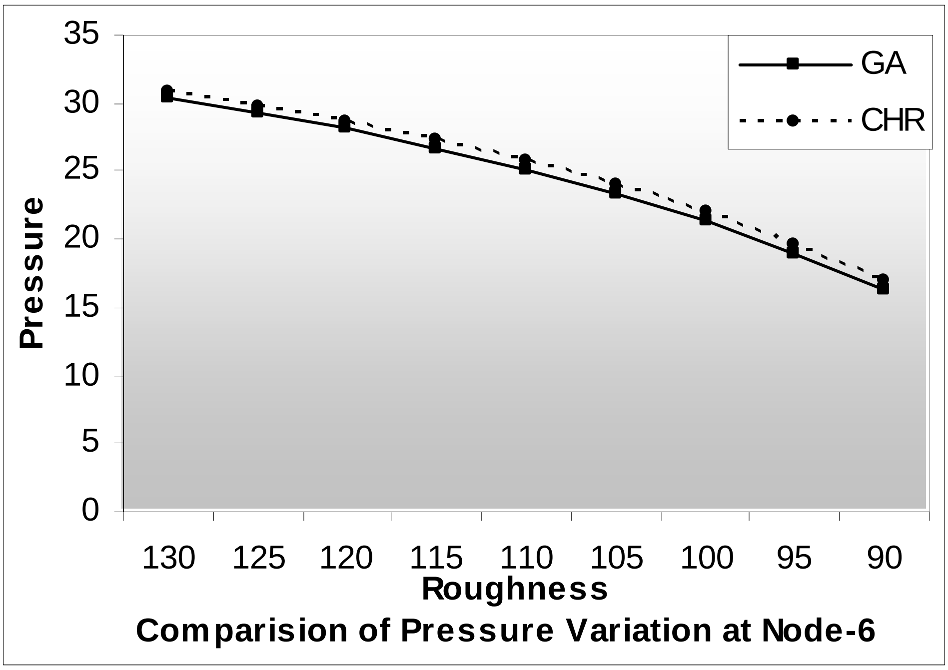
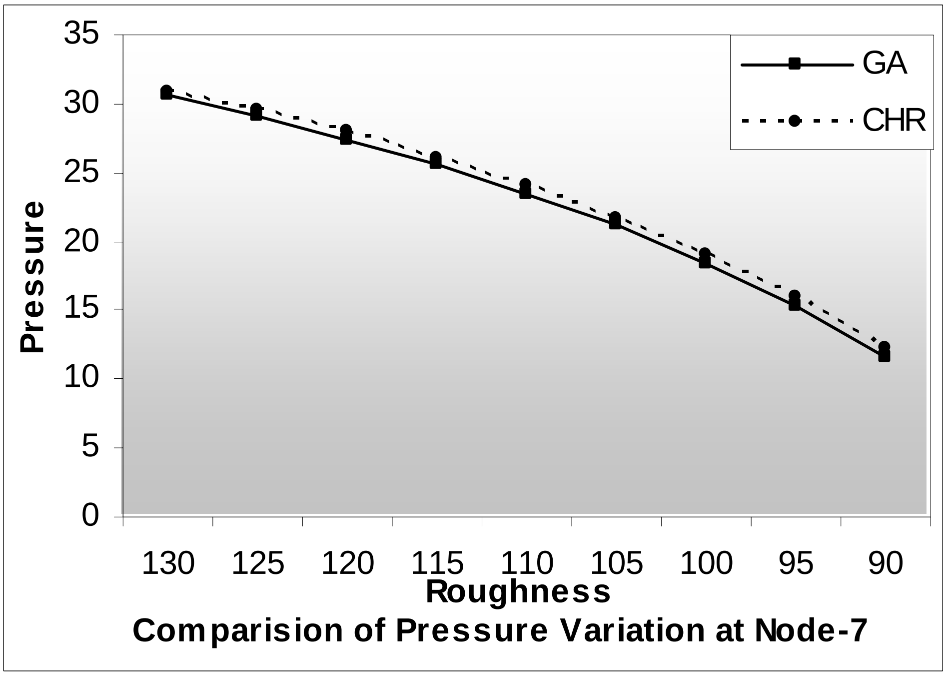


Fig 5.16 Comparison of Pressure Variation at Node-7 on yearly basis



5.3.2 DISCUSSIONS

Long term performance i.e., system’s performance for pipe roughness variations over years, for GA solutions proved to be improved over CHR solutions.

1. Lesser degradation in terminal pressure availability at nodes, in spite of increase in roughness of pipes over years, improves the systems performance and serviceability even after years.

2. It improves the networks serviceable life. For the same level of service network has greater life span or for the same life span, network has improved level of service.

3. Network encounters less cleaning and components replacement works required thus reducing maintenance expenses and overall better management of distribution system.

5.4 COMPARISON OF HYDRAULIC FAILURE PROBABILITY OF NODES AND SYSTEM RELIABILITY BETWEEN COST HEAD LOSS RATIO AND GENETIC ALGORITHM SOLUTIONS

Hydraulic failure probability of nodes and system reliability for cost headloss ratio and genetic algorithm solution were estimated, for comparison. Hydraulic failure probability of nodes was estimated based on predefined parameters [sec. 4.5.1] and system reliability was estimated based on predefined parameters [sec. 4.5.2]. The results are given in table 5.3 and 5.4 respectively, for comparison. Fig 5.17 and fig 5.18 show the graphical comparisons between cost headloss ratio and genetic algorithm solution.

5.4.1 RESULTS OF HYDRAULIC FAILURE PROBABILITY AND SYSTEM RELIABILITY EVALUATION

The following were the results of analysis:

1. Results of hydraulic failure probability analysis are shown in table 5.3. It shows the hydraulic failure probabilities at nodes for CHR and GA solutions. Hydraulic failure probability at nodes for CHR solution were found high compared to hydraulic failure probability at nodes for GA solution, thus making CHR solution network less reliable, hydraulically. Figure 5.17 summarizes the results of hydraulic failure probability analysis of various nodes between cost head loss ratio and genetic algorithm solutions. It graphically compares the hydraulic failure probability of various nodes for CHR solution and GA solution. Hydraulic failure probability for nodes 3 and 5, was low in case of GA solution over CHR solution.

2. Results of system reliability analysis are shown in table 5.4. It shows the system reliability of CHR and GA solution. Fig 5.18 graphically represents the comparison of system

hydraulic reliability between cost headloss ratio and genetic algorithm solutions. System reliability for CHR and GA solutions were found 0.989 and 0.995 respectively.

Table 5.3 Hydraulic failure probabilities at nodes for CHR and GA solutions

NODE NO.	HYDRAULIC FAILURE PROBABILITY	
	CHR SOLUTION	GA SOLUTION
2	0.000	0.000
3	0.540	0.375
4	0.250	0.250
5	0.540	0.375
6	0.375	0.375
7	0.375	0.375

Table 5.4 Hydraulic system reliability for CHR and GA solutions

CHR Solution	GA Solution
0.989	0.995

Fig 5.17 Comparison of Hydraulic Failure Probability of Nodes between Cost Headloss Ratio and Genetic Algorithm Solutions

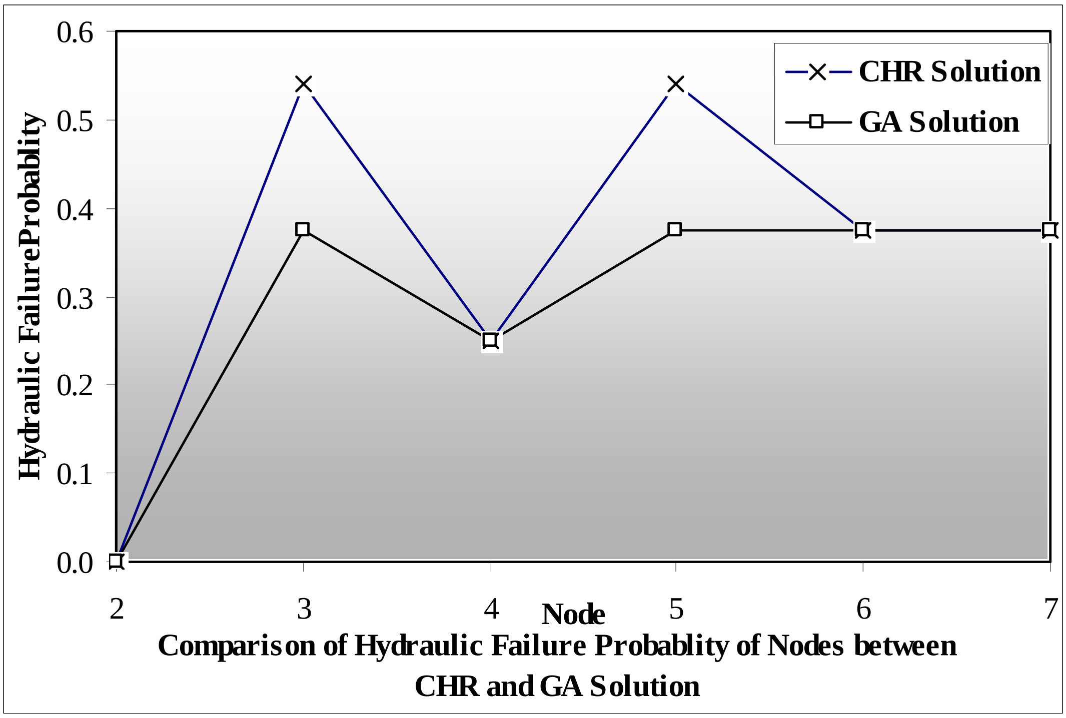
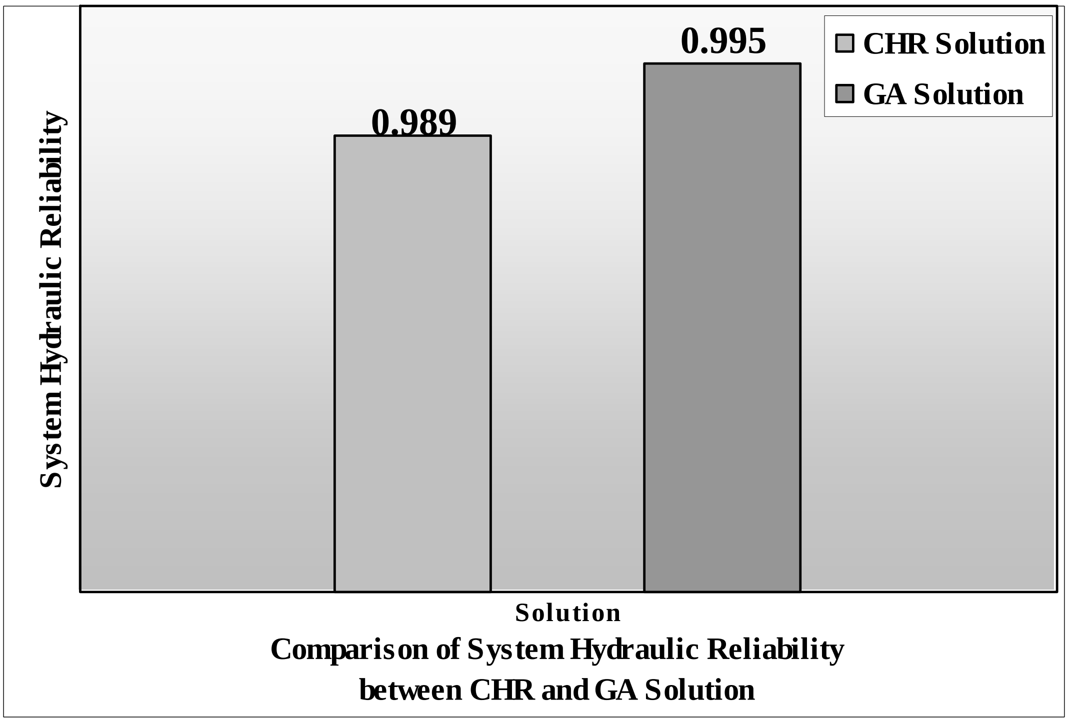


Fig 5.18 Comparison of System Hydraulic Reliability between Cost Headloss Ratio and Genetic Algorithm Solutions



5.4.2 DISCUSSION

Comparison of hydraulic failure probability of various nodes between CHR and GA solutions clearly shows the improved hydraulic reliability of network for GA solution, over CHR solution. Improved reliability at nodes improves the reliability of distribution system thus improving the level of service and overall better management of the system.

Comparison of System hydraulic reliability between CHR solution and GA solution shows the improved system hydraulic reliability for GA solution over CHR solution. Improved reliability of system improves the level of service and overall better management of the system.

Chapter-6

CONCLUSIONS

6. CONCLUSIONS

In this study, the optimization of water distribution network was done using two techniques namely cost headloss ratio technique and genetic algorithm followed by extended period simulation and performance and reliability evaluation of the network, for the comparison of two optimized design alternatives, in order to select best available optimal design alternative. Following are the conclusions of the study:

1. In terms of initial cost of network, CHR solution proves to be more economical over GA solution. Total initial cost of network for Cost Headloss Ratio optimal design solution comes to 410,000 units whereas for GA optimal design solution, it is 419,000. Further investigation on performance and reliability of the network show that overall performance and reliability of network is better for GA solution over CHR solution.

2. In terms of hydraulic performance of network, GA solution proves to be improved over CHR solution. It causes less operation and maintenance and replacement requirements for GA solution over CHR solution, thus making the system really optimal over the period of its life.

3. In terms of long term performance of network, GA solution proves to be improved over CHR solution. It increases serviceable life of the system.

4. Improved system reliability by GA solution over CHR solution, causes better serviceability and improves the marketability of services of water distribution network.

5. Choice of the design alternative is advocated to be governed by initial cost as well as overall performance and reliability of the

system. Along with initial cost, performance and reliability considerations of the system are still needed for choice of design alternatives, because costs other than capital cost, such as maintenance and replacement costs depend upon overall performance of distribution system, over the period of its life.

6. Consideration of performance and reliability of the system not only helps the designer in selecting the better alternative from economic point of view but also helps in judging the serviceability of the distribution system. This turns into improved level of service to its consumers of the essential infrastructural assets involving large investments and affecting their day to day life.

SCOPE OF FUTURE WORK

In this study, the optimization of water distribution network was done using two techniques namely cost headloss ratio technique and genetic algorithm followed by extended period simulation and performance and reliability evaluation of the network, for the comparison of two optimized design alternatives, in order to select best available optimal design alternative. Scope of future work is as follows:

1. Optimal design solutions have been achieved using Cost-Headloss Ratio method and Genetic Algorithm. Optimal design solutions using other optimization models such as Linear programming, Non-linear programming, Mixed Integer Non-linear programming, Dynamic, Shuffled Complex Evolution, Shuffled Frog Leaping Algorithm, Global optimization techniques, Simulated Annealing etc. and investigated in terms of cost and performance and reliability. Guidelines may be drawn for the best suitable optimization technique for particular type of water distribution network.

2. In this study, Performance and reliability for the optimal design solutions has been investigated after optimization. In future work, these may be considered in model building phase for the performance and reliability based optimization of water distribution networks.

3. Mechanical reliability has not been considered in this study. It is assumed to be 1. It may be considered for future work.

4. In this study, optimization of water distribution network using genetic algorithm has been achieved. In future work, optimal number of genetic runs and optimal population of genes may be worked out for refined optimization.

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APPENDIX-I

EXTENDED PERIOD SIMULATION OF COST HEADLOSS RATIO SOLUTION WITH EPANET 2.0

1. INPUT FILE OF COST HEADLOSS RATIO SOLUTION FOR EPANET 2.0

```
[TITLE]

[JUNCTIONS]
;ID        Elev    Demand          Pattern
 2         150     100                     ;
 3         160     100                     ;
 4         155     120                     ;
 5         150     270                     ;
 6         165     330                     ;
 7         160     200                     ;

[RESERVOIRS]
;ID        Head            Pattern
 1         210                     ;

[TANKS]
;ID        Elevation       InitLevel       MinLevel        MaxLevel        Diameter        MinVol

[PIPES]
;ID   Node1  Node2          Length          Diameter        Roughness       MinorLoss       Status
 1     1      2             1000.00         457.2           130             0.00            Open     ;
 2     2      3             1000.00         254             130             0.00            Open     ;
 3     2      4             1000.00         406.4           130             0.00            Open     ;
 4     5      4             1000.00         25.4            130             0.00            Open     ;
 5     4      6             1000.00         406.4           130             0.00            Open     ;
 6     7      6             1000.00         254             130             0.00            Open     ;
 7     3      5             1000.00         254             130             0.00            Open     ;
 8     5      7             1000.00         25.4            130             0.00            Open     ;

[PUMPS]
;ID        Node1           Node2           Parameters

[VALVES]
;ID        Node1           Node2           Diameter        Type    Setting         MinorLoss

[TAGS]

[DEMANDS]
;Junction  Demand          Pattern         Category

[STATUS]
;ID        Status/Setting

[PATTERNS]
;ID        Multipliers
;
 1         0.3     0.375   0.4     0.5     0.625   0.75    1       1.0     1.65    1.65    1.625   1.625
           1.0      1      0.5     0.5     0.5     0.5     1.0     1.5     1       1.75    1.75    1.5
           1.5     1.0     0.4

[CURVES]
;ID        X-Value         Y-Value

[CONTROLS]
;----------------------------------------------------------------------------
;----------------------------------------------------------------------------
[RULES]
;----------------------------------------------------------------------------
;----------------------------------------------------------------------------
```

Contd…

```
[ENERGY]
```

```
 Global Efficiency        75
 Global Price             0.0
 Demand Charge            0.0

[EMITTERS]
;Junction         Coefficient

[QUALITY]
;Node             InitQual

[SOURCES]
;Node             Type             Quality          Pattern

[REACTIONS]
;Type     Pipe/Tank               Coefficient

[REACTIONS]
 Order Bulk               1
 Order Tank               1
 Order Wall               1
 Global Bulk              0.0
 Global Wall              0.0
 Limiting Potential       0.0
 Roughness Correlation    0.0

[MIXING]
;Tank                     Model

[TIMES]
 Duration                 24
 Hydraulic Timestep       1:00
 Quality Timestep         0:05
 Pattern Timestep         1:00
 Pattern Start            0:00
 Report Timestep          1:00
 Report Start             0:00
 Start ClockTime          12 am
 Statistic                None

[REPORT]
 Status           No
 Summary          No
 Page             0

[OPTIONS]
 Units                    CMH
 Headloss                 H-W
 Specific Gravity         1.0
 Viscosity                1.0
 Trials                   40
 Accuracy                 0.001
 Unbalanced               Continue 10
 Pattern                  1
 Demand Multiplier        1.0
 Emitter Exponent         0.5
 Quality                  None mg/L
 Diffusivity              1.0
 Tolerance                0.01

[COORDINATES]
;Node             X-Coord          Y-Coord
 2                70.22            84.89
 3                32.89            84.89
 4                70.22            50.44
 5                32.89            50.44
 6                70.22            15.33
 7                32.89            15.33
 1                109.11           84.89
```

Contd…

```
[VERTICES]
;Link              X-Coord            Y-Coord

[LABELS]
;X-Coord         Y-Coord         Label & Anchor Node

[BACKDROP]
 DIMENSIONS     29.08              11.85              112.92             88.37
 UNITS          Meters
 FILE
 OFFSET         0.00               0.00

[END]
```

FIG. 1 EXTENDED PERIOD SIMULATION OF CHR SOLUTION IN EPANET 2.0

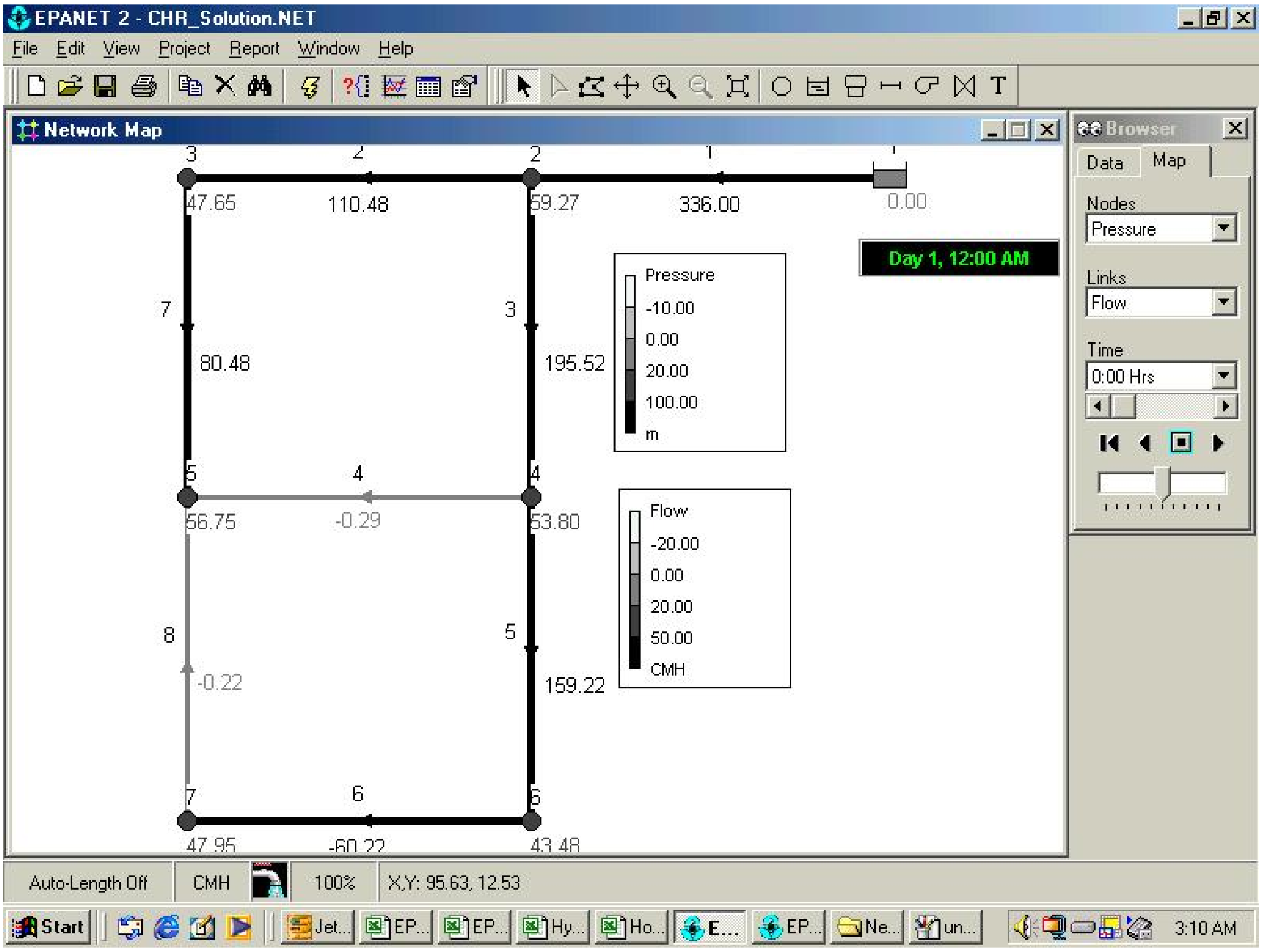

Table 1 VARIATION OF PRESSURE AT NODES AT DIFFERENT TIMES –CHR SOLUTION

	Node 2	Node 3	Node 4	Node 5	Node 6	Node 7
Time	**Pressure**	**Pressure**	**Pressure**	**Pressure**	**Pressure**	**Pressure**
Hours	**m**	**m**	**m**	**m**	**m**	**m**
0:00	59.27	47.65	53.8	56.75	43.48	47.95
1:00	58.9	46.45	53.19	55.09	42.7	46.9
2:00	58.76	46	52.96	54.46	42.41	46.51
3:00	58.13	43.95	51.91	51.63	41.08	44.72
4:00	57.17	40.86	50.33	47.35	39.07	42.02
5:00	56.04	37.19	48.46	42.26	36.69	38.81
6:00	53.25	28.17	43.85	29.78	30.84	30.94
7:00	42.93	-5.19	26.81	-16.39	9.21	1.82
8:00	42.93	-5.19	26.81	-16.39	9.21	1.82
9:00	43.4	-3.65	27.6	-14.26	10.21	3.17
10:00	43.4	-3.65	27.6	-14.26	10.21	3.17
11:00	53.25	28.17	43.85	29.78	30.84	30.94
12:00	58.13	43.95	51.91	51.63	41.08	44.72
13:00	58.13	43.95	51.91	51.63	41.08	44.72
14:00	58.13	43.95	51.91	51.63	41.08	44.72
15:00	58.13	43.95	51.91	51.63	41.08	44.72
16:00	53.25	28.17	43.85	29.78	30.84	30.94
17:00	45.69	3.74	31.37	-4.03	15	9.62
18:00	40.96	-11.55	23.57	-25.19	5.09	-3.72
19:00	40.96	-11.55	23.57	-25.19	5.09	-3.72
20:00	45.69	3.74	31.37	-4.03	15	9.62
21:00	45.69	3.74	31.37	-4.03	15	9.62
22:00	53.25	28.17	43.85	29.78	30.84	30.94
23:00	58.76	46	52.96	54.46	42.41	46.51
24:00:00	59.27	47.65	53.8	56.75	43.48	47.95

Table 2 VARIATIONS OF VARIOUS PARAMETERS AT LINKS AT DIFFERENT TIMES-CHR SOLUTION

Pipe-1				Pipe-2			
Flow	**Velocity**	**Unit Headloss**	**Friction Factor**	**Flow**	**Velocity**	**Unit Headloss**	**Friction Factor**
CMH	**m/s**	**m/km**		**CMH**	**m/s**	**m/km**	
336	0.57	0.73	0.02	110.48	0.61	1.62	0.022
420	0.71	1.1	0.02	138.11	0.76	2.45	0.021
448	0.76	1.24	0.019	147.31	0.81	2.76	0.021
560	0.95	1.87	0.019	184.14	1.01	4.18	0.02
700	1.18	2.83	0.018	230.18	1.26	6.31	0.02
840	1.42	3.96	0.018	276.21	1.51	8.85	0.019
1120	1.9	6.75	0.017	368.29	2.02	15.08	0.018
1848	3.13	17.07	0.016	607.67	3.33	38.12	0.017
1848	3.13	17.07	0.016	607.67	3.33	38.12	0.017
1820	3.08	16.6	0.016	598.47	3.28	37.06	0.017
1820	3.08	16.6	0.016	598.47	3.28	37.06	0.017
1120	1.9	6.75	0.017	368.29	2.02	15.08	0.018
560	0.95	1.87	0.019	184.14	1.01	4.18	0.02
560	0.95	1.87	0.019	184.14	1.01	4.18	0.02
560	0.95	1.87	0.019	184.14	1.01	4.18	0.02
560	0.95	1.87	0.019	184.14	1.01	4.18	0.02
1120	1.9	6.75	0.017	368.29	2.02	15.08	0.018
1680	2.84	14.31	0.016	552.43	3.03	31.95	0.017
1960	3.32	19.04	0.016	644.5	3.53	42.51	0.017
1960	3.32	19.04	0.016	644.5	3.53	42.51	0.017
1680	2.84	14.31	0.016	552.43	3.03	31.95	0.017
1680	2.84	14.31	0.016	552.43	3.03	31.95	0.017
1120	1.9	6.75	0.017	368.29	2.02	15.08	0.018
448	0.76	1.24	0.019	147.31	0.81	2.76	0.021
336	0.57	0.73	0.02	110.49	0.61	1.62	0.022

Contd…

Pipe-3				Pipe-4			
Flow	**Velocity**	**Unit Headloss**	**Friction Factor**	**Flow**	**Velocity**	**Unit Headloss**	**Friction Factor**
CMH	**m/s**	**m/km**		**CMH**	**m/s**	**m/km**	
195.52	0.42	0.47	0.022	-0.29	0.16	2.05	0.039
244.39	0.52	0.71	0.021	-0.37	0.2	3.1	0.038
260.69	0.56	0.81	0.021	-0.39	0.21	3.49	0.038
325.86	0.7	1.22	0.02	-0.49	0.27	5.28	0.037
407.32	0.87	1.84	0.019	-0.61	0.34	7.99	0.035
488.79	1.05	2.58	0.019	-0.73	0.4	11.19	0.034
651.71	1.4	4.4	0.018	-0.98	0.54	19.07	0.033
1075.33	2.3	11.12	0.017	-1.62	0.89	48.2	0.031
1075.33	2.3	11.12	0.017	-1.62	0.89	48.2	0.031
1059.03	2.27	10.81	0.017	-1.59	0.87	46.86	0.031
1059.03	2.27	10.81	0.017	-1.59	0.87	46.86	0.031
651.71	1.4	4.4	0.018	-0.98	0.54	19.07	0.033
325.86	0.7	1.22	0.02	-0.49	0.27	5.28	0.037
325.86	0.7	1.22	0.02	-0.49	0.27	5.28	0.037
325.86	0.7	1.22	0.02	-0.49	0.27	5.28	0.037
325.86	0.7	1.22	0.02	-0.49	0.27	5.28	0.037
651.71	1.4	4.4	0.018	-0.98	0.54	19.07	0.033
977.57	2.09	9.32	0.017	-1.47	0.81	40.4	0.031
1140.5	2.44	12.4	0.017	-1.71	0.94	53.75	0.03
1140.5	2.44	12.4	0.017	-1.71	0.94	53.75	0.03
977.57	2.09	9.32	0.017	-1.47	0.81	40.4	0.031
977.57	2.09	9.32	0.017	-1.47	0.81	40.4	0.031
651.71	1.4	4.4	0.018	-0.98	0.54	19.07	0.033
260.69	0.56	0.81	0.021	-0.39	0.21	3.49	0.038
195.51	0.42	0.47	0.022	-0.29	0.16	2.05	0.039

Pipe-5				Pipe-6			
Flow	**Velocity**	**Unit Headloss**	**Friction Factor**	**Flow**	**Velocity**	**Unit Headloss**	**Friction Factor**
CMH	**m/s**	**m/km**		**CMH**	**m/s**	**m/km**	
159.22	0.34	0.32	0.022	-60.22	0.33	0.53	0.024
199.03	0.43	0.49	0.021	-75.28	0.41	0.8	0.023
212.29	0.45	0.55	0.021	-80.29	0.44	0.9	0.023
265.37	0.57	0.83	0.021	-100.37	0.55	1.36	0.022
331.71	0.71	1.26	0.02	-125.46	0.69	2.05	0.022
398.05	0.85	1.76	0.019	-150.55	0.83	2.88	0.021
530.73	1.14	3.01	0.019	-200.73	1.1	4.9	0.02
875.71	1.88	7.6	0.017	-331.21	1.82	12.39	0.019
875.71	1.88	7.6	0.017	-331.21	1.82	12.39	0.019
862.44	1.85	7.39	0.017	-326.19	1.79	12.04	0.019
862.44	1.85	7.39	0.017	-326.19	1.79	12.04	0.019
530.73	1.14	3.01	0.019	-200.73	1.1	4.9	0.02
265.37	0.57	0.83	0.021	-100.37	0.55	1.36	0.022
265.37	0.57	0.83	0.021	-100.37	0.55	1.36	0.022
265.37	0.57	0.83	0.021	-100.37	0.55	1.36	0.022
265.37	0.57	0.83	0.021	-100.37	0.55	1.36	0.022
530.73	1.14	3.01	0.019	-200.73	1.1	4.9	0.02
796.1	1.7	6.37	0.017	-301.1	1.65	10.38	0.019
928.78	1.99	8.47	0.017	-351.28	1.93	13.82	0.019
928.78	1.99	8.47	0.017	-351.28	1.93	13.82	0.019
796.1	1.7	6.37	0.017	-301.1	1.65	10.38	0.019
796.1	1.7	6.37	0.017	-301.1	1.65	10.38	0.019
530.73	1.14	3.01	0.019	-200.73	1.1	4.9	0.02
212.29	0.45	0.55	0.021	-80.29	0.44	0.9	0.023
159.22	0.34	0.32	0.022	-60.22	0.33	0.53	0.024

Contd…

Pipe-7				Pipe-8			
Flow	Velocity	Unit Headloss	Friction Factor	Flow	Velocity	Unit Headloss	Friction Factor
CMH	m/s	m/km		CMH	m/s	m/km	
80.48	0.44	0.9	0.023	-0.22	0.12	1.2	0.04
100.61	0.55	1.36	0.022	-0.28	0.15	1.81	0.04
107.31	0.59	1.54	0.022	-0.29	0.16	2.05	0.039
134.14	0.74	2.32	0.021	-0.37	0.2	3.09	0.038
167.68	0.92	3.51	0.021	-0.46	0.25	4.67	0.037
201.21	1.1	4.92	0.02	-0.55	0.3	6.55	0.036
268.29	1.47	8.39	0.019	-0.73	0.4	11.16	0.034
442.67	2.43	21.2	0.018	-1.21	0.66	28.22	0.032
442.67	2.43	21.2	0.018	-1.21	0.66	28.22	0.032
435.97	2.39	20.61	0.018	-1.19	0.65	27.43	0.032
435.97	2.39	20.61	0.018	-1.19	0.65	27.43	0.032
268.29	1.47	8.39	0.019	-0.73	0.4	11.16	0.034
134.14	0.74	2.32	0.021	-0.37	0.2	3.09	0.038
134.14	0.74	2.32	0.021	-0.37	0.2	3.09	0.038
134.14	0.74	2.32	0.021	-0.37	0.2	3.09	0.038
134.14	0.74	2.32	0.021	-0.37	0.2	3.09	0.038
268.29	1.47	8.39	0.019	-0.73	0.4	11.16	0.034
402.43	2.21	17.77	0.018	-1.1	0.6	23.65	0.032
469.5	2.57	23.64	0.018	-1.28	0.7	31.47	0.032
469.5	2.57	23.64	0.018	-1.28	0.7	31.47	0.032
402.43	2.21	17.77	0.018	-1.1	0.6	23.65	0.032
402.43	2.21	17.77	0.018	-1.1	0.6	23.65	0.032
268.29	1.47	8.39	0.019	-0.73	0.4	11.16	0.034
107.31	0.59	1.54	0.022	-0.29	0.16	2.05	0.039
80.49	0.44	0.9	0.023	-0.22	0.12	1.2	0.041

APPENDIX-II

EXTENDED PERIOD SIMULATION OF GENETIC ALGORITHM SOLUTION WITH EPANET 2.0

1. INPUT FILE OF GENETIC ALGORITHM SOLUTION FOR EPANET 2.0

```
[TITLE]

[JUNCTIONS]
;ID              Elev     Demand          Pattern
 2               150.00           100.00                          ;
 3               160.00           100.00                          ;
 4               155.00           120.00                          ;
 5               150.00           270.00                          ;
 6               165.00           330.00                          ;
 7               160.00           200.00                          ;

[RESERVOIRS]
;ID              Head             Pattern
 1               210.00                           ;

[TANKS]
;ID              Elevation        InitLevel        MinLevel         MaxLevel         Diameter
[PIPES]
;ID       Node1  Node2    Length   Diameter         Roughness        MinorLoss        tatus
 1        1      2        1000.00          457.20           130     0.00             Open     ;
 2        2      3        1000.00          254.00           130     0.00             Open     ;
 3        2      4        1000.00          406.40           130     0.00             Open     ;
 4        5      4        1000.00          101.60           130     0.00             Open     ;
 5        4      6        1000.00          406.40           130     0.00             Open     ;
 6        7      6        1000.00          254.00           130     0.00             Open     ;
 7        3      5        1000.00          254.00           130     0.00             Open     ;
 8        5      7        1000.00          25.40            130     0.00             Open     ;

[PUMPS]
;ID              Node1            Node2            Parameters

[VALVES]
;ID              Node1            Node2            Diameter         Type     Setting          MinorLoss

[TAGS]

[DEMANDS]
;Junction        Demand           Pattern          Category
 2               100.00                            ;
 2               0.00                     ;
 3               100.00                            ;
 3               0.00                     ;
 4               120.00                            ;
 4               0.00                     ;
 5               270.00                            ;
 5               0.00                     ;
 6               330.00                            ;
 6               0.00                     ;
 7               200.00                            ;
 7               0.00                     ;

[STATUS]
;ID              Status/Setting

[PATTERNS]
;ID              Multipliers
;Hourly Demand Variation
 1               0.3      0.375            0.4      0.5      0.625            0.75
 1               1.0      1.65     1.65    1.625             1.625            1.0
 1               0.5      0.5      0.5     0.5      1.0      1.5
 1               1.75     1.75     1.5     1.5      1.0      0.4
```

Contd…

```
[CURVES]

;ID                     X-Value                 Y-Value

[CONTROLS]

[RULES]

[ENERGY]
 Global Efficiency  75.00
 Global Price       0
 Demand Charge                 0.0000

[EMITTERS]
;Junction              Coefficient

[QUALITY]
;Node                  InitQual

[SOURCES]
;Node                  Type                    Quality                 Pattern

[REACTIONS]
;Type       Pipe/Tank              Coefficient

[REACTIONS]
 Order Bulk         1.00
 Order Tank         1.00
 Order Wall         1
 Global Bulk        0.0000
 Global Wall        0.0000
 Limiting Potential            0
 Roughness Correlation         0

[MIXING]
;Tank                  Model

[TIMES]
 Duration           24
 Hydraulic Timestep            1:00
 Quality Timestep   0:05
 Pattern Timestep   1:00
 Pattern Start      0:00
 Report Timestep    1:00
 Report Start       0:00
 Start ClockTime    0:00
 Statistic          NONE

[REPORT]
 Status             No
 Summary            No
 Page               0

[OPTIONS]
 Units              CMH
 Headloss           H-W
 Specific Gravity   1.0000
 Viscosity          1.0000
 Trials             40
 Accuracy           0.00100000
 Unbalanced         Continue 10
 Pattern                       1
 Demand Multiplier             1.00
 Emitter Exponent              0.50
 Quality                       NONE mg/L
 Diffusivity                   1.0000
 Tolerance                     0.01000000
```

Contd…

```
[COORDINATES]
;Node            X-Coord          Y-Coord
 2               70.22            84.89
 3               32.89            84.89
 4               70.22            50.44
 5               32.89            50.44
 6               70.22            15.33
 7               32.89            15.33
 1               109.11           84.89

[VERTICES]
;Link            X-Coord          Y-Coord

[LABELS]
;X-Coord       Y-Coord        Label & Anchor Node

[BACKDROP]
 DIMENSIONS      29.08            11.85            112.92           88.37
 UNITS           None
 FILE
 OFFSET          0.00             0.00

[END]
```

FIG. 2 EXTENDED PERIOD SIMULATION OF GA SOLUTION IN EPANET 2.0

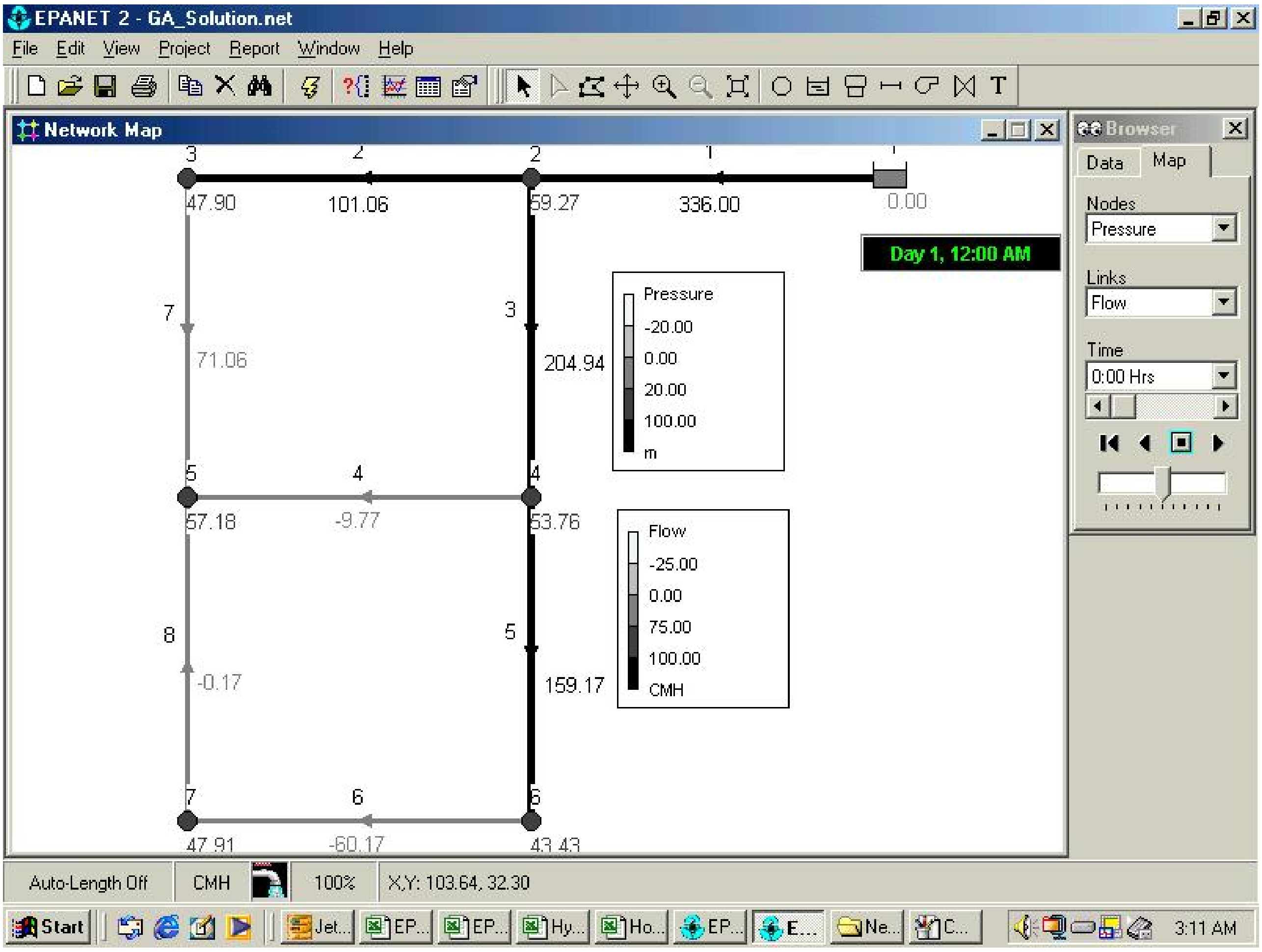

Table 1 VARIATION OF PRESSURE AT NODES AT DIFFERENT TIMES –GA SOLUTION

	Node 2	Node 3	Node 4	Node 5	Node 6	Node 7
Time	Pressure	Pressure	Pressure	Pressure	Pressure	Pressure
Hours	m	m	m	m	m	m
0:00	59.27	47.9	53.76	57.18	43.43	47.91
1:00	58.9	46.82	53.12	55.74	42.63	46.84
2:00	58.76	46.42	52.88	55.2	42.33	46.44
3:00	58.13	44.59	51.8	52.74	40.97	44.61
4:00	57.17	41.82	50.16	49.03	38.9	41.86
5:00	56.04	38.53	48.22	44.62	36.46	38.58
6:00	53.25	30.46	43.45	33.8	30.44	30.55
7:00	42.93	0.61	25.8	-6.23	8.2	0.84
8:00	42.93	0.61	25.8	-6.23	8.2	0.84
9:00	43.4	1.99	26.61	-4.38	9.23	2.21
10:00	43.4	1.99	26.61	-4.38	9.23	2.21
11:00	53.25	30.46	43.45	33.8	30.44	30.55
12:00	58.13	44.59	51.8	52.74	40.97	44.61
13:00	58.13	44.59	51.8	52.74	40.97	44.61
14:00	58.13	44.59	51.8	52.74	40.97	44.61
15:00	58.13	44.59	51.8	52.74	40.97	44.61
16:00	53.25	30.46	43.45	33.8	30.44	30.55
17:00	45.69	8.6	30.52	4.49	14.16	8.79
18:00	40.96	-5.08	22.44	-13.85	3.97	-4.83
19:00	40.96	-5.08	22.44	-13.85	3.97	-4.83
20:00	45.69	8.6	30.52	4.49	14.16	8.79
21:00	45.69	8.6	30.52	4.49	14.16	8.79
22:00	53.25	30.46	43.45	33.8	30.44	30.55
23:00	58.76	46.42	52.88	55.2	42.33	46.44
24:00:00	59.27	47.9	53.76	57.18	43.43	47.91

Table 2 VARIATIONS OF VARIOUS PARAMETERS AT LINKS AT DIFFERENT TIMES-GA SOLUTION

Pipe-1				Pipe-2			
Flow	Velocity	Unit Headloss	Friction Factor	Flow	Velocity	Unit Headloss	Friction Factor
CMH	m/s	m/km		CMH	m/s	m/km	
336	0.57	0.73	0.02	101.06	0.55	1.38	0.022
420	0.71	1.1	0.02	126.33	0.69	2.08	0.022
448	0.76	1.24	0.019	134.75	0.74	2.34	0.021
560	0.95	1.87	0.019	168.44	0.92	3.54	0.021
700	1.18	2.83	0.018	210.55	1.15	5.35	0.02
840	1.42	3.96	0.018	252.66	1.39	7.5	0.02
1120	1.9	6.75	0.017	336.88	1.85	12.78	0.019
1848	3.13	17.07	0.016	555.85	3.05	32.32	0.017
1848	3.13	17.07	0.016	555.85	3.05	32.32	0.017
1820	3.08	16.6	0.016	547.43	3	31.42	0.017
1820	3.08	16.6	0.016	547.43	3	31.42	0.017
1120	1.9	6.75	0.017	336.88	1.85	12.78	0.019
560	0.95	1.87	0.019	168.44	0.92	3.54	0.021
560	0.95	1.87	0.019	168.44	0.92	3.54	0.021
560	0.95	1.87	0.019	168.44	0.92	3.54	0.021
560	0.95	1.87	0.019	168.44	0.92	3.54	0.021
1120	1.9	6.75	0.017	336.88	1.85	12.78	0.019
1680	2.84	14.31	0.016	505.32	2.77	27.09	0.018
1960	3.32	19.04	0.016	589.54	3.23	36.04	0.017
1960	3.32	19.04	0.016	589.54	3.23	36.04	0.017
1680	2.84	14.31	0.016	505.32	2.77	27.09	0.018
1680	2.84	14.31	0.016	505.32	2.77	27.09	0.018
1120	1.9	6.75	0.017	336.88	1.85	12.78	0.019
448	0.76	1.24	0.019	134.75	0.74	2.34	0.021
336	0.57	0.73	0.02	101.06	0.55	1.38	0.022

Contd…

Pipe-3				Pipe-4			
Flow	Velocity	Unit Headloss	Friction Factor	Flow	Velocity	Unit Headloss	Friction Factor
CMH	m/s	m/km		CMH	m/s	m/km	
204.94	0.44	0.52	0.021	-9.77	0.33	1.58	0.028
256.17	0.55	0.78	0.021	-12.21	0.42	2.38	0.027
273.25	0.59	0.88	0.02	-13.02	0.45	2.68	0.027
341.56	0.73	1.33	0.02	-16.28	0.56	4.06	0.026
426.95	0.91	2.01	0.019	-20.35	0.7	6.13	0.025
512.34	1.1	2.82	0.019	-24.42	0.84	8.6	0.024
683.12	1.46	4.8	0.018	-32.56	1.12	14.65	0.023
1127.15	2.41	12.13	0.017	-53.73	1.84	37.03	0.022
1127.15	2.41	12.13	0.017	-53.73	1.84	37.03	0.022
1110.07	2.38	11.79	0.017	-52.91	1.81	35.99	0.022
1110.07	2.38	11.79	0.017	-52.91	1.81	35.99	0.022
683.12	1.46	4.8	0.018	-32.56	1.12	14.65	0.023
341.56	0.73	1.33	0.02	-16.28	0.56	4.06	0.026
341.56	0.73	1.33	0.02	-16.28	0.56	4.06	0.026
341.56	0.73	1.33	0.02	-16.28	0.56	4.06	0.026
341.56	0.73	1.33	0.02	-16.28	0.56	4.06	0.026
683.12	1.46	4.8	0.018	-32.56	1.12	14.65	0.023
1024.68	2.19	10.17	0.017	-48.84	1.67	31.03	0.022
1195.46	2.56	13.52	0.016	-56.98	1.95	41.29	0.022
1195.46	2.56	13.52	0.016	-56.98	1.95	41.29	0.022
1024.68	2.19	10.17	0.017	-48.84	1.67	31.03	0.022
1024.68	2.19	10.17	0.017	-48.84	1.67	31.03	0.022
683.12	1.46	4.8	0.018	-32.56	1.12	14.65	0.023
273.25	0.59	0.88	0.02	-13.02	0.45	2.68	0.027
204.94	0.44	0.52	0.021	-9.77	0.33	1.58	0.028

Pipe-5				Pipe-6			
Flow	Velocity	Unit Headloss	Friction Factor	Flow	Velocity	Unit Headloss	Friction Factor
CMH	m/s	m/km		CMH	m/s	m/km	
159.17	0.34	0.32	0.022	-60.17	0.33	0.53	0.024
198.96	0.43	0.49	0.021	-75.21	0.41	0.8	0.023
212.22	0.45	0.55	0.021	-80.22	0.44	0.9	0.023
265.28	0.57	0.83	0.021	-100.28	0.55	1.36	0.022
331.6	0.71	1.26	0.02	-125.35	0.69	2.05	0.022
397.92	0.85	1.76	0.019	-150.42	0.82	2.87	0.021
530.56	1.14	3	0.019	-200.56	1.1	4.89	0.02
875.42	1.87	7.59	0.017	-330.92	1.81	12.37	0.019
875.42	1.87	7.59	0.017	-330.92	1.81	12.37	0.019
862.16	1.85	7.38	0.017	-325.91	1.79	12.02	0.019
862.16	1.85	7.38	0.017	-325.91	1.79	12.02	0.019
530.56	1.14	3	0.019	-200.56	1.1	4.89	0.02
265.28	0.57	0.83	0.021	-100.28	0.55	1.36	0.022
265.28	0.57	0.83	0.021	-100.28	0.55	1.36	0.022
265.28	0.57	0.83	0.021	-100.28	0.55	1.36	0.022
265.28	0.57	0.83	0.021	-100.28	0.55	1.36	0.022
530.56	1.14	3	0.019	-200.56	1.1	4.89	0.02
795.84	1.7	6.37	0.017	-300.84	1.65	10.37	0.019
928.48	1.99	8.47	0.017	-350.98	1.92	13.79	0.019
928.48	1.99	8.47	0.017	-350.98	1.92	13.79	0.019
795.84	1.7	6.37	0.017	-300.84	1.65	10.37	0.019
795.84	1.7	6.37	0.017	-300.84	1.65	10.37	0.019
530.56	1.14	3	0.019	-200.56	1.1	4.89	0.02
212.22	0.45	0.55	0.021	-80.22	0.44	0.9	0.023
159.17	0.34	0.32	0.022	-60.17	0.33	0.53	0.024

Contd…

Pipe-7				Pipe-8			
Flow	**Velocity**	**Unit Headloss**	**Friction Factor**	**Flow**	**Velocity**	**Unit Headloss**	**Friction Factor**
CMH	**m/s**	**m/km**		**CMH**	**m/s**	**m/km**	
71.06	0.39	0.72	0.024	-0.17	0.09	0.73	0.042
88.83	0.49	1.08	0.023	-0.21	0.11	1.1	0.041
94.75	0.52	1.22	0.023	-0.22	0.12	1.24	0.041
118.44	0.65	1.84	0.022	-0.28	0.15	1.87	0.04
148.05	0.81	2.79	0.021	-0.35	0.19	2.83	0.038
177.66	0.97	3.91	0.021	-0.42	0.23	3.96	0.037
236.88	1.3	6.66	0.02	-0.56	0.31	6.75	0.036
390.85	2.14	16.83	0.018	-0.92	0.51	17.06	0.033
390.85	2.14	16.83	0.018	-0.92	0.51	17.06	0.033
384.93	2.11	16.37	0.018	-0.91	0.5	16.59	0.033
384.93	2.11	16.37	0.018	-0.91	0.5	16.59	0.033
236.88	1.3	6.66	0.02	-0.56	0.31	6.75	0.036
118.44	0.65	1.84	0.022	-0.28	0.15	1.87	0.04
118.44	0.65	1.84	0.022	-0.28	0.15	1.87	0.04
118.44	0.65	1.84	0.022	-0.28	0.15	1.87	0.04
118.44	0.65	1.84	0.022	-0.28	0.15	1.87	0.04
236.88	1.3	6.66	0.02	-0.56	0.31	6.75	0.036
355.32	1.95	14.11	0.019	-0.84	0.46	14.3	0.034
414.54	2.27	18.77	0.018	-0.98	0.54	19.03	0.033
414.54	2.27	18.77	0.018	-0.98	0.54	19.03	0.033
355.32	1.95	14.11	0.019	-0.84	0.46	14.3	0.034
355.32	1.95	14.11	0.019	-0.84	0.46	14.3	0.034
236.88	1.3	6.66	0.02	-0.56	0.31	6.75	0.036
94.75	0.52	1.22	0.023	-0.22	0.12	1.24	0.041
71.06	0.39	0.72	0.024	-0.17	0.09	0.73	0.043