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IMPLICATIONS OF THE PRESSURE DEPENDENCY OF OUTFLOWS ON DATA MANAGEMENT, MATHEMATICAL MODELLING AND RELIABILITY ASSESSMENT OF WATER DISTRIBUTION SYSTEMS

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ABSTRACT

This thesis investigates the implications of the pressure dependency of outflows in water supply network systems in aspects of data management, mathematical modelling and reliability assessment.

With regard to data management, the reliability of the collected field data for different elements of demand is increased by data reconciliation procedures developed here. By relating the leakage patterns to the pressure variations, the water consumptions and system leakage are evaluated realistically. In consequence, the uncertainties in water consumption calculations are reduced in circumstances where the domestic consumptions are not metered, as in the UK. Therefore, more reliable data are entered into the network models which are used for management and planning of water distribution networks. In addition, a minimisation procedure is performed using a best estimation parameter technique to assess different elements of water demands and leakage. This technique gives the best appraisal of the hydraulic performance of the system, producing the optimal values of flow and demands, and identifying the possibility of unknown hydraulic connection between adjacent zones.

In respect of mathematical modelling, a head driven simulation method of analysing water pipe networks is presented. Incorporating a suitable head-outflow relationship, the values of nodal outflow and head are obtained realistically. This improves the shortcomings of the conventional demand driven network analysis which considers the nodal outflows equal to pre-determined demands, regardless of the nodal pressures. It is shown that for subnormal conditions caused by mechanical or hydraulic failures, the head driven analysis produces more realistic results without any significant loss of computational efficiency, in comparison with the available demand driven simulation approaches.

Finally, a realistic, easy to implement and computationally efficient reliability measure to assess the hydraulic performance of the system is introduced. This measure which uses the results of the head driven simulation method for available outflows, is defined as the ratio of the available outflow to the demand and takes into account the probability of any component failures in the water supply system. In this investigation, an acceptable approximation is introduced first, for reliability assessment by using the improved source head method. This incorporates the nodal outflows with the source head variations of a single source network in a demand driven simulation approach. Then, a head driven simulation based reliability analysis for general multi source networks capable of including ancillary components is presented. Afterwards, applying extended period simulation, diurnal variations of nodal or system reliabilities and damage tolerances through the 24 hours of a day are obtained. Furthermore, by considering the probabilistic nature of demand, it is shown that more realistic results can be achieved for reliabilities and damage tolerances than by using deterministic demands. Finally, it is illustrated that using the demand weighted mean of the hourly reliabilities, separate calculation of the overall daily reliabilities can be avoided. Having this comprehensive and realistic reliability measure the most reliable (or critical) nodes, times and configurations are recognised in water distribution systems.

DEDICATION

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NOTATIONS

α	a coefficient in the Hazen-Williams equation
α_l	expected rate of repair of link l per unit of time
$\in$	belongs to
β_l	expected rate of failure of link l per unit of time
λ_{rs}	a permissible tolerance value for reservoir head level
μ_l	number of breaks in link l / unit of pipe length / unit of time
∂F_j	differential of function F at node j
∂H_j^m	differential of head at node j and iteration m
ΔH_j^m	head-correction at node j in iteration m
ΔH_L	total head loss in loop L
Δh_{rs}	change in reservoir water level
ΔQ_{ij}	corrective flow rate in pipe ij
Δt	time interval between times t and t+1.
Δv_{rs}	change in volume of reservoir equivalent to change of water level, h_{rs}
ε	tolerance
$\forall$	for all
Π	multiplication
Σ	summation
∞	infinity
$\int$	integral
$\lvert\ \rvert$	absolute value
$\lVert\ \rVert$	Euclidian norm
$a_j(t)$	nodal availability node j and time interval t
a_l	availability of link l
a_{lp}	availability of pump
a_{lv}	availability of link including valve
a_{tk}	availability of storage tank
$a_{t,j}$	an optimization coefficient equivalent to the demand consumption Type j and time t
A	system availability

A_i	leakage area path at state i
A_j	availability of node j
A, B, C	parameters of SCF
A_p, B_p, C_p	pump characteristics curve coefficients
ANP	average network pressure
AZNP	average zone night pressure
AZP	average zone pressure
b_j	a constant at node j
BE	a set of boundary elements like booster pumps
c_j	a constant at node j
C_1	background loss on distribution mains
C_2	background loss on service pipes
CHW_{ij}	Hazen-Williams coefficient of pipe ij
CPU	central processing unit
dH	a small portion of head
dt	a small portion of time
D_{ij}	diameter of pipe ij
D_l	diameter of link l
D-modified	modified domestic demand
D(t,t+1)	average network demand for interval of Δt
DC_s	average domestic consumption per capita of population
DF(t)	demand factor at time t
DI	distribution input
DL	distribution losses
DMA	district metered area
DOU	distribution operational use
e_{rsc}	allocated corrected error to the reservoir rs
e_{rsp}	allocated corrected error to the reservoir rs
E_c	corrected error
E_p	predicted error
F	a function
F_j	a residual value of flow balance at node j

F_j^m	a residual value of flow balance at node j and iteration m
FAVAD	fixed and variable area discharge paths concept
FCV	flow control valve
f(t)	probability density function of the time to failure of the component.
f_{ij}	a function expressing characteristics of pipe ij
f_{rs}	a function for representing head-volume relationship of reservoir rs
F_n	a node factor
F_t	a time factor
$\underline{F}(\underline{x})$	a vector of nodal residuals for unknown variables of $\underline{x}$
gpm	gallon per minute
h_{ij}	head loss in pipe ij
h_p	total head loss along the path p
h_{rs}	reservoir head level
h_{rsc}	corrected reservoir head level
h_{rsp}	predicted reservoir head level
hd	head (person)
hr	hour
H_{be}	head at boundary elements BE
H_j	available head at node j
H_j^{des}	desired head at node j
H_j^m	available head at node j and iteration m
H_j^{min}	minimum head at node j
H_p	lift head across pump
H_{prv}	outlet head of pressure reducing valve
H_s	available head at source
H_s^{des}	desired head at source
H_s^{min}	minimum head at source
$H_{s,j}^{des}$	desired head at source to satisfy the required outflow at node j
$H_{s,j}^{min}$	minimum source head corresponding to zero outflow at demand node j
HDSEPRA	head driven simulation based extended period reliability analysis
HDSRA	head driven simulation based reliability analysis

HDSRAPD	head driven simulation based reliability analysis with probabilistic demand
HDSM	head driven simulation method
IEPPD	Integrated extended period reliability analysis with probabilistic demand
IJ_d	set of all pipes in a specified path between the source and demand node
IJ_j	all links connected to node j
IJ_L	links of loop L
IJ_p	links of path p
ISHM	improved source head method
$\underline{J}$	Jacobian matrix
J_i	all nodes j in the vicinity of node i which are connected to it directly
K	a network resistance coefficient
K_{ij}	resistance coefficient of pipe ij
K_j	resistance coefficient of node j
K_v	a continuous valve control parameter
L	leakage
L_{ij}	length of pipe ij
L_l	length of link l
L_m	distribution mains length
L_t	leakage at time t
L_{MNFT}	leakage at MNF time
L-ANP	leakage based on the average network pressure profile
L-AZP	leakage based on the average zone pressure profile
L-modified	modified leakage
L-WRC	'WRc' standard Type 5 (leakage)
LF(k)	load factor at load bound k
LI	leakage index
LOUC	legitimate overnight unmetered consumption
LOUCP	legitimate overnight unmetered consumption per property
MC	total daily metered consumption
MCC	metered commercial consumption
MNF	minimum night flow

MNFT	minimum night flow time
MTBF	mean time between failures (duration of connectivity)
MTTR	mean time to repair (duration of unconnectivity and repair)
n	an exponent
n_j	an exponent at node j
N	an exponent
NB	number of breaks
NFH	number of fixed head nodes
NFN	net night flow
NJ	number of nodes
NJ_d	number of demand nodes
NL	number of loops
NLB	number of load bands
NP	number of pipes
NPP	number of paths
NRV	non-return valve
NT	number of time intervals
NV_{max}	number of time which the nodal heads are violated
NVH	number of non fixed head nodes
M	representative of unavailable components
OCC	average household occupancy rate
p(0)	probability that all links are available
$p(l)$	probability that link l is unavailable
phf_j	probability of hydraulic failure at node j
prop	property
P_t	average network (or zone) pressure value at time t
P_{MNFT}	average network (or zone) pressure value at MNF time
PCC	per capita consumption
PCC_{act}	per capita consumption value which is produced by using the field data
PCC_{ave}	network average per capita consumption
$PCC_{correct}$	unknown per capita consumption which would lead to the correct diurnal characteristics

PCF	Pressure correction factor related to Leakage Index at 40 m
PDF(t)	probability of demand factor at time t
PLF(k)	probability of load factor at load band k
Pop	population
PRV	pressure reducing valve
$q_{be}(t)$	flow rate to or from the boundary element at time t
$q_{rs}(t)$	flow rate to or from the reservoir rs at time t
Q_{be}	net outflow from boundary elements BE
Q_{ij}	flow in pipe ij
Q^{0}_{ij}	initial assumed value for flow rate at element ij
Q_{ij}^{m}	flow in pipe ij at iteration m
Q_{isol}	combined demand of the isolated nodes
Q_j	external output or input at node j
Q_j^{ave}	time averaged demand at node j
Q_j^{avl}	available outflow at node j
$Q_j^{avl}(M)$	available outflow at node j with M components unavailable
$Q_j^{avl}(k,t,l)$	available outflow at node j, time t and load band k with link l unavailable
$Q_j^{avl}(t,l)$	available outflow at node j and time t with link l unavailable
Q_j^{req}	required outflow (demand) at node j
$Q_j^{req}(k)$	required outflow at node j and load band k
$Q_j^{req}(k,t,0)$	required outflow at node j, time t and load band k with all components available
$Q_j^{req}(M)$	required outflow at node j with M components unavailable
$Q_j^{req}(t,0)$	required outflow at node j and time t with all components available
Q_p	flow delivered by pump
Q_{rs}	net outflow from reservoirs rs
Q_s^{req}	required total outflow (demand)
$Q_s^{avl}(k,t,0)$	available total outflow at time t and load band k with all components available
$Q_s^{avl}(k,t,l)$	available total outflow at time t and load band k with link l unavailable
$Q_s^{avl}(M)$	available total outflow with M components unavailable

$Q_s^{avl}(t,0)$	available total outflow at time t with all components available
$Q_s^{avl}(t,l)$	available total outflow at time t with link l unavailable
$Q_{t,0}$	net inflow to a zone at time t
r	ratio of Q_s^{avl}/Q_s^{req}
r(M)	ratio of $Q_s^{avl}(M)/Q_s^{req}$
r_j	ratio of Q_j^{avl}/Q_j^{req}
$r_j(M)$	ratio of $Q_j^{avl}(M)/Q_j^{req}$
$r_j(t,k,0)$	ratio of $Q_j^{avl}(t,k,0)/Q_j^{req}(t,k,0)$
$r_j(t,k,l)$	ratio of ${}_j^{avl}(t,k,l)/Q_j^{req}(t,k,l)$
R	system reliability
R_{FVL}	ratio of fixed area leakage to variable area leakage
R_L	system reliability, lower bound
R_U	system reliability, upper bound
R_j	reliability at node j
$R_{j,L}$	reliability at node j, lower bound
$R_{j,U}$	reliability at node j, upper bound
Rd	data reliability
Res_t	residual at each time step t
RF	ratio of the $\lvert F(H^{m+1})_{z=1}\rvert$ / $\lvert F(H^{m+1})_{z=0}\rvert$
sgn	sign
S_j	a constant dependent on the outlet characteristics
SCF	sampling period correction factor
SHM	source head method
SP	sampling period
t	time t
t′	an intermediate time between t and t+1
T	system damage tolerance
T_j	damage tolerance at node j
$T_{j,L}$	damage tolerance at node j, lower bound
$T_{j,U}$	damage tolerance at node j, upper bound
T_L	system damage tolerance
$T_{t,j}$	Type j consumption at time t

T_U	system damage tolerance
TDF	total daily net flow into DMA
TIF	total integrated flow
TT	total time
ua_l	unavailability of link l
U	unreliability
U_j	unreliability at node j
UFW	unaccounted for water
UFWM	Unaccounted for water at minimum flow condition
$v_{rs}(t)$	reservoir volume at time t
v_{rsc}	corrected reservoir volume at time t
v_{rsp}	predicted reservoir volume at time t
vol_{tk}	storage tank capacity
V_i	velocity at state i
WD	water delivered
WTM	miscellaneous water taken
$\underline{x}$	vector of x value
x_j	unknown variable for demand element j
$x_{i,j}$	unknown variable for demand element j in zone i
z	a coefficient which minimize the Euclidean norm of the single variable function $f(\underline{H} - z\,\underline{\Delta H})$

CHAPTER 1

INTRODUCTION

1.1 BACKGROUND

Design, construction, operation and maintenance of water supply systems are the most important aims of water utilities. The ultimate objective of water supply systems is to obtain water from accessible sources, treat it to an acceptable quality and then deliver the required quantity under the desired pressure to the appropriate place, at the required time without any interruption at a minimum cost. Therefore, the full provision of customer demands should be satisfied by the proper planning, design and management of the network.

To ensure reliable delivery of water to the end users, the system components are designed to satisfy a range of expected loading conditions such as average daily/hourly, maximum daily/hourly, or maximum immediate demands e.g. fire fighting demands. Furthermore, consideration of the probabilistic nature of demand helps to validate the design outputs in respect of demand variations through a period of time, based on change of climate, consumptions patterns, seasons and days, population, etc.

Water distribution modelling plays an important role in the management, future planning and design of water systems. Two essential requirements in the assessment of existing networks and the simulation of future system behaviour are that: i) the methodology should realistically evaluate the hydraulic performance of the system and; ii) accurate demand analysis is needed which produces reliable input data for different types of consumption and leakage evaluation.

In crisis events (both mechanical and hydraulic failures) maintaining adequate flows and pressures in the network are the main objectives in water system management. The individual components and their interactions must be capable of accommodating critical conditions such as failure of pipes (bursts), pumps (due to breakage or outage), valves, reservoirs, sources, etc. and also exceedance of demand values from the designed levels. Furthermore, hydraulic failure caused by any events which lead to insufficient heads and consequently outflows, must be taken into account. All these events should be evaluated to recognise and minimise their impact on the hydraulic performance of the system.

Another important aspect of water supply networks relies on determining an appropriate and realistic measure for evaluating the performance of the system under both normal and subnormal situations. The capability of the water supply networks to deliver the required flow under adequate pressures in normal and failure conditions is measured by the concept of reliability. Although there is not a universal definition for reliability in application to water distribution networks, several researchers have tried to present proper and explicit measures to calculate the reliability of these systems. Furthermore, the ability of the system to perform its mission under the failure conditions is measured by the concept of damage tolerance. Application of this concept to water supply systems measures the severity of failures on the hydraulic performance of the network.

1.2 NEED TO IMPROVE DATA MANAGEMENT, MATHEMATICAL MODELLING AND RELIABILITY ASSESSMENT OF WATER DISTRIBUTION NETWORKS

During the last two decades several measures and definitions have been developed for the reliability in water systems. Initially, some methods were adapted from other engineering fields such as power distribution and telephone networks. However, it was recognised that a realistic measure for water pipe systems should account for the specific features of these networks, i.e. pressure and demand. Therefore, as well as the mechanical failures, the effects of pressure and demand variations on the hydraulic performance of the system (i.e. hydraulic failures) was gradually recognised. Several pieces of research can be found which attempt to develop a hydraulic reliability measure (e.g. Wagner et al. 1988b; Bao and Mays 1990; Cullinane et al. 1992). The probability of pressure being less than a certain level or demand being greater than the design values, together with a failure/non-failure classification, etc. are some of these measures. Subsequently, the importance of the occurrence of shortfalls in delivery resulting from demand based evaluations was illustrated as a good indicator for the reliability.

In recent years, the ratio of the actual water delivered to the total demand has been widely accepted as one of the most appropriate definitions for reliability of water supply systems (e.g. see Carey and Hendrickson 1984; Fujiwara and De Silva 1990; Tanyimboh and Templeman 1995, etc.). For this purpose, a network model which can provide the actual outflow is highly desirable. Consequently, the necessity of addressing the pressure dependency of demand has been emphasised in order to produce a realistic reliability measure for water networks (Bhave 1981; Germanopoulos et al. 1986; Tanyimboh 1993, etc.). To address these needs, a straightforward, easy to implement and computationally inexpensive methodology is required to analyze the reliability of water supply networks more realistically, whilst the pressure dependency of demand is maintained.

Besides the necessity of head driven analysis to obtain a realistic reliability evaluation, there is a growing need for head driven simulation methodology to analyze water supply networks in general. For example, the conventional demand driven methods are not suitable for analysing the intermittent supplies in some developing countries because of very strong effects of pressure variations on the available outflows (Lumbers 1996). However, most existing methodologies in modelling applications use the method of demand driven analysis to simulate the hydraulic behaviour of the pipe systems. This kind of analysis is unable to consider the relationship between nodal flow and pressure and thus considers the system outflows equal to the demands, regardless of the actual pressures. The outputs can therefore produce unrealistic predictions in respect of available heads and outflows. Thus, use of the head driven simulation method is crucial to realistic design and operation of water supply systems.

Unfortunately, this kind of analysis has received very little attention in the past. The few available methodologies are lacking in certain respects. These include computational efficiency, ease of use and/or accuracy of results. Some of these use optimisation procedures (such as Fujiwara and De Silva 1990; Fujiwara and Tung 1991; Fujiwara and Ganesharajah 1993) or some non-straightforward head driven analysis algorithms (e.g. Chandapillai 1991; Gupta and Bhave 1996b). However, they are either complicated or computationally time consuming. Therefore, to overcome these shortcomings, a comprehensive methodology is required for head driven simulation of water distribution networks by incorporating a realistic head-outflow relationship into a straightforward algorithm.

Having a realistic model, however, is not the only requirement, because the data sets which are used by such computer models are also important. Therefore, the reliability of water consumption data is crucial not only for operational management of water supply systems, but also for future development of these systems. Unfortunately, the field data which are normally collected in the UK for different categories of demand, are error prone. Because, in this country, domestic consumptions are not metered, domestic per capita (PCC) figures follow from assumptions made regarding the diurnal variations in the leakage during 24 hours. This leads to considerable variations and uncertainties in the elements of water consumption. Consequently, considerable bias is likely to arise in total flows which may give unrealistic predictions of the future hydraulic performance of the system. Therefore, these data might be unable to represent accurately the different elements of demand. Thus, some new practical methodologies are required to maximise the quality control on estimation of the PCC and leakage and their diurnal profiles from reconciliation of the collected data sets.

1.3 OBJECTIVES OF THE RESEARCH

The main innovation in this research is the establishment of a series of new methodologies addressing implications of the pressure dependency of outflows on operational management, mathematical modelling and reliability assessment of water distribution networks. This analysis takes into account the pressure dependency of different elements of water demand and leakage to produce realistic figures for networks flows and heads. The objectives of this research can be outlined as follows:

1. To develop methodologies for data management to represent the variations of demand elements realistically and to reconcile any anomalies in the collected field data.
2. To establish a head driven simulation method to analyze the hydraulics of the water distribution networks.
3. To obtain a comprehensive measure for reliability assessment of water supply systems.

In respect of the operational management and demand analysis, the relationship between pressure with leakage will be investigated. The pressure dependency of leakage has been recognised in recent years and is used as an important tool in the area of pressure management for leakage reduction. Therefore, firstly, realistic patterns are determined for representation of the water demand and leakage variations from management and modelling points of view. Then, a series of data reconciliation and parameter estimation methodologies are developed to improve the reliability of the existing data sets collected from the field.

Regarding the mathematical modelling of water supply systems, a head driven simulation method is developed to analyze the system with respect to the nodal head-outflow relationship. This overcomes the shortcomings of the conventional demand driven methodologies.

Finally taking into account the advantage of establishing a head driven simulation method to analyze the water distribution network, a head driven simulation based reliability analysis is developed to assess the reliability of water supply systems more realistically. This method uses the actual system and nodal outflows resulting from the head driven analysis. Also, besides inclusion of the major components of general networks, it considers the possibility of both mechanical and hydraulic failures together with the probabilistic nature of demand. The method has the advantage of demonstrating the diurnal variations of system and nodal reliabilities using extended period analysis.

1.4 OUTLINE OF THE PRESENT THESIS

This thesis is organised into three main parts.

i) Demand and leakage evaluation;

ii) Mathematical modelling; and

iii) Reliability assessment of the water supply networks.

In outline, Chapters 2-3 and 4-5 cover the first two parts respectively, and Chapters 6-8 discuss the subject of the third part.

To identify the most important parameters of demand in water supply systems, Chapter 2 presents different categories of water demands. Besides the domestic and non-domestic consumptions, the concept of unaccounted for water will be discussed. Furthermore, the characteristics of leakage and its relationship with pressure are illustrated. Chapter 3 develops a series of practical methodologies to determine realistic patterns for diurnal variations of leakage to be used in simulation and management models. Considering the uncertainties in the data, water demand patterns are also evaluated. In addition, some practical techniques to reconcile the collected data sets from the field are demonstrated, including the use of an optimization procedure for best parameter estimation of demand elements.

Chapter 4 comprises a brief description of the available algorithms for simulation of water distribution networks including both the steady state and extended period analyses. Following evaluation of the existing head-outflow relationships and head driven algorithms, a methodology is developed in Chapter 5 for the head driven simulation of water supply systems which takes into account a head-outflow relationship for all demand nodes. Using a number of examples the capability of the head driven analysis methodology against the conventional demand driven simulation method is demonstrated.

A comprehensive review of literature on the available definitions and methodologies for reliability assessment of water distribution networks is made in Chapter 6. Furthermore, the concepts of mechanical and hydraulic reliability are discussed. A method of calculating the reliability in single source networks is developed in Chapter 7 which incorporates the relationship between the source head and nodal outflows using demand driven analysis. A rigorous method for reliability analysis is presented in Chapter 8 for both steady state and extended period simulation of water distribution networks using the advantages of the head driven simulation method. This method evaluates both the nodal and system reliability and damage tolerance of general multi source networks including ancillary components such as pumps, valves, reservoirs, etc. The final parts of this chapter demonstrate incorporation of probabilistic demands to the analysis procedure in order to produce more realistic results for reliability assessment.

Finally, Chapter 9 presents a general conclusion from the findings of this research. Also, some suggestions are introduced for further work to improve on the methodologies and findings of the present research.

In the appendices, besides some relevant documents to support the text materials, including the computer programs, papers which have been produced from the findings of this study, and published at the time of writing this thesis, are presented for further reference.

CHAPTER 2

WATER CONSUMPTION AND ITS CATEGORISATION

2.1 INTRODUCTION AND BACKGROUND

The provision of water supply at the appropriate quantity and quality has been the aim of all civilized societies whose standard of living has been founded upon the availability of potable water on tap 24 hours a day. The planning, design, operation, management and development of water supply and distribution systems are strongly dependent on water demand and water consumption information. Consequently, water consumption data are essential inputs to computer applications used to support the decision making process. Therefore, the accuracy of these applications is closely related to the available knowledge of the ways in which water is used by the population and the reliability of data concerning demands.

In most countries the majority of consumptions are metered. However, in the UK only trade and industrial consumptions are presently metered, most domestic and small trade supplies (more than 20 million households) are unmetered. It should be noted that this situation is special to the UK, perhaps because of its cool and usually wet climate, and efficient plumbing standards, etc. (Twort et al. 1994). However, this situation leads to a series of uncertainties in the data management process, which will be discussed in this Chapter. In view of its high cost the benefits of metering domestic demand seems uncertain in the UK. According to AWWA (1973) there is no indisputable proof that metering permanently reduces the domestic demand except when accompanied by a large increase in the price of water. However, Lambert (1997b) concluded that metering (of new households) is essential especially for effective implementation of pressure management schemes, otherwise such schemes become uneconomic for water companies.

To analyze the water supply system by a simulation model or to assess the behaviour of each water supply network from a management point of view, it is necessary to determine different demand patterns. In general, there are several types of consumption which are conveniently divided into metered and unmetered consumptions. In the UK of the total quantity put into public supply, about 14700 Ml/day on average, some 40% is used in the home and 30% is metered and is used by industry, commerce and agriculture. The remaining 30% represents unmetered supplies to commercial premises, fire fighting, sewer flushing and leakage (Phillips 1983). According to a Water Research centre (WRc) classification linked to use of their 'WATNET' network flow simulation computer package, categories of metered trade/industrial consumption are classified into domestic equivalent (i.e. shops, hotels, etc.), 10 and 24 hour commercial and industrial activities, represented as Type 2 to Type 4 respectively, and domestic consumption and leakage which are considered as Type 1 and Type 5. Furthermore, any exceptional night use, i.e. usages greater than 500 l/hr, can be classified as Type 6. It should be mentioned that there are currently several other water supply simulation models in use, for example GINAS, STONER, KYPIPE, WESNET, EPANET, etc. Each of these models has its own specific features and abilities. They also make different provision for description of types of consumptions and associated factors. In most parts of this study for consistency in the research only the 'WRc' standards are considered.

In present practices for water distribution management and leakage control, data is normally collected from District Metered Areas (DMAs) where different types of demand are usually assessed to appraise Per Capita Consumptions (PCC) and leakage figures. A DMA is a section of the distribution system supplied through one or more inlet or outlet meters and otherwise isolated by boundary valves from adjacent DMAs. Each DMA potentially has unique characteristics of the resident population, number of properties, length of trunk mains, mains length per property (urban/rural), number and types of non-households with different typical night use, number of properties with exceptional night use less than 0.5 m^3/hr, average zone night pressure AZNP (m), infrastructure condition/background loss levels and incidence of bursts on mains and services (UK/WI 1994-Report F). To calculate the distribution of domestic demand it is necessary to know the inflow to each area. For this, based on the district metering method, each area is divided into several local supply zones. Some companies refer to these zones as 'Waste' zones (districts), or 'Pressure zones' which are equivalent to the DMA. Flow is metered at strategic points throughout the distribution system, each meter recording flows into a district which has a defined and permanent boundary. Ideally, in each zone only one valve which is connected to a source is normally open. In each zone, the value of net inflow may be obtained from the sum or difference of multiple inflows/outflows. Also, in each zone the number of unmetered and equivalent properties must be ascertained. Typical district size varies between 500 and 5000 properties. However, those designed around reservoir zones or bulk meter areas, are larger, 5000-10000 properties (UK/WI 1994-Report J). Figure 2.1 represents a typical DMA.

To assess the scale of demand, a measure of overall consumption per person connected to the system is usually used. This measure which is expressed in litres per head per day (l/hd/day) can vary over a wide range due to several factors. For example, Twort et al. (1994) have reported values from 600-700 l/hd/day as the highest consumptions in highly industrialised cities to 90-159 l/hd/day for small towns with little industrial demand and a low standard of housing. The consumption figure is affected by the distribution of leakage and customer wastage, ratio of trade to domestic consumption, garden watering, use of waterborne sanitation, metering efficiency and sufficiency of water to meet total demand 24 hours of a day, etc. In a reliable water distribution network, the required water should be available at adequate pressure whenever it is needed. However, in some developing countries like India and Pakistan, etc. supplies are intermittent i.e. water is available for a few hours in a day. Evaluation of such systems needs different methods (see e.g. Lumbers 1996).

According to the (UK/WI 1994-Report B) the following balance equation should usually be satisfied.

$$DI = WD + DOU + DL \tag{2.1}$$

in which DI, WD, DOU and DL represent distribution input, water delivered, distribution operational use, and distribution losses, respectively. Values of water delivered (WD) can be obtained as:

WD = Water Delivered billed (measured + unmeasured) +
unbilled Water Taken (legally + illegally) *(2.2)*

where water delivered billed, measured and unmeasured, are the total measured and unmeasured household and non-household uses. Water taken legally unbilled includes supplies for which no charge is made e.g. fire fighting, sewer flushing, etc. DOU includes service reservoir cleaning, mains flushing, etc. Finally, distribution losses (DL) are the residual part of flow balance which accounts for losses (leakage). According to Eq. 2.1, in the case of too high and unpredicted losses which cause more head loss, some customers face insufficient head and outflow.

2.2 DEMAND ELEMENTS

According to the 'WRc' approach which is used in the 'WATNET' simulation model (WRc 1992) each type of demand is made up of three different elements which are multiplied together.

Flow (l/s) = Base demand x Demand weighting x Demand factor *(2.3)*

Base demand is specified on a node by node basis. Demand weightings are system wide. There is only one demand weighting for each demand type. Demand factors reflect differences in diurnal variations of different types of consumption. These three elements of the specification can be configured according to the users convenience. There are many possible definitions for elements of demand. One of these formats which is used in this study is represented as follows:

2.2.1 Base Demands

Type 1,5: Number of unmetered properties and type 2 equivalent properties (if selected)

Type 2,3,4: Daily consumption which is metered (m^3/day)

For Rural areas base demand for leakage (Type 5) is often represented by mains length rather than number of properties connecting to the nodes.

2.2.2 Demand Weightings

Type 2,3,4 = 1000/(24 x 3600) = 0.01157 *(2.4)*

(converts m^3/day to l/sec)

Type 1,5 = 1/3600 = 0.000278 *(2.5)*

(converts l/prop/hr to l/prop/sec)

2.2.3 Demand Factors

Demand factors describe the variation in consumption during the day. These factors for Types 2, 3, 4 and 5 are normally taken from standard 'WRc' diurnal profiles which can be seen in Table 2.1 and Figure 2.2.

In the next sections different metered and unmetered categories of demand are discussed.

2.3 DOMESTIC CONSUMPTION

Per Capita Consumption (PCC) or domestic demand is highly variable. Considering variation of climates, social, demographic and economic factors, a wide range between 120-195 l/hd/day have been reported in the literature for the UK. For example, results from a study on 700 households classified by household size, indicated that average consumption during 1993/1994 was 145 l/hd/day and 95% confidence range associated with these data was 140-150 l/hd/day (UK/WI 1994-Report D). Anderson (1997) asserted that PCC is predicted to increase at 1.1 l/hd/year, and supply pipe leakage is predicted to decrease. In reality the level and pattern of PCC is affected by the following parameters.

1. Socio-economic classification of the household.
2. Personal life pattern.
3. Climate conditions in each area and differences for each season
4. Differences between working days and holidays.
5. Specific local features, like tourism, etc.

2.3.1 Survey Data

In a survey for Great Britain the following findings have been reported by Bailey et al. (1986). The daily volume of water consumed per household for non-potable purposes is dependent on household size. However, for potable purposes it is about 8.9 litres and can be regarded as independent of household size. The daily volume of water consumed for all purposes is about 12% higher at weekends than on weekdays. Also, the daily volume of water consumed for non-potable purposes depends on social group, but appears to be independent of geographical region.

In some UK surveys the 'ACORN' classification of housing (CACI 1981) has been adopted, however, Twort et al. (1994) asserted that it is not well suited to analysis of domestic demand. The 'ACORN' system is a classification of residential neighbourhoods which classifies properties through England and Wales into a number of categories, such as modern family housing, higher income and poorest council estates, etc. (see Appendix A). Edwards and Martin (1995) presented a methodology for surveying domestic water consumption in East Anglia (England), named as 'SODCON', in which variations of PCC with several aspects like household size, housing type ('ACORN' group), socio-economic grouping, rateable value of properties, etc. were investigated. Also, Alegre and Coelho (1993) developed a method for characterization of water consumption in Portugal. It assessed the daily and weekly demand profiles over yearly periods, as well as the identification of the main socio-demographic and habitat factors affecting them. Despite the different situation in the UK in which domestic demands are not metered, their observations are more or less applicable in this country. For example, they observed that the seasonal effects are highly influenced by the private back garden. Also, the economic standards of living and the age of the population are two features that affect the percentage of people who go away for their summer holiday. Further, they found that the average consumption of residential and service apartments (offices and businesses) are similar. They also observed major differences between weekly and daily profiles. Finally, they did not find sufficient evidence for a clear influence of socio-demographic characteristics on average PCC.

Suggested design allowance for domestic demand may vary based on the range of family income. For example, 190 l/hd/day (in UK and Europe) and 230-250 l/hd/day (in warm climates) for the highest income group and 50-55 l/hd/day (in warm climates) for the lowest income group has been reported by Twort et al. (1994).

Domestic demand rises because of improvement of housing and living standards and increase of average household occupancy rate (OCC). Average occupancy rate in England and Wales varies in a range of 2.40-2.75 head per property (hd/prop) in different areas, which is less than many other countries (Twort et al. 1994).

Components of domestic demand may include drinking water, gardening demand, cooking, sanitation, etc. Drinking water consumption may vary from 0.75 to 2.00 l/hd/day. Garden watering demand which is related to the weather can increase daily consumption by 40% in dry periods. In different surveys in the UK, sanitation usage has been reported in the range of 50-113 l/hd/day and values of 38-108 l/hd/day for washing and cooking.

The maximum consumption for a day is usually expressed as a percentage of the average annual daily supply. The highest percentage occurs when a dry summer is experienced. These values have been reported in the range of 110% - 300% for different countries and climates. Also the maximum hourly demand depends on the size of area served and the nature of the demand. The peak flow factor has been reported from 1.4 to 4.9 (Twort et al. 1994).

2.3.2 Computation of PCC

The PCC can be calculated as follows

PCC (l/hd/day) = [TDF - (MC+L)] / [(No. of unmetered properties) x OCC] (2.6)

in which TDF and MC are total daily net flow into the DMA and total daily metered consumption in the district in l/day, respectively. The value of 2.75 hd/prop has been considered for OCC in this study.

Generally, domestic demand is expected to follow a pattern with the following features which can be seen in Figure 2.3.

1. Minimum value during night time.
2. A rising consumption over hours between getting up and going out for work in the morning.
3. Moderate values with some variations during working hours.
4. Further rising in late afternoon and evening times (coming back from work, eating dinner, washing, garden watering etc.).

The patterns for each property may be expected to be similar but their profiles are likely to show (random) variations, because every individual pattern is affected by the number of population, time, climate and socio-economic factors. On the other hand, an average pattern is normally implemented to represent the general characteristics of water demand in a region including several DMAs and networks. However, every regional pattern varies depending upon the size of districts. It means that for higher sample size (number of properties), less departure from the average values is observed.

2.4 NON DOMESTIC CONSUMPTION

Metered non-household demand can be divided into agriculture, energy, manufacturing, construction and service uses. Non-metered non-household demand can be divided into commercial properties such as shops, schools, offices, etc. and miscellaneous uses such as fire fighting, sewer cleaning and water mains flushing

(Anderson 1997).

Twort et al. (1994) have reported the range of 75-119 l/day/person for total non domestic consumptions (i.e. industrial, commercial and institutional uses) in England and Wales and 100-150 l/day/person for large industrial cities in western Europe. It is worth mentioning that like domestic consumptions, many small trade premises such as shops are not metered in the UK, however, a value of 20 l/day/person might be reasonable for this type of demand (TWGWW 1976). For metered use, example values of 350-500 l/day/patient for hospitals, 250 l/day/passenger for hotels, 25-75 l/day/person in schools and 65 l/day/person in offices can be mentioned as some typical consumptions in commercial and institutional usage (Twort et al. 1994).

A good classification for different industrial activities usage can be seen in the same reference. It includes cooling water demand, major industrial demand (1000-20000 m^3/day), medium to small industrial demand (less than 50 m^3/day) etc. Also, typical magnitudes of water demand by various manufacturing industries and light industrial estates are also reported. It has been found that for a large urban area, the largest industrial users use the highest proportion of the total industrial demand. Therefore, this raises the importance of accuracy of meters in such crucial consumptions.

The main sources of supply for irrigation are rivers and boreholes. In the UK some cases like dairies, cattle troughs, etc. use the public supply. For example, agricultural demand for mixed farming has been reported as 57 l/day/hectare of farmland (Twort et al. 1994).

Public usage includes parks, governmental buildings, fire fighting, sewer flushing, etc. which are not separately estimated in the UK because they are usually small. Reed (1980) reported a range of 0-104 l/hd/day for public services in some western European cities, but the nature of demand was not mentioned. There is little published data available on Miscellaneous Water Taken (WTM) i.e. distribution operation use, water taken via hydrants and water taken unbilled. Whilst WTM represents only a very small percentage of distribution input (DI), it can comprise a significant part of 'error terms' in any water balance calculation and does require particular justification in formal regulatory returns (UK/WI 1994-Report D).

2.5 LOSSES

Losses can be classified in terms of consumer waste and leakage from supply pipes (included in the non-metered household demand), distribution and trunk mains (including communication pipes), and service reservoirs (Anderson 1997).

2.5.1 Consumer Waste

Leakage and wastage on domestic premises may also include supplies by illegal connections. It varies according to the type of housing, leaking fixtures, and quality of plumbing and fittings, etc. In the UK, an average value of 15 l/hd/day is supposed to be adequate for consumer wastes e.g. dripping taps, overflowing cisterns, etc. (Twort et al. 1994).

2.5.2 Distribution Losses

Distribution losses include leakage from mains, service pipe connections and reservoirs together with reservoir overflows. Mains leakage has been reported as ranging from 100 to 300 l/hr/km or 0.2 (m^3/km/day) per year of age (UK/WI 1994-Report D). Recently, UK/WI (1994-Report B) have recommended the unit of 'm^3/km of distribution system/day' to scale distribution losses, instead of traditional measures of 'per km of main per unit of time' or 'per property per unit of time'. Reservoir losses in terms of leak and overflow are, however, reported to be not too high in the UK, possibly less than 1% of the reservoir capacity per day, on average ('WRc' 1978). Background losses, which contain leakage from mains, communication and supply pipes, in average conditions have been proposed as 40 l/km/hr for distribution mains and 4 l/prop/hr for services (UK/WI 1994-Report F).

According to the UK/WI (1994-Report B) the distribution losses vary based on the following factors: pressure, burst frequencies on mains and services, leakage control policy, PCC and age of the distribution system.

There is a range of error inherent in calculating distribution losses due to the degree of estimation which is necessary. Therefore, all derived measures of performance, simple or complex, should be expressed as falling within one of a number of 'bands', rather than as absolute values, even when all inputs and water delivered are metered (Weimer 1991).

Water leakage in a supply network may account for a large percentage of the total water supplied and can consequently represent a significant economic loss. Values as high as 50% have been reported in the case of some aging and deteriorating urban distribution networks (TWGWW 1980; Twort et al. 1994). In the UK, the leakage levels vary from one undertaking to another, depending on the system characteristics. Distribution losses have been reported to vary from 8-33% of distribution input. Twort et al. (1994) and UK/WI (1994-Report D) suggest that in the UK 22-25% of the total consumption per capita is the average amount of losses. As an indication of the potential benefits, it has been estimated that the total net savings available from the implementation of a proper leakage control strategy would be about $30m in Britain alone (Goodwin and McElory 1983). The consumption figures for the UK in 1990 can be observed in Table 2.2. Also, a comparison of the demand and leakage in three European countries in 1994/1995 is represented in Table 2.3. It shows that although the PCC values vary in a close range of 132-152 l/hd/day, leakage rates are very different. For instance, the leakage figure for England and Wales is twice as high as the rate in the Germany where the length of mains per property is higher. The most interesting characteristic is very low rate of domestic demand metering in England and Wales which probably is the main cause of the higher approximations for the leakage.

In modern leakage control methodology three categories are used: passive control, regular survey (sounding, waste metering) and leakage monitoring. The most widely used technique for leakage monitoring is 'continual night flow monitoring', by data logger or telemetry. A full review of new technology in the literature can be found in UK/WI (1994-Report J) as well as leak location and detection methods.

Leakage has proved very difficult to measure because of the lack of metering points on the water distribution network. If demand patterns and values were better understood, more accurate estimate of leakage would be possible (Clarke et al. 1997). The estimation of leakage is, however, complex, and estimations of the different component of leakage - in mains, service pipes and customer property - require different approaches. The most common practice to identify the leakage figure will be reviewed in the next section.

2.6 UNACCOUNTED FOR WATER (UFW)

The only practical way of obtaining an acceptable figure representing the level of leakage is by making an estimate of unmetered consumption (either total daily or night consumption). The inherent inaccuracies of any such estimate result in the figure obtained for leakage being somewhat approximate. Any flow measurement in the mains system is of an aggregate including household and non-household water consumption, supply pipe leakage, distribution system leakage and any operational use of water. It should be made clear that unaccounted for water includes:

i) Water which is used legitimately but which is not accounted for

ii) Leakage

iii) Error in the flow measurements (positive or negative)

Many factors influence UFW (i.e. standard of housing, rates of occupancy, age of mains, length of mains per 1000 population served, proportion of trade and bulk supplies, metering policy, distribution pressures, ground conditions, etc.) which differ from one undertaking to another. The following two methods are used for calculating UFW.

2.6.1 Total Integrated Flow (TIF)

UFW can be expressed as (TWGWW 1980)

$$UFW = TDF - (MC + DCs \cdot Pop) \qquad (2.7)$$

where UFW = unaccounted for quantities of water, TDF = total daily flow into the system, MC = sum of all water accounted for by measure (metered), DCs = average domestic consumption per capita of population plus an allowance for unmetered commercial consumption and Pop = population.

2.6.2 Minimum Night Flow (MNF)

MNF is the measured rate of flow into any distribution network or district during the night (typically the minimum one hour demand period). For this method the procedure is as follows. First the smallest hourly flow from inflow data for each zone (MNF) (l/s) is found. Then, a figure for legitimate overnight unmetered consumption (LOUC) is calculated as

$$LOUC = (No.\ of\ unmetered\ properties) \times LOUCP \qquad (2.8)$$

in which LOUCP (legitimate overnight unmetered consumption per property) often taken as 1 l/prop/hr (recommended by TWGWW, 1980) and more recently 1.7 l/household/hr (or 0.6 l/resident/hr) and 7-8 l/non-household/hr (recommended by UK/WI, 1994-Reports E and G). Generally, the average domestic night flow rate is relatively small, on average less than 2 (l/prop/hr) (TWGWW 1980). It is expected that 95% confidence limits for the average night use vary from 1.2 to 2.2 l/prop/hr (UK/WI 1994-Report D). UK/WI (1994-Report E) did not state that household night use is a fixed value (i.e. 1.7 l/household/hr) in all situations. It gave a methodology from which average values and night to night variability can be calculated (depending on the proportion of 'active' properties / population and types of night use). Seasonal variations in activity at night can be expected to produce large individual values of household night use (Lambert 1997a). The concept of 'active' properties assists in understanding the variability of LOUCP (UK/WI 1994-Report D). Obviously not all the active households have the same average use. Also, test data clearly showed the influence which a small percentage of high-use active households (e.g. washing machines, dishwashers) can have on the average household night use (UK/WI 1994-Report D). For example the LOUCP can be approximately assumed to arise from only 17% of 'active' households (or 6% of residents) using 10 l/hr each and around only 33% of non-households which are active at 24 l/hr each (UK/WI 1994-Report F).

The range of average household night use would vary depending upon size of district, the more properties the less deviation from the mean. Also, the influences on the average non-household night flow are the numbers of properties in different non-household categories (having different average night use) (UK/WI 1994-Report D). Meanwhile, where population varies seasonally (e.g. DMAs in holiday areas), or number of billed properties is not a good indicator of numbers of people (e.g. in city centres with large blocks of flats) it is preferable to use numbers of residents to estimate assessed household night use (UK/WI 1994-Report F).

Using the formulation below, metered commercial consumption (MCC) can be calculated.

$$MCC = \sum_{type\ 2,3,4} (\text{average metered consumption (l/s) for each category}) \times (\text{corresponding demand factor, at time of minimum flow}) \qquad (2.9)$$

Then, unaccounted for water at the minimum flow condition (UFWM) can be obtained as follows:

$$UFWM = MNF - LOUC - MCC \qquad (2.10)$$

UFWM represents a flow in (l/s) taken as leakage at night. Leakage is higher at night, therefore, this value is multiplied by a 'hour-day' factor to give an average flow in (l/s) for the whole day. Total daily leakage is

$$L\ (l/day) = UFWM \times (3600 \times 24) \times (20/24) \qquad (2.11)$$

Normally this factor (20/24) is less than one, as the leakage rate during the day is less than that at night. Figure 2.2 shows a leakage factor of 1.2 at midnight. Since in the MNF method the leakage is measured at the minimum night flow, the factor of 20/24 converts it to an overall daily factor. However, when pressure management is introduced, the factor can change significantly, in some cases to more than one (Lambert 1997b). Finally,

$$UFW\ (l/prop/hr) = L\ /\ [24 \times (No.\ of\ unmetered\ properties)] \quad (2.12)$$

Some source of uncertainty in UFW, which is found to vary according to the size and state of the system, might be because of the number of storage cisterns on customer's premises which fill at night and also the difficulty of estimating LOUCP (UK/WI 1994-Report D). Both methods of calculation of UFW are subject to errors, sometimes quite substantial, in the factors that are used. Errors in the measurement of metered quantities, population estimates, seasonal variations, lack of information on average domestic consumption have been noted as weaknesses of the TIF method by Phillip (1985). The use of national estimates of average domestic or domestic night consumptions can lead to totally misleading estimates of UFW. Therefore, more accurate and locally relevant figures have to be gathered for the factors used in the equations. For example, studies in the Wessex area have shown that legitimate night flow rates are commonly at least 20% higher than 2 l/prop/hr (Phillip 1985).

Both the TIF and MNF methods are acceptable as indicators of leakage. The MNF method is, however, the more widely practised and more accurate (TWGWW 1980; Twort et al. 1994 and UK/WI 1994-Report J). However, figures arising from this method are affected by the characteristics of an area. For instance, when domestic consumption is metered the UFW can represent a much smaller percentage of the total supply, because any consumption inside a property will be registered more accurately by meters and thus appear under 'consumption' and not under 'losses' (Twort et al. 1994).

Components of night flow can be seen in Figure 2.4. Also, Figure 2.5 shows how the components of MNF contribute to the series of minimum night flows which would be measured in an individual DMA over a period of year. When there are no bursts running in a DMA, it can be seen from Fig. 2.5 that the night flow consists only of customer night use and background losses. If the night flow in a district exceeds some threshold value, further investigation is undertaken to locate the source of extra losses, usually unreported bursts (UK/WI 1994-Report J).

2.7 PRESSURE DEPENDENT LEAKAGE

It is well known that there is a correlation between the levels of leakage and mains pressure in water supply distribution networks (TWGWW, 1980). The effect of pressure on the rate of leakage, which perhaps has the greatest and most immediate effect on the total leakage, is common to all systems. There are very few published references to the theory of the pressure-leakage relationship applied to water mains. Empirical analysis of limited tests have been done both in Japan and the UK, however, until recently there has been no unified theory which can be applied to data or predictions with reasonable confidence in any international situation (Lambert 1997b). Also, the concept of pressure dependency of leakage has never previously been used in the UK as an explanatory element in reconciliation of annual losses (Lambert 1994). This section will review the literature in respect of the concept of pressure dependency of leakage.

2.7.1 Pressure-Leakage Relationship

Theoretically it is known that the flow through an orifice of fixed dimensions is proportional to the square root of the pressure drop across it (see Figure 2.6). However, a series of experiments has shown that this relationship does not hold for the effect of pressure on leakage from water supply systems (see Lambert 1997b). The result of these experiments is shown in Figure 2.7. The vertical axis represents net night flow rather than leakage alone. Net night flow includes both pressure and volume dependent terms, i.e. leakage and LOUC, respectively. Therefore, the vertical scale is represented as an index (ratio) of the leakage. The average zone night pressure (AZNP) is the mean pressure occurring within the system at night taking account of variations in ground level and any hydraulic friction losses across the zone. The relationship between night pressure and index of leakage rates (as measured by night flow) has been well documented in TWGWW (1980), and confirmed by the UK/WI (1994-Report G). It can be seen that the curve is approximately linear up to 50 meters, however, it steepens at higher pressure so that, in complete contrast to the square root law (Fig. 2.6), even small reductions of high pressures can cause correspondingly large reductions in leakage. On the other hand, the leakage index has been approximated to be proportional to the average zone night pressure raised to the power 1.18 and 1.15 by Sterling and Bargiela (1984) and Miyaoka and Funabashi (1984), respectively. According to the UK/WI (1994-Report G) the pressure-leakage relationship of Figure 2.7 proposed by TWGWW (1980) can be formulated as

$$\textit{Leakage Index} = 0.5\ AZNP + 0.0042\ AZNP^2 \tag{2.13}$$

Based on the findings of recent investigations the following relationship was obtained between pressure and leakage.

$$\textit{Leakage Index} = 0.5\ AZNP + 0.007\ AZNP^2 \tag{2.14}$$

However, due to the limited scale of this study it was concluded (UK/WI, 1994-Report G) that there is no reason to reject the curve shown in Figure 2.7. Also, Figure 2.8 shows how, in a distribution system without pressure management, the leakage rate changes as the average zone pressure changes with customer use over the day.

Whilst the apparent discrepancy between the two (square root and quadratic) relationships is not completely understood, it is probably caused by the cracks becoming larger (or pipe joints which open up if badly corroded) at higher pressures. In addition, whilst the square root law only applies to a single leak, within a distribution system there may be many sources of leakage each experiencing a different pressure. Recently the concept of fixed and variable discharge paths (FAVAD) has been proposed by May (1994) to explain this situation more realistically. This subject is explained in section 2.7.3 in more detail. It is expected, therefore, that hours with high pressure, like midnight would have high values for leakage and hours when pressure is low (like morning and evening) would inversely have lower leakage values.

UK/WI (1994-Report G) indicated that in the pressure-leakage relationship of Figure 2.7, leakage measurement has been defined as net night flow, obtained by subtracting measured and assessed trade and industrial uses from minimum night flow and no allowance has been made for night time household consumption. The report then suggests that night domestic consumption should be abstracted from measured minimum night flow. The effect of this allowance is proportional to the number of domestic properties in the district and, furthermore, it is volume related and so independent of pressure. This will change the algebra of the leakage-pressure relationship of Figure 2.7.

According to Lambert (1997b) the limitations of the leakage index curve approach (Fig. 2.7), can be summarised as follows:

- It is assumed that changes in night flow are a function of pressure only and are not influenced by the amount, or individual components, of night flow.
- A single curve is used to present a variety of responses to pressure, to estimate changes in night flow when pressure is changed.
- The leakage index is based on net night flow. Although this excludes exceptional night use, it includes other night use by customers. It is, therefore, a 'net night flow' index rather than a 'leakage index'.
- It does not provide an understanding of the underlying mechanism through which leakage is influenced by pressure.
- Its application is limited to the UK, as values of customer night use in other countries can be significantly different.

2.7.2 Average Zone Night Pressure (AZNP)

Pressure in each zone can be established in different ways as follows.

1. The mean of the highest and lowest pressure gauge values weighted if necessary to take into account topography and the disposition of domestic properties which are measured at some sensitive and critical nodes in the district. To estimate the weighted average night zone pressure, three methods have been introduced by UK/WI (1994-Report F): surrogate measuring point; weighted contour method (which uses the weighted average ground level for properties); and individual property (uses GIS to obtain the weighted average ground level for individual properties).

2. Producing the pressure values for each node by a calibrated network model with measured pressure values in critical nodes. The mean of these values can be calculated as average district pressure.

In case of large differences between the calculated static pressure (by a hydraulic model) and the night pressure (measured in the field), leakage may well be the cause of such problem (Evins et al. 1989). A logical pattern for pressure variation is expected to have the highest value at night time, the lowest during the morning or evening and moderate values before and after noon. The pressure profile may not show the expected diurnal variations, because of errors in the data, errors in readings or unknown flows passing boundary valves, or where trunk mains have a prime purpose of delivering water to downstream storage reservoirs, the pressure profile may not show the expected diurnal variations.

2.7.3 The Fixed And Variable Discharge Paths, (FAVAD) Concept

Use of night flow analysis to assess losses from distribution systems over 24 hour periods requires the introduction of diurnal distribution factors, which take account of the effect of diurnal variations in pressure on leakage rates. Using the MNF method, some tests have been carried out in the UK and Japan and the results have been analyzed by Lambert (1997b) by applying a Power Law equation as follows:

$$MNF_1 / MNF_0 = (AZNP_1 / AZNP_0)^N \tag{2.15}$$

where subscripts 0 and 1 are related to different pressure conditions. A value of 0.5 is expected for 'N' if all leakage paths had fixed areas i.e. act as an orifice. However, depending upon inclusion or exclusion of the customer night use from the analysis, Table 2.4 shows a range from under 0.5 to over 2.0, with mean values between 0.62 and 1.15 for N. Therefore, it can be seen that leakage rates from distribution systems are generally more responsive to changes in pressure than the 'square-root' relation that fixed paths would suggest (Lambert 1997b).

The FAVAD concept considers the ratio of (R_{FVL}), at any particular initial pressure,

$AZNP_0$, as a key characteristic for predicting the effects of pressure on leakage rates. This is defined as

$$R_{FVL} = \textit{Fixed area leakage / Variable area leakage} \qquad (at\ AZNP_0) \qquad (2.16)$$

in which leakage estimates arise from a number of paths with fixed or variable cross section areas. Two different paths for leakage are assumed in the FAVAD concept. In contrast to the fixed area paths which act as an orifice, the variable area paths are considered to change with pressure variations.

If this ratio can be estimated, assessed or measured, predictions of the effects of increases or decreases in pressure could be significantly improved and simplified. For example, for a certain value of R_{FVL} and ranges of pressure variation, the assumption of a linear relationship between pressure and leakage rate could be justified. This could considerably simplify calculation of diurnal factors.

May (1994) provided a rational physically-based theory of why the 'N' values from tests on distribution systems could vary. His approach assumes that the area through which discharges from a distribution system occur can vary with pressure as shown in Table 2.5. For most purposes it can be assumed that variable area paths have an 'N' value of 1.5.

A relationship indicating fixed area discharge paths at night can actually arise from either leakage or customer use. By assessing the characteristic ratio R_{FVL}, with change of initial pressure, $AZNP_0$, the following relationship is obtained.

$$L_1 = L_0 \ x \ (AZNP_1 / AZNP_0)^N \qquad (2.17)$$

in which L_1 and L_0 are the leakage rates at $AZNP_1$ and $AZNP_0$, respectively. Having the values of R_{FVL} at the $AZNP_0$ from Eq. 2.15 and the ratio of $AZNP_1/AZNP_0$, the values of N can be obtained from Figure 2.9. It also shows that if R_{FVL} at $AZNP_0$ is close to 1.0, then 'N' is close to 1.0 for a large range of values of $AZNP_1/AZNP_0$ and also increases in pressure give higher 'N' values. The ratio of $AZNP_1/AZNP_0$ can be used to predict the ratio of leakage rates L_1 / L_0 for different values of R_{FVL}. It is found that the shape of the original Figure 2.7, leakage index curve, was associated with districts where the R_{FVL} ratio was low, implying that most leakage paths were expanding paths.

Lambert (1997b) asserted that for 17 data sets from the UK the comparison of predicted and calculated 'N' values showed that in each case the average is close to 1.0. It indicates that even with this very approximate assumption, the FAVAD methodology is more reliable than the simple assumptions that 'N' could be anywhere in range of 0.5-1.5, a standard 'N' value of 1.15 (as in Japan) or 0.5 (as in USA).

Finally, according to the conclusion of Lambert (1997b), the FAVAD concept is a most important advance in understanding pressure-leakage relationships, as it is based on logical physical concepts (i.e. existence of fixed and variable leakage paths which vary with pressure) rather than an empirical approach. It provides a broad explanation of why power law index values for distribution systems should vary so widely in practice. The main weaknesses of this procedure is the difficulty of assessing the values of R_{FVL} for any distribution system at $AZNP_0$ at night. No practical approach has been suggested to identify the fixed and variable area paths for thousands of metres of distribution pipe lines laid under ground.

Figure 2.9 suggests that, if the range of $AZNP_1/AZNP_0$ and the initial value of R_{FVL} are such that 'N' is close to 1.0, the assumption of a linear relationship between pressure and leakage should be valid. Pressure influences leakage and overflows from all customers' pipework subject to mains pressure. It would also be expected that the proportion of customer use which is 'open-tap' rather than 'fixed-volume' would be influenced by pressure. This would suggest that, in the UK at least, there is some element of per capita consumption, and of water delivered, which is related to pressure (Lambert 1997b).

2.7.4 Concept of Bursts and Background losses Estimate (BABE)

As was seen earlier (Fig. 2.5) distribution losses can be considered to comprise two terms, background losses and bursts. There is no precise methodology to estimate the components of distribution losses. In current practice only the arithmetical difference between distribution input (measured) and water delivered provides representation of distribution losses. However, even in fully metered situations, because the calculations are susceptible to errors, analyses show up to 50% uncertainty in the calculated annual losses (Weimer 1992). In this section the concept of bursts and background losses estimate (BABE) presented by Lambert (1994) will be introduced.

2.7.4.1 Estimates of background losses

Almost all losses from fittings on mains and services (air valves, hydrants, stop taps, dripping taps, cisterns, etc.) fall within the background category. A four part formula for net night flow (NFN) can be considered in the following form.

$$NFN\ (l/prop/hr) = [(C_1 \times L_m/prop + C_2) \times PCF \times SCF] + LOUCP \times SCF \qquad (2.18)$$

in which NFN is the minimum night flow deducted by exceptional night use (i.e. night uses greater than 500 l/hr) and the parameters inside the bracket, [ ], represent the background losses in distribution mains and services. Table 2.6 introduces the applied parameters and Table 2.7 presents the pressure correction factors (PCF). Values of PCF are applied to adjust the estimates of background losses at different pressures. In integration of the FAVAD methodology with the BABE concept the values of N for background losses is assumed to be 1.5 (Lambert 1997b).

2.7.4.2 Estimates of annual burst losses, reported and unreported

A burst is designated as such when the loss rate from an individual source of water exceeds a threshold value of 500 l/hr at 50 m pressure (or 10 l/hr per m of pressure). Smaller flow rates (including lesser leaks which may eventually exceed the threshold value and become classified as bursts) are allowed for as part of the 'background' losses. Burst losses can be estimated as follows:

Burst losses = burst flow rate x average duration x frequency *(2.19)*

The flow rate will be influenced by the size of the hole/crack and the pressure where the crack size may be changed by pressure, as well. According to the UK/WI (1994-Report E) at 40 m AZNP the average flow rates are 25 m^3/day for an underground service pipe burst, 75 m^3/day for a typical distribution mains burst and 150 m^3/day for a typical trunk mains burst. Elements affecting the average duration of bursts are awareness, location and repair (Lambert 1994). A feature of the reported burst is that it can be identified and repaired quickly. The burst rates of flow at the time of repair are generally assessed by analysis of continuously logged minimum night flows, before and after repair. A figure of 0.45 l/s at 50 m pressure is recommended for estimation of annual losses from bursts on under ground service pipes (UK/WI 1994-Report E). It can be concluded that in respect of the customer supply pipe bursts, and the local influence of burst frequency and pressure, annual supply pipe losses vary in a range of 15-80 l/prop/day (UK/WI 1994-Report E). An example of frequencies of burst in a sample area in 1992 can be seen in Table 2.8. For continuous leakage monitoring, the BABE spreadsheet approach allows for the influence of both pressure and burst frequencies (which may also be related to pressure) to be objectively considered.

2.7.5 Pressure Management

Nowadays pressure management is applied as a convenient tool for managing leakage in view of the dependencies described in the previous sections. According to the UK/WI (1994-Report G), pressure management can

- Reduce leakage, save water resource and associated costs
- Reduce pressure related consumption, such as garden watering, again yielding a saving in resources.
- Reduce frequency of bursts and consequential damage which is costly to repair.
- Provide a more constant service to customers. Large pressure variations may give customers an impression of a poorly managed service. Also, unnecessarily high pressures raise customers expectations and perceptions of what is adequate.
- Lower pressures may enable a company to standardise on pipes and fittings which

have a lower pressure rating and are therefore cheaper.

The effects of pressure management on reducing leakage can be seen in Figure 2.10 (in comparison with Fig. 2.8 i.e. before pressure management).

2.8 UNCERTAINTY IN MEASUREMENT OF DEMAND COMPONENTS

The estimation of water demand is fundamental to effective water resource management. Water supply is measured at district level but true demand is not, and therefore, mass-balance based assessment of leakage, illegal use, meter inaccuracies, etc. are compromised (Clarke et al. 1997).

Lambert (1997a) pointed out that the identification of significant balancing errors raised by evaluation of the annual water balance can be likened to the identification of significant anomalies between predicted and recorded pressures and flows in an annual model such as the BABE. The causes of significant errors, which may be due to incorrect parameter values, data efficiency or real problems, must be tracked down and resolved until achieving an acceptable degree of accuracy.

According to UK/WI (1994-Report D), in the UK where few households are metered, if the assessed components of 'water delivered' and 'water not delivered' add up to within 2 to 3% of the distribution input, the reconciliation should be considered as reasonable, as the remaining errors could be in any of the elements (including metered distribution input). However, if the sum of the elements (including calculations of losses based on recorded burst frequencies) is significantly less than the distribution input, according to the same reference, it is possible that not all the unreported bursts and overflows are being detected, and a backlog of long-running bursts or service reservoir overflows or the need for further research into key parameters (e.g. diurnal distribution factors) may exist.

Table 2.9 shows the error range for components of the balance equation. Also, from comparison between bounds of error reported from different water companies data sets, the following range can be obtained from (UK/WI 1994-Report B): 1.65-5.68% error in distribution input (DI); 1.9-5.74% in measured water delivered (WD); 4.42 - 9.84% for unmeasured WD; and finally 5.9-13.2% error in distribution losses (DL).

Passing flow to adjacent zones is another significant source of anomalies. UK/WI (1994-Report G) has introduced a zero pressure test to confirm the security of the boundaries of DMAs. This is performed by closing inputs to a system at night and observing the gauge pressure at a hydrant on high ground declining towards zero when the system inputs are closed. If the pressure is sustained then another unknown input exists which is the flow passing from the adjacent higher pressure DMAs.

In a survey for evaluation of district metering in the Sunderland and South Shields water company, Pepper (1985) in a DMA-based evaluation of water balance concluded that for different elements of demand and leakage, unbalanced results are likely either to indicate a faulty meter or meter reading, or to suggest the existence of an open boundary valve or hitherto unknown cross-connection. The discrepancy could also be due to high use by unmetered commercial premises or legitimate domestic consumption much higher than that assumed. Districts which apparently have negative UFW should be checked as there must be some anomaly or error. Some UFW can be eliminated simply by confirming that some of the assumptions made are false, meters are inaccurate, valves are not properly closed, or consumption by the consumer is not properly quantified. It is worth noting that during real surveys, at best only the largest industrial users will be 'meter-read' in real time and all other metered consumptions will be approximated from their normal (at best, quarterly) routine readings. Therefore, the appearance of imbalance in the flow balance equation such as negative values for leakage may arise from such imprecise metered data.

Finally, it can be concluded that a true data management scheme should consider the range of uncertainties in different components of demand to identify the anomalies in data sets or to reconcile them. This is incorporated in the methodologies developed in Chapter 3.

2.9 PROBABILISTIC NATURE OF DEMAND

In the previous sections it was shown that all components of water consumption are subject to random variation. However, design procedures and models which evaluate the behaviour of water supply systems normally assume a set of deterministic demands through the 24 hours of a day or as daily average or peak values. These demand values are then considered to occur synchronously and result in a set of deterministic pressure values. However, in reality all of these parameters, demands and pressures are probabilistic. Only a few researchers have attempted to take into consideration the concept of the probabilistic nature of demand. Walters and Cembrowicz (1993); Li et al. (1993) and Quimpo (1996) have acknowledged the lack of techniques and models to consider this concept. Among these few pieces of research, Tung et al. (1987) and Goulter and Bouchart (1987) included this concept in their optimization models. Assuming that the required nodal heads and demands are random variables, in both approaches the probability of nodal demands not exceeding the design levels was incorporated as a constraint. Besides Bao and Mays (1990) who used a range of randomly generated loading condition by the Monte Carlo technique, Bouchart and Goulter (1991) and Khomsi et al. (1996) have applied a probability distribution function to illustrate the variability of demand. The latter used a series of historical demand data and represented it by a normal distribution function. However, the former just used an arbitrary distribution function. These approaches were incorporated into reliability evaluation procedures for water distribution networks. Yet more research is called for and uncertainty in demand is incorporated in the methodologies presented in Chapters 3 and 8.

2.10 CONCLUSION

In this Chapter the concept of water demand and its categories have been reviewed. Existing methods and procedures for describing and categorising unmetered domestic demand, metered consumptions and leakage (losses) have been summarised and evaluated. The following conclusions can be drawn:

i) Minimum night flow is a convenient method to evaluate the unmeasured parts of demand.

ii) Metering of different components of demand can result in more accurate figures for domestic consumption and leakage in bulk water balance evaluation (e.g. annually).

iii) The concept of pressure dependency of demand and leakage, should be conveniently considered in management and modelling of water distribution systems.

iv) There are a set of uncertainties in measurement and calculation of different categories of demand (especially domestic consumption and leakage). These uncertainties can lead to large values of error in balance equation.

v) The probabilistic nature of demand should be given consideration.

vi) In the UK the available data sources assembled for network modelling studies are potentially error prone, especially in the unmetered figures of PCC and leakage. Therefore, methodologies are needed to identify the possible anomalies and improve reconciliation of such data.

Table 2.1: 'WRc' diurnal demand factors for categories of metered consumers and leakage ('WRc' 1992).

Time (hr)	Type 2 Domestic Equivalent	Type 3 10 Hour Working	Type 4 24 Hour Working	Type 5 Leakage
00-01	0.29	0.00	0.85	1.2
01-02	0.20	0.00	0.85	1.2
02-03	0.17	0.00	0.85	1.2
03-04	0.15	0.00	0.85	1.2
04-05	0.17	0.00	0.85	1.2
05-06	0.34	0.00	0.85	1.2
06-07	1.09	0.00	0.85	0.9
07-08	2.03	0.53	0.90	0.9
08-09	2.27	1.75	1.20	0.9
09-10	1.87	2.43	1.20	0.9
10-11	1.39	2.43	1.20	0.9
11-12	1.31	2.43	1.20	0.9
12-13	1.45	2.43	1.20	0.9
13-14	1.22	2.43	1.20	0.9
14-15	0.88	2.43	1.20	0.9
15-16	0.88	2.43	1.20	0.9
16-17	1.07	2.43	1.20	0.9
17-18	1.26	1.75	1.20	0.9
18-19	1.35	0.53	0.90	0.9
19-20	1.32	0.00	0.85	0.9
20-21	1.19	0.00	0.85	0.9
21-22	0.95	0.00	0.85	0.9
22-23	0.69	0.00	0.85	1.2
23-24	0.46	0.00	0.85	1.2

Table 2.2: Water consumption for the UK in 1990 (Twort et al. 1994).

Consumption	England & Wales (l/hd/day)	Scotland (l/hd/day)	N. Ireland (l/hd/day)
Metered (trade and industry)	84	123	89
Unmetered (domestic and small trade)	164	212	230
Losses (25% of total)	82	112	106
Total	330	447	425

Table 2.3: A comparison of the demand and leakage in three European countries in 1994/1995 (FT Newsletter, 1997).

Country	PCC	Leakage (including customer supply pipes)			mains per property	meter penetration
	l/hd/day	%	m^3/km of main/day	l/prop/day	km/prop	%
England & Wales	144	29	8.4	243	29	8
France	152	25	4.2	222	53	99
Germany	132	13-19	4.6	146	32	100
West Germany	139	n/a	3.7	112	32	100

Table 2.4: Power Law Index values for the UK and Japanese distribution systems (Lambert 1997b).

Country	No. of Tests	Mean value of 'N'	Median value	Range of 'N'	Base of Analysis
UK (1980)	17	1.13	1.00	0.70 - 1.68	Net Night Flow
Japan (1979)	20	1.15		0.63 - 2.12	
UK (1994/1997)	17	0.62	0.62	0.27 - 1.21	Net Night Flow
	17	0.95	0.87	0.30 - 1.98	Net Night Flow - customer night use

Table 2.5: Power law index values based on FAVAD concept (Lambert 1997b).

Type of Discharge Path	Area A_i	Velocity V_i	Discharge Q_i	'N' value
Fixed area (does not vary with pressure)	A_0	$V_0(AZNP_1/AZNP_0)^{0.5}$	$A_0V_0(AZNP_1/AZNP_0)^{0.5}$	0.5
Variable area (area proportional to pressure)	$A_0(AZNP_1/AZNP_0)$	$V_0(AZNP_1/AZNP_0)^{0.5}$	$A_0V_0(AZNP_1/AZNP_0)^{1.5}$	1.5
Variable area (area proportional to pressure squared)	$A_0(AZNP_1/AZNP_0)^2$	$V_0(AZNP_1/AZNP_0)^{0.5}$	$A_0V_0(AZNP_1/AZNP_0)^{2.5}$	2.5

Table 2.6: The applied parameters of BABE concept for Welsh Water (Lambert 1994).

Term	Explanation	Assumed value
L_m/prop	Average mains length per property (m)	From local data
C_1	Background loss on distribution mains (l/km of mains/hr) at 40 m average zone night pressure (AZNP) and 60 min sampling period (S)	0.035
C_2	Background loss on service pipes (l/prop/hr) at 40 m AZNP and 60 min sampling period (SP)	2.5
PCF	Pressure correction factor related to Leakage Index at 40 m AZNP $= LI_{(AZNP)}/LI_{(40)}$ where $LI = 0.5 \times AZNP + 0.0042 \times AZNP^2$	From local data (Equation derived from Fig. 2.7 graph)
SCF	Sampling period correction factor of the form SCF = 1/[A+B/(C+SP)]	A=0.95 B=6.00 C=60.0 SP=60.0
LOUCP	Assessed customer night flow use (l/prop/hr) for SP=60 min	1.5

Table 2.7: Values of PCF for different AZNP (UK/WI 1994-Report F).

AZNP (m)	20	30	40	50	60	70	80	90	100
PCF	0.33	0.53	0.75	1.0	1.27	1.57	1.88	2.23	2.59

Table 2.8: Frequencies of burst in a sample area in 1992 (Lambert 1994).

Infrastructure component	Reported bursts	Unreported bursts	Total bursts	Units
Trunk mains	0.12	0	0.12	per km/yr
Distribution mains	0.26	0.14	0.40	per km/yr
	5.56	3.09	8.65	per 1000 props/yr
Communication pipes	4.44	1.56	6.00	per 1000 props/yr
Supply pipes	4.46	0.92	5.38	per 1000 props/yr

Table 2.9: The error range for components of balance equation (UK/WI 1994-Report B).

Component of Balance	Assessed Standard Error of Estimates (%)
Distribution Input (DI)	± 1.5
Water Delivered (WD) :	
Billed measured	± 1.0
Meter error	± 0.5
Billed Unmeasured	± 5.0
Underground Supply Pipe Losses	± 10.0
Taken Legally (Unbilled)	± 10.0
Taken Illegally (Unbilled)	± 50.0
Distribution Operational Use (DOU)	± 10.0
Distribution Losses (DL)	± 18.6

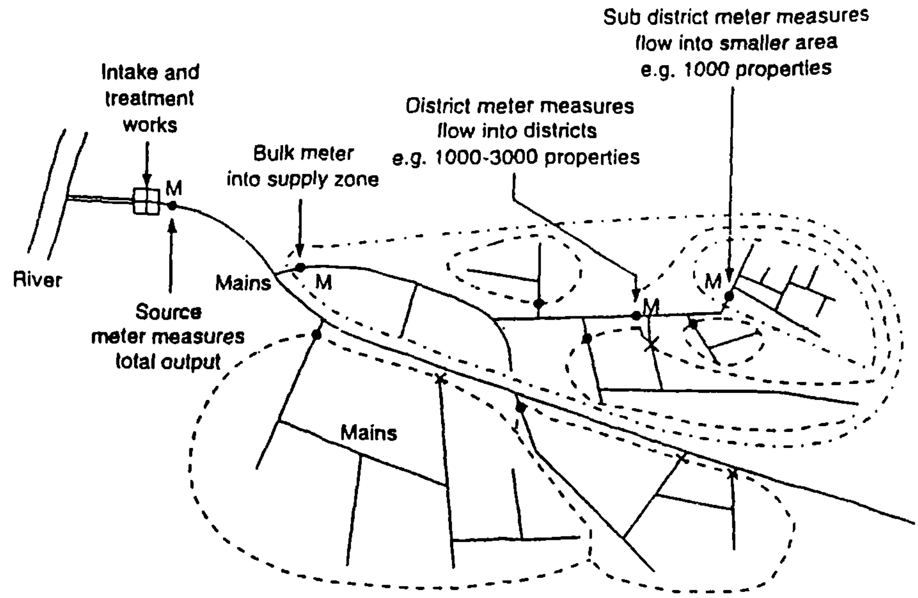

Figure 2.1: Schematic representation of a DMA (UK/WI 1994-Report J).

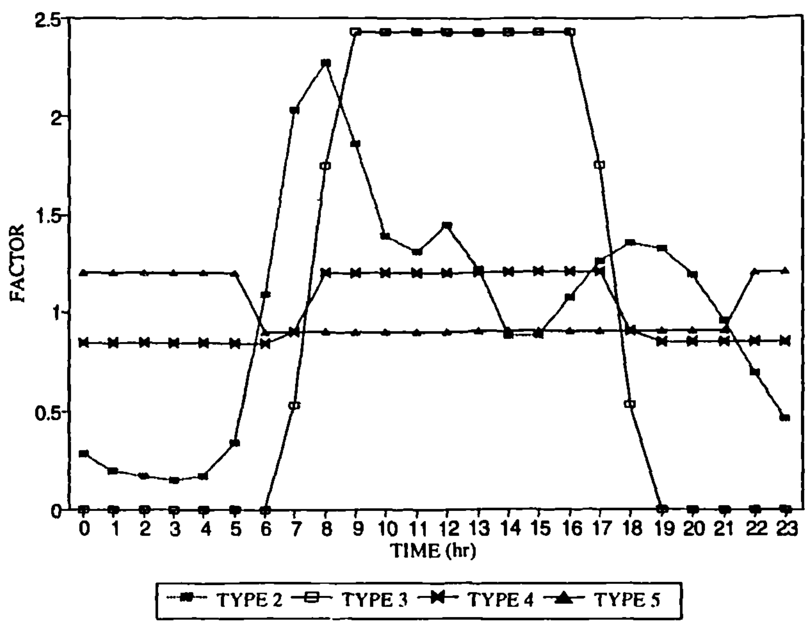

Figure 2.2: 'WRc' diurnal demand factors for categories of metered consumers and leakage.

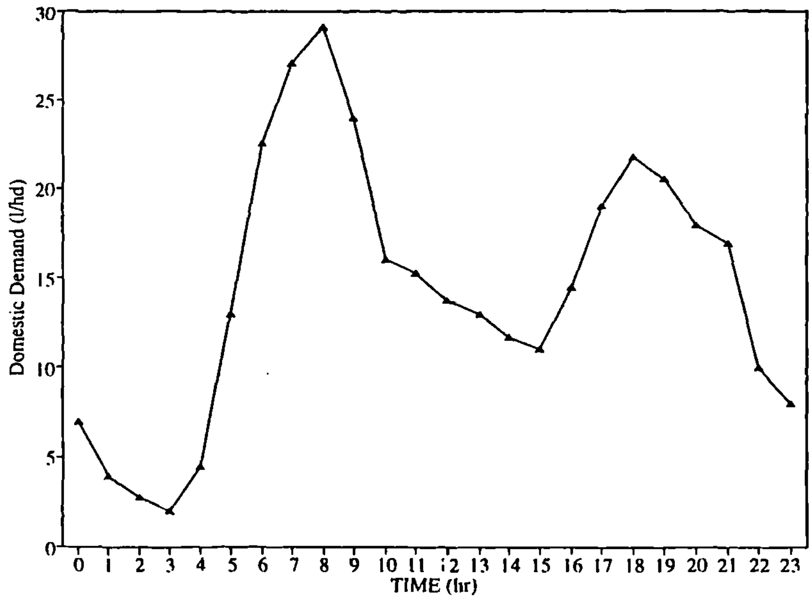

Figure 2.3: A typical diurnal pattern for domestic demand.

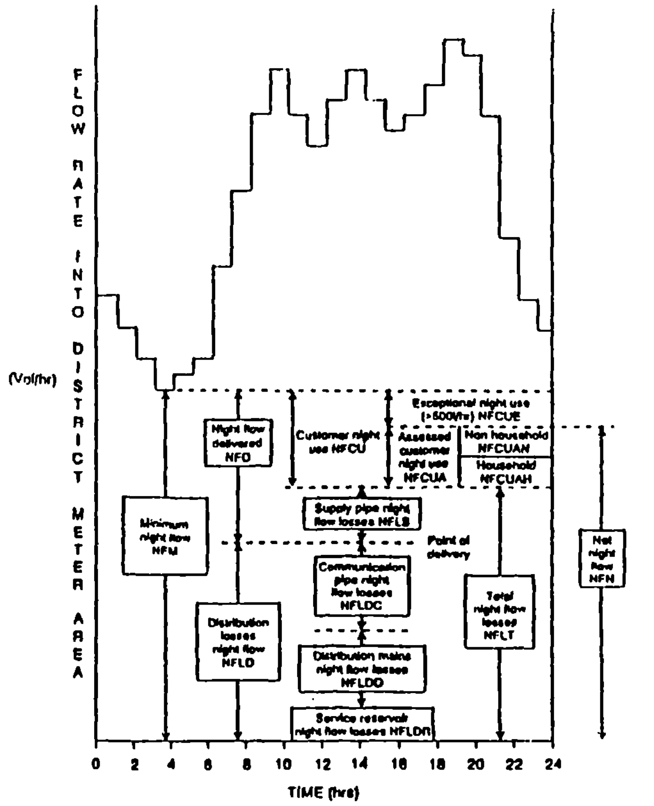

Figure 2.4: Components of night flow (UK/WI 1994-Report B).

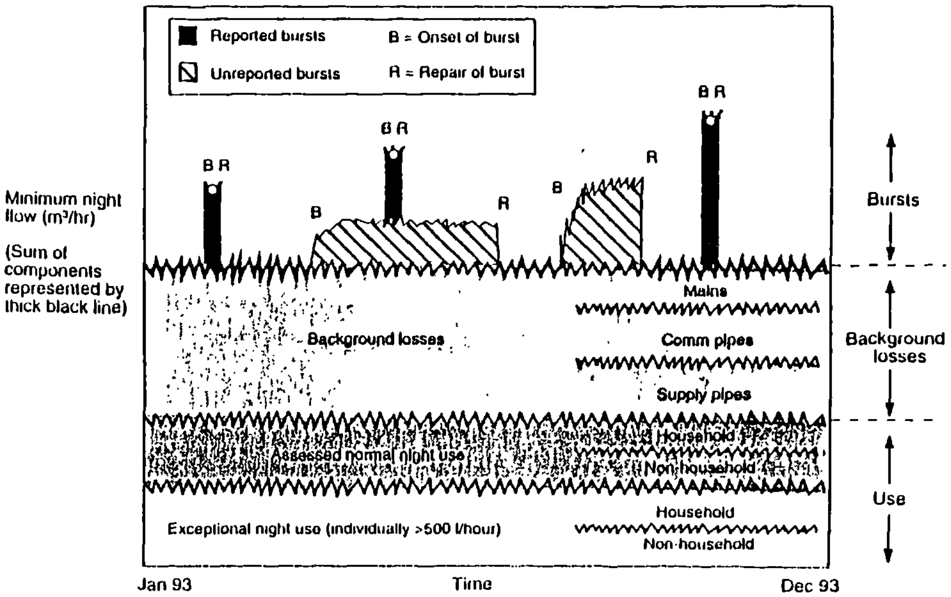

Figure 2.5: Components of minimum night flow (UK/WI 1994-Report F).

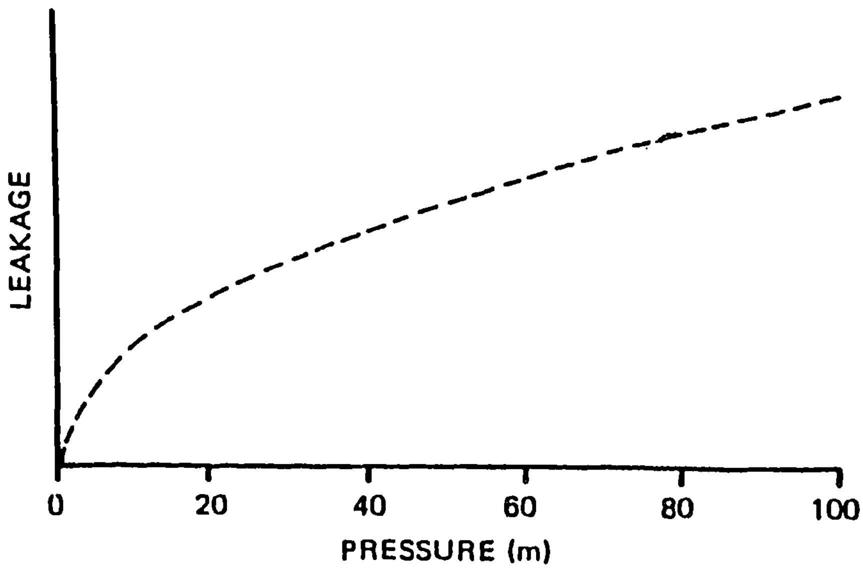

Figure 2.6: The square root relationship between leakage and pressure.

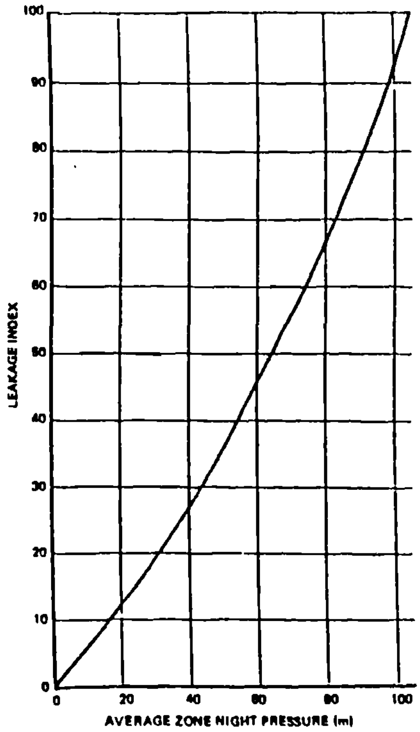

Figure 2.7: Relationship between leakage index (net night flow) and pressure (TWGWW 1980).

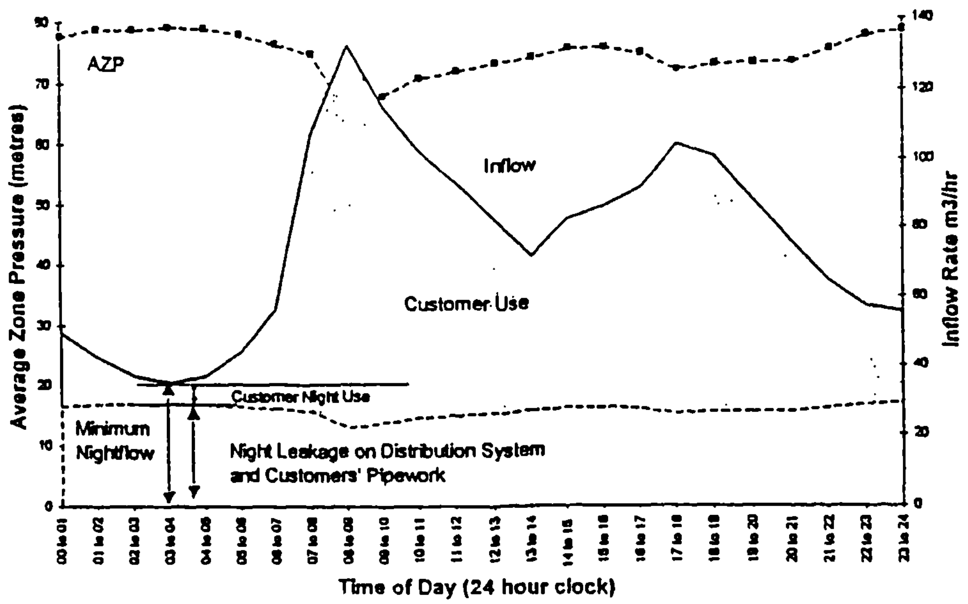

Figure 2.8: Basic components of distribution system inflow before pressure management (Lambert 1997b).

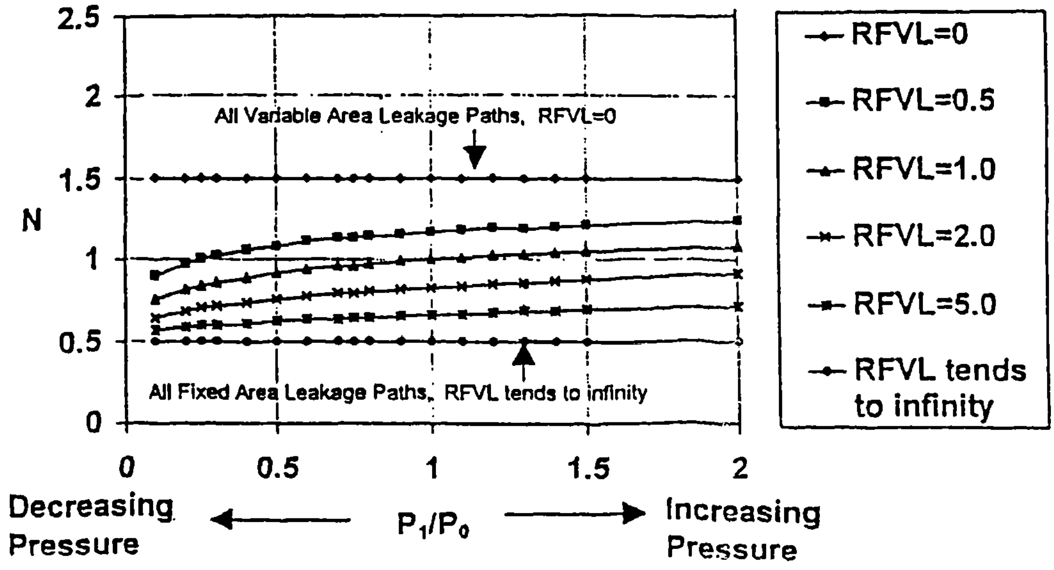

Figure 2.9: Power law index 'N' as a function of R_{FVL} and P_1 / P_0 (Lambert 1997b) in which P_0 = $AZNP_0$ and P_1 = $AZNP_1$.

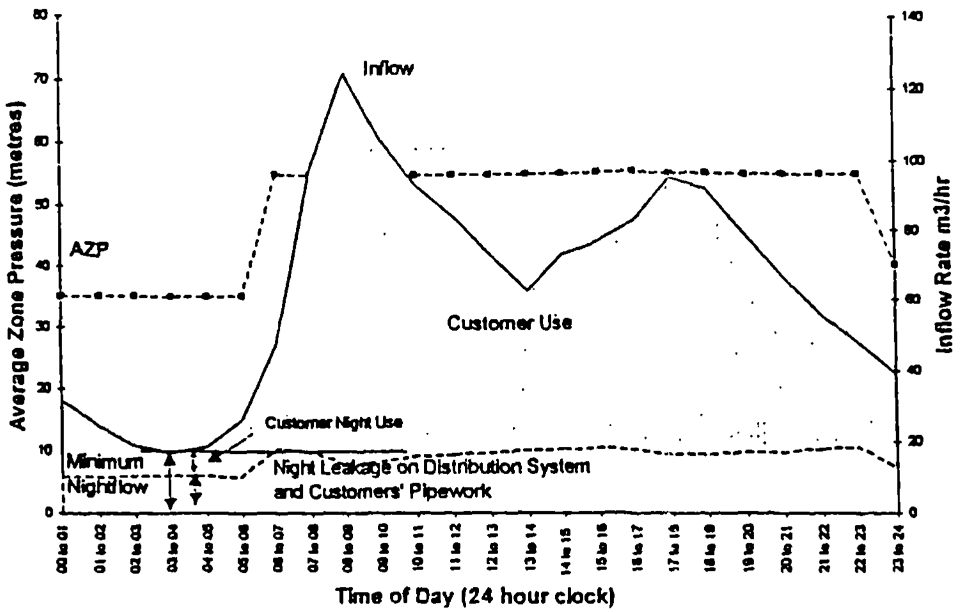

Figure 2.10: Basic components of distribution system inflow after pressure management (Lambert 1997b).

CHAPTER 3

WATER CONSUMPTION AND NETWORK LEAKAGE EVALUATION

3.1 INTRODUCTION

Computer network models of the hydraulics of pipe systems are widely used in the operational management and strategic planning of water supply networks. In the UK the data sources which are available as input to such models are potentially error prone, especially the unmetered figures for Per Capita Consumption (PCC) and leakage.

Complete details of all individual demands which really exist in a supply network cannot actually be modelled with high precision, either through limitations in the means of specification built into the modelling system or, most significantly by the absence of a practicable means of monitoring the consumption patterns of thousands of consumers. In the usual modelling approach, some typical demand types can be chosen and existing demands are forced to follow the most reasonable of these consumption categories ('types'). A 'WRc' classification used by the 'WATNET' model was shown in Chapter 2 (Table 2.1 and Figure 2.2) and is a widely used representation of demand categorisation.

Using District Metering Methods, vast amounts of data have been collected in recent years, relating to demand and consumption down to the level of the local supply zones ('Leakage District', 'Waste District' or 'District Metered Area (DMA)', see Section 2.1), for use in calibration and verification of network models. Models now have been completed for most systems in the UK, in response to the Asset Management Planning (AMP2) requirements of the Office of Water (OFWAT) in 1994. For these zones, field work studies have monitored daily inflows and databases provide population (property counts) and average consumption rates for all commercial water users. As mentioned in Chapter 2, no widespread metering of domestic supplies is made currently. Analysis of data for each zone enables the establishment of leakage levels (usually from recorded minimum flows at night). Domestic Per Capita Consumption (PCC) figures follow from some assumption regarding the diurnal variation in leakage over a 24 hour period (see Section 2.6).

Upon completion of such studies considerable variation in the crucial elements of consumption, namely PCC and leakage arise as will be shown by the case study which follows. Furthermore, significant variations in the diurnal pattern of PCC result. Whilst the cause of the variations will be partially explainable in terms of differentials in the socio-economic classifications of the housing between different supply zones, other contributory factors will be inaccuracies in flow measurement (especially lower night flows where certain measuring devices experience accuracy problems), imprecision in metered consumption figures and misrepresentation of the actual hydraulic configuration of the zone (i.e. unknown flows passing boundary valves between districts, unreported bursts, etc.).

Unfortunately, at present these potentially error prone figures for zonal PCC and leakage are the only data source available for future planning studies, based on the computer network models. These involve imposition of assumed rates of annual increase of PCC and reduction in leakage. As a consequence, considerable bias is likely to arise in total flows which may give unrealistic predictions of the future hydraulic performance of the systems, both in the individual supply zones and in the strategic trunk main system supplying them, when differential incrementation of individual components of demand (including leakage) is necessitated.

To address the shortcomings of existing methodologies, this chapter aims to present alternative procedures. These procedures maximise the quality control on the elemental components of demand (i.e. PCC and leakage and their diurnal profiles) arising from data reconciliation. These methods focus on: i) the variability of the PCC and its diurnal profile across the different supply zones in a water distribution network and ii) improved means of estimation of the diurnal patterns of leakage variation and its correlation against the zone pressures. To reconcile the data set and to identify zones producing apparently anomalous figures, an optimization approach is developed later. This produces best estimation of parameters based on an error minimization procedure, whilst applying realistic variability constraints and accuracy tolerances to the relevant data inputs.

3.2 CASE STUDY

The real data set which is used in this investigation has been taken from a supply system serving a medium sized town in an industrialized conurbation in the UK. For reasons of confidentiality it cannot be identified and is hereafter referred to as 'Onetown'. It includes 31 zones with each zone designated by code reference, e.g. NL011. Table 3.1 represents the relevant information about this area and the schematic representation which shows the layout and configuration of the system can be seen in Appendix B. Figure 3.1 indicates the variability of PCC and leakage arising from implementation of the current standard methodology i.e. the minimum night flow method (MNF), see Section 2.6.2, and the standard 'WRc' profiles (Fig. 2.2). As can be seen, the results of the existing methodology are likely to include anomalous data in some zones. It is obvious that, for example, the zones in which their PCC values are too low (or high) and also, some with high percentage of leakage are questionable and seem not to be acceptable. Because data such as this are essential as input for computer modelling of the hydraulic performance of the water distribution networks, it will be seen in the following sections that such anomalous data would lead to incorrect and illogical results for leakage and PCC values and profiles. It is therefore necessary to develop procedures to systematically identify and reconcile them.

3.3 METHODOLOGY

To find the most reliable values for domestic demand and leakage as well as their diurnal profiles, the minimum night flow (MNF) method is applied here. Then, using a spreadsheet approach, a set of coherent investigations will be performed on the data sets as follows.

3.3.1 Determination of Domestic Demand and its Diurnal Profile

The balance equation in each DMA is

$$Inflow = \sum Metered\ Consumptions + \sum Unmetered\ Consumptions \qquad (3.1)$$

in which, based on 'WRc' (1992), the components of metered consumption are Type 2 (small trade with domestic pattern) and Types 3 and 4 (10 and 24 hour industrial activities), respectively. Unmetered consumptions include Type 1 (domestic and equivalent small trade activities) and Type 5 (leakage). As can be seen, the above equation therefore includes the two unknown values, domestic consumption (or PCC) and leakage. In current practice based on the MNF method, by estimation of night time leakage the daily leakage value can be approximated and then distributed through the 24 hours of a day by using appropriate diurnal demand factors (e.g. 'WRc' factors). Finally, values of domestic demand can be obtained, as below using spreadsheet approach.

For each spreadsheet (see Table 3.2 as an example) which contains measured data as hourly net inflow (Col. 2) to the zone, the metered consumptions and leakage (Cols. 4-7) are calculated by multiplying base demands, demand weights and the 'WRc' demand factors for Types 2-5, respectively as previously described in Section 2.2. The base demand for Types 2-4 are shown in Table 3.1. They are approximated from their normal (e.g. quarterly) routine reading and only very large industrial users are likely to be metered in real time. The leakage base demand is obtained by the MNF method, Eq. (2.11). Demand weights are taken as 0.01157 and 0.000278 for Types 2-4 and Type 5 to convert the units of m^3/day and l/prop/hr to l/sec and l/prop/sec, respectively (see Section 2.2.2). Finally, demand factors for Types 2-5 are the standard profiles which have been shown in Table 2.1. Then the domestic demand and its distribution factors can be determined as

Domestic demand for each hour (l/s) =

Inflow - [$\Sigma_{type\ 2,\ 3,\ 4}$ metered consumptions + leakage (Type 5)] *(3.2)*

Domestic demand factor = Actual hourly domestic demand / Daily average domestic demand *(3.3)*

Cols. 3 and 8 of Table 3.2 show values of domestic demand and hourly factors representing the diurnal profile.

Computations were initially completed on all zones and the resulting domestic demand profiles for individual zones can be seen in Appendix C1. It was observed that some zones produce anomalous characteristics for domestic profiles, which do not follow a reasonable diurnal pattern. Figure 3.2 shows one of these zones which illustrates an illogical behaviour for domestic demand such as high demand during night time and low demand during the daytime. These results illustrate that the current procedure to calculate unmetered variables is not always reliable and some modifications should be made to improve the situation. The data under investigation are from a study where anomalous flow balances were followed up by repeated fieldwork and system configuration checks and the presence of outstanding anomalies illustrates the potential difficulties faced by system modellers.

After identifying and eliminating those zones with questionable data which lead to biased domestic demand profiles, domestic demand profiles of 22 zones (which follow logical patterns) have been selected. The average domestic demand profile of these 22 zones has been calculated as representative of the whole area. Table 3.3 represents the domestic demand factors for each zone. The last column of Table 3.3 and Figure 3.3 show the average domestic demand factors for the whole area. Figure 3.3 compares this with the standard domestic equivalent profile proposed by 'WRc' for Type 2, see Fig. 2.2 and Table 2.1. It can be seen that in this area the average domestic demand closely follows the expected pattern. Furthermore, it is observed that the two profiles have the same shape with relatively small departures. A possible explanation for some of the difference in the Type 2 profiles is that the origin of the WRC Type 2 profile is not clear, yet it is well understood that diurnal profiles for individual households are more peaked and varied than their aggregation due to the flattening effect of the lack of time synchronisation and 'random' deviations between individual households. The limit of aggregation size (i.e. number of properties) before these flattening effects diminish is not known.

3.3.2 Producing the Modified Leakage Profile

To obtain a better representation for diurnal leakage factors, the analysis can be extended to reappraise the leakage calculation as follows:

1. Adopt the average domestic demand profile at each hour with the actual zonal PCC values as first computed. Therefore,

Modified domestic demand for each hour (l/s) =
Average domestic demand factors x Actual PCC (l/hd/day) x
[No. of unmetered properties] x OCC (hd/prop) / (60 x 60 x 24) *(3.4)*

in which the occupancy rate (OCC) was taken as 2.75 hd/prop, according to the current practice of the water company.

2. Subtract the sum of revised domestic demand and metered consumptions (Types 2-4) from inflow to each zone at each hour. This produces the modified leakage values at each hour. Therefore,

Modified leakage (l/s) = Inflow - [Modified domestic demand + $\sum$ Metered consumptions] *(3.5)*

3. A modified profile for leakage is obtained using leakage factors as

Modified leakage factors = Actual hourly leakage / Daily average leakage *(3.6)*

Using the domestic average profile of 22 zones (Col. 9 of Table 3.2) and Eq. (3.4) the modified domestic values have been obtained (Col. 10). Then, applying Eqs. (3.5) and (3.6), the modified leakage values and factors have been calculated (Cols. 11-12). The resulting modified leakage profiles (L-modified) for all zones are presented in Appendix C2.

In practice the results show that in some zones, the leakage values modified in this way often include some negative values. Reasons for this eventuality and a correction method are discussed in later sections (also see Section 2.8). This is clearly infeasible and these zone were therefore ignored and their status was thereby considered as being potentially anomalous before the average leakage profile for the whole area was calculated. This modified leakage profile can be applied as an alternative to the 'WRc' standard leakage profile. It remains, however, to prove the practicability and validity of these new results.

Table 3.4 represents the 19 zones for which leakage profiles have been accepted, based on their having a reasonable and logical diurnal profile, together with the average leakage profile for this area. Figure 3.4 shows comparison between the improved and standard profiles. It is seen that the modified profile is intuitively more realistic than the standard (default) profile. According to this figure (Fig. 3.4) the values of leakage are at the highest level from midnight to 5 a.m. From 5 to 8 a.m. they show a sharp decrease; between 8 -18 hrs. a sharp increase and then a slow decrease; from 18 to 22 hrs. the leakage is low and then it increases very sharply approaching midnight. As an example, Figure 3.5 shows negative values and an illogical pattern of leakage profile for zone NT031.

Upon detailed inspection some other zones, though not producing negative leakage values, seem not to follow a logical diurnal pattern. There are 5 zones (NL014, NL044, NH022, NH024 and NH027) in which the peak value of modified leakage occurs in the day time (before or after noon). If these zones are eliminated 14 zones remain. Among these 14 zones, 7 zones show maximum leakage between 21-23 hrs. which may also be considered questionable. If these 7 zones also are ignored, finally 7 zones with an expected leakage profile remain. Figure 3.6 shows the average leakage profiles for these subgroups of 19, 14 and 7 zones, respectively. It is concluded that the more selective the choice of zones (on the basis of perceived data accuracy) the more the resulting profile departs from the standard profile.

Generally, it can be concluded that the negative values of leakage computed at certain time steps may be caused by error in PCC and UFW estimates made for the zone. According to Eq. (3.1), decrease in leakage is related to increase of PCC. One reason for a high value of PCC may be the assumed value of OCC (the occupancy rate). It is possible that there is not a constant value for all areas especially if the number of properties in the zone is too small for the full influence of averaging to be effective. Unfortunately, calculation of a more exact value of PCC is not feasible because the exact number of people living in each area and each property at the time of the field flow survey cannot be determined. Also needed is a more exact estimate of legitimate overnight unmetered consumption per property. Some errors may arise from estimation of LOUCP or in counting of the number of properties. In this research LOUCP has been taken as 1 l/prop/hr and sensitivity of results to application of the more recently introduced value of LOUCP = 1.7 l/prop/hr (UK/WI 1994), is investigated later. Because in Eq. (3.1) the summation of leakage and domestic demand is assumed to be constant (i.e. equal to the deduction of meter consumptions from the net inflow) with increasing the value of LOUC the leakage value at MNF time would be decreased and this may lead to appearance of negative hourly leakage values in the modified estimation procedure outlined above. However, the metered consumptions also include uncertainties because they are obtained at best from quarterly meter readings. These together with their assumed profile may be a cause of the negative leakage values, but the main reason is likely to be that, for some hours, the domestic demand values arising from the modified (area average) domestic demand profile are very much greater than the actual domestic demand values originally computed. This, in itself, is indicative of some anomaly being present in the zonal data.

3.3.3 Pressure Dependent Leakage

In this section another alternative approach is investigated to produce improved leakage values and diurnal profiles, this using the concept of pressure dependency of leakage. This concept was developed in Chapter 2 and different pressure-leakage relationships were presented. As expressed by UK/WI (1994-Report B) a linear relationship between pressure and leakage up to an average zone night pressure (AZNP) of about 50 m can be assumed. Also, based on the findings of May (1994) and Lambert (1997b) it was seen that an average power value close to one can be used in the power law equation (Eq. (2.15)). Therefore, for simplification and because of absence of data to identify the fixed and variable leakage paths, in this chapter a linear relationship between pressure and leakage is considered (i.e. N=1 in the power law equation).

3.3.3.1 Investigation of correlation between leakage and pressure profiles

Values of pressure at each node and each hour for every zone have been provided from a calibrated 'WATNET' model. This calibration is itself limited since generally only one or two pressure monitoring points would be present in each zone. The average value of pressure for all nodes at each hour was considered to provide the representative value of pressure for each zone. Finally, the values of average zone pressure for all zones at each hour were produced. It is expected that at midnight when there is the lowest consumption, the pressure will be high and in the morning and evening when consumption is high, it will be lower. Also, before and after noon it is expected to have moderate values. The zones which follow this logical pattern have been selected and the others have been eliminated. The average factors for pressure profiles from the 16 zones exhibiting this intuitive pattern are shown in Table 3.5.

Some possible explanations for the apparent behaviour of zones which do not conform to the expected pattern may be as follows.

1) The zone may include trunk mains supplying downstream storage so that total flow in the mains may not be following a 'normal' diurnal pattern (i.e. if service reservoirs are pumped to fill at night, etc.).
2) 'Time' or 'level' controlled pumps dictate zone pressures.
3) District data are anomalous, including unknown flows passing boundary valves.

Comparison of pressure and leakage profiles and assessment of the degree of correlation between them provides a validation check on the district data and the statistics arising. Figure 3.7 shows the average network pressure (ANP) profile arising, this is compared with the leakage profile (average of 19 zones). It can be observed that there is a strong correlation between them. In addition, using statistical procedures, a linear correlation coefficient of 90% is obtained which verifies the assumption of linear relationship between leakage and pressure in this procedure. The resulting average zone pressure (AZP) profiles compared with the modified leakage profiles (L-modified) for individual zones are presented in Appendix C2.

3.3.3.2 Pressure dependent leakage procedure

In this procedure, the diurnal variations of leakage in each zone are imposed by assuming a linear correlation with either average network pressure, or preferably average zone pressure as an improvement on the standard 'WRc' Type 5 profile (Fig. 2.2). The procedure is as follows:

1) Subtract the sum of metered consumption from net inflow for each zone at MNF time.

$$(\textit{Unmetered consumption})_{MNFT} = (\mathbf{Inflow})_{\mathbf{MNFT}} - (\Sigma\ \mathbf{Types\ 2\text{-}4})_{\mathbf{MNFT}} \qquad (3.7)$$

2) Find the domestic demand consumption at MNF time.

$$(\textit{Domestic demand})_{MNFT} = \textit{LOUCP x No. of Unmetered properties} \qquad (3.8)$$

3) Calculate the leakage value at MNF time.

$$L_{MNFT} = (\textit{Unmetered consumption})_{MNFT} - (\textit{Domestic demand})_{MNFT} \qquad (3.9)$$

4) Relate leakage at each hour to values of average network (or zone) pressure at the same time.

$$L_t = L_{MNFT}\ x\ (P_t / P_{MNFT}) \qquad (3.10)$$

where, P_t and P_{MNFT} are the average network (or zone) pressure values at each time and at MNF time, respectively.

5) Calculate the domestic demand profile by deducting these leakage figures together with corresponding hourly rates of Types 2-4 consumption from the net zonal inflows.

Firstly, values of average network pressure (ANP) are applied to produce the leakage profiles and then values of domestic demand factors are calculated. Then, the average zone pressure (AZP) variations are taken to define the leakage profile in each zone and domestic diurnal patterns have been calculated to see if this procedure produces more zones with acceptable Type 1 domestic profiles (i.e. fewer zonal failure) than the original approaches. The ANP is the average pressure profile for all accepted zones, and whilst being convenient for an initial trial, it is not so rigorous as imposition of the AZP. Domestic demand profiles for 18 and 22 zones are acceptable when L-ANP and L-AZP factors are applied, respectively. Figure 3.8 shows the average domestic demand profiles for these 18 and 22 zones and as expected, adoption of AZP improves the results for leakage values and reduces the risk of anomalies (i.e. more zones can be reconciled). The domestic demand profiles for all zones resulting from the L-AZP profiles are presented in Appendix C3.

In Section 3.3.2 it was mentioned that the value of 1 l/prop/hr has been taken for LOUCP in this research. To see the sensitivity of results to application of other LOUCP values, the value of 1.7 l/prop/hr (recommended by UK/WI 1994) was implemented to produce domestic demand profiles. The results showed a little further improvement and domestic demand profiles for 24 zones were found acceptable (compared with the 22 zones arising from application of LOUCP = 1 l/prop/hr).

Table 3.6 shows the possible explanation for zones with illogical patterns when applying AZP profiles. For those zones identified as having unreasonable pressure profiles, potential causes were outlined earlier in Section 3.3.3.1. But because these average zonal pressure profiles have been imposed to produce diurnal leakage variations, this leads to unexpected leakage patterns. In these zones it is not apparent that there is a linear relationship between leakage and AZP profile and leakage may not be related to average zonal pressure solely (TWGWW, 1980).

Applying L-AZP profiles, a set of new PCC values have been recalculated based on Eq. (2.6) for all zones. The results are presented in Table 3.7. It shows that except for those zones mentioned in Table 3.6, there is not a significant change in values of PCC. It can be concluded, therefore, that for this particular data set, imposing the average zonal pressure profiles to determine diurnal leakage profile does correct some of the illogical patterns of domestic consumption which arise when using the L-WRC in some zone. This refinement is unable, however, to fully reconcile the data sets and further development of the methodology is still called for.

3.3.4 General Comparison

The modified leakage values and profiles for the 19 zones, introduced in Section 3.3.2, can be used to reproduce domestic demands. According to the results of this recalculation, domestic demand profiles of 19 zones are acceptable. Figure 3.9 summarises the domestic demand profiles produced based on the 'WRc' factors (Type 5), modified leakage and pressure dependent leakage factors. It shows a strong correlation between all three methods. Figure 3.10 illustrates the corresponding leakage profiles which have been used for PCC calculation in Fig. 3.9.

3.3.5 Re-Calculation of Leakage Profile Arising from Input of Network Average PCC in Each Zone

To reduce the number of unknown parameters in Eq. (3.1), i.e. Types 1 and 5, and to separate the effects of these variables on each other as a further step towards data reconciliation, one way is to impose the average PCC figures as representative of domestic demand. Thus, the influence of forcing the average level of PCC of all zones, i.e. 115.5 l/hd/day (excluding the two obvious anomalous zones NL041 and NL042) on the individual zones is next investigated, using 11 satisfactory zones which originally produced reasonable leakage profiles after applying this average PCC value.

To broaden the investigation, however, other values for PCC have also been imposed.

e.g.

(i) The average PCC of all 31 zones which is equal to 200.2 l/hd/day. Using this value, the leakage values are re-calculated from which only 6 zones produce reasonable leakage profiles.

(ii) Assuming the value of 150 l/hd/day, a value often imposed in TIF calculations as the average PCC and have been confirmed by UK/WI (1994-Report D). Using this value re-calculated leakage profiles of 14 zones are acceptable.

The modified leakage profiles for individual zones resulting from imposition of different PCC values can be observed in Appendix C4. The average leakage profiles for the acceptable zones arising from imposition of the various average PCC values with use of the average domestic demand pattern (Fig. 3.3) are presented in Table 3.8. These results show that leakage based on imposed average PCC of 200.2 and 150 l/hd/day, although having a logical shape, are inferior to the results based on PCC=115.5 l/hd/day, and that the latter is very close to the results from use of the actual zonal PCC values. Figure 3.11 shows the sensitivity of these leakage factors.

By virtue of the sensitivity of results to using different PCC values, it would appear that adoption of incorrect PCC for a district (arising from initial MNF or TIF calculation) and using a network wide domestic profile can distort the computed leakage profile of some zones. This is discussed in more detail below. As seen earlier, there is an inverse relationship between PCC and leakage. Whenever PCC is higher, the leakage will be lower because domestic demand is increased (see Eq. (3.1)). In the results there are some zones in which the leakage profile exhibits negative values when applying actual or network average PCC in the revised leakage profile computations. These zones are shown in Table 3.9 and can be divided into three different groups:

(i) All applied PCC values lead to leakage profiles including negative values, such as: NL013, NH031, NH033.

(ii) By applying the average PCC, negative values have disappeared, e.g. NL041, NL043, NH032.

(iii) With average PCC some negative leakage values are found, such as: NL014, NL021, NL033, NL035, etc.

These differences can be explained as follows. In the first group, the correct value of PCC must be lower than all applied PCC values. Also, it is likely that either the district inflow readings are not correct, or actual metered consumption during the fieldwork did not conform to its average rate or to the assumed diurnal profiles. Therefore, for this type, it can be concluded that $PCC_{ave} > PCC_{act} > PCC_{correct}$ where PCC_{ave} is network average, PCC_{act} is the value produced by using the field data and leads to questionable results here. Finally $PCC_{correct}$ is the unknown value which would lead to the correct diurnal characteristics. For the second group the actual PCC is not correct and PCC_{ave} is a better estimation. Finally in the third group it can be easily seen that $PCC_{act} < PCC_{ave}$, so creating negative leakage values on the profile. Figure 3.12 shows the leakage profile for zone NL035 as an example of the third group. It can be seen that the nearer to the actual PCC value, the more realistic/logical the pattern becomes.

3.3.6 Critical Review of the Sensitivity of Results to Use of the Single Instantaneous MNF Reading

In this assessment the calculations have been repeated for various hourly 'snapshot' times (in the vicinity of the time of MNF) to see if greater stability in results is achieved. With reference to Table 3.1, it is obvious that MNF values normally occur between 1-6 a.m. and for most zones it is between 2-5 a.m. (see Figure 3.13). Therefore, it is instructive to assess the sensitivity of leakage results to the time selected for calculation. This involves recalculation of the values of L, UFW, PCC, domestic demand profile and leakage for each time selected for the 'MNF' calculation procedure.

For this assessment the average domestic demand profiles have been produced using the L-WRC profile. Then, resulting domestic demand values have been imposed to recalculate the leakage values and produced modified profiles. Figures 3.14 and 3.15 show the results of this assessment for average domestic demand and leakage profiles arising. It is worth mentioning that to avoid unnecessary calculations, the procedure of imposing the pressure dependent leakage and the average PCC values has not been repeated. The effects of different methodologies to calculate the domestic demand and leakage variations were investigated in Sections 3.3.3.2 and 3.3.5. It can be seen from both figures that all shapes are similar and follow a logical pattern. It can be concluded, therefore, that the MNF computation method is not highly sensitive to the precise selection of the computation time within the period 2-5 a.m. This fact can be seen in Figure 3.16 which shows the leakage factors based on average values obtained from four applied MNF times and the actual MNF time.

3.3.7 Variability in Diurnal Profiles

Considering the probabilistic nature of demand and its different categories, the variability in profiles arising from the computations performed for each zone in Sections 3.3.1-3, can be illustrated. A normal distribution is assumed here for variation of domestic demand and leakage profiles at each hour. Then taking the 22 and 19 zones with reasonable profiles for domestic demand and leakage, respectively (Tables 3.3 and 3.4), at each hour the mean and standard deviations of the demand factors have been calculated. The regions of 95% confidence are shown in Figures 3.17 and 3.18 for domestic demand and leakage profiles, respectively. It can be found that almost all domestic and leakage profiles arising from the different procedures described above (e.g. Figs. 3.9 and 3.10), fall within these confidence limits. Therefore, these might be introduced as the appropriate limits for these variables, in this particular area. This is introduced here as a preliminary to the probabilistic treatment of the problem which follows in Section 3.5 and Chapter 8.

3.4 RELIABILITY ASSESSMENT OF FIELD DATA

Criteria for assessment of the reliability of the field data are now considered. It is crucial that the data collected from the different DMAs is reconciled as far as is possible to eliminate errors before starting any detailed computational procedure. This may entail the repetition of fieldwork surveys where PCC and leakage figures arising from the balance studies fall outside the range of anticipated values. Table 3.10 shows values arising from the computations of Section 3.3 for each district after data reconciliation and completion of follow up surveys to eliminate anomalies. It is immediately apparent that certain districts remain anomalous. Criteria which may be imposed for reliability assessment might be as follows:

1. MNF readings which occur other than at night times are not logical in most cases.
2. A pair of minimum and maximum acceptable limits can be imposed on PCC values (say 50-200 l/hd/day). Thus, zones with higher or lower PCC values can be flagged as being anomalous.
3. Error in the flow measurements can lead to negative values in the calculation of domestic demand and leakage values, because the continuity equation cannot be satisfied.
4. Where negative values appear in domestic consumption or leakage profiles, the value of demand factors chosen for that hour and for the other consumption types are greater than the true values.
5. Districts with apparently high values of leakage (or percentage of waste) are subject to potential anomaly (say over 50% or 60%). High values of leakage together with high total daily flow (TDF) may be representative of unreported bursts.
6. It can be seen in Table 3.10 that when $0.7 < PCC_{act} / PCC_{ave} < 1.3$, the majority of zones do not include negative values in the leakage profile. Thus, a tolerance of ± 30% for this ratio would seem to be acceptable in the case of this supply network.
7. Several of the terms in the applied formulas are likely to be subject to seasonal variations and socio-economic factors. These effects should be anticipated in the data evaluation exercise.

The last column of Table 3.10 includes the reliability assessment of the district data based on the above criteria, which leads to a means of 'flagging' the likely presence of an anomaly in the data for that district.

3.5 DATA RECONCILIATION USING A BEST PARAMETER ESTIMATION TECHNIQUE

In section 3.3, a series of methods were investigated for characterisation of domestic demand and leakage as an integral part of supply system evaluation at district metered areas or zonal levels, as necessary in computer network modelling studies. These zonal level flow balance studies form an integral element in preliminary data reconciliation, the outcome of which might call for a partial repeat of the data collection procedure. This section introduces the application of a best parameter estimation technique which integrates this flow balance and data reconciliation study.

3.5.1 Introduction

It has been shown that because of the current situation in the UK, the two unknowns of domestic demand and leakage exist in the flow balance equation, Eq. (3.1). Further investigations in Section 3.3 have illustrated that the interdependency of these two variables produce some difficulties in producing realistic demand and leakage values. It is possible to improve this situation by the separation of unmetered elements of consumption and in so doing also provide a suitable measure of diurnal variations in the flow balance study.

This has been attempted here by applying the pressure dependent leakage approach (Leakage-AZP relationship) and using an average zonal PCC value of 115.5 l/hd/day for the whole network with a standard diurnal profile as a basis for specification of leakage (Type 5) and domestic consumption (Type 1), respectively. Eliminating the two zones, NH012 and NH032 (because of apparent anomaly in data files for net inflow values at MNFT), 29 zones will be analyzed in this procedure. The diurnal profile used here to distribute the total domestic demand through 24 hours of a day has been produced by averaging all non negative domestic demand profiles which are produced by the pressure dependent leakage procedure shown in Figure 3.8.

As mentioned in the last chapter, water demand is probabilistic in nature whilst being subject to a number of uncertainties in the calculation and measurement of its different components. To accommodate these random elements the best parameter estimation methodology has been conceived to systematically investigate the residual errors in zonal flow balance evaluation, when the individual elements in the calculation are constrained to values lying within the chosen validity bands. For simplification here, in this exploratory application, the uncertainties are accommodated on the basis of an assumed uniform probability of occurrence across the validity bands of each variable (i.e. PCC leakage, inflow, outflow, etc.). Consideration of the uncertainties in each demand profile within the validity bands by a probability distribution function might produce more realistic results, but complicates the procedure.

To measure uncertainty in the water system's elemental consumptions, leakage and hydraulic performance, a value of residual flow can be defined, as follows, for each zone,

$$Res_t = Net\ inflow_t - (T_{t,1} + T_{t,2} + T_{t,3} + T_{t,4} + T_{t,5}) \tag{3.11}$$

where Res_t is the residual at each time step, $T_{t,1}$ to $T_{t,5}$ are the five types of consumption in the 'WATNET' modelling procedures, and subscript t (from 0 to 23) refers to time (hrs.) over the daily cycle.

For each elemental consumption type, the following percentage errors are considered intuitively to encompass the range of uncertainty or inaccuracy in the base data.

1) ± 5% error for all flow measurements (i.e. inflows/outflows to the zone) to cover the range of instrument inaccuracies.
2) ± 5% error for all metered consumer elements, Types 2-4, obtained from consumer accounts records. This range of variation is perceived as representing possible day to day variability in metered consumption as well as seasonal drifting. These error margins are necessary since it is impracticable to monitor all metered consumers during a typical field work study.
3) ± 10% error for values of pressure dependent leakage based on the MNF method. This range is incorporated to account for the uncertainty in the relationship between leakage and zone pressure and its integration in the MNF calculation.

4) ± 25% variation for values of domestic unmetered consumption (Type 1). This makes allowance for the fact that PCC may be expected to vary, to some degree, from zone to zone, partly as a result of socio-economic factors. A later refinement could be to build in an explicit link between PCC and socio-economic make up, possibly through the 'ACORN' categorisation system (see Appendix A).

3.5.2 Optimization Procedure

The optimization procedure has been considered in three stages and provisional findings from each are presented next.

Stage 1: (Zone by zone analysis)

In this stage the following minimization procedure is carried out for each individual zone. The objective function is

$$Minimize \;\; F = \sum_{t=1}^{24} \| \, Res_t \, \|^2 \tag{3.12}$$

$$Res_t = \sum_{j=1}^{7} a_{t,j} \cdot x_j \tag{3.13}$$

subject to:

$$\mathbf{0.95 \le x_1 \le 1.05} \tag{3.14}$$

$$\mathbf{0.75 \le x_2 \le 1.25} \tag{3.15}$$

$$\mathbf{0.95 \le x_3 , x_4 , x_5 \le 1.05} \tag{3.16}$$

$$\mathbf{0.90 \le x_6 \le 1.10} \tag{3.17}$$

$$\mathbf{0.00 \le x_7 \le 1.1 \times T_{MNFT,5}} \tag{3.18}$$

$$\sum_{j=1}^{7} a_{MNFT,j} \cdot x_j = LOUC \tag{3.19}$$

in which, $a_{t,1} = Q_{t,0}$ (net inflow), $a_{t,2} = -T_{t,1}$ (domestic demand), $a_{t,3} = -T_{t,2}$ (small trade consumptions), $a_{t,4} = -T_{t,3}$ (10 hrs. industrial activities), $a_{t,5} = -T_{t,4}$ (24 hrs. industrial activities), $a_{t,6} = -T_{t,5}$ (leakage) $a_{t,7} = 0$ and $a_{MNFT,7} = 1$. x_1 to x_6 are variables which represent the optimum values of net inflow, domestic and metered consumptions and leakage, respectively. Furthermore, x_7 represents the optimum leakage value at minimum night flow time (MNFT), i.e. UFWM, and LOUC = 1 l/hr/prop x No. of unmetered properties. Finally $a_{MNFT,2}$ and $a_{MNFT,6}$ are set zero in the MNFT constraint to satisfy the flow balance equation at MNFT. This constraint represents Eq. (2.10) which produces the value of unaccounted for water at the MNF condition, i.e. UFWM. In some zones with no domestic demand ($T_{t,1}$), e.g. pure industrial zones such as NH025 and NH026, or with MNF occurring during the day rather than at night, the MNFT constraint Eq. (3.19), is eliminated because the concept of minimum night flow is not meaningful in such situations.

To calculate the optimum values of flows and consumptions, a programme has been developed using a least square minimization methodology drawing on standard routine E04NCF of the NAG library (NAG 1991). This programme minimizes the least square of the residual values (Eq. 3.13) subject to the boundary constraints for elements of demand, inflow and leakage (Eqs. 3.14-8) and the minimum night flow time constraint (Eq. 3.19). Results for individual zones are shown in Table 3.11, which includes the values of optimal variables and the objective function. Sensitivity of results was also investigated.

Table 3.11 shows that the values of the objective function for some zones with high and unreasonable original PCC (e.g. NL031, NL041, NL043, NH023, NH024) remain high and reconciliation is incomplete. In these zones it can be seen that the optimal value of x_1 has its extreme lower bound and the other variables have their extreme upper bound limits. This suggests that the values of net inflow in such zones may be excessive. One investigation showed that if the possible error for net inflow was allowed to be more than 5% (i.e from 10% to 30%), the value of the objective function would be decreased greatly. It can be concluded that in such zones values of net inflow are probably the main source of anomaly. Furthermore, change in LOUCP from 1.0 to 1.7 l/prop/hr (suggested recently by UK/WI 1994) was found (Table 3.11) to have little effect on the values of objective function. In addition, minimal residual for individual zones was observed to switch apparently arbitrarily between calculations based on 1.0 or 1.7 for LOUCP. Simultaneous change in the tolerance ranges of the constraints (increases up to 100%, in the values given in Eqs. 3.14-3.18) produced very little change in values of the objective functions or optimal value of variables from those given in Table 3.11.

Stage 1 was contrived for minimisation of flow residuals over the 24 hrs. whilst satisfying the MNF method computation (Eq. 3.19), separately. To see the effect of duplication of the MNFT condition, the problem has also been solved using only the 23 (none MNFT) hrs. and the balance equation at MNFT just considered as a constraint. Again, this resulted in little change to optimal solutions.

To see the effect of the optimization on values of PCC, new PCC estimates have been calculated based on the optimal values of x_2 for all individual zones. Table 3.11 shows that despite there being a wide range of differences, a set of intuitively reasonable values are obtained from the optimization procedure. It shows that the optimum average PCC is 102.54 l/hd/day which is 11% less than the average PCC values originally computed.

Stage 1 determines the optimum values of net inflow and consumptions for each individual zone consistent with the imposed constraints. In reality, however, zones are connected together and, therefore, it is preferable to also view the problem in respect of hydraulically inter-dependent groups of zones.

Stage 2: (Combination of individual zones)

As an intermediate step towards the hydraulically inter-dependent groups of zones, initially only the summation of zones, without considering their hydraulic inter-connection, is evaluated in this stage. This is shown below:

$$Minimize \;\; F = \sum_{t=1}^{24} \| \sum_{i=1}^{NZ} Res_{t,i} \|^2 \tag{3.20}$$

$$Res_{t,i} = \sum_{j=1}^{7} a_{t,i,j} \cdot x_{i,j} \tag{3.21}$$

subject to:

$$0.95 \leq x_{i,1} \leq 1.05 \tag{3.14}$$

$$0.75 \leq x_{i,2} \leq 1.25 \tag{3.15}$$

$$0.95 \leq x_{i,3}\ ,\ x_{i,4}\ ,\ x_{i,5} \leq 1.05 \tag{3.16}$$

$$0.90 \leq x_{i,6} \leq 1.1 \tag{3.17}$$

$$0.00 \leq x_{i,7} \leq 1.1\ x\ T_{MNFT,5} \tag{3.18}$$

$$\sum_{j=1}^{7} a_{MNFT,i,j}\ .\ x_{i,j} = LOUC_i \tag{3.22}$$

in which $x_{i,j}$ is the unknown variable for demand element j in zone i and NZ = No. of zones in each group.

It can be seen that the formulation of Stage 2 is similar to Stage 1, but extended so that a combination of zones is added to the objective function. From a study of network connectivity, the test system has been divided into 5 groups of zones. The programme is then applied to each group. Table 3.12 shows a comparison between values of objective functions for each group from Stage 2 and individual zones from Stage 1 (while LOUC = 1 l/prop/hr). It can be observed that the values of the objective function for 4 out of 5 groups is now less than the summation of those from individual zones. It is worth noting that the groups and incorporated zones have been chosen hypothetically based on the schematic representation of the network (Appendix B). Therefore, several different groups with different combination of zones could be considered. This can be a reason for exceedance of the objective function in group 1 from the summation of the individual zonal objective functions.

The approach has been extended by considering the zones in only two larger groups. The optimal variables for this, however, show little difference to the 5 groups solution. Furthermore, anomalous zones with high residual flow (objective function) simply convey this into the objective function for the group without any adjustment. As mentioned earlier, Stage 2 is considered an intermediate step towards a realistic solution. The results demonstrate a need for imposition of other physical constraints on the problem.

3.5.3 Misrepresentation of the Actual Hydraulic Configuration of Network

In the previous analysis values of measured flow elements have been considered to be reliable with ± 5% error. Normally, in the process of data collection in the field, the values of the net inflow are obtained from differences between the total inflows to, and outflows from the zone. It is assumed that all the other connections to adjacent zones are cut off by the closed status of boundary valves. In reality it is possible that some of these valves may not be closed completely and other connections or valves may have been overlooked or forgotten and, consequently, water could be passing from them. Obviously, this would disturb the balance equation in these zones and the methodology can be extended to consider the possibility of unknown flow passing between adjacent zones taking account of hydraulic head factors as follows:

Stage 3: (Network wide optimization considering hydraulic characteristics of the system)

This stage allows consideration of any possible flow passing between any pairs of adjacent zones. To simulate the inter-connectivity of adjacent zones which represents the possible flow passing between them, a new term is added to the objective function of Stage 2, together with relevant constraints. This term assumes that the flow can pass from a zone to its adjacent zone with lower average total head according to the variations of the average total head differences between each pair of adjacent zones. The formulation is as follows.

$$Minimize\ \ F = \sum_{t=1}^{24} \| \sum_{i=1}^{NZ} Res_{t,i} \|^{2} \tag{3.23}$$

$$Res_{t,i} = \sum_{j=1}^{7} a_{t,i,j} \cdot x_{i,j} + \sum_{m=1}^{K} (H_{t,i} - H_{t,m}) \cdot x_{i,m} \tag{3.24}$$

subject to:

$$0.95 \le x_{i,1} \le 1.05 \tag{3.14}$$

$$0.75 \le x_{i,2} \le 1.25 \tag{3.15}$$

$$0.95 \le x_{i,3}\,,\ x_{i,4}\,,\ x_{i,5} \le 1.05 \tag{3.16}$$

$$0.90 \le x_{i,6} \le 1.1 \tag{3.17}$$

$$0.00 \leq x_{i,7} \leq 1.1 \times T_{MNFT,5} \tag{3.18}$$

$$0 \leq x_{i,m} \leq \frac{(Ave.\ Net\ inflow)_i}{|\overline{\Delta H}_{i,m}|} \tag{3.25}$$

$$\sum_{j=1}^{7} a_{MNFT,i,j} \cdot x_{i,j} + \sum_{m=1}^{K} (H_{MNFT,i} - H_{MNFT,m}) \cdot x_{i,m} = LOUC_i \tag{3.26}$$

$$x_{i,m} - x_{m,i} = 0 \tag{3.27}$$

in which, m = 1, ... , K $\forall$ i (adjacent zones with zone i), $H_{t,i}$ and $H_{t,m}$ = average total zone heads of zones i and m, respectively, at time t, $x_{i,m} = Q_{i,m} / \overline{\Delta H}_{i,m}$ is a variable which represents values of unknown passing flow ($Q_{i,m}$) between two adjacent zones i and m and $\overline{\Delta H}_{i,m}$ = daily average of total head difference between two adjacent zones i and m.

Values of total heads within each zone have been produced by a 'WATNET' hydraulic flow model, calibrated and verified from field data (Section 3.3.3.1), then the daily average of total head for each zone has been calculated. $Q_{i,m}$ representing the average daily passing flow has so far been restricted, intuitively, to not exceed the average measured net inflow through zone i.

Because there was insufficient information to identify the actual adjacent zones with closed boundary valves from the schematic of the network (Appendix B), some groups of adjacent zones are assumed to be hydraulically linked together. These groups generally contain some validated zones as well as some anomalous ones. To account for all the adjacent zones, some zones have been considered in more than one group. Consideration of the groups helps to simplify the problem by reducing the size of the objective function and number of variables. It is worth mentioning that consideration of the entire network in only one group leads to a complicated problem with more than 230 variables.

Here the problem has been solved for five groups of zones. Results are presented in Table 3.13. It shows that the values of the objective function for all groups are less than those from Stage 2 for the same group of zones, implying that the formulation of Stage 3 is more realistic. In the ideal situation, where the systems are free from anomalies, it would be expected that terms $x_{i,m}$ (or $Q_{i,m}$) are small or zero. This would suggest that just a few adjacent zones in each group may be subject to unexpected but small passing flows. Furthermore, the ratio of the possible passing flow to the total average inflow to both adjacent zones are presented in Table 3.13, as another criterion to represent the severity of passing flows. Again it is seen that only for a few adjacent zones i.e. NL021-NL022, NL041-NL042, NL041-NL044 and NH025-NH023, is the problem serious. In addition, the optimum values of net inflow, demands and leakage for each zone are shown in Table 3.14. It can be observed that these values are different from the values from Stage 1 (Table 3.11).

Drawing from the results of the earlier data appraisal carried out so far (e.g. Tables 3.6, 3.7 and 3.10), the results of Stage 3 can be evaluated within the following categories. The first category includes those zones which in all procedures have been found to be anomalous, e.g. NL014, NL041, NL043, etc. Therefore, they might be suspected to have passing flow between their adjacent zones. The second category contains zones which have shown some unreasonable behaviour in the other procedures, however, they have been reconciled by Stage 3. The examples are zones NL013, NH024, NH031, etc. Finally in the last category, some zones can be seen which have not shown any anomalous behaviour in the previous procedures, however, apparently they still have a potentiality of flow passing between their boundaries, e.g. zones NL011, NL021, NL022, etc. It might be contended that some low amounts of passing flow arising from this procedure might rightly be ignored, however, those zones with high amounts of passing flow predicted by the procedure need further investigation. Therefore, repetition of field data collection seems to be necessary for certain zones of the first group and also for those adjacent zones for which the ratio of $Q_{i,m}$/Total inflow is high (e.g more than 10%).

It should be mentioned that the optimum values given in Tables 3.13 and 3.14 are approximations to the real values, for the following reasons. First, interaction of groups of zones could not be evaluated precisely because all zones were not evaluated in one group, because of the limitations of the NAG routine. Second, the actual adjacent zones could not be identified very accurately, because of the absence of sufficient information about network connectivity. Finally, in the procedure of Stage 3, the passing flow between adjacent zones have been assumed to vary with the average head difference between two zones. However, considering the realistic relation of $Q_{i,m}$ with $(\overline{\Delta H}_{i,m})^{0.5}$ would give more realistic results although this does not contradict validation of Stage 3 procedure to represent the possibility of passing flows. Therefore, Eqs. 3.24, 3.25 and 3.26 can be changed accordingly to relate the flow to the square root of the head differences, in subsequent application of these procedures.

3.6 SUMMARY

A systematic approach to demand evaluation and leakage computation has been conducted through applications of the procedures of Sections 3.3 and 3.5, in which the following steps are included:

(i) The sensitivity of leakage estimation and its diurnal profile to different methods of calculation and also, to the use of the single instantaneous MNF reading were investigated.

(ii) Imposing the average diurnal profile for PCC into the supply zone analysis, a more rigorous representation of leakage variation than the simple 'block' profile, widely employed, was determined.

(iii) The correlation between this leakage profile and the average pressures recorded in the zone, was investigated and two sets of pressure dependent leakage values and profiles (L-ANP and L-AZP) were applied, on the assumption of linear correlation between zone pressure and leakage, to produce revised PCC figures.

(iv) For the leakage assessment methods established (L-WRc, L-Modified, L-Pressure), variations in PCC and its diurnal profile, for all supply zones were evaluated.

(v) Leakage profiles were recalculated to see the sensitivity of results to the imposition of various average PCC values to appraise the viability of imposing PCC in the flow balance methodology.

(vi) Finally, through an optimization procedure, a best parameter estimation technique was used for flow reconciliation. To minimize the effects of unmetered consumption elements, pressure dependent leakage and average network PCC values were introduced as a better representation for Types 1 and 5 (unmetered consumptions). Thus, an error minimization approach was applied both in individual zones and in groups of adjacent zones. For individual zones without unknown boundary connections, Stage 1 produced the optimal values for elements of demand and flow. To represent the actual hydraulic configuration of the network the connections of a group of zones and inter-connection of adjacent zones were considered in Stages 2 and 3, respectively. Stage 3 led to identification of the optimum values of demand elements together with the possible flow passing between adjacent zones.

3.7 CONCLUSION

By studies such as those summarised above, it is expected that network modelling methodologies will be improved by: a) production of more realistic leakage profiles for normal application in modelling; b) reduced risk of bias in model performance under future operation scenarios by better quality assessment of base data and more reliable allocation between unmetered domestic consumption and leakage and; c) applying systematic flow reconciliation and hydraulic performance appraisal over the whole network, thus identifying serious anomalies and focusing needs for the checking of base data, field measurements and/or perceived pipe system connectivity. It can be concluded that appraisal of the computed pattern of PCC and the correlation between leakage and average zone pressures should be added to the routine procedure for reconciliation of zonal demand (consumption) data sets.

Regarding the role of the best parameter estimation technique to reconcile the field data, the Stage 3 implementation gives the best appraisal of the hydraulic behaviour of the system, producing the optimal values of flow, demands and identifying possible unknown flows passing between each pair of adjacent zones. The high likelihood of passing flows identifies zones as potentially anomalous and call for new field work studies.

The demand and leakage evaluation procedures developed in this chapter should be applied to other water supply networks to enable more thorough validation. This is especially important for the data reconciliation and parameter assessment approach, where ideally the study should follow through a real exercise. This would involve receiving and analysing the raw data on the supply network, zonal configuration and consumption data and flow monitored from the field work study using the integrated data reconciliation/flow balance procedure (Stage 3). From outcomes arising, follow up field surveys and repeated flow monitoring should be conducted followed by re-evaluation of the situation with this new data. Following from the experiences gained by applying the systematic techniques it would be possible to draw together a quantitative assessment of the improvements in demand/consumption characterisation that should be achievable (i.e. fewer situations where adjacent zones show enormous PCC variations etc.).

Table 3.1: Data information for the 'Onetown' area.

Zone Name	No. of Props.[a]	MNFT (hr)	MNF (l/s)	TDF (Ml/day)	UFW (l/prop/hr)	WASTE (%)	PCC (l/hd/day)	MC (Ml/day)
NL011	715.28	4:00	0.556	0.276	1.449	9.50	124.612	0.00603
NL012	549.25	4:00	4.607	0.593	24.179	56.49	151.515	0.04513
NL013	405.17	0:00	0.732	0.149	4.343	28.82	87.033	0.00978
NL014	479.05	2:00	1.690	0.260	9.752	44.23	112.499	0.00000
NL015	522.36	4:00	1.694	0.370	8.850	30.35	177.091	0.00420
NL021	1131.45	3:00	8.012	1.129	20.312	50.53	167.338	0.05644
NL022	365.37	5:00	8.936	0.809	72.244	81.00	141.710	0.03325
NL031	1153.42	6:00	26.101	3.263	82.962	84.02	193.919	0.35172
NL033	747.35	2:00	2.604	0.275	9.290	61.16	27.441	0.04878
NL035	238.07	1:00	1.953	0.240	22.950	61.27	85.021	0.05324
NL036	533	4:00	3.023	0.340	16.208	61.18	86.890	0.00544
NL041	73.27	8:00	10.680	1.134	318.581	81.61	1883.110	0.19406
NL042	851.19	4:00	3.507	0.469	11.412	50.41	71.708	0.06804
NL043	236.73	2:00	0.937	0.713	10.802	9.46	973.579	0.01755
NL044	724.36	2:00	7.189	0.852	18.998	45.67	112.890	0.21981
NH011	1103.47	3:00	7.137	0.951	18.507	55.75	142.383	0.02902
NH012	431.29	6:00	2.219	0.352	19.160	63.17	127.050	0.00260
NH013	601.52	4:00	1.649	0.291	6.943	36.69	75.614	0.06581
NH014	879.50	4:00	4.474	0.607	14.244	51.54	101.644	0.06046
NH021	1230.40	3:00	4.531	0.730	10.125	44.66	119.645	0.02636
NH022	836.65	3:00	3.707	0.522	12.345	48.52	111.344	0.01805
NH023	850.49	14:00	11.359	1.583	49.883	67.85	222.391	0.04448
NH024	139.86	1:00	4.200	0.480	85.229	84.34	330.168	0.06681
NH025	0.00	5:00	2.241	0.472	1152.52	5.86	0.000	0.47179
NH026	0.00	5:00	4.349	0.916	2237.09	5.86	0.000	0.91576
NH027	388.56	3:00	3.731	0.416	27.409	68.24	108.500	0.04491
NH031	1368.42	3:00	2.773	0.602	4.996	27.82	110.966	0.02037
NH032	166.3	3:00	0.472	0.086	7.685	36.31	28.688	0.04256
NH033	483.16	3:00	0.568	0.183	2.629	17.17	96.814	0.02393
NH034	548.35	3:00	0.354	0.123	1.105	11.96	68.948	0.00476
NT031	843.19	3:00	0.694	0.420	1.620	7.87	165.821	0.00276

[a] Represents unmetered and equivalent Type 2 properties. Values after decimal points arise where small metered users (<1000 m^3/yr) are categorised as equivalent properties by dividing their total consumption to a regional average consumption figure.

Table 3.2: An example of the spreadsheet procedure applied to calculate domestic demand and modified leakage profiles (zone NL021).

Time (hr) (1)	Inflow (l/s) (2)	Type 1 (l/s) (3)	Type 2 (l/s) (4)	Type 3 (l/s) (5)	Type 4 (l/s) (6)	Type 5 (l/s) (7)	Type 1 Profile (8)	Type 1 Ave-Prof. (9)	D-Modif. (l/s) (10)	L-Modif. (l/s) (11)	L-Modif. Profile (12)
0	11.38	3.64	0.071	0.00	0.00	7.67	0.605	0.40	2.40	8.91	1.39
1	12.20	4.48	0.049	0.00	0.00	7.67	0.745	0.26	1.55	10.60	1.66
2	9.67	1.96	0.042	0.00	0.00	7.67	0.325	0.19	1.12	8.51	1.33
3	8.01	0.31	0.037	0.00	0.00	7.67	0.051	0.16	0.95	7.03	1.1
4	10.47	2.77	0.042	0.00	0.00	7.67	0.459	0.19	1.15	9.28	1.45
5	10.50	2.75	0.083	0.00	0.00	7.67	0.456	0.27	1.65	8.77	1.37
6	11.32	5.30	0.266	0.00	0.00	5.75	0.880	0.92	5.52	5.53	0.87
7	11.70	5.24	0.496	0.22	0.00	5.75	0.87	1.56	9.42	1.57	0.25
8	14.83	7.81	0.555	0.72	0.00	5.75	1.297	1.77	10.65	2.91	0.46
9	17.49	10.29	0.457	0.99	0.00	5.75	1.709	1.62	9.74	6.60	0.99
10	16.53	9.44	0.340	0.99	0.00	5.75	1.569	1.47	8.85	6.35	0.99
11	15.79	8.73	0.320	0.99	0.00	5.75	1.450	1.26	7.59	6.89	1.08
12	13.31	6.21	0.354	0.99	0.00	5.75	1.031	1.16	6.98	4.98	0.78
13	15.37	8.32	0.298	0.99	0.00	5.75	1.383	1.12	6.76	7.32	1.15
14	12.76	5.80	0.215	0.99	0.00	5.75	0.964	1.01	6.05	5.50	0.86
15	12.21	5.25	0.215	0.99	0.00	5.75	0.873	0.95	5.74	5.27	0.82
16	15.78	8.77	0.262	0.99	0.00	5.75	1.457	1.10	6.61	7.91	1.24
17	13.94	7.17	0.308	0.72	0.00	5.75	1.191	1.27	7.66	5.26	0.82
18	13.77	7.47	0.330	0.22	0.00	5.75	1.241	1.60	9.62	3.60	0.56
19	14.18	8.10	0.323	0.00	0.00	5.75	1.346	1.49	8.98	4.87	0.76
20	15.26	9.21	0.291	0.00	0.00	5.75	1.530	1.45	8.72	6.24	0.98
21	15.10	9.12	0.232	0.00	0.00	5.75	1.514	1.29	7.78	7.09	1.11
22	10.96	3.12	0.169	0.00	0.00	7.67	0.518	0.86	5.20	5.58	0.87
23	11.02	3.24	0.112	0.00	0.00	7.67	0.538	0.64	3.84	7.07	1.11

Table 3.3: Diurnal factors of domestic demand profile for 22 zones.

Time	L011	L012	L013	L014	L015	L021	L022	L035	L036	L042	L043	L044	H011	H013	H014	H021	H022	H023	H031	H033	H034	T031	Ave.
0	0.40	0.52	0.10	0.37	0.32	0.61	0.43	0.28	0.28	0.53	0.08	0.34	0.39	0.44	0.29	0.32	0.36	1.17	0.27	0.30	0.54	0.41	0.40
1	0.20	0.19	0.46	0.18	0.16	0.75	0.28	0.10	0.20	0.33	0.04	0.09	0.13	0.26	0.23	0.15	0.17	1.09	0.10	0.12	0.29	0.17	0.26
2	0.11	0.10	0.31	0.08	0.12	0.32	0.26	0.25	0.13	0.26	0.01	0.08	0.08	0.16	0.16	0.11	0.12	0.99	0.09	0.11	0.14	0.08	0.19
3	0.08	0.11	0.23	0.09	0.08	0.05	0.18	0.20	0.11	0.19	0.09	0.12	0.06	0.24	0.11	0.07	0.08	1.026	0.08	0.09	0.13	0.05	0.16
4	0.07	0.06	0.43	0.10	0.05	0.46	0.18	0.25	0.10	0.12	0.09	0.11	0.09	0.11	0.08	0.10	0.10	1.30	0.08	0.12	0.14	0.06	0.19
5	0.21	0.09	0.66	0.19	0.10	0.46	0.06	0.79	0.16	0.24	0.30	0.30	0.15	0.20	0	0.10	0.12	1.20	0.12	0.14	0.18	0.13	0.27
6	0.66	0.74	1.23	0.75	0.58	0.88	1.42	2.37	0.91	0.85	1.10	1.08	0.71	0.73	0.77	0.59	0.59	2.13	0.56	0.49	0.38	0.68	0.92
7	1.78	1.27	1.74	1.92	1.54	0.87	1.59	2.31	1.28	1.52	2.07	1.56	1.47	1.68	1.54	1.56	1.38	1.07	1.82	1.77	0.75	1.96	1.57
8	2.13	1.70	1.92	2.48	2.08	1.30	1.45	1.05	1.66	1.78	2.42	1.34	1.97	1.65	1.61	1.87	1.74	0.61	2.13	1.95	1.85	2.23	1.77
9	1.87	1.38	1.68	2.41	1.81	1.71	1.25	0.97	1.65	1.30	2.01	1.26	1.96	1.25	1.44	1.61	1.48	0.76	1.38	1.89	2.87	1.63	1.62
10	1.53	1.56	1.59	2.08	1.49	1.57	1.23	1.46	1.65	1.22	1.48	1.27	1.56	1.20	1.31	1.58	1.57	0.47	1.39	1.57	2.15	1.39	1.47
11	1.23	1.33	1.08	1.70	1.44	1.45	1.10	0.95	1.39	1.07	0.91	1.85	1.32	1.12	1.49	1.48	1.41	0.09	1.29	1 .0	1.49	1.15	1.26
12	1.13	1.28	0.98	1.58	1.23	1.03	1.19	0.74	1.37	1.12	1.14	1.31	1.29	0.97	1.18	1.36	1.30	0.82	1.24	1.12	1.11	1.03	1.16
13	1.13	1.21	0.91	1.67	1.13	1.38	1.25	1.18	1.23	0.84	1.13	1.51	1.26	0.87	1.15	1.23	1.20	0.38	1.12	1.01	1.06	0.81	1.12
14	0.87	1.08	0.95	0.88	1.05	0.96	1.14	1.31	1.17	0.81	0.98	1.54	1.05	0.93	1.19	1.09	1.25	0.04	1.16	0.86	0.95	0.85	1.01
15	0.85	0.93	1.09	0.81	0.91	0.87	1.21	1.10	1.16	0.55	0.87	1.60	1.03	0.81	1.09	0.96	1.18	0.43	0.97	0.77	0.86	0.79	0.95
16	1.08	1.03	1.27	0.93	1.14	1.46	1.17	0.65	1.29	1.05	1.23	1.11	1.04	1.14	1.18	1.16	1.35	0.43	1.27	1.03	1.02	1.13	1.10
17	1.18	1.22	1.21	0.92	1.35	1.19	1.35	1.24	1.50	1.21	1.35	1.44	1.25	1.40	1.43	1.32	1.44	1.08	1.40	1.26	1.08	1.16	1.27
18	1.54	1.88	1.23	1.17	1.63	1.24	1.61	1.45	1.58	1.81	1.56	1.51	1.52	1.83	1.80	1.58	1.58	1.85	1.74	1.81	1.42	1.82	1.60
19	1.48	1.64	0.10	1.15	1.37	1.35	1.68	1.64	1.31	1.98	1.47	1.18	1.53	1.93	1.59	1.46	1.51	1.31	1.54	1.59	1.44	1.65	1.49
20	1.45	1.60	1.29	0.96	1.54	1.53	1.68	1.69	1.43	1.66	1.25	1.41	1.41	1.71	1.31	1.34	1.30	1.68	1.40	1.37	1.31	1.54	1.45
21	1.06	1.32	1.37	0.76	1.21	1.51	1.74	1.50	1.36	1.71	1.09	1.24	1.18	1.41	1.22	1.27	1.19	1.57	1.10	1.26	1.36	1.23	1.29
22	1.08	1.13	0.60	0.51	0.98	0.52	0.34	0.44	0.65	1.12	0.81	0.45	0.90	1.12	0.91	0.93	0.95	1.22	1.03	1.23	0.87	1.24	0.86
23	0.79	0.65	0.69	0.31	0.70	0.54	0.20	0.09	0.44	0.72	0.53	0.3	0.66	0.84	0.80	0.75	0.64	1.30	0.73	0.75	0.81	0.81	0.64

Table 3.4: Diurnal factors of modified leakage profile for 19 zones.

Time	NL012	NL014	NL021	NL022	NL031	NL033	NL035	NL036	NL042	NL044	NH011	NH012	NH013	NH014	NH021	NH022	NH023	NH024	NH027	Ave.
0	1.29	1.17	1.40	1.21	1.13	1.26	1.15	1.13	1.29	1.16	1.19	1.31	1.26	1.11	1.10	1.16	1.59	1.12	1.09	1.22
1	1.15	1.09	1.66	1.21	1.15	1.27	1.13	1.17	1.25	1.08	1.07	1.56	1.20	1.18	1.06	1.11	1.62	1.10	1.12	1.22
2	1.14	1.06	1.33	1.22	1.16	1.24	1.23	1.17	1.26	1.13	1.11	1.87	1.17	1.18	1.10	1.13	1.61	1.13	1.15	1.23
3	1.17	1.11	1.10	1.21	1.17	1.27	1.22	1.17	1.23	1.18	1.11	1.91	1.31	1.16	1.08	1.12	1.64	1.15	1.16	1.23
4	1.10	1.08	1.45	1.20	1.31	1.25	1.22	1.14	1.15	1.15	1.11	1.86	1.10	1.11	1.08	1.11	1.77	1.13	1.15	1.24
5	1.07	1.09	1.37	1.15	1.18	1.22	1.42	1.13	1.18	1.22	1.09	1.80	1.10	1.08	0.96	1.05	1.67	1.17	1.11	1.21
6	0.77	0.68	0.87	1.01	0.67	0.92	1.52	0.89	0.85	1.01	0.72	1.38	0.66	0.78	0.46	0.56	1.52	1.29	0.75	0.91
7	0.69	1.36	0.25	0.91	0.53	0.69	1.21	0.73	0.87	0.89	0.82	1.01	1.04	0.88	0.89	0.71	0.65	0.81	0.56	0.81
8	.085	1.84	0.46	0.83	0.54	0.63	0.60	0.83	0.91	0.61	1.08	1.05	0.75	0.77	1.03	0.87	0.31	0.83	0.60	0.81
9	0.73	1.95	0.99	0.82	0.50	0.55	0.63	0.92	0.67	0.66	1.20	1.17	0.44	0.76	0.89	0.76	0.46	1.00	0.82	0.84
10	0.97	1.71	0.99	0.85	0.81	0.78	0.90	1.01	0.72	0.77	0.98	1.23	0.56	0.77	1.06	1.01	0.39	1.05	0.93	092
11	0.95	1.47	1.08	0.86	0.77	0.89	0.77	0.98	0.76	1.30	0.95	1.20	0.73	1.09	1.20	1.06	0.30	1.12	1.07	0.98
12	0.99	1.45	0.78	0.91	0.71	0.87	0.72	1.03	0.87	1.00	1.02	1.09	0.67	0.91	1.17	1.05	0.73	1.31	1.12	0.97
13	0.97	1.62	1.15	0.93	0.96	0.89	0.93	0.97	0.70	1.16	1.02	0.69	0.58	0.92	1.05	0.98	0.52	1.11	1.18	0.96
14	0.96	0.73	0.86	0.93	1.12	0.93	1.03	1.00	0.76	1.27	0.94	0.53	0.81	1.05	1.01	1.15	0.41	1.32	1.24	0.95
15	0.88	.71	0.82	0.96	1.08	0.98	0.96	1.03	0.61	1.34	0.97	0.34	0.72	1.01	0.90	1.13	0.64	1.24	1.12	0.92
16	0.85	.68	1.24	0.92	0.99	0.91	0.71	1.02	0.87	0.91	0.85	0.39	0.95	0.97	0.99	1.16	0.56	1.14	0.99	0.90
17	0.86	0.44	0.82	0.92	0.88	0.89	0.88	1.04	0.86	1.01	0.88	0.61	1.06	1.03	0.97	1.07	0.80	0.86	1.01	0.89
18	1.10	0.34	0.56	0.90	1.09	0.94	0.84	0.89	1.05	0.84	0.83	0.67	1.19	1.07	0.88	0.88	1.03	0.63	0.87	0.87
19	1.01	0.46	0.76	0.94	1.12	1.04	0.96	0.79	1.25	0.69	0.93	0.58	1.44	0.98	0.86	0.92	0.81	0.60	0.89	0.90
20	1.01	0.26	0.98	0.95	1.03	1.09	1.00	0.89	1.05	0.87	0.87	0.19	1.23	0.79	0.76	0.74	1.02	0.51	0.76	0.84
21	0.92	0.19	1.11	1.00	1.25	1.07	0.99	0.94	1.20	0.87	0.80	0.18	1.05	0.84	0.87	0.79	1.04	0.59	0.89	0.87
22	1.39	0.74	0.87	1.08	1.49	1.24	1.02	1.07	1.38	0.92	1.23	0.51	1.52	1.23	1.28	1.29	1.38	0.82	1.19	1.14
23	1.21	0.76	1.11	1.10	1.36	1.17	0.97	1.08	1.26	0.97	1.22	0.86	1.47	1.33	1.35	1.20	1.54	1.00	1.22	1.17

Table 3.5: Diurnal factors of pressure profile for 16 zones.

Time	NL012	NL013	NL014	NL015	NL021	NL022	NL031	NL033	NL035	NL036	NL041	NL042	NL043	NL044	NH034	NT031	Ave.
0	1.07	1.12	1.10	1.09	1.13	1.18	1.17	1.22	1.18	1.16	1.17	1.21	1.21	1.20	1.06	1.11	1.15
1	1.13	1.14	1.10	1.09	1.11	1.16	1.17	1.22	1.18	1.15	1.16	1.21	1.20	1.20	1.08	1.12	1.15
2	1.15	1.16	1.11	1.10	1.14	1.18	1.19	1.25	1.19	1.17	1.18	1.23	1.22	1.22	1.08	1.12	1.17
3	1.15	1.16	1.11	1.10	1.16	1.18	1.19	1.25	1.20	1.17	1.18	1.24	1.22	1.22	1.08	1.13	1.17
4	1.15	1.16	1.12	1.10	1.15	1.19	1.19	1.26	1.20	1.18	1.18	1.24	1.23	1.22	1.08	1.13	1.17
5	1.13	1.14	1.12	1.10	1.15	1.20	1.20	1.26	1.20	1.18	1.19	1.25	1.23	1.23	1.07	1.12	1.17
6	1.07	1.08	1.06	1.08	1.09	1.08	1.13	1.16	1.09	1.10	1.11	1.15	0.89	0.94	1.04	0.83	1.06
7	0.95	0.93	1.16	0.89	0.97	0.93	0.95	0.93	0.90	0.94	0.94	0.92	0.79	0.86	0.88	0.74	0.92
8	0.79	0.79	0.83	0.84	0.88	0.85	0.87	0.81	0.85	0.87	0.87	0.79	0.79	0.86	0.88	0.74	0.83
9	0.87	0.87	0.85	0.88	0.86	0.87	0.89	0.84	0.87	0.89	0.89	0.84	0.84	0.88	0.68	0.91	0.86
10	0.85	0.85	0.88	0.93	0.88	0.88	0.88	0.83	0.87	0.89	0.89	0.85	0.88	0.88	0.85	0.97	0.88
11	0.93	0.94	0.91	0.95	0.91	0.90	0.90	0.85	0.90	0.91	0.90	0.88	0.93	0.88	0.97	1.02	0.92
12	0.95	0.95	0.93	0.98	1.03	0.92	0.92	0.94	0.92	0.92	0.91	0.89	0.93	0.91	1.01	1.04	0.95
13	0.97	0.97	0.93	1.00	0.93	0.93	0.92	0.85	0.92	0.93	0.93	0.92	0.95	0.92	1.03	1.07	0.95
14	0.99	1.00	0.98	1.02	0.98	0.96	0.94	0.92	0.94	0.95	0.95	0.95	0.97	0.94	1.04	1.06	0.98
15	1.01	1.01	0.99	1.00	0.99	0.97	0.95	0.93	0.96	0.96	0.91	0.97	0.99	0.96	1.05	1.07	0.98
16	0.98	0.98	0.97	1.00	0.93	0.93	0.92	0.89	0.93	0.94	0.94	0.91	0.94	0.93	1.03	1.02	0.95
17	0.96	0.96	0.97	0.97	0.94	0.93	0.93	0.89	0.92	0.93	0.94	0.91	0.93	0.92	1.02	1.02	0.95
18	1.01	0.87	0.93	0.91	0.91	0.88	0.87	0.84	0.88	0.89	0.89	0.84	0.88	0.88	0.97	0.87	0.90
19	0.94	0.95	0.95	0.96	0.93	0.91	0.91	0.88	0.91	0.92	0.92	0.88	0.91	0.92	0.98	0.91	0.92
20	0.93	0.93	0.97	0.94	0.93	0.94	0.94	0.91	0.94	0.95	0.95	0.93	0.96	0.95	1.00	0.94	0.95
21	0.98	0.98	1.00	1.00	0.96	0.98	0.96	0.97	0.98	0.97	0.98	0.96	0.99	0.99	1.03	1.00	0.98
22	0.98	0.99	1.01	1.02	1.02	1.01	0.98	1.00	1.01	1.00	1.01	1.00	1.03	1.02	1.05	1.00	1.01
23	1.06	1.07	1.03	1.05	1.03	1.03	1.02	1.06	1.05	1.03	1.03	1.05	1.07	1.06	1.06	1.07	1.05

Table 3.6: Zones with illogical PCC diurnal pattern based on applying AZP profiles.

Zone Name	Remarks
NL015	Includes negative values
NL041	MNFT=8:00 (is wrong), Unreasonable pressure profile
NH012, NH032	Appear to have wrong inflow values (abstracted from database)
NH023	MNFT=14:00 (is wrong), Unreasonable pressure profile
NH024	Including negative values, PCC is high (330), Unreasonable pressure profile
NH025, NH026	Including negative values, PCC is low (0), Unreasonable pressure profile
NH027	Including negative values, Unreasonable pressure profile

Table 3.7: Values of PCC for all zones arising from adoption of average zone pressure dependent leakage (L-AZP).

Zone Name	Initial PCC	New PCC (based on L-AZP)	Zone Name	Initial PCC	New PCC (based on L-AZP)
NL011	124.612	122.095	NH012[a]	127.050	-67.993
NL012	151.515	142.229	NH013	75.614	55.140
NL013	87.033	84.294	NH014	101.644	66.307
NL014	112.499	105.638	NH021	119.645	93.650
NL015[a]	177.091	26.33	NH022	111.344	91.023
NL021	167.338	160.465	NH023[a]	222.391	306.206
NL022	141.710	143.179	NH024[a]	330.168	114.018
NL031	193.919	339.599	NH025[a]	0.000	0.000
NL033	27.441	23.593	NH026[a]	0.000	0.000
NL035	85.021	81.144	NH027[a]	108.500	56.851
NL036	86.89	81.815	NH031	110.966	107.242
NL041[a]	1883.110	1782.736	NH032[a]	28.688	29800.250
NL042	71.708	75.256	NH033	96.814	92.097
NL043	973.579	973.986	NH034	68.948	67.816
NL044	112.890	130.160	NT031	165.821	164.669
NH011	142.383	108.772			

[a] Represents the zones with questionable data from Table 3.6.

Table 3.8: Modified leakage factors based on imposition of different average PCC values and the average domestic diurnal profile from Figure 3.3.

Time (hr)	PCC_{ave} = 200.2	PCC_{ave} = 150	PCC_{ave} = 115.5	PCC_{act}[a]
00	1.4180	1.3832	1.2268	1.2162
01	1.3951	1.4164	1.2644	1.2206
02	1.4170	1.4403	1.2825	1.2307
03	1.4282	1.4335	1.2673	1.2343
04	1.4520	1.4460	1.2977	1.2366
05	1.3640	1.3921	1.2625	1.2141
06	0.9241	0.8716	0.9970	0.9104
07	0.5929	0.5819	0.7842	0.8149
08	0.5647	0.4853	0.7195	0.8097
09	0.6307	0.5793	0.7749	0.8383
10	0.7174	0.7500	0.8952	0.9202
11	0.7820	0.9221	0.9399	0.9759
12	0.8890	0.9131	0.9220	0.9678
13	0.8850	0.9564	0.9183	0.9646
14	0.8905	1.0228	0.9310	0.9499
15	0.9262	1.0178	0.9132	0.9179
16	0.8021	0.9022	0.8843	0.8982
17	0.8014	0.8704	0.9028	0.8895
18	0.8320	0.7271	0.8961	0.8727
19	0.8441	0.7448	0.8869	0.8968
20	0.8660	0.7448	0.8624	0.8423
21	0.9197	0.8445	0.9085	0.8730
22	1.3189	1.2390	1.1123	1.1396
23	1.3393	1.3155	1.1504	1.1654

[a] This column arises from imposition of the actual PCC values and L-modified profile for 19 zones to produce the average diurnal profile for domestic demand (see Fig. 3.6).

Table 3.9: Quality of leakage profiles based on using different PCC values.

Zone Name	Actual PCC l/hd/day	PCC_{act}	PCC_{ave}= 200.2	PCC_{ave} = 150	PCC_{ave} = 115.5
NL013	87.0334	-	- , F	- , F	- , F
NL014	112.4987	+ , F	- , F	- , F	+ , F
NL021	167.3380	+	-	+	+
NL033	27.4414	+	- , F	- , F	- , F
NL035	85.0213	+	- , F	+	+
NL036	86.8900	+	-	+	+
NL041	1883.1116	-	+	+	+
NL042	71.7075	+	- , F	-	-
NL043	973.5787	- , F	+ , F	+ , F	+ , F
NL044	112.8902	+	-	+	+
NH012	127.0504	+	-	- , F	+ , F
NH013	75.6143	+	- , F	-	-
NH014	101.6436	+	-	+	+
NH021	119.6452	+	- , F	+	+
NH022	111.3444	+	-	+	+
NH027	108.5001	+	-	+	+
NH031	110.9661	- , F	- , F	- , F	- , F
NH032	28.6880	- , F	+ , F	+ , F	+ , F
NH033	96.8143	- , F	- , F	- , F	-

(+): Logical pattern; (-): Including negative values; (F): Illogical pattern

Table 3.10: Reliability assessment of base data.

Zone Name	PCC_{act} l/hd/day	PCC_{act} / 200.2	PCC_{act} / 150	PCC_{act} / 115.5	UFW l/prop/hr	Waste (%)	Assessment[a]
NL011	124.612	0.622	0.831	1.079	1.449	9.50	Rd
NL012	151.515	0.757	1.010	1.312	24.179	56.49	Rd
NL013	87.033	0.435	0.580	0.754	4.343	28.82	3,4
NL014	112.499	0.562	0.750	0.974	9.752	44.23	3,4
NL015	177.091	0.885	1.181	1.533	8.850	30.35	Rd
NL021	167.338	0.836	1.116	1.449	20.312	50.53	Rd
NL022	141.710	0.708	0.945	1.227	72.244	81.00	Rd
NL031	193.919	0.969	1.293	1.679	82.962	84.02	5
NL033	27.441	0.137	0.183	0.238	9.290	61.16	Rd
NL035	85.021	0.425	0.569	0.736	22.950	61.27	Rd
NL036	86.890	0.434	0.579	0.752	16.208	61.18	Rd
NL041	1883.11	9.406	12.554	16.303	318.581	81.61	1,2,5,6
NL042	71.708	0.358	0.478	0.621	11.412	50.41	Rd
NL043	973.579	4.863	6.491	8.428	10.802	9.46	2,5,6
NL044	112.890	0.564	0.753	0.977	18.998	45.67	Rd
NH011	142.383	0.711	0.949	1.233	18.507	55.75	Rd
NH012	127.050	0.635	0.847	1.100	19.160	63.17	Rd
NH013	75.614	0.378	0.504	0.655	6.943	36.69	Rd
NH014	101.644	0.508	0.678	0.880	14.244	51.54	Rd
NH021	119.645	0.598	0.798	1.036	10.125	44.66	Rd
NH022	111.344	0.556	0.742	0.964	12.345	48.52	Rd
NH023	222.391	1.111	1.483	1.925	49.883	67.85	1,2,5,6
NH024	330.168	1.649	2.201	2.859	85.229	84.34	2,5,6
NH025	0.000	0.000	0.000	0.000	1152.52	5.86	2,3,4
NH026	0.000	0.000	0.000	0.000	2237.09	5.86	2,3,4
NH027	108.500	0.542	0.723	0.939	27.409	68.24	Rd
NH031	110.966	0.554	0.740	0.961	4.996	27.82	3,4
NH032	28.688	0.143	0.191	0.248	7.685	36.31	2,3,4
NH033	96.814	0.484	0.645	0.838	2.629	17.17	3,4
NH034	68.948	0.344	0.460	0.597	1.105	11.96	Rd
NT031	165.821	0.8283	1.106	1.436	1.620	7.87	Rd

[a] Rd signifies the reliability of zone's data and numbers (see Section 3.4) represent the probable cause of data anomaly.

Table 3.11: Results of Stage 1 for optimal values of variables and objective functions.

Zone Name	x_1	x_2	x_3	x_4	x_5	x_6	OBJ-F (LOUC P= 1)	OBJ-F (LOUC P= 1.7)	Original PCC	Optimal PCC
NL11	0.95	1.06	1.05	0.0	0.0	0.90	2.408	2.462	124.61	122.25
NL12	0.95	1.25	1.05	1.02	0.0	0.90	1.709	1.735	151.52	144.38
NL13	0.97	0.75	0.95	0.0	0.0	0.90	0.842	0.857	87.04	86.63
NL14	0.95	0.96	0.0	0.0	0.0	0.90	5.828	5.663	112.50	110.88
NL15	0.95	1.25	1.05	0.0	0.0	1.02	5.013	5.692	177.09	144.38
NL21	0.95	1.17	0.95	1.05	0.0	1.04	24.035	23.391	167.34	134.81
NL22	0.95	1.24	0.95	0.95	0.0	0.94	0.333	0.935	141.71	143.28
NL31	0.95	1.25	1.05	1.05	1.05	1.10	484.460	497.960	193.92	144.38
NL33	1.05	0.75	0.95	0.95	0.0	0.90	21.499	18.097	27.44	86.63
NL35	0.95	0.75	0.95	0.95	0.0	0.91	0.722	0.771	85.02	86.63
NL36	0.99	0.82	0.0	1.05	0.0	0.90	0.483	0.412	86.89	94.24
NL41	0.95	1.25	1.05	0.0	1.05	1.10	115.560	116.270	1883.11	144.38
NL42	1.05	0.82	0.95	0.95	0.0	0.92	1.634	1.667	71.71	94.53
NL43	0.95	1.25	1.05	1.05	0.0	1.10	606.280	613.600	973.58	144.38
NL44	0.95	0.93	1.05	1.05	1.05	0.90	7.329	7.153	112.89	107.89
NH11	1.03	1.21	1.05	1.05	0.0	0.90	4.435	4.151	142.38	139.83
NH13	1.05	0.75	0.95	0.95	0.0	0.90	1.703	1.183	75.61	86.63
NH14	1.01	0.75	0.95	0.95	0.0	0.90	0.938	0.848	101.64	86.64
NH21	1.05	1.02	1.05	1.05	0.0	0.90	2.316	1.934	119.65	117.76
NH22	0.99	0.88	1.05	0.0	0.0	0.90	1.251	1.033	111.34	101.66
NH23	0.95	1.25	1.05	0.95	0.0	1.10	415.620	431.860	222.39	144.38
NH24	0.95	0.75	1.05	1.05	1.05	0.95	4.478	4.462	330.17	86.63
NH25	0.97	0.00	0.95	0.0	0.0	0.90	67.990	67.990	. 0.00	0.00
NH26	0.95	0.00	1.05	0.0	0.0	1.10	521.530	521.530	0.00	0.00
NH27	1.00	0.75	0.95	1.05	0.0	0.90	1.799	1.617	108.50	86.63
NH31	0.95	0.94	1.05	0.0	0.95	0.90	8.246	8.841	110.97	108.74
NH33	0.95	0.81	1.05	0.95	0.0	0.90	0.848	0.768	96.81	94.01
NH34	1.05	0.75	0.0	0.95	0.0	0.90	3.144	2.592	68.95	86.63
NT31	0.95	1.25	1.05	0.0	0.0	1.10	14.631	17.462	165.82	144.38

Table 3.12: Comparison of objective functions from Stage 1 and Stage 2.

Group	OBJ-F	Zone	OBJ-F [a]
1	72.84	NL011	2.408
		NL012	1.709
		NL013	0.842
		NL014	5.828
		NL015	5.013
		NT031	14.630
	Sum.		30.430
2	18.68	NL021	24.035
		NL022	0.333
		NH013	1.703
		NH014	0.938
	Sum.		27.009
3	245.26	NL031	484.460
		NL033	21.499
		NL036	0.483
		NL035	0.722
		NL042	1.634
		NL043	606.280
		NL044	7.329
		NL041	115.560
	Sum.		1237.977
4	229.31	NH011	4.435
		NH031	8.246
		NH033	0.848
		NH034	3.144
		NH023	415.620
	Sum.		432.293
5	245.237	NH021	2.316
		NH022	1.251
		NH024	4.478
		NH027	1.799
		NH025	67.990
		NH026	521.530
		NH023	415.620
	Sum.		1014.364

[a] Arising from solution of Stage 1

Table 3.13: Optimum values of $x_{i,m}$ and $\overline{\Delta H}_{i,m}$ for adjacent zones from Stage 3.

Group	OBJ-F	Adjacent Zones	$\overline{\Delta H}_{i,m}$ (m)	$x_{i,m}$ (l/s/m)	$Q_{i,m}$ (l/s)	$Q_{i,m}$/Net Inflow (%)
1	50.26	NL011 - NT031	53.16	0.001	0.053	0.66
		NL013 - NL012	0.65	0.0	0.0	0.0
		NL012 - NL014	41.30	0.005	0.202	2.05
		NL015 - NL012	19.51	0.0	0.0	0.0
		NL012 - NT031	37.66	0.0	0.0	0.0
		NL013 - NL014	41.95	0.0	0.0	0.0
		NL015 - NT031	57.16	0.0	0.0	0.0
2	0	NL021 - NL022	1.75	2.373	4.145	18.48
		NH014 - NL021	71.27	0.014	1.026	5.11
		NH024 - NL022	75.67	0.0	0.0	0.0
		NH013 - NH014	1.59	0.067	0.107	1.03
3	82.64	NL031 - NL033	3.64	0.0	0.0	0.0
		NL031 - NL042	2.13	0.0	0.0	0.0
		NL036 - NL031	0.52	0.798	0.415	0.10
		NL036 - NL042	2.65	0.0	0.0	0.0
		NL036 - NL035	0.94	0.0	0.0	0.0
		NL041 - NL042	2.31	2.349	5.426	29.20
		NL043 - NL042	1.77	0.0	0.0	0.0
		NL043 - NL044	0.24	0.199	0.047	2.59
		NL041 - NL043	0.55	0.0	0.0	0.0
		NL041 - NL044	0.78	12.610	9.836	42.79
4	154.17	NH011 - NH031	4.90	0.0	0.0	0.0
		NH033 - NH031	0.88	0.0	0.0	0.0
		NH023 - NH031	2.68	0.0	0.0	0.0
		NH033 - NH034	32.80	0.001	0.016	0.45
		NH023 - NH034	34.61	0.0	0.0	0.0
5	242.48	NH021 - NH022	19.98	0.013	0.256	1.77
		NH023 - NH021	8.59	0.0	0.0	0.0
		NH023 - NH022	28.57	0.0	0.0	0.0
		NH023 - NH027	33.56	0.144	0.511	2.20
		NH025 - NH023	3.66	1.492	5.460	22.96
		NH026 - NH023	2.35	0.0	0.0	0.0

Table 3.14: Optimum values of variables arising from Stage 3.

Zone Name	x_1	x_2	x_3	x_4	x_5	x_6
NL011	1.01	1.25	0.95	0.00	0.00	0.90
NL012	0.95	1.25	1.05	1.05	0.00	0.90
NL013	0.95	1.25	1.05	0.00	0.00	0.90
NL014	0.95	1.25	0.00	0.00	0.00	0.90
NL015	0.95	0.75	0.95	0.00	0.00	0.90
NL021	0.95	1.11	0.95	0.95	0.00	0.90
NL022	0.95	1.00	1.00	1.00	0.00	1.00
NL031	0.95	1.25	1.05	1.00	1.05	1.00
NL033	0.95	1.00	1.00	1.00	0.00	1.10
NL035	0.95	0.75	0.95	0.95	0.00	1.10
NL036	0.95	1.25	0.00	1.00	0.00	1.00
NL041	0.95	1.25	0.95	0.00	1.05	0.90
NL042	0.95	1.00	1.00	1.05	0.00	1.00
NL043	0.95	1.00	0.95	1.00	0.00	1.00
NL044	0.95	1.00	1.00	1.05	1.05	1.00
NH011	0.95	0.85	0.95	0.95	0.00	1.10
NH013	0.95	0.75	0.95	1.00	0.00	1.10
NH014	0.95	1.00	0.95	1.00	0.00	1.00
NH021	0.95	1.25	0.95	1.05	0.00	1.10
NH022	0.95	1.22	0.95	0.00	0.00	1.10
NH023	0.95	1.25	0.95	1.05	0.00	1.10
NH024	0.95	1.00	1.00	1.00	1.00	1.00
NH025	0.95	0.00	0.95	0.00	0.00	0.90
NH016	0.95	0.00	0.95	0.00	0.00	1.10
NH017	0.95	0.75	0.95	1.05	0.00	1.10
NH031	0.95	0.75	1.05	0.00	1.05	1.10
NH033	1.05	1.25	1.05	0.95	0.00	1.10
NH034	1.05	0.75	0.00	0.95	0.00	0.90
NT031	1.05	1.25	1.05	0.00	0.00	0.90

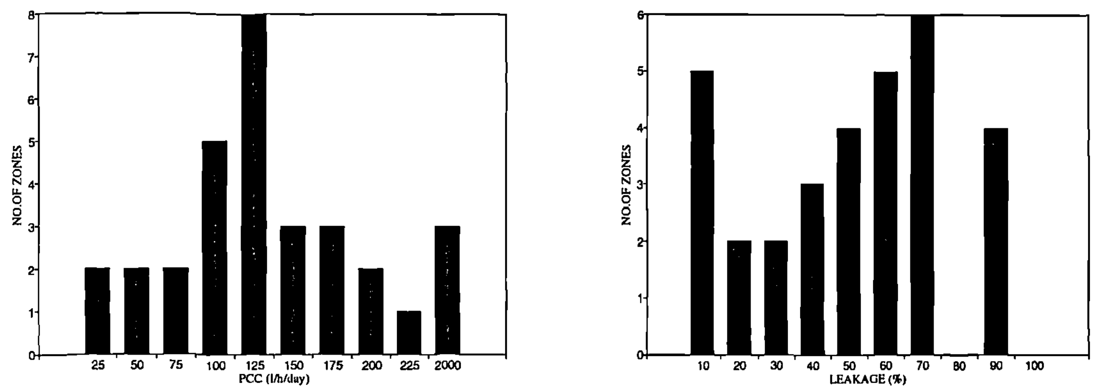

Figure 3.1: Variation of PCC and Leakage in the 'Onetown' study.

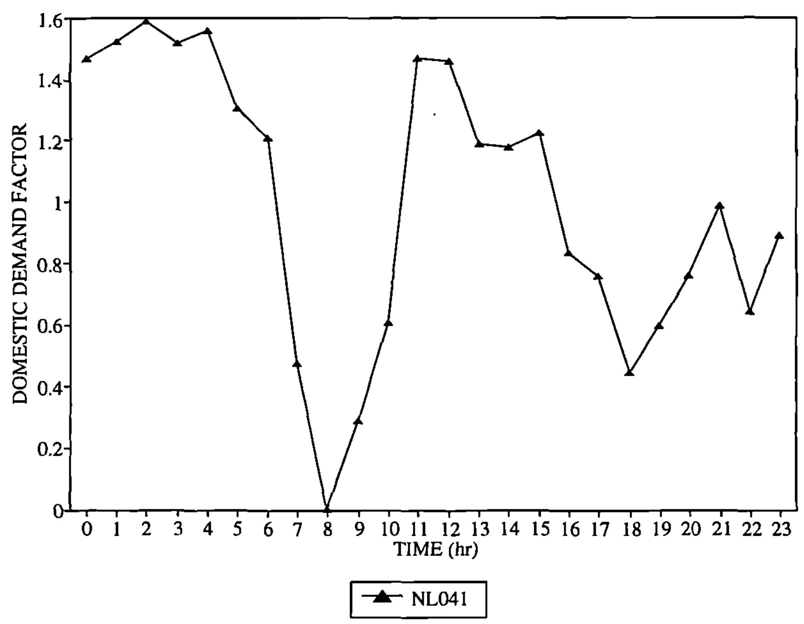

Figure 3.2: Illogical pattern for domestic demand raised from imposition of the 'WRc' factors (Table 2.1).

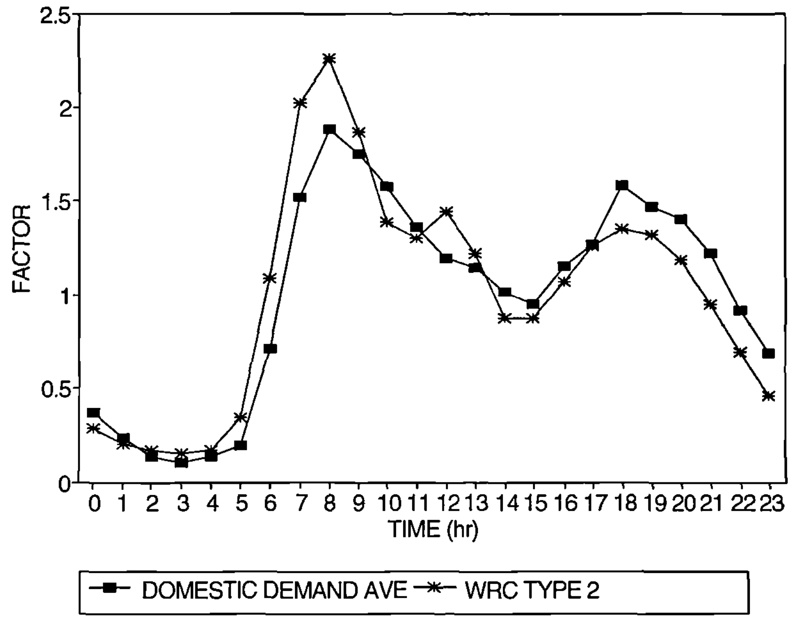

Figure 3.3: Average domestic demand profile for 22 zones and 'WRc' Type 2 profile for equivalent domestic consumption.

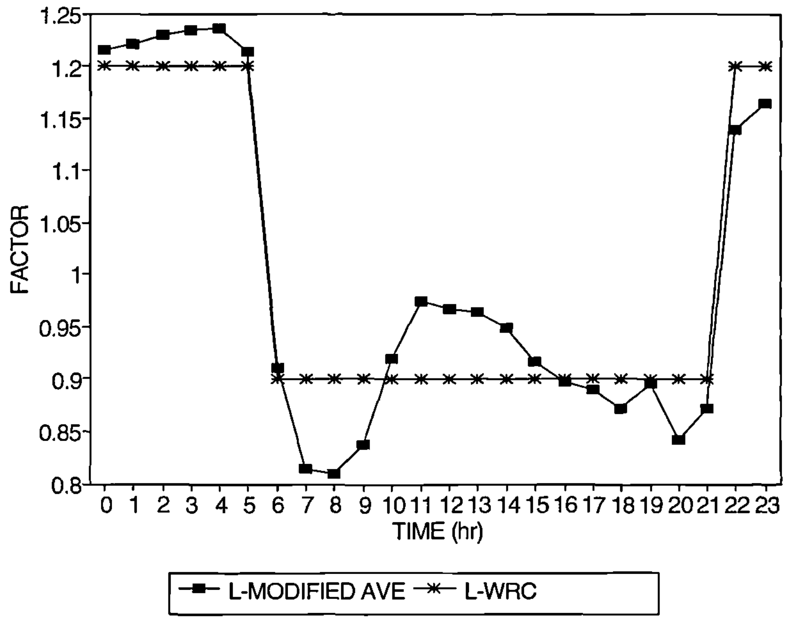

Figure 3.4: Average modified leakage profile for 19 zones and 'WRc' Type 5 (standard) profile.

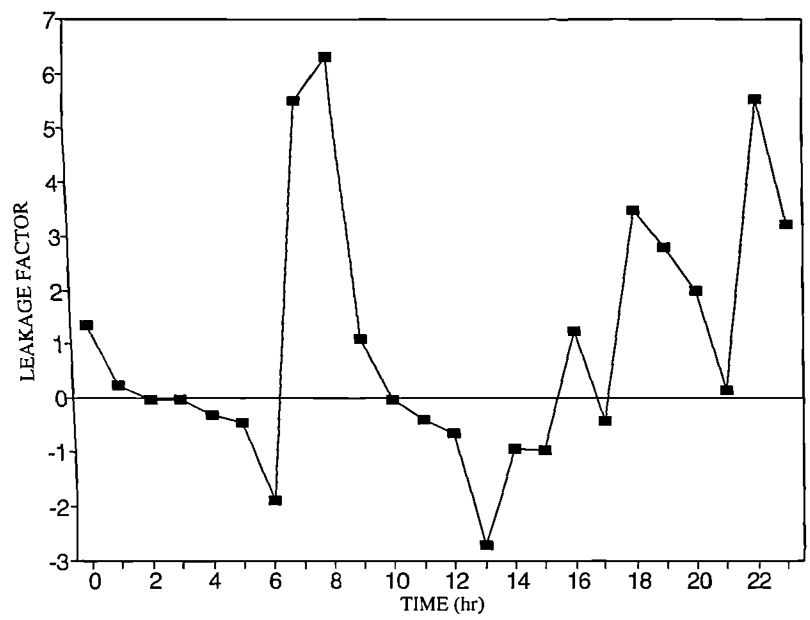

Figure 3.5: Negative values and illogical pattern for modified leakage in zone NT031.

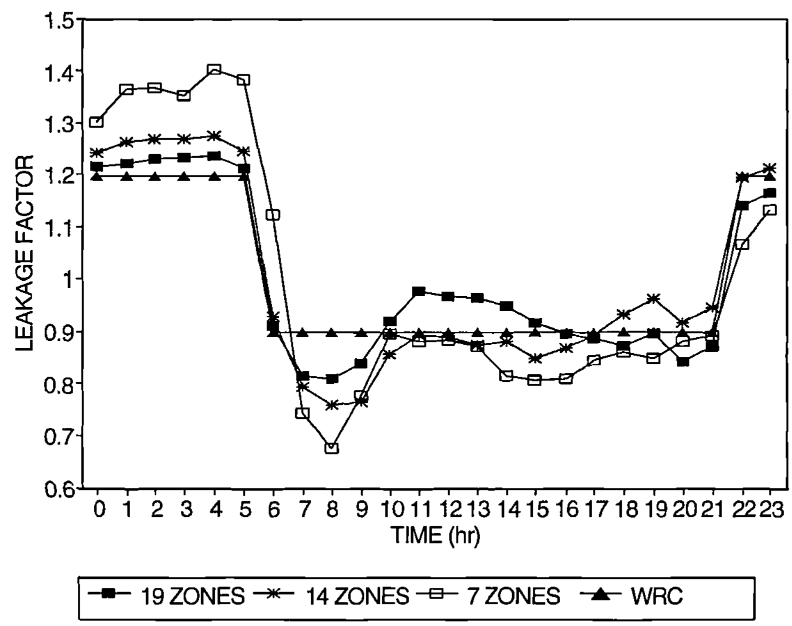

Figure 3.6: Modified leakage profiles for different selected subgroups of zones.

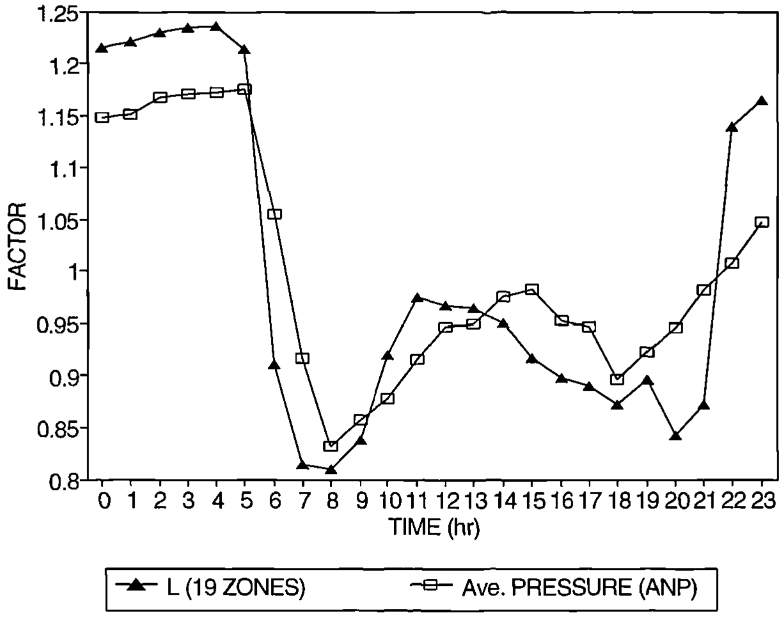

Figure 3.7: The average modified leakage profile for 19 zones together with the average network pressure profile for 16 zones.

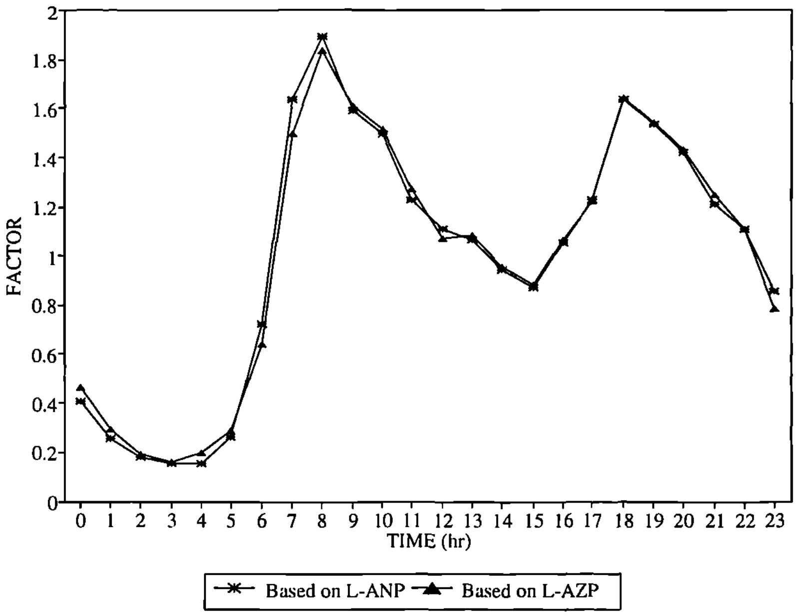

Figure 3.8: Average domestic demand profiles for 18 and 22 zones, arising from imposition of average network and zonal pressure dependent leakage profiles.

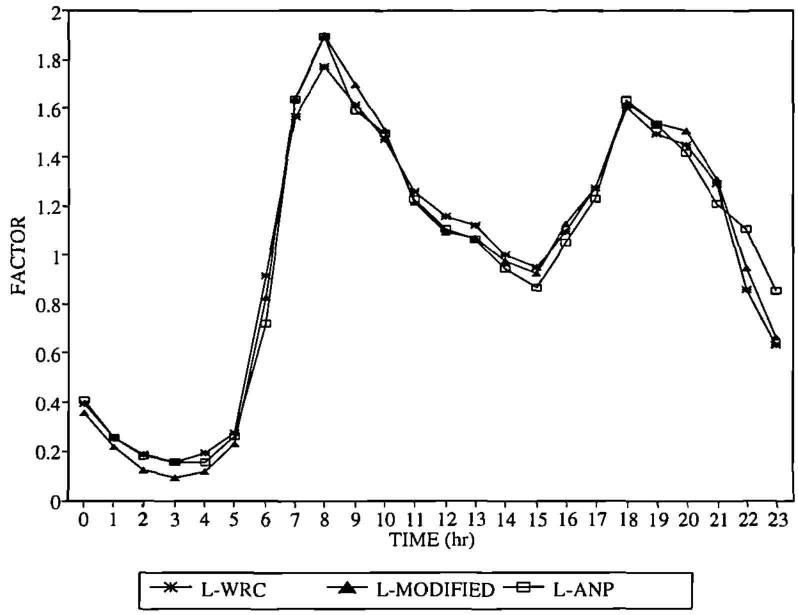

Figure 3.9: Average domestic demand profiles arising from adoption of different average leakage profiles.

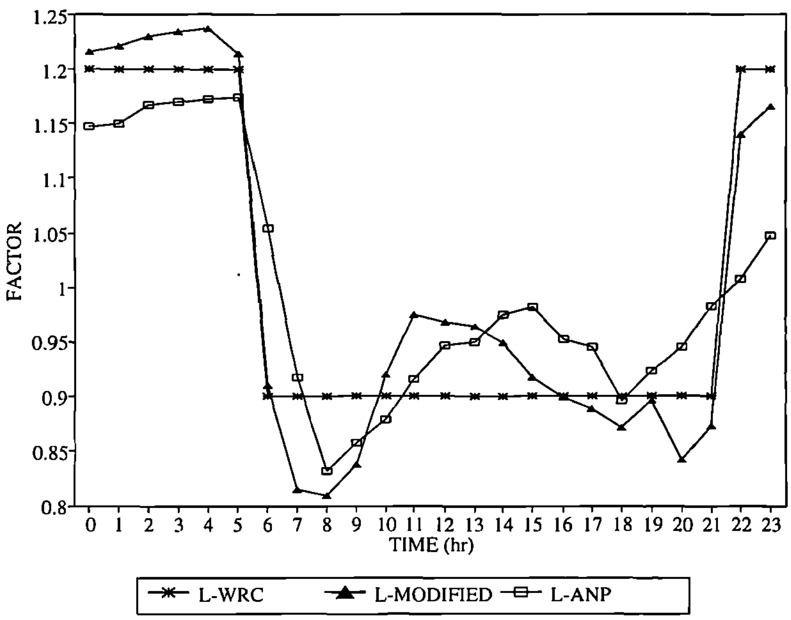

Figure 3.10: The applied leakage profiles to obtain the domestic demand profiles of Fig 3.9.

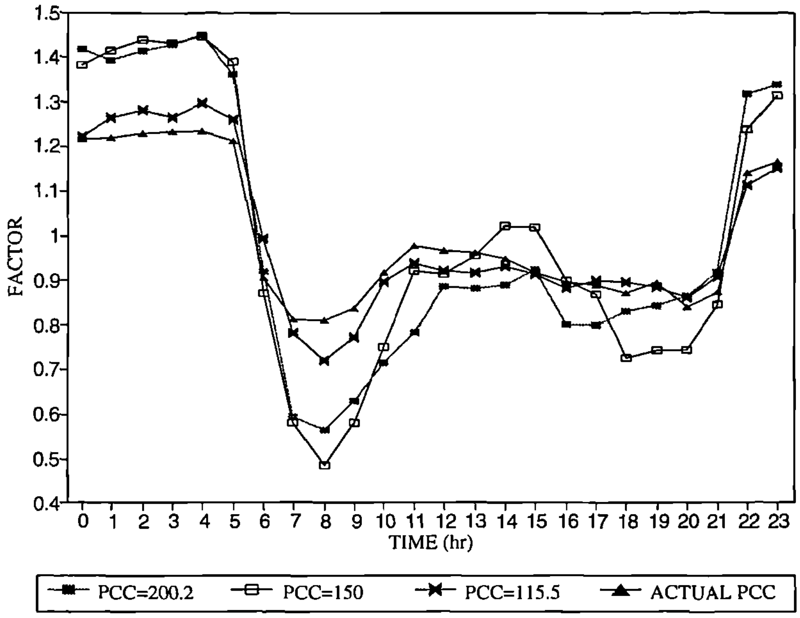

Figure 3.11: Leakage profiles based on different PCC values.

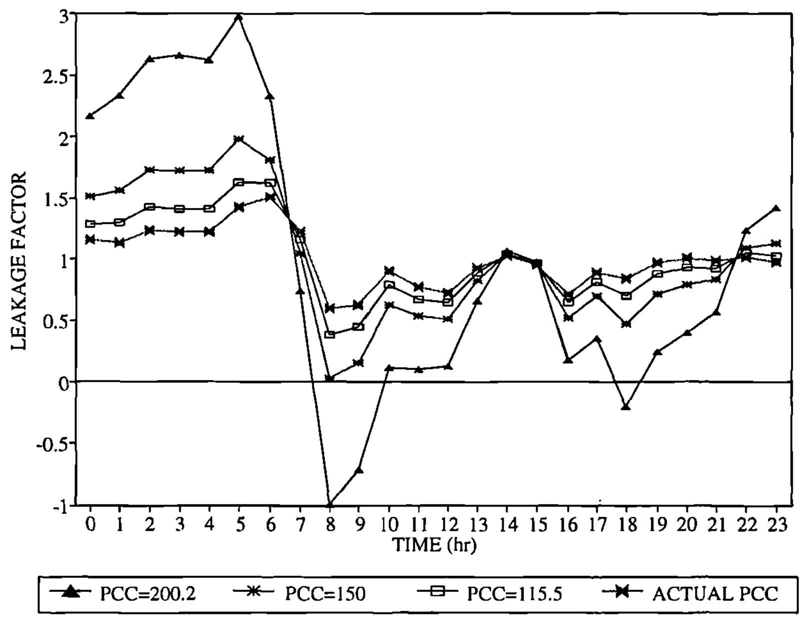

Figure 3.12: Leakage profile based on variations of PCC for zone NL035 (group (iii)).

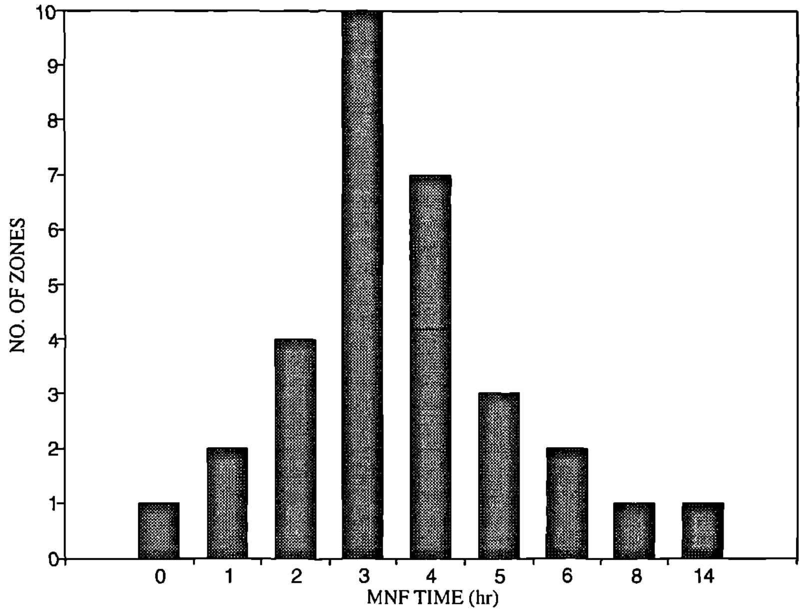

Figure 3.13: Variations of MNF times in the studied area.

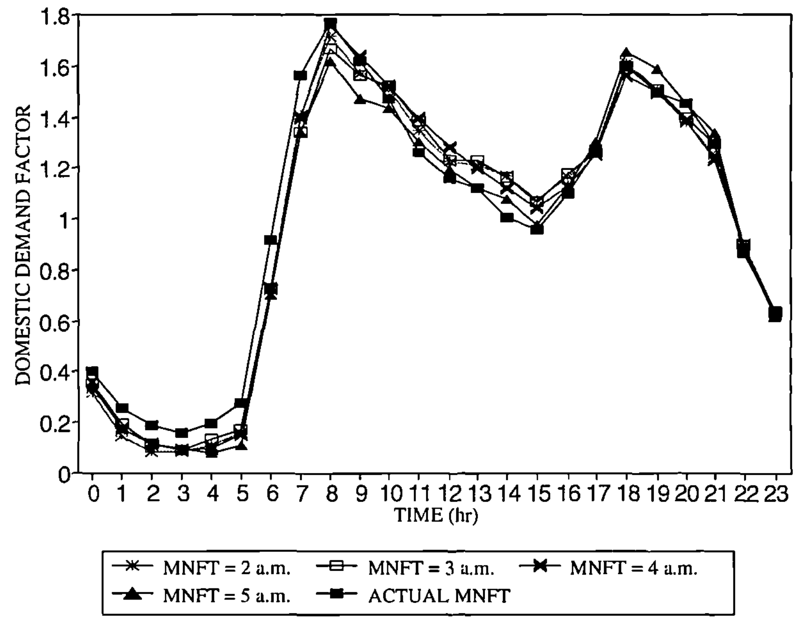

Figure 3.14: Average domestic demand profiles based on variations of MNF time arising from imposition of L-WRC.

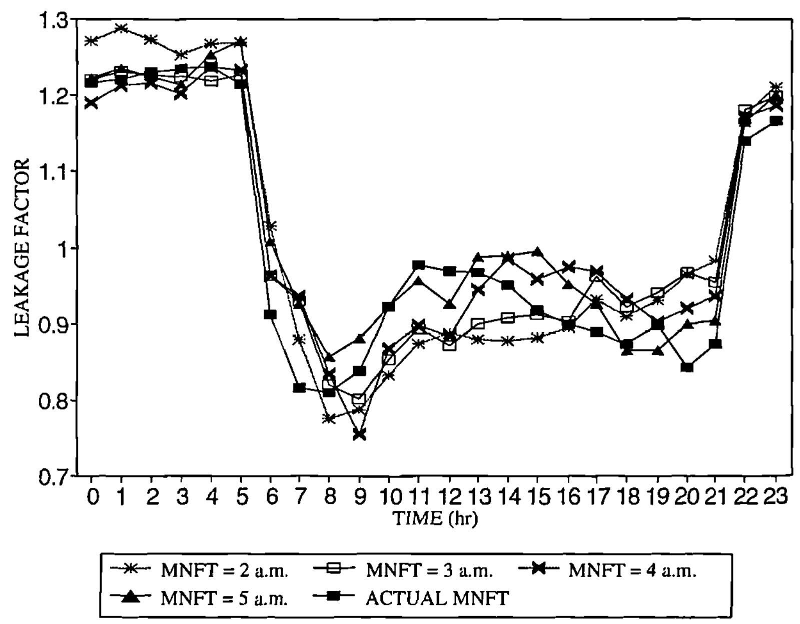

Figure 3.15: Modified leakage profiles based on variations of MNF time arising from imposition of the average domestic profile of Fig. 3.14.

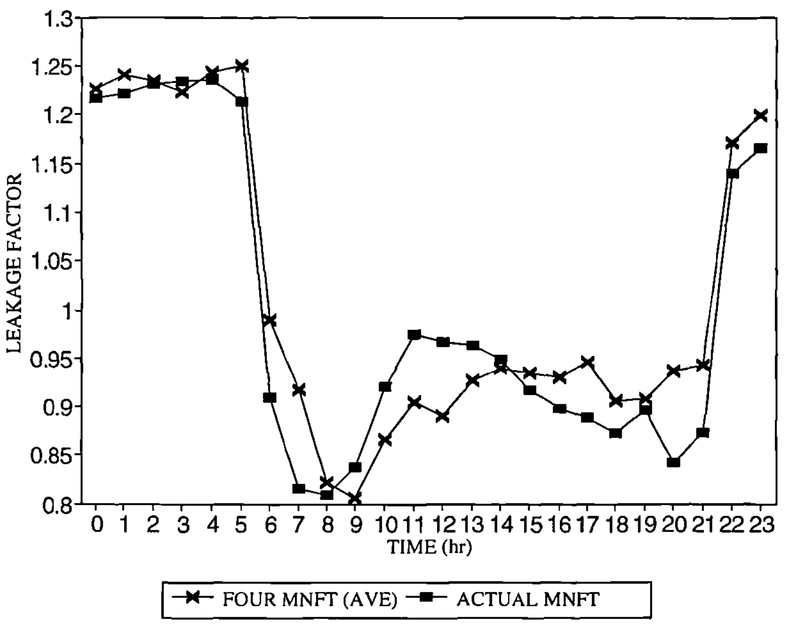

Figure 3.16: Average leakage profiles based on the four different MNF times and L-Modified based on the actual MNF time.

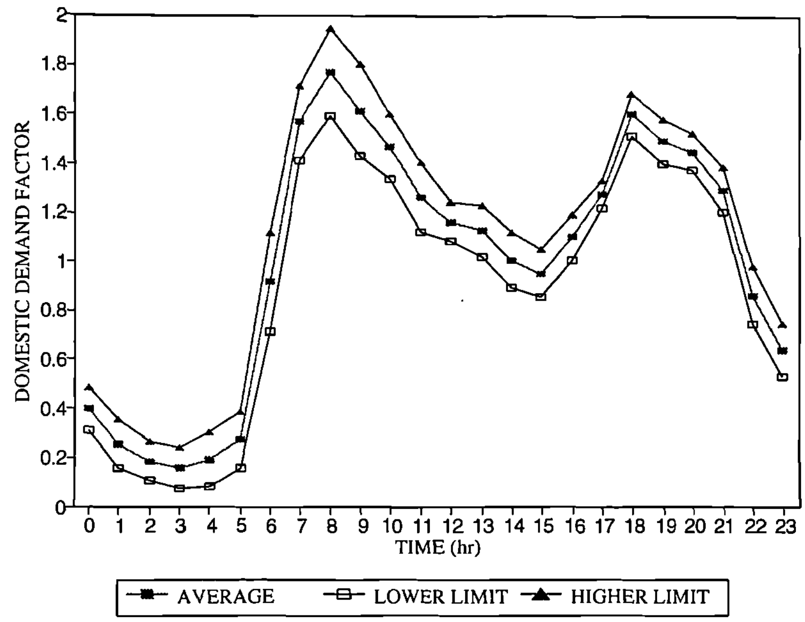

Figure 3.17: 95% confidence bound for domestic demand profile.

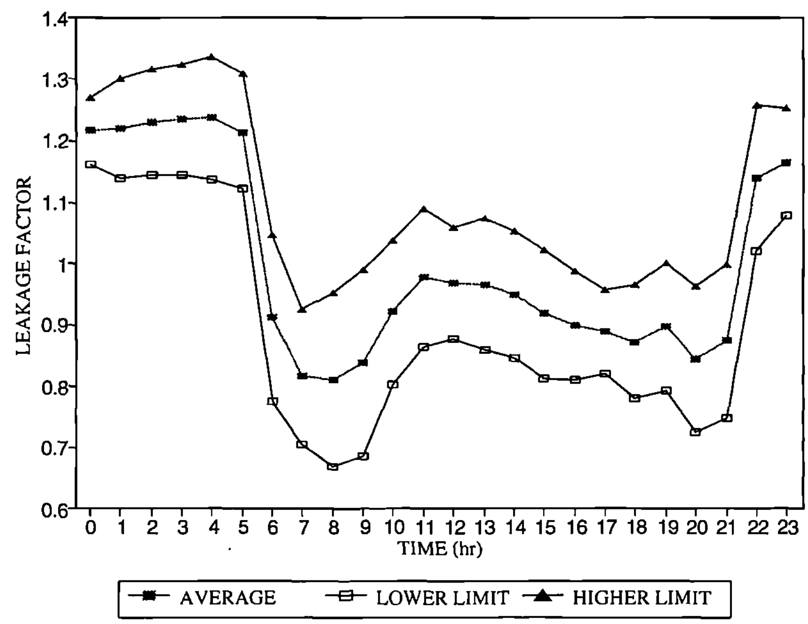

Figure 3.18: 95% confidence bound for leakage profile.

CHAPTER 4

WATER DISTRIBUTION NETWORK ANALYSIS

4.1 INTRODUCTION

The mathematical modelling of water distribution systems has been well established over past decades for the design of new networks and for operational control of existing systems. Solving the governing nonlinear hydraulic equations of distribution networks needs some mathematical techniques to obtain the required efficiency and accuracy of the solutions. Pressure and flow are two major unknowns in these systems. Besides steady state analysis which is equivalent to one snapshot with fixed nodal demands and constant water levels in reservoirs, the operational control of the system under time varying conditions is also required. The assumption of constant water demands and reservoir water levels can be valid for a short period. However, in actual practice none of them are constant. The extended period simulation (dynamic analysis) of water supply networks tends to analyze the system over a long period. The objectives are the knowledge of the impact of demand fluctuation on the level of the service and evaluation of the adequacy of the storage for the expected increase in total demand.

This chapter reviews the existing methodologies to simulate the steady state and extended period simulation of water distribution networks. The governing hydraulic equations and head flow relations of some major components are also presented.

4.2 HEAD-FLOW RELATIONSHIPS OF NETWORK COMPONENTS

Each water supply network includes hydraulic elements such as pipes, pumps, valves, reservoirs, etc. The characteristics of each element can be described by the nodal heads and flows in that element and for which the latter itself is related to the head differences across the component. The head-flow relationship for some of these elements are described next.

4.2.1 Pipes

The head flow relation through pipes are usually expressed by the Hazen-Williams or the Darcy-Weisbach equations. The Hazen-Williams equation which is one of the most commonly used ones may be written as:

$$h_{ij} = H_i - H_j = K_{ij}\, Q_{ij}^n \tag{4.1}$$

in which h_{ij} is head loss, Q_{ij} and K_{ij} are flow rate and resistance coefficient of pipe ij, respectively. H_i and H_j are heads at nodes i and j, respectively. n is a coefficient usually between 1.5 and 2. Herein n is considered to be equal to 1.852. Value of K_{ij} can be obtained as follows:

$$K_{ij} = \frac{\alpha \,.\, L_{ij}}{CHW_{ij}^{1.852} \,.\, D_{ij}^{4.87}} \tag{4.2}$$

where α = 10.675 in SI units, L_{ij} = length of pipe ij (m), CHW_{ij} = Hazen-Williams coefficient for pipe ij and; D_{ij} = diameter of pipe ij (m). Values of CHW_{ij} which depend on the pipe conditions (e.g. material and age, etc.) can be found in many text books (see e.g. Jeppson 1976). Also most recently Savic and Walters (1997) have presented the basis of the Hazen-Williams equation and summarised a range of different values for the α coefficient, used in the literature. Therefore, the head-flow relation of pipe may be written as:

$$Q_{ij} = \left(\frac{|H_i - H_j|}{K_{ij}}\right)^{(\frac{1}{n})} sgn\,(H_i - H_j) \tag{4.3}$$

in which

$$sgn\,(H_i - H_j) = 1 \qquad ;\ if\ H_i \succ H_j \tag{4.4a}$$

$$sgn\,(H_i - H_j) = 0 \qquad ;\ if\ H_i = H_j \tag{4.4b}$$

$$sgn\ (H_i - H_j) = -1 \quad ;\ if\ H_i < H_j \tag{4.4c}$$

In this research the Hazen-Williams formulation is used. For more information about the Darcy-Weisbach equation the reader is referred to Jeppson (1976) and Walski (1984). The head-flow relationship for the other network components (e.g. pumps, valves, reservoirs, etc.) can be incorporated with the Hazen-Williams formulation in network analysis as follows.

4.2.2 Pumps

The head flow relationship of a pump can be typically approximated by a parabolic curve as:

$$H_p = H_j - H_i = A_p Q_p^2 + B_p Q_p + C_p \tag{4.5}$$

where constant values of the A_p, B_p, and C_p are empirically determined (usually set by the manufacturer), however, they are dependent on the pump speed if a variable speed pump be used. H_p is the head lift across the pump and Q_p is the flow delivered by pump from node i to node j. H_i and H_j denote heads at upstream and downstream nodes of the pump, respectively. The flow can be thus expressed in terms of the nodal heads as follows:

$$Q_p = \frac{-B_p \pm \left(B_p^2 - 4A_p\,[C_p - (H_j - H_i)]\right)^{0.5}}{2A_p} \tag{4.6}$$

If the above equation results in a negative value of Q_p, the negative root is chosen and if there is no real root, Q_p is set to zero. Also, no flow can be delivered by the pump when it is switched off.

4.2.3 Non-Return Valves (NRV)

The head flow relation for a pipe fitted with a non-return valve which allows flow in one direction only is

$$Q_{ij} = (\frac{H_i - H_j}{K_{ij}})^{(\frac{1}{n})} \quad ; if \;\; H_i \succ H_j \tag{4.7a}$$

$$Q_{ij} = 0 \quad ; if \;\; H_i \leq H_j \tag{4.7b}$$

in which Q_{ij} expresses flow between nodes i and j.

4.2.4 Flow Control Valves (FCV)

The head flow relationship for a FCV is given as:

$$Q_{ij} = K_v \, (\frac{|H_i - H_j|}{K_{ij}})^{(\frac{1}{n})} \, sgn \, (H_i - H_j) \tag{4.8}$$

where K_v is a valve control parameter which varies from zero to one, to represent the various stages of closure.

4.2.5 Pressure Reducing Valve (PRV)

A pressure reducing valve produces a constant outlet pressure for a range of higher inlet pressures. For a PRV the head flow relationship is expressed as:

$$Q_{ij} = (\frac{|H_{prv} - H_j|}{K_{ij}})^{(\frac{1}{n})} \quad ; if \;\; H_j \leq H_{prv} \leq H_i \tag{4.9a}$$

$$Q_{ij} = (\frac{H_i - H_j}{K_{ij}})^{(\frac{1}{n})} \quad ; if \;\; H_j \leq H_i \prec H_{prv} \tag{4.9b}$$

$$Q_{ij} = 0 \quad ; if \;\; H_j \geq H_{prv} \tag{4.9c}$$

in which H_{prv} is the pressure reducing valve setting corresponding to the constant outlet head sought. In the above equation the pressure reducing valve is considered to be adjacent to node i. The presence of the PRV has no effect on the flow between nodes i and j when H_i drops below H_{prv}. Also, the pressure reducing valve acts as a non-return valve to prevent reverse flow from node j to node i.

4.3 STEADY STATE ANALYSIS

4.3.1 Basic Laws in Pipe Networks

The governing nonlinear equations for flow in water supply networks can be obtained by considering the two basic laws of fluid motion, continuity equation and conservation of energy.

4.3.1.1 Continuity Equation

Based on the continuity equation applied at each node, the inflows should be equal to the outflows.

$$(\sum_{ij \in IJ_j} Q_{ij})_{out} - (\sum_{ij \in IJ_j} Q_{ij})_{in} = Q_j \qquad \forall j = 1,\ldots, NJ \tag{4.10}$$

in which Q_j is the external output or input at node j, IJ_j denotes all links connected to node j and, NJ is number of nodes.

4.3.1.2 Conservation of Energy

Conservation of energy applied to each loop expresses that the cumulative head loss around a loop must be zero.

$$\sum_{ij \in IJ_L} h_{ij} = 0 \qquad ; L = 1,\ldots, NL \tag{4.11}$$

where IJ_L represents links of loop L and NL is number of loops. The conservation of energy may be expressed through consideration of a path between any two points on the loop. Along each path, the total head loss should be equal to the head difference between two end nodes of the path.

$$\sum_{ij \in IJ_p} h_{ij} = h_p \qquad \forall p = 1,\ldots, NPP \tag{4.12}$$

in which h_p is the total head loss along the path p, IJ_p represents links of path p and, NPP is number of paths.

4.3.2 Network Governing Equations

In analysis of water distribution networks, commonly three different systems of equations (or three alternative unknown variables) can be used namely 'Flow', 'Nodal' and 'Loop' equations. System equations combine the hydraulic characteristics of all elements and produce the nonlinear governing equation of the system. These three alternative systems of equations which are determined using one or both basic laws in pipe networks are briefly described next.

4.3.2.1 Flow Equations (Q- Equations)

Equations which are based on the (unknown) pipe flow rates are called Q- Equations. According to the two basic laws of continuity and conservation of energy, two linear and nonlinear sets of equations are produced. In any water supply network the following equation for the number of independent equations can be written

$$NP = (NJ-1) + NL \tag{4.13}$$

where NP is number of pipes. In Q- Equations the (NJ-1) linear continuity equations in terms of the elemental flows can be written at node j as follows:

$$\sum_{ij \in IJ_j} Q_{ij} + Q_j = 0 \qquad \forall j = 1,..., NJ \tag{4.14}$$

Equations which represent the conservation of energy at each independent loop L are

$$\sum_{ij \in IJ_L} f_{ij}\,(Q_{ij}) = 0 \qquad \forall L = 1,..., NL \tag{4.15}$$

in which f_{ij} is the nonlinear function expressing head loss in terms of flow in the hydraulic elements linking nodes i and j. Therefore, with the NL loop equations and the (NJ-1) continuity equations the total number of NP independent equations would be set up.

4.3.2.2 Nodal Equations (H- Equations)

A set of nonlinear equations in terms of the unknown heads at the nodes can be obtained as follows:

$$\sum_{ij \in IJ_j} f_{ij}\,(H_i - H_j) + Q_j = 0 \qquad \forall\, j = 1,\ldots,\, NJ \tag{4.16}$$

This equation is based on the continuity equation considering the flow in the network's hydraulic elements, as a function of head. Using fixed head nodes as known variables, solution of (NJ-NFH) independent nonlinear equations leads to determination of nodal heads and pipe flows. NFH is the number of fixed head nodes and normally is at least 1 in the H-Equations. Therefore, like the Q- Equations the (NJ-NFH) independent continuity equations can be solved.

4.3.2.3 Loop Equations (ΔQ- Equations)

This approach relates the system variables by a set of energy conservation equations around all loops. It involves first assuming pipe flow rates Q_{ij}, which satisfy the continuity equation at each node but may not satisfy conservation of energy around the loop. Then NL nonlinear equations for NL loops can be written in which a set of corrective flow rates (ΔQ_{ij}) is used in an iterative procedure to reduce the differences in head loss at each loop. The formulation can be presented as follows:

$$\sum_{ij \in IJ_L} f_{ij}\,(Q_{ij}^0 + \Delta Q_{ij}) = \Delta H_L \qquad \forall\, L = 1,\ldots,\, NL \tag{4.17}$$

where ΔH_L is the total head loss in loop L and Q^0_{ij} is the initial assumed value for flow rate at each element. The procedure should be continued until a pre-specified tolerance value is met by ΔH_L. At each iteration the updated flow rates will satisfy the (NJ-1) continuity equations.

The nodal equations (H- Equations) has been chosen as being most convenient in this research as it can readily incorporate the head-flow relations of all system elements and it does not require determination of loops.

4.4 SOLUTION METHODS

To solve any different set of system equations which are nonlinear, a numerical method is needed. Three numerical methods which are commonly used are described next.

4.4.1 Hardy-Cross Method

The most widely used method of analysing water distribution systems was developed by Cross (1936). This method can be used for Q-, ΔQ- and H- Equations. However, most text books have used it just for ΔQ- Equations. This method is commonly applied for hand calculation of simple networks. Also this method solves the composed equations successively but not simultaneously at each iteration. For H-Equations the method can be presented as follows:

1) Assume an initial head for each non fixed head node in the network.
2) For each node, calculate a head-correction to satisfy the flow continuity at that node.
3) For the updated nodal heads, repeat step 2 until achieving the required precision.

The residual flow at node j and iteration m, F_j^m, can be written as:

$$F_j^m = \sum_{ij \in IJ_j} f_{ij} \, (H_i^m - H_j^m) + Q_j \qquad \forall j = 1,..., NJ \tag{4.18}$$

in which H_j^m is the head value at node j in iteration m. Based on the Newton-Raphson iteration procedure, the head-correction at node j in iteration m can be calculated by

$$\Delta H_j^m = - \frac{F_j^m}{\left(\dfrac{\partial F_j}{\partial H_j^m}\right)} \tag{4.19}$$

Then the new head value at node j can be obtained by

$$H_j^{m+1} = H_j^m + \Delta H_j^m \tag{4.20}$$

in which H_j^{m+1} is the updated head value for iteration m+1 at node j. Because only one equation is dealt with at each time, each iteration involves the solution of Eq. (4.19) for each individual node. The final solution can be obtained by achieving a required precision for head-corrections, ΔH_j^m, or residual flow, F_j^m, at node j. The application of this method for Q- or ΔQ- Equations can be seen in Jeppson (1976); Wood and

Reys (1981) and Walski (1984).

4.4.2 Linear Theory Method

This method was first proposed by Wood and Charles (1972) for the Q- Equations in which external flows were known. Later Isaacs and Mills (1980) used it for solution of the H- Equations. They stated that for problems which consist of a number of known nodal heads like reservoirs, a linear theory method based on nodal heads as unknowns should prove more efficient. The nonlinear head flow relationship for pipes can be linearized as follows:

$$Q_{ij}^{m+1}|Q_{ij}^{m}|^{(n\ -\ 1)} = \frac{(H_i^{m+1}\ -\ H_j^{m+1})}{K_{ij}} \tag{4.21}$$

Knowing all Q_j^m and H_j^m values at iteration m, a linearized pipe head-flow relation can be written as:

$$Q_{ij}^{m+1} = \frac{(H_i^{m+1}\ -\ H_j^{m+1})}{K_{ij}|Q_{ij}^{m}|^{(n\ -\ 1)}} \tag{4.22}$$

Substitution of the above equation in the continuity equation, Eq. (4.10), leads to a set of linear equations whose solution provides values of H_j^{m+1}. Using these updated head values the new pipe flows Q_{ij}^{m+1} can be obtained. The linear theory method may be outlined as follows:

1) Assume an initial guess for pipe flows Q_{ij}^m.
2) Linearize the continuity equations for nodal flows.
3) Solve the linearized nodal flow continuity equations to obtain new values for the unknown nodal heads H_j^{m+1}.
4) Compute the new pipe flows Q_{ij}^{m+1}, then return to step 2 until achieving the required precision.

The first assumed flow values at step 1 do not need to satisfy continuity equation and their values have no effect on the convergence of the solution (Isaacs and Mills 1980). Also, the obtained flow values at each iteration will satisfy continuity and the iterative procedure tends toward a solution that will also satisfy the nonlinear head-flow relations of each component. The procedure would be continued until a pre-specified tolerance value is achieved for differences of Q_{ij} in the two successive iterations m and m+1.

4.4.3 Newton-Raphson Method

The Newton-Raphson method which solves all the nodal continuity equations simultaneously, was originally presented by Martin and Peters (1963). Shamir and Howard (1968) and; Zarghamee (1971) developed the method to incorporate pumps and valves. The former also used it for mixed unknown variables e.g. diameters and nodal heads. Epp and Fowler (1970) used this method for loop equations and also Lemieux (1972); Lam and Wolla (1972); Donachi (1974); Rao and Bree (1977) and Nogueira (1993) applied some techniques to modify, improve and develop the method, alternatively.

In respect of the continuity equation at each node, unbalanced residual flow at each node (see Eq. 4.18) can be written as:

$$\underline{F}(\underline{x}) = \underline{0} \tag{4.23}$$

in which $\underline{F}$ is a vector of nodal residuals and $\underline{x}$ is vector of unknowns at each node. Considering nodal heads H as unknowns, the Newton-Raphson formula can be written as:

$$\underline{H}^{m+1} = \underline{H}^{m} - (\underline{J}^{m})^{-1}\underline{F}(\underline{H}^{m}) \tag{4.24}$$

in which $\underline{H}^m$ and $\underline{H}^{m+1}$ are vectors of the nodal head values at two consecutive iterations of m and m+1 and $(\underline{J}^m)^{-1}$ is the inverse of the Jacobian matrix, $\underline{J}^m$. For a network with NVH (= NJ-NFH) unknown non fixed head nodes, the Jacobian matrix $\underline{J}^m$ with NVHxNVH dimension can be written as follows:

$$\underline{J}^m = \begin{vmatrix} \frac{\partial F_1}{\partial H_1^m} & \cdots & \frac{\partial F_1}{\partial H_{NVH}^m} \\ \cdots & \cdots & \cdots \\ \frac{\partial F_{NVH}}{\partial H_1^m} & \cdots & \frac{\partial F_{NVH}}{\partial H_{NVH}^m} \end{vmatrix} \tag{4.25}$$

in which

$$\frac{\partial F_i}{\partial H_j^m} = -\left(\frac{1}{n}\right)\left(\frac{|H_i^m - H_j^m|^{\frac{1}{n}-1}}{K_{ij}^{\left(\frac{1}{n}\right)}}\right) = \frac{\partial F_j}{\partial H_i^m} \qquad ; i \neq j \tag{4.26a}$$

$$\frac{\partial F_i}{\partial H_i^m} = -\sum_{j \in J_i} \frac{\partial F_i}{\partial H_j^m} \tag{4.26b}$$

where J_i denotes all nodes j in the vicinity of node i which are connected to it directly. It is obvious that $\partial F_i / \partial H_j$ and $\partial F_j / \partial H_i$ are zero if there is no connection between node i and j. Because inversion of $\underline{J}$ is computationally expensive, the vector of nodal-head corrections may be obtained by the following equation

$$\underline{J}^m \underline{\Delta H}^m = \underline{F}(\underline{H}^m) \tag{4.27}$$

Finally the updated values of head can be calculated as:

$$\underline{H}^{m+1} = \underline{H}^m - \underline{\Delta H}^m \tag{4.28}$$

Therefore, to solve the (NJ-NFH) simultaneous nonlinear equations the following steps could be followed.

1) Assume an initial guess for head value at node j, H_j^m.
2) Evaluate the residual flow vector and the Jacobian matrix, $\underline{F}(\underline{H}^m)$ and $\underline{J}^m$.
3) Calculate the vector of nodal head corrections $\underline{\Delta H}^m$ to obtain the updated nodal heads $\underline{H}^{m+1}$.
4) Repeat steps 2-3 to achieve the required precision for ΔH_j^m or F_j^m.

Using the final solutions for nodal head values, flow rates in pipes and at fixed head nodes can be obtained.

4.4.4 Computational Characteristics

The advantages and disadvantages of the above alternative methods for steady state analysis of water supply networks have been reviewed by several researchers (e.g. Jeppson 1976; Wood and Rays 1981 and Wood and Funk 1993). They are summarised below.

The Hardy-Cross algorithm is easy to implement but it is not computationally efficient because it solves one equation at each time. Also it is not suitable and reliable for complex and large networks since it converges slowly or may not converge at all (Jeppson 1976).

Of the Newton Raphson and Linear Theory methods researchers have reported good convergence for both (Lemieux 1972; Rao and Bree 1977; Isaacs and Mills 1980; Wood and Rayes 1981; Nielsen 1989; Altman and Boulos 1992 and Wood and Funk 1993). However, the former needs a proper initial guess for unknown values. The H-Equations approach is recommended for the Newton-Raphson method, which also can save more computational time by considering symmetrical and sparsity features of Jacobian matrix using particular techniques (see Tinney and Walker 1967 and Chandrashekar and Stewart 1975).

Unlike the other methods, the Linear Theory method does not need to initially guess values and its convergence is fast (Wood and Charles 1972). Also it is simple and no derivation is needed. Isaacs and Mills (1980) stated that a method based on flows is more suitable for problems of linear theory and can be expected to converge more rapidly than one based on nodal heads. However, they observed that the results of their proposed procedure based on nodal heads can be used with reasonable efficiency to solve these problems.

In this research the Newton-Raphson method is chosen to analyze the water supply system equations because head-flow relationships for hydraulic elements can be easily included in the terms of the Jacobian matrix and it is more convenient for the specific purposes of this research (i.e. inclusion of pressure dependency of demand) which will be described in the next chapter.

4.5 EXTENDED PERIOD SIMULATION OF WATER SUPPLY NETWORKS (DYNAMIC ANALYSIS)

4.5.1 Introduction

The previous sections of this chapter discussed steady state analysis of water supply networks. Nodal demands and reservoir water levels are assumed to remain constant in this kind of analysis. These assumptions are valid for a short period and for certain applications, such as modelling the network operation at the design stage for predicted peak demands. However, in the real situation over a long period, neither the nodal demands nor the reservoir water levels remain constant. As an example, the diurnal profile of different types of demand through 24 hours of a day can be seen in Chapter 2. To guarantee an adequate level of service to the customers under the changing demand patterns, proper operational planning of the water system is necessary. Maintenance of pressures and flow rates, with proper management of the storage to balance the supply and distribution of water are two main criteria which should be achieved (Rao and Bree 1977; Bhave 1991). To obtain these objectives the analysis of the system over a period of 24-48 hours under varying demand conditions, reservoir water levels and boundary conditions should be performed. The necessity of dynamic analysis of water supply networks was recognised by Shamir and Howard (1968); Tart (1973) and Demoyer et al. (1973) and some alternative methodologies for extended period simulation were proposed by Rao and Bree (1977) and Bhave (1991), these are described next.

4.5.2 Predictor-Corrector Iterative Procedure

Rao and Bree (1977) were perhaps the first to present a comprehensive methodology for extended period simulation of water distribution systems. This technique consists of a sequence of static solutions that are performed at pre-specified time intervals (usually between 1/2 to 2 hours). The simulation comprises the following steps.

1) At time step t, the reservoir levels and volumes $h_{rs}(t)$ and $v_{rs}(t)$, $\forall$ $rs \in RS$ and nodal demands $Q_j^{req}(t)$, $\forall$ $j \in NJ$, are known.

2) Perform a static solution at time step t to determine heads and flow rates and $q_{rs}(t)$, flow rates to or from the reservoirs.

3) Assuming $q_{rs}(t)$ is constant in the interval (t,t+1), the reservoir depletion is

$$Q_{rs}(t,t+1) = q_{rs}(t)\ \Delta t \tag{4.29}$$

in which Δt = time interval between t and t+1.

4) Compute the net outflow from the reservoirs and boundary elements BE as follows:

$$Q_{rs} = \sum_{rs \in RS} q_{rs}(t)\ \Delta t \tag{4.30}$$

$$Q_{be} = \sum_{be \in BE} q_{be}(t)\ \Delta t \tag{4.31}$$

where BE is the set of boundary elements like booster pumps which bring water to the network from external sources such as neighbouring pressure zones.

5) From the load curve obtain the average network demand for interval of Δt, D(t, t+1).

6) Calculate the predicted error as:

$$E_p(t,t+1) = \sum_{rs \in RS} q_{rs}(t)\ \Delta t + \sum_{be \in BE} q_{be}(t)\ \Delta t + D(t,t+1) \tag{4.32}$$

7) Allocate this error to the (rs)th reservoir in proportion to the flow rates

$$e_{rsp}(t,t+1) = \frac{q_{rs}(t)}{\sum_{rs \in RS} q_{rs}(t)}\ E_p(t,t+1) \tag{4.33}$$

8) Predict the reservoir volumes at time (t+1)

$$v_{rsp}(t+1) = v_{rs}(t) + q_{rs}(t)\ \Delta t + e_{rsp}(t,t+1) \tag{4.34}$$

9) From the head-volume curve of each reservoir, calculate predicted reservoir level $h_{rsp}(t+1)$, using $v_{rsp}(t+1)$.
10) Perform a network balance to determine $q_{rs}(t+1)$ for all $rs \in RS$, at time step (t+1), given h_{rsp}, D, H_{be} and v_{rsp}, in which H_{be} is head at boundary elements, BE.
11) Check pre-set switch points for valves or pumps. If any were switched in time interval (t,t+1), go to step 16, otherwise calculate a correction error as:

$$E_c(t,t+1) = \sum_{rs \in RS} [q_{rs}(t) + q_{rs}(t+1)]\frac{\Delta t}{2} + \sum_{be \in BE} [q_{be}(t) + q_{be}(t+1)]\frac{\Delta t}{2} + D(t,t+1) \tag{4.35}$$

12) Allocate the computed error to each reservoir as:

$$e_{rsc}(t,t+1) = \frac{q_{rs}(t) + q_{rs}(t+1)}{\sum_{rs \in RS} q_{rs}(t) + \sum_{rs \in RS} q_{rs}(t+1)}\ E_c(t,t+1) \tag{4.36}$$

13) Compute the corrected reservoir volumes as:

$$v_{rsc}(t+1) = v_{rs}(t) + [q_{rs}(t) + q_{rs}(t+1)]\frac{\Delta t}{2} + e_{rsc}(t,t+1) \tag{4.37}$$

14) Obtain the corrected reservoir levels $h_{rsc}(t+1)$ from the head-volume curve.
15) Compare the differences between the predicted and corrected reservoirs levels with a permissible tolerance value for each reservoir rs, λ_{rs}.

$$\mathit{If}\ |h_{rsp}(t+1) - h_{rsc}(t+1)| \le \lambda_{rs} \tag{4.38}$$

then set $h_{rsc}(t+1)$ and $v_{rsc}(t+1)$ to $h_{rs}(t+1)$ and $v_{rs}(t+1)$, respectively. The predictor-corrector integration step should be repeated for the same time step next.
16) In the occurrence of a pump or valve switching at intermediate time t′, where (t ≤ t′ ≤ t+1), separate the time interval to two intervals (t, t′) and (t′, t+1), then proceed with the method as before.

4.5.3 Direct Method

The above methodology needs to evaluate two separate predicted and corrected values for head and flow at storage elements. However, Bhave (1988) showed that it is possible to obtain the steady state analysis for time (t+Δt) directly from the steady state analysis of time t, by obtaining a relationship between $h_{rs}(t+\Delta t)$ and $q_{rs}(t+\Delta t)$ and use it in the analysis. This avoids the predictor-corrector iterative procedure. From the known head-volume curve of each reservoir rs, $v_{rs} = f_{rs}(h_{rs})$, variation in reservoir volume can be obtained as follows:

$$\Delta v_{rs} = f'_{rs}(h_{rs})\ \Delta h_{rs} \tag{4.39}$$

in which Δv_{rs} is the change in volume of reservoir rs equivalent to change of water level h_{rs}, and $f'_{rs}(h_{rs})$ is the first derivation which represents the cross section area of the reservoir rs at h_{rs}. Therefore, the reservoir head at the updated time interval is

$$h_{rs}(t+1) = h_{rs}(t) + \frac{\Delta v_{rs}(t,t+1)}{f'_{rs}(h_{rs}(t))} \tag{4.40}$$

Detailed application of the Direct method, which has many of steps similar to the procedure of Rao and Bree (1977), can be seen in the Bhave (1988, 1991).

For this research an extended period simulation model has been developed based on the predictor-corrector procedure to synthesise the 24-hour variation in levels of service and reservoir variations. Applications of this programme will be evaluated in the following chapters.

CHAPTER 5

HEAD DRIVEN SIMULATION OF WATER SUPPLY NETWORKS

5.1 INTRODUCTION

In the recent past, the subject of analysis and optimal design of water supply networks consisting of a number of sources, pipes, valves, pumps and reservoirs has drawn the attention of several researchers. These analyses usually have as their objective the need for an efficient technique to compute the pressure and flows in a defined network at defined levels of computational accuracy, given the characteristics of the network such as the pipe lengths, diameters and friction factors as well as the hydraulic characteristics of all ancillary plants such as pumps and valves.

Most network simulation models currently used in engineering practice are based on the conventional Demand Driven Simulation Method (DDSM). They assume that nodal outflows are fixed and are provided regardless of network pressures. The assumption simplifies the mathematical solution of the problem but is not always appropriate because it is clear that the amount of outflow at nodal outlets depends on the actual network pressures. If the pressure falls below a minimum required level, due to some critical events in the system such as mechanical and hydraulic failure or excess demand, the flow provided to consumers will be significantly reduced. Although some nodes may be able to completely satisfy their demands, others may meet the demand partially while the rest may completely fail and may not provide any water at all. The assumption that nodal consumptions are fixed irrespective of the network pressures is therefore valid only under normal conditions when the pressures can be expected to be adequate to satisfy the stipulated demands. If the operation of the system is simulated under pressure-critical conditions, the relationship between pressure and outflow should, therefore, be taken into account if the simulation results are to be realistic (Germanopoulos 1985; Germanopoulos et al. 1986; Reddy and

Elango 1989; Lumbers 1996). This kind of analysis in which the relationship between nodal outflow and pressure is explicitly considered and is referred to hereafter as the Head Driven Simulation Method (HDSM).

There are a number of circumstances, in water supply and irrigation networks where the assumption of fixed nodal demand is unsatisfactory. In water supply networks, problems such as leakage modelling are also pressure dependent. Most network operators are well aware that if they reduce the pressure in the distribution system, the total water consumption will be reduced. This leads to the standard practice of reducing night pressure in order to control system leakage (see Section 2.7). Also, fire protection and irrigation systems and garden watering, etc. are based on sprinklers which deliver water in an amount that is pressure dependent (Sterling and Bargiela 1984; Lonsdale 1985; Jowitt and Xu 1990; Zepeda and Rojo 1991).

The terms 'outflow' and 'demand' should be clearly distinguished. Demand is the quantity of water required at the nodal outlet but outflow is the quantity which the network actually yields, this being influenced by the hydraulic characteristics of the network as a whole including the outflows at other nodes (Reddy and Elango 1989).

From the early 1980s different researchers have referred to the importance and necessity of considering the pressure dependency of nodal consumption in water distribution systems modelling from different points of view (e.g. Bhave 1981; Germanopoulos et al. 1986; Tanyimboh 1993; Lumbers 1996; etc.). For example, Tanyimboh (1993) and Tanyimboh and Templeman (1994) stated that reduced service (i.e. 0 < nodal outflow < demand) should be recognized and accounted for somehow and any shortfall in flow should be reflected by network reliability measures (see also Bhave 1981 and Jowitt et al. 1989). In addition, Chandapillai (1991) pointed out that in some developing countries where the water distribution systems operate intermittently, the lack of adequate pressure leads to substantially less discharge than the requirement (demand) and very short duration of supply. It is, therefore, necessary to develop a network analysis methodology that explicitly and automatically takes into account the variation of outflows with pressure.

The aim of this chapter is to present an analysis algorithm in which a realistic pressure-outflow relationship is incorporated directly into the main set of nonlinear hydraulic equations of the water distribution network. Through a number of examples, the accuracy of results and the computational efficiency of the methodology are discussed. The results suggest that the proposed head-driven simulation method (HDSM) can simulate networks with insufficient pressure in a realistic way (unlike DDSM) without any significant loss of computational efficiency (compared to DDSM). Furthermore, unlike the cumbersome nature of previous HDSM methods (Gupta and Bhave, 1996b; Chandapillai, 1991) the present formulation is easy to implement. First of all, the following section looks for the most relevant and convenient pressure-outflow relationship.

5.2 NODAL PRESSURE-OUTFLOW RELATIONSHIP

During the last decade, several equations have been suggested to describe the pressure dependency of nodal consumption (outflow). The major proposed relationships are assessed next.

5.2.1 Discontinuous Relationships

Goulter and Coals (1986); Cullinane (1986) and Su et al. (1987) suggested a zero-one relationship for head-outflow as below,

$$\frac{Q_j^{avl}}{Q_j^{req}} = 1 \quad ; if \;\; H_j \geq H_j^{des} \tag{5.1a}$$

$$\frac{Q_j^{avl}}{Q_j^{req}} = 0 \quad ; if \;\; H_j < H_j^{des} \tag{5.1b}$$

where Q_j^{avl} is the available outflow, Q_j^{req} is the required outflow (demand), H_j is the available pressure (head) and H_j^{des} is the required pressure at node j. Using this approach, total nodal demand is considered to be available when pressure is equal to or more than a minimum desired value and no flow would be available otherwise (see

Figure 5.1a).

A major disadvantage of this relationship is that it does not adequately represent the engineering realities of the problem (Cullinane et al. 1992), because it cannot account for the equivalent values of outflow when $H_j^{min} < H_j < H_j^{des}$. H_j^{min} is the absolute minimum head at node j below which no nodal outflow is available.

5.2.2 Continuous Relationship

This kind of relationship has been used in two different ways by earlier researchers, with or without imposing an upper limit for outflow.

5.2.2.1 Relationships without an upper limit for outflow

The following equation has been proposed for networks with completely uncontrolled outlets, i.e. with no upper limit on the outflow (Reddy and Elango 1989 and Salgado et al. 1993),

$$Q_j^{avl} = S_j \, (H_j - H_j^{min})^{\frac{1}{n_j}} \tag{5.2}$$

where $(H_j - H_j^{min})$ is the residual head, S_j is a constant dependent on the outlet characteristics, and n_j is an exponent (usually between 1.5-2). Thus, the actual flow can be significantly higher than the demand when the residual pressure at a node is more than the design head H_j^{des}, for which the outflow equals the demand, Q_j^{req} (Reddy and Elango 1989), see Figure 5.1b.

It seems that equations with no upper limit on outflow are more appropriate for those networks with completely uncontrolled outlets such as irrigation systems and some special cases like sprinkler valves operating under emergency conditions in water supply networks. Consequently, for water supply networks, this does not usually lead to a realistic representation of hydraulic behaviour of the system other than for description of the uncontrolled leakage elements. Because, if the network pressure keeps rising, the consumer outflow will not necessarily follow that increase as there is normally a limit to the total amount of water the consumers require at any given time (Germanopoulos 1985).

5.2.2.2 Relationships with an upper limit imposed for outflow

Bhave (1981, 1991) presented a head-outflow relationship, with imposed upper limit as below.

$$Q_j^{avl} = Q_j^{req} \qquad ; if \;\; H_j > H_j^{des} \tag{5.3a}$$

$$0 < Q_j^{avl} < Q_j^{req} \qquad ; if \; H_j = H_j^{des} \tag{5.3b}$$

$$Q_j^{avl} = 0 \qquad ; if \;\; H_j < H_j^{des} \tag{5.3c}$$

Naming critical the nodes which meet the minimum desirable head, H_j^{des}, he considered partial flow at those nodes. However, as can be seen this relationship cannot quantify the exact value of abstracted flow at each node in partial flow mode. Furthermore, it does not consider the absolute minimum head, H_j^{min}, and partial flow when $H_j^{min} < H_j < H_j^{des}$ (see Figure 5.1c). Therefore, outflows from Eqs. 5.3b and 5.3c are not realistic.

Germanopoulos (1985) asserted that the exact form of the pressure-consumption relationship for each network node will depend on the hydraulic configuration between the node and the consumers downstream. The basic characteristics that are expected in a pressure-consumption relation can be therefore displayed, (i.e. a fall in nodal outflow for pressures below a certain limit as well as a levelling out for higher pressures corresponding to the maximum flow that the consumers are likely to require). Consequently, Germanopoulos (1985) suggested a pressure-consumption relationship which was later used by Jowitt and Xu (1990) as

$$Q_j^{avl} = Q_j^{req} \left(1 - b_j \, e^{-c_j \left[\frac{H_j - H_j^{min}}{H_j^{des} - H_j^{min}} \right]} \right) \tag{5.4}$$

where b_j, c_j and (H_j^{des} - H_j^{min}) are constants for the particular node j. In the absence of detailed field data, b_j and c_j have been assumed as 10 and 5, respectively by Germanopoulos (1985) which lead to an approximation for the head-outflow relationship. However, these values are generally specific for each network and should be obtained for each region, separately. Therefore, $(H_j^{des} - H_j^{min})$ corresponds to the pressure that provides 93.2% of the nominal consumer demand Q_j^{req}, i.e. when available head, H_j, reaches the desired head, H_j^{des}, only 93.2% of Q_j^{req} is delivered. On the other hand, when $(H_j - H_j^{min}) = 0.46\ (H_j^{des} - H_j^{min})$, no flow is delivered, i.e. $Q_j^{avl} = 0$, see Figure 5.1d. Thus, the formula does not evaluate consumer flows properly.

Gupta and Bhave (1996b) modified the Germanopoulos (1985) equation as follows:

$$Q_j^{avl} = Q_j^{req} \left(1 - 10^{-c_j \left[\frac{H_j - H_j^{min}}{H_j^{des} - H_j^{min}}\right]}\right) \tag{5.5}$$

This equation (shown by Figure 5.1e) introduces a considerable amount of simplicity and clarity to the original relationship and consequently, its ease of use and general applicability has been increased (Tanyimboh and Tabesh 1997). In the modified equation, firstly, the values of coefficients b_j and c_j are changed and secondly the exponential form of main equation is changed to power of ten. Thus, $Q_j^{avl} = 0$ when $H_j = H_j^{min}$ and $Q_j^{avl} \rightarrow Q_j^{req}$ when $H_j \leq H_j^{des}$ and c_j becomes larger, thereby removing one of the main restrictions of the basic Germanopoulos relationship, Eq. (5.4) from which $Q_j^{avl} = 0.932\ Q_j^{req}$ when $H_j = H_j^{des}$ and $Q_j^{avl} = 0$ when $(H_j - H_j^{min}) = 0.46\ (H_j^{des} - H_j^{min})$. However, this modified approach also does not give a good representation of the network behaviour because when $H_j = H_j^{des}$ a lower c_j value gives Q_j^{avl} considerably less than Q_j^{req}, while when H_j is much less than H_j^{des}, a higher c_j value gives larger Q_j^{avl} values throughout and are practically equal to Q_j^{req} (see Gupta and Bhave 1996b).

In another approach, a parabolic head-outflow relationship shown graphically in Figure 5.1f, was suggested by Wagner et al. (1988b) and Chandapillai (1991) as follows:

$$Q_j^{avl} = Q_j^{req} \qquad ; if \ \ H_j \geq H_j^{des} \tag{5.6a}$$

$$Q_j^{avl} = Q_j^{req} \left(\frac{H_j - H_j^{\min}}{H_j^{des} - H_j^{\min}}\right)^{(\frac{1}{n_j})} \quad ; \; if \; H_j^{\min} < H_j < H_j^{des} \tag{5.6b}$$

$$Q_j^{avl} = 0 \quad ; \; if \; H_j \leq H_j^{\min} \tag{5.6c}$$

in which a value of 2 is applied for n_j. It can be seen that the above equation is able to quantify the outflow while the network is in reduced mode (i.e. $H_j^{min} < H_j < H_j^{des}$).

The available head (H_j) is determined from the hydraulic solution of the network. The minimum head (H_j^{min}) below which no flow can be discharged may be taken as the minimum outlet level in the locality served by the node. In the absence of field data it may be set equal to ground elevation. The desired head (H_j^{des}) below which the nodal demand cannot be totally satisfied might typically be about 14 to 15 m or more (Insurance Service Office 1980; Twort et al. 1994; UK/WI 1994-Report G). Under certain circumstances, the absolute minimum desired pressure is suggested to be 7 m (US Army Corps of Engineers 1984; OFWAT 1996). However, practical and physical requirements, for example topographical features, may dictate that pressure as high as 75 m must be tolerated at some properties (UK/WI 1994-Report G). The desired head can also be calculated through the following equation (Gupta and Bhave 1996b)

$$H_j^{des} = H_j^{\min} + K_j \, (Q_j^{req})^{n_j} \tag{5.7}$$

in which K_j is an empirical resistance factor at node j, with unit of s^2/m^5 when n_j = 2. In addition, the required outflow is normally taken from 9 l/min to 20 l/min (at peak daily time) at the stop tap (UK/WI 1994-Report G). Regarding the above demand values and the above-mentioned limits for desirable head values, K_j may be considered to vary in a range of 21000-100000 s^2/m^5.

Considering the required and absolute minimum pressure as 14 and 7 m respectively, Cullinane et al. (1992) used a fuzzy relationship between the nodal availability (equivalent to Q_j^{avl}/Q_j^{req}) and head similar to the cumulative normal distribution to represent variation of available outflow for residual pressure below the desired value, see Figure 5.1g. However, their lower limit of nodal availability seems to be questionable because when pressure reaches the absolute minimum value, 50% of required demand is still available. This would appear to be because of their consideration of an absolute minimum pressure required for proper system operation which results from hydraulic constraints on the operation of fire fighting equipment. However, as fire fighting operations would be infrequent, their formulation would appear to be a special case which, consequently, overestimates values of available outflow. (Also, see Twort et al. 1994).

Later Fujiwara and Ganesharajah (1993) proposed an expected served demand concept which took into account both insufficient heads and flows at individual nodes in the network. The relative effectiveness of nodal head (equivalent to Q_j^{avl}/Q_j^{req}), termed nodal hydraulic availability therein, was defined as a non-decreasing smooth function of head, taking values between zero and one, the values being zero below minimum head level and one above the desired head level. The approach further developed the availability concept in that at H_j^{min} availability was zero and at H_j^{des}, one. Furthermore, the nodal hydraulic availability during reduced service mode in which H_j is not fully satisfactory was defined as a differentiable function of head as follows:

$$\frac{Q_j^{avl}}{Q_j^{req}} = \frac{\int_{H_j^{min}}^{H_j} (H-H_j^{min})\,(H_j^{des}-H)\,dH}{\int_{H_j^{min}}^{H_j^{des}} (H-H_j^{min})\,(H_j^{des}-H)\,dH} \quad ;\ if\ H_j^{min} < H_j < H_j^{des} \tag{5.8}$$

Figure 5.1h shows the graphical representation of this formulation. Although the above equation can be applied to any network, it can be seen that it is not as straight forward as the Eq. (5.6b) and more computational effort is needed for its evaluation.

From comparison of all presented head-outflow relationships, the parabolic relationship can be concluded as the best for prediction of deficient-network performance (Gupta and Bhave 1996b). Herein Eqs. (5.6) is used to represent the head-outflow relationship because it is a continuous function with realistic upper and lower bounds for outflow. Also, it is not complicated, is easy to use and can represent behaviour of the system reasonably.

5.3 REVIEW OF ALGORITHMS FOR HEAD-DRIVEN NETWORK ANALYSIS

In the review of algorithms to analyze the hydraulic equations of the system including pressure dependency of demand, different approaches can be seen in the literature. Bhave (1981); Wagner et al. (1988b) and Gupta and Bhave (1996b) used a two-phase formulation. Thus, using a conventional demand-driven simulation the head value at each node was obtained. Then the respective head-outflow relationships of Eqs. (5.4b and 5.6b) were used, respectively, in the former and latter approaches to calculate the outflows for those nodes with head values less than the desired ones. In addition, the iterative scheme of Gupta and Bhave (1996b) repeats the above procedure until there are no significant changes in nodal outflows or pressures between successive iterations.

Calculating nodal heads by DDSM, a corrected nodal outflow was obtained by Chandapillai (1991) using the Newton-Raphson iterative formula, i.e.

$$(Q_j^{avl})^{m+1} = (Q_j^{avl})^m - \frac{H_j^{\min} + K_j\,[(Q_j^{avl})^m]^{n_j} - H_j^m}{n_j\,K_j\,[(Q_j^{avl})^m]^{n_j-1}} \tag{5.9}$$

in which $(Q_j^{avl})^{m+1}$ is the updated outflow for nodes with less than fully satisfactory pressure, H_j, in the range $H_j^{min} < H_j < H_j^{des}$ and $(Q_j^{avl})^m$ represents the value of nodal outflow at the previous iteration. Also, for nodes with $H_j < H_j^{min}$, $Q_j^{avl} = 0$ and for nodes with $H_j > H_j^{des}$, Q_j^{avl} = demand. Although this analysis incorporated the pressure dependency of demand, it was also a two-phase approach. The disadvantage of this method is that there is an intermediate step between iterations in which nodal pressures are checked and modified outflows calculated.

The algorithm of Fujiwara and Ganesharajah (1993) to calculate the available outflows at each node was based on an optimization procedure which maximized the sum of the available outflows over all demand nodes. The head-outflow relationship was incorporated by means of the nodal hydraulic availability approach of Eq. (5.8) while the other hydraulic characteristics of the system were considered as constraints. The disadvantage of the approach is that it involves the solution of a difficult-to-solve nonlinear programming problem which is computationally expensive.

Germanopoulos (1985) and Reddy and Elango (1989) presented a head-driven algorithm incorporated with the head-outflow relationships of Eqs. 5.4 and 5.2, respectively. Although they used a one phase approach to update the nodal heads at each iteration, because of the weaknesses of Eqs. 5.4 and 5.2 (see Section 5.2.2.2) they obtained approximate results.

To improve the weaknesses of the above-mentioned algorithms, a fully integrated algorithm incorporated with a realistic head-outflow relationship is clearly needed to carry out the pressure-dependent network analysis of water distribution networks. A fully integrated algorithm for head-driven simulation is presented in the next section.

5.4 STEADY STATE HEAD-DRIVEN ANALYSIS OF WATER DISTRIBUTION NETWORKS

The governing equations for flow in water supply networks can be set up by considering the basic physical laws, i.e. the equation of continuity applied at each node and conservation of energy applied to each loop or path.

Different methods of computation have been developed (e.g. Hardy Cross, Newton-Raphson and Linear theory, etc.) and many computer programs produced to solve the conventional network analysis problem (Shamir and Howard 1968; Epp and Fowler 1970; Zarghamee 1971; Lemieux 1972; Wood and Charles 1972 and Wood and Reyes 1981; etc.). In comparison with other solution methods, the Newton-Raphson method has good convergence characteristics (Lemieux 1972; Rao and Bree 1977; Wood and Funk 1993). (Also, see Section 4.4.4). Herein the Newton Raphson method has been chosen and the pressure dependency of demand is incorporated in the system of equations as shown in the following pages.

5.4.1 Main Equations

The continuity equation at each node j may be written as

$$(\sum_{ij \in IJ_j} Q_{ij})_{out} - (\sum_{ij \in IJ_j} Q_{ij})_{in} = Q_j \qquad \forall j = 1,..., NJ \tag{5.10}$$

where Q_{ij} is flow in pipe ij, IJ_j denotes all links connected to node j and NJ is the number of the nodes in the network. Using the Hazen-Williams equation for flow in a pipe, Eq. (5.10) becomes

$$F_j = [\sum_{ij \in IJ_j} (\frac{H_j - H_i}{K_{ij}})^{(\frac{1}{n})}]_{out} - [\sum_{ij \in IJ_j} (\frac{H_j - H_i}{K_{ij}})^{(\frac{1}{n})}]_{in} - Q_j^{avl} = 0 \tag{5.11}$$

in which, n=1.852 and F_j *represents the continuity equation for node* j. H_i and H_j are piezometric heads at nodes i and j. K_{ij} is resistance coefficient for pipe ij and Q_j^{avl} is the outflow of node j (see Section 4.2.1).

Other network components (e.g. pumps, valves and reservoirs, etc.) can be included in this equation in a similar way. The head-flow relation for some of these components were indicated in Chapter 4 (Sections 4.2.2-5).

5.4.2 Incorporation of Pressure Dependent Outflow in the Governing Equations

As concluded earlier from the various head-outflow relationships proposed in the literature, the Wagner et al. (1988b) approach (Eqs. 5.6a-c) has been chosen as a good representation of the pressure dependency of nodal outflows. Because available outflow is a function of available head, it seems to be more reasonable that this dependency be included in the main set of hydraulic equations throughout the analysis procedure.

The head-dependent outflow term can be added to the continuity equations of the system as follows, giving in general NJ equations in NJ unknowns,

$$F_j = \sum_{i=1}^{NJ_j} (\frac{|H_i - H_j|}{K_{ij}})^{0.54} sgn\,(H_i - H_j) + Q_j^{req} (\frac{H_j - H_j^{min}}{H_j^{des} - H_j^{min}})^{0.5} = 0 \tag{5.12}$$

in which $H_j^{min} \le H_j \le H_j^{des}$, NJ_j is the number of nodes directly connected to node j and

$$sgn\ (H_i - H_j) = 1 \qquad ;\ if\ H_i > H_j \tag{5.13a}$$

$$sgn\ (H_i - H_j) = 0 \qquad ;\ if\ H_i = H_j \tag{5.13b}$$

$$sgn\ (H_i - H_j) = -1 \qquad ;\ if\ H_i < H_j \tag{5.13c}$$

From Eqs. (5.6), the second term of Eqs. (5.12) is equal to Q_j^{req}, if $H_j \ge H_j^{des}$ and is zero when $H_j \le H_j^{min}$. Based on the Newton-Raphson method and choosing the piezometric nodal heads as unknown parameters, Eq. (5.12) would be solved by the following iterative formula (see Section 4.4.3),

$$\underline{J}^m\ \underline{\Delta H}^m = \underline{F}\ (\underline{H}^m) \tag{5.14a}$$

$$\underline{H}^{m+1} = \underline{H}^m - \underline{\Delta H}^m \tag{5.14b}$$

where $\underline{H}$ is the vector of unknown heads, the matrix $\underline{J}$ is the Jacobian of the set of equations, $\underline{\Delta H}$ is the vector of the respective changes in nodal heads and $\underline{F}$ is the vector of the respective values of the nodal continuity expressions, i.e. F_j, j=1, ..., NJ. The iteration number is denoted by m.

The elements of the Jacobian matrix for each nodal equation are given by

$$\frac{\partial F_j}{\partial H_i} = 0.54\ \left(\frac{|H_i - H_j|^{-0.46}}{K_{ij}^{0.54}}\right) \quad \forall j;\ \forall i:\ i \ne j \tag{5.15a}$$

$$\frac{\partial F_j}{\partial H_j} = -0.54 \sum \left(\frac{|H_i - H_j|^{-0.46}}{K_{ij}^{0.54}}\right) + \frac{0.5\ Q_j^{req}}{(H_j^{des} - H_j^{min})} \left(\frac{H_j - H_j^{min}}{H_j^{des} - H_j^{min}}\right)^{-0.5} \tag{5.15b}$$

The second term of Eq. (5.15b) representing Q_j^{avl} becomes operable when $H_j^{min} \le H_j \le H_j^{des}$ and it is zero otherwise, since Q_j^{avl} is zero when head drops below H_j^{min} and $Q_j^{avl} = Q_j^{req}$ (a constant) when the head exceeds H_j^{des}.

As can be seen, the incorporation of pressure dependent demand term in the main set of nonlinear equations does not lead to any new equations or unknown variables. As such, the basic structure of the Jacobian remains unchanged. It can, therefore, be expected that the computational characteristics of the solution methodology will not be highly affected.

To improve the computational efficiency (faster convergence), some modifications have been made in the Newton-Raphson method herein. First, instead of using Eq. (4.14b), an approximation to the values of z which minimize the Euclidean norm of the single variable function $f(\underline{H} - z\,\underline{\Delta H})$ is found. Then the new point in the sequence is now given by

$$\underline{H}^{m+1} = \underline{H}^{m} - z^{m}\,\underline{\Delta H}^{m} \tag{5.16}$$

The above procedure ensures that $\underline{H}^{m+1}$ is a better approximation to the solution than $\underline{H}^{m}$ (Lemieux 1972; Lam and Wolla 1972; Todini 1997). Several methods can be used to calculate the value of z (see e.g. Burden and Fairs, 1993). For instance, a Fibonacci search can be used to calculate the z values. According to Broyden (1965) the following relationship can be assumed to minimize the norm of $\underline{F}\,(\underline{H}^{m+1})$,

$$z = \frac{(1-6RF)^{0.5} - 1}{3RF} \tag{5.17}$$

in which $RF = |F(H^{m+1})_{z=1}| \,/\, |F(H^{m+1})_{z=0}|$. At each iteration if $F(H^{m+1})_{z=1} > F(H^{m+1})_{z=0}$ then the z value is operable in Eq. (5.16). Secondly, to avoid head oscillations for some demand nodes, a modification is made by averaging the computed values of head obtained at the (m)th and (m-1)th iterations (Shamir and Howard 1968).

The proposed algorithm can be summarised as follows:

1) Assume initial heads, H_j^0 for all nodes other than fixed head nodes.
2) Solve the system of equations, Eqs. (5.12) - (5.15).
3) Determine improved estimates of nodal heads using Eq. (5.16).
4) Repeat steps 2-3 until the convergence criteria are satisfied.
5) Calculate available nodal outflows, Q_j^{avl}.

A flowchart of the HDSM is presented in Figure 5.2.

A Fortran computer programme has been developed based on the above methodology. This has been implemented using a pentium processor 75 MHz and 8 Mbyte RAM. In addition to the normal operating condition, the programme developed for HDSM is capable of simulating failure of any component. Using only the data for the fully connected network, the program can automatically simulate the consequences, in terms of available flow, of the failure of up to any two network components. The accuracy of the results and efficiency of the above methodology is illustrated by the following examples.

5.5 EXAMPLES

To illustrate different aspects of the HDSM a number of networks have been analyzed. Four examples are evaluated in this chapter to illustrate different advantages of the HDSM. In addition, to show the capability of the method three more examples are presented in Appendix D. The tolerances used in the examples were 0.001 m for nodal heads and 0.001 m^3/s for nodal flow equilibrium. The first example, Example 5.1, is taken from Gupta and Bhave (1996b) and the layout of the network is shown in Figure 5.3. The lengths and Hazen-Williams coefficients for all pipes are 1000 m and 130, respectively. The diameters for pipes 1 through 4 are 400, 350, 300 and 300 mm, respectively. The node resistance coefficient, K_j and available flow exponent, n_j, are equal to 360 (s^2/m^5) and 2, respectively. The node data of the network along with the HDSM analysis results are presented in Table 5.1. The DDSM results are also shown for comparison.

It can be seen from the head-driven simulation (HDSM) results in Table 5.1 that the network is pressure deficient as the demand of node 4 of 3 m^3/min is only partially satisfied, the actual outflow being 0.381 m^3/min. To check the accuracy of the results of the proposed formulation and to demonstrate the effects of variations in the source head on available outflows, the source head for this network has been varied from 85 to 110.89 m, and the available outflow at each node based on HDSM can be observed in Table 5.2. As can be seen, these values are essentially the same as the results of Gupta and Bhave (1996b) and therefore confirm the accuracy of the present formulation. The results in Table 5.2 demonstrate the reliability of the model in terms of its ability to produce the correct results where pressure/outflows are less than fully satisfactory. As expected, when pressures are fully satisfactory both HDSM and DDSM give identical results.

Figure 5.4 shows the layout of Example 5.2 which is a well-known four loop network used by several researchers (e.g. Goulter and Coals 1986, Fujiwara and De silva 1990, Fujiwara and Tung 1991; Awumah and Goulter 1992 and; Tanyimboh and Templeman 1995). Table 5.3 shows the pipe data for this network and the nodal data are presented in Table 5.4. First the network is analyzed by the demand-driven simulation method and results are shown in Table 5.4. It can be seen that available head at some nodes is less than H_j^{min}. In reality these values cannot satisfy the required demands at that nodes. However, the conventional DDSM which is used widely by water supply network models, is not able to represent this situation and shows that all nodal demands are satisfied (although pressures are unrealistic). Here the real performance could be simulated by head driven analysis. Results are included in Table 5.4, as well. It shows that all nodal demands could be satisfied except the demand of node 9 which is the critical node and its actual head leads to only partial outflow. The values of H_j and Q_j^{avl} show the qualitative difference between HDSM and DDSM. In fact the HDSM has limited the deficiency of heads around the critical node and has prevented the other parts of the network from be affected in the numerical solution. It means that the numerical representation of the performance of the hydraulic of system has been improved.

Examples 5.3 and 5.4 consider the applicability of the HDSM for more complicated networks, including ancillary components (e.g. pumps, reservoirs, etc.). Figure 5.5 shows the layout of a sample network which is taken from Jeppson (1976). The Hazen-Williams coefficient is 120 for all pipes and H_j^{min} = 25.908 m for all nodes. The equation of each pump is represented by $H_p = -3823.64\, Q_p^2 + 27.172\, Q_p + 6.819$, where H_p is the head provided by the pump. Other features of the network are given in Table 5.5.

To assess the effects of pipe failures on the hydraulic performance of the system, the HDSM is used assuming one pipe fails in each case. Table 5.6 shows values of inflow from the three sources and available flow at the demand node. It can be seen that in all cases of pipe failure, demand node 1 is in reduced service mode with all available outflows being less than the required demand of 0.0566 m^3/s. It can, therefore, be seen that HDSM can simulate the effects of mechanical failure on the hydraulic performance of the system. Results show that critical situation occurs when pipe 2-1, with the largest length and diameter, fails. In this case conveying of flow through the only alternative pipe 6-1 with smaller diameter, leads to more head loss and the nodal head falls below required value.

The last example, Example 5.4, shows the application of the HDSM to a small real-world network. Figure 5.6 and Table 5.7, respectively, show the layout and physical characteristics of this branched system. In this network values of the minimum nodal heads are, somewhat optimistically, taken to be 7 m for each node. The hydraulic characteristics of the pump were represented by $H_p = -11478.421\ Q_p^2 - 13822.773\ Q_p + 51.647$. For peak demand time (9:00 am), the network is analyzed by the HDSM and the results are presented in Table 5.8. Values of available outflow at the nodes are identical to the respective demands except for nodes 2 and 14. It can be observed that available head at node 2 is greater than the minimum but less than the desired head and so the demand is only partially satisfied. Also, at node 14 the available head is less than the assumed minimum head of 7 m and so the outflow is zero. This means that there could potentially be a shortfall in supply at nodes 2 and 14 during periods of high demand. These results suggest that any programme based on DDSM cannot be relied upon to reproduce the real situation when available heads are inadequate. However, in normal DDSM in any situation when nodal pressures drop much below zero the system is regarded as operationally inadequate and efforts would be made to remove this physical anomaly before end results were used. It can, therefore, be concluded that in comparison with DDSM, HDSM is better able to simulate the actual performance of the system and might lead to more accurate and realistic results in terms of nodal head and flow.

To investigate the effects of nodal heads on the nodal outflows during 24 hour a day, an extended period simulation is performed and Figures 5.7 and 5.8 show the diurnal profile of available head and outflow for node 14 (the critical node) obtained from application of the HDSM. Values of H_{14} are shown in Figure 5.7, in comparison with the desired and minimum heads. In some periods of the day when demand is high, the available head is less than the desired head, even less than the stipulated minimum head. This means that partial or no outflow should be expected at this particular node during these periods. Figure 5.8 represents the available outflow at node 14 during 24 hour a day.

To investigate other differences between the DDSM and HDSM, the sensitivity of the head-driven analysis to variation of the nodal demands and desired heads are investigated next. The network of Fig. 5.6 is re-analyzed by both methods with variable nodal demands of 9, 12 and 15 l/min and also values of 7, 14 and 30 m desired head above the minimum nodal heads, i.e. H_j^{des} - H_j^{min}. Performing several DDSM and HDSM analyses the following results are obtained. Tables 5.9 and 5.10 show the results of the both methods for nodal outflows and heads with the required flow (demand) of 9 l/min at all demand nodes. It can be seen from the HDSM results that for each set of the fixed nodal demands, the values of available head increase with increase of the desired head values. However, values of outflow are equal to or less than the nodal demands when H_j^{des} increases, because the range of the reduced service mode would be larger and more nodes would face pressure deficiency. On the other hand, the DDSM results are not sensitive to variation of H_j^{des} and both nodal heads and outflows are constant.

Further, Table 5.11 shows the fraction of unsatisfied nodal outflows for (H_j^{des}- H_j^{min}) of 30 m when nodal demands vary from 9 to 15 l/min. It can be observed that the percentage of shortfall is increased when nodal demands increase. However, the abstraction is zero when the DDSM is used for any demand value, because in DDSM the nodal outflows are (inappropriately) fully satisfied regardless of variation in nodal pressures. The above analyses show the sensitivity of HDSM results to variations of nodal required flow and head. It means that the head driven simulation method responds realistically to variations of the required demand and head, compared with the demand driven simulation method. Therefore, it can be recommended as a useful tool for the modelling of network operations.

It has recently been recognised that the actual volumetric shortfall in nodal water supplied can form the basis of robust reliability assessment (Bao and Mays 1990; Fujiwara and Ganesharajah 1993; Tanyimboh 1993). Results from above examples show that with the HDSM the values of nodal and system shortfall can be obtained directly and with ease. If compared with complicated procedures to calculate maximum available inflow or total shortfall, e.g. the approach of Fujiwara and Ganesharajah (1993) which needs optimization procedure, another advantage of the HDSM arises. This aspect will be illustrated in Chapters 7 and 8.

5.6 COMPUTATIONAL EFFICIENCY

The computational efficiency of the HDSM can be assessed in terms of the number of iterations required in the achievement of a solution to a chosen accuracy together with an overall accuracy measure for successive iterations. One such measure is the Euclidian norm defined as

$$\|\underline{\Delta H}\| = [\sum_{j=1}^{NJ} (\Delta H_j)^2]^{1/2} \tag{5.18}$$

where $\| \ \|$ represents the Euclidian norm.

Figure 5.9 illustrates the rapid convergence of the HDSM. Because the norm only measures the magnitude of the changes in head for all nodes, it may also be useful to examine the changes in head for successive iterations at some critical nodes. The critical node can be taken as the node with the largest discrepancy between demand and available flow at the end of solution procedure, i.e. the most pressure-critical nodes. Figure 5.10 represents the variations of available heads at critical nodes against number of iterations. It is seen that convergence of solution using HDSM is good.

In Table 5.12 numbers of iterations and computer run time for the fully connected network of all the examples considered previously are presented. It can be seen that in these particular examples with the same initial values for nodal heads, results are often obtained using HDSM without any significant loss of computational efficiency compared to the DDSM. Therefore, the HDSM realistically represents not only the behaviour of the physical system, but also has good computational efficiency in comparison with the DDSM.

5.7 SUMMARY AND CONCLUSION

A methodology for pressure-driven analysis of water supply networks has been developed and its capability examined through a number of examples. It was observed that the head-driven simulation methodology works both for simple and realistic networks. However, unlike other formulations for pressure-driven simulation (Bhave 1981; Wagner et al. 1988b; Chandapillai 1991; etc.) the methodology presented herein does not require a separate step in which nodal outflows are adjusted at the end of each iteration. The proposed procedure explicitly incorporates a realistic head-outflow relationship in the continuity equations.

The present method is equivalent to demand-driven simulation when flows and pressures are adequate such that designated demands are fully satisfied. In typical water supply applications this would usually be representative of normal operation conditions. However, under subnormal conditions e.g. pipe failure, pressure-driven analysis (HDSM) can simulate the partial flow delivery realistically, whilst DDSM can only indicate that a supply problem will arise.

Finally, regarding computational efficiency, it has been observed that convergence of the iterations to the solution using HDSM compares favourably to an efficient DDSM implemented herein both in terms of CPU time and number of iterations. It would appear, therefore, that the methodology proposed has the potential to produce hydraulically more realistic results without any significant loss of computational efficiency compared to demand-driven analysis. Furthermore, it represents the ability of method to predict the actual shortfall of nodal and network consumptions. It might therefore be introduced as a suitable tool to be included in reliability measurement.

Table 5.1: Nodal data and results for the network of Example 5.1 (Fig. 5.3).

Node	Input Data			Output Results			
				DDSM		HDSM	
	H_j^{min} (m)	H_j^{des} (m)	Q_j^{req} (m³/min)	H_j (m)	Q_j^{avl} (m³/min)	H_j (m)	Q_j^{avl} (m³/min)
1[a]	-	100.0[b]	11.0	100.000	11.000	100.000	8.381
2	90.0	90.4	-2.0	95.131	-2.000	97.053	-2.000
3	88.0	88.4	-2.0	88.698	-2.000	93.647	-2.000
4	90.0	90.9	-3.0	80.139	-3.000	90.015	-0.381
5	85.0	86.6	-4.0	77.103	-4.000	86.982	-4.000

[a] Source
[b] Available source head

Table 5.2: Available nodal outflows for different source head values in Example 5.1.

Source Head (m)	Available outflow (m³/min) at node:				Total supply to the network (m³/min)
	2	3	4	5	
85.00	-0.000 (-0.000)[a]	-0.000 (-0.000)	-0.000 (-0.000)	-0.000 (-0.000)	0.000 (0.000)
88.87	-0.000 (-0.000)	-0.000 (-0.000)	-0.000 (-0.000)	-2.424 (-2.420)	2.424 (2.420)
90.88	-0.000 (-0.000)	-1.790 (-1.787)	-0.000 (-0.000)	-2.560 (-2.553)	4.350 (4.340)
91.96	-1.621 (-1.616)	-2.000 (-2.000)	-0.000 (-0.000)	-2.592 (-2.586)	6.214 (6.202)
92.33	-2.000 (-2.000)	-2.000 (-2.000)	-0.000 (-0.000)	-2.645 (-2.629)	6.645 (6.629)
98.50	-2.000 (-2.000)	-2.000 (-2.000)	-0.000 (-0.000)	-4.000 (-4.000)	8.000 (8.000)
98.84	-2.000 (-2.000)	-2.000 (-2.000)	-0.000 (-0.000)	-4.000 (-4.000)	8.000 (8.000)
110.89	-2.000 (-2.000)	-2.000 (-2.000)	-3.000 (-3.000)	-4.000 (-4.000)	11.000 (11.000)

[a] Indicates Gupta and Bhave (1996b) results

Table 5.3: Pipe data for the network of Example 5.2 (Fig. 5.4).

Pipe	Diameter (mm)	CHW	Length (m)
1-2, 2-4	250	130	1000
2-3, 4-7	175	130	1000
2-5, 4-5	145	130	1000
3-6, 7-8	115	130	1000
5-6, 5-8	100	130	1000
6-9, 8-9	100	130	1000

Table 5.4: Results of DDSM and HDSM analysis of the network of Example 5.2 (Fig. 5.4).

Node	Input Data			Output Results			
				DDSM		HDSM	
	H_j^{min} (m)	H_j^{des} (m)	Q_j^{req} (m^3/sec)	H_j (m)	Q_j^{avl} (m^3/sec)	H_j (m)	Q_j^{avl} (m^3/sec)
1[a]	-	100[b]	0.2081	100.000	0.2081	100.000	0.1710
2, 4	0	30	-0.0208	83.174	-0.0208	88.015	-0.0208
3, 7	0	30	-0.0208	57.106	-0.0208	70.942	-0.0208
5	0	30	-0.0208	56.783	-0.0208	71.579	-0.0208
6, 8	0	30	-0.0208	-20.338	-0.0208	35.328	-0.0208
9	0	30	-0.0625	-177.633	-0.0625	4.817	-0.0255

[a] Source
[b] Available source head

Table 5.5: Pipe data for Figure 5.5 (Example 5.3).

Pipe	Length (m)	Diameter (mm)
2-1	609.6	203.2
3-2	304.8	152.4
10-3	pump	-
4-2	304.8	203.2
5-4	304.8	152.4
9-5	pump	-
6-4	304.8	152.4
8-7	pump	-
7-6	304.8	203.2
6-1	609.6	152.4

Table 5.6: Actual nodal inflows and outflow for the network of Figure 5.5 (Example 5.3).

Pipe failed	Node			
	1[b]	8[c]	9[c]	10[c]
None	-0.05660 (-0.05663)[d]	0.02373 (0.02381)	0.01127 (0.01127)	0.02160 (0.02155)
2-1	-0.02563	0.01481	0.00000	0.01082
6-1	-0.04331	0.01354	0.00850	0.02127
3-2	-0.04670	0.02874	0.01796	0.00000
4-2	-0.04689	0.01945	0.00000	0.02744
10-3[a]	-0.04670	0.02874	0.01796	0.00000
5-4	-0.05302	0.02766	0.00000	0.02436
6-4	-0.05623	0.02033	0.01339	0.02251
9-5[a]	-0.05202	0.02766	0.00000	0.02436
7-6	-0.04546	0.00000	0.01938	0.02608
8-7[a]	-0.04546	0.00000	0.01938	0.02608
	Available nodal outflow/inflow (m^3/sec)			

[a] Indicates pipe including pump, [b] Demand node, [c] Source node
[d] Indicates Jeppson (1976) results

Table 5.7: Input data for the real network of Figure 5.6 (Example 5.4).

Link	Diameter (mm)	CHW	Length (m)
1-2	76	100	1
2-3	pump	-	-
3-4	150	140	40
4-5	180	140	455
5-6	100	43	755
6-7	125	130	65
7-8	125	130	160
8-9	100	130	10
9-10	76	100	215
9-13	125	130	155
10-11	76	100	75
10-12	76	5	150
13-14	20	50	115
13-15	125	2	655
15-16	76	100	40
15-18	125	130	390
16-17	80	120	210

Table 5.8: HDSM results for the network of Fig. 5.6 with peak demands.

Node	H_j^{min} (m)	H_j^{des} (m)	Q_j^{req} (l/s)	H_j (m)	Q_j^{avl} (l/s)
1 (Source)	84.3	86.0	2.130	86.000	1.780
2	84.3	91.3	-0.020	85.992	-0.010
3	84.3	91.3	-0.000	109.987	-0.000
4	84.0	91.0	-0.020	109.976	-0.020
5	72.0	79.0	-0.320	109.938	-0.032
6	83.0	90.0	-0.000	102.779	-0.000
7	82.8	89.8	-0.280	102.567	-0.280
8	82.6	89.6	-0.100	101.431	-0.100
9	82.6	89.6	-0.140	101.267	-0.140
10	84.0	91.0	-0.070	97.785	-0.070
11	87.0	94.0	-0.200	97.126	-0.200
12	86.0	93.0	-0.210	94.299	-0.210
13	83.5	90.5	-0.320	101.105	-0.320
14	89.5	96.5	-0.340	41.113	-0.000
15	63.9	70.9	-0.110	96.554	-0.110
16	63.8	70.8	-0.000	96.554	-0.000
17	61.6	68.6	-0.000	96.554	-0.000
18	64.6	71.6	-0.000	96.554	-0.000

Table 5.9: Sensitivity of available nodal heads of the network of Fig. 5.6 to variation of H_j^{des} using the DDSM and HDSM while required flow at all demand nodes is 9 l/min.

Node	DDSM	HDSM		
		$H_j^{des} - H_j^{min}$ =		
		7 (m)	14 (m)	30 (m)
1[a]	86.000[b]	86.000[b]	86.000[b]	86.000[b]
2	85.992	85.993	85.993	85.994
3	107.173	109.081	109.234	110.444
4	107.167	109.076	109.229	110.441
5	107.145	109.057	109.210	110.424
6	102.484	104.920	105.114	106.733
7	102.467	104.905	105.100	106.720
8	102.432	104.876	105.070	106.693
9	102.427	104.871	105.066	106.690
10	102.289	104.753	104.948	106.597
11	102.282	104.747	104.943	106.282
12	99.050	101.887	102.115	104.481
13	102.422	104.867	105.061	106.685
14	79.142	90.425	91.145	92.945
15	95.582	96.706	96.813	97.263
16	95.582	96.706	96.813	97.263
17	95.582	96.706	96.813	97.263
18	95.582	96.706	96.813	97.263
	Available head (m)			

[a] Source
[b] Available source head

Table 5.10: Sensitivity of available nodal outflows of the network of Fig. 5.6 to variation of H_j^{des} using the DDSM and HDSM while required flow at all demand nodes is 9 l/min (= 15 l/s).

Node	DDSM	HDSM		
		$H_j^{des} - H_j^{min}$ =		
		7 (m)	14 (m)	30 (m)
1[a]	1.800	1.628	1.630	1.431
2	-0.150	-0.074	-0.052	-0.036
3	0.000	0.000	0.000	0.000
4	-0.150	-0.150	-0.150	-0.141
5	-0.150	-0.150	-0.150	-0.150
6	0.000	0.000	0.000	0.000
7	-0.150	-0.150	-0.150	-0.134
8	-0.150	-0.150	-0.150	-0.134
9	-0.150	-0.150	-0.150	-0.134
10	-0.150	-0.150	-0.150	-0.130
11	-0.150	-0.150	-0.150	-0.121
12	-0.150	-0.150	-0.150	-0.118
13	-0.150	-0.150	-0.150	-0.132
14	-0.150	-0.055	-0.051	-0.050
15	-0.150	-0.150	-0.150	-0.150
16	0.000	0.000	0.000	0.000
17	0.000	0.000	0.000	0.000
18	0.000	0.000	0.000	0.000
	Available outflow (l/s)			

[a] Source

Table 5.11: Abstraction of nodal outflows of the network of Fig 5.6 when H_j^{des} - H_j^{min} = 30 m.

Node	DDSM	HDSM		
		Nodal demands (l/min) =		
		9	12	15
1[a]	0.00	20.50	24.00	27.50
2	0.00	76.27	76.25	76.25
3	-	-	-	-
4	0.00	6.13	9.65	12.44
5	0.00	0.00	0.00	0.00
6	-	-	-	-
7	0.00	10.73	15.20	18.76
8	0.00	10.40	14.85	18.44
9	0.00	10.40	14.90	18.44
10	0.00	13.20	17.90	21.60
11	0.00	19.20	24.25	28.28
12	0.00	21.53	27.30	31.88
13	0.00	12.07	16.65	20.32
14	0.00	66.13	76.50	83.48
15	0.00	0.00	0.00	0.00
16	-	-	-	-
17	-	-	-	-
18	-	-	-	-
	Abstraction of nodal outflows (%)			

[a] Source

Table 5.12: Summary of computational efficiency for fully-connected networks of Examples 5.1-4 (with all components available).

Example	Type of Analysis	No. of iterations	CPU time (sec)
1[a]	DDSM	5	0.113
	HDSM	16	0.164
2	DDSM	21	0.275
	HDSM	9	0.273
3	DDSM	17	0.275
	HDSM	17	0.275
4	DDSM	23	0.172
	HDSM	23	0.172

[a] Source head = 100 m

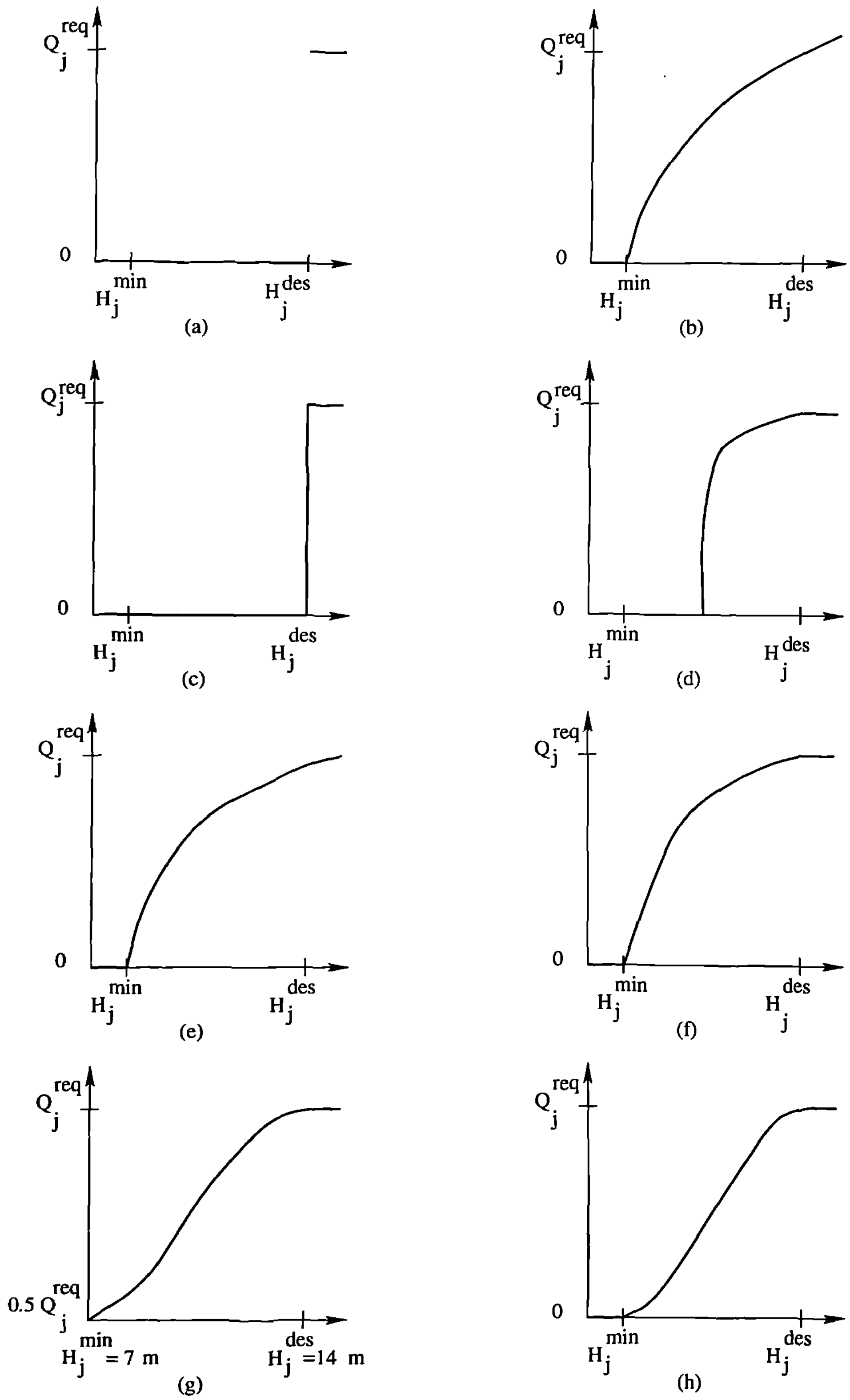

Figure 5.1: Head-outflow relationships: a) Su et al. (1987), b) Reddy and Elango (1989), c) Bhave (1981), d) Germanopoulos (1985), e) Gupta and Bhave (1996b), f) Wagner et al. (1988b), g) Cullinane et al. (1992) and h) Fujiwara and Ganesharajah (1993).

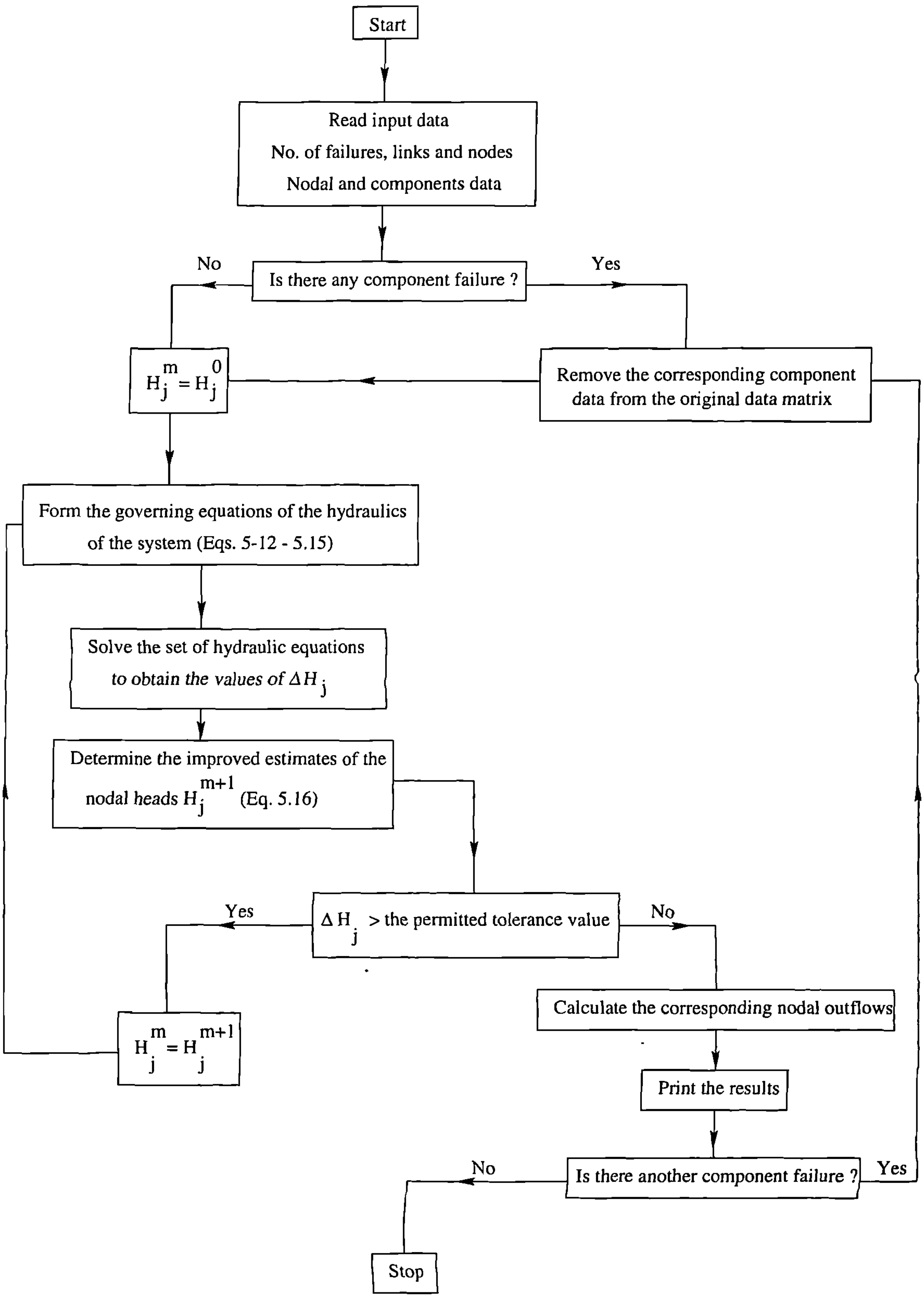

Figure 5.2: Flowchart of the head-driven simulation method.

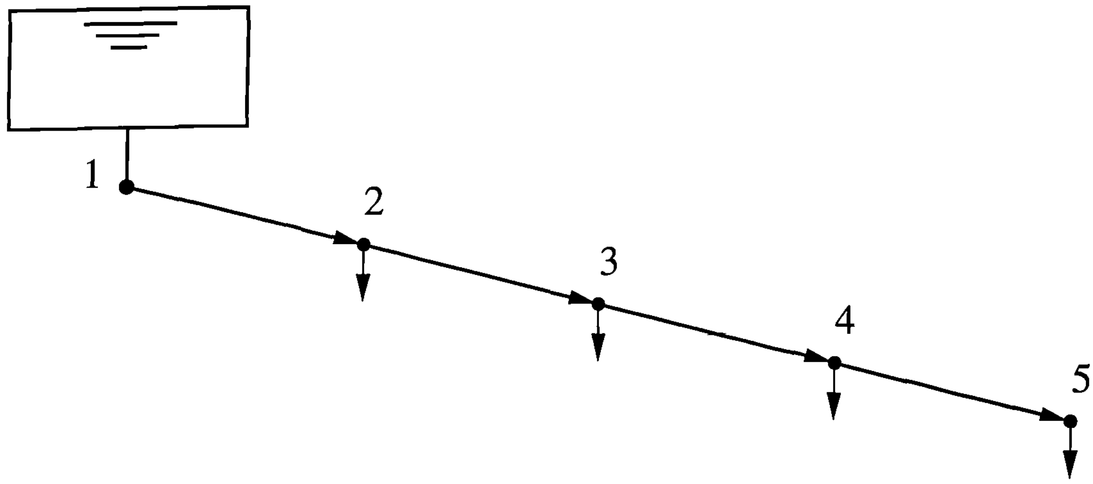

Figure 5.3: Simple network of Example 5.1; adapted from Gupta and Bhave (1996b).

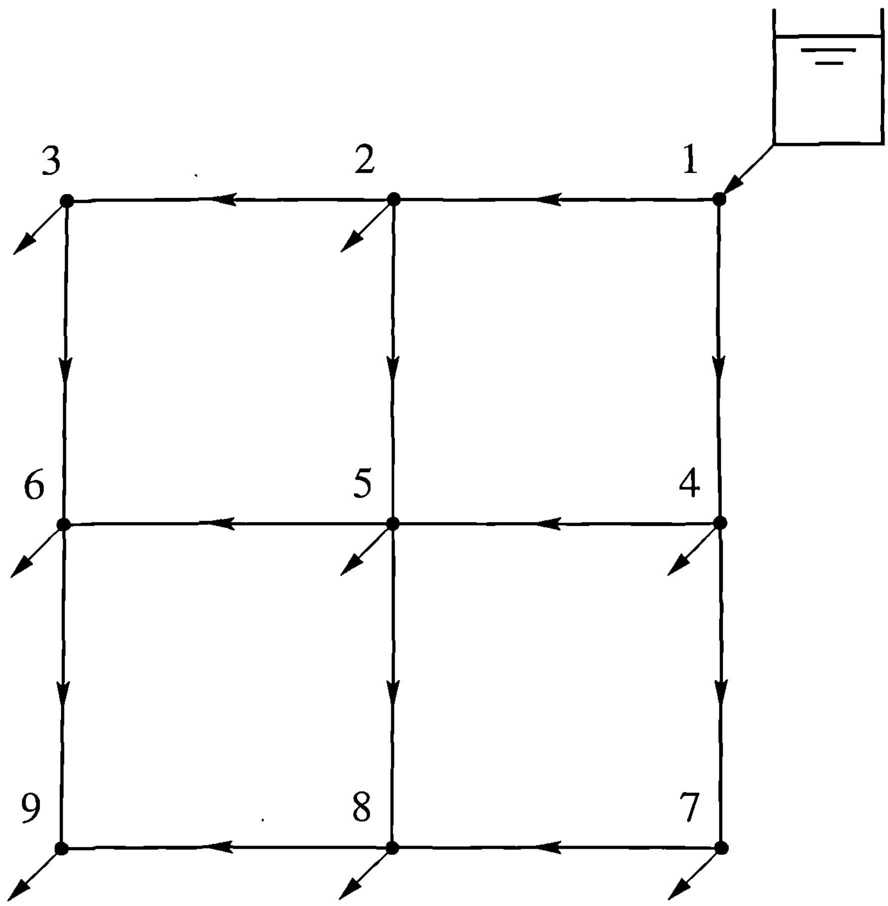

Figure 5.4: Layout of Example 5.2.

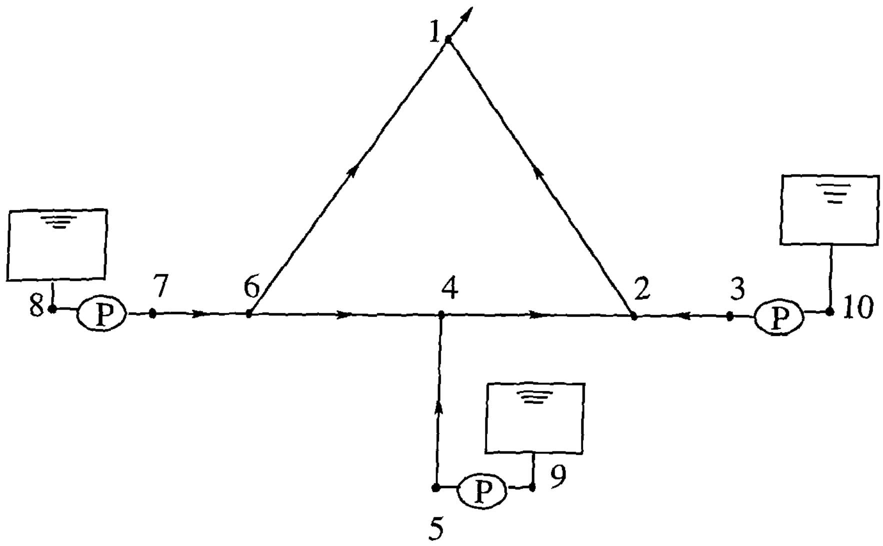

Figure 5.5: Layout of Example 5.3; adapted from Jeppson (1976).

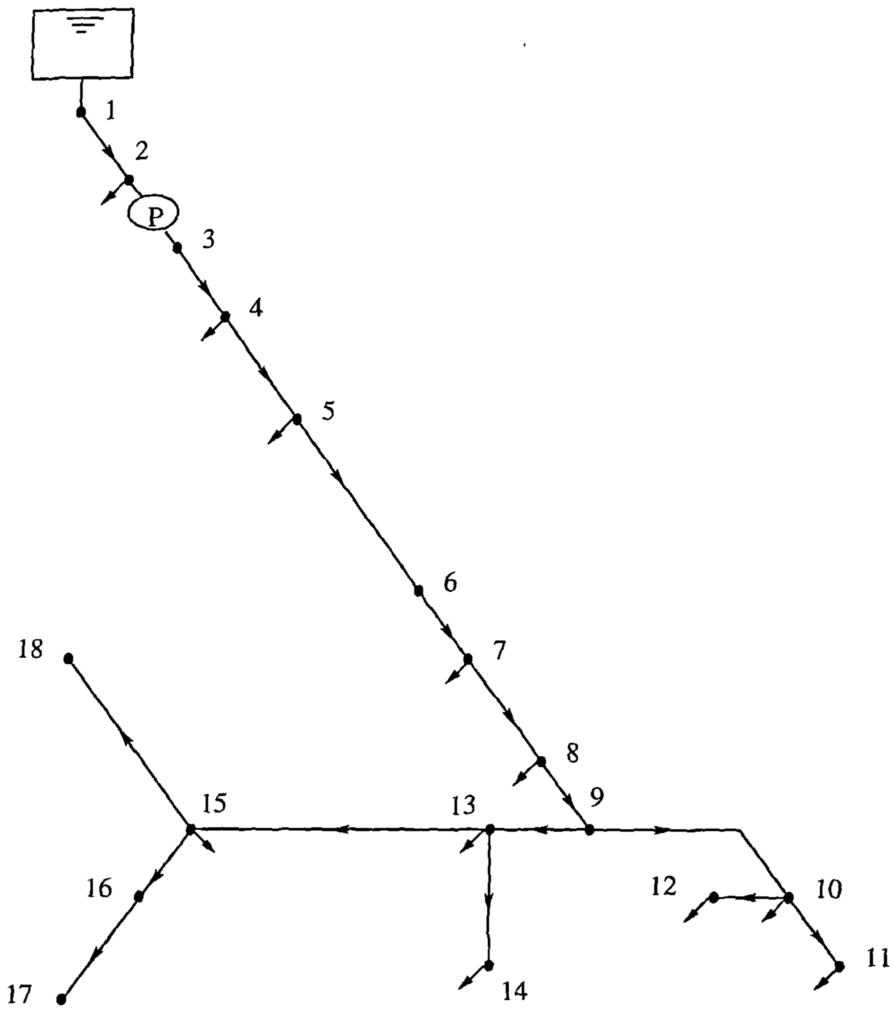

Figure 5.6: Layout of Example 5.4 (a real world case study).

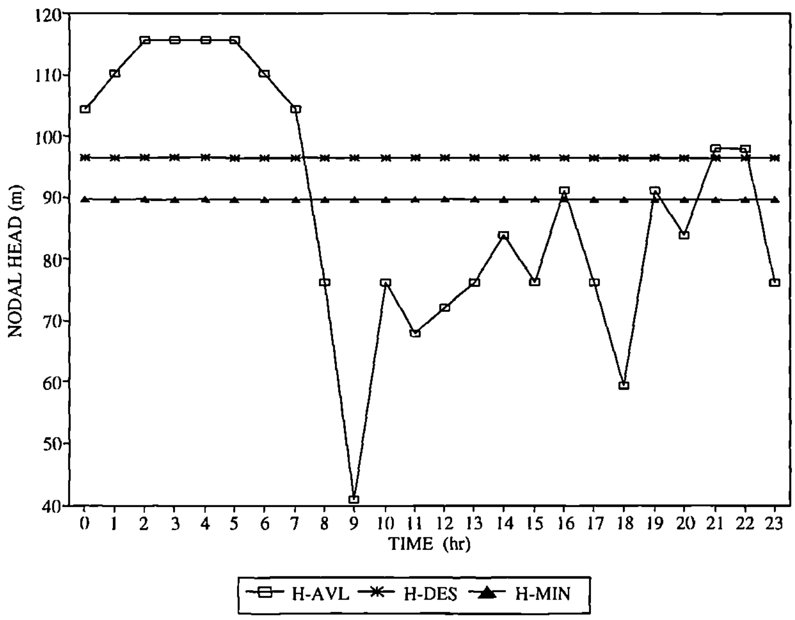

Figure 5.7: Diurnal profile of available head at node 14 of Fig. 5.6.

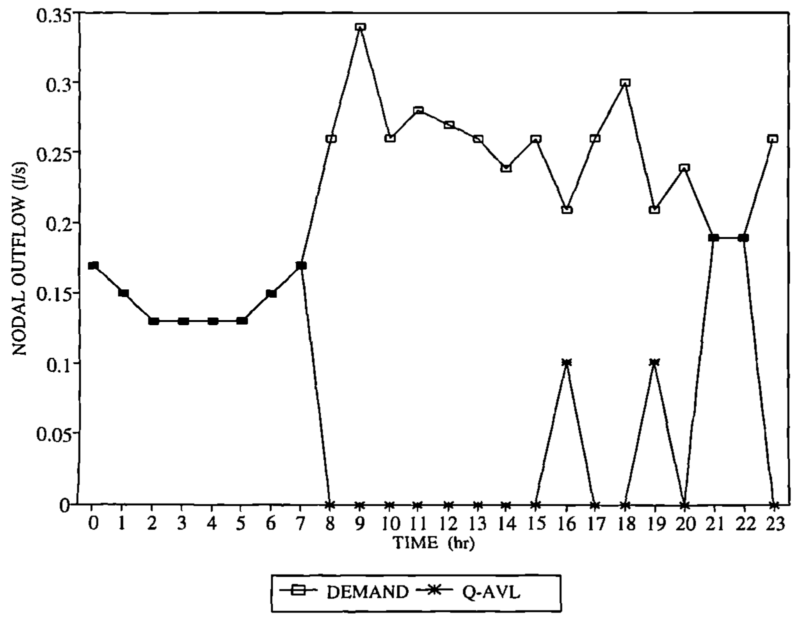

Figure 5.8: Diurnal profile of available outflow at node 14 of Fig. 5.6.

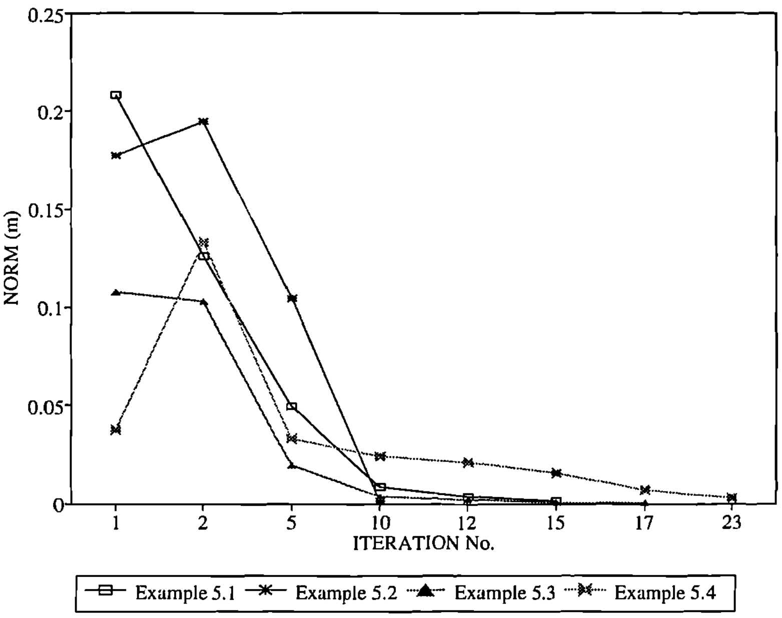

Figure 5.9: Values of convergence measure, norm, against iteration number for Examples 5.1-4 using HDSM.

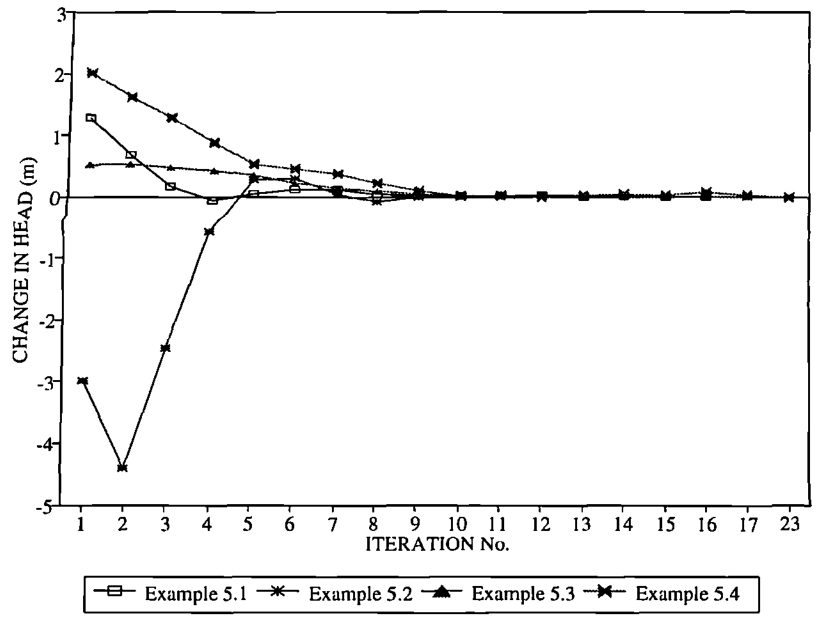

Figure 5.10: Changes in available head at critical nodes for Examples 5.1-4 using HDSM.

CHAPTER 6

THE RELIABILITY CONCEPT IN WATER SUPPLY NETWORKS

6.1 INTRODUCTION

Reliability of water supply systems is becoming one of the most important issues in respect of the design and management of these systems under normal operation and failure conditions. There is much emphasis on operational control and modifications of the network regarding the capacity and level of service in existing distribution systems. From a general point of view reliability can be defined as the probability that a system performs its mission within specific limits for a given period of time in a specified environment (Bazovski, 1961). In particular, for a water supply network there is a common concern about the ability of the system to supply water at each demand point at the required flow rate and head level, under the random failure of system components or under fluctuating demand rates.

Nothing is 100% reliable and water supply systems are no exception. The failure state of the network has multiple aspects. Drought or pollution of streams, outage of pumps, leakage and burst in pipes, failure of treatment systems, exceedance of demands and inadequacy of pressures to guarantee a certain level of service are a number of crises which affect, often severely, the reliability of a water supply system. These crises can lead to large economic losses and even threats to public health. The frequency of a failure state is also important, both as a general event and impact on a particular set of consumers. It should be noted that whilst the average performance of the network is good, network failures may be concentrated both in frequency and effect on a small number of customers.

Reliability measures should be reasonable, understandable and must have a sound theoretical basis (Walski et al. 1987). However, because of the lack of a comprehensive, computationally feasible and easy reliability measure, there is no simple and universally accepted way to predict network reliability for design purposes or evaluation of the operating networks. For a large system with many interactive subsystems such as water distribution systems, accurate calculation of reliability is extremely difficult because it requires knowledge of the precise reliability of the basic subsystems or components and the impact on mission satisfaction caused by all possible subsystem failures.

This chapter aims to review the reliability concept in water supply systems. Since reliability measures have been adopted in some other fields involving networks, e.g. power, telephone and computer engineering, first a short history of reliability in other fields is presented. Then through reviewing several network reliability measures in water supply systems, some of the most acceptable definitions and classifications of reliability indices are evaluated.

6.2 GENERAL REVIEW

Several studies on reliability measures in other fields had been developed before being investigated in water distribution networks. Some of these applied measures can be useful for assessing reliability of water distribution systems, however, the different physical laws that govern flow in these networks and the different effects that failures have in these services should be paid due attention. Wagner et al. (1988a) presented a brief review of the literature on reliability of networks in general and water supply systems in particular. More detail and a critical evaluation can be found in Wagner et al. (1986) which also contains a comprehensive bibliography.

Reliability measures in other fields, according to Satanarayana and Prabhakar (1978), generally used standard definitions which have been presented in major text books such as state enumeration (Wing and Demetrio 1964; Shooman 1968), factoring (Moskovitz 1958), reduction to series-parallel networks (Misra 1970), path enumeration (Lee 1955; Misra and Rao 1970; Brown 1971; Henley and Williams

1973; Frata and Montanari 1973 and Lin et al. 1976) and Cut Set and Tie Set enumeration (Jensen and Bellmore 1969; Hansler et al. 1970 and Lin et al. 1976). In some of these texts a number of system reliability characteristics like availability, mean time to failure and mean time to repair are discussed (e.g. Pearson 1977).

Many of the network reliability measures studied in the fields of communications theory and operations research were concerned with the probability that a continuous path exists between two specified nodes (connectivity), that all nodes in the network are connected, or that groups of nodes are connected (Ball 1980). The specific advantage of these measures was considered to be their capability to analyse complex networks. Also, Kim et al. (1972); Rosenthal (1977) and; Rai and Aggarwal (1978) presented a method for computing complex system reliability (none series-parallel) in general networks. Several researchers have shown that for general networks, exact calculation of these reliability measures is very difficult (e.g. Provan and Ball 1983). In power and communications networks some of the simpler measures and approximations of more difficult ones have been developed (Rosenthal 1977; Buzacott 1980; Satanarayana and Wood 1982; Ball and Provan 1983; Agrawal and Barlow 1984; Agrawal and Satanarayana 1984; Johnson 1984 and Provan and Ball 1984). For general non-series-parallel networks the only known practical methods of reliability analysis were supposed to be the Path and Cut Set methods (Biegel 1977 and Nelson et al. 1970). However, they are not easy to apply to large networks (Jensen and Bellmore 1969; Nelson et al. 1970 and Batts 1971).

Some works were based on graph theory and considered only one source for the network (e.g. Satanarayana and Hagstrom 1981). System reliability analysis often assumes that the system is represented by a probabilistic graph, and the system is functioning if there exists a path from the input node to the output node (Lee 1980). Thus reliability is considered a matter of connectivity only and reliability has been obtained primarily with the enumeration of paths or cuts in the graph (Arnborg 1978 and Arunkumar and Lee 1979). But in many physical systems such as power transmission systems and oil or water pipeline networks, there will be numbers associated with every branch, for example, the flow capacity of the branch. Therefore, reliability of a network cannot necessarily be characterized by only connectivity. Lee (1980) asserted that a network is good if a specified amount of flow can be conveyed from the input node to the output node.

Kessler et al. (1990) pointed out that the design of invulnerable water distribution networks is more complicated than other networks. For example in computer and telephone networks the path capacity would be automatically satisfied when a path exists. However, in water networks the situation is more complicated because of the nonlinear characteristics of hydraulic elements (head-flow relationship along a pipe, valve and pump). Also, unlike some systems such as electrical networks, the cost of a path is dependent on its capacity. The rest of this chapter will review the literature in respect of the reliability measures in water supply systems.

6.3 RELIABILITY MEASURES IN WATER SUPPLY SYSTEMS

Numerous indices and methods for the evaluation of reliability in water distribution networks are available in the literature (e.g. Mays 1989). However, none of them are universally accepted. According to Goulter and Bouchart (1990) most of the research activity in the probabilistic aspects of reliability in water distribution systems can be separated into two main groups. The first group is concerned with approaches and models that address the reliability of the system as a whole. These approaches examine the reliability of major sections such as the supply, the treatment and the distribution stages, in terms of the performance of the overall system (e.g. Hobbs 1985a,b; Shamir and Howard 1985; Germanopoulos et al. 1986 and Hobbs and Beim 1986). The second group is concerned with the reliability of specific components of the overall system (e.g. the distribution system). This group can be divided into two subgroups of direct and surrogate reliability measures. The latter consists of measures for aspects that are inherently related to reliability.

Since this research deals with reliability in water distribution systems the second group is discussed in more detail while the first group is briefly presented next. It is worth noting that some aspects of reliability have been considered as part of a few optimization studies for the design of water distribution networks but not comprehensively (Su et al. 1987). Because this research does not focus on the optimum design of water distribution networks, herein only a few reliability measures which have been incorporated in such models are summarized.

6.3.1 Reliability of a System as a Whole

A water supply system may be considered as a single demand area connected to a single supply area. Reliability measures for such models have been developed by Endrenyi (1978); Billinton and Allan (1984); Shamir and Howard (1981, 1985) and Hobbs (1985b). Wagner et al. (1988a) stated that measures applicable to these systems include the expected percentage of time during which demand will exceed capacity and the expected number of shortfall events per unit time. Billinton (1972) and an IEEE subcommittee on the application of probability methods (1978) have presented the bibliographies and the applications of these methods. The concept of frequency and analysis which indicates how frequently and for how long shortfalls of a given severity occur, was also used by Hobbs (1985a), Hobbs and Beim (1986) and Duan and Mays (1987, 1990).

Hobbs and Beim (1988) presented an approach for analysing a bulk water supply system with unreliable capacity. They addressed the computation of the unreliability and expected unserved demand of a water supply system having random demand, finite water storage, and unreliable capacity components. In addition, water supply systems reliability was evaluated by using a Markov chain (Beim and Hobbs 1988). Also, source failure was considered in the methodology of Germanopoulos et al. (1986).

The system supply may be modelled with further subsystems, like aquifers, reservoirs, treatment facilities, etc. for which each one can be characterized by the probability function of time to failure and time to repair. For water systems these methods have been applied by Tangena and Koster (1983), Shamir and Howard (1985) and Hobbs (1985a). For example Vogel and Bolognese (1995) developed a general approach to describe the overall behaviour of water supply systems. Using the Monte Carlo simulation with a two state Markov model, generalised relationships among reservoir system storage, yield, reliability and resilience were introduced for water supply systems fed by normal and lognormal annual inflows. Also, using two stage linear programming to integrate long term and short term supply enhancement and demand management options for least cost shortage management, Wilchfort and Lund (1997) presented a measure for the reliability of urban water supply systems in which the effects of hydrological uncertainties, availability of resources, water uses, and costs were incorporated. The next subsection introduces reliability measures in water distribution systems which is the main concern of this research.

6.3.2 Reliability in Water Distribution Systems

Reliability in water distribution systems is categorised into direct and surrogate measures. In this subsection, direct measures which include some analytical methods, mechanical and hydraulic reliability indices are first discussed and then some surrogate measures are introduced. Mays and Cullinane (1986) provided a good review of previous works on various aspects of reliability in water distribution networks. Also, Mays (1989) reported on current and future tendency in the analysis of water distribution system reliability including concepts, techniques, and methodologies for the evaluation of these systems.

According to Goulter (1987), there is no satisfactory measure for the reduction in performance of the system caused by component failure. A number of studies have made attempts to define parts of the problem and incorporate them into design models. For example the probability of component failure, probability of actual demands being greater than design values, and the system redundancy inherent within the layout of the network are some related reliability issues which have been addressed. However, the joint characteristics of these aspects of reliability are not well determined. Also, Goulter and Bouchart (1990) argued the necessity of two major issues which should be addressed to improve system reliability, an acceptable definition and the parameters which must be evaluated.

The reliability concept has been determined through a number of different names and definitions. For instance, Goulter (1987) expressed the concept of resilience as the ability of a distribution system to supply demands in times of component failure. Hashimoto et al. (1982) pointed out that resiliency (frequency and duration indices) indicate how often failures of a given severity occur and how long they last. Hashimoto et al. (1982); Norrie (1983) and Charles Howard and Associates (1984) defined vulnerability and risk as indices of the economic consequences of shortage and Goulter (1987) quoted it as maximum deficit in supply in terms of network failure. Reliability has also been determined as the ratio of available annual supply to demand (Norrie 1983; Charles Howard and Associate 1984 and Randall et al. 1984). System unavailability has been defined as the probability of failures which equals the chance that the system will be at risk such that demand exceeds available supply/capacity or that operating conditions are otherwise unsatisfactory (Loucks et al. 1981). The following subsections will look at different measures of reliability in water distribution networks.

6.3.2.1 Direct reliability measures

Reliability in water distribution networks can be assessed by means of both analytical and simulation methods. By these, system and nodal reliability can be evaluated in terms of mechanical and/or hydraulic failures which may occur within a water distribution network. However, it might be said that the analytical methods are more appropriate to evaluate mechanical rather than hydraulic failures. Mechanical or component reliability is the probability of a particular component remaining in operation over a specified time period (Ormsbee and Kessler, 1990). Hydraulic reliability is the probability that the system withdraws the required demand under certain pressure. This evaluates the severity of hydraulic failure which may be caused by mechanical failures or excessive demands caused e.g. by fire fighting. This situation has been also quoted as demand failure (Goulter 1987).

Wagner et al. (1988a) showed that a number of reliability indices in water distribution networks can be calculated analytically. However, most algorithms which are used by these methods are adopted from other fields, some of which were presented in Section

6.2. These analytical methods can provide a fast initial assessment of the reliability of a simple system. However, because of the assumptions which are needed to simplify the network description, their applicability to real systems may be restricted. For example behaviour of pumps, tanks, supply rates, etc. cannot be easily represented by analytical methods. Also, these measures are few and do not cover reliability issues comprehensively. Reachability, connectivity and topological reliability are some examples of analytical measures which may be characterized as the probability of a network remaining physically connected over a specified period of time (Ormsbee and Kessler 1990).

For a more detailed analysis that considers the hydraulic behaviour of the distribution system itself, a simulation model is needed, particularly to analyze the reduced networks caused by different kinds of failures (Wagner 1988a). These models solve the nonlinear hydraulic equations for the heads and flow in both normal and subnormal conditions. They appear in different forms as follows:

i) demand-driven simulation
ii) head-driven simulation
iii) optimization procedures

Different methods and algorithms for demand and head driven analysis of water distribution networks were described in detail in Chapters 4 and 5. An optimization procedure, unlike the former, avoids the repetition of trial and error while it converges to a solution (Yeh 1985). Different objectives such as maximizing nodal outflows over all demand nodes (Fujiwara and Tung 1991, Fujiwara and Ganesharajah 1993), minimizing the total shortage (Yang et al. 1996b), etc. have been used in these procedures. Optimization models are likely to be more complicated and time consuming than the other simulation procedures, regarding their computational characteristics.

6.3.2.1.1 Analytical methods

Two probabilistic measures, reachability and connectivity, were defined by Wagner et al. (1988a) as analytical methods. They determined connectivity as the probability that a given demand node in a system is connected to a source and reachability as the probability that all demand nodes in a system are connected to a source. In addition, the probability that a given node receives sufficient supply was suggested as a reliability measure.

Henley and Kumamoto (1981) and Ang and Tang (1984) presented an introduction to the fault-tree analysis. It considers the different ways in which component failures lead to supply shortfall and calculates the associated probabilities. Also, Willie (1978) and De Jong et al. (1983) provided application of these methods to power and water systems respectively.

Among six analytical techniques developed to evaluate the mechanical reliability of water supply networks with complex configuration (i.e. Cut Set method, Tie Set analysis, event tree technique, fault tree analysis, conditional probability approach and connection matrix method), Tung (1985) concluded that all methods except the connection matrix method yield practically the same system reliability. However, from the computational point of view, the Cut Set method with a first-order approximation was introduced as the most efficient method. Billinton and Allan (1992) state: "A minimum Cut Set is a set of system components which, when failed, causes failure of the system. However, when any one component of the set has not failed, it does not cause system failure".

A detailed description of the Cut Set method can be found in Mays and Cullinane (1986) and Billinton and Allan (1992). The former introduced methods for evaluating the reliability of individual water distribution system components, which included the concepts of mean time to failure analysis and stress-strength or load-resistance analysis. They also concluded that the most promising methods for determining the system reliability and availability for simple series-parallel combination systems are the Cut Set method and the path enumeration methods.

In the reliability based optimal procedure of Su et al. (1987) the value of the reliability constraint was determined by the minimum Cut Set method. Reliability was defined as the probability of achieving sufficient flow and pressure at each node, but values of shortfall in flow and pressure in reduced service mode were not included in the definition of reliability. At first, some pipe(s) were removed and then the minimum Cut Sets were obtained by simulation of the reduced network. If nodal heads were too low and not satisfactory it was quoted as a failure which determined those pipes as a minimum Cut Set of the system. Although this model incorporated the reliability constraint into the optimization model, the whole procedure proved to be very time consuming.

Shamsi (1990) used a simple reliability measure based on network connectivity to compute subnetwork reliability by the path enumeration techniques such as minimal Cut Sets method which was based on water availability only, regardless of its quantity or quality. He concluded that more research was needed to recommend computationally better algorithms that consider more complicated reliability measures such as quantity and/or quality of water supply.

Yang et al. (1996a) demonstrated a method focused on the impact of link failures on source-demand connectivity, which was used as a measure of mechanical reliability. The mechanical reliability was computed using the minimum Cut Set method. The operations of the water distribution network were simulated by an optimization model with the objective of minimizing water shortage at the demand nodes. Regarding the Cut Set method it can be said that use of this method for water supply systems may not be justified because in real networks the probability of simultaneous failure of a combination of components from a node to the source is very low (Walters and Knezevic 1989). Therefore, consideration of such a situation is not realistic.

More recently Quimpo and Wu (1997) developed a method for calculating the spatial variation in reliability throughout a water distribution network. Measure of reliability was based on meeting nodal demands dependent on the hydraulic capacity of all the network elements leading from the source. Then, nodal reliability was calculated, using a measure of network connectivity.

In conclusion it can be said that neither connectivity nor reachability are adequate measures of reliability because they only address the existence of a path between nodes and the capacity of the path to supply required water at adequate pressure is not considered.

6.3.2.1.2 Mechanical reliability / Component availability

The mechanical reliability of a network can be defined as the ability of the distribution system to provide continuing and long term operation without the need for frequent repairs, modifications, or replacement of components or subcomponents (AWWA 1980). It depends on the arrangement or layout of its components and the mechanical reliability of the individual components. Thus the mechanical reliability of a component is usually defined as the probability that the component or subcomponent performs its mission within specified limits for a given period of time in a specified environment (Mays 1989). When quantified, mechanical reliability is simply an expression of the probability that a piece of equipment is operational at any given time. The mathematical evaluation of mechanical reliability is well developed and has been used in the analysis of mechanical and electrical systems (Billinton and Allan 1992, 1984, Henley and Kumamoto 1981).

Mathematically, reliability R(t) of a component can be expressed as follows:

$$R(t) = \int_{t}^{\infty} f(t)\, dt \qquad (6.1)$$

where f(t) is the probability density function of the time to failure of the component.

Mays (1989) asserted that for repairable components such as those often found in water distribution systems, it is much more appropriate to use the concept of availability. While the reliability is the probability that the component experiences no failures during period of [0-t], the availability of a component is the probability that the component is in operational condition at time t and can be expressed as the percentage of time that the component is in an operational state.

Mechanical reliability measures can be used to check the level of connectivity or availability of network components at any time. The following subsections will investigate component availability and its relation to the mechanical reliability of pipes and other components (pumps, valves and tanks).

i) Pipe availability / reliability

The concept of burst and its effects on water losses was developed in Chapter 2. The largest proportion of total annual losses from bursts generally occurs on service pipes. However, there is little published information on its frequency (Lambert 1997a). Water main failures (bursts) are due to excessive load, temperature, or corrosion (O'Day 1982). Break rates are dependent upon pipe size, geographic location, method of pipe manufacture, soil type etc. For instance, a strong relationship between the failure rate and pipe diameter was shown by Kettler and Goulter (1983) for cast iron pipes in Winnipeg, Canada. Kettler and Goulter (1985); Su et al. (1987); Goulter and Kazemi (1988, 1989); Mays (1989); Cullinane et al. (1992) and Goulter et al. (1993), have carried out investigations to determine the pipe breakage rate based on diameters which can be used in the pipe reliability/availability formulations. Also, Habibian (1994) has related the pipe breakage to temperature.

Based on pipe qualities such as age, material, etc. a few studies have tried to quantify the number of pipe breaks/pipe length unit/time unit. Walski (1984) and O'day (1982) presented some data on pipe break inter-arrival times, besides some factors which affect these inter-arrival times qualitatively. A relationship for the increase of pipe breaks with pipe age was described by Shamir and Howard (1979) using an exponential model. Walski and Pelliccia (1982) added some corrections to this model for the factors of pipe size and number of previous breaks. However, the inter-arrival time between individual breaks of the same pipe was not covered by either of these models. A hazard failure model was presented by Marks et al. (1985) giving the probability, at any small time interval, that a pipe will break based on several factors including the age of the pipe, the number of previous breaks and the time since the last break. The hazard failure model has been also developed by Cox and Oakes (1984); Andreou et al. (1987) and Al-Humoud et al. (1990). A brief description of this model can be found in Quimpo and Wu (1997).

It has been commonly assumed, by several studies, that failures of different pipes occur independently. By presenting some examples, Su et al. (1987) illustrated that the simultaneous failure of a combination of pipes may not have a major impact on the reliability of the system because of the small probability of the joint failures.

The mechanical availability was evaluated by Cullinane et al. (1992) in terms of availability of individual pipes. Following the definition of Ang and Tang (1984), the probability of the operational state of link (pipe) l, a_l, can be represented as:

$$a_l = \frac{MTBF}{MTBF + MTTR} \tag{6.2}$$

where MTBF = mean time between failures (duration of connectivity) and MTTR = mean time to repair i.e. duration of unconnectivity and repair. Then using the data sets of Mays (1989) and Walski and Pelliccia (1982), the following relationship for pipe availability was obtained,

$$a_l = \frac{0.21218\ D_l^{1.462131}}{(0.00074 D_l^{0.285} + 0.21218 D_l^{1.462131})} \qquad \forall\ l = 1, ..., NP \tag{6.3}$$

in which a_l is availability of link (pipe) l and D_l is pipe diameter (in inches).

Another expression for probability of an operational state of a component has been given as follows (Fujiwara and Tung 1991):

$$a_l = \frac{\alpha_l}{\alpha_l + \beta_l} \tag{6.4}$$

where α_l = expected number of repairs of pipe l per unit of time, and β_l = expected number of failures of pipe l over the same period. α_l is obtained by evaluation of historic data and β_l can be demonstrated as:

$$\beta_l = L_l \, \mu_l \qquad \forall \, l = 1, ..., NP \tag{6.5}$$

in which L_l is pipe length and μ_l is number of breaks/ unit of pipe length/ unit of time which has been expressed by Kettler and Goulter (1985) by the following relationship

$$\mu_l = (2.002 - 0.0064 \, D_l) \qquad \forall \, l = 1, ..., NP \; and \; 100 \leq D_l \leq 300 \tag{6.6}$$

μ_l is number of breaks/km/year and L_l and D_l are in (km) and (mm), respectively. Finally, applying Eqs. 6.4-6 and assuming $\alpha_l = 0.64$ repair per day, the following formula was obtained by Fujiwara and Tung (1991).

$$a_l = \frac{0.64}{0.64 + L_l \, (0.005485 - 0.0000175 \, D_l)} \qquad \forall \, l = 1,..., NP \tag{6.7}$$

where L_l and D_l are in (km) and (mm), respectively, and day is used as the unit of time.

Furthermore, a Poisson distribution has been used as the probability function for pipe failure by some researchers (e.g. Kettler and Goulter 1983; Coals and Goulter 1985; Goulter and Coals 1986; Germanopoulos et al. 1986 and Su et al. 1987; etc.) by which the probability of NB link breaks can be determined as:

$$p(NB) = \sum_{NB=1}^{\infty} \frac{e^{-\beta_l} \, \beta_l^{NB}}{NB!} \tag{6.8}$$

and NB = number of breaks.

For instance, Germanopoulos et al. (1986) described the random occurrence of the failure events in time by a Poisson distribution. The duration of a failure event was taken to be exponentially distributed and the probability of occurrence of failure events of duration greater than a given time was also represented by the Poisson distribution. Also as an example of above, the mechanical reliability (availability) of pipe l was determined by Su et al. (1987) as follows:

$$a_l = e^{[-L_l \cdot \mu_l]} \qquad \forall\, l = 1, ..., NP \tag{6.9}$$

in which L_l is the pipe length (in miles) and μ_l, the number of breaks/mile/year in pipe l, was determined using the failure data obtained from the city of St. Louis, as below

$$\mu_l = \frac{0.6858}{D_l^{3.28}} + \frac{2.7158}{D_l^{1.3131}} + \frac{2.7685}{D_l^{3.5792}} + 0.042 \qquad \forall\, l = 1, ..., NP \tag{6.10}$$

where D_l is pipe diameter in (inches).

The concept of pipe availability has been considered through different approaches as part of reliability measures. For example, the probability that the number of breaks in a link are greater than a specified value together with the probability that nodal demands and pressures are greater than certain values were used as a measure of reliability in a chance-constraint model by Kettler and Goulter (1983); Tung (1986) and; Goulter and Bouchart (1987).

Coals and Goulter (1985) demonstrated three alternative approaches which related the probability of failure of individual pipes to a system reliability measure through a least cost procedure. In the first approach the probability of failure of the path supplying a node was addressed. The second one considered the probability of all pipes connected to a node failing simultaneously. Finally, the last approach maintained a pre-determined reliability at nodes while minimizing the differences in the diameters of the pipes connected to that node. Pipe failure probabilities were considered using the Poisson distribution. Both Goulter and Coals (1986) and Su et al. (1987) also showed how the concept of pipe breakage probability could be incorporated into a least cost design model.

The first approach of the Goulter and Coals (1986) addressed the probability of isolation of a node through simultaneous failure of all links connected directly to that node. Their second approach attempted to recognize redundancy by minimizing the deviations in the reliabilities of all pipes connected to each node within the network. One theoretical weakness of the node isolation approach was that it considered a node to be able to be supplied adequately as long as there was at least one link connecting it to the rest of the network, which was somewhat optimistic. Also Su et al. (1987) pointed out that the other disadvantage of the model was the assumption that all the pipes connecting a node had similar diameters and hence had similar values of failure probability, which was not applicable in real pipe networks.

Goulter and Bouchart (1990) pointed out the important fact that there are a number of ways in which improvement in the measure of reliability can be obtained by improvement in some other parts of the network. A number of researchers e.g. Clark et al. (1982); Ciottoni (1983) and Kettler and Goulter (1985) have shown that pipe failure rates are strongly related to diameter, with the larger diameter pipes having lower rates of failure than pipes with smaller diameters. Therefore, improvement in mechanical reliability can be achieved by selecting larger diameter pipes. Similarly, reducing the probabilities of flow exceedance from the design levels at nodes will also improve mechanical reliability by causing the selection of larger diameter, high capacity pipes which, in turn, have reduced rates of breakage.

ii) Availability of other components

Goulter (1987) asserted that many of the concepts used in the pipe breakage case are equally applicable to the failure of other components or even whole parts of the network. Damelin et al. (1972) developed a model to evaluate the reliability of supplying a known demand pattern in a water supply system in which shortfalls were caused by random failures of the pumping equipment. They defined the average reliability factor as the probability that the equipment will supply the total required demand in a year. Using the work of Arad (1968) the following procedure was carried out for pump reliability calculation. Synthetic data for inter-failure times and repair durations were generated by Monte Carlo method. Inter-failure times of pumping equipment were assumed to be random variables with an exponential distribution function. The probability that the time to the next failure is less than or equal to a certain amount was given by a cumulative probability function. Repair duration was assumed to be a random variable with a log normal distribution and finally the probability that the duration of a repair be less than or equal to a certain value was given by a log normal cumulative probability function.

Duan and Mays (1990) developed a methodology for reliability analysis of pumping stations. It considers both mechanical and hydraulic failures of pumps and models the available capacity of a pump station as a continuous-time Markov process, using bivariate analysis and conditional probability approaches in a frequency and duration analysis framework. Furthermore, Duan et al. *(1990)* proposed a procedure for design of the pumping system using a reliability based procedure considering both type of failures.

Some availability measures were presented by Cullinane et al. (1992) for components other than pipes as follows:

$$a_{lv} = 0.9278\ D_{lv}^{0.000118} \tag{6.11}$$

$$a_{tk} = 1.000997\ Vol_{tk}^{-0.000118} \tag{6.12}$$

$$a_{lp} = 1.046943\ Q_p^{-0.01634} \tag{6.13}$$

in which a_{lv}, a_{tk} and a_{lp} are the hydraulic availability of a pipe linked with valves, distribution storage tank and pump, respectively. D_{lv} is the diameter of the pipe including the valve (in inches), Vol_{tk}, is the tank capacity and Q_p is the pump's design flow rate (in gpm). Full details of the above formulations can be found in Mays (1989).

6.3.2.1.3 Hydraulic reliability / availability

Hydraulic reliability which can be dependent on mechanical reliability, is a measure of the performance of the distribution system. The hydraulic performance of the distribution system depends on the following factors: 1) interaction between the piping system, distribution storage, and all other ancillary plants like pumps, valves, etc.; 2) reliability of the individual system components; 3) spatial variation of demands in the system and; 4) temporal variation in demands on the system (Cullinane 1989).

Jowitt et al. (1989) pointed out that networks cannot be assessed by the simple pair of (failure, no failure) alone and it is insufficient to consider reliability as the probability of non-connectivity. They asserted that the level of service is a function of probability and consequence, for which the latter was determined from topology, network characteristics, and normal and abnormal performance.

The consequence of a component failure, while it is out of service to be repaired, is that extra flow and head loss are expected in the reduced network. This can lead to hydraulic failure if the shortfall in pressure reaches a critical level. For hydraulic failure, use of a zero-one relationship to describe the effects of nodal pressures on nodal outflows is not appropriate because in reality partial flow can be supplied while pressure is between the desired and minimum levels. A full explanation of this situation was explained in Chapter 5.

Calculation of the hydraulic reliability is said to be difficult. In fact, the problem of finding the probability that each node in a distribution network will receive sufficient supply is extremely difficult to solve (Valliant 1979 and Wagner et al. 1988a). Herein, some of the available indices for hydraulic reliability/availability in the literature, will be reviewed.

A measure of the level of service was used by Germanopoulos et al. (1986). Determination of the duration and frequency of occurrence of interruption in normal supplies to the network was required for assessing the level of service. The level of service was dependent on the frequency of occurrence and the duration of the network breakdowns and source failures that cause these interruptions. These factors are probabilistic in character and the level of service was expressed as the probability that no more than a certain number of interruptions in normal supplies of a given duration will occur over a given period of time. They found that the assessment of supply reliability obtained using the above methodology was considerably different from that suggested by the conventional approach at that time period, which simply relates supply reliability to the amount of emergency storage available in the network. Finally, it was concluded that use of head-driven simulation and extended period models could be important both operationally and in assessing the reliability of water supplies. However, their model was based on demand driven simulation method. They found that the use of normal operating scenarios during failure events could lead to serious reductions in pressures in the network. This fact was illustrated in chapter 5 (see Example 5.1).

Considering possible component failures, Wagner et al. (1988a) introduced a reliability measure as the probability that a given node receives sufficient supply. They asserted that pipes in a distribution network do not actually have a capacity because, generally, the pipe flow rates are determined by the amount of available pressure in the network. To calculate this reliability measure all reduced configurations caused by failures should be analyzed. Therefore, both probability that a given node receives sufficient supply and probability of the configurations which cannot supply the required flow should be considered. A maximum gradient was specified to determine the capacity of each link.

Wagner et al. (1988b) presented a simulation approach to assess the reliability of water distribution networks subject to failure due to pipe breaks and pump outage. This model can be used to calculate a variety of reliability measures relating to the number, location, duration, and effects of failures. The concept of pressure dependency of demand at nodes was considered in the model to account for nodal outflows when nodal heads were inadequate. The calculated reliability values were regarded as approximate because they were based on a finite number of random events. The annual shortfall and the percentage of time spent in the reduced (failure) mode (for every node) were applied as the reliability measures. Finally, the shortfall measure was mentioned as a good overall indicator of the reliability of the system.

Generally, expected shortfalls can be expressed in several different ways. The total expected shortage may be found by summing the nodal shortages as one measure of reliability. However, in some cases, the shortfall at a particular node may be more important. Alternatively, reliability may be indicated as a vector of expected shortages at all nodes, or a demand-weighted sum of these expected shortages. It also may be considered as a fraction of demand which has the advantage of providing a direct comparison with demand.

Lansey et al. (1989) introduced a chance constrained model for a least cost design which attempted to account for the uncertainties in required demands, required pressure heads, and pipe roughness coefficients. These parameters were considered as independent random variables. By the chance constraint formulation the probability that demand and required pressure are equal or greater than design values were restricted to be greater than certain levels. They concluded that inclusion of the uncertainties into the design procedure leads to more reliable design than would be obtained by using an average condition.

Based on the definition of Carey and Hendrickson (1984), Fujiwara and De Silva (1990) and Fujiwara and Tung (1991) measured reliability as the ratio of the expected minimum total shortfall in flow to the total demand, i.e.

$$R = 1 - \frac{\textit{Expected minimum total shortfall in flow}}{\textit{Total demand}} \qquad (6.14)$$

In the former, reliability was improved by increasing link flows (or link capacity) along the longest path from the source to the node. A single source and single demand pattern was used. In both, it was assumed that the probabilities of configurations with multiple failed links were negligible. Then, for each configuration, the maximum flow delivered was obtained by an optimization procedure which maximized nodal flows subject to flow capacity in each link and all hydraulic conditions. Then, the minimum shortfall for the system was calculated by the sum of the shortfall for each state, weighted according to the respective state probabilities as follows:

$$R = 1 - \sum_{l=0}^{NP} p(l) \left(1 - \frac{Q_s^{avl}(l)}{Q_s^{req}} \right) \qquad (6.15)$$

in which p(l) is the probability that link l is unavailable, Q_s^{req} is the total demand and $Q_s^{avl}(l)$ is the total available outflow when link l is unavailable.

An overestimation of reliability values was observed in Fujiwara De Silva (1990), because the maximum flow model did not take into account the pressure requirement at each node and the hydraulic consistency along each loop. On the other hand, considering these two requirements increases the computational time.

Bao and Mays (1990) quantified a measure for system reliability based on hydraulic reliability. The method used random demands, pressure heads and pipe roughnesses generated using a Monte Carlo technique and hydraulic uncertainty due to variability of water demand was regarded by using an appropriate probability distribution for demand and pressure over a time period. Monte Carlo simulation is a suitable technique for evaluation of the reliability of even complex systems. However, it is expensive to run and cannot provide precise estimates of reliability without long run times. This limits its practical application especially within an optimization scheme for water networks. A demand-driven simulation was carried out, then for each node the hydraulic reliability was defined as the probability that the actual nodal pressure is equal to or greater than the minimum required pressure, i.e.

$$R_j = p(H_j > H_j^{des}) = \int_{H_j^{des}}^{\infty} f(H_j)\, dH_j \tag{6.16}$$

where R_j is the nodal reliability at node j, H_j and H_j^{des} are nodal available and minimum required heads at node j, respectively, and $f(H_j)$ is the probability density function.

As one disadvantage, the reduced mode was not recognized by this approach because their demand driven simulation model just satisfied the nodal demands regardless of nodal heads. Therefore, the probability that the available head is in reduced mode, i.e. $p(H_j^{min} < H_j < H_j^{des})$ was not determined by this approach. They used three measures for the system reliability. The first was reliability of the critical node, however, according to many health department rules all nodes must meet head requirements, not just critical ones. The other measures were the average of the nodal reliabilities and the sum of the nodal reliabilities weighted according to the respective demands. The demand-weighted mean of nodal reliabilities was concluded as being the best way to obtain the system reliability.

Bouchart and Goulter (1991) proposed a measure of reliability in which the concept of pipe failure and actual demand exceeding design values was combined. The measure was based upon calculation of the expected volume of deficit associated with each type of network failure. To calculate the expected volumes of deficit, the probabilities of both types of failure and the shortfall in supply caused by those failures were used. In the case of violation in the demand and minimum pressure levels, the volume of deficit was estimated from the difference between design demands and the actual values of outflow.

Cullinane et al. (1992) presented a measure based on hydraulic availability defined as the ability of the system to provide service with an acceptable level of interruption in spite of abnormal conditions. The hydraulic availability was evaluated based on delivery of the specified quantity of water to the appropriate place at the required time under the desired pressure. The availability was defined as the percentage of time that the demand can be supplied at or above the desired pressure. The pressure dependency of demand was also recognized by the model. The overall nodal availability A_j was determined as

$$A_j = \sum_{t=1}^{NT} \frac{a_j(t)\ \Delta t}{TT} \tag{6.17}$$

in which $a_j(t)$ is the nodal availability at time t and node j, Δt is the time interval, TT is the total time and NT is the number of time intervals, respectively. Values of $a_j(t)$ were approximated by a fuzzy function (see Section 5.2.2.2). Then, the system availability, A, was obtained using the arithmetic mean of the nodal availabilities as follows:

$$A = \sum_{j=1}^{NJ} \frac{A_j}{NJ} \tag{6.18}$$

Considering the probability of one link failure, the nodal availability is then stated as

$$A_j = \sum_{l=1}^{NP} \frac{p(l)\, A_j(l)}{NP} \tag{6.19}$$

where p(l) is the probability that link l is unavailable and $A_j(l)$ is nodal availability when link l is unavailable. NP is number of links.

Some points should be raised concerning the above procedure. First, although the nodal availability has been defined as the percentage of time that the nodal pressure is greater than a required value, Eq. (6.17) incorporates the available flow during the time interval. This causes confusion because the reduced pressure mode (i.e. $H_j^{min} < H_j < H_j^{des}$) is considered in calculation of the available outflow. Second, Eq. (6.19) has been obtained by an arithmetic mean formulation over all failure conditions which does not seem to be able to account for the severity of shortfalls during each failure state. Finally, the system availability (Eq. 6.18) has been calculated as the arithmetic mean of the nodal reliabilities which is unable to represent the effects of the magnitude of shortfall during each time period.

In addition, they asserted that since the overall average system availability may be an important indication of performance, any measure of availability should be capable of computing both the availability at a point and the average system availability. However, hydraulic performance at the critical nodes may be more important than the system average, because demand is spatially and temporally distributed. Critical nodes may be those nodes which are closely associated with high economic losses, those that cause a threat to public health if failure occurs, or some zones with low pressures. Following identification of critical nodes, links (pipes, pumps, or tanks) that have high potential for disrupting service at those nodes were identified.

An expected served demand was employed by Fujiwara and Ganesharajah (1993) to measure reliability taking into account both insufficient heads and flows at individual nodes in the network (by adopting a head-outflow relationship). The average value of the maximum effective served system demand relative to the total system demand over all system states was defined as system reliability. The nodal reliability for each demand node was similarly defined. The maximum nodal outflows were evaluated through an optimization procedure instead of a hydraulic simulation model. Their results showed that the system reliability is not a simple arithmetical average of nodal reliabilities and that nodal reliabilities are significantly different from each other. They concluded that the nodal and system reliabilities are very sensitive to both nodal head requirements and the distribution of nodal flows. The major weaknesses of this approach were its complication and also high computational time was required when the system becomes large.

The inadequacy of a network under component failure was presented by Park and Liebman (1993) in two forms: insufficient outflow (shortage) and insufficient head. Redundancy was quantified using the expected shortage due to failure of individual pipes as a measure of reliability in which some consideration of frequency, duration, and severity of failure were incorporated. Shortage was determined directly from the optimization model. Shortages at each node were determined with at least minimum required head. Any nodal head below the specified minimum value under failure condition was set to the minimum head. The model constrained the shortage at each node in the network to be less than or equal to some specified fraction of demand while determining a set of optimal pipe sizes. Their model's deficiency was its inability to include multiple loading condition because of the size of the model.

Tanyimboh (1993) and Tanyimboh and Templeman (1995) proposed a source-head approach (SHM) to calculate reliability of a single source network subject to mechanical and hydraulic failure. In this method a relationship between source head and source outflow (inflow to the system) was applied to determine the flow supplied when service was subnormal. The formulation was as follows:

$$Q_s^{avl} = Q_s^{req} \left(\frac{H_s}{H_s^{des}}\right)^{\frac{1}{n}} \qquad H_s \leq H_s^{des} \tag{6.20}$$

in which Q_s^{avl} and Q_s^{req} were the available and required source flows, respectively. H_s and H_s^{des} were the available and desired heads at source, respectively, and n = 2. The values of the desired source head which satisfy all network demands were obtained applying a demand driven simulation. The reliability was defined as a ratio between the expected total outflows under adequate pressure and the total demand. Then the reliability at each state was obtained as

$$R = \frac{Q_s^{avl}}{Q_s^{req}} \propto \left(\frac{H_s}{H_s^{des}}\right)^{0.5} \tag{6.21}$$

Tanyimboh (1993) asserted that the actual flow supplied when service is subnormal, should be determined using the head driven simulation. He added that the total supply which should be used in the calculation for network reliability, is the sum of the actual nodal outflows. He pointed out that the proposed method can remove the difficulties arising from the interdependencies of nodal reliabilities because network reliability should not be simply calculated by averaging the reliabilities of the individual nodes. However, the above formulation is unable to calculate the real head dependent nodal outflows and represents only an approximate head-outflow relationship at source. Also, because the actual nodal outflows cannot be calculated, the method is not able to evaluate the nodal reliabilities. Therefore, the results of the source head method should be regarded just as an approximation.

Yang et al. (1996b) presented a stochastic simulation for reliability analysis, which used an optimization model as the simulation process. A performance reliability index was used as an indicator for the ability of a network to meet the demands. It was defined as the probability that a chance-constrained criterion for determining the success of a system was met. A user-specified criterion was defined as follows. In at least a certain percentage of the time during the planning horizon, the shortage (expressed as a percentage of demand) at each of the demands of concern must not exceed a certain value. The system was failed if the criterion was not met. Although the performance of a network was also affected by uncertainties in supply and demands, in order to focus attention on system component failure, quantities of supply and demands were assumed deterministic and taken from various forecasting models.

Using a reliability tester, Khomsi et al. (1996) incorporated both mechanical failure caused by pipe breakages and hydraulic failure caused by insufficient pipe capacity into a simple stochastic model. The model identifies nodal pressure under single pipe failure conditions and probabilistic demands, with known probabilities. The probabilities of pressure deficiency were calculated at nodes for which the availability of supply was determined. In this approach a zero-one criterion was described to identify the hydraulic failure caused by pipe failures. i.e. when $H_j < H_j^{des}$ it is considered as a hydraulic failure. Combinations of pipe failures and loading conditions probabilities were accounted for by function of $f_j(nv)$ in the failure mode ($H_j < H_j^{des}$) in which nv is an index for counting violations of minimum desired head at node j. The probability of hydraulic failure at node j, phf_j, was presented by summation of all nodal head violations, i.e.

$$phf_j = \sum_{nv=1}^{NV_{max}} f_j(nv) \tag{6.22}$$

where NV_{max} was number of occasions the nodal heads were violated. The nodal reliability, R_j, (assumed to be the same as the nodal availability, A_j) was obtained as

$$R_j = A_j = 1 - phf_j \tag{6.23}$$

Finally the system reliability (availability) was calculated as the demand-weighted mean of the nodal reliabilities.

The weaknesses of the Khomsi et al. (1996) method can be described as follows: First, the zero-one method is unable to evaluate the reliability in the reduced mode ($H_j^{min} < H_j < H_j^{des}$), therefore, the results are lower bound for nodal reliabilities. Second, use of the demand driven analysis does not allow calculation of actual nodal outflows because they are always taken equal to the demands regardless of the nodal heads. Third, the nodal shortfalls could not be quantified and were not considered by the reliability measure. Forth, the system reliability cannot be calculated without prior calculation of the nodal reliabilities.

Considering the pressure dependency of demand Gupta and Bhave (1994) developed a reliability measure as

$$R = rv\ F_t\ F_n \tag{6.24}$$

in which R is the system reliability, rv is the volume reliability, F_t is time factor and F_n is a node factor, respectively. The factors are determined as follows:

$$rv = \frac{\sum_{t=1}^{NT} \sum_{j=1}^{NJ} Q_j^{avl}(t)\ \Delta t}{\sum_{t=1}^{NT} \sum_{j=1}^{NJ} Q_j^{req}(t)\ \Delta t} \tag{6.25}$$

$$F_t = \frac{\sum_{t=1}^{NT} \sum_{j=1}^{NJ} a_j(t)\ \Delta t}{NJ\ x\ TT} \tag{6.26}$$

$$F_n = \left(\prod_{j=1}^{NJ} R_j \right)^{\frac{1}{NJ}} \tag{6.27}$$

where $a_j(t)$, the availability factor, is equal to 1 if the ratio of Q_j^{avl}/Q_j^{req} for a particular state is equal to or more than an acceptable value and $a_j(t) = 0$, otherwise. Also the nodal reliability was presented as

$$R_j = \frac{\sum_{t=1}^{NT} Q_j^{avl}(t)\ \Delta t}{\sum_{t=1}^{NT} Q_j^{req}(t)\ \Delta t} \tag{6.28}$$

Although this method includes the pressure dependency of demand, because of using the node flow analysis (see Section 5.2.2.2, Eqs. 5.3), it is not able to quantify the partial flows in the reduced mode. Therefore, the values of Q_j^{avl} are obtained approximately. In addition, the availability factor which acts as a zero-one parameter according to a pre-set value for the ratio of Q_j^{avl}/Q_j^{req}, does not recognize the gradual variations of the available nodal outflow from zero to the Q_j^{req} according to the variations of H_j from H_j^{min} to H_j^{des}. Also, difficulties with system reliability measure may arise if any node , however insignificant, has a reliability of zero because it results in a system reliability of zero. Furthermore, it is not clear how the probabilities of the failures at each stage can be incorporated into the reliability measure.

Later, Gupta and Bhave (1996a) used the same approach for a reliability-based design of water distribution systems. In this approach one of the main weaknesses of their last work in (1994) was improved by using a head-outflow relationship which was able to quantify the partial nodal outflows in reduced mode. However, the other disadvantages of the method remained unchanged.

6.3.2.2 Surrogate reliability approaches

Surrogate measures of system reliability have been used because of difficulties in definition and quantification of the reliability concept. Some of these surrogate approaches are redundancy (Bhave 1978; Ormsbee and Kessler 1990), graph theory (Wagner et al 1988a; Jacobs and Goulter 1989; Kessler et al. 1990; Quimpo and Wu 1997) and entropy (Awumah et al. 1990,1991,1992; Tanyimboh and Templeman 1993a, b, c).

Goulter (1987) asserted that improving reliability can be achieved by ensuring redundancy in the network. Redundancy means having extra components e.g. loops, or increasing the size of components and the traditional approach to improve redundancy is to provide loops throughout the system. Observations of Alprovits and Shamir (1977) showed that the optimal design of a network will have a branched configuration unless a minimum permissible diameter or multiple load cases are specified. Also, Bhave (1978) through a simple technique, determined a least cost minimal branched network and simply joined ends of the branches to create redundancy loops.

In water networks, redundancy may be measured by the number of separate hydraulic paths that exist between a source and every demand node. A hydraulic path is a series of pipes which have adequate hydraulic capacity, i.e. provides an acceptable pressure and flow at the demand node. Two paths are distinct if they contain no common nodes other than the initial and final nodes (Ormsbee and Kessler 1990). In the procedure of Rowell and Barnes (1982) and Longanathan et al. (1990) to satisfy the reliability measures, a number of links were added to the system which was supposed to be a spanning tree at first.

Jacobs and Goulter (1988) assessed some different methods used in reliability concept of water distribution systems (e.g. state enumeration methods, filtering methods, heuristics and graph theory). Then the combination of filters with results of graph theory were suggested to obtain better indicators of reliability.

Later, Jacobs and Goulter (1989) presented an optimization-based approach to maximize reliability using some graph theory results. Layouts were defined to be as regular in degree as possible at all nodes while the degree of a node was defined as the number of links connected to it. Maximum regularity was obtained by minimizing the sum of the deviations at each node, in terms of the number of links incident upon it, from the average number of links incident on a node over the whole network.

Kessler et al. (1990) developed a methodology for least cost design of a single source network which adopted an invulnerability degree of two alternative paths calculated by graph theory algorithm. Invulnerability was obtained by a number of separate paths which satisfied demand and minimum pressure at nodes. The need to calculate the actual level of network reliability by assigning a failure probability for each hydraulic component and calculating the failure probability were avoided.

Awumah and Goulter (1992) showed a correlation between entropy and reliability. Their curves for entropy and reliability against cost were quite similar. For the same demands and supply, and for different designs based on the range of layouts, reliability was determined by the average node pair reliability of the network. For a pair of nodes, the node pair reliability was the probability that those nodes were linked.

Through a least cost design of a single source network subject to pipe failure, Tanyimboh and Templeman (1993a, b, c) asserted that some flexibility can be achieved by maximizing the entropy of the flows using an entropy based approach. They claimed that their methodology is able to produce resilient designs without a notable increase in cost. Also, Tanyimboh (1993) introduced the network flow entropy as a good surrogate measure for reliability which can be incorporated into least cost procedures. He observed that the reliability of a network generally increases as the entropy increases and also that the entropy constraint increases the resilience of the network by making the pipes larger and more uniform than they would otherwise be. However, the approach was not applied to networks with components other than pipes.

6.4 SUMMARY AND CONCLUSION

In this chapter a literature review has been carried out on different aspects of the reliability concept. Commencing with general networks in other fields rather than water supply systems, a range of reliability measures have been reviewed. Then, for water distribution networks, a classification has been presented and several reliability indices evaluated. From this, the most important aspects of an overall realistic reliability measure can be summarised as follows:

i) A reliability measure should address both mechanical and hydraulic failures.

ii) It should reflect the amount of required flow that is not supplied (shortfall).

iii) The duration and/or frequency of service interruptions and supply shortfalls should be considered. It must therefore be a time-based measure.

iv) A true reliability assessment must recognize in some way both the probabilistic issues of failure and their effects on the performance of the system.

v) Considering demand failure rather than pipe breaks, the division of supply levels into three modes of failure, i.e. no supply, reduced service, and full service is an important insight into the nature of this type of failure which represents the situation more realistically than just failure/non failure mode consideration. This situation can be regarded only with respect to nodal head-outflow relationship. Therefore, there is need to model pressure dependency of delivered supplies throughout the network.

vi) Regarding the uncertainties and variability in demands through a long period of time a stochastic simulation can take into account the probabilistic nature of demand and lead to more realistic reliability assessment.

vii) The interdependency of nodal reliabilities should be appreciated. Therefore, system reliability should not be a simple arithmetic average of nodal reliabilities.

viii) A reliability measure should be realistic, computationally feasible and easy to implement.

The available reliability indices are not comprehensive and include only some of these capabilities. Therefore, there is a need for a comprehensive realistic reliability measure which considers all of the above-mentioned aspects. The following chapters provide a step by step construction of a time-based reliability model which uses the results of the head driven simulation of water supply networks. In this reliability model, besides the effects of mechanical and hydraulic failures, the probabilistic nature of demand and probabilistic nature of failures are considered. Using extended period simulation, the variability in reliabilities through a period of time is also evaluated.

CHAPTER 7

THE IMPROVED SOURCE HEAD METHOD OF CALCULATING DISTRIBUTION NETWORK RELIABILITY

7.1 INTRODUCTION

Reliability has in recent years been firmly established as an important parameter in distribution network design. It is widely accepted, however, that reliability is difficult both to define and calculate in the context of water distribution. This concept was comprehensively reviewed in the previous chapter. It was concluded that a real reliability measure should address the hydraulic reliability as well as the mechanical reliability. Therefore, it should be able to address the issue of supply shortfall and consider the pressure dependency of demand. Also to be realistic, reliability indices must be computationally efficient and easy to implement. Another aspect which was found to be important is the issue of the strong interdependency between the reliability of demand nodes. However, most available reliability measures do not address this issue and so produce questionable reliability values (Tanyimboh and Tabesh 1997; Fujiwara and Ganesharajah 1993). Furthermore, many of the existing methodologies that attempt to calculate reliability realistically are difficult to implement and/or make high demands on computational resources (Walters and Knezevic 1989; Jacobs and Goulter 1991; Tanyimboh and Templeman 1995). Monte Carlo simulation (Bao and Mays 1990; Wagner et al. 1988b), minimum Cut Set (Su et al. 1987; Yang et al. 1996a), advanced mathematical programming-based methods (Fujiwara and De Silva 1990; Fujiwara and Tung 1991), etc. are some of the examples of such approaches.

Despite initial uncertainty about the essence of water distribution network reliability, it has been established during the last few years that reliability should be a function of actual flow delivered to the required flow (see e.g. Fujiwara and De Silva 1990; Fujiwara and Ganesharajah 1993 and Tanyimboh and Templeman 1995). To do this, the actual outflow delivered under unsatisfactory nodal heads needs to be calculated. However, the available conventional demand driven simulation methodologies are not satisfactory for quantifying partial flow when the nodal head is insufficient. This is primarily due to the fact that existing algorithms for network analysis generally treat nodal outflows as constants with pre-determined values, regardless of nodal head variations.

The spatial nature of the hydraulic performance of distribution networks is another important issue which should be recognised by reliability measures. Early results from pressure-dependent network analysis (Chapter 5) showed that with the use of the head driven simulation method the spatial nature of the hydraulic performance of distribution systems can be represented realistically. According to these results, the consequences of insufficient supply/pressure are normally localised around the mechanically/hydraulically failed components and critical nodes, with conditions elsewhere often being largely unaffected (see also, Bhave 1991; Gupta and Bhave 1996b; Tanyimboh and Tabesh 1997).

There are many water distribution network design test problems in the literature, and many of the well-known test problems are based on small networks often having a single source. Therefore, there is an immediate need for a realistic, easily-implemented, straightforward and fast methodology that can be used to assess the reliability of such simple networks. Additionally, such a measure would considerably simplify reliability-related comparisons between different reliability-based optimal design procedures.

To improve the shortcomings of the existing methodologies, Tanyimboh and Templeman (1995) presented a source head method to calculate the reliability of single source networks which was described in Section 6.3.2.1.3. The source head method (SHM) considers the variations of source flow with the source head in which the source flow is equal to the combined actual outflows of the demand nodes of the single-source network. The modelling effects of lumping the demands according to the foregoing procedure have been detailed in Gupta and Bhave (1996b) where it was concluded that such approaches generally provided approximate rather than accurate results. As such, the SHM can at best be expected to give reasonable estimates of reliability values. On the other hand, network models in which the demand nodes are considered individually are generally more accurate (Gupta and Bhave 1996b). Furthermore, the formulation of Tanyimboh and Templeman (1995) for the SHM was a special case and more significantly, their method could not recognise sufficiently the spatial nature of hydraulic performance in a pipe network.

The aim of this chapter is to present an Improved Source Head Method (ISHM) of approximating nodal outflows and system reliability for single source networks. Unlike the SHM, the present improved source head method uses the values of source head which are required to satisfy the demands of individual nodes to estimate the available flow at those nodes. The theoretical formulation of the ISHM approach to reliability analysis of water distribution networks is based on pressure-driven network analysis principles (see Chapter 5) which relates the available nodal outflows to the available head in the system. However, its practical implementation depends on a novel interpretation and use of the traditional demand-driven analysis results. Furthermore, a comprehensive reliability model is presented to measure the system or nodal reliability and damage tolerances in water supply networks.

It is shown that reliability values calculated using the proposed method (ISHM) are more realistic than the results of the SHM, in comparison with the head driven simulation (HDSM) results. Also, all the identified advantages of the SHM including computational efficiency are retained. The issue of the interdependency between the flows available at the demand nodes and its significance in system reliability calculations is also addressed.

7.2 AVAILABLE FLOW

In this section a general formulation to calculate the available source flow is developed first. Then a new formulation for the nodal outflow based on the source head values is presented.

7.2.1 Available Source Flow

The quantity of water which a distribution network can supply at adequate pressure is one of the principal factors determining the reliability of the system. This point, however, is usually not taken into account in the existing network modelling approaches. Therefore, the relationship between actual nodal outflows and pressure should be incorporated in any realistic network reliability measure (see Chapters 5 and 6). The basic nodal head-outflow relationship was presented in Chapter 5 as follows:

$$H_j = H_j^{\min} + K_j \, (Q_j^{avl})^{n_j} \tag{7.1}$$

The available nodal head and outflow, H_j and Q_j^{avl}, the minimum nodal head, H_j^{min}, in addition to the nodal parameters of K_j and n_j were determined previously (see Chapter 5). It has been shown that a similar approach to the above equation can be applied to small portions of distribution networks (Gupta and Bhave, 1996b), for example, housing estates or industrial complexes, small communities, etc. with a single input point from the main distribution system. Using the pressure-driven analysis approach, it is possible to determine how much water would really be available from a subnetwork such as a DMA with a single feed for any given value of the subnetwork source head or pressure head at the input point of the DMA. Of course, the same is true for small single-source networks in general and underpins the derivation which follows.

The system source head-discharge relationship may be written as

$$H_s = H_s^{\min} + K \, (Q_s^{avl})^{n} \tag{7.2}$$

where H_s represents the available source head. H_s^{min} is the source head below which there would be no outflow at the demand nodes or, conversely, the source head above which outflow begins at least at one demand node. The value of H_s^{min} therefore, corresponds to the smallest H_j^{min} of the network. Also, Q_s^{avl} denotes the sum of the nodal outflows. K and n are a resistance coefficient and an exponent, respectively, for the network. The value of the exponent n, usually between 1.5-2 (Chandapillai, 1991; Gupta and Bhave, 1996b), is determined by calibration. An expression for K is derived shortly.

To determine the value of Q_s^{avl} for any given source head, Eq. (7.2) can be rearranged as

$$Q_s^{avl} = \left(\frac{H_s - H_s^{\min}}{K}\right)^{\frac{1}{n}} \tag{7.3}$$

When $H_s = H_s^{des}$, the desirable source head, $Q_s^{avl} = Q_s^{req}$, the required source flow or total demand. It follows from Eq. (7.3) that

$$Q_s^{req} = \left(\frac{H_s^{des} - H_s^{\min}}{K}\right)^{\frac{1}{n}} \tag{7.4}$$

from which

$$K^{\frac{1}{n}} = \frac{(H_s^{des} - H_s^{\min})^{\frac{1}{n}}}{Q_s^{req}} \tag{7.5}$$

Substituting for K in Eq. (7.3) gives the available flow as

$$Q_s^{avl} = Q_s^{req} \left(\frac{H_s - H_s^{\min}}{H_s^{des} - H_s^{\min}}\right)^{\frac{1}{n}} \qquad H_s^{\min} \leq H_s \leq H_s^{des} \tag{7.6}$$

As mentioned in Section 6.3.2.1.3, Tanyimboh and Templeman (1995) have previously presented a similar equation to Eq. (7.6). However, this was in reality a special case of Eq. (7.6) in which $H_s^{min} = 0$ and $n = 2$. Furthermore, Wagner et al. (1988b) presented a corresponding equation for individual nodes, but without a formal derivation. The desired source head, H_s^{des} is the required source head which fully satisfies all the nodal demands. Using a demand driven simulation, values of head loss at each link are produced for any normal or reduced network configuration. Then, the desired source head is obtained by summation of these head losses along each path from the critical node to the source.

If there are tree-type portions in the network or if two or more simultaneous link failures are considered in the analysis, then the possibility of demand node isolation due to link unavailability exists and should be addressed. Thus, if a reduced network configuration is such that some nodes are disconnected from the rest of the network, the available flow at source is given by

$$Q_s^{avl} = (Q_s^{req} - Q_{isol}) \left(\frac{H_s - H_s^{\min}}{H_s^{des} - H_s^{\min}}\right)^{\frac{1}{n}} \qquad H_s^{\min} \le H_s^{avl} \le H_s^{des} \tag{7.7}$$

in which Q_{isol} represents the combined demand of the isolated nodes. H_s^{des} is calculated for the full demands of the reachable nodes. H_s^{min} may or may not have a different value from that in Eq. (7.6), depending on the nodes which are isolated. In other words, H_s^{min} in Eq. (7.7) refers to the reduced network. If $Q_{isol} = 0$ then Eq. (7.7) reduces to Eq. (7.6). The original source head method (SHM) uses either Eq. (7.6) or Eq. (7.7) to calculate the available source flow.

7.2.2 Nodal Outflow

Regarding the concepts of pressure dependency of demand (Chapter 5), nodal head-outflow relationship (Eq. (7.1)) and formulation of the available source flow (Eq. (7.6)) for a single source network, the head-outflow relationship can be approximated so that nodal outflows are related to the source head as follows,

$$Q_j^{avl} = Q_j^{req} \left(\frac{H_s - H_{s,j}^{\min}}{H_{s,j}^{des} - H_{s,j}^{\min}}\right)^{\frac{1}{n_j}} \qquad H_{s,j}^{\min} \le H_s \le H_{s,j}^{des} \quad ; \forall j \tag{7.8}$$

where Q_j^{req} and Q_j^{avl}, respectively, represent the demand and actual outflow at node j. H_s is the available source head. $H_{s,j}^{min}$ is the source head below which outflow at node j is zero. $H_{s,j}^{des}$ is the source head above which the demand at node j is fully satisfied. n_j is an exponent whose value, usually between 1.5 and 2, can be determined by calibration (Gupta and Bhave 1996b).

To calculate the values of the Q_j^{avl} a demand-driven analysis of the network is first performed using the demand values, Q_j^{req}, to obtain the nodal heads, H_j. Then the desirable source head to satisfy full demand at node j is determined as

$$H_{s,j}^{des} \approx H_s - H_j = H_j^{\min} + \sum_{ij \in IJ_d} h_{ij} \qquad ; \forall j \tag{7.9}$$

in which H_j represents the available head at node j. h_{ij} is the head loss of pipe ij and IJ_d is the set of all pipes in a specified path between the source and demand node j. The minimum nodal head (H_j^{min}) below which no flow can be discharged may be taken as the minimum outlet level in the locality served by the node. In the absence of field data it may be set equal to ground elevation.

The value of $H_{s,j}^{min}$, the source head corresponding to zero outflow at demand node j, can be found using pressure-driven simulation or field tests. Inherently, however, the present formulation does not have the full capabilities of pressure-driven analysis. The $H_{s,j}^{min}$ values can also be approximated using H_s^{min}, the source head above which outflow just begins at any node of the network or taken as the elevation of the lowest node.

7.3 RELIABILITY ANALYSIS

As well as the hydraulic reliability, mechanical reliability should also be considered. To calculate the mechanical reliability the random nature of any component (pipe) failure should be accounted for by a suitable measure (see Chapter 6). This section presents a reliability model which includes different pipe availability formulations, in addition to nodal and system reliability.

7.3.1 Component Availability/Unavailability

Pipe availability, a_l can be considered in different ways (see Section 6.3.2.1.2). The reliability model developed for this research incorporated a number of link (pipe) availability/reliability formulations (mostly introduced in Chapter 6) as follows.

i) The Cullinane et al. (1992) formulation in which the concept of the mean time to failure and duration of repair time is included, as follows:

$$a_l = \frac{0.21218\ D_l^{1.462131}}{(0.00074 D_l^{0.285} + 0.21218 D_l^{1.462131})} \quad ; \forall\ l = 1, ..., NP \qquad (7.10)$$

in which a_l is availability of link (pipe) l, D_l is pipe diameter (in inches) and NP is the number of links (pipes).

ii) The Fujiwara and Tung (1991) pipe availability formulation in which the concept of repair and failure rates are incorporated, is given by

$$a_l = \frac{0.64}{0.64 + L_l\,(0.005485 - 0.0000175\,D_l)} \quad ; \forall\, l = 1, ..., NP \qquad (7.11)$$

where L_l is the pipe length in (km) and D_l is in (mm), respectively, for which $100 \leq D_l \leq 300$. Also, days are used as the unit of time and the value of 0.64 denotes the number of repairs per day.

iii) A Poisson-based formulation for determination of the mechanical reliability (availability) of link (pipe) l as follows (Su et al. 1987):

$$a_l = e^{(-L_l \cdot \mu_l)} \quad ; \forall\, l = 1, ..., NP \qquad (7.12)$$

in which L_l is pipe length (in miles) and μ_l is the number of breaks/mile/year in pipe l. Using failure data obtained from the city of St. Louis, a regression equation was performed by Su et al. (1987) to determine the number of breaks as follows:

$$\mu_l = \frac{0.6858}{D_l^{3.28}} + \frac{2.7158}{D_l^{1.3131}} + \frac{2.7685}{D_l^{3.5792}} + 0.042 \quad ; \forall\, l = 1, ..., NP \qquad (7.13)$$

where D_l is pipe diameter of pipe l (in inches).

iv) Having the rate of pipe breaks per km length of pipe per year, μ_l, by averaging the rate of pipe breaks for the cities of New York, Philadelphia and St. Louis, Khomsi et al. (1996) presented the mean probability of failure of pipe l for a day (i.e. $\mu_l.L_l$ / 365) for a set of pipe diameters. These values can be seen in Table 7.1. Arising from this data, the reliability (availability) of pipe l (i.e. the probability that pipe l functions) is

$$a_l = 1 - \frac{\mu_l \cdot L_l}{365} \qquad (7.14)$$

in which D_l is the pipe diameter (in mm) and L_l is the pipe length (in km).

All the above formulations are applied in Chapters 7 and 8 wherever is appropriate and the corresponding results for reliability will be compared.

7.3.2 Nodal and System Reliability

For any given source head, the available flow obviously depends on the configuration of the distribution system, i.e. whether the system is in a reduced state or not. The components (pipes) of the distribution network can be unavailable for use due to failure/bursts and/or repair/maintenance, etc. It is commonly assumed that pipe bursts are not interdependent (Su et al. 1987; Fujiwara and Tung, 1991; etc.). With this assumption, the probability p(0) that all pipes are available is

$$p(0) = \prod_{l=1}^{NP} a_l \tag{7.15}$$

in which a_l is the probability that pipe (link) l is available and NP is the number of links. Also, the probability that only M specified links are unavailable, p(M), while the remaining (NP-M) are available is given by (Tanyimboh 1993)

$$p(M) = p(0)\prod_{l=1}^{M} \frac{ua_l}{a_l} \qquad M = 1, ..., NP \tag{7.16}$$

in which ua_l denotes the unavailability of component l (= 1 - a_l). For any given normal or subnormal configuration, the nodal reliability can be defined as the ratio of the available flow to the required flow at each node, i.e.

$$r_j(M) = \frac{Q_j^{avl}(M)}{Q_j^{req}} \qquad M = 0, ..., NP \quad ; \forall j \tag{7.17}$$

in which $r_j(M)$ is the reliability of node j for the given configuration with M specified pipes unavailable and $Q_j^{avl}(M)$ is the available flow at node j when the M specified pipes are unavailable and is obtained from Eq. (7.8). Using Eqs. (7.15-17) and taking all network configurations into consideration, the reliability R_j of the node is given by the expectation of the nodal reliabilities for the various configurations, i.e.

$$R_j = p(0)\; r_j(0) + \sum_{l=1}^{NP} p(l)\; r_j(l) + \sum_{l=1;\; m\neq l}^{NP} p(l,m)\; r_j(l,m) + \sum_{l=1;\; m\neq l;\; n\neq l;\; n\neq m}^{NP} p(l,m,n)\; r_j(l,m,n) + \ldots \tag{7.18a}$$

For the purpose of computational efficiency the above equation can be represented as follows.

$$R_j = p(0) \sum_{M=0}^{NP} \left(r_j(M) \prod_{l=1}^{M} \frac{ua_l}{a_l} \right) \tag{7.18b}$$

If no more than two simultaneously unavailable links are considered, then a lower bound to reliability, $R_{j,L}$ is given by

$$\begin{aligned} R_{j,L} &= \frac{1}{Q_j^{req}} \left(\sum_{l=0}^{NP} p(l)\; Q_j^{avl}(l) + \sum_{l=1;\; m\neq l}^{NP} p(l,m)\; Q_j^{avl}(l,m) \right) \\ &= p(0) \left(r_j(0) + \sum_{l=1}^{NP} r_j(l) \frac{ua_l}{a_l} + \sum_{l=1;\; m\neq l}^{NP} r_j(l,m) \frac{ua_l}{a_l} \frac{ua_m}{a_m} \right) \quad ; \forall j \end{aligned} \tag{7.19}$$

in which p(l) and p(l,m) are the respective probabilities that only pipes l and both l and m, together, are unavailable. Similarly, $r_j(l)$ and $r_j(l,m)$ are the respective nodal reliabilities with pipe l and both pipes l and m simultaneously unavailable. $Q_j^{avl}(l)$ and $Q_j^{avl}(l,m)$ are the respective available flows at node j with pipes l, and both l and m, together, unavailable. Eq. (7.19) represents lower bound reliability because further terms are added if more than two simultaneous link failures are considered. The nodal unreliability U_j, can be determined from Eq. (7.19) simply by replacing the available flow, Q_j^{avl}, by shortfall in supply, (Q_j^{req} - Q_j^{avl}). Thus an upper bound to nodal reliability, $R_{j,U}$ is given by 1 - U_j, i.e.

$$\begin{aligned} R_{j,U} &= 1 - \frac{1}{Q_j^{req}} \left(\sum_{l=0}^{NP} p(l)\; (Q_j^{req} - Q_j^{avl}(l)) + \sum_{l=1;\; m\neq l}^{NP} p(l,m)\; (Q_j^{req} - Q_j^{avl}(l,m)) \right) \\ &= 1 - p(0) \left(u_j(0) + \sum_{l=1}^{NP} u_j(l) \frac{ua_l}{a_l} + \sum_{l=1;\; m\neq l}^{NP} u_j(l,m) \frac{ua_l}{a_l} \frac{ua_m}{a_m} \right) \quad ; \forall j \end{aligned} \tag{7.20}$$

in which $u_j = 1 - r_j$. $R_{j,U}$ together with the lower bound of reliability, Eq. (7.19), can be used to determine the point in the summation of Eq. (7.18a) beyond which further terms need not be included for a given accuracy. That is, when for a certain value of M, the values of $R_{j,L}$ and $R_{j,U}$ are identical or very close to each other, there is no need to consider the simultaneous failures of more than M links.

The reliability, R, of the network as a whole can be determined in a similar way by simply writing Eqs. (7.17-20) without the subscript j. To this end,

$$Q_s^{req} = \sum_{j=1}^{NJ_d} Q_j^{req} \tag{7.21}$$

$$Q_s^{avl} = \sum_{j=1}^{NJ_d} Q_j^{avl} \tag{7.22}$$

in which Q_s^{req} and Q_s^{avl} are, respectively, the sum of the nodal demands and available flow and NJ_d represents the number of demand nodes.

Finally, besides the reliability values, the proposed reliability model is capable of calculating the damage tolerance values. Damage tolerance represents the ability of the system to continue functioning even under both mechanical and hydraulic failure conditions. Tanyimboh and Templeman (1995) state: "High values of damage tolerance represent high degree of redundancy, i.e. low vulnerability to component failure. This redundancy could be a combination of alternative supply paths to demand points, large diameter pipes with additional capacity, nearby service reservoirs with emergency storage, etc.".

Using the definition of Tanyimboh (1993) for lower bound system damage tolerance, if the second and third terms of Eq. (7.19) which represent the probability that node j functions with unavailability of up to two links in the network, is divided by the probability (1-p(0)) that the network is not fully connected, the lower bound damage tolerance at node j, $T_{j,L}$, is obtained as

$$T_{j,L} = \frac{p(0)\left(\sum_{l=1}^{NP} r_j(l)\frac{ua_l}{a_l} + \sum_{l=1;\, m\neq l}^{NP} r_j(l,m)\frac{ua_l}{a_l}\frac{ua_m}{a_m}\right)}{1 - p(0)} \quad ; \forall j \tag{7.23}$$

Combining Eqs. 7.19 and 7.23 gives the following formulation for $T_{j,L}$.

$$T_{j,L} = \frac{R_{j,L} - r_j(0)\, p(0)}{1 - p(0)} \tag{7.24}$$

Similarly, the upper bound damage tolerance at node j, $T_{j,U}$, is obtained as

$$T_{j,U} = \frac{R_{j,U} - r_j(0)\, p(0)}{1 - p(0)} \tag{7.25}$$

The T_L and T_U, lower and upper bounds for system damage tolerance can also be calculated in a similar way by dropping the index of j from Eqs. 7.24 and 7.25.

7.4 APPRAISAL

To demonstrate the advantages of the improved source head based reliability measure, the network of Figure 7.1 is used in which H_s^{min} = 0 m, H_s = 100 m and n_j = 2. This simple symmetric four-loop network was previously used in Chapter 5 (Example 5.2) and has also been used by a number of researchers to represent several aspects of design and reliability. In addition, this example demonstrates the sensitivity of the proposed reliability measure using a progression of designs for the network taken from Fujiwara and Tung (1991). The designs therein are obtained by generating minimum cost designs respectively satisfying a progression of specified levels of reliability. Reliability is calculated therein as the ratio of the expected maximum flow supplied to the total system demand for all network configurations with up to two simultaneous link failures. The maximum available nodal outflow is obtained using an optimization procedure while a maximum hydraulic gradient limit of 0.01 is imposed. Pipe data for the 16 reliability-constrained minimum cost designs are given in Table 7.2. By using these designs, it is relatively easy to judge whether any reliability indicator is consistent and, therefore, reliable. Also as shown shortly, the hydraulic performance of the designs cannot be properly simulated using demand-driven analysis because there is insufficient pressure in the system to drive the required flows.

Fortran 77 programs were written for the network analysis (Newton-Raphson method) for both the demand and head-driven simulation methods and the reliability calculations. The ISHM uses the demand driven simulation method while applying the head-flow formulation of Section 7.2. The programs were run using a 75 MHz Pentium PC with 8 Mbyte RAM.

7.5 DISCUSSION OF RESULTS

For a better understanding of the hydraulic behaviour of the respective designs, the following indicators of hydraulic performance are calculated. First, the total available flow (i.e. flow supplied at available source head) for each design is calculated using the head-driven analysis (as described in Chapter 5) and the results are presented in Table 7.3. This table also includes results for two less than normal source heads. It can be seen that for all values of source head, i.e. 100, 80 and 50 m the network is deficient and the required source flow of 0.2081 m^3/sec cannot be satisfied. The actual consumptions increase from design 1 to 16. This actual abstraction parameter shows that hydraulic performance is improving from design 1 to 16. It can therefore be concluded that any measure of reliability of the designs should increase from designs 1 to 16. Second, the required source heads to satisfy the full demand at individual node j for the fully connected network are calculated by the ISHM. The results are shown in Table 7.4. Also, Table 7.5 presents fractions of nodal demands satisfied by the available source head, i.e. 100 m, in the fully connected network using the ISHM. It can be observed from Tables 7.4 and 7.5 that the shortfall in supply due to insufficient pressure is localised around the nodes 6, 8 and 9 while supply elsewhere is not affected. Therefore, by considering the demand nodes individually, the ISHM recognises the spatial characteristics of the distribution system. As such, the ISHM has successfully addressed the main weakness of the source head approach. This interesting feature of the ISHM will be demonstrated further in the forthcoming results.

Because the layout of the designs does not change, their mechanical reliabilities can be compared using p(0), the probability that no component is unavailable. The p(0) values obtained from Eq. (7.15) can be seen in Table 7.5, as well. The results show that all the designs are deficient, however, they form a gradual and smooth progression. Furthermore, it also follows that the mechanical reliability of each design is greater than that of its predecessor, as numerous studies have shown that large diameter pipes are generally more reliable. The approximations to actual system flow given by the ISHM, compared with the SHM and HDSM are presented in Table 7.6 and shown graphically in Figure 7.2. Table 7.6 shows that while the SHM underestimates the actual flow delivered by about 25%, the ISHM overestimates it by only about 5% in this example. It is reminded that in the HDSM the nodal outflows are obtained directly by the head-outflow relationship at any individual demand node. However, the ISHM calculates the nodal outflows indirectly according to variations of the source head.

For better comparison between the ISHM and the SHM, H_s^{des}, the required source head values to satisfy full demands with single isolated pipes are given in Tables 7.7 and 7.8. It can be seen that the values of the required source head by the SHM is equivalent to the required source head to satisfy full demand at the critical node (i.e. node 9, Table 7.8). Therefore, to satisfy the full demand in other nodes, smaller source head values are required which can be obtained by the ISHM as given in Table 7.8. It can also be observed that the required source head for full demand satisfaction (Table 7.7) decreases from designs 1 to 16 for each isolated pipe. This is the expected result because each of the 16 designs is obtained from the preceding one by increasing only two pipe diameters (each by 5 mm). It can therefore be said, confidently, that the hydraulic performance of each design is superior to that of its predecessor. Consequently, the ISHM can produce the values of nodal outflow using the required heads of Table 7.8.

The fractions of total demand satisfied by the available source head from ISHM are shown in Table 7.9. The results show the increase of the nodal outflows following the decrease of the required source head for each isolated pipe. As can be seen, the maximum shortfall occurs when pipes 1-2 (or 1-4) with the largest diameter fail. Alternatively for the internal pipes 2-5 and 4-5, for which the lowest source heads are required, the maximum source flow is supplied.

Considering up to two simultaneous pipe failures, the lower and upper bounds of reliability and damage tolerance have been calculated by the ISHM using appropriate equations from Section 7.3.2. The results are shown in Table 7.10 as the averages of the upper and lower bounds. It is seen that the upper and lower bounds are identical if up to two simultaneous pipe failures are considered for the Cullinane et al. (1992) pipe availability formulation (Eq. 7.10). The reliability values calculated by averaging the lower and upper bounds derived using single-link failures only are virtually identical to the reliability values determined using one- and two- link failures (Table 7.10) with the maximum difference being 3 x 10^{-6}. It may be noted that the former values are all higher than the latter. The above two observations also apply to the damage tolerance values, except that the differences are somewhat larger ($\approx 5.5 \times 10^{-4}$ or less). The values of damage tolerance show the severity of the component failures to the hydraulic performance of the system. Table 7.10 shows a difference of about 10% between the R and T values which represents the sensitivity of the network to failure conditions.

As demonstrated in Table 7.10, the reliability and damage tolerance values for up to two simultaneous pipe failures are very close to the results of one pipe failure, when expressed as the average of the lower and upper bounds. Therefore, it can be concluded that expressing the results as $(R_L + R_U)/2$ or $(T_L + T_U)/2$ for one pipe failure, has the advantage of considerable computational time saving while the R and T values are estimated quite accurately. Bear in mind that in this case for up to two simultaneous pipe failures, 78 simulations are required in comparison with only 12 simulations for the one pipe failure case. This conclusion is practically valuable when the method is applied to realistic size networks.

Table 7.11 and Figure 7.3 show the reliability and damage tolerance values from the ISHM compared with values from the SHM. The reliability values from Fujiwara and

Tung (1991) are also given in Table 7.11, Although a fair correlation is seen between the ISHM and Fujiwara and Tung (1991) reliability values, they are not directly comparable because pipe availability and nodal outflows are defined in different ways. For instance, the nodal outflows have been calculated by an optimization procedure with certain constraints such as restriction of the maximum allowable hydraulic gradient to a fixed value. It is clear from Table 7.11 and Figure 7.3 that the present method (ISHM) gives significantly higher values of reliability and damage tolerance than the previous formulation (SHM). The reason for the difference is that the ISHM gives more accurate values of flow delivered at available source head than the SHM (see Table 7.6). A major advantage of the ISHM (unlike the SHM) is its ability to provide nodal reliability values, which are shown in Table 7.12. As seen earlier, again Tables 7.11 and 7.12 clearly show that the reliability value given by the original SHM is in reality an approximation to the reliability of the most critical node, i.e. node 9, in the present example. Also, it is worth noting that there is a good correlation between the available flow values of the approximate SHM and ISHM methods and the more accurate head-driven simulation method (HDSM) as demonstrated graphically in Figure 7.4.

For comparison, the system reliability values are also shown in Table 7.12 along with the arithmetic and demand-weighted means of the nodal reliabilities. It can be seen that the nodal means are different from the real reliability values of each network (also see Fujiwara and Ganesharajah 1993) though the demand-weighted means are remarkably similar to the system reliability values. An important reason for these differences is that the nodal outflows (and, hence, their reliabilities) are not mutually independent (Tanyimboh 1993 and Tanyimboh and Tabesh 1997). This issue is addressed herein by first calculating the nodal outflows, the sum of which is then used to calculate system reliability (Tanyimboh 1993). In this way, double counting was avoided.

Also, Table 7.13 shows the system damage tolerance values along with the arithmetic and demand-weighted means of the nodal damage tolerance values. The same as Table 7.12, the demand-weighted mean of the damage tolerance values are very close to the system damage tolerances. In comparison with Table 7.12, the severity of pipe failures on the hydraulic performance of the system are well illustrated. About 14% difference in node 9, 10-19% in nodes 6 and 8, 5% in nodes 3 and 7, 3% in node 5 and 2% in nodes 2 and 4 can be seen between the nodal damage tolerance and reliability values. High values of the nodal reliabilities (except node 9) indicate that the probability of failures are low. However, the severity of failures, when they occur, is high especially in node 6, 8 and 9.

Table 7.14 shows the nodal reliability values for the fully connected network, $r_j(0)$, using the ISHM. It represents further evidence that the shortfall is localised around the critical node and confirms the fact that the SHM reliability results (Table 7.11) are actually the reliability of the critical node.

To illustrate differences of the results using different pipe availability formulations, presented in Section 7.3.1, Tables 7.15 and 7.16 show the values of system reliability and damage tolerance arising from Eqs. 7.10-14, respectively. Among the four applied pipe availability formulations reliability results the Cullinane et al. (1992), Fujiwara and Tung (1991) and Khomsi et al. (1996) equations show a strong correlation. However, a significant discrepancy is observed between the results of Su et al. (1987) with the other equation's results (about 34% and 29% in R and T values, respectively). Actually, this equation results in very small values for the lower bound reliabilities. For instance, R_L varies from 0.202358 to 0.2689 for designs 1 to 16. Also, the lower bound damage tolerance value varies from 0.159478 to 0.21101, respectively. One possible reason perhaps is that the data set which has been used by Su et al. (1987) to produce the pipe break rate is imperfect. Also, it can be seen that both R and T values from the Su et al. (1987) equation are very close to each other which indicates that the network is not able to function well during normal and failure conditions. This, however, is not realistic and is caused by the formulation's weaknesses.

Figures 7.5 and 7.6 demonstrate the reliability and damage tolerance results based on different pipe availability formulations. Therefore, it can be concluded that except the

Su et al. (1987) formulation, any of the other pipe availability equations of Cullinane et al. (1992), Fujiwara and Tung (1991) and Khomsi et al. (1996) can be used in the reliability analysis. However, of the three remaining equations the Cullinane et al. (1992) formula is the most reliable one. The reason is that the Fujiwara and Tung (1991) formula (Eq. 7.11) has a limitation of 100-300 mm for pipe diameters. Also, the Khomsi et al. (1996) formula (Eq. 7.14) is discontinuous in pipe diameters in which interpolation has to be made for values between known diameters (Table 7.1). On the other hand, the mean probability for pipe diameter of 250 mm seems unreasonable because naturally the larger the diameter, the lower the probability of pipe failure. The validity of this conclusion will be examined further in Chapter 8.

Finally, the efficiency of the method has been reported in Table 7.17 in terms of CPU times. The obvious inference from the reported CPU time (i.e. less than 2.64 seconds with 78 failure cases) is that the computational efficiency of the proposed formulation is very good.

7.6 SUMMARY AND CONCLUSION

This chapter has formulated the improved source head method of calculating distribution network reliability. Using various parameters it has been shown that the designs appraised herein form a good progression in terms of hydraulic performance and reliability. They would therefore be suitable as a test bed for any reliability measure proposed in future.

In order to ensure fair reliability comparisons between different water distribution systems, the present formulation retains the requirement that demands be fully satisfied throughout the network, and this is achieved using demand-driven simulation to determine nodal heads. Then, calculating the desirable source head to satisfy full demand at each node, the available flow at individual nodes is found using an approximation to the relationship between nodal outflow and source head. By so doing, the proposed (ISHM) method allows for spatial variations in the performance of the distribution network. This leads to much improved estimates of system reliability compared to the previous (SHM) formulation. Thus the ISHM has successfully addressed the main weakness of the source head approach to reliability calculation. Furthermore, it has the advantage that nodal reliability values can also be computed for any nodes of particular interest.

It has also been demonstrated that instead of calculating the reliability of a large number of simultaneous component failures, it is sufficient to express the reliability as the mean of the lower and upper bounds of a few simultaneous component failures which leads to significant saving of CPU time. For instance, for a 4 loops network with 9 nodes and 12 pipes, consideration of $(R_L+R_U)/2$ for just one pipe failure led to 77% decrease in the CPU time in comparison with the two simultaneous pipe failures case. Furthermore, it has been shown that the reliability value of a distribution network is not equal to the arithmetic mean of the respective nodal reliability values. However, the demand-weighted mean which reflects the relative importance of the nodal demands turned out to be very close to the calculated system reliability values for the examples considered.

The efficacy and excellent computational efficiency of the method have also been demonstrated. A key feature of the proposed formulation is that, both conceptually and in terms of implementation, it is a very simple realistic method for calculating distribution network reliability. The simulations of network performance following pipe failures can be carried out using any programme which use the conventional demand driven simulation method. Finally, it has been illustrated that the improved source head method has very good correlation with results of the head-driven analysis, which is believed to be the most realistic one. In comparison with the head driven simulation method, the indirect approach of the ISHM requires nearly the same CPU time to calculate the nodal outflows. However, its results show an overestimation in a range of 5-10% (based on the results from a few networks) and its use is limited to the single source networks.

For the mechanical availability of pipe failures, different formulations were applied in this chapter. Formulations of Cullinane et al. (1992); Fujiwara and Tung (1991) and

Khomsi et al. (1996) produced very close reliability results. However, results from Su et al. (1987) formulation was found to be significantly different from does of other formulations and it can therefore be concluded that this equation is not suitable for calculation of the pipe availability, especially for lower bound reliabilities. Among the former equations, the formulation of Cullinane et al. (1992) appears to be the most reliable.

Table 7.1: Mean probability of pipe failures (Khomsi et al. 1996).

Diameter (mm)	Mean probability (km^{-1} day^{-1})
100	0.000901
150	0.000468
200	0.000192
250	0.000370
300	0.000107
350	0.000107
400	0.000071

Table 7.2: Pipe data of the network of Fig. 7.1.

Design	Pipe						Cost[b] ($ 10^6)
	1-2, 1-4[a]	2-3, 4-7	2-5, 4-5	3-6, 7-8	5-6, 5-8	6-9, 8-9	
1	250	175	145	115	100	100	0.255
2	250	175	145	115	105	100	0.257
3	250	180	145	115	105	100	0.259
4	250	180	145	120	105	100	0.261
5	250	180	145	125	105	100	0.263
6	250	185	145	125	105	100	0.266
7	250	185	145	130	105	100	0.268
8	250	185	145	135	105	100	0.270
9	250	190	145	135	105	100	0.272
10	250	190	145	140	105	100	0.274
11	250	190	145	140	110	100	0.276
12	250	190	150	140	110	100	0.278
13	250	190	150	140	115	100	0.280
14	255	190	150	140	115	100	0.283
15	255	190	155	140	115	100	0.285
16	255	190	155	140	120	100	0.287
	Diameter (mm)						

[a] The network and all designs are symmetrical about the line joining nodes 1, 5 and 9; Length = 1000 m and CHW = 130 for all pipes. [b] From Fujiwara and Tung (1991).

Table 7.3: Pressure-dependent outflows for different source heads.

Design	Source Head (m)		
	100	80	50
1	0.171951	0.164525	0.145240
2	0.173044	0.166080	0.156221
3	0.173323	0.166581	0.147354
4	0.174264	0.168615	0.149245
5	0.174327	0.168595	0.151073
6	0.174653	0.168891	0.151360
7	0.175417	0.169752	0.151705
8	0.176089	0.170640	0.152519
9	0.176424	0.170898	0.153642
10	0.177033	0.171625	0.154581
11	0.177137	0.172049	0.155661
12	0.177361	0.172108	0.155718
13	0.177711	0.172457	0.156373
14	0.178042	0.172801	0.157078
15	0.178263	0.173019	0.157486
16	0.178822	0.173380	0.158135
	Total consumption (m^3/s)		

Table 7.4 : Required source heads to satisfy full demand at individual nodes for fully connected network (ISHM).

Design	Node		
	2-5, 7	6, 8	9
1	< 100	120.3	277.6
2	< 100	113.2	270.5
3	< 100	111.3	268.6
4	< 100	103.1	260.4
5	< 100	< 100	253.2
6	< 100	< 100	251.4
7	< 100	< 100	245.1
8	< 100	< 100	239.6
9	< 100	< 100	237.8
10	< 100	< 100	232.9
11	< 100	< 100	230.1
12	< 100	< 100	229.0
13	< 100	< 100	226.3
14	< 100	< 100	224.8
15	< 100	< 100	223.7
16	< 100	< 100	221.2
	Require source head (m)		

Table 7.5: Fractions of nodal demands satisfied by fully connected network and mechanical reliability (ISHM).

Design	Node			p(0)
	2-5, 7	6, 8	9	
1	1.000	0.912	0.600	0.993972
2	1.000	0.940	0.608	0.994049
3	1.000	0.948	0.610	0.994072
4	1.000	0.985	0.620	0.994129
5	1.000	1.000	0.628	0.994182
6	1.000	1.000	0.631	0.994204
7	1.000	1.000	0.639	0.994251
8	1.000	1.000	0.646	0.994296
9	1.000	1.000	0.648	0.994316
10	1.000	1.000	0.649	0.994357
11	1.000	1.000	0.659	0.994427
12	1.000	1.000	0.661	0.994461
13	1.000	1.000	0.665	0.994524
14	1.000	1.000	0.667	0.994535
15	1.000	1.000	0.669	0.994568
16	1.000	1.000	0.673	0.994625
	Proportion of total demand satisfied			

Table 7.6: Fractions of total demand satisfied by the available source head at fully connected network (equivalent to the values of r(0)).

Design	Fraction of total demand satisfied			Error[a] (%)	
	SHM	ISHM	HDSM	SHM	ISHM
1	0.600156	0.862239	0.826290	-27.40	4.35
2	0.608066	0.870303	0.831543	-26.87	4.66
3	0.610132	0.872460	0.832883	-26.74	4.75
4	0.619686	0.882734	0.837405	-26.00	5.41
5	0.628385	0.888390	0.837808	-25.00	6.04
6	0.630685	0.889082	0.839274	-24.85	5.93
7	0.638796	0.891519	0.842946	-24.22	5.76
8	0.646091	0.893710	0.846175	-23.65	5.62
9	0.648594	0.894445	0.847785	-23.50	5.50
10	0.655285	0.896468	0.850711	-22.97	5.38
11	0.659220	0.897650	0.851211	-22.56	5.46
12	0.660846	0.898140	0.852287	-22.46	5.38
13	0.664712	0.899298	0.853969	-22.16	5.31
14	0.666996	0.899986	0.855560	-22.04	5.19
15	0.668625	0.900475	0.856622	-21.95	5.12
16	0.672434	0.901619	0.859308	-21.75	4.92
Mean Error				-24.01	5.30

[a] % of corresponding HDSM value

Table 7.7: Required source heads to satisfy full demands with single isolated pipes (SHM).

Design	Isolated Pipe[a]					
	1-2, 1-4	2-3, 4-7	2-5, 4-5	3-6, 7-8	5-6, 5-8	6-9, 8-9
1	561.946	566.601	304.845	422.845	341.004	808.534
2	555.453	514.933	300.758	391.264	335.845	789.016
3	552.586	513.511	298.388	389.996	332.274	785.974
4	535.351	505.554	286.513	383.464	312.663	768.162
5	519.474	498.494	278.289	377.675	296.537	752.582
6	516.308	497.004	273.936	376.352	293.237	749.456
7	501.293	490.639	265.004	371.133	279.841	735.632
8	487.708	485.029	257.328	366.537	268.762	723.593
9	484.334	483.515	255.049	365.200	265.758	720.473
10	471.924	478.477	248.346	361.074	256.493	709.814
11	467.358	440.566	247.175	338.955	254.660	701.476
12	446.058	434.617	244.860	334.703	254.316	700.619
13	442.159	402.747	243.708	316.187	252.557	692.722
14	436.575	400.867	241.919	314.462	250.984	691.021
15	417.782	395.548	239.669	310.660	250.656	690.162
16	414.408	368.871	238.537	295.202	248.978	682.747
	Required source head (m)					

[a] The table shows pairs of pipes whose respective removal have identical effects.

Table 7.8: Required source heads to satisfy full demand at each node for single isolated pipes in Design 1 (ISHM).

Node	Isolated Pipe					
	1-2, (1-4)	2-3, (4-7)	2-5, (4-5)	3-6, (7-8)	5-6, (5-8)	6-9, (8-9)
2	418.038 (60.737)	9.399 (26.188)	9.785 (25.578)	12.289 (22.011)	18.513 (15.214)	13.004 (21.097)
3	433.144 (113.428)	570.700 (63.300)	40.990 (55.147)	17.138 (54.457)	61.476 (43.294)	26.577 (65.371)
4	60.737 (418.039)	26.188 (9.399)	25.579 (9.785)	22.011 (12.289)	15.214 (18.513)	21.097 (13.004)
5	333.921 (333.921)	83.192 (83.192)	97.557 (97.557)	65.427 (65.427)	31.064 (31.064)	42.249 (42.249)
6	460.701 (337.473)	533.228 (205.197)	144.721 (150.358)	343.010 (164.728)	229.435 (130.870)	48.213 (240.760)
7	113.428 (433.144)	63.300 (570.700)	55.147 (40.990)	54.457 (17.138)	43.294 (61.477)	65.371 (26.577)
8	337.473 (460.701)	205.197 (533.228)	150.358 (144.721)	164.728 (343.010)	130.870 (229.436)	240.760 (48.213)
9	561.946 (561.946)	566.601 (566.601)	304.845 (304.845)	422.845 (422.845)	341.004 (341.004)	808.534 (808.534)
	Required source head (m)					

Table 7.9: Values of nodal outflows using the required source heads of Tale 7.6 for single isolated pipes in Design 1 (ISHM).

Node	Isolated Pipe[a]					
	1-2, (1-4)	2-3, (4-7)	2-5, (4-5)	3-6, (7-8)	5-6, (5-8)	6-9, (8-9)
2	-0.0018 (-0.0208)	-0.0208 (-0.0208)	-0.0208 (-0.0208)	-0.0208 (-0.0208)	-0.0208 (-0.0208)	-0.0208 (-0.0208)
3	-0.0100 (-0.0195)	-0.0087 (-0.0208)	-0.0208 (-0.0208)	-0.0208 (-0.0208)	-0.0208 (-0.0208)	-0.0208 (-0.0208)
4	-0.0208 (-0.0102)	-0.0208 (-0.0208)	-0.0208 (-0.0208)	-0.0208 (-0.0208)	-0.0208 (-0.0208)	-0.0208 (-0.0208)
5	-0.0114 (-0.0114)	-0.0208 (-0.0208)	-0.0208 (-0.0208)	-0.0208 (-0.0208)	-0.0208 (-0.0208)	-0.0208 (-0.0208)
6	-0.0097 (-0.0113)	-0.0090 (-0.0145)	-0.0173 (-0.0170)	-0.0112 (-0.0162)	-0.0137 (-0.0182)	-0.0208 (-0.0134)
7	-0.0195 (-0.0100)	-0.0208 (-0.0087)	-0.0208 (-0.0208)	-0.0208 (-0.0208)	-0.0208 (-0.0208)	-0.0208 (-0.0208)
8	-0.0113 (-0.0097)	-0.0145 (-0.0090)	-0.0170 (-0.0173)	-0.0162 (-0.0112)	-0.0182 (-0.0137)	-0.0134 (-0.0208)
9	-0.0264 (-0.0264)	-0.0263 (-0.0263)	-0.0358 (-0.0358)	-0.0304 (-0.0304)	-0.0339 (-0.0339)	-0.0220 (-0.0220)
Available source flow	0.1193	0.1417	0.1741	0.1618	0.1698	0.1602
	Available nodal flow (m^3/s)					

[a] The table shows pairs of pipes whose respective removal have identical effects on the available source flow

Table 7.10: Summary of network reliability measures from ISHM.

Design	$(R_L+R_U)/2$ [a]	R [b]	$(T_L+T_U)/2$ [a]	T [b]
1	0.861654	0.861656	0.765086	0.765560
2	0.869715	0.869717	0.771417	0.771899
3	0.871873	0.871875	0.773356	0.773837
4	0.882136	0.882139	0.780823	0.781304
5	0.887806	0.887809	0.787959	0.788458
6	0.888508	0.888510	0.789944	0.790451
7	0.890971	0.890974	0.796127	0.796639
8	0.893186	0.893189	0.801778	0.802303
9	0.893931	0.893933	0.803869	0.804397
10	0.895972	0.895975	0.808525	0.809053
11	0.897176	0.897178	0.812464	0.813000
12	0.897676	0.897678	0.814270	0.814811
13	0.898853	0.898856	0.818006	0.818552
14	0.899549	0.899551	0.819782	0.820327
15	0.900047	0.900050	0.821716	0.822270
16	0.901211	0.901213	0.825547	0.826098
	Reliability		Damage tolerance	

[a] Based on one-link failure, [b] Based on one and two-link failures in which for results arising from Cullinane et al. (1992) Eq. (R_L and R_U are identical).

Table 7.11: Reliability and damage tolerance values based on different methods.

Design	R (SHM)[a]	R (ISHM)[a]	Fujiwara[b]	T (SHM)[c]	T (ISHM)[c]
1	0.599357	0.861656	0.81855	0.467883	0.764816
2	0.607279	0.869717	0.83345	0.476057	0.771417
3	0.609345	0.871875	0.83368	0.477667	0.773356
4	0.618897	0.882139	0.85017	0.485695	0.780823
5	0.627596	0.887809	0.86149	0.493025	0.787959
6	0.629896	0.888510	0.86360	0.494857	0.789944
7	0.638008	0.890974	0.88216	0.501722	0.796127
8	0.645301	0.893189	0.89072	0.507904	0.801778
9	0.647755	0.893933	0.89780	0.509897	0.803869
10	0.654495	0.895975	0.91661	0.515614	0.808525
11	0.658447	0.897178	0.92173	0.520798	0.812464
12	0.660079	0.897678	0.92467	0.522573	0.814270
13	0.663961	0.898856	0.93533	0.527683	0818006
14	0.666243	0.899551	0.94231	0.529357	0.819782
15	0.667877	0.900050	0.94362	0.531172	0.821716
16	0.671701	0.901213	0.95094	0.536179	0.825555

[a] Based on up to two simultaneous link failures
[b] From Fujiwara and Tung (1991)
[c] Based on up to two simultaneous link failures

Table 7.12: Nodal reliabilities (ISHM) for Designs 1 to 16 with one and two simultaneous pipe failures, expressed as the average of the lower and upper bounds.

Design	Node					Mean	Demand-Weighted Mean	System Reliability
	2, 4	3, 7	5	6, 8	9			
1	0.999872	0.999553	0.999775	0.910446	0.599358	0.927373	0.861644	0.861656
2	0.999880	0.999663	0.999780	0.938864	0.607278	0.935105	0.869519	0.869717
3	0.999880	0.999677	0.999786	0.946537	0.609344	0.937665	0.871874	0.871875
4	0.999882	0.999677	0.999792	0.983533	0.618898	0.947234	0.881441	0.882139
5	0.999884	0.999677	0.999798	0.998818	0.627597	0.953019	0.887810	0.887809
6	0.999884	0.999690	0.999799	0.998852	0.629896	0.953318	0.888510	0.888510
7	0.999886	0.999690	0.999805	0.998980	0.638006	0.954479	0.890699	0.890974
8	0.999888	0.999690	0.999810	0.999096	0.645302	0.955308	0.893187	0.893189
9	0.999888	0.999702	0.999811	0.999130	0.647755	0.955626	0.893933	0.893933
10	0.999890	0.999703	0.999816	0.999219	0.654495	0.956492	0.895976	0.895975
11	0.999891	0.999713	0.999816	0.999290	0.658447	0.957006	0.897180	0.897178
12	0.999897	0.999723	0.999828	0.999311	0.660079	0.957221	0.897678	0.897678
13	0.999897	0.999733	0.999829	0.999373	0.663960	0.957724	0.898859	0.898856
14	0.999901	0.999743	0.999836	0.999393	0.666242	0.958019	0.899005	0.899551
15	0.999907	0.999753	0.999848	0.999415	0.667877	0.958234	0.900051	0.900050
16	0.999908	0.999764	0.999848	0.999476	0.671701	0.958731	0.901002	0.901213

Table 7.13: Nodal damage tolerances (ISHM) for Designs 1 to 16 with one and two simultaneous pipe failures, expressed as the average of the lower and upper bounds.

Design	Node					Mean	Demand-Weighted Mean	System Damage Tolerance
	2, 4	3, 7	5	6, 8	9			
1	0.979458	0.942513	0.962369	0.722413	0.467693	0.839854	0.765279	0.764816
2	0.979697	0.943442	0.963071	0.741957	0.475711	0.846122	0.771897	0.771417
3	0.979755	0.944493	0.963773	0.746835	0.477321	0.847908	0.773648	0.773356
4	0.979898	0.944944	0.964561	0.772214	0.485363	0.855505	0.781334	0.780823
5	0.980068	0.945444	0.965284	0.796876	0.492714	0.862847	0.788678	0.787959
6	0.980952	0.946464	0.965343	0.802020	0.494552	0.864846	0.790645	0.789944
7	0.980204	0.946840	0.966026	0.822598	0.501428	0.870842	0.796817	0.796127
8	0.980351	0.947162	0.966664	0.841506	0.507627	0.876541	0.802616	0.801778
9	0.980539	0.947635	0.966748	0.846979	0.509623	0.878335	0.804451	0.803869
10	0.980767	0.947908	0.967343	0.861607	0.515348	0.882907	0.809254	0.808525
11	0.980915	0.948443	0.967631	0.872578	0.520550	0.886507	0.813175	0.812464
12	0.981316	0.949910	0.968378	0.875617	0.522332	0.888125	0.814825	0.814270
13	0.981680	0.951266	0.968681	0.885465	0.527456	0.891618	0.818645	0818006
14	0.981885	0.953022	0.969958	0.888846	0.529131	0.893324	0.820346	0.819782
15	0.982838	0.954499	0.971679	0.892371	0.530954	0.895256	0.822256	0.821716
16	0.982915	0.956110	0.971706	0.902562	0.535971	0.898856	0.826140	0.825555

Table 7.14: Nodal Reliability *For Fully Connected Network*, $r_j(0)$, *(ISHM)*.

Design	Node		
	2-5, 7	6, 8	9
1	1.0000	0.9115	0.6002
2	1.0000	0.9399	0.6080
3	1.0000	0.9476	0.6101
4	1.0000	0.9846	0.6197
5	1.0000	1.0000	0.6283
6	1.0000	1.0000	0.6307
7	1.0000	1.0000	0.6389
8	1.0000	1.0000	0.6461
9	1.0000	1.0000	0.6485
10	1.0000	1.0000	0.6488
11	1.0000	1.0000	0.6592
12	1.0000	1.0000	0.6608
13	1.0000	1.0000	0.6648
14	1.0000	1.0000	0.6670
15	1.0000	1.0000	0.6686
16	1.0000	1.0000	0.6725

Table 7.15: System reliabilities arising from different pipe availability equations, expressed as the average of the lower and upper bounds for one pipe failure.

Design	Cullinane et al. Eq. (Eq. 7.10)	Fujiwara & Tung Eq. (Eq. 7.11)	Khomsi et al. Eq. (Eq. 7.14)	Su et al. Eq. (Eq. 7.12)
1	0.861653	0.856903	0.861494	0.574280
2	0.869714	0.864930	0.869551	0.578128
3	0.871873	0.867105	0.871715	0.579247
4	0.882136	0.877259	0.881977	0.583064
5	0.887806	0.883016	0.887652	0.586451
6	0.888507	0.883806	0.888362	0.587518
7	0.890971	0.886461	0.890834	0.590442
8	0.893186	0.888852	0.893057	0.593165
9	0.893930	0.889686	0.893810	0.594276
10	0.895972	0.891859	0.895859	0.596735
11	0.897175	0.893239	0.897064	0.599999
12	0.897676	0.893812	0.897568	0.601559
13	0.898853	0.895148	0.898746	0.604658
14	0.899548	0.895938	0.899453	0.605442
15	0.900047	0.896509	0.899956	0.607008
16	0.901210	0.897828	0.901120	0.610035

Table 7.16: System damage tolerances arising from different pipe availability equations, expressed as the average of the lower and upper bounds for one pipe failure.

Design	Cullinane et al. Eq. (Eq. 7.10)	Fujiwara & Tung Eq. (Eq. 7.11)	Khomsi et al. Eq. (Eq. 7.14)	Su et al. Eq. (Eq. 7.12)
1	0.765058	0.761128	0.763453	0.555568
2	0.771417	0.767981	0.769524	0.558306
3	0.773355	0.769987	0.771777	0.559136
4	0.780823	0.777439	0.779232	0.561876
5	0.787959	0.784524	0.786352	0.564510
6	0.789943	0.786596	0.788707	0.565377
7	0.796126	0.792774	0.794854	0.567781
8	0.801777	0.798383	0.800496	0.570028
9	0.803869	0.800573	0.803038	0.570944
10	0.808525	0.805103	0.807781	0.572951
11	0.812463	0.809745	0.811129	0.575451
12	0.814269	0.811445	0.812646	0.576669
13	0.818006	0.815738	0.815723	0.579063
14	0.819782	0.818045	0.818686	0.579733
15	0.821715	0.819779	0.820598	0.580977
16	0.825547	0.824068	0.823778	0.583357

Table 7.17: Computational efficiency statistics of the ISHM.

	one-pipe failures	one & two-pipe failures
No. of Simulations	12	78
CPU[a] (Seconds)	< 0.6 per design	< 2.64 per design

[a] Includes simulations and computation of R_L, R_U, T_L and T_U

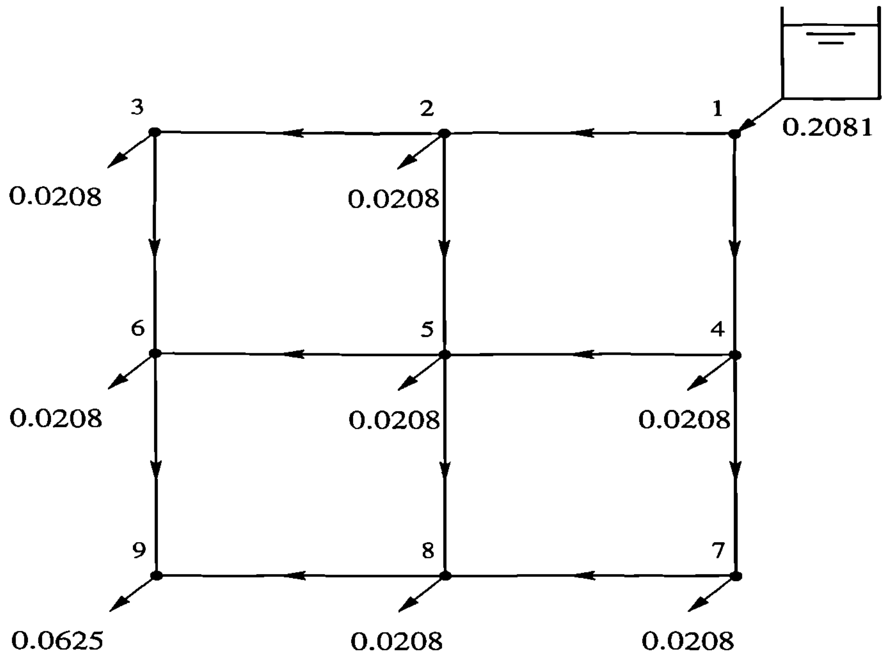

Figure 7.1: Layout of the sample network.

Flows are in m^3/s, node elevations are all 0 m and source head = 100 m.

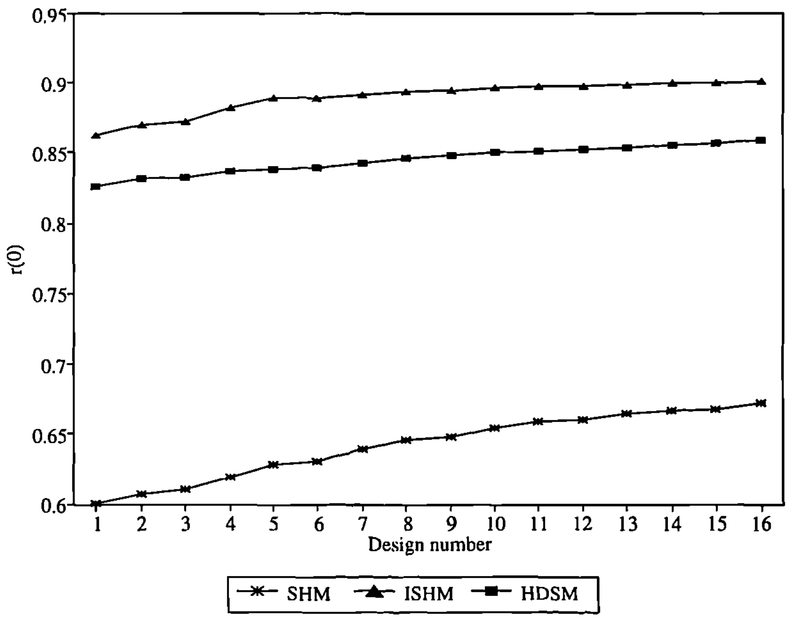

Figure 7.2: Fractions of total demand satisfied by fully connected network.

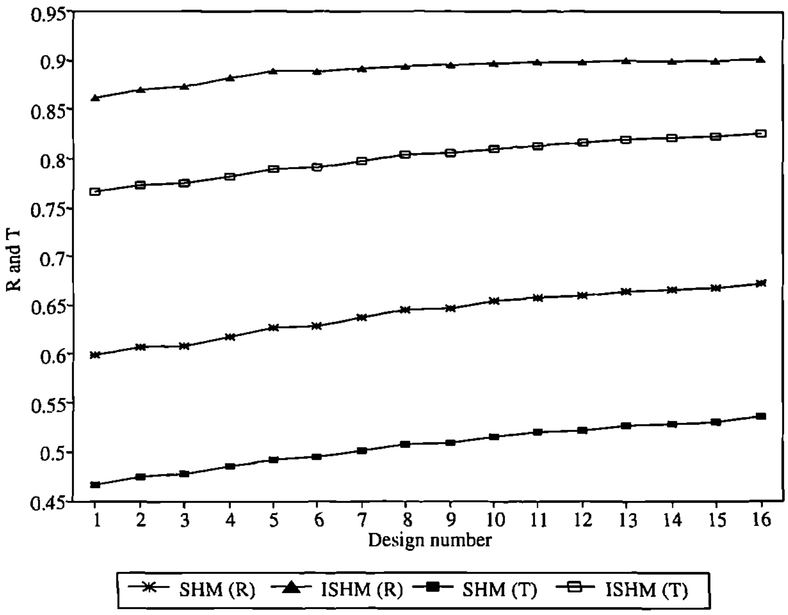

Figure 7.3: Reliability and damage tolerance.

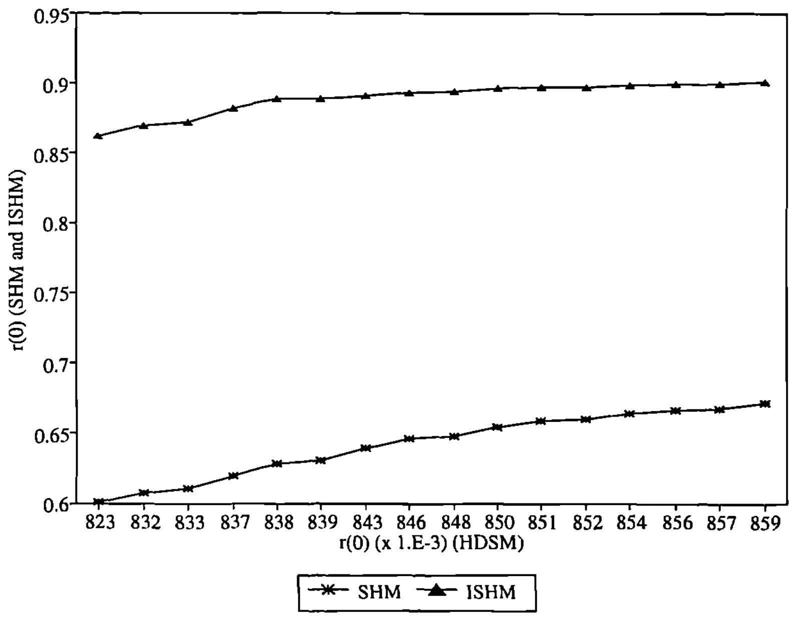

Figure 7.4: SHM and ISHM fraction of total demand against HDSM value.

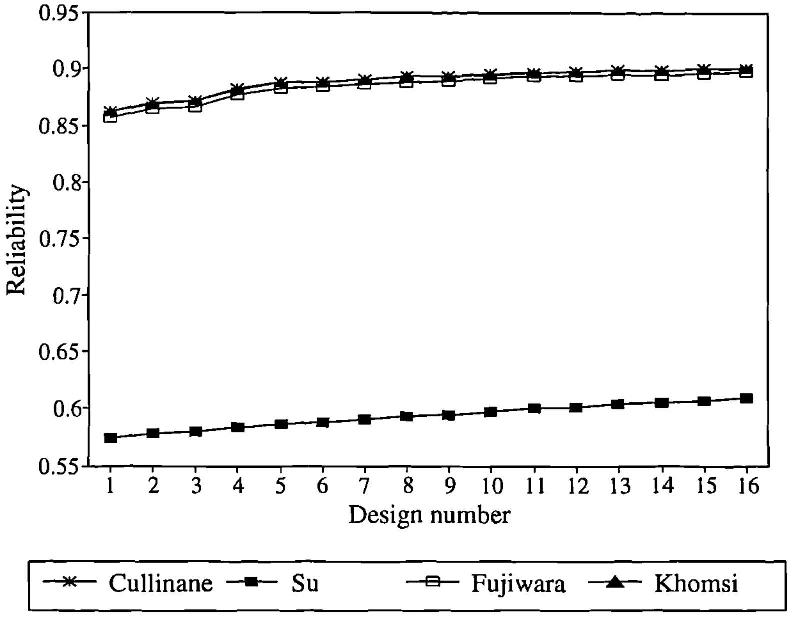

Figure 7.5: Reliability values arising from different pipe availability formulations.

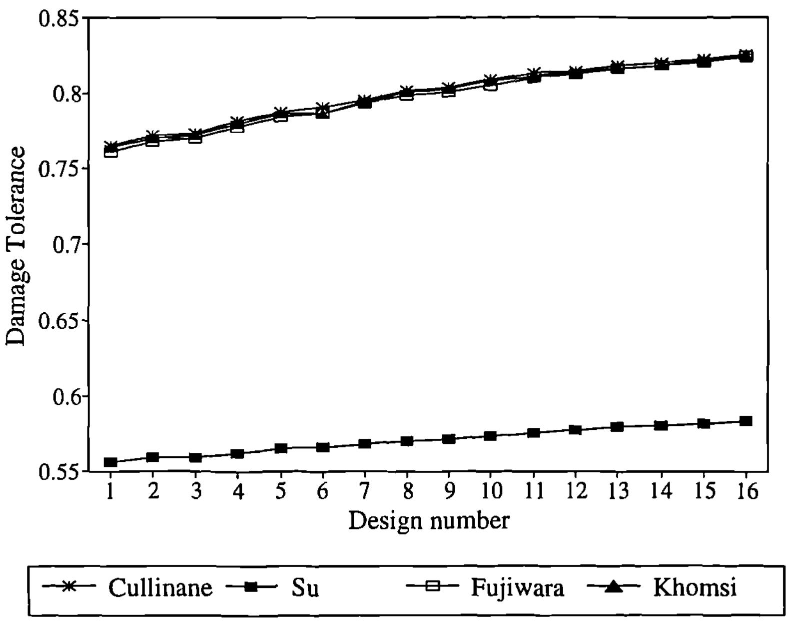

Figure 7.6: Damage tolerance values arising from different pipe availability formulations.

CHAPTER 8

HEAD DRIVEN SIMULATION BASED RELIABILITY ANALYSIS OF WATER SUPPLY NETWORKS

8.1 INTRODUCTION

The concept of reliability in water distribution networks was reviewed in Chapter 6. It was seen that although there is not a universal definition for reliability indices in water supply networks, one of the most acceptable definitions during recent years has been determined as the ratio of the available outflow to the required flow (demand).

It was recognised from Chapter 5 that the actual outflow delivered under insufficient nodal heads cannot be evaluated by the conventional demand driven simulation method. Results of that chapter showed that by relating the nodal outflow to the nodal pressure, the head driven simulation method can provide the values of actual nodal outflows and consequently, values of shortfall in water distribution networks, in a realistic manner. It was seen that this method is relatively easy and straightforward, in comparison with more complicated methods such as the advanced mathematical programming based methods (Fujiwara and De Silva 1990; Fujiwara and Tung 1991), or the indirect formulation of the improved source head method, ISHM (see Chapter 7).

Furthermore, Chapter 7 showed that consideration of a pressure dependent formulation for nodal demand can lead to great improvement in calculation of available nodal and source flows. However, it was observed that using the conventional demand driven analysis of water distribution networks in conjunction with ISHM, is only able to produce approximate results. In addition, the improved source head method is applicable only to single source networks and there is a need for a realistic reliability measure for general networks, capable of including ancillary components such as pumps, valves, reservoirs, etc.

Another important consideration for a more realistic evaluation of the reliability of a water supply system is to account for the variations in demand. It was argued in Chapter 2 that all design procedures and hydraulic models assume a set of deterministic demands through a 24 hour day or as daily average or peak demands, to assess the behaviour of water distribution networks. However, demands are not constant over a period of time. This casts doubt on whether snapshot analysis using daily average demands can represent a realistic overall figure for the reliability evaluation of a system over a period of time e.g. 24 hrs. Therefore, an extended period simulation is required to evaluate the system based on hourly distributed demands during a day to see the diurnal variations of reliability. To have an overall reliability assessment different criteria can be considered e.g. choosing the most critical snapshot or making an average for all snapshots, etc. However, accuracy of the mean and demand-weighted mean of the hourly nodal reliabilities to represent the overall daily system reliability needs to be examined.

In the literature few studies can be found in which extended period simulation has been applied for reliability purposes (e.g. Wagner et al. 1988b; Cullinane et al. 1992; Gupta and Bhave 1994, 1996a). However, none of them have focused on the variations of reliability during a time period and the extended period results have only been used to calculate the daily system reliability. Furthermore, most of these attempts use the arithmetic mean of the nodal reliabilities/availabilities to represent the system reliability. The approaches also require high execution time.

Distribution of the average demand values through a day is not fully representative of actual variations of demand. Additionally, the daily average or peak demands vary over a long period of time and change of climate, season, days of week, socio-economic aspects, populations, etc. also causes variations. Therefore, consideration of the probabilistic nature of demand is crucial for a realistic evaluation of the system especially when demand exceeds the design values. Exceptional demand at one or several nodes, which lead to more head loss through that part of system is one cause of hydraulic failure. It follows, therefore, that a real reliability measure should consider a range of probable loading conditions for true assessment of the system.

Walters and Cembrowicz (1993) emphasised the probabilistic nature of demand as an important aspect of water distribution systems, alongside pipe failures, connectivity and link capacity (see also Section 2.9). Also, Wagner et al. (1988b) stated the usefulness of stochastic simulation methods which can incorporate more complicated features of distribution networks and allow calculation of any desired set of reliability indices. They mentioned the inclusion of the uncertain nature of failure events and repair times. However, the stochastic nature of demand was not discussed.

Bao and Mays (1990) used the Monte Carlo simulation technique to evaluate a different range of demands. However, inclusion of both probabilistic pipe failures and demands by the Monte Carlo simulation technique requires very high computer time (Walters and Cembrowicz 1993). As an alternative, a probability distribution function can be applied to demonstrate the variability of demands (Bouchart and Goulter 1991; Khomsi et al. 1996).

This chapter aims to present a more realistic reliability measure to evaluate general water distribution networks including the ancillary components, using the results of the head driven analysis of hydraulics of the system. This head driven simulation based reliability analysis measure will be hereafter referred to as HDSRA. To address the effects of variable and probabilistic demands on the reliability indices, a set of extended period analyses, with both deterministic and probabilistic demands through a 24 hour day, are assessed by the proposed reliability measure. It will be seen how the proposed head driven simulation based reliability indices for general networks can incorporate the variation of demands (including its probabilistic nature) while avoiding the complication and time consuming features of some existing methodologies such as the Monte Carlo simulation method.

8.2 SNAPSHOT ANALYSIS OF GENERAL WATER DISTRIBUTION NETWORKS

The comprehensive reliability analysis of water distribution systems includes two major hydraulic and reliability models as follows:

8.2.1 Hydraulic Model

The head driven simulation of the hydraulics of pipe systems (HDSM) is considered as the hydraulic model to produce the available nodal heads and, consequently, available outflows. This method which was described in Chapter 5 in detail, uses the following nodal head-outflow relationship to calculate the value of nodal outflows,

$$Q_j^{avl} = Q_j^{req} \qquad ; if \;\; H_j \geq H_j^{des} \tag{8.1a}$$

$$Q_j^{avl} = Q_j^{req} \left(\frac{H_j - H_j^{\min}}{H_j^{des} - H_j^{\min}}\right)^{\left(\frac{1}{n_j}\right)} \qquad ; if \; H_j^{\min} < H_j < H_j^{des} \tag{8.1b}$$

$$Q_j^{avl} = 0 \qquad ; if \;\; H_j \leq H_j^{\min} \tag{8.1c}$$

in which Q_j^{avl} and Q_j^{req} are the available and required nodal outflows, respectively. H_j, H_j^{des} and H_j^{min} are the respective available, desirable and minimum nodal heads and n_j is a coefficient (between 1.5-2). Consequently, the total outflow from the network is obtained by summation of outflow at all demand nodes, i.e.,

$$Q_s^{avl} = \sum_{j=1}^{NJ_d} Q_j^{avl} \tag{8.2}$$

where Q_s^{avl} is sum of the nodal available flow and NJ_d represents the number of demand nodes. Obviously, the total demand is the summation of all nodal demands.

8.2.2 Reliability Model

Considering the more recently and widely accepted definition for reliability as the ratio of the available flow to the required flow (demand) in conjunction with link failure probabilities, a reliability model was built in Chapter 7. This model is restated briefly.

8.2.2.1 Component reliability/availability

Four pipe availability formulations from Cullinane et al. (1992), Fujiwara and Tung (1991), Khomsi et al. (1996) and Su et al. (1987) were presented in Section 7.3.1 through Eqs. 7.10 to 7.14. For the purposes of this chapter, the following additional equations suggested by Cullinane et al. (1992), are added to the reliability model for availability of pumps and pipes, including valves.

$$a_{lv} = 0.9278\ D_l^{0.000118} \tag{8.3}$$

$$a_{lp} = 1.046943\ Q_p^{-0.01634} \tag{8.4}$$

in which a_{lv} and a_{lp} are the hydraulic availability of a pipe linked with valve and pump, respectively. D_l is the diameter of the pipe which includes the valve (in inches) and Q_p is the value of the design flow rate of the pump (in gpm).

8.2.2.2 System and nodal reliability

The system reliability can be defined as the ratio of the available flow to the required flow for the entire network at any normal or subnormal configuration, i.e.

$$r(M) = \frac{Q_s^{avl}(M)}{Q_s^{req}} \qquad M = 0, \ldots, NP \tag{8.5}$$

in which r(M) is the system reliability with M specified links (pipes) unavailable and $Q_s^{avl}(M)$ is the total available flow when the M specified links (pipes) are unavailable. Q_s^{req} denotes total demands. Taking all network configurations into consideration, the system reliability R, is

$$R = p(0)\ r(0) + \sum_{l=1}^{NP} p(l)\ r(l) + \sum_{l=1;\ m\neq l}^{NP} p(l,m)\ r(l,m) + \sum_{l=1;\ m\neq l;\ n\neq l;\ n\neq m}^{NP} p(l,m,n)\ r(l,m,n) + \ldots \tag{8.6a}$$

For the purpose of computational efficiency the above equation can be represented as follows:

$$R = p(0) \sum_{M=0}^{NP} \left(r(M) \prod_{l=1}^{M} \frac{ua_l}{a_l} \right) \tag{8.6b}$$

where a_l and ua_l ($=1-a_l$) are the availability and unavailability of link l, respectively. p(M) is the probability that only M specified links are unavailable. Therefore, p(0) is the probability that all links are available (see Eq. 7.15).

It can be assumed that the link failure occurrences are statistically independent (Su et al. 1987; Wagner et al. 1986). Therefore, the probability of simultaneous failures of several links is very low. For two simultaneous link failures, the lower and upper bound to system reliability, R_L and R_U, are as follows:

$$\begin{aligned} R_L &= \frac{1}{Q_s^{req}} \left(\sum_{l=0}^{NP} p(l)\ Q_s^{avl}(l) + \sum_{l=1;\ m\neq l}^{NP} p(l,m)\ Q_s^{avl}(l,m) \right) \\ &= p(0) \left(r(0) + \sum_{l=1}^{NP} r(l) \frac{ua_l}{a_l} + \sum_{l=1;\ m\neq l}^{NP} r(l,m) \frac{ua_l}{a_l} \frac{ua_m}{a_m} \right) \end{aligned} \tag{8.7}$$

$$\begin{aligned} R_U &= 1 - \frac{1}{Q_s^{req}} \left(\sum_{l=0}^{NP} p(l)\ (Q_s^{req} - Q_s^{avl}(l)) + \sum_{l=1;\ m\neq l}^{NP} p(l,m)\ (Q_s^{req} - Q_s^{avl}(l,m)) \right) \\ &= 1 - p(0) \left(u(0) + \sum_{l=1}^{NP} u(l) \frac{ua_l}{a_l} + \sum_{l=1;\ m\neq l}^{NP} u(l,m) \frac{ua_l}{a_l} \frac{ua_m}{a_m} \right) \end{aligned} \tag{8.8}$$

where all the parameters were defined in Section 7.3.2.

Damage tolerance, the probability that the system is functioning under failure conditions, was also introduced in Chapter 7. The damage tolerance of the system, lower and upper bounds (T_L and T_U), can be calculated as

$$T_L = \frac{R_L - r(0)\, p(0)}{1 - p(0)} \tag{8.9}$$

$$T_U = \frac{R_U - r(0)\, p(0)}{1 - p(0)} \tag{8.10}$$

As mentioned in Section 7.3.2, for nodal reliability and damage tolerance calculation index of j is added to the above formulations for the parameters R, T, U, r and u.

The HDSRA is a combination of the same programmes developed in Chapters 5 and 7 for the head driven simulation analysis (HDSM) and reliability assessment. They are applied in this section as the hydraulic and reliability models. To solve the examples, a Pentium PC with 75 MHz speed and 8 Mbyte RAM has been used.

8.2.3 Numerical Examples

To provide an appropriate basis for comparisons of results, the example used for appraisal of Chapter 7 is described here as Example 8.1. The layout of the network is shown in Figure 8.1 and the pipe and nodal data for the 16 successive designs of this example is presented in Tables 8.1 and 8.2. Using the HDSRA the values of available nodal outflows are calculated first by the head driven simulation method. The system reliability and damage tolerance values are then calculated using the three different pipe availability formulations (Eqs. 7.10-12) considering up to two simultaneous pipe failures. The results are presented in Table 8.3 as the average of the lower and upper bounds. Figure 8.2 shows the reliability values of all 16 successive designs graphically. It is seen that as expected, the results show systematic improvement in hydraulic performance of the system from design 1 to 16, while the mechanical reliability is improved, by increasing the pipe diameter of two pipes at each successive design. It leads to lower head loss and thus, higher nodal heads and smaller shortfalls are obtained.

Also, looking at Table 8.3 again, it can be seen that these designs on average satisfy only about 83-86% of the demand. Furthermore, under failure conditions only about

75-79% of demand would be satisfied on average. Improvement in the reliability and damage tolerance results are only about 3% to 5% from design 1 to 16, respectively. This is brought about by the rather small increase (just 5 mm) in diameter of two pipes between the successive designs.

Among the three applied pipe availability formulations shown in Table 8.3, the results of Su et al. (1987) are observed to have a considerable discrepancy with the results of the other equations which further verifies the decision not to adopt it, as the conclusion of Chapter 7.

In addition, the results of indirect formulations of the problem, namely source head method (SHM) and the improved source head method (ISHM) from Chapter 7, are compared with the reliability values from the HDSRA in Figure 8.2. The figure confirms the conclusions of Chapter 7 that the SHM results depart substantially, underestimating system reliabilities by about 25%, compared with the realistic values arising from HDSRA. The ISHM, in contrast, shows good correspondence with the HDSRA results, overestimating values by about 5%.

One of the advantages of the head driven simulation based reliability analysis is that nodal reliability and damage tolerance values can be calculated directly, using appropriate expressions from Section 7.3.2. The results are shown in Table 8.4 for designs 1 and 16. As can be seen, the critical node (i.e. node 9) has the smallest reliability and damage tolerance and the farther the nodes are from the critical one (i.e. the nearer to the source), the higher the reliability they have. Also, comparison of the R and T values indicates that for nodes 2, 3, 4, 5 and 7 the probability of failure and its impact on the system functioning are low. However, at nodes 6, 8 and especially node 9, the severity of failure is higher.

Table 8.4 also presents the arithmetic and demand-weighted mean of the nodal reliabilities, together with the system reliability for these two designs. As expected and discussed in Chapter 7, the results show that the system reliability may not be calculated as the arithmetic mean of nodal reliabilities. However, the demand-weighted mean values of the nodal reliabilities are much closer to the system reliabilities. The reason for this closeness can be seen in Appendix E in which the relationship between the system and demand-weighted of the nodal reliabilities are derived. It can be concluded therefore that in the current reliability analysis, based on equation 8.6, the system reliability and demand-weighted mean of the nodal reliabilities are equivalent. This conclusion is also valid for the damage tolerance values.

It is clear that calculation of the system reliability in a straightforward way (i.e. using Eqs. 8.7 and 8.8) when the nodal reliabilities are not required, is more convenient than using the demand-weighted mean of nodal reliabilities, because it just considers the total available flow in the reliability model with no need to calculate the nodal reliabilities, in advance. The required computer time to calculate the available outflows for 78 different failure cases together with the system and nodal reliabilities was about 2.5 seconds.

The next two examples show the capability of the reliability measure (HDSRA) to calculate the reliability of more complicated networks including ancillary components such as pumps, valves, reservoirs. etc. Example 8.2 includes a two looped network with single source and two pressure reducing valves. The layout of this network is shown in Figure 8.3. Tables 8.5 and 8.6 present the pipe and nodal data, respectively. The outlet heads of PRV1 and PRV2 are set to 292.608 m and 289.56 m, respectively. The results of the reliability analysis for different pipe availability formulas are shown in Table 8.7. In addition, values of nodal reliability and damage tolerance can be observed in the same table. It is seen that all the nodal and system reliabilities and damage tolerances are very high. This demonstrates the very good hydraulic performance of the system. Also, the closeness of the reliability and damage tolerance values indicates that the severity of even up to two simultaneous link failures (including pipes and PRVs) is very low and does not affect the system and nodal reliabilities, even in the most critical node, i.e. node 3. This situation arises because of the well maintained head values in all nodes. i.e. during failure conditions sufficient pressure still exists to run the system without serious deficiency. As a result, the nodal and system reliabilities are also very close together.

The other finding from Table 8.7 is that the system reliabilities are equivalent to the demand-weighted mean of nodal reliabilities and damage tolerances, as expected. The computer time required to compute the available outflows for 45 different failure cases (including pipes and PRVs) together with the reliability calculations was about 3 seconds.

The network of Example 8.3 includes three reservoirs and three pumps. Its layout is shown in Figure 8.4 and Table 8.8 represents the pipe data. Demand at node 1 is 0.0566 m^3/s. Also, the minimum and desired heads for all nodes are set equal to 15.24 m and 29.24 m, respectively. Furthermore, the head-flow relation for all pumps is given by $H_p = -382364\ Q_p^2 + 27172\ Q_p + 6.819$. The reliability and damage tolerance values resulting from the HDSRA are shown in Table 8.9. High values for the reliability illustrates the high performance of the system under both normal and subnormal conditions. In fact, existence of three different reservoirs and pumps helps the system to meet the required head in most situations. The severity of the failures can be appraised by the damage tolerance value which is around 90% (from Eqs. 7.10 and 7.11). In this example which consists of only one demand node, the system and nodal reliabilities and damage tolerances are identical. The required CPU time is about 5 seconds for 91 different failure cases (including pipes and pumps) for both hydraulic and reliability analyses.

So far, this section has shown that the head driven simulation based reliability analysis is able to calculate the reliability of general networks in a straightforward manner. The above examples show the capability of the method to deal with general and multi source networks, which is another advantage of this method against the indirect ISHM approach. It can therefore, be concluded that the method can be applied to realistic networks. To further applicability, the following sections consider a combination of the extended period analysis and probabilistic nature of demand with the HDSRA method.

8.3 EXTENDED PERIOD RELIABILITY ANALYSIS

It was mentioned earlier that demands are not fixed through the 24 hours of a day and therefore, any reliability analysis based on just one snapshot analysis, which considers daily average or peak demands, cannot represent the overall system reliability realistically. In this section the variations of system and nodal reliability values are investigated through a period of time (herein 24 hrs.). The main task is the combination of the extended period simulation of water distribution networks and the head driven simulation based reliability analysis. The methodology is described below.

8.3.1 Methodology

To perform an extended period analysis, the diurnal profile of nodal demands is needed. The required flows at each demand node are obtained by the multiplication of the daily average values with the corresponding demand factor at each time. The demand factors themselves vary with time of day and from node to node. However, for any particular distribution zone a set of fixed demand factors can be assumed for all nodes, based on demand analysis for a set of the historical data. For design purposes hydraulic models use these factors, or a set of factors for different types of demand. A detailed procedure was described in Chapters 2 and 3. It is worth noting that the resulting demand values are still deterministic for each individual time.

The extended period simulation procedure was described in Chapter 4. The hydraulic model is extended by combination of the extended period simulation algorithm and the head driven simulation method for analysis of water distribution networks. Now having demand values at each time (hour), the nodal and system outflows and shortfalls are determined by the hydraulic model. Finally, using the reliability measure (i.e. appropriate equations from Sections 7.3.2 and 8.2.2.2), the nodal and system reliabilities can be calculated at each individual time snapshot. This approach is referred to hereafter as the head driven simulation based extended period reliability analysis (HDSEPRA).

8.3.2 Numerical Example

To demonstrate the outcome of the HDSEPRA on reliability results, Example 8.4 is performed. This example is taken from Khomsi et al. (1996) and the layout of the network is shown in Figure 8.5. The pipe and nodal data are presented in Tables 8.10 and 8.11, respectively. Table 8.12 shows the diurnal profile of demand factors for 'domestic type' consumptions which are taken from the results of Chapter 3 and assumed to be applicable to this example. Figure 8.6 also shows this diurnal profile, graphically. To be able to compare the results, besides the previous pipe availability formulations (Eqs. 7.10-12), Eq. 7.14, used by Khomsi et al. (1996), is also considered in the procedure of reliability calculation.

By performing a head driven simulation based extended period reliability analysis (HDSEPRA) the system and nodal reliability and damage tolerance values are determined. The results at each individual hour are presented in Table 8.13. It should be mentioned that to avoid complication of the solution and save computer time, in the rest of this chapter the possibility of just one pipe failure is considered. Furthermore, using the conclusions of Chapter 7, all the results are presented as the mean of the lower and upper bounds.

8.3.3 Discussion

As expected, in times of low demand (i.e. midnight, mid noon and late evening) the reliability values are high. In contrast, increase in demand leads to decrease in reliability values. Figure 8.7 represents the diurnal profile of reliability and damage tolerance values, graphically. It can be seen that during the night both the possibility of failures and the impact of failures on the operation of the system is very low. However, with increasing demand, both of them increase. In addition, as concluded in Chapter 7, Table 8.13 shows that results from the two pipe availability formulations are very close together. Therefore, there is no reason for preference of the Khomsi et al. (1996) formulation against the Cullinane et al. (1992) formulation, as was seen in Chapter 7.

The mean and demand-weighted mean of reliabilities and damage tolerances for the

24 hours have also been calculated and are shown in Table 8.13, using the demand factors from Table 8.12 at each hour as the appropriate weighting. Furthermore, for better comparison, the daily system reliability is calculated using the daily average demand values (Table 8.11) by the snapshot analysis, i.e. a steady state analysis is used with daily average demands to produce the daily average available outflows. Then using the reliability measure, the daily system reliability and damage tolerances are obtained. Considerable differences can be observed between the daily system reliability and both the sets of the arithmetic and demand-weighted mean values of the hourly reliabilities. Therefore, it is clear that using the HDSEPRA, neither mean nor demand-weighted mean of the hourly reliability values are equivalent to the daily system reliability resulting from snapshot analysis of the daily average demands. Snapshot analysis using the daily average demand values is not able to show the severity of shortfall in the system during the times of peak demands. Therefore, this does not seem to be a good representation of the overall daily reliability of the system. Values of the demand-weighted mean of hourly reliabilities look more meaningful in representing the overall daily reliability of the system.

To assess the critical situations, Table 8.14 demonstrates the hourly reliability and damage tolerance values at the critical node (i.e. node 4). Also Figure 8.8 shows diurnal profile of reliabilities and damage tolerances at node 4. Comparison of results with the system reliabilities at each time (Table 8.13) shows that the hourly reliabilities and damage tolerances for the critical node are lower, as might be expected. However, the other features are the same.

Furthermore, the nodal reliability and damage tolerance values at the critical time (with peak demand, i.e. 8 am) are presented in Table 8.15. As expected, the greater the distance from the source, the smaller the reliability and damage tolerance values are. This table shows that from node 2 to nodes 3, 6, 5 and 4, the possibility of failure and the severity of failures on the performance of the networks are increased, by about 40% in node 4. Node 2, the most reliable one, has the highest R and T values and the lowest differences between them, in contrast with node 4. Also, the demand-weighted mean of the nodal reliabilities is again seen to be equivalent to the system reliabilities during this time period.

To evaluate the variations of nodal reliabilities during a day, Figures 8.9 and 8.10 present the diurnal profile of reliabilities and damage tolerances respectively, at each individual node. It can be seen that the reliability decreases from node 2 (the nearest to the source) to nodes 3, 6, 5 and 4 (the farthest from the source). Also, it is observed that although node 6 has the same distance from the source as node 2, its reliability is less than that of node 2. The reason is the higher required head value at node 6, i.e. 194 m, against 178 m for node 2. Therefore, it faces greater shortfall and lower reliabilities. During the peak demand time (i.e. morning and evening) the reliability values tend to decrease. It is observed that the variations of the reliability profile is the converse of the demand profile (Fig. 8.6).

Table 8.16 shows the daily system and nodal reliability and damage tolerance values which have been produced by the snapshot analysis using the deterministic average demands. The high reliability values indicate that deterministic daily average demand values, even when considering the possibility of one pipe failure, do not affect the hydraulic performance of the system. The same order in nodal reliabilities from node 2 to 4 is also observed. Very high values of R represent the rare possibility of failures even in node 4. The impact of failures can be seen by the values of damage tolerance which are up to 12%, at node 4. Comparison of Tables 8.14-16 indicates that snapshot analysis using the daily average demands seems to be unable to produce a realistic figure for overall nodal and system reliability or damage tolerance values because it cannot represent the effects of severe shortfalls at the peak demand times.

The required computer time for extended period simulation of the reduced network (with one pipe failure) during the 24 hours together with the calculations of the system and nodal reliabilities at each hour was about 48 seconds.

8.4 HEAD DRIVEN SIMULATION BASED RELIABILITY ANALYSIS USING VARIABLE (PROBABILISTIC) DEMAND (HDSRAPD)

The conclusions of Chapters 2 and 6 showed that the probabilistic (variable) nature of demand is important when analysing reliability of water distribution systems. This section aims to incorporate the variability of demand into the head driven simulation based reliability analysis . This analysis will be referred to as HDSRAPD, herein.

To consider the probabilistic nature of demand, two alternative approaches may be applied. First is generation of a set of random values to represent probabilistic (random) demands by a random number generation approach such as Monte Carlo simulation method. It was mentioned earlier that high computational requirement, is one of the disadvantages of the Monte Carlo technique. For instance, about 500 simulations were required from random number generation in the Bao and Mays (1990) case for just one snapshot analysis.

Alternatively, a probability density function can be considered to account for variations of demand. This function can be made more realistic by using a set of historical field data for demands for each region (Khomsi et al. 1996). In the absence of such a historical data set, a hypothetical distribution function can be assumed (Bouchart and Goulter 1991). Herein to have a sound base for comparison of the results of different reliability methodologies, the demand model and data sets of Khomsi et al. (1996) are chosen for probabilistic demand analysis.

8.4.1 Methodology

According to the Khomsi et al. (1996) approach, variability of different loading conditions may be expressed by a Load Factor (LF). Using a probability density function, the probability of each load factor can be obtained. To do this, the area under the probability density function curve can be divided into several, usually equal load bands across the x-axis (such as Fig. 8.11). Load factor may be defined as the ratio of the required outflow to the average load for the network. Thus, load factor for any load band k, LF(k), is given by

$$LF(k) = \frac{Q_j^{req}(k)}{Q_j^{ave}} \tag{8.11}$$

in which $Q_j^{req}(k)$ is the required outflow (demand) at node j averaged over load band k. Q_j^{ave} is the demand at node j averaged over all load bands. In the absence of detailed field data for particular nodes or networks, the load factor is determined based on the data set for a particular region. Because the summation of the load factor probabilities over the all bands is equal to one, the following relationship should be satisfied for the corresponding load factors and their probabilities.

$$\sum_{k=1}^{NLB} PLF(k) = 1 \tag{8.12}$$

where NLB is the total number of load bands and PLF(k) is the probability of the load factor at band k.

A general formulation which accounts for probabilistic demand and extended period simulation, is built up as follows. Considering the possibility of one link failure, Eq. 8.7 for lower bound system reliability (R_L) can be re-written as

$$R_L = \frac{1}{Q_s^{req}} \left(\sum_{l=0}^{NP} p(l)\ Q_s^{avl}(l) \right) \tag{8.13}$$

which is used for any snapshot analysis. An overall daily system reliability for deterministic demand can be given as

$$R_L = \sum_{t=1}^{NT} \frac{1}{Q_s^{req}(t,0)} \left(\sum_{l=0}^{NP} p(l)\ Q_s^{avl}(t,l) \right) PDF(t) \tag{8.14}$$

where $Q_s^{req}(t,0)$ is the total required outflow at time t with all links available, $Q_s^{avl}(t,l)$ is the total available outflow at time t when link l is unavailable. PDF(t) is the probability of the demand factor at time t which can be obtained by a set of historical data for any particular region and NT is the number of time intervals. The summation of demand factor probabilities over a period of time is equal to one, i.e.

$$\sum_{t=1}^{NT} PDF(t) = 1 \tag{8.15}$$

Considering the probabilistic nature of demand and a series of load bands under the probability distribution function, the overall system reliability is

$$R_L = \sum_{t=1}^{NT} \sum_{k=1}^{NLB} \frac{1}{Q_s^{req}(k,t,0)} \left(\sum_{l=0}^{NP} p(l)\ Q_s^{avl}(k,t,l) \right) PLF(k)\ .\ PDF(t) \tag{8.16}$$

in which $Q_s^{req}(k,t,0)$ is the total required outflow at band k and time t with all links available and $Q_s^{avl}(k,t,l)$ is the total available outflow at load band k, and time t, respectively, with link l unavailable. These can be obtained as follows.

$$Q_s^{avl}(k,t,l) = \sum_{j=1}^{NJ_d} Q_j^{avl}(k,t,l) \tag{8.17}$$

$$Q_s^{req}(k,t,0) = \sum_{j=1}^{NJ_d} Q_j^{req}(k,t,0) \tag{8.18}$$

It is worth noting that to calculate the system reliability at each individual time the summation over time and the PDF(t) factor are omitted from Eq. (8.16).

Having Eqs. 8.12 and 8.15 for summation of load and demand factors, the following relationship should be satisfied when combination of the extended period simulation with probabilistic demand is used.

$$\sum_{t=1}^{NT} \sum_{k=1}^{NLB} PDF(t)\ .\ PLF(k) = 1 \tag{8.19}$$

Finally the overall system reliability can be written as

$$R_L = \sum_{j=1}^{NJ_d} \sum_{t=1}^{NT} \sum_{k=1}^{NLB} \frac{1}{Q_j^{req}(k,t,0)} \left(\sum_{l=0}^{NP} p(l)\ Q_j^{avl}(k,t,l) \right) PLF(k)\ .\ PDF(t) \tag{8.20}$$

or

$$R_L = p(0)\,\Big(\sum_{j=1}^{NJ_d}\sum_{t=1}^{NT}\sum_{k=1}^{NLB} r_j(k,t,0)\ PLF(k)\ .\ PDF(t)\ +$$
$$\sum_{j=1}^{NJ_d}\sum_{t=1}^{NT}\sum_{k=1}^{NLB}\sum_{l=0}^{NP} r_j(k,t,l)\ \frac{ua_l}{a_l}\ PLF(k)\ .\ PDF(t)\Big) \tag{8.21}$$

in which

$$r_j(k,t,0) = \frac{Q_j^{avl}(k,t,0)}{Q_j^{req}(k,t,0)} \quad and \quad r_j(k,t,l) = \frac{Q_j^{avl}(k,t,l)}{Q_j^{req}(k,t,0)} \tag{8.22}$$

To evaluate the overall daily nodal reliability the first summation over demand nodes is omitted from Eqs. 8.20 and 8.21. Therefore, values of the nodal outflow and demand are calculated. The upper bound reliability and also damage tolerance formulations can be obtained in the same way.

8.4.2 Application

To be able to compare the results of the various methodologies, again the network of Fig. 8.5 (taken from Khomsi et al. 1996), i.e. Example 8.4 is chosen. The pipe and nodal data was seen in Tables 8.10 and 8.11. The probabilistic distribution of demands has been obtained from a set of 14 years daily data recorded for a region in Southwest England from 1976 to 1989. According to the investigations of Khomsi et al. (1996) the normal distribution was found to be the closest theoretical distribution function for this sample of data. Figure 8.11 shows this probability density function. As can be seen, the area under the curve of Figure 8.11 is divided into five equal bands using equal divisions along the x-axis. Based on Eq. 8.11 the load factors and their corresponding probabilities for each load band are calculated and shown in Table 8.17. The diurnal profile of demand factors and their corresponding probability values at each hour, PDF(t), was presented in Table 8.12.

Using the proposed methodologies of head driven simulation based extended period reliability analysis (HDSEPRA) (Section 8.3.1) and head driven simulation based reliability analysis with probabilistic demand (HDSRAPD) (Section 8.4.1), a set of analyses have been performed for this example. To make sense of the progression of the analyses, both the extended period and steady state analyses (with the daily average values) have been performed using probabilistic demands. The results are presented and discussed in the following subsections.

8.4.3 Head Driven Simulation Based Extended Period Reliability Analysis Using Probabilistic Demands

In this part of the analysis, by applying HDSEPRA and HDSRAPD (Sections 8.3 and 8.4), a set of probabilistic demand values have been used by an extended period reliability analysis through 24 hours. Besides the hourly system reliabilities and damage tolerances, values of mean and demand-weighted mean of the hourly reliabilities together with the daily average system values are calculated and shown in Table 8.18. The diurnal profile of reliability and damage tolerance values with probabilistic demands are shown in Figure 8.12. Comparison of the results from the probabilistic and deterministic demands (Tables 8.18 and 8.13) show that they are close together. From Figures 8.7 and 8.12 very slight differences between the results of the two sets of demand can be observed which suggest that both the reliabilities and damage tolerance results of probabilistic demands are less than the deterministic results. It is expected that the probabilistic demands lead to lower reliability than the deterministic demands, because when probabilistic demands are considered, higher demand values are permitted and more nodes can face insufficient heads and consequently, more shortfalls.

Table 8.19 shows the values of reliability and damage tolerance for the critical node which has the highest shortage and the lowest reliability (i.e. node 4) using probabilistic demands. The diurnal profile of reliability values have been shown graphically in Figure 8.13. As for the system reliabilities (Tables 8.18 and 8.13), there is a small difference between the results of probabilistic and deterministic demands for this node (Tables 8.19 and 8.14) in which the results of the HDSRAPD are less than the results of the HDSEPRA. Furthermore, Table 8.20 shows the reliability and damage tolerance values at the individual nodes for the peak demand time (i.e. 8 a.m.). As expected, the greater the distance from the source, the lower the reliability and damage tolerance values are. The most reliable node is node 2 and node 4 is the most critical one. Again the probabilistic and deterministic results are close together (compared with Table 8.15).

Also, Table 8.21 shows the daily nodal and system reliabilities using probabilistic daily average demands produced by the steady state analysis. In addition, the results of Khomsi et al. (1996) are presented for comparison. This table illustrates that the probabilistic results are slightly lower than the deterministic ones (Table 8.16). It can also be seen that the nodal and system reliability results of Khomsi et al. (1996) are lower than those from the HDSRAPD. Some explanations for differences in the two methods are presented next.

Table 8.21 shows differences between the results of the head driven based analysis (HDSRAPD) and the demand driven based analysis (Khomsi et al. 1996). Using a demand driven simulation (DDSM), Khomsi et al. (1996) have shown that many nodes with the load factors of 0.77 and 0.99 and all nodes with the load factors of 1.21 and 1.43, face unsatisfactory heads and therefore, suffer hydraulic failure. As illustrated in Chapters 5 and 7, the head driven analysis of the hydraulic performance of the system recognises the spatial nature of the shortfall and its results tend to be localised around critical nodes. Generally, the resulting head loss from the HDSM is less than that from the DDSM. Therefore, by using the head driven simulation based reliability measure, fewer nodes are expected to face shortfall in heads and outflows. Thus, the head driven simulation based reliability values are seen to be higher than the results of the demand driven simulation based measures.

As mentioned in Chapter 6 (Section 6.3.2.1.3) the approach applied by Khomsi et al. (1996) is just an approximation, i.e. a [0-1] measure for reliability values which does not recognise the reduced head and partial flow modes. As the demand driven simulation cannot quantify the values of shortfall for nodal demand, they have simply used a function which takes into account the number of times at which nodal heads are insufficient (Eqs. 6.22-23). In this [0-1] method, which has been used by other researchers (including Goulter and Coals 1986; Cullinane 1986 and Su et al. 1987), when nodal head, H_j, falls below the desired head, H_j^{des}, the situation is considered as a complete hydraulic failure. Such failures summed against non failures, without quantifying the actual shortfall, provides their simplistic reliability measure.

However, as mentioned in Chapter 6, recognising the pressure dependency of demands, the nodal outflow varies from 100% to 0% of the demand at the node. It means that in the reduced mode from H_j^{des} to H_j^{min} there is still outflow, albeit less than full demand, at the node. Thus, in contrast to the HDSRAPD, the reliability measure of Khomsi et al. (1996) is not able to incorporate the reduced outflow and evaluates the case as a no outflow situation. Therefore, consideration of either demand driven simulation and a [0-1] reliability measure leads to lower reliability values than the head driven simulation based reliability method. This situation is illustrated especially at node 4.

Furthermore, recognising the reduced mode for partial flows and heads, the values of shortfall are quantified in terms of the required and minimum nodal heads, in the HDSM. Therefore, the higher the required nodal heads, the lower the nodal reliabilities. For instance, the reliability of node 6 which is close to the source is lower than node 3. The cause of this is the high required head value, i.e. 194 m, at this node which by application of the HDSM, leads to higher shortfall and therefore, lower reliability. The Khomsi et al. (1996) method is unable to handle the above situation.

Another point regarding the approach of Khomsi et al. (1996) is that, in contrast with the head driven simulation based reliability analysis used in this research, it does not determine the system reliabilities directly and just calculates them as the demand-weighted mean of the nodal reliabilities. Although these two sets proved to be equivalent, if only system reliabilities were required, calculation of nodal reliabilities and their demand-weighted mean can be avoided. By having a direct relationship for calculation of the system reliability, using the results of the HDSM (Eq. 8.21), more computational time can be saved.

To evaluate the variations of nodal reliabilities during a day, the extended period analyses have been performed by the head driven simulation method for probabilistic demands. The total required computer time to calculate the system and nodal reliabilities based on the HDSRAPD is about 5 seconds for each individual time, for both the hydraulic and reliability analyses, or in other words, about 100 seconds which includes 960 simulations. Nodal reliability results are presented in Table 8.22. Figures 8.14-15 illustrate the diurnal profile for nodal reliability and damage tolerance through 24 hours, graphically. As for the system reliabilities, nodal reliability and damage tolerance values are higher at low demand times and are lower at peak demand times. For example, at the times of 8 a.m. and 18 with the highest demands, all the system and nodal reliabilities and damage tolerances experience their lowest values. Furthermore, the severity of the failures on the performance of the system can be seen in the profile of damage tolerances at the critical times and nodes, respectively. These show reductions from the reliability values by up to 25%.

It can be observed that combination of the extended period simulation and the probabilistic demand in the HDSRAPD has the advantage of having nodal and system reliabilities at each individual time and therefore, the diurnal profile of reliabilities through the 24 hour day. However, using the probabilistic daily average values for just one steady state analysis is unable to produce a realistic overall system reliability, in which the effects of critical times with smaller reliabilities can be accounted for. For instance, as during the critical times (i.e. morning and early evening) and for critical nodes (e.g. 4, 5 and 6) reliability values as low as 74% (at node 4) are obtained, the overall system or nodal values of more than 99% is grossly misleading. Thus, to overcome the above-mentioned shortcomings, a further step is performed by integrating the extended period and the probabilistic demands to be able to obtain realistic overall daily nodal and system reliabilities and damage tolerances.

8.4.4 Fully Integrated Extended Period and Probabilistic Analysis for Daily Reliabilities (IEPPD)

It was seen in Section 8.4.3 that the HDSRAPD produced realistic reliability results for individual hours. However, it could not represent logical daily reliability results when using the average daily demands by a steady state analysis (see Tables 8.18 and 8.21), because the severity of failures at individual times was not reflected. For further investigation of the effects of probabilistic demands, a fully integrated probabilistic and extended period reliability analysis is performed to evaluate the overall daily nodal and system reliabilities. For this purpose, the full capability of Eq. 8.21 is used, i.e. NT=24 and NLB=5. According to this procedure the simulation's results for every node at each hour, considering the probabilistic nature of demands (using the probability density function of Fig. 8.11, including five different loading conditions) and the possibility of one pipe failure at each case are used as the available probabilistic nodal and system outflows. Then, using the head driven simulation based reliability measure, values of nodal and system reliability and damage tolerance are produced. These are presented in Table 8.23.

Table 8.23 shows that the nodal reliabilities are of the same order from node 2 to node 4 (the highest and the lowest reliability). Also, values of the demand-weighted mean of nodal reliabilities are equivalent to the system reliability values. Furthermore, because of the superimposition of the probabilistic demands and extended period analysis, the effects of the probabilistic demands are clearly illustrated. In comparison with Table 8.21, the overall system and nodal reliabilities and damage tolerances from the fully integrated method (Table 8.23) shows lower values (by about 10%) and represents the differences of the results at each node, clearly. It is seen that the results from the probabilistic demands are smaller than those from the deterministic demands (Table 8.16) by 11% for reliability and damage tolerance values. While the HDSRAPD with steady state analysis for daily average demand (Table 8.18) produced just 1% difference with the deterministic results (Table 8.16) for the overall daily reliabilities, Table 8.23 resulting from the fully integrated method (IEPPD) produces difference of 11%. This represents a great improvement because it is 10% less than the steady state results from Table 8.21. The total required CPU time to obtain all the nodal and system outflows during the 24 hrs. and to produce the nodal and system reliabilities was about 100 seconds.

It can be observed from Tables 8.13 and 8.18 that consideration of only daily average demand values in the snapshot analysis cannot properly represent the overall system and nodal reliabilities through 24 hours. However, use of a fully integrated extended period simulation with probabilistic demand, which produces the most realistic results, leads to a large number of simulations and therefore, high computer time. To find an appropriate way to represent the overall daily reliabilities for probabilistic demands, from comparison of Tables 8.23 and 8.18, it can be found that the system reliabilities and damage tolerances arising from the fully integrated method are very close to the demand-weighted mean of the hourly reliabilities and damage tolerances through a period of 24 hours. The same similarity can be seen in the results of nodal reliabilities in Table 8.23 and the demand-weighted mean of hourly reliabilities in Table 8.22. The situation is similar for the damage tolerances. The relationship between the system reliabilities and demand-weighted mean of the hourly reliabilities can be seen in Appendix E. From Appendix E follows that the system reliabilities and damage tolerances arising from the fully integrated method are equivalent to the demand-weighted mean of the hourly reliabilities and damage tolerances in a period of 24 hrs. arising from the HDSRAPD.

It can be concluded therefore that use of the time consuming fully integrated extended period analysis using probabilistic demands to represent the overall daily nodal and system reliabilities can be avoided by using the demand-weighted mean of the hourly system and nodal reliabilities and damage tolerances from HDSRAPD. Therefore, with the same computational effort, the HDSRAPD produces extra information. If the variations of reliability and damage tolerance are not required, application of IEPPD leads to realistic system reliability values.

8.5 SUMMARY AND CONCLUSION

This chapter presents a step by step progressive reliability measure which evaluates the hydraulic performance of water supply systems based on the head driven simulation of the network. First, a head driven simulation based reliability measure (HDSRA) has been introduced to evaluate the reliability and damage tolerance of general networks including ancillary components such as pumps, valves, reservoirs, etc. using snapshot analysis. It is seen that the head driven simulation of the hydraulics of the pipe system, which produces the nodal heads and outflows realistically, is an appropriate basis for a direct reliability measure which calculates nodal and system reliability according to the more widely accepted definition of reliability as the ratio of the available flow to the demand. This, takes into account the possibility of any link failures and their corresponding probabilities. Therefore, this measure can evaluate the hydraulic reliability realistically and accurately.

Based on the Khomsi et al. (1996) results, it has been shown that the demand driven analysis is not a suitable means to evaluate the availability of nodal heads and outflows and consequently, the nodal and system shortfalls in flow delivery. Therefore, it can be concluded that any reliability measure based on results of the conventional demand driven simulation is not appropriate for water distribution networks which face insufficient head to meet demands in any part of the system.

Comparison of the results of the head driven simulation based reliability (HDSRA) with those from SHM and ISHM (from Chapter 7) confirms the conclusions that the ISHM is a close approximate to the HDSRA results but it is only applicable to single source networks.

As demands are not fixed during the 24 hours of a day, the reliability calculations based on just one snapshot analysis (considering daily average or peak demands) is not able to represent reliability realistically throughout the day. To address this, a head driven simulation based on extended period reliability analysis (HDSEPRA) has been presented which is able to represent the diurnal profile of reliability and damage tolerance values through the 24 hours. In the HDSEPRA the reliability method uses the system or nodal outflows which have been produced by the extended period head driven simulation of the network. According to the results, as expected, the higher the demand factors at any time, the lower the values for nodal and system reliability or damage tolerance obtained. In addition, based on the required criteria, reliability values for the critical time or node are crucial to properly show the severity of mechanical and hydraulic failures on the hydraulic performance of the system.

As water demands are not fixed, consideration of the probabilistic nature of demand leads to more realistic assessment of the system. A head driven simulation based reliability analysis with probabilistic demands (HDSRAPD) has been presented to account for this, which uses a probability density function, obtained from a set of historical data, to account for the variability of demands in both steady state and extended period analysis of the system. With probabilistic analysis, nodal demands can be higher than the design levels for the required outflows, so more head loss and consequently more shortfalls are obtained in the demand points. Results from an example network show that in both the steady state and extended period analyses, the HDSRAPD provides smaller reliability or damage tolerance values than those obtained from using deterministic demand specifications.

The results of the steady state analysis using the average daily conditions, from both deterministic and probabilistic demands, did not produce a reasonable figures for overall daily reliabilities. i.e. the severity of the shortfalls which would occur at critical times and certain nodes are not incorporated into the results. Therefore, to obtain a realistic overall daily reliability, a fully integrated extended period analysis with probabilistic demands (IEPPD) was performed. This method recognises the effects of the probabilistic nature (variations) of demands over a long period (including different hourly, day of the week, seasonal affects, etc.) on the overall reliability figures. Therefore, the overall daily nodal and system reliabilities and damage tolerances can be produced very realistically. However, this approach is computationally demanding.

It was recognised that there are usually differences between the system reliabilities and the mean and demand-weighted mean of nodal reliability and damage tolerance values. The mean of nodal reliability values was found not to give a proper representation for the overall system reliability, but the demand-weighted mean of the nodal reliabilities has been demonstrated to be theoretically equivalent to the system reliabilities. When only system reliability is required, the direct procedure of the head driven simulation based reliability analysis has the advantage of avoiding unnecessary calculations of nodal reliabilities and therefore, saves computer time. From the extended period simulation it was observed that the snapshot analysis using average daily demands is not capable of properly representing the overall daily system or nodal reliabilities. On the other hand, the demand-weighted mean of the hourly reliability and damage tolerance values has been shown to be theoretically equivalent to the overall daily values. Therefore, it can be concluded that to express the overall daily system and nodal reliability or damage tolerance values realistically, the demand-weighted mean of the hourly values can be used.

Comparison of the results from different procedures presented in this chapter, i.e. steady state and extended period analyses with deterministic or probabilistic demands show that the fully integrated extended period analysis with probabilistic demands (IEPPD) can represent more realistically the overall system and nodal reliabilities and damage tolerances. However, it is time consuming and is unable to represent the variations of reliability for individual times of the day.

On the other hand, by means of the extended period analysis, the diurnal profile of variations of reliability and damage tolerance values through the 24 hours can be obtained. Therefore, based on whichever criterion has been chosen by the decision maker to evaluate the reliability of the network (i.e. critical node, critical hour, overall system reliability, etc.) the extended period analysis can identify the range of reliability over the network, during a period of 24 hours and produce the diurnal profile of reliability values. Furthermore, to avoid using the time consuming integrated procedure, IEPPD, which provides only the overall daily reliability values, an overall daily reliability can be determined using equivalent values of demand-weighted mean of the hourly reliabilities resulting from the HDSRAPD.

Finally, it has been clearly demonstrated that the head driven simulation based extended period reliability analysis of water distribution networks, together with the imposition of probabilistic demands can be introduced as a powerful tool to evaluate the daily and hourly system or nodal reliabilities more realistically and accurately than the other available procedures.

Table 8.1: Pipe data for the network of Example 8.1 (Fig. 8.1).

Design	Pipe					
	1-2, 1-4[a]	2-3, 4-7	2-5, 4-5	3-6, 7-8	5-6, 5-8	6-9, 8-9
1	250	175	145	115	100	100
2	250	175	145	115	105	100
3	250	180	145	115	105	100
4	250	180	145	120	105	100
5	250	180	145	125	105	100
6	250	185	145	125	105	100
7	250	185	145	130	105	100
8	250	185	145	135	105	100
9	250	190	145	135	105	100
10	250	190	145	140	105	100
11	250	190	145	140	110	100
12	250	190	150	140	110	100
13	250	190	150	140	115	100
14	255	190	150	140	115	100
15	255	190	155	140	115	100
16	255	190	155	140	120	100
	Diameter (mm)					

[a] The network and all designs are symmetrical about the line joining nodes 1, 5 and 9; Length = 1000 m and CHW = 130 for all pipes.

Table 8.2: Nodal data[a] for the network of Fig. 8.1.

Node	Demand (m^3/s)	Required Head (m)
1 (Source)	0.2081	100
2	-0.0208	30
3	-0.0208	30
4	-0.0208	30
5	-0.0208	30
6	-0.0208	30
7	-0.0208	30
8	-0.0208	30
9	-0.0625	30

[a] Minimum head is 0 m at all nodes.

Table 8.3: Reliability and damage tolerance values for Example 8.1.

Design	Cullinane Eq. (Eq. 7.10)		Fujiwara & Tung Eq. (Eq. 7.11)		Su et al. Eq. (Eq. 7.12)	
	R	T	R	T	R	T
1	0.825802	0.745301	0.821998	0.744951	0.616208	0.602556
2	0.831056	0.749748	0.827260	0.749978	0.620534	0.606219
3	0.832401	0.751603	0.828653	0.751928	0.622060	0.607604
4	0.836921	0754929	0.833147	0.755503	0.625946	0.610994
5	0.837254	0.759671	0.833700	0.760242	0.630245	0.615169
6	0.838823	0.761423	0.835302	0.762118	0.631585	0.616337
7	0.842507	0.766499	0.839059	0.767060	0.635859	0.620623
8	0.845745	0.770720	0.842354	0.771180	0.639623	0.623205
9	0.847358	0.772569	0.844000	0.773123	0.640762	0.624670
10	0.850286	0.775323	0.846936	0.775887	0.643987	0.627582
11	0.850806	0.778612	0.847633	0.779907	0.647791	0.631015
12	0.851890	0.780566	0.848768	0.781811	0.650098	0.633130
13	0.853585	0.783866	0.850583	0.785784	0.653903	0.636524
14	0.855178	0.78596	0.852232	0.788189	0.655447	0.637980
15	0.856250	0.788099	0.853369	0.790421	0.658002	0.640384
16	0.858942	0.791277	0.856125	0.794180	0.661892	0.643830

Table 8.4: Nodal reliability and damage tolerance values for the network of Fig. 8.1 using Cullinane equation (Eq. 7.10).

Node	Design 1		Design 16	
	R	T	R	T
2, 4	0.999878	0.979750	0.999907	0.982614
3, 7	0.999598	0.933237	0.999732	0.950144
5	0.999711	0.965228	0.999810	0.973722
6, 8	0.999190	0.865565	0.999680	0.940487
9	0.420875	0.298139	0.530784	0.389659
Mean	0.927240	0.852559	0.941154	0.888734
Demand-Weighted Mean	0.825772	0.741462	0.858922	0.788727
System Reliability	0.825802	0.745301	0.858942	0.791277

Table 8.5: Pipe data for the network of Example 8.2 (Fig. 8.3).

Pipe	Diameter (mm)	CHW	Length (m)
1-2	20.32	100	304.8
6-1	30.48	100	609.6
2-3	15.24	100	304.8
5-2	20.32	100	609.6
4-3	15.24	100	609.6
5-4	15.24	100	304.8
6-5	20.32	100	304.8

Table 8.6: Nodal data for the network of Fig. 8.3 (Example 8.2).

Node	Demand (m^3/sec)	Required Head (m)	Minimum Head (m)
1	-0.0269	302.858	256.032
2	-0.0269	300.319	257.556
3	-0.0269	287.123	258.775
4	-0.0085	287.639	249.936
5	0.0000	302.264	254.508
6 (Source)	0.0892	304.800	259.080

Table 8.7: Reliability and damage tolerance values for Example 8.2, considering up to two simultaneous link failures (including pipes and PRVs).

Node	Cullinane et al. Eq. (Eq. 7.10)		Fujiwara & Tung Eq. (Eq. 7.11)		Su et al. Eq. (Eq. 7.12)	
	R	T	R	T	R	T
1	0.999831	0.998799	0.999644	0.997740	0.971961	0.938438
2	0.999647	0.997427	0.998596	0.991104	0.960037	0.912258
3	0.999502	0.996337	0.998030	0.987479	0.955135	0.901494
4	0.999814	0.998457	0.999232	0.995135	0.972207	0.938977
Mean	0.999699	0.997755	0.998876	0.992865	0.964835	0.922792
Demand-Weighted Mean	0.999657	0.997610	0.998802	0.992396	0.963314	0.919452
System	0.999650	0.997510	0.998781	0.992284	0.963256	0.919325

Table 8.8: Pipe data for the network of Example 8.3 (Fig. 8.4).

Pipe	Diameter (mm)	CHW	Length (m)
2-1	203.2	120	609.6
3-2	152.4	120	304.8
10-3	pump	-	-
4-2	203.2	120	304.8
5-4	152.4	120	304.8
9-5	pump	-	-
6-4	152.4	120	304.8
8-7	pump	-	-
7-6	203.2	120	304.8
6-1	152.4	120	609.6

Table 8.9: Reliability and damage tolerance values for Example 8.3, considering up to two simultaneous link failures (including pipes and pumps).

Reliability	Cullinane Eq. (Eq. 7.10)		Fujiwara & Tung Eq. (Eq. 7.11)		Su et al. Eq. (Eq. 7.12)	
	R	T	R	T	R	T
System and Nodal	0.984029	0.911147	0.980334	0.903705	0.914289	0.816032

Table 8.10: Pipe data for the network of Example 8.4 (Fig. 8.5).

Pipe	Diameter (mm)	CHW	Length (m)
1-2	250	130	1000
1-6	300	130	1000
2-3	250	130	1000
2-6	150	130	1000
3-4	100	130	1000
3-5	150	130	1000
5-4	200	130	1000
6-5	200	130	1000

Table 8.11: Nodal data for the network of Fig. 8.5 (Example 8.4).

Node	Daily Average Demand (m^3/s)	Minimum Head (m)	Required Head (m)
1	0.150	200	200
2	-0.020	158	178
3	-0.030	158	178
4	-0.040	148	168
5	-0.030	155	175
6	-0.030	174	194

Table 8.12: Hourly demand factors and their corresponding probabilities (PDF).

Time (hr)	Demand Factor	PDF	Time (hr)	Demand Factor	PDF
0	0.3983	0.01660	12	1.1589	0.04829
1	0.2567	0.01070	13	1.1220	0.04675
2	0.1856	0.00773	14	1.0050	0.04188
3	0.1577	0.00657	15	0.9528	0.03970
4	0.1916	0.00798	16	1.0978	0.04574
5	0.2736	0.01140	17	1.2718	0.05299
6	0.9170	0.03821	18	1.5983	0.06660
7	1.5649	0.06520	19	1.4910	0.06213
8	1.7686	0.07369	20	1.4484	0.06035
9	1.6170	0.06738	21	1.2917	0.05382
10	1.4691	0.06121	22	0.8642	0.03601
11	1.2607	0.05253	23	0.6378	0.02658

Table 8.13: Daily and hourly reliability[a] and damage tolerance[a] values of the network of Fig. 8.5 from the HDSEPRA, considering deterministic demands and possibility of one pipe failure.

Time	Cullinane Eq. (Eq. 7.10)		Khomsi Eq. (Eq. 7.14)	
	R	T	R	T
0	0.999975	0.992362	0.999984	0.994719
1	0.999996	0.998733	0.999997	0.999018
2	0.999998	0.999403	0.999998	0.999368
3	0.999998	0.999403	0.999998	0.999368
4	0.999998	0.999403	0.999998	0.999368
5	0.999996	0.998465	0.999997	0.998881
6	0.999808	0.932222	0.999830	0.944385
7	0.906904	0.796689	0.906897	0.800287
8	0.860357	0.758334	0.860340	0.760427
9	0.892670	0.786577	0.892662	0.789751
10	0.929982	0.816368	0.929978	0.821935
11	0.974049	0.865037	0.974046	0.872436
12	0.989785	0.888906	0.989789	0.897953
13	0.988812	0.899039	0.988823	0.909516
14	0.998011	0.919267	0.998028	0.930647
15	0.999794	0.927166	0.999813	0.939065
16	0.993184	0.902540	0.993194	0.912683
17	0.970992	0.862247	0.970989	0.869437
18	0.897712	0.790172	0.897705	0.793473
19	0.913331	0.811734	0.913321	0.816074
20	0.924128	0.820507	0.924119	0.825457
21	0.965537	0.857259	0.965532	0.864107
22	0.999830	0.939947	0.999854	0.952468
23	0.999911	0.978658	0.999935	0.978865
Mean	0.967146	0.897518	0.966493	0.902904
Demand-Weighted Mean	0.949568	0.858457	0.948742	0.864633
Daily System Reliability[b]	0.998265	0.920015	0.998282	0.931453

[a] All values are mean of the lower and upper bounds
[b] Using daily average demands by the snapshot analysis

Table 8.14: Daily and hourly reliability and damage tolerance values at the critical node (node 4) of the network of Fig. 8.5 from the HDSEPRA, considering deterministic demands and possibility of one pipe failure.

Time	Cullinane Eq. (Eq. 7.10)		Khomsi Eq. (Eq. 7.14)	
	R	T	R	T
0	0.999954	0.983379	0.999974	0.988887
1	0.999998	0.999403	0.999998	0.999368
2	0.999998	0.999403	0.999998	0.999368
3	0.999998	0.999403	0.999998	0.999368
4	0.999998	0.999403	0.999998	0.999368
5	0.999998	0.999403	0.999998	0.999368
6	0.999717	0.900169	0.999799	0.934357
7	0.835042	0.665553	0.835096	0.694691
8	0.747178	0.595203	0.747223	0.621719
9	0.817733	0.645463	0.817784	0.683712
10	0.862073	0.698753	0.862090	0.739121
11	0.960255	0.775393	0.960315	0.809058
12	0.999502	0.823961	0.999568	0.859134
13	0.999543	0.838619	0.999611	0.873218
14	0.999657	0.878782	0.999739	0.914846
15	0.999692	0.891265	0.999774	0.926245
16	0.999579	0.851346	0.999657	0.887924
17	0.954176	0.770184	0.954235	0.803654
18	0.822701	0.646165	0.822755	0.684997
19	0.861360	0.691084	0.861415	0.721173
20	0.872402	0.706843	0.872466	0.747313
21	0.931593	0.769229	0.931649	0.800554
22	0.999755	0.913472	0.999835	0.946194
23	0.999862	0.951327	0.999913	0.971619
Mean	0.944240	0.833050	0.944287	0.858579
Demand-Weighted Mean	0.913477	0.768448	0.913533	0.800524
Daily Nodal Reliability[a]	0.999660	0.879900	0.999742	0.915887

[a] Using daily average demands by the snapshot analysis

Table 8.15: Reliability and damage tolerance values of the network of Fig 8.5 at the peak demand time (8 am) considering deterministic demand and possibility of one pipe failure.

Node	Cullinane Eq. (Eq. 7.10)		Khomsi Eq. (Eq. 7.14)	
	R	T	R	T
2	0.999871	0.954333	0.999798	0.954172
3	0.930055	0.846714	0.924759	0.857758
4	0.747178	0.594606	0.747223	0.621719
5	0.832009	0.695170	0.832120	0.706515
6	0.876938	0.820020	0.882017	0.772869
Mean	0.877210	0.782288	0.877183	0.782607
Demand-Weighted Mean	0.860364	0.758346	0.860345	0.760443
System Reliability	0.860357	0.758334	0.860340	0.760427

Table 8.16: System and nodal reliability and damage tolerance values for the network of Fig. 8.5 considering deterministic demands and possibility of one pipe failure using daily average demands by the snapshot analysis.

Node	Cullinane Eq. (Eq. 7.10)		Khomsi Eq. (Eq. 7.14)	
	R	T	R	T
2	0.999941	0.979087	0.999908	0.969975
3	0.998823	0.937552	0.998824	0.948825
4	0.997526	0.879900	0.997573	0.911395
5	0.997756	0.913697	0.997788	0.915887
6	0.998124	0.922301	0.998130	0.930690
Mean	0.998434	0.926507	0.998447	0.935354
Demand-Weighted Mean	0.998273	0.920095	0.998289	0.931449
System Reliability	0.998265	0.920015	0.998282	0.931453

Table 8.17: Mean load factors and their corresponding probabilities for the network of Fig. 8.5.

Load Factor	0.56	0.77	0.99	1.21	1.43
Probability	0.0209	0.2127	0.4900	0.2545	0.0219

Table 8.18: Daily and hourly reliability and damage tolerance values of the network of Fig. 8.5 considering probabilistic demands and possibility of one pipe failure, using the HDSRAPD.

Time	Cullinane Eq. (Eq. 7.10)		Khomsi Eq. (Eq. 7.14)	
	R	T	R	T
0	0.999939	0.991297	0.999073	0.994225
1	0.999996	0.998612	0.999996	0.998872
2	0.999997	0.998889	0.999997	0.998939
3	0.999998	0.999038	0.999998	0.999113
4	0.999173	0.998686	0.999173	0.998805
5	0.997481	0.997521	0.997483	0.998152
6	0.991072	0.925282	0.991092	0.937402
7	0.904215	0.792062	0.904211	0.799528
8	0.856374	0.751135	0.856365	0.755220
9	0.892241	0.781134	0.892236	0.787872
10	0.923635	0.812712	0.923634	0.821045
11	0.967636	0.857365	0.967641	0.867466
12	0.980755	0.879396	0.980765	0.890510
13	0.984761	0.887003	0.984773	0.898455
14	0.996157	0.909384	0.996173	0.921384
15	0.997283	0.918903	0.997302	0.930900
16	0.987580	0.892152	0.987593	0.903805
17	0.965651	0.854917	0.965656	0.864900
18	0.896709	0.785195	0.896704	0.792176
19	0.901452	0.793772	0.901452	0.802122
20	0.919101	0.807603	0.919099	0.815492
21	0.961888	0.850750	0.961892	0.860554
22	0.999230	0.934110	0.999252	0.946300
23	0.999799	0.968658	0.999821	0.978285
Mean	0.963422	0.891066	0.963391	0.898397
Demand-Weighted Mean	0.944912	0.850644	0.944902	0.859245
Daily System Reliability[a]	0.996285	0.910811	0.996302	0.922598

[a] Using daily average demands by steady state analysis

Table 8.19: Daily and hourly reliability and damage tolerance values at the critical node (node 4) of the network of Fig 8.5 considering probabilistic demands and possibility of one pipe failure, using the HDSRAPD.

Time	Cullinane Eq. (Eq. 7.10)		Khomsi Eq. (Eq. 7.14)	
	R	T	R	T
0	0.999951	0.982830	0.999969	0.989787
1	0.999996	0.998638	0.999997	0.998920
2	0.999998	0.999403	0.999998	0.999368
3	0.999998	0.999403	0.999998	0.999368
4	0.999998	0.999403	0.999998	0.999368
5	0.999993	0.997661	0.999995	0.998366
6	0.998466	0.888174	0.998544	0.922000
7	0.825230	0.656564	0.825292	0.689531
8	0.745932	0.591428	0.745987	0.621135
9	0.804409	0.638771	0.804468	0.670734
10	0.859278	0.690391	0.859343	0.724542
11	0.941739	0.767928	0.941809	0.804012
12	0.968325	0.808810	0.968400	0.845478
13	0.974714	0.823401	0.974791	0.860276
14	0.996930	0.861547	0.997007	0.896968
15	0.997840	0.877049	0.997918	0.911580
16	0.979748	0.833157	0.979827	0.870092
17	0.936601	0.763440	0.936671	0.799456
18	0.812271	0.644945	0.812331	0.677162
19	0.851296	0.682638	0.851359	0.716336
20	0.866922	0.698069	0.866988	0.732488
21	0.927373	0.755883	0.927443	0.791741
22	0.999459	0.902077	0.999535	0.934142
23	0.999857	0.949354	0.999908	0.970082
Mean	0.936930	0.825457	0.936982	0.850956
Demand-Weighted Mean	0.904075	0.759499	0.904139	0.791293
Daily Nodal Reliability[a]	0.997157	0.863243	0.997234	0.898611

[a] Using daily average demands by steady state analysis

Table 8.20: Reliability and damage tolerance values of the network of Fig. 8.5 at the peak demand time (8 am) considering probabilistic demands and possibility of one pipe failure, using the HDSRAPD.

Node	Cullinane Eq. (Eq. 7.10)		Khomsi Eq. (Eq. 7.14)	
	R	T	R	T
2	0.999862	0.951366	0.999795	0.933161
3	0.923866	0.836096	0.923715	0.854865
4	0.745932	0.591428	0.745987	0.621135
5	0.822913	0.682983	0.822942	0.703141
6	0.873938	0.813803	0.873986	0.772857
Mean	0.873302	0.775135	0.873285	0.777032
Demand-Weighted Mean	0.856374	0.751139	0.856365	0.756230
System Reliability	0.856374	0.751135	0.856365	0.755220

Table 8.21: System and nodal reliability and damage tolerance values for the network of Fig. 8.5 considering probabilistic demands and possibility of one pipe failure (NLB=5; NT=1), using the daily average values with the steady state analysis.

Node	Cullinane Eq. (Eq. 7.10)		Khomsi Eq. (Eq. 7.14)		Khomsi et al. (1996)
	R	T	R	T	R
2	0.999937	0.977762	0.999902	0.968069	0.999552
3	0.999811	0.933152	0.999712	0.944381	0.977169
4	0.990638	0.863243	0.990230	0.898611	0.722810
5	0.997282	0.899456	0.997319	0.919011	0.978755
6	0.997867	0.918584	0.997930	0.906005	0.977022
Mean	0.996907	0.918439	0.996919	0.927216	0.931062
Demand-Weighted Mean	0.996287	0.910805	0.996307	0.922585	0.912600
System Reliability	0.996285	0.910811	0.996302	0.922598	0.912600[a]

[a] The system reliability has been calculated as weighted mean of nodal reliabilities by Khomsi et al. (1996)

Table 8.22: Nodal reliability values for the network of Fig. 8.5 considering probabilistic demands and possibility of one pipe failure, based on the Cullinane equation (Eq. 7.10).

Time	Reliability at node :				
	2	3	4	5	6
0	0.999998	0.999998	0.999951	0.999998	0.999998
1	0.999998	0.999998	0.999996	0.999998	0.999998
2	0.999998	0.999998	0.999998	0.999998	0.999998
3	0.999998	0.999998	0.999998	0.999998	0.999998
4	0.999998	0.999998	0.999998	0.999998	0.999998
5	0.999998	0.999998	0.999993	0.999998	0.999998
6	0.999949	0.999842	0.998466	0.999482	0.999516
7	0.999872	0.970528	0.825230	0.881759	0.890896
8	0.999862	0.923866	0.745932	0.822913	0.873938
9	0.999869	0.959814	0.804409	0.867850	0.884418
10	0.999871	0.979452	0.859278	0.909764	0.911678
11	0.999899	0.998118	0.941739	0.969348	0.973458
12	0.999903	0.999075	0.968325	0.983150	0.989840
13	0.999908	0.999488	0.974714	0.988359	0.994727
14	0.999936	0.999809	0.996930	0.998261	0.998749
15	0.999944	0.999829	0.997840	0.998978	0.999524
16	0.999911	0.999725	0.979748	0.991811	0.996421
17	0.999898	0.998017	0.936601	0.967888	0.984953
18	0.999870	0.964212	0.812271	0.872715	0.897014
19	0.999880	0.977368	0.851296	0.903230	0.913259
20	0.999885	0.981429	0.866922	0.916008	0.919914
21	0.999896	0.997633	0.927373	0.964710	0.974005
22	0.999955	0.999866	0.999459	0.999804	0.999825
23	0.999990	0.999953	0.999857	0.999934	0.999942
Mean	0.999929	0.989501	0.936930	0.959831	0.966753
Demand-Weighted Mean	0.999924	0.983174	0.904075	0.938191	0.948754
Daily Nodal Reliability[a]	0.999937	0.999811	0.990638	0.997282	0.997867

[a] Using daily average demands

Table 8.23: Overall daily system and nodal reliability and damage tolerance values for the network of Fig. 8.5 considering probabilistic demands and possibility of one pipe failure (NLB=5; NT=24) by the fully integrated approach (IEPPD).

Node	Cullinane Eq. (Eq. 7.10)		Khomsi Eq. (Eq. 7.14)	
	R	T	R	T
2	0.999913	0.962019	0.999861	0.971043
3	0.983163	0.893505	0.983050	0.904154
4	0.904065	0.759852	0.904128	0.780325
5	0.938180	0.823463	0.938212	0.833225
6	0.941828	0.885970	0.947035	0.897555
Mean	0.953430	0.864962	0.954457	0.877260
Demand-Weighted Mean	0.947039	0.851484	0.948075	0.864546
System Reliability	0.947000	0.851490	0.948008	0.860149

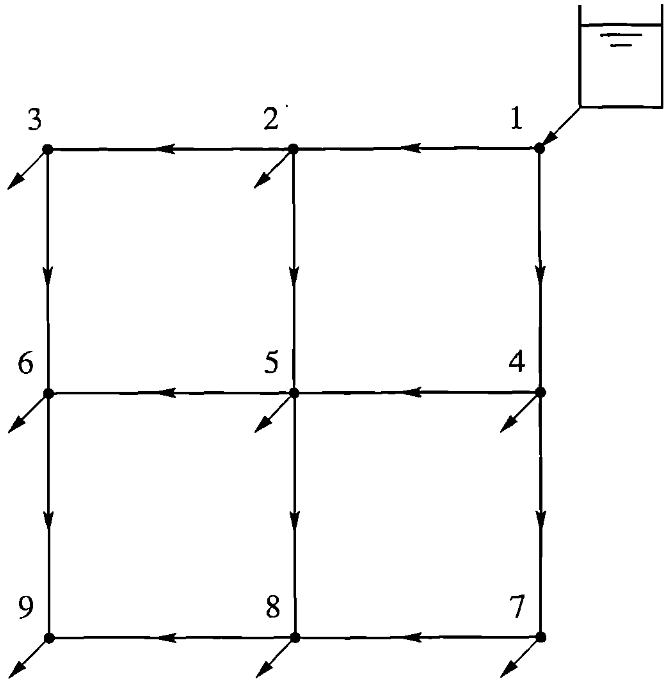

Figure 8.1: Layout of Example 8.1.

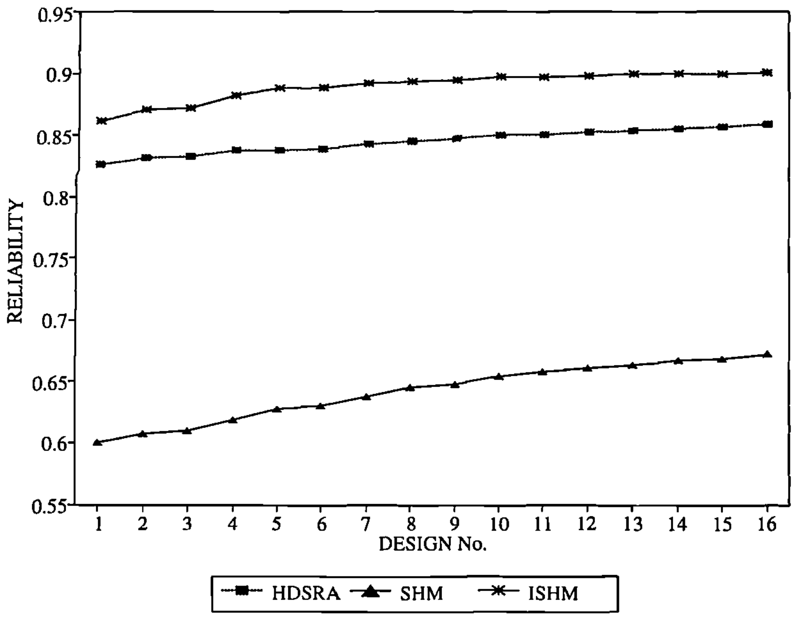

Figure 8.2: Reliability values for the network of Fig. 8.1 from different methods.

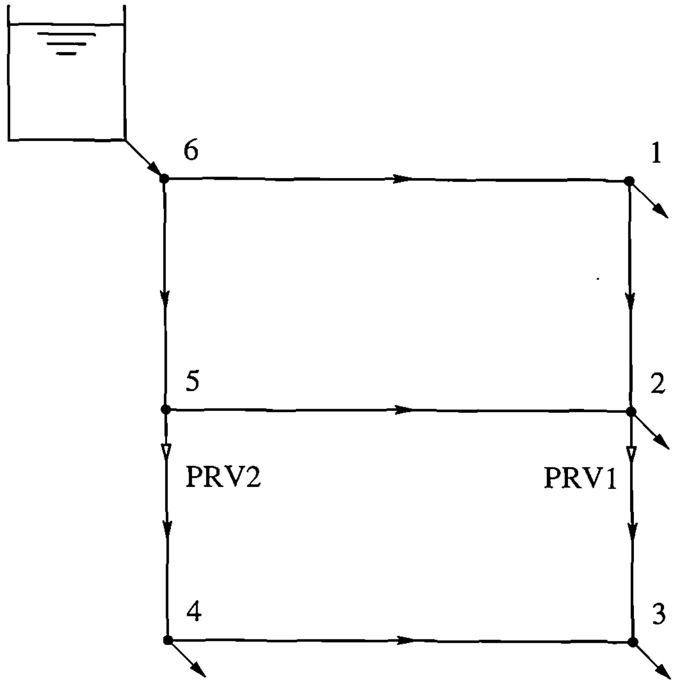

Figure 8.3: Layout of Example 8.2.

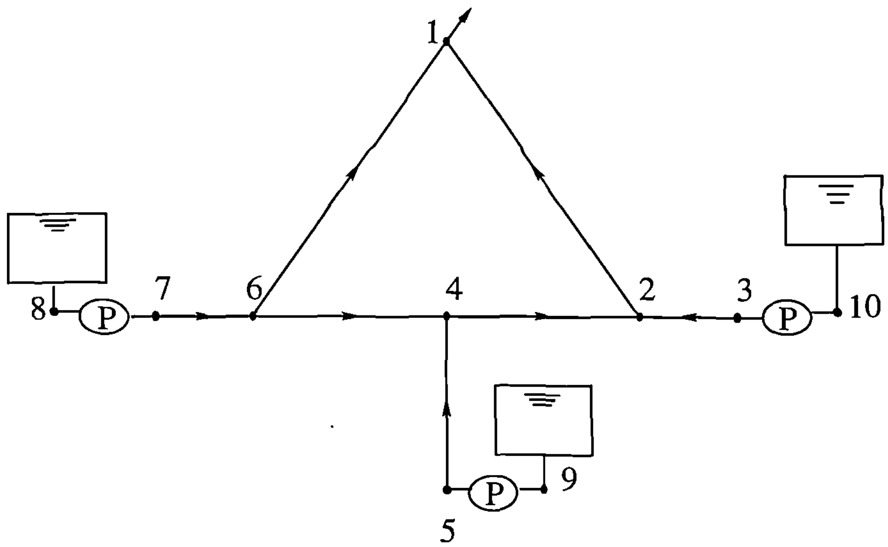

Figure 8.4: Layout of Example 8.3; adapted from Jeppson (1976).

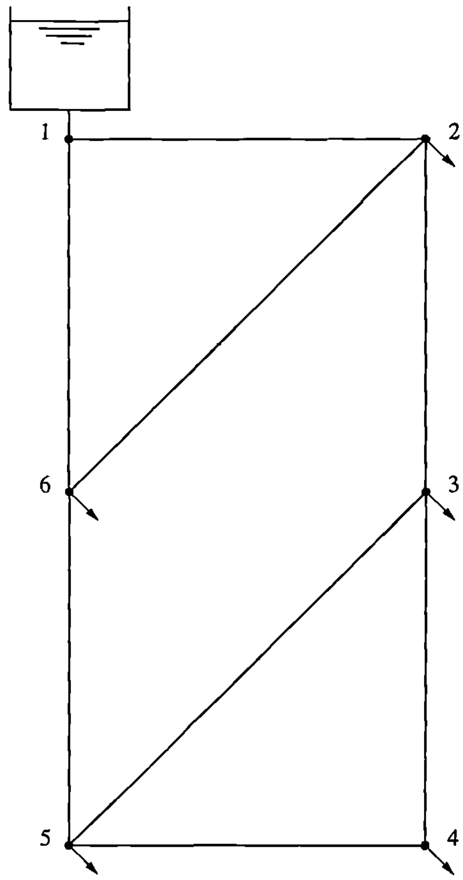

Figure 8.5: Layout of Example 8.4, taken from Khomsi et al. (1996).

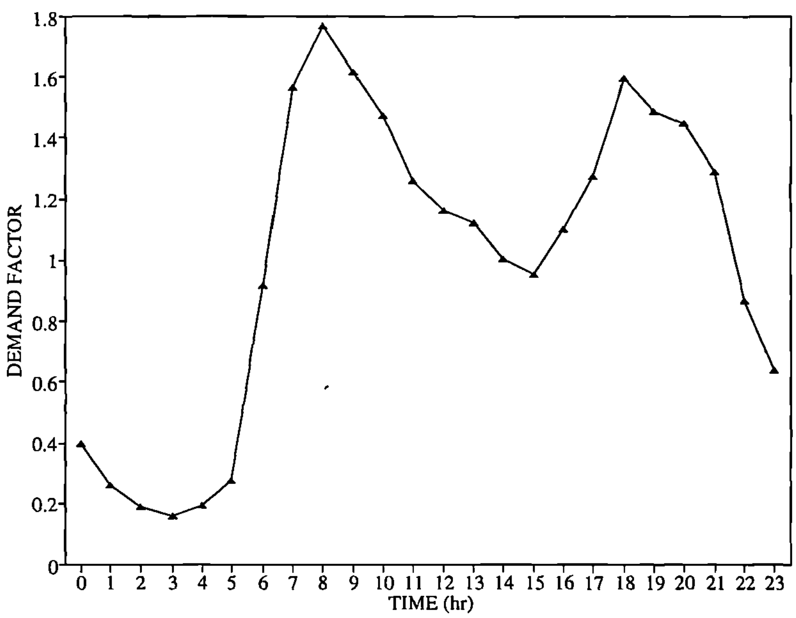

Figure 8.6: Diurnal profile of demand factors for the network of Fig. 8.5.

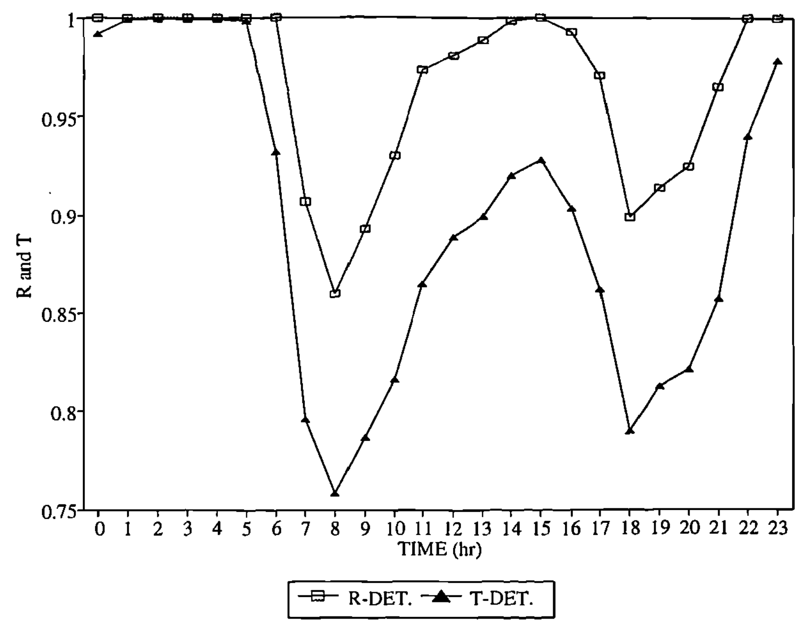

Figure 8.7: Diurnal profile of reliability and damage tolerance values for the network of Fig. 8.5, using the HDSEPRA and Cullinane equation.

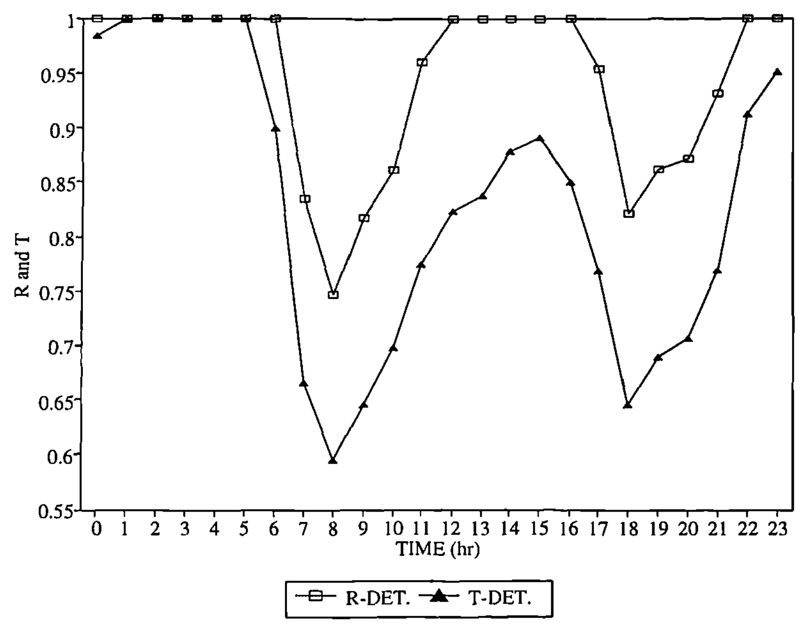

Figure 8.8: Diurnal profile of reliability and damage tolerance values at the critical node (node 4) of the network of Fig. 8.5, using the HDSEPRA and Cullinane equation.

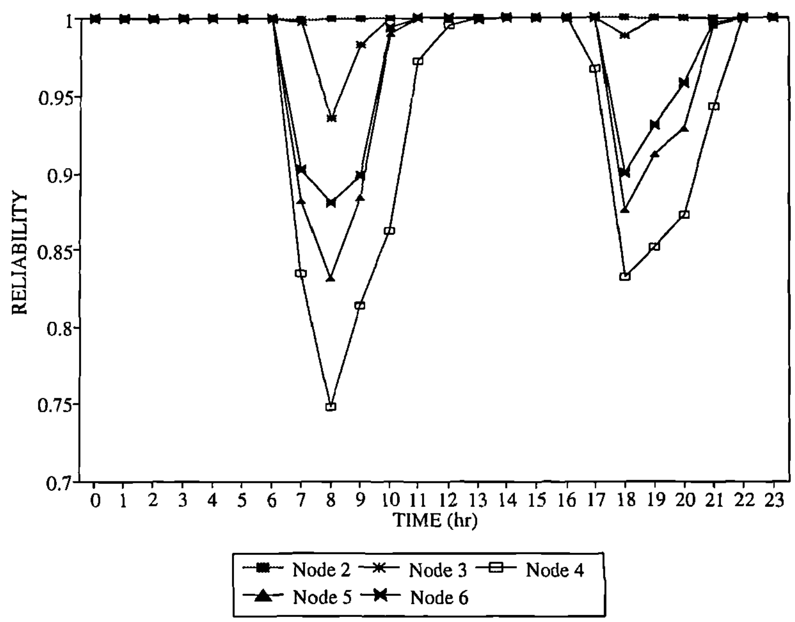

Figure 8.9: Diurnal profile of reliability values at individual nodes of the network of Fig. 8.5, considering deterministic demands and using the HDSEPRA and Cullinane equation.

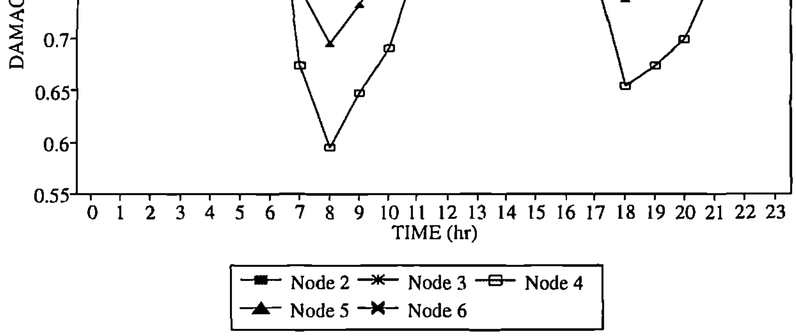

Figure 8.10: Diurnal profile of damage tolerance values at individual nodes of the network of Fig. 8.5, considering deterministic demands and using the HDSEPRA and Cullinane equation.

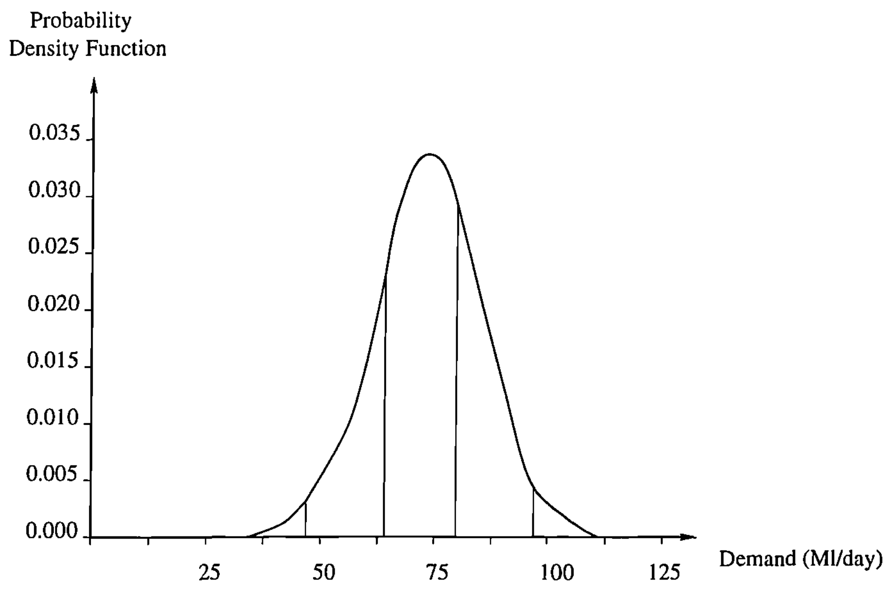

Figure 8.11: Probability density function of demand for a region in Southwest England, taken from Khomsi et al. (1996).

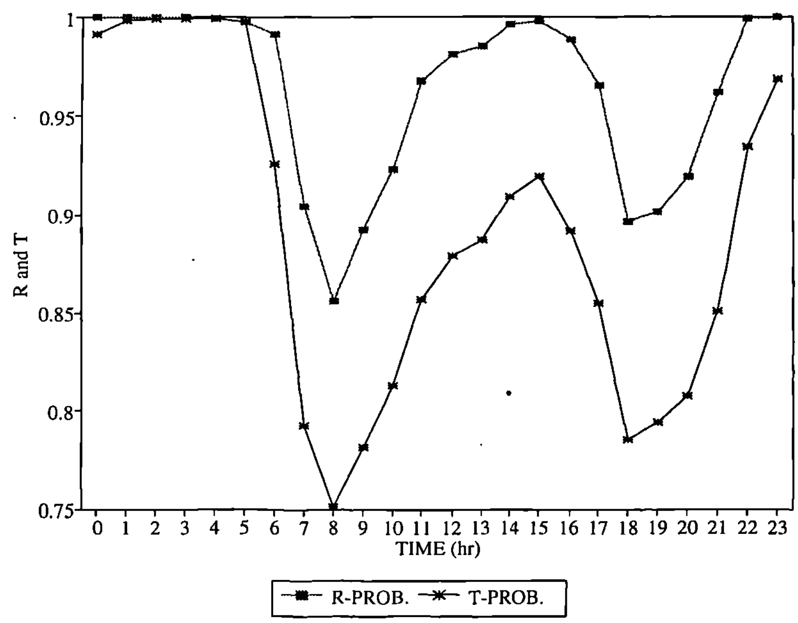

Figure 8.12: Diurnal profile of reliability and damage tolerance values for the network of Fig. 8.5, using the HDSRAPD and Cullinane equation.

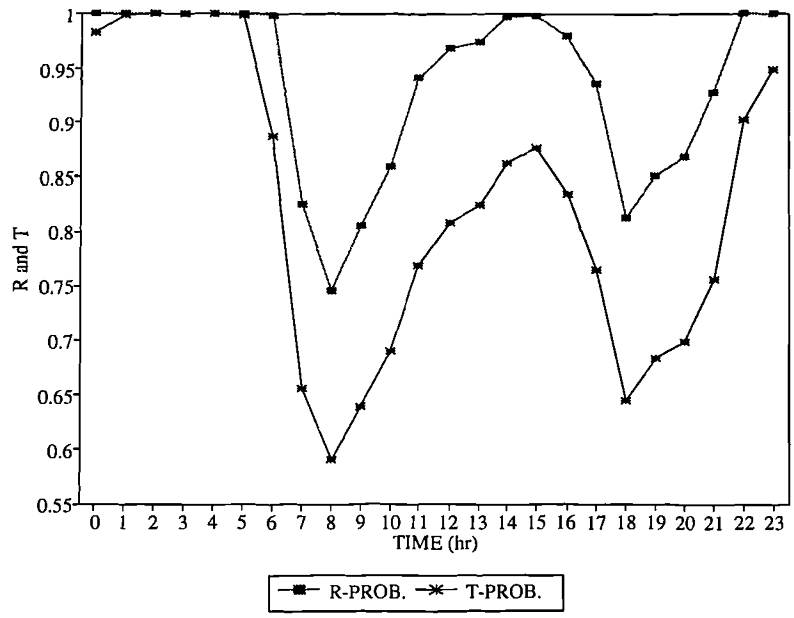

Figure 8.13: Diurnal profile of reliability and damage tolerance values at the critical node (node 4) of the network of Fig. 8.5, using the HDSRAPD and Cullinane equation.

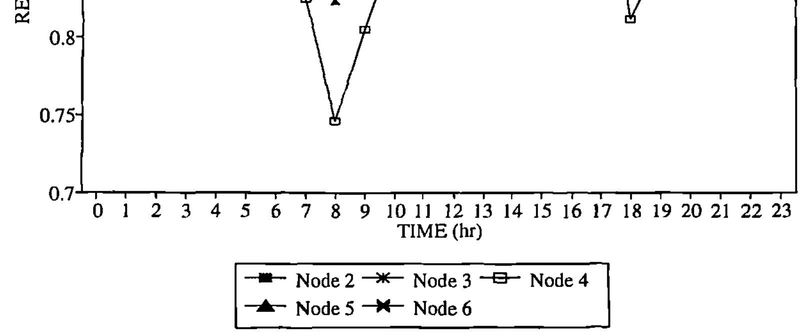

Figure 8.14: Diurnal profile of reliability values at individual nodes of the network of Fig. 8.6, using the HDSRAPD and Cullinane equation.

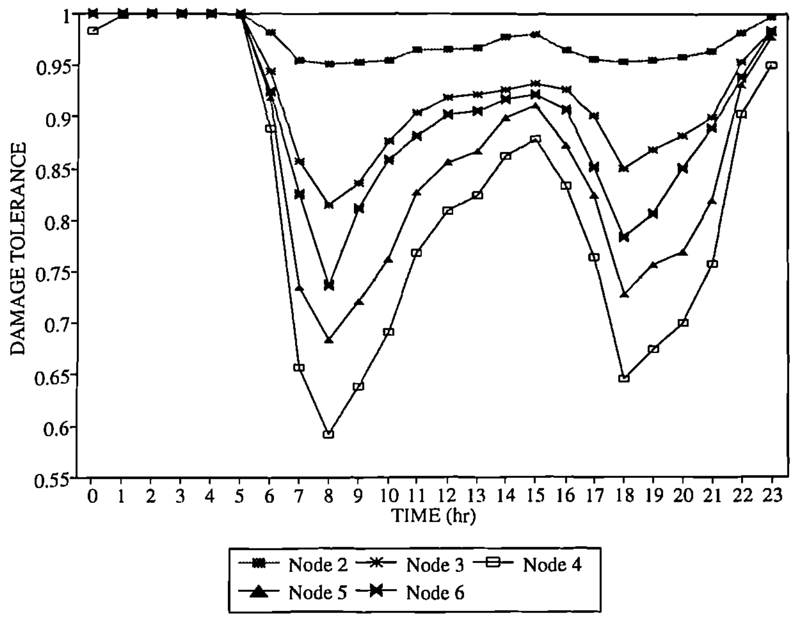

Figure 8.15: Diurnal profile of damage tolerance values at individual nodes of the network of Fig. 8.5, using the HDSRAPD and Cullinane equation.

CHAPTER 9

CONCLUSIONS AND RECOMMENDATIONS

9.1 INTRODUCTION

Full summaries of the methodologies and conclusions from findings have been presented at the end of each chapter. This chapter focuses on general conclusions and some recommendations for further work.

9.2 GENERAL CONCLUSIONS

In the three parts of this research methodologies have been developed for data management, mathematical modelling and reliability evaluation of water distribution networks.

Besides a systematic approach to demand evaluation and leakage computation, a best parameter estimation technique was developed in Chapters 2 and 3 to reconcile the collected field data and various demand figures. By such studies, it is expected that network modelling methodologies will be improved by:

a) production of more realistic leakage profiles for normal application in modelling;
b) reduced risk of bias in model performance under future operation scenarios by better quality assessment of base data and more reliable allocation between unmetered domestic consumption and leakage;
c) applying systematic flow reconciliation and hydraulic performance appraisal over the whole network, thus identifying serious anomalies and focusing needs for the checking of base data, field measurements and/or perceived pipe system connectivity.

To accommodate the random variations of different categories of demand, the best parameter estimation technique was conceived to systematically investigate the residual errors in zonal flow balance evaluation when the individual elements in the calculation are constrained to values lying within the chosen validity bounds. In this regard, the final stage which considers a combination of zones with possible unclosed boundary valves, leads to a better expression for hydraulic behaviour of the system, producing the optimal values of flow, demands and identifying possible unknown flows passing between each pair of adjacent zones.

In Chapters 4 and 5, a fully integrated methodology was developed for pressure driven analysis of water supply networks. The proposed procedure explicitly incorporates a realistic head-outflow relationship in the continuity equations. When nodal heads are adequate the designated demands are fully satisfied, but under subnormal conditions e.g. component failures or exceedance of demand, the head-driven analysis method (HDSM) can simulate the partial flow delivery realistically, whilst the demand driven simulation method (DDSM) can only indicate that a supply problem will arise. It would appear, therefore, that the methodology proposed has the potential to produce hydraulically more realistic results without any significant loss of computational efficiency compared to demand-driven analysis. This follows because it uses an integrated straightforward algorithm in which the head-outflow relationship is incorporated in the main set of equations. Furthermore, the ability of the method to predict the actual shortfall of nodal and network consumptions has been demonstrated.

Using advantages of the pressure dependency of demands, a number of reliability measures were introduced in the third part of this research (Chapters 6-8). First of all, an improved source head method (ISHM) of calculating reliability of single source networks was developed. This measure approximates the nodal and system reliabilities and damage tolerances with an acceptable overestimate of about 5%-10%. The method incorporates the relationship between the nodal outflows and source head in a demand driven simulation approach. In addition, a head driven simulation based reliability analysis (HDSRA) method was developed to assess the reliability of general water networks involving different ancillary components. The approach has the advantage that it uses the head driven simulation results and reliability is measured using the definition of reliability as the ratio of the available outflow to the demand. It also considers the probability of components failing. The accuracy of results and efficiency of the method are believed to be high in comparison with other available procedures.

Furthermore, in respect of the extended period simulation of water supply networks the head driven simulation based extended period reliability approach (HDSEPRA) proved able to produce realistic diurnal profile for variations of nodal and system reliability and damage tolerance values in a given period of time. In addition, by inclusion of the probabilistic nature of demand, the head driven simulation based reliability analysis with probabilistic demand (HDSRAPD) produced more realistic results for nodal or system reliability and damage tolerance values in which the effects of demand variations through a long period of time and different situations were accounted for.

Finally, it was shown that the demand-weighted mean of nodal reliabilities and damage tolerances are equivalent to the system reliability and damage tolerance values, using the proposed reliability measure. Also, the demand-weighted mean of the hourly nodal or system reliability and damage tolerance values are equivalent to the overall daily system or nodal reliabilities and damage tolerances. In contrast to the steady state analysis using daily average demands, the demand-weighted mean of hourly reliabilities produced more realistic results in that the magnitude and severity of shortfalls at critical times and nodes were fully incorporated.

9.3 SUGGESTIONS FOR FURTHER WORK

The following points are suggested to further develop the presented methodologies and improve the findings of this research.

9.3.1 Data Management

The proposed practical data management procedures were examined for a case study from a particular region in the UK. However, the demand and leakage evaluation procedures developed in Chapter 3 should be applied to other water supply networks to enable the formulation of fully verified procedures for data reconciliation and parameter assessment suitable for implementation in water management activities.

With respect to the pressure dependency of leakage it can be recommended that appraisal of the computed pattern of per capita consumption (PCC) and the correlation between leakage and average zone pressures should be added to the routine procedure for reconciliation of zonal demand (consumption) data sets.

In addition, the more recently proposed fixed and variable discharge paths (FAVAD) concept (May 1994, Lambert 1997b) merits consideration as the leakage-pressure relationship and the results should be compared with the linear relationship applied in this research.

The methodologies of Chapters 2 and 3 on water demand and system leakage evaluation are applicable to the areas in which both the domestic demand and leakage are unmetered such as the UK. However, because the domestic demand in most countries is metered, it may be worth-while to examine the validity of the proposed procedure for data reconciliation in such situations.

The best parameter estimation technique can be used as a practical tool in real engineering studies. In this technique, variations of possible passing flow and zonal head differences between any two adjacent zones are assumed to be linear. However, consideration of the realistic variation of flows with square root of head differences can improve the accuracy of the method. In addition, this methodology can be developed further with consideration of more realistic probability distribution functions instead of the uniform distribution used herein to represent the variability of different demand categories. As well as the average magnitudes for different types of demand, the random variability and uncertainty in the demand profiles should be taken into account. The effects of the sample size dependency on the domestic consumption profiles should also be considered.

9.3.2 Head Driven Simulation Method

Several head-outflow relationships were evaluated in Chapter 5. However, the real relationship is still unknown. Therefore, to find the most realistic relationship more investigation is required. For instance, an experimental procedure with different pressure-outflow conditions could usefully be developed and its results compared with different theoretical formulations. Most importantly, however, an extended field survey is called for to investigate the pressure dependency of consumption of a wide range of different consumer types.

The procedure for the head driven simulation used herein does not consider explicitly the pressure dependency of some categories of demand such as non-domestic consumptions and leakage. With respect to the pressure dependency of leakage introduced in Chapters 2 and 3, it was recognised that different relationships are applicable for 可应用于 leakage 泄漏, because it is in fact an uncontrolled outflow. Therefore, to have a comprehensive 全面的 head driven simulation method the head-outflow relationship of each type of demand should be introduced within the head driven simulation algorithm and accuracy and efficiency of the solution should be investigated.

In this research a number of sample networks were evaluated by the head driven analysis method. Applying this methodology to further networks which are more realistic and complicated and comparing the results with the demand driven results would be useful. On the other hand, the applicability of other hydraulic simulation approaches such as the Gradient Method (Todini and Pilati, 1988) to the head driven analysis can also be usefully investigated.

Despite identifying the pressure dependency of demand and introducing several relationships together with some methodologies for head driven simulation of water distribution networks, the conventional demand driven analysis is still applied widely in the engineering field. However, it has been identified that during failure conditions its results for pressures and water withdrawal are questionable. Perhaps it is not clear to modellers which kind of demands are really pressure dependent. It seems that in normal engineering applications only leakage has been recognised as pressure dependent and the other categories of demand are assumed to behave as volume related values. Therefore, an investigation is required to determine scale of sensitivity of different consumer types to supply pressure. More recently, however, the importance of the HDSM has been recognised at least for intermittent water networks in which water outflows are available only for a short period of time due to lack of adequate pressure (see Lumbers 1996). To have a realistic design, the HDSM should be developed as a commercial code.

9.3.3 Reliability Assessment

In the interim, while the head driven analysis of the water distribution networks has not become popular, the improved source head method of calculating single source network reliability has the advantage of being easily added to the current commercial modelling codes which use conventional demand driven simulation (DDSM).

The capability of the head driven simulation based reliability analysis method (HDSRA) should be investigated for more realistic and complicated networks. In addition, comparison of results from other available reliability approaches and the HDSRA is suggested by evaluation of the accuracy and efficiency of each method for different network sizes.

Generally, it is considered that producing commercial codes based on the HDSM for design and management purposes would be crucial to progressing the analysis of water systems towards the real situation. Such models would be able to evaluate the network reliability under critical events as well as the simulation of its normal function.

To compare different reliability measures which use probabilistic demands, it would be worthwhile to examine the same examples using other methods such as Monte Carlo simulation against the HDSRAPD, for appraisal of respective accuracies of results and computational requirements.

The methodology of HDSRAPD merely extends the range of demand variability (included in a 24 hour simulation) since the load factor is applied equally to all demand nodes at the same time. However, the variable nature of demand may be approached from another view point, which permits individual nodes to take demands from the distribution function, randomly (i.e. not all nodal demands take the same load zone at the same time).

One of the disadvantages of using the probabilistic demands in reliability analysis, is the huge number of simulations needed by all the identified methods. An investigation is required to find out if the resulting accuracy of the solutions is worth the extra effort, in comparison with the results based on deterministic demands. A criterion could then be established to determine a balance between accuracy of results and computational effort.

One of the most likely places for leakage is joints or fittings in water distribution networks. Therefore, investigation on the reliability of joints would be important especially for nodal reliability assessment.

As illustrated in Chapter 8, mechanical reliability/availability formulations for different components in water pipe networks produce different reliability results. One of the reasons for this discrepancy is absence of enough historical data for failure rates, repair times or in other words, frequency and duration of failure-non failure conditions for all available components such as pipes, pumps, valves, reservoirs, etc. Therefore, to obtain a proper and realistic figure for reliability/availability of all components, more field data from different climates and geological conditions should be collected and published. Normally, different water companies collect such data in their region and would keep them confidential. Thus, publicising such valuable data sets would significantly help reliability analysis of existing and new networks in each region. Statistical analysis of such data is a basic step in this procedure.

9.3.4 Optimal Network Design and Operation

Usually reliability measures are included inside some optimization schemes for water supply systems to obtain reliable and cost effective designs. Therefore, the reliability measures introduced in this research could be combined with optimization procedures.

The results could then be compared with several available reliability-based optimal solutions in terms of the accuracy and efficiency. Furthermore, since the use of Genetic Algorithms in water distribution systems is growing, combination of the proposed head driven simulation based reliability analysis with a GA approach could be investigated.

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APPENDIX A

THE 'ACORN' CLASSIFICATION

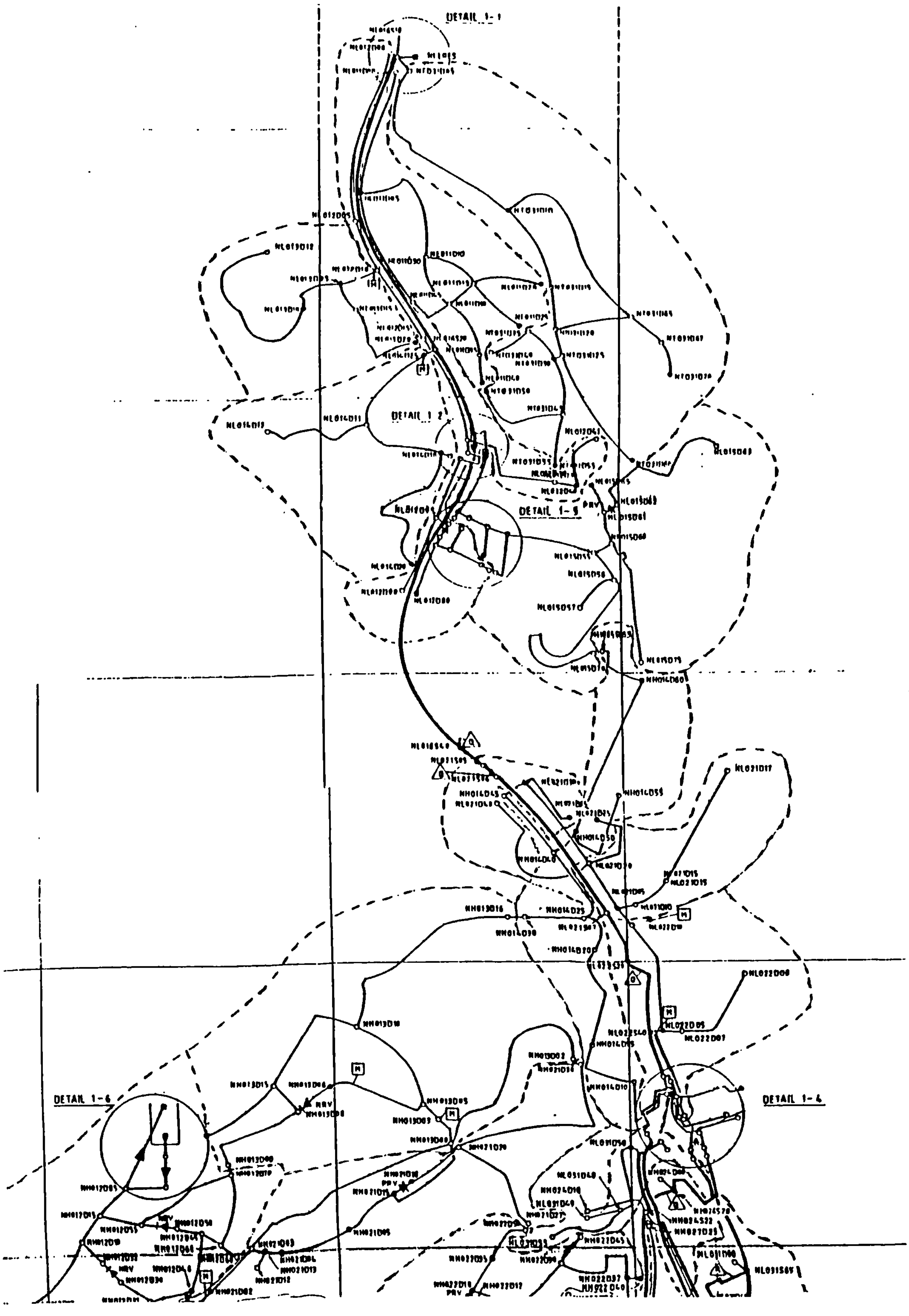
The 'ACORN' (A Classification Of Residential Neighbourhoods) is a geo-demographic classification system for small areas, which classifies enumerate districts (EDs) according to similarity across selected census variables.

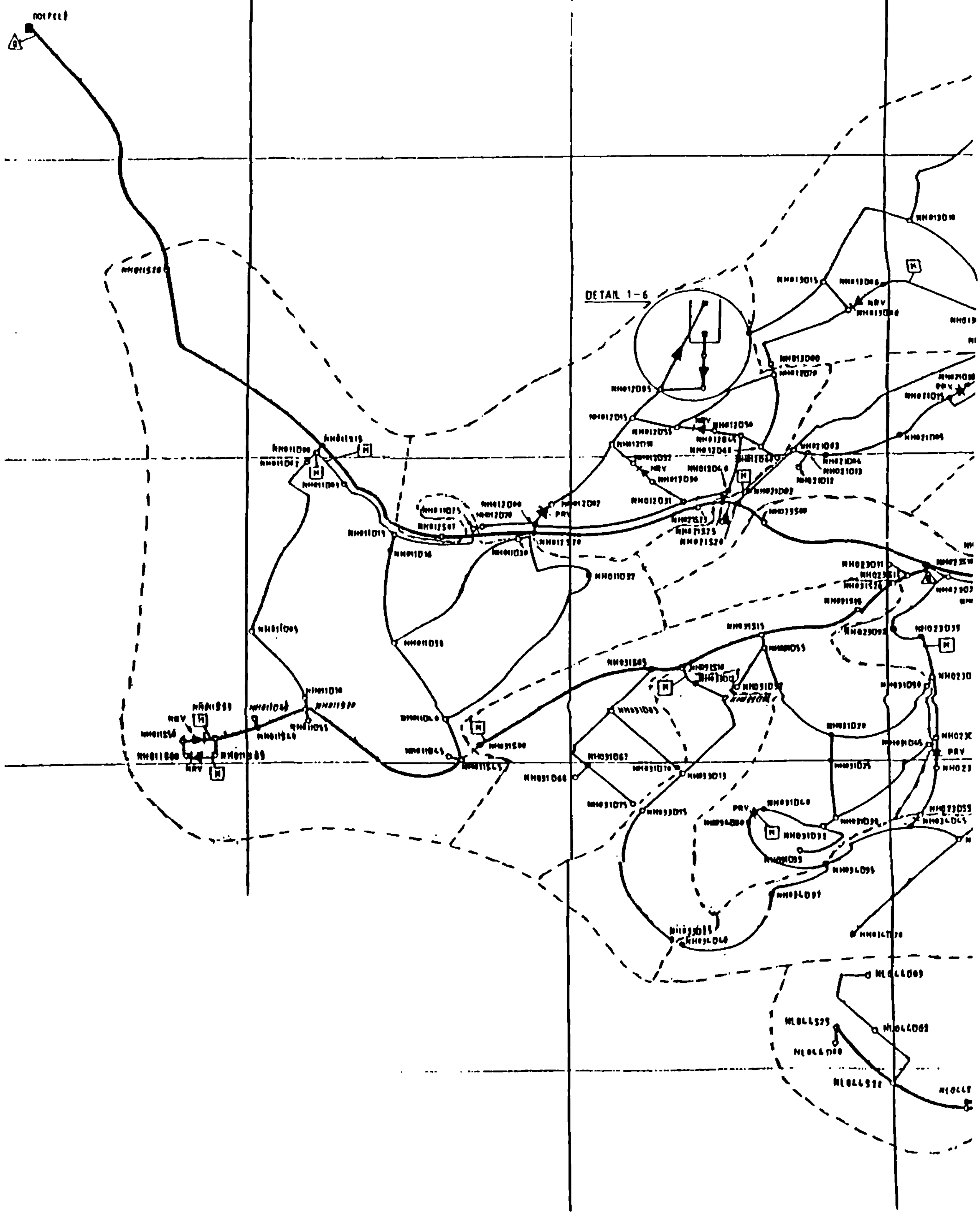
According to Edwards and Martin (1995), this concept which has been suggested by CACI (1981) classifies properties through England and Wales into a number of categories as follows.

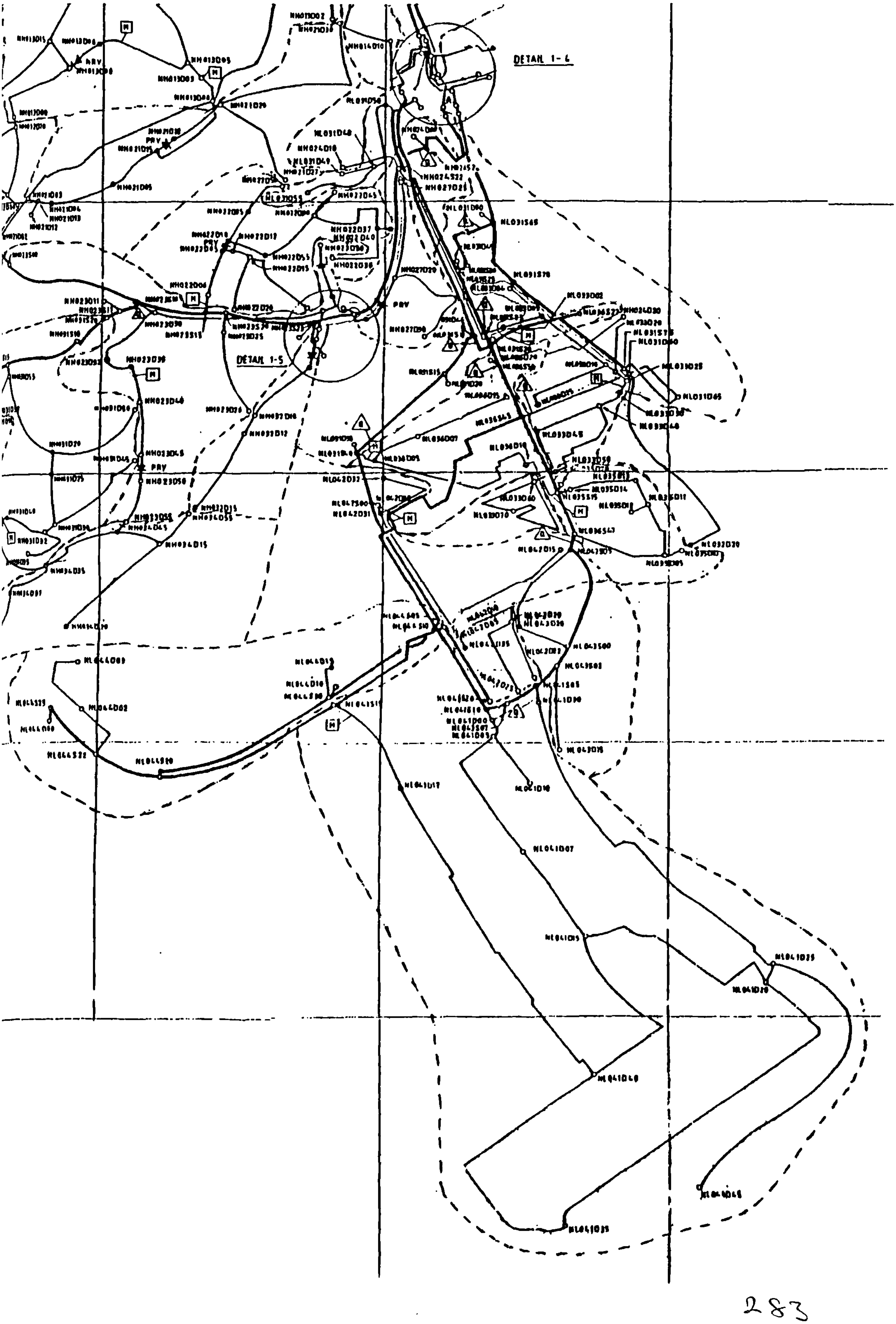
a) Housing in agricultural areas.
b) Modern family housing, higher income bracket.
c) Older housing of intermediate status.
d) Older traced housing.
e) Council housing (in three categories)
f) High status non-family areas.
g) Affluent suburban housing.
h) Better-off retirements areas.

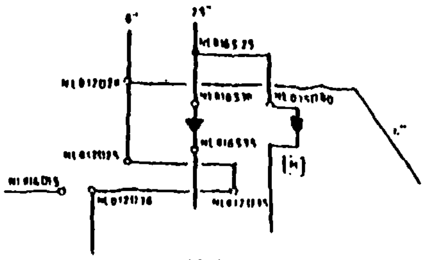
APPENDIX B

LAYOUT AND CONFIGURATION OF THE 'ONETOWN' WATER SUPPLY SYSTEM

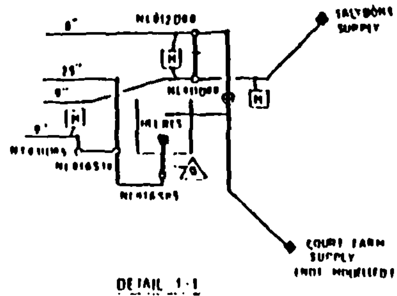

DETAIL 1-1
DETAIL 1-2
DETAIL 1-5
DETAIL 1-6
DETAIL 1-4
PRV


DETAIL 1-6

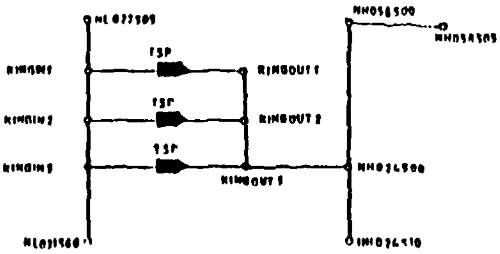

DETAIL 1-4
DETAIL 1-5



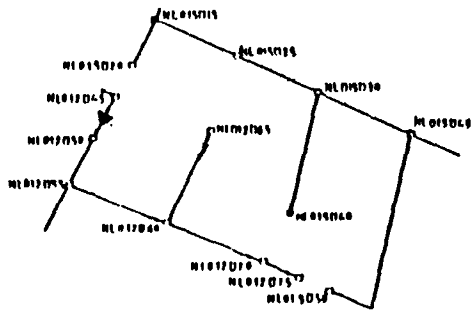
DETAIL 1-2
MALPAS VALVE HOUSE
& BRYNGLAS PUMPS



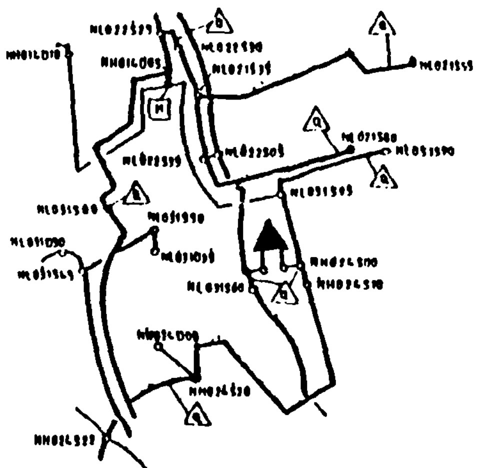
DETAIL 1-1
LLANTARNAM S R



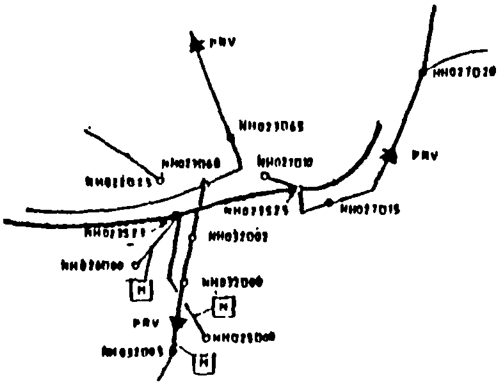
KINGSWAY P.S.
DETAIL 1-4



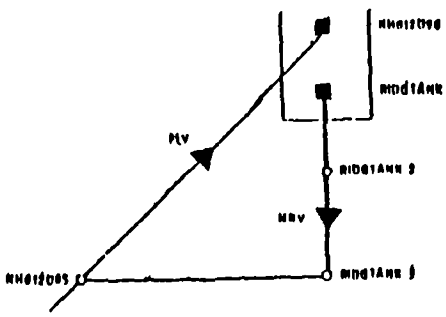
DETAIL 1-3



DETAIL 1- 6



DETAIL 1-5



DETAIL 1 - 6

LEGEND

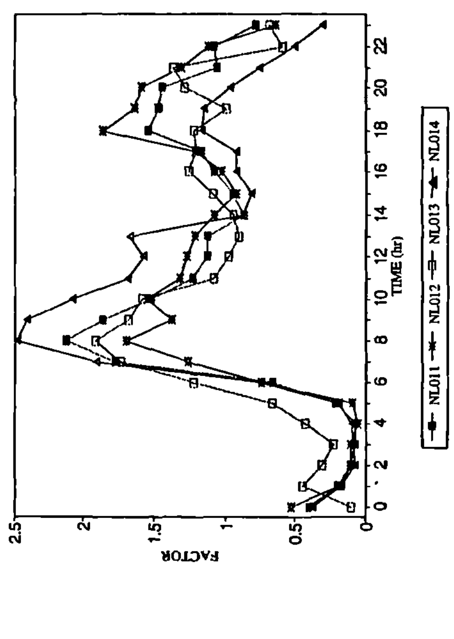
- NODE
- VARIABLE HEAD SOURCE
- FIXED HEAD SOURCE
- PRESSURE LOGGING SITE
- [M] FLOW MONITORING SITE
- QUADRINA MONITORING SITE
- UP UNAVAILABLE PIPE OR SHUT VALVE
- BOOSTER PUMP
- NRV NON RETURN VALVE
- PRV PRESSURE REDUCING VALVE
- THV THROTTLE VALVE

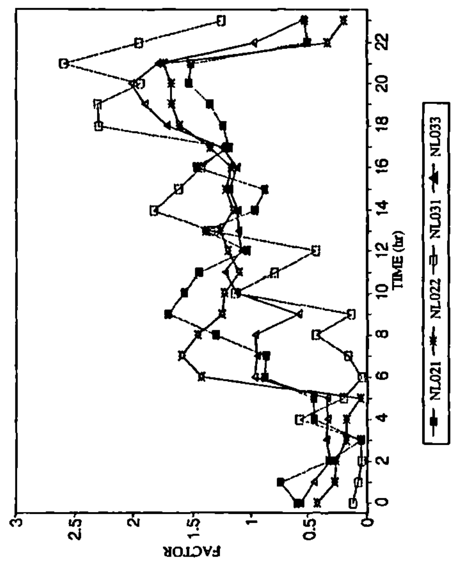
APPENDIX C

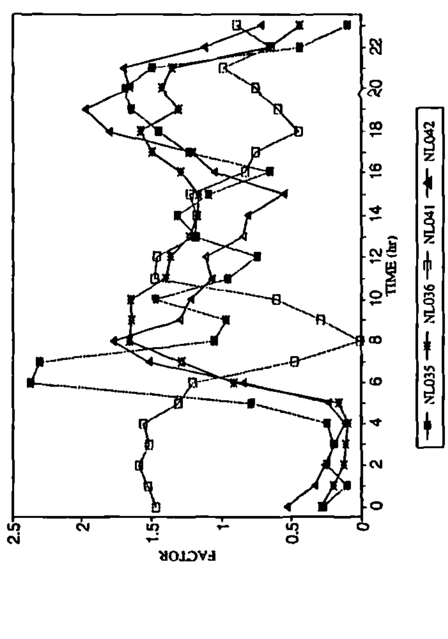
DIURNAL PROFILES OF DOMESTIC DEMAND, LEAKAGE AND PRESSURE FOR INDIVIDUAL ZONES IN 'ONETOWN' WATER SUPPLY SYSTEM

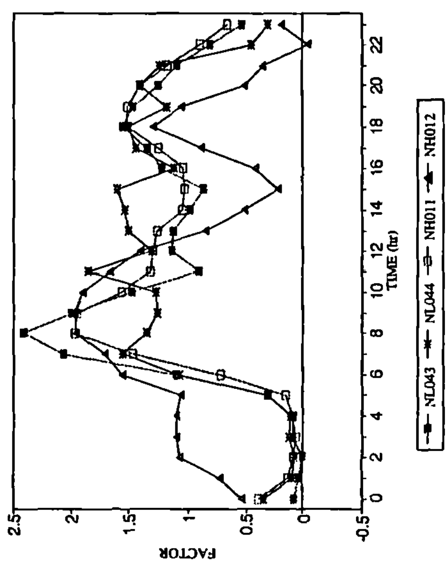
APPENDIX C1

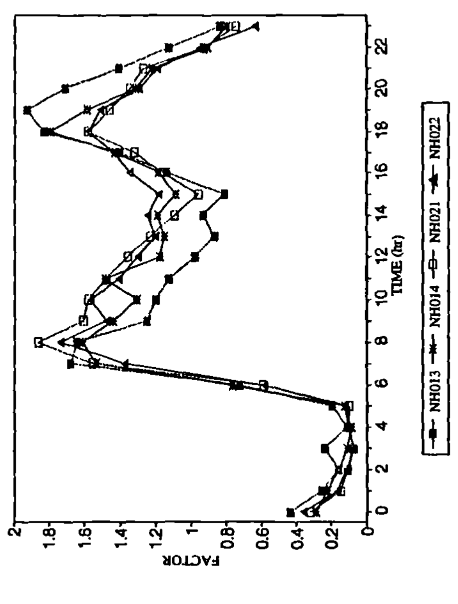
DOMESTIC DEMAND PROFILES BASED ON 'WRC - TYPE 5' LEAKAGE PROFILE (L-WRC)

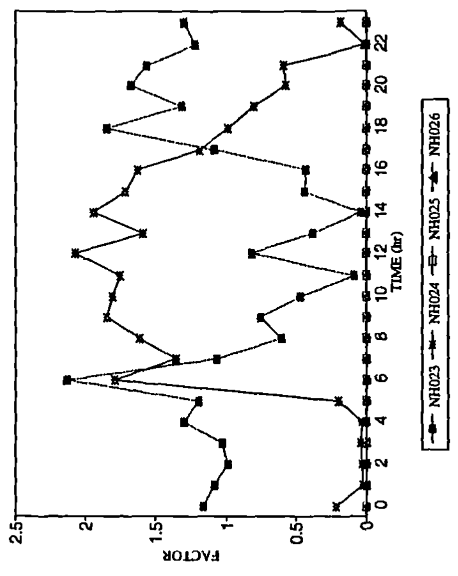

FACTOR
TIME (hr)
NL011
NL012
NL013
NL014


FACTOR
TIME (hr)
NL021
NL022
NL031
NL033


FACTOR
TIME (hr)
NL035
NL036
NL041
NL042


FACTOR
TIME (hr)
NL043
NL044
NH011
NH012


FACTOR
TIME (hr)
NH013 NH014 NH021 NH022


FACTOR
TIME (hr)
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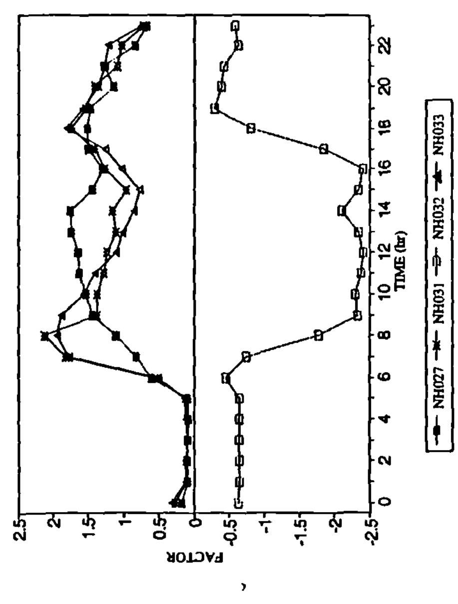
FACTOR
TIME (hr)
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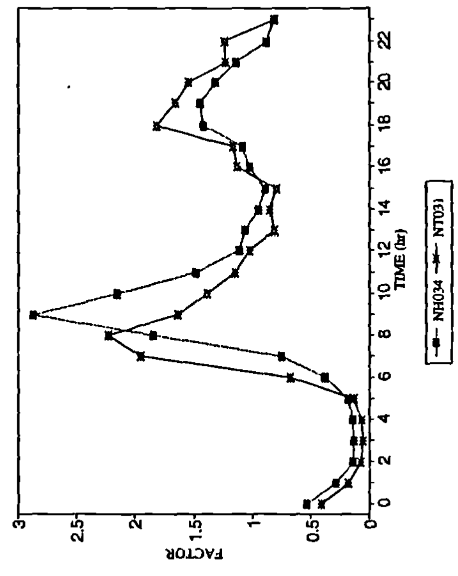
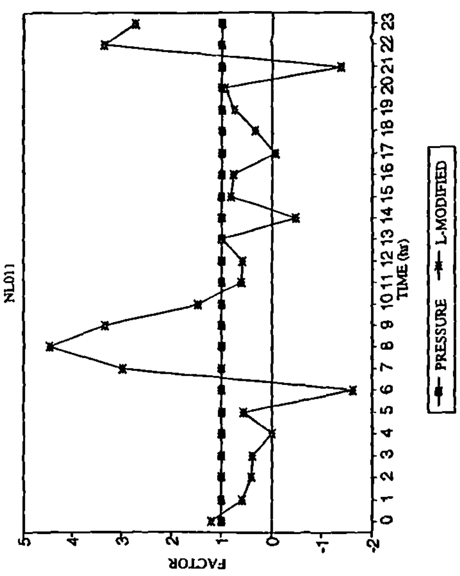
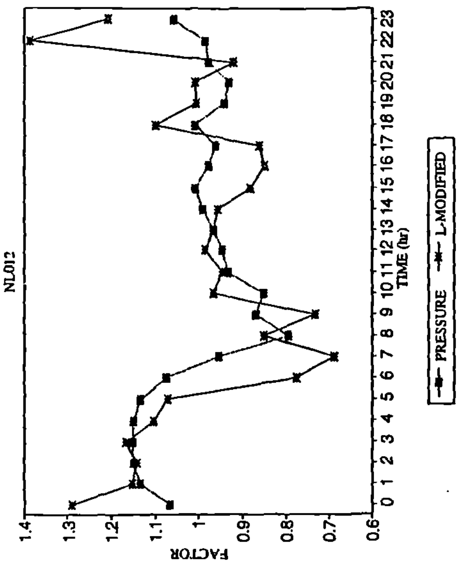
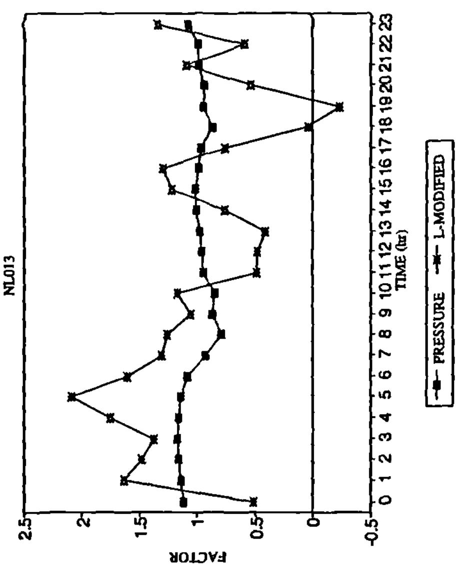

FACTOR
TIME (hr)
NH034 NT031

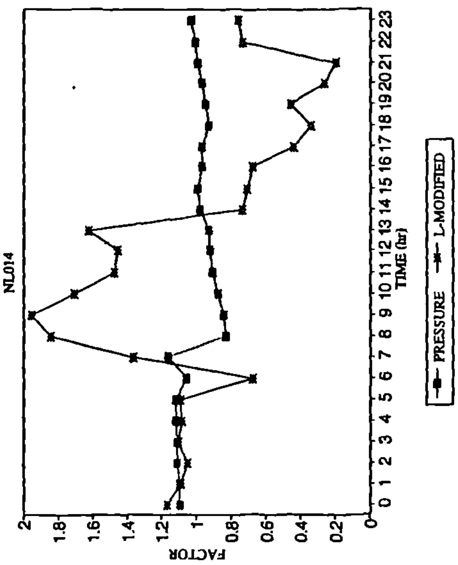
APPENDIX C2

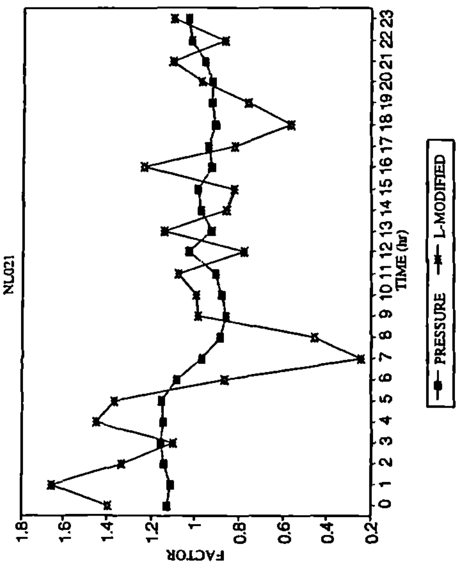
AVERAGE ZONAL PRESSURE AND MODIFIED LEAKAGE PROFILES (AZP AND L-MODIFIED)

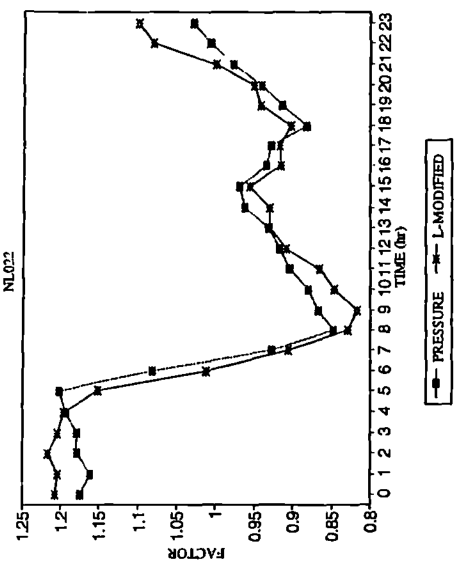

NL011
FACTOR
TIME (hr)
PRESSURE
L-MODIFIED

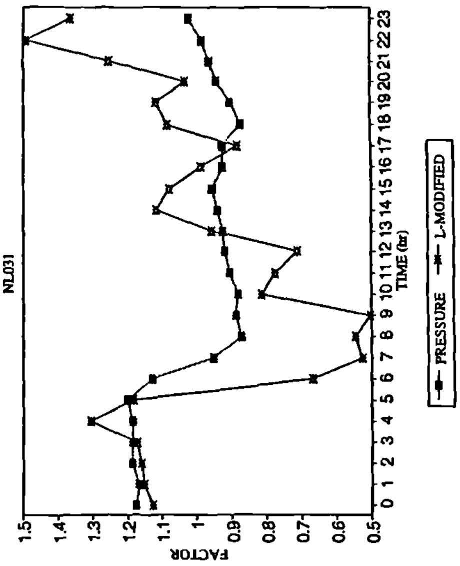

NL012
FACTOR
TIME (hr)
PRESSURE
L-MODIFIED

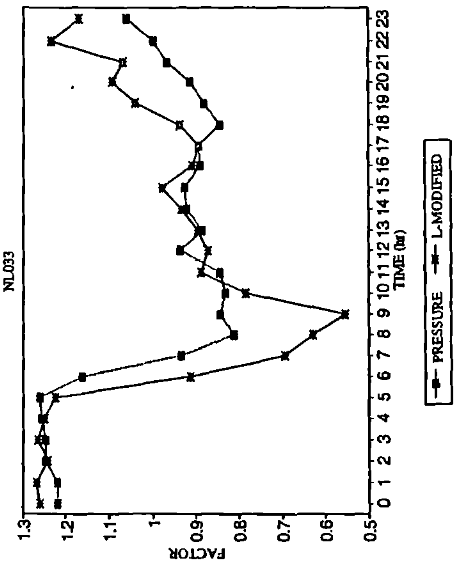

NL013
FACTOR
TIME (hr)
PRESSURE
L-MODIFIED


NL014
FACTOR
TIME (hr)
PRESSURE
L-MODIFIED


NL021
FACTOR
TIME (hr)
PRESSURE
L-MODIFIED


NL022
FACTOR
TIME (hr)
PRESSURE
L-MODIFIED


NL031
FACTOR
TIME (hr)
PRESSURE
L-MODIFIED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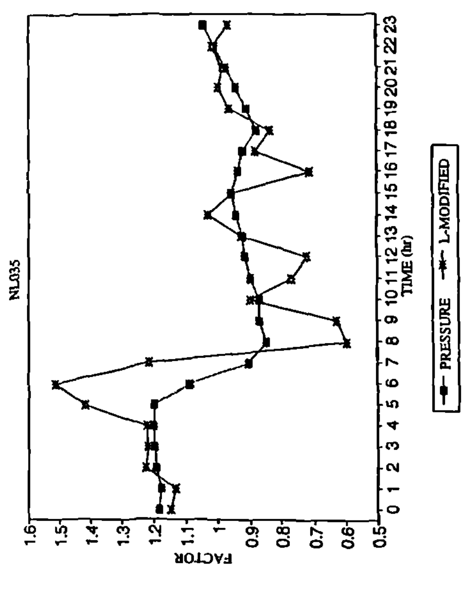
FACTOR
TIME (hr)
PRESSURE
L-MODIFIED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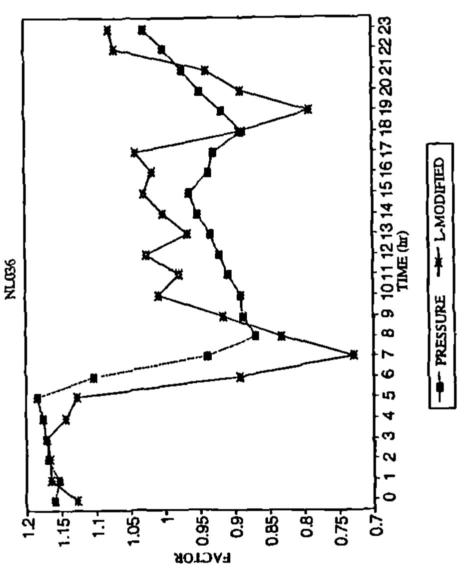
FACTOR
TIME (hr)
PRESSURE
L-MODIFIED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TIME (hr)
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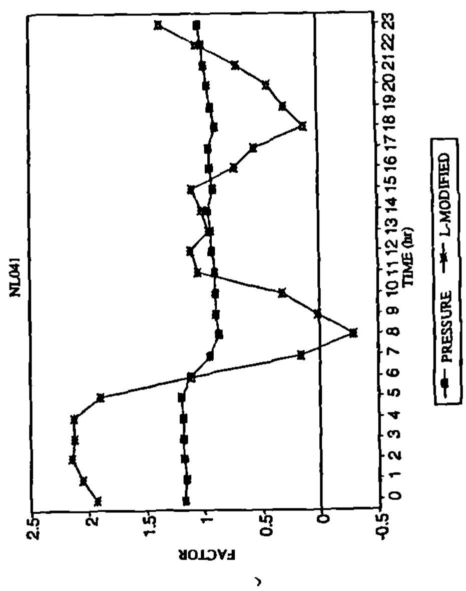
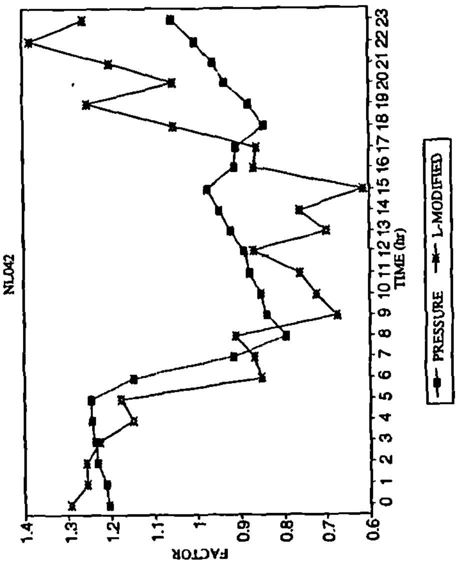
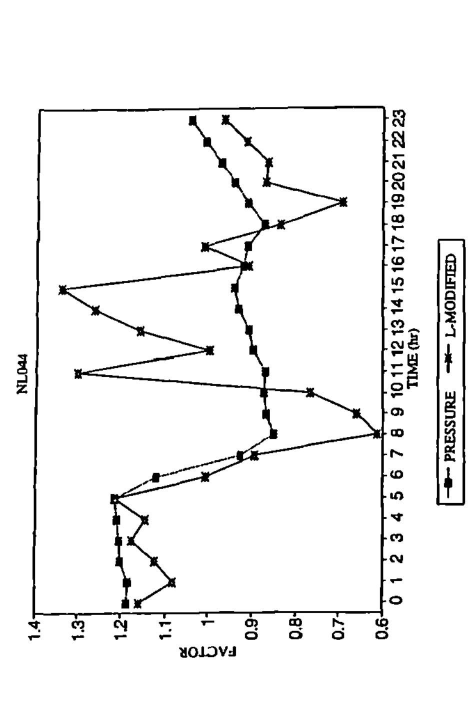
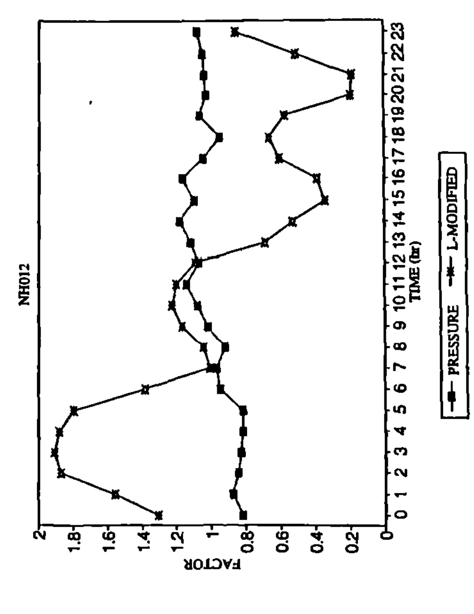
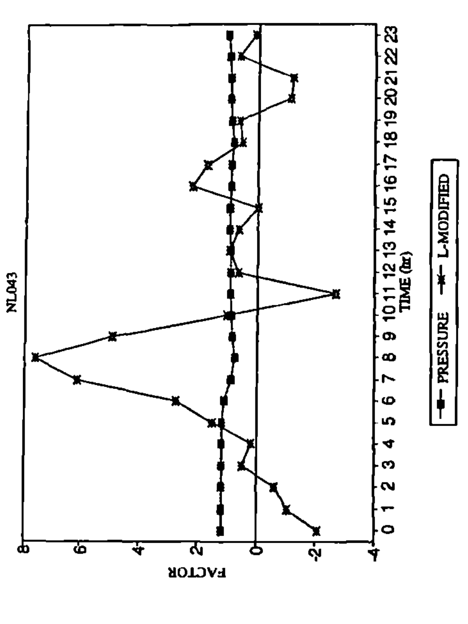
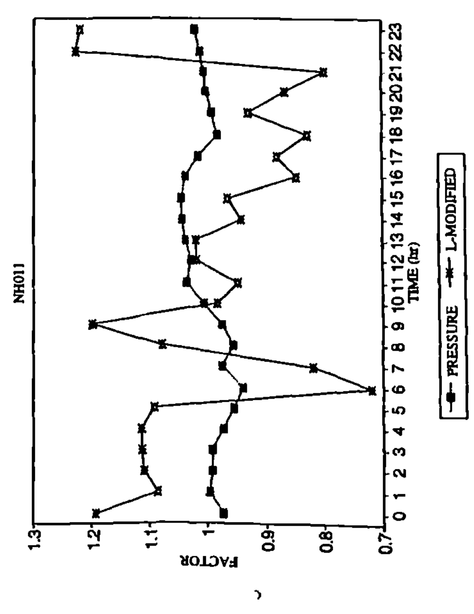

NL041
FACTOR
TIME (hr)
PRESSURE
L-MODIFIED


NL042
FACTOR
TIME (hr)
PRESSURE
L-MODIFIED


NL044
FACTOR
TIME (hr)
PRESSURE
L-MODIFIED


NH012
FACTOR
TIME (hr)
PRESSURE
L-MODIFIED


NL043
FACTOR
TIME (hr)
PRESSURE
L-MODIFIED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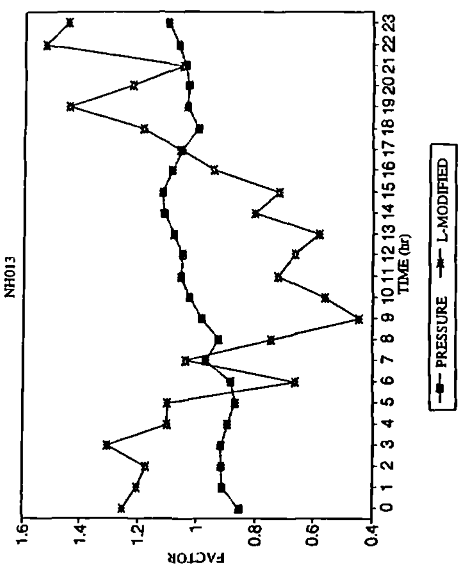
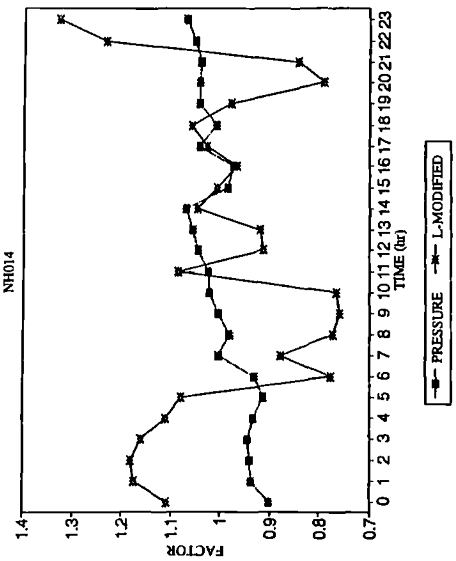
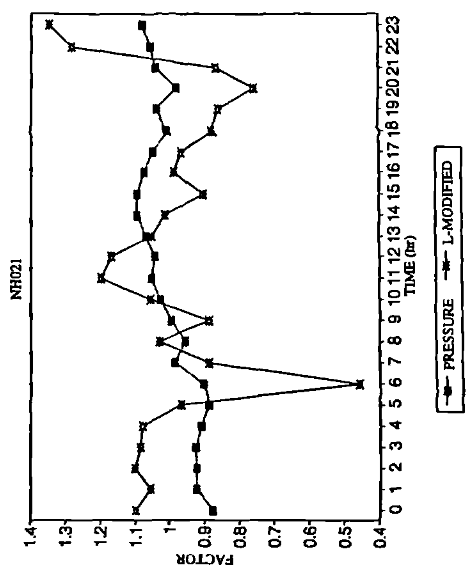
FACTOR
TIME (hr)
PRESSURE
L-MODIFIED


NH013
FACTOR
TIME (hr)
PRESSURE
L-MODIFIED


NH014
FACTOR
TIME (hr)
PRESSURE
L-MODIFIED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FACTOR
TIME (hr)
PRESSURE
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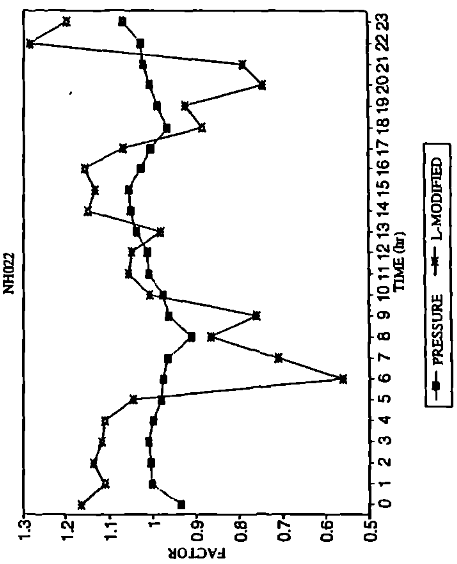
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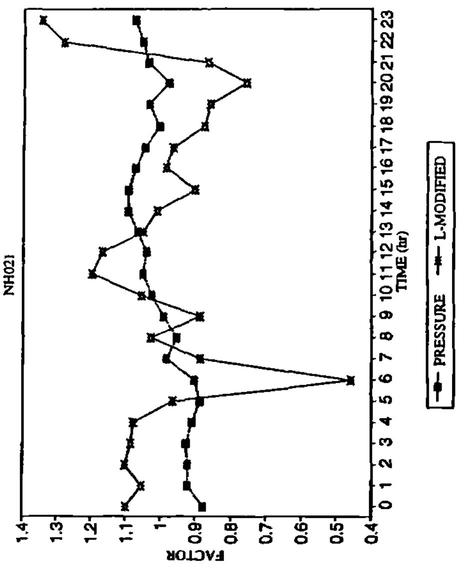
FACTOR
TIME (hr)
PRESSURE
L-MODIFIED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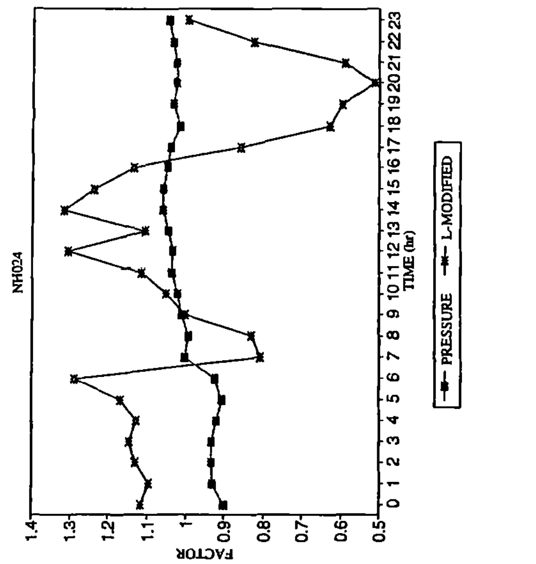
FACTOR
TIME (hr)
PRESSURE
L-MODIFIED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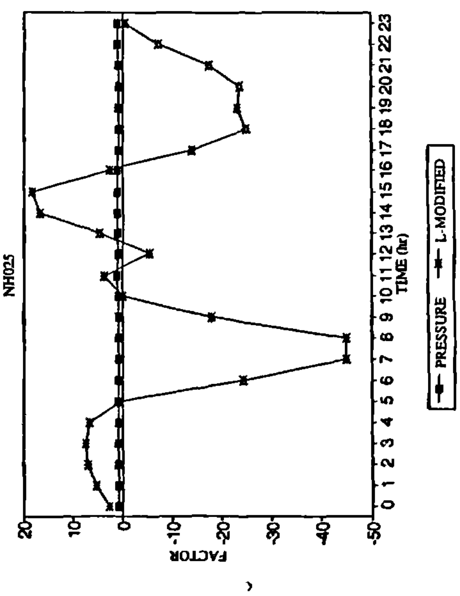
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FACTOR
TIME (hr)
PRESSURE
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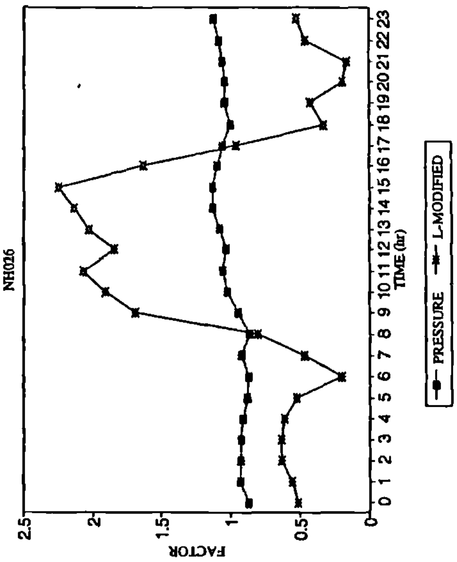
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FACTOR
TIME (hr)
PRESSURE
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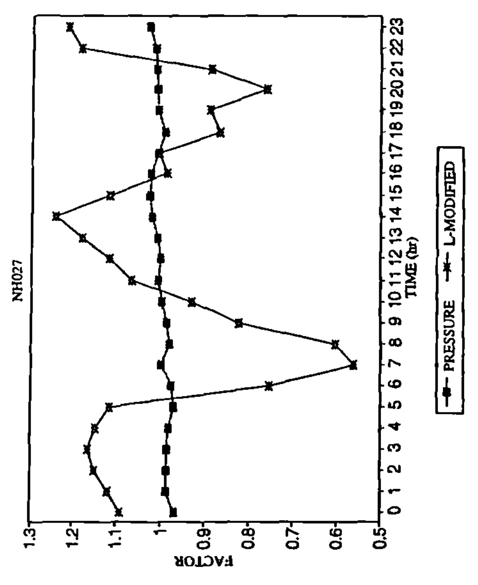
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FACTOR
TIME (hr)
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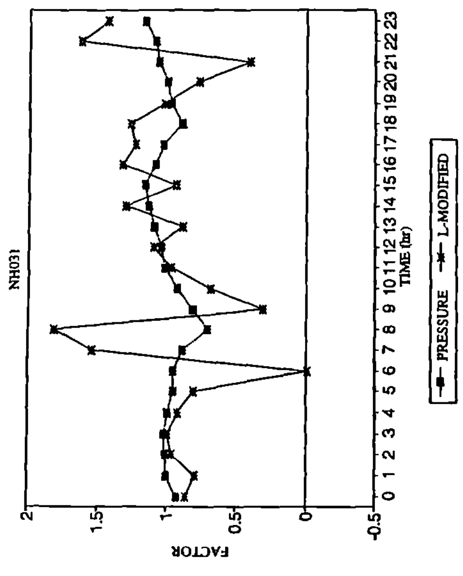
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FACTOR
TIME (hr)
PRESSURE
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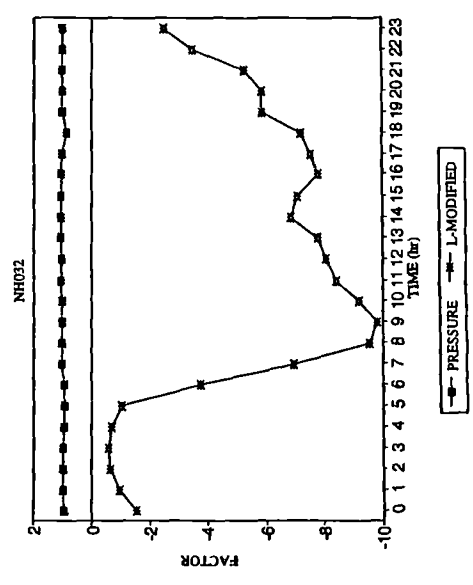
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TIME (hr)
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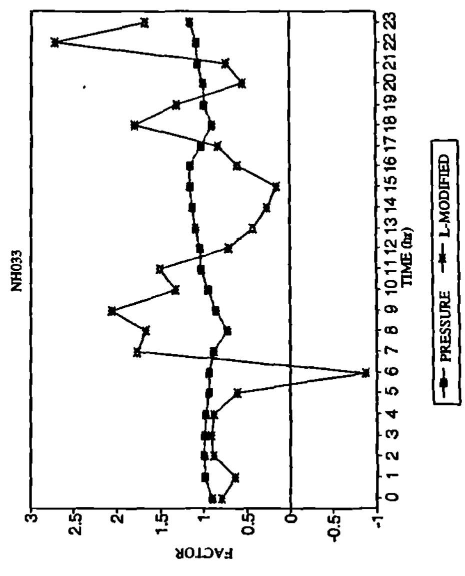
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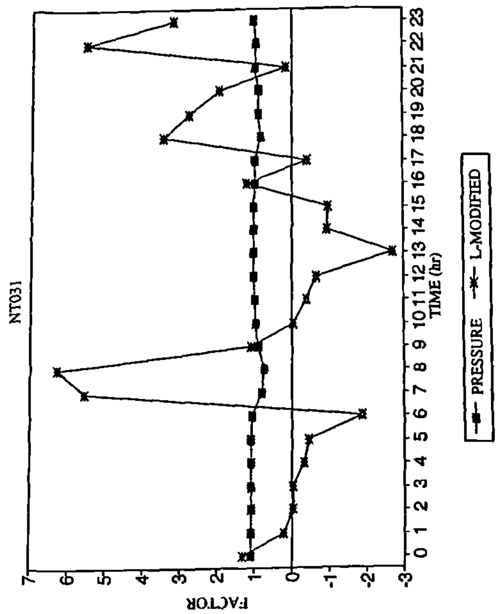
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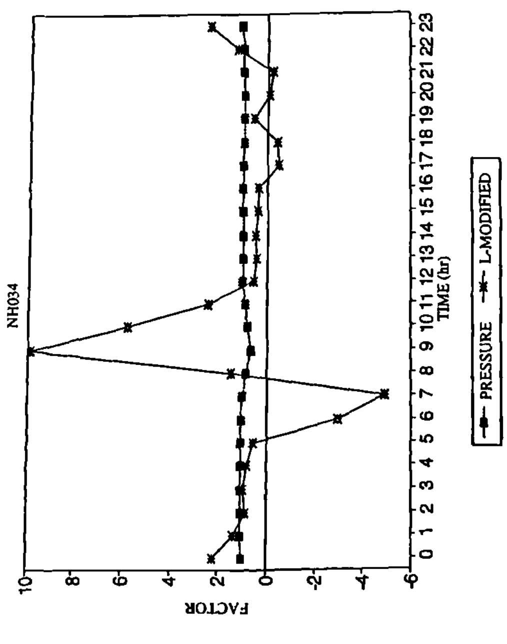
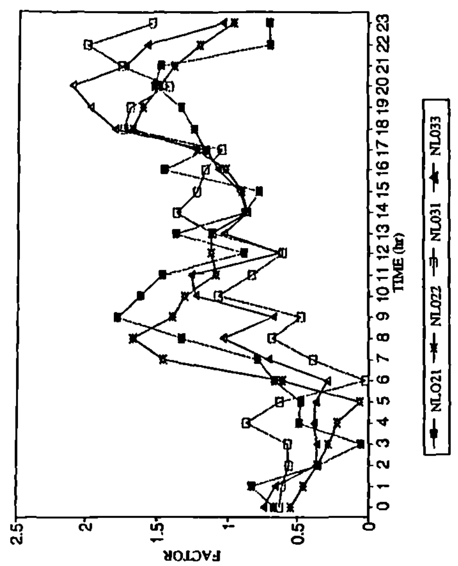
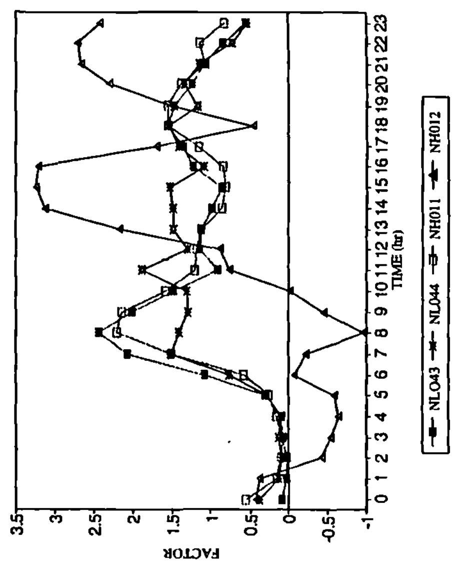
FACTOR
TIME (hr)
PRESSURE
L-MODIFIED

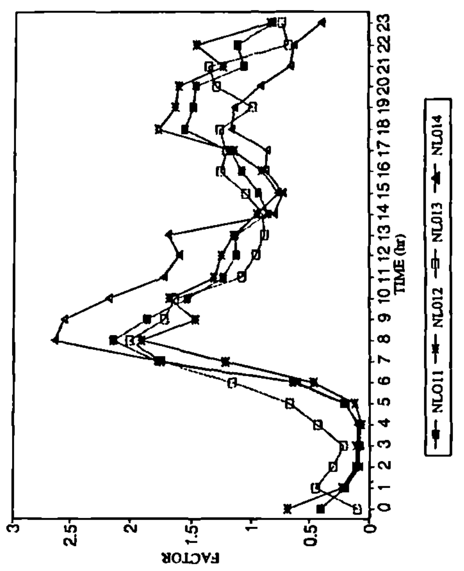

NH034
FACTOR
TIME (hr)
PRESSURE
L-MODIFIED

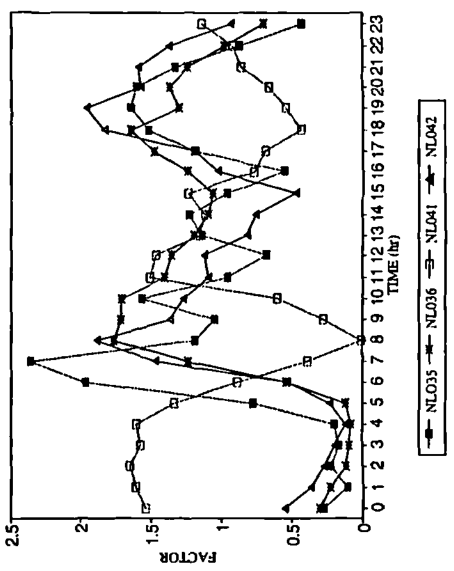
APPENDIX C3

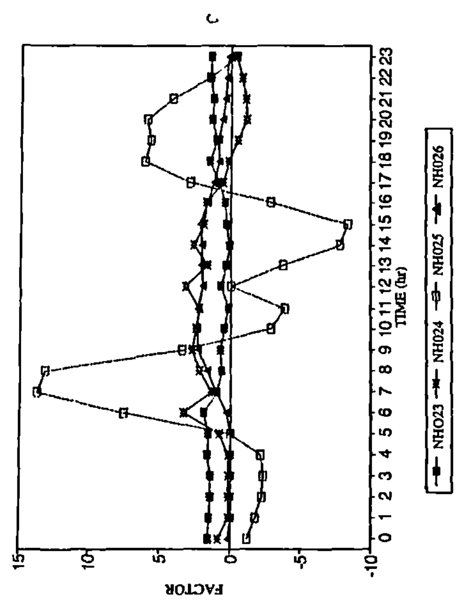
DOMESTIC DEMAND PROFILES BASED ON AVERAGE ZONAL PRESSURE DEPENDENT LEAKAGE (L-AZP)

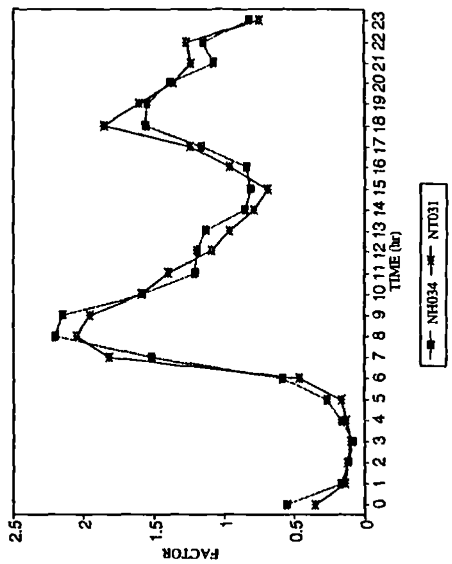

FACTOR
TIME (hr)
NL021
NL022
NL031
NL033

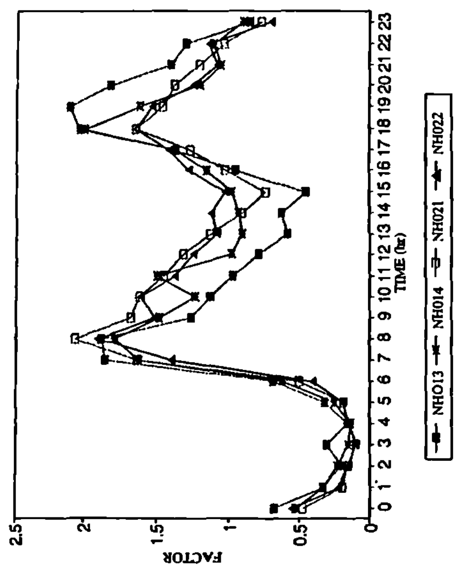

FACTOR
TIME (hr)
NL043
NL044
NH011
NH012

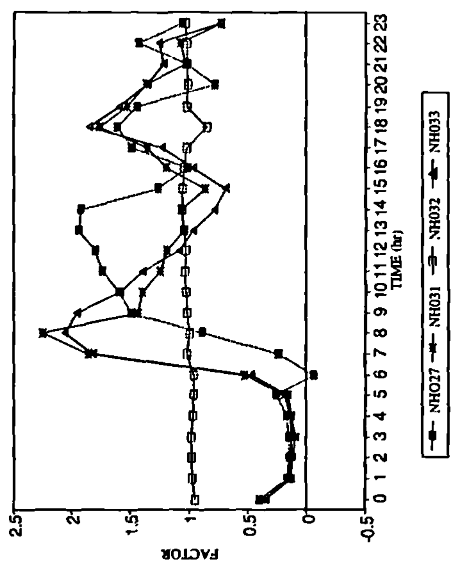

FACTOR
TIME (hr)
NL011
NL012
NL013
NL014


FACTOR
TIME (hr)
NL035
NL036
NL041
NL042


FACTOR
TIME (hr)
NHO23
NHO24
NHO25
NHO26

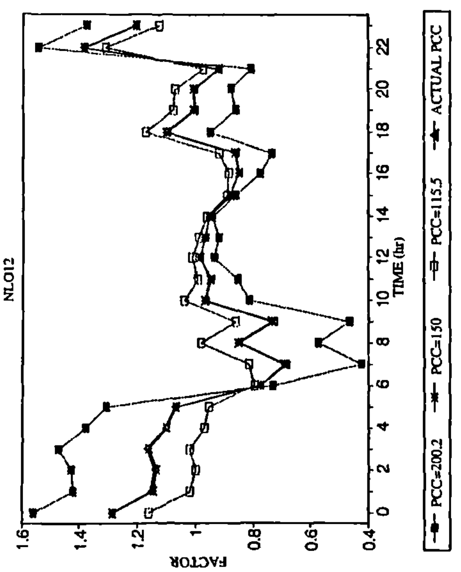

FACTOR
TIME (hr)
NH034
NT031

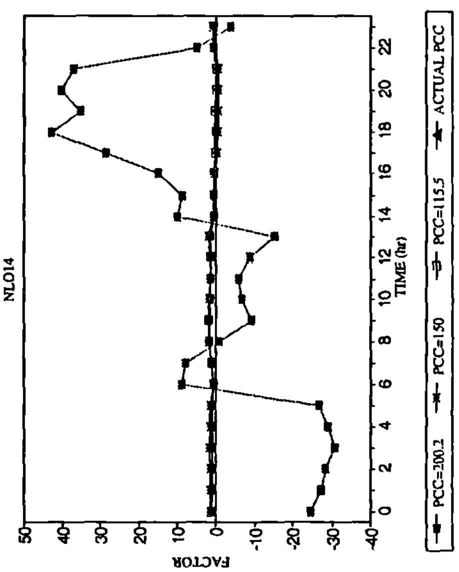

FACTOR
TIME (hr)
NHO13
NHO14
NHO21
NHO22

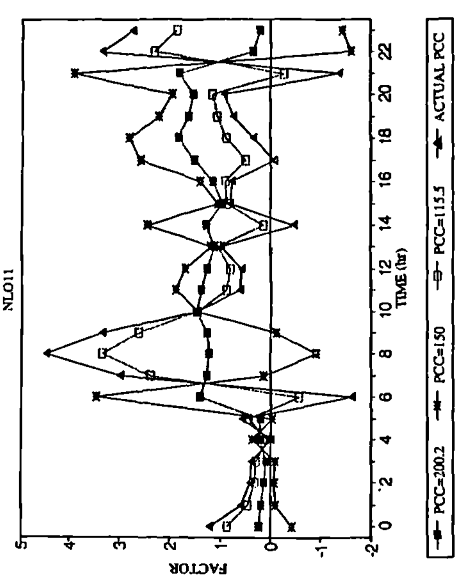

FACTOR
TIME (hr)
NHO27
NHO31
NHO32
NHO33

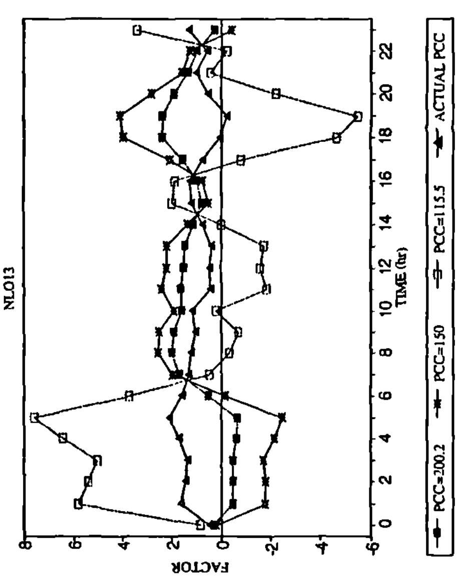
APPENDIX C4

MODIFIED LEAKAGE PROFILES BASED ON DIFFERENT PCC VALUES


NLO12
FACTOR
TIME (hr)
PCC=200.2
PCC=150
PCC=115.5
ACTUAL PCC


NLO14
FACTOR
TIME (hr)
PCC=200.2
PCC=150
PCC=115.5
ACTUAL PCC


NLO11
FACTOR
TIME (hr)
PCC=200.2
PCC=150
PCC=115.5
ACTUAL PCC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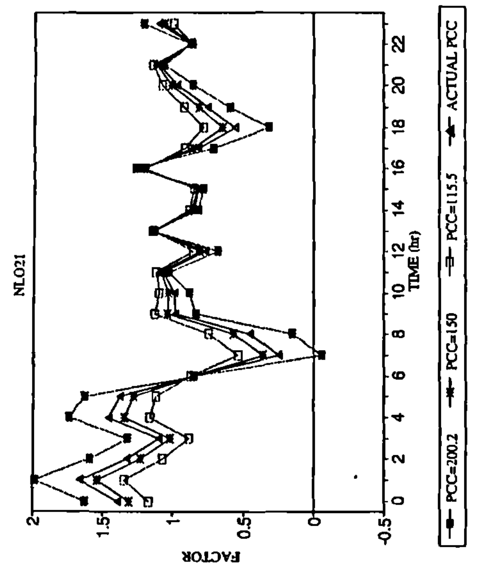
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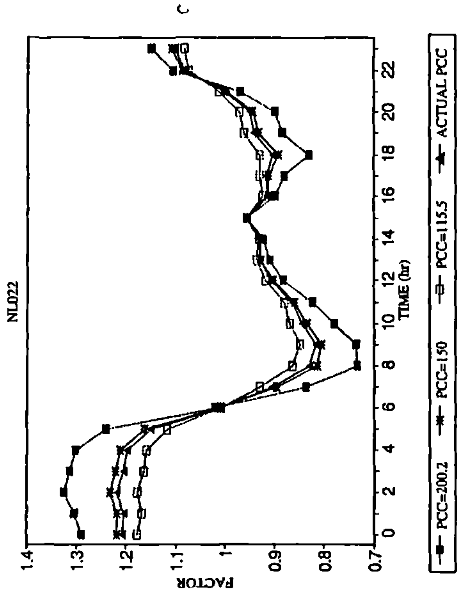
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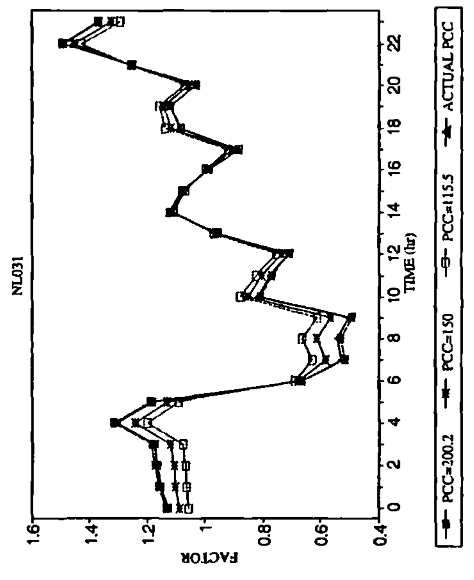
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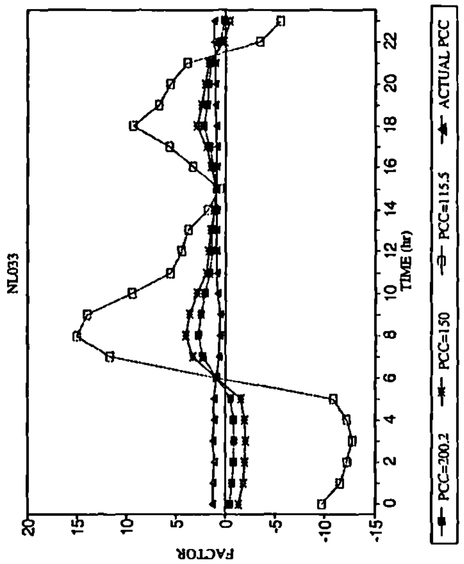
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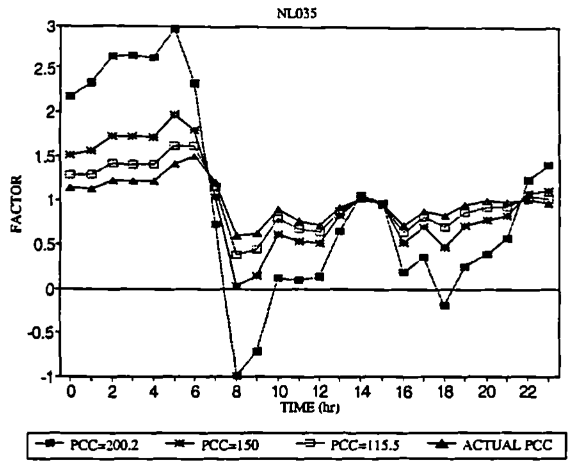
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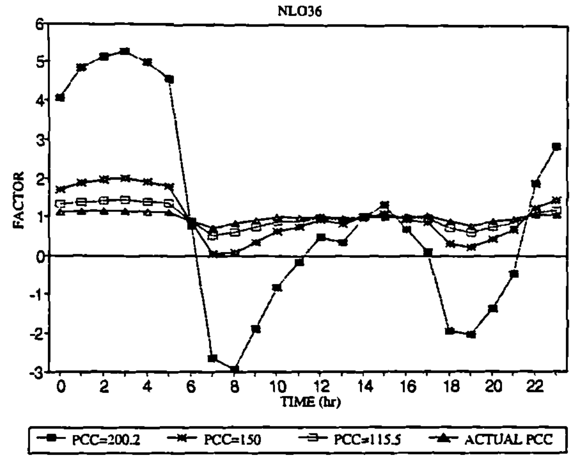
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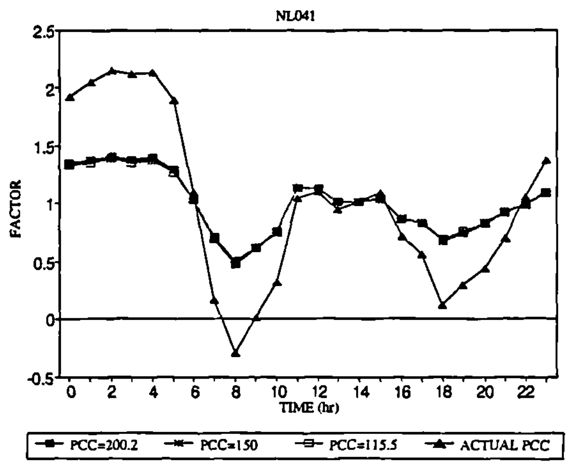
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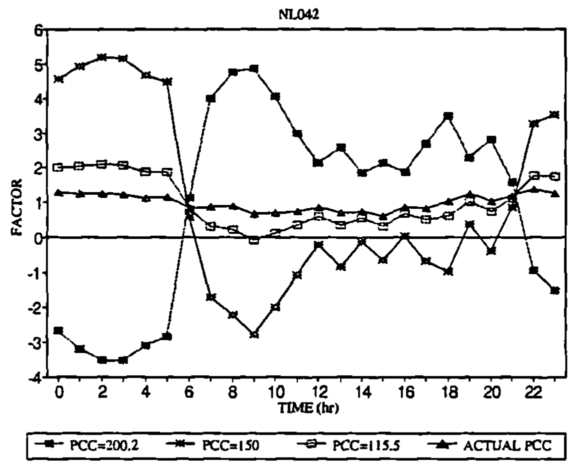
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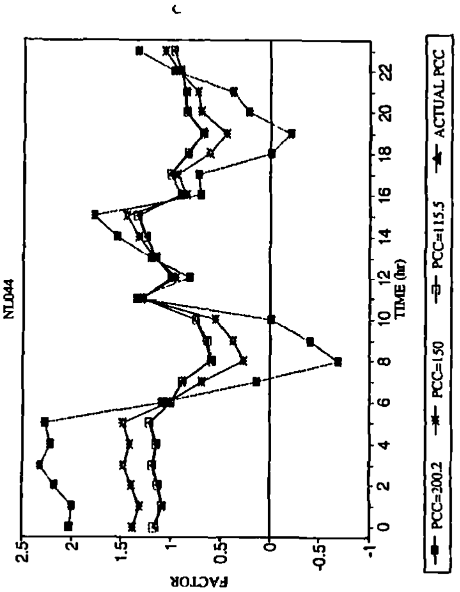
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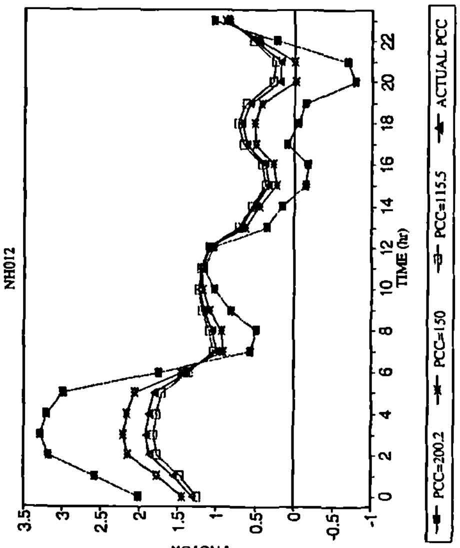
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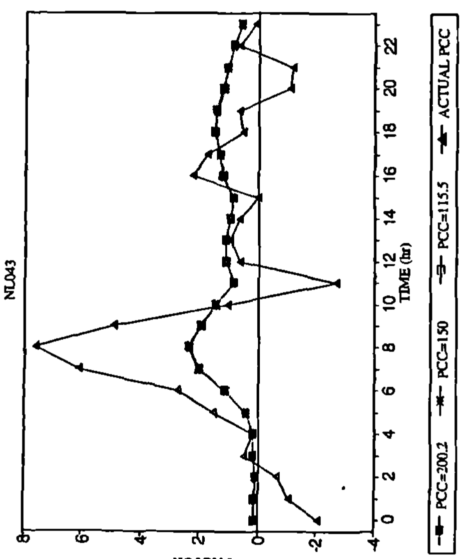
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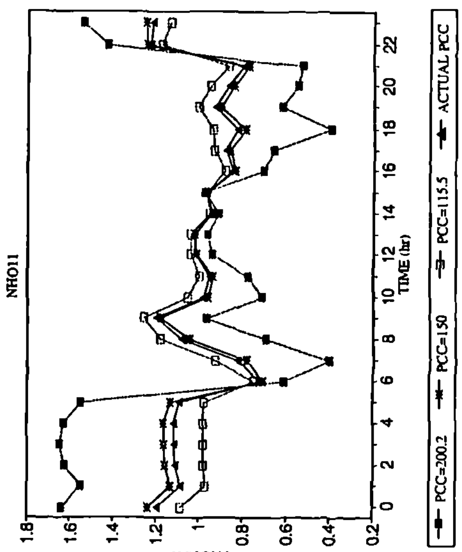
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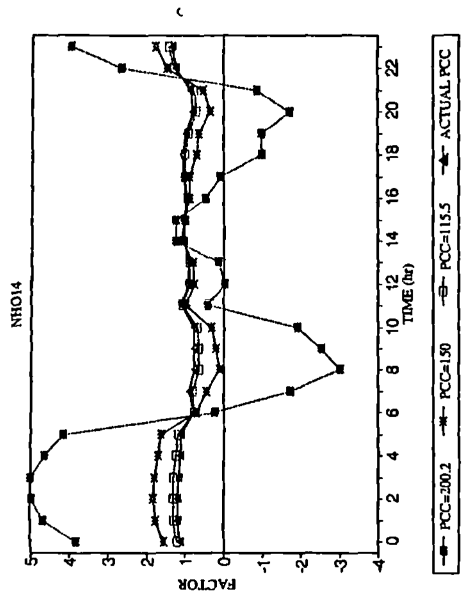
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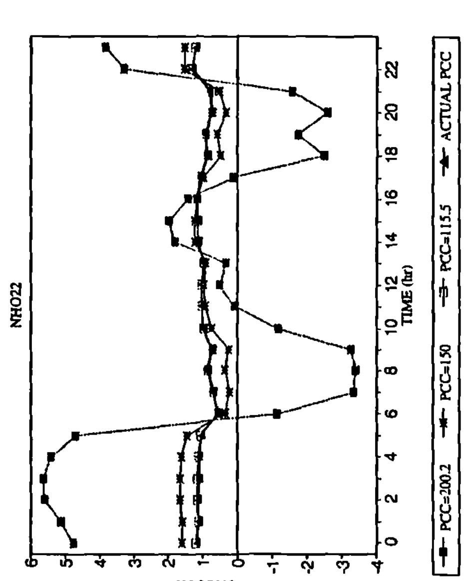
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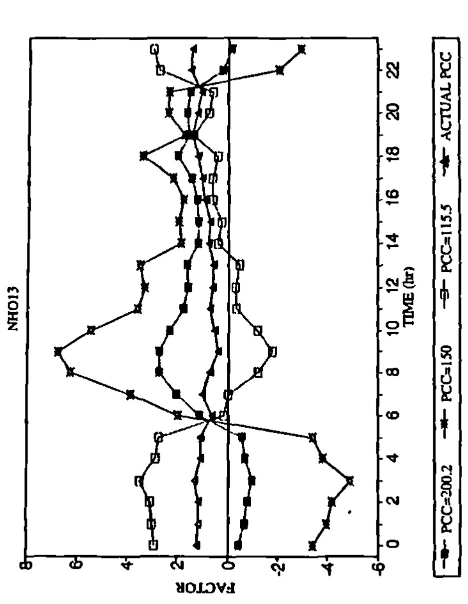
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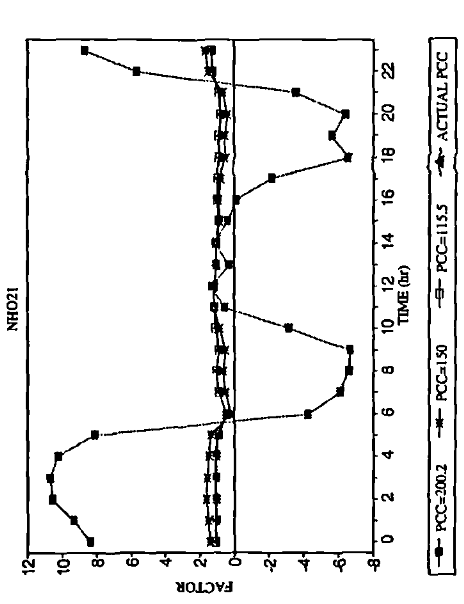
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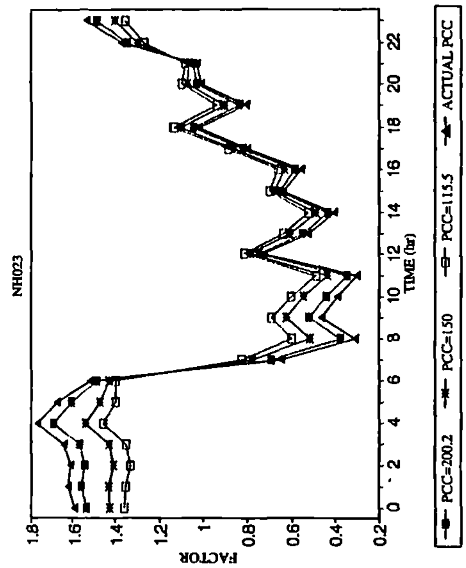
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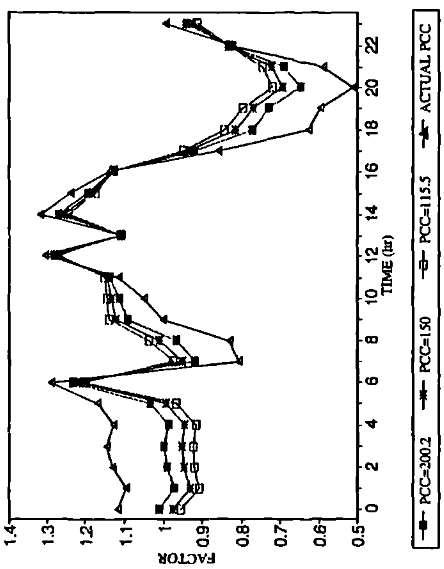
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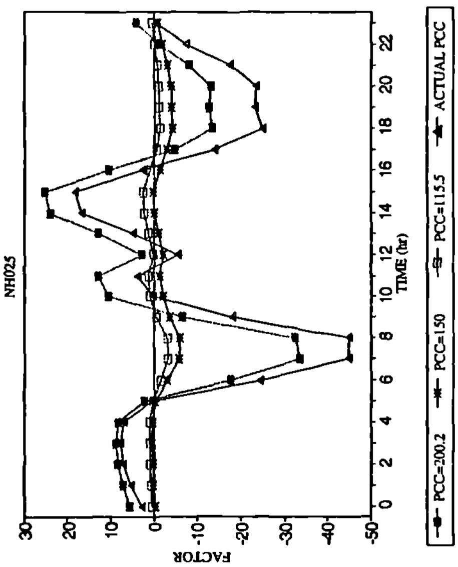
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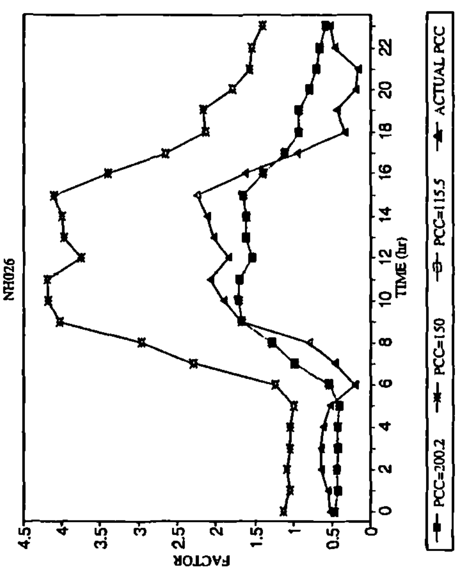
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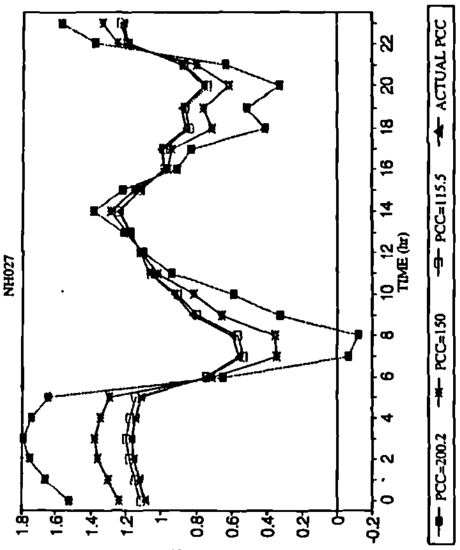
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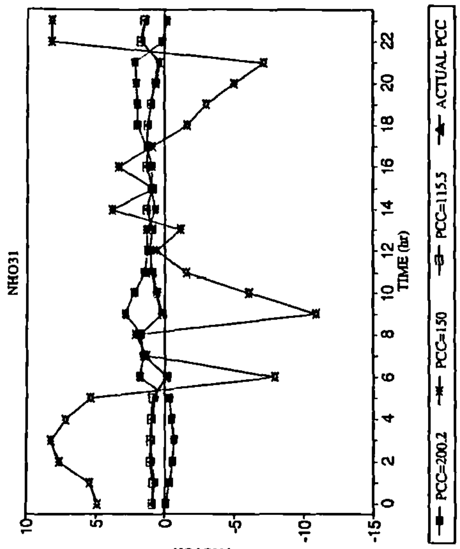
ACTUAL PCC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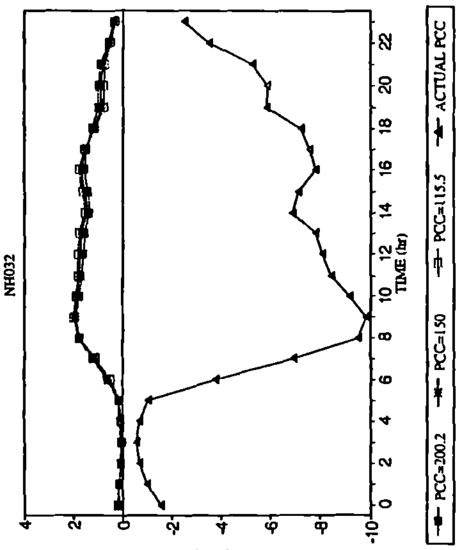
ACTUAL PCC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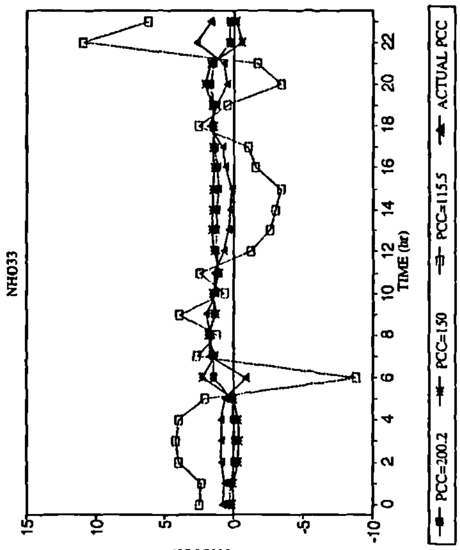
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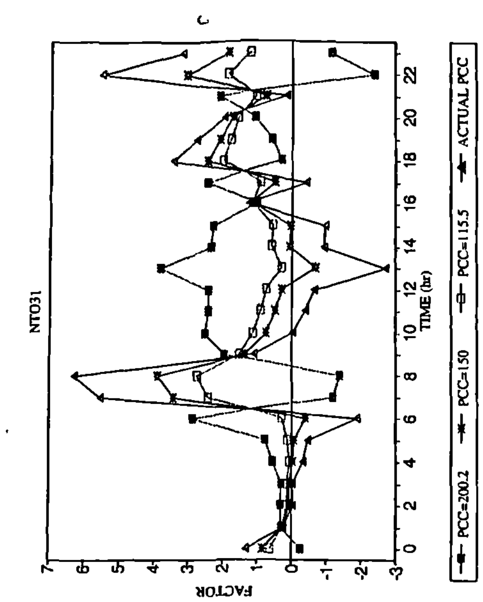
ACTUAL PCC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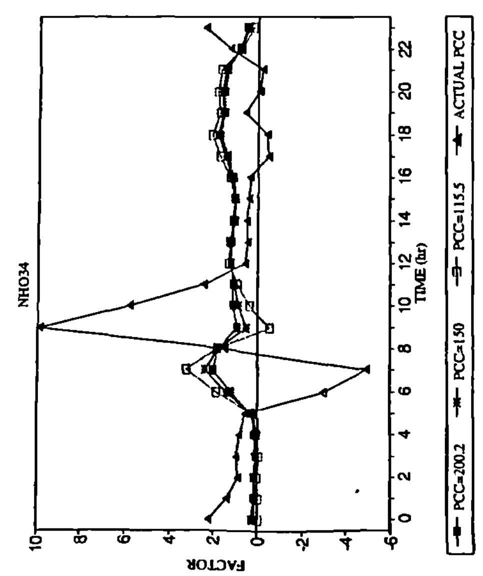
ACTUAL PCC


NHO34
FACTOR
TIME (hr)
PCC=200.2
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PCC=115.5
ACTUAL PCC

APPENDIX D

MORE HEAD DRIVEN SIMULATION EXAMPLES

D.1 INTRODUCTION

To evaluate the head driven simulation method (HDSM), several networks were analyzed. Accuracy and efficiency of the method were examined by a number of examples in Chapter 5. Herein the ability of the method to analyze some networks which were reported to have problem when solved by the traditional methods such as the Simultaneous path adjustment method, Linear theory method, etc. is demonstrated.

D.2 EXAMPLES

The first two examples are taken from Salgado et al. (1988). Example D.1, shown in Figure D.1, represents a single source network comprising 10 nodes, 13 pipes and four loops. The pipe data are shown in Table D.1 and the required heads, demands and the minimum heads can be seen in Table D.2. The desirable heads are assumed to be 15 m above the minimum heads. The example has been solved by both the DDSM and HDSM and the available heads and outflows are presented in Table D.2. It can be observed that the network is deficient and the available heads cannot satisfy the desired values over parts of the network, i.e. nodes 4, 7, 8 and 10. Further, the HDSM results show the ability of the method to calculate the real available nodal outflows which are different from the demands. Also, an improvement in nodal heads is seen in applications of the HDSM which show the tendency of the head shortfalls to be localised around the critical nodes.

The second example, Example D.2, is taken from the same reference. The layout of this network which is shown in Figure D.2 includes 2 reservoirs, 14 nodes, 16 pipes and 3 loops. Tables D.3 and D.4, respectively, show the input data and output results for this example. Again a desirable head of 15 m above the minimum head is assumed for all nodes. The analysis show that because of the two reservoirs, the nodal heads are well maintained and all the nodal outflows and heads are satisfactory. In this case the results of the DDSM and HDSM are identical because in fact the same algorithm is used to solve the governing equations of hydraulics of the system, i.e. the head driven related terms would not be operational.

Finally, a more complicated network with 2 reservoirs, one pump, one PRV and one non-return valve (NRV) is evaluated as Example D.3 which has been taken from Naeeni (1992). Figure D.3 shows the layout of this network which comprises 12 nodes and 18 links. Pipe data and nodal inflows can be seen in Tables D.5 and D.6, respectively. Here 30 m head is desired above the minimum value. The pump characteristics is represented by the following head-flow relationship, $H_p = -2000\ Q_p^2 - 150\ Q_p + 95$. Heads and flows are in m and m^3/s, respectively. The value of pressure set as output for the PRV is 358 m. The results of the DDSM and HDSM for this network is presented in Table D.6. It can be seen that existence of the two reservoirs and the pump lead to maintenance of heads in most nodes. However, node 11 cannot fulfil the total demand because of its high desired head and partial flow would be delivered at this node. Also, it is observed that the PRV is not operational because the pre set value for head at the outlet of the PRV is less than the available head at nodes 16 and 4. In fact the left hand side of the network is isolated from the rest and in this case operation of the pump leads to filling of the reservoir at node 2.

Table D.1: Pipe data for the network of Example D.1 (Fig. D.1).

Pipe	Diameter (mm)	CHW	Length (m)
1-2	450	60	270
1-5	375	60	150
2-6	225	60	150
5-6	375	60	270
2-3	375	60	90
3-4	225	60	120
4-8	375	60	150
3-7	225	60	150
8-7	450	60	120
7-10	225	60	180
6-10	225	60	210
5-9	375	60	180
9-10	225	60	360

Table D.2: Nodal data and results for the network of Example D.1 (Fig. D.1).

Node	Input Data			Output Results			
				DDSM		HDSM	
	H_j^{min} (m)	H_j^{des} (m)	Q_j^{req} (m^3/sec)	H_j (m)	Q_j^{avl} (m^3/sec)	H_j (m)	Q_j^{avl} (m^3/sec)
1 (Source)	-	100[a]	1.110	100.000	1.110	100.000	0.965
2	10	25	0.000	69.277	0.000	76.204	0.000
3	10	25	-0.185	47.641	-0.185	59.431	-0.185
4	10	25	0.000	-3.542	0.000	24.406	0.000
5	10	25	0.000	75.783	0.000	81.039	0.000
6	10	25	-0.074	65.359	-0.074	73.128	-0.074
7	10	25	0.000	-7.840	0.000	21.595	0.000
8	10	25	-0.370	-8.904	-0.370	20.738	-0.313
9	10	25	-0.111	66.551	-0.111	73.582	-0.111
10	10	25	-0.370	-11.538	-0.370	20.101	-0.304

[a] Available source head

Table D.3: Pipe data for the network of Example D.2 (Fig. D.2).

Pipe	Diameter (mm)	CHW	Length (m)
1-2	250	140	460
2-3	250	140	400
3-10	250	140	320
1-4	250	140	140
4-5	250	140	200
5-6	250	140	300
1-7	250	140	500
7-10	250	140	620
10-9	250	140	285
4-11	5	140	460
11-12	250	140	160
12-13	250	140	170
13-14	250	140	145
14-9	250	140	340
14-8	250	140	320
6-8	250	140	130

Table D.4: Nodal data and results for the network of Example D.2 (Fig. D.2).

Node	Input Data			Output Results	
	H_j^{min} (m)	H_j^{des} (m)	Q_j^{req} (m^3/sec)	H_j (m)	Q_j^{avl} (m^3/sec)
1 (Source)	-	100[a]	-	100.000	-0.090
2	10	25	-0.005	100.333	-0.005
3	10	25	-0.005	100.765	-0.005
4	10	25	-0.005	100.454	-0.005
5	10	25	-0.005	101.235	-0.005
6	10	25	-0.010	102.626	-0.010
7	10	25	-0.005	100.447	-0.005
8	10	25	-0.005	103.455	-0.005
9	10	25	-0.005	103.154	-0.005
10	10	25	-0.010	101.245	-0.010
11 (Source)	-	120[a]	-	120.000	0.159
12	10	25	-0.005	114.877	-0.005
13	10	25	-0.005	108.790	-0.005
14	10	25	-0.005	105.744	-0.005

[a] Available source head

Table D.5: Pipe data for the network of Fig. D.3.

Pipe	Diameter (mm)	CHW	Length (m)
1-3	406.4	130	1200
1-16	406.4	130	900
2-3	609.6	130	700
3-15	406.4	130	750
4-6	254.0	95	600
4-8	304.8	100	1800
4-16 PR ✓	406.4	130	3
5-6	254.0	95	600
5-7	304.8	100	1500
5-10	355.6	130	1800
5-15	pump	-	-
7-14	304.8	100	1200
8-9	304.8	100	600
9-10	304.8	100	600
10-11	457.2	130	2100
10-13	203.2	90	1200
11-12	457.2	130	2200
13-14 NR ✓	203.2	90	1100

Table D.6: Nodal data and results for the network of Example D.3 (Fig. D.3).

Node	Input Data			Output Results			
				DDSM		HDSM	
	H_j^{min} (m)	H_j^{des} (m)	Q_j^{req} (m^3/sec)	H_j (m)	Q_j^{avl} (m^3/sec)	H_j (m)	Q_j^{avl} (m^3/sec)
1	345	375	-0.025	410.050	-0.025	410.050	-0.025
2 (Source)	-	410[a]	-	410.000	-0.110	410.000	-0.110
3	365	395	-0.006	410.170	-0.006	410.170	-0.006
4	330	360	-0.012	371.658	-0.012	371.658	-0.012
5	335	365	-0.012	371.122	-0.012	371.129	-0.012
6	328	358	-0.020	371.109	-0.020	371.115	-0.020
7	340	370	-0.003	371.349	-0.003	371.352	-0.003
8	340	370	-0.030	373.271	-0.030	373.276	-0.030
9	335	365	-0.030	375.323	-0.030	375.338	-0.030
10	330	360	-0.012	379.778	-0.012	379.822	-0.012
11	380	410	-0.006	389.443	-0.006	389.553	-0.003
12 (Source)	-	400[a]	-	400.000	0.259	400.000	0.256
13	338	368	-0.006	374.145	-0.006	374.150	-0.006
14	338	368	-0.003	371.591	-0.003	371.593	-0.003
15	335	365	0.000	410.889	0.000	410.889	0.000
16	330	360	0.000	410.050	0.000	410.050	0.000

[a] Available source head

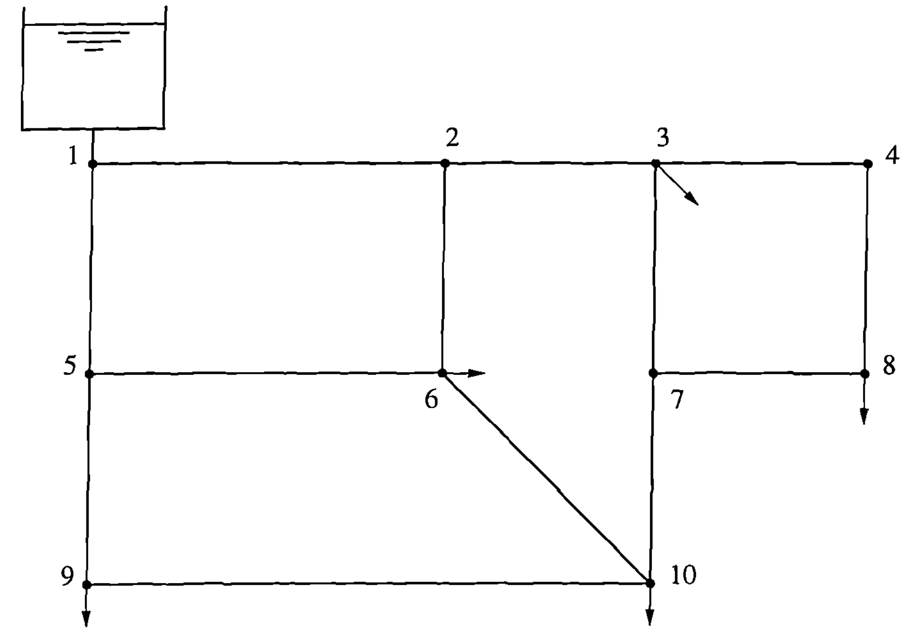

Figure D.1: Layout of Example D.1.

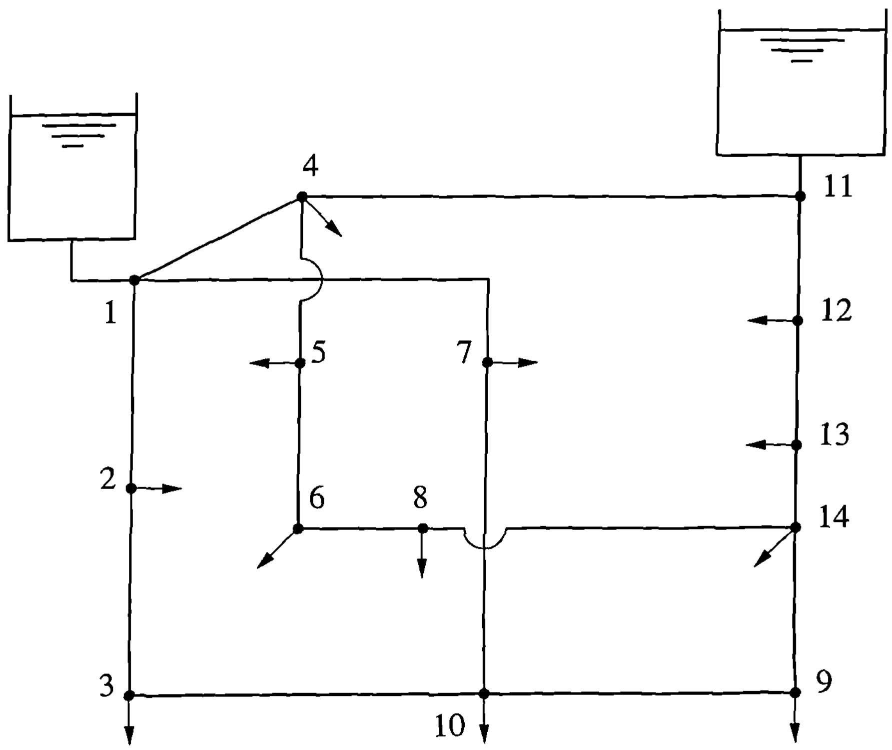

Figure D.2: Network of Example D.2.

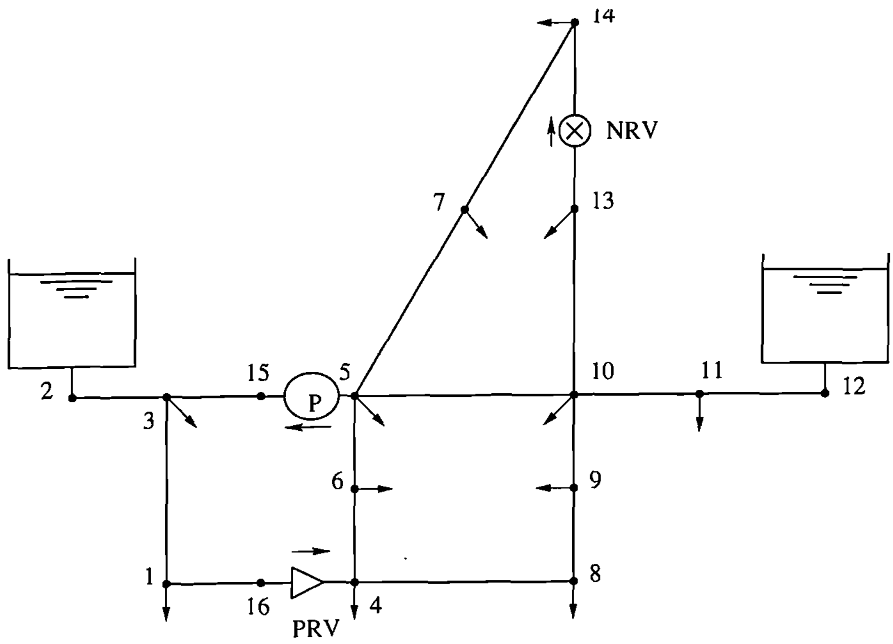

Figure D.3: Layout of Example D.3.

APPENDIX E

CLARIFICATION OF THE RELATIONSHIPS BETWEEN NODAL AND SYSTEM AND DAILY AND HOURLY RELIABILITIES

E.1 SYSTEM AND DEMAND-WEIGHTED MEAN OF NODAL RELIABILITIES RELATIONSHIP

As mentioned in Sections 8.2.3, there is a relationship between the system and demand-weighted mean of nodal reliabilities. Consider the nodal reliability with the possibility of one link failure as below,

$$R_{j,L} = \frac{1}{Q_j^{req}} \left(\sum_{l=0}^{NP} p(l)\ Q_j^{avl}(l) \right) \tag{E.1}$$

in which $R_{j,L}$ is the lower bound nodal reliability, $Q_j^{avl}(l)$ is the available outflow at node j with link l unavailable. Q_j^{req} is the required demand at node j with all links available, p(l) is the probability that link l is unavailable and NP is the number of links. Making a demand-weighted mean for the nodal reliabilities results the following equation,

$$\frac{\sum_{j=1}^{NJ} R_{j,L} \cdot Q_j^{req}}{Q_s^{req}} = \frac{\sum_{j=1}^{NJ} \frac{1}{Q_j^{req}} \left(\sum_{l=0}^{NP} p(l)\ Q_j^{avl}(l) \right) Q_j^{req}}{Q_s^{req}} \tag{E.2}$$

Where Q_s^{req} is the total system demand and NJ is the number of nodes. Consequently, the following formula is obtained which is in fact the system reliability formulation,

$$R_L = \frac{1}{Q_s^{req}} \left(\sum_{l=0}^{NP} p(l)\ Q_s^{avl}(l) \right) \tag{E.3}$$

in which R_L is the lower bound system reliability and $Q_s^{avl}(l)$ is the available system outflow when link l is unavailable. Therefore, it is seen that the system and demand-weighted nodal reliabilities are equivalent.

E.2 THE OVERALL DAILY AND DEMAND-WEIGHTED MEAN OF HOURLY RELIABILITIES RELATIONSHIP

It was recognised in Section 8.4.4 that there is a relationship between the overall daily system or nodal and the demand-weighted mean of hourly system and nodal reliabilities. This subsection investigates this relationship. The hourly system reliability with possibility of one link failure can be expressed as follows,

$$R_{t,L} = \sum_{k=1}^{NLB} \frac{1}{Q_s^{req}(k,t,0)} \left(\sum_{l=0}^{NP} p(l)\ Q_s^{avl}(k,t,l) \right) PLF(k) \tag{E.4}$$

where $R_{t,L}$ is the lower bound system reliability at time t, Q_s^{avl} (k,t,l) is the total available system outflow at band k (when demand is probabilistic) and time t when link l is unavailable. Q_s^{req}(k,t,0) is the required system demand at time t and band k when all links are available. PLF(k) is the probability of load factor at band k and NLB is the number of load bands (when demand is probabilistic). Making demand-weighted mean of the hourly system reliabilities can be represented as follows,

$$\frac{\sum_{t=1}^{NT} R_{t,L}\ Q_s^{ave}(t)}{Q_s^{ave}\ NT} = \frac{\sum_{t=1}^{NT} \sum_{k=1}^{NLB} \frac{1}{Q_s^{req}(k,t,0)} \left(\sum_{l=0}^{NP} p(l)\ Q_s^{avl}(k,t,l) \right) PLF(k)\ Q_s^{ave}\ DF(t)}{Q_s^{ave}\ NT} \tag{E.5}$$

in which $Q_s^{ave}(t)$ and DF(t) are the average system demand and demand factor at time t, respectively. Q_s^{ave} is the total daily average system demand and NT is the number of time intervals. Finally the following equation results,

$$R_L = \sum_{t=1}^{NT} \sum_{k=1}^{NLB} \frac{1}{Q_s^{req}(k,t,0)} \left(\sum_{l=0}^{NP} p(l)\ Q_s^{avl}(k,t,l) \right) PLF(k)\ PDF(t) \tag{E.6}$$

where R_L is the lower bound overall daily system reliability and PDF(t) is the demand factor probability at time t. This equation is in fact equivalent to the overall daily system reliability formulation. For the nodal reliabilities the same procedure can be followed by replacing the subscript s with j.

APPENDIX F

COMPUTER PROGRAMS AND DATA FILES

APPENDIX F1

E04NCF PROGRAMME

F1.1 LIST OF PROGRAMME

```
*     E04NCF Program Text
*     Mark 15 Revised.  NAG Copyright 1991.
*     .. Parameters ..
      INTEGER          NIN, NOUT
      PARAMETER        (NIN=5,NOUT=6)
*     .. External Subroutines ..
      EXTERNAL         EX1
*     .. Executable Statements ..
      open(unit=5,file='ncfe.dat')
      open(unit=6,file='ncfe.out')
      WRITE (6,*) 'E04NCF Example Program Results'
*     Skip heading in data file
c      READ (NIN,*)
c      READ (NIN,*)
      CALL EX1
      END
*
      SUBROUTINE EX1
*
*     Example 1.  A linear least-squares problem.
*
*     .. Parameters ..
      implicit double precision (a-h,o-z)
C M = NO. OF ROWS IN THE MATRIX A
C N = NO. VARIABLES
C NCLIN = NO. OF GENERAL LINEAR CONSTRAINTS
      INTEGER          M, N, NCLIN
      PARAMETER        (M=24,N=44,NCLIN=13)
      INTEGER          NROWC, NROWA
      PARAMETER        (NROWC=NCLIN,NROWA=M)
      INTEGER          NBND
      PARAMETER        (NBND=N+NCLIN)
      INTEGER          LIWORK
      PARAMETER        (LIWORK=N)
      INTEGER          LWORK
      PARAMETER        (LWORK=2*N*N+9*N+6*NCLIN)
      INTEGER          NOUT, IOPTNS
      PARAMETER        (NOUT=6,IOPTNS=5)
*     .. Local Scalars ..
      DOUBLE PRECISION BIGBND, OBJ
      INTEGER          I, IFAIL, INFORM, ITER, J
      CHARACTER*10     CBGBND
      character*400    cptn
*     .. Local Arrays ..
      DOUBLE PRECISION A(NROWA,N), B(M), BL(NBND), BU(NBND), C(NROWC,N),
     +                 CLAMDA(NBND), CVEC(N), WORK(LWORK), X(N)
      INTEGER          ISTATE(NBND), IWORK(LIWORK), KX(N)
*     .. External Subroutines ..
      EXTERNAL         E04NCF, E04NDF, E04NEF, X04ABF
*     .. Executable Statements ..

      open(unit=1,file='ncf.dat')
      read(1,40)cptn
 40   format(a400)
      write(6,*)cptn
*     Set the unit number for advisory messages to NOUT.
      CALL X04ABF(1,NOUT)
```

```
*
      BIGBND = 1.0D15
      CBGBND = '1.0E+15'
*
*     Form the data for the problem.
*     A     = the observation matrix.
*     B     = the vector of observations.
*     C     = the general constraint matrix.
*     BL    = the lower bounds on  x  and  C*x.
*     BU    = the upper bounds on  x  and  C*x.
*
      read(1,*)(b(i),i=1,m)
      WRITE(6,*)(b(i),i=1,m)
      WRITE(6,*)'BBBBBBBBBBBBBBBBBBBBB.............................'
      do 100 i=1,m
      read(1,*)(a(i,j),j=1,n)
      WRITE(6,*)(a(i,j),j=1,n)
      WRITE(6,*)'AAAAAAAAAAAAAAAAAAAA................................'
 100  continue
      do 111 i=1,NCLIN
      read(1,*)(c(i,j),j=1,n)
      WRITE(6,*)(c(i,j),j=1,n)
      WRITE(6,*)I,'CCCCCCCCCCCCCCCCCCCCCCCCCC..................'
 111  continue
      read(1,*)(bl(k),k=1,n+NCLIN)
      WRITE(6,*)(bl(k),k=1,n+NCLIN)
      WRITE(6,*)'BLBLBLBLBLBLBLBLBLB.....................'
      read(1,*)(bu(k),k=1,n+NCLIN)
      WRITE(6,*)(bu(k),k=1,n+NCLIN)
      WRITE(6,*)'BUBUBUBUBUBUBUB.................'
      read(1,*)(x(k),k=1,n)
      WRITE(6,*)(x(k),k=1,n)
      WRITE(6,*)'XXXXXXXXXXX...........'
*
*     Re-set the defaults (only necessary if this is not the first call
*     to E04NCF) and set an option using E04NEF.
*
      CALL E04NEF('   Defaults')
      CALL E04NEF('   Infinite Bound Size = '//CBGBND)
*
*     Read the options file for the remaining options.
*
      CALL E04NDF(IOPTNS,INFORM)
*
      IF (INFORM.NE.0) THEN
         WRITE (NOUT,*)
         WRITE (NOUT,99999) ' E04NDF terminated with  INFORM =', INFORM
         GO TO 140
      END IF
*
*     Solve the problem.
      IFAIL = -1
*
      CALL E04NCF(M,N,NCLIN,NROWC,NROWA,C,BL,BU,CVEC,ISTATE,KX,X,A,B,
     +            ITER,OBJ,CLAMDA,IWORK,LIWORK,WORK,LWORK,IFAIL)
*
      write(6,*)' '
      do 150 i=1,n
      write(6,*)'x(',i,')=',x(i)
  150 continue
  140 CONTINUE
*
99999 FORMAT (1X,A,I3)
      END
```

F1.2 INPUT FILES

INPUT FILES FOR GROUP1 AT STAGE 3

F1.2.1 NCFE.DAT

```
Begin   Example options file for  E04NCF

   Problem type        =   Least squares
   Print level        =   1
   Feasibility phase  =   900
   Optimality  phase  =   900
   Rank tolerance     =   1.0E-4

End
```

F1.2.2 NCF.DAT

```
INPUT FILES FOR GROUP1 AT STAGE 3 INCLUDING
NL11 T3=T4=0,NL12 T4=0,NL13 T3=T4=0,NL14 T2=T3=T4=0,NL15 T3=T4=0,NT31 T3=T4=0

0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0
-1.5028  1.3938  0.0202  0.3455 0 50.58
-5.8653  1.0702  0.0471  0.0000  4.1031 0 -.66 43.71 -18.84 41.5
-0.7319  0.7895  0.0328  0.5866 0 .66 44.37
-2.1986  0.9335  1.5398  0 -43.71 -44.37
-2.4986  1.0179  0.0141  1.6795 0 60.34 18.84
-2.2917  1.6430  0.0093  0.4484  0 -50.58 -41.5 -60.34

-0.9167  1.0501  0.0140  0.3455 0 50.20
-4.9597  0.8063  0.0324  0.0000  4.3656 0 -.4 45.23 -17.3 42.66
-1.1097  0.5948  0.0226  0.5990 0 .4 45.63
-1.8597  0.7033  1.5409  0 -45.23 -45.63
-2.0231  0.7669  0.0097  1.6797 0 59.96 17.3
-1.2361  1.2379  0.0064  0.4538 0 -50.2 -42.66 -59.96

-0.6612  0.8176  0.0119  0.3455 0 50.12
-4.7278  0.6278  0.0276  0.0000  4.4260 0 -.24 45.99 -16.59 43.61
-0.9528  0.4631  0.0192  0.6076 0 .24 46.23
-1.6903  0.5476  1.5573  0 -45.99 -46.23
-1.9038  0.5971  0.0083  1.6876 0 60.2 16.59
-1.2361  1.2379  0.0064  0.4538 0 -50.12 -43.61 -60.2

-0.5711  0.7396  0.0105  0.3455 0 50.10
-4.7458  0.5680  0.0243  0.0000  4.4353 0 -.39 45.96 -16.66 43.59
-0.8653  0.4190  0.0170  0.6090 0 .39 46.36
-1.7056  0.4954  1.5584  0 -45.96 -46.36
-1.7851  0.5402  0.0073  1.6892 0 60.25 16.66
-0.8194  0.9638  0.0054  0.4551 0 -50.1 -43.59 -60.25

-0.5560  0.8492  0.0119  0.3454 0 50.11
-4.6069  0.6521  0.0276  0.0000  4.4267 0 -.33 45.72 -16.99 43.47
-1.0875  0.4811  0.0192  0.6072 0 .33 46.05
-1.7361  0.5688  1.5652  0 -45.72 -46.05
-1.6977  0.6202  0.0083  1.6940 0 60.46 16.99
-0.6944  0.8719  0.0048  0.4554 0 -50.11 -43.47 -60.46

-0.9618  0.9941  0.0237  0.3455 0 50.16
-4.7305  0.7633  0.0552  0.0000  4.3669 0 -.31 44.79 -17.95 42.63
-1.3667  0.5631  0.0385  0.5983 0 .31 45.1
-1.8861  0.6658  1.5669  0 -44.79 -45.1
-1.8532  0.7260  0.0165  1.6954 0 60.57 17.95
-0.7361  1.0011  0.0054  0.4553 0 -50.16 -42.63 -60.57

-2.2091  1.9073  0.0761  0.3454 0 51.21
```

-5.4625 1.4645 0.1769 0.0000 4.1334 0 -.31 42.69 -19.6 40.26
-1.9403 1.0804 0.1234 0.5651 0 .31 43
-2.4542 1.2774 1.4909 0 -42.69 -43
-2.9175 1.3929 0.0530 1.6527 0 59.86 19.6
-1.0556 1.1719 0.0109 0.4543 0 -51.21 -40.26 -59.86

-5.4401 3.7356 0.1417 0.3448 0 58.40
-7.1958 2.8684 0.3294 0.1903 3.6743 0 -.49 38.15 -18.89 39.3
-2.6236 2.1161 0.2299 0.4872 0 .49 38.64
-4.4542 2.5019 1.6321 0 -38.15 -38.64
-5.7898 2.7281 0.0988 1.3702 0 58.2 18.89
-3.4028 2.2484 0.0348 0.4390 0 -58.4 -39.3 -58.2

-6.4619 4.3983 0.1584 0.3445 0 60.73
-8.8125 3.3773 0.3683 0.6285 3.0581 0 -.79 32.2 -20.47 33.69
-2.8542 2.4915 0.2570 0.4130 0 .79 32.98
-5.4264 2.9458 1.1653 0 -32.2 -32.98
-7.3915 3.2120 0.1105 1.2859 0 54.17 20.47
-9.1250 4.4037 0.0648 0.3340 0 -60.73 -33.69 -54.17

-5.6805 3.9607 0.1305 0.3447 0 55.90
-8.1555 3.0413 0.3034 0.8727 3.3464 0 -.67 36.19 -19.01 33.15
-2.5389 2.2436 0.2117 0.4537 0 .67 36.86
-5.3056 2.6527 1.1851 0 -36.19 -36.86
-6.5771 2.8925 0.0910 1.3543 0 52.67 19.01
-10.3333 5.1849 0.0725 0.2996 0 -55.9 -33.15 -52.16

-4.6886 3.7501 0.0970 0.3450 0 54.35
-8.5528 2.8796 0.2255 0.8727 3.2693 0 -.85 34.26 -23.43 30.41
-2.3778 2.1243 0.1574 0.4443 0 .85 35.11
-4.7403 2.5117 1.2285 0 -34.26 -35.11
-5.6115 2.7387 0.0676 1.4321 0 53.84 23.43
-7.6667 4.6690 0.0597 0.3698 0 -54.35 -30.41 -53.84

-3.8472 3.2359 0.0914 0.3451 0 53.07
-7.9194 2.4847 0.2125 0.8727 3.5929 0 -.832 38.23 -19.39 33.97
-1.8028 1.8330 0.1483 0.4911 0 .832 39.06
-4.0750 2.1672 1.2794 0 -38.23 -39.06
-5.4604 2.3631 0.0637 1.4521 0 53.36 19.39
-6.5694 4.4208 0.0444 0.3925 0 -53.07 -33.97 -53.36

-3.5616 2.8490 0.1012 0.3452 0 52.51
-7.8139 2.1877 0.2353 0.8727 3.6454 0 -.826 38.62 -20.84 34.17
-1.7000 1.6139 0.1642 0.4986 0 .826 39.45
-3.8722 1.9082 1.3020 0 -38.62 -39.45
-4.8489 2.0806 0.0706 1.5056 0 55.02 20.84
-5.5000 3.8146 0.0418 0.4115 0 -52.51 -34.17 -55.02

-3.5616 2.8176 0.0852 0.3452 0 51.63
-7.6083 2.1636 0.1979 0.8727 3.7211 0 -.8 39.75 -20.8 34.44
-1.6000 1.5961 0.1381 0.5093 0 .8 40.55
-4.0333 1.8871 1.3025 0 -39.75 -40.55
-4.5432 2.0577 0.0594 1.5316 0 55.24 20.8
-4.9583 3.3586 0.0463 0.4199 0 -51.63 -34.44 -55.24

-2.7801 2.5033 0.0614 0.3453 0 51.77
-7.2056 1.9222 0.1428 0.8727 3.8208 0 -.7 39.91 -20.46 36.06
-1.6000 1.4181 0.0996 0.5228 0 .7 40.61
-2.6764 1.6766 1.3782 0 -39.91 -40.61
-4.2911 1.8282 0.0428 1.5595 0 56.52 20.46
-4.0000 3.3215 0.0390 0.4325 0 -51.77 -36.06 -56.52

-3.0056 2.3675 0.0614 0.3453 0 51.57
-6.7903 1.8180 0.1428 0.8727 3.8859 0 -.56 40.65 -20.76 36.84
-1.7653 1.3411 0.0996 0.5309 0 .56 41.21
-2.5556 1.5857 1.3912 0 -40.65 -41.21
-3.8789 1.7290 0.0428 1.5349 0 57.6 20.76

-4.1389 2.9510 0.0281 0.4308 0 -51.57 -36.84 -57.6

-3.4113 2.7101 0.0747 0.3453 0 52.97
-7.0847 2.0810 0.1736 0.8727 3.7676 0 -.57 39.48 -20.12 36.47
-1.9847 1.5352 0.1212 0.5139 0 .57 40.05
-2.7611 1.8151 1.3573 0 -39.48 -40.05
-4.5654 1.9792 0.0521 1.5320 0 56.59 20.12
-3.9028 2.7910 0.0281 0.4337 0 -52.97 -36.47 -56.59

-3.6818 3.1057 0.0879 0.3452 0 53.13
-7.3750 2.3848 0.2044 0.6285 3.7018 0 -.68 38.53 -19.05 35.65
-1.9444 1.7593 0.1427 0.5054 0 .68 39.21
-2.7472 2.0801 1.3551 0 -38.53 -39.21
-5.1929 2.2681 0.0613 1.4831 0 54.7 19.05
-5.5417 3.6612 0.0402 0.4110 0 -53.13 -35.65 -54.7

-4.7338 4.0905 0.0942 0.3450 0 57.26
-8.6986 3.1410 0.2190 0.1903 3.8799 0 -1.12 33.73 -21.39 34.18
-1.9681 2.3172 0.1529 0.4551 0 1.12 34.86
-3.1778 2.7396 1.3071 0 -33.73 -34.86
-6.0218 2.9873 0.0657 1.4042 0 55.57 21.39
-8.4722 4.8221 0.0431 0.3508 0 -57.26 -34.18 -55.57

-4.5383 3.8303 0.0921 0.3450 0 56.06
-7.8764 2.9412 0.2142 0.0000 3.6195 0 -.97 37.73 -19.88 37.37
-1.7069 2.1698 0.1495 0.4962 0 .97 38.7
-3.1486 2.5654 1.3302 0 -37.73 -38.7
-5.2547 2.7973 0.0642 1.4731 0 57.25 19.88
-7.7361 4.5153 0.0422 0.3682 0 -56.06 -37.37 -57.25

-4.4633 3.6392 0.0831 0.3450 0 55.30
-7.7431 2.7945 0.1931 0.0000 3.5816 0 -.85 69.62 -19.37 36.04
-2.0208 2.0615 0.1347 0.4896 0 .85 70.46
-2.8222 2.4374 1.3610 0 -69.62 -70.46
-5.7490 2.6577 0.0579 1.4475 0 55.41 19.37
-7.2361 4.2901 0.0380 0.3792 0 -55.3 -36.04 -55.41

-3.3361 3.2285 0.0663 0.3453 0 53.48
-6.9667 2.4791 0.1541 0.0000 3.7676 0 -.62 38.79 -20.15 36.98
-2.0889 1.8288 0.1076 0.5144 0 .62 39.41
-2.4667 2.1623 1.3963 0 -38.79 -39.41
-4.7656 2.3577 0.0462 1.5330 0 57.13 20.15
-5.8333 3.8059 0.0303 0.4061 0 -53.48 -36.98 -57.13

-3.4714 2.9224 0.0482 0.3452 0 53.63
-7.5236 2.2441 0.1120 0.0000 3.7915 0 -.97 38.76 -21.21 37.5
-1.3375 1.6555 0.0781 0.5212 0 .97 39.72
-2.4347 1.9573 1.4183 0 -38.76 -39.72
-4.4613 2.1343 0.0336 1.5666 0 58.71 21.21
-5.9722 3.4451 0.0220 0.4038 0 -53.63 -37.5 -58.71

-2.6299 2.2140 0.0321 0.3454 0 51.72
-6.2222 1.7000 0.0746 0.0000 4.0746 0 -.61 42.53 -18.99 39.82
-1.4167 1.2542 0.0521 0.5588 0 .61 43.14
-2.0847 1.4828 1.4460 0 -42.53 -43.14
-3.6257 1.6169 0.0224 1.6153 0 58.8 18.99
-4.0694 2.6099 0.0147 0.4317 0 -51.72 -39.82 -58.8

.556 0 -.0119 0 -1 50.11 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0
0 0 0 0 0 0 4.6069 0 -.0276 0 0 -1 -.33 45.72 -16.99 43.47 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0
0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 .7319 0 -.0328 0 -1 .66 44.37 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0
0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 1.6903 0 0 -1 -45.99 -46.23 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0
0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 1.6977 0 -.0083 0 -1 60.46 16.99 0 0 0 0 0 0 0 0
0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 .6944 0 -.0048 0 -1 -50.1 -43.59 -60.25

0 0 0 0 0 1 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 -1 0 0
0 0 0 0 0 0 0 0 0 0 0 0 -1 0 0 0 0 0 0 0 0 1 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0

0 0 0 0 0 0 0 0 0 0 0 0 0 1 0 0 0 0 0 0 0 0 0 0 0 0 0 -1 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0
0 0 0 0 0 0 0 0 0 0 0 0 0 0 -1 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 1 0 0 0 0 0 0 0 0
0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 1 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 -1 0
0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 1 0 0 0 0 0 -1 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0
0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 0 1 0 0 0 0 0 0 0 0 -1

.95 .75 .95 .9 0 0
.95 .75 .95 .95 .9 0 0 0 0 0
.95 .75 .95 .9 0 0 0
.95 .75 .9 0 0 0
.95 .75 .95 .9 0 0 0
.95 .75 .95 .9 0 0 0 0
.19869 .15269 .11249 .13309 .14509 .23419 0 0 0 0 0 0 0
1.05 1.25 1.05 1.1 .3799 .061
1.05 1.25 1.05 1.05 1.1 4.8694 10.6081 .1662 .3519 .1823
1.05 1.25 1.05 1.1 .6453 2.6655 .0411
1.05 1.25 1.1 1.713 .0729 .0718
1.05 1.25 1.05 1.1 1.8634 .0749 .2196
1.05 1.25 1.05 1.1 .501 .0914 .1291 .0851
.1987 .1527 .1125 .1331 .1451 .2342 0 0 0 0 0 0 0
1 1 1 1 .2 .01
1 1 1 1 1 2.5 5 .05 .01 .05
1 1 1 1 .4 1 .01
1 1 1 1 .01 .01
1 1 1 1 1 .01 .1
1 1 1 1 .5 .01 .01 .01

F1.3 OUTPUT FILE

GROUP1.OUT

OUTPUT FILE FOR GROUP1 AT STAGE 3
E04NCF Program Results For Group1 Includig the Following Zones.

NL11 T3=T4=0,NL12 T4=0,NL13 T3=T4=0,NL14 T2=T3=T4=0,NL15 T3=T4=0,NT31 T3=T4=0

```
Calls to E04NEF
---------------

     Defaults
     Infinite Bound Size = 1.0E+15


OPTIONS file
------------

   Begin   Example options file for  E04NCF

     Problem type       =  Least squares
     Print level        =  1
     Feasibility phase  =  200
     Optimality  phase  =  200
     Rank tolerance     =  1.0E-4

   End

*** E04NCF
*** Start of NAG Library implementation details ***

Implementation title: DOS Salford FTN77/386
           Precision: Double Precision
        Product Code: FLIBP15DS
                Mark: 15

*** End of NAG Library implementation details ***

Parameters
----------

Problem type...........    LS1

Linear constraints.....     13      Feasibility tolerance.. 1.05D-08
Variables..............     44      Crash tolerance........ 1.00D-02
Objective matrix rows..     24      Rank tolerance......... 1.00D-04

Infinite bound size.... 1.00D+15    COLD start.............
Infinite step size..... 1.00D+20    EPS (machine precision) 1.11D-16

Print level............      1      Feasibility phase itns.    200
                                    Optimality  phase itns.    200

Workspace provided is     IW(    44),  W(  4346).
To solve problem we need  IW(    44),  W(  4346).

Rank of the objective function data matrix =    19

Exit from LS problem after   79 iterations.  IFAIL =  0


Varbl State    Value    Lower Bound   Upper Bound   Lagr Mult   Residual

V  1   FR    1.00812    0.950000      1.05000     0.0000E+00  4.1879E-02
V  2   UL    1.25000    0.750000      1.25000     -16.98      0.0000E+00
```

V 3	LL	0.950000	0.950000	1.05000	821.0	0.0000E+00
V 4	LL	0.900000	0.900000	1.10000	0.2136	0.0000E+00
V 5	UL	0.379900	0.000000E+00	0.379900	-6.520	0.0000E+00
V 6	FR	5.863087E-04	0.000000E+00	6.100000E-02	0.0000E+00	5.8631E-04
V 7	LL	0.950000	0.950000	1.05000	32.35	0.0000E+00
V 8	UL	1.25000	0.750000	1.25000	-2.612	0.0000E+00
V 9	UL	1.05000	0.950000	1.05000	-0.1687	0.0000E+00
V 10	UL	1.05000	0.950000	1.05000	-1.040	0.0000E+00
V 11	LL	0.900000	0.900000	1.10000	9.252	0.0000E+00
V 12	FR	4.41767	0.000000E+00	4.86940	0.0000E+00	0.4517
V 13	LL	0.000000E+00	0.000000E+00	10.6081	85.96	0.0000E+00
V 14	FR	4.873066E-03	0.000000E+00	0.166200	0.0000E+00	4.8731E-03
V 15	LL	0.000000E+00	0.000000E+00	0.351900	8.891	0.0000E+00
V 16	FR	-1.505686-307	0.000000E+00	0.182300	0.0000E+00	-1.5057-307
V 17	LL	0.950000	0.950000	1.05000	138.1	0.0000E+00
V 18	UL	1.25000	0.750000	1.25000	-2.575	0.0000E+00
V 19	UL	1.05000	0.950000	1.05000	-17.56	0.0000E+00
V 20	LL	0.900000	0.900000	1.10000	1.294	0.0000E+00
V 21	FR	0.548365	0.000000E+00	0.645300	0.0000E+00	9.6935E-02
V 22	FR	-7.588771E-17	0.000000E+00	2.66550	0.0000E+00	-7.5888E-17
V 23	FR	3.841327E-20	0.000000E+00	4.110000E-02	0.0000E+00	3.8413E-20
V 24	LL	0.950000	0.950000	1.05000	14.46	0.0000E+00
V 25	UL	1.25000	0.750000	1.25000	-11.37	0.0000E+00
V 26	LL	0.900000	0.900000	1.10000	4.262	0.0000E+00
V 27	FR	1.24857	0.000000E+00	1.71300	0.0000E+00	0.4644
V 28	FR	4.873066E-03	0.000000E+00	7.290000E-02	0.0000E+00	4.8731E-03
V 29	LL	0.000000E+00	0.000000E+00	7.180000E-02	7010.	0.0000E+00
V 30	LL	0.950000	0.950000	1.05000	29.71	0.0000E+00
V 31	LL	0.750000	0.750000	1.25000	1.636	0.0000E+00
V 32	LL	0.950000	0.950000	1.05000	4.2234E-04	0.0000E+00
V 33	LL	0.900000	0.900000	1.10000	2.668	0.0000E+00
V 34	FR	1.45983	0.000000E+00	1.86340	0.0000E+00	0.4036
V 35	LL	0.000000E+00	0.000000E+00	7.490000E-02	233.2	0.0000E+00
V 36	FR	-1.395487E-19	0.000000E+00	0.219600	0.0000E+00	-1.3955E-19
V 37	UL	1.05000	0.950000	1.05000	-1.727	0.0000E+00
V 38	UL	1.25000	0.750000	1.25000	-35.81	0.0000E+00
V 39	UL	1.05000	0.950000	1.05000	-0.3503	0.0000E+00
V 40	LL	0.900000	0.900000	1.10000	1.595	0.0000E+00
V 41	FR	0.460516	0.000000E+00	0.501000	0.0000E+00	4.0484E-02
V 42	FR	5.863087E-04	0.000000E+00	9.140000E-02	0.0000E+00	5.8631E-04
V 43	LL	0.000000E+00	0.000000E+00	0.129100	194.0	0.0000E+00
V 44	LL	0.000000E+00	0.000000E+00	8.510000E-02	64.47	0.0000E+00

L Con	State	Value	Lower Bound	Upper Bound	Lagr Mult	Residual
L 1	LL	0.198690	0.198690	0.198700	2.484	2.4980E-16
L 2	UL	0.152700	0.152690	0.152700	-1.972	1.5543E-15
L 3	UL	0.112500	0.112490	0.112500	-157.4	2.3592E-16
L 4	UL	0.133100	0.133090	0.133100	-1.962	-3.8858E-16
L 5	UL	0.145100	0.145090	0.145100	-2.477	0.0000E+00
L 6	LL	0.234190	0.234190	0.234200	2.484	8.3267E-17
L 7	EQ	-1.084202E-19	0.000000E+00	0.000000E+00	-128.5	-1.0842E-19
L 8	EQ	-7.588771E-17	0.000000E+00	0.000000E+00	89.37	-7.5888E-17
L 9	EQ	0.000000E+00	0.000000E+00	0.000000E+00	93.66	0.0000E+00
L 10	EQ	-1.395487E-19	0.000000E+00	0.000000E+00	34.13	-1.3955E-19
L 11	EQ	-1.505686-307	0.000000E+00	0.000000E+00	179.0	-1.5057-307
L 12	EQ	3.841327E-20	0.000000E+00	0.000000E+00	7041.	3.8413E-20
L 13	FR	0.000000E+00	0.000000E+00	0.000000E+00	0.0000E+00	0.0000E+00

Exit E04NCF - Optimal LS solution.

Final LS objective value = 50.26423

x(	1)=	1.00812063280
x(	2)=	1.25000000000
x(	3)=	0.950000000000

x(	4)=	0.900000000000
x(	5)=	0.379900000000
x(	6)=	5.863086841877E-04
x(	7)=	0.950000000000
x(	8)=	1.25000000000
x(	9)=	1.05000000000
x(	10)=	1.05000000000
x(	11)=	0.900000000000
x(	12)=	4.41767157592
x(	13)=	0.000000000000E+0000
x(	14)=	4.873065965046E-03
x(	15)=	0.000000000000E+0000
x(	16)=	-1.505685741790E-0307
x(	17)=	0.950000000000
x(	18)=	1.25000000000
x(	19)=	1.05000000000
x(	20)=	0.900000000000
x(	21)=	0.548365000000
x(	22)=	-7.588770838377E-17
x(	23)=	3.841327103068E-20
x(	24)=	0.950000000000
x(	25)=	1.25000000000
x(	26)=	0.900000000000
x(	27)=	1.24857269627
x(	28)=	4.873065965046E-03
x(	29)=	0.000000000000E+0000
x(	30)=	0.950000000000
x(	31)=	0.750000000000
x(	32)=	0.950000000000
x(	33)=	0.900000000000
x(	34)=	1.45983000000
x(	35)=	0.000000000000E+0000
x(	36)=	-1.395486780601E-19
x(	37)=	1.05000000000
x(	38)=	1.25000000000
x(	39)=	1.05000000000
x(	40)=	0.900000000000
x(	41)=	0.460515934922
x(	42)=	5.863086841877E-04
x(	43)=	0.000000000000E+0000
x(	44)=	0.000000000000E+0000

APPENDIX F2

HDSM PROGRAMME

F2.1 PROGRAMME LIST

```
C HDSM.FOR
C
C Written by Massoud Tabesh, 1995.
C
C This program analyses water distribution networks based on the head driven simulation method
C (HDSM) using the Newton Raphson iterative approach. The Wagner at al. (1988) head-outflow
C relationship is used to represent the pressure dependency of demand.
C
      PROGRAM HDSM
      PARAMETER (MXP=50)
      PARAMETER (MXN=50)
      PARAMETER (MXPAPUMP=5)
      PARAMETER (MXCHP=5)
      PARAMETER (MXNRV=5)
      PARAMETER (MXPRV=10)
      PARAMETER (MXFCV=5)
      PARAMETER (MXPTON=7)
      INTEGER N1(MXP),N2(MXP),NN(MXN),JB(MXN,MXPTON)
      REAL H(MXN),D(MXP),Q(MXP),CHW(MXP),QJ(MXN),K(MXP),F(MXN,MXN+1)
      REAL QA(MXN,MXN),QB(MXN,MXN),P(MXN),HS(MXN),HA(MXN),HMIN(MXN)
      REAL AA(MXPAPUMP),BB(MXPAPUMP),CC(MXPAPUMP),HPP(MXPAPUMP)
      REAL HPC(MXCHP),HPRV(MXPRV),KV(MXFCV),FIXHEAD(MXN),L(MXP),DIF
      REAL DIFLAST(MXN),DIFH(MXN),ZCOF(MXN),ZCOF1(MXN)
      CHARACTER CAPTN*80
C
C -------------------------------------------------------------------
C           READING INPUT DATA
C -------------------------------------------------------------------
C
      CALL CLOCK@(START)
      OPEN(UNIT=6,FILE='HDSM.OUT')
 98   OPEN(UNIT=5,FILE='HDSM.IN')
      READ(5,40)CAPTN
      WRITE(6,40)CAPTN
 40   FORMAT(A80)
      WRITE(6,*)'                '
C  NF= NO. OF FAILURES (NF=0 WHEN NO FAILURE)
C  NP= NO. OF PIPES
C  NJ= NO. OF JUNCTIONS (NODES)
C  NRES= NO. OF RESERVOIRS WITH FIXED HEADS
C  MAXNCT= MAXIMUM NO. OF ITERATION
C  ERR= PRECISION OF CALCULATED HEADS
C  NUNIT= SYSTEM OF EQUATIONS
C  NUNIT=1 IF SYSTEM UNIT=SI & NUNIT=2 IF SYSTEM UNIT=ENGLISH
C  NOEQ= TYPE OF EQUATIONS
C  NOEQ=0 WHEN DEMANDS ARE FIXED (DEMAND DRIVEN SIMULATION)
C  NOEQ=1 IF USE GERMANOPOLOUS'S EQ. & NOEQ=2 IF USE WAGNER'S EQ.
C  A,B= IMPERICAL PARAMETERES FOR GERMANOPOULOS EQUATION
      READ(5,*)NF,NP,NJ,NRES,MAXNCT,ERR,NUNIT,NOEQ,A,B,ZT
C  QB(I,J)= INPUT DATA MATRIX
C  0= NO COMPONENT, 1= PIPE, 2=NRV, 3=PRV, 4=FCV, 7=PAPUMP, 8=CHP
C  DIAGONAL INCLUDES EXTERNAL FLOW AT NODE ( -  IF IT IS OUTFLOW)
      DO 190 I=1,NJ
      READ(5,*)(QB(I,J),J=1,NJ)
 190  CONTINUE
C  D(I)=PIPE DIAMETER (m or in)
C  CHW(I)=HAZEN WILLIAM COEFFICIENT
```

```
C  L(I)=PIPE LENGHT (m or ft)
      DO 2 I=1,NP
      READ(5,*)D(I),CHW(I),L(I)
      IF(NUNIT .EQ. 1)GO TO 300
      D(I)=D(I)/12
      K(I)=4.727328*L(I)/(CHW(I)**1.85185185*D(I)**4.87037)
      GO TO 2
 300  K(I)=10.675*L(I)/(CHW(I)**1.85185185*D(I)**4.87037)
 2    CONTINUE
      NJM=NJ-NRES
C  HA(I)= INITIAL HEAD AT EACH NODE (m or ft)
C  P(I)= THE VALUE OF HEAD WHICH SATISFIES 93.2% OF DEMAND AT NODE I
C  HS(I)= SERVICE HEAD AT NODE I WHICH SATISFIES 100% DEMAND
C  HMIN(I)= MINIMUM HEAD AT NODE I WHICH SATISFIES NO DEMAND
      DO 4 I=1,NJ
      READ(5,*)HA(I),P(I),HS(I),HMIN(I)
 4    CONTINUE
C  FIXHEAD(I) =THE NODES WHICH HAVE FIX HEAD LIKE RESERVIORS
C  FIXHEAD(I)=1 IF HEAD IS FIXED OTHERWISE =0
      READ(5,*)(FIXHEAD(N),N=1,NJ)
C
C  NOPAPUMP= NO. OF PARABOLIC PUMPS
C  NOCHP= NO. OF CONSTANT HEAD PUMPS
C  NOPRV= NO. OF PRESSURE REDUCING VALVES
C  NOFCV= NO. OF FLOW CONTROL VALVES
C  NRV= NONE RETURN VALVES
      READ(5,*)NOPAPUMP,NOCHP,NOPRV,NOFCV
C  AA(I),BB(I),CC(I)= CONSTANT PARAMETERES OF EACH PARABOLIC PUMPS
C  HPP= HEAD OF PARABOLIC PUMP
C  HPC= HEAD OF CONSTANT PUMP
C  KV= CONSTANT VALVE CONTROL PARAMETERE (0-1)
C  HPRV= SETTING HEAD OF PRV
      IF(NOPAPUMP .EQ. 0.0)GO TO 330
      DO 320 N=1,NOPAPUMP
      READ(5,*)AA(N),BB(N),CC(N)
 320  CONTINUE
 330  IF(NOCHP .EQ. 0.0)GO TO 311
      DO 340 N=1,NOCHP
      READ(5,*)HPC(N)
 340  CONTINUE
 311  IF(NOPRV .EQ. 0.0)GO TO 350
      DO 360 N=1,NOPRV
      READ(5,*)HPRV(N)
 360  CONTINUE
 350  IF(NOFCV .EQ. 0.0)GO TO 1000
      DO 370 N=1,NOFCV
      READ(5,*)KV(N)
 370  CONTINUE
C
C --------------------------------------------------------------------
C              MAIN PROGRAM FOR CALCULATION FAILUR'S EFFECTS
C --------------------------------------------------------------------
C
 1000 NFF=1
      IF(NF .NE. 0)NFF=NF
      DO 9900 KKK=1,NFF
      PRINT*,'pipe failured no:',kkk
      DO 9010 KI=1,NJ
      DO 9020 KJ=1,NJ
      QA(KI,KJ)=QB(KI,KJ)
 9020 CONTINUE
      H(KI)=HA(KI)
 9010 CONTINUE
      NOP=0
      DO 200 I=1,NJ
      DO 210 J=1,NJ
      IF(I.EQ.J)QJ(I)=QA(I,J)
```

```
      IF(QA(I,J) .EQ. 0.0 .OR. J .LE. I)GO TO 210
      NOP=NOP+1
      IF(NF .EQ. 0)GO TO 205
      IF(NOP .NE. KKK)GO TO 205
      QA(I,J)=0
      QA(J,I)=0
 205  N1(NOP)=I
      N2(NOP)=J
      Q(NOP)=QA(I,J)
      IF(Q(NOP) .GE. 0.0)GO TO 210
      KOLD=N1(NOP)
      N1(NOP)=N2(NOP)
      N2(NOP)=KOLD
      Q(NOP)=-QA(I,J)
 210  CONTINUE
 200  CONTINUE
C
C -------------------------------------------------------------------
C                    HYDRAULIC SIMULATION
C  1. CALCULATION OF NO. OF PIPES AT EACH NODE (KK) & SIGNS BY JB MATRIX
C -------------------------------------------------------------------
C
      DO 5 J=1,NJ
      NNP=0
      DO 6 I=1,NP
      IF(NF .EQ. 0)GO TO 77
      IF(I .EQ. KKK)GO TO 6
 77   IF(N1(I).NE.J)GO TO 7
      NNP=NNP+1
      JB(J,NNP)=I
      GO TO 6
 7    IF(N2(I).NE.J)GO TO 6
      NNP=NNP+1
      JB(J,NNP)=-I
 6    CONTINUE
      NN(J)=NNP
 5    CONTINUE
      NCT=0
C
C -------------------------------------------------------------------
C  2. CALCULATION OF Qij at Fj
C     a. PIPES
C -------------------------------------------------------------------
C
 20   SUM=0
      NZCOUNT=0
      NCT=NCT+1
 2000 NPAPUMP=0
      NCHP=0
      NNRV=0
      NPRV=0
      NFCV=0
      IPRV=0
      IFCV=0
      IPAPUMP=0
      ICHP=0
      JE=0
      DO 10 J=1,NJ
      IF(FIXHEAD(J) .EQ. 1)GO TO 10
      JE=JE+1
      JJE=J-JE
      DO 15 JJ=1,NJM+1
      F(JE,JJ)=0
 15   CONTINUE
      NNP=NN(J)
      DO 11 KK=1,NNP
      II=JB(J,KK)
```

```
      I=ABS(II)
      I1=N1(I)
      I2=N2(I)
      I1I1=I1
      I2I2=I2
      FAC=II/I
      FAC1=1
      IF(ABS(QA(I1,I2)) .EQ. 1)THEN
      ARG=H(I1)-H(I2)
      ARGE=(ARG/K(I))**.54
      ENDIF
C
C ----------------------------------------------------------------------
C   b. OTHER COMPONENTS
C ----------------------------------------------------------------------
C
      IF(ABS(QA(I1,I2)) .NE. 2)GO TO 120
      NNRV=NNRV+1
      IF(QA(I1,I2) .LT. 0)THEN
      I1=N2(I)
      I2=N1(I)
      ENDIF
      IF(H(I1) .GE. H(I2))ARG=ABS(H(I1)-H(I2))
      IF(H(I1) .LT. H(I2))ARG=0
      ARGE=(ARG/K(I))**.54
 120  IF(ABS(QA(I1,I2)) .NE. 3)GO TO 130
      IF(I .EQ. IPRV)GO TO 123
      NPRV=NPRV+1
      IPRV=I
 123  IF(Q(I) .NE. QA(I1,I2))THEN
      I1=N2(I)
      I2=N1(I)
      ENDIF
      IF(HPRV(NPRV) .GE. H(I2) .AND. HPRV(NPRV) .LE. H(I1))THEN
      ARG=ABS(HPRV(NPRV)-H(I2))
      ENDIF
      IF(HPRV(NPRV) .LT. H(I2))ARG=0
      IF(HPRV(NPRV) .GT. H(I1) .AND. H(I1) .GE. H(I2))THEN
      ARG=ABS(H(I1)-H(I2))
      ENDIF
      ARGE=(ARG/K(I))**.54
 130  IF(ABS(QA(I1,I2)) .NE. 4)GO TO 110
      NFCV=NFCV+1
      IF(I .EQ. IFCV)GO TO 135
      NFCV=NFCV+1
      IFCV=I
 135  IF(Q(I) .NE. QA(I1,I2))THEN
      I1=N2(I)
      I2=N1(I)
      ENDIF
      ARG=ABS(H(I1)-H(I2))
      ARGE=(ARG/K(I))**.54
      ARGE=ARGE*KV(NFCV)
 110  IF(ABS(QA(I1,I2)) .NE. 7)GO TO 150
      IF(I .EQ. IPAPUMP)GO TO 113
      NPAPUMP=NPAPUMP+1
      IPAPUMP=I
 113  IF(Q(I) .NE. QA(I1,I2))THEN
      I1=N2(I)
      I2=N1(I)
      ENDIF
C
      ROOT=BB(NPAPUMP)**2-4*AA(NPAPUMP)*(CC(NPAPUMP)-(H(I2)-H(I1)))
      IF(ROOT .LT. 0)THEN
      ARGE=0
      GO TO 150
      ENDIF
```

```
      ARGE=(-BB(NPAPUMP)+ROOT**.5)/(2*AA(NPAPUMP))
      IF(ARGE .LT. 0)THEN
      ARGE=(-BB(NPAPUMP)-ROOT**.5)/(2*AA(NPAPUMP))
      ENDIF
 150  IF(ABS(QA(I1,I2)) .NE. 8)GO TO 1313
      NCHP=NCHP+1
      IF(I.EQ.ICHP)GO TO 155
      NCHP=NCHP+1
      ICHP=I
 155  IF(Q(I) .NE. QA(I1,I2))THEN
      I1=N2(I)
      I2=N1(I)
      ENDIF
      ARG=ABS(H(I1)+HPC(NCHP)-H(I2))
      ARGE=(ARG/K(I))**.54
      IF((H(I1)+HPC(NCHP)-H(I2)) .GT. 0.0)FAC1=1
      IF((H(I1)+HPC(NCHP)-H(I2)) .EQ. 0)THEN
      FAC1=0
      ELSE
      FAC=-1
      ENDIF
 1313 FAC5=.54*FAC*FAC1
C
 13   IF(ABS(QA(I1,I2)) .NE. 7)GO TO 1315
      IF(J .EQ. I1)FAC=1
      IF(J .EQ. I2)FAC=-1
 1315 F(JE,NJM+1)=F(JE,NJM+1)+ARGE*FAC
C
C --------------------------------------------------------------------
C   c. CALCULATION OF DERIVATIVES ON H(I1)
C --------------------------------------------------------------------
C
      IF(FIXHEAD(I1I1) .EQ. 1)GO TO 14
      IF(KK .NE. NNP)GO TO 8000
      IF(NOEQ .EQ. 0)GO TO 8000
      IF(NOEQ .EQ. 1)THEN
      PRESSURE=H(J)-HMIN(J)
      PSTAR=HS(J)-HMIN(J)
      F(JE,I1I1)=F(JE,I1I1)+(QA(J,J)*A*B/PSTAR)*EXP(-B*PRESSURE/PSTAR)
      ELSE
      IF(H(J) .GT. HS(J) .OR. H(J) .LT. HMIN(J))GO TO 8000
      F(JE,I1I1)=F(JE,I1I1)+(.5*QA(J,J)/(HS(J)-HMIN(J)))*((H(J)-HMIN(J))
     &/(HS(J)-HMIN(J)))**(-.5)
      ENDIF
 8000 IF(ABS(QA(I1,I2)) .NE. 2)GO TO 170
      IF(H(I1) .GE. H(I2))ARG=ABS(H(I1)-H(I2))
      IF(H(I1) .LT. H(I2))ARG=0
 170  IF(ABS(QA(I1,I2)) .NE. 3)GO TO 180
      IF(HPRV(NPRV) .GE. H(I1) .AND. H(I1) .GE. H(I2))THEN
      ARG=ABS(H(I1)-H(I2))
      ELSE
      ARG=0
      ENDIF
 180  IF(ABS(QA(I1,I2)) .NE. 4)GO TO 160
      FAC5=.54*FAC*KV(NFCV)
 160  IF(ABS(QA(I1,I2)) .NE. 7)GO TO 230
      IF(ROOT .LT. 0)THEN
      FS=0
      GO TO 230
      ENDIF
      FS=-1*ROOT**(-.5)
      IF(ARGE .LT. 0)FS=1*ROOT**(-.5)
 230  IF(ABS(QA(I1,I2)) .NE. 8)GO TO 2424
      ARG=ABS(H(I1)+HPC(NCHP)-H(I2))
      IF((H(I1)+HPC(NCHP)-H(I2)) .GT. 0.0)FAC1=1
      IF((H(I1)+HPC(NCHP)-H(I2)) .EQ. 0)THEN
      FAC1=0
```

```
      ELSE
      FAC1=-1
      ENDIF
 2424 FAC5=.54*FAC*FAC1
C
 240  IF(ABS(QA(I1,I2)) .NE. 7)GO TO 245
      F(JE,I1I1)=F(JE,I1I1)+FS
      GO TO 14
 245  IF(ARG .EQ. 0)THEN
      F(JE,I1I1)=F(JE,I1I1)
      ELSE
      F(JE,I1I1)=F(JE,I1I1)+(FAC5/K(I)**.54)*(ARG**(-.46))
      ENDIF
C
C ------------------------------------------------------------------
C   d. CALCULATION OF DERIVATIVES ON H(I2)
C ------------------------------------------------------------------
C
 14   IF(FIXHEAD(I2I2) .EQ. 1)GO TO 11
      IF(KK .NE. NNP)GO TO 8010
      IF(NOEQ .EQ. 0)GO TO 8010
      IF(NOEQ .EQ. 1)THEN
      PRESSURE=H(J)-HMIN(J)
      PSTAR=HS(J)-HMIN(J)
      F(JE,I2I2)=F(JE,I2I2)+(QA(J,J)*A*B/PSTAR)*EXP(-B*PRESSURE/PSTAR)
      ELSE
      IF(H(J) .GT. HS(J) .OR. H(J) .LT. HMIN(J))GO TO 8010
      F(JE,I2I2)=F(JE,I2I2)+(.5*QA(J,J)/(HS(J)-HMIN(J)))*((H(J)-HMIN(J))
     &/(HS(J)-HMIN(J)))**(-.5)
      ENDIF
 8010 IF(ABS(QA(I1,I2)) .NE. 2)GO TO 260
 260  IF(ABS(QA(I1,I2)) .NE. 3)GO TO 270
      IF(H(I1) .GE. HPRV(NPRV) .AND. HPRV(NPRV) .GE. H(I2))THEN
      ARG=ABS(HPRV(NPRV)-H(I2))
      ELSEIF(HPRV(NPRV) .GT. H(I1) .AND. H(I1) .GT. H(I2))THEN
      ARG=ABS(H(I1)-H(I2))
      ELSEIF(HPRV(NPRV) .LT. H(I2))ARG=0
      ELSEIF(H(I1) .LE. HPRV(NPRV) .AND. HPRV(NPRV) .LE. H(I2))ARG=0
      ENDIF
 270  IF(ABS(QA(I1,I2)) .NE. 4)GO TO 250
      FAC5=.54*FAC*KV(NFCV)
 250  IF(ABS(QA(I1,I2)) .NE. 7)GO TO 280
      IF(ROOT .LT. 0)THEN
      FS=0
      GO TO 280
      ENDIF
      FS=ROOT**(-.5)
      IF(ARGE .LT. 0)FS=-1*ROOT**(-.5)
 280  IF(ABS(QA(I1,I2)) .NE. 8)GO TO 8015
C
 8015 IF(ABS(QA(I1,I2)) .NE. 7)GO TO 8020
      F(JE,I2I2)=F(JE,I2I2)+FS
      GO TO 11
 8020 IF(ARG .EQ. 0)THEN
      F(JE,I2I2)=F(JE,I2I2)
      ELSE
      F(JE,I2I2)=F(JE,I2I2)-(FAC5/K(I)**.54)*(ARG**(-.46))
      ENDIF
 11   CONTINUE
      I1=I1I1
      I2=I2I2
      IF(NOEQ .EQ. 0)GO TO 8888
      IF(NOEQ .EQ. 1)THEN
      PRESSURE=H(J)-HMIN(J)
      PSTAR=HS(J)-HMIN(J)
      QJ(J)=QA(J,J)*(1-A*EXP(-B*PRESSURE/PSTAR))
      GO TO 8888
```

```
      ENDIF
      IF(H(J) .LE. HMIN(J))THEN
      QJ(J)=0
      GO TO 8888
      ENDIF
      IF(H(J) .GE. HS(J))THEN
      QJ(J)=QA(J,J)
      GO TO 8888
      ENDIF
      QJ(J)=QA(J,J)*((H(J)-HMIN(J))/(HS(J)-HMIN(J)))**.5
 8888 F(JE,NJM+1)=F(JE,NJM+1)-QJ(J)
      ZCOF1(JE)=F(JE,NJM+1)
 10   CONTINUE
C
      IF(NZCOUNT.EQ.1)GO TO 2010
C
      CALL SOLVE(F,NP,NJ,NJM,NCT,DIFMAX)
C
      JE=0
      DO 2050 J=1,NJ
      IF(FIXHEAD(J).EQ.1)GO TO 2050
      JE=JE+1
      DIFH(JE)=F(JE,NJM+1)
 2050 CONTINUE
C
C --------------------------------------------------------------------
C   e. CALCULATION OF NEW H AND Q
C --------------------------------------------------------------------
C
      NZCOUNT=0
 2060 ZIGDIFSQ=0
      ZIGDIFRT=0
      JE=0
      DIFMAX=0
      IF(NCT .NE. 1)GO TO 272
      DO 242 MD=1,NJ
      DIFLAST(MD)=0
 242  CONTINUE
 272  DO 24 J=1,NJ
      IF(FIXHEAD(J) .EQ. 1)GO TO 24
      JE=JE+1
      DIF=DIFH(JE)
      SUM=SUM+ABS(DIF)
      IF(ABS(DIF) .GT. DIFMAX)DIFMAX=ABS(DIF)
      IF((H(J)-DIF) .LT. 0.0)DIF=DIF/2
      DIF=(DIF+DIFLAST(J))/2
      H(J)=H(J)-DIF*ZT
      DIFSQ=(DIF)**2
      ZIGDIFSQ=ZIGDIFSQ+DIFSQ
      DIFLAST(J)=DIF
      IF(NZCOUNT.EQ.1)GO TO 24
      ZCOF(JE)=ZCOF1(JE)
 24   CONTINUE
      DO 25 I=1,NP
      IF(NF .EQ. 0)GO TO 3333
      IF(I .EQ. KKK)GO TO 25
 3333 I1=N1(I)
      I2=N2(I)
      IF(H(I1).GT.H(I2))GO TO 25
 225  FORMAT('FLOW IS REVERSED IN PIPE',3I5)
      N1(I)=I2
      N2(I)=I1
      II=I1
 28   NNP=NN(II)
      DO 26 KK=1,NNP
      IF(IABS(JB(II,KK)).NE.I)GO TO 26
      JB(II,KK)=-JB(II,KK)
```

```
      GO TO 27
 26   CONTINUE
 27   IF(II.EQ.I2)GO TO 25
      II=I2
      GO TO 28
 25   CONTINUE
      ZIGDIFRT=ZIGDIFSQ**.5
      GO TO 2070
      IF(NZCOUNT.EQ.1)GO TO 2070
      NZCOUNT=1
      GO TO 2000
      JE=0
 2010 DO 2020 J=1,NJ
      IF(FIXHEAD(J).EQ.1)GO TO 2020
      JE=JE+1
      FZ1=ABS(ZCOF1(JE))
      FZ0=ABS(ZCOF(JE))
      IF(ZCOF1(JE) .LE. ZCOF(JE))GO TO 2020
      RF=FZ1/FZ0
      Z=((1-6*RF)**(.5)-1)/(3*RF)
      DIFH(J)=DIFH(JE)*Z
2020  CONTINUE
      GO TO 2060
 2070 WRITE(6,*)'ITERATION NO.=',NCT,'      NORM=',ZIGDIFRT
      IF(NCT .LT. MAXNCT .AND. DIFMAX .GT. ERR)GO TO 20
C
C ----------------------------------------------------------------------
C     f. PRINTTIG THE OUTPUT
C ----------------------------------------------------------------------
C
      WRITE(6,*)'                    '
      WRITE(6,*)'--------------------------------------------------------'
      WRITE(6,*)' NO. OF ITERATIONS=',NCT,'      DIFMAX=',DIFMAX
      WRITE(6,*)'   REQUIRED PRECISION=',ERR
      WRITE(6,*)'       '
      IF(NF .EQ. 0)GO TO 107
      WRITE(6,*)'  ****** PIPE FAILURED =',KKK
 103  FORMAT(' HEADS AT JUNCTIONS',/,(1H ,13F10.3))
 107  WRITE(6,104)
 104  FORMAT(' FROM   TO  DIAMETER    LENGTH        CHW  FLOWRATE HEAD LO
     &SS   HEADS AT NODES')
      NPRV=0
      NPAPUMP=0
      NCHP=0
      DO 17 I=1,NP
      IF(NF .EQ. 0)GO TO 33
      IF(I .EQ. KKK)GO TO 17
 33   I1=N1(I)
      I2=N2(I)
      DH=H(I1)-H(I2)
      IF(ABS(QA(I1,I2)) .EQ. 3)THEN
      NPRV=NPRV+1
      IF(QA(I1,I2) .EQ. 3)THEN
      IF(HPRV(NPRV) .GT. H(I1))DH=H(I1)-H(I2)
      IF(HPRV(NPRV).LT.H(I1).AND.HPRV(NPRV).GT.H(I2))DH=HPRV(NPRV)-H(I2)
      IF(HPRV(NPRV) .LT. H(I2))DH=0
      ENDIF
      ENDIF
      IF(QA(I1,I2) .EQ. -3)THEN
      Q(I)=0
      GO TO 2222
      ENDIF
      Q(I)=(DH/K(I))**.54
 2222 IF(ABS(QA(I1,I2)). EQ. 7)DH=-DH
      IF(ABS(QA(I1,I2)) .NE. 7)GO TO 19
      NPAPUMP=NPAPUMP+1
      IF(QA(I1,I2) .EQ. 7)THEN
```

```
      Q(I)=0
      GO TO 19
      ENDIF
 1234 IF(Q(I) .NE. QA(I1,I2))THEN
      I1=N2(I)
      I2=N1(I)
      ENDIF
      ROOT=BB(NPAPUMP)**2-4*AA(NPAPUMP)*(CC(NPAPUMP)-(H(I2)-H(I1)))
      IF(ROOT .LT. 0)THEN
      Q(I)=0
      GO TO 19
      ENDIF
      Q(I)=(-BB(NPAPUMP)+ROOT**.5)/(2*AA(NPAPUMP))
      IF(Q(I) .LT. 0)THEN
      Q(I)=(-BB(NPAPUMP)-ROOT**.5)/(2*AA(NPAPUMP))
      ENDIF
 19   WRITE(6,105)I1,I2,D(I),L(I),CHW(I),Q(I),DH,H(I1),H(I2)
 105  FORMAT(2I5,2F10.3,F10.0,4F10.3)
 17   CONTINUE
C
      DO 4647 J=1,NJ
      IF(FIXHEAD(J) .EQ. 1)GO TO 4647
      IF(NOEQ .EQ. 0)GO TO 4647
      IF(NOEQ .EQ. 1)THEN
      PRESSURE=H(J)-HMIN(J)
      PSTAR=HS(J)-HMIN(J)
      QJ(J)=QA(J,J)*(1-A*EXP(-B*PRESSURE/PSTAR))
      GO TO 4647
      ENDIF
      IF(H(J) .LE. HMIN(J))THEN
      QJ(J)=0
      GO TO 4647
      ENDIF
      IF(H(J) .GE. HS(J))THEN
      QJ(J)=QA(J,J)
      GO TO 4647
      ENDIF
      QJ(J)=QA(J,J)*((H(J)-HMIN(J))/(HS(J)-HMIN(J)))**.5
 4647 CONTINUE
C
      DO 4001 JZ=1,NJ
      IF(FIXHEAD(JZ) .NE. 1)GO TO 4001
      PRINT*,JZ
      QJ(JZ)=0
      DO 4002 NZ=1,NP
      IF(N1(NZ) .NE. JZ)THEN
      IF(N2(NZ) .NE. JZ)GO TO 4002
      ENDIF
      IF(ABS(QA(N1(NZ),N2(NZ))) .EQ. 7 .AND. N2(NZ) .EQ. JZ)THEN
      QJ(JZ)=(Q(NZ))
      GO TO 4002
      ENDIF
      IF(JZ .EQ. N1(NZ))THEN
      QJ(JZ)=QJ(JZ)+Q(NZ)
      GO TO 4002
      ENDIF
      IF(JZ .EQ. N2(NZ))THEN
      QJ(JZ)=QJ(JZ)-Q(NZ)
      ENDIF
 4002 CONTINUE
 4001 CONTINUE
      ZIGMAQJ=0
      ZIGINFLW=0
      DO 4005 J=1,NJ
      IF(FIXHEAD(J) .EQ. 1)GO TO 4004
      ZIGMAQJ=ZIGMAQJ-QJ(J)
      GO TO 4005
```

```
4004 ZIGINFLW=ZIGINFLW+QJ(J)
4005 CONTINUE
     X2=ZIGMAQJ
     WRITE(6,*)' '
     DO 433 J=1,NJ
     WRITE(6,*)'H(',J,')=',H(J),'QJ(',J,')=',QJ(J)
433  CONTINUE
     WRITE(6,*)' '
     WRITE(6,*)'>>>>> SUMMATION OF NODAL OUTFLOWS AT DEMAND NODES =',X2
     WRITE(6,*)' '
     GO TO 9901
9901 IF(NF .EQ. 0)GO TO 99
9900 CONTINUE
99   CALL CLOCK@(FINISH)
     TIME=FINISH-START
     WRITE(6,*)'    '
     WRITE(6,*)'----------- TIME=',TIME,'SECOND  ------------------'
     STOP
     END
C
C -------------------------------------------------------------------
C              END OF THE MAIN PROGRAM
C -------------------------------------------------------------------
C
C -------------------------------------------------------------------
C  g. SUBROUTIN FOR CALCULATION OF EQUATIONS SET
C -------------------------------------------------------------------
C
     SUBROUTINE SOLVE (F,NP,NJ,NJM,NCT,DIFMAX)
C
     REAL F(45,46),Z(45)
     REAL C,DET
C.................. START OF PROGRAM
     NN1=NJM
     DO 600 I=1,NN1
     Z(I)=F(I,NJM+1)
 600 CONTINUE
     EPI=0.00000001
     N1=NN1-1
     DO 610 K=1,N1
     K1=K+1
     C=F(K,K)
     IF (ABS(C) .LE. EPI) THEN
     DO 620 J=K1,NN1
C........... TRY TO INTERCHANGE ROWS
     IF (ABS(F(J,K)) .GT. EPI)THEN
     DO 630 LL=K,NN1
     C=F(K,LL)
     F(K,LL)=F(J,LL)
     F(J,LL)=C
 630 CONTINUE
     C=Z(K)
     Z(K)=Z(J)
     Z(J)=C
     C=F(K,K)
     GO TO 640
     ENDIF
 620 CONTINUE
     GO TO 650
     ENDIF
C.... DIVID ROW BY DIAGONAL COEFFICIENT
 640 CONTINUE
     DO 660 J=K1,NN1
     F(K,J)=F(K,J)/C
 660 CONTINUE
     Z(K)=Z(K)/C
C.... ELIMINATE UNKNOWN X(K) FROM ROW I
```

```
      DO 670 I=K1,NN1
      C=F(I,K)
      DO 680 J=K1,NN1
      F(I,J)=F(I,J)-C*F(K,J)
  680 CONTINUE
      Z(I)=Z(I)-C*Z(K)
  670 CONTINUE
  610 CONTINUE
C................ COMPUTE LAST UNKNOWN
      IF (ABS(F(NN1,NN1)) .GT. EPI) THEN
      Z(NN1)=Z(NN1)/F(NN1,NN1)
      DO 690 LL=1,N1
      K=NN1-LL
      K1=K+1
      DO 700 J=K1,NN1
      Z(K)=Z(K)-F(K,J)*Z(J)
  700 CONTINUE
  690 CONTINUE
      DO 720 I=1,NN1
      F(I,NJM+1)=Z(I)
  720 CONTINUE
C......... COMPUTE VALUE OF DETERMINANT
      DET=1.
      DO 710 I=1,NN1
      DET=DET*F(I,I)
  710 CONTINUE
      RETURN
      ENDIF
  650 DET=0.0
      RETURN
      END
```

F2.2 INPUT FILE

```
EXAMPLE 5.1 (NO FAILURE)
0 4 5 1 10000 .001 1 2 1 1.14 17
-.066666667 -1 0 0 0
1 -.05 -1 0 0
0 1 -.033333333 -1 0
0 0 1 -.033333333 -1
0 0 0 1 .133333333
 130 1000
 130 1000
5 130 1000
 130 1000
91.6 86.6 86.6 85
91.7 90.9 90.9 90
91.8 88.4 88.4 88
91.9 90.4 90.4 90
100 100 100 100
0 0 0 0 1
0 0 0 0
```

F2.3 OUTPUT FILE

```
EXAMPLE 5.1 (NO FAILURE)

 ITERATION NO.=       1    NORM=   0.104122
 ITERATION NO.=       2    NORM=   0.112433
 ITERATION NO.=       3    NORM=   9.063007E-02
 ITERATION NO.=       4    NORM=   6.441569E-02
 ITERATION NO.=       5    NORM=   4.779814E-02
 ITERATION NO.=       6    NORM=   3.611581E-02
 ITERATION NO.=       7    NORM=   2.442037E-02
 ITERATION NO.=       8    NORM=   1.462449E-02
 ITERATION NO.=       9    NORM=   8.377761E-03
 ITERATION NO.=      10    NORM=   6.465737E-03
 ITERATION NO.=      11    NORM=   5.060641E-03
 ITERATION NO.=      12    NORM=   3.457203E-03
 ITERATION NO.=      13    NORM=   2.385431E-03
 ITERATION NO.=      14    NORM=   1.982849E-03
 ITERATION NO.=      15    NORM=   1.516812E-03
 ITERATION NO.=      16    NORM=   1.050120E-03

-----------------------------------------------------
 NO. OF ITERATIONS=       16    DIFMAX=   5.775443E-04
   REQUIRED PRECISION=   1.000000E-03

FROM  TO  DIAMETER   LENGTH    CHW  FLOW  HEAD LOSS  HEADS AT NODES
  2    1    0.300    1000.000  130.  0.067     3.033   90.015   86.982
  3    2    0.300    1000.000  130.  0.073     3.633   93.647   90.015
  4    3    0.350    1000.000  130.  0.106     3.406   97.053   93.647
  5    4    0.400    1000.000  130.  0.140     2.947  100.000   97.053

H(      1)=   86.9816   QJ(     1)=  -6.666667E-02
H(      2)=   90.0145   QJ(     2)=  -6.350572E-03
H(      3)=   93.6473   QJ(     3)=  -3.333333E-02
H(      4)=   97.0528   QJ(     4)=  -3.333333E-02
H(      5)=  100.000    QJ(     5)=   0.139804

>>>>> SUMMATION OF NODAL OUTFLOWS AT DEMAND NODES =   0.139684

----------- TIME=   0.164   SECOND  ------------------
```

APPENDIX F3

RELIABLE PROGRAMME

F3.1 PROGRAMME LIST

```
C RELIABLE.FOR
C
C Written by Massoud Tabesh, (1996).
C
C This program calculates the system reliability. Three different pipe availability formulations of
C Cullinane et al. (1992), Su et al. (1987) and Fujiwara and Tung (1991) are
C used, respectively. The programme considerers up to two simultaneous link
C failures including pumps and PRVs.
C
      PROGRAM RELIABLE
      PARAMETER (MXN=50, MXP=50)
      PARAMETER (NOPUMP=3)
      REAL DIA(MXP),R1(MXP),R2(MXP),U1(MXP),U2(MXP),TOTCONS(MXN,MXN),MDI
     &A(MXP),LEN(MXP),TCONS(MXN),R3(MXP),U3(MXP),TYPELINK(20),Q(NOPUMP)
      CHARACTER CAPTN*80
C NLINK= NO. OF LINKS
C NF= NO. OF FAILURES
C NP= NO. OF PIPES
C T0= TOTAL DEMANDS
C TOTCONS= TOTAL CONSUMPTIONS
C LEN(I)= LENGHT OF PIPE I (m)
C MDIA(I)= DIAMETERE OF PIPE I (mm)
C R1= CULLINANE ET AL EQ. (1992)
C R2= SU ET AL EQ. (1987)
C R3= FUJIWARA & TUNG EQ. (1991)
C TYPELINK = A NUMBER SHOWS TYPE OF LINK
C 1=PIPE, 2=PUMP, 3=PIPE INCLUDING PRV
C Q=DESIGN FLOW RATE IN PUMP IN (GPM)
C
C NCODE=1 ONE FAILURE, NCODE=2 TWO SIMULTANEOUS FAILURE
C
      CALL CLOCK@(START)
      OPEN(UNIT=5,FILE='RELIABLE.DAT')
      OPEN(UNIT=6,FILE='RELIABLE.OUT')
C
      READ(5,40)CAPTN
      WRITE(6,*)CAPTN
      WRITE(6,*)' '
      WRITE(6,*)' '
      READ(5,*)NCODE
      READ(5,*)NLINK,NP,R0
      READ(5,*)T0
C
      DO I=1,NP
        READ(5,*)LEN(I)
      END DO
C VERTICAL INPUT
      DO 555 I=1,NLINK
      IF(NCODE.EQ.1)READ(5,*)TOTCONS(I,I)
      IF(NCODE.EQ.1)GO TO 555
      DO J=I,NLINK
      READ(5,*)TOTCONS(I,J)
      END DO
 555  CONTINUE
C
C MATRIX INPUT
```

```
C      DO I=1,NP
C        READ(5,*)(TOTCONS(I,J),J=I,NP)
C      END DO
C
       READ(5,*)(TYPELINK(I),I=1,NLINK)
       READ(5,*)(Q(I),I=1,NOPUMP)
C
      NPUMP=0
      DO I=1,NLINK
      IF(TYPELINK(I).EQ.2)GO TO 510
      READ(5,*)MDIA(I)
      DIA(I)=MDIA(I)/25.4
      IF(TYPELINK(I).EQ.3)GO TO 550
      A=0.21218*DIA(I)**1.462131
      B=0.00074*DIA(I)**0.285
      R1(I)=A/(A+B)
C
      BMY=0.6858/DIA(I)**3.26 + 2.7158/DIA(I)**1.3131
     & + 2.7685/DIA(I)**3.5792 + 0.042
C BMY (BREAKS/MILE/YEAR)
      BKY=BMY*0.625
C BKY (BREAKS/KM/YEAR)
C LEN(I)*1.0E-3 (LENGHT IN KM)
      R2(I)=EXP(-BKY*LEN(I)*1.0E-3)
      U1(I)=1.0-R1(I)
      U2(I)=1.0-R2(I)
      R3(I)=.64/(.64+((2.002-.0064*MDIA(I))/365))
      U3(I)=1.0-R3(I)
      GO TO 500
550   R1(I)=0.9278*DIA(I)**0.000118
      U1(I)=1.0-R1(I)
      R2(I)=R1(I)
      R3(I)=R1(I)
      U2(I)=U1(I)
      U3(I)=U1(I)
      GO TO 500
510   NPUMP=NPUMP+1
      R1(I)=1.046943*Q(NPUMP)**(-0.01634)
      U1(I)=1.0-R1(I)
      R2(I)=R1(I)
      R3(I)=R1(I)
      U2(I)=U1(I)
      U3(I)=U1(I)
500     END DO
C
      S1=1.0
      S2=1.0
      S3=1.0
C
      DO I=1,NLINK
        S1=S1*R1(I)
        S2=S2*R2(I)
        S3=S3*R3(I)
      END DO
C
      T1=0.0
      T2=0.0
      T3=0.0
      FT1=0.0
      FT2=0.0
      FT3=0.0
C
      DO 307 I=1,NLINK
      DO 300 J=1,NLINK
      IF(J .LT. I)GO TO 300
      IF(J .EQ. I)THEN
        T1=T1+(U1(I)/R1(I))*TOTCONS(I,J)
```

```
      T2=T2+(U2(I)/R2(I))*TOTCONS(I,J)
      T3=T3+(U3(I)/R3(I))*TOTCONS(I,J)
      GO TO 300
     ENDIF
      IF(NCODE.NE.2)GO TO 307
      FT1=FT1+((U1(I)*U1(J))/(R1(I)*R1(J)))*TOTCONS(I,J)
      FT2=FT2+((U2(I)*U2(J))/(R2(I)*R2(J)))*TOTCONS(I,J)
      FT3=FT3+((U3(I)*U3(J))/(R3(I)*R3(J)))*TOTCONS(I,J)
 300  CONTINUE
 307  CONTINUE
C
      RSYSTL1=S1*(R0+(T1/T0)+(FT1/T0))
      RSYSTL2=S2*(R0+(T2/T0)+(FT2/T0))
      RSYSTL3=S3*(R0+(T3/T0)+(FT3/T0))
C
      TSYSTL1=(RSYSTL1-(S1*R0))/(1.0-S1)
      TSYSTL2=(RSYSTL2-(S2*R0))/(1.0-S2)
      TSYSTL3=(RSYSTL3-(S3*R0))/(1.0-S3)
C
      T1=0.0
      T2=0.0
      T3=0.0
      FT1=0.0
      FT2=0.0
      FT3=0.0
C
      DO 407 I=1,NLINK
      DO 400 J=1,NLINK
      IF(J .LT. I)GO TO 400
      IF(J .EQ. I)THEN
       T1=T1+(U1(I)/R1(I))*(T0-TOTCONS(I,J))
       T2=T2+(U2(I)/R2(I))*(T0-TOTCONS(I,J))
       T3=T3+(U3(I)/R3(I))*(T0-TOTCONS(I,J))
       GO TO 400
      ENDIF
       IF(NCODE.NE.2)GO TO 407
       FT1=FT1+((U1(I)*U1(J))/(R1(I)*R1(J)))*(T0-TOTCONS(I,J))
       FT2=FT2+((U2(I)*U2(J))/(R2(I)*R2(J)))*(T0-TOTCONS(I,J))
       FT3=FT3+((U3(I)*U3(J))/(R3(I)*R3(J)))*(T0-TOTCONS(I,J))
 400  CONTINUE
 407  CONTINUE
C
      RSYSTU1=1-(S1*(1-R0+(T1/T0)+(FT1/T0)))
      RSYSTU2=1-(S2*(1-R0+(T2/T0)+(FT2/T0)))
      RSYSTU3=1-(S3*(1-R0+(T3/T0)+(FT3/T0)))
C
      TSYSTU1=(RSYSTU1-(S1*R0))/(1.0-S1)
      TSYSTU2=(RSYSTU2-(S2*R0))/(1.0-S2)
      TSYSTU3=(RSYSTU3-(S3*R0))/(1.0-S3)
C
      WRITE(6,*)' '
      WRITE(6,*)'...........LOWER BOUND SYSTEM RELIABILITY.......... '
      WRITE(6,*)' '
      WRITE(6,*)'   R1L     R2L     R3L    '
      WRITE(6,80)RSYSTL1,RSYSTL2,RSYSTL3
      WRITE(6,*)' '
      WRITE(6,*)'   T1L     T2L     T3L'
      WRITE(6,80)TSYSTL1,TSYSTL2,TSYSTL3
      WRITE(6,*)' '
C
      WRITE(6,*)'........... UPPER BOUND SYSTEM RELIABILITY.......... '
      WRITE(6,*)' '
      WRITE(6,*)'   R1U     R2U     R3U'
      WRITE(6,80)RSYSTU1,RSYSTU2,RSYSTU3
      WRITE(6,*)' '
      WRITE(6,*)'   T1U     T2U     T3U'
      WRITE(6,80)TSYSTU1,TSYSTU2,TSYSTU3
```

```
      WRITE(6,*)'  '
C
      AVERS1=(RSYSTL1+RSYSTU1)/2
      AVERS2=(RSYSTL2+RSYSTU2)/2
      AVERS3=(RSYSTL3+RSYSTU3)/2
      AVETS1=(TSYSTL1+TSYSTU1)/2
      AVETS2=(TSYSTL2+TSYSTU2)/2
      AVETS3=(TSYSTL3+TSYSTU3)/2
C
      WRITE(6,*)'  '
      WRITE(6,*)'------------------------------------------------ '
      WRITE(6,*)'  '
      WRITE(6,*)'......... AVERAGE RELIABILITY VALUES [(RL+RU)/2]......'
      WRITE(6,*)'  AVER1      AVER2    AVER3'
      WRITE(6,80)AVERS1,AVERS2,AVERS3
      WRITE(6,*)'  '
      WRITE(6,*)'...... AVERAGE DAMAGE TOLERANCE VALUES [(TL+TU)/2]....'
      WRITE(6,*)'  AVET1      AVET2    AVET3'
      WRITE(6,80)AVETS1,AVETS2,AVETS3
C
40    FORMAT(A80)
80    FORMAT(4(2X,F8.6))
C
      CALL CLOCK@(FINISH)
      TIME=FINISH-START
      WRITE(6,*)'  '
      WRITE(6,*)'----------- TIME =',TIME,'  SECOND -----------------'
      STOP
      END
```

F3.2 INPUT FILE

```
EXAMPLE 8.3
2
13 10 .99534
.0566337
609.6
609.6
304.8
304.8
.0003048
304.8
304.8
.0003048
304.8
.0003048
.03273
0
.03179
.03197
.03167
.03262
.03044
.03274
.02713
.02715
.03102
.03042
.02816
.04847
.04148
.03604
.04149
.04609
.04565
.04607
.04564
.04564
.04504
.04305
.04255
.05087
.03244
.05087
.04138
.04920
.04131
.03457
.03457
.05085
.04598
.04667
.05227
.03260
.05191
.05190
.05189
.04841
.04841
.04884
.04982
.04855
.05086
.04128
.04915
.04124
```

.03455
.03461
.05085
.04595
.04665
.05390
.05196
.05390
.03772
.03774
.05019
.05388
.04900
.05624
.05198
.04571
.04587
.05302
.05059
.05267
.05362
.03769
.03764
.05016
.05388
.04899
.04975
.04975
.04646
.04471
.04992
.04844
.04643
.04471
.04992
.05303
.04903
.05215
.05067
.04830
.05129
1 1 1 1 1 1 1 1 1 1 1 2 2 2
372.018 177.293 374.783
203.2
152.4
152.4
203.2
152.4
152.4
152.4
152.4
203.2
203.2

F3.3 OUTPUT FILE

EXAMPLE 8.3 RESULTS

...........LOWER BOUND SYSTEM RELIABILITY..........

R1L	R2L	R3L
0.983970	0.905891	0.980132

T1L	T2L	T3L
0.910712	0.797453	0.902474

.... UPPER BOUND SYSTEM RELIABILITY..........

R1U	R2U	R3U
0.984087	0.922687	0.980535

T1U	T2U	T3U
0.911581	0.834610	0.904936

.... AVERAGE RELIABILITY VALUES [(RL+RU)/2]......

AVER1	AVER2	AVER3
0.984028	0.914289	0.980334

...... AVERAGE DAMAGE TOLERANCE VALUES [(TL+TU)/2]....

AVET1	AVET2	AVET3
0.911147	0.816032	0.903705

----------- TIME = 0.109375 SECOND -----------------

APPENDIX G

PUBLICATIONS

During the period of this study a number of papers were prepared to demonstrate the methodologies and findings of this research. This appendix presents the five refereed papers which have been published by the time of submission of this thesis.

APPENDIX G1

INVESTIGATION ON ASPECTS OF WATER CONSUMPTION AND SYSTEM LEAKAGE IN THE UK

Proceedings of Regional Conference on Water Resources Management (*WRM'95*), 28-30 August, (1995), Isfahan, Iran, S.F. Mousavi & M. Karamooz (Eds.), PP 423-432. It was also published in the I.J. of Water Resources Engineering, Vol. 3, No. 1, PP 35-46, 1995.

Proceedings of Regional Conference on Water Resources Management, Isfahan, Iran, 1995

INVESTIGATION ON ASPECTS OF WATER CONSUMPTION AND SYSTEM LEAKAGE IN THE UK

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ABSTRACT

Computer models have yet to be fully integrated into the operational management and strategic functions of many water companies. At present the only data sources available for future planning studies in some countries (e.g. UK) are potentially error prone figures for zone domestic Per Capita Consumption (PCC) and leakage. The present study's aim is to maximize the quality control on estimation of the unmetered domestic consumption, leakage and their diurnal profiles from flow surveys implemented down to the level of local supply zones . The investigation draws from analysis of data from a number of supply zones in the UK, and addresses the following issues: (i) production of realistic representative diurnal patterns for domestic demand and leakage; (ii) establishment of the sensitivity of leakage estimation to different methods of estimation (Minimum Night Flow (MNF) and Total Integrated Flow (TIF)), its correlation with average zone pressure and sensitivity of results to use of the single instantaneous MNF reading. By these studies more realistic leakage and domestic demand profiles can be obtained for incorporation in computer modelling of supply networks. Also a better quality assessment of the base data is obtained, enabling a more objective approach to the reconciliation of the base data, including flow balance 'closure' exercises, and greater opportunity for detection of the root cause of anomalies.

INTRODUCTION AND BACKGROUND

Today, vast amounts of data have been collected and collated relating to demand and consumption down to the level of the local supply zones (sometimes referred to as leakage districts, waste zones, or district metered areas (DMA's)). For these zones, field work studies have monitored daily inflows and databases provide population (property counts) and average consumption rates for all commercial water users, no widespread metering of domestic supplies is made currently in the UK. Analysis of data for each zone enables the establishment of leakage levels (usually from recorded minimum flows at night). Domestic Per Capita Consumption figures follow from some assumption regarding the diurnal variation in leakage over the 24 hour period. Figure 1 shows 'WRC' diurnal demand factors for categories of metered consumers (type 2 domestic equivalent, type 3- 10 hour working and type 4- 24 hour working for industry) and type 5- leakage which are widely used in the UK (DoE/NWC 1980).

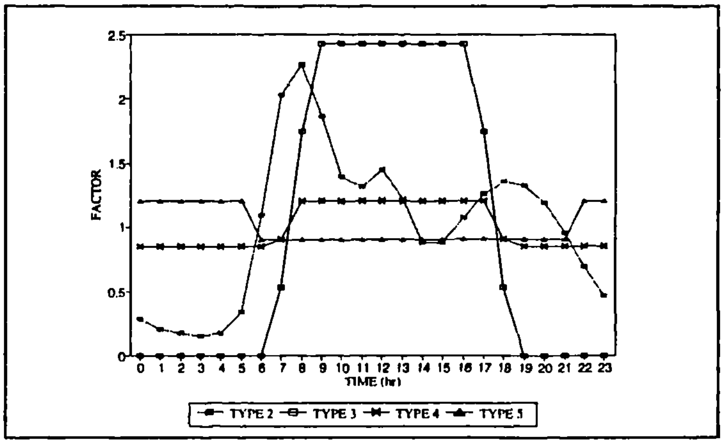


Figure 1: WRC diurnal demand factors for categories of metered consumers and leakage.

Upon completion of such studies considerable variation in the crucial elements of consumption, namely PCC and leakage, often arise between the different supply zones which make up a typical supply network. Furthermore, significant variations in the diurnal pattern of PCC result. Whilst the cause of the variations will be partially explainable in terms of differentials in the socio-economic classifications of the housing between different supply zones, other contributory factors will be inaccuracies in flow measurement (especially lower night flows since certain measuring devices experience accuracy problems) and misrepresentation of actual hydraulic configuration of the zone (i.e. unknown flows passing boundary valves between districts etc).

At present, reconciliation of such base data is largely restricted to ensuring that zonal PCC and % leakage lie with an 'acceptable' range whilst recorded flows through the network balance, when accounting for the zonal 'consumptions'. In large networks with many zones this process can prove problematic, since in many cases zones will have multiple (metered) inflows and outflows and reliance on intuitive adjustments as part of the reconciliation process can become convoluted, even after any anomalies have been resolved by follow up field work.

The ultimate aim of the study programme is to develop a more objective approach to such data reconciliation, possibly using optimisation techniques, to provide best estimates of zonal domestic consumption and leakage. This would make systematic allowance for potential error in figures for zonal metered consumptions and in the meter readings themselves and also ensure suitable correlation between diurnal leakage profiles arising and observed pressure variations in the district.

Data arising from flow evaluation exercises has now enabled the computer modelling of almost all supply networks in the UK. A strategic role for these models has been, and will be increasingly in future, to enable simulation of performance of systems for a future time horizon, when PCC and other consumption will have incremented upwards considerably, whilst leakage figures are subjected to a forecast reduction, arising from intervening mitigation initiatives. Under the recent Assessment Management Plan 2, in the UK, such models were used to ascertain deficiencies down to the level of individual supply pipes and for costing of their relining or replacement. In the absence of a more refined and systematic reconciliation of the supply zone data, as outlined above, these exercises are subject to significant risk of error and bias.

Only the initial phase of the ongoing study programme is reported here. This focuses on: (i) the variability of the PCC and its diurnal profile across the different supply zones in a single water supply network; and (ii) improved means of estimation of the diurnal patterns on leakage variation and its correlation against the observed internal zone pressures.

METHODOLOGY

The only practical way of obtaining an acceptable figure representing the level of leakage is by making an estimate of unmetered consumption (either total daily or night consumption). The inherent inaccuracies of any such estimate result in the figure obtained for leakage being somewhat approximate. It should be make clear that unaccounted for water (UFW) includes:

(i) error , in the flow measurements (positive or negative);
(ii) water which is used legitimately but which is not accounted for;
(iii) leakage

There are two methods for calculating UFW:

a. Total Integrated Flow (TIF)

The formula suggested for estimation of UFW by the TIF method is:

$$U = S - (M+AP) \tag{1}$$

where: U = Unknown or unaccounted for quantities of water
S = Sum of all inputs into the system
M = Sum of all water accounted for by measure
A = Average domestic consumption per capita of population plus an allowance for unmetered commercial consumption
P = Population

b. Minimum Night Flow (MNF)

This method assumes a small value, tiypically 1 ℓ/prop/hr[1], as legitimate overnight

[1] The results from the experimental programme have shown that the average domestic night flow is relatively small, on average less than 2 (l/prop/hr) (DoE/NWC 1980).

unmetered consumption. Unaccounted for water at minimum flow condition (UFWM) is then obtained by subtraction of the summation of total metered and unmetered consumptions from inflow at the MNF time. Because leakage is higher at night, this value is factored by (20/24) to give an average flow for whole day (L). This term (20/24) is the weighting to correct night time leakage according to stipulated profile, see Figure 1. Then:

$$\text{UFW } (\ell/\text{prop/hr}) = L \,/\, [24 \times (\text{No. of properties})] \tag{2}$$

Also Per Capita Consumption (PCC) can be obtained as follows:

$$\text{PCC } (\ell/\text{hd/day}) = [\text{TDF} - (\text{MC}+\text{L})] \,/\, [(\text{No. of properties}) \times \text{O.C.C}] \tag{3}$$

where, TDF = Total daily flow, MC = Total metered consumption, L = Daily leakage, and O.C.C. = Occupancy rate which is typically taken as about 2.75 (head/property) in the UK at present.

Of the two methods the latter is more accurate (DoE/NWC 1980). Therefore, in this research the MNF method is used firstly and the TIF method is referred to only as a comparison.

Data following from engineering reconciliation and WATNET computer model verification (WRC 1992), for a network with 31 zones has been analyzed. The minimum night flow (MNF) method has been employed to establish PCC and its diurnal pattern, taking metered profiles from WRC (Figure 1). In the calculations, leakage variation has been considered either to follow Figure 1 or to be correlated to the observed zone pressure. At various stages of the calculations some zones were eliminated from further treatment as a result of anomalous behaviour.

DISCUSSION

Figure 2 indicates the variability in PCC and leakage arising from the implementation of the standard methodology (DoE/NWC 1980). Figure 3 shows that the average domestic demand profile is relatively insensitive to variations in the diurnal profile of leakage (i.e. L-WRC = standard leakage profile; L-MODIFIED = leakage profile arising from imposition of the network average domestic profile; L-PRESSURE = leakage varying linearly with changes in zonal pressure) and closely approximates the domestic equivalent quoted by WRC type 2, see Figure 1.

The pressure dependency of leakage is well known (DoE/NWC 1980), and in Figure 4, it can be seen that the modified average leakage profile has a reasonable correlation with the average pressure profile for the zones in the network and is ,therefore, more realistic than the 'WRC' profile. The correlation between adjusted leakage profiles and pressure for individual zones are detailed by Tabesh and Burrows, 1994.

Figure 5 demonstrates the sensitivity of the computed network average profile to the value of PCC imposed under a TIF calculation in each district (itself with the network average profile taken from Figure 2 (L-MODIFIED)). The average profile obtained from the original

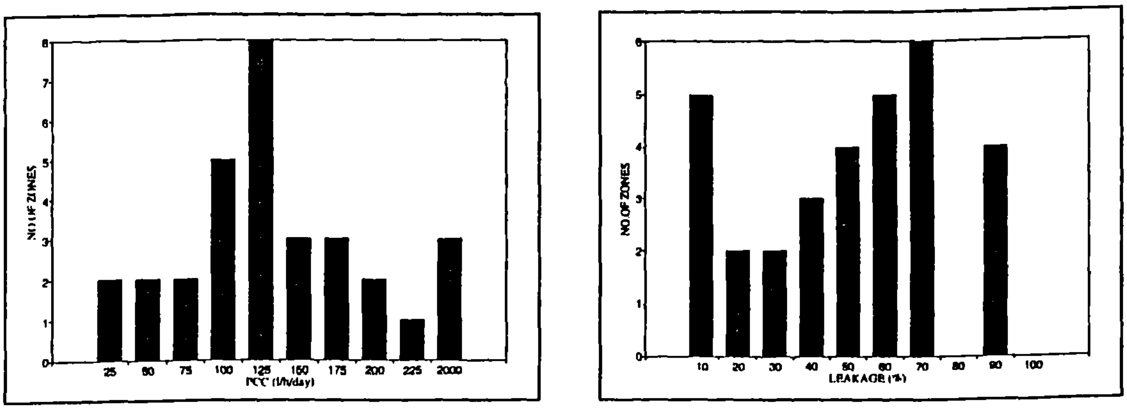


Figure 2: Variations of PCC and Leakage (%) in studied area.

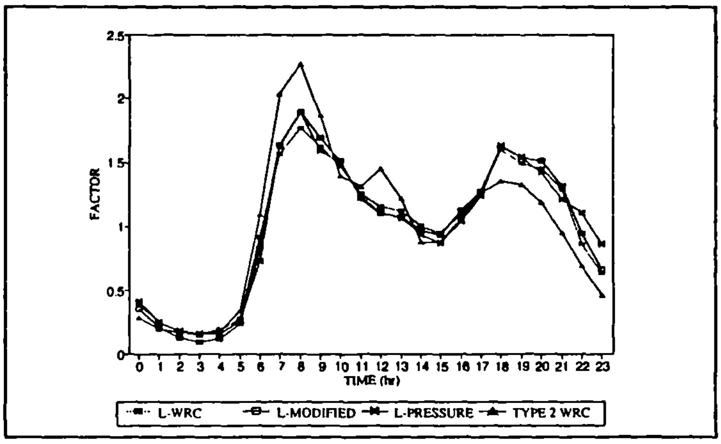


Figure 3: Domestic demand profiles based on different methods.

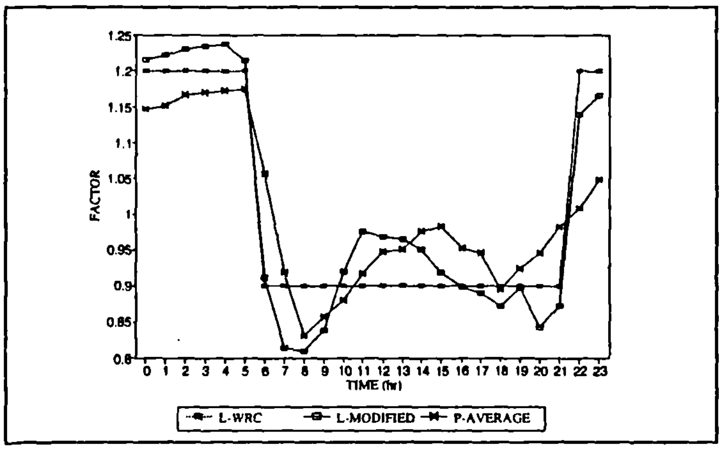


Figure 4: Leakage profiles based on different methods.

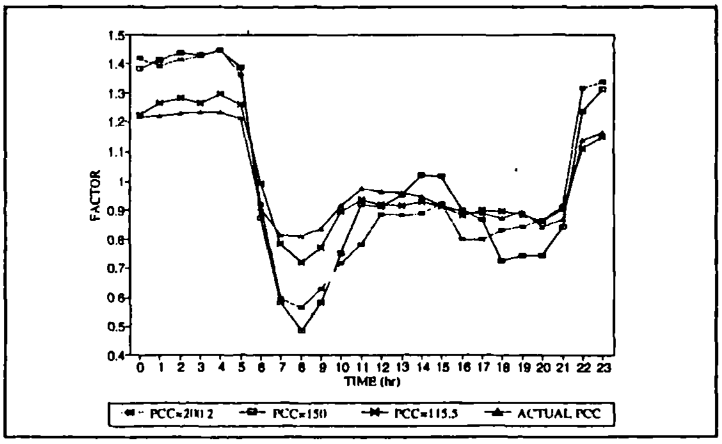


Figure 5: Leakage profiles based on different PCC values.

PCC values is shown for comparison. By virtue of this sensitivity, it would appear that adoption of incorrect PCC for a district (arising from initial MNF or TIF calculation) and using a network wide (standard) domestic profile would distort the computed leakage profile which would be shown up by compromising the correlation between leakage and zone pressure. This is discussed in more detail below.

As seen earlier, there is an inverse relationship in the evaluation exercise between PCC and leakage. Whenever PCC is higher, the leakage will be lower because domestic demand is increased (Eq 3). In the results there are some zones in which the leakage profile exhibits negative values when applying actual or network average PCC in the revised leakage profile computations. These zones, which can be divided to three different types, are shown in Table 1.

1. All applied PCC values lead to leakage profiles including negative values, like; NL013, NH031,NH033.
2. By applying the average PCC, negative values have disappeared, like; NL041, NL043, NH032.
3. With average PCC some negative leakage values are found, like; NL014, NL021, NL033, NL035, etc.

These differences can be explained as follows:
In the first group the correct value of PCC must be lower than all applied PCC values. Also it is likely that either the district inflow readings are not correct, or actual metered consumption during the fieldwork did not conform to its average rate or to the assumed diurnal profiles. Therefore, for this type, it can be concluded that $PCC_{ave} > PCC_{act} > PCC_{correct}$ where PCC_{ave} is network average, PCC_{act} is the value which is produced by using field data which is leading to questionable results here, and finally $PCC_{correct}$ is the unknown value which would lead to the correct diurnal characteristics. For the second group the actual PCC is not correct and average PCC is a better estimation. Finally in the third group it can be easily seen that $PCC_{act} < PCC_{ave}$, so creating negative leakage values on the profile.

Sensitivity of results to use of single instantaneous MNF is investigated next. This has

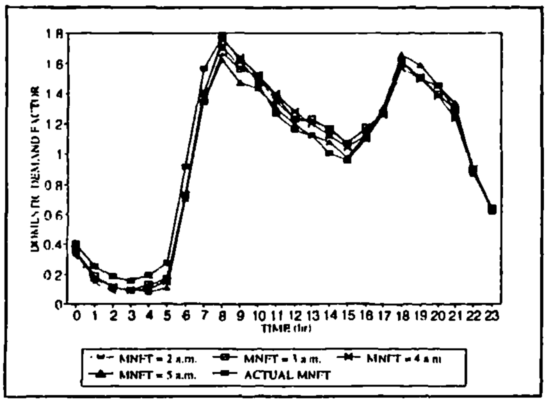
Table 1: Quality of Leakage Profiles Based on Using Different PCC Values.

WASTE DISTRICT	ACTUAL PCC l/hd/day	PCC_{act} SIGN	PCC_{ave}= 200.2 SIGN	PCC_{ave} = 150 SIGN	PCC_{ave} = 115.5 SIGN
NL013	87.0334	-	- , F	- , F	- , F
NL014	112.4987	+ , F	- , F	- , F	+ , F
NL021	167.3380	+	-	+	+
NL033	27.4414	+	- , F	- , F	- , F
NL035	85.0213	+	- , F	+	+
NL036	86.8900	+	-	+	+
NL041	1883.1116	-	+	+	+
NL042	71.7075	+	- , F	-	-
NL043	973.5787	- , F	+ , F	+ , F	+ , F
NL044	112.8902	+	-	+	+
NH012	127.0504	+	-	- , F	+ , F
NH013	75.6143	+	- , F	-	-
NH014	101.6436	+	-	+	+
NH021	119.6452	+	- , F	+	+
NH022	111.3444	+	-	+	+
NH027	108.5001	+	-	+	+
NH031	110.9661	- , F	- , F	- , F	- , F
NH032	28.6880	- , F	+ , F	+ , F	+ , F
NH033	96.8143	- , F	- , F	- , F	-

(+): Logical Pattern; (-): Including negative values; (F): Illogical Pattern

been achieved by applying the leakage evaluation to a period of 3-4 hrs around the time of minimum night flow to see if greater stability in results is achieved. It involves recalculation of the values of L, UFW, PCC, Domestic demand and leakage profiles for various times (in the vicinity of the time of MNF). Figure 6 shows the results of this assessment for domestic demand and leakage profiles. It can be seen that all shapes are similar and it can be concluded, therefore, that the MNF computation method is not highly sensitive to the precise selection of the computation time within the period 2 a.m. to 5 a.m.

Table 2 summarise the quality of domestic demand profile estimates based on several procedures to obtain the leakage profiles. It can be concluded that the three procedures applied to calculate the leakage profile appear equally acceptable in this case. The modified leakage and pressure dependent leakage profiles give a more realistic assessment for leakagecategory during the 24 hrs, however, they do not have a great effect on the average domestic demand profiles arising.



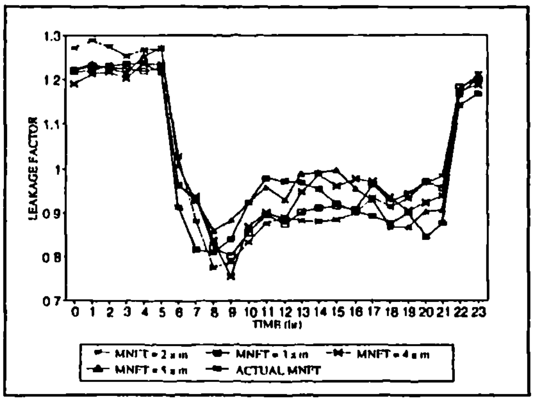


Figure 6: Domestic demand and leakage profiles based on variations of MNF times.

Table 2: Influence of Different Leakage Procedures on Quality of Domestic Demand Profiles.

Based on L-WRC	Based on L-Modified	Based on L-Pressure	No. of Zones
+	+	+	15
- , F	+	+	1
+	- , F	+	1
+	+	- , F	3
-	-	+	0
-	-	-	4

(+): Logical Pattern; (-): Including negative values; (F): Illogical Pattern

RELIABILITY ASSESSMENT OF FIELD DATA

Criteria for assessment of the reliability of the field data are now considered. It is important to assess the data collected from different waste districts and eliminate the obvious errors before starting the main procedure. This may entail the repetition of fieldwork surveys where PCC and leakage figures arisings fall outside the range of anticipated values. Table 3 shows values arising for each district after data reconciliation and follow up surveys, to eliminate anomalies. It is immediately apparent that certain districts remain anomalous. Criteria which may imposed for reliability assessment might be as follows:

1. MNF readings which occur other than at night times are not logical.
2. Impose limits on minimum and maximum acceptable values for PCC. Thus zones with higher or lower PCC values can be eliminated. (say be 50 - 200 l/h/prop)
3. Error in the flow measurements can lead to negative values in the calculation because the equilibrium equation cannot be satisfied.
4. Where negative values appear on domestic consumption or leakage profiles, the value of

Table 3: Reliability Assessment of Base Data.

ZONE NAME	PCC l/hd/day	PCC / 200.2	PCC / 150	PCC / 115.5	UFW l/pr/hr	WASTE (%)	Assessment
NL011	124.612	0.622	0.831	1.079	1.449	9.50	R
NL012	151.515	0.757	1.010	1.312	24.179	56.49	R
NL013	87.033	0.435	0.580	0.754	4.343	28.82	3 , 4
NL014	112.499	0.562	0.750	0.974	9.752	44.23	3 , 4
NL015	177.091	0.885	1.181	1.533	8.850	30.35	R
NL021	167.338	0.836	1.116	1.449	20.312	50.53	R
NL022	141.710	0.708	0.945	1.227	72.244	81.00	R
NL031	193.919	0.969	1.293	1.679	82.962	84.02	5
NL033	27.441	0.137	0.183	0.238	9.290	61.16	R
NL035	85.021	0.425	0.569	0.736	22.950	61.27	R
NL036	86.890	0.434	0.579	0.752	16.208	61.18	R
NL041	1883.11	9.406	12.554	16.303	318.581	81.61	1,2,5,6
NL042	71.708	0.358	0.478	0.621	11.412	50.41	R
NL043	973.579	4.863	6.491	8.428	10.802	9.46	2,6
NL044	112.890	0.564	0.753	0.977	18.998	45.67	R
NH011	142.383	0.711	0.949	1.233	18.507	55.75	R
NH012	127.050	0.635	0.847	1.100	19.160	63.17	R
NH013	75.614	0.378	0.504	0.655	6.943	36.69	R
NH014	101.644	0.508	0.678	0.880	14.244	51.54	R
NH021	119.645	0.598	0.798	1.036	10.125	44.66	R
NH022	111.344	0.556	0.742	0.964	12.345	48.52	3,4
NH023	222.391	1.111	1.483	1.925	49.883	67.85	1,2,5,6
NH024	330.168	1.649	2.201	2.859	85.229	84.34	2,5,6
NH025	0.000	0.000	0.000	0.000	1152.52	5.86	2,3,4,6
NH026	0.000	0.000	0.000	0.000	2237.09	5.86	2,3,4,6
NH027	108.500	0.542	0.723	0.939	27.409	68.24	R
NH031	110.966	0.554	0.740	0.961	4.996	27.82	3,4
NH032	28.688	0.143	0.191	0.248	7.685	36.31	2,3,4
NH033	96.814	0.484	0.645	0.838	2.629	17.17	R
NH034	68.948	0.344	0.460	0.597	1.105	11.96	R
NT031	165.821	0.8283	1.106	1.436	1.620	7.87	R

Note: (R) signifies the reliability of zone's data and numbers represent the probable cause of data anomaly.

demand factors chosen for that hour and for the other consumption types are greater than the true values.

5. Districts with apparently high value of UFW (or % waste) are subject to potential anomaly (say over 50% or 60%).
6. It can be seen in Table 3 that when $0.75 < PCC_{nct} / PCC_{ave} < 1.25$, the majority of zones do not include negative values in the leakage profile. Thus a tolerance of (+/-) 30% for this ratio would seem to be acceptable in the case of this supply network.
7. Several of the terms in the applied formulas are likely to be subject to seasonal variation and socio-economic factors. These effects should be anticipated in the data evaluation exercise.

The last column of Table 3 includes the reliability assessment of the district data based on the above criteria, which leads to a means of 'flagging' the likely presence of anomaly in the data for that district.

CONCLUSION

By studies such as those summarised here it is expected that network modelling methodologies will be improved by: a) Production of more realistic leakage profiles for normal application in modelling; b) Reduced risk of bias in model performance under future operation scenarios by better quality assessment of base data and more reliable allocation between unmetered domestic consumption and leakage; c) Identifying supply zones for which the base data appears anomalous, rendering the need for further field work studies. Also it can be concluded that appraisal of the computed pattern of PCC and the correlation between leakage and average zone pressures, should be added to the routine procedure for reconciliation of zonal demand/consumption data sets.

ACKNOWLEDGMENT

Appreciation is extended to the Ministry of Culture and Higher Education of the Islamic Republic of IRAN for financial support of the first author. The authors also are indebted to George Crowder Associates who made available the data sets upon which this study has been made, as part of ongoing UK Department of Trade and Industry Teaching Company Project with the University of Liverpool.

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APPENDIX G2

WATER CONSUMPTION AND NETWORK LEAKAGE EVALUATION USING A BEST PARAMETER ESTIMATION TECHNIQUE

Hydroinformatics' 96, Proceeding of the Second International Conference on Hydroinformatics, Zurich, Switzerland, 9-13 September, (1996), A. Muller (Ed.), Vol. 1, A.A. Balkema Publishers, PP 387-394.

Hydroinformatics'96, Müller (ed.) © 1996 Balkema, Rotterdam. ISBN 90 5410 852 5

Water consumption and network leakage evaluation using a best parameter estimation technique

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ABSTRACT: Vast amounts of data have been collected in recent years for use in calibration and verification of computer network models of water supply systems, now completed for most systems in the UK. Upon completion of such studies considerable variability between adjacent zones often arises in crucial elements of demand, such as levels of per capita consumption (*PCC*) and leakage, as well as their diurnal patterns. Unfortunately, with application of these potentially error prone figures, considerable bias is likely to arise which may give unrealistic predictions of the future hydraulic performance of the system, when differential incrementation of individual components of demand (including leakage) is necessitated. This paper presents an optimized approach for best estimation of parameters based on an error minimization procedure, whilst applying realistic variability constraints and accuracy tolerances on the relevant data inputs. A computer code is generated and the model is integrated to a database information source from a real network in the UK. Using this case study, a systematic approach to demand evaluation and leakage computation is demonstrated. Operating at individual supply zone level, this enables flow reconciliation and hydraulic performance appraisal over the whole network, so identifying serious anomalies, as a focus to the need for the checking of base data, field measurements and/or perceived pipe system connectivity.

INTRODUCTION

Computer network models of the hydraulics of the pipe system are widely used in operational management and strategic planning of water supply networks. In the UK the data sources which are available as input to such models are potentially error prone, especially in the unmetered figures for Per Capita Consumption (*PCC*) and leakage.

Because it is not possible to model all details of real water distribution systems, a reduced network including major links, nodes, and the other components like pumps, valves, etc is normally constructed. Also, complete details of all individual demands which really exist cannot actually be modelled, either through limitations in the means of specification built into the modelling system or, most significantly by the absence of a practicable means of monitoring the consumption patterns of thousands of consumers. In the usual modelling approach, some typical demand types can be chosen and existing demands are forced to follow the most reasonable of these consumption categories ('types'). For example in the WATNET model which is chosen in this investigation, as mentioned by Tabesh and Burrows (1994, 1995), five types of demand are considered. Type 2 is metered consumption with an equivalent domestic demand profile, types 3 and 4 are 10 and 24 hrs profiles for trade and industrial activities (which are metered). Type 1 (domestic demand) and Type 5 (leakage) are unmetered consumption categories. Figure 1 shows the diurnal variations in types 2 to 5 from standard WRC profiles.

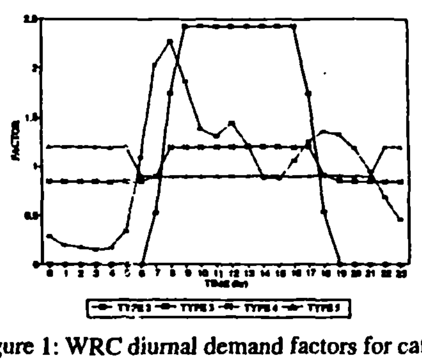

Figure 1: WRC diurnal demand factors for categories of metered consumers and leakage (WRC 1992).

Vast amounts of data have been collected in recent years for use in calibration and verification of network models, now completed for most systems in the UK, in response to the Asset Management Planning (AMP2) requirements of the Office of Water (OFWAT) in 1994.

Methods for demand allocation, such as the District Metering Method, relate demand and consumption down to the level of the local supply zones of perhaps only several thousand properties. Upon completion of such studies considerable variability between adjacent zones often arises in crucial elements, such as levels of *PCC* and leakage, as well as their diurnal patterns. The causes of these variations, besides differences in socio-economic classifications in the case of *PCC* and different pipe age or condition in the case of leakage, will be inaccuracies in flow measurement, imprecision in metered consumption figures and misrepresentation of actual hydraulic configuration of the zone.

Unfortunately, with application of these potentially error prone figures, considerable bias is likely to arise which may cause unrealistic predictions of the future hydraulic performance of the system, when differential incrementation of individual components of demand (including leakage) is necessitated.

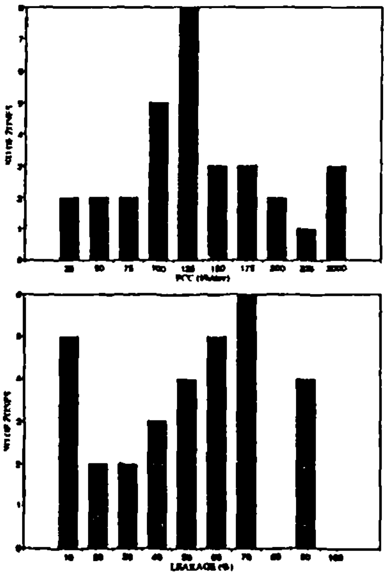

Figure 2: variations of *PCC* and Leakage (%) in the studied area.

Real data which has been used in this investigation is taken from a supply system serving a medium sized town in an industrialized conurbation in the UK. Figure 2 indicates the variability of *PCC* and leakage arising from implementation of the standard methodology. As can be seen, the results of the existing methodology include anomalous data in some zones. It is obvious that the zones in which *PCC* values are between 0-50 or 225-2000 (lit/hd/day) and also some with high percentage of leakage are questionable and cannot be accepted.

The authors have earlier investigated some alternative procedures to maximize the quality control on the elemental components of demand (*PCC*, leakage and their diurnal profiles) arising from data reconciliation and to identify zones producing apparently anomalous figures (Tabesh and Burrows 1994, 1995).

This paper presents the development of an optimized approach for best estimation of parameters based on an error minimization procedure, whilst applying realistic variability constraints and accuracy tolerances on the relevant data inputs.

METHODOLOGY

Each network can be divided to individual districts (zones) in which the inflow and outflow are metered. Some consumptions in each zone are also metered (mean daily flow rates for type 2, 3, 4 are taken from customers accounts data bases collected by the water companies). In the flow balance equation for each zone, there are two unknown parameters (i.e. the unmetered domestic type 1 and type 5 leakage figures). The most accurate method of demand reconciliation used in the UK is the Minimum Night Flow (MNF) approach (TWG/NWC 1980). This assumes a small value, typically 1 ℓit/prop/hr, as legitimate overnight domestic unmetered consumption (*LOUCP*). Unaccounted For Water at the minimum flow condition (*UFWM*) is then obtained by subtraction of the summation of total metered and unmetered consumptions from inflow at the MNF time. Because leakage is higher at night, this value is conventionally factored by (20/24) to give an average flow for whole day consistent with the simple 'block' type leakage profile adopted, see figure 1. Unaccounted for water (*UFW*) and Per Capita Consumption (*PCC*) can then be obtained as follows:

UFW (ℓ/prop/hr) = L / [24 x (No. of properties)] (1)

PCC (ℓ/hd/day) = [TDF - (MC+L)] / [(No. of properties) x $O.C.C$] (2)

where, TDF = Total daily flow, MC = Total metered consumption, L = Daily leakage, and $O.C.C.$ = Occupancy rate which is typically taken as about

2.75 (head/property) in the UK at present. For details see Tabesh and Burrows (1994, 1995).

It is possible to improve on this separation of unmetered elements of consumption and in so doing also provide suitable measure of diurnal variations. To determine values of leakage and *PCC* the following extended procedure has been carried out.

a) *Pressure Dependent Leakage*

It is well known that there is a correlation between the levels of leakage and mains pressure in water supply distribution networks (TWG/NWC 1980 and EOC/WRC 1994). The effect of pressure on the rate of leakage, which perhaps has the greatest and most immediate effect on the total leakage, is common to all systems. From field data, TWG/NWC (1980) developed a relationship to show the effect of pressure on leakage for water supply systems. A nearly linear variation between a leakage index and average zone night pressure (*AZNP*) was found but which beyond a certain value of *AZNP* changes to an exponential function. It expresses a higher sensitivity of leakage to higher zone pressure values. The average zone night pressure is defined as the mean pressure occurring within the system at night taking account of variations in ground level and any hydraulic friction losses across the zone.

A high degree of correlation between diurnal profiles of leakage and average zone pressure is, therefore, expected. Pressure in each zone can be established in different ways (TWG/NWC 1980): - (i) the mean of the highest and lowest pressure values which are measured at some sensitive and critical nodes in the district; (ii) the mean of pressure values for each node by output from a calibrated network model with measured pressure values only at critical nodes.

Here leakage values are determined by the Minimum Night Flow (MNF) method and its diurnal variation can be imposed by correlation with average zonal pressure, as an improvement on the standard WRC type 5 profile (figure 1). The procedure is described below:

1) Subtract the sum of metered consumption from net inflow of each zone

Unmetered consumption =

Net inflow - $\Sigma_{type\ 2,3,4}$ Metered consumption (3)

2) Find the domestic demand consumption at MNF time

$(\text{Domestic demand})_{MNFT}$ =

LOUCP x No. of properties (4)

3) Calculate the leakage value at MNF time

$L_{MNFT} = (\text{Net inflow})_{MNFT} - (\Sigma\ \text{Type } 2,3,4)_{MNFT} - (\text{Domestic demand})_{MNFT}$ (5)

4) Relate leakage at each hour to value of average zone pressure at the same time

$L_t = L_{MNFT} \times (P_t / P_{MNFT})$ (6)

where, P_t and P_{MNFT} are the average zone pressure values at each time and at MNF time respectively.

5) Calculate the domestic demand profile by deducting these leakage figures together with corresponding hourly rates of types 2, 3 and 4 consumption from the net zonal inflows (Tabesh and Burrows 1994, 1995).

b) *Average Network PCC*

As mentioned earlier, the existing methodology will produce anomalous data in some zones which leads to unrealistic values of *PCC*. Accepting that variations arise from socio-economic differences , it is expected that zonal *PCC* values of a network will have a reasonable variation. Nevertheless, an average zonal *PCC* value for the whole network can be used as the basis for the type 1 domestic demand specification following the approach below. The diurnal profile used to distribute the total domestic demand through a 24 hr day can be similarly produced by averaging all non negative domestic demand profiles which are produced by the above pressure dependent leakage procedure. Details of this are explained by Tabesh and Burrows (1994, 1995).

c) *Optimization Procedure*

This methodology systematically investigates the residual errors in zonal flow balance evaluation when the individual elements are constrained within the chosen validity bands. This is initially investigated on the basis of an assumed uniform probability across the validity bands.

To measure uncertainty in the water system's elemental consumptions, leakage and hydraulic performance, a value of residual can be defined as follows for each zone,

$R_t = \text{Net inflow}_t - (T_1 + T_2 + T_3 + T_4 + T_5)_t$ (7)

where R_t is the residual at each time, T_1 to T_5 are the five types of consumption in the WATNET modelling procedures, and subscript t (from 0 to 23) refers to time (hrs.) over daily cycle. For each elemental consumption type the following percentage errors are considered (at this stage intuitively) to encompass the range of uncertainty or inaccuracy in the base data:

1) +/- 5% error for all flow measurements (i.e. inflows/outflows to the zone) to cover the range of instrument accuracies.

2) +/- 5% error for all metered consumer elements, type 2 to 4, obtained from consumer accounts records. This range of variation is perceived as representing possible day to day variability in metered consumption as well as seasonal drifting. These error margins are necessary since it is impracticable to monitor all metered consumers during a typical field work study.

3) +/- 10% error for values of pressure dependent leakage based on the MNF method. This range is incorporated to account for the uncertainty in the relationship between leakage and zone pressure.

4) +/- 25% variation for values of domestic unmetered consumption (type 1). This makes allowance for the fact that *PCC* may be expected to vary, to some degree, from zone to zone, partly as a result of socio-economic factors. A later refinement would be to build in an explicit link between *PCC* and socio-economic make up, possibly through the ACORN categorisation system (EOC/WRC 1994).

The optimization problem has been considered in three steps and provisional findings only from a preliminary implementation are presented herein.

Problem 1: (zone by zone analysis)

The objective function is

$$\text{Minimize} \quad F = \sum_{t=1}^{24} | R_t |^2 \tag{8}$$

$$R_t = \sum_{j=1}^{7} a_{t,j} \cdot x_j \tag{9}$$

subject to:

$$0.95 \le x_1 \le 1.05 \tag{10}$$

$$0.75 \le x_2 \le 1.25 \tag{11}$$

$$0.95 \le x_3 , x_4 , x_5 \le 1.05 \tag{12}$$

$$0.90 \le x_6 \le 1.10 \tag{13}$$

$$0.00 \le x_7 \le 1.1 \times T_{5,MNFT} \tag{14}$$

$$\sum_{j=1}^{7} a_{MNFT,j} \cdot x_j = LOUC \tag{15}$$

in which:

$a_{1,t} = Q_{0t}$ (Net inflow)
$a_{2,t} = -T_{1t}$ (Domestic demand)
$a_{3,t} = -T_{2t}$ (Small trade consumptions)
$a_{4,t} = -T_{3t}$ (10 hrs. industrial activities)
$a_{5,t} = -T_{4t}$ (24 hrs. industrial activities)
$a_{6,t} = -T_{5t}$ (Leakage)
$a_{7,t} = 0$

x_7 represents the optimum leakage value at MNFT
MNFT = minimum night flow time
LOUC = legitimate over night consumption
= 1 lit/hr/prop x No. properties (16)

Finally $a_{2,MNFT}$ and $a_{6,MNFT}$ are set zero in the MNFT constraint. (In some zones with no domestic demand (T_1) or with MNF occurring during the day rather than night times, the MNFT constraint (Eq. 15) is eliminated). To calculate the optimum values of flows and consumptions, a program has been developed using a least square minimization methodology drawing on standard routine E04NCF of the NAG library (NAG 1991).

Results for 29 individual zones are shown in Table 1, which includes values of optimal variables and objective functions. Sensitivity of results was also investigated. Simultaneous change in the tolerance ranges of the constraints (increases up to 100%) produced very little change in values of objective functions or optimal variables.

Table 1 shows that the values of the objective function for some zones with high and unreasonable original *PCC* (e.g. NL31, NL41, NL43, NH23, NH24) remain high and reconciliation is incomplete. In these zones can be seen that the optimal value of x_1 has its extreme lower bound and the other variables have got their extreme upper bound limits. This suggests that the values of net inflow in such zones may be excessive. One investigation showed that if the possible error for net inflow was allowed to be more than 5% (i.e from 10% to 30%), the value of the objective function would be decreased greatly. It can be concluded that in such zones values of net inflow are probably the main source of anomaly. This result supports the conclusion of Tabesh and Burrows (1995) that zones with values of *PCC* outside a certain range (say 50 - 200 lit/hd/day) are likely to be anomalous.

Change in *LOUCP* from 1.0 to 1.7 lit/prop/hr (suggested by EOC/WRC 1994) was found (Table 1) to have little effect and minimal residual for individual zones was found to switch apparently arbitrarily between the values of 1.0 and 1.7.

Problem 1 was contrived for optimization of flow residuals over the 24 hrs whilst satisfying the MNF method computation separately. To see the effect of duplication at MNFT, the problem has also been solved using only the 23 (none MNFT) hrs and the balance equation at MNFT just considered as a constraint. Again, this resulted in little change to optimal solutions.

To see the effect of the optimization on values of *PCC*, new *PCC* estimates have been calculated based on the optimal values of x_2 for all individual zones. Table 1 shows that despite there being a wide range of differences, a set of intuitively reasonable values are obtained from the optimization procedure. It shows that the optimum average *PCC* for all 29 zones is 103.56 (lit/hd/day) which is 10% less than the average *PCC* values originally computed.

Table 1: Results of problem 1 for optimal values of variables and objective functions.

Zone Name	x_1	x_2	x_3	x_4	x_5	x_6	OBJ-F (*LOUCP* = 1)	OBJ-F (*LOUCP* = 1.7)	Original *PCC*	Optimal *PCC*
NL11	0.95	1058	1.05	0	0	0.9	2.408	2.462	124.61	122.25
NL12	0.95	1.25	1.05	1.02	0	0.9	1.709	1.735	151.52	144.41
NL13	0.965	0.75	0.95	0	0	0.9	0.842	0.857	87.04	86.63
NL14	0.95	0.96	0	0	0	0.9	5.828	5.663	112.50	110.91
NL15	0.95	1.25	1.05	0	0	1.02	5.013	5.692	177.09	143.93
NL21	0.95	1.167	0.95	1.05	0	1.035	24.035	23.391	167.34	134.82
NL22	0.95	1.241	0.95	0.95	0	0.939	0.333	0.935	141.71	143.34
NL31	0.95	1.25	1.05	1.05	1.05	1.1	484.460	497.960	193.92	144.39
NL33	1.05	0.75	0.95	0.95	0	0.9	21.499	18.097	27.44	87.66
NL35	0.95	0.75	0.95	0.95	0	0.907	0.722	0.771	85.02	86.61
NL36	0.988	0.816	0	1.05	0	0.9	0.483	0.412	86.89	94.24
NL41	0.95	1.25	1.05	0	1.05	1.1	115.560	116.270	1883.11	151.99
NL42	1.05	0.818	0.95	0.95	0	0.92	1.634	1.667	71.71	94.54
NL43	0.95	1.25	1.05	1.05	0	1.1	606.280	613.600	973.58	144.39
NL44	0.95	0.934	1.05	1.05	1.05	0.9	7.329	7.153	112.89	122.81
NH11	1.033	1.211	1.05	1.05	0	0.9	4.435	4.151	142.38	139.84
NH13	1.05	0.75	0.95	0.95	0	0.9	1.703	1.183	75.61	86.64
NH14	1.012	0.75	0.95	0.95	0	0.9	0.938	0.848	101.64	86.64
NH21	1.05	1.02	1.05	1.05	0	0.9	2.316	1.934	119.65	117.76
NH22	0.987	0.88	1.05	0	0	0.9	1.251	1.033	111.34	101.76
NH23	0.95	1.25	1.05	0.95	0	1.1	415.620	431.860	222.39	144.36
NH24	0.95	0.75	1.05	1.05	1.05	0.953	4.478	4.462	330.17	86.58
NH25	0.973	0	0.95	0	0	0.9	67.990	67.990	0.00	0.00
NH26	0.95	0	1.05	0	0	1.1	521.530	521.530	0.00	0.00
NH27	0.996	0.75	0.95	1.05	0	0.9	1.799	1.617	108.50	86.61
NH31	0.95	0.942	1.05	0	0.95	0.9	8.246	8.841	110.97	108.76
NH33	0.95	0.814	1.05	0.95	0	0.9	0.848	0.768	96.81	94.03
NH34	1.05	0.75	0	0.95	0	0.9	3.144	2.592	68.95	86.64
NT31	0.95	1.25	1.05	0	0	1.1	14.631	17.462	165.82	144.37

Problem 1 has determined the optimum values of net inflow and consumptions for each individual zone consistent with the imposed constraints. In reality, however, zones are connected together and, therefore, it is necessary also to view the problem in respect of inter-dependent groups of zones.

Problem 2: (combination of individual zones)

$$Minimize\ \ F = \sum_{t=1}^{24} | \sum_{i=1}^{NZ} R_{t,i} |^2 \tag{17}$$

$$R_{t,i} = \sum_{j=1}^{7} a_{t,i,j} \cdot x_{ij} \tag{18}$$

subject to:

$0.95 \leq x_{1i} \leq 1.05$ (19)

$0.75 \leq x_{2i} \leq 1.10$ (20)

$0.95 \leq x_{3i}\ , x_{4i}\ , x_{5i} \leq 1.05$ (21)

$0.90 \leq x_{6i} \leq 1.1$ (22)

$0.00 \leq x_{7i} \leq 1.1\ \text{x}\ T_{5,MNFT}$ (23)

$$\sum_{j=1}^{7} a_{MNFT,i,j} \cdot x_{ij} = LOUC_i \tag{24}$$

in which NZ = No. of zones in each group.

It can be seen that the formulation of problem 2 is similar to problem 1, but extended so that a combination of zones is added to the objective function. From a study of network connectivity the test system has been divided to 5 groups of zones. The program is then applied to each group. Table 2 shows a comparison between values of objective functions for each group from problem 2 and individual zones from problem 1. It can be seen that the values of objective function for most groups is now less than summation of those from individual zones. Solution to problem 2, therefore, gives a better representation of behaviour of the network than problem 1.

The approach has been extended by considering the zones in only two larger groups. The optimal variables for this, however, show little difference to the 5 group solution. Furthermore, anomalous zones with high residual flow (objective function) simply convey this into the objective function for the group without any adjustment. This demonstrates a need for imposition of other physical constraints on the problem.

Misrepresentation of the actual hydraulic configuration of network.

In the forgoing analysis values of measured flow elements have been considered to be reliable with +/- 5% error. Normally, in the process of data collection in the field, the values of the net inflow is obtained from differences between the total inflows to, and outflows from the zone. It is assumed that all the other connections to adjacent zones are cut off by the closed status of boundary valves. In reality it is possible that some of these valves may not be closed completely and other connections or valves may have been overlooked or forgotten and, consequently, water could be passing from them. Obviously, this would disturb the balance equation in these zones and the methodology can be extended to consider the possibility of unknown flow passing between adjacent zones taking account of hydraulic head factors.

Problem 3: (Network wide optimization considering hydraulic characteristics of the system)

$$Minimize\ \ F = \sum_{t=1}^{24} | \sum_{i=1}^{NZ} R_{t,i} |^2 \tag{17}$$

$$R_{t,i} = \sum_{j=1}^{7} a_{t,i,j} \cdot x_{ij} + \sum_{m=1}^{K} (H_{t,i}-H_{t,m}) \cdot x_{i,m} \tag{25}$$

subject to:

$0.95 \leq x_{1i} \leq 1.05$

(19)

$0.75 \leq x_{2i} \leq 1.10$ (20)

$0.95 \leq x_{3i}\ , x_{4i}\ , x_{5i} \leq 1.05$ (21)

$0.90 \leq x_{6i} \leq 1.1$ (22)

$0.00 \leq x_{7i} \leq 1.1\ \text{x}\ T_{5,MNFT}$ (23)

$$0 \leq x_{i,m} \leq \frac{(Ave.\ Net\ inflow)_i}{|\Delta \bar{H}_{i,m}|} \tag{26}$$

$$\sum_{j=1}^{7} a_{MNFT,i,j} \cdot x_{ij} + \sum_{m=1}^{K}(H_{MNFT,i} - H_{MNFT,m}) \cdot x_{i,m} = LOUC_i \tag{27}$$

$$x_{i,m} - x_{m,i} = 0 \tag{28}$$

in which:

$m = 1, \ldots\ldots, k \quad \forall\ i$ (adjacent zones with zone i)

Table 2: Comparison of objective functions from problem 1 and problem 2.

Group	OBJ-F	Zone	OBJ-F
1	72.84	NL11	2.408
		NL12	1.709
		NL13	0.842
		NL14	5.828
		NL15	5.013
		NT31	14.630
2	18.68	NL21	24.035
		NL22	0.333
		NH13	1.703
		NH14	0.938
3	245.26	NL31	484.460
		NL33	21.499
		NL36	0.483
		NL35	0.722
		NL42	1.634
		NL43	606.280
		NL44	7.329
		NL41	115.560
4	229.31	NH11	4.435
		NH31	8.246
		NH33	0.848
		NH34	3.144
		NH23	415.620
5	245.237	NH21	2.316
		NH22	1.251
		NH24	4.478
		NH27	1.799
		NH25	67.990
		NH26	521.530
		NH23	415.620

Table 3: Optimum values of $x_{l,m}$ and $\Delta\bar{H}_{l,m}$ for adjacent zones from problem 3.

Adjacent Zones	Ave $\Delta H_{l,m}$ (m)	$x_{l,m}$ (lit/s/m)
L11 - T31	53.16	0.0006
L13 - L12	0.65	0.0
L12 - L14	41.30	0.0049
L15 - L12	19.51	0.0
L12 - T31	37.66	0.0
L13 - L14	41.95	0.0
L15 - T31	57.16	0.0
L21 - L22	1.75	2.3728
H14 - L21	71.27	0.0144
H24 - L22	75.67	0.0
H13 - H14	1.59	0.0671
L31 - L33	3.64	0.0
L31 - L42	2.13	0.0
L36 - L31	0.52	0.7978
L36 - L42	2.65	0.0
L36 - L35	0.94	0.0
L41 - L42	2.31	2.3489
L43 - L42	1.77	0.0
L43 - L44	0.24	0.1989
L41 - L43	0.55	0.0
L41 - L44	0.78	12.61
H11 - H31	4.90	0.0
H33 - H31	0.88	0.0
H23 - H31	2.68	0.0
H33 - H34	32.80	0.0005
H23 - H34	34.61	0.0
H21 - H22	19.98	0.0128
H23 - H21	8.59	0.0
H23 - H22	28.57	0.0
H23 - H27	33.56	0.1435
H25 - H23	3.66	1.4918
H26 - H23	2.35	0.0

$H_{t,i}$ = Average total zone heads of zone i at time t.
$x_{i,m} = Q_{i,m} / \Delta\bar{H}_{i,m}$ is a variable which represents values of unknown passing flow ($Q_{i,m}$) between two adjacent zones *i* and *m*.
$\Delta\bar{H}_{i,m}$ = Daily average of total head difference between two adjacent zones i and m.

This methodology allows consideration of any possible flow passing between two adjacent zones. It is assumed that there is a linear relationship between diurnal variations of these passing flows and variations of total average head differences between each pair of adjacent zones. Values of total heads within each zone have been produced by a 'WATNET' hydraulic flow model, calibrated and verified from field data, then average total head for each zone has been calculated. $Q_{i,m}$ representing the average daily passing flow has so far been restricted intuitively to not exceed the average measured net inflow through zone i. Therefore, a set of new terms of the form ($\Delta H_{i,m,t} \cdot x_{i,m}$) are added to the objective function and relevant constraints incorporated to identify probable passing flows. The formulation allows flow to go only to the adjacent zones with lower average total head.

So far the problem has been solved for groups of zones. In the normal situation, where the systems are free from anomalies, it would be expected that terms $x_{i,m}$ are small or zero. Results are presented in Table 3. This suggests that just a few adjacent zones in each group may be subject to unexpected passing flow. It can be tentatively concluded that when values of the objective function are reasonably low, problem 3 leads to optimum values of variables and estimation of passing flows consistent with minimization of the errors. When values of the objective function are unreasonably high, however, the performance of the system still cannot be reconciled and here the scale of the anomalies are such that repeated fieldwork studies are called for.

SUMMARY AND CONCLUSION

A systematic approach to demand evaluation and leakage computation is demonstrated. A best parameter estimation technique, through an optimization procedure, is used for flow reconciliation. To minimize the effects of unmetered consumption elements, pressure dependent leakage and average network *PCC* values are introduced as a better representation for types 1 and type 5 consumption. A three stage error minimization approach is applied both in individual zones and groups of adjacent zones. For individual zones without unknown boundary connections, problem 1 produces the optimal values for elements of demand and flow. Considering inter-connection of zones in a network, problem 3 gives a better expression for hydraulic behaviour of the system, producing the optimal values of flow, demands and identifying possible unknown flows passing between each pair of adjacent zones. The procedure facilitates systematic flow reconciliation and hydraulic performance appraisal over the whole network, so identifying serious anomalies and focusing needs for the checking of base data, field measurements and/or perceived pipe system connectivity.

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APPENDIX G3

THE BASIS OF THE SOURCE HEAD METHOD OF CALCULATING DISTRIBUTION NETWORK RELIABILITY

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The basis of the source head method of calculating distribution network reliability

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Abstract

This paper formalises the the derivation of the hitherto intuitive source head method of calculating water distribution network reliability. The derivation is inspired by recent developments in pressure-driven network analysis. The reliability analysis uses a parameter referred to as the required source head for full demand satisfaction together with the widely accepted definition of reliability as the expectation of the ratio of available flow to required flow. The methodology is conceptually simple and straightforward to implement. The calculated reliability values are probabilistic in that the random nature of pipe failures is accounted for. The computational efficiency of the formulation is very high, and this is demonstrated using a sample network. The results suggest that if reliability is calculated as the average of the upper and lower bounds, large savings in computational effort can be made.

1 Introduction

There are many water distribution network design test problems in the literature, and many of the well-known test problems are based on small networks often having a single source. Also, reliability and damage tolerance have in recent years been firmly established as important distribution network design parameters. It is widely accepted, however, that reliability is difficult both to define and calculate in the context of water distribution. Existing methodologies that attempt to calculate reliability realistically use Monte Carlo simulation (Bao and Mays, 1990; Wagner et al., 1988), minimum cut sets (Su et al., 1987; Yang et al., 1996) or advanced mathematical programming methods (Fujiwara et al., 1990, 1991). These approaches are difficult to implement and/or make high demands on computational resources (Jacobs and Goulter, 1991; Tanyimboh and Templeman, 1995; Walters and Knezevic, 1989). Furthermore, existing methodologies generally do not address the issue of the strong interdependencies between the reliabilities of demand nodes and, as such, produce questionable network reliability values (Tanyimboh and Tabesh, 1997; Fujiwara and Ganesharajah, 1993). There is, therefore, an immediate need for a realistic, easily-implemented and fast methodology that can be used to asssess the reliability of simple networks.

The above considerations were the driving force behind the development of this new method of calculating reliability. This paper aims to place the source head method on a firm footing by formalising its derivation and so obviating the need for intuition (Tanyimboh, 1993). The present formulation draws from pressure-dependent network analysis (Gupta and Bhave, 1996). It should be noted, however, that reliability values are calculated herein using demand-driven analysis. Existing procedures for pressure-dependent network analysis are cumbersome and, in general, slower (Gupta and Bhave, 1996; Chandapillai, 1991; Bhave, 1981). The formulation herein is more general than the original (Tanyimboh and Templeman, 1995) in several respects including the capability of handling failure cases involving demand node isolation. The efficacy of the method, including computational efficiency, is demonstrated using a distribution network.

2 Reliability Analysis

The quantity of water which a distribution network can supply at adequate pressure is one of the principal factors determining the reliability of the system. Therefore, the relationship between actual nodal outflows and pressures should be incorporated in any realistic network reliability measure. This relationship can be expressed as (Chandapillai, 1991)

$$H_j = H_j^{min} + K_j q_j^{n_j} \tag{1}$$

in which: H_j represents the total head at demand node j. H_j^{min} represents the pressure head below which service at demand node j is unsatisfactory and therefore unacceptable (Twort et al., 1994). K_j is a resistance coefficient for node j and its value depends on the characteristics of the service connection at that node. q_j is the outflow at node j. n_j is an exponent, often taken as 2 (Wagner et al., 1988; Gupta and Bhave, 1996).

It has been shown that a similar approach to that described above can be applied to small portions of distribution networks (Gupta and Bhave, 1996), for example, housing estates, small communities, etc. with a single input point from the main distribution system. Using the pressure-driven analysis approach, it is possible to determine how much water would be abstractable from the subnetwork for any given value of the subnetwork source head. Of course, the same is true for small single-source networks in general and underpins the derivation which follows.

The system source head vs discharge relationship may be written as

$$H_0 = H_0^{min} + KQ^n \tag{2}$$

in which: H_0 represents the source head. H_0^{min} represents the source head above which there would be outflow — not necessarily full flow — at at least one demand node. K and n are a resistance coefficient and an exponent, respectively, for the network. The value of the exponent n, usually about 2 (Chandapillai, 1991; Gupta and Bhave, 1996), is determined by calibration. An expression for K is derived shortly. Q denotes the sum of the nodal outflows. To determine the value of Q for any given source head, it is sufficient to rearrange Eq. (2) as

$$Q = \left(\frac{H_0 - H_0^{min}}{K}\right)^{\frac{1}{n}} \tag{3}$$

When $Q = Q_{req}$, the required flow or total demand, $H_0 = H_0^{des}$, the desirable source head. It follows from Eq. (3) that

$$Q_{req} = \left(\frac{H_0^{des} - H_0^{min}}{K}\right)^{\frac{1}{n}} \Rightarrow \frac{1}{K^{\frac{1}{n}}} = \frac{Q_{req}}{\left(H_0^{des} - H_0^{min}\right)^{\frac{1}{n}}} \tag{4}$$

Substituting for K in Eq. (3) gives the available flow as

$$Q = Q_{req}\left(\frac{H_0 - H_0^{min}}{H_0^{des} - H_0^{min}}\right)^{\frac{1}{n}} \quad H_0^{min} \leq H_0 \leq H_0^{des} \tag{5}$$

Tanyimboh and Templeman (1995) have previously presented a similar equation to Eq. (5). It turns out that the former is in reality a special case of the latter. Furthermore, Wagner et al. (1988) presented a corresponding equation for individual nodes, but without a formal derivation.

The technique for calculating Q using demand-driven analysis consists of determining the value of H_0^{des} with all demands fully satisfied for any given (reduced) network configuration (Tanyimboh and Templeman, 1993). Eq. (5) is then used, knowing that H_0 represents the available source head. If the possibility of demand node isolation due to link unavailability exists, the available flow is given by

$$Q = (Q_{req} - Q_{isol})\left(\frac{H_0 - H_0^{min}}{H_0^{des} - H_0^{min}}\right)^{\frac{1}{n}} \quad H_0^{min} \leq H_0 \leq H_0^{des} \tag{6}$$

in which: Q_{isol} represents the combined demand of the isolated nodes. H_0^{des} is calculated for the full demands of the reachable nodes. H_0^{min} in Eq. (6) refers to the reduced network.

For any given source head, the available flow obviously depends on whether the system is in a reduced state or not. The components (pipes) of a distribution network can be unavailable for use due to failure/bursts and/or repair/maintenance, etc. It is commonly assumed that pipe bursts are not interdependent (Fujiwara and Tung, 1991; Su et al., 1987; etc.). With this assumption, the probability $p(0)$ that no pipe is unavailable is

$$p(0) = \prod_{l=1}^{NL} a_l \tag{7}$$

in which a_l is the probability that pipe (link) l is available and NL is the number of links. A function for calculating the availability of individual pipes is given shortly.

Also, the probability $p(N)$ that only N specified links are unavailable while the remaining $(NL - N)$ are available is given by

$$p(N) = p(0) \prod_{n=1}^{N} \frac{u_n}{a_n} \quad N = 1, ..., NL \tag{8}$$

in which u denotes the unavailability of a component.

Pipe availability can be taken as the ratio of the mean time between failures to the mean time between failures plus the mean failure, including repair, duration. This can be calculated using (Cullinane et al., 1992)

$$a_l = \frac{0.21218 D_l^{1.462131}}{0.00074 D_l^{0.285} + 0.21218 D_l^{1.462131}} \quad l = 1, ..., NL \tag{9}$$

in which D_l is the diameter of pipe l in inches (1 in. = 25.4 mm).

For any given normal or subnormal configuration, the reliability can be defined as the ratio of the available flow to the required flow, i.e.

$$r(N) = \frac{Q(N)}{Q_{req}} \quad N = 0, ..., NL \tag{10}$$

in which $r(N)$ is the sytem reliability with N specified pipes unavailable and $Q(N)$ is the available flow when the N specified pipes are unavailable (Eq. (6)). Using Eq. (8) and taking all network configurations into consideration, the reliability R of the distribution system is given by the expectation of the state reliabilities, i.e.

$$R = p(0) \sum_{N=0}^{NL} \left(r(N) \prod_{n=1}^{N} \frac{u_n}{a_n} \right) \tag{11}$$

If no more than two simultaneously unavailable links are considered, then a lower bound to reliability, R_L, is given by

$$\begin{aligned} R_L &= \frac{1}{Q_{req}} \left(\sum_{l=0}^{NL} p(l)Q(l) + \sum_{\substack{l=1 \\ m:m>l}}^{NL-1} p(l,m)Q(l,m) \right) \\ &= p(0) \left(r(0) + \sum_{l=1}^{NL} r(l)\frac{u_l}{a_l} + \sum_{\substack{l=1 \\ m:m>l}}^{NL-1} r(l,m)\frac{u_l}{a_l}\frac{u_m}{a_m} \right) \end{aligned} \tag{12}$$

in which: $p(l)$ and $p(l,m)$ are the respective probabilities that only pipes l and both l and m, together, are unavailable. Similarly, $r(l)$ and $r(l,m)$ are the respective system reliabilities with pipe l and both pipes l and m simultaneously unavailable. $Q(l)$ and $Q(l,m)$ are the respective available flows with pipes l, and both l and m, together, unavailable. The system unreliability U_L (lower bound) can be determined from Eq. (12) by simply replacing the available flow, Q, by the shortfall in supply, $(Q_{req} - Q)$. Thus an upper bound to system reliability R_U can be calculated as $R_U = 1 - U_L$. To determine the overall reliability of the reduced configurations, the damage tolerance T_L (lower bound) can be calculated using (Tanyimboh and Templeman, 1995)

$$T_L = \frac{R_L - r(0)p(0)}{1 - p(0)} \tag{13}$$

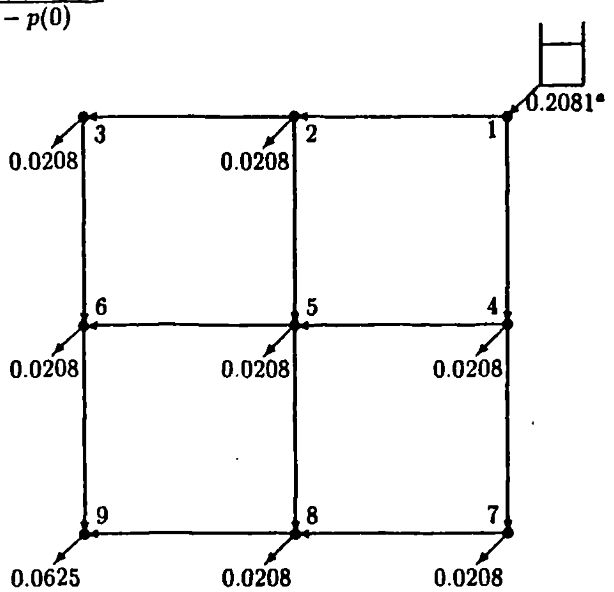

Figure 1: Sample network
* Flows are in m^3/s; node elevations are all 0 m; source head = 100 m.

3 Appraisal

To demonstrate both the efficacy of the source head reliability measure and the computational efficiency of the present formulation, the network of Figure 1 is used. This simple symmetric four-loop network has previously been used by a number of researchers to demonstrate several aspects of design and reliability (Awumah and Goulter, 1992; Fujiwara et al., 1990, 1991; Tanyimboh et al., 1993, 1995; Goulter and Coals, 1986). This example also demonstrates the sensitivity of the proposed reliability measure using a progression of designs for the network of Figure 1.

The designs upon which the present appraisal is based are taken from Fujiwara and Tung (1991). The designs therein are obtained by generating minimum cost designs respectively satisfying a progression of specified levels of reliability, including a maximum hydraulic gradient limit of 0.01. Pipe data for the 16 designs are given in Tables 1 and 2. For a better understanding of the hydraulic behaviour of the respective designs, the following two indicators of hydraulic performance were also calculated. The required source head values H_0^{des} for the 16 designs are given in Table

3 for single link failures. The total available flow (i.e. flow supplied at adequate pressure) for each design was calculated as described in Gupta and Bhave (1996) using head-driven analysis and the values are presented in Tables 4 and 5.

Reliability and damage tolerance values were calculated using the source head method for both single and up to two simultaneous link failures with $H_0^{min} = 0$, $n = 2$ and $H_0 = 100m$. The results are reported in Table 6. As the upper and lower bounds are identical if up to two simultaneous pipe failures are considered, these values are shown in Table 6 as the correct values, R and T. Fortran 77 programs were written for the network analysis (Newton-Raphson method) and the reliability calculations. The programs were run using a Compaq Pentium PC (75 MHZ, 8 MB RAM). The total CPU time to calculate the reliability measures (R_L, R_U, T_L and T_U) for each design was less than 3.2 seconds when up to two simultaneous link failures were considered.

Pipe	Diameter (mm)							
1-2, 1-4[a]	250	250	250	250	250	250	250	250
2-3, 4-7	175	175	180	180	180	185	185	185
2-5, 4-5	145	145	145	145	145	145	145	145
3-6, 7-8	115	115	115	120	125	125	130	135
5-6, 5-8	100	105	105	105	105	105	105	105
6-9, 8-9	100	100	100	100	100	100	100	100
Cost[b] ($ 10^6)	0.255	0.257	0.259	0.261	0.263	0.266	0.268	0.270
Design	1	2	3	4	5	6	7	8

Table 1: Pipe Data (Designs 1–8)

[a] All designs are symmetrical about the line joining nodes 1, 5 and 9; all length = 1000m, C value = 130. [b] From Fujiwara and Tung (1991)

4 Discussion

The indicators of hydraulic performance and reliability shown in the tables are either monotonic increasing or decreasing sequences. The required source head for full demand satisfaction (Table 3) decreases from Designs 1 to 16 for each isolated pipe. This is the expected result because each of the 16 designs is obtained from the preceding one by increasing only two pipe diameters (Tables 1 and 2). It can therefore be said, confidently, that the hydraulic performance of each design is superior to that of its predecessor. Furthermore, it also follows that the mechanical reliability of each design is greater than that of its predecessor, for numerous studies have shown that larger diameter pipes are generally more reliable. Also, the actual consumptions increase from Designs 1 to 16 (Tables 4 and 5). This provides further direct evidence that hydraulic performance is improving from Designs 1 to 16. There is, therefore, no doubt that any measures of the reliability of the designs should increase monotonically from Designs 1 to 16. This fundamental condition is satisfied by the values calculated (Table 6).

The reliability values calculated by averaging the lower and upper bounds derived using single-link failures only are virtually identical to the true values (Table 6)

Pipe	Diameter (mm)							
1-2, 1-4[a]	250	250	250	250	250	255	255	255
2-3, 4-7	190	190	190	190	190	190	190	190
2-5, 4-5	145	145	145	150	150	150	155	155
3-6, 7-8	135	140	140	140	140	140	140	140
5-6, 5-8	105	105	110	110	115	115	115	120
6-9, 8-9	100	100	100	100	100	100	100	100
Cost[b] ($ 10^6)	0.272	0.274	0.276	0.278	0.280	0.283	0.285	0.287
Design	9	10	11	12	13	14	15	16

Table 2: Pipe Data (Designs 9–16)
[a] All designs are symmetrical about the line joining nodes 1, 5 and 9; all length = 1000m, C value = 130. [b] From Fujiwara and Tung (1991)

with the maximum difference being 2×10^{-6}. It may be noted that the former values are all higher than the latter. The above two observations also apply to the damage tolerance values (Table 6), except that the differences are somewhat larger ($\approx 3 \times 10^{-4}$ or less). The damage tolerance parameter is quite sensitive to component reliability and/or omission of network configurations with multiple failed components from the analysis (Eq. (12)). It should therefore be used with care and should preferably be computed by averaging the upper and lower bounds.

As demonstrated above, calculating reliability as the mean of the lower and upper bounds improves the accuracy of the results significantly with virtually no increase in computational effort or CPU time. This is easy to verify by factorising the expression for $\frac{1}{2}(R_L+R_U)$. This means that when the method is eventually applied to realistic size networks, it may be possible to estimate reliability quite accurately by considering the simultaneous failure of no more than a few components. Also, the obvious conclusion from the reported CPU time (i.e. less than 4 seconds with 78 failure cases) is that the computational efficiency of the proposed formulation is excellent.

Perhaps the main weakness of the present method is that it does not recognise sufficiently the spatial nature of hydraulic performance in a pipe network. For example, if a pipe near the downstream end of the distribution system (node 9) is unavailable, this should not have any serious consequences for demand nodes further upstream. It would therefore be expected that the problem of insufficient supply/pressure would be localised around the failed component. For this reason, the proposed source head method tends to underestimate reliability. Further research is required to address this issue.

5 Summary and Conclusions

A formal derivation of the source head method of calculating the reliability of water distribution networks has been provided. The methodology is realistic in that it incorporates the pressure dependency of nodal outflows and does not calculate network reliability as the average of the demand node reliabilities. However, the requirement

Design	Isolated Pipe					
	1-2 1-4	2-3 4-7	2-5 4-5	3-6 7-8	5-6 5-8	6-9 8-9
1	561.946	570.700	304.845	422.845	341.004	808.534
2	555.453	514.933	300.758	391.264	335.845	789.016
3	552.586	513.511	298.388	389.996	332.274	785.974
4	535.351	505.554	286.513	383.464	312.663	768.162
5	519.474	498.494	276.289	377.675	296.537	752.582
6	516.308	497.004	273.936	376.352	293.237	749.456
7	501.293	490.639	265.004	371.133	279.841	735.632
8	487.708	485.029	257.328	366.537	268.762	723.593
9	484.334	483.515	255.049	365.200	265.758	720.473
10	471.924	478.477	248.346	361.074	256.493	709.814
11	467.358	440.566	247.175	338.955	254.660	701.476
12	446.058	434.617	244.860	334.703	254.316	700.619
13	442.159	402.747	243.708	316.187	252.557	692.722
14	436.575	400.867	241.919	314.462	250.984	691.021
15	417.782	395.548	239.669	310.660	250.656	690.162
16	414.408	368.871	238.537	295.202	248.978	682.747
	Required source head (m)					

Table 3: Required source heads for single isolated pipes

that all demands be fully satisfied leads to an underestimation of system-wide available flows. Further research is necessary to address this issue. A well-known sample network has been used to show that the method is computationally efficient and is capable of differentiating between a range of nearly identical designs. By avoiding complicated and/or time-consuming procedures (minimum cut sets, etc.), the method can be used in a routine way to estimate the reliability of simple networks.

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Design	Source Head (m)		
	100	80	50
1	0.171951	0.164525	0.145240
2	0.173044	0.166080	0.146221
3	0.173323	0.166581	0.147354
4	0.174264	0.168615	0.149245
5	0.174327	0.168595	0.151073
6	0.174653	0.168891	0.151360
7	0.175417	0.169752	0.151705
8	0.176089	0.170640	0.152519
	Total consumption (m^3/s)		

Table 4: Pressure-dependent outflows for several source heads (Designs 1–8)

Design	Source Head (m)		
	100	80	50
9	0.176424	0.170898	0.153642
10	0.177033	0.171625	0.154581
11	0.177137	0.172049	0.155661
12	0.177361	0.172108	0.155718
13	0.177711	0.172457	0.156373
14	0.178042	0.172801	0.157078
15	0.178263	0.173019	0.157486
16	0.178822	0.173380	0.158135
	Total consumption (m^3/s)		

Table 5: Pressure-dependent outflows for several source heads (Designs 9–16)

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Design	$\frac{R_L+R_U}{2}$ [a]	R [b]	Fujiwara[c]	$\frac{T_L+T_U}{2}$ [a]	T [b]
1	0.599359	0.599357	0.81851	0.467883	0.467538
2	0.607280	0.607279	0.83341	0.476057	0.475736
3	0.609347	0.609345	0.83364	0.477667	0.477353
4	0.618899	0.618897	0.85013	0.485695	0.485405
5	0.627598	0.627596	0.86145	0.493025	0.492750
6	0.629898	0.629896	0.86356	0.494857	0.494583
7	0.638010	0.638008	0.88212	0.501722	0.501566
8	0.645303	0.645301	0.89069	0.507904	0.507660
9	0.647756	0.647755	0.89777	0.509897	0.509659
10	0.654497	0.654495	0.91657	0.515614	0.515393
11	0.658449	0.658447	0.92170	0.520798	0.520590
12	0.660080	0.660079	0.92463	0.522573	0.522373
13	0.663962	0.663961	0.93530	0.527683	0.527498
14	0.666244	0.666243	0.94228	0.529357	0.529177
15	0.667879	0.667877	0.94359	0.531172	0.530997
16	0.671702	0.671701	0.95091	0.536179	0.536022
	Reliability			Damage Tolerance	

Table 6: Summary of network reliability measures

[a] Based on 1-link failures; [b] Based on 1- and 2-link failures; [c] Fujiwara and Tung (1991) — comparison with the present R values is difficult because pipe availability is defined in a different way by Fujiwara and Tung.

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APPENDIX G4

AN IMPROVED SOURCE HEAD METHOD FOR CALCULATING THE RELIABILITY OF WATER DISTRIBUTION NETWORKS

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An Improved Source Head Method for Calculating the Reliability of Water Distribution Networks

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Abstract

In order to calculate the reliability of single-source networks, this paper presents a new way of interpreting results of the traditional demand-driven network analysis as an approximation to pressure-driven simulation. This approach is useful because most network modelling software packages use the demand-driven analysis approach and are incapable of simulating pressure-deficient conditions properly. Furthermore, as far as pressure-driven analysis is concerned, many questions remain unanswered. The method proposed herein can be used to calculate the reliability of an entire water distribution network and/or that of the individual demand nodes. Using a sample network, it is shown that the method is much more accurate than previous formulations and retains the proven advantages of the source head approach to reliability calculation.

1 Introduction

For a long time, little attention was paid to the definition and quantification of reliability in the context of water distribution. Early research in the area was mostly concerned, initially, with the transfer of well-established approaches in other fields such as power transmission. It soon became clear that the uniqueness of water distribution networks required methods specifically developed for these networks. Despite initial uncertainty about the essence of water distribution network reliability, it has been established during the last five years or so that reliability should be a function of the ratio of actual flow delivered to the required flow.

For a while, it was believed that calculated reliability values for a distribution network could be raised artificially by reducing the flow to the parts of the network with insufficient pressure [1]. It was felt that this would make it possible to increase available flow in the remainder of the network. Furthermore, there were no satisfactory methods for quantifying partial failure. This was primarily due to the fact that algorithms for network analysis generally treated nodal outflows as constants with pre-determined values. Therefore, to enable like-with-like comparisons between different networks and avoid the above-mentioned uncertainties and difficulties, the source head approach to reliability calculation was developed using the notional source head for full satisfaction of all demands in a single-source network [2–5; also see 6].

However, early results from pressure-dependent network analysis have shown that the consequences of insufficient pressure (i.e. insufficient flow) tend to be localised, with conditions elsewhere often being largely unaffected [7–9]. In other words, the spatial nature of the hydraulic performance of distribution networks should be recognised by reliability measures. Furthermore, the basic source head method (SHM) [2, 4–5] tacitly treated the source node as if it were a demand node whose demand is equal to the combined demand of the actual demand nodes of the single-source network. The modelling effects of lumping the demands in the above way have been detailed in [7] where it was concluded that such approaches generally provide approximate rather than accurate results. As such, the SHM can at best be expected to give reasonable estimates of reliability values. On the other hand, network models in which the demand nodes are considered individually are generally more accurate [7]. Therefore, unlike previous formulations, the present improved source head method (ISHM) uses the required source heads to satisfy the demands of individual nodes to estimate the available flow at those nodes.

The theoretical formulation of the ISHM approach to reliability analysis is based on pressure-driven network analysis principles [5]. However, its practical implementation depends on a novel interpretation and use of the traditional demand-driven analysis results. The aim of this paper is to demonstrate a method of approximating nodal outflows and system reliability using demand-driven simulation and an evaluation of the required pressures at individual nodes. Using a much-studied sample network, it is shown that reliability values calculated using the proposed method (ISHM) are far more accurate than previous results and that all the previously identified advantages of the SHM including computational efficiency [4-5] are retained. An issue which has not received sufficient attention is the interdependency between the flows available at the demand nodes and its significance in system reliability calculations. This issue is also addressed in this paper.

2 Nodal Outflow

The quantity of water which a distribution network can supply at adequate pressure is one of the principal factors determining the reliability of the system. Therefore, the relationship between actual nodal outflows and pressures should be incorporated in any realistic network reliability measure. Thus, following [5], for a single-source network the relationship can be approximated as

$$Q_j = Q_j^{req}\left(\frac{H_s - H_{s,j}^{min}}{H_{s,j}^{des} - H_{s,j}^{min}}\right)^{\frac{1}{n_j}} \quad H_{s,j}^{min} \leq H_s \leq H_{s,j}^{des};\ \forall j \tag{1}$$

in which: Q_j^{req} and Q_j, respectively, represent the demand and actual outflow at node j. H_s is the available source head. $H_{s,j}^{min}$ is the source head below which outflow at node j is zero or deemed unsatisfactory. $H_{s,j}^{des}$ is the source head above which the demand at node j is fully satisfied. n_j is an exponent whose value, usually between about 1 and 2 [1, 7, 10], can be determined by calibration.

To calculate the values of the Q_j a demand-driven analysis of the network is performed using the demands Q_j^{req}. This gives

$$H_{s,j}^{des} \approx H_s - H_j \tag{2}$$

in which H_j represents the head at node j. The value of $H_{s,j}^{min}$, the source head corresponding to zero outflow at demand node j can be found using pressure-driven simulation [7] or field tests. Inherently, however, the present formulation does not have the full capabilities of pressure-driven analysis. Therefore, $H_{s,j}^{min}$ is approximated using H_s^{min}, the source head above which outflow just begins at any node of the network. Thus $H_s^{min} = Min < H_{s,j}^{min}, \quad \forall j >$, which may be taken as the elevation of the lowest node.

3 Reliability Analysis

For any given source head, the available flow obviously depends on whether the system is in a reduced state or not. The components (pipes) of a distribution network can be unavailable for use due to failure/bursts and/or repair/maintenance, etc. It is commonly assumed that pipe bursts are not interdependent [11-12]. With this assumption, the probability $p(0)$ that no pipe is unavailable is

$$p(0) = \prod_{l=1}^{NL} a_l \tag{3}$$

in which a_l is the probability that pipe (link) l is available and NL is the number of links. A function for calculating the availability of individual pipes is given shortly.

Also, the probability $p(N)$ that only N specified links are unavailable while the remaining $(NL - N)$ are available is given by

$$p(N) = p(0) \prod_{n=1}^{N} \frac{u_n}{a_n} \quad N = 1, ..., NL \tag{4}$$

in which u denotes the unavailability of a component.

Pipe availability can be taken as the ratio of the mean time between failures to the mean time between failures plus the mean failure, including repair, duration. This can be calculated as in [13] using

$$a_l = \frac{0.21218D_l^{1.462131}}{0.00074D_l^{0.285} + 0.21218D_l^{1.462131}} \quad l = 1, ..., NL \tag{5}$$

in which D_l is the diameter of pipe l in inches (1 in. = 25.4 mm).

For any given normal or subnormal configuration, nodal reliability can be defined as the ratio of the available flow to the required flow, i.e.

$$r_j(N) = \frac{Q_j(N)}{Q_j^{req}} \quad N = 0, ..., NL; \ \forall j \tag{6}$$

in which $r_j(N)$ is the nodal reliability with N specified pipes unavailable and $Q_j(N)$ is the available flow when the N specified pipes are unavailable (Eq. (1)). Using Eq. (6) and taking all network configurations into consideration, the reliability R_j of the node is given by the expectation of the nodal reliabilities for the various configurations, i.e.

$$R_j = p(0) \sum_{N=0}^{NL} \left(r_j(N) \prod_{n=1}^{N} \frac{u_n}{a_n} \right) \quad \forall j \tag{7}$$

If no more than two simultaneously unavailable links are considered, then the reliability is given by

$$\begin{aligned} R_j &= \frac{1}{Q_j^{req}} \left(\sum_{l=0}^{NL} p(l) Q_j(l) + \sum_{\substack{l=1 \\ m:m>l}}^{NL-1} p(l,m) Q_j(l,m) \right) \\ &= p(0) \left(r_j(0) + \sum_{l=1}^{NL} r_j(l) \frac{u_l}{a_l} + \sum_{\substack{l=1 \\ m:m>l}}^{NL-1} r_j(l,m) \frac{u_l}{a_l} \frac{u_m}{a_m} \right) \quad \forall j \end{aligned} \tag{8}$$

in which: $p(l)$ and $p(l,m)$ are the respective probabilities that only pipes l and both l and m, together, are unavailable. Similarly, $r_j(l)$ and $r_j(l,m)$ are the respective nodal reliabilities with pipe l and both pipes l and m simultaneously unavailable. $Q_j(l)$ and $Q_j(l,m)$ are the respective available flows at node j with pipes l, and both l and m, together, unavailable. The nodal unreliabilities, U_j, can be determined from Eqs. (8) by simply replacing the available flow, Q_j, by the shortfall in supply, $(Q_j^{req} - Q_j)$. The values of nodal reliability, R_j, and unreliability, U_j, given by the above equations are lower bounds because the terms for three or more simultaneous link failures are not included. Thus an upper bound to nodal reliability is given by $1 - U_j$ which, together with the lower bound of Eq. (8), can be used to determine the point in the summation of Eq. (7) beyond which further terms need not be included for a given accuracy.

The reliability, R, of the network as a whole can be determined in a similar way by simply writing Eqs. (6–8) without the subscript j. To this end,

$$Q_{req} = \sum_{j=1}^{J} Q_j^{req} \tag{9}$$

$$Q = \sum_{j=1}^{J} Q_j \tag{10}$$

in which Q_{req} and Q are, respectively, the sum of the nodal demands and available flow and J represents the number of demand nodes.

Finally, the damage tolerance, T, of the system [2, 4–5] can be calculated as

$$T = \frac{R - r(0)p(0)}{1 - p(0)} \tag{11}$$

in which $r(0)$ represents the system reliability (Q/Q_{req}) with all components available. The damage tolerance of individual nodes can also be calculated in a similar way if required.

4 Appraisal

The advantages of the source head approach (SHM), namely simplicity, sensitivity and computational efficiency have previously been demonstrated [4–5]. The main aim of the following appraisal is to show the present improved source head method (ISHM) is a great deal more accurate than the previous SHM formulation. The present demonstration is based on the network of Figure 1. This simple symmetric four-loop network has previously been used by a number of researchers to demonstrate several aspects of design and reliability [4–5, 11, 14–16] and, as such, is well known.

The designs upon which the present appraisal is based were taken from [11]. These designs represent a progression in that they were obtained by generating minimum cost designs respectively satisfying a progression of specified levels of reliability, including a maximum hydraulic gradient limit of 0.01. Pipe data for the

16 designs are given in Tables 1 and 2. These designs were selected because successive designs in the progression are virtually identical. Consequently, by using these designs, it is relatively easy to judge whether any reliability indicator is consistent and, therefore, reliable. Also, as shown shortly, the hydraulic performance of the designs cannot be properly simulated using demand-driven analysis because there is insufficient pressure in the system to drive the required flows.

The hydraulic performance of the designs has previously been assessed [5] in terms of the required source head to fully satisfy demands throughout the network (demand-driven analysis) with individual pipes isolated in turn. Also, the actual nodal outflows were calculated for a range of values of the source head (pressure-driven analysis [7]). It was concluded that, in terms of hydraulic performance, the designs do constitute a gradual and smooth progression. Because the layout of the designs does not change, their mechanical reliabilities can be compared using $p(0)$, the probability that no component is unavailable. The $p(0)$ values are shown in Table 3 along with the required source heads, $H_s(0)$, for full satisfaction of the total demand of the network with all pipes available. The total flow supplied at adequate pressure [7] for the available source head of 100m for the fully connected network is also shown. Table 3 shows that the designs are deficient and provides further evidence that the designs form a good progression.

Reliability and damage tolerance values were calculated using the improved source head method (ISHM) with $H_s^{min} = 0m$, $n_j = 2$ and $H_s = 100m$. The results are reported in Figure 2 and Table 4 where SHM values are also shown for comparison. The approximations to actual system flow given by the ISHM are compared to the SHM in Table 5 and shown graphically in Figure 3. Corresponding values for individual nodes and their respective reliabilities are given in Tables 6 and 7, respectively.

Fortran 77 programs were written for the network analysis (Newton-Raphson method) and the reliability calculations. The programs were run using a PC with a Pentium processor (75 MHZ, 8 MB RAM). The total CPU time to simulate all combinations of up to two isolated pipes and calculate the system and nodal reliability values including damage tolerance was less than 5 seconds per design.

5 Discussion

It is clear from Table 4 and Figure 2 that the present method (ISHM) gives significantly higher values of reliability and damage tolerance than the previous formulation (SHM). The reason for the difference is that the ISHM gives more accurate values of flow delivered at adequate pressure than SHM (Table 5). While the SHM

Pipe	Diameter (mm)							
1-2, 1-4[a]	250	250	250	250	250	250	250	250
2-3, 4-7	175	175	180	180	180	185	185	185
2-5, 4-5	145	145	145	145	145	145	145	145
3-6, 7-8	115	115	115	120	125	125	130	135
5-6, 5-8	100	105	105	105	105	105	105	105
6-9, 8-9	100	100	100	100	100	100	100	100
Cost[b] ($ 10^6)	0.255	0.257	0.259	0.261	0.263	0.266	0.268	0.270
Design	1	2	3	4	5	6	7	8

Table 1: Pipe Data (Designs 1–8)

[a] All designs are symmetrical about the line joining nodes 1, 5 and 9; all length = 1000m, C value = 130. [b] From Fujiwara and Tung (1991)

Pipe	Diameter (mm)							
1-2, 1-4[a]	250	250	250	250	250	255	255	255
2-3, 4-7	190	190	190	190	190	190	190	190
2-5, 4-5	145	145	145	150	150	150	155	155
3-6, 7-8	135	140	140	140	140	140	140	140
5-6, 5-8	105	105	110	110	115	115	115	120
6-9, 8-9	100	100	100	100	100	100	100	100
Cost[b] ($ 10^6)	0.272	0.274	0.276	0.278	0.280	0.283	0.285	0.287
Design	9	10	11	12	13	14	15	16

Table 2: Pipe Data (Designs 9–16)

[a] All designs are symmetrical about the line joining nodes 1, 5 and 9; all length = 1000m, C value = 130. [b] From Fujiwara and Tung (1991)

underestimates the actual flow delivered by about 25%, the ISHM overestimates it by only about 5%. As shown in Tables 3 and 6, the shortfall in supply due to insufficient pressure is localised around nodes 6, 8 and 9 while supply elsewhere is not affected. Therefore, by considering the demand nodes individually, the ISHM recognises the spatial characteristics of the distribution sytem. As such, the ISHM has successfully addressed the main weakness of the previous network-wide required source head approach which often underestimated the quantity of actual flow delivered. In particular, the present analysis clearly shows that the reliability value given by the original SHM is in reality an approximation to the reliability of the most critical node — node 9 in the present example (cf. Table 4: SHM and Table 7: node 9). Also, it is worth observing that there is good correlation between the available flow values of the approximate SHM and ISHM methods and the more accurate head-driven simulation method (HDSM) as demonstrated graphically in Figure 4.

A major advantage of the ISHM (unlike SHM) is its ability to provide nodal reliability values. These are shown in Table 7. For comparison, the system reliability values are also shown in Table 7 along with the arithmetic and demand-weighted means of the nodal reliabilities. It can be seen that the nodal means are different from the real reliability value of each network (also see [17]) though the weighted means are remarkably similar to the true values. An important reason for these differences is that the nodal outflows (and, hence, their reliabilities) are not mutually independent [2, 7–9]. This issue was addressed herein by first calculating nodal outflows the sum of which was then used to calculate system reliability [2]. This way, double counting was avoided.

6 Conclusions

Using various parameters it has been shown that the designs appraised herein form a good progression in terms of hydraulic performance and reliability. They would therefore be suitable as a test bed for any reliability measures proposed in future. In the past, one of the major difficulties with research into reliability analysis has been the lack of results against which rigorous comparisons could be made.

In order to ensure fair reliability comparisons between different water distribution systems, the present formulation retains the requirement that demands be fully satisfied throughout the network, and this is achieved using demand-driven simulation to determine nodal pressures corresponding to those demands. The available flow at individual nodes is then found using an approximation to the relationship between nodal outflow and pressure. By so doing, the proposed (ISHM) method

Design[a]	$Q(0)^a$ (m^3/s)	$H_s^{req}(0)^b$ (m)	$p(0)$	$H_{s,6}^{req}(0)^c$ (m)
1	0.171951	277.6	0.993972	120.3
2	0.173044	270.5	0.994049	113.2
3	0.173323	268.6	0.994072	111.3
4	0.174264	260.4	0.994129	103.1
5	0.174327	253.2	0.994182	<100
6	0.174653	251.4	0.994204	<100
7	0.175417	245.1	0.994251	<100
8	0.176089	239.6	0.994296	<100
9	0.176424	237.8	0.994316	<100
10	0.177033	232.9	0.994357	<100
11	0.177137	230.1	0.994427	<100
12	0.177361	229.0	0.994461	<100
13	0.177711	226.3	0.994524	<100
14	0.178042	224.8	0.994535	<100
15	0.178263	223.7	0.994568	<100
16	0.178822	221.2	0.994625	<100

Table 3: Hydraulic performance and mechanical reliability indicators

[a] All 16 designs are deficient: $Q < Q_{req}$ and $H_s^{req} > H_s$ for full network.

[b] Equivalent to the head loss from the source to node 9 with all demands fully satisfied.

[c] Nodes for which required source head exceeds available source head are 6 ($\equiv$ 8) and 9.

allows for spatial variations in the performance of the distribution network. This leads to much improved estimates of system reliability compared to the previous (SHM) formulation. Thus the ISHM has successfully addressed the main weakness of the source head approach to reliability calculation. Furthermore, it has the advantage that nodal reliability values can also be computed for any nodes of particular interest.

Finally, it has been shown that the reliability value of a distribution network is not equal to the arithmetic mean of the respective nodal reliability values. Surprisingly, however, the demand-weighted mean which reflects the relative importance of the nodal demands turned out to be very close to the true reliability of the system for the examples considered. Further investigation is required to assess the significance of this.

Acknowledgment

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Design	Reliability		Damage tolerance	
	ISHM	SHM	ISHM	SHM
1	0.861656	0.599357	0.765560	0.467538
2	0.869717	0.607279	0.771899	0.475736
3	0.871875	0.609345	0.773837	0.477353
4	0.882139	0.618897	0.781304	0.485405
5	0.887809	0.627596	0.788458	0.492750
6	0.888510	0.629896	0.790451	0.494583
7	0.890974	0.638008	0.796639	0.501566
8	0.893189	0.645301	0.802303	0.507660
9	0.893933	0.647755	0.804397	0.509659
10	0.895975	0.654495	0.809053	0.515393
11	0.897178	0.658447	0.813000	0.520590
12	0.897678	0.660079	0.814811	0.522373
13	0.898856	0.663961	0.818552	0.527498
14	0.899551	0.666243	0.820327	0.529177
15	0.900050	0.667877	0.822270	0.530997
16	0.901213	0.671701	0.826098	0.536022

Table 4: Values of system reliability and damage tolerance

Design	Fraction of total demand satisfied			Error[b] (%)	
	SHM	ISHM	HDSM[a]	SHM	ISHM
1	0.600156	0.862239	0.826290	-27.40	4.35
2	0.608066	0.870303	0.831543	-26.87	4.66
3	0.610132	0.872460	0.832883	-26.74	4.75
4	0.619686	0.882734	0.837405	-26.00	5.41
5	0.628385	0.888390	0.837808	-25.00	6.04
6	0.630685	0.889082	0.839274	-24.85	5.93
7	0.638796	0.891519	0.842946	-24.22	5.76
8	0.646091	0.893710	0.846175	-23.65	5.62
9	0.648594	0.894445	0.847785	-23.50	5.50
10	0.655285	0.896468	0.850711	-22.97	5.38
11	0.659220	0.897650	0.851211	-22.56	5.46
12	0.660846	0.898140	0.852287	-22.46	5.38
13	0.664712	0.899298	0.853969	-22.16	5.31
14	0.666996	0.899986	0.855560	-22.04	5.19
15	0.668625	0.900475	0.856622	-21.95	5.12
16	0.672434	0.901619	0.859308	-21.75	4.92
Mean Error				-24.01	5.30

Table 5: Fractions of total demand satisfied by fully connected network
[a] Head-Driven Simulation Method [7]. [b] % of corresponding HDSM value

Node	Proportion of nodal demand satisfied					
2–5, 7	1.000	1.000	1.000	1.000	1.000	1.000
6, 8	0.912	0.940	0.985	1.000	1.000	1.000
9	0.600	0.608	0.620	0.646	0.661	0.673
Design	1	2	4	8	12	16

Table 6: Fractions of nodal demands satisfied by fully connected network (ISHM)

Node	Reliability				
2, 4	0.999879	0.999882	0.999888	0.999897	0.999908
3, 7	0.999653	0.999677	0.999690	0.999723	0.999764
5	0.999775	0.999792	0.999810	0.999828	0.999848
6, 8	0.910446	0.983533	0.999096	0.999311	0.999476
9	0.599358	0.618898	0.645302	0.660079	0.671701
Mean	0.927373	0.947234	0.955308	0.957221	0.958731
Weighted Mean	0.861644	0.881441	0.893187	0.897678	0.901002
System	0.861656	0.882139	0.893189	0.897678	0.901213
Design	1	4	8	12	16

Table 7: Reliabilities of individual nodes

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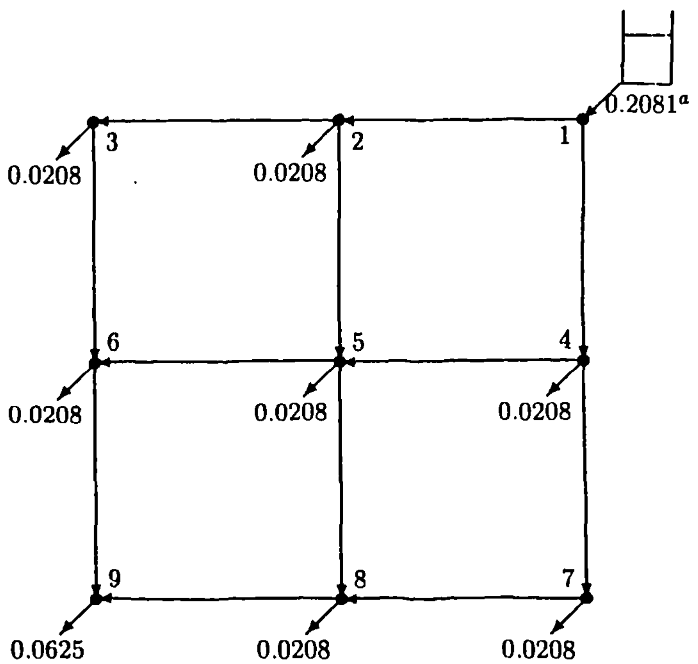

Figure 1: Sample network
[a] Flows are in m^3/s; node elevations are all 0 m; source head = 100 m.

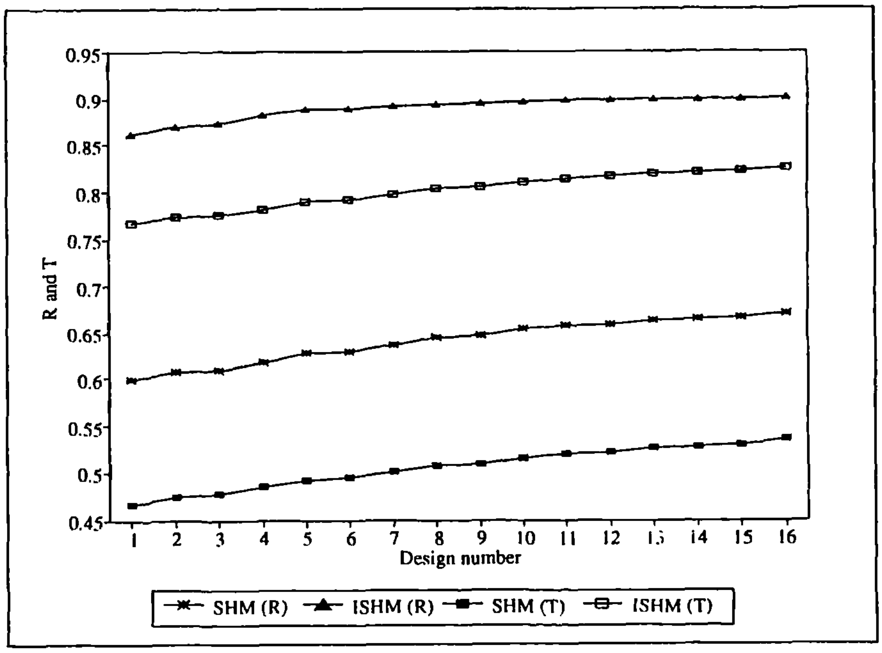

Figure 2: Network reliability and damage tolerance values

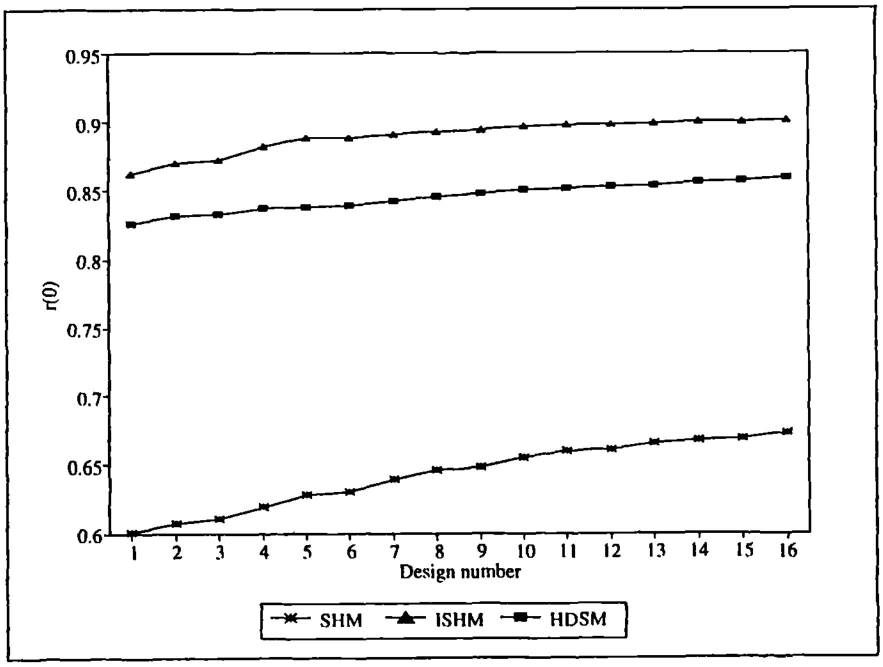

Figure 3: Fractions of total demand satisfied by fully connected networks

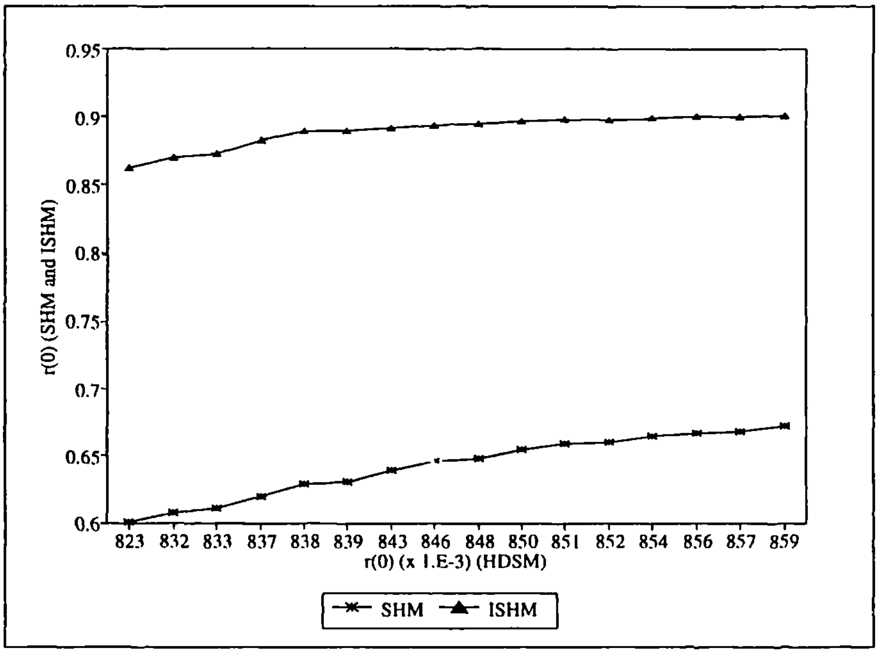

Figure 4: SHM and ISHM fraction of total demand against HDSM value

APPENDIX G5

DISCUSSION OF 'COMPARISON OF METHODS FOR PREDICTING DEFICIENT-NETWORK PERFORMANCE'

J. Water Resources Planning and Management, ASCE, Vol. 123, No. 6, PP 369-370, 1997.

COMPARISON OF METHODS FOR PREDICTING DEFICIENT-NETWORK PERFORMANCE*

Discussion by T. T. Tanyimboh[3] and M. Tabesh[4]

The authors have shed some much needed light on the poorly understood and much neglected issue of the pressure-driven analysis of water distribution networks with particular reference to less than fully satisfactory performance (Lumbers 1996). The authors' algorithm for network analysis with pressure-dependent nodal outflows appears in essence to be an iterative two-phase formulation in which each major iteration consists of a traditional demand-driven analysis followed by a formula-based calculation of improved nodal outflows. Taken together with Chandapillai (1991), the aforementioned two-level iterative technique is useful for appraising the performance of distribution networks if nodal outflows and pressures are less than fully satisfactory. Such a situation is commonly encountered in distribution network reliability analysis (Tanyimboh and Templeman 1993, 1994, 1995). The discussers' preliminary results would appear to suggest that further research into algorithms for pressure-driven simulation is required to improve the computational efficiency of the approach. Nevertheless, the authors have presented some interesting results worth commenting on.

Eq. (6), a special case of (8) (Chandapillai 1991), is quite useful

$$H_j^{avl} = H_j^{min} + R_j(q_j^{avl})^n \tag{8}$$

To a certain extent, (6) enables the desirable head to be determined in a formalized way given the minimum head and the required flow. It is often the case that the minimum flow and residual pressure are legislated for, but not desirable pressure. Thus, for example, the required flow and minimum residual pressure in distribution mains fitted with hydrants are often specified to satisfy fire-fighting requirements. Similarly, water undertakings in the United Kingdom are required by law to state the minimum pressure at the stop tap and the expected flow rate at the first tap in a property (Twort et al. 1994). (It is worth point out, however, that the minimum pressures in the preceding examples do not necessarily correspond to the pressures below which there would be no flow. Rather, they are probably more indicative of the pressures below which the service would be considered substandard and, therefore, unacceptable.)

Eq. (5) introduces a considerable amount of simplicity and clarity to the head-flow relationship suggested by Germanopoulos (1985) and, consequently, its ease of use and general applicability are increased. The previous development is important for two reasons. First, as the authors have noted, pressure-driven analysis using (5) gave fairly accurate results when the actual abstraction points of the distribution system were considered; furthermore, the results were similar to those obtained using the Wagner et al. equations [(7a)–(7c)]. Second, (5) is analytically easier to handle than (7a)–(7c), and, as such, may have certain computational advantages.

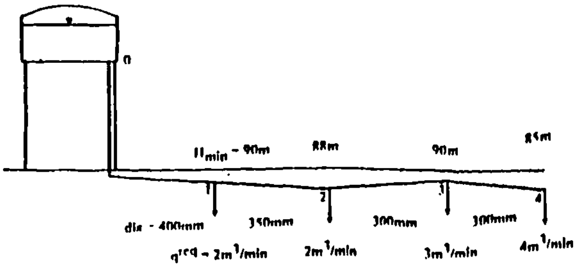

FIG. 4. Serial Network with Extra, Fire-Fighting Demand at Node 4 [Adapted from Gupta and Bhave (1996)]

Also, Fig. 3 is interesting for a number of reasons. The individual nodal outflow curves show quite clearly that nodal outflows are strongly interdependent (Tanyimboh 1993; Tanyimboh and Templeman 1995). Looking at the nodal head-discharge curves, it can be seen that there is a change in the respective shapes of the curves as outflow begins or stops at other nodes. The significance of this observation in the context of distribution network reliability is that the usual method of calculating reliability as the arithmetic mean of nodal reliabilities is inappropriate. It was the realization of this fact that first prompted the discussers' investigations into pressure-driven distribution network analysis.

Another remarkable feature of Fig. 3 is the ragged nature of the total available flow versus source head curve. The shape of this curve would appear to suggest that the relationship between source head and total outflow is perhaps more complex than has been assumed (Tanyimboh 1993; Tanyimboh and Templeman 1995). The systemwide available flow curve in the figure is, in essence, a superimposition of the individual nodal available flow curves. From this it may be inferred that conclusions about the value of the exponent, n, for a given network would be quite difficult to arrive at in the absence of extensive field data. In turn, this leads to the conclusion that some form of calibration will generally be necessary for the determination of the networkwide R, n, and c values for any distribution network.

One more point highlighted by the authors' paper concerns the difference in quality between the results of demand- and pressure-driven simulation. With reference to Fig. 4, the authors observed that demand-driven analysis flagged nodes 3 and 4 as deficient in that the available heads were less than the minimum heads with the full nodal demands imposed. However, pressure-driven analysis revealed that the deficiency was in fact localized at node 3, with node 4 being fully satisfactory in terms of both flow and pressure. This demonstrates quite clearly that it would be better for network upgrading and long-term planning decisions to be based on pressure- rather than demand-driven network modeling, which is currently the norm.

In conclusion, the authors have shed some light on the importance of incorporating explicit head-nodal outflow relationships in water distribution network analysis. Their work has shown that there is a clear need for further research into pressure-dependent network analysis including fully integrated algorithms. The paper has shown in a graphic way that nodal outflows and, therefore, reliabilities are not mutually independent. Finally, perhaps the most important lesson to be learned from their results is that only with fully satisfactory flows and pressures is it definitely safe to model networks using demand-driven analysis. Otherwise, pressure-driven analysis is indicated irrespective of whether the supply is continuous or intermittent.

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APPENDIX II. NOTATION

The following symbols are used in this paper:

- c = coefficient of exponential pressure-available flow function;
- H_j^{avl} = available residual pressure at node j;
- H_j^{min} = minimum residual pressure at node j;
- n = exponent;
- q_j^{avl} = available, pressure-dependent outflow at node j;
- q_j^{req} = required flow at node j; and
- R_j = resistance coefficient for node.

Closure by Rajesh Gupta[5] and Pramod R. Bhave[6]

The writers appreciate the interest shown and the thoughtful discussion provided by the discussers. It is true that (5) introduces simplicity and clarity to the head-discharge relationship suggested by Germanopoulos (1985), and consequently, its ease of use and general applicability is increased. However, the writers did not observe any significant computational advantage of (5) over (7*a*)–(7*c*). Furthermore, it should be remembered, as shown in the paper, that (5) gives q_j^{avl} considerably less than q_j^{req} for lower values of c_j even when $H_j^{avl} \geq H_j^{des}$, while the relationship gives q_j^{avl} much larger throughout and practically equal to q_j^{req} even when H_j^{avl} is much less than H_j^{des}. Since (7*a*)–(7*c*) give a better performance, they were recommended in the paper, and also used by the writers for reliability-based design of water distribution systems (WDSs) (Gupta and Bhave 1996).

As remarked by the discussers, the nodal outflows are interdependent in deficient networks. Therefore, nodal heads and flows should be considered together through node flow analysis for systems, becoming temporarily deficient in reliability consideration of WDSs. The writers agree with the discussers that the usual method of calculating reliability of a WDS as the arithmetic mean of nodal reliabilities is inappropriate. In a recent study, the writers used a geometric mean of nodal reliabilities as one of the factors in determining the system reliability in reliability-based analysis and design of WDSs (Gupta and Bhave 1994; Gupta and Bhave 1996).

The writers agree with the discussers that the behavior of a WDS under deficient conditions is quite complex, that extensive field data would be necessary to determine R, n, and c values for a WDS, and also that further research is necessary in this field.

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