Risk and hydraulic reliability analysis of water distribution systems

Juned Laiq Syed

Civil Engineering

November 2003

Abstract

In this study, reliability analysis of water distribution systems is performed by focusing on the hydraulic failure of the water distribution system. It considers systems failures due to pressure heads at the junctions which are unable to satisfy the requirements. At first input random variables are generated for the Monte Carlo approach by using the statistical analysis package STATISTICA, and for bootstrapping input random variables are generated by MATLAB. The hydraulic simulation software EPANET is used to perform hydraulic analysis for each set of generated input random variables, and nodal pressures are calculated. Finally, nodal and system reliability is calculated by adopting a minimum cut set approach, which involves the use of pipe failure probabilities. A methodology based on Generic Expectation Function (GEF) is developed to calculate a pipe failure probability which is found to be more realistic.

In this study, based on the described methodology, reliability analysis is performed for Al-Khobar water distribution system. The results show that the hydraulic reliability of Al-Khobar water distribution system is approximately 67%, including low hydraulic reliability compared to similar distribution systems. Accordingly, the study proposes several recommendations to improve the hydraulic reliability of Al-Khobar water distribution system.

RISK AND HYDRAULIC RELIABILITY ANALYSIS OF WATER DISTRIBUTION SYSTEMS

JUNED LAIQ SYED

CIVIL ENGINEERING

NOVEMBER 2003

KING FAHD UNIVERSITY OF PETROLEUM & MINERALS

DHAHRAN 31261, SAUDI ARABIA

DEANSHIP OF GRADUATE STUDIES

This thesis, written by **JUNED LAIQ SYED** under the direction of his thesis advisor and approved by his thesis committee, has been presented to and accepted by the Dean of Graduate Studies, in partial fulfillment of the requirements for the degree of

MASTER OF SCIENCE IN CIVIL ENGINEERING.

Thesis Committee

____

Dr. Muhammad. Al-Zahrani. (Chairman)

____

Dr. Rashid.I.Allayla (Member)

____

Dr. Mohammad S. Al-Suwaiyan (Member)

____

Dr. Hamad I. Al-Abdul Wahhab
Department Chairman

____

Dr. Osama Ahmed Jannadi
Dean of Graduate Studies

____

Date

Dedicated to

My Parents, Nephews and Nieces

ACKNOWLEDGEMENTS

I am extremely grateful to Almighty Allah who alone made this accomplishment possible. Research is basically unveiling the mysteries of the universe by trying to understand the laws of nature as set by the Creator.

Acknowledgements are due to King Fahd University of Petroleum & Minerals for providing support and computational facilities in carrying out this research. Acknowledgements are also extended to the Graduate School and SABIC for necessary funding support throughout the work under the Grant No. 01.

I would like to express my gratitude to my main thesis advisor Dr. Muhammad Al-Zahrani for his constant help and guidance throughout this work. My deep thanks are offered to my thesis committee members Dr. Rashid.I. Allayla and Dr. Mohammad Saleh Al-Suwaiyan for their sincere guidance and advice.

I also appreciate the assistance and encouragement received from the Chairman, faculty members, staff and graduate students of the department. Also, I am thankful to Al-Khobar Water Authority for their cooperation in providing necessary data.

Special thanks to Abid, Ahmar, Akhtar Ghazi, Asif, Moin Bhai, Mudassir and Saad Bhai for both moral and technical support. I would also like to thank Amir Bhai for his useful suggestions during preparation of my thesis presentation. An expression of gratitude for all friends in KFUPM, specially Dr. Kalim and Faisal Alvi for their cooperation and guidance.

Finally, I am grateful to my parents and all family members for their extreme moral support, encouragement and patience during the course of my studies here.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

THESIS ABSTRACT

Name: JUNED LAIQ SYED

Title: Risk and Hydraulic Reliability Analysis of Water Distribution Systems

Degree: Master of Science

Major Field: Civil Engineering

(Water Resources and Environmental Engineering)

Date of Degree: November 2003

In this study, reliability analysis of water distribution systems is performed by focusing on the hydraulic failure of the water distribution system. It considers systems failures due to pressure heads at the junctions which are unable to satisfy the requirements. At first, input random variables are generated for the Monte Carlo approach by using the statistical analysis package STATISTICA, and for bootstrapping input random variables are generated by MATLAB. The hydraulic simulation software EPANET is used to perform hydraulic analysis for each set of generated input random variables, and nodal pressures are calculated. Finally, nodal and system reliability is calculated by adopting a minimum cut set approach, which involves the use of pipe failure probabilities. A methodology based on Generic Expectation Function (GEF) is developed to calculate a pipe failure probability which is found to be more realistic.

In this study, based on the described methodology, reliability analysis is performed for Al-Khobar water distribution system. The results show that the hydraulic reliability of Al-Khobar water distribution system is approximately 67%, indicating low hydraulic reliability compared to similar distribution systems. Accordingly, the study proposes several recommendations to improve the hydraulic reliability of Al-Khobar water distribution system.

Master of Science Degree

King Fahd University of Petroleum and Minerals

Dhahran, Saudi Arabia

November 2003

الخلاصة

اسم الطالب : جنيد ليك سيد

عنوان الرساله : تحليل الموثوقيه والأخطار الهيدروليكيه المرتبطه المتعلقه بشبكات توزيع المياه

الدرجه : ماجستير علوم هندسيه

التخصص : هندسه مدنيه (هندسة مصادر مياه وبيئه)

تاريخ الدرجه : نوفمبر 2003

في هذه الدراسه تم تطوير طريقة يمكن من خلالها تقييم الموثوقيه الهيدروليكيه لنظم توزيع شبكات المياه آخذاً في الإعتبار التغير العشوائي لضغط المياه في شبكات التوزيع الناتجه عن المدخلات الغير مؤكده لخصائص الشبكه . ولتحقيق أهداف الدراسه تم استخدام برنامج يحاكي حركة المياه في الشبكات (EPANET) ، وتم توليد مدخلات البرنامج العشوائيه باستخدام طريقتين هما : مونت كارلو (Monte Carlo) ، و بوت سترابنق (boot strapping) ، ومن ثم تم تبني طريقة (minimum cut set) المجموعات الصغرى المتقطعه لحساب الموثوقيه الهيدروليكيه لشبكة التوزيع . وتم تطبيق الطريقه المقترحه على شبكة مياه فرضيه للتأكد من كفاءتها ومن ثم تم تطبيقها على شبكة مياه مدينة الخبر . وأوضحت نتائج الدراسه بأن الموثوقيه الهيدروليكيه لشبكة توزيع مياه الخبر تقدر بـ 67% مما يدل على موثوقيه غير عاليه لشبكة توزيع الخبر مقارنةً بالمقاييس العالميه .

وخلصت الدراسه إلى بعض التوصيات التي من شأن تبنيها تحسين الأداء الهيدروليكي لشبكة توزيع المياه بمدينة الخبر ومن ثم تحقيق موثوقيه عاليه للشبكه .

درجة الماجستير

جامعة الملك فهد للبترول والمعادن

الظهران – المملكه العربيه السعوديه

نوفمبر 2003

Chapter 1

INTRODUCTION

1.1 General

The primary purpose of water distribution systems is to provide safe water, in adequate amounts, at reasonable pressure, to all users, at all times and at the lowest cost possible under economic and other constraints which exist at any specific time. The most important consideration in the planning and operation of the water distribution system is to satisfy consumer demands.

A water distribution system is an interconnected collection of sources, pipes, and hydraulic control elements (e.g., pumps, valves, regulators, and tanks) aimed at delivering water to the consumers in prescribed quantities at desired pressures. Such systems are often described in terms of a graph, with links representing the pipes, and nodes representing connections between pipes, hydraulic control elements, consumers and sources. The behavior of a water distribution system is governed by:

1. Physical laws that describe the flow relationships in the pipes and hydraulic control elements
2. Consumer demand, and
3. System layout.

1.2 Importance of Risk and Reliability

Water distribution systems are among the most essential municipal systems and require extensive planning and maintenance for supplying good quality water to consumers. The main components of water distribution systems are pipes, pumps, storage tanks, reservoirs, and groundwater wells. The planning process involves considerations of the available resources, anticipated demands, location of facilities and other economic considerations. System reliability is considered as part of the design. However, the reliability is typically not quantified; rather engineering judgment plays an important role in the design.

The reliability of water distribution systems is defined by Goulter (1995) and Cullinane et al (1992) as, “The ability of a distribution system to meet the demands that are placed on it, where demands are specified in terms of:

- the flow to be supplied, and
- the range of pressure at which these flow rates must be provided.”

The reliability of water distribution systems is also defined by Kaufmann et al. (1977) as, “The probability of the system that it will perform specified tasks under specified conditions and during a specified time”.

On the other hand, risk is associated with decision making in stochastic environments. The stochastic environment in water distribution networks arises from a number of sources, broadly categorized as follows:

- external factors to the system: e.g. uncertainty in the hydrologic regime which provides the water supply to the system and which also impacts upon the demands on the system, and
- internal factors to the system: e.g. uncertainties in the performance of the components of the system.

Risk differs from reliability in that it is the statement of the probabilities of occurrences of events and the impacts of those events, whereas reliability describes how a system responds or reacts to events. An important feature of risk is that the consequences of events may be described in quantitative or qualitative terms.

It is extremely difficult to define risk in a single set of words because of the high level of confusion surrounding the aspects of this subject. In general, risk could be established in qualitative aspect as well as in quantitative aspect. The latter one is usually called Engineering Risk Analysis. It is important to the observer that qualitative aspect of risk conveys a level understanding about failure or success of some defined event. In such way, risk comes relative to hazard and safeguards, where hazard is defined as a source of damage or injury. The quantification of risk involves looking for answers for three basic questions:

- What can happen?
- How often failure is expected?
- What are the likely consequences?

Reliability considerations for water distribution systems are an integral part of all decisions regarding the planning, design and operational phases. A major problem in

reliability analysis of water distribution systems is to define reliability measures that are meaningful and appropriate, while still being computationally feasible. Traditionally, reliability is provided by following certain heuristic guidelines, like ensuring two alternative paths to each demand node from at least one source, or having the entire pipe diameters greater than a minimum prescribed value. By using these guidelines it is implicitly assumed that reliability is assured, but the level of reliability provided is not quantified or measured. Therefore only limited confidence can be placed in these guidelines, since reliability is not considered explicitly.

Reliability is an important factor that requires due consideration during optimal design of water distribution networks. It has currently no universally accepted definition or measure. Nevertheless, it is generally understood that reliability is concerned with the ability of the network to provide an adequate supply to the consumers, under both normal and abnormal conditions (Xu and Goulter, 1999). Major abnormal operating conditions commonly arise in two ways: when demand exceeds the design value and causes what is known as performance failure of the network; and when one or more network components fail, resulting in what is known as mechanical failure of the distribution network. Since water distribution networks are normally designed to satisfy any foreseeable high demands, such as maximum hourly consumption and fire demands, the performance failure may not be severe. Therefore, if one is concerned only about the reliability of the pipe network, then the water distribution system reliability can be defined as the probability of satisfying nodal demand under normal and possible pipe failure conditions.

Application of reliability analysis to hydraulic engineering covers a wide scope of sub-fields, ranging from data collection and gauging network design to turbulence loading

on structures; and from inland surface water to groundwater to coastal water. In terms of the system scale, it could involve entire river basins containing many components, or a large dam and reservoir, or a single culvert or pipe. The analysis objectives could cover the following: designing the geometry and dimension of hydraulic facilities, planning of a hydraulic project, the operation procedure, management strategy, risk-cost analysis, or risk-based decision making.

Reliability analysis for designing water distribution systems has proven to be a fruitful area of research. However, progress in this field is not as rapid as one might wish. This is due to the fact that huge and detailed data are required in order to perform reliability analysis (Plate, 1993).

1.3 Objectives of the study

The nodal and system reliability analysis of water distribution systems is useful in identifying the critical locations of junctions in the network where the pressure heads are not enough to fulfill the minimum pressure requirements which are required to satisfy consumer demand. Reliability analysis of water distribution systems helps in improving the reliability level by providing some pragmatic solution of the problem.

This study covers the following objectives:

1. Perform hydraulic reliability analysis of water distribution system under stochastic conditions.
2. Apply Monte Carlo method and Bootstrapping for generating model input data.
3. Develop a methodology for calculating pipe failure probability based on Generic Expectation Function (GEF) and compare this developed methodology with alternate method based on Poisson method.

4. Compare the final results to see the effect of stochasticity of random variables on nodal and system reliability.

1.4 Methodology

Reliability analysis of water distribution systems is performed by focusing on the hydraulic failure of the water distribution system which results when junction pressures are unable to satisfy the minimum requirements. At first, input random variables are generated for Monte Carlo approach by using the statistical analysis package STATISTICA® (1997) and for bootstrapping input random variables are generated by MATLAB®. The hydraulic simulation software EPANET® (2000) is used to calculate nodal pressures for each set of generated input random variables and for the random combinations of pipe closures. Finally, nodal and system reliability is calculated by adopting a minimum cut set approach which involves the use of pipe failure probabilities. A methodology based on Generic Expectation Function (GEF) is developed to calculate a pipe failure probability which is found to be more realistic and conservative. In order to determine the combination of pipe closures, a purely random approach is adopted for a fixed number of pipe closure combinations by using MATHEMATICA® (1999).

In this study, based on the described methodology, reliability analysis is performed for the Al-Khobar water distribution system.

Chapter 2

LITERATURE REVIEW

2.1 Risk and Reliability Analysis in Water Resources Engineering

Risk and reliability analysis is presently being performed in almost all fields of engineering depending upon the specific field and its particular area. In software engineering, reliability of software is considered as important as its efficiency. Glenford (1976) defines software reliability as the probability that the software will execute for a particular period of time without a failure, weighted by the cost to the user of each failure encountered. Industrial engineers applied reliability analysis techniques for product quality control more than half a century ago. Aeronautical engineering has applied it for aircraft and spacecraft design. Communication engineering incorporates it for reliability of communication systems. Nuclear engineering applies it for safety evaluation. Much has been developed in structural engineering relating to hazard control in earthquakes and wind engineering. Several countries in the world already have probabilistic-based building codes in parallel to the traditional deterministic codes. On the other hand, hydraulic engineering, perhaps paying too much attention only to frequency analysis and its statistical distribution in the past, has been rather slow in expanding its horizon.

In civil engineering, reliability analysis is performed in almost all branches, such as structural engineering, earthquake engineering, environmental engineering and water

resources engineering. Ganoulis (1991) performed stochastic modeling and used fuzzy-set theory to define loads and resistance of hydrological systems under risk conditions. This is done by considering hydrological variables, parameters, inputs, and outputs as random or fuzzy variables.

El Maalouf and Young (1992) evaluated analytical algorithms of three methods for applications to water quality analysis in pipe systems. These algorithms were based on nodal mass balance formulation and concentration of contaminant in the pipes. They examined the reliability of these algorithms by modeling a water distribution network with each algorithm and comparing the results of each solution. They concluded that the comparisons of concentrations for the modeled network ranged from poor to good but did not provide conclusive evidence that any of these algorithms is reliable.

Goderya and Adelman (1996) introduced a risk/cost analysis technique for finding an optimum well location and accordingly developed a pump and treat clean up strategy. They performed the analysis for a small community. The aquifer around the drinking water supply wells is identified to be contaminated with carbon tetrachloride, the source being the grain storage facility. Carbon tetrachloride was used as a fumigant in the grain bins. In a related study, a lumped parameter model was developed to simulate clean up of the upper aquifer using one well and the plume boundary as the model area boundary. They developed a distributed parameter model to locate the most optimum well location in the model area grid network which covers a larger area than the plume. They developed a computer program to model the transport of the contaminant using a mixing cell model and Monte Carlo simulation techniques to generate risk and cost criteria for different well

locations and pumping rates. Their model included a stochastic groundwater recharge variable to reflect variability of recharge with annual rainfall. They computed risk values for different grid network cells versus normalized cost and as a result risk/cost plots were generated. They concluded that the existing well location may be the best option for the pump and treat clean up strategy.

Dominique et al. (1999) compared the probabilistic and possibilistic approaches to uncertainty in risk assessment modeling. They performed fuzzy and Monte Carlo calculations to determine the risk of excess cancer from groundwater contamination and concluded that fuzzy calculations are more conservative because they consider all possible combinations of parameter values, while in the Monte Carlo calculations, scenarios that combine low probability parameter values have very little chance of being randomly selected. However, in an environmental context, where human health is often at stake, mere possibility that a scenario might occur can be an important element in the decision-making process. In this case, the possibilistic approach may seem more appropriate.

Verdonck et al. (2000) applied bootstrapping, the maximum likelihood method (MLE) and Bayesian approaches to estimate uncertainty in the toxicity test results, which are used in probabilistic risk assessment. They determined the most suitable and reliable technique to determine the uncertainty bands on a cumulative distribution function. They concluded that all methods give similar uncertainty estimates, considering the fact that other, large uncertainties exist in the risk assessment process.

Souza et al. (2001) applied the methods of probabilities to evaluate the uncertainties present in the water pollution analysis processes and showed that there are some restrictions in using the method of probability with inconsistent sets of data. This is due to the fact that the probability theory needs, in its application, the probability density function for all sets of random variables that should be analyzed in order to measure the risk of any environmental system. This cannot be done with an inconsistent data set.

Wong and Yeh (2002) present a systematic approach for solving the management problem of a contaminated groundwater supply system. They selected supply water quality criteria based on stochastic health risk assessment and concluded with the establishment of a trade-off relationship between the increased management cost and the desired level of protection. They performed uncertainty analysis by using Gaussian quadrature numerical integration as an alternative to the nested Monte Carlo simulation. They adopted two management approaches - treating the contaminated groundwater with granular activated carbon and using imported water to lower the contamination level in the water supply. They used a random hydraulic conductivity field to produce the contamination variability at each extraction well. They obtained a trade-off relationship between the increased management cost and the level of protection by performing the uncertainty analysis at several supply water quality criteria for each of the two management approaches.

2.2 Application of Risk and Reliability in Water Distribution Systems

In order to analyze risk and reliability in water distribution systems, different approaches are presently being employed by different researchers.

Germanopoulos et al. (1986) presented a methodology for assessing the security of supply from a water distribution system associated with different network failure events such as burst mains or source failures. They used a network simulation model to study the operation of the network both under normal operating conditions and conditions arising from crisis events. They identified the effect of different failure events on the supplies of the area and they used probability analysis of the occurrence of such events to provide an assessment of the security of water supply. They also identified the operational responses that should be triggered by crisis events. They concluded that the assessment of supply reliability obtained using the adopted methodology is considerably different from that suggested by the conventional approach, which simply relates supply reliability to the amount of emergency storage available in the network.

Bargiela and Hainsworth (1989) examined the problem of telemetry-related uncertainty in water systems, its causes, and its consequences. In the monitoring of water distribution systems, the inaccuracy of input data contributes greatly to the inaccuracy of system state estimates calculated from them. It is important, therefore, that the system operators are given not only the values of flows and pressures in the network at any instant of time but also that they have some indication of how reliable these values are. The quantification of the inaccuracy of calculated flows and pressures caused by the input data uncertainty is called confidence limit analysis. Bargiela and Hainsworth presented several confidence limit analysis algorithms. These include a Monte Carlo simulation method, an optimization method, and a sensitivity matrix technique. The performance of

these algorithms is assessed in terms of their suitability for real-time control or design stage applications.

Lansey et al. (1989) presented a constrained model for the minimum cost design of water distribution networks. Their methodology attempted to account for the uncertainties in required demands, required pressure heads, and pipe roughness coefficients. They formulated an optimization problem as a nonlinear programming model which is solved using a generalized reduced gradient method. Their results show that uncertainties in future demands, pressure head requirements, and pipe roughness can have significant effects on the optimal design and cost. They observed that the cost versus reliability relationship is convex, which means an incremental amount at a higher reliability level will result in a greater increase in the system cost than for an incremental change at a lower level.

Quimpo and Shamsi (1991) developed a strategy for prioritizing decisions for the maintenance of a water distribution system. Using component and network reliability based on time-varying connectivity concepts, the probabilities that the water will be available at demand points in the system are calculated to determine a reliability surface. At any time, this surface is used to locate low reliability areas, which identify parts of the system that need maintenance priority. The specific components that must be repaired or replaced are determined using a component importance criterion that measures the overall effect of component maintenance on the system reliability.

Mays (1993) computed the reliability of the water distribution system by treating the demand, pressure head and pipe roughness as random variables. He assumed that the

water demand and pipe roughness coefficient follows a probability distribution, and then he used a random number generator to generate the values of random variables for each node and each pipe. Then he performed hydraulic simulation and computed the pressure heads at the demand nodes, provided that the demands are satisfied. Then finally he computed nodal and system hydraulic reliabilities.

Calvin et al. (1996) investigated capacity reliability which is defined as the probability that the carrying capacity meets the flow demand. They described the use of capacity reliability for networks with more than one demand node through finding the probability of a feasible flow given probability distributions of flow capacities in the pipes and fixed nodal demand. The solution procedure generates a set of inequalities that represents a necessary and sufficient condition for feasible flow. They proposed a solution procedure for evaluating the probability that all the inequalities are satisfied by eliminating redundant inequalities and by determining bounds for the probability of feasible flow. They developed a decision-making framework that applies both the capacity reliability measure and the solution approach for maintenance and rehabilitation decision making.

Xu and Goulter (1996) applied three uncertainty analysis techniques, derived from inventory theory, probability theory and Fuzzy logic theory to examine the effects of these uncertainties on the results of hydraulic modeling. They examined the effects of uncertainty and imprecision in nodal demands and the values of pipe parameters on the output of simulation studies of water distribution networks. They performed hydraulic analysis associated with the uncertainty analysis by an approximate and computationally

efficient hydraulic model developed by linearization of the non-linear hydraulic equations at selected points.

Kumar et al. (1996) proposed an analytical approach to compute the probabilistic reliability measures for a water distribution network under drought induced failure conditions. According to them, in a water distribution network water supply and demand are both driven by the variability of climatic conditions such as rainfall and temperature. Traditionally, a water distribution network is designed according to certain guidelines which represent reliability considerations. Most of the present studies related to hydraulic reliability of a water distribution network have assumed that supply is always greater than or equal to demand. In developing countries, water supply to the consumer is quite often less than the actual requirement and is also negatively correlated. Even when the supply available is more than the demand, the system may not satisfy the requirement because of the pressure or capacity constraint.

Quimpo (1996) suggested restoration maintenance prioritization using reliability measures as management tools. His suggestion was based on the fact that the aging and deterioration of water supply infrastructure have resulted in performance deficiencies. Limited budgets require that resources for maintenance and rehabilitation be allocated in the most efficient manner. If resources are insufficient to correct all the deficiencies, restoration maintenance must be prioritized. He examined the developments in the field of restoration maintenance and explained some of the approaches, and discussed some of the conceptual and computational difficulties encountered. He concluded that if a polynomial time algorithm is successfully developed then it will facilitate the reliability analysis of

large water distribution systems and encourage water authorities to adopt the resulting decision making tool for infrastructure assessment and rehabilitation.

Xu and Goulter (1997) presented a framework for hydraulic reliability analysis of water distribution networks based on the first order reliability method. The method is capable of taking into consideration the stochastic nature of demands and random values of pipe roughness coefficients. They developed an efficient algorithm using simulation and sensitivity analysis techniques to identify the failure point. They showed that their proposed method is able to accurately determine the hydraulic reliability for a wide range of nodal demands and pipe roughnesses.

Goulter (1999) proposed a conceptual framework for consideration and management of risk and reliability in urban water supply systems. His conceptual framework draws on developments in the electricity supply industry for management of reliability and uses economic principles rather than engineering standards to define the optimal reliability for a system. The application of these economic principles involves use of “total societal cost”, as defined by the sum of the cost to the utility of supplying the water at specified levels of reliability of service and the cost to the customers of that level of service, as the means of identifying the optimal level of reliability for a system. His proposed framework also incorporates aspects of the value of service (VOS) and the cost of service (COS) concepts which differentiate between the cost of providing a level of reliability (service) and the value to the customers of that level of reliability (service).

Tanyimboh et al. (1999) highlighted the important role of pressure-driven simulation in the management of water distribution systems. They described two cases for

a real water distribution system of a big residential area. One case involves the simulation of the effects of the failure of a major system component. The other case is concerned with the capacity of a distribution system with reference to growing demands over a planning horizon spanning two decades. The examples considered demonstrate the primacy of pressures in water distribution systems and highlight some of the shortcomings of demand-driven methods for analyzing water distribution systems. Tanyimboh et al. also mentioned new developments in system performance and reliability assessment brought about by recent research in pressure-dependent network analysis. Much of the reliability analysis research to date has been based on the concept of average demands. In reality, however, demand exhibits both diurnal and seasonal variations. The authors also mentioned possible ways of incorporating these variations into reliability analysis models, to enhance the value of these models as an important tool for the management of water distribution system.

Ezell et al. (2000) developed the Infrastructure Risk Analysis Model (IRAM) to perform risk analysis of water distribution systems. According to IRAM, first the water distribution is decomposed along the dimensions of function, component, structure, state, and vulnerability, while considering other perspectives such as political, temporal and economic. Component vulnerability is subjectively assessed in terms of exposure and access. Based on vulnerability analysis and expert opinion, a willful water contamination attack scenario is developed and modeled using an event tree. Expected and extreme risks are then measured using exceedence probability. Lastly, alternatives are generated and the results are presented in a multi-objective framework.

Shinstine et al. (2000) performed reliability analyses on two large scale water distribution systems. They defined reliability as the probability of satisfying nodal demands and pressure heads for various possible pipe failures (breaks) in the water distribution system. They linked an existing reliability model based on a minimum cut-set method to a steady-state simulation model that implicitly solves the continuity and energy equations. The results from the simulation model are used in the reliability model to define minimum cut-sets and determine the values of system and nodal reliability assuming deterministic conditions. A discrete failure relationship is used with absolute failure if pressure heads fall below a prescribed minimum. Comparisons of results illustrate the similarities and differences in the design of each system under varying operating conditions.

Ostfeld (2001) developed tailor-made reliability methodology for the reliability assessment of regional water distribution and applied it to the regional water distribution system. Hydraulic reliability refers directly to the basic function of a water distribution system: conveyance of desired water quantities at desired pressures to desired appropriate locations at desired appropriate times. Furthermore, the straightforward way to evaluate the hydraulic reliability of a water distribution system is through stochastic simulation, therefore Ostfeld adopted a methodology comprising of the following two interconnected stages:

1) Analysis of storage/conveyance properties of the system, and

2) Implementation of stochastic simulation through use of the software "US Air Force Rapid Availability Prototyping for Testing Operational Readiness (RAPTOR).

Tyagi and Haan (2001) developed Generic Expectation Functions (GEF) as a function of means and coefficient of variations of input random variables with different probability distributions by considering a power function and taking higher order moments of it about the origin. They used GEF to calculate the risk which was defined as the probability of failure of a storm sewer system by calculating expectations of the input random variables.

Shinstine et al. (2002) applied reliability models based on the minimum cut-set method to large scale municipal water distribution systems and examined the reliability levels that engineers implicitly design into their systems.

Aklog and Hosoi (2003) introduced a new reliability-based optimal design formulation and examined the effect of specifying minimum allowable pipe sizes during least-cost designs on system reliability. They estimated system reliability using the minimum cut-set method. They used a pressure-driven network simulation model to determine the actual supply at each demand point when a component fails. Their results show that the new model preserves loops and results in a system with better reliability.

The current study for calculating nodal and system reliability of water distribution systems is based on the minimum cut-set method. In this study, four input parameters (pipe roughness, junction water demand, tank and reservoir water level required for modeling the water distribution system) are considered as stochastic or random variables. In the literature review, different approaches for calculating nodal and system reliability

were presented, but no one has considered the above mentioned four model input parameters as random variables. Although Mays (1993) computed the reliability of the water distribution system by treating the demand, pressure head, and pipe roughness as random variables, he did not consider tank and reservoir water level. Also, in this study, a new method is introduced for calculating the pipe failure probability using Generic Expectation Function (GEF), which takes into account randomness in pipe roughness, pipe diameter, number of breaks in the pipe, and repair and replacement costs of the pipe. Previously, Shinstine et al. (2002) had applied reliability models based on the minimum cut-set method and calculated pipe failure probabilities based on the Poisson method which only takes into account the annual number of breaks in the pipe assuming deterministic conditions.

2.3 Hydraulic Simulation Models

In order to quantify water distribution system reliability, hydraulic simulation of the system is required. Several commercial simulation models are available, but in this study EPANET® (2000) will be used to perform hydraulic simulation. It was selected because it fulfills our requirement of calculating nodal pressures, and also its source code is available free of cost in the public domain, and can be applied to large water distribution networks with unlimited pipe numbers. EPANET® uses the same numerical engines as WaterCAD® (2001) and other commercially available software and therefore results of hydraulic simulation obtained from them are expected to be similar. Although it lack features such as links to SCADA systems and GIS, for ordinary hydraulic modeling it is suitable to use EPANET®.

2.3.1 EPANET

EPANET® performs extended period simulation of hydraulic and water quality behavior within pressurized pipe networks. A network consists of pipes, nodes (pipe junctions), pumps, valves and storage tanks or reservoirs. EPANET® 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.

Running under Windows, EPANET® provides an integrated environment for editing network input data, running hydraulic and water quality simulations, and viewing the results in a variety of formats. These include color-coded network maps, data tables, time series graphs, and contour plots.

EPANET® was developed by the Water Supply and Water Resources Division (formerly the Drinking Water Research Division) of the U.S. Environmental Protection Agency's National Risk Management Research Laboratory.

Full-featured and accurate hydraulic modeling is a prerequisite for doing effective water quality modeling. EPANET® contains a state-of-the-art hydraulic analysis engine that includes capabilities such as placing no limit on the size of network that can be analyzed, and computing friction headloss using either Hazen-Williams, Darcy-Weisbach, or Chezy-Manning equations. It includes minor headlosses for bends, fittings, etc; models constant or variable speed pumps; computes pumping energy and cost; models various types of valves including shutoff, check, pressure regulating, and flow control valves;

allows storage tanks to have any shape (i.e., diameter can vary with height); considers multiple demand categories at nodes, each with its own pattern of time variation; models pressure-dependent flow issuing from emitters (sprinkler heads); and can base system operation on both simple tank level or timer controls and on complex rule-based controls.

EPANET's hydraulic simulation model computes hydraulic heads at junctions and flow rates through links for a fixed set of reservoir levels, tank levels, and water demands over a succession of points in time. From one time step to the next, reservoir levels and junction demands are updated according to their prescribed time patterns, while tank levels are updated using the current flow solution. The solution for heads and flows at a particular point in time involves solving simultaneously the conservation of flow equation for each junction and the headloss relationship across each link in the network. This process, known as hydraulically balancing the network, requires using an iterative technique to solve the nonlinear equations involved. EPANET® employs the Gradient Algorithm for this purpose.

The hydraulic time step used for extended period simulation (EPS) can be set by the user. A typical value is 1 hour. Shorter time steps than normal will occur automatically whenever one of the following events occurs: the next output reporting time period occurs, the next time pattern period occurs, a tank becomes empty or full then a simple or rule-based control is activated.

Chapter 3

DEVELOPED METHODOLOGY

3.1 General

The methodology adopted for performing nodal and system reliability analysis is presented in the flow chart shown in Figure 3.1. According to the developed methodology, the first step is to select the random variables and random pipe closure combinations for the modeled water distribution network. Next, random variables are generated by adopting either a Monte Carlo simulation or the Bootstrapping method. Then hydraulic simulation is performed for all selected combinations of pipe closures by considering all generated random variables. Pipe failure probabilities are then calculated by each of the following methods: Poisson and Generic Expectation Function (GEF). Finally, nodal and system reliabilities are calculated.

The methodology adopted for calculating nodal and system reliability is based on the "Minimum Cut-Set" and is presented in the form of a flowchart as shown in Figure 3.2.

3.2 Random Variables Selection

Four input parameters of the water distribution system model are selected as random variables, as follows:

1. Pipe roughness coefficients
2. Nodal demand
3. Tank water level

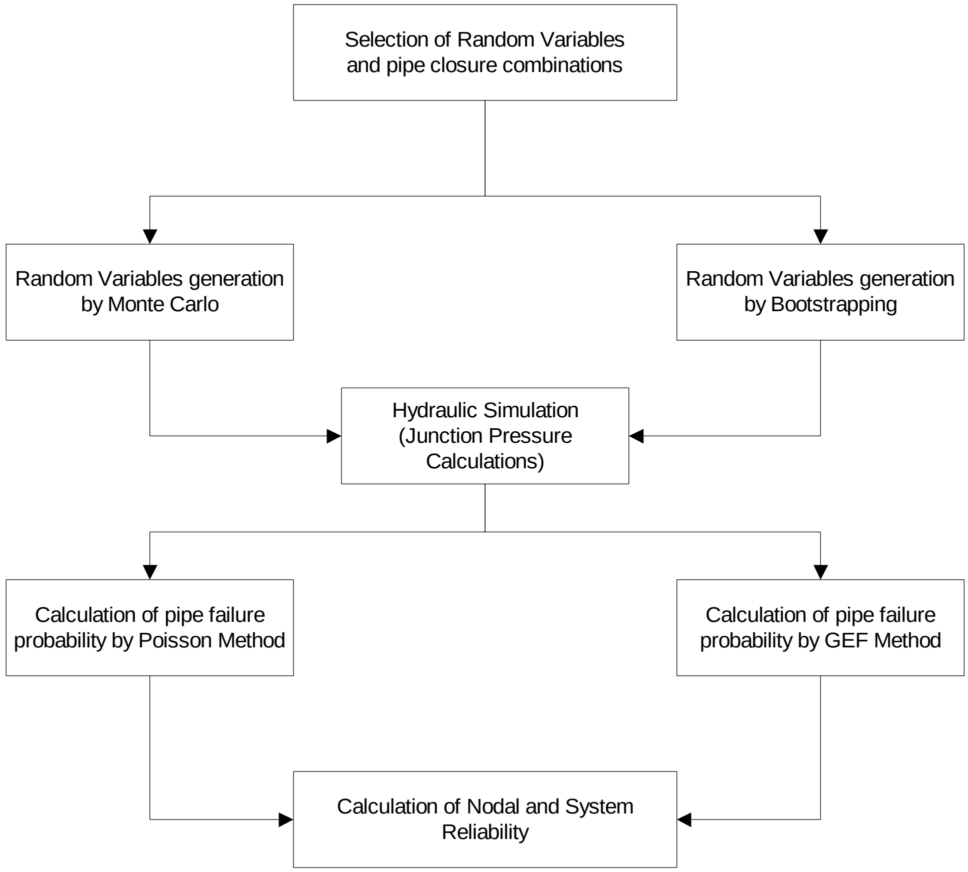


Figure 3.1: Flow chart of the developed methodology

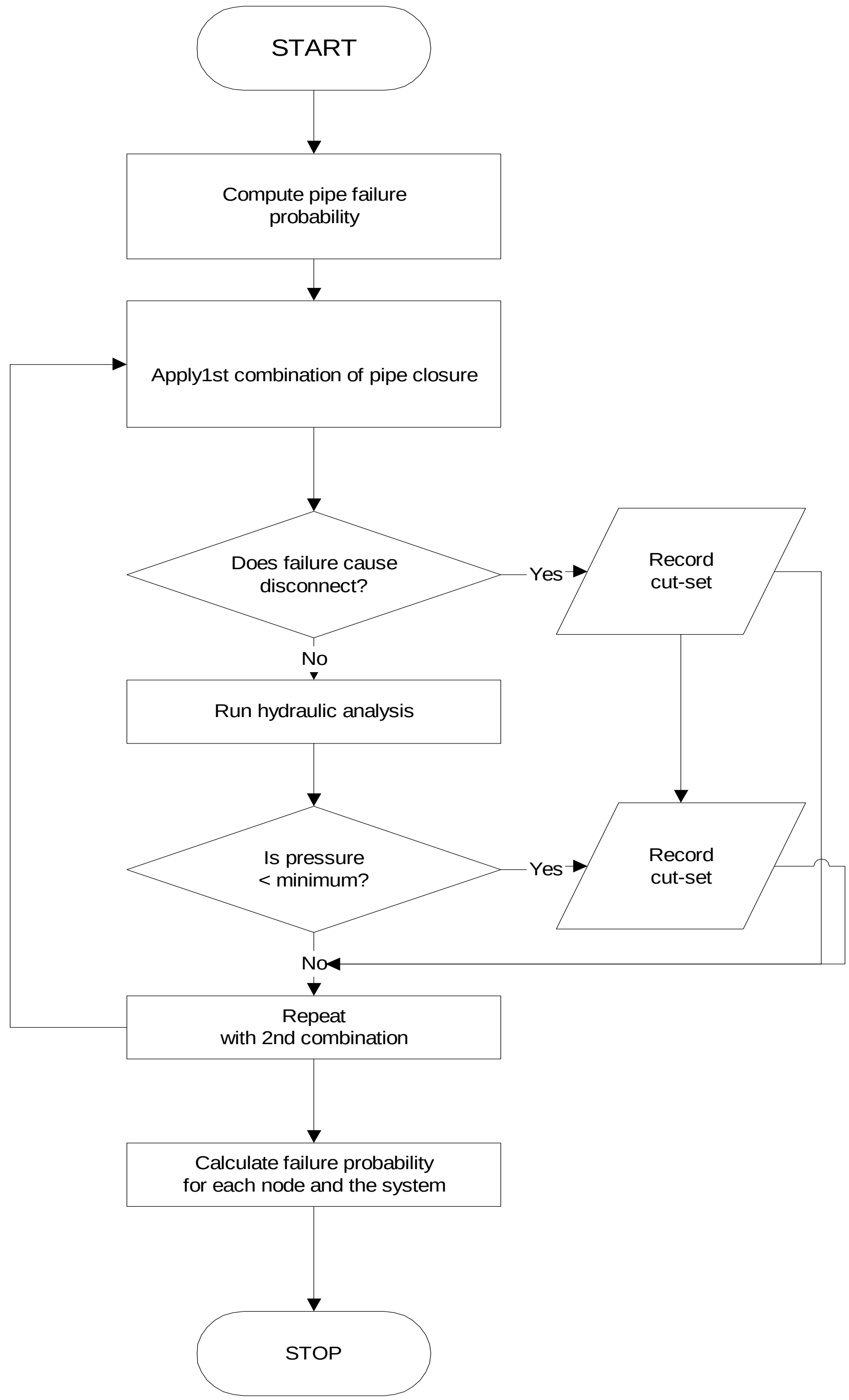


Figure 3.2: Minimum Cut Set Reliability flow chart

4. Reservoir water level

The random values are generated for the above selected random variables by Monte Carlo and Bootstrapping methods. The methodologies for generating them are explained in Sections 3.4 and 3.5 respectively.

3.3 Pipe Closure Combinations Selection

The pipe closure combinations are randomly selected using the computing software MATHEMATICA® (1999) by considering all pipes in the modeled water distribution network. The nodal pressures are calculated by EPANET® (2000) for cases when all pipes are open, and all combinations of pipe closures are used in calculating the nodal and system reliability by the "Minimum Cut-Set" method. This will be explained in Section 3.8.

3.4 Random Variables Generation by the Monte Carlo Method

In recent years, simulation techniques have been applied to many problems in the various sciences, and if the processes which are being simulated involve an element of chance, these techniques are referred to as Monte Carlo methods (Johnson, 1994). Monte Carlo simulation generates random values for uncertain variables over and over to simulate a model.

In the Monte Carlo method, for each random variable the possible values are defined with probability distribution, and Monte Carlo simulation generates random values corresponding to the probability distribution of each random variable.

In order to generate random values by the Monte Carlo Method, the probability distribution functions of the selected random variables (which are pipe roughness

coefficients, nodal demand, tanks and reservoirs water level) are assumed. One thousand random values of the selected random variables are generated assuming normal distribution. These random values are generated by using the statistical analysis package STATISTICA® by inputting the corresponding mean, standard deviation and probability distribution function of each random variable.

3.5 Random Variables Generation by Bootstrapping

The bootstrap method is a computer intensive method like Monte Carlo used frequently in applied statistics. The bootstrap method is a type of Monte Carlo method applied based on observed data (Efron and Tibshirani 1993). The bootstrap method was described by Efron (1979) and since then he has written much about the method and its generalizations. The bootstrap method can be used for several purposes such as robust estimation of sample variances or standard errors and (asymmetrical) confidence intervals. It has found use in the estimation of model selection frequencies and a variety of other applications.

Bootstrap refers to a method for estimating confidence intervals for a statistic such as population mean, by re-sampling a data set with replacements to form new data sets called bootstrap samples (U.S EPA 1999)**.**

In order to generate random values by Bootstrapping, the random values (bootstrapped samples) of selected random variables are generated from samples assigned to each random variable. One thousand random values of the selected random variables are generated by using the package MATLAB®.

3.6 Pipe Network Analysis

Pipe network analysis seeks to determine the discharge and pressure at every node. To accomplish this, the physical features of the network must be known. These features include pipe diameter, length, and roughness, as well as the location of the reservoir, pumps, pressure reduction valves, and fittings.

Walski (1984) defines two types of pipe networks, namely looped and un-looped, depending on whether flow in each pipe can be determined without solving the energy equation. According to Walski (1984) if the head is specified only at the single point then flow in each pipe can be determined without solving the energy equation. However, if the head is specified at more than one location then the energy equation needs to be solved, as flow in the pipe can be determined by applying the energy equation between the junctions connecting the pipe.

3.6.1 Definition of Pipe Network

In practice, pipe networks consist not only of pipes, but also miscellaneous fittings, services, storage tanks and reservoirs, meters, regulating valves, pumps, and electronic and mechanical controls. For modeling purposes, these system elements are most commonly organized into three fundamental categories:

1. Junction Nodes: Junctions are specific points (nodes) in the system at which an event of interest is occurring. This includes points where pipes intersect, major demands on the system (such as a large industry, a cluster of houses, or a fire hydrant), or critical points in the system where pressures are important for analysis purposes.

2. Boundary Nodes: Boundaries are nodes in the system of known hydraulic grade which define the initial hydraulic grades for any computational cycle. They form the

baseline hydraulic constraints used to determine the condition of all other nodes during system operation. Boundary nodes are elements such as tanks, reservoirs, and pressure sources.

3. Links: Links include pipes, pumps, and various valves. These are system components which connect to junctions or boundaries, and control the flow rates and energy losses (or gains) between nodes.

3.6.2 Governing Principles

The fundamental equations that govern flow in pipe line systems are: the Conservation of Mass or continuity equation and the Conservation of Energy equation.

a) Conservation of Mass (Continuity Equation)

According to this principle, at any node in the system under incompressible flow conditions, the total volumetric or mass flows in must equal the flows out (less the change in storage). Separating these into flows from connecting pipes, demands, and storage, we obtain:

$$\Sigma Q_{in} \Delta t = \Sigma Q_{out} \Delta t + \Delta V_s \tag{3.1}$$

where Q_{in} = Total flow into the node,

Q_{out} = Total demand at the node,

ΔVs = Change in storage volume,

Δt = Change in time.

b) Conservation of Energy

The conservation of energy is also simple: conceptually the head losses through the system must balance at each point. For pressure networks, this means that the total head loss between any two nodes in the system must be the same regardless of what path is taken between the two points. The head loss must be sign consistent with the assumed flow direction (i.e. gain head when proceeding opposite the flow and lose head when proceeding with the flow).

3.6.3 Friction Losses in Pipes

There are many equations that approximate friction losses associated with the flow of a liquid through a pressure pipe. The three most commonly used methods are as follows:

i) Hazen-Williams Equation

ii) Manning's Equation

iii) Darcy-Weisbach Equation

i) Hazen-Williams Equation

The Hazen-Williams equation is the most frequently used in the design and analysis of pressure pipe systems for water distribution. The equation was developed experimentally and therefore should not be used for fluids other than water (within temperatures normally experienced in potable water distribution systems).

The Hazen-Williams equation includes a roughness factor, C, which is constant over a wide range of (turbulent) flows. Mathematically, it is expressed as

$$h_f = \frac{3.02L}{D^{1.16}}\left(\frac{V}{C}\right)^{1.85} \text{(English Units)} \quad (3.2)$$

where h_f = Head loss (feet)

L = Pipe length (feet)

D = Pipe diameter (feet)

V = Velocity (ft/sec)

C = Hazen-Williams friction coefficient

ii) Manning's Equation

The Manning's Equation is most frequently used in the analysis of water flow in open channels, but can be applied to water flow in closed conduits as well. The resistance component of this equation includes a factor n, which is generally a function of pipe material and condition. The Manning's Equation is expressed as:

$$h_f = \left(\frac{n.V}{1.49.R_H^{2/3}}\right).L \quad \text{(English Units)} \quad (3.3)$$

where h_f = Head loss (feet)

L = Pipe length (feet)

R_H = Hydraulic Radius = Area/Wetted Perimeter (feet)

V = Velocity (ft/sec)

n = Manning's friction coefficient

iii) Darcy-Weisbach Equation

The Darcy-Weisbach Equation is a theoretically-based equation for use in the analysis of pressure pipe systems. It is a general equation that applies equally well to any

flow rate and any incompressible fluid. The Darcy-Weisbach equation is expressed as:

$$h_f = f \frac{L}{D}\frac{V^2}{2g} \quad \text{(English Units)} \tag{3.4}$$

where h_f = Head loss (feet)

f = Pipe friction factor

L = Pipe length (feet)

D = Pipe diameter (feet)

V = Velocity (ft/sec)

g = Acceleration due to gravity (ft/s^2)

3.6.4 Modes of Analysis

a) Steady State Network Hydraulics

Steady state analyses determine the operating behavior of the system at a specific point in time, or under steady-state (unchanging) conditions. This type of analysis can be useful for determining short term effects on the system due to fire flows or average demand conditions.

For this type of analysis, the network equations are determined and solved with tanks being treated as fixed grade boundaries. The results that are obtained from this type of analysis are instantaneous values, and may or may not be representative of the values of the system a few hours or even a few minutes later in time.

b) Extended Period Simulation

When the effects on the system over time are important, an extended period simulation is appropriate. This type of analysis allows the user to model tanks filling and draining, regulating valves opening and closing, and pressures and flow rates changing throughout

the system in response to varying demand conditions and in response to automatic control strategies formulated by the modeler.

While a steady state model may tell whether or not the system has the capability to meet a certain average demand, an extended period simulation indicates whether or not the system has the ability to provide acceptable levels of service over a period of minutes, hours, or days. Extended period simulations can also be used for energy consumption and cost studies, as well as water quality modeling.

Data requirements for extended period simulations are greater than for steady state runs. In addition to the information required by a steady state model, the user also needs to determine water usage patterns, more detailed tank information, and operational rules for pumps and valves.

3.6.5 Hydraulic Simulation

The hydraulic model EPANET® (Rossman, 2000) is used to perform the hydraulic simulation of the water distribution system as explained previously in Section 2.4.6.

The hydraulic simulation is performed on the modeled water distribution system by considering two scenarios, taking each random variable independently and taking all of them collectively. The simulation is performed for all combinations of pipe closures and for both steady and extended period conditions. For example, when only pipe roughness is considered as a random variable then hydraulic simulation is performed for all one thousand generated data sets of pipe roughness values, and for each set of pipe roughness values all pipe closure combinations are considered along with the case when all pipes are open. While considering pipe roughness as the only random variable, all other input variables are taken as deterministic values. A similar procedure is applied to all other

selected random variables independently, and finally all variables are considered as random variables when executing the hydraulic model.

Since the hydraulic simulation model EPANET® (2000) is a windows based version, it can not handle input data sets iteratively. Therefore MS Visual C++ computer code of EPANET® (2000) along with another code written in MS Visual C++ is used for applying the above methodology.

3.7 Calculation of Pipe Failure Probability

A final step prior to the application of the reliability model is to calculate the pipe failure probability for all of the pipes considered in the cut-sets. The adopted minimum cut-set method of calculating nodal and system reliability is described in Section 3.8.
The pipe failure probabilities are calculated by two different methods: Poisson method and Generic Expectation Function Method (GEF), and finally the nodal and system reliability are calculated by both of these methods.

3.7.1 Poisson Method

Goulter and Coals (1986) and Su et al. (1987) used the following method to determine the probability of failure of individual pipes. The probability of failure of pipe *i*, P*i*, is determined using the Poisson probability distribution.

$$P_i = 1 - e^{-\beta_i} \tag{3.5}$$

where r_i = Expected number of failures per year per unit length of pipe *i*

L_i = Length of pipe *i*

and

$$\beta_i = r_i L_i = \text{Expected number of failures per year for pipe } i \qquad (3.6)$$

3.7.2 Generic Expectation Function Method (GEF)

Tyagi and Haan (2001) used Generic Expectation Functions for calculating first and higher order moments of an output random variable defined by multiplicative, additive, and combination models. These functions are straightforward to develop and apply.

Most hydrologic and hydraulic engineering problems are based on derived equations that involve several uncertain parameters that may be difficult to quantify accurately. Further, an equation g ($\underline{X}$), may have different degrees of nonlinearity with respect to its uncertain parameters represented as an array $\underline{X}$. Bates (1988) and Stevens (1993) indicated that the predictors for nonlinearity developed so far work well only in specific applications, and that no well-accepted, generalized nonlinearity measure is available. Uncertainty, risk and reliability analysis of a system defined by a number of functional forms can be analyzed using generic expectation functions of individual component functions of input parameters.

3.7.2.1 Considered Functional Forms and Their Moments about Origin

Multiplicative models are frequently encountered in hydrological and hydraulic studies and most of them are of multiplicative form. Examples are flow over control structures such as weirs, spillways, overfalls and sluices (Haan et al 1994), channel control equations such as Manning's equation, and pipe flow resistance equations such as Hazen -Williams and Darcy-Weisbach equations. Tung and Mays (1980), Lee and Mays (1986), and Tung (1990) present examples of an uncertainty analysis of multiplicative

forms encountered in hydraulic/hydrologic systems. In this form, the output random variable *Y* is calculated by multiplication of *N* power functions

$$Y = C_0 \prod_{i=1}^{N} X_i^{ri} \tag{3.7}$$

where C_o and r_i = constants ; and X_i = *N* independent stochastic input random variables. The k_{th} moment of *Y*, about the origin, μ'_k, is defined by Haan (1977) as

$$\mu'_k = E\,[Y^k] = C^k{}_0 \prod_{i=1}^{N} E[X_i^{kri}] \tag{3.8}$$

where *E* [ ] = expectation operator. The general additive form is given as

$$Y = \sum_{j=1}^{M} W_j \tag{3.9}$$

where *M* = number of additive terms $W_{j.,}$ which is defined either by a multiplicative form such as equation (3.7) or by an individual power function. Mathematically, $W_{j.,}$ is given as

$$W_j = C_j \sum_{i=1}^{N} X_i^{ri} \tag{3.10}$$

The k_{th}–order statistical moment of *Y* about the origin can be obtained by expanding the expression Y^k and then taking its expectation. For $Y = W_1 - W_2$, the first four moments of *Y* about the origin are given as

$$E[Y] = E[W_1] - E[W_2] \tag{3.11}$$

$$E[Y^2] = E[W_1^2] - 2E[W_1]E[W_2] + E[W_2^2] \tag{3.12}$$

$$E[Y^3] = E[W_1^3] - 3E[W_1^2]E[W_2] + 3E[W_1]E[W_2^2] - E[W_2^3] \tag{3.13}$$

$$E[Y^4] = E[W_1^4] - 4E[W_1^3]E[W_2] + 6E[W_1^2]E[W_2^2] - 4E[W_1]E[W_2^3] + E[W_2^4] \tag{3.14}$$

3.7.2.2 Central Moments and Distributional Characteristics

The k_{th} central moment of Y, μ_k, can be obtained using the following equation by Haan (1977) as

$$\mu_k = E[(Y - \mu_Y)^k] = \sum_{i=0}^{k} (-1)^i \binom{k}{i} \mu_Y^i \mu'_{k-i} \tag{3.15}$$

where μ_Y = Mean of Y, given by

$$\mu_Y = E[Y] \tag{3.16}$$

Equation (3.15) shows that central moments of Y of any order k can be obtained if expectations of individual power functions are known.

In most situations, distributional properties of a random variable can be characterized in terms of its mean, variance, coefficient of skewness, and coefficient of kurtosis. The variance of Y, σ^2_Y, is defined as the second moment about the mean. Substituting $k = 2$ into equation (3.15), σ^2_Y is given as

$$\mu_2 = \sigma_Y^2 = E[Y^2] - \mu_Y^2 \tag{3.17}$$

where μ_2 = second moment of Y about the mean. The coefficient of skewness of Y, γ_Y, is defined by Haan (1977) as

$$\gamma_Y = \frac{\mu_3}{\mu_2^{3/2}} \tag{3.18}$$

where μ_3 = third moment of Y about the mean, which can be obtained by substituting $k = 3$ into equation (3.15) as

$$\mu_3 = E[Y^3] - 3\mu_Y E[Y^2] + 2\mu_Y^3 \tag{3.19}$$

The kurtosis of *Y*, κ_Y, is defined by Haan (1977) as

$$k_Y = \frac{\mu_4}{\mu_2^2} \tag{3.20}$$

where μ_4 = forth moment of *Y* about the mean, which can be obtained by substituting k = 4 into equation (3.15) as

$$\mu_4 = E[Y^4] - 4\mu_Y E[Y^3] + 6\mu_Y^2 E[Y^2] - 3\mu_Y^4 \tag{3.21}$$

3.7.2.3 Determination of Risk and Reliability

It is better to measure the reliability of a system in terms of probability. The failure of a system can be considered as an event whereby the demand or loading, *L*, on the system exceeds the capacity or resistance, *R*, of the system so that the system fails to perform satisfactory for its intended use. The objective of reliability analysis is to ensure that the probability of event ($R > L$) throughout the specified useful life is acceptably small. To study this event, a performance function, *Z*, is defined as (Ang and Tang, 1984;Tung, 1990; Mays and Tung, 1992)

$$Z = R - L \tag{3.22}$$

The risk is defined as the probability of failure of the system, which can be written as

$$P_f = P(Z < 0) = \int_{-\infty}^{0} p_Z(z)dZ \tag{3.23}$$

where P_f = probability of failure; P = probability operator; and $p_z(z)$ = probability density function of Z. The reliability of the system, R , can be written as

$$R = P\ (Z>0) = 1\text{-}P_f \tag{3.24}$$

Equation (3.22) is an additive form with two functions, R and L. To determine moments of Z, the moments of individual functions are determined first, and then the first four moments of the overall form of Z about the origin are determined using equations (3.11), (3.12), (3.13) and (3.14).

3.7.2.4 Development of Generic Expectation Functions

Consider a power function

$$Y = X^r \tag{3.25}$$

The k_{th} order moment of Y about the origin can be obtained as

$$\mu_k' = E[Y^k] = E[X^{kr}] = \int_{-\infty}^{\infty} X^{kr} p_X(x)dX \tag{3.26}$$

where $p_x(x)$ = probability density function of X.

The expectations can be calculated assuming different probability density functions like Uniform, Triangular, Lognormal, Gamma, Exponential and Normal Distribution as summarized in Table 3.1.

Lets calculate expectations of Triangular and Gamma distributions.

3.7.2.5 Triangular Distribution

The probability density function $p_x\ (x)$ for any triangular distribution is

Table 3.1: r^th Moment about Origin with respect to probability distribution

Probability Distribution	**E[X^r]**
Uniform	$\frac{\mu_x^r}{2\sqrt{3}(r+1)CV_x}\left[\left(1+CV_x\sqrt{3}\right)^{r+1}-\left(1-CV_x\sqrt{3}\right)^{r+1}\right]$
Triangular	$E[X^r]=\frac{\mu_x^r}{6(r+1)(r+2)CV_x^2}\left[\left(1+CV_x\sqrt{6}\right)^{r+2}+\left(1-CV_x\sqrt{6}\right)^{r+2}-2\right]$
Lognormal	$\mu_x^r(1+CV_x^2)^{r(r-1)/2}$
Gamma	$\mu_x^r CV_x^{2r}\exp\left\{\ln\left[\Gamma(CV_x^{-2}+r)\right]-\ln\left[\Gamma(CV_x^{-2})\right]\right\}$
Exponential	$\mu_x^r\Gamma(r+1)$
Normal	$\mu_x^r\left[1+\frac{r(r-1)}{2}CV_x^2+\ldots+\frac{r(r-1)(r-2)\ldots(r-n+1)}{2^{n/2}(n/2)!}CV_x^n+\ldots\ldots\right]$

μ_x = Mean

CV_X = Coefficient of Variation

Γ = Gamma Function

$$p_X(x) = \frac{2}{b-a}\frac{(X-a)}{(c-a)} \quad \text{when } a \le X \le c \tag{3.27}$$

$$p_X(x) = \frac{2}{b-a}\frac{(b-X)}{(b-c)} \quad \text{when } c \le X \le b \tag{3.28}$$

where a, b, c = minimum, maximum, and mode values of X. These parameters can be obtained by the following equation given by Tyagi (2000) as :

$$\underline{\mathbf{a}} = \mu_X \left\{ 1 + 2\sqrt{2} CV_X \cos\left[\frac{2\pi n}{3} + \frac{1}{3}\cos^{-1}\left(\frac{5}{2\sqrt{2}} \gamma_X \right) \right] \right\} \tag{3.29}$$

where $\underline{\mathbf{a}}$ = vector containing b, a, and c, which can be obtained by substituting n = 0, 1, and 2, respectively into equation (3.29); and γ_x = coefficient of skewness of X. Using equations (3.23), (3.24) and (3.25), the $E\,[X^r]$ is given by Tyagi (2000) as :

$$E[X^r] = \frac{2\left[(b-c)a^{r+2} + (c-a)b^{r+2} + (a-b)c^{r+2} \right]}{(r+1)(r+2)(b-c)(c-a)(b-a)} \tag{3.30}$$

For a symmetrical triangle, γ_x = 0, and the parameters a, b, and c can be obtained corresponding to n = 1, 0, and 2. The obtained c is the μ_X, and the parameters a and b are the same as those obtained using the method of moments. The estimates of a and b are given as

$$\hat{a} = \mu_X (1 - \sqrt{6} CV_X) \tag{3.31}$$

$$\hat{b} = \mu_X (1 + \sqrt{6} CV_X) \tag{3.32}$$

Using equations (3.26), (3.27), (3.28), (3.31), and (3.32), the $E\,[X^r]$ is given by Tyagi (2000) as:

$$E[X^r] = \frac{\mu_X^r}{6(r+1)(r+2)CV_X^2}\left[\left(1 + CV_X\sqrt{6}\right)^{r+2} + \left(1 - CV_X\sqrt{6}\right)^{r+2} - 2 \right] \tag{3.33}$$

3.7.2.6 Gamma Distribution

The gamma density function is given by

$$p_X(x) = \frac{\lambda^{\alpha} e^{-\lambda X} X^{(\alpha-1)}}{\Gamma(\alpha)}\text{; where } X, \alpha, \text{ and } \lambda > 0 \tag{3.34}$$

where α and λ = distribution parameters. Using the method of moments, α and λ are expressed by Haan (1977) as:

$$\hat{\lambda} = \frac{\mu_X}{\sigma_X^2} = \frac{1}{\mu_X CV_X^2} \tag{3.35}$$

$$\hat{\alpha} = \frac{\mu_X^2}{\sigma_X^2} = \frac{1}{CV_X^2} \tag{3.36}$$

Substituting equation (3.34) into equation (3.26), the $E\,[X^r]$ is written as

$$E[X^r] = \frac{\lambda^{\alpha}}{\Gamma(\alpha)} \int_{-\infty}^{\infty} e^{-\lambda X} X^{(\alpha+r-1)} dX = \frac{\Gamma(\alpha+r)}{\lambda^r \Gamma(\alpha)} \tag{3.37}$$

Replacing α and λ in equation (3.37) by their estimates given in equation (3.35) and equation (3.36), equation (3.37) is rewritten as

$$E[X^r] = \frac{CV_X^{2r} \mu_X^r \Gamma(CV_X^{-2} + r)}{\Gamma(CV_X^{-2})} \tag{3.38}$$

For the convenience of calculations equation (3.38) can be expressed as

$$E[X^r] = \mu_X^r CV_X^{2r} \exp\left\{\ln\left[\Gamma(CV_X^{-2} + r)\right] - \ln\left[\Gamma(CV_X^{-2})\right]\right\} \tag{3.39}$$

3.7.2.7 Application of Generic Expectation Function (GEF)

Two events are statistically independent if the probability of any one of them is unaffected by the occurrence of the other (Lapin, 1997). According to the multiplication law for independent events, the probability that two independent events will both occur is simply the product of their probabilities (Johnson, 1994).

Mathematically, the probability of occurrence of two independent events A and B is given by

$$P(A \cap B) = P(A).P(B) \tag{3.40}$$

3.7.2.7.1 Calculation of failure probability [$P(A)$] of pipe to fulfill the demand

Assume that if the pipe is unable to satisfy the nodal demand is an event.

For pipes in water distribution systems, failure is assumed to occur when the flow in the pipe exceeds the capacity of the pipe. According to the Hazen-Williams equation the flow in the pipe is given by:

$$Q_p = 0.849 C_{HW} A R^{0.63} S^{0.54} \quad \text{(SI Units)} \tag{3.41}$$

where C_{HW} =Hazen-Williams coefficient

A = Pipe cross-sectional area (m^2)

R = Hydraulic Radius = Area/Wetted Perimeter (m)

S = Slope of Hydraulic Grade line

If the pipe is considered as flowing full, then

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

$$P = \pi d$$

By substituting the values of A and R in equation (3.38) we have,

$$Q_p = 0.27842 C_{HW} d^{2.63} S^{0.54} \tag{3.42}$$

The total flow directed into the pipe will be equal to pipe distribution factor multiplied by the demand at the junction. Mathematically, it is given by:

$$Q_D = D_p Q J_i \tag{3.43}$$

where D_p = Distribution factor of the pipe

Qj_i = Water Demand at Junction *i*

Therefore, performance function, *Z*, of the pipe can be defined as

$$Z = Qp - Q_D \tag{3.44}$$

The above equation (3.44) is used to find the failure probability [*P(A)*] of the pipe to fulfill the demand of the junction.

3.7.2.7.2 Calculation of pipe replacement probability [*P (B)*]:

Consider the break rate equation (Shamir and Howard 1979)

$$N(t) = N(t_0)e^{A(t-t_0)} \tag{3.45}$$

where *N(t)* = number of breaks per 1,000 feet length of pipe in year t;

t = time in years;

t_0 = base year for the analysis;

A = growth rate coefficient (1/year).

The threshold break rate Brk_{th} (Loganathan et. al, 2002) gives the critical break rate for optimal replacement of the pipe. Mathematically, it can be expressed as:

$$Brk_{th} = \frac{\ln(1+R)F_n}{C_{n+1}} \tag{3.46}$$

where, R = Discount Rate;

F_n = Replacement cost at time t_n;

C_{n+1} = Repair cost of (n+1) th break.

Equation (3.45) and equation (3.46) can be written as:

$$Z = Brk_{th} - N(t)$$

$$Z = \frac{\ln(1+R)F_n}{C_{n+1}} - N(t_0)e^{A(t-t_0)} \quad (3.47)$$

The above equation (3.47) is used to find the replacement probability [*P (B)*] of the pipe.

By using equation (3.44) and equation (3.47), we can find the "complete failure probability" of the pipe.

Mathematically, complete failure probability, P_{com}, can be written as:

$$P_{com} = P(A).P(B) \quad (3.48)$$

3.8 Calculation of Nodal and System Reliability

The proposed methodology for calculating Nodal and System Reliability is based on "Minimum Cut-Sets"' in which pipe roughness, nodal demands, tank and reservoir water levels are considered as random variables and finally "Minimum Cut-Sets Methodology" is applied to calculate Nodal and System Reliability. In order to explain this methodology, the discussion of the following is necessary.

3.8.1 Hydraulic Availability

"Hydraulic Availability" is defined by Cullinane et al. (1992) as the ability of the water distribution system to provide service with an acceptable level of interruption in spite of abnormal conditions The evaluation of hydraulic availability relates directly to the

basic function of the water distribution system, i.e., delivery of the specified quantity of water to a specific location at the required time under desired pressure.

Availability is evaluated in terms of developing a required minimum pressure. Pressures between 20 psi to 80 psi (Shinstine et al. 2000) are considered to be desirable pressures under normal daily demands.

Goulter and Coals (1986) proposed the use of a discrete relationship between availability and pressure, as shown in Figure 3.3. The availability during a time period *t* can be expressed by the following mathematical relationship:

$$\left.\begin{aligned} HAj &= 1 \ldots\ldots\ldots\ldots for Pj \geq PR \\ \\ HAj &= 0 \ldots\ldots\ldots\ldots for Pj < PR \end{aligned}\right\} \tag{3.49}$$

where,

HAj = Hydraulic Availability of node j

Pj = Pressure at node j

PR = Required Minimum Pressure

The network hydraulic availability is the product of the nodal hydraulic availabilities. With this approach, a zero availability index is assigned for all pressure values below the required minimum pressure. For example, if the required minimum pressure is set at 20 psi, a residual of 19.9 psi results in the same availability index as a

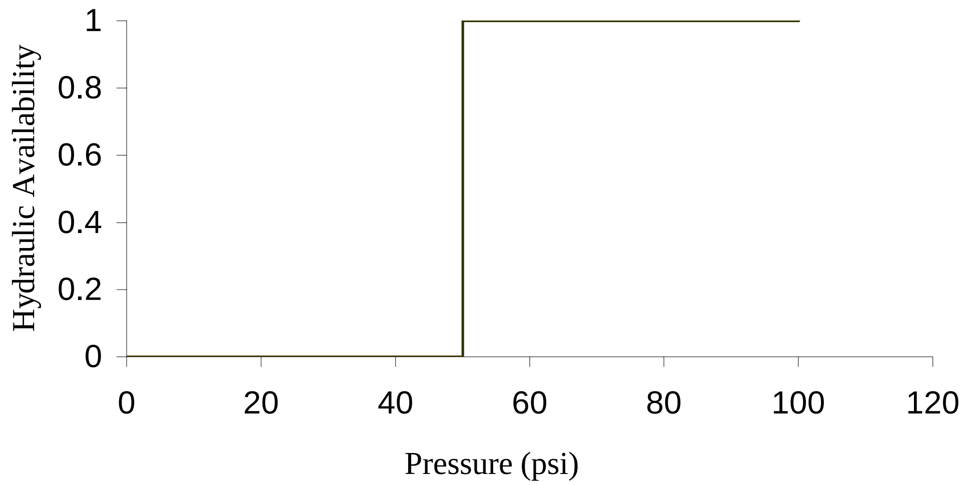


Figure 3.3: Hydraulic availability time step

residual pressure of 1 psi. Thus, the use of this discontinuous relationship does not adequately represent the engineering reality of the problem.

Cullinane et al. (1992) adopted a more appropriate representation that describes availability index as a continuous "fuzzy" function. Using a continuous function of this shape, a significant index value may be assigned to pressure values slightly less than the arbitrary assigned required minimum pressure value, *PR*

The shape of the curve is shown in Figure 3.4; it is similar to the cumulative normal distribution, which is mathematically stated as follows:

$$HAj = (PR \leq Pj)$$

$$HAj = \frac{1}{\sqrt{2\pi}} \int_{-\infty}^{\frac{(H-\mu_H)}{\sigma_H}} e^{-\frac{t^2}{2}} dt = P[\frac{(H-\mu_H)}{\sigma_H}] \qquad (3.50)$$

where,

Pj = Value of nodal pressure,

μ_H = Mean nodal pressure,

σ_H = Standard deviation of pressure

Values of "μ_H" and "σ_H" can be selected to adjust the position and shape of the function, respectively.

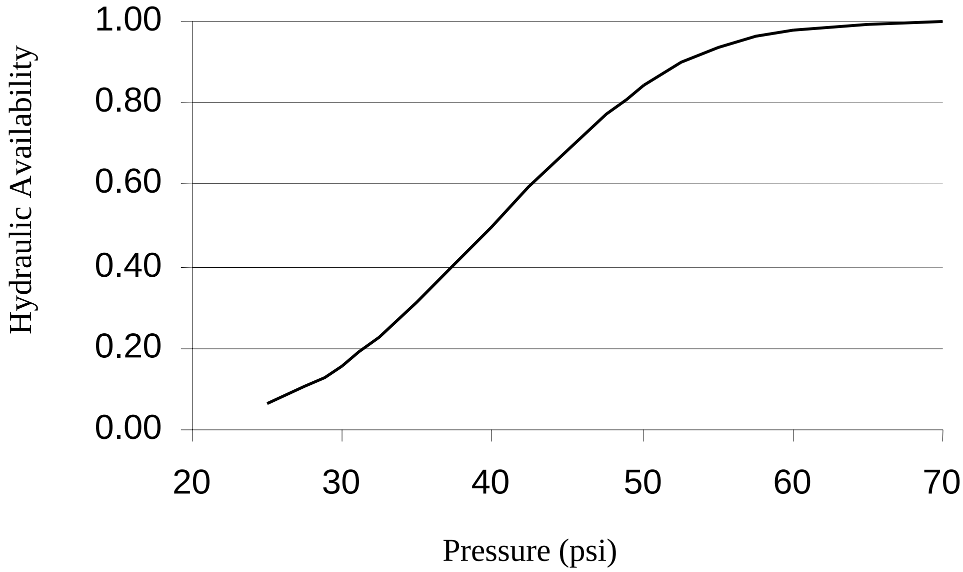


Figure 3.4: Continuous hydraulic availability function

3.8.2 Minimum Cut-Set Method

The minimum cut-set approach is adopted to calculate the nodal and system reliability, R_{node} and R_s. According to Su et al. (1987), the minimum cut set can be defined as "a set of system components (e.g., pipes) which when fails, causes failure of the system". However, when any component of the set has not failed, it does not cause system failure (Billinton and Allan, 1983).

Assuming that a pipe break can be isolated from the rest of the system, the minimum cut-sets are determined by closing a pipe or combination of pipes in the water distribution system and by using a hydraulic simulation model to determine the values of pressure head at each demand node of the system. In this study, EPANET (2000) was used (Rossman, 2000). By comparing these pressure heads with the minimum pressure head requirements, the reliability model can determine whether or not this pipe or combination of pipes is a minimum cut set of the system or an individual demand node. A minimum cut set for a node is one that causes reduced hydraulic availability at that node, while a minimum cut set for the system is a cut set that reduces the hydraulic availability for any node in the system. To calculate the number of combinations for pipe closure for the cut set determination, it is observed that failure of two or three pipes is purely a "random" phenomenon. Therefore, in order to determine the pipe combinations for the cut set determination, subsets of pipe combinations should be determined by applying a random approach. For instance, if there are K numbers of pipes in the water distribution system, then out of those K pipes, T subsets should be randomly generated and each

subset could have only one pipe or a combination of two or three pipes. A flow chart of the procedure is shown in Figure 3.2.

According to Shinstine et al. (2002), for *n* components (pipes) in the i^{th} minimum cut-set of a water distribution system, the failure probability of the i^{th} minimum cut-set (*MCi*) is

$$P(MC_i) = \prod_{i=1}^{n} P_i = P_1 \bullet P_2 \bullet P_3 \bullet .... \bullet P_n \tag{3.51}$$

Using the step function for hydraulic availability and assuming that the occurrence of the failure of the components within a minimum cut set is statistically independent, for a water distribution network with four minimum cut-sets (*MCi*) with the system reliability, R_s, the failure probability of the system P_s is then defined (Billinton and Allan, 1983) as

$$P_s = P(MC_1 \cup MC_2 \cup MC_3 \cup MC_4) \tag{3.52}$$

By applying the principle of inclusion and exclusion (Ross, 1985), equation (3.52) can be reduced to

$$P_s = P(MC_1) + P(MC_2) + P(MC_3) + P(MC_4) \tag{3.53}$$

$$P_S = \sum_{i=1}^{4} P(MC_i) \tag{3.54}$$

In general form,

$$P_s = \sum_{i=1}^{M} P(MC_i) \tag{3.55}$$

The system reliability, R_s, is expressed as

$$R_s = 1 - P_s = 1 - \sum_{i=1}^{M} P(MC_i) \qquad (3.56)$$

where M = number of minimum cut-sets in the system.

It is possible to weigh the nodal terms as the function of the nodal demand. Nodal reliabilities can be computed with the same relationship including only failures that affect the individual node.

Using the continuous hydraulic availability concept, a true minimum cut set does not exist. The probability of a cut set occurring is consistent; however, reliability is defined as the pipe reliability and hydraulic unavailability (1–HA). The system reliability is then expressed as

$$R_s = 1 - P_s = 1 - \sum_{i=1}^{M} (1 - HA_{net}^{i}) P(MC_i) \qquad (3.57)$$

where HA_{net}^{i} = network hydraulic availability (Fujivara and DeSilva, 1990)

$$HA_{net} = \prod_{j=1}^{J} HA_j \qquad (3.58)$$

where HA_j = hydraulic availability of node j.

If HA equals one, the failure probability of the cut set is not included in equation (3.57); thus, it is identical to equation (3.56) for the step function hydraulic availability case. To compute the system reliability with continuous hydraulic availability, all cut sets are included.

Chapter 4

APPLICATION OF THE DEVELOPED METHODOLOGY

4.1 General

The developed methodology was first applied to a small hypothetical water distribution network and then applied to a real water distribution network in Al-Khobar city.

4.2 Application to Hypothetical Water Distribution Network

Consider the hypothetical water distribution network as shown in Figure 4.1 which consists of one pump, one reservoir, two tanks, eighteen pipes and twelve junctions. The characteristics of the hypothetical water distribution network are shown in Tables 4.1 to 4.5.

4.2.1 Selection of Random Variables

The roughness coefficients of the pipes, nodal demands, tanks water level, and reservoir water level are selected as random or stochastic variables. These random variables are among the input parameters required for modeling the water distribution network.

4.2.2 Pipe Closure Combinations Selection

The pipe closure combinations are randomly selected using MATHEMATICA® by considering all pipes in the modeled hypothetical water distribution network.

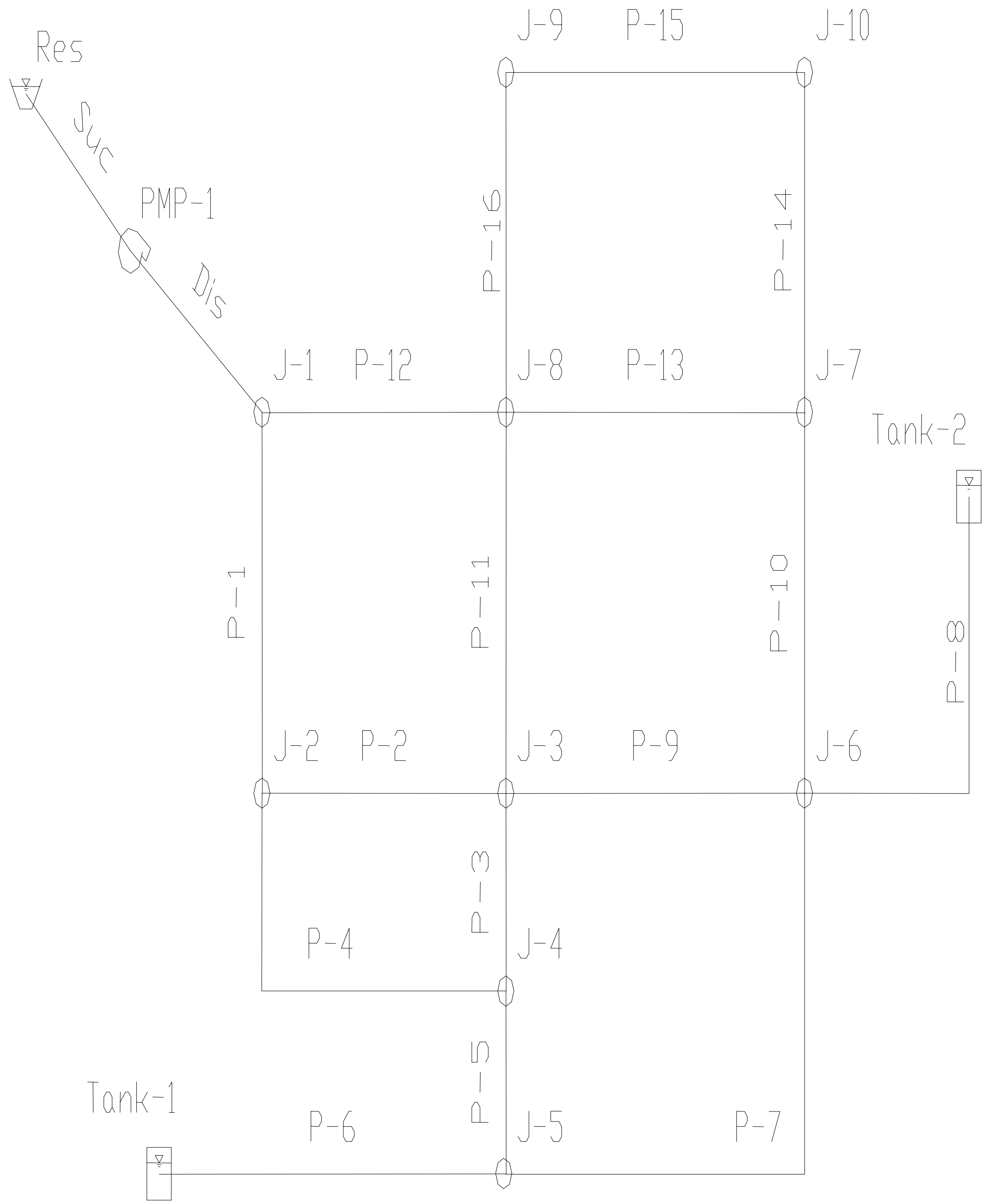


Figure 4.1: Hypothetical water distribution network (Haestad Methods, 2001)

Table 4.1: Pipe characteristics of hypothetical water distribution network

Pipe Label	Length	Diameter	Hazen-Williams
	(ft)	(in.)	C-factor
Suction	25	24	120
Discharge	220	21	120
P-1	1250	6	110
P-2	835	6	110
P-3	550	8	130
P-4	1010	6	110
P-5	425	8	130
P-6	990	8	125
P-7	2100	8	105
P-8	560	6	110
P-9	745	8	100
P-10	1100	10	115
P-11	1330	8	110
P-12	890	10	115
P-13	825	10	115
P-14	450	6	120
P-15	690	6	120
P-16	500	6	120

Table 4.2: Nodal characteristics of hypothetical water distribution network

Node Label	Elevation	Demand
	(ft)	(gpm)
Res	320	N/A
J-1	390	120
J-2	420	75
J-3	425	35
J-4	430	50
J-5	450	0
J-6	445	155
J-7	420	65
J-8	415	0
J-9	420	55
J-10	420	20
J-11	330	0
J-12	330	0

Table 4.3: Pump characteristics of hypothetical water distribution network

	Head (ft)	Flow (gpm)
Shutoff	245	0
Design	230	1100
Max Oper	210	1600

Table 4.4: Tank characteristics of hypothetical water distribution network

	Tank-1	Tank-2
Base Elevation (ft)	0	0
Min Elevation (ft)	535	525
Initial Elevation (ft)	550	545
Max Elevation (ft)	570	565
Tank Diameter (ft)	49.3	35.7

Table 4.5: Water demand pattern of hypothetical water distribution network

Time of Day	Multiplication Factor
Midnight (0 - 6^{th} hr)	1
6.00 A.M(6-12^{th} hr)	0.75
Noon(12-18^{th} hr)	1
6.00 P.M(18-24^{th} hr)	1.2
Midnight	1

As the network is small, fifteen pipe closure combinations are selected for the modeled hypothetical water distribution network as shown in Table 4.6. Since the probability of simultaneous failures of three or more pipes in a network is very small, a maximum of three pipes is considered in pipe closure combinations (Walski, 2003).

4.2.3 Random Variables Generation by Monte Carlo Method

Initially, one thousand random values were generated for the selected random variables by using the statistical analysis package STATISTICA® (1997). The statistical characteristics of the input variables, i.e. mean, standard deviation and probability distribution function, are assumed and the random variables are generated accordingly as shown in Tables 4.7 to 4.10. In this study, 1000 realizations or random values are generated for each variable and the junction and system reliability values are calculated, but it is observed that between 200 to 250 realizations of steady state conditions are reached. Therefore, in all the reliability plots for the hypothetical network, the results up to 500 realizations are plotted.

4.2.4 Random Variables Generation by Bootstrapping

Initially, one thousand random values (bootstrapped samples) are generated for the selected random variables by using the package MATLAB® (2001) as shown in Appendix-A. The random values (bootstrapped samples) of selected random variables are generated from samples assigned to each random variable. The reliability plots for the thousand bootstrapped values show that the steady state conditions are reached between 200 to 250 realizations. Therefore, in all the reliability plots for the hypothetical network, the results up to 500 realizations are plotted.

Table 4.6: Pipe Closure Combinations for Hypothetical Network

Closure Comb No	Pipe ID	Pipe ID	Pipe ID
1	DISCHARGE	-	-
2	P-6	P-8	-
3	P-6	DISCHARGE	-
4	P-6	-	-
5	P-8	DISCHARGE	-
6	P-8	-	-
7	P-4	P-7	-
8	P-3	P-6	P-10
9	P-15	-	-
10	P-2	-	-
11	P-9	P-16	SUCTION
12	P-1	P-11	-
13	P-5	P-15	SUCTION
14	P-10	P-14	-
15	P-1	P-10	-

Table 4.7: Input Parameters for Generating Pipe Roughness "C" by Monte Carlo Method

Pipe ID	**Mean**	**Stand Dev**	**Prob Dist**
P-1	100	10	Normal
P-2	100	10	Normal
P-3	100	10	Normal
P-4	100	10	Normal
P-5	100	10	Normal
P-6	100	10	Normal
P-7	100	10	Normal
P-8	100	10	Normal
P-9	100	10	Normal
P-10	100	10	Normal
P-11	100	10	Normal
P-12	100	10	Normal
P-13	100	10	Normal
P-14	100	10	Normal
P-15	100	10	Normal
P-16	100	10	Normal
Suction	100	10	Normal
Discharge	100	10	Normal

Table 4.8: Input Parameters for Generating Junction Demand “D" by Monte Carlo Method

Junction ID	Mean	Stand Dev	Prob Dist
J-1	120	5	Normal
J-2	75	5	Normal
J-3	35	5	Normal
J-4	50	5	Normal
J-5	0	0	Normal
J-6	155	5	Normal
J-7	65	5	Normal
J-8	0	0	Normal
J-9	55	5	Normal
J-10	20	5	Normal

Table 4.9: Input Parameters for Generating Reservoir water level (ft) "RL" by Monte Carlo Method

Reservoir ID	Mean	Stand Dev	Prob Dist
Res1	320	5	Normal

Table 4.10: Input Parameters for Generating Tank water level (ft) "TL" by Monte Carlo Method

Tank ID	Mean	Stand Dev	Prob Dist
Tank1	550	5	Normal
Tank2	545	5	Normal

4.2.5 Hydraulic Simulation

The hydraulic simulation of the modeled water distribution is performed by using EPANET (Rossman, 2000). The MS Visual C++ computer code of EPANET along with another code written in MS Visual C++ as shown in Appendix-B is used for hydraulic simulation and nodal pressures are calculated for all combinations of pipe closures and for the case when pipes are all open.

4.2.6 Calculation of pipe failure probability

The pipe failure probabilities are calculated by two different methods, Poisson method and Generic Expectation Function Method (GEF).

4.2.6.1 Poisson Method

The input parameters required for calculating pipe failure probability using Poisson method are expected number of failures per year per unit length of pipe and the length of pipe (already explained in Section 3.7.1). For the hypothetical water distribution network, the expected number of failures per year per unit length of pipe is assumed to be 0.122 breaks per mile per year (Kleiner and Rajani, 1999) for all the pipes. The pipe failure probabilities of all the pipes in the modeled network calculated by using Poisson method are shown in Table 4.11.

4.2.6.2 Generic Expectation Function (GEF) Method

According to GEF method, the complete pipe failure probability, P_{com}, is the product of failure probability of pipe [*P(A)*] to fulfill the demand and the pipe replacement probability [*P(B)*]. The methodology for calculating the complete pipe failure probability is already explained in Section 3.7.2. The failure probability [*P(A)*] and [P*(B)*] are calculated by using Generic Expectation Function (GEF) Method. For example, in order

Table 4.11: Pipe failure probabilities of hypothetical network using Poisson Method

Pipe	Pipe Failure Probaility
	Pi(Poisson)
P1	0.134
P2	0.092
P3	0.062
P4	0.110
P5	0.048
P6	0.108
P7	0.215
P8	0.063
P9	0.082
P10	0.119
P11	0.142
P12	0.098
P13	0.091
P14	0.051
P15	0.077
P16	0.056
Psuc	0.003
Pdis	0.025

to calculate [*P(A)*] of Pipe P-1 of the hypothetical water distribution network as shown in Figure 4.1, the statistics of input variables of equation (3.39) and equation (3.40) are required as shown in Table 4.12. Then, higher order expectations are calculated as shown in Table 4.13. By using these higher order expectations distributional characteristics are calculated as shown in Table 4.14. Similarly, for calculating [*P(B)*], the statistics of input variables of equation (3.42) and equation (3.43) are required as shown in Table 4.15. Then, higher order expectations are calculated as shown in Table 4.16. By using these higher order expectations distributional characteristics are calculated as shown in Table 4.17. Finally, complete pipe failure probability, P_{com}, is calculated. The pipe failure probabilities of all the pipes in the modeled network calculated by using GEF method are shown in Table 4.18.

4.2.7 Calculation of Nodal and System Reliability

In order to determine nodal and system reliability based on minimum cut-set method, a computer program is developed in MATLAB® (2001) as shown in Appendix-C. This computer program first determines the hydraulic availability of the junctions when all pipes of the network are open and for all cases of pipe closure combinations. Then, it compares the hydraulic availability of each junction of the network for the case when all pipes of the network are open to the hydraulic availability calculated for all assigned pipe closure combinations. If the hydraulic availability of the considered junction is reduced for any pipe closure combination then this pipe closure combination is recorded as a "cut-set". Similarly, for calculating system reliability, the program determines the "cut-sets" for calculating system reliability by comparing the hydraulic availability of all the junctions of the network considering when all pipes of the network are open to the

Table 4.12: Statistics of input variables ([*P(A)*])

Pipe P-1:

Input Variable	Mean	CV	Distribution
C_{HW}	100	0.1	Normal
d (ft)	0.5	0.025	Triangular
S	0.00408	0.0514	Triangular
D_p	2.0348	0.0692	Normal
QJ (cfs)	0.16712475	0.0133	Normal

Table 4.13: Order of Expectations ([*P(A)*])

	r	*K=1*	*K=2*	*K=3*	*K=4*
$E[C^K_{HW}]$	*K*	100	10100	1031596	106668307.8
$E[d^{2.63k}]$	*2.63*K*	0.161761	0.026279	0.004288	0.000702512
$E[S^{0.54K}]$	*0.54*K*	0.051241	0.002628	0.000135	6.92566E-06
$E[D_p^K]$	*K*	2.0348	4.160238	8.550395	17.67299357
$E[QJ^K]$	*K*	0.167125	0.027936	0.00467	0.000780963
$E[Q_P^K]$	*K*	0.358288	0.130314	0.048173	0.018118845
$E[Q_D^K]$	*K*	0.340065	0.116219	0.039934	0.01380195
$E[Z^K]$	*K*	0.018223	0.00285	0.000213	3.09152E-05

Table 4.14: Distributional Characteristics ([*P(A)*])

Output Var	Mean	*Variance*	*Stand Dev*	*Coeff of Var*	*Skewness*	*Kurtosis*
	μ	σ^2	σ	*CV*	γ	κ
Q_P	0.3583	0.0019	0.0441	0.123	1.0604	3.4439
Q_D	0.3401	0.0006	0.024	0.0705	1.5371	3.0573
Z	0.0182	0.0025	0.0502	2.7535	0.5516	3.2675

Table 4.15: Statistics of input variables ([*P(B)*]

Input Variable	Mean	Distribution
N(to)	0.0288825	Exponential
A	0.051	Exponential
R	0.06	Exponential
Fn	115962.5	Exponential
C_{n+1}	2814	Exponential

Table 4.16: Order of Expectations ([*P(B)*])

	r	*K=1*	*K=2*	*K=3*	*K=4*
$E[N(t_o)^K]$	*K*	0.0289	0.0017	0.0001	0.0000
$E[A^k]$	*K*	0.0510	0.0052	0.0008	0.0002
$E[R^K]$	*K*	0.0600	0.0072	0.0013	0.0003
$E[F_n^K]$	*K*	115962	26894602809	9356296134568020	4339917961935790000000
$E[C_{n+1}^K]$	*K*	2814	15837192	133697574844	1504899902420550
$E[N(t)^K]$	*K*	0.0373	0.0017	0.0001	0.0000
$E[B_{th}^K]$	*K*	2.4012	12.1832	90.6367	896.8558
$E[Z^K]$	*K*	2.36	12.01	89.29	910.49

Table 4.17: Distributional Characteristics ([*P(B)*])

Output Variable	Mean	Variance	Stand Dev	Coeff of Var	Skewness	Kurtosis
	μ	σ^2	σ	*CV*	γ	κ
N(t)	0.0373	0.0003	0.0180	0.4823	9.8498	34.0829
Bth	2.4012	6.4174	2.5333	1.0550	1.8800	8.4513
Z	2.3639	6.4177	2.5333	1.0716	1.8799	9.1069

Table 4.18: Pipe failure probabilities using Generic Expectation Function (Hypothetical)

Pipe	Pipe Failure Probability
	P_f(GEF)
P1	0.063
P2	0.089
P3	0.027
P4	0.101
P5	0.100
P6	0.107
P7	0.053
P8	0.063
P9	0.113
P10	0.156
P11	0.117
P12	0.076
P13	0.105
P14	0.163
P15	0.015
P16	0.012
Psuc	0.091
Pdis	0.118

hydraulic availability calculated for all assigned pipe closure combinations. If the hydraulic availability of any junction of the network is reduced for any pipe closure combination then this pipe closure combination is recorded as a “cut-set” for calculating system reliability. Finally the program calculates the nodal and system reliability. The mean and standard deviation of pressure equal to 35 psi and 5 psi are selected respectively for hydraulic availability calculations. The mean and standard deviation are calculated by considering all the junctions of the Hypothetical Network and taking out mean and standard deviation of their pressures.

In order to calculate the mean of the reliability values, an iterative code in MATLAB® (2001) as shown in Appendix-D is written which reads the file and calculates the mean. Another program in Java is written as shown in Appendix-E, which merges several files into one file.

4.3 Application to Al-Khobar City Water Distribution Network

Al-Khobar city is located on the Eastern Coast of Saudi Arabia. It has an area of approximately 64 square kilometers with a population of about 300,000. The population of the city is expected to double by the year 2020. Increase in population as well as comprehensive development resulted in a tremendous increase in water consumption in Al-Khobar city.

The water distribution system of Al-Khobar city is divided into three portions, namely Al-Rakah, Central and Al-Fawaziah. For the purposes of the application, only the central part of the city was selected, as this part covers most of the residential areas, as shown in Figure 4.2. The central network consists of twelve reservoirs, two tanks, one hundred and ninety one pipes, one hundred and thirty one junctions and twelve pumps. The characteristics of the Al-Khobar network are mentioned in Appendix-F.

4.3.1 Selection of Random Variables

The roughness coefficients of the pipes, nodal demands, tanks water level, and reservoir water level are selected as random or stochastic variables. These random variables are among the input parameters required for modeling the water distribution network.

4.3.2 Pipe Closure Combinations Selection

The pipe closure combinations were randomly selected using the computing software MATHEMATICA® by considering all pipes in the modeled water distribution network. Since the network is large, fifty pipe closure combinations were selected, as

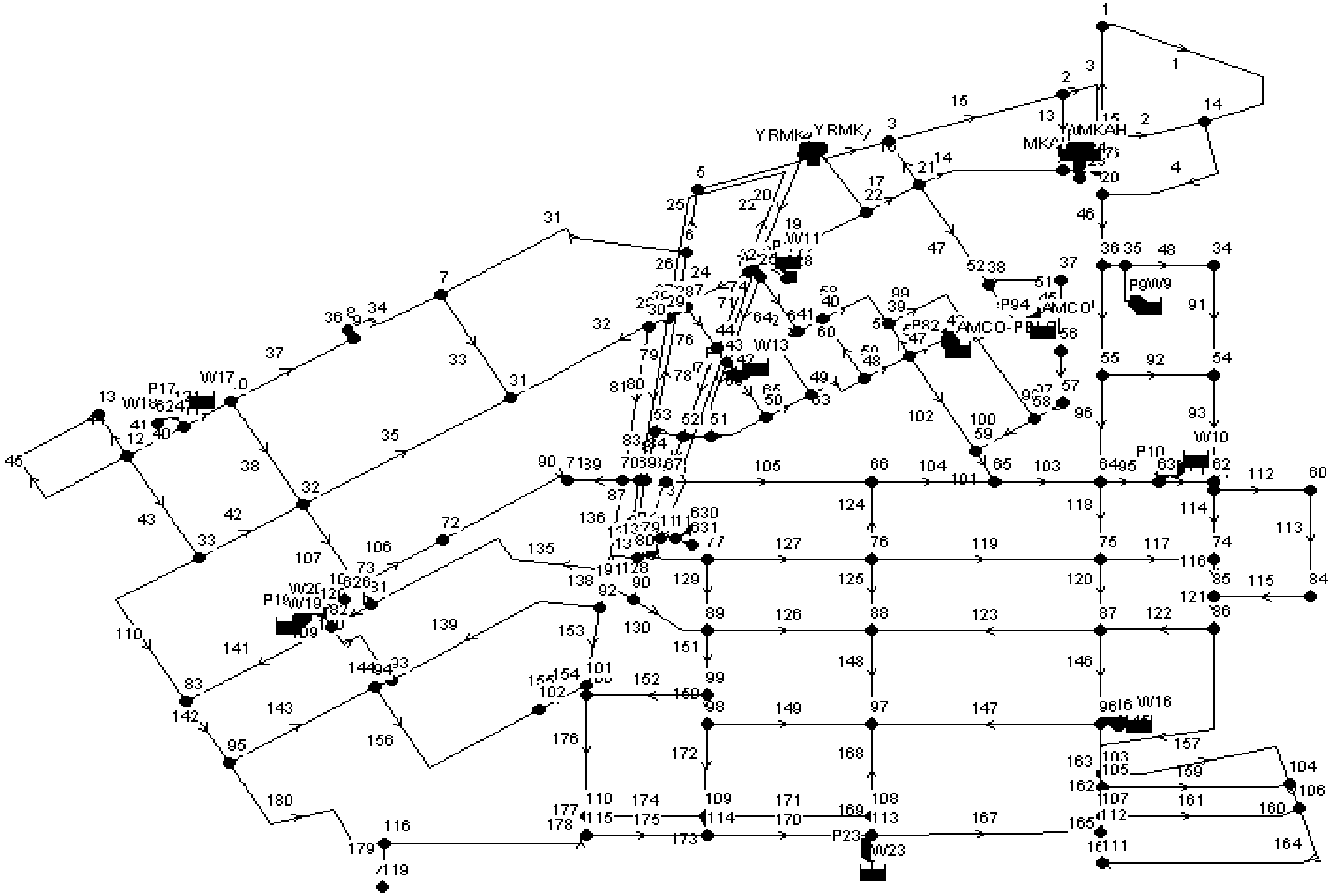


Figure 4.2 : Al-Khobar water distribution network (Central

shown in Table 4.19, for the modeled Al-Khobar water distribution network after consulting Al-Khobar Water Authority and experts. Since the probability of simultaneous failures of three or more pipes in a network is very small, a maximum of three pipes is considered in pipe closure combinations (Walski, 2003).

4.3.3 Random Variables Generation by Monte Carlo Method

In this study, for the Al-Khobar network, 1000 realizations or random values were generated for each variable and the junction and system reliability values were calculated, but it was observed from the results of the hypothetical network that between 200 and 250 realizations of steady state conditions were reached. Therefore, in all the reliability plots for Al-Khobar network, 300 realizations are used for plotting.

4.3.4 Random Variables Generation by Bootstrapping

The reliability plots of the hypothetical network for the thousand bootstrapped values show that the steady state conditions were reached between 200 and 250 realizations. Therefore, in all the reliability plots for Al-Khobar network, the results up to 300 realizations are plotted.

4.3.5 Hydraulic Simulation

The hydraulic simulation of the modeled water distribution is performed by using EPANET (Rossman, 2000). The MS Visual C++ computer code of EPANET along with another code written in MS Visual C++ (as shown in Appendix-C) is used for hydraulic simulation, and nodal pressures are calculated for all combinations of pipe closures and for the case when all pipes are open. The developed MS Visual C++ code reads the input data set of the considered random variable and runs hydraulic simulation software EPANET for all assigned combinations of pipe closures and calculates pressure at all the

Table 4.19: Pipe Closure Combinations for Al-Khobar Network

Closure Comb No	Pipe ID	Pipe ID	Pipe ID
1	160	111	20
2	83	-	-
3	146	-	-
4	150	-	-
5	112	119	-
6	164	21	-
7	37	-	-
8	175	44	-
9	12	-	-
10	150	-	-
11	114	27	-
12	158	142	-
13	17	96	-
14	141	73	-
15	7	-	-
16	7	YRMK-900	-
17	166	158	-
18	50	132	-
19	10	142	-
20	21	-	-
21	41	26	-
22	39	116	163
23	170	-	-
24	24	25	-
25	37	127	-
26	170	26	55
27	7	-	-
28	127	-	-
29	158	150	128
30	10	26	-
31	10	-	-
32	178	115	-
33	27	163	-
34	50	-	-
35	170	20	-
36	162	-	-
37	110	128	-
38	96	132	-
39	164	172	-
40	180	146	-
41	83	150	-
42	119	-	-
43	114	86	-
44	6	43	-
45	41	172	-
46	83	-	-
47	178	50	-
48	83	-	-
49	160	-	-
50	19	21	-

junctions in the network. It is observed from the result of hypothetical network that between 200 and 250 realizations of steady state conditions are reached. Therefore, for Al-Khobar network, 300 input data sets of the selected random variables are used for hydraulic simulation.

4.3.6 Calculation of pipe failure probability

The pipe failure probabilities are calculated by two different methods, Poisson method and Generic Expectation Function Method (GEF). The application is the same as explained in Section 3.7 The failure probabilities of pipes considered in selected pipe closure combinations for the modeled Al- Khobar network calculated by using Poisson method are shown in Table 4.20 and by using Generic Expectation Functions are shown in Table 4.21.

4.3.7 Calculation of Nodal and System Reliability

The nodal and system reliability is calculated by the minimum cut-set method (already explained in Section 3.8). The mean and standard deviation of nodal pressures equal to 33 psi (23.2 m of H_2O) and 5 psi (3.515 m of H_2O) are selected respectively for hydraulic availability calculations of Al-Khobar Network. The mean and standard deviation are calculated by considering all the junctions of the Al-Khobar Network and taking out mean and standard deviation of their pressures.

Table 4.20: Pipe failure probabilities for selected pipes of Al-Khobar network using Poisson Method

Pipe ID	Pipe Failure Probability	Pipe ID	Pipe Failure Probability
	Pi (Poisson)		Pi (Poisson)
6	0.00726	112	0.02216
7	0.00067	114	0.01527
10	0.00022	115	0.02242
12	0.00431	116	0.01061
17	0.01447	119	0.05589
19	0.02821	127	0.03266
20	0.06063	128	0.01341
21	0.02190	132	0.00022
24	0.01687	141	0.03865
25	0.01766	142	0.01952
26	0.01527	146	0.02532
27	0.00270	150	0.00807
37	0.03319	158	0.00337
39	0.01234	160	0.00673
41	0.01554	162	0.00780
43	0.03188	163	0.02900
44	0.01301	164	0.06097
50	0.03005	166	0.00860
55	0.00834	170	0.03240
73	0.07598	172	0.02453
83	0.01341	175	0.02835
86	0.02295	178	0.04744
96	0.02848	180	0.05052
110	0.05308	YRMK-900	0.00022
111	0.00243		

Table 4.21: Pipe failure probabilities for selected pipes of Al-Khobar network using GEF

Pipe ID	Pipe Failure Probability	Pipe ID	Pipe Failure Probability
	Pi (GEF)		Pi (GEF)
6	0.02004	112	0.01892
7	0.00000	114	0.02298
10	0.02071	115	0.01260
12	0.02064	116	0.02020
17	0.02062	119	0.02045
19	0.02077	127	0.02012
20	0.01656	128	0.02066
21	0.00000	132	0.01993
24	0.00589	141	0.01917
25	0.00013	142	0.01890
26	0.00275	146	0.02053
27	0.02017	150	0.01985
37	0.01755	158	0.01911
39	0.02042	160	0.02959
41	0.02051	162	0.01782
43	0.01897	163	0.01726
44	0.00000	164	0.04163
50	0.01956	166	0.04071
55	0.02118	170	0.04112
73	0.01867	172	0.01971
83	0.02346	175	0.02031
86	0.01961	178	0.02095
96	0.01758	180	0.01934
110	0.03316	YRMK-900	0.01973
111	0.02124		

The developed methodology was applied first to a small hypothetical water distribution network and then applied to the water distribution network of Al-Khobar city. The nodal and system reliability is calculated for the following cases :

Case 1: *Steady state analysis, model input data generated by Monte Carlo method and pipe failure probabilities calculated by Generic Expectation Function (GEF) method.*

Case 2: *Steady state analysis, model input data generated by Monte Carlo method and pipe failure probabilities calculated by Poisson method.*

Case 3: *Steady state analysis, model input data generated by Bootstrapping and pipe failure probabilities calculated by Generic Expectation Function (GEF) method.*

Case 4: *Steady state analysis, model input data generated by Bootstrapping and pipe failure probabilities calculated by Poisson method.*

Case 5: *Extended period analysis, model input data generated by Monte Carlo method and pipe failure probabilities calculated by Generic Expectation Function (GEF) method.*

Case 6: *Extended period analysis, model input data generated by Monte Carlo method and pipe failure probabilities calculated by Poisson method.*

Case 7: *Extended period analysis, model input data generated by Bootstrapping and pipe failure probabilities calculated by Generic Expectation Function (GEF) method.*

Case 8: *Extended period analysis, model input data generated by Bootstrapping and pipe failure probabilities calculated by Poisson method.*

The above cases are summarized and shown in Table 4.22.

Table 4.22: Cases for calculating Nodal and System Reliability

Case No.	Model		Data Generation Method		Pipe Failure Probability Calculation Method	
	SS	**EPS**	**MC**	**BS**	**GEF**	**Poisson**
1	√		√		√	
2	√		√			√
3	√			√	√	
4	√			√		√
5		√	√		√	
6		√	√			√
7		√		√	√	
8		√		√		√

SS = Steady State
EPS = Extended Period Simulation
MC = Monte Carlo
BS = Bootstrapping
GEF = Generic Expectation Method

All graphs generated represent the reliability values calculated by considering four random variables independently and also considering all of them collectively. The four random variables are defined as follows:

1. Hazen William pipe roughness coefficient (C)
2. Nodal or junction water demand (D)
3. Tank water level (TL)
4. Reservoir water level (RL).

For steady state analysis, graphs are plotted between the moving average of reliability values and the number of iterations. Both junction and system reliability are analyzed separately.

The extended period analysis is performed for duration of 24 hours with a time step of 1 hour. The 0^{th}, 6^{th}, 12^{th}, 16^{th}, 18^{th} and 24^{th} hour are selected for analysis purpose and for each selected hour graphs are plotted between the moving average of reliability values and the number of iterations. Similar to the case of steady state condition, both junction and system reliability are analyzed separately.

Moreover, for extended period analysis, graphs are also plotted for the mean reliability values versus the 24 hour duration.

Chapter 5

ANALYSIS OF RESULTS

5.1 Hypothetical Water Distribution Network

For the hypothetical water distribution network, one junction out of twelve junctions is selected for analysis purpose. Since most of the junctions in the hypothetical network have very high pressures, therefore junction J-5 which has relatively lower pressure is selected for nodal reliability analysis. The nodal and system reliability are calculated for all cases already mentioned in Table 4.22.

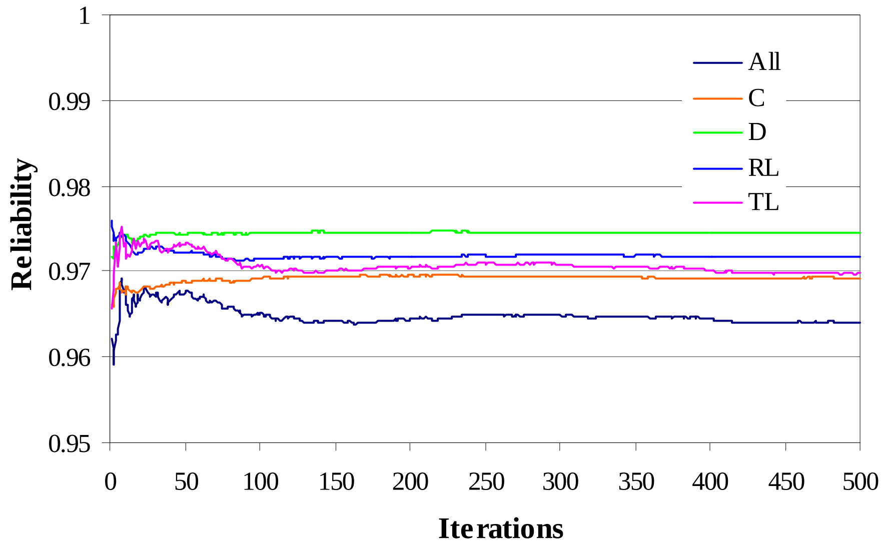
5.1.1 Nodal Reliability

The nodal reliability values are calculated for the selected junction J-5 of the hypothetical network considering all the cases summarized in Table 5.1. The junction reliability plots for steady state analysis are shown in Figure 5.1 while the plots for the mean reliability values versus the 24 hours duration are shown in Figure 5.2. For extended period analysis, the graphs plotted between the moving average of reliability values and the numbers of iterations are shown in Appendix-G.

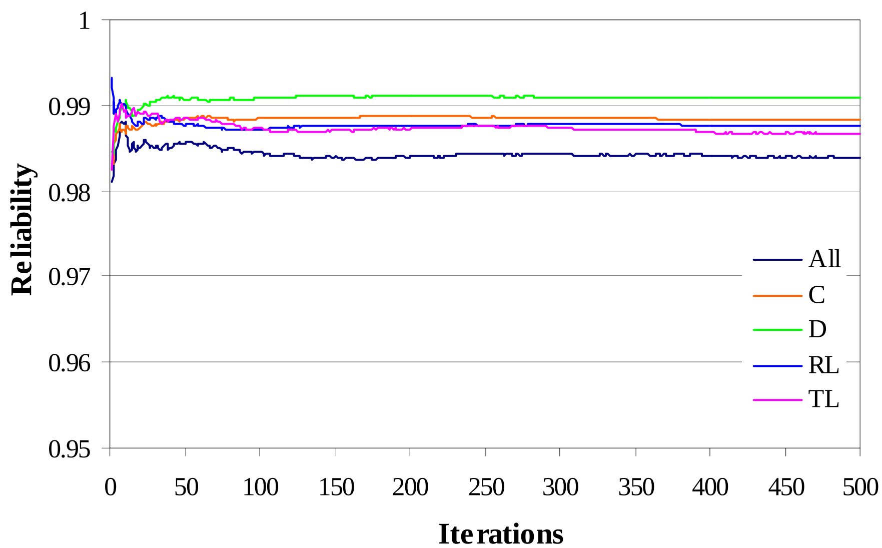
Table 5.1: Summary of Reliability Values (%) for Junction J-5 of the Hypothetical Network

Data Generation Method	**Monte Carlo Method**										**Bootstrap Method**									
Pipe failure probability	**GEF**					**Poisson**					**GEF**					**Poisson**				
Random Variable (RV)	**C**	**D**	**RL**	**TL**	**All**	**C**	**D**	**RL**	**TL**	**All**	**C**	**D**	**RL**	**TL**	**All**	**C**	**D**	**RL**	**TL**	**All**
	STEADY STATE ANALYSIS																			
	96.93	97.45	97.18	96.98	96.41	98.83	99.08	98.76	98.68	98.40	96.94	97.44	97.18	97.02	96.51	98.83	99.07	98.76	98.69	98.46
Time	**EXTENDED PERIOD SIMULATION**																			
0th hour	99.35	99.40	99.20	99.15	99.10	99.71	99.77	99.60	99.63	99.50	99.32	99.36	99.19	99.22	99.12	99.71	99.77	99.61	99.62	99.52
6th hour	99.25	99.26	99.23	99.22	99.16	99.55	99.59	99.6	99.6	99.5	99.25	99.26	99.24	99.24	99.17	99.56	99.59	99.56	99.58	99.50
12th hour	99.24	99.26	99.23	99.22	99.13	99.55	99.59	99.54	99.56	99.46	99.23	99.26	99.23	99.23	99.15	99.55	99.59	99.54	99.57	99.48
18th hour	99.23	99.25	99.07	99.18	99.01	99.55	99.57	99.40	99.53	99.35	99.23	99.25	99.07	99.20	99.03	99.55	99.57	99.40	99.54	99.37
24th hour	99.24	99.28	98.97	99.14	98.94	99.58	99.62	99.30	99.50	99.30	99.24	99.28	98.99	99.16	98.99	99.58	99.62	99.30	99.51	99.34

C = Pipe roughness
D = Junction demand
TL = Tank level
RL = Reservoir level
All = All above four as RVs

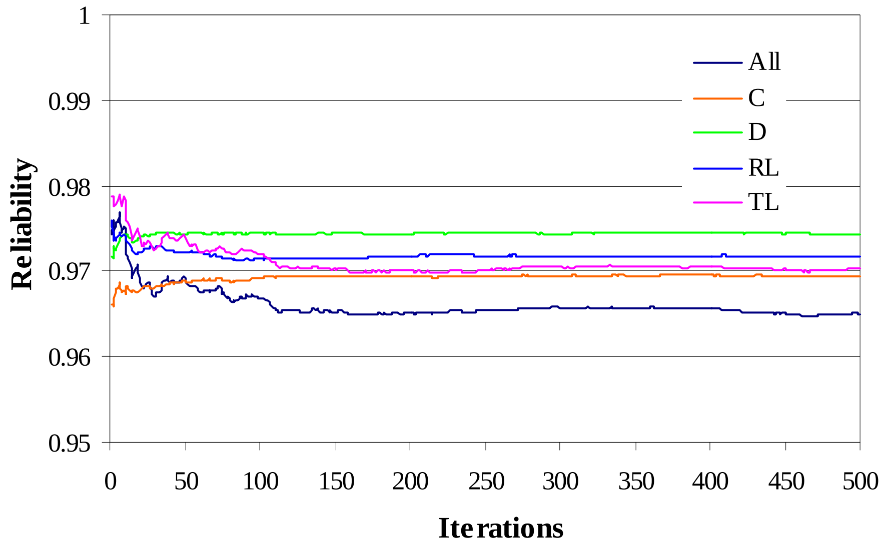


(a)

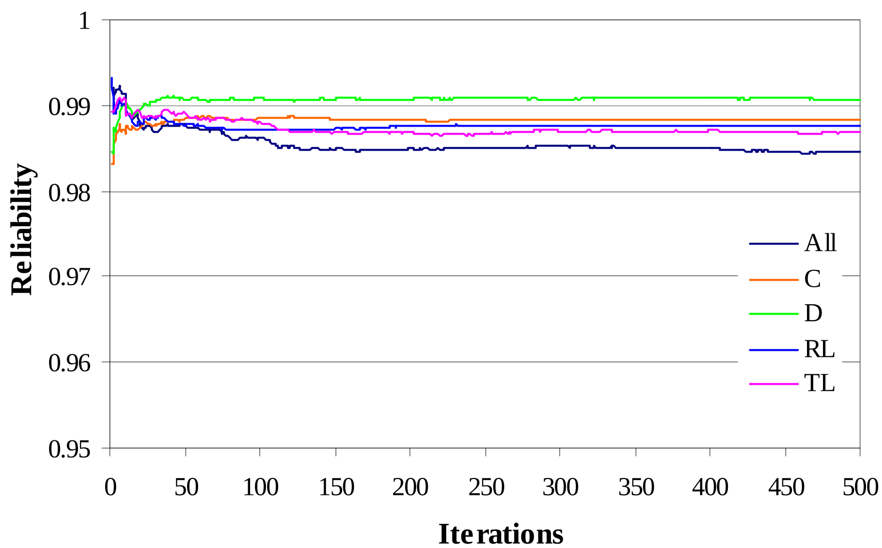


(b)

Figure 5.1: Variation of the reliability of junction J-5 for (a) Case 1, (b) Case 2, (c) Case 3 and (d) Case 4

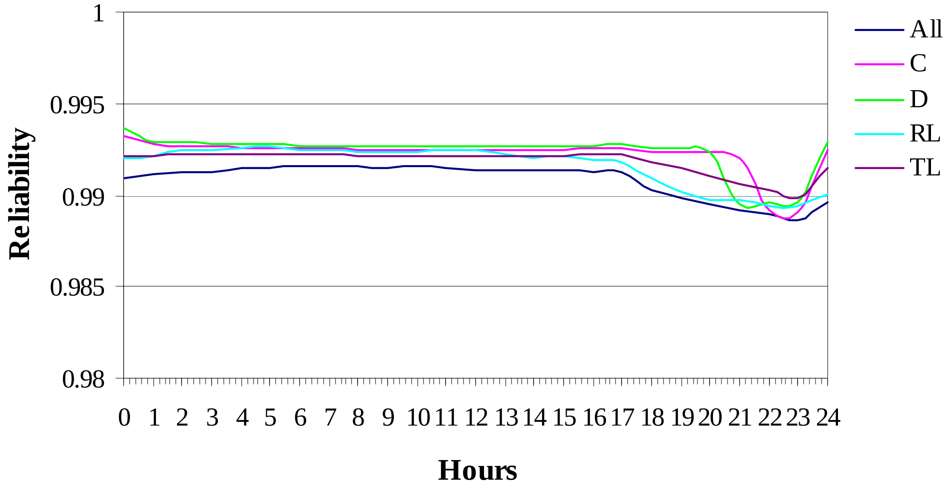


(c)

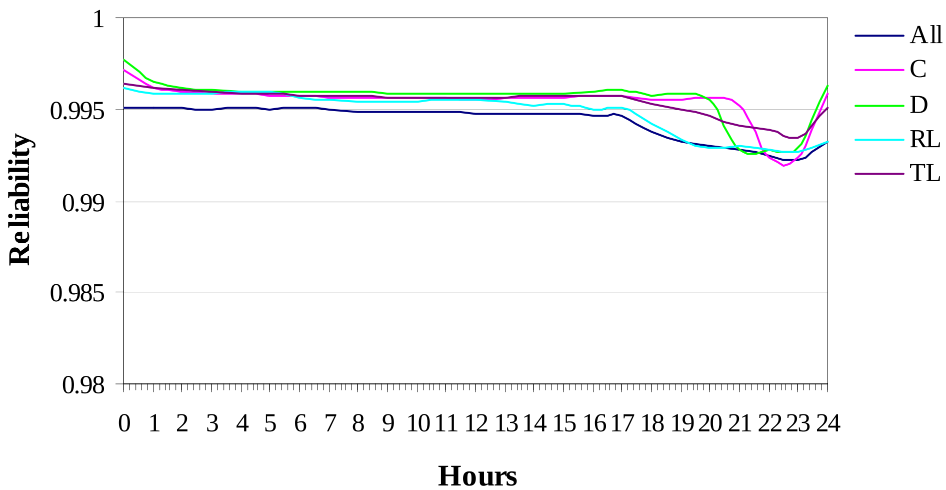


(d)

Figure 5.1: "Continued"

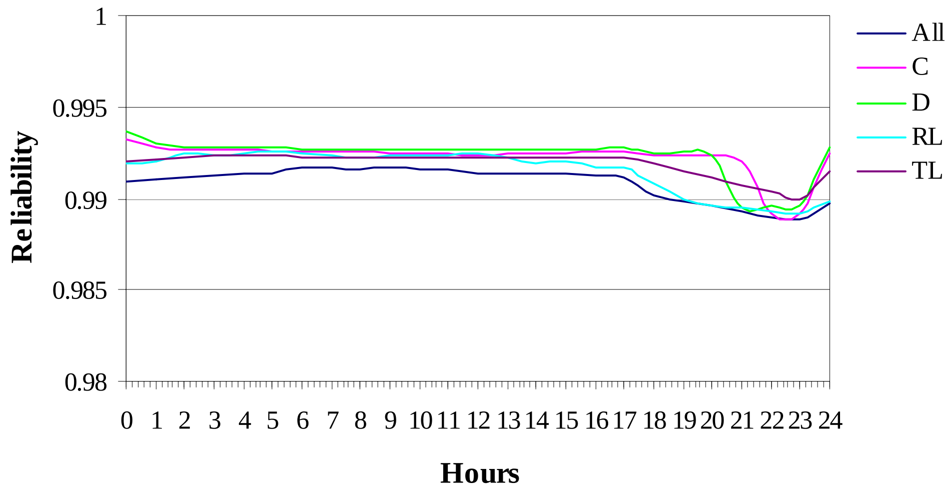


(a)

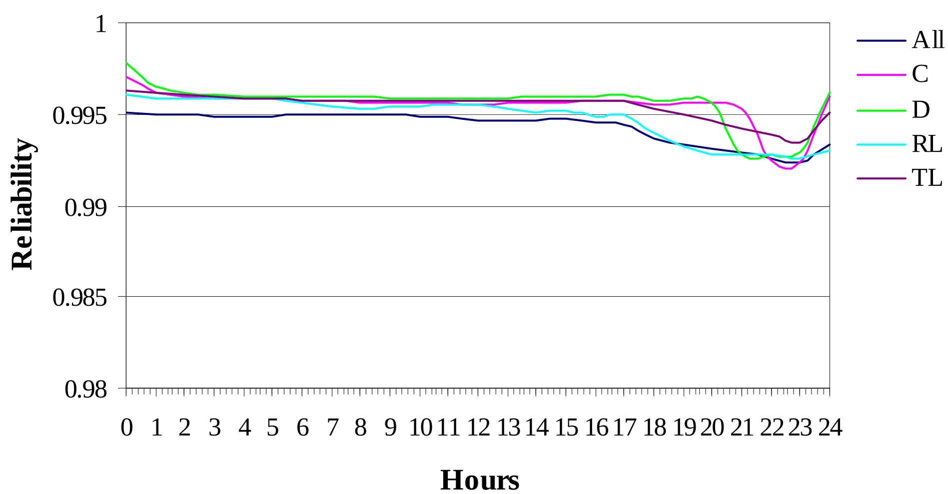


(b)

Figure 5.2: Variation of the mean reliability of junction J-5 during 24-hr period for (a) Case 5 (b) Case 6 (c) Case 7 and (d) Case 8



(c)



(d)

Figure 5.2: "Continued"

By comparing the reliability values summarized in Table 5.1 for steady state and extended period reliability analysis, it is seen that the reliability values for junction J-5 of the hypothetical network are affected by the method of calculating pipe failure probabilities. For most of the pipes selected for pipe closure combinations, the Poisson method gives higher pipe failure probability values as compared to the Generic Expectation Function (GEF) method. The reason is that the Poisson Method only requires the expected number of failures per year of pipe for calculating pipe failure probabilities while the GEF method requires many parameters, such as randomness in pipe roughness, pipe diameter, and number of breaks in the pipe, and repair and replacement costs of the pipe.

For junction J-5, it is observed that reliability values are not much affected by the methods adopted for generating input data, which are Monte Carlo and Bootstrapping; both methods give nearly the same reliability values. Both methods have been assigned the same ranges of model input parameters; therefore it is observed that the hypothetical network is not sensitive to the assigned ranges of model input parameters.

It is observed from Figure 5.1 and Table 5.1 that for all steady state and extended period cases, the reliability values are minimum for cases when collectively all four selected input variables (which are pipe roughness, junction demand, tank water level, and reservoir water level) are considered as random variables. The reason is that each random variable has its effect on nodal pressures, which in turn affects the reliability values, therefore from Table 5.1 it is seen that the collective effect “All” of these four random variables for junction J-5 of hypothetical network gives the minimum reliability values for

junction J-5 as compared to the cases when only one input parameter is considered as a random variable. For instance, referring to Tables 4.22 and 5.1 for interpreting the reliability values for Case 1 (which is for steady state analysis, data generated by Monte Carlo method and pipe failure probability calculated by GEF method), the reliability of junction J-5 while considering only the pipe roughness (C) as the random variable is 96.93 %, for junction demand (D) as only random variable, it is 97.45 %, for reservoir water level (RL) only as random variable, it is 97.18 % , for tank water level (TL) as only random variable, it is 96.98 % and while considering “All” input parameters i.e “C’, “D”, “RL” and “TL” as random variables, junction reliability is 96.41 %. It can be seen that for independent case, pipe roughness “C” gives the minimum reliability value, which is 96.93 %. Therefore, for having the minimum reliability value for the case when “All” input parameters are considered as random variables, the most significant effect among the four random variables must be of pipe roughness “C”, and all other random variables must have some slight effect.

Similar interpretations can be made for the reliability values of junction J-5 for all the other cases from Tables 4.22 and 5.1.

For steady state reliability analysis results, from Tables 4.22 and 5.1, it is observed that the minimum reliability value is 96.41 % for Case 1, which means that there is a probability of 96.41 % that the pressure at junction J-5 will be greater than or equal to 35 psi or there is a risk of (1-0.9641)*100 = 3.59 % that the pressure at this junction will be less than 35 psi.

For extended period reliability analysis results, it is observed from Figure 5.2 that the reliability value is almost constant from 0^{th} to 17^{th} hour and it decreases from the 18^{th}

to the 23rd hour and again it increases up to the 24th hour. The reason for this decrease is the change in demand pattern multiplier from 1.0 to 1.2 during these hours, as shown in Table 4.5. The demand pattern has an effect by slightly decreasing the nodal pressures during these hours, which in turn has an effect on the reliability values.

For instance, referring to Figure 5.2 for Case 5, it is concluded that at the 23rd hour of Extended Period Simulation (EPS), the reliability value of junction J-5 is minimum and is equal to 98.86 % when "All" input parameters "C", "D", "RL" and "TL" are considered as random variables. It is also observed that for independent case, reservoir level "RL" gives the minimum reliability value at the 23rd hour of EPS and is equal to 98.93 %.

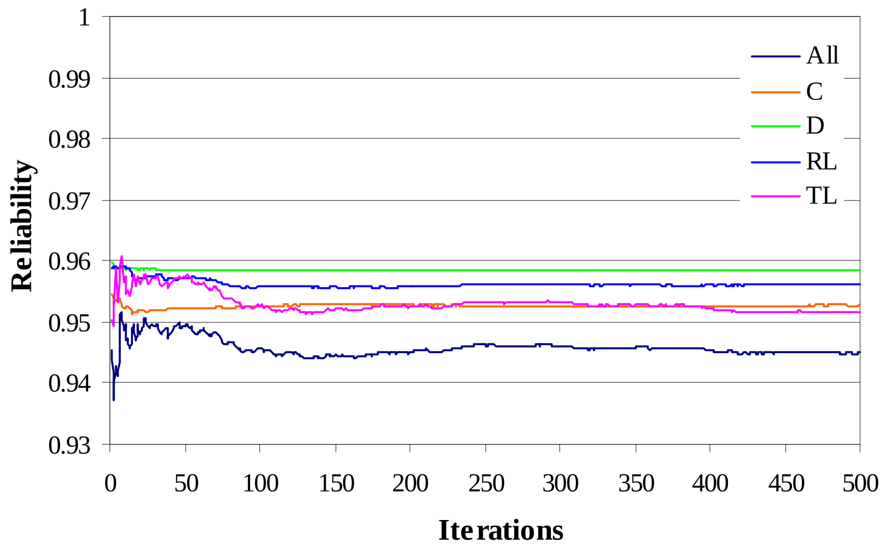
5.1.2 System Reliability

The system reliability values are calculated for the hypothetical network considering all the cases; they are summarized in Table 5.2. The system reliability plots for steady state analysis are shown in Figure 5.3 while the plots for the mean reliability values versus the 24 hours duration are shown in Figure 5.4. The graphs plotted between the moving average of reliability values and the numbers of iterations are shown in Appendix-G.

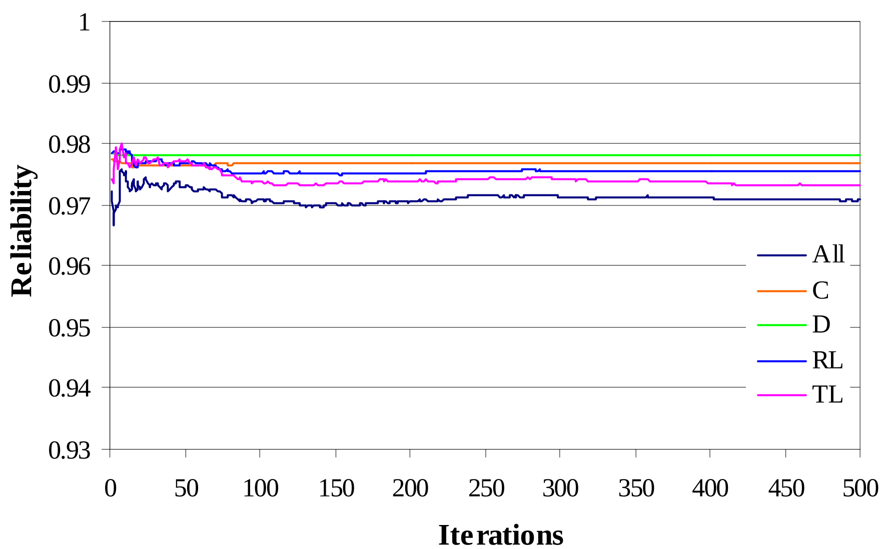
Table 5.2: Summary of System Reliability Values (%) of the Hypothetical Network

Data Generation Method	Monte Carlo Method										Bootstrap Method									
Pipe failure probability	GEF					Poisson					GEF					Poisson				
Random Variable (RV)	C	D	RL	TL	All	C	D	RL	TL	All	C	D	RL	TL	All	C	D	RL	TL	All
	STEADY STATE ANALYSIS																			
	95.27	95.85	95.61	95.15	94.48	97.67	97.80	97.55	97.31	97.07	95.29	95.85	95.61	95.22	94.59	97.67	97.80	97.56	97.35	97.15
Time	EXTENDED PERIOD SIMULATION																			
0^{th} hour	95.27	95.88	95.60	95.15	94.48	97.67	97.80	97.55	97.31	97.07	95.29	95.85	95.61	95.22	94.59	97.67	97.80	97.56	97.35	97.15
6^{th} hour	94.48	95.06	94.99	94.61	93.99	96.88	97.00	96.94	96.8	96.58	94.53	95.05	95.02	94.69	94.15	96.88	97.00	96.97	96.83	96.71
12^{th} hour	94.48	95.06	94.98	94.58	93.95	96.88	97.00	96.92	96.74	96.53	94.53	95.05	95.00	94.66	94.10	96.88	97.00	96.94	96.80	96.67
18^{th} hour	94.48	95.06	94.99	94.58	93.96	96.88	97.00	96.94	96.74	96.55	94.50	95.05	95.02	94.46	94.13	96.88	97.00	96.97	96.80	96.69
24^{th} hour	94.48	95.06	95.03	94.58	94.00	96.88	97.00	96.98	96.74	96.60	94.50	95.05	95.07	94.68	94.18	96.88	97.00	97.04	96.80	96.75

C = Pipe roughness
D = Junction demand
TL = Tank level
RL = Reservoir level
All = All above four as RVs

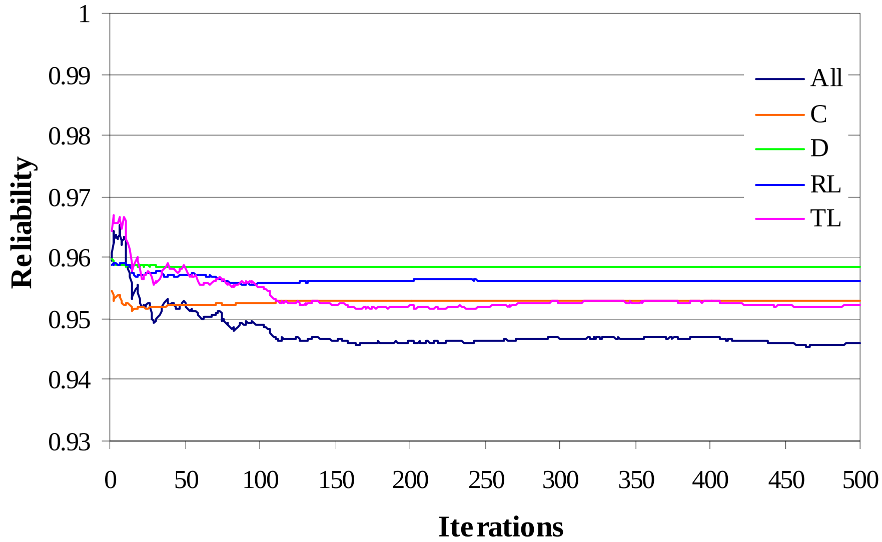


(a)

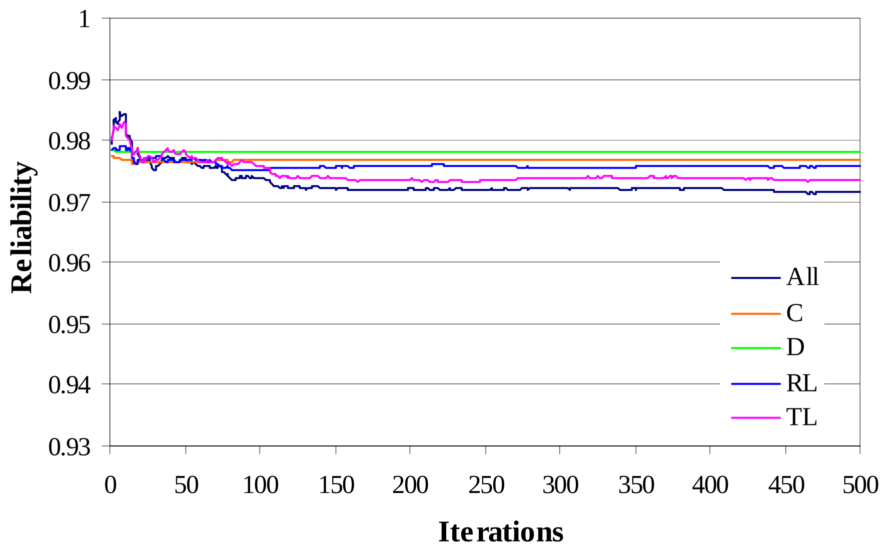


(b)

Figure 5.3: System reliability plot of the hypothetical network under steady state condition for (a) Case 1 (b) Case 2, (c) Case 3 and (d) Case 4

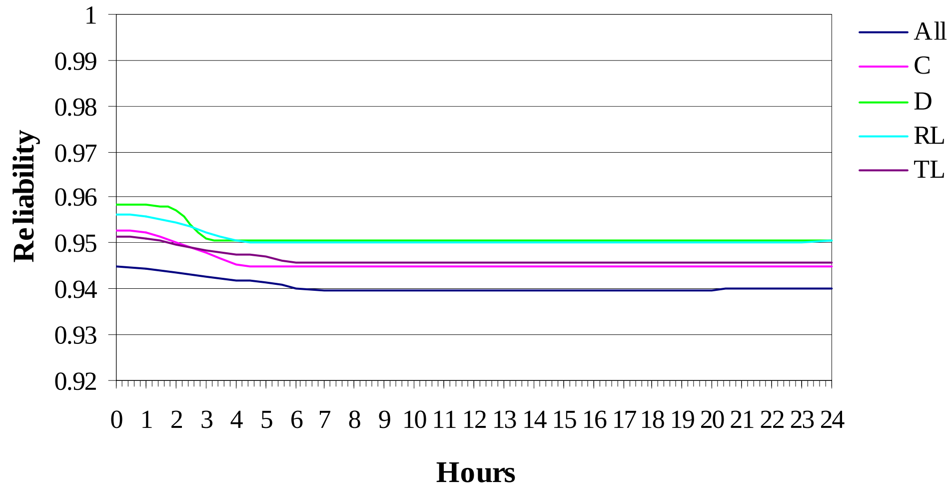


(c)

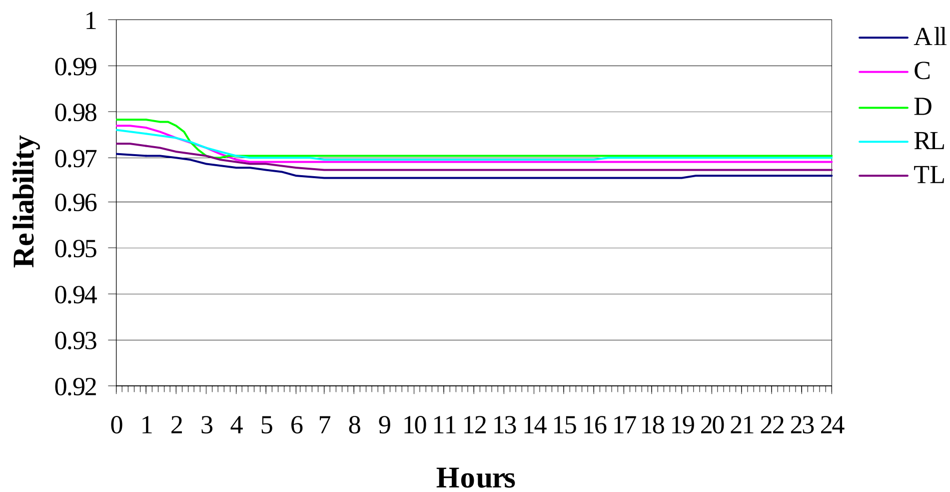


(d)

Figure 5.3: "Continued"

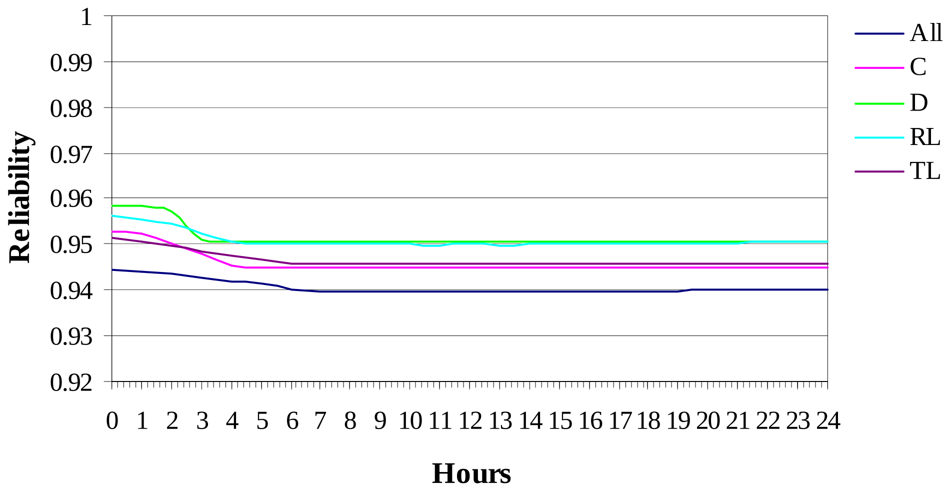


(a)

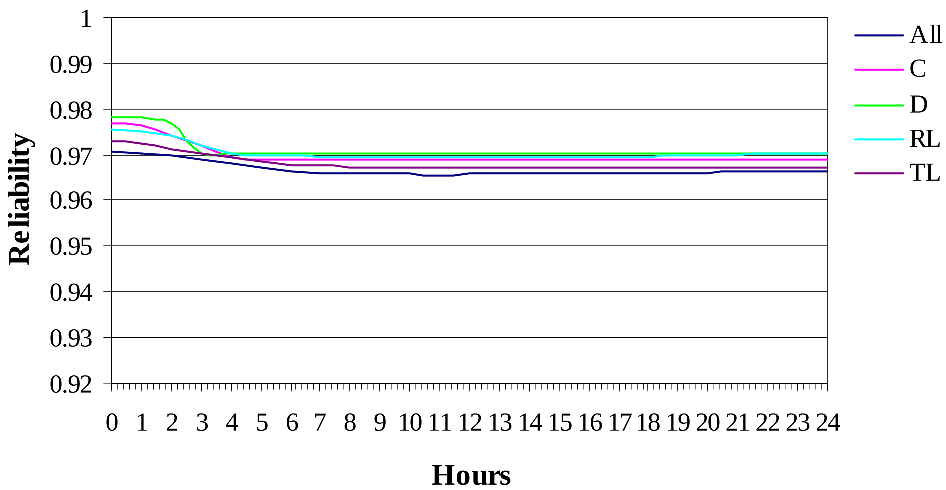


(b)

Figure 5.4: System reliability variation of the hypothetical network during 24-hr period for (a) Case 5, (b) Case 6, (c) Case 7 and (d) Case (8)



(c)



(d)

Figure 5.4: "Continued"

By comparing the reliability values summarized in Table 5.2 for steady state and extended period reliability analysis, it is observed that similar to the junction reliability values, system reliability values for the hypothetical network are also affected by the method of calculating pipe failure probabilities. For most of the pipes selected for pipe closure combinations, the Poisson method gives higher pipe failure probability values as compared to the Generic Expectation Function (GEF) method. The reason is that the Poisson Method only requires the expected number of failures per year of pipe for calculating pipe failure probabilities while the GEF method requires many parameters, such as randomness in pipe roughness, pipe diameter, and number of breaks in the pipe, and repair and replacement costs of the pipe.

Similar to the behavior of junction reliability, it is observed that system reliability values are not much affected by the methods adopted for generating input data which are Monte Carlo and Bootstrapping; both methods give nearly the same reliability values. Both methods have been assigned the same ranges of model input parameters; therefore it is observed that the hypothetical network is not sensitive to the assigned ranges of model input parameters.

It is observed from Figure 5.3 and Table 5.2 that for all steady state and extended period cases, the reliability values are minimum for cases when collectively all four selected input variables (which are pipe roughness, junction demand, tank water level, and reservoir water level) are considered as random variables. The reason is that each random variable has its effect on nodal pressures, which in turn affects the reliability values, therefore from Table 5.2 it is seen that the collective effect “All” of these four random

variables for junction J-5 of the hypothetical network gives the minimum reliability values for junction J-5, as compared to the cases when only one input parameter is considered as the random variable.

For steady state reliability analysis results, from Tables 4.22 and 5.2, it is observed that the minimum reliability value is 94.48 % for Case 1, which means that there is a probability of 94.48 % that the pressure at all junctions of the hypothetical network will be greater than or equal to 35 psi or there is a risk of (1-0.9448)*100 = 5.52 % that the pressure at all junctions would be less than 35 psi.

For extended period reliability analysis results, it is observed from Figure 5.4 that for all Cases, there is a slight decrease in system reliability from 0^{th} to 6^{th} hour, after that system reliability becomes almost constant from 7^{th} to 24^{th} hour. The reason for this decrease in reliability values is that when some of the pipes that are connected to sources are closed, the nodal pressures start to drop suddenly, therefore in the early hours there is a “lower number of cut-sets” which results in higher reliability values.

From Table 5.2, it is observed that the minimum system reliability value is 93.95 % for Case 5 at the 12^{th} hour, as some pipe closure combinations result in very low pressures during this hour, which results in a lower reliability value.

5.2. Al-Khobar Water Distribution Network

The central network of Al-Khobar Water Distribution Network, which is the major part of the distribution system, consists of twelve reservoirs, two tanks, one hundred and ninety one pipes having pipe diameters of 150 cm or higher, one hundred and thirty one junctions and twelve pumps. For analysis purposes, the junctions are categorized with respect to their pressure ranges, and 4 junctions out of 131 are selected for analysis purposes. These junctions are selected according to the following ranges of nodal pressures (metres of water).

Pressure Range	2m to 10m	11m to 20m	21m to 30m	31m to 40m
Junction ID	J-14	J-16	J-31	J-116

5.2.1 Nodal Reliability

The nodal reliability values are calculated for the above selected junctions of Al-Khobar network considering all previously mentioned cases and summarized in Tables 5.3 to 5.6. The junction reliability plots for all the cases of junction J-16 for steady state analysis are shown in Figure 5.5 while the plots of the selected junctions for the mean of reliability values versus the 24 hours duration are shown in Figures 5.6 to 5.9. The graphs are plotted for other junctions J-14, J-31 and J-116, between the moving average of reliability values and the number of iterations for both steady and extended period conditions.

Table 5.3: Summary of Reliability Values (%) for Junction J-14 of Al-Khobar Network

Data Generation Method	**Monte Carlo Method**										**Bootstrap Method**									
Pipe failure probability	**GEF**					**Poisson**					**GEF**					**Poisson**				
Random Variable (RV)	**C**	**D**	**RL**	**TL**	**All**	**C**	**D**	**RL**	**TL**	**All**	**C**	**D**	**RL**	**TL**	**All**	**C**	**D**	**RL**	**TL**	**All**
	STEADY STATE ANALYSIS																			
	85.20	85.22	85.27	85.52	85.65	84.50	84.50	84.50	84.50	84.50	85.26	85.22	85.27	85.45	85.56	84.46	84.46	84.46	84.46	84.46
Time	**EXTENDED PERIOD SIMULATION**																			
0^{th} hour	86.23	86.39	87.21	86.21	86.03	84.88	84.89	84.86	84.88	84.86	86.12	86.32	87.22	86.31	86.02	84.87	84.89	84.86	84.88	84.84
6^{th} hour	80.04	80.03	76.53	79.79	79.66	82.58	82.62	80.66	82.49	82.31	79.95	80.06	76.45	79.99	79.48	82.51	82.64	80.62	82.63	82.22
12^{th} hour	78.04	76.66	79.26	76.59	77.61	79.67	78.62	80.73	78.89	79.32	77.50	76.73	79.37	76.76	77.52	79.58	78.67	80.77	78.95	79.31
18^{th} hour	81.19	80.45	80.94	80.27	80.39	82.30	82.09	81.86	81.96	82.04	80.95	80.23	80.89	80.20	80.70	82.06	82.04	81.85	81.84	82.25
24^{th} hour	79.15	78.79	78.28	78.53	78.54	82.09	81.66	81.58	81.73	81.50	78.93	78.56	78.38	78.53	79.07	81.96	81.47	81.64	81.74	81.74

C = Pipe roughness
D = Junction demand
TL = Tank level
RL = Reservoir level
All = All above four as RVs

Table 5.4: Summary of Reliability Values (%) for Junction J-16 of Al-Khobar Network

Data Generation Method	**Monte Carlo Method**										**Bootstrap Method**									
Pipe failure probability	**GEF**					**Poisson**					**GEF**					**Poisson**				
Random Variable (RV)	**C**	**D**	**RL**	**TL**	**All**	**C**	**D**	**RL**	**TL**	**All**	**C**	**D**	**RL**	**TL**	**All**	**C**	**D**	**RL**	**TL**	**All**
	STEADY STATE ANALYSIS																			
	85.48	85.48	85.44	85.74	85.85	84.65	84.65	84.64	84.72	84.71	85.43	85.46	85.44	85.70	85.75	84.63	84.65	84.64	84.69	84.70
Time	**EXTENDED PERIOD SIMULATION**																			
0th hour	86.63	86.63	87.50	86.36	86.16	85.02	85.06	85.09	85.03	85.00	86.29	86.52	87.50	86.47	86.18	85.02	85.06	85.09	85.03	85.00
6th hour	79.04	79.04	78.85	78.98	79.47	82.25	81.99	81.97	81.91	82.14	79.38	79.07	78.77	79.15	79.53	82.19	82.04	81.93	82.14	82.23
12th hour	76.99	76.99	76.35	76.94	77.63	79.82	78.98	79.13	79.23	79.43	77.81	76.93	76.15	76.90	77.71	79.69	78.98	79.06	79.21	79.44
18th hour	79.65	79.65	80.43	79.99	79.86	81.35	81.41	79.89	81.79	81.47	80.03	79.72	80.27	79.93	79.85	81.31	81.54	79.56	81.68	81.45
24th hour	78.00	78.00	76.71	78.19	78.28	81.64	81.13	80.70	81.32	81.30	78.34	77.97	76.60	78.32	78.43	81.47	81.13	80.65	81.41	81.33

C = Pipe roughness
D = Junction demand
TL = Tank level
RL = Reservoir level
All = All above four as RVs

Table 5.5: Summary of Reliability Values (%) for Junction J-31 of Al-Khobar Network

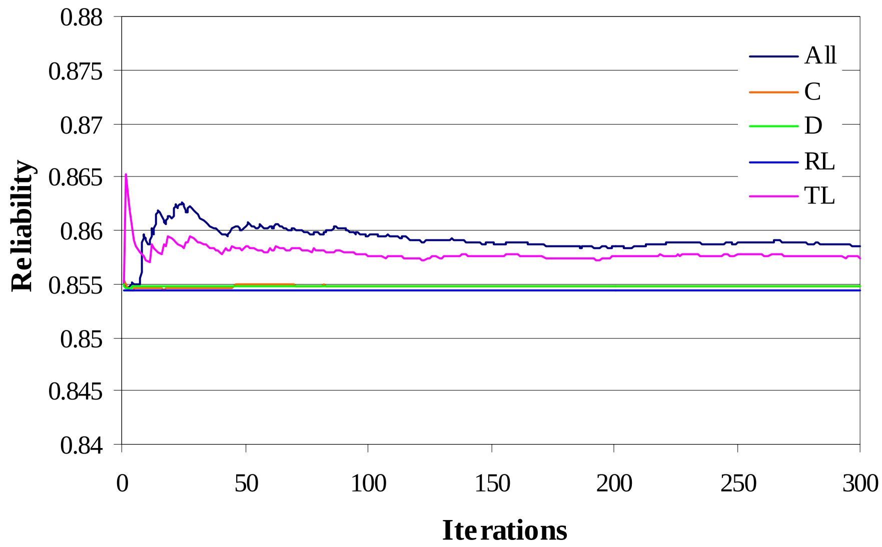
Data Generation Method	Monte Carlo Method										Bootstrap Method									
Pipe failure probability	**GEF**					**Poisson**					**GEF**					**Poisson**				
Random Variable (RV)	**C**	**D**	**RL**	**TL**	**All**	**C**	**D**	**RL**	**TL**	**All**	**C**	**D**	**RL**	**TL**	**All**	**C**	**D**	**RL**	**TL**	**All**
	STEADY STATE ANALYSIS																			
	96.40	96.40	96.47	96.30	96.28	96.67	96.66	96.69	96.57	96.57	96.37	96.41	96.47	96.37	96.33	96.64	96.66	96.69	96.63	96.61
Time	**EXTENDED PERIOD SIMULATION**																			
0th hour	86.95	87.11	88.99	86.96	86.72	90.22	90.30	90.99	90.56	90.03	86.83	86.93	88.99	87.21	86.68	90.18	90.26	90.99	90.64	90.01
6th hour	83.06	82.98	82.62	83.53	82.17	88.00	87.90	87.98	88.05	87.62	82.83	82.89	82.65	83.73	81.88	87.87	87.81	87.97	88.18	87.53
12th hour	73.92	73.84	74.86	74.23	72.93	82.63	82.25	83.71	82.44	82.05	73.71	73.59	75.10	74.35	72.76	82.48	82.17	83.81	82.52	82.11
18th hour	78.00	78.65	77.84	77.91	77.43	84.71	85.09	85.22	85.07	84.46	77.79	78.44	78.04	78.22	77.09	84.47	85.01	85.25	85.27	84.38
24th hour	80.57	79.96	79.82	80.49	79.50	86.45	86.56	87.25	86.73	86.20	80.40	79.91	79.89	80.68	79.18	86.31	86.55	87.28	86.81	86.13

C = Pipe roughness
D = Junction demand
TL = Tank level
RL = Reservoir level
All = All above four as RVs

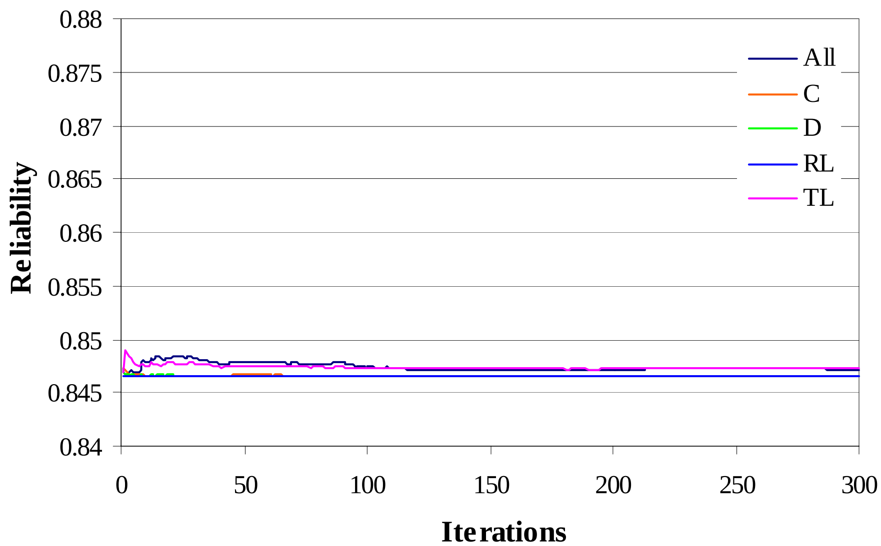
Table 5.6: Summary of Reliability Values (%) for Junction J-116 of Al-Khobar Network

Data Generation Method	Monte Carlo Method										Bootstrap Method									
Pipe failure probability	GEF					Poisson					GEF					Poisson				
Random Variable (RV)	C	D	RL	TL	All	C	D	RL	TL	All	C	D	RL	TL	All	C	D	RL	TL	All
	STEADY STATE ANALYSIS																			
	99.98	99.98	99.98	99.98	99.98	99.98	99.98	99.98	99.98	99.98	99.98	99.98	99.98	99.98	99.98	99.98	99.98	99.98	99.98	99.97
Time	EXTENDED PERIOD SIMULATION																			
0th hour	93.56	93.94	97.83	93.61	93.56	95.13	95.28	96.76	95.15	95.11	93.63	93.81	97.83	93.66	93.56	95.12	95.24	96.76	95.13	95.01
6th hour	89.79	88.85	86.63	88.18	89.61	94.18	93.65	92.74	93.27	94.14	89.99	88.83	86.67	88.39	89.70	94.27	93.69	92.75	93.37	94.15
12th hour	89.67	88.14	87.85	88.14	88.68	93.63	92.87	92.81	92.81	93.35	89.76	88.07	87.90	88.38	88.36	93.68	92.96	92.82	92.81	93.27
18th hour	88.87	88.29	88.29	87.74	88.74	92.84	92.43	92.55	92.14	92.75	89.14	88.37	88.31	87.81	88.54	92.95	92.59	92.55	92.19	92.76
24th hour	87.56	87.17	85.68	86.87	87.48	92.49	92.41	91.90	91.87	92.73	87.72	87.27	85.72	86.93	87.19	92.56	92.57	91.93	91.89	92.66

C = Pipe roughness
D = Junction demand
TL = Tank level
RL = Reservoir level
All = All above four as RVs

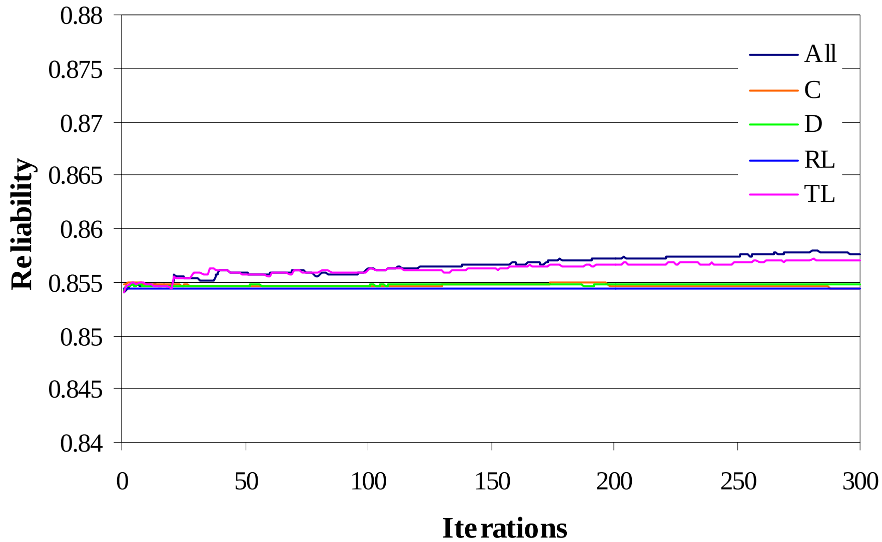


(a)

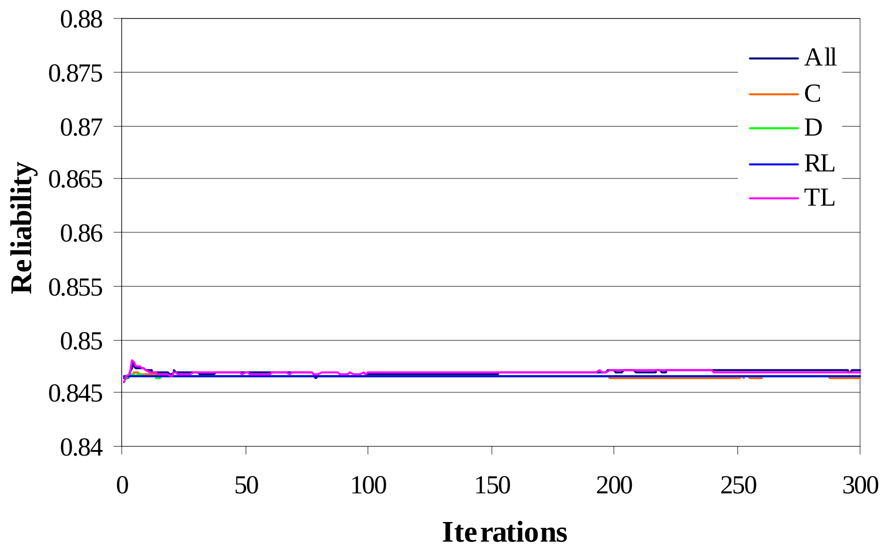


(b)

Figure 5.5: Reliability variation of junction J-16 under steady state condition for (a) Case 1, (b) Case 2, (c) Case 3 and (d) Case 4

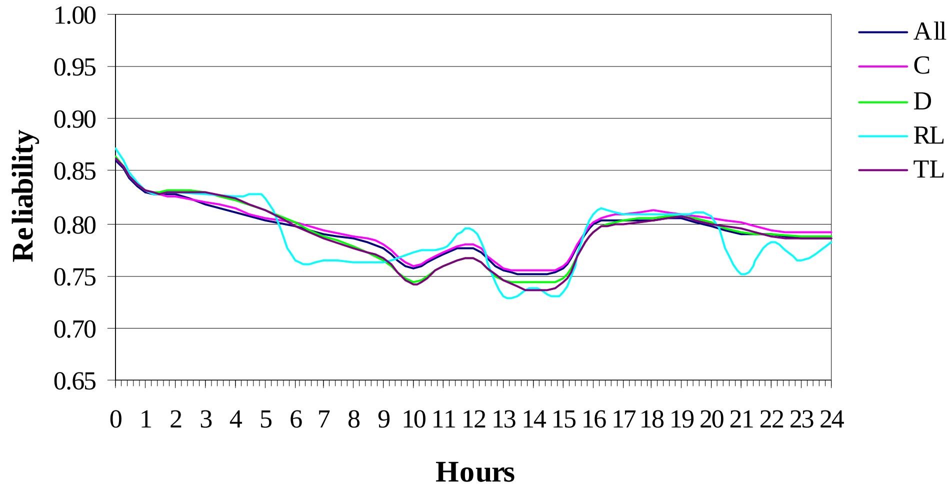


(c)

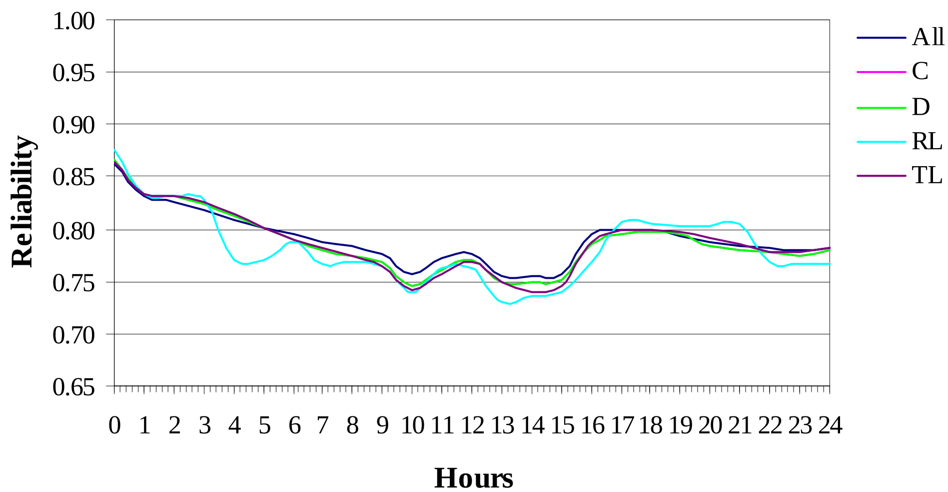


(d)

Figure 5.5: "Continued"

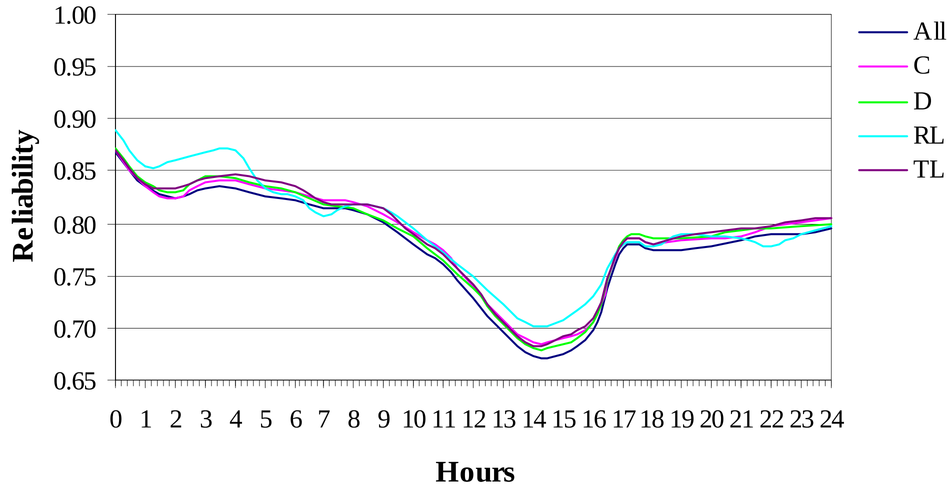


(a)

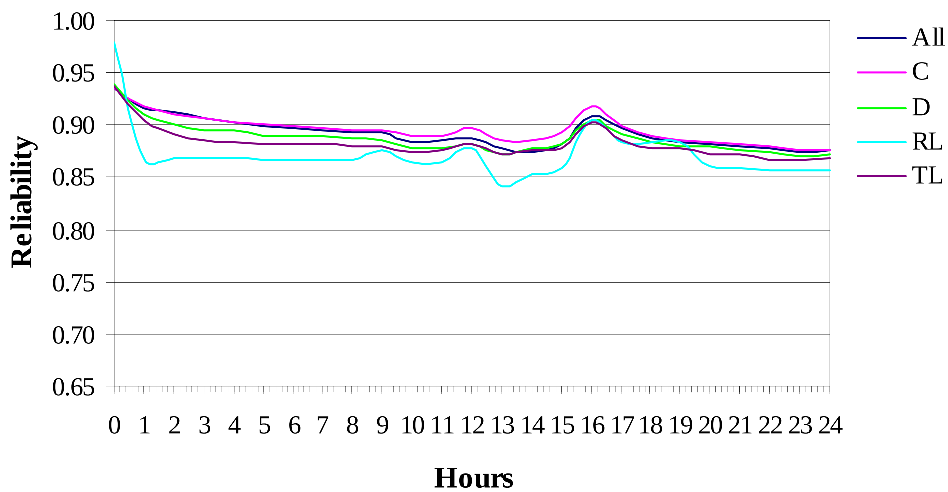


(b)

Figure 5.6: Reliability variation during the 24-hr period of Case 5 for junctions (a) J-14, (b) J-16, (c) J-31 and (d) J-116

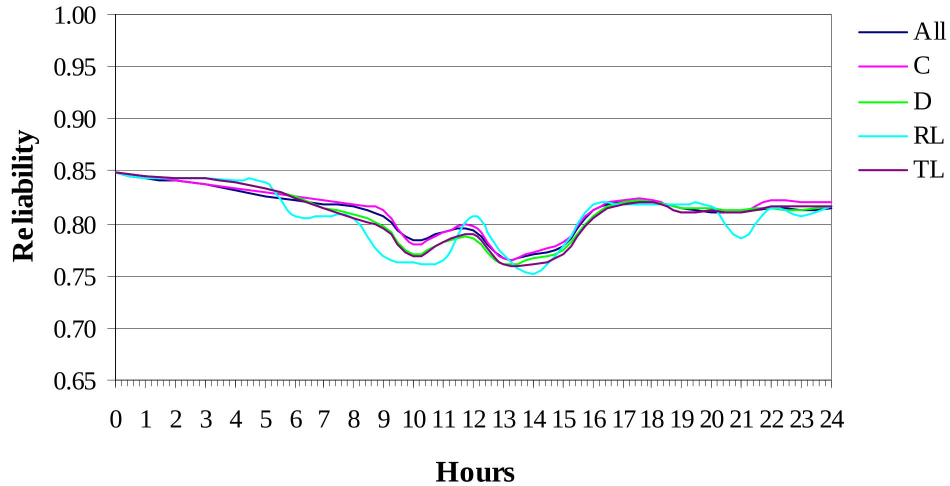


(c)

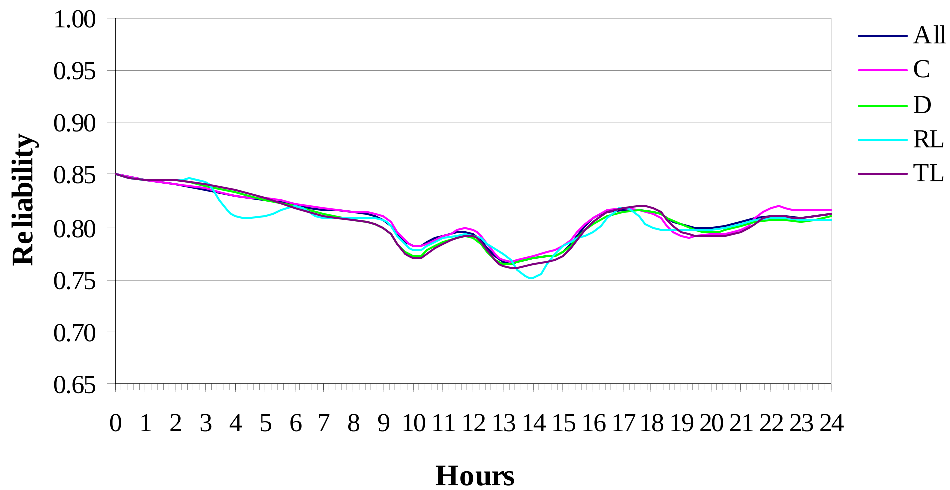


(d)

Figure 5.6: "Continued"

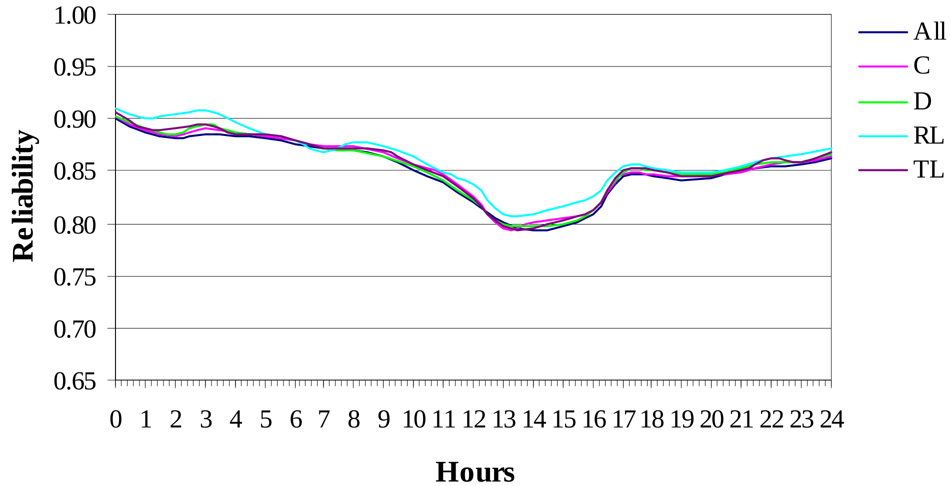


(a)

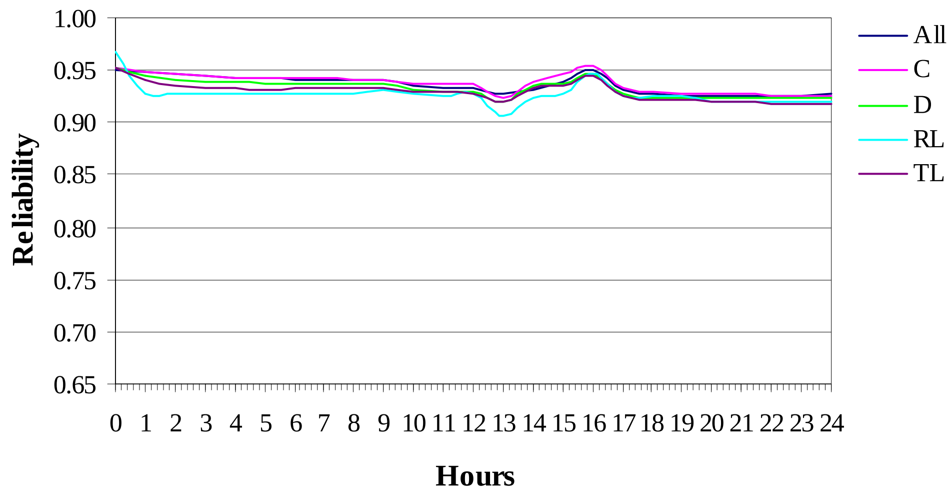


(b)

Figure 5.7: Reliability variation during the 24-hr period of Case 6 for junctions (a) J-14, (b) J-16, (c) J-31 and (d) J-116

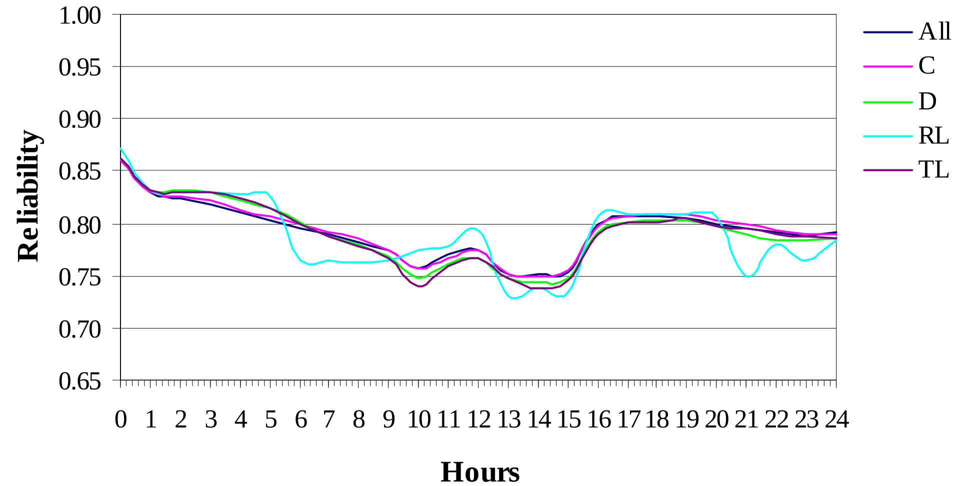


(c)

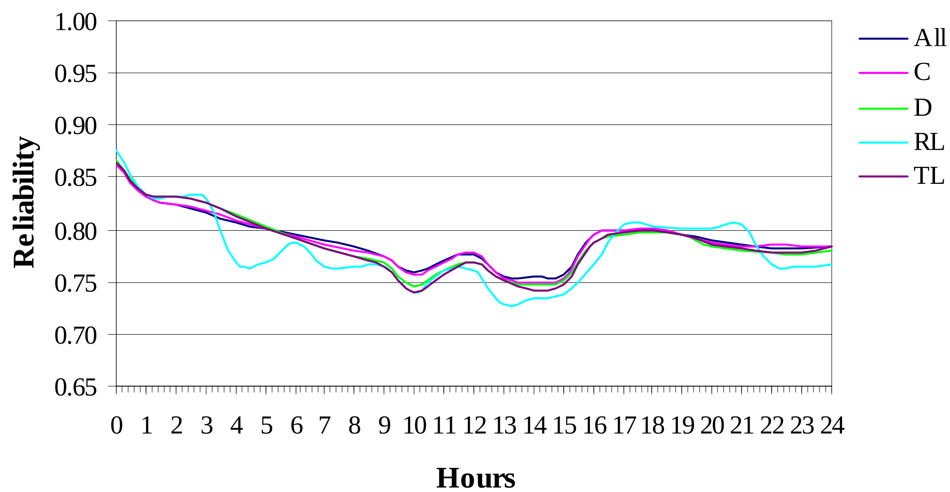


(d)

Figure 5.7: “Continued”

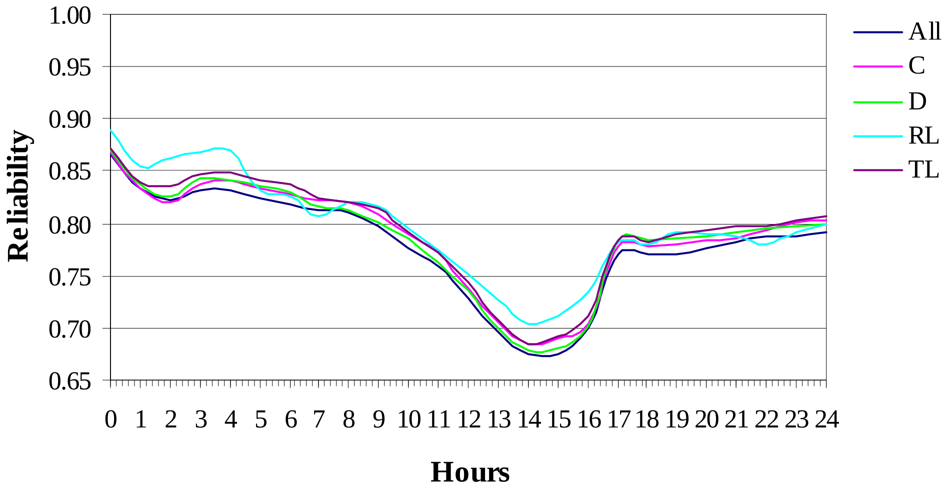


(a)

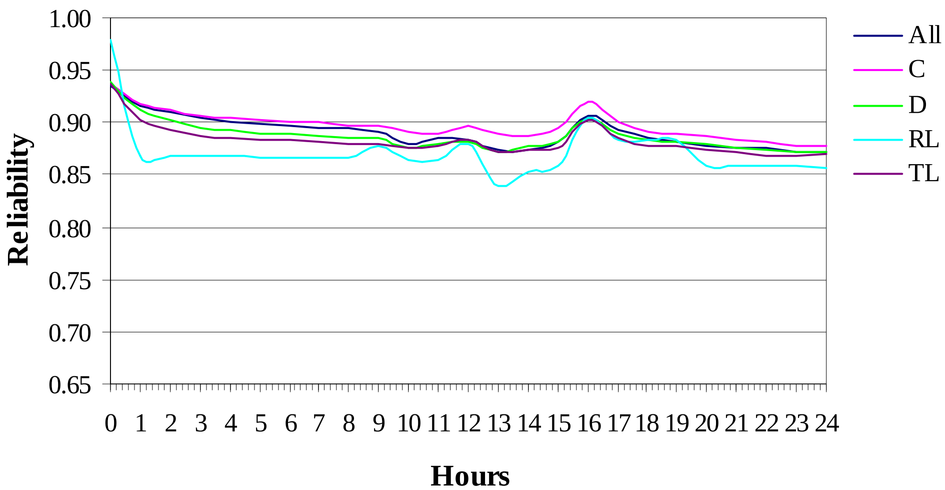


(b)

Figure 5.8: Reliability variation during the 24-hr period of Case 7 for junctions (a) J-14, (b) J-16, (c) J-31 and (d) J-116

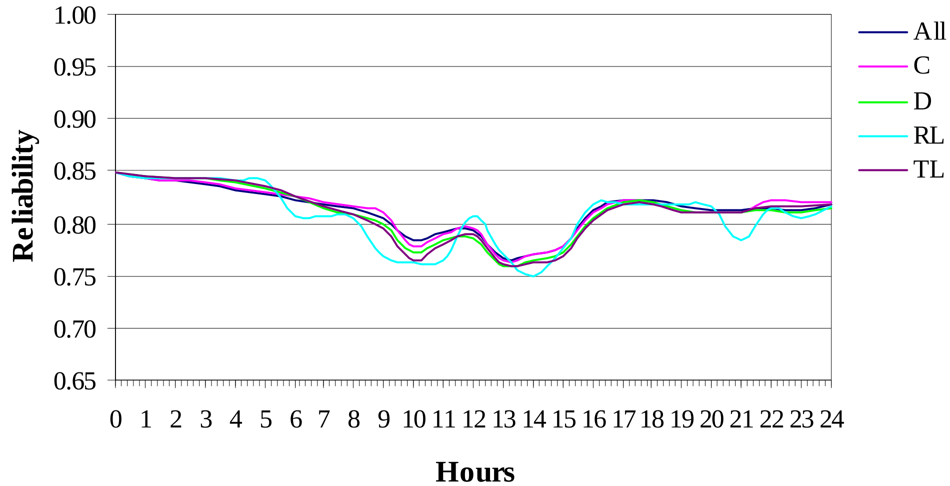


(c)

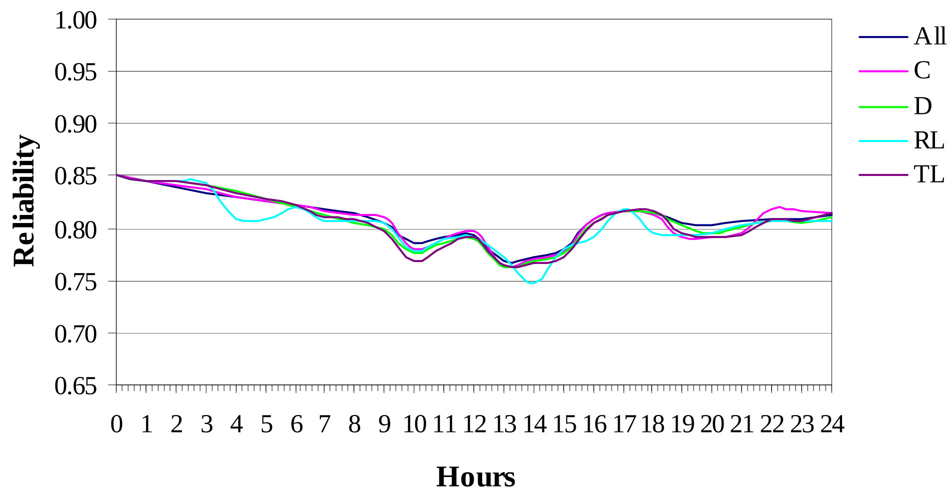


(d)

Figure 5.8: "Continued"

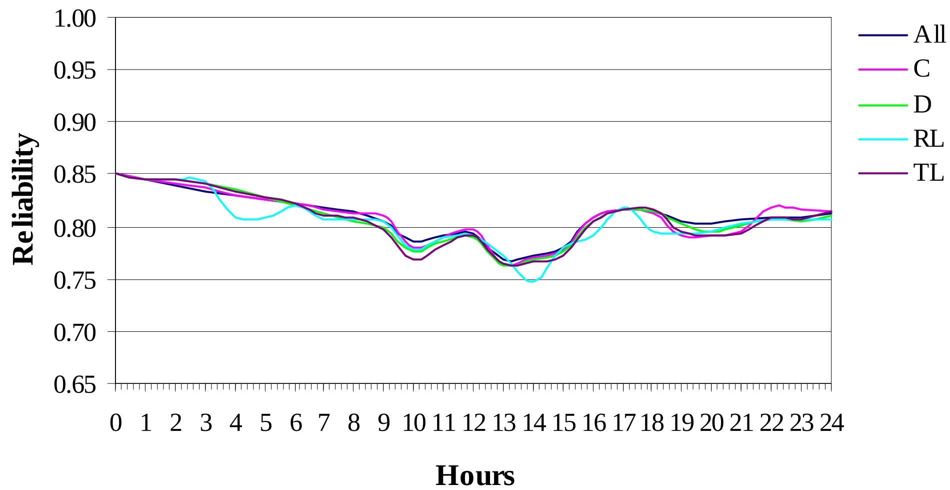


(a)

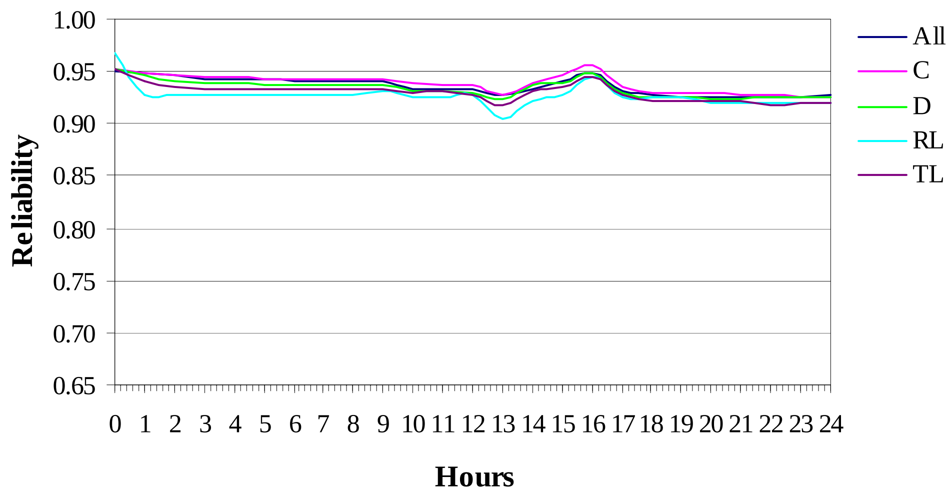


(b)

Figure 5.9: Reliability variation during the 24-hr period of Case 8 for junctions (a) J-14, (b) J-16, (c) J-31 and (d) J-116



(c)



(d)

Figure 5.9: "Continued"

By comparing the reliability values of junctions J-14, J-16, J-31 and J-116 obtained from Tables 5.3 to 5.6 respectively, it is observed that for steady state and extended period reliability analysis, the reliability values for all these selected junctions are affected by the method of calculating pipe failure probabilities, while the input data generation methods do not have much significant effect.

For steady state analysis, it is observed that for junctions J-14 and J-16 the Poisson Method gives lower reliability values as compared to the GEF Method, while for junction J-31 GEF gives lower reliability values as compared to the Poisson Method. For junction J-116, reliability values are the almost same from both methods. This is due to the fact that the Poisson method requires only annual break rate of pipe to calculate pipe failure probability, while the GEF method considers many parameters, such as randomness in pipe roughness, pipe diameter, and number of breaks in the pipe, and repair and replacement costs of the pipe.

A minimum cut-set approach is adopted for determining reliability, which requires failure probability of pipes selected in pipe closure combinations for determining "cut-sets failure probability". Since Al-Khobar water distribution system is a large network, fifty pipe closure combinations were randomly selected; therefore pipe failure probabilities significantly affect the reliability values.

For steady state analysis, it is observed that for junctions J-14 and J-16, the reliability values calculated by considering all four input parameters (i.e. pipe roughness, junction demand, reservoir water level and tank water level) collectively as random variables are higher than the case when only one input parameter is considered as the

random variable. This means that reliability can also be significantly affected by considering only one input parameter as the random variable. For junction J-31, the collective effect of random variables is more significant than the independent effect, which is similar to the case of the hypothetical network. For junction J-116, as shown in Table 4.6, the reliability is around 99.98 % for all cases of steady state analysis. This is because of high pressure at this junction.

For extended period analysis, it is observed from Tables 5.3 to 5.6 that for junctions J-14, J-16 and J-116, the Poisson Method give lower reliability values as compared to GEF Method, while for junction J-31 GEF gives lower reliability values as compared to Poisson Method. The reason is that the GEF and Poisson methods of calculating pipe failure probability are different. The Poisson method requires only annual break rate of pipe, while the GEF method considers many parameters, such as randomness in pipe roughness, pipe diameter, and number of breaks in the pipe, and repair and replacement costs of the pipe.

From Figures 5.6 to 5.9, it is observed that during 24 hours for all four selected junctions, the reliability values are maximum at 0^{th} hour and minimum around 13^{th} to 15^{th} hour. The reason for this decrease in reliability values is that when some of the pipes that are connected to sources are closed, the nodal pressures start to drop suddenly; therefore these pipe closure combinations cause lower pressures than the mean pressure, resulting in lower reliability values.

For junction J-14, it is observed that the reliability value is highest at 0^{th} hour for Case 5 and is equal to 86.39 % when “D” is considered as an independent random

variable, while it is minimum for Case 5 & Case 7 at 13^{th} hour and is equal to 72.5 % when "RL" is considered as an independent random variable. Since two reservoirs "W6" and "W7" are located near junction J-14, as shown in Figure 4.2, the water levels in these reservoirs have some direct effect on the reliability values of junction J-14.

For junction J-16, it is observed that the reliability value is highest at 0^{th} hour for Case 5 & Case 7 and is equal to 87.5 % when "RL" is considered as an independent random variable, while it is minimum for Case 5 & Case 7 at 13^{th} hour and is equal to 72.5 %. Since junction J-16 is located near the reservoirs, the reservoir water levels have some direct effect on the reliability values of junction J-16.

For junction J-31, it is observed that the reliability value is highest at 0^{th} hour for Case 6 & Case 8 and is equal to 90.99 % when "RL" is considered as an independent random variable, while it is minimum for Case 5 & Case 7 between 14^{th} and 15^{th} hour and is equal to 67.5 % when "All" input variables collectively are considered as random variables. Since junction J-31 is located away from the tanks and reservoirs, it is not directly affected by them. But, collectively "All" random variables slightly affect the reliability value.

For junction J-116, it is observed that the reliability value is highest at 0^{th} hour for Case 5 & Case 7 and is equal to 97.83 % when "RL" is considered as an independent random variable, while it is minimum for Case 5 & Case 7 at 13^{th} hour and is equal to 84.0 % when reservoir water level "RL" is independently considered as the random variable. Since junction J-116 is located near the reservoirs "W19" and "W20", as shown in Figure 4.2, reliability values of junction J-116 are directly affected by these reservoirs.

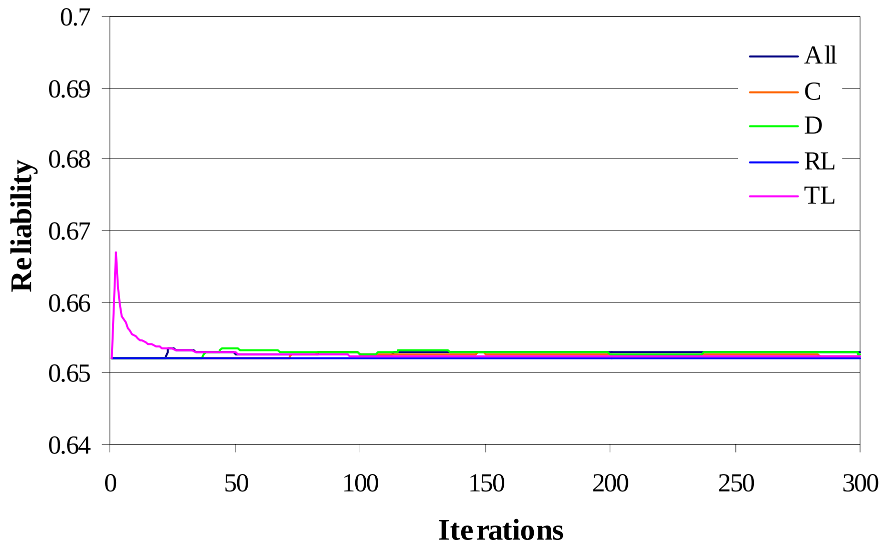
It is observed that values of reliability of junctions are dependent on the range of their pressures. For instance, junction J-14, which is under the lower pressure range, has lower reliability values as compared to junction J-31, which is under moderate pressure range. Since junction J-116 has the highest pressure range, its reliability values are highest among all of the junctions.

5.2.2 System Reliability

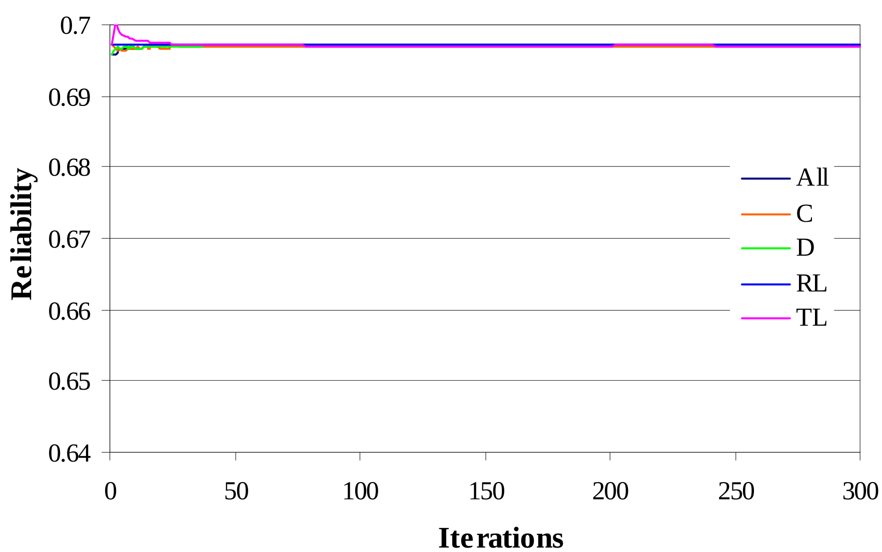
The system reliability values calculated for the Al-Khobar network considering all the cases are summarized in Table 5.7. The system reliability plots for steady state analysis are shown in Figure 5.10, while the plots for the mean reliability values versus the 24 hours duration are shown in Figure 5.11. The graphs are plotted between the moving average of reliability values and the number of iterations for the extended period reliability analysis for 0^{th}, 6^{th}, 12^{th}, 18^{th} and 24^{th} hour.

Table 5.7: Summary of System Reliability Values (%) of Al-Khobar Network

Data Generation Method	**Monte Carlo Method**										**Bootstrap Method**									
Pipe failure probability	**GEF**					**Poisson**					**GEF**					**Poisson**				
Random Variable (RV)	**C**	**D**	**RL**	**TL**	**All**	**C**	**D**	**RL**	**TL**	**All**	**C**	**D**	**RL**	**TL**	**All**	**C**	**D**	**RL**	**TL**	**All**
	STEADY STATE ANALYSIS																			
	65.24	65.27	65.21	65.23	65.28	69.68	69.71	69.72	69.70	69.68	65.28	65.27	65.22	65.22	65.28	69.69	69.70	69.73	69.69	69.68
Time	**EXTENDED PERIOD SIMULATION**																			
0^{th} hour	66.14	66.36	67.29	65.62	69.64	69.64	69.65	69.61	69.64	69.64	66.07	66.26	67.29	65.70	65.60	69.65	69.65	69.61	69.64	69.64
6^{th} hour	83.08	82.61	83.10	83.10	78.47	79.15	78.78	79.15	79.15	78.47	83.08	82.51	83.10	83.10	81.80	79.15	78.71	79.15	79.14	78.30
12^{th} hour	65.22	65.22	65.22	65.22	67.40	67.40	67.40	67.40	67.40	67.40	65.22	65.22	65.22	65.22	65.22	67.40	67.40	67.40	67.40	67.40
18^{th} hour	65.22	65.31	65.22	65.22	67.44	67.40	67.48	67.40	67.40	67.44	65.22	65.31	65.22	65.22	65.37	67.40	67.48	67.40	67.40	67.52
24^{th} hour	65.22	68.49	65.22	65.22	70.26	67.40	69.59	67.40	67.40	70.26	65.22	69.39	65.22	65.22	69.73	67.40	70.20	67.40	67.40	70.52

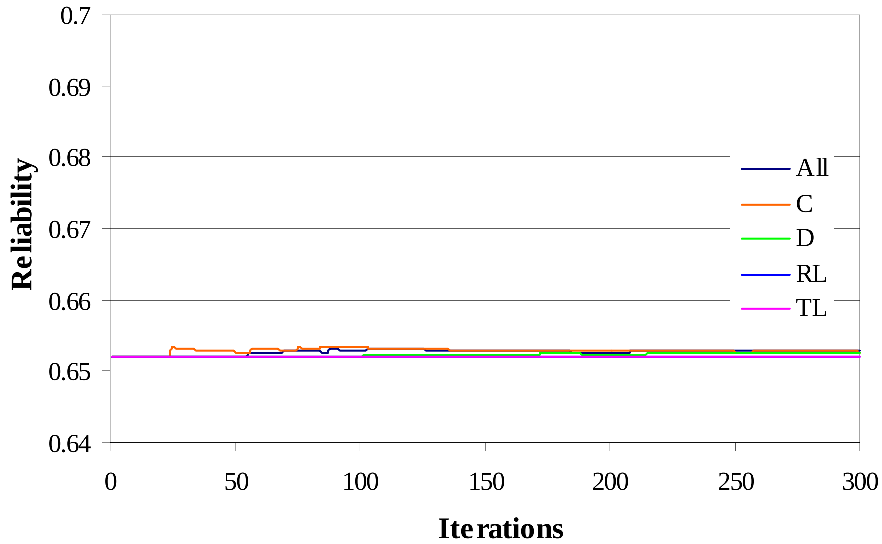


(a)

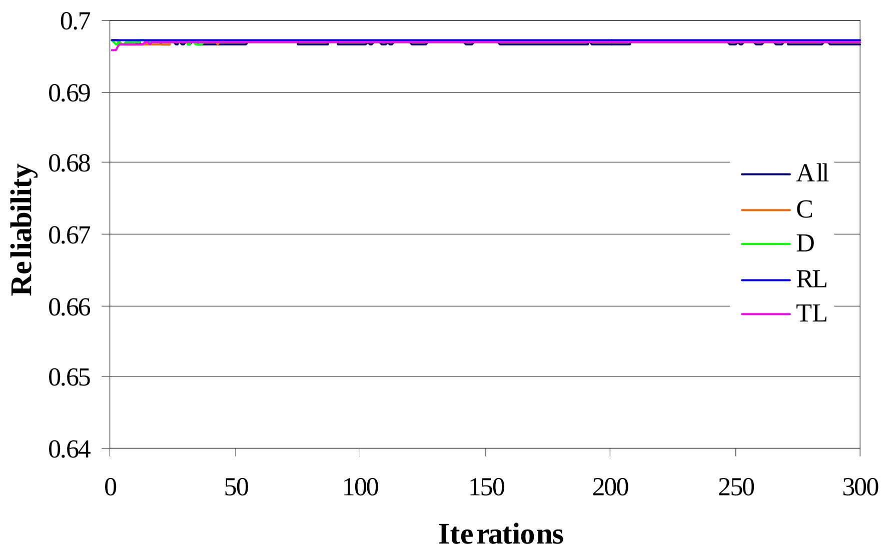


(b)

Figure 5.10: System reliability variation of Al-Khobar Network for (a) Case 1, (b) Case2, (c) Case 3, (d) Case 4

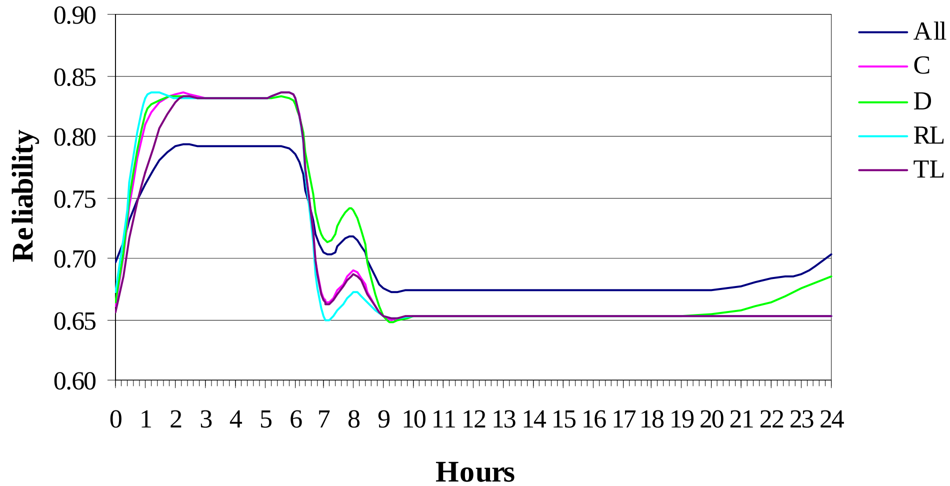


(c)

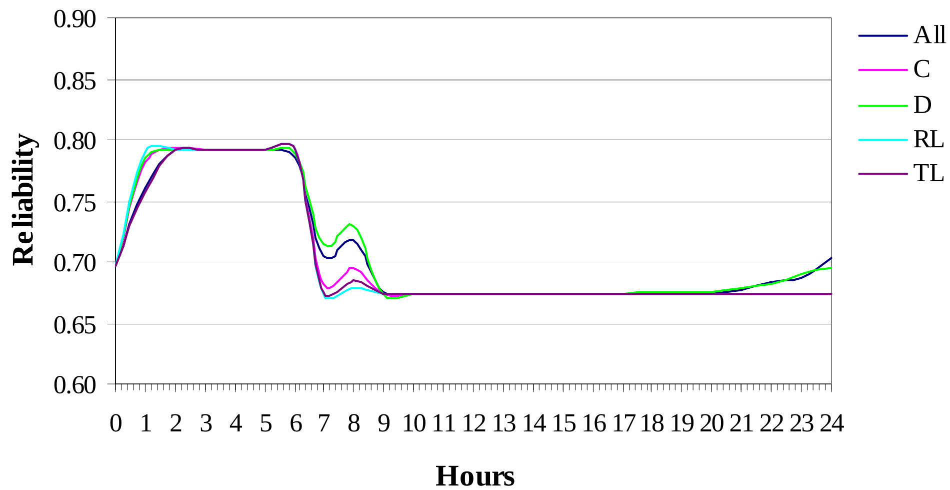


(d)

Figure 5.10: "Continued"

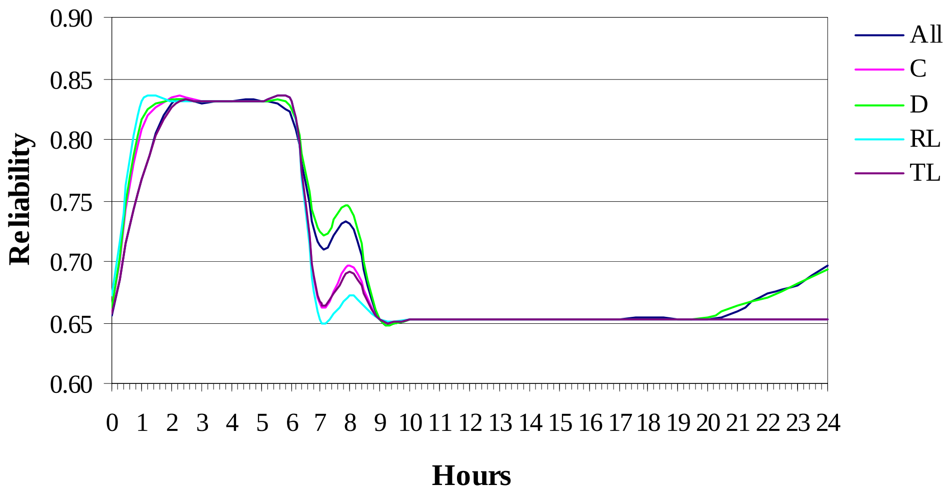


(a)

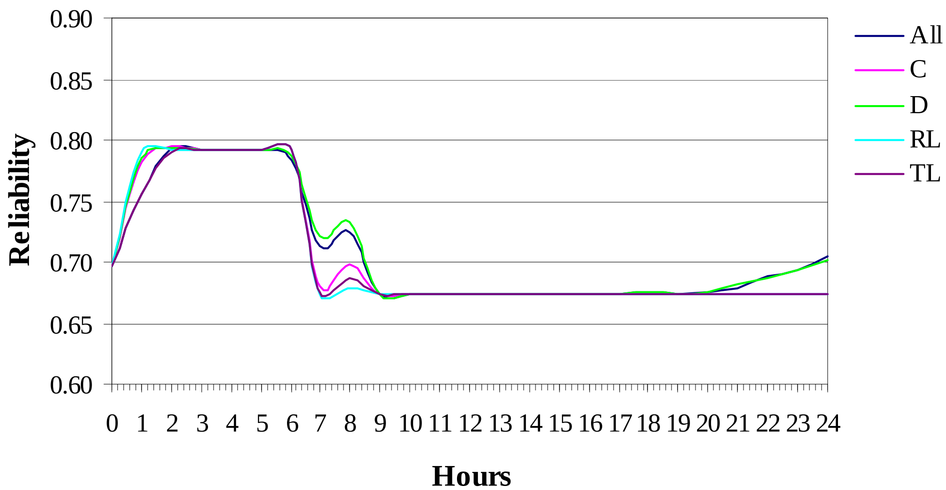


(b)

Figure 5.11: System reliability variation of the Al-Khobar network during 24 hours for (a) Case 5, (b) Case 6, (c) Case 7 and (d) Case (8)



(c)



(d)

Figure 5.11: "Continued"

By comparing the system reliability values of Al-Khobar network, as shown in Table 5.7, for steady state and extended period reliability analysis, it is observed that the system reliability values are affected by the method of calculating pipe failure probabilities. Since two different methods (i.e. Poisson and Generic Expectation Function (GEF) method) are used to calculate pipe failure probabilities, the pipe failure probabilities calculated by these methods are different. The reason is that the Poisson Method only requires the expected number of failures per year for calculating pipe failure probabilities, while the GEF method requires many parameters, such as randomness in pipe roughness, pipe diameter, and number of breaks in the pipe, and repair and replacement costs of the pipe.

It is observed from Table 5.7 that the system reliability values are somewhat sensitive to the method adopted for generating random input data (either Monte Carlo or Bootstrapping). Although both methods were assigned the same ranges of model input parameters, for the Al-Khobar network the reliability values are affected by the method adopted for generating random input data. This means that the Al-Khobar network is somewhat sensitive to the assigned ranges of model input parameters.

For steady state reliability analysis results, as shown in Tables 4.18 and 5.7, it is observed that the minimum reliability value is 65.21 % for Case 1 when reservoir water level “RL” is considered independently as the random variable, which means that there is a probability of 65.21 % that the pressure at all junctions of the Al-Khobar network will be greater than or equal to 33 psi, or there is a risk of (1-0.6521)*100 = 34.8 % that the pressure at all junctions would be less than 33 psi.

For extended period reliability analysis results, it is observed from Figure 5.11 that, for all cases, the system reliability values are highest and almost constant from the 1^{st} to the 6^{th} hour, while they are lowest and constant between the 9^{th} and the 20^{th} hours. The reason for this decrease in system reliability values is that when some of the pipes that are connected to sources are closed the nodal pressures start to drop suddenly. Therefore between the 9^{th} and the 20^{th} hour there are a larger number of “cut-sets” than in the early hours, which results in lower reliability values.

It is observed from Figure 5.11(a) that for Case 5, when the input data is generated by the Monte Carlo method and pipe failure probability calculated by Generic Expectation Method, there is a significant effect of independent random variables on reliability values. For instance, from 1^{st} to 6^{th} hour, the reliability values are almost constant for independent random variables (i.e. pipe roughness, junction demand, reservoir water level and tank water level) while the combined effect “All” gives a lower reliability value. But from 10^{th} to 21^{st} hour, the opposite is true and the collective effect “All” of random variables gives a higher reliability value as compared to the independent effects. It is observed that from 21^{st} to 24^{th} hour, there is a slight increase in reliability values for the case when junction demand “D” is independently considered, and also for the case when “All” input parameters are considered as random variables. The reason for this sudden increase and decrease in reliability values is the effect of pipe closures on junction pressures which ultimately influence the reliability values. By comparing Figures 5.11 (a) to 5.11 (d), it can be observed that the increase in reliability values from 21^{st} to 24^{th} hour is independent of the methods adopted for generating input data or methods adopted to calculate pipe failure probabilities. It is observed from Figure 5.11 that for the Al-Khobar network, the

random variables “C”, “D”, “RL” and “TL” have their individual effect on reliability values between the 7^{th} and 9^{th} hours.

For instance, referring to Figure 4.2 and Table 4.19, when pipes P-160, P-111 and P-20 are closed, it is observed that this pipe closure combination results in reduced pressures at almost all the junctions of the Al-Khobar network. The reason is that pipe P-20 is connected to a main pipe YRMK-900 coming directly from Yarmok tank. Similarly, all pipe closure combinations have their individual effect on junction pressures, which ultimately affects the reliability values.

By performing the hydraulic reliability analysis on the Al-Khobar water distribution system, it is concluded that there is a probability of 65.0 % to 70.0 % that the pressure at all junctions of the Al-Khobar network will be greater than or equal to the mean pressure of 33.0 psi. From the literature (Shinstine et al. 2002), it is seen that for a Tucson (Arizona, USA) water distribution system of 109 pipes and 89 nodes, the system reliability calculated by using the minimum cut-set method comes out to be 96.0 %. The reason for this higher reliability value is the selection of a lower mean nodal pressure of 20.0 psi and lower pipe failure rates as compared to Al-Khobar network.

It is observed that the main cause of lower reliability values in the Al-Khobar network is the failure of main pipes which are connected directly to the reservoirs and tanks. Therefore, in order to improve the reliability, it is necessary to provide precautionary measures by providing alternative pipes leading to the sources so that in case of failure these alternative pipes could be used immediately.

Chapter 6

CONCLUSIONS AND RECOMMENDATIONS

The main purpose of this research was to calculate the hydraulic reliability of a water distribution system which requires extensive planning and maintenance in order to ensure that it will provide consumers with safe water in adequate amounts at all times. Hydraulic reliability of water networks comprises junction and system reliability; therefore reliable networks should fulfill the minimum pressure head requirements at all the junctions of the network so that consumers can meet their water demands.

In this research, a hydraulic reliability model based on minimum cut-sets was developed using Generic Expectation Function (GEF) and compared with an alternate model based on Poisson method. The developed model considers input parameters (i.e. pipe roughness, junction demand, reservoir water level and tank water level) for hydraulic simulation as random variables. The developed model is applied to a hypothetical network and to the central part of Al-Khobar City water distribution system. The calculated system reliability values for Al-Khobar Water Distribution System range between 65.0 % to 70.0 %, which means that there is a probability of 65.0 % to 70.0 % that the pressure at all junctions of the Al-Khobar network will be greater than or equal to the mean pressure of 33.0 psi. If this reliability value is compared with the reliability value of Tucson, Arizona, U.S.A which is 96.0 %, it is observed that the Al-Khobar network is less reliable. The reason of high reliability value of Tucson water distribution network is the proper maintenance of the network by the private water business companies, as in U.S.A most of

the municipal water distribution networks are managed and maintained by private water business companies. In addition to the higher pipe failure rates, the reason of low reliability value of Al-Khobar network is the selection of high mean pressure. If the mean pressure of Al-Khobar network were reduced from 33 psi then the system reliability value would increase. The judgment of the acceptable system reliability values of Al-Khobar network can be made by looking at the complaints related to lower pressures of supplied water coming from consumers. For instance, if it is observed that there are complaints of low pressures from consumers residing in a particular area covering 30% of the water distribution system; this means that the pressures at this particular area are not meeting the minimum pressure requirements, therefore the probability that the minimum pressure requirements at all the junctions of the water distribution system are fulfilled would be 70%.

After performing the hydraulic simulation of Al-Khobar Water Distribution System, it was found that the main cause of low reliability values of Al-Khobar network is the failure of main pipes which are connected directly to the reservoirs and tanks. Therefore, in order to improve the reliability it is necessary to provide precautionary measures by providing alternative pipes leading to the sources so that in case of failure these alternate pipes could be used immediately. It is also suggested to properly maintain the pipes which are directly connected to the reservoirs and tanks and if necessary they should be replaced.

The results of this research can contribute greatly in assessing the hydraulic reliability of water distribution methods by two different approaches i.e Poisson and Generic Expectation Function. In addition, it will help the municipilties, Water and Rural Affairs,

and Water Authorities, to set guidelines for establishing reliability levels with respect to the pressure head requirements of the consumers that would be useful for satisfying the water requirements of the consumers, as presently there are no proper guidelines available in the literature to establish the reliability levels.

Based on this research, the following recommendations are made:

- A Law needs to be promulgated to set the hydraulic reliability levels of the water distribution system in all the metropolitan cities. In order to establish reliability levels, a "Code of Reliability Levels" needs to be developed by the consensus of the water and municipal authorities of the Kingdom. After developing the "Code of Reliability Levels", water authorities should be made responsible for implementing the code and monitoring the reliability levels of their respective water distribution systems.

- It is suggested that in all the metropolitan cities of the Kingdom, data regarding water distribution systems such as pipe break data, pipe repair and replacement costs, etc, should be documented properly, so that it can be easily accessible and available to the concerned authorities.

- Extension of the developed model is suggested in future by generating model input data by assuming different probability distributions other than normal distribution. Moreover, other random variables e.g. pump efficiency; can be considered to investigate their effects on the hydraulic reliability of the water distribution system.

- The developed model must be converted in the form of software using Graphical User Interface (GUI) to merge the developed codes with the hydraulic simulation software, i.e. EPANET, for easy use by the engineers and operators in the field.

REFERENCES:

Aklog, D., and Hosoi, Y. (2003), "Reliability-based optimal design of water distribution networks", *Water Science and Technology: Water Supply*, Vol. 3, No. 1-2, pp 11-18.

Ang, A.H.S., and Tang, W.H. (1984), "Probability concepts in engineering planning and design". Volume II: Decision, risk and reliability, Wiley, New York.

Bargiela, A., and Hainsworth, G. D. (1989), "Pressure and Flow Uncertainty in Water Systems", *ASCE Journal of Water Resources Planning and Management*, Vol. 115, No. 2 pp 212-229.

Bates, B.C (1988), "Nonlinear discrete flood event models.2: Assessment of statistical nonlinearity". *Journal of Hydrology,* Amsterdam, 99,pp. 77-89.

Billinton, R., and Allan, R. N (1983), "Reliability Evaluation of Engineering Systems,: Concepts and Techniques" , Pitman London.

Calvin, R, S., Yacov, Y.H., Duan, L., and James, H.L., (1996), "Capacity reliability of water distribution networks and optimum rehabilitation decision making". *Water Resources Research*, Vol. 32, No. 7 pp 2271-2278.

Cullinane, M.J., Lansey K. E., and Mays, L. W (1992), "Optimization availability-based design of water distribution networks", *ASCE Journal of Hydraulic Engineering*., Vol. 118, No. 3 pp 420-441.

Dominique, G., Bernard, C., Pierre, C., and Aurele, P. (1999), "Comparing two methods for addressing uncertainty in risk assessments", ASCE Journal of Environmental Engineering, Vol. 125, No. 7, pp 660-666.

Efron, B. (1979), "Bootstrap Methods: Another look at the jackknife", *Annals of Statistics,* Vol. 7, No. 1, pp 1-26.

Efron, B, and Tibshirani, R. J. (1993), "An Introduction to Bootstrap", Monographs of Statistics and Applied Probability, No. 57. Chapman and Hall, London. 436 pp.

El Maalouf, S. and Young, C.K (1992), "Reliability of Algorithms for Water Quality Analysis in Hydraulic Networks", In: Coping with Scarcity and Abundance, Los Angeles, California, U.S.A, pp 887-892

EPANET (2000), Hydraulic Simulation Software developed by *Drinking Water Research Division, Risk Reduction Engineering Laboratory, U.S Environmental Protection Agency, Cincinati, Ohio.*

Ezell, B.C., Farr, J.V., and Weise, I. (2000), "Infrastructure Risk Analysis of Municipal Water Distribution Systems," *ASCE Journal of Infrastructure Systems*, Vol. 6, No. 3, pp 118-122.

Fujiwara, O., and De Silva, A.U (1990), "Algorithm for reliability based optimum design of water networks," *ASCE Journal of Environmental Engineering*, Vol. 116, No. 3, pp 75-87.

Ganoulis, J.G. (1991) "Risk Analysis in Water Resources Engineering: Development and Application," Conference Proceeding, Part of "*Risk-Based Decision Making in Water Resources V*", pp 1-10.

Germanopoulos, G., Jowitt, P.W., and Lumbers, J. P., (1986), "Assessing the reliability of supply and level of service for water distribution systems", *Proc. Instn Civ. Engrs*, Part 1, Water Engineering Group, pp 413-428.

Glenford, J.M. (1976) *Software Reliability: Principles and Practices*, John Wiley & Sons, New York , pp 7-14

Goderya, F.S and Adelman, D.D (1996), "A Risk/Cost Analysis of Hazardous Waste Sites", Conference Proceeding, Part of: *After the Rain Has Fallen: Ground Water Management Symposium*, Memphis, Tennessee, U.S.A, pp 45-50.

Goulter, I.C. (1995) "Analytical and simulation models for reliability analysis in water distribution systems", In: E. Cabrera and A. Vela (eds), Improving Efficiency and Reliability in Water Distribution Systems, Kluwer Academic Publishers, Dordrecht, 235-266.

Goulter, I.C. (1999), "Reliability and Risk in a Water Supply System Emphasizing Drought Periods", In: E. Caberra and J. Garcia-Serra (eds), Drought Management Planning in Water Supply Systems, Kluwer Academic Publishers, Dordrecht, pp 128-147.

Goulter, I.C and Coals (1986), "Quantitative approaches to reliability assessment in pipe networks", *ASCE Journal of Transportation Engineering*, Vol. 112, No.3, pp 287-301.

Haan C. T (1977), "Statistical Methods in Hydrology", Iowa State University Press, Ames , Iowa.
Haan, C. T., Barfield, B.J., and Hayes, J.C (1994), "Design Hydrology and Sedimentology for Small Catchments", Academic Press , San Diego.

Haestad Methods Inc. (2001), "Water Distribution Modeling Exam Workbook", 1st Edition, Haestad Press, Waterbury, CT, USA.

Johnson, R. A. (1994). “Conditional Probability”. In: *Miller and Freund’s Probability and Statistics for Engineers*, 5th Edition, Prentice-Hall International, Inc., pp-71-74.

Kaufmann, A. D., Grouchko, D., and Croun, R. (1977). *Mathematical Models for the Study of the Reliability of Systems*. Academic Press, New York, N.Y.

Kleiner, Y. and Rajani , B. B. (1999). “Using limiting data to assess future needs.” *Journal of the American Water Works Association,* Vol. 91, No.7,pp.47-62.

Kumar, A., Kansal, M,L., and Kumar, S., (1996), “Stochastic hydraulic reliability of a water distribution network”, Stochastic Hydraulics’ 96,Tickle, Goulter, Xu, Wasimi & Bouchart (eds), Balkema, Rotterdam.

Lansey, K.E., Duan, N., and Mays, L.W., (1989), “Water Distribution System Design Under Uncertainties”, *ASCE Journal of Water Resources Planning & Management*, Vol. 115, No. 5, pp 630-644.

Lapin, L. L. (1997). “Probabilities for compound events”. In: *Modern Engineering Statistics*, Duxbury Press, Washington, pp 162-167

Lee, H. L and Mays, L. W. (1986) , “Hydraulic Uncertainties in Flood Levee Capacity”, *ASCE Journal of Hydraulic Engineering*., Vol. 112, No. 10 pp 928-934.

Loganathan, G. V., Park, S., and Sherali, H. D. (2002). “Threshold Break Rate for Pipeline Replacement in Water Distribution Systems.”, *ASCE Journal of Water Resources Planning and Management*, Vol. 128, No. 4 pp 271-279.

MATHEMATICA (1999), Windows based mathematical computational software developed by *Wolfram Research Inc.* U.S.A

MATLAB (2001), Technical computing software developed by *The MathWorks Inc. U.S.A*

Mays, L. W. (1993), “Methods for Risk and Reliability Analysis,” Conference Proceeding, *Risk-Based Decision Making in Water Resources VI*, Santa Barbara, California, U.S.A, pp 26-44.

Mays, L. W., and Tung, Y. K. (1992), “Hydrosystems Engineering and Management”, McGraw-Hill, New York.

Plate, E. J. (1993), “Some Remarks on the Use of Reliability Analysis in Hydraulic Engineering Applications”, *Reliability and Uncertainty Analysis in Hydraulic Design*, pp 5-15.

Ostfeld, A. (2001), “Reliability Analysis of Regional Water Distribution Systems,” *Urban Water* 3, pp 253-260.

Quimpo, R.G. (1996), “Reliability Analysis of Water Distribution Systems”, In: *Risk-Based Decision Making in Water Resources, VII*, Ed. by Y. Y. Haimes, D. A. Moser and E. Z. Stakhiv, ASCE. New York, pp. 388-395, 1996.

Quimpo, R. G. and Shamsi, U. M. (1991), “Reliability-Based Distribution System Maintenanace”, *ASCE Journal of Water Resources Planning & Management*, Vol. 117, No. 3, pp 321-339.

Ross, S.M. (1985), “Introduction to Probability Models”, Academic Press, NewYork.

Rossman, L. A., (2000), “EPANET Users Manual”, *Drinking Water Research Division, Risk Reduction Engineering Laboratory, U.S Environmental Protection Agency, Cincinati, Ohio.*

Shamir, U. and Howard, C. D. D. (1979). “An analytical approach to scheduling pipe replacement.” *J. Am. Water Works Assoc.*, 71(5), 248-258.

Shinstine, D.S., Ahmed, I., and Lansey, K.E. (2000), “How Reliable are Water Distribution Networks”. *Conf. Proc: Building Partnerships Joint Conf. on Water Resourcses Engg & Water Resources Planning & Management*, Section. 49, Chapter 4 , Minneapolis , USA.

Shinstine, D.S., Ahmed, I. and Lansey, K.E. (2002), “Reliability/Availability Analysis of Municipal Water Distribution Networks: Case Studies,” *ASCE Journal of Water Resources Planning & Management*, Vol. 128, No. 2, pp 140-151.

Souza, R. and Chagas, P. (2001), “Risk Analysis in Water Quality Problems”, *Proceedings of the World Water and Environmnetal Resources*, Orlando, Florida, U.S.A, section 1, chapter 379.

STATISTICA (1997), Statistical computational software developed by *StatSoft Inc*, Oklohoma, U.S.A.

Stevens, E (1993), “A method to predict the accuracy of a first order approximation of model output variance.” PhD Thesis, Department of Biosystems and Agricultural Engineering, Oklahoma State University, Stillwater.

Su, Y.C, Mays, L.W., and Lansey, K.E (1987), “Reliability-based optimization models for water distribution systems,” *ASCE Journal of Hydraulic Engineering* ,Vol. 113, No.12, pp 1539-1556.

Tanyimboh, T.T., Burd, R., Burrows, R., and Tabesh, M. (1999), “Modeling and Reliability Analysis of Water Distribution Systems”. *Wat. Sci. Tech.* Vol. 39, No. 4 , pp 249-255.

Tung, Y. K (1990), “Mellin transform applied to uncertainty analysis in hydrology/hydraulics”, *ASCE Journal of Hydraulic Engineering*., Vol. 116, No. 5 pp 659-674.

Tung, Y. K., and Mays, L .W (1980), “Optimal risk-based design of water resources engineering projects.” *Tech Rep. No. 171,* Center for Research in Water Resources, University of Texas, Austin.

Tyagi, A. (2000), “*A simple approach to reliability, risk, and uncertainty analysis of hydrologic, hydraulic, and environmental engineering systems*.” PhD Thesis, Oklahoma State University, Stillwater.

Tyagi, A. and Haan C.T. (2001), “Reliabilty, Risk, and Uncertainty Analysis Using Generic Expectation Functions,” *ASCE Journal of Environmental Engineering,* Vol. 127, No. 10, pp 938-945.

U.S EPA (1999), *Risk Assessment Guidance for Superfund, Volume 3 Part A: Process for Conducting Probabilistic Risk Assessment-Appendix D: Estimating Uncertainty in the Mean Concentration*, Office of Solid Waste and Emergency Response, Draft.

Verdonck, F., Jaworska, J., Thas,O. and Vanrolleghem, P.A. (2000), “Uncertainty Techniques in Environmental Risk Assessment”, *Med. Fac. Landbouw. Univ. Gent,* 65/4, pp 247-252.

Walski, T. M. (1984). *Analysis of Water Distribution Systems.* Van Nostrand, New York.

Walski, T.M. (2003). “*Watertalk Forum*”, An online discussion forum by Haestad Methods, Inc, USA.

WaterCAD (2001), Hydraulic Simulation Software developed by *Haestad Methods Inc.* Waterbury, CT, USA.

Wong, H. S. and Yeh, W.G. (2002), “Uncertainty Analysis in Contaminated Aquifer Management”, *ASCE Journal of Water Resources Planning and Management*, Vol. 128, No. 1 pp 33-45.

Xu, C., and Goulter, I. (1996), “Uncertainty Analysis in Water Distribution Networks,” *Stochastic Hydraulics*’96 Tickle, Goulter, Xu, Wasimi & Bouchart (eds), Rotterdam, Balkema, pp 609-616.

Xu, C. and Goulter, I.C. (1997), "Reliability Assessment of Water Distribution Networks Using the First Order Reliability Method", *Conf. Proc: "Managing Water: Coping with Scarcity and Abundance"*, 27th Congress of Intl. Ass. For Hydraulic Research, San Francisco, CA, USA.

Xu, C. and Goulter, I.C. (1999), "Reliability-based optimal design of water distribution networks" *J. Water Resour. Plng. and Mgmt.*, 125(6), 352-361.

APPENDIX-A

(Matlab Code to Generate Random Values by Bootstrapping)

*****************Matlab Code to generate random values by Bootstrapping*****************

```
load Khobar1000tank.txt
cv=Khobar1000tank;
[r,c]=size(cv);

in = randint(r,1,[1 r]);
for i= 1:r
   for j = 2:c
      A(i,j-1) = cv(in(i),j);
   end
end

save khobartank.txt -ascii A

load Khobar1000junction.txt
cv=Khobar1000junction;
[r,c]=size(cv);

in = randint(r,1,[1 r]);
for i= 1:r
   for j = 2:c
      A(i,j-1) = cv(in(i),j);
   end
end

save khobarjunc.txt -ascii A
load Khobar1000Reservoir.txt
cv=Khobar1000Reservoir;
[r,c]=size(cv);

in = randint(r,1,[1 r]);
for i= 1:r
   for j = 2:c
      A(i,j-1) = cv(in(i),j);
   end
end

save khobarres.txt -ascii A

load Khobar1000tank.txt
cv=Khobar1000tank;
[r,c]=size(cv);

in = randint(r,1,[1 r]);
for i= 1:r
   for j = 2:c
      A(i,j-1) = cv(in(i),j);
   end
end

save khobartank.txt -ascii A
```

APPENDIX-B

(Program to Run EPANET for Applying Developed Methodology)

****** Program to Run EPANET for Applying Developed Methodology******

```
// SmallCodes.cpp : Defines the entry point for the console application.
//
#include <stdlib.h>
#include <string.h>
#include <afxcoll.h>  // for the use of CStringList class
#include "stdafx.h"
#include "SmallCodes.h"
#include <process.h>
#include <time.h>



#ifdef _DEBUG
#define new DEBUG_NEW
#undef THIS_FILE
static char THIS_FILE[] = __FILE__;
#endif

/////////////////////////////////////////////////////////////////////////////
// The one and only application object

CWinApp theApp;

using namespace std;

int getTotalCols(char* Filename);   // prototypes
char* getCols(char* Filename,int rowNumber);
int getTotalRows(char *Filename);
void FillValues(int current_row, int command);
int FindColNumber(char *Colname,char *Filename);
int getColNumber(char *Colname,char *String);
void Fill_pclosed(char* Filename);
int getNumberofCols(char *nameOfPipes);
void Edit_Pclosed(char *Filename, char *nameOfClosedPipes, char *cFileName);
void Edit_CValues(char *Filename, char *nameOfPipes, char *values);
void Edit_DValues(char *Filename, char *Junctions, char *values);
void Edit_RLs(char *Filename, char *Resers, char *values);
void Edit_TLs(char *Filename, char *Tanks, char *values);
void GenEdit(char *Filename,char *tagname, char *idname, char *colNames, char *values);
void myStrtok(char *Originalstring,char *returnString, char *seps); // my implementation of strtok
bool IsSeparater(char ch,char *seps);  // used in my implementation of strtok
void EditLine(char *newLine, char *values, int indexinValues, int col2Edit);
void Filecopy(char *Filename,char *newFilename); // copies file Filename to File newFilename
void Filecreate(char *Filename,char *String,int strigsize); // copies the whole string String into Filename
//////////////////GLOBALS ////////////////////////
/////////////////////////////////////////////////
// The rows returned by getCols are stored in C,D,TL and RL. All rows are strings instead of float.
char* C;
char* D;
char* TL;
```

```
char* RL;
#define default_row 1

CPtrList p_closed;
//////////////////////////////////////////////////////
//////////////GLOBAL ENDS///////////////////////////

int _tmain(int argc, TCHAR* argv[], TCHAR* envp[])
{
	int nRetCode = 0;

	// initialize MFC and print and error on failure
	if (!AfxWinInit(::GetModuleHandle(NULL), NULL, ::GetCommandLine(), 0))
	{
		// TODO: change error code to suit your needs
		cerr << _T("Fatal Error: MFC initialization failed") << endl;
		nRetCode = 1;
	}
	else
	{
		// TODO: code your application's behavior here.
		CString strHello;
		strHello.LoadString(IDS_HELLO);
		cout << (LPCTSTR)strHello << endl;



		/*****************  From Pipes closed file *****************/

/*		//// all rows in pclosed.dat are now in p_closed object.
	Fill_pclosed("D:\\Thesis work 19 March\\STATISCA Random Input\\Hypthetical Network 6th 
April\\Monte Carlo\\pclosed.dat");

	POSITION lineno;//,pipe;
	CString *list;
	for(lineno = p_closed.GetHeadPosition();lineno != NULL ;)
		{
			list = (CString *)p_closed.GetNext(lineno);
			list->FreeExtra();
			char *value = list->GetBuffer(1000);
			cout<<value;
		}
*/

	/**************************** Pipes closed file ***************/

/*



		char* Filename="D:\\Thesis work 19 March\\STATISCA Random Input\\Hypthetical 
Network 6th April\\Monte Carlo\\1000Hypo.txt";
		int column_no = FindColNumber("P9_C",Filename);
```

```
cout<<column_no;
float *values=getCols(Filename,2);
cout<<values[column_no]<<" ";  // print the element in 2nd row of column column_no



char* Filename="D:\\Thesis work 19 March\\STATISCA Random Input\\Hypthetical
Network 6th April\\Monte Carlo\\1000Hypo.txt";

int totalrs=getTotalRows(Filename);

cout<<"Total number of rows are "<<totalrs<<endl;

char *values=getCols(Filename,totalrs);

int totalcs=getTotalCols(Filename);

cout<<"Total number of columns are "<<totalcs<<endl;


for (int i=0;i<totalcs;i++)
{
        cout<<values[i]<<" ";
}

*/



/****************************************/
/********** REAL CODE BEGINS*************/
/****************************************/

char inpZerofile[] = "Basic.inp";//argv[1]; // First argument should be the 0 hour inp file to
edit.
char inpExtfile[] = "Extended.inp";//argv[2]; // Second argument should be the 24 hour
inp file to edit.
char temp24hourfile[100]="\0";
char tempzerohourfile[100]="\0";
char pclosefile[] = "pclosed.dat";
FILE *fp;
char Filename[200]="C.dat";//"D:\\Thesis work 19 March\\STATISCA Random
Input\\Hypthetical Network 6th April\\Monte Carlo\\1000Hypo.txt";
char newFilename[200]="\0";
int no_of_rows=getTotalRows(Filename); // get the number of rows
char buffer[15]="\0"; // buffer for storing _itoa integers
/*
strcpy(Filename,pclosefile); // pipeclosed.dat contains the number of combinations for
pipe closure in separate row.
int total_closed_iter=getTotalRows(Filename); // get the number of rows

fp=fopen(Filename,"r");  // opening the pclosed file.

if (fp==NULL)
```

```
                    {cout<<"Error in the pclosed file: Cannot open it.";
                    }else{
*/
                              // Following four statements fetch the first row of these files to get their names"
                    char *nameOfPipes = getCols("C.dat",0);
                    char *Junctions = getCols("D.dat",0);
                    char *Resers = getCols("RL.dat",0);
                    char *Tanks = getCols("TL.dat",0);
                    char dosCommand[1000]="\0"; // command to be issued in each loop.
                    char nameOfClosedPipes[2000]="\0";



                    for (int command=0;command<=4;command++) // For C,D, TLs, RLs and ALL
                    {
                              for(int current_row=2;current_row<no_of_rows;current_row++)
                              {
/**********************/
/* I've checked for current_row=2. Now check for next rows. */

/********************************/

                                        FillValues(current_row, command); // Fill values of
                                        strcpy(Filename,pclosefile); // pipeclosed.dat contains the number of
combinations for pipe closure in separate row.
                                        int total_closed_iter=getTotalRows(Filename); // get the number of
rows
                                        fp=fopen(Filename,"r");  // opening the pclosed file.

                                        for(int pipeclosed=0;pipeclosed<total_closed_iter;pipeclosed++)
                                        {
                                        fgets(nameOfClosedPipes,2000,fp); // takes one line from pclosed and
generates inp file
                                                                                          // for it.

                                        for(int steady=0;steady<=1;steady++)
                                                  {  // For zero hour steady =0;
                                                    // for 24 hour steady = 1;

                                                            if (steady==0)
                                                                      {
                                                                      strcpy(Filename,inpZerofile);
                                                                      strcpy(newFilename,inpZerofile);
                                                                      strcpy(tempzerohourfile,inpZerofile);

          myStrtok(tempzerohourfile,newFilename,".");
                                                                      strcat(newFilename,"_");

          strcat(newFilename,itoa(command,buffer,10));
                                                                      strcat(newFilename,"_");

          strcat(newFilename,itoa(pipeclosed,buffer,10));
                                                                      strcat(newFilename,"_");
                                                                      strcat(newFilename,"row");
```

```
            strcat(newFilename,itoa(current_row,buffer,10));
                                                            strcat(newFilename,"_");
                                                            strcat(newFilename,"zero.inp");

                                                      }
                                                      else
                                                            {
                                                            strcpy(Filename,inpExtfile);
                                                            strcpy(newFilename,inpExtfile);
                                                            strcpy(temp24hourfile,inpExtfile);
                                                            myStrtok(temp24hourfile,newFilename,".");

                                                            strcat(newFilename,"_");

            strcat(newFilename,itoa(command,buffer,10));
                                                            strcat(newFilename,"_");

            strcat(newFilename,itoa(pipeclosed,buffer,10));
                                                            strcat(newFilename,"_");
                                                            strcat(newFilename,"row");

            strcat(newFilename,itoa(current_row,buffer,10));
                                                            strcat(newFilename,"_");
                                                            strcat(newFilename,"Ext.inp");
                                                            }
                                                      Filecopy(Filename,newFilename);// copies Filename
into newFilename
                                                      Edit_Pclosed(newFilename,
nameOfClosedPipes,"C.dat"); // Filename is the inp file to edit
                                                      Sleep(100); // sleep 0.1 second
                                                      //putdelay if necessary
                                                      Edit_CValues(newFilename, nameOfPipes, C); //
Filename is the inp file to edit
                                                      Sleep(100); // sleep 0.1 second
                                                      //putdelay if necessary
                                                      Edit_DValues(newFilename, Junctions, D); //
Filename is the inp file to edit
                                                      Sleep(100); // sleep 0.1 second
                                                      //putdelay if necessary
                                                      Edit_RLs(newFilename, Resers, RL);// Filename is
the inp file to edit
                                                      Sleep(100); // sleep 0.1 second
                                                      //putdelay if necessary
                                                      Edit_TLs(newFilename, Tanks, TL); // Filename is
the inp file to edit
                                                      Sleep(100); // sleep 0.1 second

                                                strcpy(dosCommand,"C:\\EPANET\\source-
code\\epanet2\\Debug\\epanet.exe ");
                                                strcat(dosCommand,newFilename);
                                                //First argument is the name of inp file.
                                                strcat(dosCommand," ");
```

```
                                        // Second argument outputfile where epanet will store the
results.
                                        strcat(dosCommand,"outputfile.rpt");
                                        system(dosCommand); // runs EPANET program with the new
file as parameter.

                                   //   Sleep(500); // sleep 0.5 seconds to generate and save results

                                        }  // end zero/24 hour loop
                                   }    //end pipe closed loop
                              //Freeing the memory
                              delete(C);
                              delete(D);
                              delete(TL);
                              delete(RL);
                              } // end current row loop

                    } // End Command loop
                    delete(nameOfPipes); // freeing allocated memory
                    delete(Junctions);
                    delete(Resers);
                    delete(Tanks);

                    /********** REAL CODE  ENDS*************/
//                  } // else if file opens ends
          }// else of MFC environment

          return nRetCode;
} // end of _tmain function



char* getCols(char* Filename,int rowNumber)
{
          // fetches row rowNumber from file Filename and return it as a float array
          FILE* fp;

          int totalCols=getTotalCols(Filename);
          char *oneLine= new char[6*totalCols+500];


          fp=fopen(Filename,"r");
          if (fp==NULL)
          {cout<<"Error in the input file: Cannot open it.";
          }else{

          for (int i=0;i<=rowNumber;i++)
                    fgets(oneLine,6*totalCols+500,fp); // number of columns X 4, i.e. for float variable
                                                                                           //plus
500 for the first row that contains column names..
/*        for (i=0;i<totalCols;i++)  // checks whether retrieved row is correct.
                    printf("%.2f ",value[i]);
*/
          fclose(fp);
```

```
        }

        return (oneLine); // return float array that contains the requested row from the file Filename.
}


int getTotalCols(char* Filename)
{ // get total columns in a row.
        FILE* fp;
        char oneLine[2000]="\0";
        char seps[]   = "\t \n";
        char* token;
        int colCount=0;//  Number of columns in the file
        fp=fopen(Filename,"r");

        if (fp==NULL)
        {cout<<"Error in the input file: Cannot open it.";
        }else{

        fgets(oneLine,2000,fp);

         token = strtok( oneLine, seps );
   while( token != NULL )
         {
    /* While there are tokens in "string" */
   //printf( " %s\n", token );
    /* Get next token: */
    token = strtok( NULL, seps );
          colCount++;
         }
          fclose(fp);
        }

        return (colCount-1);
}




void FillValues(int current_row, int command)
{
// Fill the values of C,D,TL and RL for the current row.
// Command's status. 0  Take from C
                //                              1  Take from D
        //                                      2  Take from TL
        //                                      3  Take from RL
        //                                      4  Take from ALL


        // Value of current_row starts from 2 to n+1 because row 1 is the caption and row 2 is default row.

        switch(command)
        {
```

```
case 0: {
        C= getCols("C.dat",current_row); // fetch current row from C.dat file
        D= getCols("D.dat",default_row); // fetch default values of D from
default row of D.dat
        TL= getCols("TL.dat",default_row); // fetch default values of TL from
default row of TL.dat
        RL= getCols("RL.dat",default_row); // fetch default values of RL from
default row of RL.dat
        break;
    }// case 0 ends   Fill from file C.dat

case 1: {
        C= getCols("C.dat",default_row);  // fetch default values of C from
default row of C.dat
        D= getCols("D.dat",current_row);  // fetch current row from D.dat file
        TL= getCols("TL.dat",default_row); // fetch default values of TL from
default row of TL.dat
        RL= getCols("RL.dat",default_row); // fetch default values of RL from
default row of RL.dat
        break;
    }// case 1 ends       Fill from D.dat

case 2: {
        C= getCols("C.dat",default_row); // fetch default values of C from
default row of C.dat
        D= getCols("D.dat",default_row); // fetch default values of D from
default row of D.dat
        TL= getCols("TL.dat",current_row); // fetch current row from TL.dat
file
        RL= getCols("RL.dat",default_row); // fetch default values of RL from
default row of RL.dat
        break;
    }// case 2 ends       Fill from TL.dat

case 3:{
        C= getCols("C.dat",default_row); // fetch default values of C from
default row of C.dat
        D= getCols("D.dat",default_row); // fetch default values of D from
default row of D.dat
        TL= getCols("TL.dat",default_row); // fetch default values of TL from
default row of TL.dat
        RL= getCols("RL.dat",current_row); // fetch current row from RL.dat
file
        break;
  } // case 3 ends   Fill from RL.dat

case 4:{
        C= getCols("C.dat",current_row); // fetch current row from C.dat file
        D= getCols("D.dat",current_row); // fetch current row from D.dat file
        TL= getCols("TL.dat",current_row); // fetch current row from TL.dat
file
        RL= getCols("RL.dat",current_row); // fetch current row from RL.dat
file
        break;
```

```
                } // case 4 ends   Fill from ALL

        } // switch ends



}  // fill values ends

int getTotalRows(char *Filename)
{
        FILE* fp;
        char oneLine[2000]="\0";
        int rowCount=0;//  Number of columns in the file
        fp=fopen(Filename,"r");

        if (fp==NULL)
                cout<<"Error in the input file: Cannot open it.";

        else{

                while(!feof(fp))
                {
                        fgets(oneLine,2000,fp);
                        rowCount++;
                }

                fclose(fp);
                } // else ends

        return (rowCount-1);


}

int FindColNumber(char *Colname,char *Filename)
{ // Finds the column number of column name Colname.
        FILE* fp;
        char oneLine[2000]="\0";
        char seps[]   = "\t \n";
        char* token;
        int colCount=0;//  Number of columns in the file
        fp=fopen(Filename,"r");

        if (fp==NULL)
        {cout<<"Error in the input file: Cannot open it.";
        }else{

        fgets(oneLine,2000,fp);

         token = strtok( oneLine, seps );
   while( strcmp(token,Colname)!=0)
         {
    /* While token is not equal to column Colname in "string" */
   //printf( " %s\n", token );
```

```
      /* Get next token: */
      token = strtok( NULL, seps );
                 colCount++;
                fclose(fp);
                }


                }

                return (colCount);
}// FindColNumber Ends




//CStringList
void Fill_pclosed(char* Filename) //  Fills the file pclosed.dat into global CPtrList object
{
                //CStringList pClosed;

                FILE* fp;
                char oneLine[2000]="\0";
                fp=fopen(Filename,"r");
                if (fp==NULL)
                                cout<<"Error in the input file: Cannot open it.";

                else{
                                CString line;
                                while(!feof(fp))
                                {
                                                fgets(oneLine,2000,fp);
                                    /* Get next token: */
                                                p_closed.AddTail(new CString(oneLine));
                                //              line.RemoveAll();
                                }

                                fclose(fp);
                                } // else ends

                //return (pClosed);
}   // FIll_pclosed ends



void Edit_Pclosed(char *Filename, char *nameOfClosedPipes, char* cFileName)
// Filename is .inp file     cFileName is C.Dat file
{
                FILE* fp;
                char pipenames[2000]="\0";
                char pipenamesCopy[2000]="\0";
                char RemTokensClosePipes[2000]="\0";
                char values[2000]="\0";
                char closedPipe[30]="\0"; // will be used to store a pipe name.
                char token[100]="\0";
```

```
            char seps[]= "\t \n\0";

            fp=fopen(cFileName,"r");  // open file C
            if (fp==NULL)
            {cout<<"Error in the input file: Cannot open it.";
            }else{

            fgets(pipenames,2000,fp); // get the first line containing SNO and all other column names
            fclose(fp);
            int flag =0;
            int maxpipes = getTotalCols(cFileName);
            int maxclosedPipes = getNumberofCols(nameOfClosedPipes);//Nocount the number of pipes in
string nameOfPipes ****************/

            /*token = strtok( pipenames, seps );*/
            strcpy(pipenamesCopy,pipenames);
            myStrtok(pipenamesCopy,token,seps);                  // eliminate serial number. pipenamesCopy is
passed to myStrTOK instead of pipenames so that pipenames remains intact.
            strcat(values,"Open\t");        // Put 'Open' in place of Serial number.
            for(int column=0;column<maxpipes;column++) // start from the first pipe till last pipe
            {                                                                                                          // and mark
closed if the current pipe is
                                                                                                                           //
present in nameOfPipes
                        myStrtok(pipenamesCopy,token,seps);// Next pipe name from the first row of
C.dat.

                        /*closedPipe=strtok(nameOfClosedPipes,seps);*/
                        strcpy(RemTokensClosePipes,nameOfClosedPipes);
                        myStrtok(RemTokensClosePipes,closedPipe,seps);
                        for(int closedPipes=0;closedPipes<maxclosedPipes;closedPipes++)
                        {
                        //          if the current token is equal to any of the closed pipes, break and put
flag =1;
                        //          *******************************************/
                                    if (strcmp(token,closedPipe)==0)
                                    {
                                                flag=1;
                                                break;
                                    }

                                    /*closedPipe=strtok(NULL,seps);*/
                                    //myStrtok(pipenamesCopy,token,seps);
                                    myStrtok(RemTokensClosePipes,closedPipe,seps);
                        }

                        if(flag==0)
                                    strcat(values,"Open\t"); // in the string 'values'
                        else
                                    strcat(values,"Closed\t"); // strcat "closed" in the string 'values'

                        flag =0;
            } // for ends.
```

```
            GenEdit(Filename,"[PIPES]","Status",pipenames,values);

            } // else fp!=NULL ends


} // editing pclosed ends


void Edit_CValues(char *Filename, char *nameOfPipes, char *values) // Filename is .inp file
{

            GenEdit(Filename,"[PIPES]","Roughness",nameOfPipes,values);

} // editing cValues ends


void Edit_DValues(char *Filename, char *Junctions, char *values) // Filename is .inp file
{

            GenEdit(Filename,"[JUNCTIONS]","Demand",Junctions,values);


} // editing DValues ends

void Edit_RLs(char *Filename, char *Resers, char *values)
{

            GenEdit(Filename,"[RESERVOIRS]","Head",Resers,values);

} // editing Reservoirs ends



void Edit_TLs(char *Filename, char *Tanks, char *values)
{

            GenEdit(Filename,"[TANKS]","InitLevel",Tanks,values);

} // editing TLs ends




int getNumberofCols(char *nameOfPipes)
{

            char seps[]   = "\t \n";
            char* token;
            int colCount=0;//  Number of columns in the file
            char CopynameOfPipes[2500]="\0";
            strcpy(CopynameOfPipes,nameOfPipes);

             token = strtok( CopynameOfPipes, seps );
    while( token != NULL )
```

```
            {
     /* While there are tokens in "string" */
    //printf( " %s\n", token );
     /* Get next token: */
     token = strtok( NULL, seps );
             colCount++;
            }
           return (colCount);


}



void GenEdit(char *Filename,char *tagname, char *idname, char *headNames, char *values)
{
        // GenEdit edits the existing .inp file's specific tagname portion only.
        // So for editing C,D,TL,RL etc, we have to call it 4 times with different parameters.
        // It is being called by
        //          void Edit_Pclosed(char *Filename, char *nameOfClosedPipes);
        //void Edit_CValues(char *Filename, char *nameOfPipes, char *values);
        //void Edit_DValues(char *Filename, char *Junctions, char *values);
        //void Edit_RLs(char *Filename, char *Resers, char *values);
        //void Edit_TLs(char *Filename, char *Tanks, char *values);

        FILE* fp;
        char oneLine[3000]="\0";
                char titleLine[3000]="\0";
        char seps[]= "\t \n";
        char elemName[50]="\0";
        char stToToken[3000]="\0";
        int colCount=0;//  Number of columns in the file
        fp=fopen(Filename,"r");
        CString EditedFile;
        if (fp==NULL)
        {cout<<"Error in the input file: Cannot open it.";
        }else{

                do{
                        fgets(oneLine,3000,fp);
                        EditedFile+=oneLine;
                        }while(strstr(oneLine,tagname)-oneLine!=0); // put all lines into EditedFile uptil
(including) tagname

                fgets(titleLine,3000,fp);
                int col2Edit=getColNumber(idname,titleLine);

                EditedFile+=titleLine; // adding title line into modified file.


                while(!feof(fp)) // keep reading inp file
                {
                        fgets(oneLine,3000,fp);
                        if (strlen(oneLine)==1)
```

```
                                        {
                                        EditedFile+=oneLine;
                                        //EditedFile+="\n";
                                        break;
                                        }
                              strcpy(stToToken,oneLine);  // copying oneLine to stToToken because
stToToken will be chaged after call to myStrtok
///Changed
                              myStrtok(stToToken,elemName,seps);  // first token is extracted and put into
elemName
///Changed

                              int indexinValues = getColNumber(elemName,headNames); //TO CHECK THIS
-1 From Edit_pclosed // get the tokennumber of elemName in headNames
                              EditLine(oneLine,values,indexinValues,col2Edit); // will edit oneLine with
corresponding indexinValuesTH value from string values. The result is also returned in oneLine
                              EditedFile+=oneLine;
                    }         // while ends. i.e. this tag portion ends


                    while(!feof(fp))
                    {
                              fgets(oneLine,3000,fp);
                              if (!feof(fp))
                                        EditedFile+=oneLine;
                    }

                    fclose(fp); // close the file opened previously in reading mode

          Filecreate(Filename,EditedFile.GetBuffer(EditedFile.GetLength()),EditedFile.GetLength());
          } // else ends

}  // genEdit ends here.

void EditLine(char *newLine, char *Invalues, int indexinValues, int col2Edit)
{
          char values[3000]="\0";
          char *token;
          char seps[] ="\t \n";
          char EditedLine[3000]="\0";
          char* tempToken;

          strcpy(values,Invalues);
          token= strtok(values,seps);
          for(int index=1;index<indexinValues;index++) // get the token to be inserted in newLine
          {
                    token = strtok(NULL,seps);
          }

          tempToken = strtok(newLine,seps);
          EditedLine[0]='\0';  // initialize the string

          index =1;
          while(tempToken!=NULL)
```

```
        {
                if(index!=col2Edit) // TO CHECK THIS +1
                        strcat(EditedLine,tempToken); // copy old values if condition is not true

                else{
                        strcat(EditedLine,token); // copy the new value of this line.
                        strcat(EditedLine,"\t");
                        break;
                        }
                strcat(EditedLine,"\t");
                index++;
                tempToken=strtok(NULL,seps);
        } // while ends

        tempToken=strtok(NULL,seps);
        while(tempToken!=NULL)
        {       strcat(EditedLine,tempToken);
                strcat(EditedLine,"\t");
                tempToken=strtok(NULL,seps);
        } // while ends


        strcat(EditedLine,"\n");
        strcpy(newLine,EditedLine); // copy the new line into the newLine
}



int getColNumber(char *idname,char *titleLine)
{

        char seps[]   = "\t \n";
        char tempTitle[2000]="\0";
        char* token;
        int colCount=0;//  Number of columns in the file

        strcpy(tempTitle,titleLine);
        token = strtok(tempTitle, seps );
        colCount++;
   while( strcmp(token,idname)!=0)
         {
     /* While token is not equal to column Colname in "string" */
    //printf( " %s\n", token );
     /* Get next token: */
     token = strtok( NULL, seps );
          colCount++;
         }

        return (colCount);

}
```

```
void myStrtok(char *Originalstring,char *returnString, char *seps)
{// change the original string to point to the rest of the string after token.
        // returns token in returnString

        char temp[1000]="\0";
        int index=0;
        bool IsSeps=true;
        while(IsSeparater(Originalstring[index],seps)){
        index++;
        }

        int returnIndex=0;
        do{
                returnString[returnIndex]=Originalstring[index];
        returnIndex++;
        index++;
        }while(!IsSeparater(Originalstring[index],seps));

        returnString[returnIndex]='\0'; // terminate the returning string.

        strcpy(temp,&Originalstring[index]); // modifying orginal string and eliminate token returnstring.
        strcpy(Originalstring,temp);

}


bool IsSeparater(char ch,char *seps)
{
                bool isseperator=false;
                for(int i=0;i<=strlen(seps);i++)
                {
                        if (ch==seps[i])
                                isseperator=true;
                }

        return (isseperator);
}


void Filecopy(char *Filename,char *newFilename)
{

FILE *fpin,*fpout;
fpin=fopen(Filename,"r");
fpout=fopen(newFilename,"w");
char ch;

//char buffer[2000];

/*while(!feof(fpin)){
        //fputc(fgetc(fpin),fpout);
        fputs(fgets(buffer,2000,fpin),fpout);
```

```
}*/

while((ch=fgetc(fpin))!=EOF)
 fputc(ch,fpout);

fclose(fpin);
fclose(fpout);

}


void Filecreate(char *Filename,char *String,int length)
{
FILE *fpout;
fpout=fopen(Filename,"w");

//char buffer[2000];

/*while(!feof(fpin)){
		//fputc(fgetc(fpin),fpout);
		fputs(fgets(buffer,2000,fpin),fpout);
}*/

for (int index=0;index<length;index++)
 fputc(String[index],fpout);

fclose(fpout);

}
```

APPENDIX-C

(MATLAB Code to Calculate Reliability)

*****************MATLAB Code to Calculate Reliability******************

```
close all
clear all

start = 2;
last = 101;
jun_rel=[];

load CutsetprobGEF.txt   %%% F1, F2, ..., F15

for FileCounter = start:last
   DataSource = load(strcat('Extended_4_row', num2str(FileCounter), '_Ext.pre'));
   data = DataSource(:,1:end-5);

   [Press,J] = size(data);
   % J is the number of junctions & P is the pipe closure combinations
   prob = CutsetprobGEF;
   mu=35;
   sigma = 5;
   % disribution
   P = normcdf(data, mu, sigma);

   for k=1:25

      for j = 1:J % J is the number of junctions
         row = find(P(k:25:end-25+k,j)<P(k,j)); % find values less then " all open"

         if isempty(row)==0
            for r = 1:length(row)
               cutsetfail(r,j) = (1-P(row(r),j))*prob(row(r)-1); % cut set for jth column
            end
            tot(j) = sum(cutsetfail(:,j)); % sum of cutset
            jun_rel1(k,j) = 1-tot(j);


         elseif isempty(row)==1
            jun_rel1(k,j)=1;
         end


      end

   end

    jun_rel = [jun_rel; jun_rel1];
   %sys_rel =1-sum((1-prod(P(2:end,:),2)).*prob);


   for k=1:25

      tt=2;

      for t = 26:25:Press-25+k
```

```
            check(k,tt-1)=sum(P(k,:) > P(t,:));
            if check(k,tt-1)>0

               sys_rel(k,tt-1) =  (1-prod(P(t,:))).*prob(tt-1);
            elseif check(k,tt-1)== 0
               sys_rel(k,tt-1) = 0;
            end


            tt=tt+1;

         end

         if isequal(sys_rel(k,:),zeros(1,15))== 1
            sys_reliability(FileCounter-start+1,k)=1;
         else
            sys_reliability(FileCounter-start+1,k)=1-sum(sys_rel(k,:));
         end

   end


   clear cutsetfail sys_rel;
end



%--------------FILE OPERATIONS----------------%
fid = fopen('RelMatExt.txt','w');
[Rows, Cols]=size(jun_rel);
for I=1:Rows
   for J=1:Cols
      fprintf(fid,'%f %s',jun_rel(I,J),' ');    % Writing the Matrix Data to File

   end
   fprintf(fid,'\n');
end
fclose(fid);

fid = fopen('SysRelExt.txt','w');
[Len Wed]=size(sys_reliability);
for I=1:Len
   for J=1:Wed
   fprintf(fid,'%f %s',sys_reliability(I,J),' ');    % Writing the vector Data to File
   end
   fprintf(fid,'\n');
end
fclose(fid);
%--------------FILE OPERATIONS----------------%


%****************************Particular Hour Calculations for Junction
Reliability*************************%
```

```
%%%%%%%%%%%%%%%%% 0 Hour Data

sechr=jun_rel(1:25:2476,:);
fid = fopen('Rx_0.txt','w');
[Rows, Cols]=size(sechr);
for I=1:Rows
   for J=1:Cols
      fprintf(fid,'%f %s',sechr(I,J),' ');    % Writing the Matrix Data to File

   end
   fprintf(fid,'\n');
end
fclose(fid);


%%%%%%%%%%%%%%%%% 1 Hour Data

sechr=jun_rel(2:25:2477,:);
fid = fopen('Rx_1.txt','w');
[Rows, Cols]=size(sechr);
for I=1:Rows
   for J=1:Cols
      fprintf(fid,'%f %s',sechr(I,J),' ');    % Writing the Matrix Data to File

   end
   fprintf(fid,'\n');
end
fclose(fid);

%%%%%%%%%%%%%%%%% 2 Hour Data

sechr=jun_rel(3:25:2478,:);
fid = fopen('Rx_2.txt','w');
[Rows, Cols]=size(sechr);
for I=1:Rows
   for J=1:Cols
      fprintf(fid,'%f %s',sechr(I,J),' ');    % Writing the Matrix Data to File

   end
   fprintf(fid,'\n');
end
fclose(fid);

%%%%%%%%%%%%%%%%% 3 Hour Data

sechr=jun_rel(4:25:2479,:);
fid = fopen('Rx_3.txt','w');
[Rows, Cols]=size(sechr);
for I=1:Rows
   for J=1:Cols
      fprintf(fid,'%f %s',sechr(I,J),' ');    % Writing the Matrix Data to File
```

```
    end
    fprintf(fid,'\n');
end
fclose(fid);

%%%%%%%%%%%%%%%%% 4 Hour Data

sechr=jun_rel(5:25:2480,:);
fid = fopen('Rx_4.txt','w');
[Rows, Cols]=size(sechr);
for I=1:Rows
    for J=1:Cols
        fprintf(fid,'%f %s',sechr(I,J),' ');    % Writing the Matrix Data to File

    end
    fprintf(fid,'\n');
end
fclose(fid);

%%%%%%%%%%%%%%%%% 5 Hour Data

sechr=jun_rel(6:25:2481,:);
fid = fopen('Rx_5.txt','w');
[Rows, Cols]=size(sechr);
for I=1:Rows
    for J=1:Cols
        fprintf(fid,'%f %s',sechr(I,J),' ');    % Writing the Matrix Data to File

    end
    fprintf(fid,'\n');
end
fclose(fid);

%%%%%%%%%%%%%%%%% 6 Hour Data

sechr=jun_rel(7:25:2482,:);
fid = fopen('Rx_6.txt','w');
[Rows, Cols]=size(sechr);
for I=1:Rows
    for J=1:Cols
        fprintf(fid,'%f %s',sechr(I,J),' ');    % Writing the Matrix Data to File

    end
    fprintf(fid,'\n');
end
fclose(fid);

 %%%%%%%%%%%%%%%%% 7 Hour Data

sechr=jun_rel(8:25:2483,:);
fid = fopen('Rx_7.txt','w');
[Rows, Cols]=size(sechr);
for I=1:Rows
    for J=1:Cols
```

```
        fprintf(fid,'%f %s',sechr(I,J),' ');    % Writing the Matrix Data to File

    end
    fprintf(fid,'\n');
end
fclose(fid);

%%%%%%%%%%%%%%%% 8 Hour Data

sechr=jun_rel(9:25:2484,:);
fid = fopen('Rx_8.txt','w');
[Rows, Cols]=size(sechr);
for I=1:Rows
    for J=1:Cols
        fprintf(fid,'%f %s',sechr(I,J),' ');    % Writing the Matrix Data to File

    end
    fprintf(fid,'\n');
end
fclose(fid);

%%%%%%%%%%%%%%%% 9 Hour Data

sechr=jun_rel(10:25:2485,:);
fid = fopen('Rx_9.txt','w');
[Rows, Cols]=size(sechr);
for I=1:Rows
    for J=1:Cols
        fprintf(fid,'%f %s',sechr(I,J),' ');    % Writing the Matrix Data to File

    end
    fprintf(fid,'\n');
end
fclose(fid);

%%%%%%%%%%%%%%%% 10 Hour Data

sechr=jun_rel(11:25:2486,:);
fid = fopen('Rx_10.txt','w');
[Rows, Cols]=size(sechr);
for I=1:Rows
    for J=1:Cols
        fprintf(fid,'%f %s',sechr(I,J),' ');    % Writing the Matrix Data to File

    end
    fprintf(fid,'\n');
end
fclose(fid);

%%%%%%%%%%%%%%%% 11 Hour Data

sechr=jun_rel(12:25:2487,:);
fid = fopen('Rx_11.txt','w');
[Rows, Cols]=size(sechr);
```

```
for I=1:Rows
   for J=1:Cols
      fprintf(fid,'%f %s',sechr(I,J),' ');    % Writing the Matrix Data to File

   end
   fprintf(fid,'\n');
end
fclose(fid);
%%%%%%%%%%%%%%%% 12 Hour Data

sechr=jun_rel(13:25:2488,:);
fid = fopen('Rx_12.txt','w');
[Rows, Cols]=size(sechr);
for I=1:Rows
   for J=1:Cols
      fprintf(fid,'%f %s',sechr(I,J),' ');    % Writing the Matrix Data to File

   end
   fprintf(fid,'\n');
end
fclose(fid);

%%%%%%%%%%%%%%%% 13 Hour Data

sechr=jun_rel(14:25:2489,:);
fid = fopen('Rx_13.txt','w');
[Rows, Cols]=size(sechr);
for I=1:Rows
   for J=1:Cols
      fprintf(fid,'%f %s',sechr(I,J),' ');    % Writing the Matrix Data to File

   end
   fprintf(fid,'\n');
end
fclose(fid);

%%%%%%%%%%%%%%%% 14 Hour Data

sechr=jun_rel(15:25:2490,:);
fid = fopen('Rx_14.txt','w');
[Rows, Cols]=size(sechr);
for I=1:Rows
   for J=1:Cols
      fprintf(fid,'%f %s',sechr(I,J),' ');    % Writing the Matrix Data to File

   end
   fprintf(fid,'\n');
end
fclose(fid);

%%%%%%%%%%%%%%%% 15 Hour Data

sechr=jun_rel(16:25:2491,:);
fid = fopen('Rx_15.txt','w');
```

```
[Rows, Cols]=size(sechr);
for I=1:Rows
   for J=1:Cols
      fprintf(fid,'%f %s',sechr(I,J),' ');    % Writing the Matrix Data to File

   end
   fprintf(fid,'\n');
end
fclose(fid);

%%%%%%%%%%%%%%%% 16 Hour Data

sechr=jun_rel(17:25:2492,:);
fid = fopen('Rx_16.txt','w');
[Rows, Cols]=size(sechr);
for I=1:Rows
   for J=1:Cols
      fprintf(fid,'%f %s',sechr(I,J),' ');    % Writing the Matrix Data to File

   end
   fprintf(fid,'\n');
end
fclose(fid);

%%%%%%%%%%%%%%%% 17 Hour Data

sechr=jun_rel(18:25:2493,:);
fid = fopen('Rx_17.txt','w');
[Rows, Cols]=size(sechr);
for I=1:Rows
   for J=1:Cols
      fprintf(fid,'%f %s',sechr(I,J),' ');    % Writing the Matrix Data to File

   end
   fprintf(fid,'\n');
end
fclose(fid);

%%%%%%%%%%%%%%%% 18 Hour Data

sechr=jun_rel(19:25:2494,:);
fid = fopen('Rx_18.txt','w');
[Rows, Cols]=size(sechr);
for I=1:Rows
   for J=1:Cols
      fprintf(fid,'%f %s',sechr(I,J),' ');    % Writing the Matrix Data to File

   end
   fprintf(fid,'\n');
end
fclose(fid);


%%%%%%%%%%%%%%%% 19 Hour Data
```

```
sechr=jun_rel(20:25:2495,:);
fid = fopen('Rx_19.txt','w');
[Rows, Cols]=size(sechr);
for I=1:Rows
   for J=1:Cols
      fprintf(fid,'%f %s',sechr(I,J),' ');    % Writing the Matrix Data to File

   end
   fprintf(fid,'\n');
end
fclose(fid);

%%%%%%%%%%%%%%%%% 20 Hour Data

sechr=jun_rel(21:25:2496,:);
fid = fopen('Rx_20.txt','w');
[Rows, Cols]=size(sechr);
for I=1:Rows
   for J=1:Cols
      fprintf(fid,'%f %s',sechr(I,J),' ');    % Writing the Matrix Data to File

   end
   fprintf(fid,'\n');
end
fclose(fid);

%%%%%%%%%%%%%%%%% 21 Hour Data

sechr=jun_rel(22:25:2497,:);
fid = fopen('Rx_21.txt','w');
[Rows, Cols]=size(sechr);
for I=1:Rows
   for J=1:Cols
      fprintf(fid,'%f %s',sechr(I,J),' ');    % Writing the Matrix Data to File

   end
   fprintf(fid,'\n');
end
fclose(fid);

%%%%%%%%%%%%%%%%% 22 Hour Data

sechr=jun_rel(23:25:2498,:);
fid = fopen('Rx_22.txt','w');
[Rows, Cols]=size(sechr);
for I=1:Rows
   for J=1:Cols
      fprintf(fid,'%f %s',sechr(I,J),' ');    % Writing the Matrix Data to File

   end
   fprintf(fid,'\n');
end
fclose(fid);
```

```
%%%%%%%%%%%%%%%%% 23 Hour Data

sechr=jun_rel(24:25:2499,:);
fid = fopen('Rx_23.txt','w');
[Rows, Cols]=size(sechr);
for I=1:Rows
    for J=1:Cols
        fprintf(fid,'%f %s',sechr(I,J),' ');    % Writing the Matrix Data to File

    end
    fprintf(fid,'\n');
end
fclose(fid);

 %%%%%%%%%%%%%%%%% 24 Hour Data

sechr=jun_rel(25:25:2500,:);
fid = fopen('Rx_24.txt','w');
[Rows, Cols]=size(sechr);
for I=1:Rows
    for J=1:Cols
        fprintf(fid,'%f %s',sechr(I,J),' ');    % Writing the Matrix Data to File

    end
    fprintf(fid,'\n');
end
fclose(fid);
```

APPENDIX-D

(MATLAB Program to Calculate Statistics by Reading the Files Iteratively)

MATLAB Program to Calculate Statistics by Reading the Files Iteratively

```
close all
clear all
start = 0;
last = 24;

for FileCounter = start:last

temp = strcat('ExtendedGEF_All_Rx_',num2str(FileCounter))

DataSource = load(strcat('ExtendedGEF_All_Rx_',num2str(FileCounter),'.txt'));

data = DataSource(:,1:end);

mean_values= mean(data);

max_values = max(data);

min_values = min(data);

Stand_dev=std(data);

UCL = (mean_values)+(1.96*(Stand_dev)/(sqrt(300)));

LCL = (mean_values)-(1.96*(Stand_dev)/(sqrt(300)));



%--------------FILE OPERATIONS----------------%
fid = fopen('mean.txt','a');
[Rows, Cols]=size(mean_values);
for I=1:Rows
   for J=1:Cols
      fprintf(fid,'%f %s',mean_values(I,J),' ');    % Writing the Matrix Data to File

   end
   fprintf(fid,'\n');
end
fclose(fid);



fid = fopen('max.txt','a');
[Rows, Cols]=size(max_values);
for I=1:Rows
   for J=1:Cols
      fprintf(fid,'%f %s',max_values(I,J),' ');    % Writing the Matrix Data to File

   end
```

```
    fprintf(fid,'\n');
end
fclose(fid);


fid = fopen('min.txt','a');
[Rows, Cols]=size(min_values);
for I=1:Rows
    for J=1:Cols
        fprintf(fid,'%f %s',min_values(I,J),' ');    % Writing the Matrix Data to File

    end
    fprintf(fid,'\n');
end
fclose(fid);

fid = fopen('stdev.txt','a');
[Rows, Cols]=size(Stand_dev);
for I=1:Rows
    for J=1:Cols
        fprintf(fid,'%f %s',Stand_dev(I,J),' ');    % Writing the Matrix Data to File

    end
    fprintf(fid,'\n');
end
fclose(fid);


fid = fopen('upper.txt','a');
[Rows, Cols]=size(UCL);
for I=1:Rows
    for J=1:Cols
        fprintf(fid,'%f %s',UCL(I,J),' ');    % Writing the Matrix Data to File

    end
    fprintf(fid,'\n');
end
fclose(fid);


fid = fopen('lower.txt','a');
[Rows, Cols]=size(LCL);
for I=1:Rows
    for J=1:Cols
        fprintf(fid,'%f %s',LCL(I,J),' ');    % Writing the Matrix Data to File

    end
    fprintf(fid,'\n');
end
fclose(fid);
end
%--------------FILE OPERATIONS----------------%
```

APPENDIX-E

(Java Code for Merging Files)

*************************Java Code for Merging Files*******************

```
import java.io.*;
public class ExtendedGEF_C_Rx0
{
	public static void main(String args[])
	{
		PrintWriter pw = null;

		File f1 = new File("MC_Row 1 to 25/C_values/Rx_0.txt");//input file1
		File f2 = new File("MC_Row 26 to 50/C_values/Rx_0.txt");//input file2
		File f3 = new File("MC_Row 51 to 75/C_values/Rx_0.txt");//input file3
		File f4 = new File("MC_Row 76 to 100/C_values/Rx_0.txt");//input file4
		File f5 = new File("MC_Row 101 to 125/C_values/Rx_0.txt");//input file5
		File f6 = new File("MC_Row 126 to 150/C_values/Rx_0.txt");//input file6
		File f7 = new File("MC_Row 151 to 175/C_values/Rx_0.txt");//input file7
		File f8 = new File("MC_Row 176 to 200/C_values/Rx_0.txt");//input file8
		File f9 = new File("MC_Row 201 to 225/C_values/Rx_0.txt");//input file9
		File f10 = new File("MC_Row 226 to 250/C_values/Rx_0.txt");//input file10
		File f11 = new File("MC_Row 251 to 275/C_values/Rx_0.txt");//input file11
		File f12 = new File("MC_Row 276 to 300/C_values/Rx_0.txt");//input file12

		File f13 = new File("ExtendedGEF_C_Rx0.txt");//output file
		try
		{
	pw = new PrintWriter(new FileWriter(f13));
		}
		catch(IOException e) { }
		try
		{
			BufferedReader br1 = new BufferedReader(new FileReader(f1));
			BufferedReader br2 = new BufferedReader(new FileReader(f2));
			BufferedReader br3 = new BufferedReader(new FileReader(f3));
			BufferedReader br4 = new BufferedReader(new FileReader(f4));
			BufferedReader br5 = new BufferedReader(new FileReader(f5));
			BufferedReader br6 = new BufferedReader(new FileReader(f6));
			BufferedReader br7 = new BufferedReader(new FileReader(f7));
			BufferedReader br8 = new BufferedReader(new FileReader(f8));
			BufferedReader br9 = new BufferedReader(new FileReader(f9));
			BufferedReader br10 = new BufferedReader(new FileReader(f10));
			BufferedReader br11 = new BufferedReader(new FileReader(f11));
			BufferedReader br12 = new BufferedReader(new FileReader(f12));
					for (int skcnt=0; skcnt<=24; skcnt++)
					pw.println(br1.readLine());

					for(int wrcnt=25;wrcnt<=49;wrcnt++)
					pw.println(br2.readLine());

					for(int wrcnt1=50;wrcnt1<=74 ;wrcnt1++)
					pw.println(br3.readLine());

					for(int wrcnt2=75;wrcnt2<=99;wrcnt2++)
					pw.println(br4.readLine());
```

```
                    for(int wrcnt3=100;wrcnt3<=124;wrcnt3++)
                    pw.println(br5.readLine());

                    for(int wrcnt4=125;wrcnt4<=149;wrcnt4++)
                    pw.println(br6.readLine());

                    for(int wrcnt5=150;wrcnt5<=174;wrcnt5++)
                    pw.println(br7.readLine());

                    for(int wrcnt6=175;wrcnt6<=199;wrcnt6++)
                    pw.println(br8.readLine());

                    for(int wrcnt7=200;wrcnt7<=224;wrcnt7++)
                    pw.println(br9.readLine());

                    for(int wrcnt8=225;wrcnt8<=249;wrcnt8++)
                    pw.println(br10.readLine());

                    for(int wrcnt9=250;wrcnt9<=274;wrcnt9++)
                    pw.println(br11.readLine());


                    for(int wrcnt12=275;wrcnt12<=299 ;wrcnt12++)
                    pw.println(br12.readLine());




            }
                catch (Exception e){}

                pw.close();
            }
}
```

APPENDIX-F

(Characteristics of Al-Khobar Water Distribution Network)

Pipe characteristics of Al-Khobar Network

Pipe Label	Length (m)	Diameter (mm)	Hazen-Williams C-factor
Pipe 2	498	300	100
Pipe 3	672	300	100
Pipe 8	714	225	100
Pipe 13	462	280	100
Pipe 7	15	400	100
Pipe 5	201	300	100
Pipe 6	162	500	100
Pipe 4	858	300	100
Pipe 15	948	280	100
Pipe 14	786	400	100
Pipe 16	300	250	100
Pipe 17	324	400	100
Pipe 25	396	500	100
Pipe 26	342	600	100
Pipe 46	429	500	100
Pipe 47	711	280	100
Pipe 91	678	400	100
Pipe 52	393	200	100
Pipe 54	732	280	100
Pipe 57	237	200	100
Pipe 59	276	300	100
Pipe 60	423	200	100
Pipe 61	420	200	100
Pipe 62	507	200	100
Pipe 63	354	300	100
Pipe 65	270	300	100
Pipe 68	450	200	100
Pipe 67	318	300	100
Pipe 92	570	200	100
Pipe 97	366	150	100
Pipe 98	204	200	100
Pipe 100	348	200	100
Pipe 102	660	150	100
Pipe 93	645	400	100
Pipe 101	216	150	100
Pipe 103	534	200	100
Pipe 104	750	200	100
Pipe 105	1080	200	100
Pipe 111	54	400	100
Pipe 112	498	400	100
Pipe 113	642	300	100
Pipe 114	342	400	100
Pipe 118	465	500	100
Pipe 124	465	150	100

Pipe 117	594	200	100
Pipe 119	1278	600	100
Pipe 127	738	600	100
Pipe 115	504	400	100
Pipe 116	237	400	100
Pipe 121	180	400	100
Pipe 122	570	200	100
Pipe 120	420	500	100
Pipe 125	420	150	100
Pipe 126	738	250	100
Pipe 130	492	300	100
Pipe 146	570	500	100
Pipe 147	1290	300	100
Pipe 148	570	150	100
Pipe 149	738	300	100
Pipe 151	372	600	100
Pipe 150	180	600	100
Pipe 152	636	300	100
Pipe 160	150	400	100
Pipe 166	192	400	100
Pipe 167	1296	400	100
Pipe 168	546	150	100
Pipe 169	111	150	100
Pipe 171	732	250	100
Pipe 170	732	400	100
Pipe 172	552	600	100
Pipe 173	111	600	100
Pipe 174	639	300	100
Pipe 175	639	400	100
Pipe 31	1428	400	100
Pipe 32	870	300	100
Pipe 33	726	250	100
Pipe 34	552	400	100
Pipe 36	63	150	100
Pipe 37	750	400	100
Pipe 35	1257	300	100
Pipe 38	720	300	100
Pipe 42	624	300	100
Pipe 43	720	500	100
Pipe 44	291	500	100
Pipe 45	1275	200	100
Pipe 81	948	300	100
Pipe 88	129	300	100
Pipe 89	288	300	100
Pipe 90	558	250	100
Pipe 106	444	200	100
Pipe 107	564	200	100
Pipe 108	138	150	100
Pipe 136	630	350	100

Pipe 135	1626	600	100
Pipe 181	246	600	100
Pipe 141	876	600	100
Pipe 138	129	350	100
Pipe 139	1188	300	100
Pipe 140	558	150	100
Pipe 142	438	600	100
Pipe 137	180	300	100
Pipe 153	492	350	100
Pipe 144	120	300	100
Pipe 143	990	300	100
Pipe 154	57	350	100
Pipe 155	318	250	100
Pipe 156	1194	200	100
Pipe 176	726	300	100
Pipe 177	108	300	100
Pipe 77	102	200	100
Pipe 158	75	400	100
Pipe 159	978	200	100
Pipe 162	174	400	100
Pipe 161	1020	400	100
Pipe 165	282	400	100
Pipe 163	654	500	100
Pipe 99	1230	200	100
Pipe 70	150	300	100
Pipe 82	144	300	100
Pipe 78	567	200	100
Pipe 84	780	200	100
Pipe 87	408	150	100
Pipe 134	114	200	100
Pipe 85	372	200	100
Pipe 123	1278	200	100
Pipe 129	420	600	100
Pipe 110	1212	500	100
Pipe 64	390	200	100
Pipe 71	543	200	100
Pipe 75	516	200	100
Pipe 23	30	400	100
Pipe 83	300	600	100
Pipe 86	516	600	100
Pipe 80	1032	200	100
Pipe 58	618	200	100
Pipe 22	1845	200	100
Pipe 55	186	800	100
Pipe 131	390	300	100
Pipe 128	300	800	100
Pipe 20	1390	1000	100
Pipe 24	378	400	100
Pipe 27	60	400	100

Pipe 76	390	200	100
Pipe 29	78	200	100
Pipe 30	24	200	100
Pipe 66	222	200	100
Pipe 69	162	200	100
Pipe 51	321	300	100
Pipe 53	615	300	100
Pipe 56	144	300	100
Pipe 12	96	400	100
Pipe 9	96	400	100
Pipe 180	1152	400	100
Pipe 178	1080	400	100
Pipe 39	276	400	100
Pipe 41	348	400	100
Pipe 21	492	1000	100
Pipe 50	678	500	100
Pipe 49	204	200	100
Pipe 48	390	200	100
Pipe 96	642	500	100
Pipe 95	312	200	100
Pipe 94	258	200	100
Pipe 74	60	200	100
Pipe 19	636	400	100
Pipe 28	162	300	100
Pipe 79	780	600	100
Pipe MKAH	90	400	100
Pipe YRMK-1600	10	1600	100
Pipe 73	1756	1000	100
Pipe 1	1110	300	100
Pipe 157	1158	400	100
Pipe YRMK-900	5	900	100
Pipe 164	1398	400	100
Pipe 18	720	280	100
Pipe 145	1800	400	100
Pipe 109	5	200	100
Pipe 40	5	200	100
Pipe 72	5	200	100
Pipe BLND-2	10	900	100
Pipe YRMK-900-CV	5	900	100
Pipe W18	5	250	100
Pipe W20	5	200	100
Pipe 179	5	200	100
Pipe W12	5	250	100
Pipe 11	5	200	100
Pipe P420	10	250	100
Pipe P220	10	200	100
Pipe 132	5	600	100
Pipe 133	5	1000	100

Pipe 10	5	400	100

Nodal characteristics of Al-Khobar Network

Node Label	Elevation (m)	Demand (LPS)
Junc 1	30.1	0.9259
Junc 2	21.5	0.9722
Junc 3	25.71	2.1296
Junc 4	24.1	0
Junc 5	16.55	1.4468
Junc 6	13.76	2.8356
Junc 7	10.96	13.9244
Junc 8	9.44	6.0494
Junc 9	9.6	6.0494
Junc 10	11.38	11.7207
Junc 11	9.2	0
Junc 12	9.1	8.912
Junc 13	8.95	3.2407
Junc 14	27.5	1.929
Junc 15	31.73	1.929
Junc 16	18.21	1.7809
Junc 17	18.24	0.7778
Junc 18	27.54	0
Junc 19	22.59	1.75
Junc 20	22.5	1.7809
Junc 21	22.4	4.7078
Junc 22	21.98	4.6939
Junc 23	17.75	5.2148
Junc 24	18.6	1.6782
Junc 25	18.83	3.0874
Junc 26	15.18	2.3449
Junc 27	15.19	0.8981
Junc 28	14.2	2.1134
Junc 29	14.6	4.5231
Junc 30	14.2	3.8565
Junc 31	10	21.9614
Junc 32	8	23.3333
Junc 33	7.13	10.8025
Junc 34	11.45	1.0127
Junc 35	11.31	0
Junc 36	12.19	1.7905
Junc 37	13	1.75
Junc 38	14.59	3.5504
Junc 39	13.56	3.305

Junc 40	14.57	3.0157
Junc 41	14.28	3.3768
Junc 42	11.2	0
Junc 43	11.8	2.3426
Junc 44	11.7	0.8981
Junc 45	11.04	0
Junc 46	11.5	0
Junc 47	12.53	4.7299
Junc 48	9.4	2.7431
Junc 49	9.66	3.1042
Junc 50	10.19	2.9722
Junc 51	9.61	2.9722
Junc 52	9.15	1.4444
Junc 53	8.94	3.4537
Junc 54	7.81	1.765
Junc 55	8.3	1.765
Junc 56	9.5	0
Junc 57	8.5	1.3889
Junc 58	8.45	3.0864
Junc 59	8.39	3.2253
Junc 60	6.1	0.2315
Junc 61	6.25	0.5787
Junc 62	7.9	1.0995
Junc 63	8.4	0
Junc 64	7.4	6.0995
Junc 65	7	6.5278
Junc 66	6.98	10.4167
Junc 67	8	5.8056
Junc 68	8.8	5.1636
Junc 69	8.75	5.1636
Junc 70	8.72	1.1603
Junc 71	6.3	12.8117
Junc 72	7.84	0
Junc 73	7.8	13.9892
Junc 74	7.05	1.088
Junc 75	7.56	18.3565
Junc 76	6.85	27.8472
Junc 77	6.07	15.2855
Junc 78	6.8	14.8245
Junc 79	5.8	2.7654
Junc 80	6	7.985
Junc 81	6.79	13.1162
Junc 82	6.16	15.9248
Junc 83	2.45	10.3164
Junc 84	0.56	0.2315
Junc 85	5.6	0.7407
Junc 86	3.88	1.1603
Junc 87	0.95	46.4728
Junc 88	3.29	68.6458

Junc 89	6.12	29.7597
Junc 90	6.98	12.0348
Junc 91	7.52	7.0966
Junc 92	7.09	25.3257
Junc 93	3.95	29.0228
Junc 94	3.21	30.9144
Junc 95	1.25	12.6852
Junc 96	1.14	62.3524
Junc 97	0	93.5764
Junc 98	0.9	40.7639
Junc 99	1.23	27.252
Junc 100	3.27	33.5251
Junc 101	3.27	42.8654
Junc 102	2.2	0
Junc 103	0.26	1.1333
Junc 104	0.15	4.0992
Junc 105	0.15	4.7502
Junc 106	0.06	7.9572
Junc 107	0.6	8.6082
Junc 108	0.32	50.5556
Junc 109	-1.52	35.1852
Junc 110	0.18	44.1667
Junc 111	-1.5	4.9913
Junc 112	0.36	29.5399
Junc 113	-0.59	35.5556
Junc 114	-0.71	11.2963
Junc 115	0.21	35.2778
Junc 116	0.75	7.5
Junc 117	6.7	0
Junc 118	6.7	0
Junc 122	21.27	0
Junc 121	9.2	0
Junc 120	6.16	0
Junc 520	0	0
Junc 624	9.2	-91.66
Junc 625	50	-337
Junc 626	6.16	-70.83
Junc 119	0.75	-88.88
Junc 628	21.27	-63.88
Junc 123	27.54	-50
Junc 630	6.7	-420
Junc 631	6.7	-220
Junc 124	0	

Tank characteristics of Al-Khobar Network

	MKAH	YRMK
Base Elevation (m)	30	27.45
Min Elevation (m)	0	1
Initial Elevation (m)	4	10
Max Elevation (m)	20	24.5
Tank Diameter (m)	30	40

Demand pattern of Al-Khobar Network

Time Period (hr)	Multiplication Factor
1	0.75
2	0.65
3	0.58
4	0.58
5	0.64
6	0.86
7	1.13
8	1.18
9	1.11
10	1.1
11	1.14
12	1.18
13	1.16
14	1.21
15	1.33
16	1.32
17	1.22
18	1.14
19	1.15
20	1.1
21	1.05
22	1.06
23	0.99
24	0.86

APPENDIX-G

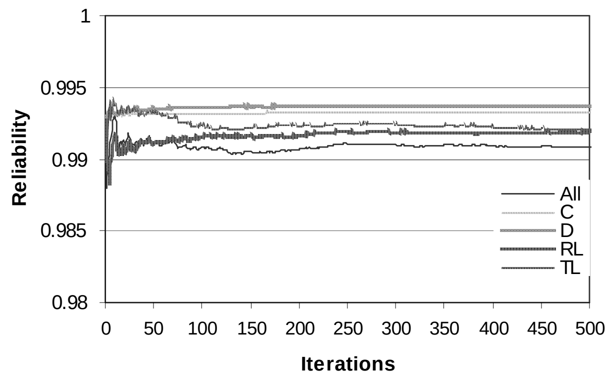
(Extended Period Junction & System Reliability Plots for Hypothetical Nework)

Extended Period Junction Reliability Plots for Hypothetical Nework

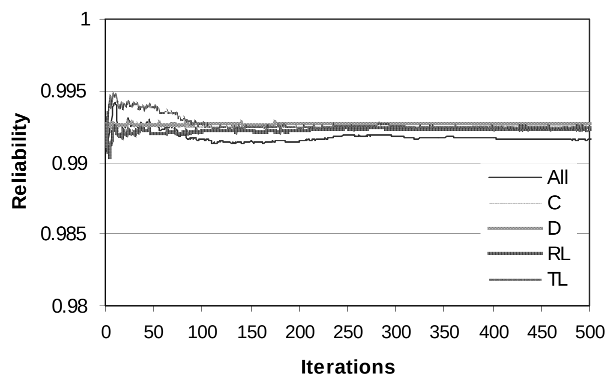
Case No.	Model		Data Generation Method		Pipe Failure Probability Calculation Method	
	SS	EPS	MC	BS	GEF	Poisson
1	√		√		√	
2	√		√			√
3	√			√	√	
4	√			√		√
5		√	√		√	
6		√	√			√
7		√		√	√	
8		√		√		√

Legend

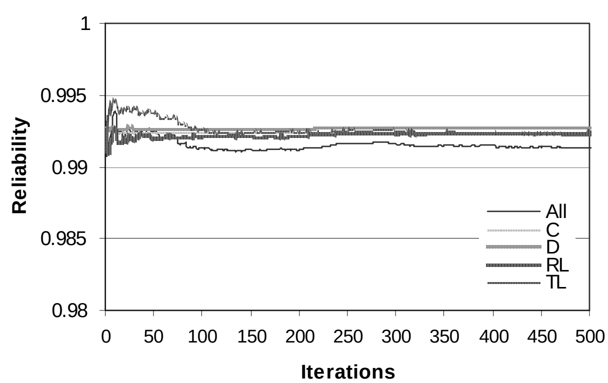
SS = Steady State
EPS = Extended Period Simulation
MC = Monte Carlo
BS = Bootstrapping
GEF = Generic Expectation Method



(a) 0^{th} hour

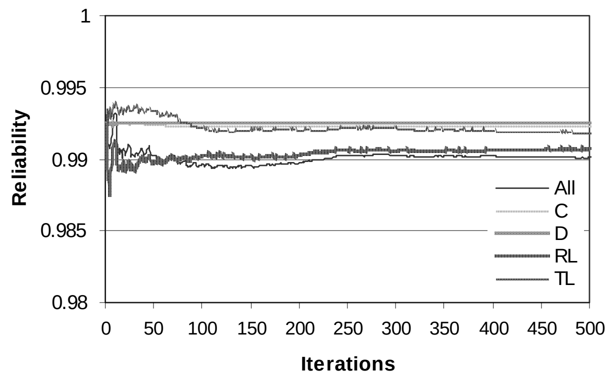


(b) 6^{th} hour

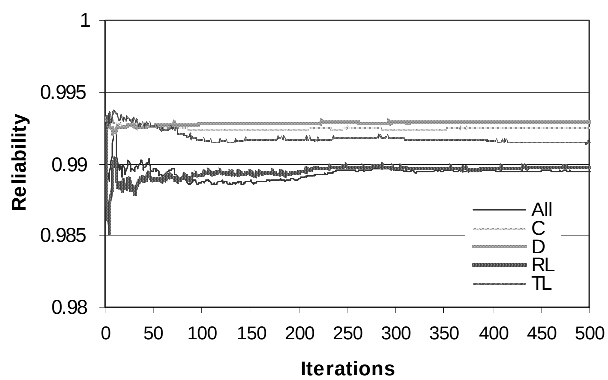


(c) 12^{th} hour

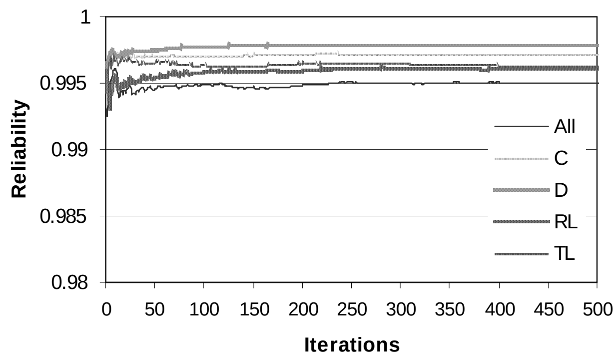
Figure A4.1: Reliability of Junction J-5 for Case 5 at (a) 0^{th}, (b) 6^{th}, (c) 12^{th}, (d) 18^{th} and (e) 24^{th} hour



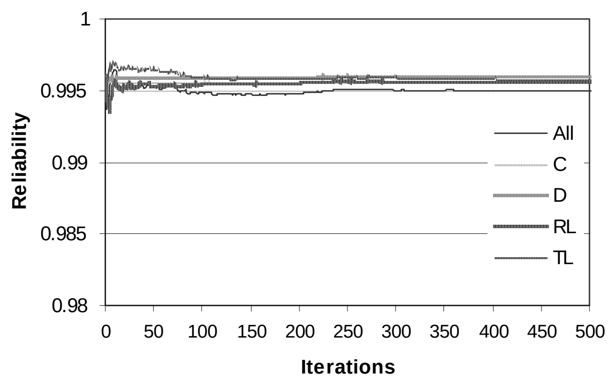
(c) 18th hour



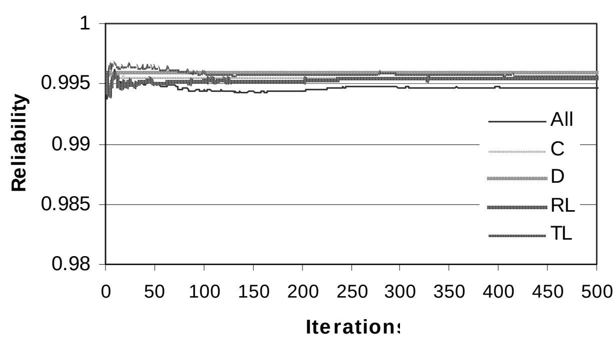
(c) 24th hour



(a) 0th hour

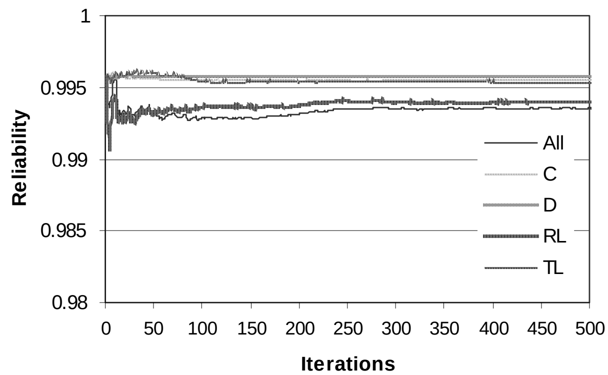


(b) 6th hour

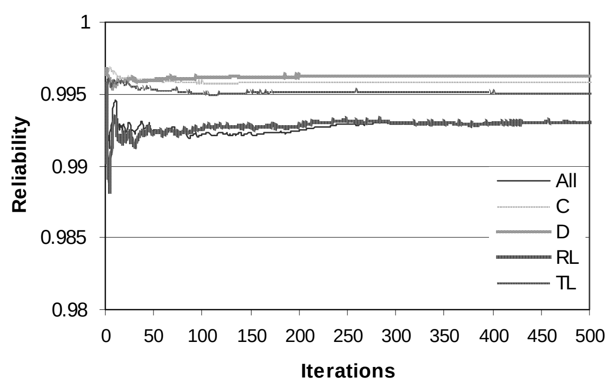


(c) 12th hour

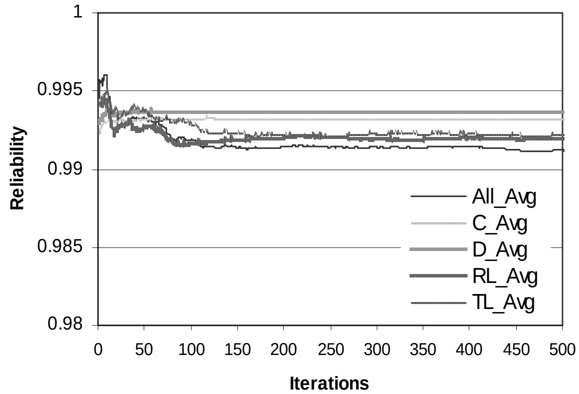
Figure A4.2: Reliability of Junction J-5 for Case 6 at (a) 0th, (b) 6th, (c) 12th, (d) 18th and (e) 24th hour



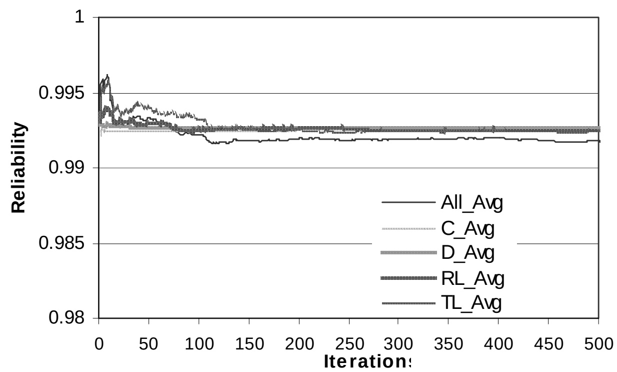
(d) 18th hour



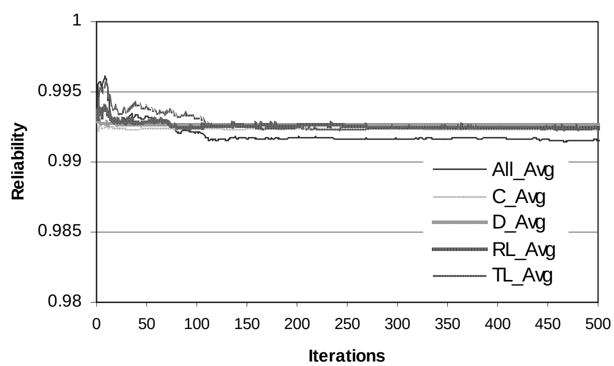
(e) 24th hour



(a) 0th hour

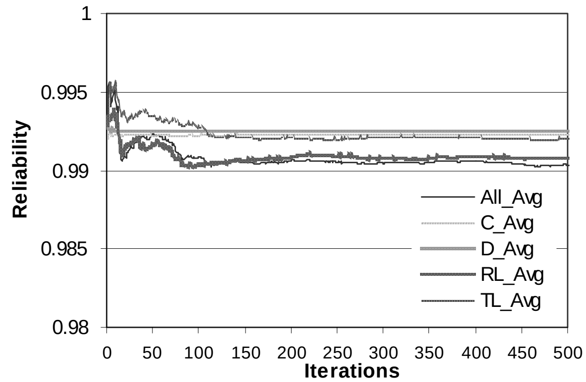


(b) 6th hour

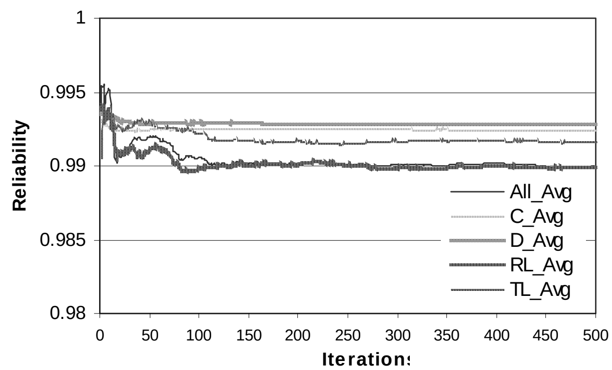


(c) 12th hour

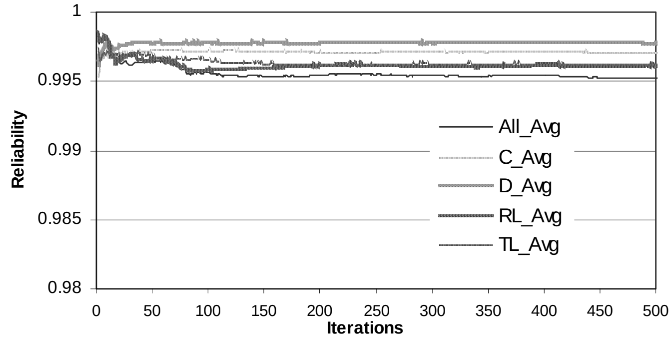
Figure A4.3: Reliability of Junction J-5 for Case 7 at (a) 0^{th}, (b) 6^{th}, (c) 12^{th}, (d) 18^{th} and (e) 24^{th} hour



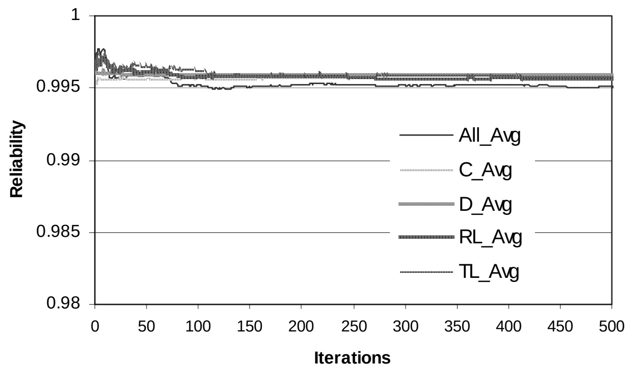
(d) 18th hour



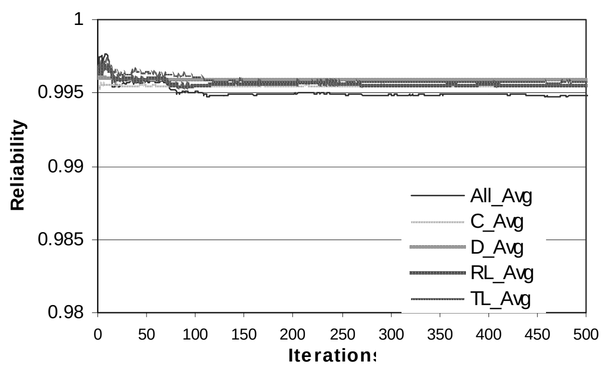
(e) 24th hour



(a) 0th hour

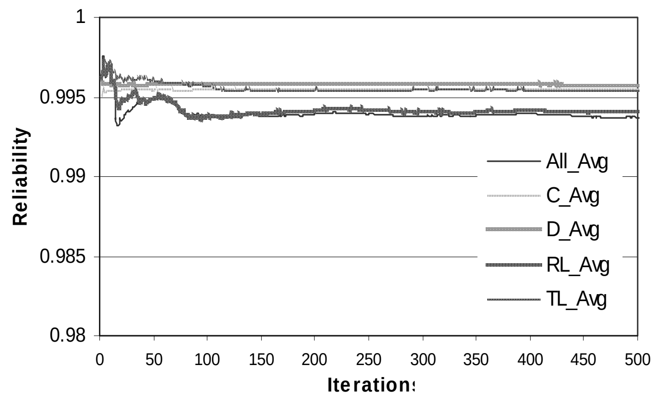


(b) 6th hour

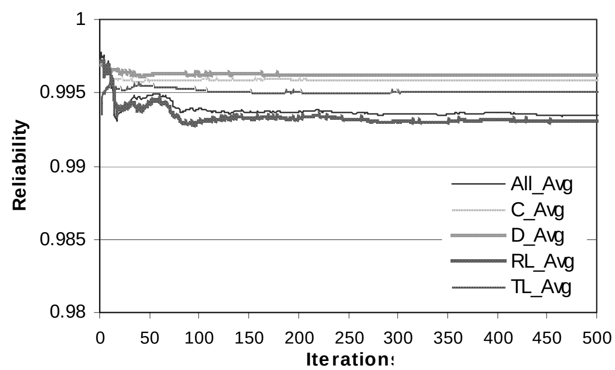


(c) 12th hour

Figure A4.4: Reliability of Junction J-5 for Case 8 at (a) 0th, (b) 6th, (c) 12th, (d) 18th and (e) 24th hour

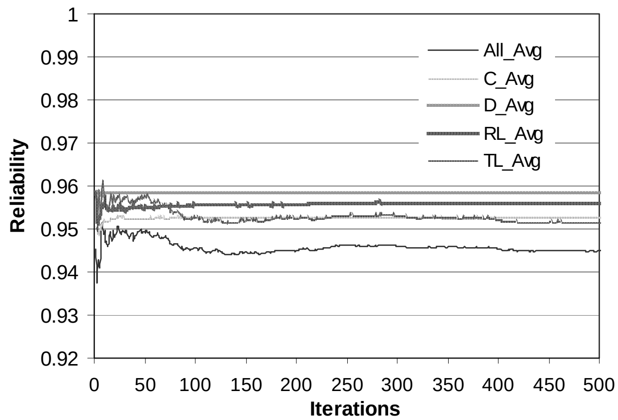


(d) 18th hour

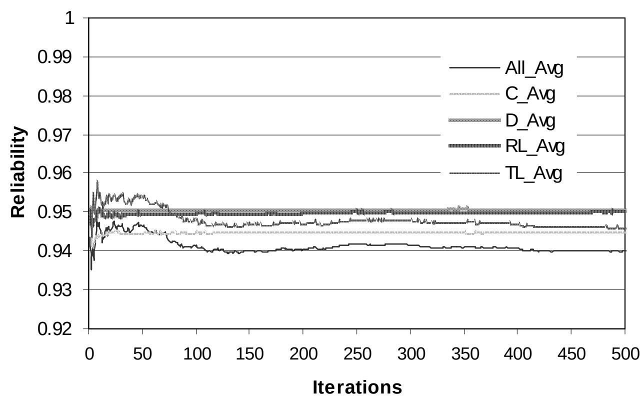


(e) 24th hour

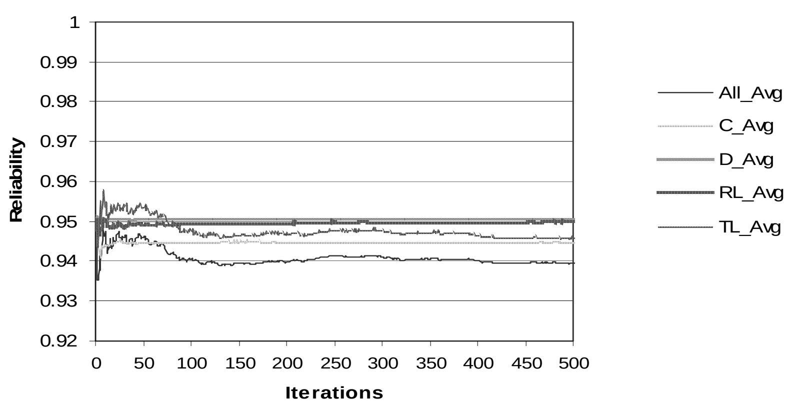
Extended Period System Reliability Plots for Hypothetical Nework



(a) 0th hour

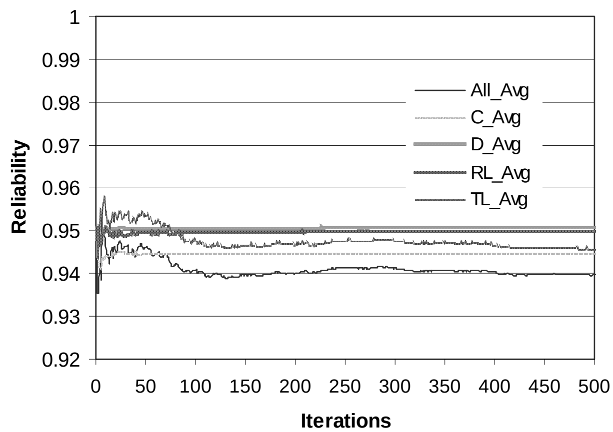


(b) 6th hour

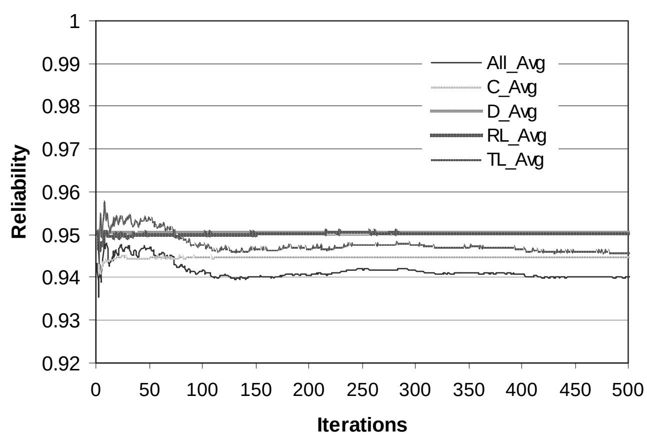


(c) 12th hour

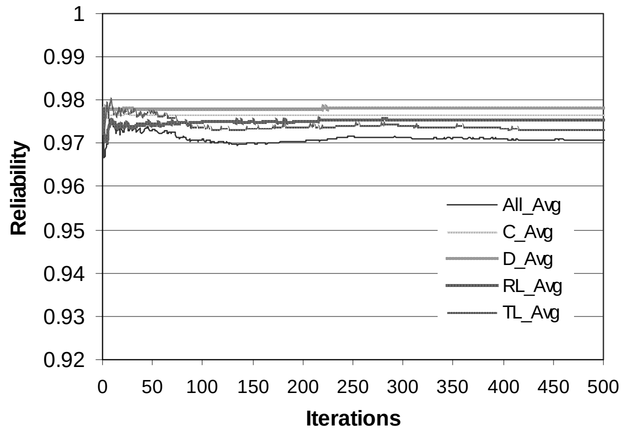
Figure A4.5: System Reliability for Case 5 at (a) 0th, (b) 6th, (c) 12th, (d) 18th and (e) 24th hour



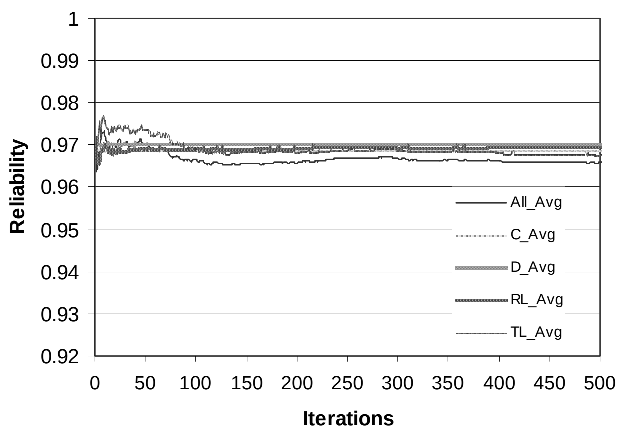
(d) 18th hour



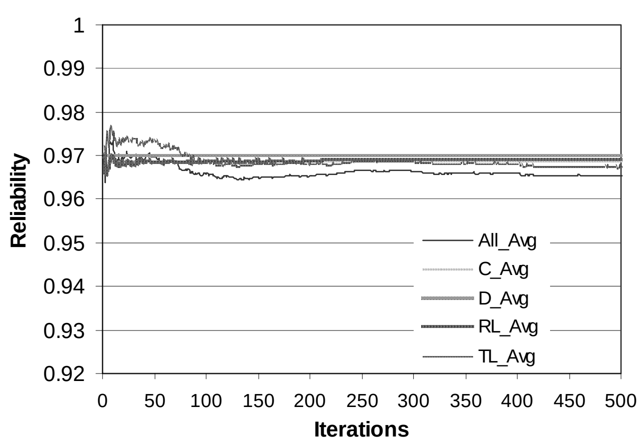
(e) 24th hour



(a) 0th hour

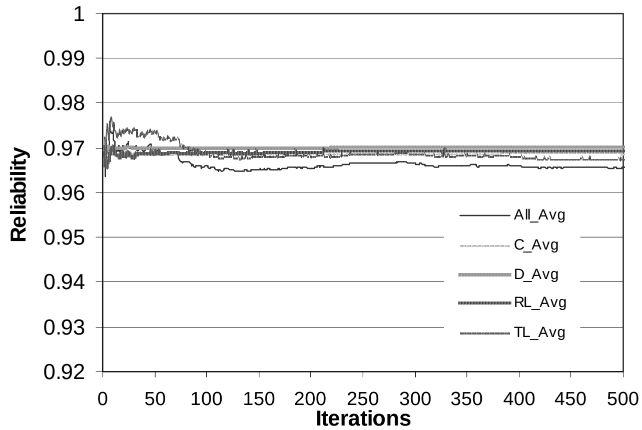


(b) 6th hour

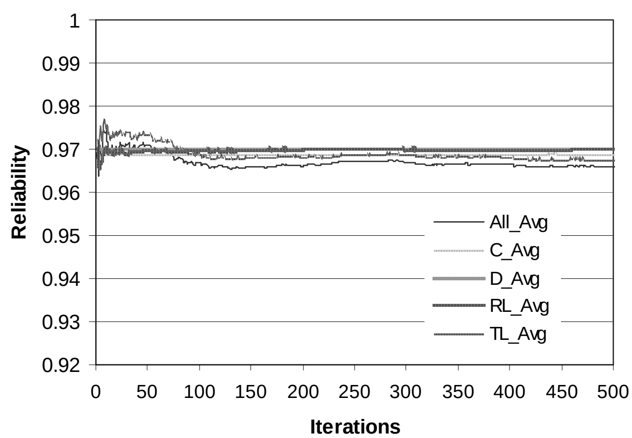


(c) 12th hour

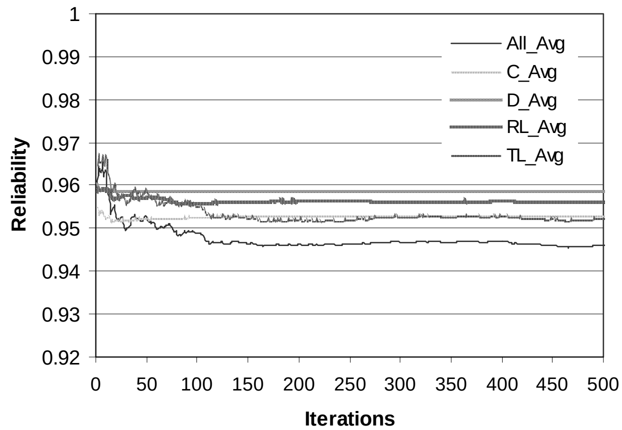
Figure A4.6: System Reliability for Case 6 at (a) 0th, (b) 6th, (c) 12th, (d) 18th and (e) 24th hour



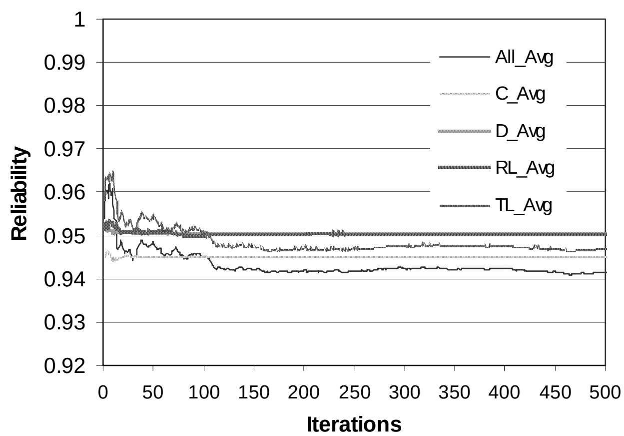
(d) 18th hour



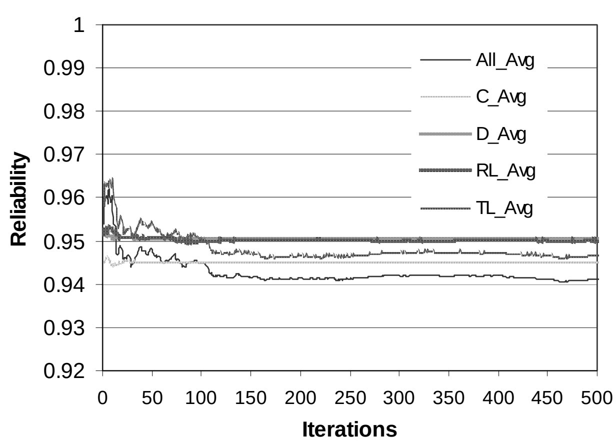
(e) 24th hour



(a) 0th hour

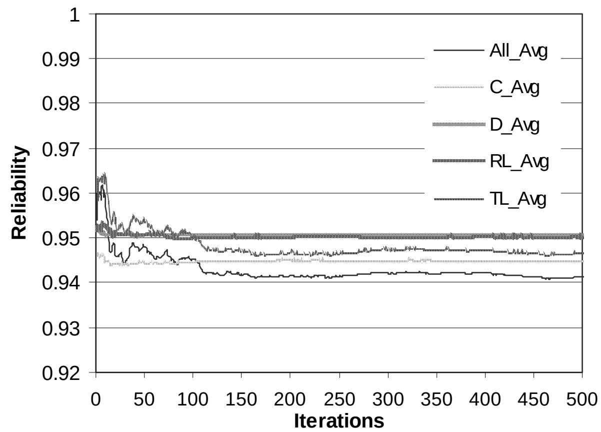


(b) 6th hour

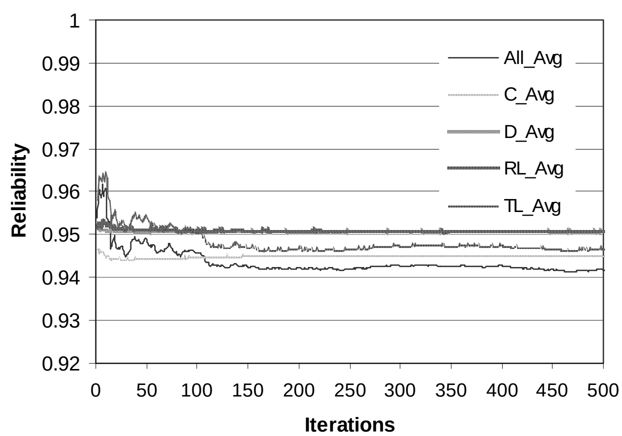


(c) 12th hour

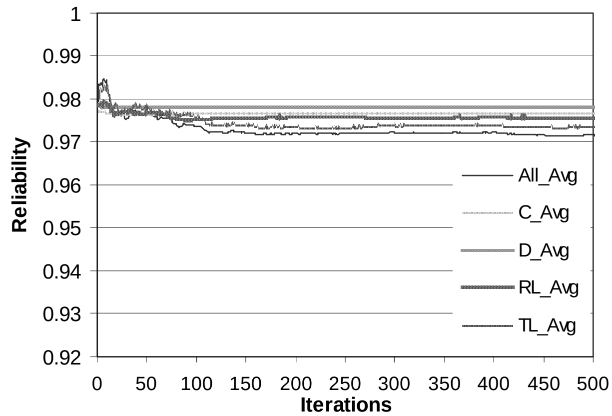
Figure A4.7: System Reliability for Case 7 (a) 0th, (b) 6th, (c) 12th, (d) 18th and (e) 24th hour



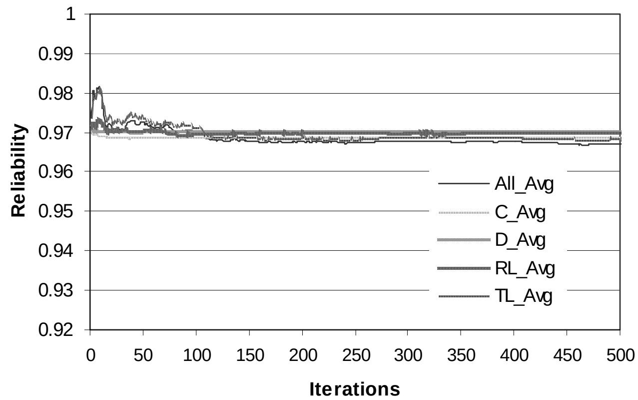
(d) 18th hour



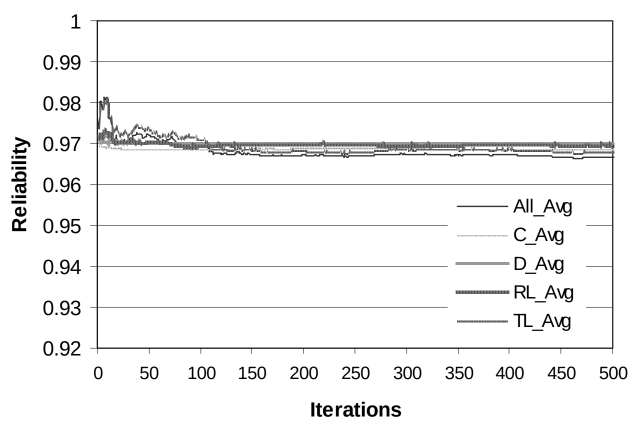
(e) 24th hour



(a) 0th hour

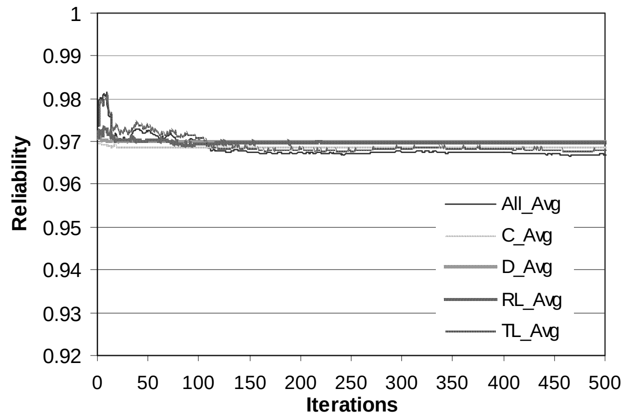


(b) 6th hour

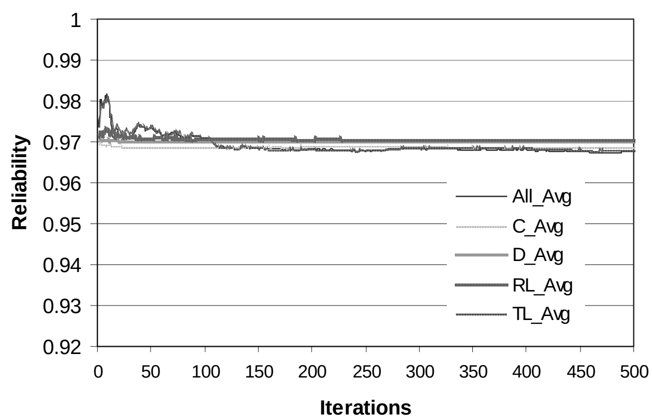


(c) 12th hour

Figure A4.8: System Reliability for Case 8 (a) 0th, (b) 6th, (c) 12th, (d) 18th and (e) 24th hour



(d) 18th hour



(e) 24th hour

VITAE

- Juned Laiq Syed
- Born in Hyderabad, Pakistan in 1975
- Received Bachelor of Engineering (BE) in Civil Engineering from NED University of Engineering and Technology, Karachi, Pakistan in 1999
- Worked as a Lecturer at the Civil Engineering Department of NED University of Engineering and Technology, Karachi, Pakistan from 2000 to 2001
- Received Master of Science (MS) in Civil Engineering from King Fahd University of Petroleum and Minerals, Dhahran, Saudi Arabia in 2003