THE UNIVERSITY *of* LIVERPOOL

A HOLISTIC APPROACH TO THE OPTIMAL LONG-TERM UPGRADING OF WATER DISTRIBUTION NETWORKS

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ABSTRACT

As water networks get older, increasing maintenance costs impose serious restrictions on the operators necessitating proper planning for future upgrading to minimise costs. This thesis investigates this problem while specifically focussing on the optimal scheduling and magnitude of long-term upgrading strategies for water distribution networks in a hydraulically consistent and economical manner, with due regard to the performance of the network, social and environmental issues. In order to address these issues comprehensively, an Integrated Model has been formulated in this research, with separate modules for Network Design, Hydraulic Simulation, Assessment of System Performance, and the Analytic Hierarchy Process.

The features of novelty and originality done in this research are presented next.

A rule-based algorithm for head driven analysis of water supply networks is developed. It is used to effectively model the network in situations when conventional demand driven analysis techniques on which most commercial software are based, cannot cope, e.g., when some components are unavailable due to failure or maintenance and the pressure in the network is insufficient to meet demand fully. Using the algorithm, nodes with reduced outflow or no outflow due to insufficient pressure are identified systematically and their outflows are obtained in an iterative scheme. The algorithm is computationally efficient and gives results that compare very well with other methods of head driven analysis.

A model for optimal design and upgrading of a deteriorating network to minimise the present value of capital, repair and damage costs is developed. It uses linear programming to carry out a combined analysis of water distribution network economics and hydraulic performance over a predefined design horizon. The timing of upgrading over the entire planning horizon is based on dynamic programming. Maximum entropy link flows or least biased flows are used in the design to reduce the dimensionality of the upgrading problem. The model explicitly considers deterioration over time of both the structural integrity and hydraulic capacity of every pipe. It simultaneously considers the upgrading options of paralleling and replacement of pipes and it can also be used for rehabilitation strategies. It has the capability of considering joint water pricing and network upgrading policies.

The Integrated Model is set up in a multi-objective framework by inclusion of the module for the Analytic Hierarchy Process (AHP). The AHP is a popular multi-criteria decision-making tool for combining qualitative and quantitative decision factors by assigning them with relative importance weights to obtain an overall ranking of upgrading options. In this thesis, the AHP is used for the first time in the Integrated Model in the manner described herein, to solve the long-term upgrading problem. This is done in a holistic fashion to cover hydraulic, economic, social and environmental issues together with issues related to the level of service like reliability and failure tolerance.

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DEDICATION

This thesis is dedicated to my parents Mr. and Mrs. Paul Kawooya, my wife Sylvia and children Paula and Paul-Solomon; all my sisters and brothers, Regina, Pauline, Winfred, Anna, Barbara, Henry, Joseph, Emmanuel and Martin.

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NOTATION

A	a matrix of comparisons.
A_{pu}	pump characteristics curve coefficients.
age_{ij}	number of years from the time of installation for link *ij*.
age_{ijm}	number of years from the time of installation for segment *m* of link *ij*.
a_{ijm}	roughness growth rate (mm/year) for segment *m* of link *ij*.
a_{ij}	roughness growth rate (mm/year) for link *ij*.
a_{jk}	elements of matrix A, for $j, k = 1, 2, \ldots, n$.
a_l	the probability that link *l* is available.
α_τ	the discount factor, $(1+r)^{-v}$.
B_{pu}	pump characteristics curve coefficients.
$BC(d_{ij})$	burst cost or direct repair cost for existing link *ij*.
$BC(dn_{ij})$	burst cost or direct repair cost for new replaced link *ij*.
$BR(d_{ij}, st)$	failure rate for link of diameter d_{ij} at time step *st* in expected bursts/km/year.
$BR(dn_{ij}, st)$	failure rate for pipe *ij* of diameter dn_{ij} at time step *st* in expected bursts/km/year.
b	annually compounded interest rate at which borrowed capital is raised.
b_j	node specific constant.
β_τ	product of a discount factor $(1+r)^{-v}$ and a price increase factor $(1+c)^{v}$.
Ce	Hazen William's coefficient for existing pipe.
C_{eq}	equivalent Hazen-Williams coefficient for link segments in series.
C_{ij}	Hazen-Williams coefficient of link *ij*.
$C_{ij}(t)$	Hazen-Williams hydraulic conductivity value for link *ij* coefficient in year *t*.
C_{net}	cost of network pipes.
C_{peq}	equivalent Hazen-Williams coefficient for parallel link segments.
Cp	Hazen William's coefficient for parallel pipe.
C_{p1}, C_{p2}	Hazen-Williams coefficients for link segments 1 and 2 in parallel.
C_{pu}	pump characteristics curve coefficients.
C_τ	cost of adding capacity in an upgrading model in design phase τ.
C_1, C_2	Hazen-Williams coefficients for link segments 1 and 2 in series.
CB_{ijm}	repair cost per break of segment *m* of link *ij*.
c	general annual rate of increase in construction costs.
c_j	node specific constant.
c_p	parallel pipe cost constant.
c_r	replaced pipe cost constant.
D_D	set of commercially available link diameters on the candidate list.
D_{eq}	equivalent diameter for link segments in series.
D_{peq}	equivalent diameter for link segments in parallel.
D_{ij}	diameter of link *ij*.
D_{ijm}	diameter of segment *m* of link *ij*.
$D_{ijm,max}$	upper bound on diameters of segment *m* of link *ij*.
$D_{ijm,min}$	lower bounds on diameters of segment *m* of link *ij*.

$D_{ij,max}$	upper bound on diameters of link ij.
$D_{ij,min}$	lower bounds on diameters of link ij.
Dp	existing pipe diameter of the upgrading model in Phase II
D_{p1}, D_{p2}	diameters for parallel link segments $p1$ and $p2$.
DGR	percentage annual rate of increase of the base demand.
Dn	new pipe diameter of the upgrading model in Phase I
DN_{ij}	diameter of the new water main, ij, after rehabilitation
DO_{ij}	diameter of the existing water main for link ij in rehabilitation model.
Dp	parallel pipe diameter of the upgrading model in Phase II
Dr	replaced pipe diameter of the upgrading model in Phase II
D_t	community demand (including losses) when the price of water is P_t in year t (e.g. in kilolitres/year).
D_τ	capacity required at the end of period τ.
D_1, D_2	diameters for link segments 1 and 2 in series.
d	length of the design horizon in years.
d'	percentage difference of compared values on the Scale of Relative Intensities.
d_{ij}	diameter of existing link ij in the rehabilitation model.
dn_{ij}	diameter of new replaced link ij in the rehabilitation model.
$\Delta q_{lp}^{(k)}$	correction to be applied to the estimated link flow rate.
$\underline{\Delta H}$	the vector of the respective corrections to nodal heads.
$\underline{\Delta Q}$	the vector of the respective corrections to nodal outflows.
e_{0ijm}	initial roughness (mm) for segment m of link ij at time of installation.
e_{0ij}	initial roughness (mm) of link ij at time of installation.
$E_1, E_2, \ldots, E_n$	general set of activities in the analytic hierarchy process.
$\underline{F}$	vector of respective values of the nodal continuity expressions.
$\mathbf{F}$	system of equations.
f	subscript for demand patterns or flow regime.
$f1$	pipeline costs which include installation, paralleling, replacement, and repair costs.
$f2$	cost of setting up construction plant and machinery at the beginning of each design phase.
$f3$	costs that vary with the magnitude of installed capacity.
$f1_a$	costs of new pipeline including supply, installation and the related discounted future failure costs.
$f1_b$	costs of parallel pipeline including supply, installation and the related discounted future failure costs.
F_j	continuity equation for node j.
$FCF(LU_{ij})$	failure cost factor for land use, LU_{ij}, for link ij.
Φ	break repair cost exponent.
φ	factor for reducing the Newton step.
G_{ijm}	hydraulic gradient for segment m of link ij (dimensionless).
g	master function to be minimised.
γ_{br}	break repair cost constant.
γ_p	parallel pipe cost constant.
γ_r	replaced pipe cost constant.

$\underline{H}$	vector of unknown heads.
H_{critN1}	pressure of most critical node during identification of no-flow nodes.
H_{critN2}	pressure of next most critical node during identification of no-flow nodes.
$H_{critP1}, \ldots, H_{critPn}$	respective pressure values of critical nodes in the same pressure contours during the stage of identifying partial-flow nodes and the sets of nodes are designated sets $P1, \ldots, Pn$ respectively.
$H_{critK1}, \ldots, H_{critKn}$	respective pressure values of critical nodes in the same pressure contours during the stage of identifying partial-flow nodes and the sets of nodes are designated sets $K1, \ldots, Kn$ respectively.
H_{elevj}	nodal elevation for node j.
H_i	piezometric head at node i.
H_j	piezometric head at node j.
H_j^{des}	desired head to satisfy the demand.
H_j^{min}	minimum required head at node j.
H_s^{des}	required source head to fully satisfy all the demand nodes.
H_{min}	absolute minimum pressure for outflow to be possible
H_{minj}	lower bound on the nodal head of node j.
H_{maxj}	upper bound on the nodal head of node j.
$H_{min,tn}$	minimum allowable head at terminal node.
H_{res}	desirable pressure above which nodal outflow can be fully satisfied.
H_s	the head available at the source.
H_s^{min}	the source head above which outflow just begins at any node of the network.
$H_{s,j}^{min}$	the head at the source below which outflow at node j is zero or deemed unsatisfactory.
$H_{s,j}^{res}$	the head at the source above which the demand at node j is fully satisfied.
H_{statj}	static head of node j.
H_p	head lift of the pump.
H_{prv}	pressure reducing valve setting for constant outlet head.
HG_{ijm}	hydraulic gradient for segment m of link ij (m/km).
h_{ij}	head loss for link ij.
h_{ijm}	head loss for segment m of link ij.
h_{ijmf}	head loss for segment m of link ij for the f^{th} flow regime.
h_{sp}	known head loss for path between fixed-grade nodes.
h_{spf}	known head loss for path between fixed-grade nodes for the f^{th} flow regime.
η	dimensionless conversion factor for units.
IJ_j	total number of links incident on node j.
IJ_{lp}	set of all links in loop lp.
IJ_{sp}	set of all links in a path specified between fixed-grade nodes.
IJ_{sj}	set of all links in a path specified between the source node and node j.
IJ_{tn}	links in a path specified between a source and a terminal node.
IJ_{tnf}	links in a path specified between a source and a terminal node for the f^{th} flow regime
J_H	Jacobian matrix for heads.
J_x	Jacobian matrix for x's.
J_{HQ}	Jacobian matrix for the head-flow system of equations.

$J(t)_{ijm}$ break rate of segment m of link ij (breaks/km/year) in year t.
K_{ij} resistance coefficient of link ij.
L_{ij} length of link ij.
Le existing pipe length of the upgrading model in Phase II.
Le_{ij} existing pipe length for link ij of the upgrading model in Phase II.
Le_p existing paralleled pipe length of the upgrading model in Phase II.
L_{eq} equivalent length for link segments in series.
L_{peq} equivalent length for parallel link segments.
lp_{ij} consists of all loops sharing link ij.
l_{ijm} length of segment m of link ij.
Ln the new pipe length of the upgrading model in Phase I.
LN length of the water main to be replaced with new pipes.
$LN_{ij,1}$ length of link ij in design period 1, to be replaced with new pipes.
$LN_{ij,2}$ length of link ij in design period 2, to be replaced with new pipes.
$LO_{ij,1}$ length of link ij in design period 1, to be left as is.
$LR_{ij,1}$ length of link ij in design period 1, to be relined.
$LR_{ij,2}$ length of link ij in design period 2, to be relined.
$LT_{ij,1}$ total length of reach ij in design period 1 in the rehabilitation model.
LO length of the existing main to be left in its present condition.
Lp parallel pipe length of the upgrading model in Phase II.
Lp_{ij} parallel pipe length for link ij of the upgrading model in Phase II.
Lr_{ij} replaced pipe length for link ij of the upgrading model in Phase II
LU_{ij} land use related to link ij.
L_1, L_2 lengths for link segments 1 and 2 in series.
L_{p1}, L_{p2} lengths for link segments $p1$ and $p2$ in parallel.
Λ total number of phases or periods.
λ_{max} the maximum or principal eigenvalue.
$MAXDIAL_{ij}$ maximum link segment diameter for the new link ij, in Phase I.
$MINDIAL_{ij}$ minimum link segment diameter for the new link ij, in Phase I.
$MAXDIAP_{ij}$ maximum parallel link segment diameter for link ij.
$MINDIAP_{ij}$ minimum parallel link segment diameter for link ij.
$MAXDIAR_{ij}$ maximum replaced link segment diameter for link ij.
$MINDIAR_{ij}$ minimum replaced link segment diameter for link ij.
$N1$ set of nodes with pressure values about H_{critN1}.
$N2$ set of nodes with pressure values about H_{critN2}.
NF total number paths specified between fixed-grade nodes the network.
NFH number of fixed head nodes.
N_{ij} number of segments specified for link ij
NJ number of nodes in the network.
NL number of primary loops in the network.
Nl total number of links in the network.
Nn_j nodes connected to node j.
NT total number paths specified between a source and a terminal node.
NPR_{ij} present value of the replacement cost of a pipe with diameter dn_{ij} in time period T_{rij}.
NS number of source nodes in the network.
n_j nodal exponent whose value ranges from 1.5 to 2.
nc an exponent.
$p(0)$ probability that no pipe is unavailable.
$p(l)$ probability that only pipe l is unavailable.

$p(l,m)$ probability that only pipes l and m are unavailable.

P_0 price per unit of water or water tariffs (e.g. in \$/m^3, dollars/kilolitre) in the base year.

P_t price per unit of water or water tariffs (e.g. in \$/m^3, dollars/kilolitre) in the year t.

POP_0 population served in base year.

POP_t population served in year t.

$PREL$ price elasticity of demand.

$PVNB_{ij}$ repair costs related to new replaced link ij in rehabilitation model.

$PVOB_{ij}$ repair costs related to existing link ij in rehabilitation model.

Pr_j pressure at node j.

$Pr_j^{\#}$ nodal pressure at which a given proportion of the demand is provided.

Q_{inst} installed capacity in a particular phase in l/s.

Q_j^{avl} available outflow at node j.

Q_s^{avl} sum of all available nodal outflows.

Q_j^{req} required outflow (demand) at node j.

Q_j inflow or outflow at node j and it represents the nodal demand in cases where j is a demand node.

Q_{jf} inflow or outflow at node j for the fth flow regime; and it represents the nodal demand in cases where j is a demand node.

Q_{req} sum of the nodal demands.

Q_s^{req} sum of all nodal demands.

Qe_p existing pipe flow in upgrading model.

Q_{0j} base demand for node j.

Qp parallel pipe flow in upgrading model.

Qp_{ijm} parallel link segment flow for link ij.

Q_{pu} flow delivered by the pump.

$Q(l)$ total network outflow when link l is unavailable.

$Q(0)$ total network outflow when all components are available.

$Q(l,m)$ total network outflow when links l and m are unavailable.

q_{ij} discharge or flow in link ij.

$q_{ij}^{(k)}$ the corrected flow rate for link ij.

$q_{ij}^{(k-1)}$ an estimated flow rate for link ij.

q_{ijmf} flow in segment m of link ij for the fth flow regime.

R^2 regression correlation coefficient.

$R(dn_{ij})$ cost/m length of replacing the pipe with diameter dn_{ij}.

R_j resistance constant for node j.

R_s the resistance constant.

Re network reliability.

REP_{ijm} failure costs for segment m of link ij.

r_τ installed network capacity in design phase τ.

r discount rate.

$r(0)$ ratio of total nodal outflows to the total demand when all pipes are available.

st time step in the rehabilitation model.

s_1 final capacity for each design phase τ.

s_τ existing capacity at the beginning of design phase τ.

$T1$ lower limit for the end of Phase I.

$T2$	upper limit for the end of Phase I.
$TDEL$	delay in years caused by the pricing policy.
T_{rij}	time step when the existing pipe is replaced for link ij given a discount rate of r.
t	time in years.
τ	design phase.
tb	the time when a pipe in a given design period starts to incur repair costs.
ts	first year of a given design period.
tr	last year of a design period.
V_{costs}	generalised costs that vary with the magnitude of installed capacity.
VC	installed capacity cost coefficient.
VE	installed capacity cost exponent.
v	number of years preceding a design phase.
v_{ijm}	flow velocity in segment m of link ij.
$v_{ijm,min}$	lower bound on link segment velocity.
$v_{ijm,max}$	upper bound on link segment velocity.
$v_{ij,min}$	lower bound on link velocity.
$v_{ij,max}$	upper bound on link velocity.
w	fractional growth rate of population for linear growth relationship.
$w_1, w_2, \ldots, w_n$	actual physical measurements or elements of a priority vector.
ω	factor for the initial rate of decrease of g.
$\underline{x}$	system of equations.
y_{AC}	failure rate for asbestos cement (AC) pipes.
y_{CICL}	failure rate for cast iron cement-lined (CICL) pipes.
ψ	step-length adjustment parameter.

CHAPTER ONE

INTRODUCTION

1.1 BACKGROUND

A water distribution network is designed to supply water to the consumers to meet standards of quantity, quality and pressure. Spiralling demand as the years go by culminates in the impaired ability of the network to achieve these functions. Furthermore, the aging network also experiences a deterioration of its components. At this point, the quality of service to the consumers drops significantly and the need to upgrade or rehabilitate the network is inevitable. Considering the vast investment required, careful planning for long-term strategies is an economic and effective approach to solving the problem. The water distribution network is typically the most expensive component of a water supply system in terms of capital cost, operation and maintenance (Selvakumar et al., 2002) and for this reason, this research focuses on the network.

The basic network upgrading or master-planning problem is concerned with how and when to increase network capacity in order to minimise the present worth of all costs involved. Demand plays an important role in capacity planning. Present demands are forecast or projected into the future, and the flow at the end of the design period is selected as the target for setting the capacity of the system. The policy of upgrading considers the trade-off between the economies-of-scale savings of large expansion sizes, versus the cost of installing capacity before it is really needed. Definitely, overbuilding ties up valuable resources, yet under sizing the system may result in its inadequacy. The tendency has been to tackle this problem of sizing increments to capacity using conservative designs for arbitrarily long planning periods, that is, to overbuild in order to ensure safe and adequate supplies. With increasing budget constraints, this approach is no longer as effective.

Upgrading or expansion of an existing system to meet current and future demands of flow rate and pressure and quality, has always been a field of interest to engineers. Both the adaptation of existing technologies, and the development of new innovative technologies, is required to improve the efficiency and cost effectiveness of future and existing water supply systems. This process is particularly enhanced by the rapidly evolving computer technology, whose role lies at the forefront of solving the complex problem of capacity expansion. A good long-term upgrading or rehabilitation strategy should be able to improve service levels to customers by improving water quality, improving pressure, reducing leakage, bursts and interruptions to supply, all at an economic cost. Bursts and interruptions to supply are caused by poor structural condition and workmanship. Poor pressure arises from inadequate size either due to pipes being too small or due to the build up of encrustation.

Other concerns worth consideration are the performance of the network with regard to key parameters such as reliability, social and environmental issues. Reliability is associated with two types of failure, hydraulic failure and mechanical failure. Hydraulic failure refers to the inability of the network to meet demand at adequate pressure. Mechanical failure relates to components out of service (e.g. due to bursts and valve blockages) and their impact on the network's performance. An aging network tends to exhibit frequent incidences of failure to cope with emergency conditions like fire demands; broken pipes, and pump failures. Such incidences are bound to have a significant impact on the reliability of the system; for this reason, reliability has been selected as one of the key performance assessment parameters in this research. Another important performance assessment parameter is redundancy, which addresses existence of spare capacity for use in abnormal operating conditions. It can be quantified by a parameter that has been introduced by Tanyimboh and Templeman (1998) and referred to as component failure tolerance. This parameter has been selected as a key performance assessment parameter because it can effectively be used to show the degree of vulnerability of a network when some of its components are unavailable or taken out of service for repair.

Social and environmental issues are not easy to quantify, but a provision should be made for assessing the impact of the strategy adopted on the society and on the environment. Any strategy that is deemed appropriate for the long-term upgrading problem should not have an adverse impact on the social well being of the customers and on the environment.

Various researchers have attempted to develop models to address this problem using different approaches. Some of the models have laid emphasis on network economics concentrating on cost–flow functions; minimisation of total costs; and/or the relationship between supply, demand and the price of water e.g. the model by Dandy et al. (1985). A number of simplifications in the hydraulic analysis area are usually made to ease the network-upgrading problem. There are individual asset-based models, which deal with network components in isolation of their interactive hydraulic and performance nature e.g., the models by Walski and Pellicia (1982) and Loganathan et al. (2002). Models that provide a more comprehensive approach to the problem are referred to as system-wide models. These models consider the network economics, hydraulics and performance explicitly with the full interaction of all network components. They address the timing and magnitude of upgrading and some of the models can be used to identify the particular components for rehabilitation and/or replacement e.g., models by Lansey et al. (1992) and Kleiner et al. (2001). However these models are limited to very small networks save for a few that use genetic algorithms for optimisation and are thus crippled by the high computational requirements. Due to the level of complexity, these models neither include performance assessment measures nor provide for socio-environmental issues.

It is needless to mention that the highly non-linear nature of the network-upgrading problem implies that there are complications involved and a requirement for powerful optimisation techniques. However, the fact that these methods have the disadvantage of being time-consuming only serves as a disincentive to use them. Also, despite all the advancement in developing computer programs for simulating network performance, there are no commercially available packages for pressure dependent network analysis. This analysis is the most appropriate technique for modelling abnormal network operating conditions or situations when the pressure in the network is insufficient.

This research seeks to address most of the weaknesses mentioned above.

1.2 OBJECTIVES OF THE RESEARCH

The objectives of this research are as follows:

1) To develop a reliable planning tool or model, that can be used to carry out the long-term upgrading of water distribution networks in an economically efficient and hydraulically consistent manner, giving due attention to social and environmental issues.

2) To ensure that the model can be used to determine the magnitude and the optimal scheduling of the upgrading and to assess the performance of the networks.

3) To test the practical capability of the model by applying it to hypothetical and real-life networks.

1.3 A BRIEF DESCRIPTION OF THE METHODOLOGY

In order to achieve these aims and address the optimal long-term upgrading problem in a holistic fashion, an Integrated Model has been formulated with separate modules for network design, hydraulic simulation, assessment of network performance and the Analytic Hierarchy Process. These modules have interactive roles in the Integrated Model, in that data output from one module is the input to another module as described next. For various design horizons and design options, the Network Design Module is used to obtain least cost designs by optimization, together with the timing and magnitude for the long-term upgrading strategies. The Hydraulic Simulation Module is then used to ensure that each network design is hydraulically consistent, and to simulate stressed network conditions. The results are fed into the Performance Assessment Module to establish the reliability and failure tolerance of each network design. The Analytic Hierarchy Process is then used to obtain priority weights or ranking of each design option with respect to various criteria like the present value of project costs, performance, environmental and social issues. The choice of the best upgrading design option is made on the basis of these priority weights. The incorporation of the module for the Analytic Hierarchy Process implies that the model is set up in a multi-objective framework that enables it to handle the complex problem of network upgrading in an efficient manner. The novelty here lies in the fact that though the Analytic Hierarchy Process is a very important decision-making tool that has been used for numerous projects, it has never been applied to water distribution networks in the manner described in this research.

A key driving force in the solution methodology used in this research is the bid to reduce the dimensionality of the problem especially in the network design module. For example, the use of the maximum entropy flow distribution approach (Yassin-Kassab et al., 1999) to provide least biased flows has reduced the problem into one that can be solved by linear optimisation. The idea of the segmental approach for optimisation by Alperovits and Shamir (1977), coupled with techniques of reducing the number variables in terms of commercially available diameters on the candidate list for each individual link, are some of the other features that have been used to facilitate this process.

1.4 LAYOUT OF THESIS

The main background and literature review material in this thesis is arranged as described next. In Chapter 2, the analysis of water distribution networks is addressed, with due regard to the conventional demand driven analysis, extended period simulation and the concept of pressure-dependent network analysis. Methods of designing water distribution networks at minimum cost are also reviewed. The concepts of dynamic programming and the assessment of network performance using reliability and failure tolerance are then presented. Chapter 3 presents a detailed review of long-term rehabilitation and upgrading techniques. This chapter reviews models that are mainly based on network economics, individual asset-based models and system-wide models.

Chapter 4 presents a new model for network upgrading, which considers deterioration over time of both the structural integrity and hydraulic capacity of every pipe, allows for the direct and indirect failure costs and can be used to obtain the timing and magnitude of upgrading. Methods of reducing the dimensionality of the upgrading problem as mentioned earlier in Section 1.3, are also presented. In Chapter 5, a new algorithm for performing head-dependent modelling of water distribution networks is presented. This algorithm is then used to develop the Hydraulic Simulation Module and the Performance Assessment Module, followed by a number of examples to demonstrate the efficacy of these modules.

Chapter 6 introduces the notion of multiple-criteria decision-making, giving a review of different methods and focusing on a method called the Analytic Hierarchy Process. Chapter 7 dwells upon the module that has been developed for the Analytic Hierarchy Process and its application to a sample network. The Integrated Model is formulated in Chapter 8 and the interaction of the various modules developed in Chapters 4, 5, 6 and 7 is detailed. This model can be used for optimal upgrading strategies, ensuring system reliability, with due consideration of socio–environmental issues, network economics and hydraulics. The integrated model is applied to a hypothetical network and a real-life water distribution network as a case study to demonstrate its practicability and efficiency. Each of the Chapters 4, 5, 6, 7 and 8, has a detailed section for analysis and discussion of results related to the examples solved in the chapter. Finally, Chapter 9 winds up with a general summary of the main ideas, overall conclusions and suggestions for further research.

CHAPTER TWO

DESIGN, ANALYSIS AND PERFORMANCE ASSESSMENT OF WATER DISTRIBUTION NETWORKS

2.1 INTRODUCTION

A water distribution network is composed of nodes (reservoirs and pipe junctions); and links, which comprise of pumps, valves and pipes that form loops. The Operation and management of water distribution networks inevitably requires the building of network models to simulate the behaviour of the actual network. The models can then be used for operational analysis involving optimum pump scheduling and pressure and leakage control activities, assessing the performance of the networks, predicting the behaviour of the network as a result of an event such as a pipe burst, valve closure, etc. There are two general steps involved in creating a network model. The first one involves defining the network in terms of the nodes, their demands locations and elevations, and determining the sizes of the links. The second one involves analysing the network by solving the constitutive or governing equations for a general network, in order to obtain the flows in the links and piezometric heads at the nodes; and to satisfy pre-set hydraulic conditions such as minimum nodal pressure requirements.

Bhave (1978) classified the techniques for performing these general steps into two main categories namely: (a) "check design and optimisation" techniques and (b) "direct design and optimisation" techniques. The "check design and optimisation" techniques for a defined network involve making assumptions for parameters like link sizes, pumping pressures, etc. The network is analysed and checked repetitively with the adjustment of some parameters until hydraulic consistency is achieved. The network analysis checks are made by methods such as: the Linear theory method (Wood and Charles, 1972); the Newton-Raphson method (Martin and Peters, 1963) and the Hardy Cross method (Cross, 1936). The "check design and optimisation" techniques may require several trials and do not guarantee an optimum solution. On the other hand, the "direct design and optimisation" techniques can be applied for the least-cost sizing of network components using methods like linear programming (Alperovits and Shamir, 1977), non-linear programming (Jacobs, 1968) and dynamic programming (Liang, 1971, Templeman, 1982a). For a given network layout and nodal elevations, these methods are aimed at determining the appropriate network diameters and pump operating conditions so as to minimise the total cost while satisfying minimum and maximum allowable nodal heads and link velocities.

Analysis of the network may be done under the assumption that the nodal demands and reservoir water levels are constant. This assumption can be valid for a short period of time and the analysis is referred to as steady state analysis or static analysis (Bhave, 1991). In reality however, nodal demands and reservoir water levels vary with time. To maintain a satisfactory level of service to customers, the impact of the variation in demand has to be put into consideration. This requires the analysis of the network over a longer period of time typically referred to as extended period analysis, dynamic analysis or extended period simulation (Bhave, 1991).

The network analysis method based on the assumption that nodal demands can be fully satisfied at all times regardless of the network pressure is referred to as Demand Driven Analysis (DDA). Most of the commercial network analysis packages are based on DDA. However, it should be noted that the demand driven analysis technique is not effective for situations when the network pressures are insufficient. These low pressures can be caused by a burst pipe, pump or valve failures or excessive demand at some nodes. Such a deficient network condition requires pressure-dependent network analysis (Germanopoulos, 1985, Bhave, 1991, Tanyimboh, 1993) or Head Driven Analysis (HDA).

Assessment of the network performance is essential as an audit to gauge the extent to which the network meets the goals for which it was designed. Some of the important parameters used are reliability and failure tolerance (Tanyimboh and Templeman, 1998). Reliability is used to assess the network's ability to meet demand at adequate pressure under normal and abnormal conditions (e.g. when some components are unavailable due to bursts or maintenance activities). Failure tolerance is used to assess the vulnerability of the network to the unavailability of some of its components.

Section 2.2 presents the constitutive equations for water distribution network analysis. Sections 2.3 and 2.4 cover the analysis of water distribution networks for cases where the network layout is defined. Demand Driven Analysis is detailed first and it covers steady state analysis together with a summary on extended period analysis. Head driven analysis of water networks is then presented with a review of various methods for this type of network analysis. The optimum design problem, i.e. the least capital cost design of water networks, is also presented in this chapter and it covers the concepts of linear, non-linear and dynamic programming. Finally, the performance assessment of water distribution networks is briefly described, along with two key performance assessment parameters called reliability and failure tolerance.

2.2 CONSTITUTIVE OR GOVERNING EQUATIONS

The constitutive or governing equations for flow in water distribution networks are the pipe head loss equations, nodal flow equilibrium equations together with the equations of energy conservation for the loops and paths. These constitutive equations have to be simultaneously satisfied in network analysis.

2.2.1 Head Loss Equations

A broadly used equation for the head loss due to flow in a pipe or link ij is the Hazen-Williams equation (Bhave, 1991), which is given by

$$h_{ij} = \left(\frac{\eta L_{ij}}{C_{ij}^{1.852} D_{ij}^{4.87}}\right) q_{ij}^{1.852} = K_{ij} q_{ij}^{1.852} \qquad \forall ij \tag{2.1}$$

in which q_{ij} is the discharge or flow in the link ij; h_{ij} and C_{ij} are the head loss and Hazen-Williams coefficient for link ij respectively. Values of C_{ij} vary according to pipe conditions such as the material, age, diameter, etc., (Bhave, 1991, Jeppson, 1976). L_{ij} and D_{ij} are the length and diameter of link ij respectively. η is a dimensionless conversion factor for units (η = 10.68 in S.I. units). K_{ij} is the resistance coefficient for link ij and it has the form

$$K_{ij} = \frac{\eta \cdot L_{ij}}{C_{ij}^{1.852} \cdot D_{ij}^{4.87}} \tag{2.2}$$

There are other equations for head loss due to flow in a pipe such as the Darcy-Weisbach equation and the Manning formula (Bhave, 1991, Jeppson, 1976). However, Hazen-William's Eq. (2.1) is used throughout this thesis.

2.2.2 Continuity Equations

The algebraic sum of inflows to and outflows from a node must be zero for steady incompressible flow in a network. Thus, the flow continuity or equilibrium equations for each node j, j=1,..., NJ, may be written as

$$\sum_{i:H_i<H_j} q_{ij} - \sum_{i:H_i>H_j} q_{ij} = Q_j \tag{2.3}$$

where NJ is the number of nodes in the network; Q_j is the inflow or outflow at node j and it represents the nodal demand in cases where j is a demand node; H_i and H_j are piezometric heads at nodes i and j respectively. These piezometric heads are also referred to as total heads and for each node, they comprise of the sum of the nodal pressure and the nodal elevation.

2.2.3 Equations for Conservation of Energy

2.2.3.1 Loop Equations

Conservation of energy in a pipe network implies that the algebraic sum of head losses in pipes forming each loop must equal to zero. Thus the equation for each loop is

$$\sum_{ij \in IJ_{lp}} h_{ij} = 0 \qquad lp = 1,\ldots,Nl \tag{2.4}$$

in which IJ_{lp} represents a set of all links in loop lp and Nl is the total number of primary loops in the network.

2.2.3.2 *Path Equations*

For a given path in the network, the total head loss along the path should equal the difference in head between the end nodes of the path. Independent energy conservation equations can be written for individual paths. For paths with end nodes that are fixed-grade nodes, an equation for each path may be written as

$$\sum_{ij \in IJ_{sp}} h_{ij} = h_{sp} \qquad sp = 1,\ldots,NF \tag{2.5}$$

in which IJ_{sp} represents a set of all links in a path specified between fixed-grade nodes, NF is the total number of such specific paths in the network and h_{sp} is the known head loss for the path between the fixed-grade nodes.

If the end nodes of the path are a source s (where $s \in NS$) and a terminal node tn i.e., a node with no other nodes downstream of it; an equation for each path of this type is given by

$$\sum_{ij \in IJ_{tn}} h_{ij} = H_s - H_{tn} \qquad tn = 1,\ldots,NT \tag{2.6}$$

in which IJ_{tn} represents a set of all links in a path between a source and a terminal node; NT is the total number of paths specified between sources and terminal nodes; and NS is the total number of source nodes.

2.2.4 Head-flow Relationships of Network Components

A water supply network may include components such as pipes, pumps, tanks and valves. Bhave (1991) has shown how these components can be represented by head-flow relationships and incorporated into the network. A few examples are briefly described next.

2.2.4.1 Pipes

The head-flow relationship based on the Hazen-Williams equation may be written as

$$h_{ij} = H_i - H_j = K_{ij}\left|q_{ij}\right|^{nc-1} q_{ij} \qquad \forall ij \tag{2.7}$$

in which nc is considered to be equal to 1.852 and this value is used throughout this thesis. Whereas $\left|q_{ij}\right|^{nc-1}$ is always positive, q_{ij} and h_{ij} are both positive when $H_i > H_j$; and negative when the flow reverses (i.e. when $H_j > H_i$).

2.2.4.2 Pumps

The main purpose of a pump in a pipeline is to supply extra head. Pumps are provided in a network to boost up pressure at some points within the system. The head-flow relationship of a pump is referred to as the pump head-discharge curve and it may be approximated by a parabolic curve as

$$H_p = A_{pu}Q_{pu}^2 + B_{pu}Q_{pu} + C_{pu} \tag{2.8}$$

in which A_{pu}, B_{pu}, and C_{pu} are constants that are usually set by the pump manufacturer. These constants may also be determined by fitting Eq. (2.8) to three points taken from the expected working range of the pump head-discharge curve. Q_{pu} is the flow delivered by the pump. H_p is the head lift of the pump or the difference between the heads at the upstream and downstream nodes of the pump.

2.2.4.3 Non-Return Valve (NRV)

Non-return valves permit flow in one direction only. The head-flow relationship for a pipe fitted with a non-return valve is

$$q_{ij} = \begin{cases} \dfrac{H_i - H_j}{K_{ij}^{0.54}\left|H_i - H_j\right|^{0.46}} & \text{if} \quad H_j \le H_i \\ 0 & \text{if} \quad H_i > H_j \end{cases} \tag{2.9}$$

2.2.4.4 Pressure Reducing Valve (PRV)

A pressure-reducing valve produces a constant outlet pressure for a range of higher inlet pressures. It is used to ensure that the pressure downstream of where it is positioned do not exceed a pre-set value. An example of such a situation is where the downstream pressure is considered excessive. The head-flow relationship for a pipe fitted with a PRV is given by

$$q_{ij} = \begin{cases} \dfrac{H_{prv} - H_j}{K_{ij}^{0.54}\left|H_{prv} - H_j\right|^{0.46}} & \text{if} \quad H_j \le H_{prv} \le H_i \\ \dfrac{H_i - H_j}{K_{ij}^{0.54}\left|H_i - H_j\right|^{0.46}} & \text{if} \quad H_j < H_i < H_{prv} \\ 0 & \text{if} \quad H_j > H_{prv} \end{cases} \tag{2.10}$$

in which H_{prv} is the pressure reducing valve setting.

It should be noted that the relationships for pumps and valves have been included in this thesis for completeness and that this research is not particularly concerned with them.

2.3 DEMAND DRIVEN ANALYSIS

This section presents the analysis of the network based on the assumption that nodal demand satisfaction can always be attained even when the network pressure is insufficient. This analysis, which is carried out in an instantaneous moment or over a very short period of time, is referred to as steady state analysis and is presented first. A summary of the analysis over a longer period or extended period analysis then follows.

2.3.1 Steady State Analysis

Nodal demands and reservoir water levels are assumed to remain constant. These assumptions are valid for a very short period of time and for certain applications e.g. modelling the network operation at the design stage for predicted peak demands. The external flows, pipe lengths, diameters and roughness characteristics are often pre-specified e.g. when using the technique referred to by Bhave (1978) as "check design and optimisation". The analysis problem has three types of variables and these are the pipe head losses h_{ij}, the nodal heads H_j and the pipe flow rates q_{ij}. The constitutive equations may be set up as systems of equations, with any of these three variables as the basic unknown parameters. Such formulated equations are designated by these unknown parameters as described next.

2.3.1.1 Systems of Equations

A - Pipe flow rates as unknowns

Network analysis equations formulated with the pipe flow rates as the basic unknown parameters or independent variables are referred to as q-equations (Bhave, 1991; Jeppson, 1976). For example, the equations for head loss and continuity, Eqs. (2.1) and (2.3), respectively, have been written with flow rates, q_{ij} as the unknowns. This implies that the basic unknowns in the loop and path equations, i.e. Eqs. (2.4), (2.5) and (2.6), are the flow rates q_{ij}.

B - Nodal heads as unknowns

Equations formulated with nodal heads as the basic unknowns are referred to as H-equations (Bhave, 1991). The flow rate in each pipe is expressed in terms of the nodal heads. For example, Eq. (2.1) may be re-written by substituting h_{ij} with the nodal heads from Eqs. (2.7) to give

$$q_{ij} = \eta C_{ij} D_{ij}^{2.63} \frac{(H_i - H_j)}{L_{ij}^{0.54} |H_i - H_j|^{0.46}} \qquad \forall ij \in Nl \qquad (2.11)$$

in which, here, $\eta = 0.2785$ in S.I. units.

The continuity equations, Eqs. (2.3) can incorporate Eq. (2.11) to become

$$F_j = \sum_{i:H_i > H_j} \left(\frac{H_i - H_j}{K_{ij}} \right)^{(0.54)} - \sum_{i:H_i < H_j} \left(\frac{H_j - H_i}{K_{ij}} \right)^{(0.54)} - Q_j = 0; \ \forall j = 1, \ldots, NJ \qquad (2.12)$$

in which F_j represents the continuity equation for node j. Equations (2.12) describe the flow in a pipe network without any need for the loop or path equations. Thus, these H-equations can conveniently be set up for the nodes and solved simultaneously. For a feasible solution to be obtained, the number of continuity equations required should be equal to the unknown nodal heads and at least one nodal head should have a fixed value.

C – Loop-flow corrections as unknowns

Equations formulated by taking loop-flow corrections as the basic unknowns are known as Δq equations (Bhave, 1991). These equations are formulated by making an initial assumption of the pipe flow rates to satisfy nodal flow continuity (Eq. 2.3). However, the assumed flows will not generally satisfy the loop-head loss relationship of Eq. (2.4). Thus, the pipe flow rates are adjusted by applying loop-flow corrections around each loop and treating these corrections as the basic variables. Hence,

$$q_{ij}^{(k)} = q_{ij}^{(k-1)} + \sum_{lp \in lp_{ij}} \Delta q_{lp}^{(k)} \qquad \forall ij \in Nl \tag{2.13}$$

in which $\Delta q_{lp}^{(k)}$ is a loop-flow correction applied according to the flow direction for all pipe flows in loop *lp*; $q_{ij}^{(k-1)}$ is an estimated flow rate and $q_{ij}^{(k)}$ is the corrected flow rate. It should be noted that bracketed superscripts and subscripts have been adopted to indicate values that apply in successive iterations in any iterative scheme and this convention is maintained throughout herein. lp_{ij} consists of all loops sharing link *ij*, which implies that $\sum_{lp \in lp_{ij}} \Delta q_{lp}^{(k)}$, $\forall ij \in Nl$, is the summation of the corrections of all loops to which link *ij* belongs.

The unknows in Eqs. (2.13) are the Δq_{lp}; and from the loop-head loss relationship of Eq. (2.4), a set of Δq equations based on the unknowns can be formulated as

$$\sum_{ij \in IJ_{lp}} K_{ij} \left(q_{ij}^{(k-1)} + \sum_{lp \in lp_{ij}} \Delta q_{lp}^{(k)} \right)^{nc} = 0 \qquad \forall ij \in Nl; \quad lp = 1, \ldots, NL \tag{2.14}$$

Each loop has its individual equation making a total of *NL* equations in the set of Eqs. (2.14). These equations are solved simultaneously in an iterative manner to meet pre-specified convergence criteria.

2.3.1.2 Solution of the Equations

The solution of the conventional pipe network analysis problem has evolved over time into three main numerical approaches. These are the Linear theory method (Wood and Charles, 1972), the Newton-Raphson Method (Martin and Peters, 1963) and the Hardy-Cross Method (Cross, 1936, Jeppson, 1976). These methods are simply techniques of solving a system of non-linear stationary point conditions.

Each is iterative in nature and begins with an initial trial solution. Procedures that involve solving a system of equations are applied to obtain a new solution. The new solution is compared with the previous solution and this procedure is repeated until the difference between consecutive solutions is less than a pre-specified tolerance. A brief description of each of these methods follows next.

A – Linear theory method

This method involves linearisation of the non-linear term in the Hazen-Williams Eq. (2.1) and merging it together with the pipe resistance coefficient (Eq. 2.2) to give

$$h_{ij}^{(k)} = \left(K_{ij}\left[q_{ij}^{(k-1)}\right]^{nc-1}\right) \cdot q_{ij}^{(k)} \qquad \forall ij, \quad k=1,2 \tag{2.15a}$$

in which $q_{ij}^{(0)}$, $\forall ij$, is usually set to unity as suggested by Wood and Charles (1972).

$$h_{ij}^{(k)} = \left(K_{ij}\left[\frac{q_{ij}^{(k-1)} + q_{ij}^{(k-2)}}{2}\right]^{nc-1}\right) \cdot q_{ij}^{(k)} \qquad \forall ij, \quad k=3,4,5,6,\ldots \tag{2.15b}$$

The linear equations (2.15) are used to form a set of loop head loss and nodal flow continuity equations. This set or system of linear equations formed, can be solved simultaneously by any suitable algorithm, for example Gaussian elimination (Burden and Faires, 1993). The solution obtained is an approximation since assumed values are used for $q_{ij}^{(k-1)}$ to obtain $q_{ij}^{(k)}$. To improve on the accuracy of the solution, an iterative scheme has to be used. This scheme starts off by using Eq. (2.15a) for the first and second iterations. Then, from the third iteration onwards, the flows for each iteration $q_{ij}^{(k)}$ are obtained by using the average of the assumed value $q_{ij}^{(k-2)}$ and the obtained value $q_{ij}^{(k-1)}$ from the preceding iteration (k-1) as shown in Eq. (2.15b). This scheme by Wood and Charles (1972) avoids oscillations in the value of $q_{ij}^{(k)}$ and ensures faster convergence. The iterations are continued until convergence is achieved i.e. when the flows obtained from successive sets of calculations have almost the same values.

The main advantage of this method is that there is no need for making initial estimates for flows and it has good convergence characteristics (Wood and Charles, 1972).

B – Newton-Raphson method

The Newton-Raphson iterative scheme for the solution of a system of non-linear equations was first proposed by Martin and Peters (1963). Since then, it has been used by many researchers e.g. Shamir and Howard (1968), Lemieux (1972), Rao and Bree (1977) and others. For a single function, $F(x)=0$, in one variable, x, the Newton-Raphson formula can generally be stated as

$$x^{(k+1)} = x^{(k)} - \frac{F\left(x^{(k)}\right)}{dF\left(x^{(k)}\right)/dx} \tag{2.16}$$

in which $dF\left(x^{(k)}\right)/dx$ is the derivative of $F(x)$ evaluated at x^k. For a system of equations, the corresponding formula is as follows (Burden and Faires, 1993)

$$\underline{x}^{(k+1)} = \underline{x}^{(k)} - \left(J_x\right)^{-1}_{(k)} \underline{F}\left(\underline{x}^{(k)}\right) \tag{2.17}$$

in which $\underline{x}$ is the vector of the variables; $\underline{F}$ is the vector of function values for the system of simultaneous equations at the point $\underline{x}$ and J_x is the Jacobian or the matrix of the first partial derivatives of each F with respect to each of the x's.

By analogy with the continuity equations at the nodes, Eqs. (2.12), unbalanced residual flow at the nodes can be written as

$$\underline{F}\left(\underline{H}\right) = 0 \tag{2.18}$$

in which, here, $\underline{F}$ is the vector of respective values of the nodal continuity expressions and $\underline{H}$ is the vector of unknown nodal heads.

From Eqs. (2.17), the Newton-Raphson formulation for the continuity expressions is

$$\underline{H}^{(k+1)} = \underline{H}^{(k)} - \left(J_H\right)^{-1}_{(k)} \underline{F}\left(\underline{H}^{(k)}\right) \tag{2.19}$$

in which J_H is the Jacobian matrix for the unknown nodal heads. The inversion of the Jacobian is computationally expensive and thus, this process can be avoided by using the updating scheme (Shamir and Howard, 1968) that is described next.

$$\underline{\Delta H}^{k} = -\left(J_H\right)^{-1}_{(k)} \underline{F}\left(\underline{H}^{(k)}\right) \tag{2.20}$$

in which $\underline{\Delta H}$ is the vector of the respective corrections to nodal heads. Multiplying both sides of Eq. (2.20) by $\left(J_H\right)_k$ yields

$$\left(J_H\right)_{(k)} \underline{\Delta H}^{(k)} = -\underline{F}\left(\underline{H}^{(k)}\right) \tag{2.21}$$

The elements of the Jacobian matrix, J_H, are given by

$$\frac{\partial F_j}{\partial H_i} = 0.54\left(\frac{|H_i - H_j|^{-0.46}}{K_{ij}^{\ 0.54}}\right) = \frac{\partial F_i}{\partial H_j}; \quad \forall j, \forall i : i \neq j \tag{2.22}$$

$$\frac{\partial F_j}{\partial H_j} = -0.54 \sum_{\substack{i \in Nn_j \\ i \neq j}} \left(\frac{|H_i - H_j|^{-0.46}}{K_{ij}^{\ 0.54}}\right); \quad \forall j \tag{2.23}$$

where Nn_j represents nodes connected to node j. For a network with NFH fixed head nodes, the size of the Jacobian matrix is $(NJ\text{-}NFH)$ x $(NJ\text{-}NFH)$.

The linear Eqs. (2.21) are solved simultaneously for the corrections $\underline{\Delta H}^{(k)}$ using any suitable algorithm, e.g., Gaussian elimination (Burden and Faires, 1993). The corrections are then used to obtain new estimates for the nodal heads using

$$\underline{H}^{(k+1)} = \underline{H}^{(k)} + \underline{\Delta H}^{(k)} \tag{2.24}$$

To obtain the final solution for nodal head values, an iterative scheme is used. This scheme involves making initial estimates H_j^k for head values at nodes j; then for each iteration, evaluating the vector $\underline{F}(\underline{H}^{(k)})$ and the Jacobian matrix, $(J_H)_k$, obtaining the nodal head corrections $\underline{\Delta H}^{(k)}$ and updating the nodal heads to get the new heads $\underline{H}^{(k+1)}$. The iterative scheme ends when the pre-specified convergence criteria is attained e.g. the required precision for $\Delta H_j^{(k)}$ or $F_j^{(k)}$.

Flow rates in pipes and at fixed head nodes can be calculated using the final solutions for nodal head values. The H-equations approach is recommended for the Newton-Raphson method; and this results in computational timesavings due to the symmetrical and sparsity features of the Jacobian matrix. Given a good estimate of initial heads, the Newton-Raphson method has very good convergence characteristics (Lemieux, 1972) and has been adopted for network analysis in this thesis.

C – Hardy-Cross method

The Hardy-Cross method is based on loop-flow correction equations (Cross, 1936). The method involves making flow corrections to initial assumed pipe flow rates.

Applying a first order expansion of Taylor's series to the Δq equations of each loop and then solving the resulting equations for the loop-flow corrections, is then done. Bhave (1991) presented a step-by-step implementation of the Hardy-Cross method based on the Δq-equations. In brief, initial link flows are assumed for a loop in the network, then the Δq-equation for the loop is set up in a similar way as in Eqs. (2.14) to give

$$\sum_{ij \in IJ_{lp}} K_{ij}\left(q_{ij}^{(k-1)} + \sum \Delta q_{lp}^{(k)}\right)^{nc} = 0 \qquad \forall lp \in LP \tag{2.25}$$

Applying a first order Taylor's series to the Δq-equation for the loop yields

$$\sum_{ij \in IJ_{lp}} K_{ij}\left(q_{ij}^{(k-1)}\right)^{nc} + \Delta q_{lp}^{(k)} \sum_{ij \in IJ_{lp}} \left| nc \cdot K_{ij}\left(q_{ij}^{(k)}\right)^{nc-1} \right| = 0 \qquad \forall lp \in LP \tag{2.26}$$

Eq. (2.26) can be rearranged to give

$$\Delta q_{lp}^{(k)} = -\frac{\sum_{ij \in IJ_{lp}} K_{ij}\left(q_{ij}^{(k-1)}\right)^{nc}}{\sum_{ij \in IJ_{lp}} \left| nc \cdot K_{ij}\left(q_{ij}^{(k)}\right)^{nc-1} \right|} \qquad \forall lp \in LP \tag{2.27}$$

The loop-flow correction obtained in the k^{th} iteration from Eq. (2.27) is used to update the pipe flow rates as follows:

$$q_{ij}^{(k)} = q_{ij}^{(k-1)} + \Delta q_{lp}^{(k)} \qquad \forall lp; \quad \forall ij \in IJ_{lp} \tag{2.28}$$

After updating the flows in the links of loop lp, the next loop is then considered. This implies that loop corrections are calculated and applied sequentially until all loops are covered as originally proposed by Hardy Cross (Bhave, 1991) and this ends the first iteration. It also means that for adjacent loops or loops that share a link, the effect on the loop-flow corrections is neglected. The next iteration involves using the updated flows as the new estimates $q_{ij}^{(k-1)}$ for pipe flow rates in Eqs. (2.25) to (2.27). This process is repeated until pre-specified convergence criteria are satisfied. Examples of such criteria could be a point when the correction values become very small; or when the differences in link flow rates obtained from successive cycles become insignificant, and the loop and path equations are satisfied. The Hardy-Cross algorithm is easy to implement but is computationally inefficient because it solves one equation at each time. It is unsuitable and unreliable for complex and large networks since it converges slowly or may not converge at all (Jeppson, 1976).

2.3.2 Extended Period Analysis – a Summary

The previous section of this chapter has focused on steady state analysis of water supply networks in which nodal demands and reservoir water levels are assumed to remain constant. These assumptions are valid for a short period of time. However, the reality is that over a long period, neither the nodal demands nor the reservoir water levels remain constant. Nodal demand fluctuates with a typical 24-hour pattern depending upon the type of demand. Examples of typical demand types are residential, commercial, institutional and industrial demand. It is the obligation of those who manage and operate water supply systems to provide a satisfactory level of service to the consumers. This obligation demands that the operators ensure that flow rates and nodal pressures at various times are sufficient; and that an evaluation of storage adequacy is made to balance the supply and distribution of water (Bhave, 1991). To achieve these objectives, extended period or dynamic analysis should be carried out.

Dynamic analysis of a water supply system is normally performed over a period of 24-48 hours under varying demand conditions; reservoir water levels and boundary conditions. The technique involves sub-dividing the dynamic analysis period into several time intervals (typically with a duration of 15minutes to 1 hour); and performing a sequence of steady state solutions at these intervals. The inputs to the static solution in every time interval are updated using appropriate data e.g., from the dynamics of filling and emptying of the reservoirs; pump schedules and valve settings. See, for example, Bhave (1991) for a methodology of performing extended period analysis.

Walski (1995) has noted that extended period analysis is a means of checking whether a design accounts for the fact that the sizing of some pipes in distribution network is controlled by the need to refill a service reservoir through the network pipes, during low demand periods. However, he has pointed out the fact that generally, extended period analysis cannot be linked with optimisation in an efficient manner. Such a problem could end up taking a lot of time to solve with no guarantee of an optimum solution. Walski (1995) has also noted that for master planning

(planning for the long-term upgrading of the network), decision-makers are aware of the fact that water use patterns and various other assumptions are bound to change over the years; and are thus interested in a realistic forecast of the future water use, a reasonable set of pipes for the design, their costs and when they should be laid. Therefore, this summary on extended period analysis has been included in this thesis for completeness only; otherwise it is not particularly dealt with in this research.

2.4 PRESSURE–DRIVEN NETWORK ANALYSIS

2.4.1 Basic concept

Demand driven analysis is sufficient for normal network operating conditions and it is based on the assumption that demands can always be fully satisfied. However, there are various situations when the pressures in the network are less than fully satisfactory, leading to nodal outflows that are less than the actual demands. Such situations can be due to excessive demands due to an increased rate of growth of demand or excessive withdrawal at some nodes for fire fighting purposes, pipes that have been isolated for repair or maintenance, very old mains whose internal roughness has increased greatly, pump or valve failures, to mention but a few. A network in this state is deficient and requires pressure-dependent network analysis or head driven analysis, rather than demand-driven analysis for more accurate results (Germanopoulos, 1985; Germanopoulos et al., 1986 and Bhave, 1991).

Using demand driven analysis for a network in a deficient state tends to yield pressures that are very low or negative at some of the nodes, and may also give nodal pressures that are sufficient to meet the demand at other nodes. One would then conclude that there is a problem in the area where the nodes have very low or negative pressures without quantifying the magnitude of the problem. On the other hand, analysis of such a deficient network using head driven analysis methods would yield results which show that some of the nodes in the network would have no outflow; some nodes would have a proportion of the nodal demand as their outflow, and the rest would have the full nodal demand values as the outflow.

In simplistic terms, the main difference between pressure-dependent network analysis or head-driven analysis and demand-driven analysis is the fact the latter assumes that nodal demands are fixed; while the former does not but rather, gives the actual quantity of water the network yields through the nodes (nodal outflows). To elaborate on this, referring to the nodal continuity Eqs. (2.12), pressure-dependent network analysis involves substituting Q_j, the demand with Q_j^{avl}, which is the available outflow at node j. This gives a set of pressure-dependent continuity equations as

$$F_j \equiv \sum_{i:H_i>H_j}\left(\frac{H_i-H_j}{K_{ij}}\right)^{(0.54)} - \sum_{i:H_i<H_j}\left(\frac{H_j-H_i}{K_{ij}}\right)^{(0.54)} - Q_j^{avl} = 0 \quad \forall j=1,\ldots,NJ \qquad (2.29)$$

in which Q_j^{avl} depends on the total head, H_j, at the node, such that $0 \le Q_j^{avl} \le Q_j^{req}$ whenever $H_j^{min} \le H_j \le H_j^{des}$. H_j^{des} is the desired head to fully satisfy the nodal demand. H_j^{min} is the minimum required head at node j or the head below which service at demand node j is unsatisfactory and therefore unacceptable (Twort et al., 2000). It may also be referred to as the head below which outflow is assumed to be zero or the performance is unacceptable. $Q_j^{avl} = Q_j^{req}$ (demand at node j) when $H_j \ge H_j^{des}$ and $Q_j^{avl} = 0$ when $H_j < H_j^{min}$. The Newton-Raphson method can then be used to solve the set of equations (Eqs. 2.29) as detailed earlier. Other network components like pumps and valves can be added using their head-flow relationships as mentioned earlier, with appropriate adjustments.

2.4.2 Review

2.4.2.1 Methods Based on Demand Driven Analysis

Various researchers have used different methods of obtaining the available nodal outflow for pressure dependent network analysis as reviewed shortly. Bhave (1991) developed a heuristic method of obtaining the available nodal flow. The method is referred to as the Node Flow Analysis (NFA) and is set up as an iterative scheme involving a series of demand driven analyses. Bhave (1991) noted that flows in a deficient network adjust so as to produce the maximum possible total outflow under the given conditions; and developed the NFA based on this fact. He classified nodes according to available head as follows: "critical" nodes (with heads equal to the minimum required value), "supercritical" nodes (with heads more than the minimum required value) and the "sub-critical" nodes (with heads less than the minimum required). He has also classified nodes according to flow as "zero-flow" nodes (nodes with no demand); "no-flow" nodes (demand nodes with no outflow); "partial-flow" nodes (nodes with partially satisfied demand); "adequate-flow" nodes (nodes with fully satisfied demand); "surplus-flow" nodes (nodes with outflow in excess of demand) and "negative-flow" nodes (nodes with negative demand). The surplus-flow and negative-flow nodes are categories that transitionally occur only during the NFA iterations.

For the first iteration, all demand nodes are assigned a category of "adequate-flow" nodes, the problem is solved for nodal heads using demand driven analysis. From the resulting nodal head values, a check is made to find out whether the nodal flows are compatible with their head-dependent categories according to the respective requirements for compatibility of node categories as defined by Bhave (1991). If compatibility is achieved for all nodes, the solution of the optimisation problem is achieved and this marks the end of the NFA. For example, if all demand nodes had been assumed to be "adequate-flow" nodes, and optimisation yields heads that are higher than the minimum required implying that all nodes are supercritical, compatibility is deemed to have been achieved based on the requirements for compatibility of node categories and the NFA solution is achieved. If not, nodes are re-assigned categories of flow using the requirements for compatibility of node categories, based on their assumed nodal head categories. For example, for nodes whose heads are less than the minimum required and had been assumed to be "adequate-flow" nodes, a "critical-flow" category is assigned, etc. Nodes for which compatibility has been achieved retain the assumed categories. The iterations are continued until category compatibility is achieved for all nodes. The general relationship between nodal outflows and heads that is used for the NFA method was presented by Gupta and Bhave (1996) as follows:

$$Q_j^{avl} = Q_j^{req} \text{ (adequate-flow), if } H_j \geq H_j^{min} \quad (2.30a)$$

$$0 < Q_j^{avl} < Q_j^{req} \text{ (partial-flow), if } H_j = H_j^{min} \quad (2.30b)$$

$Q_j^{avl} = 0$ (no-flow), if $H_j \leq H_j^{\min}$ (2.30c)

NFA method is advantageous in that there is no requirement for directly including a nodal head-outflow relationship in the constitutive equations since demand driven analysis is used for each of the iterations. Bhave (1991) stated that there is no proof that the NFA solution is the global optimal solution. He also noted that violation of constraints occurs for some of the iterations. This is especially true when nodes are assumed to be "adequate-flow" nodes for a network with insufficient pressures. Such constraint violation may perhaps increase the time required for analysing a given problem.

Tanyimboh and Templeman (1995) and Tanyimboh and Tabesh (1997a) developed a method referred to as the Source Head Method (SHM). It was based on the net source head required for fully satisfying the network demand. They used the following source head-discharge relationship

$$H_s = H_s^{\min} + R_s\left(Q_s^{avl}\right)^{n_s} \tag{2.31}$$

in which H_s is the head available at the source, Q_s^{avl} is the sum of all nodal outflows, R_s is the resistance constant and the exponent $n_s = 2$. $H_s^{\min}$ is the head at the source when any node of the network just begins deliver water. Therefore, the expression for the sum of the actual nodal outflows is

$$Q_s^{avl} = \left(\frac{H_s - H_s^{\min}}{R_s}\right)^{1/n_s} \tag{2.32}$$

The sum of all available nodal outflows $Q_s^{avl} = Q_s^{req}$ or the sum of all nodal demands when $H_s = H_s^{des}$. H_s^{des} is the required source head to fully satisfy all the demand nodes. It is the sum of head losses (obtained from demand driven analysis) in links along a path from the source to the most critical node. Substituting for H_s and Q_s^{avl} in Eq. (2.31) with H_s^{des} and Q_s^{req} respectively, an expression for R_s was obtained. This expression for R_s was then used in Eq. (2.32) to obtain the total flow delivered. Tabesh (1998) has presented the details of the above derivation. Therefore, the source head-discharge relationship for the source head method (SHM) is as follows:

$$Q_s^{avl} = Q_s^{req}\left(\frac{H_s - H_s^{\min}}{H_s^{des} - H_s^{\min}}\right)^{\left(\frac{1}{n_s}\right)}; \qquad H_s^{\min} \le H_s \le H_s^{des} \tag{2.33}$$

This method is simple and provides a clearer picture of the behaviour of a pressure-deficient system than demand driven analysis. However, this approximation has the tendency of underestimating the total outflow from deficient networks. This is probably due to the fact that demand driven analysis results for a deficient network often give large negative heads for the most critical node on which the method is based. Consequently a high value of H_s^{des} is obtained and thus an under estimated total flow delivered value. The method cannot be used for multiple-source networks.

Tabesh (1998) has proposed an improvement of the SHM called the Improved Source Head Method (ISHM). The method has been derived in a similar way to the SHM and it relates the head at the source to the outflow at a node, giving an approximation of the actual flow delivered as (Tanyimboh et. al, 2001)

$$Q_j^{avl} \approx Q_j^{req}\left(\frac{H_s - H_s^{\min}}{H_{s,j}^{des} - H_s^{\min}}\right)^{\left(\frac{1}{n_j}\right)}; \qquad H_s^{\min} \le H_j \le H_{s,j}^{des}, \forall j \tag{2.34}$$

in which the exponent n_j varies between 1.5 and 2 (Gupta and Bhave, 1996). $H_{s,j}^{des}$ is the head required at the source for full satisfaction of demand at node j. To obtain $H_{s,j}^{des}$, demand driven analysis of the network is carried out with the nodal demands Q_j^{req}. $H_{s,j}^{des}$ for a given node j, is taken as $H_j^{\min}$ plus the sum of the head losses in pipes that form a path between node j and the source. $H_j^{\min}$ may be taken as the ground level of the location of the node.

ISHM does not estimate the actual flow delivered based on just the critical node (as SHM); instead, it considers each individual node and calculates the nodal demands (Q_j^{req}) base on the respective values of ($H_{s,j}^{des}$) as shown in Eq. (2.34). The method is applicable to both single- and multiple-source networks and it is easy to implement since it only requires head loss values of links, which are obtained from demand driven analysis. However, its approximation for nodal flow delivered has the tendency of overestimating the outflow at nodes with a shortfall in head (Tanyimboh et. al, 2001). Also, it uses an index n_j, whose accuracy requires a considerable amount of effort in field data collection, analysis and network calibration (Tanyimboh and Tabesh, 1997b).

2.4.2.2 Methods Based on Head-Outflow relationships

Some researchers have defined the available flow by a head-outflow relationship showing the variation of flow with pressure. Gupta and Bhave (1996) made a comparison of various formulae for describing the pressure dependency of nodal consumption and concluded that the following parabolic relationship (Wagner et al., 1988a; Chandapillai, 1991) is sufficiently accurate.

$$H_j = H_j^{min} + R_j\left(Q_j^{avl}\right)^{n_j} \tag{2.35}$$

where R_j is a resistance coefficient and its value depends on the characteristics of the service connection at that node. Generally, the exponent n_j is both node and network specific and it often varies between 1.5 and 2 (Gupta and Bhave, 1996).

Thus

$$Q_j^{avl} = Q_j^{req}; \qquad if \quad H_j \geq H_j^{des} \tag{2.36a}$$

$$Q_j^{avl} = Q_j^{req}\left(\frac{H_j - H_j^{min}}{H_j^{des} - H_j^{min}}\right)^{\left(\frac{1}{n_j}\right)}; \qquad if \quad H_j^{min} < H_j < H_j^{des} \tag{2.36b}$$

$$Q_j^{avl} = 0; \qquad if \quad H_j \leq H_j^{min} \tag{2.36c}$$

Germanopoulous (1985) noted that the hydraulic configuration between each node in the network and the consumers downstream of it governs the precise form of the pressure-consumption relationship for the node. He developed a head-outflow relationship for the available outflow at a node as follows:

$$Q_j^{avl} = Q_j^{req}\left(1 - b_j e^{-c_j\left[\frac{\Pr}{\Pr_j^{\#}}\right]}\right) \tag{2.37}$$

where b_j and c_j are coefficients for node j. $\Pr_j$ is the pressure at node j. $\Pr_j^{\#}$ is the nodal pressure at which a given proportion of the demand at node j is provided.

Germanopoulous (1985) suggested that the coefficients b_j and c_j may be taken as 10 and 5, respectively, in the event of no detailed field data. In such an instance, he noted that $Pr_j^{\#}$ is the pressure at which 93.2% of the nodal demand is satisfied. He also noted that Eq. (2.37) is an approximation of the available outflow at a node for a deficient network.

This relationship is very useful in that it can be used to estimate the available outflow at a node for a deficient network. However, the values for the coefficients b_j and c_j vary from network to network and they are node specific. Additional field data and calibration is therefore required. Generally, for values of expected nodal outflows above 93.2% of the demand, Eq. (2.37) does not model the outflows accurately.

Eq. (2.37) was modified by Gupta and Bhave (1996) to give

$$Q_j^{avl} = Q_j^{req}\left(1 - 10^{-c_j\left[\frac{H_j - H_j^{min}}{H_j^{des} - H_j^{min}}\right]}\right) \tag{2.38}$$

This Eq. (2.38) caters for the weaknesses of the relationship proposed by Germanopoulos (1985) by that fact that it yields $Q_j^{avl} = 0$ when $H_j = H_j^{min}$ and $Q_j^{avl} = Q_j^{req}$ when $H_j = H_j^{des}$ i.e. it can be used to model nodes with outflows above 93.2% of the demand. Eq. (2.38) is more practical and has a better representation of deficient network behaviour than Eq. (2.37). However, its representation of reduced outflow at a node varies according the value of c_j used. For a given node with available head H_j, the lower the value of c_j used, the lower the fraction of nodal demand satisfied, i.e. available flow Q_j^{avl} reduces as the value of c_j is lowered. This means that extra effort is required to calibrate the model and obtain the correct values of c_j for the nodes.

Tabesh (1998) and Tanyimboh et al. (2001) have developed a method called the Head-Driven Simulation Method (HDSM) by setting up a set of pressure-dependent continuity equations as in Eqs. (2.29) and taking Q_j^{avl} as

$$Q_j^{avl} = Q_j^{req}\left(\frac{H_j - H_j^{min}}{H_j^{des} - H_j^{min}}\right)^{\left(\frac{1}{n_j}\right)}; \quad if \quad H_j^{min} \le H_j \le H_j^{des}, \forall j \tag{2.39}$$

HDSM is based on the Newton-Raphson technique and explicitly incorporates the head–outflow relationship in the continuity equations as shown in Eqs. (2.39). The method has a provision for the elimination of oscillations to ensure faster convergence, based on a step-length adjustment parameter (SAP). The step-length adjustment parameter is incorporated in the equations for updating the head estimates (Eqs. 2.24) to give

$$\underline{H}^{(k+1)} = \underline{H}^{(k)} + \psi \Delta \underline{H}^{(k)} \tag{2.40}$$

in which ψ is the step-length adjustment parameter obtained by trial and error.

The value of this step-length adjustment parameter of Eq. (2.39) is difficult to ascertain (Kalungi and Tanyimboh, 2000). Not only are the indices R_j (Eq. 2.35) and n_j node and network specific but also, their determination requires field data collection and network calibration (Tanyimboh and Tabesh, 1997b).

To address the uncertainties and weaknesses of the above-mentioned approaches, a simple heuristic algorithm for determining pressure-dependent outflows in water distribution networks has been developed in this study and is presented in Chapter 5. In Chapter 5, a comparison is made between this method and recently developed head driven analysis methods by Ackley et al. (2001) and Tanyimboh et al. (2002).

2.5 MINIMUM COST DESIGN OF A WATER NETWORK

This section presents some methods of designing a water distribution network at minimum cost. It starts off with linear and non-linear optimisation, followed by dynamic programming.

2.5.1 Linear and Non-linear Programming Formulations

The goal of the optimisation process, is to minimise the cost of the network of the network components by obtaining the cheapest set of component sizes of the network. The minimisation process has to be done within pre-specified restrictions or constraints. In reality, this problem is non-linear because its objective function and some of its constraints are non-linear (Lansey and Mays, 1989). Thus, in this section, the problem is set up as a non-linear programming problem and suggestions are made for converting it into a linear-programming problem, which is easier to solve (Lansey and Mays, 1989). A linear programming problem based on the approach of Alperovits and Shamir (1977) is presented later in Chapter 4.

The design of the network has to be considered under different demand loads or patterns to ensure that redundancy is added and the design is reliable (Templeman, 1982b). Thus, multiple loading considerations with respect to this optimisation problem are also presented in this sub-section. Solution methods of the optimisation problem are briefly mentioned.

The type of problem dealt with in this sub-section is for a network whose layout, flow directions and demand patterns are specified for each link.

2.5.1.1 *Objective Function*

The objective function for this optimisation problem is that of minimising the total cost of the network pipes. For a given type of pipe material, the cost varies according to the diameter and length. The total cost of the network pipes is given by

$$C_{net} = \lambda \sum_{ij \in Nl} L_{ij} D_{ij}^{e_1} \qquad (2.41)$$

where C_{net} is the total cost of network pipes; the coefficient λ and exponent e_1 depend on the units of D_{ij} and they can be obtained by regression analysis. Given that the layout of the network is known, the link lengths L_{ij} are constant and known.

The minimisation of the objective function in Eq. (2.41) is subject to constraints, which are subdivided into main constraints and additional constraints (Orth, 1986) due to practical considerations as detailed next.

2.5.1.2 *The Constraints*

A - Main constraints

The main constraints are the head loss equations (Eqs. 2.2), the continuity equations (Eqs. 2.3) and the equations for conservation of energy (Eqs. 2.4, 2.5 and 2.6).

B - Additional constraints

Head bounds

$$H_{max,j} \geq H_j \geq H_{min,j} \qquad \forall j \qquad (2.42)$$

in which $H_{max,j}$ and $H_{min,j}$ are the upper and lower bounds on the nodal heads, H_j, respectively.

From Eq. (2.6) for the head loss along any path between a source s and a terminal node tn, the head at node tn is given by

$$H_{tn} = H_s - \sum_{ij \in IJ_{tn}} h_{ij} \qquad tn = 1, \ldots, NT; \qquad \forall s \in NS \qquad (2.43)$$

Thus, substituting Eq. (2.43) into Eq. (2.42) yields

$$H_s - H_{min,tn} \geq \sum_{ij \in IJ_{tn}} h_{ij} \geq H_s - H_{max,tn} \qquad tn = 1, \ldots, NT \qquad (2.44)$$

in which $H_{max,tn}$ and $H_{min,tn}$ are the upper and lower bounds on the terminal nodal head H_{tn}, respectively. Similarly, an equation may be obtained for the head loss along any path between a source s and a node j by substituting for $H_{max,tn}$ and $H_{min,tn}$ with $H_{max,j}$ and $H_{min,j}$, respectively, in Eq. (2.44).

Bounds on link-flow velocities

$$v_{ij,max} \geq v_{ij} \geq v_{ij,min} \qquad \forall ij \in Nl \qquad (2.45)$$

in which $v_{ij,max}$ and $v_{ij,min}$ are the upper and lower bounds on link velocities, respectively; and v_{ij} is the flow velocity of link ij. The velocity, v_{ij}, is obtained by

$$v_{ij} = \frac{4q_{ij}}{\pi D_{ij}^{2}} \qquad \forall ij \in Nl \qquad (2.46)$$

Bounds on pipe diameters

$$D_{ij,max} \geq D_{ij} \geq D_{ij,min} \qquad \forall ij \in Nl \qquad (2.47)$$

where $D_{ij,max}$ and $D_{ij,min}$ are the upper and lower bounds on diameters of link ij, respectively.

Non-negativity of link flows

$$q_{ij} \geq 0 \qquad \forall ij \in Nl \qquad (2.48)$$

In the next sub-section, the objective function for minimising the cost of pipes and all the constraints are presented in a problem statement.

2.5.1.3 Problem Statement – Single Loading Condition

Minimise $C_{net} = \lambda \sum_{ij \in Nl} L_{ij} D_{ij}^{e_1}$ (2.49a)

Subject to:

$$h_{ij} = \alpha L_{ij} \left(q_{ij} / C_{ij} \right)^{1.852} D_{ij}^{-4.87} \qquad \forall ij \in Nl \tag{2.49b}$$

$$\sum_{ij \in IJ_j} q_{ij} = Q_j \qquad j = 1, \ldots, NJ - 1 \tag{2.49c}$$

$$\sum_{ij \in IJ_{lp}} h_{ij} = 0 \qquad lp = 1, \ldots, NL \tag{2.49d}$$

$$\sum_{ij \in IJ_{sp}} h_{ij} = h_{sp} \qquad sp = 1, \ldots, NF \tag{2.49e}$$

$$H_s - H_{\min,tn} \geq \sum_{ij \in IJ_{tn}} h_{ij} \geq H_s - H_{\max,tn} \qquad tn = 1, \ldots, NT \tag{2.49f}$$

$$v_{ij,\max} \geq \frac{4 q_{ij}}{\pi D_{ij}^{\,2}} \geq v_{ij,\min} \qquad \forall ij \in Nl \tag{2.49g}$$

$$D_{ij,\max} \geq D_{ij} \geq D_{ij,\min} \qquad \forall ij \in Nl \tag{2.49h}$$

$$q_{ij} \geq 0 \qquad \forall ij \in Nl \tag{2.49i}$$

The above problem is non-linear (Lansey and Mays, 1989) because its objective function, Eq. (2.49a) is non-linear and its main constraints Eqs. (2.49b), (2.49c), (2.49d) and (2.49e) are also non-linear. However, the problem can be converted into a linear problem by linearising the objective function and pre-specifying the flow distribution after solving the main constraints using a hydraulic simulator or otherwise; and then using discrete commercially available diameters and the segmental approach of Alperovits and Shamir (1977) as described later in Chapter 4.

2.5.1.4 Multiple Loading Problem

It is important to design a water distribution network to satisfy different operational states or loading conditions (Alperovits and Shamir, 1977); and some examples of loading states are: peak hour demand, average day demand, maximum day demand, fire demand, periods of low demand when re-filling of reservoirs takes place, etc. One of the practical examples where designing for multiple loading applies is that when a network is designed to meet the design period's demand, it may not necessarily satisfy the minimum velocity bound in the first few years of the planning period. In such an instance, both scenarios have to be considered. Another practical application of designing for multiple loading conditions is the design of a water distribution network with a source and storage or service reservoir situated on opposite ends of the network. The network has to be designed for peak demand when it is being supplied from the source and service reservoir; and for low demand e.g., during the night, when the reservoir is being filled from the source (Orth, 1986).

Catering for demands for fire-fighting purposes is an important consideration in the design, given the damage a fire can cause if it is not put out in time. Ideal fire flow requirements (Twort et al., 2000) for residential areas range from 8 l/s to 35 l/s; and those for industrial and commercial areas range from 20l/s to 75l/s. Commonly used fire appliances have built-in pumps for capacities 2.3 and 4.5m^3/min (Twort et al., 2000). To a certain extent, these pumps can boost the available flow and pressure at a fire hydrant. Twort et al. (2000) have noted that in the USA, the minimum fire flow capacity of mains is stipulated depending upon the size of area and nature of property served. They asserted that these standards tend to be in excess of the peak domestic demand and hence govern the design of distribution mains; an issue that raises concern as to whether such high rates are necessary. With this information in mind, one has to make a careful judgement of the quantity of fire flow to allow for in design depending upon the loading case. For example, taking a loading case where peak demands are combined with fire flows, it might perhaps be reasonable to use a proportion of the fire demand to avoid over designing.

In general, the multiple loading problem requires that the system is designed to simultaneously satisfy the design criteria for various steady states. In brief, this problem may be set up as follows (Orth, 1986): establish the design criteria and flow patterns separately for the different states; obtain the diameters, which collectively satisfy the velocity constraints of all states (candidate diameters); formulate the objective function for candidate diameters in the same way as for a single state model; establish the hydraulic constraints separately for all states avoiding any repetition.

The objective function and the resulting hydraulic constraints together form the planning problem for multiple loading cases.

2.5.1.5 Solution of the Optimisation Problems

Though the non-linear problem of Sub-section 2.5.1.3 may be solved by non-linear programming techniques; Yates et al. (1984) have asserted that it is extremely difficult to solve due to its high non-linearity in q_{ij}, and D_{ij}. However, a considerable effort has gone into developing solution methods for the problem. For some examples of solution approaches that are used for solving the non-linear network design problem, the interested reader may consult the original publication of Jacobs' (1968) approach; or that of Lansey and Mays (1989) who have developed an algorithm that combines the use of a hydraulic simulator and a general non-linear programming code by Lasdon and Waren (1983). For examples of solution approaches that are used for solving the linear network design problem, refer to the original publications of Alperovits and Shamir (1977), and Quindry et al. (1981).

2.5.2 Dynamic Programming

This section presents another technique for minimum cost design of water distribution networks called dynamic programming, in the context of designs for master planning or long-term upgrading which can be done economically in stages. A general formulation of the upgrading or capacity expansion problem is also presented. An example of a model for capacity expansion is briefly mentioned.

2.5.2.1 Background

A numerical problem may be classified in various ways depending upon its major features e.g. linear, non-linear, continuous, discrete, and stochastic, etc. The group of problems that exhibits serial or sequential features and tends to cross these conventional group boundaries is one whose problems are solved by a technique called dynamic programming (Templeman, 1982a).

Templeman, (1982a) has noted that dynamic programming (DP) is best referred to as a solution philosophy rather than a numerical method without any standard algebraic or algorithmic form as does, for example, the simplex method of linear programming. Before solving any problem, it has to be formulated into a unique serial form. He has also noted that the advantages of dynamic programming include an efficient and rapid solution technique, and the ease of handling discontinuous functions or tabular values of functions with the same effectiveness as continuous ones.

To avoid designing the network for a very long period and therefore installing components well before they are actually needed, staging or phasing of the network design over the design horizon is necessary (Walski, 1992). In this thesis, dynamic programming has been applied in the network upgrading process and thus the problem formulation is described in this perspective.

2.5.2.2 Generalised Problem Formulation

Loucks et al., (1981) have considered the capacity expansion or network upgrading problem in a generalised form as presented next. They have considered a water company planning for the future upgrading of its water supply system. For a given design horizon, the network is to be upgraded in stages or phases. It is assumed that the required capacity for each phase or period τ has been approximated as D_τ. For each period τ, the cost $C_\tau(s_\tau, r_\tau)$, of adding capacity, r_τ, is a function of that added capacity and the existing capacity, s_τ, at the beginning of the period or the design phase. This cost may be obtained by formulating and solving the problem for the specific design period using non-linear programming or linear programming as described earlier. The planning problem is to obtain sequence of capacity expansions in terms of timing and magnitude, that minimises the present value of total future costs, and satisfies the projected requirements and it may be written as follows:

$$\text{minimize} \sum_{\tau=1}^{\Lambda} \alpha_\tau C_\tau(s_\tau, r_\tau) \qquad (2.50)$$

in which Λ is the total number of phases or periods and α_τ is the discount factor $(1+r)^{-v}$. This discount factor assumes an interest rate of r in each period τ, and that the construction costs are incurred at the beginning of each phase. The symbol v represents the number of years preceding a design phase. For example, in Phase I, $v = 0$, and if Phase II is carried out in the 8th year of the design horizon, $v = 7$ in Phase II.

The constraints define the final capacity of each period, s_1, which is also the initial capacity of the next period, and each increase in capacity, r_τ, up through period τ.

$$s_{\tau+1} = s_1 + \sum_{\tau=1}^{\Lambda} r_\tau \quad \text{for } \tau = 1,2,\ldots,\Lambda \tag{2.51}$$

A constraint is needed to ensure that required capacity, D_τ, at the end of each future period τ, does not exceed the actual capacity, $s_{\tau+1}$, at the end of that period. Thus,

$$D_\tau \le s_{\tau+1} \qquad \text{for } \tau = 1,2,\ldots,\Lambda \tag{2.52}$$

Any other constraints may be added as appropriate.

Loucks et al. (1981) have asserted that the constrained optimisation model described above by Eqs. (2.50), (2.51) and (2.52) is a multistage decision-making process that can be solved using either a forward or a backward-moving dynamic programming solution procedure. The periods in which upgrading decisions are made may be taken as the stages of the model. If a forward-moving solution procedure is adopted, the states may be taken as the capacity at the end of a stage or period τ. On the other hand, if a backward-moving solution procedure is utilised, the states may be taken as the capacity s_τ at the beginning of a stage or period τ. In this thesis, a model similar to the one described above has been adopted with some modifications as detailed in Chapter 4.

An example of a model for the capacity expansion of water supply systems by Dandy et al. (1985) is reviewed in Chapter 3. Their model focuses on pricing or policies for varying the water tariffs and their impact on expansion of a water supply system. It mainly dwells on the economics of capacity expansion. In this research, the effect of pricing on decisions of long-term upgrading of water distribution networks is investigated in Chapter 4.

2.6 PERFORMANCE ASSESSMENT OF WATER DISTRIBUTION NETWORKS

This section is about the assessment of network performance as an audit or check to assess whether the network design can meet the expectations. Important performance assessment parameters of reliability and failure tolerance that are used in this research are introduced; followed by a brief review of work done using these parameters.

Assessment of the performance of water distribution systems is a complicated process involving many different important issues. These include reliability of individual components, possible variations in demands, fire flows and their locations to mention but a few. The difficulty in defining useful performance measures and determining what constitutes acceptable levels for these measures further complicates matters. Network reliability is often considered as the extent to which the network can meet customer demands at adequate pressure under normal and abnormal operating conditions (Tanyimboh et al., 2001). It is a performance assessment measure that puts more emphasis on the hydraulic perspective and less upon the underlying robustness of the network in terms of its layout.

Closely related to reliability, an aspect of the overall system performance that is often neglected is redundancy. Redundancy addresses the resilience or the network robustness with respect to the layout, in a more effective manner. It is the existence of alternative pathways from the sources to demand nodes or excess capacity in normal operating conditions, for use in abnormal operating conditions e.g., when components become unavailable (Park and Liebman, 1993). To ensure an uninterrupted albeit reduced supply of water, distribution network designs should include some amount of redundancy. Conventionally, redundancy is assumed to be present if the network has many loops. However the task of quantifying redundancy is a difficult one. A quantified measure for redundancy called failure tolerance (Tanyimboh and Templeman, 1998) is presented next.

Tanyimboh and Templeman (1998) have presented a parameter called failure tolerance, which is a quantified performance assessment parameter concerned with the operation of water distribution networks under conditions of partial system failure or in a reduced state. Such situations may arise due to various circumstances, e.g. pump failures, pipe bursts and unavailability of components due to maintenance or rehabilitation. Under these circumstances, there will often be insufficient pressure heads at the demand nodes, which are incapable of fully satisfying demands.

A brief review of independent work on water distribution network reliability and failure tolerance by Tanyimboh and Templeman (1998), together with that of Tanyimboh et al. (2001) is presented next. The formulae for calculating reliability and failure tolerance, which are the key parameters that are used for network performance assessment in this thesis, are taken from their work. The main reason for this choice is because the methods they have developed are simple, easy to understand and interpret. It should be noted that a detailed extension of reliability theory has not been included herein.

2.6.1 Reliability

Water distribution systems are designed with the aim of minimising system failures and ensuring sufficient reliability. The two main types of failures in WDS are mechanical failure of components like valves, pumps, pipes, etc. and hydraulic failure when the system is incapable of delivering the right quantity of water at the right pressure. Therefore reliability assessment involves a combination of mechanical reliability and hydraulic reliability. Mechanical reliability measures the probability that a particular component or system is operational at any time. Mechanical reliability of a network depends upon the layout of its components and the mechanical reliability of individual components. Hydraulic reliability is the probability that the system can supply the right amount of water at adequate pressure and it is largely governed by the hydraulic performance of the network. Mechanical reliability and hydraulic reliability are interdependent to a certain extent (Tanyimboh and Templeman, 1998).

2.6.1.1 Mechanical Reliability

A parameter that is indicative of network's mechanical reliability and easy to calculate is the probability $p(0)$ that the network is fully connected. Based on the assumption that pipe unavailabilities are independent, the probability, $p(0)$, that no pipe is unavailable (Tanyimboh and Templeman, 1998) is

$$p(0)=\prod_{l=1}^{Nl} a_l \qquad (2.53)$$

in which a_l is the probability that link l is available or the link's mechanical reliability. Pipe availability can be taken as the ratio of the mean time between failures to the sum of the mean time between failures and the failure duration. For example, this may be obtained using the formula developed by Cullinane et al. (1992) as follows

$$a_l=\frac{0.21218D_l^{1.462131}}{0.00074D_l^{0.285}+0.21218D_l^{1.462131}} \qquad \forall l\in Nl \qquad (2.54)$$

in which D_l is the link diameter in inches.

2.6.1.2 Network Reliability Calculation

Assuming a constant demand value, and taking only one and two unavailable components into consideration, the reliability, Re, of a water distribution system can be taken as (Tanyimboh and Sheahan, 2002)

$$\mathrm{Re}=\frac{1}{Q^{req}}\left(p(0)Q(0)+\sum_{l=1}^{NL}p(l)Q(l)+\sum_{\substack{l=1\\ m=l+1}}^{NL-1}p(l,m)Q(l,m)\right)$$
$$+\frac{1}{2}\left(1-p(0)-\sum_{l=1}^{NL}p(l)-\sum_{\substack{l=1\\ m=l+1}}^{NL-1}p(l,m)\right) \qquad (2.55)$$

in which: $p(0)$ is the probability that no link is unavailable, $p(l)$ is the probability that only link l is unavailable and $p(l,m)$ corresponds to the probability that two components l and m are unavailable. $Q(0)$, $Q(l)$, and $Q(l,m)$ are the respective actual total outflows when zero components, components l and, l and m are unavailable. These outflows are obtained using any appropriate head-dependent network analysis method. Q^{req} is the total demand for the network.

2.6.1.3 *Failure Tolerance*

Tanyimboh and Templeman (1998) have pioneered the use of the concept of component failure or damage tolerance in the context of water distribution. They have defined damage or failure tolerance as a quantified measure of how reliable on average a distribution system is when any number of components are not operational. Tanyimboh et al. (2001) have noted that since most systems are expected to perform satisfactorily under normal conditions, it may be instructive to carry out a separate analysis of the behaviour under sub-normal conditions and an appropriate measure is the tolerance, *FT*, of the system to component unavailability, which is given by

$$FT = \frac{\mathrm{Re} - r(0)p(0)}{1 - p(0)} \tag{2.56}$$

in which $r(0)$ is the ratio of total outflow to the total demand when all pipes are available.

As defined above, failure tolerance is the expectation of the proportion of nodal or system demand that is satisfied during the periods in which there are mechanical failures in the system or when some components are taken out of service for repair or maintenance. It is used as a measure of the average system performance when components are out of action. The formulation for *FT* has the advantage that it can be calculated easily without any further hydraulic simulations, after the system reliability has been obtained. Thus, it does not impose a major computational burden. Values of *FT* lie between 0 and 1 (Tanyimboh and Templeman, 1998).

All in all, *FT* provides an estimate of the total demand that can be met when some components in the distribution system are out of service. Low values of *FT* are indicative of high vulnerability to component failure or a low degree of redundancy. Tanyimboh and Templeman (1998) have emphasised that *FT* rather than *Re* may be the more useful measure in many situations, e.g. for very reliable systems, the behaviour of the system when components are unavailable becomes the more relevant consideration. Therefore, both *FT* and *Re* should be evaluated in order to obtain a better representation of the performance of a network (Kalungi and Tanyimboh, 2001).

2.7 SUMMARY AND CONCLUSION

The design, operation, maintenance and upgrading of water distribution networks is a very expensive process that inevitably requires least cost optimisation techniques to meet budget constraints. This chapter has presented the steady state demand driven analysis of pipe networks followed by a summary of extended period analysis, which involves analysis of the network over a longer period of time, under changing demand patterns, reservoir water levels and boundary conditions. A brief introduction to the pressure-dependent network analysis and a review of various methods has been done. The fact that demand driven analysis is not sufficient when the network is in a deficient state and the importance of using pressure-dependent network analysis in such instances has been highlighted.

The least cost design of pipe networks has been detailed, with methods such as linear programming, non-linear programming and dynamic programming as optimisation techniques for the design of water networks. Finally, the performance assessment of water distribution networks using parameters such as reliability and failure tolerance has been presented.

The following chapter introduces the concepts of network rehabilitation and upgrading, together with a detailed review of long-term rehabilitation and upgrading techniques for water distribution networks.

CHAPTER THREE

LONG-TERM NETWORK REHABILITATION AND UPGRADING

3.1 INTRODUCTION

An aging water distribution system experiences a general increase in demand and its ability to transport water diminishes. This leads to problems of inadequate water supply and reduced network performance. In addition, there are direct economic impacts of a failing system. For example, industrial growth can be seriously deterred by lack of adequate water supplies. Other problems caused by a deteriorating network are: water quality problems, internal pipe corrosion, low pressure, high head loss problems, leakages, pipe breaks and inoperable valves.

Consequently, the rehabilitation and expansion of an existing system to meet current and future demands of flow rate and pressure head, has always been a field of interest to engineers. Rehabilitation and expansion of a water distribution network generally involves large capital outlays, as well as the continuing operation, maintenance and repair costs. Most water companies spend more than half of their total budgets to address this problem (Selvakumar et al., 2002). For example, the U.S. Environmental Protection Agency (USEPA) conducted a survey whose findings show that $138 billion will be needed for maintenance and replacement of existing drinking water systems over the next 20 years (USEPA, 1997); with 56% of this amount dedicated to pipelines (Selvakumar et al., 2002). This significant pipe network expenditure is one of the reasons why this thesis has focused on the pipeline network of water supply systems. It also implies that large savings can be gained if a cost-effective long-term upgrading strategy is adopted right from the design stage of a new network.

Designing the network for a very long design horizon inevitably yields an expensive design, which requires locking up capital investment in the project whose full capacity will not be required until after many years. It is more practical and economical to design the network in stages or phases for shorter time periods and upgrade it as the need arises. This strategy provides the opportunity of investing the rest of the capital and getting interest on it, instead of locking it all up in the project. However, the number of phases, their length and the magnitude by which the network may be expanded, together with any related issues are matters that require careful planning at the onset of the project. The main objective of the long-term upgrading problem therefore is to obtain the phasing, timing and magnitude of upgrading over the entire design horizon considering water quality and network performance requirements within a limited budget.

A long-term network upgrading strategy generally involves planning for a staged or phased future increase of the network capacity; while improving the hydraulic and structural capacity. The length of these phases varies widely; for example, from 5 years to 20 years or more; and the number of phases in a design horizon can be taken as two or more (Lansey et al., 1992). Shorter phases (of less than 5 years) may not be practical/economical considering that the goal of increasing capacity implies large capital outlays that include fixed costs of setting up construction equipment for each phase. The cost of adding capacity in each phase is a function of the added capacity as well as the existing capacity. Thus, the higher the number of phases opted for in a given design horizon, the higher the fixed costs associated with the upgrading strategy, increasing its chances of becoming uneconomical.

Rehabilitation can be defined as improvement of the functional service of an existing network. In other words, the main purpose of rehabilitation of the distribution network is to improve service levels to customers by ensuring better water quality, reducing interruptions to supply and poor pressure; and reducing leakage at an economic cost. As the network ages the water quality tends to deteriorate. Interruptions to supply are can be caused by poor structural conditions of the pipes and bad workmanship. Poor pressure arises from inadequate size either due to the pipes being too small, or more usually, the build up of encrustation (Burgess, 2001). Rehabilitation should put into consideration any regulatory requirements. For example, the majority of mains rehabilitation in the United Kingdom is subject to legal undertakings with the Drinking Water Inspectorate (DWI) under the provisions of Section 19 of the Water Industry Act, 1991 (Burgess, 2001).

Whereas network rehabilitation is generally limited to restoring the hydraulic and structural capacity of a deteriorating network; a long-term rehabilitation strategy involves planning for future rehabilitation with due regard to the timing and magnitude and in some instances may involve the increase in network capacity. There are examples of long-term rehabilitation models that can utilise multiple time steps and determine the timing of rehabilitation simultaneously for all pipes with individual pipes having different timings (Kleiner et al., 2001, Dandy and Engelhardt, 2001, and others). The length of time-steps or phases varies from 1 year to about 10 years, meaning that the rehabilitation actions can be implemented as frequently as annually, probably for a few pipes in some years.

Long-term rehabilitation strategies as opposed to those of long-term upgrading or capacity expansion, are more flexible in accommodating shorter time-steps (less than 5 years long). Shorter time-steps for rehabilitation can probably be approximated to routine maintenance, which may not require the magnitude of capital outlays needed for long-term upgrading and the associated fixed costs of setting up construction equipment for each time step. There is an overlap in what constitutes an optimal long-term upgrading strategy and a long-term rehabilitation strategy. To a certain extent, these two definitions can be used interchangeably, for example, when the long-term rehabilitation strategy involves long time-steps and a significant increase in network capacity.

Comprehensive long-term upgrading and rehabilitation strategies should satisfy network hydraulics and economics, performance and water quality criteria. The hydraulics criterion is concerned with ensuring that minimum service pressures at the nodes and consumer demands are met; and minimising head losses in the network. Network economics involves monetary costs including capital costs of installing the network components like pipes; maintenance costs associated with repairing and preventing failures in the network; costs arising from pipe bursts like property damage and any inconvenience caused. The performance criterion may address issues of reliability and failure tolerance (Tanyimboh and Templeman, 1998) as detailed earlier in Chapter 2.

The fourth criterion which is water quality, is mainly concerned with the fact that the aging of pipes is accompanied with build up of deposits and microbiologic slime growths that lead to the deterioration of water quality and reduction in carrying capacity of pipes. Burgess, (2001) has noted that the effect of deteriorating water quality manifests itself at the consumer's tap through discoloured water, taste or odour. He has also asserted that deposits in the pipe system such as iron and/or manganese, are the major source of discolouration in water supplied to consumers. Areas of low velocity are the most likely locations for deposition in the pipe system. The presence of flocculating agents from the water treatment process, such as aluminium salts in the water, may further assist the build up of deposits.

Encrustation causes the reduction in carrying capacity of pipes. For example, aggressive water may corrode iron mains internally; the iron loss could pass into the water or form part of an encrustation on the internal wall of the pipe. The encrustation builds up over the years leading to tuberculation in the pipe and a reduced internal diameter (Burgess, 2001).

Previous work in this area involved models that can be categorised into three main groups. The first group is made up of models based mainly on economics with minimum regard to technical details, e.g. the model by Dandy et al. (1985). The second group consists of individual asset-based models that do not explicitly consider network hydraulics and performance; and impart rehabilitation and upgrading decisions for individual components, e.g. the models by Shamir and Howard (1979), Walski and Pellicia (1982), Loganathan et al. (2002), and others. The third group is that of system-wide models that incorporate budget constraints and explicitly consider network hydraulics and performance, e.g. the models by Lansey et al. (1992), Halhal et al. (1997), Kleiner et al. (2001), Dandy and Engelhardt (2001) and others.

The preceding chapter has dealt with a review of the design, analysis and performance assessment of water distribution networks. This chapter presents a review of long-term rehabilitation and upgrading techniques. Specific attention has been paid to the models by Dandy et al. (1985), Lansey et al. (1992) and, Dandy and Engelhardt (2001) as these models are of particular relevance to the present research.

3.2 REVIEW OF LONG-TERM NETWORK REHABILITATION AND UPGRADING MODELS

This section presents a review of long-term upgrading and rehabilitation models in their different generalised categories.

3.2.1 Models Based Predominantly on Network Economics

Dandy et al. (1985) presented an economics-based upgrading or capacity expansion model that concentrated on optimum pricing and its concomitant impact on capacity expansion of a water supply system. The main objective of their model was to identify optimum water pricing and capacity expansion policies for water supply in the presence of administrative constraints on price. The relevance of this model to the current research lies in the fact that joint pricing and capacity expansion policies have been utilised in this thesis as one of the possible alternative strategies for network upgrading. The effect of these policies on upgrading strategies has also been assessed. Dandy et al. (1985) used the model to obtain the timing and magnitude of upgrading at a reduced cost; as a result of optimum pricing based on the price elasticity of demand, to delay the need to expand the network. This concept has been exploited in this research.

It is known that the price elasticity of demand for water, although small, is not zero (Carver and Boland, 1980 and Twort et al., 2000). This implies that an increase in the price of water may be accompanied with a reduction in demand. Typical values of price elasticity of demand for water lie in the range of -0.1 to -1.1 (Hanke, 1978). Based on this information, Dandy et al. (1985) developed a model for the maximisation of the present value of net benefits (NPV), subject to a series of constraints. The economic benefits were measured in terms of revenue from water sales. The constraints included one to ensure that peak demand cannot exceed capacity, no obsolescence of plant components, upper and lower bounds on prices, bounds on price changes from year to year, and bounds on revenue in relation to system costs.

They applied a relationship to model the community demand (including losses), D_t in m^3/year, as a function of the price of water as follows:

$$D_t(P_t) = q_0 POP_t \left(\frac{P_t}{P_0} \right)^{PREL} \qquad (3.1)$$

in which q_0 is the annual demand per capita in the base year, in m^3/person per year;, P_t is the price per unit (m^3) of water in year t, in dollars/m^3 and $PREL$ is the price elasticity of demand. For all variables the subscript zero refers to base year conditions. POP_t is the population served in year t for which a linear growth in population was assumed as follows:

$$POP_t = (1 + wt) POP_0 \qquad (3.2)$$

in which w is the fractional growth rate of the initial population per year.

The capital costs considered in the model were those associated with new supply sources. Their model was set up to cater for the fact that the total revenue from water sales and the sewerage surcharge must cover (within an acceptable range) the cost of both water supply and wastewater collection plus treatment. Thus, the short run marginal costs considered were those related to the operation, maintenance and repair (OMR) costs for water supply, transmission and distribution together with wastewater collection, treatment and disposal. The model also included short-run fixed costs such as administration and billing costs.

Dandy et al. (1985) demonstrated the utility of the model to explore pricing decisions in a case study application to the twin cities of Kitchener-Waterloo (KW) in Ontario, Canada. The interested reader may consult their publication for the details.

The results of applying this model to the twin cities of Kitchener-Waterloo (KW) showed that the price should be varied to maximise the economic efficiency and utilise the system capacity as fully as possible. In particular, when adequate capacity exists, the price should be adjusted to approach the short-run marginal cost of supply. The price should be increased in the last few years to full capacity utility in order to delay future expansions in capacity. Finally, a point is reached where the benefits of expanding the capacity (and thus supplying more water) exceed the costs of new construction. At this point, the capacity should be expanded. Dandy et al. (1985) found that administration and minor capital costs were insensitive to price, and merely increased with population increase. Optimum pricing and capacity expansion decisions were shown to be compatible with a requirement of financial cost recovery on the part of the supply authority.

Significant benefits can be attained by jointly optimising decisions about pricing and capacity expansion of water supply systems. As a first step towards achieving efficient policies, water resource planners should recognise that water pricing can be used as a soft alternative to investment in major new capital works. The model presented is of a practical nature and can be applied to other cities with appropriate adjustments. However, the model does not consider the structural and hydraulic integrity of the network and it does not incorporate important network performance parameters such as reliability. It does not directly identify the components to be upgraded, but rather the funds that are required for upgrading purposes, the timing and magnitude of upgrading and the benefits accrued by optimum pricing and capacity expansion policies.

3.2.2 Individual Asset-based Models

Shamir and Howard (1979) used an economic decision model formulation to provide the optimal time to replace a pipe. They applied regression analysis techniques to obtain an exponential relationship between the pipe breakage rate and its age. They classified reasons for breaks as follows: (i) pipe quality and age together with the fittings, (ii) environment in which the pipe is laid, e.g., the corrosiveness of the soil, susceptibility to external loads, etc. (iii) pipe laying workmanship, (iv) service conditions e.g. pressure and the water hammer effect. The procedure they developed uses the history of mains breaks to forecast how the number of breaks would change with time if the pipe were not replaced. A separate analysis predicts the failure rate of newly installed pipes. These forecasts are combined with cost data and a discount rate that accounts for inflation to determine the optimal replacement date. This optimal time of replacement can then be used to prioritise the pipes that should be given further consideration in determining the final rehabilitation schedule. The break prediction model is homogeneous in that all pipes of the same characteristics (i.e., pipe material and age, soil and temperature, operating pressure, etc.) are assumed to deteriorate at that same rate. The economic analysis developed is simple to calculate and can be programmed on a hand held calculator. The work of Shamir and Howard (1979), focused mainly on developing an analytical approach and a sensitivity analysis for forecasting water mains breaks and determining the year in which a pipe should be replaced. Little attention was given to providing data on the cost of main breaks and estimating the parameters of their break model. Their model does not include network hydraulics.

Walski and Pellicia (1982) introduced the idea of the threshold break rate. They adopted Shamir and Howard's (1979) model for predicting break rates and derived an optimal replacement time estimator by setting the total repair costs over a period of time to be equal to the replacement cost. They made a considerable effort in collecting additional data (e.g., data for the cost of main breaks and estimation of parameters) and several modifications to convert the model by Shamir and Howard (1979) into a tool for practicing engineers. Walski and Pellicia (1982) provided a slightly different economic criterion, by which a pipe should be replaced if its failure rate is above a critical value. For the break equations, they added two factors to the exponential relationship between the pipe breakage rate and its age by Shamir and Howard (1979); one factor relating to the size of the pipe and the other relating its previous failure history. Walski and Pellicia (1982) based their decision of whether and when to replace a pipe on the cost per break, the cost of replacement pipe, the total length of pipe to be replaced, pipe diameter, previous break history, the interest rate, pipe material and the rate at which the pipe is aging. Whereas the model of Shamir and Howard (1979) is useful for deciding whether to replace entire groups of pipes, the Walski and Pellicia's (1982) model is more useful in analysing the economic replacement on a pipe-by-pipe basis. However, the model does not incorporate the hydraulic analysis of the network and it does not consider the impact of deterioration of hydraulic integrity of the pipes on the network performance.

Loganathan et al. (2002) have presented an economically sustainable threshold break rate for replacement of pipelines in deteriorating water distribution systems, which is derived from a comprehensive review of pipe replacement analyses. They have defined the threshold break rate as an unacceptable break rate for a pipe to remain in service. The derivation does not embed a rate of break occurrence model as in previous studies and it only involves the discount rate, repair and replacement costs. The steps involved are as follows: a) derivation of an analytical model for an economically sustainable (critical) break rate, b) derivation of the equivalence relationships between critical break rate and statistical models suitable for repairable systems, c) the utility of the proposed procedure is established with the help of the equivalence relationship and a flexible probability function. The methodology is based on a time-truncated probability function, which means that an incomplete data set can be handled and that it does not require the full failure history of a pipe. The threshold break rate yields an optimal time of replacement as obtained by Shamir and Horward (1979) but with less restrictive assumptions. A cost data analysis has been used to provide a functional relationship between the threshold break rate and the pipe diameter. Unfortunately, the model does not consider the impact of deteriorating pipes on the performance of the network. Also it does not consider the fall in the water quality standard due to deterioration of the hydraulic integrity of the pipes. In determining the optimal replacement time of individual pipes, no regard is given to the effect of replacing some pipes on the rate of growth of breaks for the remaining pipes.

In general, individual asset-based models do not explicitly incorporate factors such as the hydraulics and performance of the rehabilitated network. As an example, a deteriorating water distribution network is quite complex and it exhibits increasing roughness of pipes, which lead to increased pipe head losses; an issue that should not be neglected. The hydraulic behaviour of the network should be determined simultaneously for all pipes and interactions between them because any change in the hydraulic properties of any pipe causes a redistribution of flows in the network. The use of individual asset-based models results in a simplified approach that involves making decisions in isolation of the network hydraulics and performance criteria. However, these simplifications also mean that the models are easy to implement.

3.2.3 System-wide Models

These models incorporate the network hydraulics and performance explicitly. Most of these models integrate budget constraints into the formulation and some include multiple time steps such as that of Lansey et al. (1992) and Kleiner et al. (2001). This means that system-wide models provide a holistic approach, but are generally limited to small networks with a few exceptions e.g. Halhal et al. (1997) and Dandy and Engelhardt (2001). A more detailed presentation of the models of Lansey et al. (1992) and Dandy and Engelhardt (2001) has been done because these models are specifically relevant to the present research.

Lansey et al. (1992) presented a model for scheduling of maintenance for water distribution systems as detailed herein. They considered major piping alternatives of replacing, cleaning and relining for strategies of rehabilitation and/or expansion. They developed a non-linear optimisation model and linked it with a network simulation model to solve the problem. This model has been given special attention due to the fact that it tackles the problem of rehabilitation and/or expansion effectively by using the segmental approach whereby each link is sub-divided into sections that form the decision variables. This idea has been utilised in this thesis.

To implement the rehabilitation alternatives, they suggested that subdividing the planning horizon into two planning periods or phases would suffice for most planning purposes; for example, a typical planning horizon of 15 years could be sub-divided into two, a 5-year period followed by a 10-year (long-term) period. They noted that the sub-division would sort out any short-term problems, with due consideration to projected growth and the implications of short-term decisions on long-term costs. They used a segmental approach by assigning four decision variables to each existing link in the network during the first phase in the design horizon. These were defined as follows: a) *LN* was the length to be replaced with new pipe, b) *DN* was the diameter of the new pipe, c) *LR* was the length of the old water main to be relined, and d) *LO* was the length of existing main to be left unaltered. In the second phase, they assumed that any link replaced or relined would be left as it is. Thus, the length, *LO*, was subdivided into three sections. *LN2* with diameter *DN2*, *LR2* and *LO2* corresponding to link segments to be replaced, relined and left as it is respectively.

The water network optimal rehabilitation model they formulated comprised the options of installing new pipes and relining existing ones. The main objective was to minimise the cost of rehabilitation and expansion of the network over the first and second planning periods and was stated as follows:

$$\text{Minimize} \quad F=\sum_{\tau=1}^{2}\left[\sum_{ij=1}^{IJ}\left(ar_{ij}LN_{ij,\tau}DN_{ij,\tau}^{nr_{ij}}+br_{ij}LR_{ij,\tau}DO_{ij,\tau}^{mr_{ij}}\right)\right] \tag{3.3a}$$

subject to the following constraints:

The nodal flow continuity equations, the conservation of energy equations for loops and paths between nodes of known total head, together with upper and lower bounds on nodal pressure heads. These were the main constraints.

Other constraints were as follows

$$LN_{ij,1}+LR_{ij,1}+LO_{ij,1}=LT_{ij} \qquad ij=1,\ldots,IJ \tag{3.3b}$$

$$LN_{ij,2}+LR_{ij,2}\leq LO_{ij,1} \qquad ij=1,\ldots,IJ \tag{3.3c}$$

$$DN_{ij,1}, LN_{ij,1}, LR_{ij,1}, LO_{ij,1}, DN_{ij,2}, LN_{ij,2}, LR_{ij,2}\geq 0 \qquad ij=1,\ldots,IJ \tag{3.3d}$$

Where $LR_{ij,\tau}$ is the length of existing main to be relined in period τ for each reach *ij*. $DN_{ij,\tau}$ and $LN_{ij,\tau}$ are the diameter and length of the new pipe installed in reach *ij* during period τ. $LO_{ij,\tau}$ is the length of the pipe of reach *ij* left in its initial condition in period τ. ar_{ij}, br_{ij}, nr_{ij}, and mr_{ij} are regression coefficients relating the decision variable to actual cost for replacing and relining pipe *ij*. The total number of links in the network is *IJ*.

The first term in the objective function (Eq. 3.3a) represents the cost of installing a new pipe in a given reach during a particular period. The second term represents the cost of replacing and relining a given pipe in a given reach during a particular period. Eq. (3.3b) states that the sum of the sections of the existing main that is replaced, relined and left unaltered is equal to the total length of the reach in the first phase. During the second phase, the sum of the replaced and relined lengths is equal to the length of main that was left unaltered in the first phase. Eq. (3.3d) is for the non-negativity of variables since they represent physical quantities. They also gave a suggestion for including costs for repairing water mains and for pump station expansion in terms of two main costs, namely: the cost of expansion (construction and installation) and the increased energy cost associated with pumping water.

Lansey et al. (1992) avoided simplifying or linearizing the hydraulic equations; instead, they used a network simulation model to solve constitutive equations or the main constraints, thus reducing the number of constraints. This reduced the problem to one of solving the objective function of Eq. (3.3a), subject to the constraints in Eqs. (3.3b), (3.3c) and (3.3d). They solved the problem using a non-linear programming scheme in which an augmented Lagrangian penalty term was introduced in the objective function in order to incorporate the violation of the bounds on the implicit pressure heads. The penalty term is based on Lagrange multipliers and penalty weights that are adjusted in an iterative procedure until all pressure heads are within their bounds or up to a limit number of iterations. The interested reader may consult the original publication of Lansey et al. (1992) for details.

The above formulation was applied to a system with two sources and 14 links for two examples. The design horizon was taken as 15 years with an initial planning period of 5 years and a second period of 10 years. In the first example, it was assumed that only pipes could be replaced or rehabilitated during the two planning periods. In the second example, it was assumed that a pump at the source would be sized during the first period and a second one added in the second period, if deemed economical.

The model execution time (CPU time) was about 33 minutes for the first example on a PC with a microprocessor speed of 386Mhz and about 2.8 hours for the second example. Lansey et al. (1992) noted that this execution time could be reduced slightly with better starting estimates which in turn improves confidence of finding the best solution. They also observed that the chances of obtaining a global optimum solution were enhanced by repeated trial runs and good user judgment which facilitated the process of selecting pipe and pump size estimates, together with that of variable scaling which is required in the optimisation process.

The model developed by Lansey et al. (1992) can assist decision-makers in solving the complex problem of maintenance of water distribution systems. The approach used of solving the problem in its true non-linear form by linking the optimisation algorithm with a network simulation model, greatly reduces the computer time required. The model can be analysed for multiple-planning periods to account for staged growth. It directly identifies the pipes that require maintenance by relining and/or replacement together with the associated costs.

However, the computer execution time is fairly high, even for small networks. The model is deterministic and does not account for uncertainties in system decay and demand forecasting. Since the problem is solved in its non-linear form, there is no guarantee that a global optimum can be found. The model greatly relies upon good engineering judgement for its proper implementation and selection of parameters so as to ensure that a global optimum is achieved. This judgement may be quite subjective depending upon the user and this subjectivity may require fine-tuning or adjusting of the final results to address this problem. Deterioration of pipes and its impact on important aspects of network performance such as reliability are issues that are not considered in the model. The timing of the rehabilitation is pre-defined by the user and not determined automatically by the model.

Halhal et al. (1997) have discussed the importance of water distribution network rehabilitation, replacement and expansion. The problem of choosing the best possible set of network improvements within a limited budget has been presented as a large optimisation problem to which conventional optimisation techniques are poorly suited. They have proposed a multi-objective rehabilitation formulation that uses a technique, which they have called, structured messy genetic algorithms (SMGAs). The SMGAs incorporate some of the principles of the messy algorithm, such as strings whose length increases during the evolution of designs. The two objectives considered are the benefits to the system and the rehabilitation cost, with the latter constrained to a predetermined budget limit. The benefits include the improvements to the system in hydraulic performance, increase in flexibility, savings in maintenance and operating costs, and improvements in water quality. These have been combined using weights set by the decision maker. The formulation permits a wise choice of solution along a trade-off curve between benefit and cost, according to the total funds available and the benefit resulting from the rehabilitation scheme. The effectiveness of the algorithm for the current optimisation problem has been demonstrated by applying it to a large network in Morocco, thereby stressing the fact that the algorithm performs much better than a standard genetic algorithm. However, multiple time steps and the timing of rehabilitation are not included in the model. Deterioration or the aging effect on the carrying capacity of pipes is not considered in the model. The weights used to represent the benefits may be rather subjective, leading to some inconsistence in the network behaviour. Such inconsistency is probably one of the reasons why one of the examples solved using the algorithm shows a violation of minimum nodal pressure requirements at all the nodes.

Kleiner et al. (2001) have proposed a model that considers the life-cycle time for each pipe. This model considers that each pipe can be rehabilitated by any available rehabilitation option. These options may be implemented any number of times, in any year of the design horizon. The time to the next replacement depends upon both the structural and hydraulic deterioration experienced by a pipe. The time between subsequent replacements is purely a function of the expected structural deterioration. The model considers network economics and hydraulic capacity analysed simultaneously over a predefined analysis period. The deterioration over time of both the structural integrity and the hydraulic capacity of every pipe in the system is explicitly considered in the process of obtaining the timing of rehabilitation. They have used the deterioration model proposed by Sharp and Walski (1988) as presented next, to depict the effect of aging on the carrying capacity of pipes as

$$C_{ij}(t) = 18.0 - 37.2\log\left[\frac{e_{0ij} + a_{ij}(age_{ij})}{D_{ij}}\right] \tag{3.4}$$

in which, for link ij, $C_{ij}(t)$ is the Hazen-Williams hydraulic conductivity coefficient in year t, e_{0ij} is the initial roughness (mm) at time of installation, a_{ij} is the roughness growth rate (mm/year), D_{ij} is the diameter in millimetres, and age_{ij} is the number of years from the time of installation.

This problem involves a huge combinatorial search space that consists of all combinations of pipes, rehabilitation alternatives and the related sequencing. In order to search this vast combinatorial space, Kleiner et al. (2001) have used a dynamic programming approach, combined with a partial and implicit enumeration scheme. They have noted that in a network of p pipes, R rehabilitation alternatives and a planning horizon of TS time steps, the total number of combinations is $(R+1)^{pTS}$; which is very high even for a small system. The model has a capacity of comparing projected cost streams that are independent of the selected analysis period by considering rehabilitation cycles to infinity. The model directly identifies the pipes that require rehabilitation, the funds to be assigned for rehabilitation purposes and the timing of rehabilitation for individual pipes, which may vary from pipe to pipe. However, the model does not incorporate important network performance parameters such as reliability. Judging from the vast combinatorial space that has to be searched for a solution, bearing in mind that it increases exponentially as explained before, the computational requirements of the method are high. For example, the Central Processing Unit (CPU) time required when using a 500-MHz Pentium III processor; for a four-looped network with 12 pipes, a single source and 4 demand nodes, is 32 minutes. This is when time-steps of one year are used in a 30-year planning horizon. If longer time steps of 6 years are used, the CPU time reduces to about 5 minutes.

Dandy and Engelhardt (2001) have developed a model for scheduling water pipe replacement as detailed next. They have developed a decision model for making rehabilitation strategies using genetic algorithms and applied it to a real-life water distribution system. The relevance of this model to the current research is based on the way it handles repair and damage costs. The model considers direct and indirect costs associated with pipe failure by using failure cost factors for land use to reflect the variation in the impact of pipe failure with its location. Also, they have incorporated equations for predicting the failure rate of pipes based on their diameter and age. These ideas have been utilised in this thesis for modelling the costs related to the failure of pipes.

In brief, the model for planning the rehabilitation of water supply systems that has been developed by Dandy and Engelhardt (2001) considers pipe replacement as the sole rehabilitation option due to the high cost of relining pipes. They have used three different models to carry out economic analysis of the system. The first model that is referred to as the single time-step case is for the decision to replace a pipe now or not all. This model explores the effectiveness of the genetic algorithm technique in identifying the pipes that are due for replacement. The second model builds up on the first and it provides for staging of pipe replacement activities over the design horizon in time steps or phases. It includes a constraint as an upper limit on available funds in each phase. The third model is an extension of the second model, with the introduction of the new pipe's diameter as a decision variable. The details of the model are presented next, beginning with some of the key equations involved, followed by the details of the three individual models and a brief description of genetic algorithms.

Dandy and Engelhardt (2001) have considered direct and indirect costs as the cost components of repairing a pipe failure. The direct costs cover pipe repair and replacement. The indirect costs comprise of consequential costs such as inconvenience caused to traffic and any damage incurred by third parties. The direct costs have been obtained from water authorities and the indirect costs have been modelled using failure cost factors for land use. For example, failure cost factors for pipes in rural or residential areas are generally lower than those used in commercial areas or along major roads. This is mainly due to the fact that the potential for interruption to traffic and third party damage is relatively lower in the rural and residential areas.

Failure costs for link *ij* include repair costs related to the existing pipes and those that are related to the new pipe. For a design horizon of 50 years, Dandy and Engelhardt (2001) have calculated the expected repair costs in ten 5-year time steps or phases. The failure rate at each 5-year time step is assumed to be averaged over the 5-year time period. The present value of repair costs of existing pipes ($PVOB_{ij}$) is

$$PVOB_{ij}\left(T_{rij}\right)=\sum_{st=0}^{T_{rij}}\frac{k\cdot BR\left(d_{ij},st\right)\cdot BC\left(d_{ij}\right)\cdot FCF\left(LU_{ij}\right)L_{ij}}{\left(1+r\right)^{5st}} \qquad (3.5)$$

where T_{rij} is the time step when the existing pipe is replaced for link ij given a discount rate of r; st = time step (0, 1, 2, ..., T_{rij}); d_{ij} is the diameter of existing link ij; $BR(d_{ij}, st)$ is the failure rate for pipe ij of diameter d_{ij} at time step st in expected bursts/km/year; $BC(d_{ij})$ is the burst cost or direct repair costs for link ij and $FCF(LU_{ij})$ is the failure cost factor for land use LU_{ij}. L_{ij} is the link length (km) and k = 2.5 for st =0 or T_{rij}, otherwise it equals 5. Thus, k represents the number of years in time step st = 0, 1, 2, ..., T_{rij}; over which the failure rate of existing pipes has been averaged. This implies that the numerator in Eq. (3.5) is the repair cost related to the existing pipe ij for each phase or time step st = 0, 1, 2, ..., T_{rij}. The denominator is a factor for discounting the repair cost to the base year at the beginning of the design horizon.

The present value of the future repair costs for a new replaced pipe of diameter dn_{ij} when installed in the time period T_{rij} is referred to as $PVNB_{ij}$ and given by

$$PVNB_{ij}\left(dn_{ij},T_{rij}\right)=\sum_{st=T_{rij}}^{10}\frac{k\cdot BR\left(dn_{ij},st\right)\cdot BC\left(dn_{ij}\right)\cdot BCF\left(LU_{ij}\right)L_{ij}}{\left(1+r\right)^{5st}} \qquad (3.6)$$

in which $BR(dn_{ij}, st)$ is the failure rate for a new replaced pipe ij of diameter dn_{ij} at time step st in expected bursts/km/year; the time step st = T_{rij}, T_{rij} +1, T_{rij} +2, ..., 10; $BC(dn_{ij})$ is the burst cost for the new replaced pipe ij of diameter dn_{ij}. k = 2.5 for st =T_{rij} or 10, otherwise it equals 5. Thus, k represents the number of years in time step st = T_{rij}, T_{rij} +1, T_{rij} +2, ..., 10; over which the failure rate of new replaced pipes has been averaged. This implies that the numerator in Eq. (3.6) is the repair cost related to a new replaced pipe for each phase or time step step st = T_{rij}, T_{rij} +1, T_{rij} +2, ..., 10.

The present value of the replacement cost of a pipe with diameter dn_{ij} in time period T_{rij} is referred to as NPR_{ij} and given by

$$NPR_{ij}\left(dn_{ij},T_{rij}\right)=\left(\frac{1{,}000\cdot R\left(dn_{ij}\right)\cdot L_{ij}}{\left(1+r\right)^{5T_{rij}}}\right) \qquad (3.7)$$

where $R(dn_{ij})$ = cost/m length of replacing the pipe with diameter dn_{ij}. The numerator in Eq. (3.7) is the replacement cost related for a pipe in each phase or time step and the factor of 1000 is for converting the units of pipe length from kilometres to metres. The denominator is a factor for discounting the pipe replacement cost to the base year at the beginning of the design horizon.

Dandy and Engelhardt (2001) have used regression analysis techniques to obtain the failure rate of pipes. Equations that can be used for the prediction of pipe failure have been developed based on recorded failure data. For example, the equation that has been obtained for estimating the failure rate for cast iron cement-lined (CICL) pipes, y_{CICL}, is

$$y_{CICL} = 0.02214 \cdot \exp\left(-0.00864 * d_{ij}\right) * age_{ij}^{1.337} \tag{3.8}$$

in which age_{ij} is number of years from installation of link ij.

The equation that has been obtained for estimating the failure rate for asbestos cement (AC) pipes, y_{AC}, is

$$y_{AC} = 0.001974 * \exp\left(-0.00974 * d_{ij}\right) * age_{ij}^{1.808} \tag{3.9}$$

In the case of replaced pipes, Eqs. (3.8) and (3.9) can be used by simply substituting d_{ij} with dn_{ij}. From these equations, the longer a pipe is left without being replaced, the higher the failure rate. They have assumed that failure curves of mild steel cement-lined pipes follow those of CICL pipes due to their metallic nature. They have also assumed that ductile iron cement-lined (DICL) and poly-vinyl chloride (PVC) pipe types follow the failure curves of asbestos cement (AC) pipes. The interested reader may consult Dandy and Engelhardt (2001) for details of the above study on the failure rate of pipes.

The rehabilitation model is set up as an optimisation problem for minimising network pipe costs, e.g. repair costs, replacement costs, supply and installation costs; and solved using a genetic algorithm technique, in three separate models which are described next.

The first of the three models that have been developed to obtain a rehabilitation strategy is the single time-step case model. This model is a function that aims at minimising the system cost with an option for each link, of immediate replacement or being left as it is. Any pipe that is not replaced incurs repair costs over the entire design horizon. The costs associated with pipes that are replaced immediately are replacement costs of the pipes and the future repair costs for the new pipes, which are considerably lower than those of the pipes that are not replaced, due to the pipe age difference.

The second model is referred to as the multiple time-step case and is concerned with the scheduling of the works over the design horizon, which is sub-divided into time steps. These time steps or phases are the periods in which a pipe should be replaced. This model has a constraint as an upper bound on the funds that can be spent in any time step ensuring an even spread of pipe replacements over the design horizon. This constraint on the available funds is modelled using genetic algorithms by imposing penalty costs for constraint violation.

The third model is the multiple time-step case, which introduces the diameter of the replaced main as a decision variable. The reduction of rehabilitation costs by use of smaller diameter pipes can end up compromising network pressures and flow velocities in the pipes, thus violating the acceptable limits for these values. This model is similar to the second model but it also includes a set of constraints on the hydraulic performance of the rehabilitated system. The genetic algorithm model that has been used is interfaced with a hydraulic simulator to solve the constitutive equations and obtain nodal pressures and link velocities. The nodal pressures are checked against allowable minimum values and the pipe velocities against allowable maximum values. Modelling of these constraints by the genetic algorithm model involves imposing penalty costs whenever these constraints are violated.

Dandy and Engelhardt (2001) have noted that genetic algorithms are very effective at manipulating integer decision variables such as discrete pipe diameters, a problem that is not easily solved by other optimisation methods. Thus, the reason for the choice of genetic algorithms as the optimisation technique for the rehabilitation problem. The details of genetic algorithms are presented next.

Savic and Walters (1997) have noted that genetic algorithms have several formulations and they have given a general description that covers the key features as presented next. Genetic algorithms are a type of optimisation technique that is used to obtain a solution to a problem by replicating the process of natural selection in genetics. Genetic algorithms are based on processing a set of trial solutions referred to as a population. Each trial solution is made up of a set of assumed values for the decision variables, which are represented using sets of binary character strings to form a chromosome.

The general procedure (Savic and Walters, 1997) involves allowing the population to evolve over a number of generations. At each generation, the binary strings are converted into parameter values that are used to evaluate the objective function in order to obtain a measure of how good (fitness) each chromosome is with respect to the objective function. Then, based on the fitness values of individual solutions, assumed values for decision variables from different solution trials are recombined to produce offspring for the next generation. The recombination operation is referred to as "crossover" since it involves the transfer of genetic material from one chromosome to another. There is a higher chance of a chromosome with a high fitness value to be selected for producing offspring and this ensures that the fitness level of next the generation is higher than that of the older one. Alternatively, assumed values for decision variables from different solution trials may be adjusted slightly in a process called mutation to form new trial solutions for the next generation. It is not always possible for the objective function to be evaluated using all the chromosomes within the constraints of the problem. Penalty costs are introduced whenever constraints are violated to reflect the fact that the objective function has been poorly evaluated for a given chromosome. Thus, the fitness of such a chromosome would be low due to the high cost obtained from evaluating the objective function.

The above processes are continued until an optimal solution or a solution that cannot be improved any further is obtained. The interested reader may consult Savic and Walters, (1997) and Dandy and Engelhardt (2001) for the detailed application of genetic algorithms to water distribution networks.

The model by Dandy and Engelhardt (2001) for rehabilitation strategies has been applied to a large system in Adelaide with 488 pipes with diameters ranging from 150mm to 300mm. This model has been successfully used to identify pipes that require rehabilitation, the scheduling of works within budget constraints, and the diameters of replaced pipes in each time-step. The computational time increases as the complexity of the problem increases from the single time-step case, through to the multiple time-step case with changing diameters. For example, the multiple time-step case, which includes pipe diameters as decision variables, requires a computational time of about 1.45 hours. It is not clear from their publication what type of computer was used in terms of microprocessor speeds and memory capacity. The genetic algorithm has also been used to investigate the sensitivity of the results to various parameters used in the rehabilitation process e.g. indirect repair cost factors whose role in a rehabilitation policy remains an unresolved issue.

All in all, the model presented is a powerful optimisation tool for rehabilitation policies. It is based on genetic algorithms and applicable to large water distribution systems. The formulation allows for pre-defined multiple time steps and is used to directly identify pipes for rehabilitation. The methodology used can be extended to include scheduling of works for system upgrading. The genetic algorithm technique used for optimisation has various advantages. Unlike many other optimisation techniques, it has the ability to use a population of solutions and search a vast solution space that greatly increases the chances of convergence on a local minimum solution. It also has the flexibility of accommodating any integer, non-linear, logical or discontinuous objective function or constraints in the optimisation. However, due to its stochastic nature, a global optimum solution is not guaranteed (Dandy and Engelhardt, 2001). Worse still, the need of carrying out many simulations of the physical system inevitably implies that the computational requirements are high.

Despite the complexity of the system-wide model that has been presented, it does not consider the deterioration of hydraulic capacity of pipes and the timing of rehabilitation explicitly. Network performance in terms of reliability and failure tolerance is also not handled. Dandy and Engelhardt (2001) have noted this fact and concluded that the rehabilitation strategy should be extended to include a reliability criterion in a multi-objective framework so that the cost and timing of replacement may be obtained for a given level of service.

3.3 SUMMARY AND CONCLUSION

This chapter has provided a review of optimal long-term rehabilitation and upgrading techniques with special attention being paid to selected models with features that have been adopted and/or modified in this research to produce a model for the optimal long-term upgrading of water distribution networks. The complexity of the long-term rehabilitation/upgrading problem has been shown.

A number of shortcomings of the models by past researchers have been highlighted. In brief, some models do not consider the structural and hydraulic integrity of the network. Others do not directly identify the components to be upgraded or consider the timing of upgrading. Individual asset-based models do not explicitly consider the fact that the hydraulic behaviour of the network should be determined simultaneously for all pipes and interactions between them. System-wide models are generally complex, have high computational demands and are limited to simplistic networks.

In the next chapter, a module for the optimal design and long-term upgrading of water distribution networks is presented. This module applies some of the concepts that have been discussed in this chapter and attempts to address some of the shortcomings identified. The overall complex problem of long-term upgrading and related issues presented in the review in Chapters 2 and 3 is best tackled by a holistic approach. This approach has been implemented in this research, by developing an Integrated Model that is detailed in Chapter 8. The Integrated Model has separate modules for network design, hydraulic simulation, assessment of network performance and the analytic hierarchy process; and it is set in a multi-objective framework in that it includes network performance, economic, social and environmental issues. The individual modules of the Integrated Model are developed and presented in Chapters 4 to 7.

CHAPTER FOUR

OPTIMAL DESIGN AND UPGRADING MODEL

4.1 INTRODUCTION

The concepts of network rehabilitation and upgrading have been introduced in the previous chapter and a detailed review of various network rehabilitation and upgrading techniques has been presented. The shortcomings of these models have also been detailed in Chapter 3. In this chapter, a new model for the optimal long-term upgrading of water distribution networks is described in detail. This model is referred to herein as the Network Design Module. A FORTRAN 95 computer program has been encoded to execute the module. This module forms part of the Integrated Model for long-term upgrading of water distribution networks. The module is primarily for the optimal network design process and for determining the scheduling and magnitude of capacity expansion of new and existing networks.

As mentioned earlier in Chapter 3, a good upgrading strategy should be able to meet the network hydraulics and economics, water quality and performance requirements. The proposed model in this chapter concentrates on the first three of these criteria. The performance aspect is demonstrated in Chapter 5 and also included later in Chapter 8, when the Network Design Module is presented in a multi-criteria decision making framework of the Integrated Model.

As detailed earlier in Chapter 3, previous work in this area did not address the issues involved in network upgrading and/or rehabilitation in a holistic fashion. For example, the model of Lansey et al. (1992) did not tackle the issue of timing automatically but used pre-defined periods for upgrading; the models by Walski and

Pellicia (1982) and that of Lansey et al. (1992) did not address the deterioration of pipes over time; and the model of Dandy et al. (1985) did not consider the deterioration of structural and hydraulic integrity of the network. Also, the model proposed by Kleiner et al. (1998) does not directly identify the pipes due for replacement in the network but rather, the funds required for replacement of the pipes. Although the model of Halhal et al. (1997) and that of Dandy and Engelhardt (2001) can be applied to real-life systems using genetic algorithms, they are quite complex and have high computational demands.

This chapter presents a model that can assist in decision-making for a network upgrading strategy with due consideration of the pipe condition over time. The significance of the proposed method is in its ability to explicitly consider deterioration over time of both the structural integrity and hydraulic capacity of every pipe, and to allow for the direct and indirect failure costs. The model directly identifies pipes for upgrading based on the options of paralleling and replacement. The model's basic formulation uses the segmental approach to implement the upgrading strategy in which each link is sub-divided into sections as decision variables. It considers multiple time-steps in that the overall design horizon is subdivided into two phases whose duration is automatically determined by the model. For each phase, a pre-defined feasible flow distribution is used to reduce the dimensionality of the problem. This enables the application of linear programming in the simultaneous analysis of the distribution network economics and hydraulic performance for various design periods. The model uses dynamic programming by varying the duration of the first and second phase to obtain the cheapest combination and thus, the timing and magnitude of the upgrading. It considers joint water pricing and capacity expansion policies and it has links with a network performance module that calculates the network reliability and failure tolerance. The model can also be used for rehabilitation strategies.

In the next section, the details of the formulation of the upgrading problem are presented. Techniques that have been used to reduce the dimensionality of the upgrading problem are then presented. This is followed by the details of the computer program for the Network Design Module. The network design module is then applied to some examples, followed by the presentation and discussion of the results together with a sensitivity analysis.

4.2 FORMULATION OF UPGRADING PROBLEM

The main options for upgrading used in this model are replacing and paralleling of pipes. The structural integrity and water quality is improved by replacing old pipes in the network with new ones of similar or larger diameters, and with lower roughness. The carrying capacity of a given reach in the network is improved by using a parallel pipe. In this study, relining an existing pipe with a smooth liner, after scouring away encrusted material, has not been considered as one of the options for upgrading the network because it is limited to a maximum diameter of the existing pipe. It is therefore not likely to cope with significant future increases in demand that dictate the need for network upgrading. Relining is generally not much cheaper than paralleling or replacement especially for smaller pipe sizes less than 250mm in diameter (Walski et al., 2001).

In this research, the upgrading strategy has been assumed to be over a planning horizon of about 20 years, sub-divided into two design periods or phases. This provides a chance to evaluate and adjust any related uncertainties like interest rates, population and demand growth forecasts, inflation rates, growth rates for pipe failures, etc., at the end of the first phase. The two types of pipe deterioration considered in the upgrading model are hydraulic capacity and structural integrity. Deterioration of the hydraulic capacity of pipes results in a reduction of supply; and the deterioration of structural integrity results in increased breakage rates. These two types of pipe deterioration result in increased pipe maintenance costs.

In the first design period (Phase I) of the planning horizon, each reach or link has two key design variables, *Ln*, the length and *Dn*, the diameter. In the second design period (Phase II), each reach is assigned five variables and a typical link representation is shown in Figure 4.1. The five variables are: (i) *Le*, the existing length in section A-B, (ii) *Lp*, the parallel pipe length, (iii) *Lr*, the length of replaced pipe, (iv) *Dp* the diameter of the parallel pipe, (v) *Dr* the diameter of the replaced pipe.

In reality, the upgrading problem is a non-linear problem that would require non-linear programming. But given the total number of decision variables that would be involved, non-linear programming would be limited to small networks. To reduce the dimensionality of the problem, an entropy-based approach is used for feasible flow distribution (Yassin-Kassab et al., 1999) and then linear programming is used directly to obtain the optimal solution. The use of segmental pipes to make up a link has been applied in several optimal design models for looped networks (e.g. Alperovits and Shamir, 1977). The timing or optimal scheduling, cost and magnitude for the long term upgrading strategy of each design option is carried out using dynamic programming.

The planning problem is to find that time sequence of capacity expansions that minimises the present value of total future costs (e.g. in United States dollars as all costs that have been used in this study) and meets the projected requirements; giving due consideration to the fact as the network ages, there is an increase in the breakage rate of pipes and a reduction in their hydraulic carrying capacity. For a given design horizon, the aim is to minimise the total cost of upgrading or increasing the capacity of the network by replacing and/or paralleling of pipes, subject to a number of constraints. The main constraints are: pipe head loss equations; nodal flow continuity equations; and equations of energy conservation for loops and paths in the network. There are additional constraints namely: bounds on nodal heads, link-flow velocities and pipe diameters; summation of section lengths of each reach or link; summation of link segment lengths; and non-negativity of link-segment lengths.

The details of the costs and constraints involved are presented next, followed by a statement of the planning problem.

4.2.1 Details of the Various Costs Involved

The cost, $C_\tau(s_\tau, r_\tau)$, of adding capacity, r_τ, in each period or design phase τ, is a function of that added capacity, as well as the existing capacity, s_τ, at the beginning of the period or the design phase. In this study, it has been assumed that construction costs are incurred at the beginning of each period. Capital is to be raised by borrowing at an annually compounded interest rate of b%. All borrowed capital has to be paid back at the end of the design horizon, i.e. after d years. Thus, the overall objective function cost represents the present value of the total end debt as follows:

$$Cost = \sum_{\tau=1}^{2} \beta_\tau C_\tau(s_\tau, r_\tau)(1+b)^{(d-v)} \tag{4.1}$$

where $C_\tau(s_\tau, r_\tau) = f1 + f2 + f3$ (4.2)

$f1$ represents pipeline costs which include installation, paralleling, replacement, and repair costs. $f2$ is the cost of setting up construction plant and machinery at the beginning of each phase. $f3$ represents costs that vary with the magnitude of installed capacity. Detailed descriptions of these costs ($f1$, $f2$, and $f3$) will be presented shortly. The term $(1+b)^{(d-v)}$ is the compound factor and the symbol v represents the number of years preceding a design phase. $v = 0$ when $\tau = 1$; and $v = T1, \ldots, T2$ when $\tau = 2$; where $T1$ is the lower limit for the end of Phase I and $T2$ is the upper limit for the end of Phase I. For example, in Phase I, $v = 0$, and if Phase II is carried out in the 9th year of the design horizon, $v = 8$ in Phase II; also, for a given design horizon of 20 years (d=20), typical values of $T1$ and $T2$ can be selected from 5, 6,..., 15 years. Eq (4.1) represents a two-Phase sequence of upgrading and is the sum of the Phase I and II costs. Various such costs can be obtained by varying v from $T1$ to $T2$.

β_τ is a product of a discount factor, $(1+r)^{-v}$, and a price increase factor, $(1+c)^{v}$. This means that the discount rate is assumed to be r% per annum, in each period (τ), and that there is a general increase in construction costs at a rate of c% per annum. It has also been assumed that the values of c and r are equal, thus β_τ has unit value. This assumption is based on the fact that the requirement for this study is an economic analysis aimed at comparing alternatives based on the present value of their costs;

but would probably be unsatisfactory for a financial analysis, which aims at assessing the profitability of a project with emphasis on cash inflows and outflows (Potts, 1996).

4.2.1.1 Pipeline Costs

To minimise pipeline costs is an optimisation problem that is non-linear with many variables. Such a problem is extremely difficult to solve (Yates et al., 1984). The problem may be simplified by using discrete commercially available pipe diameters as opposed to continuous diameters. This strategy should be combined with the use of a pre-specified pipe flow distribution, facilitating the formulation of the problem as a linear programming problem in which lengths of discrete diameters are unknowns rather than the uniform diameters in which the problem is non-linear. The pipeline costs are detailed below as follows:

$$f1 = (f1_a + f1_b + f1_c) \qquad (4.3)$$

where $f1_a$ and $f1_b$ represent costs of new and parallel pipeline and include supply, installation and the related discounted future failure costs. The cost relationship is the same for $f1_a$ and $f1_b$ because in each case, it covers newly supplied pipes installed in freshly excavated trenches and the related failure costs. The only difference is that parallel pipeline trench parallels an existing pipeline trench. All pipes used for this study are made of polyvinyl chloride (PVC) material. However, the model can accommodate pipes of other materials such as asbestos cement (AC), ductile iron (DI), galvanised iron, (GI), etc. All that is required is to change the cost constants that will be presented shortly, according to the pipe material.

The unit costs of new and parallel pipes to cover supply and installation have been taken as those used for the Wobulenzi Water Supply Project (Associated Consulting Engineers, 1995a). The details of this project are presented later as a case study in Chapter 8. These costs have been plotted on a scatter plot or a graph that shows how data points are scattered (Johnson, 2000), as Wobulenzi costs in Figure 4.2. In this study, a curve fitting technique called the method of least squares (Johnson, 2000) has been used to fit a regression curve to the data and obtain the equation of this curve as shown in the figure. This curve of best fit represents the relationship between the unit costs of PVC pipes and their diameters. The regression correlation coefficient, R^2, which shows how well the equation has fitted to the data or the goodness of fit is also shown. The value for R^2 lies between zero and one and a value close to one suggests that the curve fits well to the data (Kirkpatrick, 1974; Johnson, 2000). Since the R^2 value of the curve for this data is 0.996 (Fig. 4.2), this exponential curve is a good fit and a satisfactory representation of the relationship between the unit costs and diameters. The unit costs used in this study have also been compared to the costs for the supply and installation of PVC pipes obtained from Langdon and Everest (2000), which are shown in Figure 4.2 as Langdon and Everest costs. From this comparison, the coefficient of linear correlation of the two sets of costs has been calculated as 0.991. Since this value is close to one, it shows that there is a strong positive correlation between the two sets of unit costs (Kirkpatrick, 1974).

Thus, the cost relationship for $f1_a$ and $f1_b$ is detailed as follows:

$$f1_a = f1_b = \sum_{ij\in IJ}\sum_{m=1}^{N_{ij}}\left(\gamma_p * \exp\left(c_p * D_{ijm}\right) * l_{ijm} + REP_{ijm}\right) \tag{4.4}$$

in which D_{ijm} and l_{ijm} are the diameter and length (in meters) of segment m of link ij respectively. N_{ij} is the number of segments specified for link ij. The first term in the outer brackets represents the exponential cost relationship for the supply and installation costs of new and parallel pipes. γ_p and c_p are empirical cost constants for new and parallel pipes that are specified by the user; typical values that have been used in this study are γ_p = 32.093 and c_p = 3.7 (Figure 4.2), where the costs are in US dollars and the pipe material is polyvinyl chloride (PVC). For pipes of other materials, these cost constants may be obtained in a similar manner by regression analysis or otherwise, and used as appropriate. REP_{ijm} represents the failure costs for segment m of link ij consisting of the present value of direct and indirect failure costs for any pipe older than 5 years. New pipes are assumed to have no repair costs during their first 5 years (Directorate of Water Development, 1999) or 10 years (Halhal et al., 1997) as they are usually under warranty.

The relationship for failure costs is given by Dandy and Engelhardt, (2001), as

$$REP_{ijm} = \sum_{t=tb}^{tr} \frac{J(t)_{ijm} * CB_{ijm} * FCF(LU_{ij}) * l_{ijm}}{(1+r)^{t-ts+1}} \tag{4.5}$$

where r is the discount rate; ts is the first year of a given design period and tr is the last year of a design period, e.g. if the second design period starts in the 13[th] year of a 20-year design horizon, ts = 13 and tr = 20. tb is the time when a pipe in a given design period starts to incur repair costs. tb has been taken as 6 years for replaced and parallel pipes (a warranty period of 5 years has been assumed); and for new pipes in the first design period. $tb = ts$ for existing pipes in the second design period. For the optional provision of obtaining the best rehabilitation strategy of an existing network in the first design period, $ts = tr$, i.e. the time for the first phase is set to equal the design life of the existing network. $FCF(LU_{ij})$ is the failure cost factor for land use, LU_{ij}, for link ij. The failure cost factors cater for indirect costs caused by pipe failures like disruption to traffic and damage incurred by third parties (Dandy and Engelhardt, 2001). For segment m of link ij, CB_{ijm} is the repair cost per break and $J(t)_{ijm}$ is the break rate (breaks/km/year) in year t.

The repair costs per break have been obtained in this study based on figures by Dandy and Engelhardt (2001) by using construction indices for the general annual increase in construction costs (Tweeds, 1994, Langdon and Everest, 2000), to obtain figures for 2002. These costs have been plotted on a scatter plot in Figure 4.3. In this study, the method of least squares (Johnson, 2000) has been used to fit a regression curve to the data and obtain the equation of this curve as shown in the figure. This curve represents the relationship between the repair costs of the pipes and their diameters. The regression equation of this curve is shown in the figure, together with its R^2 value of 0.9885. Since the R^2 value of the curve is close to one, this curve is a good fit and a satisfactory representation of the relationship between the repair costs and the diameters (Kirkpatrick, 1974).

Thus, the relationship for repair costs per break has been obtained as

$$CB_{ijm} = \gamma_{br}(D_{ijm} * 1000)^{\Phi} \tag{4.6}$$

where γ_{br} and Φ are the empirical break repair cost constants that are specified by the user. Typical values that have been used in this study are γ_{br} = 108.87 and Φ =

0.6067 (Figure 4.3) where the diameter D_{ijm} is in metres, the pipe material is PVC and the costs are in US dollars.

The break rate for asbestos cement pipes is given by (Dandy and Engelhardt, 2001)

$$J(t) = 0.001974 * \exp(-0.00974 * D_{ijm}) * age_{ijm}^{1.808} \tag{4.7}$$

where age_{ijm} is number of years from installation for segment m of link ij. Dandy and Engelhardt, 2001 have assumed that failure curves for PVC pipes just like those of ductile iron-cement lined pipes (DICL), follow the failure curves of asbestos cement (AC) pipes on the grounds that PVC and DICL pipes are a relatively new type of pipes on the market and are more likely to have a low failure rate as that of asbestos cement pipes. This assumption has been adopted throughout this thesis.

The costs, $f1_c$, of replacing deteriorated pipes cover the actual pipe replacement costs together with the repair costs associated with the new pipes. The actual pipe replacement costs are assumed to be about 5% higher than parallel pipe costs (Directorate of Water Development, 1999). This may be due to the fact that apart from supply and installation of new pipes, replacement costs may also involve uninstalling existing pipes. The related coefficients and constant for replaced pipes have been obtained based on this assumption. Thus, the cost relationship for $f1_c$ is given by

$$f1_c = \sum_{ij \in U} \sum_{m=1}^{N_{ij}} \left(\gamma_r * \exp(c_r * D_{ijm}) * l_{ijm} + REP_{ijm} \right) \tag{4.8}$$

where the diameter D_{ijm} and length l_{ijm} are in metres; the first term in the outer brackets represents the actual pipe replacement costs in which γ_r and c_r are empirical cost constants for replaced pipes that are specified by the user. Typical values that have been used in this study are γ_r = 33.928 and c_r= 3.7. REP_{ijm} represents the repair costs associated with these new pipes and the repair costs $f1_c$ are in US dollars.

4.2.1.2 Setting-up Costs

The cost, $f2$, of setting up construction plant and machinery at the beginning of each phase also includes mobilisation and setting up all the necessary facilities for the contractor and his workforce.

4.2.1.3 Costs that Vary with the Magnitude of Installed Capacity

The costs, $f3$, that vary with the magnitude of installed capacity in a particular phase, were included on the grounds that the total volume of water released from a reservoir into the system has a proportion of the costs attributed to treatment, transmission of water to the main reservoir, energy costs of water transmission by pumping, capital costs associated with an increase in water supply (e.g. new wells, pumps, etc.), capital costs of expanding sewage treatment plants, operation and maintenance costs, and, any other overheads like administration and billing costs, etc. (Directorate of Water Development, 1999). The generalised relationship that was assumed to represent all these costs can be stated as follows:

$$V_{costs} = VC * Q_{inst}^{VE} \tag{4.9}$$

where Q_{inst} is the installed capacity in a particular phase in l/s, and V_{costs} is in US dollars, VC and VE which are a cost coefficient and exponent respectively, vary depending on the above factors and are specified by the user. Typical values e.g. those used in this study for VC and VE are 130 and 1.6 respectively. Conversion factors can be used as appropriate for other currencies.

4.2.2 Main Constraints

The main constraints are the governing equations for flow in water distribution and their details are presented in this sub-section. These constraints consist of pipe head loss equations, nodal flow continuity equations, and equations of energy conservation for loops and paths in the network.

4.2.2.1 Head Loss Equations

The Hazen-Williams equation for the head loss in a pipe is given by

$$h_{ijm} = \alpha l_{ijm} \left(q_{ijm} / C_{ijm}\right)^{1.852} D_{ijm}^{-4.87} \qquad \forall ijm \tag{4.10}$$

in which q_{ijm}, h_{ijm} and C_{ijm} are the flow rate, head loss and Hazen-Williams hydraulic conductivity value for segment m of link ij respectively. α is a dimensionless conversion factor for units = 10.67 in S.I. units.

Equation (4.10) is non-linear in q_{ijm}, C_{ijm} and D_{ijm}. But with the segmental approach (Alperovits and Shamir, 1977) used in this study, the link segment flows are pre-defined and pipe diameters are selected from a candidate list of commercially available diameters with known Hazen-Williams coefficients. Thus Equation (4.10) is linearised by rewriting it in terms of the segment length l_{ijm} as follows,

$$h_{ijm} = G_{ijm} l_{ijm} \qquad \forall ijm \tag{4.11}$$

where G_{ijm} is the hydraulic gradient or the head loss per unit length that is dimensionless and is given by

$$G_{ijm} = \alpha \left(q_{ijm} / C_{ijm} \right)^{1.852} D_{ijm}^{-4.87} \qquad \forall ijm \tag{4.12}$$

C_{ijm} deteriorates over time at a rate that varies according to the pipe type, supplied water quality, and the operation and maintenance practices. To model the effect of aging on the carrying capacity of pipes, the equation of Sharp and Walski (1988) has been used in this study as

$$C_{ijm}(t) = 18.0 - 37.2 \log \left[\frac{e_{0ijm} + a_{ijm}\left(age_{ijm}\right)}{D_{ijm}} \right] \tag{4.13}$$

in which, for segment m of link ij, $C_{ijm}(t)$ is the Hazen-Williams hydraulic conductivity coefficient in year t that is used to replace C_{ijm} in equation (4.10), e_{0ijm} is the initial roughness (mm) at time of installation and a_{ijm} is the roughness growth rate (mm/year) and D_{ijm} is the segment diameter in millimetres.

4.2.2.2 *Continuity Equations*

Inflows and outflows must balance at each node. Thus, the nodal flow equilibrium or continuity equations are

$$\sum_{ij \in IJ_j} \sum_{m=1}^{N_{ij}} q_{ijm} = Q_j \qquad j = 1, \ldots, NJ \tag{4.14}$$

in which N_{ij} is the number of segments specified for link ij, IJ_j is the total number of links incident on node j. Q_j = inflow or outflow at node j and it represents the nodal demand in cases where j is a demand node.

The demand value used for Q_j is the design demand value for the given design period based on nodal base demand or the demand in the first year of the entire design horizon. The design demand is obtained by projecting or forecasting the nodal base demand from the first year of the entire design horizon to the end of the respective design period. In this study, optimum pricing or setting water tariffs and its effect on network upgrading has also been investigated. The combined strategy of pricing and upgrading exploits the fact that the price elasticity of demand is not zero. Thus, an increase in the price of water, especially as the demand approaches system capacity, can lead to a reduction in the demand (Twort et al., 2000) and consequently delay the need to upgrade the network. Such a delay results in reduced costs as detailed later in this chapter. Therefore, a demand function that caters for the growth in base demand and the pricing issues (Dandy et al., 1985) has been adopted to represent the nodal demand, Q_j, in Eq. (4.14) as

$$Q_j = Q_{0j}\left(1 + {DGR}/{100}\right)^t \cdot \left({P_t}/{P_0}\right)^{PREL} \tag{4.15}$$

where Q_{0j} is the base demand for node j, DGR is the percentage annual rate of increase of the base demand. DGR is assumed to follow demographic data patterns closely, i.e. it can be approximated to the population growth rate with typical values of 2 to 5 (Dandy et al., 1985). t is the time in years, P_t and P_0 are the price per unit of water or water tariffs (e.g. in \$/m^3) for the year t and the base year respectively, (P_t/P_0) is the tariff increase ratio and $PREL$ is the price elasticity of demand. For cases where the pricing policy is not enforced, $P_t = P_0$. Typical values of $PREL$ are –0.2 to –0.5 (Dandy et al. 1985, Twort et al., 2000). Generally, it is advisable to increase the price of water in the last one or two years of the design period for the best results in order to delay the need to upgrade the network. When there is adequate capacity after upgrading, prices can be reduced in order to utilise the system capacity as fully as possible and maximise economic efficiency (Dandy et al. 1985).

The following examples show how enforcement of a pricing policy leads to a reduction in demand (Example 1) and a delay in the need to upgrade the network (Example 2).

Example 1

Using Eq. (4.15), assume $t = tr = 12$ as the last year of design period I, the base demand $Q_{0j} = 1$ unit and the demand growth rate (DGR) = 4%. When pricing is not enforced and $P_t = P_0$, the design demand or capacity, Q^{req}, for the network is given by projecting the base demand over the design period as

$$Q^{req} = 1\left(1 + \frac{4}{100}\right)^{12} = 1.601 \tag{4.16}$$

Assuming a tariff increase ratio (P_t/ P_0) of 1.333 (a 33.33% increase in the price of water in the last one or two years of the design period) and a price elasticity of demand ($PREL$) of –0.3, the design demand or capacity, Q^{req}, for the network is given by Eq. (4.15) as

$$Q^{req} = 1\left(1 + \frac{4}{100}\right)^{12}(1.333)^{-0.3} = 1.4687 \tag{4.17}$$

The capacity utility of the network is 1.4687/1.601 = 0.917, that is, the network capacity utility is reduced to 91.7% by enforcing the pricing policy. Capacity utility herein refers to actual amount of water consumed by the population as a percentage of the total design capacity.

Example 2

Using the same values for the t, tr, Q_{0j}, DGR, P_t, P_0, and $PREL$ as in Example 1, assume that the period required to give 100% capacity utility with the enforcement of the pricing policy is $tr + TDEL$ years, where $TDEL$ is the delay in years caused by the pricing policy, the relationship for 100% capacity utility is

$$\frac{Q_{0j}\left(1 + \frac{DGR}{100}\right)^{tr+TDEL} \cdot \left(\frac{P_t}{P_0}\right)^{PREL}}{Q_{0j}\left(1 + \frac{DGR}{100}\right)^{tr}} = 1.00 \tag{4.18}$$

cross-multiplying and taking natural logarithms on both sides gives $TDEL$ as

$$TDEL = \frac{\log_e\left[\left(1+{DGR}/{100}\right)^{tr} \Big/ \left({P_t}/{P_0}\right)^{PREL}\right]}{\log_e(1+DGR/100)} - tr = 2.2 \tag{4.19}$$

Thus the pricing policy involving a tariff increase of 33.33% and a price elasticity of demand of –0.3 leads to a delay to upgrade the network (delay in attaining capacity utility), of 2 years. A tariff increase of 25% with $PREL$ = -0.4 also gives a delay of 2 years. This means that one could design a system for a 10-year demand to serve for 12 years. Eq. (4.19) has been used in this study to obtain the delay in the need for upgrading introduced by enforcement of the pricing policy.

4.2.2.3 Equations for Conservation of Energy

For a network, the conservation of energy principle implies that the net loss of energy around each loop is equal to zero. Therefore the equation for each loop is

$$\sum_{ij \in IJ_{lp}} \sum_{m=1}^{N_{ij}} G_{ijm} l_{ijm} = 0 \qquad lp = 1,\ldots,NL \tag{4.20}$$

in which IJ_{lp} represents the links in loop and NL is the total number of primary loops in the network.

For a specific path in the network, the total head loss along the path must equal the difference in head between the end nodes of that path and independent energy conservation equations can be written for pipes along each path. If the end nodes of the path are fixed-grade nodes, an equation for each path may be written as

$$\sum_{ij \in IJ_{sp}} \sum_{m=1}^{N_{ij}} G_{ijm} l_{ijm} = h_{sp} \qquad sp = 1,\ldots,NF \tag{4.21}$$

in which IJ_{sp} represents the links in a path specified between fixed-grade nodes, NF is the total number of such specific paths in the network and h_{sp} is the known head loss for the path between the fixed-grade nodes.

If the end nodes of the path are a source and a terminal node, an equation for each path may be written as

$$\sum_{ij \in IJ_{tn}} \sum_{m=1}^{N_{ij}} G_{ijm} l_{ijm} = H_s - H_{tn} \qquad tn = 1,\ldots,NT \tag{4.22}$$

in which IJ_{tn} represents the links in a path between a source and a terminal node and NT is the total number of paths specified between a source and a terminal node.

4.2.3 Additional Constraints

In this sub-section, the following additional constraints are presented: bounds on nodal heads, link-flow velocities and pipe diameters; summation of section lengths of each link; summation of link segment lengths; and non-negativity of link-segment lengths.

4.2.3.1 Head Bounds

From Chapter 2, Eq. (2.42) of the bounds on heads is reproduced below as

$$H_{\max,j} \geq H_j \geq H_{\min,j} \qquad \forall i \tag{4.23}$$

The Eq. (4.22) for the head loss along any path between a source, s, and a terminal node tn can be rearranged to give

$$H_{tn} = H_s - \sum_{ij \in IJ_{tn}} \sum_{m=1}^{N_{ij}} G_{ijm} l_{ijm} \qquad tn = 1,\ldots,NT \tag{4.24}$$

Therefore substituting Eq. (4.24) into Eq. (4.23) yields

$$H_s - H_{\min,tn} \geq \sum_{ij \in IJ_{tn}} \sum_{m=1}^{N_{ij}} G_{ijm} l_{ijm} \geq H_s - H_{\max,tn} \qquad tn = 1,\ldots,NT \tag{4.25}$$

4.2.3.2 Bounds on Flow-velocities in Link Segments

$$v_{ijm,\max} \geq v_{ijm} \geq v_{ijm,\min} \qquad \forall ijm \tag{4.26}$$

in which v_{ijm} is the flow velocity in segment m of link ij; $v_{ijm,min}$ and $v_{ijm,max}$ are the lower and upper bounds, respectively, on velocities. The velocity, v_{ijm}, is given by

$$v_{ijm} = \frac{4q_{ijm}}{\pi D_{ijm}^{\ 2}} \qquad \forall ijm \tag{4.27}$$

Substituting for v_{ijm} in Eq. (4.26) and rearranging,

$$V_{ijm,\max} \geq \frac{4q_{ijm}}{\pi D_{ijm}^{\ 2}} \geq V_{ijm,\min} \qquad \forall ijm \tag{4.28}$$

4.2.3.3 *Bounds on Diameters of Link Segments*

$$D_{ijm,\max} \geq D_{ijm} \in D_D \geq D_{ijm,\min} \qquad \forall ijm \tag{4.29}$$

where D_D is the set of commercially available link diameters on the candidate list; $D_{ijm,max}$ and $D_{ijm,min}$ are the upper and lower bounds, respectively, on diameters of segment m of link ij.

4.2.3.4 *Summation of Link Section Lengths*

The sum of the individual lengths of the sections for each link in phase II, i.e. the existing section length, Le_{ij}, the parallel section length, Lp_{ij}, and the replaced section length, Lr_{ij}, is equal to the total length of the link. Thus,

$$Le_{ij} + Lp_{ij} + Lr_{ij} = L_{ij} \qquad \forall ij \tag{4.30}$$

4.2.3.5 *Summation of Link Segment Lengths*

The sum of all the link segment lengths equals the total link length.

$$\sum_{m=1}^{N_{ij}} l_{ijm} = L_{ij} \qquad \forall ijm \tag{4.31}$$

4.2.3.6 *Non-negativity of Link Segment Lengths*

$$l_{ijm} \geq 0 \qquad \forall ijm \tag{4.32}$$

4.2.4 Problem Statement

The overall objective is to minimise the total costs associated with pipe breakage repairs, designing a network in Phase I and upgrading it by paralleling and/or replacement in Phase II. The objective function cost of Eq. (4.1), and all the constraints are put together to represent the planning model or the optimal upgrading problem. The following is the formal statement of the optimal upgrading problem described to this point and stated as Problem 4.1.

Problem 4.1

$$\text{Minimise } Cost = \sum_{\tau=1}^{2} \beta_\tau C_\tau(s_\tau, r_\tau)(1+b)^{(d-v)} \tag{4.33a}$$

$$\text{where } C_\tau(s_\tau, r_\tau) = f1 + f2 + f3 \tag{4.33b}$$

in which $f1$ represents pipeline costs which include installation, paralleling, replacement, and repair costs. $f2$ is the cost of setting up construction plant and machinery at the beginning of each phase. $f3$ represents costs that vary with the magnitude of installed capacity. $v = 0$ when $\tau = 1$; and $v = T1, \ldots, T2$ when $\tau = 2$.

subject to

$$h_{ijm} = \alpha l_{ijm} \left(q_{ijm} / C_{ijm} \right)^{1.852} D_{ijm}^{-4.87} \qquad \forall ijm \tag{4.33c}$$

$$G_{ijm} = \alpha \left(q_{ijm} / C_{ijm} \right)^{1.852} D_{ijm}^{-4.87} \qquad \forall ijm \tag{4.33d}$$

$$\sum_{ij \in IJ_j} \sum_{m=1}^{N_{ij}} q_{ijm} = Q_j \qquad j = 1, \ldots, NJ \tag{4.33e}$$

$$\sum_{ij \in IJ_{lp}} \sum_{m=1}^{N_{ij}} G_{ijm} l_{ijm} = 0 \qquad lp = 1, \ldots, NL \tag{4.33f}$$

$$\sum_{ij \in IJ_{sp}} \sum_{m=1}^{N_{ij}} G_{ijm} l_{ijm} = h_{sp} \qquad sp = 1, \ldots, NF \tag{4.33g}$$

$$H_s - H_{\min,tn} \geq \sum_{ij \in IJ_{tn}} \sum_{m=1}^{N_{ij}} G_{ijm} l_{ijm} \geq H_s - H_{\max,tn} \qquad tn = 1, \ldots, NT \tag{4.33h}$$

$$V_{ijm,\max} \geq \frac{4 q_{ijm}}{\pi D_{ijm}^{\ 2}} \geq V_{ijm,\min} \qquad \forall ijm \tag{4.33i}$$

$$D_{ijm,\max} \geq D_{ijm} \in D_D \geq D_{ijm,\min} \qquad \forall ijm \tag{4.33j}$$

$$Le_{ij} + Lp_{ij} + Lr_{ij} = L_{ij} \qquad \forall ij \tag{4.33k}$$

$$\sum_{m=1}^{N_{ij}} l_{ijm} = L_{ij} \qquad \forall ijm \tag{4.33l}$$

$$l_{ijm} \geq 0 \qquad \forall ijm \tag{4.33m}$$

Only a single design load or demand pattern has been considered in Problem 4.1. It is important however to include multiple demand patterns, and this may be done by considering each flow regime explicitly in the constraints. An additional subscript, f, is used to identify the variables in the fth flow regime. Thus, for segment m of link ij, the flow rate, head loss and hydraulic gradient are represented by q_{ijmf}, h_{ijmf} and G_{ijmf}, respectively, for the fth flow regime; $\forall ij$, f=1,..., ND, where ND is the number of demand patterns. Similarly, H_{jf} $\forall j$, $\forall f$, is the head at the node j for the fth load case, etc. All other adjustments are made accordingly to give the planning problem for multiple loading cases, which is stated as Problem 4.2 below.

Problem 4.2

$$\text{Minimise } Cost = \sum_{\tau=1}^{2} \beta_\tau C_\tau(s_\tau, r_\tau)(1+b)^{(d-v)} \tag{4.34a}$$

$$\text{where } C_\tau(s_\tau, r_\tau) = f1 + f2 + f3 \tag{4.34b}$$

subject to

$$h_{ijmf} = \alpha l_{ijm}\left(q_{ijmf} / C_{ijm}\right)^{1.852} D_{ijm}^{-4.87} \qquad \forall ijm, \quad \forall f \tag{4.34c}$$

$$G_{ijmf} = \alpha\left(q_{ijmf} / C_{ijmf}\right)^{1.852} D_{ijmf}^{-4.87} \qquad \forall ijm, \quad \forall f \tag{4.34d}$$

$$\sum_{ij \in U_j} \sum_{m=1}^{N_{ij}} q_{ijmf} = Q_{jf} \qquad j = 1,\ldots,NJ, \quad \forall f \tag{4.34e}$$

$$\sum_{ij \in U_{lp}} \sum_{m=1}^{N_{ij}} G_{ijmf} l_{ijmf} = 0 \qquad lp = 1,\ldots,NL, \quad \forall f \tag{4.34f}$$

$$\sum_{ij \in U_{sp}} \sum_{m=1}^{N_{ij}} G_{ijmf} l_{ijmf} = h_{spf} \qquad sp = 1,\ldots,NF, \quad \forall f \tag{4.34g}$$

$$H_s - H_{\min,tn} \geq \sum_{ij \in U_{nf}} \sum_{m=1}^{N_{ij}} G_{ijmf} l_{ijmf} \geq H_s - H_{\max,tn} \qquad tn = 1,\ldots,NT, \quad \forall f \tag{4.34h}$$

$$v_{ijm,\max} \geq \frac{4q_{ijmf}}{\pi D_{ijm}^{2}} \geq v_{ijm,\min} \qquad \forall ijm, \quad \forall f \tag{4.34i}$$

$$D_{ijm,\max} \geq D_{ijm} \in D_D \geq D_{ijm,\min} \qquad \forall ijm \tag{4.34j}$$

$$Le_{ij} + Lp_{ij} + Lr_{ij} = L_{ij} \qquad \forall ij \tag{4.34k}$$

$$\sum_{m=1}^{N_{ij}} l_{ijm} = L_{ij} \qquad \forall ijm \tag{4.34l}$$

$$l_{ijm} \geq 0 \qquad \forall ijm \tag{4.34m}$$

4.2.5 Details of the Upgrading Methodology

The foregoing formulation in this research has been used for upgrading of the network by combined paralleling and replacement of deteriorating pipes, which in a way caters for network rehabilitation too. This has been done in two distinct phases of the design horizon. The first phase involves the design of a new network and the second phase involves building up on this existing network and expanding it by paralleling and/or replacement of pipes. In order to incorporate the properties of the existing pipes (length, diameter and roughness) in the second phase, the constraints of the optimisation problem must be modified to cater for situations where a link is paralleled or when a link is subject to combined paralleling and replacement. The large number of variables involved in this problem greatly increases its dimensionality. To reduce the complexity and dimensionality of the problem, the following steps have been used in this study: a) the link flows have been determined using appropriate flow distribution methods to facilitate the sizing of links by linear programming (Alperovits and Shamir, 1977), b) a relationship for the flow in a parallel pipe in terms of the total link flow has been utilised, c) an algorithm for determining the design flow for parallel links and limiting the segment diameters on the candidate list has been developed, and d) an equivalent diameter to combine individual segments of each link has been obtained for ease of manipulation and related calculations, e.g. head loss calculations. These issues are presented in the following sub-sections.

4.2.5.1 Flow Distribution Methods

The flow distribution methods that have been used in this study, together with their underlying principles are briefly described in this sub-section.

A – Maximum Entropy Flow Distribution Method

The entropy concept was presented in the context of informational theory by Shannon (1948), who proposed a function as a quantitative measure of the amount of uncertainty in a finite probability distribution (one with outcomes whose probabilities add up to unity). Jaynes (1957) developed the maximum entropy formalism using Shannon's entropy measure and found that a probability distribution scheme whose outcomes have equal probabilities has the maximum value of entropy or the amount of uncertainty. Thus, the uniform distribution, whose outcomes have equal probabilities, has the highest uncertainty (Tanyimboh and Templeman, 1993c).

Tanyimboh and Templeman (1993c) related the entropy concept to water distribution systems based on the fact that there is a substantial amount of uncertainty linked to their design and management e.g., demand variations, change in the growth rate of demand, change in valve status, change in hydraulic capacity of pipes over time, random bursts, fire demands and where they occur, etc. Tanyimboh (1993) developed a function for measuring entropy in water distribution networks.

Tanyimboh and Templeman (1993b) showed that designs with higher entropy values could cope better with eventualities not designed for than those with lower entropy values. They noted that these designs with maximum entropy values had uniform flows (flows that are non-committal to uncertain factors). Based on the maximum entropy formalism, Tanyimboh and Templeman (1993c) concluded that there are advantages in sizing pipes to carry uniform flows especially in the increased flexibility of the design to handle unforeseen changes.

Tanyimboh and Templeman (1993a) developed a maximum entropy flow distribution method for obtaining the link flows of a single-source network. These maximum entropy flows are such that the total outflow at a node (nodal demand and total flow in links with outflow at that node) is equally distributed between the supply paths from the source to that node. This method has been used in this study due to the considerable evidence of a strong relationship between entropy and reliability (Tanyimboh, 1993, Tanyimboh and Templeman, 1993a, Tanyimboh and Templeman, 1993b, Tanyimboh and Templeman, 2000, and Tanyimboh and Sheahan, 2002) implying that maximum entropy flow distribution designs have a high reliability. Furthermore, the method is simple, non-iterative and quickly yields flow distribution results (Tanyimboh and Templeman, 1993a). The description of the method, which has also been used in this thesis, is presented in Appendix A1 and it is followed by an illustration of the method on a two-loop network example (Figure A1.1).

Yassin-Kassab et al. (1999) extended the method of calculating maximum entropy flows for single-source networks and developed a method for calculating maximum entropy flows in multi-source, multi-demand networks. The interested reader may consult the original publication for the details.

B – Shortest Path Flow Distribution Method

The shortest path flow distribution method is a commonly used method for determination of link flows (Orth, 1986). It involves concentrating the bigger proportion of the total inflow to a node (about two-thirds) on the shortest path from the source to the node or the path with the shortest total link length between the source and the node. Loop-completing links are then used to carry the remainder of the flow, which is distributed in such a way as to satisfy nodal flow continuity (Orth, 1986). This method has been used in this study for purposes of comparison with the maximum entropy flow distribution method.

A description of the shortest path flow distribution algorithm (Orth, 1986) is presented in Appendix A2, followed by an illustration of the algorithm on network example (Figure A2.1).

4.2.5.2 Constraint Modification for Flow in the Parallel Pipe

From the flow distribution algorithm, the total flow in each link is known. The aim of this sub-section is to obtain a relationship for flow in the parallel pipe in terms of the total flow in the link. Consider Figure 4.1, for section B-C that is paralleled. The existing and parallel lengths are Le_p and Lp respectively. Let De be the diameter of the existing pipe and Dp that of the parallel pipe; Qt the total link flow; and, Qe_p and Qp the existing and parallel pipe flows respectively. Also, let Ce and Cp be the Hazen William's coefficients for existing and parallel pipe respectively. Then by flow continuity

$$Qt = Qe_p + Qp \tag{4.35}$$

The head loss in the parallel pipe is equal to that in the existing pipe across section B-C and from Eq. (4.10), this head loss is given by

$$\frac{\alpha Le_p Qe_p^{\;1.852}}{Ce^{1.852} De^{4.87}} = \frac{\alpha Lp Qp^{1.852}}{Cp^{1.852} Dp^{4.87}} \tag{4.36}$$

Considering the fact that for most cases in which a parallel pipe is economical, the entire section is paralleled giving a ratio $Lp/Le_p = 1$ (Walski, 1985), it can be assumed that $Le_p = Lp$ for section B-C. Thus, the flow in the existing pipe is

$$Qe_p = Qp\left(\frac{Ce}{Cp}\right)\left(\frac{De}{Dp}\right)^{\left(\frac{4.87}{1.852}\right)} \tag{4.37}$$

substituting Eq (4.37) into Eq (4.35) gives

$$Qp + Qp\left(\frac{Ce}{Cp}\right)\left(\frac{De}{Dp}\right)^{\left(\frac{4.87}{1.852}\right)} = Qt \tag{4.38}$$

and thus, the flow in the parallel pipe expressed in terms of the total link flow is

$$Qp = Qt \bigg/ \left[1 + \left(\frac{Ce}{Cp}\right)\left(\frac{De}{Dp}\right)^{\left(\frac{4.87}{1.852}\right)}\right] \tag{4.39}$$

Therefore, knowing the total link flow, which is also the flow in section A-B of the link, the flow in the parallel pipe of section B-C can be obtained using Equation (4.39). The flow in the replaced section C-D is the same as the total link flow. The head losses for segments in these sections for each link can be obtained using Eq. (4.11) to form the constraints of the linear programming formulation.

4.2.5.3 Algorithm for Flow in Parallel Pipes and Limiting Segment Diameters

It is important to limit the number of segment diameters on the candidate list of diameters for each link in order to reduce the number of variables. This action is aimed at reducing the computational effort and does not reflect a real constraint. Alperovits and Shamir (1977) have asserted that the implicit constraint introduced by such an action may be binding at the computed optimum or may lead to overly stringent conditions, making it difficult for a true optimal solution to be reached. Limiting the segment diameters has to take this fact into consideration.

The hydraulic gradient (Eq. 4.12) for a given link segment is the head loss per unit length of the segment and is dimensionless. Assuming the segment length is measured in metres, the hydraulic gradient can also be expressed in metres per kilometre (HG_{ijm}), by rearranging Eq. (4.10), i.e., dividing the head loss, h_{ijm}, by the link segment length and multiplying it by 1000m giving

$$HG_{ijm} = 1000\alpha\left(q_{ijm} / C_{ijm}\right)^{1.852} / D_{ijm}^{4.87} \qquad \forall ijm \tag{4.40}$$

This equation shows that one does not need to know the link segment length in order to determine its hydraulic gradient of flow. The equation can therefore be used conveniently to check whether violation of a pre-specified limiting condition on the hydraulic gradient has occurred before the optimisation process. Eq. (4.40) has been applied in the process of determining the design flow in parallel pipes and the limits for the largest and smallest link segment diameters for the parallel link.

The appropriate design parallel link flow has to be obtained in order to set limits for the largest and smallest link segment diameters for the parallel link B-C (Fig. 4.1). It should be noted that the parallel pipe flow given by Eq. (4.39) is a function of the segment diameter, *Dp*, and therefore changes with the variation in segment diameters of the candidate list. The limiting conditions used to facilitate this process are a pre-specified minimum velocity, and a maximum hydraulic gradient. The minimum velocity condition is a requirement to minimise stagnant water and ensure that the age of the water in the pipe does not become excessive leading to water quality problems. Typical values of minimum pipe velocity are 0.3m/s (Orth, 1985) and 0.6m/s (Orr et al., 2001). In this study, a minimum velocity of 0.5m/s has been adopted as a compromise between these two values. The maximum hydraulic gradient is set to limit head losses in the pipes. Typical values of maximum hydraulic gradient values are 0.01 and 0.5 (Twort et al., 2000). In this study, a maximum hydraulic gradient of 0.5 or a unit head loss of 50m/km has been adopted. This value is probably reasonable considering that pipe deterioration, which leads to loss of carrying capacity, is considered in the Network Design module. Pre-specifying a very low limit maximum hydraulic gradient could probably lead to overly stringent conditions and infeasibility.

An algorithm for obtaining the parallel link design flow, the minimum and maximum parallel link segment diameters has been developed in this study and used in the Network Design module as detailed shortly. The algorithm is sub-divided into three parts. In Part 1, the maximum link segment diameter is obtained. The parallel link flow is then obtained in Part 2 and finally the minimum link segment diameter is determined in Part 3.

Part 1: Maximum Parallel Link Segment Diameter

1) For a particular link, set the lowest link flow from the demand patterns (each demand pattern has different link flows) as Qt. Use Eq. (4.39) to obtain the parallel link segment flow (Qp_{ijm}), by substituting for Dp with the link segment diameter (D_{ijm}) beginning with the largest available commercial diameter. If the flow (Qp_{ijm}) and diameter (D_{ijm}) give a velocity (Eq. (4.27), with q_{ijm}=Qp_{ijm}) that is less than the minimum velocity ($v_{ijm,min}$), select the next commercial diameter that is a size smaller and repeat this Step. Otherwise go to Step 2.
2) For the parallel link segment flow, Qp_{ijm}, if the corresponding existing link flow for section B-C of Figure 4.1 (Qt - Qp_{ijm}) and the diameter, De, give a velocity that is less than the minimum velocity ($v_{ijm,min}$), select the next commercial diameter that is a size smaller and go to Step 1.
3) If a diameter meets the minimum velocity criterion, take it as the maximum link segment diameter ($MAXDIAP_{ij}$) for the parallel link and perform Part 2. If none of the commercially available link diameters meet the minimum velocity criterion, the parallel pipe is not viable for the link, exit.

Part 2: Design Flow for the Parallel Link

1) Set the largest link flow from the demand patterns as Qt. Use Eq. (4.39) to obtain the parallel link segment flow (Qp_{ijm}), by substituting for Dp with $MAXDIAP_{ij}$. If the flow (Qp_{ijm}) and diameter give a velocity that is less than the minimum velocity ($v_{ijm,min}$) or a hydraulic gradient (Eq. 4.40, with q_{ijm}=Qp_{ijm}) that is greater than the maximum allowable value, change $MAXDIAP_{ij}$ to the next commercial diameter that is a size smaller and repeat this Step. Otherwise, the flow (Qp_{ijm}) is confirmed as the design flow (QPF_{ij}) for the parallel link.

Part 3: Minimum Parallel Link Segment Diameter

1) Set the segment link flow, q_{ijm}, as QPF_{ij} and use Eq. (4.40) to obtain the hydraulic gradient by substituting for D_{ijm} with different segment

diameters. Start with a size smaller than $MAXDIAP_{ij}$, followed by the next commercial diameters that are a size smaller until a hydraulic gradient that is greater than the maximum allowable value is obtained. At this point, the next commercial diameter that is a size larger is the minimum link segment diameter ($MINDIAP_{ij}$) for the parallel link.

Upper Bound for New or Replaced Link Segment Diameter

The limiting maximum diameters of new links in Phase I or replaced link sections in Phase II, can be obtained as detailed in this paragraph. For a particular link, set the lowest link flow from the demand patterns as q_{ijm}. Use Eq. (4.27) to obtain link segment velocity by substituting for D_{ijm} with the largest available commercial diameter. If the velocity is greater than or equal to the minimum allowable velocity ($v_{ijm,min}$), then this diameter is limiting maximum diameter for the new link in Phase I, $MAXDIAL_{ij}$, or $MAXDIAR_{ij}$ for the replaced link section, as appropriate. Otherwise, select the next commercial diameter that is a size smaller and repeat the process of checking that the minimum velocity is exceeded, until a commercial diameter that satisfies the conditions is obtained.

Lower Bound for New or Replaced Link Segment Diameter

The limiting minimum diameters of new links in Phase I or replaced link sections in Phase II, can be obtained as detailed in this paragraph. This process requires a pre-specified maximum allowable link velocity and hydraulic gradient. A maximum allowable velocity of 3m/s (Dandy and Engelhardt, 2001) has been used in this thesis. For a particular link, set the highest link flow from the demand patterns as q_{ijm}. Use Eq. (4.27) to obtain link segment velocity, by substituting for D_{ijm} starting with the largest available commercial diameter. Use Eq. (4.40) to obtain the hydraulic gradient by substituting for D_{ijm} starting with the largest available commercial diameter. If the velocity is less than or equal to the maximum allowable velocity and the hydraulic gradient is less than or equal to maximum allowable hydraulic gradient, then this diameter is the limiting minimum diameter for the new link in Phase I, $MINDIAL_{ij}$, or $MINDIAR_{ij}$ for the replaced link section, as appropriate. Otherwise, select the next commercial diameter that is a size smaller and repeat the process of checking that the maximum allowable velocity and hydraulic gradient are not exceeded, until a commercial diameter that satisfies the conditions is obtained.

4.2.5.4 Equivalent Pipe Diameters

An equivalent diameter is a uniform diameter whose head loss and flow is the same as that of a combination of segment diameters for a link. Equivalent diameters are convenient to deal with in the model and they help to reduce the computational effort by reducing the number of variables required for some of the calculations. For example, a check that utilises equivalent diameters has been provided in the model to ensure that head losses are not excessive for each link or reach. Also, the optimal solution for each design phase needs to be checked to ensure that path constraints are not violated. This feature has also been incorporated into the program using equivalent diameters.

The optimal solution normally consists of link sections that have at most two segments, their diameters being adjacent on the candidate list (Alperovits and Shamir, 1977), e.g. the parallel pipe of section B-C of Figure 4.1 would be made up of at most two diameters that are adjacent on the candidate list. Considering all the various issues involved in this problem, the possibility of making a mistake e.g. in data entry, exists. It is therefore important to check the optimal solution for hydraulic consistence (e.g. whether minimum service pressures are met at all nodes) using a hydraulic simulation model. However, the numerous optimal link segments can be quite cumbersome and difficult to handle. Thus, for each link, it is convenient to convert these optimal segment diameters into an equivalent diameter to provide an easy means of checking the optimal solution for hydraulic consistence and to reduce on the effort required for data entry into the hydraulic simulation model.

Consider Figure 4.1, for each link, this problem involves finding the equivalent diameter for segments in series for the individual sections (e.g. segments of sections, A-B, B-C and C-D), followed by the equivalent diameter for the link sections themselves (e.g. sections, A-B, B-C and C-D). The procedure used in the Network Design Program is as follows: 1) the equivalent diameter for segments in series for the replaced link section C-D is obtained, 2) the equivalent diameter for segments in series for the parallel link of section B-C is obtained, 3) the equivalent diameter for parallel links (the equivalent parallel link and the existing link that is paralleled) of section B-C is then obtained, 4) the equivalent diameter of link sections in series for the equivalent replaced link section C-D and the equivalent link for the paralleled section B-C is obtained as section B-C-D; and finally, 5) the overall equivalent diameter is obtained for sections in series, the existing link section A-B and the equivalent link section B-C-D.

A – Equivalent Pipe Diameters for Pipes in Series

Consider a link section made up of two optimal link segments, 1 and 2, in series; let Segment 1 have a diameter, Hazen-Williams coefficient and length of D_1, C_1 and L_1, respectively. Let Segment 2 have a diameter, Hazen-Williams coefficient and length of D_2, C_2 and L_2, respectively. Also assume that the equivalent link for Segment 1 and Segment 2 has a diameter, Hazen-Williams coefficient and length of D_{eq}, C_{eq} and L_{eq}, respectively. By continuity, each of these segments in series has the same flow (q_{ijm}). Assume that the length of the link section is L_{eq}, then

$$L_{eq} = L_1 + L_2 \tag{4.41}$$

From Eq. (4.41), the sum of the head losses in each of the segments equals the head loss in the equivalent link for the two segments. Thus from Eq. (4.10),

$$\frac{\alpha L_{eq} q_{ijm}^{1.852}}{C_{eq}^{1.852} D_{eq}^{4.87}} = \frac{\alpha L_1 q_{ijm}^{1.852}}{C_1^{1.852} D_1^{4.87}} + \frac{\alpha L_2 q_{ijm}^{1.852}}{C_2^{1.852} D_2^{4.87}} \tag{4.42}$$

From which the equivalent link diameter is given by

$$D_{eq} = \left[L_{eq} \Big/ \left(C_{eq}^{1.852} \left(\frac{L_1}{C_1^{1.852} D_1^{4.87}} + \frac{L_2}{C_2^{1.852} D_2^{4.87}} \right) \right) \right]^{\left(\frac{1}{4.87}\right)} \tag{4.43}$$

In which equivalent Hazen-William's coefficient is the average of the respective values for the individual segments i.e. $C_{eq} = 0.5(C_1 + C_2)$. From Eq. (4.43), one has the choice of fixing the value of C_{eq} and obtaining D_{eq} or vice versa. Ideally, the value of D_{eq} should lie between D_1 and D_2; and that of C_{eq} should lie between C_1 and C_2. Typical values of D_1, C_1, D_2 and C_2 are 0.08m, 112.7, 0.1m and 116.4 respectively. The values of C_1 and C_2 are those obtained after the adjacent link segments have been subjected to deterioration (Eq. 4.13). In this study, different values have been substituted into Eq. (4.41) to assess the sensitivity of choice of D_{eq} to variations C_{eq} and vice versa. It has been found that the values of C_{eq} are quite sensitive to the choice of D_{eq}, in that a small change in the value of D_{eq} makes a significant change in C_{eq}. It is not easy to select a value of D_{eq} that guarantees C_{eq} to lie between C_1 and C_2. On the other hand, for this model, taking C_{eq} as the average of C_1 and C_2 guarantees that D_{eq} will lie between D_1 and D_2. The significant impact of variation of the value of D_{eq} on C_{eq} might probably be due to the fact that in Eq. (4.43), the exponent for the diameters (4.87) is higher than the exponent for the Hazen-William's coefficients (1.852).

B - Equivalent Pipe Diameter for Parallel Pipes

Consider a link section with two optimal parallel link segments, 1 and 2; let Segment 1 have a diameter, Hazen-Williams coefficient and length of D_{p1}, C_{p1} and L_{p1}, respectively. Let Segment 2 have a diameter, Hazen-Williams coefficient and length of D_{p2}, C_{p2} and L_{p2}, respectively. Also assume that the equivalent link for Segment 1 and Segment 2 has a diameter, Hazen-Williams coefficient and length of D_{peq}, C_{peq} and L_{peq}, respectively. For parallel segments, the head loss in each of the segments is the same and is also equal to that of the equivalent section. The sum of the flow in each segment is equal to the flow in the equivalent link. Thus, from Eq. (4.10),

$$\frac{\alpha L_{peq} Q_{eq}^{\;1.852}}{C_{peq}^{\;1.852} D_{peq}^{\;4.87}} = \frac{\alpha L_{p1} Q_{p1}^{\;1.852}}{C_{p1}^{\;1.852} D_{p1}^{\;4.87}} = \frac{\alpha L_{p2} Q_{p2}^{\;1.852}}{C_{p2}^{\;1.852} D_{p2}^{\;4.87}} \qquad (4.44)$$

assuming $L_{eq} = L_1 = L_2$, justification for this assumption has been given in Sub-section 4.2.5.2, and $C_{eq} = 0.5(C_1 + C_2)$; the equivalent diameter is given by

$$D_{eq} = D_1 \left[\left(\frac{C_1}{C_{eq}} \right)^{1.852} \left(\frac{Q_{eq}}{Q_1} \right)^{1.852} \right]^{\left(\frac{1}{4.87}\right)} = D_2 \left[\left(\frac{C_2}{C_{eq}} \right)^{1.852} \left(\frac{Q_{eq}}{Q_2} \right)^{1.852} \right]^{\left(\frac{1}{4.87}\right)} \qquad (4.45)$$

4.3 COMPUTER PROGRAM FOR MODEL EXECUTION

A computer program coded in FORTRAN 95 has been developed in this study, to execute this model. The program is called UPSIZE and it has the capability of carrying out the maximum entropy flow distribution and the shortest path flow distribution. The program prepares all the linear programming constraint details, together with all the details covered in the upgrading methodology of Section 4.2.5. This model can also be used for rehabilitation purposes. UPSIZE considers both the structural and hydraulic deterioration experienced by pipes. It incorporates indirect and direct pipe failure costs. UPSIZE has a link with the performance assessment model in that it converts the optimal link segments into equivalent links that are required as input data for the performance assessment model. UPSIZE includes paralleling, replacement and pricing policy for upgrading strategies.

The overall upgrading strategy used in this study is a two-phase strategy mainly because designing a network once over the entire design horizon tends to be very expensive considering the large capital outlays involved and the deterioration of the components. In addition, the two-phase strategy provides some flexibility to allow some of the predicted parameters to be adjusted e.g. demand growth rates, interest rates, etc. The two-phase strategy generally involves designing and installing a new network to serve a number of years in Phase I. The second phase follows at the end of Phase I and it involves upgrading the existing network by paralleling and/or replacing pipes to add incremental capacity to the network in order to meet the demand of the entire design horizon. The program therefore designs the network for different complementary design periods as the various sequences of capacity expansions, e.g. assuming a 20-year design horizon, designing Phase I for a demand of 8 years implies designing for an incremental capacity to cover 12 years in the second phase; similarly a design for a demand of 9 years in the first phase implies designing for an incremental capacity to cover 11 years in the second Phase. Each of these sequences of upgrading has a different overall cost (sum of Phase I and Phase II costs). The aim is to find the cheapest sequence.

The Network Design Module comprises of a number of subroutines. The detailed functions of each of these subroutines and how they are interlinked follows shortly. A flowchart illustrating these details is presented in Figure 4.4 and a typical input and output file has also been presented in Appendix B.

4.3.1 Details of the Computer Program

UPSIZE has seven main subroutines namely, MAXENTFLOWS, DATPREP1, PRINTRES1, EZLP, DATPREP2, PRINTRES2 and DYNAP. The functions of each of these subroutines and how they interact together are described below.

For a given design horizon in Phase I, MAXENTFLOWS is used for link flow distribution. This can be based on the maximum entropy flow distribution algorithm or the shortest path flow algorithm described earlier in sub-section 4.2.5.1. The input data required consists of the nodal base demands and link flow directions defined by entering the upstream and down stream nodes for each link. The program then calculates the nodal and link design demands. There is also an option for the user to enter any other flow distribution. The flows are input into subroutine DATPREP1 which is used for preparing all the input data including calculating all the hydraulic gradients or head loss coefficients for the variables just before the linear programming subroutine is invoked in Phase I. Calculations are based on the planning problem 4.2 (Eq. 4.34) with the omission of equations related to paralleling and replacement ($f1_b$ = $f1_c$= 0). All related costs as described in Eqs. (4.3), (4.4), (4.5), (4.6), (4.8) and (4.9) are also included.

The prepared data is input into subroutine EZLP. This is a subroutine for linear programming by Templeman (1989) and it has been incorporated into the program. The main time consuming element for this subroutine is the preparation of data input especially the calculation of hydraulic gradients, G_{ijm} (Eq. 4.12), or the head loss coefficients for all the variables involved. These hydraulic gradients are part of the linearised Equation (4.11), in which the link segment lengths l_{ijm} are the unknown variables. In this thesis, this problem of data preparation has been solved by using DATPREP1, which prepares all this information automatically. The output from EZLP is then fed into PRINTRES1, which tabulates all the Phase I results and prints them to an output file.

In Phase II, MAXENTFLOWS is used for link flow distribution by projecting base demands over the entire design horizon, less the time delays caused by a pricing option if any. These flows together with the network design details from Phase I are input into DATPREP2. This subroutine is for preparing linear programming data in Phase II. Calculations are based on the planning problem 4.2 (Eq. 4.34) including all the equations related to paralleling and replacement. All related costs as described in Eqs. (4.3) to (4.9) are also included. The prepared data is then fed into EZLP for an optimal solution to be obtained by linear programming. The results of the optimisation are input into PRINTRES2, which tabulates all the Phase II results and prints them to an output file.

The linear programming subroutine is used for obtaining least cost designs for various design horizons in Phases I and II. For example for a given design horizon of 20 years, designing for a 9-year demand in Phase I has a cost related to it and designing for the incremental capacity to obtain a system that can meet the 20-year demand in Phase II also has its cost. The total cost for this sequence of capacity expansion is the sum of the Phases I and II costs. DYNAP, the subroutine for dynamic programming, uses Eq. (4.1) to obtain the total costs for various combinations of Phases I and II, in which the design period for Phase I varies from *T1* to *T2*. This subroutine uses Eq. (4.34a) to determine from the different cost combinations of the first phase and the second phase, the sequence with the lowest cost, its timing and magnitude of upgrading. These details of cost timing and magnitude constitute the best long-term upgrading strategy.

A notable feature of the program is that use of the data preparation subroutines, DATPREP1 and DATPREP2, results in major timesavings. This is mainly due to the fact that given very basic data input, the hydraulic gradients (taking deterioration of hydraulic capacity into consideration) for all the link segments together with all the related costs are calculated by these subroutines and the data is fed directly into the linear programming subroutine. For example, loop and path constraints (Eqs. (4.34f), (4.34g), and (4.34h)) are simply defined in the input data by listing the link numbers that form the loops or paths. The detailed calculations to obtain respective hydraulic gradients are done automatically.

4.3.2 Program Input

For the input, the following information is required:

1) The number of nodes, pipes, loop equations, head loss inequalities, and demand patterns and the type of design option e.g. upgrading with or without pricing.
2) Upstream and downstream nodes for each link.

3) Links that form loop head loss equality equations, numbering the pipes in the loop with a convention of the clockwise direction being positive.
4) Links that form the pre-specified paths between a selected source to nodes for each demand pattern, and the allowable head loss for each path.
5) Link lengths and initial Hazen-Williams coefficients for links.
6) An optional provision for user to enter data of an existing network including link lengths and the values of the Hazen-Williams coefficient (in case the network rehabilitation choice is preferred).
7) Base demands at each node, the fire demands, fire flow node(s) and the demand growth rate.
8) Initial pipe roughness and its growth rate.
9) Minimum allowable velocity, maximum allowable velocity and maximum allowable hydraulic gradient.
10) Lower limit for end of Phase I (*T1*), upper limit for end of Phase I (*T2*) and the overall planning horizon (*d*).
11) The discount factor, water tariff increase ratio, price elasticity of demand and link failure cost factors.
12) Construction setting-up and mobilisation costs, compound interest rate, variable cost constant (*VC*) and the variable cost exponent (*VE*).

4.3.3 Program Output

The output from the program consists of two main sections, namely: the results of linear programming for Phase I and II for various upgrading sequences and the results of dynamic programming as detailed below.

4.3.3.1 Linear Programming Results

1) The first section is for Phase I output consisting of tabulated results including the link label, link diameter, link length and the Hazen-William's coefficient for each link segment. This is followed by the link flow velocities, unit head losses for each link, the allowable head loss along each pre-specified path, the number of iterations and the optimum cost for Phase I.
2) The second section is for Phase II output consisting of tabulated results including the link label, link diameter, link length and the Hazen-William's coefficient for

each link segment. These results are tabulated separately for the different link sections (Figure 4.1), i.e. the existing link section A-B, the paralleled section B-C and the replaced section C-D. The link flow velocities, hydraulic gradients or unit head losses for each link, the allowable head loss along each pre-specified path, the number of iterations and the optimum cost for Phase II follow at this point.

These results for Phases I and II are presented in the same format for various combinations or sequences upgrading.

4.3.3.2 Dynamic Programming Results

1) For the different combinations or sequences of upgrading, the costs for Phases I and II are presented. The results of the dynamic programming which include the pipeline cost summary and the overall system costs depending upon the design capacity, are tabulated for the best long-term upgrading strategy. Summaries of typical results are in Tables 4.1 to 4.9.
2) Finally, a statement of the best long-term upgrading strategy, the timing, magnitude and cost of the strategy is presented. Alternatively, for a network rehabilitation problem, the best rehabilitation strategy is presented. The overall CPU time is also given.

4.4 APPLICATION OF THE NETWORK DESIGN MODULE

The Network Design Module has been applied to three networks as possible design options for serving four demand nodes. The problem involves selecting the best design option for optimal long-term upgrading of a water distribution network. The first option is a three-loop layout in Figure 4.5 (Three-loop network A), which is designed with maximum entropy flows combined with a pricing policy. The pricing policy involves increasing the price of water as the system capacity is being approached, resulting in reduced consumption of water and a delay in the timing of expansion or upgrading of the network. The second option is a three-loop layout in Figure 4.6 (Three-loop network B) similar to that of the first option and designed with maximum entropy flows, but without any pricing policy. The third option is the single-loop layout in Figure 4.7 designed using the shortest path flow distribution method for link-flow allocation (Orth, 1986). This network is partially looped and partially dendritic as a compromise design between a highly redundant fully looped network and a branched network with low redundancy. The Network Design Module is used to obtain the optimal pipe diameters for each design period by linear programming, and then the optimal designs over the entire design horizon, timing and magnitude of the upgrading for each of these three options are obtained by dynamic programming.

The overall design horizon selected for this study is 20 years. The nodal base demands or the nodal demands in the first year of the design horizon for Nodes 2, 3, 4 and 5 are 3l/s, 4l/s, 12l/s and 12l/s. The network loading that has been adopted for this study is that of a combination of fire demand and the peak hour demands with a peak hour factor of 2.0 (Directorate of Water Development, 1999). The nodal base demands are assumed to increase at an annual rate (*DGR*) of 4%. Thus, to obtain the design demands for the entire design horizon, for example, the nodal base demands are multiplied by the peak hour factor to obtain peak hour demands. The resulting values are then forecast or projected over a period of 20 years. Finally the fire demand is added to obtain the 20-year nodal design demands for Options 2 and 3 as shown in Figures 4.6 and 4.7.

The design demands for Option 1 are reduced due to enforcement of the pricing policy. For this design option, a tariff increase ratio, (P_t/P_0), has been taken as 1.333 (a 33.33% increase in the water tariff normally implemented in the last one or two years of a given design period). The price elasticity of demand, *PREL*, has been assumed to be –0.3 (Dandy et al. 1985). This combination of tariff increase ratio and price elasticity of demand, yields reduced design demands for Nodes 2, 3, 4, and 5 as shown in Figure 4.5.

The fire demand has been taken as 20l/s (Twort et al., 2000) for the network loading combination (peak hour demands plus fire demands) that has been adopted in this study. It is supposed that this multiple loading combination should stress the system most and give a better representation of the damage tolerance. However, designing for peak demand and the total fire flow is a combination likely to dominate the design of distribution mains, yet this scenario is not likely to occur frequently. Thus, the nodal design demands adopted in this study are a combination of the peak hour design demands and a proportion (25%) of the fire demand (at Node 2) to avoid over-designing the network. The basis of using a proportion of the fire demand when combined with peak demands is detailed in Chapter 2.

All links are 1000m long and the pipes used are made of PVC. The water level at the source is 70m while demand nodes have elevations of 0m and minimum service heads of 15m. Hazen-Williams coefficient is 130 for all new pipes. Compound interest rate, b = 8, design horizon d = 20 years, v for Phase II varies from 7 to 13 years i.e. $T1$=7 and $T2$=13. All costs in this study are in US dollars. $FCF(LU_i)_{ij}$, the failure cost factors for land use are taken as 4 each of the links. r, the discount rate = 8%, e_{0ijm}, the initial roughness = 0.0021mm and a_{ijm}, the roughness growth rate = 0.025 (mm/year) (Bhave, 1991). Pipe cost constants for Eqs. (4.4), (4.6) and (4.8) are taken as γ_p = 32.093; c_p = c_r= 3.7; γ_r = 33.928 and γ_{br} = 108.87. The pipe cost exponent Φ = 0.6067.

The costs of parallel pipes to cover supply and installation; together with replacement costs that involve uninstalling existing pipes and installing new ones, have been obtained in this study as detailed in Sub-section 4.2.1.1.

In order to limit the segment diameters, the minimum and maximum allowable velocities that have been used in this study are 0.5m/s and 3m/s, respectively (Dandy and Engelhardt, 2001). A maximum hydraulic gradient of 50m/km (Twort et al., 2000) has been used. Setting-up costs at the beginning of each design period or phase = $100,000. VC, the cost coefficient = 130 and VE, the cost exponent = 1.6. These coefficients have been obtained as detailed in 4.2.1.1.

The Network Design module has been used to obtain the best upgrading strategy and the timing of the upgrading for the three design options. The detailed results of Option 2 are presented in Section I of Appendix B, as an example of typical results from the program. The results for the three design options are summarised and shown in Tables 4.1 to 4.6 for the first and second phases of the three design options. The overall costs for the three designs are also shown in Tables 4.7, 4.8 and 4.9.

4.5 RESULTS AND DISCUSSION

4.5.1 Results

The pricing policy for the first option results in a two-year delay of expansion. From Table 4.7, the cheapest cost strategy for the first option has a value of $3,810,851.25 and it is to design and install a capacity for a 9-year demand in Phase I; delay expansion or upgrading for 2 years and then upgrade the network by paralleling and/or replacing to a 16-year demand (20 years less the delays due to pricing of 2 years in each design phase), to serve the entire 20-year design horizon. Note that the 9-year demand, is obtained by taking the base demand or the demand of the first year of the 20-year design horizon and projecting or forecasting it over a period of 9 years. Similarly, the 16-year demand is obtained by projecting the demand of the first year of the 20-year design horizon over a period of 16 years.

From Table 4.8, the cheapest cost strategy for the second design option has a value of $4,214,803 and it is to design and install a capacity for a 14-year demand in Phase I; and then upgrade the network by paralleling and/or replacement to the ultimate or 20-year demand capacity in Phase II. Table 4.9 shows that cheapest cost strategy for the third design option (single loop network) has a value of $3,529,782.75 and it is to design and install a capacity for an 11-year demand in Phase I; and then upgrade the network by paralleling and/or replacement to the ultimate or 20-year demand capacity in Phase II.

Table 4.10 gives a summary of results for the optimal design options. The optimal design for Option 1 requires 20 variables in Phase I and 46 variables in Phase II. The overall CPU-time required for execution of the Network Design Module for this option is 0.391 seconds on a PC with 128 MB RAM, and 1.2 MHz microprocessor speed. Option 2 requires 23 variables in Phase I and 48 variables in Phase II. The overall CPU-time required for execution of the Network Design Module for this option is 0.488 seconds. Option 3 requires 20 variables in Phase I and 40 variables in Phase II. The overall CPU-time required for execution of the Network Design Module for this option is 0.289 seconds. The overall CPU-time referred to covers the different linear programming designs for Phase I and Phase II design periods of the various sequences of upgrading as shown in Tables 4.7, 4.8 and 4.9. The time also covers the dynamic programming to determine the timing and magnitude of the upgrading for each design option.

4.5.2 Discussion

In this thesis, energy and pump related costs are not considered mainly because they require several extended period simulations, which are bound to greatly increase the computational requirements of the problem. This omission may be justified in a practical engineering context. Kleiner et al. (1998) have noted that the typical operating range of pressure heads is narrow, ranging from 30m to 70m of the water column. This is further reduced by the variations in the topology of the terrain on which the networks lie. Thus, the possibility of increasing the pressure in the network using boosters is limited. Also booster pumps are more likely to be required in the final years of a particular design period (when the network's hydraulic performance is reduced due to aging and pressures are reduced) despite having been installed at the beginning of the design horizon. Thus, the overall capacity utility of these booster pumps over the entire design horizon is probably low and not as economical.

The results of the three design options in Tables 4.1 to 4.6 generally show that paralleling is preferred to replacement probably because it is cheaper and that the rate of deterioration of the pipes is quite low. A higher rate of deterioration might perhaps have required a more balanced use of both paralleling and replacement to meet the velocity and maximum hydraulic gradient requirements. Another reason that might influence a more balanced use of both paralleling and replacement might probably be the state of the links to be paralleled in terms of hydraulic and structural integrity. If these existing links deteriorate to a level where violation of the allowable hydraulic gradient and velocity is inevitable, there is a more likely chance for the requirement of both replacement and paralleling of these links.

The detailed costs for the three hypothetical network designs shown in Tables 4.7, 4.8 and 4.9 show an increasing trend in costs for Phase I and a decreasing trend in costs for Phase II, as the Phase I design period increases. This is generally due to the fact the as time for the Phase I design period increases, the design demand increases too and thus a larger capacity has to be designed for. A larger capacity implies an increase in installed capacity costs. On the other hand, the decreasing trend in Phase II costs as the Phase I design period increases, is due to the fact that the incremental capacity to be designed for in Phase II decreases as the Phase I design period increases. The higher the capacity designed for in Phase I, the lower the incremental capacity one would have to design for in Phase II to meet the ultimate demand over the 20-year design horizon.

The optimal design costs for Options 1, 2, and 3 are \$3,810,851.25, \$4,214,803 and \$3,529,782.75. The single loop network has the lowest cost probably due to the fact that it has fewer links than the other two options. Option 1 is cheaper than Option 2 mainly due to enforcement of the pricing policy. The increase in the price of water combined with the price elasticity of demand, leads to reduced consumption (Twort et al., 2000) and a delay in the need to upgrade the network. It also means that overall total demand to be designed for is lower as seen in Figures 4.5 and 4.6; where the total demands are 121.2 l/s and 140.9l/s for the network design with the pricing policy enforced and the one without respectively.

From the summary of results in Table 4.10, the values of overall CPU time for execution of Network Design Module on Options 1, 2 and 3, which are 0.391, 0.488 and 0.289 seconds respectively, are quite low considering that they cover different linear programming designs for various Phase I and II design periods, together with dynamic programming to determine the timing and magnitude of the upgrading for each design option. This is a good reflection of the efficiency of the Network Design Module. Design Option 3 has the lowest overall CPU time probably because it has only one loop and five links compared to Options 2 and 3 which have three loops and seven links each. Thus Option 3 has the lowest number of constraints and decision variables, 20 and 40 variables in the respective Phases I and II; compared to 20 and 46 variables in the respective Phases I and II for Option 1; and to 23 and 48 variables in Phase I and II, respectively, for Option 3. Option 2 has the highest CPU time probably because it has the largest number of decision variables.

4.6 SENSITIVITY ANALYSIS AND DISCUSSION

To assess the effect of variation of different parameters on the upgrading strategies for individual options, a sensitivity analysis involving various scenarios has been carried out. The results of these scenarios are in Tables 4.11 to 4.19.

4.6.1 Sensitivity Analysis

Scenario A of the sensitivity analysis is to check the effect of variation of the price elasticity of demand on the timing and magnitude of upgrading with respect to Option 1. This scenario involves maintaining the annual demand growth rate (*DGR*) of 4% up to the overall design horizon demands shown in Figure 4.8. For this design option, the tariff increase ratio, (P_t/P_0), is maintained as 1.333 (a 33.33% increase in the water tariff). The price elasticity of demand, *PREL*, has been taken as – 0.4. This results in a three-year delay to expansion in each design phase and a reduction of the 20-year total demand of 140.9 l/s (as in Figs. 4.6 and 4.7) to a total demand of 112.5 l/s as shown in Figure 4.8 (Three-loop network C). The optimal network design details are shown in Table 4.11 for Phase I and Table 4.12 for Phase II. The costs for Scenario A are shown in Table 4.13. The cheapest cost strategy for this scenario has a value of $3,674,676.50. This strategy is to design and install a capacity for a 7-year demand in phase one; delay expansion or upgrading for 3 years and then upgrade the network to a 14-year demand capacity (20 years less the delays due to pricing of 3 years in each design phase), to serve the entire 20-year design horizon. The summary of the results for the optimal design options are presented in Table 4.14.

Table 4.15 presents sensitivity analysis Scenario B to check the impact of varying compound interest rate, *b*%, at which borrowed capital is raised. This rate has been changed from 8% to 6% in this scenario, for the three-loop network B of Figure 4.6. This option has been designed with maximum entropy flows but without the pricing policy. The design details of the optimal design are exactly the same as those earlier obtained in Tables 4.2 and 4.5. The costs for this scenario are presented in Table 4.15. The cheapest cost strategy for the second design option has a value of \$3,033,587. It entails designing and installing a capacity for a 14-year demand in phase one; and then upgrading the network by paralleling and/or replacement of pipes to the ultimate or 20-year design demand in phase two.

Sensitivity analysis Scenario C is aimed at assessing the impact of varying the failure cost factors for land use, $FCF(LU_{ij})$, that were assigned to each link. The network that has been selected for this assessment is the three-loop network B of Figure 4.6, for which each of its links had been originally assigned a failure cost factor of 4 using Cost Case I in Table 4.16. This Cost Case I assumes the same weight of indirect costs for different overlying land uses like residential, industrial, commercial activities and major roads (results are in Tables 4.2, 4.5 and 4.8). For Scenario C, the failure cost factors have been set according to Cost Case II in Table 4.16. For each of the links 1-2, 1-3, 1-4, 2-3 and 2-5 the failure cost factor has been set to 1.5 with the assumption that the overlying land use for these links is residential, industrial, or both. Links 3-4 and 3-5 have each been assigned a failure cost factor of 3 with the assumption that the overlying land use is for commercial activities and major roads. The results of optimal design results of Scenario C are in Tables 4.17 and 4.18. Table 4.19 shows the costs for Scenario C. The cheapest cost for this strategy is \$4,177,958.25. The strategy for this scenario is to design and install a capacity for a 13-year demand in Phase I; and then upgrade the network by paralleling and/or replacing pipes to the 20-year demand capacity in Phase II.

4.6.2 Discussion

Scenario D shows that combining a water tariff increase ratio of 1.333 with a reduction in the price elasticity of demand from –0.3 to –0.4 for the three-loop network by enforcing a pricing policy, culminates in reduced optimal design costs and a new optimal design (Tables 4.11 and 4.12). This is probably because the overall total design demand decreases from 140.9 l/s (as in Figs. 4.6 and 4.7) to a total demand of 112.5 l/s as shown in Figure 4.8 (Three-loop network C). The other reason is due to the fact that the timing of upgrading changes from designing for a 9-year demand in Phase I (Table 4.7) to designing for a 7-year demand for Scenario A, which requires a lower capital outlay. The design, cost, timing and magnitude of upgrading are sensitive to the price elasticity of demand. Generally the lower the price elasticity of demand (for a given increase in water tariffs), the lower the overall cost of upgrading.

From Table 4.14 showing the summary of results of Scenario A, the optimal design of Option 1 requires 18 variables in Phase I and 44 in Phase II. The CPU time is 0.344 seconds compared to that of 0.391 seconds that was obtained for Option 1 in Table 4.10. This reduction in CPU time might be due to the fact that the overall number of variables has reduced in this scenario, from the values of 20 and 46 in Phase I and II (Table 4.10).

From Table 4.15 it can be concluded that a lower compound interest rate at which borrowed capital must be paid back, implies a cheaper optimal design. The timing of upgrading and the actual network design is not affected by this compound interest rate, *b*, probably due to the fact that it is not included directly in the optimisation equations, and that it is rather introduced in the overall capital cost equation (Eq. 4.1).

Scenario F of the sensitivity analysis which involves applying failure cost factors of Cost Case II (Table 4.16) to the three-loop network B without pricing, gives optimal design results in Tables 4.17 and 4.18 and costs in Table 4.19. The optimum cost of this strategy is $4,177,958.25 which is lower than the optimum cost for the strategy obtained earlier (Table 4.8) using Cost Case I ($4,214,803). The reduced costs for Scenario F are partly due to the fact that the failure cost factors in Cost Case II are lower than those in Cost Case I. Secondly the fact that the timing of upgrading changes from designing for a 14-year demand in phase one (Table 4.8) to designing for a 13-year demand in phase one (Table 4.19) means a lower capital outlay in the first phase for Scenario F and thus lower overall costs. Indirect failure costs are sensitive to the timing, magnitude and cost of upgrading. This inevitably means that due consideration is required when specifying these indirect failure or repair costs.

4.7 SUMMARY AND CONCLUSIONS

The large capital resources required for upgrading of a water distribution network dictate the need for careful planning and great diligence at the onset to determine the phasing, timing and magnitude of upgrading.

A comprehensive long-term upgrading model called the Network Design Module has been presented in this chapter. The model is used for obtaining cost-effective optimal long-term upgrading strategies that can satisfy hydraulic, network economics and water quality requirements. The model explicitly considers deterioration over time of both the structural integrity and hydraulic capacity of every pipe and it allows for the direct and indirect failure costs. It simultaneously considers the upgrading options of paralleling and replacement of pipes; and can determine exactly which pipes in the network to be upgraded. Distribution network economics and hydraulic performance for various design periods are done using linear programming. The timing and magnitude of upgrading over the planning horizon is based on dynamic programming. The model can also be used for rehabilitation strategies and it has the capability of considering joint pricing and network upgrading or capacity expansion policies.

Considering the issues involved in this planning problem makes it quite complex with numerous variables involved. Notable features of the Network Design Module are in its ability to reduce the dimensionality of the problem by using an entropy-based algorithm for feasible flow distribution. This is coupled with an algorithm for limiting the number of link segment diameters on the candidate list thus reducing the number of variables and the computational effort. The input data required has a simple format and the process of data entry is not time-consuming. Given these modifications, the model is very efficient as reflected by the CPU time requirements. Typical overall CPU time on a PC with 128 MB RAM, and 1.2 MHz microprocessor speed, is about 0.5 seconds for a small network of about seven links. This CPU time covers the different linear programming analyses for Phase I and Phase II design periods of the various sequences of upgrading, and the dynamic programming to determine the timing and magnitude of the upgrading for each design option. Larger networks are bound to require more execution time since they inevitably involve more constraints and variables. However, the CPU time for larger networks with about 25 links is relatively low and in the region of 5 seconds as detailed later in Chapter 8.

The Network Design Module has been applied to three design options to demonstrate its capabilities. For each of these examples, two-phase optimal upgrading strategies have been obtained. The advantage of the two-phase strategy is that it is cheaper than designing a network once over the entire design horizon, which involves large capital outlays and excessive deterioration of the components. Secondly, uncertainty is catered for with the flexibility that allows one to adjust some of the predicted parameters at the end of Phase I, e.g. demand growth rates and interest rates.

The importance of using joint pricing and capacity expansion policies has also been demonstrated. Enforcement of a pricing policy, which involves an increase in the water tariff combined with the price elasticity of demand, leads to a reduction in demand and a delay in the need to expand or upgrade the network. Consequently, a reduction in the overall costs associated with such a strategy is attained. The sensitivity of the design, cost, timing and magnitude of upgrading to the price elasticity of demand is significant. Generally, the lower the price elasticity of demand (for a given increase in water tariffs), the lower the overall cost of upgrading. As a first step towards achieving efficient policies, water distribution network managers and planners should recognise that water pricing can be used as a soft alternative for network upgrading.

Sensitivity analysis shows that for a particular design option, the lower the compound interest rate at which borrowed capital must be paid back, the cheaper the optimal design. However, the timing of upgrading and the actual network design is not affected. The fact that indirect failure costs are sensitive to the timing, magnitude and cost of upgrading has been demonstrated. The higher the failure cost factors, the higher the overall costs. This certainly means that due attention is required when specifying indirect failure costs through the failure cost factors.

In the next chapter the hydraulic simulation module of the Integrated Model shall be presented. This module is based on a new method for head-driven analysis of water distribution networks.

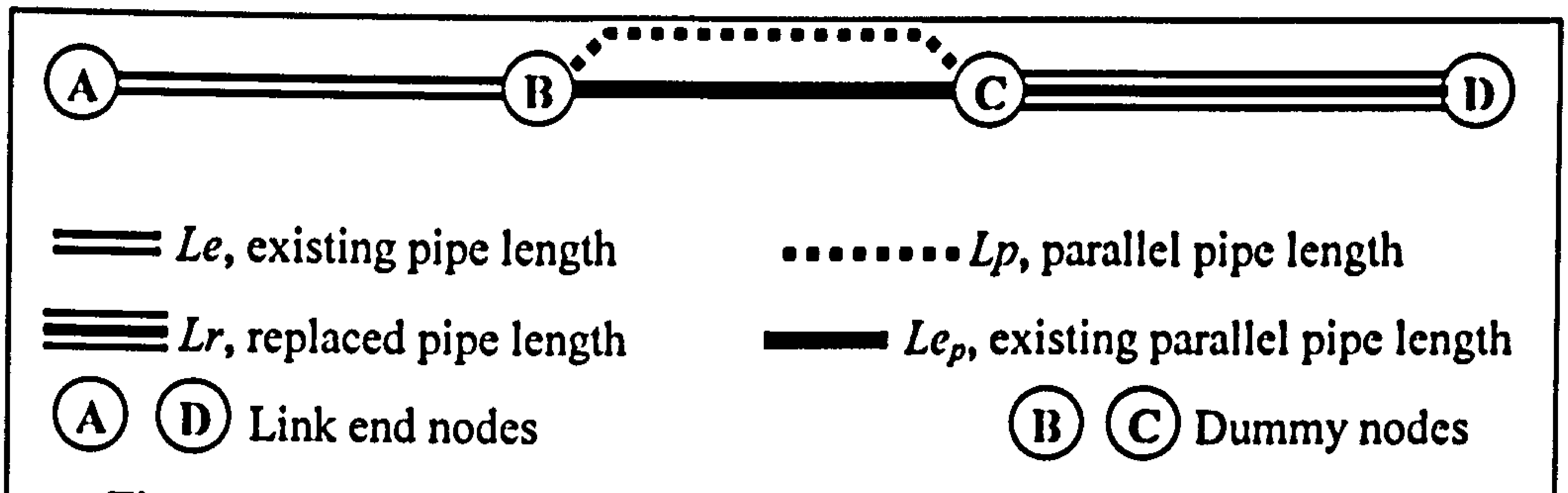

Figure 4.1 Typical Link Representation for the Second Design Period

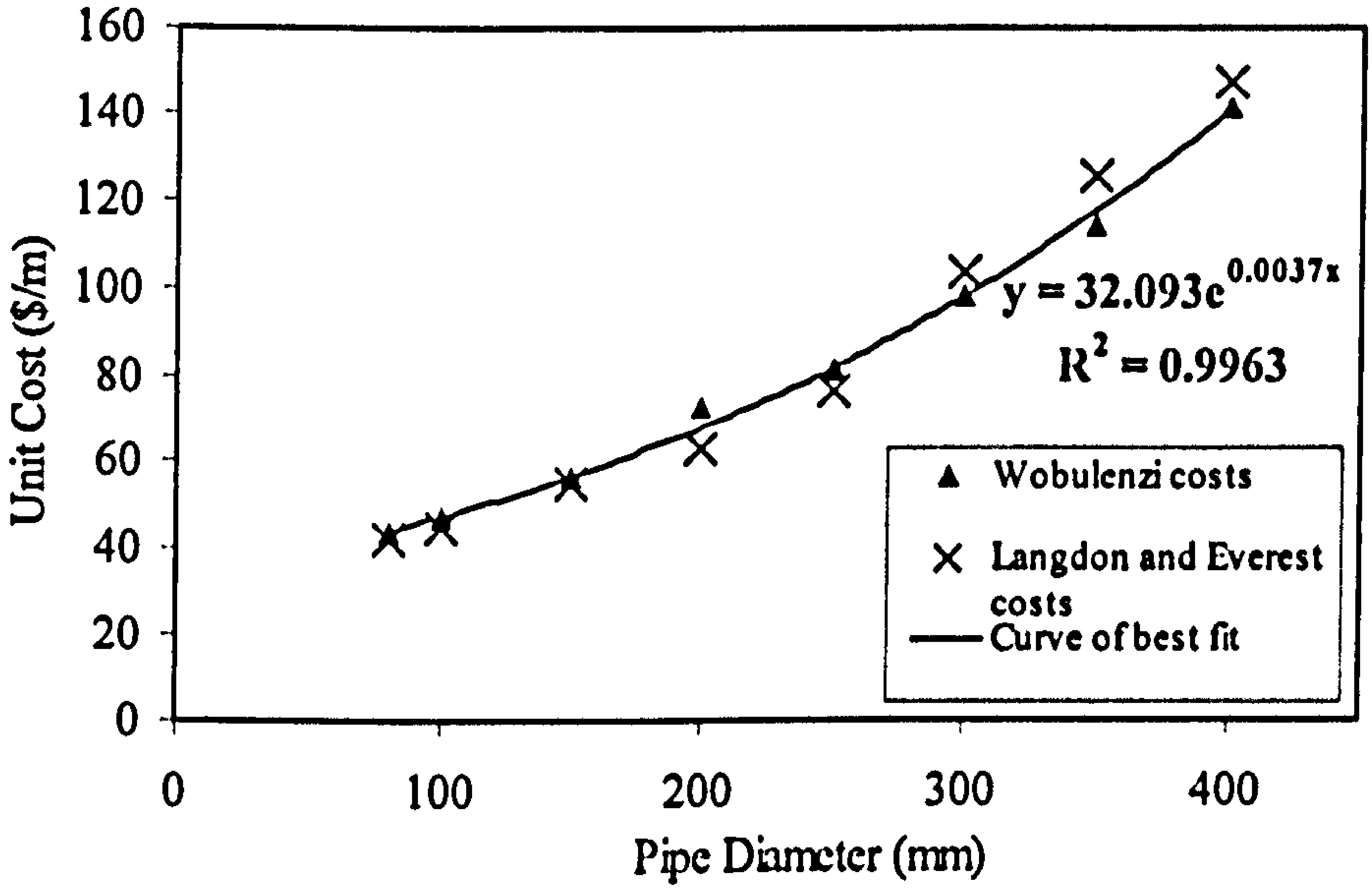

Figure 4.2 Unit Costs for New and Parallel Pipes

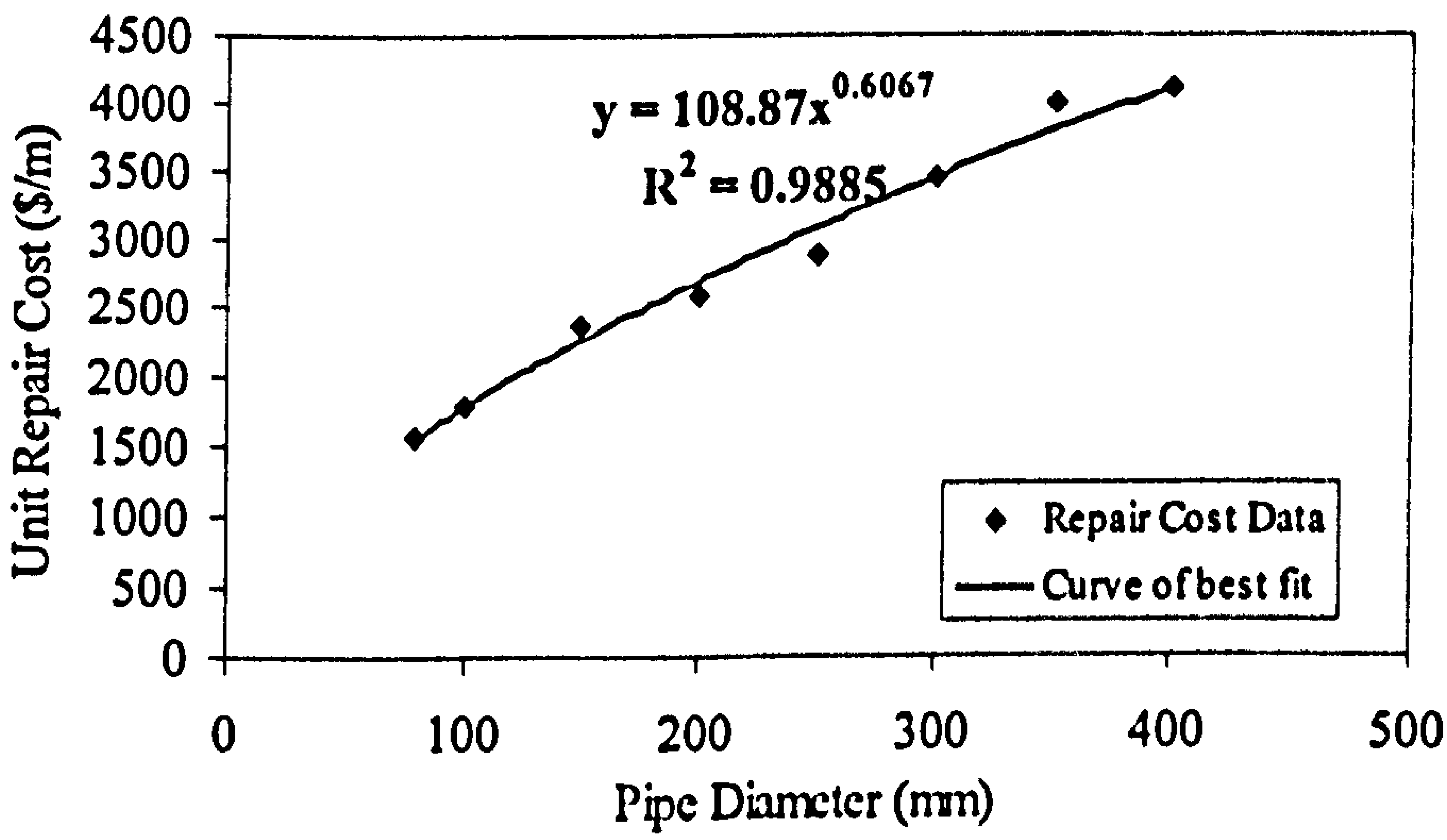

Figure 4.3 Unit Repair Costs for Pipes

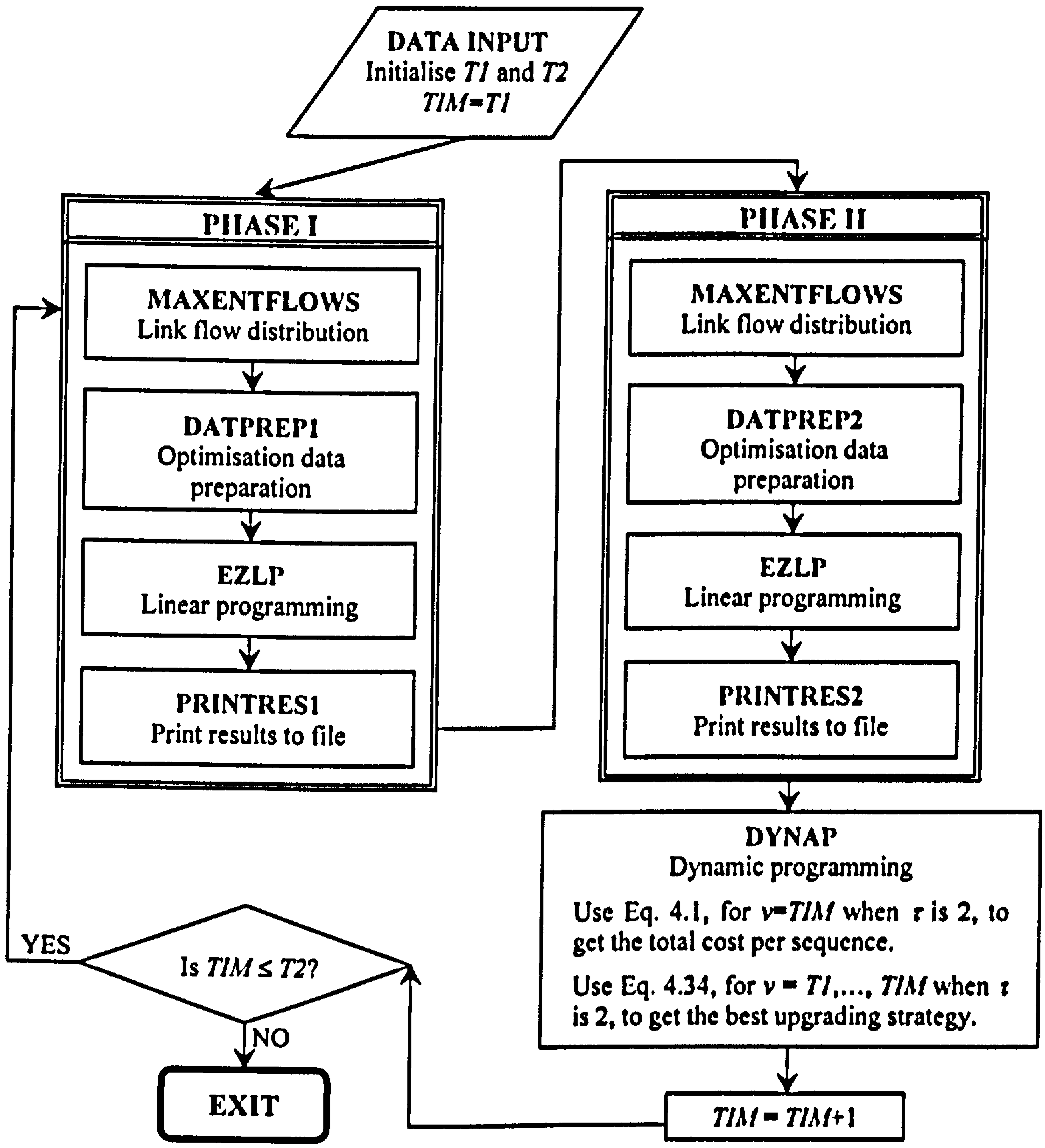

Figure 4.4 Flow Diagram for the Network Design Module

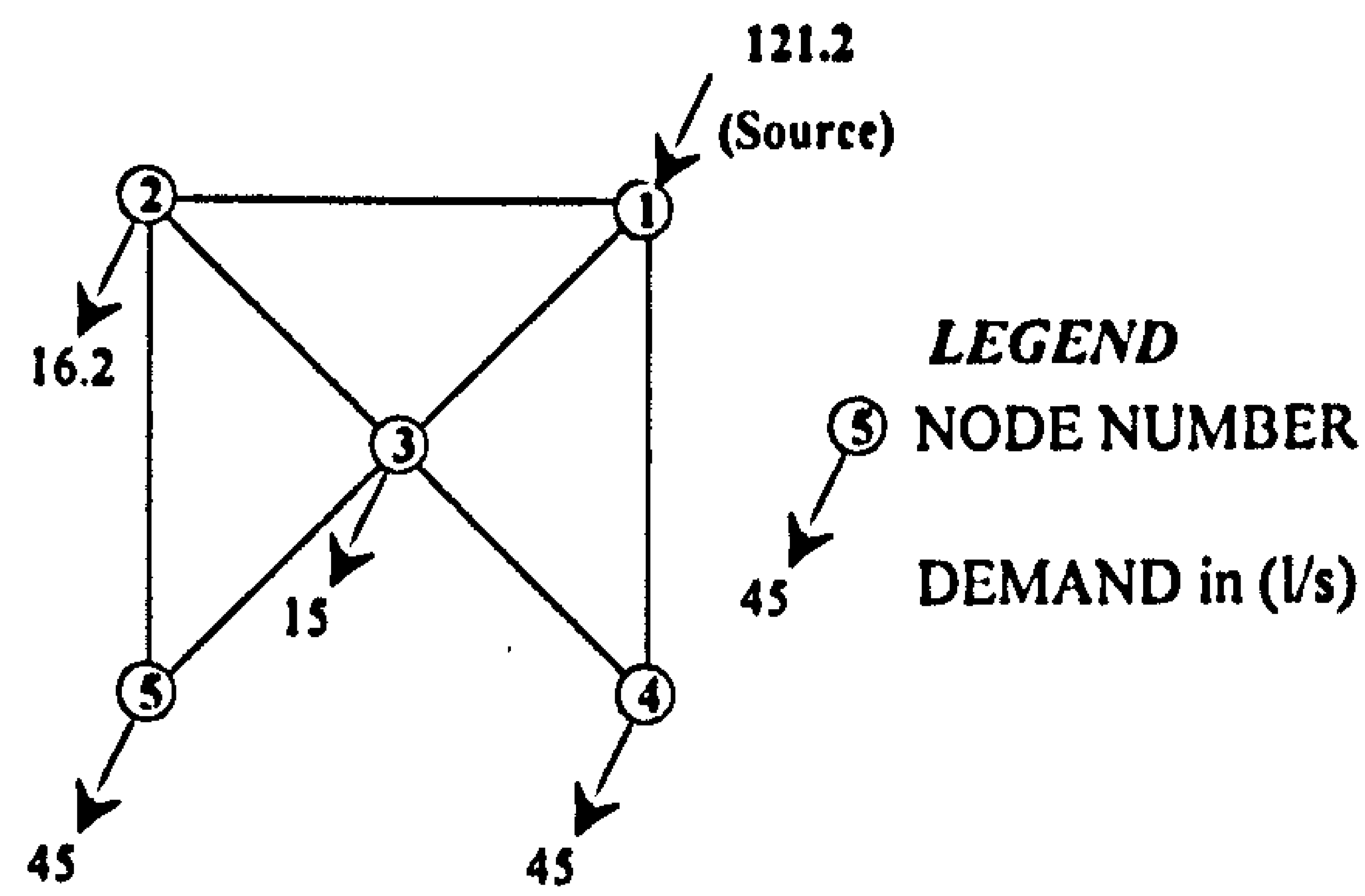

Figure 4.5 Three-loop Network A

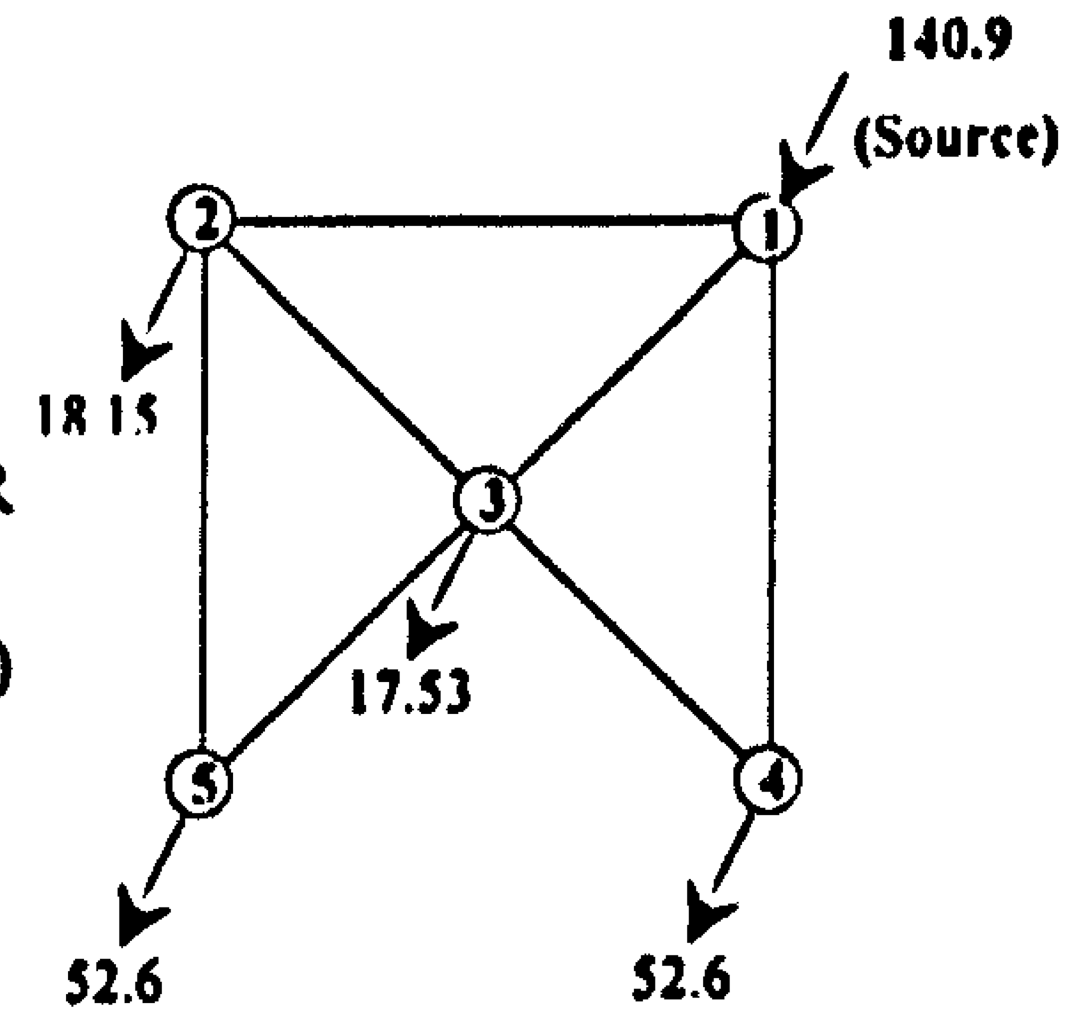

Figure 4.6 Three-loop Network B

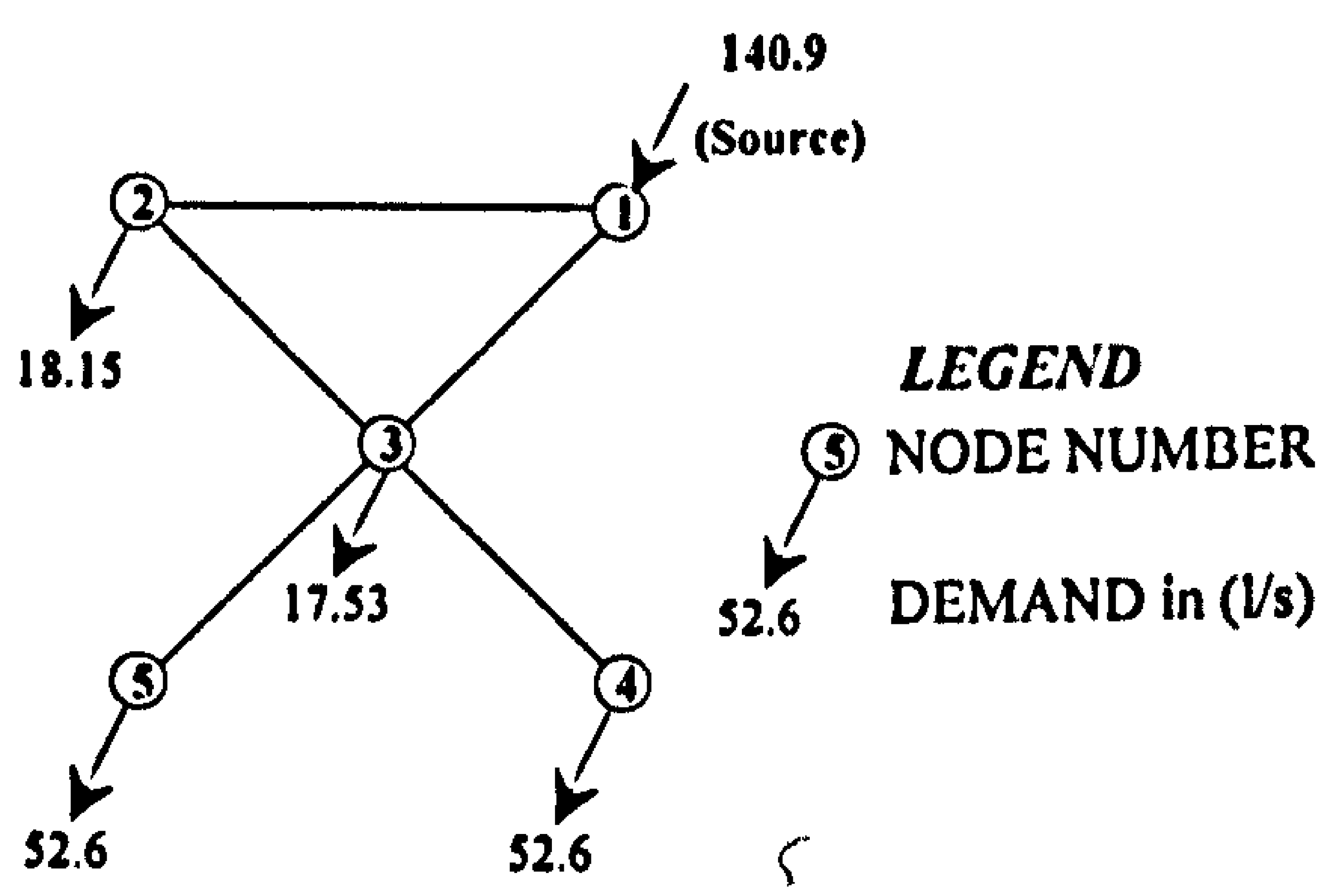

Figure 4.7 Single-loop Network

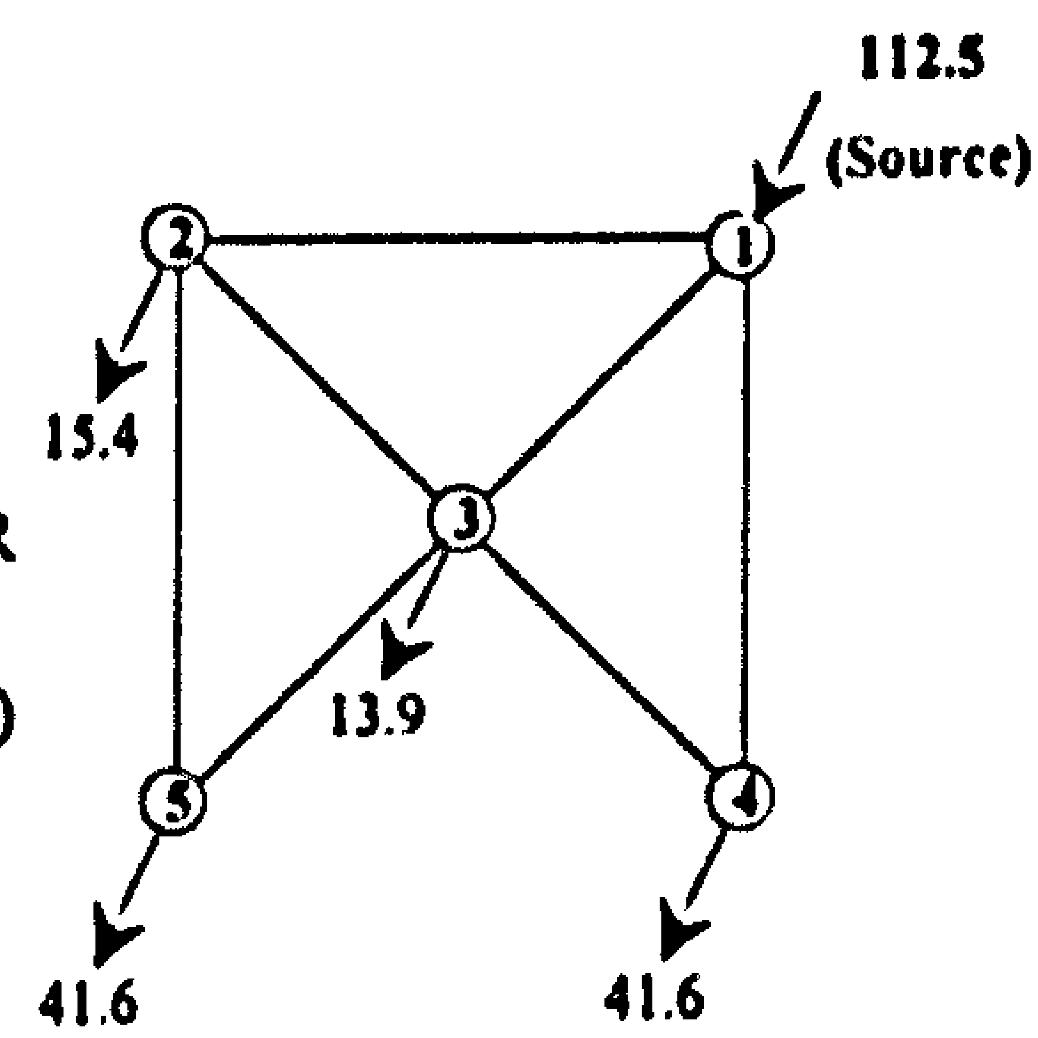

Figure 4.8 Three-loop Network C

Table 4.1 Three-loop Design with Pricing - Phase I

	SEGMENT ONE		SEGMENT TWO		EQUIVALENT DIAMETER LINKS		
LINK	DIAMETER (m)	LENGTH (m)	DIAMETER (m)	LENGTH (m)	DIAMETER (m)	LENGTH (m)	CHW
1 - 2	0.200	1000.00	-	-	0.200	1000	124.3
1 - 3	0.150	1000.00	-	-	0.150	1000	119.7
1 - 4	0.100	1000.00	-	-	0.100	1000	113.1
2 - 3	0.150	44.91	0.200	955.09	0.196	1000	122
2 - 5	0.100	579.19	0.150	420.81	0.109	1000	116.4
3 - 4	0.150	614.55	0.200	385.45	0.160	1000	122
3 - 5	0.150	1000.00	-	-	0.150	1000	119.7

Table 4.2 Three-loop Design without Pricing - Phase I

	SEGMENT ONE		SEGMENT TWO		EQUIVALENT DIAMETER LINKS		
LINK	DIAMETER (m)	LENGTH (m)	DIAMETER (m)	LENGTH (m)	DIAMETER (m)	LENGTH (m)	CHW
1 - 2	0.200	1000.00	-	-	0.200	1000	120.5
1 - 3	0.150	1000.00	-	-	0.150	1000	115.8
1 - 4	0.100	809.01	0.150	190.99	0.103	1000	112.5
2 - 3	0.150	76.97	0.200	923.03	0.192	1000	118.1
2 - 5	0.100	595.63	0.150	404.37	0.108	1000	112.5
3 - 4	0.150	143.12	0.200	856.88	0.186	1000	118.1
3 - 5	0.150	1000.00	-	-	0.150	1000	115.8

Table 4.3 Single-loop Design without Pricing - Phase I

	SEGMENT ONE		SEGMENT TWO		EQUIVALENT DIAMETER LINKS		
LINK	DIAMETER (m)	LENGTH (m)	DIAMETER (m)	LENGTH (m)	DIAMETER (m)	LENGTH (m)	CHW
1 - 2	0.200	1000.00	-	-	0.200	1000	124.3
1 - 3	0.200	1000.00	-	-	0.200	1000	124.3
2 - 3	0.150	150.81	0.200	849.19	0.185	1000	122
3 - 4	0.150	712.23	0.200	287.77	0.157	1000	122
3 - 5	0.150	1000.00	-	-	0.150	1000	119.7

Table 4.4 Three-loop Design with Pricing - Phase II

		(A-B): EXISTING UNPARALLELED LINK			(B-C): PARALLEL LINK			(C-D): REPLACED LINK		
LINK	LINK FLOW (l/s)	DIAM. (m)	LENGTH (m)	CHW	DIAM. (m)	LENGTH (m)	CHW	DIAM. (m)	LENGTH (m)	CHW
1 - 2	68.68	0.200	501.42	114.7	0.250	498.58	130.0	-	-	-
1 - 3	37.46	0.150	626.55	110.1	0.200	373.45	127.6	-	-	-
1 - 4	14.98	-	-	-	0.080	760.72	112.7	-	-	-
	14.98				0.100	239.28	116.4			
2 - 3	37.46	0.196	1000.00	114.4	-	-	-	-	-	-
2 - 5	14.98	0.109	944.25	104.9	-	-	-	0.150	55.75	122.9
3 - 4	29.97	0.160	580.85	111.1	0.200	419.15	127.6	-	-	-
3 - 5	29.97	0.150	1000.00	110.1	-	-	-	-	-	-

Table 4.5 Three-loop Design without Pricing - Phase II

LINK	LINK FLOW (l/s)	(A-B): EXISTING UNPARALLELED LINK			(B-C): PARALLEL LINK			(C-D): REPLACED LINK		
		DIAM. (m)	LENGTH (m)	CHW	DIAM. (m)	LENGTH (m)	CHW	DIAM. (m)	LENGTH (m)	CHW
1 - 2	79.50	0.200	93.37	114.7	0.250	906.63	130.0	-	-	-
1 - 3	43.82	-	-	-	0.150	861.88	129.4	0.150	138.12	129.4
1 - 4	17.53	-	-	-	0.100	1000.00	122.8	-	-	-
2 - 3	43.82	0.192	1000.00	114.1	-	-	-	-	-	-
2 - 5	17.53	-	-	-	0.100	1000.00	122.8	-	-	-
3 - 4	35.06	0.186	1000.00	113.5	-	-	-	-	-	-
3 - 5	35.06	0.150	468.65	110.1	0.200	531.35	130.0	-	-	-

Table 4.6 Single-loop Design without Pricing - Phase II

LINK	LINK FLOW (l/s)	(A-B): EXISTING UNPARALLELED LINK			(B-C): PARALLEL LINK			(C-D): REPLACED LINK		
		DIAM. (m)	LENGTH (m)	CHW	DIAM. (m)	LENGTH (m)	CHW	DIAM. (m)	LENGTH (m)	CHW
1 - 2	59.05	0.200	1000.00	114.7	-	-	-	-	-	-
1 - 3	81.80	0.200	948.65	114.7	0.250	51.35	130.0	-	-	-
2 - 3	40.90	0.185	1000.00	113.5	-	-	-	-	-	-
3 - 4	52.59	-	-	-	0.200	513.93	127.6	-	-	-
	52.59				0.250	486.07	130.0			
3 - 5	52.59	-	-	-	0.150	189.98	122.9	-	-	-
	52.59				0.200	810.02	127.6			

Table 4.7 Costs for the Three-loop Design with Pricing

PHASE I TIME (Yrs)	PHASE I COSTS ($)	PHASE II COSTS ($)	OVERALL COSTS ($)
7	3,120,086.75	708,928.00	3,829,014.75
8	3,169,715.50	645,930.00	3,815,645.50
9	3,222,733.00	588,118.31	3,810,851.25
10	3,279,293.75	535,002.06	3,814,295.75
11	3,339,562.00	486,315.16	3,825,877.25
12	3,411,014.25	431,524.91	3,842,539.25
13	3,495,779.50	357,232.38	3,853,011.75

Table 4.8 Costs for the Three-loop Design without Pricing

PHASE I TIME (Yrs)	PHASE I COSTS ($)	PHASE II COSTS ($)	OVERALL COSTS ($)
9	3,214,985.75	1,099,575.38	4,314,561.00
10	3,270,748.25	1,006,002.38	4,276,750.50
11	3,330,287.75	914,353.13	4,244,641.00
12	3,393,784.75	836,443.44	4,230,228.00
13	3,478,529.25	737,166.81	4,215,696.00
14	3,566,436.75	648,366.06	4,214,803.00
15	4,172,562.25	200,822.39	4,373,384.50

Table 4.9 Costs for the Single-loop Design without Pricing

PHASE I TIME (Yrs)	PHASE I COSTS ($)	PHASE II COSTS ($)	OVERALL COSTS ($)
8	2,734,724.50	835,699.75	3,570,424.25
9	2,786,066.25	756,062.88	3,542,129.25
10	2,847,015.25	682,930.31	3,529,945.50
11	2,914,191.00	615,591.88	3,529,782.75
12	2,989,242.50	553,982.13	3,543,224.50
13	3,070,870.75	497,955.81	3,568,826.50
14	3,155,013.25	447,086.56	3,602,099.75

Table 4.10 Summary of Results for the Optimal Design Options

PARAMETER	OPTION 1	OPTION 2	OPTION 3
VARIABLES IN PHASE I	20	23	20
VARIABLES IN PHASE II	46	48	40
CPU-TIME (secs)	0.391	0.488	0.289
PRESENT VALUE OF COSTS ($)	3,810,851.25	4,214,803.00	3,529,782.75

Table 4.11 Sensitivity Analysis: Three-loop Design with Pricing - Phase I (Scenario A)

	SEGMENT ONE		SEGMENT TWO		EQUIVALENT DIAMETER LINKS		
LINK	DIAMETER (m)	LENGTH (m)	DIAMETER (m)	LENGTH (m)	DIAMETER (m)	LENGTH (m)	CHW
1 - 2	0.200	1000.00	-	-	0.200	1000	125.9
1 - 3	0.150	1000.00	-	-	0.150	1000	121.2
1 - 4	0.100	1000.00	-	-	0.100	1000	114.7
2 - 3	0.150	30.25	0.200	969.75	0.197	1000	123.5
2 - 5	0.100	571.42	0.150	428.58	0.109	1000	117.9
3 - 4	0.150	610.56	0.200	389.44	0.160	1000	123.5
3 - 5	0.150	1000.00	-	-	0.150	1000	121.2

Table 4.12 Sensitivity Analysis: Three-loop Design with Pricing - Phase II (Scenario A)

	TOTAL LINK	(A-B): EXISTING UNPARALLELED LINK			(B-C): PARALLEL LINK			(C-D): REPLACED LINK		
LINK	FLOW (l/s)	DIAM. (m)	LENGTH (m)	CHW	DIAM. (m)	LENGTH (m)	CHW	DIAM. (m)	LENGTH (m)	CHW
1 - 2	63.88	0.200	862.33	114.7	0.250	137.67	129.5	-	-	-
1 - 3	34.63	0.150	856.50	110.1	0.200	143.50	125.9	-	-	-
1 - 4	13.85	-	-	-	0.080	931.34	111.1	-	-	-
					0.100	68.66	114.7			
2 - 3	34.63	0.197	1000.00	114.5	-	-	-	-	-	-
2 - 5	13.85	0.109	943.37	104.9	-	-	-	-	-	-
3 - 4	27.71	0.160	445.62	111.2	0.200	554.38	125.9	0.150	56.63	121.2
3 - 5	27.71	0.150	1000.00	110.1	-	-	-	-	-	-

Table 4.13 Sensitivity Analysis: Costs for the Three-loop Design with Pricing (Scenario A)

PHASE I TIME (Yrs)	PHASE I COSTS ($)	PHASE II COSTS ($)	OVERALL COSTS ($)
5	3,033,161.50	661,983.44	3,695,145.00
6	3,076,915.50	604,235.81	3,681,151.25
7	3,123,748.25	550,928.56	3,674,676.50
8	3,173,793.25	504,350.34	3,678,143.50
9	3,227,193.25	458,679.06	3,685,872.25
10	3,284,100.00	417,513.34	3,701,613.25
11	3,344,679.00	385,185.34	3,729,864.25

Table 4.14 Sensitivity Analysis: Summary of Results for the Optimal Design Options (Scenario A)

PARAMETER	OPTION 1	OPTION 2	OPTION 3
VARIABLES IN PHASE I	18	23	20
VARIABLES IN PHASE II	44	48	40
CPU - TIME	0.344	0.488	0.289
PRESENT VALUE OF COSTS ($)	3,674,676.50	4,214,803.00	3,529,782.75

Table 4.15 Sensitivity Analysis: Costs for the Three-loop Design (Scenario B)

PHASE I TIME (Yrs)	PHASE I COSTS ($)	PHASE II COSTS ($)	OVERALL COSTS ($)
8	2,176,294.75	959,237.06	3,135,531.75
9	2,212,180.25	895,215.94	3,107,396.25
10	2,250,549.50	834,487.25	3,085,036.75
11	2,291,517.75	772,774.19	3,064,292.00
12	2,335,209.00	720,266.44	3,055,475.50
13	2,393,520.25	646,755.75	3,040,276.00
14	2,454,008.00	579,579.13	3,033,587.00
15	2,871,073.25	182,903.72	3,053,977.00

Table 4.16 Failure Cost Factors

LAND USE	COST CASE I	COST CASE II
Residential	4	1.5
Industrial	4	1.5
Residential/ Industrial	4	1.5
Commercial	4	3
Major roads	4	3
Rural	1	1

Values adopted from Dandy and Engelhardt (2001)

Table 4.17 Sensitivity Analysis: Three-loop Design without Pricing - Phase I (Scenario C)

	SEGMENT ONE		SEGMENT TWO		EQUIVALENT DIAMETER LINKS		
LINK	DIAMETER (m)	LENGTH (m)	DIAMETER (m)	LENGTH (m)	DIAMETER (m)	LENGTH (m)	CHW
1 - 2	0.200	1000.00	-	-	0.200	1000	121.7
1 - 3	0.150	1000.00	-	-	0.150	1000	117
1 - 4	0.100	901.40	0.150	98.6	0.101	1000	113.7
2 - 3	0.150	70.73	0.200	929.27	0.193	1000	119.3
2 - 5	0.100	592.78	0.150	407.22	0.108	1000	113.7
3 - 4	0.150	373.22	0.200	626.78	0.170	1000	119.3
3 - 5	0.150	1000.00	-	-	0.150	1000	117

Table 4.18 Sensitivity Analysis: Three-loop Design without Pricing - Phase II (Scenario C)

LINK	TOTAL LINK FLOW (l/s)	(A-B): EXISTING UNPARALLELED LINK			(B-C): PARALLEL LINK			(C-D): REPLACED LINK		
		DIAM. (m)	LENGTH (m)	CHW	DIAM. (m)	LENGTH (m)	CHW	DIAM. (m)	LENGTH (m)	CHW
1 - 2	79.50	-	-	-	0.250	1000.00	130.0	-	-	-
1 - 3	43.82	-	-	-	0.150	918.72	126.9	-	-	-
					0.200	81.28	130.0			
1 - 4	17.53	-	-	-	0.100	1000.00	120.4	-	-	-
2 - 3	43.82	0.193	1000.00	114.1	-	-	-	-	-	-
2 - 5	17.53	-	-	-	0.100	1000.00	120.4	-	-	-
3 - 4	35.06	0.170	900.43	112.2	0.200	99.57	130.0	-	-	-
3 - 5	35.06	0.150	505.53	110.1	0.200	494.47	130.0	-	-	-

Table 4.19 Sensitivity Analysis: Costs for the Three-loop Design without Pricing (Scenario C)

PHASE I TIME (Yrs)	PHASE I COSTS ($)	PHASE II COSTS ($)	OVERALL COSTS ($)
9	3,209,103.25	1,064,566.13	4,273,669.50
10	3,262,813.50	974,008.06	4,236,821.50
11	3,320,093.50	885,386.63	4,205,480.00
12	3,381,139.75	810,553.06	4,191,692.75
13	3,463,404.50	714,553.81	4,177,958.25
14	3,548,717.25	629,544.44	4,178,261.75
15	4,158,327.75	189,830.38	4,348,158.00

CHAPTER FIVE

A NEW METHOD FOR HEAD-DRIVEN ANALYSIS OF WATER NETWORKS

5.1 INTRODUCTION

A vast amount of attention has been put in sophisticating water distribution network analysis packages and this is greatly supported by the ever-increasing efficiency of computers. These packages provide a basic tool for design, operation and management policies aimed at reducing costs, increasing reliability and reducing water wastage. Analysis packages that are commercially available are based on demand-driven analysis (DDA), which is sufficient for the analysis of a water distribution network if the available nodal heads are not less than the desired values. Unfortunately, nodal heads are not always satisfactory. A reduction in the available system pressures may be caused by pump or valve failures, pipe bursts, excessive demand e.g. for fire fighting at some nodes, etc. (Tanyimboh et al., 1999). This is a deficient network condition for which conventional methods like DDA are unable to accurately simulate the performance. These deficient WDS states can be handled using pressure-dependent network analysis as mentioned earlier in Chapter 2. The importance of simulating these deficient states cannot be overemphasized since such simulations are required for operational management aspects like reliability analysis, pressure-dependent leakage analysis and control, contingency planning, and water quality modelling.

Pressure-dependent network analysis or head driven analysis (HDA) differs from DDA in that the former recognises the primacy of pressures over demands, and considers nodal heads and flows simultaneously in the solution procedure. The objective of HDA is to establish the actual supply quantity from each node based on inherent characteristics and available pressures in the WDS. The previous chapter gives details of Network Design Module for the long-term optimal design of water distribution networks. This chapter presents a new algorithm called the Critical-node Head Driven Simulation Method (CHDSM), for performing head-dependent modelling of water distribution networks. This algorithm forms the engine of the Hydraulic Simulation Module and the Performance Assessment Module of the Integrated Model for long-term upgrading of water distribution networks. The Hydraulic Simulation Module and the Performance Assessment Modules are also described and examples are used to show their efficiency and practical application.

5.2 A NEW HEAD-DEPENDENT NETWORK ANALYSIS APPROACH

Many researchers have developed methods for head driven analysis of water distribution networks as detailed in Chapter 2 e.g. Germanopoulous (1985), Wagner et al. (1988a), Bhave (1991), Chandapillai (1991), Tanyimboh and Templeman (1998), Tabesh (1998), and Tanyimboh et al. (2001); and various weaknesses of these methods have been highlighted. For example, considerable underestimation of the total outflow from a deficient network (Tanyimboh and Templeman, 1995); the use of a step-length adjustment parameter that is network specific and obtained by trial and error (Tabesh, 1998); the use of head-outflow relationships with indices whose accuracy requires a considerable amount of effort in field data collection, analysis and network calibration (e.g. Tanyimboh and Tabesh, 1997b; and Tabesh, 1998). To address these uncertainties and weaknesses, a simple heuristic algorithm for determining pressure-dependent outflows in water distribution networks has been developed in this study and is detailed next.

A detailed study of various deficient networks (e.g. Fujiwara and Tung, 1991, Gupta and Bhave, 1996) to assess the effect of variation of outflow at the critical nodes of water distribution systems has been carried out in this research. The critical nodes are, generally, critical monitoring points in the network (nodes at isolated high points for regular pressure monitoring), nodes with abnormally high demands, nodes at the extreme ends of the network with respect to the distance water travels from a source to the node and nodes representing areas of persistent low pressure problems. From this study, it was observed that a deficient network has four categories of nodes including no-flow nodes, partial-flow nodes, key partial-flow nodes and nodes whose demands are fully satisfied. No-flow nodes are those with pressures below the absolute minimum pressure, H_{min} for outflow to be possible. Partial-flow nodes are those with pressures between H_{min} and H_{res}, the latter being a desirable pressure above which nodal outflow can be fully satisfied. Nodes whose demands are fully satisfied are those nodes with pressures above H_{res}. Key partial-flow nodes (nodes whose outflows affect outflows at other nodes) are generally nodes at isolated high points in the network, and in some cases, nodes could fall into this category depending upon the magnitude of their nodal demands and their location in the network.

The main conclusion of the study was that a distribution network in a stressed state experiences a reduction in the nodal outflow, in a decreasing progression from the most critically affected nodes to the least critically affected ones. From this conclusion, an algorithm for carrying out pressure-driven network analysis has been developed based on a technique involving a systematic identification of no-flow nodes, partial-flow nodes and key partial-flow nodes as shown by the flowchart in Figure 5.1. The details of the algorithm follow next.

5.2.1 Main Features of the Proposed Method

Recall that in Chapter 2, Newton-Raphson's method is used for demand driven analysis to obtain the nodal piezometric heads in an iterative scheme given by Eqs. (2.21) and reproduced below in Eqs. (5.1) as follows:

$$(J_H)_{(k)}\underline{\Delta H}^{(k)} = -\underline{F}\left(\underline{H}^{(k)}\right) \qquad (5.1)$$

in which $\underline{\Delta H}$ is the vector of the respective corrections to nodal heads. The nodal head estimates are updated by Eqs. (2.24) which are reproduced here as Eqs. (5.2)

$$\underline{H}^{(k+1)} = \underline{H}^{(k)} + \underline{\Delta H}^{(k)} \qquad (5.2)$$

where $\underline{H}^{(k+1)}$ is the vector of the adjusted nodal heads for an iteration and $\underline{H}^{(k)}$ is the vector of the nodal head estimates for the preceding iteration.

The proposed head-driven analysis model, builds up on these equations. Subnormal nodal flows ($0 < Q_j^{avl} < Q_j^{req}$) are obtained by converting the head-equations of Eq. (5.1) to head-flow equations as shown in Eq. (5.3). Head-flow equations have both nodal flows and heads as the unknown basic variables (Bhave, 1991). Thus

$$J_{HQ}^{(k)}\begin{bmatrix}\underline{\Delta H}^{(k)}\\ \cdots\cdots \\ \underline{\Delta Q}^{(k)}\end{bmatrix} = -\underline{F}\left(\underline{H}^{(k)},\underline{Q}^{(k)}\right) \tag{5.3}$$

in which, for the k^{th} iteration, $\underline{Q}^{(k)}$ is the vector of unknown nodal outflows, $\underline{\Delta Q}^{(k)}$ is the vector of the respective corrections to nodal outflows and $J_{HQ}^{(k)}$ is the Jacobian matrix for the head-outflow equations.

The nodal heads in these equations are updated by Eqs. (5.2) and successive values of the nodal outflows for partial-flow nodes are updated and given by

$$\underline{Q}^{(k+1)} = \underline{Q}^{(k)} + \underline{\Delta Q}^{(k)} \tag{5.4}$$

Similarly, the elements of the Jacobian matrix for the unknown nodal heads, J_H, (Eqs. 2.22 and 2.23) are modified with respective adjustments to give the new Jacobian, J_{HQ}, as follows:

$$\frac{\partial F_j}{\partial Q_p} = -1; \quad \forall j, \forall p : p = j \tag{5.5}$$

$$\frac{\partial F_j}{\partial Q_p} = 0; \quad \forall j, \forall p : p \neq j \tag{5.6}$$

where p is a partial-flow node.

The Newton-Raphson technique is used for the solution of the set of simultaneous equations using a refined form of Gaussian Elimination involving Scaled Column Pivoting (Burden and Faires, 1993). Although the method does not use any explicit head-discharge relationship, no-flow and partial-flow nodes are identified in a systematic way. The algorithm automatically carries out DDA or HDA as appropriate. The main driving force of the proposed formulation, called Critical-node Head Driven Simulation Method (CHDSM) is a pre-specified residual pressure, H_{res}, for nodal outflow at a desirable pressure. H_{res} can be set at a minimum nodal residual pressure value according to the terrain, locality served, plumbing arrangements and the general bylaws regarding residual heads. Under certain circumstances, the absolute minimum desired pressure is suggested to be 7m (US Army Corps of Engineers, 1984).

For purposes of calculating the reliability of a WDS, H_{res} can be set at a desirable pressure below which flow cannot be fully satisfied, with typical values being about 14m to 15m (Insurance Service Office, 1980; Twort et al., 2000) or any other standard value set by a regulatory water supply organisation. The absolute minimum residual pressure, H_{min}, below which no flow is possible at a node, can be set at a bare minimum value or zero. Under certain critical operating conditions, for example bursts in trunk mains, H_{res} can be set to the lowest value of those obtained from pressure loggers put at the critical points in the network and H_{min}, can be set at a bare minimum value or zero. The actual pressures and actual outflows can then be obtained for the entire network using the model.

A brief characterisation of some of the key concepts and variables used in the algorithm on which the model is based follows. H_{elevj} is the nodal elevation for node j. H_{statj} is the static head of node j. No-flow nodes are either nodes with no initial base demands or nodes whose outflows are confirmed and fixed as zero during the course of executing the algorithm. The most critical node and the next most critical node are the nodes with the lowest and second lowest nodal residual pressure in a specified category, X, of nodes and their pressures are denoted by H_{critX1} and H_{critX2} respectively. X takes on the value of N, P and K for no-flow nodes, partial-flow nodes and key partial-flow nodes, respectively. To enhance the computational efficiency, critical nodes in a given category X, whose residual pressures are approximately the same are considered to be in the same pressure contour and are processed together in sets. Thus, the set $X1$ of the most critical node(s) refers to one or more nodes with almost equal pressure values of H_{critX1}. H_{critN1} and H_{critN2} (where $H_{critN1} < H_{critN2}$) represent the respective pressure values of critical nodes in the same pressure contours during the stage of identifying no-flow nodes; the sets of nodes are designated sets $N1$ and $N2$ respectively. H_{critP1}, H_{critP2}, ..., H_{critPn} (where $H_{critP1} <$ H_{critP2} ..., $< H_{critPn}$) represent the respective pressure values of critical nodes in the same pressure contours during the stage of identifying partial-flow nodes and the sets of nodes are designated sets $P1$, $P2$, ..., Pn respectively. Similarly, H_{critK1}, H_{critK2}, ..., H_{critKn} (where $H_{critK1} < H_{critK2}$ ..., $< H_{critKn}$) represent the respective pressure values of critical nodes in the same pressure contours during the stage of identifying key partial-flow nodes and the sets of nodes are designated sets $K1$, $K2$, ..., Kn respectively. No-flow nodes do not belong to any of the $P1$, ..., Pn and $K1$, ..., Kn sets.

5.2.2 Algorithm for Head-Dependent Water Network Modelling

5.2.2.1 Part I: Identification of No-flow Demand Nodes

1) Given nodal demands, assume initial heads, H_j for all nodes other than fixed head nodes.
2) Calculate the nodal heads using DDA (Eqs. 5.1 and 5.2).
3) Identify all nodes whose static heads are less than their respective minimum heads, (H_{elevj}+ H_{min}). Fix the demands at these no-flow nodes to zero and perform Step 2.
4) Identify the most critical node of all non-zero demand nodes. If its pressure, H_{critN1} is less than H_{res}, then this node, together with any other nodes in the same pressure contour, should be taken as the critical nodes (designated set $N1$). Otherwise, exit.
5) Set the demands of the node(s) in set $N1$ to zero, and perform Step 2.
6) If the pressure H_{critN1} of the node(s) in set $N1$ is less than or equal to H_{min}, and the pressure, H_{critN2}, of the next most critical node is less than H_{res}, confirm the nodes in set $N1$ as no-flow nodes by fixing their demand values to zero, and return to Step 2. Otherwise, the nodes of set $N1$ are categorised as partial-flow nodes. Go to Part II.

5.2.2.2 Part II: Identification of Partial-flow Nodes

1) The set of partial-flow nodes should be designated set $P1$. If H_{critP1} of the node(s) in set $P1$ is less than H_{res}, all nodes with pressures between H_{min} and H_{res} should be categorised as partial-flow nodes with heads, H_{critP1}, H_{critP2}, H_{critP3}, H_{critP4}, ..., H_{critPn}, and grouped together with nodes of the same pressure contours in sets $P1$, $P2$, $P3$, $P4$, ..., Pn respectively. Set their pressures as $0.5(H_{critP1}+H_{min})$ for the node(s) in set $P1$, $H_{min}+0.75(H_{critP1}-H_{min})$ for the node(s) in set $P2$,

$0.5(H_{critP2}+H_{critP3})$, $0.5(H_{critP3}+H_{critP4})$, ..., and $0.5(H_{critPn-1}+H_{critPn})$, for nodes in sets $P3$, $P4$, ..., Pn, respectively. In effect, the averages between consecutive pressure values are used as appropriate at this stage. Otherwise, (i.e. if H_{critP1} is greater than or equal to H_{res}) set their pressures to H_{res}.

2) Convert the system of head-equations in Eqs. (5.1) into a system of head-flow equations as shown in Eqs. (5.3). Solve Eqs. (5.3) and update the nodal heads and flows using Eqs. (5.2) and (5.4) respectively (i.e. HDA).
3) Proceed to Part III.

5.2.2.3 Part III: Identification of Key Partial-flow Nodes

1) The set of partial-flow nodes in $P1$ should be designated set $K1$.
2) H_{critK2} is the lowest nodal residual pressure amongst pressures of nodes that are not no-flow nodes and do not belong to sets $P1$, ..., Pn. If H_{critK2} is less than H_{res}, the outflow (Q_{critK2}) of this node together with outflows of any other nodes in the same pressure contour should be set to zero, the nodes designated set $K2$ and then Step 2 in Part I performed. Otherwise, exit.
3) If the pressure of the node(s) of set $K2$ is less than H_{res}, confirm them as no-flow nodes by fixing their outflows to zero, otherwise go to Step 4. Set the pressure of nodes of set $K1$ to H_{res} and perform Step 2 in Part II. Go to Step 2.
4) For the two sets of key partial-flow nodes $K1$ and $K2$, set the pressure of nodes in set $K1$ to H_{res} and that of nodes in set $K2$ to $(H_{res}+\varepsilon)$, where ε is a small tolerance of about 0.05m. Perform Step 2 in Part II.
5) Get the next most critical node of the non-zero demand nodes that do not belong to sets $K1$, ..., Kn-1. If its pressure, H_{critKn}, is less than H_{res}, then this node together with any other nodes in the same pressure contour should be designated the nth set of key partial-flow nodes, Kn. The flows of all key partial-flow nodes should be obtained using Step 2 in Part II, by setting the pressure of nodes in the latest set, Kn, to $(H_{res}+\varepsilon)$, that of nodes in the first set, $K1$, to H_{res}, and setting the pressures of the remaining sets using a constant increment of $\varepsilon/(n-1)$.
6) Repeat Step 5 until there are no more key partial-flow nodes.
7) End the algorithm and exit.

The introduction of the equations for determining partial-flow in Step 2 in Part II of the algorithm, neither alters the basic structure and size of the Jacobian nor leads to an increase in the number of basic unknowns. Moreover, their introduction occurs at a point where values of all other unknowns have been obtained. This leads to the application of Newton's method in the immediate neighbourhood of the solution, and convergence is attained after a few additional iterations. It can therefore be expected that in general, the basic computational characteristics of the solution methodology will not be highly affected. In identifying critical nodes, nodes that have residual heads in the same pressure contour are treated together. This speeds up the process of identifying no-flow and partial-flow nodes, and improves the efficiency of the algorithm. The sets of partial-flow nodes tend to be few and consequently, obtaining outflow at partial-flow nodes does not involve many extra iterations.

To further improve on the efficiency of the algorithm, a line search and backtracking numerical routine has been incorporated as detailed next.

5.2.3 Improving the Efficiency of the Algorithm

5.2.3.1 Introduction

The Newton-Raphson technique used in this study is a very efficient method for solving non-linear equations given a set of initial estimates for the unknown variables. However, if these initial estimates are far from the actual solution, the method tends not to converge. In this research, a line search and backtracking numerical routine (Press et al., 1992) has been incorporated in the CHDSM algorithm to improve its convergence characteristics. This routine has been linked to the CHDSM algorithm at the point of solving the non-linear equations, i.e., Step 2 in Part I for calculating the nodal heads and Step 2 in Part II for solving the head-flow equations. A brief description of the algorithm that has been used to implement the numerical routine in this thesis follows shortly. The description of the algorithm below has been based upon Step 2 in Part I for calculating nodal heads. However, the same algorithm has been used for Step 2 in Part II to solve the head-flow equations, with appropriate adjustments to accommodate Eqs. (5.3) and (5.4).

As reviewed earlier in Chapter 2, the Newton-Raphson set of equations is given by Eqs. (2.18) which are reproduced here as

$$\underline{F}(\underline{H})=0 \tag{5.7}$$

These equations can be solved simultaneously for the corrections given by Eqs (2.20) which are reproduced as

$$\underline{\Delta H}^{k}=-(J_H)^{-1}_{(k)}\underline{F}(\underline{H}^{(k)}) \tag{5.8}$$

in which J_H is the Jacobian matrix for the unknown nodal heads. $\underline{\Delta H}^{(k)}$ is the vector of adjustments to the nodal head estimates in a given iteration and is also referred to as the full Newton step. These corrections are used to obtain new estimates of nodal heads using Eqs. (5.2) as in Step 2 in Part I of the CHDSM algorithm.

To implement the line search and backtracking method, a factor φ for the Newton step has been introduced (Press et al., 1992). For example, Eq. (5.2) has been modified to

$$\underline{H}^{(k+1)}=\underline{H}^{(k)}+\varphi\underline{\Delta H}^{(k)} \qquad 0<\varphi\leq 1 \tag{5.9}$$

In brief, the numerical routine involves a line search process in the major iterations (Eqs. 5.8 and 5.9) of Step 2 in Part I of the CHDSM algorithm, using the full Newton step with $\varphi = 1$ in Eq. (5.9). A check is made at each iteration to ensure that the proposed step $\varphi\underline{\Delta H}^{k}$ decreases the value of g (Press et al., 1992), which is given by

$$g=\frac{1}{2}\underline{F}\cdot\underline{F} \tag{5.10}$$

If the Newton step does not decrease the value of g, backtracking is carried out in a series of minor iterations to obtain a better step. To guarantee a sufficient decrease in g, the criterion for acceptance of a Newton step $\varphi\underline{\Delta H}^{k}$ that has been adopted is (Press et al., 1992)

$$g(\underline{H}^{k+1})\leq g(\underline{H}^{k})+\omega J_H^k\cdot\underline{F}(\underline{H}^{k})\cdot(\varphi\underline{\Delta H}^{k}) \qquad 0<\omega<1 \tag{5.11}$$

in which $g(\underline{H}^{k+1})$ is the evaluation of Eq. (5.10) at $\underline{H}^{k+1}$; $g(\underline{H}^{k})$ is the evaluation of Eq. (5.10) at $\underline{H}^{k}$; $J_H^k\cdot\underline{F}(\underline{H}^{k})\cdot(\varphi\underline{\Delta H}^{k})$ is the initial rate of decrease of g and ω is a fraction with a typical value of 0.0001.

5.2.3.2 Algorithm for the Line Search and Backtracking Routine

1) Set the iteration number $k = 0$; and assume initial nodal heads $\underline{H}^k$.
2) Calculate the "Euclidean" norm (Kreyszig, 1993) for the function values based on nodal head estimates, i.e. $\|\underline{F}(\underline{H})\|_2 = \sqrt{|\underline{F}(\underline{H})|^2}$. As a test for convergence, if the "Euclidean" norm is less than a pre-defined tolerance, exit.
3) Major iteration: $k = k+1$; compute $\underline{\Delta H}^{(k)}$ using Eqs. (5.8)
4) Adjust the nodal head estimates using the full Newton step by setting $\varphi =$ 1 in Eq. (5.9).
5) Check for acceptance of a Newton step: if the criterion for acceptance of a step (Eq. 5.11) is met, go to Step 2. Otherwise, perform backtracking (Press et al., 1992) along the Newton direction to find a smaller value of φ that decreases $g(\underline{H}^k + \varphi\underline{\Delta H}^k)$ sufficiently. Backtracking involves a series of minor iterations for minimising $g(\underline{H}^k + \varphi\underline{\Delta H}^k)$ modelled as a quadratic function in the first minor iteration and as a cubic function in subsequent minor iterations until the acceptance criterion of Eq. (5.11) is met. Adopt the Newton step $\varphi\underline{\Delta H}^k$ and obtain new head estimates $\underline{H}^{k+1}$ using Eq. (5.9). Go to Step 2.

The interested reader may consult Press et al. (1992) for the details of the line search and backtracking method for ensuring global convergence for systems of non-linear equations. The next two sections present details of computer programs for the Hydraulic Simulation Module and the Performance Assessment Module of the Integrated Model that have been developed in this research. Typical input and output files are presented in Appendix B.

5.3 HYDRAULIC SIMULATION MODULE

A computer program coded in FORTRAN 95 has been developed in this study for the realistic simulation of the network behaviour. The program is the hydraulic simulation module of the Integrated Model and it is based on the Critical-node Head Driven Simulation Method (CHDSM). The program has the capability of automatically performing demand driven analysis (DDA) and head driven analysis (HDA) of water distribution networks, as appropriate. It can handle networks with single and multiple sources. The program input requirements and output are detailed next.

5.3.1 Program Input

The input data for the program comprises of the following information:

1) The number of nodes, pipes, reservoirs, fixed head nodes, a pre-defined tolerance on nodal head estimates as a check for convergence, an option for DDA or HDA and an option for simulating the unavailability of pipes.
2) The connectivity matrix for the nodes showing how the nodes are interconnected to each other and the nodal demands (l/s).
4) Hazen-Williams coefficients for the links, their lengths and diameters in meters.
5) Nodal elevations and initial total head at nodes in meters.

5.3.2 Program Output

The output from the program consists of two main sections consisting of results for the links and those of the nodes as detailed shortly. Values for the total number of major iterations and overall CPU time are also given.

5.3.2.1 Results for the Links

These consist of tabulated results including the link label, upstream and downstream nodes, diameter, length, Hazen-William's coefficient, the flow rate and the hydraulic gradient or the unit head loss in each link.

5.3.2.2 Results for the Nodes

The results for the nodes are in a tabular format and they include the node number, demand, elevation, total head, pressure and nodal outflow.

5.3.3 Examples and Discussion of Results

This sub-section presents some examples for which the Hydraulic Simulation Module has been applied. For convenience, each example is described in detail followed by its results and a discussion of the results.

5.3.3.1 Example 1

In Chapter 2, a method by Tabesh (1998) and Tanyimboh et al. (2001) called the Head-driven simulation method (HDSM) has been detailed. The method uses a step-length adjustment parameter to eliminate oscillations and ensure faster convergence. This parameter is integrated in the equations for updating the head estimates at the end of each iteration. However, the determination of this parameter is time consuming and not a straightforward process as detailed next. To elaborate on the difficulty encountered in selecting the step-length adjustment parameter (SAP) when using HDSM, a number of trials have been made on a simple network. A FORTRAN program for HDSM by Tabesh (1998) has been used in this study to carry out the trials and generate all the results of this example. The network for this example, which is shown in Figure 5.2, has been used by several researchers to demonstrate several aspects of design and reliability of water distribution networks (Fujiwara et al., 1990, 1991; Awumah and Goulter, 1992; Tanyimboh et al., 1993, 1995). The designs upon which the present appraisal is based are taken from Tanyimboh and Tabesh (1997a) based on Fujiwara and Tung (1991). The pipe diameters for the sixteen different designs are presented in Table 5.1. All pipes are 1000m long and the Hazen Williams coefficient for each pipe is 130. The minimum and required nodal heads are 0 and 30m respectively, and the source head, Hs, is 100m.

To obtain the best SAP value, various runs of the model have been made in this study; each run with values for total inflow and outflow at the point of convergence. These values for each run have been plotted on a graph and the best choice of the SAP identified from the graph as the one that yields equal inflow and outflow. The results are shown in Figures 5.3 and 5.4. From these figures, it is clear that different choices of the step adjustment parameter lead to a variation between nodal inflow and outflow. For Design 1, with all links available (Fig. 5.3a), the best SAP value is about 300 and with link 1–2 unavailable (Fig. 5.3b), the best SAP is about 150. For Design 16, with all links available (Fig. 5.4a), the best SAP value is about 230 and with link 1–2 unavailable (Fig. 5.4b), the best SAP is about 130.

When the right choice of SAP value is used, HDSM gives very good results. Each individual run or model execution takes approximately 0.25 seconds on a Pentium I, 75MHz, 8 MB. Considering the numerous runs of the model required in the search for the best SAP value, this accumulated run time is probably the basis of the method's computational inefficiency. For example, an average of 10 trial runs of the program (each requiring about 0.25seconds) were required to obtain the best SAP implying an average CPU time of 2.5seconds for each design. The fact that the SAP value for each design is different and the effort required to obtain each SAP value emphasises the fact that it is network specific.

5.3.3.2 Example 2

To demonstrate efficacy of the Critical-node Head Driven Simulation Method (CHDSM), the formulation has been applied to the sample network in Figure 5.2; and the sixteen designs in Table 5.1 with the same data as in Example 1. The results have been compared to those generated by Tabesh (1998) using the Improved Source Head Method (ISHM) that has been reviewed in Chapter 2. The CHDSM results show that all the designs have deficient outflow at node 9 only, leading to total outflows supplied falling short of the total demand of 208.10 *l/s*. On the other hand, ISHM results show that Designs 1 to 4 have nodes 6, 8, and 9 with deficient outflow (Tabesh, 1998). Thus, CHDSM has a better recognition of the spatial performance characteristics of the network than ISHM.

Using the same designs, fractions of total demand satisfied by the fully connected network have been generated with CHDSM and presented in Figure 5.5. For comparison purposes, proportions of total demand satisfied by a fully connected network using different methods namely Source Head Method (SHM) by Tanyimboh and Templeman (1998), the Improved Source Head Method (ISHM) by Tanyimboh et al. (1997) and Head Driven Simulation Method (HDSM) by Tabesh (1998) are presented in the same Figure.

Figure 5.5 clearly shows that the CHDSM formulation gives values of flow delivered at adequate pressure that are closer to those obtained by HDSM than the values got using the ISHM and SHM formulations. Table 5.2 shows the deviations of proportions of total demand satisfied obtained by SHM, ISHM and CHDSM from the control values of HDSM. Apart from the CHDSM results, the results in this table are taken from Tabesh (1998). Whereas on average, SHM underestimates total outflow by about 24% and ISHM overestimates the total outflow by about 5%, CHDSM underestimates the total outflow by only 1%. Therefore the present formulation has an advantage that reasonable estimates of system performance can be obtained in a manner that avoids the difficulties associated with HDSM.

Another aspect worth emphasising is the computational efficiency of the present formulation. CHDSM results were generated for each of the sixteen designs, using a 400MHz Ultra Sparc Sun system. For each of the sixteen designs, the CPU time required was approximately 0.26 seconds. A single DDA run using CHDSM for the same network with a higher source head to ensure that all demands are satisfied requires a CPU time of about 0.12 seconds.

The performance of CHDSM for head-dependent network analysis has also been compared to two new methods. The first one, which has been developed by Tanyimboh et al. (2002), is presented next in Sub-section 5.3.3.3. This is followed by the second method in Sub-section 5.3.3.4, which has been developed by Ackley et al. (2001). In each of these sub-sections, a brief review of the methods is presented, followed by the application of the methods to sample networks. The reason for reviewing these methods in this chapter and not in Chapter 1 is because they have been developed recently and after CHDSM.

5.3.3.3 Example 3

The first of these methods for head-driven analysis has been developed by Tanyimboh et al. (2002) and is called the Newton Raphson plus Line Search

Algorithm (NRLSA). The method is used for solving the constitutive equations to obtain nodal pressures and outflows for water distribution systems with (or without) insufficient pressure. It is an improvement of HDSM and it uses the Newton-Raphson technique in which a line minimisation (Press et al., 1992) is performed at each iteration to obtain a step-length adjustment parameter (Eq. 5.10). NRSLA overcomes the weakness of HDSM of having to determine the step-length adjustment parameter by trial and error. The ability of this method to incorporate a range of head-outflow relationships into the constitutive equations is one of its main strengths (Tanyimboh et al., 2002). NRLSA has improved convergence characteristics, low computational requirements, and it gives results that are both accurate and hydraulically feasible. However, Tanyimboh et al. (2002) have noted that different assumed head outflow relationships yield different results in the prediction of network performance under subnormal conditions, and thus, the need for field data to ascertain the most appropriate relationship. It might even be possible that a network may have different head outflow relationships for nodes in different parts of the system.

In this study, CHDSM has been applied to the looped network in Figure 5.2, for the first twelve of the sixteen designs by Tanyimboh and Tabesh (1997a) that are shown in Table 5.1 and have been used in Examples 1 and 2. Two source heads of 100m and 50m have been used for the twelve designs. These source heads are not high enough to satisfy all the nodal demands implying that the network has deficient operating conditions. The purpose of this example is to compare the results obtained by CHDSM to those obtained by Tanyimboh et al., (2002) and Tabesh (1998) who have also applied NRLSA and HDSM respectively, to this network under similar conditions with minimum and required nodal heads of 0 and 30m. The limitations of demand driven analysis for deficient network analysis are also demonstrated.

Table 5.3 shows the nodal demands and the results for Design 1 with a source head H_s of 100m, obtained for the conventional demand driven analysis using HDSM (Tabesh, 1998), NRLSA (Tanyimboh et al., 2002), CHDSM and EPANET2 (Rossman, 2000). CHDSM like HDSM and NRLSA gives results that match closely with the EPANET2, confirming its accuracy. The fact that demand driven analysis results yield negative results at nodes 7, 8 and 9 probably means that the demands at some or all of these nodes cannot be fully satisfied, though the magnitude of shortfall in demand cannot be quantified.

The head-driven analysis results in terms of nodal heads and actual outflows at the nodes are also presented in Table 5.3 for NRLSA, HDSM and CHDSM. Unlike the demand driven analysis results, these results give the actual outflow at the nodes showing that apart from node 9, full nodal demand satisfaction is achieved at all the other nodes. Thus, CHDSM, like NRLSA and HDSM, can effectively represent the spatial performance of the network in terms of the locations and magnitudes of the shortfalls in demand. Also, the fact that head-driven analysis methods are superior to demand driven analysis for deficient network conditions as noted by Tabesh (1998), is further confirmed by the CHDSM results for HDA.

A check for hydraulic feasibility has been done using EPANET2 to confirm whether the HDA results obtained by the three methods are consistent. This involves taking the actual nodal outflows obtained for HDA using each method (CHDSM, HDSM and NRLSA) as input data for the nodal demands in EPANET2. Demand driven analysis is then carried out using EPANET2 to obtain total heads that are compared to the corresponding total heads obtained for HDA using each of the methods. The results are presented as EPANET2 output for the feasibility check in Table 5.3. Generally, the results obtained using CHDSM to perform HDA when compared to those of DDA using EPANET2 in the feasibility check (e.g. for node 9, 8m vs 7.89m), exhibit a better conformity just like those of NRLSA (e.g. for node 9, 5.27m vs 5.29m), as compared to those of HDSM that do not match as closely (e.g. for node 9, 4.28m vs 7.75m).

Figure 5.6 presents the fractions of total demand satisfied for the twelve different designs using CHDSM, HDSM and NRLSA for a source head, H_s, of 100m together with that of 50m. When the source head is 100m, CHDSM slightly underestimates the total outflow from the network compared to HDSM and NRLSA whose results match very closely. On the other hand, when the source head is reduced to 50m, CHDSM results though slightly lower than those of NRLSA, are in better agreement as compared to the results of HDSM. All in all, the variation in the demand satisfaction ratio between the results of the different methods is very small, and this confirms the hydraulic feasibility and accuracy of CHDSM.

5.3.3.4 Example 4

The second of these methods for head-driven analysis has been developed by Ackley et al. (2001) and is referred to herein as the outflow maximisation method. The method involves the use of non-linear programming techniques to solve the network analysis problem with the objective of maximising the available outflow from all the demand nodes. In this method, the network analysis problem has been set up as an optimisation problem for normal and abnormal network operating conditions. The objective function has been taken as a maximisation of the sum of nodal outflows subject to the constitutive equations, and bounds on nodal outflows together with the non-negativity of link flows. The main variables have been taken as the link flow rates.

Ackley et al. (2001) have applied the method to a network while varying the source heads to simulate deficient network conditions, obtaining results that give an accurate representation of the behaviour of deficient networks. They have noted that the network in such stressed states yields pipe flow rates that are close to zero, leading to the violation of loop constraints and prompting the need for these constraints to be reformulated whenever this occurs. This problem has been solved by using nodal heads as the basic variables. The method gave a good representation of deficient network performance. However, computational difficulties were encountered for very low source heads.

In this research, CHDSM has been applied to the two-loop sample network in Figure 5.7. Ackley et al. (2001) have applied the outflow maximisation method to this network. Tanyimboh et al., (2002) have also applied NRLSA to this network with its design data as described next. The intent herein is to show further evidence of the accuracy and robustness of CHDSM by comparison of results from the three methods. Each pipe has a length of 1000m and a Hazen-Williams coefficient of 140. Pipe 1-2 has a diameter of 0.5m; pipes 2-3, 2-4 and 3-5 each have a diameter of 0.4m, and pipes 4-5, 4-6, 5-7 and 6-7 each have a diameter of 0.25m. The minimum nodal heads for outflow to be possible at a node have been taken as 50m for nodes 2, 3, 7 and 6; and 45m for nodes 4 and 5. The nodal demands are 77.8 l/s for node 4; 55.6 l/s for node 6; 88.9 l/s for node 7, and 41.7 l/s for each of the nodes 2, 3 and 5, respectively. The source head at node 1 has been varied from 76m down to 45m to show the impact of insufficient network pressures on the total outflow from the network. Each network with a different source head represents a different design.

For each design, the sum of the nodal outflows has been divided by the total network demand to obtain the total demand satisfaction ratio. All the results are shown in Figure 5.8, which has been adopted from Tanyimboh et al. (2002) apart from the CHDSM results that have been included. Naturally, for all the three methods, the higher the source head, the higher the demand satisfaction ratio. For all the designs, CHDSM gives results that are very close to the outflow maximisation method and the NRSLA whose results are identical. Despite the fact that CHDSM results are slightly lower than those obtained from the two recent methods, they provide a very good representation of the performance of a deficient network. Tanyimboh et al. (2002) have noted that below a source head of 51m, the outflow maximisation method did not produce any results due to computational problems as shown in Figure 5.8. However, CHDSM just like NRLSA can be used to predict the network performance right down to a demand satisfaction ratio very close to zero, i.e., when the source head is very low, confirming that CHDSM is a stable and robust method.

5.3.3.5 Example 5

CHDSM has been applied to the serial network shown in Figure 5.9, which has been adapted from Gupta and Bhave (1996). The lengths and the Hazen-Williams coefficients for all pipes are 1000m and 130 respectively. The diameters of pipes 1 to 4 are 400mm, 350mm, 300mm and 300mm respectively. The nodal outlet elevations of nodes 1 to 4 are 90m, 88m, 90m, and 85m respectively, have been taken as the respective minimum nodal heads. The demands Q_j^{req} for nodes 1 to 4 are 2m^3/min, 2m^3/min, 3m^3/min and 4m^3/min, respectively. This is a deficient network that requires head-driven analysis.

The source head for the serial network has been varied from 85m to 110.89m in order to check the accuracy of CHDSM and to demonstrate the effects of variations in the source head on available nodal outflows. Gupta and Bhave (1996) have used a method known as Node Flow Analysis to obtain the total network supply the available nodal outflows as shown in Figure 5.10. The thick continuous curve in the figure is the related head-discharge curve that depicts the actual quantity of water or the network's total supply at the different source heads. For comparison purposes, CHDSM has been applied to this network design and the results of the network's total supply together with the available nodal flows are shown in Figure 5.10. The predicted total supply results by Wagner et al. (1988a) are also presented in the figure.

Comparing the CHDSM results to those of Gupta and Bhave (1996) reveals that there is a very close relationship. The total supply to the network is exactly the same. The curves for the available nodal flows follow the same trend and they are similar. Therefore this approach probably gives an accurate representation of the network behaviour in terms of the total network supply. CHDSM gives good results for available nodal outflows, and a realistic prediction of deficient network performance.

5.3.3.6 Example 6

The Critical-node Head Driven Simulation Method has been applied to the multiple-source Network A in Figure 5.11 for demand driven analysis and head driven analysis. In this Figure, nodes 1 and 2 are the source nodes, and nodes 3, 4, 5 and 6 are demand nodes. The node and link data are shown in Table 5.4, in which the source heads for nodes 1 and 2 are 76m and 54m respectively. H_{res}, has been taken as 15m, and the absolute minimum residual pressure, H_{min}, as zero. The results are presented in Table 5.5. From this table, considering the DDA results, it can be seen that the residual head (total head at node less the nodal elevation) at node 6 (10.77m) is less than H_{res}. Thus, the network is deficient and should be analysed using head driven analysis techniques. CHDSM results of HDA are shown in the same table and they reflect the network deficiency clearly by the fact that the outflow for node 6 is

75.38l/s, which falls short of the demand (88.9l/s). The CPU time for HDA is about 0.06 secs on a 1.2 MHz PC with 128 MB RAM.

Confirmation of hydraulic feasibility of the HDA results obtained by CHDSM has been achieved by comparing the output with that of EPANET2 (Rossman, 2000). This involves taking the actual nodal outflows obtained for HDA using CHDSM as input data for the nodal demands in EPANET2. Demand driven analysis is then performed using EPANET2 and the resulting nodal residual heads are compared to those obtained from HDA using CHDSM. Table 5.5 shows the results of this verification. It is evident that the results from CHDSM match very closely with those from EPANET2 and are therefore hydraulically feasible.

There are various practical applications of pressure dependent network modelling that would improve the operation and management of water distribution systems. Performance Assessment of water distribution systems requires HDA and a program for this purpose is presented in the next section. In Leakage control by pressure management, (considering that excessive pressures in the system leads to increase in leakage), use of HDA is more likely to lead to a better leakage control decision being made in terms of the choice of a pressure reducing valve or PRV settings. Water quality modelling for networks in stressed states involves key issues like:

-Water age or travel time of water from source to node, generally, older water has poorer quality.

-Simulation of the movement of a substance through a network e.g., a pollutant or chlorine, from one or more sources as a function of time.

Since water-distribution networks and the processes within them are time dependent (Rossman and Boulos, 1996), HDA can provide more realistic pipe flow rates and a more accurate picture of the water quality with respect to these key issues.

Given that the proposed technique is an extension of the conventional technique, it can be applied along with any conventional methods of network analysis, and all components like pumps and valves can be included. It can easily be tailored to perform extended period analysis. In this thesis however, the pumps have not been included as explained in Chapter 4. Also, extended period analysis has not been done due to the reasons stated earlier in Chapter 2.

5.4 PERFORMANCE ASSESSMENT MODULE

This section presents a program that has been coded in this research using FORTRAN 95. This program is based on the CHDSM algorithm. It involves the random simulation of unavailable links and the probability of a network being in a given full or reduced state in terms of availability of components. The key performance assessment parameters that have been used in this study are reliability and failure tolerance. These parameters have been reviewed in detail in Chapter 2. This section starts off by presenting the formulae for network reliability and failure tolerance, followed by required input and output data for the program and some examples.

5.4.1 Network Reliability and Failure Tolerance Formulae Used

The formulae for these parameters have been presented earlier in Chapter 4 and only reproduced here for ease of reference. Assuming a constant demand value, and taking only one and two unavailable components into consideration, the reliability, *Re*, of a water distribution system can be taken as (Tanyimboh and Sheahan, 2002)

$$\mathrm{Re} = \frac{1}{Q_{req}}\left(p(0)Q(0) + \sum_{l=1}^{NL} p(l)Q(l) + \sum_{\substack{l=1 \\ m=l+1}}^{NL-1} p(l,m)Q(l,m) \right) + \frac{1}{2}\left(1 - p(0) - \sum_{l=1}^{NL} p(l) - \sum_{\substack{l=1 \\ m=l+1}}^{NL-1} p(l,m) \right) \tag{5.12}$$

Herein, the nodal demands are taken as constants. In practice, however, they vary in a random fashion and this issue, with respect to reliability assessment of water distribution systems is currently an area of active research.

Tanyimboh and Templeman (2000) have noted that the first term of Eq. (5.12) corresponds to the basic definition of reliability of the distribution system. However, since all possible combinations are not included in practice due to the excessive computational time requirements, it underestimates the reliability. They have asserted that the advantage of Eq. (5.12) is that the second term, which improves the reliability estimates, involves only pipe availabilities or unavailabilities and its evaluation does not require extra hydraulic analysis. Furthermore, its magnitude can be used to ascertain whether extra simulations of link unavailability can improve the reliability results. This fact has been fully exploited in this research and an in-built check has been included in the computer program to monitor the cumulative sum of the pipe availabilities or unavailabilities in the second term of Eq. (5.12) after each simulation of link unavailability. This sum approaches unity as more simulations are carried out and thus, the second term approaches zero. When this happens, the program execution is stopped since this is an indication that there can be no further significant improvement on reliability. This improves the efficiency of the program in that unnecessary simulations of link unavailability can be avoided without compromising the system reliability.

The formula used for failure tolerance, *FT*, which is a quantified measure for redundancy is (Tanyimboh and Templeman, 1998),

$$FT = \frac{\mathrm{Re} - r(0)p(0)}{1 - p(0)} \tag{5.13}$$

5.4.2 Program Input

The input data for the program comprises of the following information:

1) The number of nodes, pipes, reservoirs, fixed head nodes, a pre-defined tolerance on nodal head estimates as a check for convergence and an option for the maximum number of unavailable links for each simulation.
2) The connectivity matrix for the nodes showing how the nodes are interconnected to each other and the nodal demands.
6) Link diameters, Hazen-Williams coefficients and lengths.
7) Nodal elevations and initial total head at nodes.
8) Values for H_{min} and H_{res}.

5.4.3 Program Output

The output from the program consists of two main sections as described next.

5.4.3.1 Link Availability Values

The availability values for the links consist of tabulated results including the link label, diameter and the availability value, a_l. The probability that all links are available is also shown.

5.4.3.2 Results for Reliability and Failure Tolerance

These results consist of a tabular format showing a summary for each of the configurations of link unavailability. They include the configuration number, unavailable link number, the probability that the network has the respective configuration, and the total outflow. The values for system reliability, component failure tolerance, overall number of iterations and the overall CPU time are also given.

5.4.4 Examples and Discussion of Results

This sub-section presents some examples for which the Performance Assessment Module has been applied to show its practical application. Once again, each example is described in detail, followed by its results and a discussion of the results. Sample networks have been analysed with up to two components simultaneously unavailable. This implies that each network is analysed with different combinations of unavailable links in order to obtain $Q(0)$, $Q(l)$ and $Q(l, m)$, these being the respective actual total outflows when zero components, component l and components l and m are unavailable. The pipe availabilities have been calculated based on the diameters, using Eq. (2.54) by Cullinane et al. (1992). The results have then been used to calculate the $p(0)$, $p(l)$ and $p(l, m)$ values. Finally, Re and FT have been calculated using Eqs. (5.12) and (5.13). The lengths and the Hazen-Williams coefficients for all pipes are 1000m and 130 respectively.

5.4.4.1 Example 7

The purpose of this example is to demonstrate the importance of the key performance assessment parameters used in this thesis and to highlight the need of using both parameters together in assessing the performance of water networks. The serial network used in Example 5, shown in Figure 5.9 has been used for this example. The network details and nodal demands are exactly the same as those in Example 5. The source head has been varied to correspond to a range of reservoir conditions from 85m to 110.89m and the performance of the system assessed in each case. It is worth noting that below a source head of 110.89m the network is stressed in that the source head is not sufficient to satisfy all the nodal demands. The detailed results are shown in Table 5.6. For comparison purposes, EPANET2 (Rossman, 2000) has been used with nodal outflows obtained from the proposed performance assessment module, in order to obtain the source heads required to satisfy these outflows and confirm that they are hydraulically feasible. The results have also been presented graphically in Figure 5.12, in which the $r(0)$ values of CHDSM coincide with those of EPANET2 confirming the accuracy of the hydraulic analysis model used.

Figure 5.12 suggests that compared to reliability, it is quite difficult to improve failure tolerance or redundancy in a cost-effective manner, especially for branched networks. Regardless of the source head value used, the network's failure tolerance value of about 0.35 is low, because there are no alternative supply paths from the source to the demand nodes. Thus, the degree of redundancy of the design is low. An inexperienced designer may not necessarily recognise this vulnerability through lack of redundancy. However, if the failure tolerance value is calculated explicitly, this vulnerability can be easily identified. Failure tolerance is therefore a useful measure of redundancy since it reflects this degree of vulnerability, with the advantage of being easily quantifiable using Eq. (5.13) as noted by Kalungi and Tanyimboh (2001).

Consider two reservoir conditions when the source head is 98.5m and 110.89m respectively. The redundancy value is 0.325 for both conditions and the respective reliabilities are 0.7270 and 0.9995 (Table 5.6). The values for failure tolerance or redundancy clearly expose the fact that the network is highly vulnerable to component failure in both cases, which undermines the higher reliability value of the latter case. Consideration of reliability values alone would not necessarily make this fact obvious. The redundancy values are equal mainly due to the layout of the network. Being a serial or branched network, supply of water to the demand nodes depends largely on connectivity such that above a certain source head, the expectation of total flow delivered is the same. On the other hand, the variation of reliabilities is mainly due to the change in total outflow for the various reservoir conditions. This is indicated by the trends of results of reliability and the proportion, $r(0)$, of total flow delivered; coupled with the fact that these results match very closely (Figure 5.12). Therefore, it does not necessarily mean that a network with a high reliability has a high failure tolerance level and this emphasizes the need for calculating reliability and failure tolerance together in order for a better judgement to be made on the performance of a network design (Kalungi and Tanyimboh, 2001).

5.4.4.2 Example 8

This example further stresses the importance of the key performance assessment parameters used in this thesis and the need of using both parameters together in assessing the performance of water networks. The sample network used in this example is the two–loop network in Figure 5.13, which has been used by other researchers (e.g. Fujiwara and Tung, 1991, Tanyimboh, 1993, etc.). The pipe data for six candidate designs have been taken from Tanyimboh (1993) and are presented in Table 5.7. The source node has a fixed head of 35m, and all elevations of demand nodes are 0m. H_{min}, the absolute minimum residual head for flow to be possible at a node has been taken as 0m, while H_{res}, the residual head for nodal outflow at a desirable pressure, has been set at 15m (OFWAT, 1998). The proposed performance assessment model has been used to obtain a range of performance data detailed in Table 5.8 and presented graphically in Figure 5.14. Typical details of the program's output showing 29 different layouts obtained by simulating the unavailability of up to two components are given in Appendix B, for Design 1. The average CPU time for the combined simulation of all the 29 layouts together with the reliability and failure tolerance calculation is about 1.2 seconds on a 1.2MHz computer with 128MB of RAM.

The graphs in Figure 5.11 suggest, that the use of components that are more mechanically reliable to ensure a low frequency of a failure occurring, leads to improved reliability or overall performance. Neglecting the issue of redundancy however, could lead to serious problems when components are taken out of service for maintenance purposes. Kalungi and Tanyimboh (2001) have noted that the conformity in the trend of network reliability and the probability, $p(0)$, that all links are available, is attributed to the fact that the network would be fully connected most of the time (see Eq. (2.53) and the $p(0)$ values in Figure 5.14). However, the trend of failure tolerance or redundancy is different. The unavailability of individual components has a significant influence on redundancy values. From Table 5.7, there is a general increase in the link diameters for links 2-4, 4-6, and 1-2, from Design 1 to 6. This probably indicates that unavailability of these links would lead to a decreasing trend in total network outflow from Design 1 to 6, hence the trend in failure tolerance. Of the six different designs, the lowest failure tolerance value is about 0.8, suggesting that all the designs have a reasonable degree of redundancy. Designs 5 and 6 have the highest and comparable network reliability values (Fig. 5.14 and Table 5.8), which is due to the fact that they have the highest $p(0)$ values and the highest total outflow when all links are available. However these designs are the most vulnerable to component unavailability. Design 5 could be considered better that Design 6 on the grounds of marginal superiority with respect to redundancy, which can be quantified using failure tolerance. All in all, although reliability is a commonly used assessment parameter, for a more comprehensive performance appraisal, reliability and failure tolerance should be used together (Kalungi and Tanyimboh, 2001).

5.4.4.3 *Example 9*

To further confirm the robustness and computational efficiency of CHDSM, it has been applied to the multiple-source Network B shown in Figure 5.15. The lengths and the Hazen-Williams coefficients for all pipes are 1000m and 130 respectively. Pipes S1-S2, 4-5 and 4-6 each have a diameter of 50mm; pipes S1-3 and 3-5 have a diameter of 250mm, pipe S2-4 has a diameter of 80mm and pipe 5-6 has a diameter of 150mm. The ground elevation for all nodes is zero. The demand for each of the nodes 3 and 4 is 10l/s; that for node 5 is 30l/s and for node 6 is 45l/s. H_{min}, the absolute minimum residual head for flow to be possible at a node has been taken as 0m, while H_{res}, the residual head for nodal outflow at a desirable pressure, has been set at 15m.

The source heads have been varied to obtain a range of performance data for six different scenarios as detailed in Table 5.9 and shown graphically in Figure 5.16. Starting with Scenario 1 whose respective source heads for S1 and S2 are 28m and 20m, which are insufficient for full satisfaction of nodal demands; each source head has been increased by 10m for every individual operational scenario to improve the pressure conditions. Thus, the respective source heads for S1 and S2 for Scenario 2 are 38m and 30m; for Scenario 3, 48 and 40; and so on up to Scenario 6 when the network can fully satisfy nodal demands with source heads of 78m and 70m for S1 and S2, respectively.

Figure 5.16 suggests that there is a considerably high level of reliability and failure tolerance with values being higher than 0.5 in all cases, probably due to the fact that the network has 2 sources and is looped. This means that there are alternative supply paths, which allow a fraction of the demand to be satisfied when some pipes are not available. The figure shows that there is a general increasing trend in the reliability, redundancy and the proportion of total demand satisfied *r(0)* from Scenario 1 to 6. However, a closer look shows that the rate of increase of redundancy as the source heads increase (from Scenario 1 to 6), is lower than that of reliability. Thus, the importance of using reliability and redundancy together for network performance appraisal is evident. The typical CPU time required to obtain the reliability, which in this case involved a total of 29 full and degraded network configurations, is about 2.5 seconds. This confirms the computational efficiency of the technique and its applicability to multiple-source networks. The technique is robust in that feasible results are obtained even when the source heads are low.

Another practical aspect of the Performance Assessment Module is in the removal of excessive redundancy in existing and aging networks that have too many loops. These loops could easily lead to low velocities and long residence times culminating in water quality problems. An attempt to reduce these loops can be carried out while ensuring that the resulting failure tolerance value does not fall below a pre-specified value, in order to ensure that the robustness of the network is not compromised in the process.

5.5 CONCLUSIONS

A technique called the Critical-node Head Driven Simulation Method (CHDSM) that can be used for determining head-dependent outflows in WDS has been presented. CHDSM can also perform conventional demand driven analysis (DDA). It simultaneously considers nodal heads and flows in the prediction of deficient-network performance and it does not require independent head-discharge relationships in its solution procedure. It uses the Newton-Raphson technique and Gaussian Elimination refined with the Scaled Column Pivoting method to solve the linear equations. This has been further coupled with a line search and backtracking numerical routine to further improve on the efficiency and convergence characteristics of the technique.

CHDSM can be used to assess the effect of a random combination of up to two simultaneously unavailable components. The method can be applied to networks with single and multiple sources. Using examples, it has been demonstrated that CHDSM is capable of producing results that are accurate, hydraulically feasible and compare favourably with other methods. The method is robust and stable and can give hydraulically feasible results even when the pressure in the network falls to very low values. The method has been encoded into a computer program using the FORTRAN 90 programming language. This program has been shown to have a high efficiency with values of CPU program execution time to convergence that are relatively low compared to a program like the Head Driven Simulation Method (HDSM) which requires additional effort to establish the best SAP.

Another FORTRAN 90 program, the Performance Assessment Module, has also been presented. It involves the use of CHDSM for simulating networks under abnormal loading conditions, and/or, with random unavailability of components. The results obtained are then used to calculate key performance assessment parameters called reliability and failure tolerance. The importance of these parameters in the process of assessing the performance of water networks has been demonstrated and the need for using these two parameters together has been highlighted. The Hydraulic Simulation Module and the Performance Assessment Module are appropriate tools for the routine operation and management of water distribution systems, and, proper simulation of low-supply situations.

The next chapter presents a detailed review of various methods for multi-criteria decision-making and gives an introduction to a method called the Analytic Hierarchy Process (AHP).

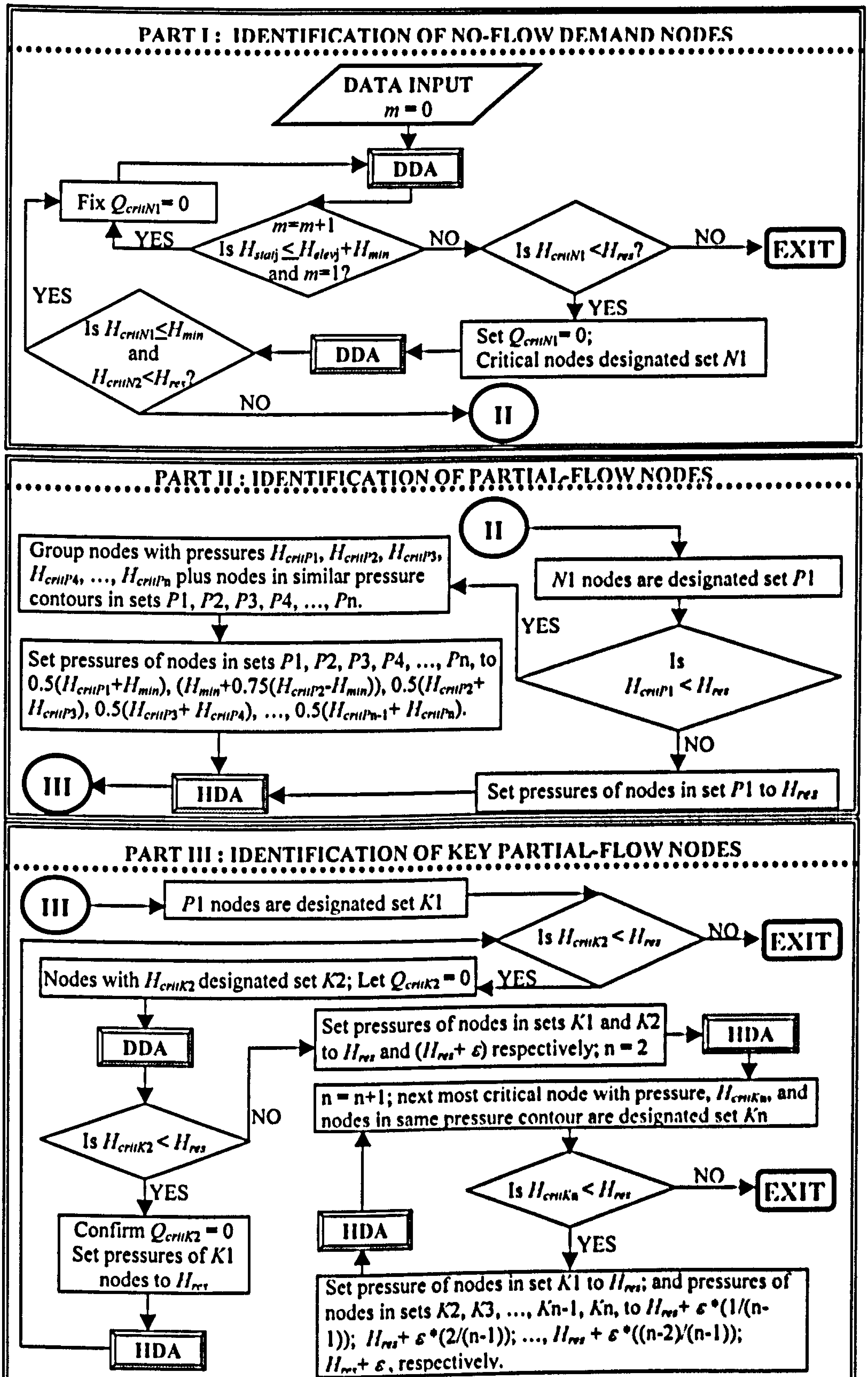

Figure 5.1. Flow Diagram for the CHDSM Model

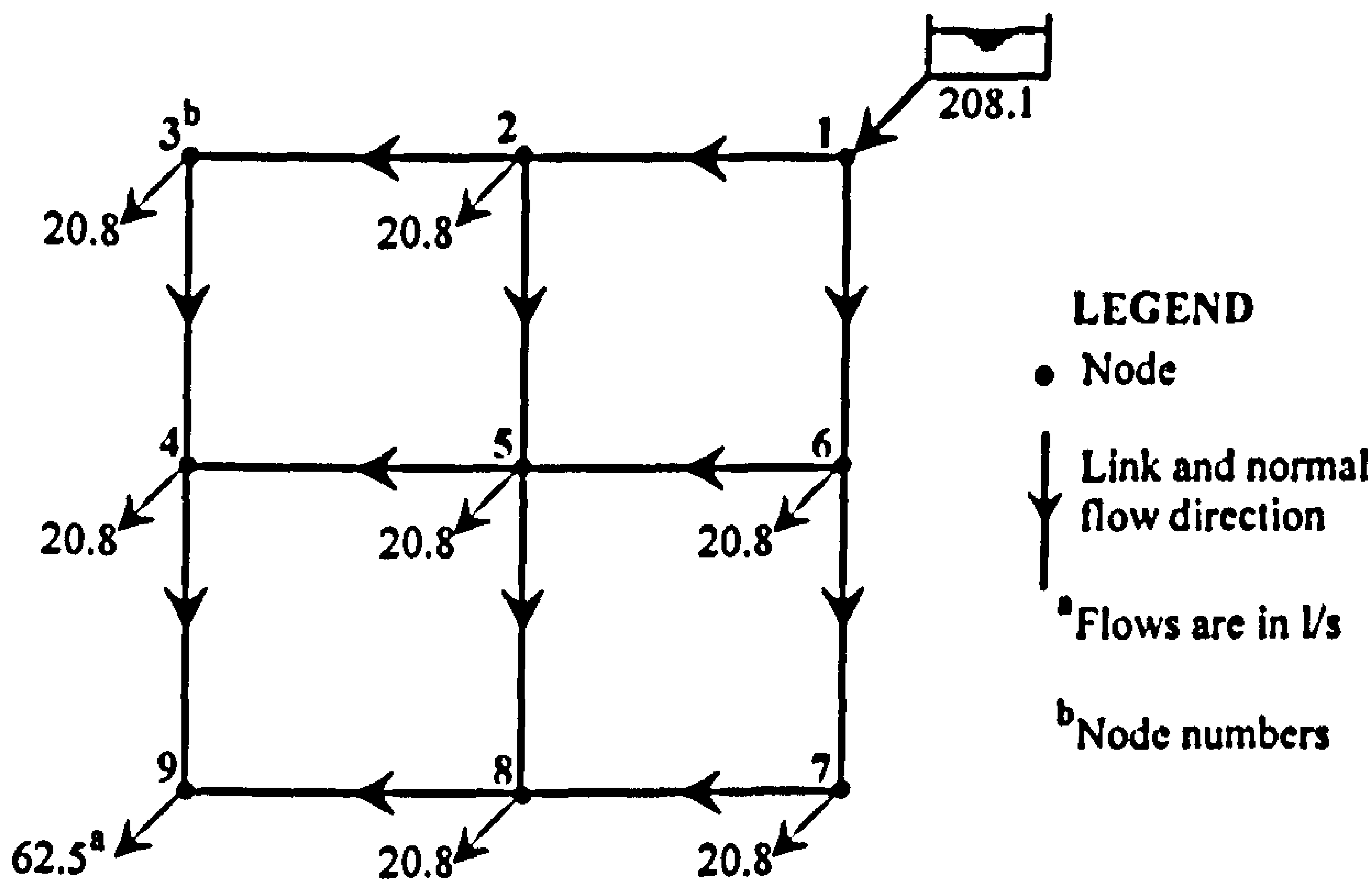

Figure 5.2. Looped Network

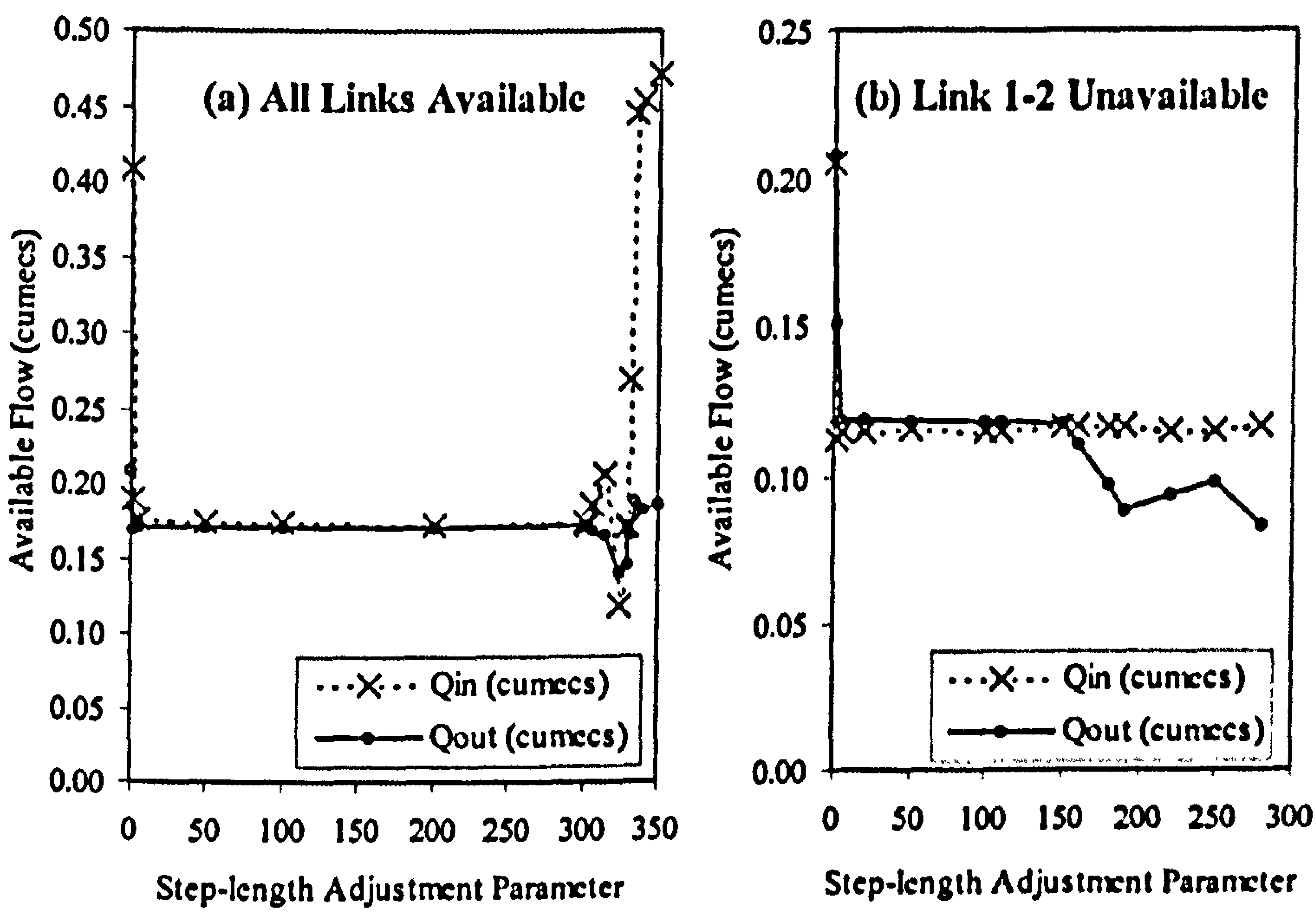

Figure 5.3. Total Inflow, Qin, and Outflow, Qout, vs Step-length Adjustment Parameter, for Design 1.

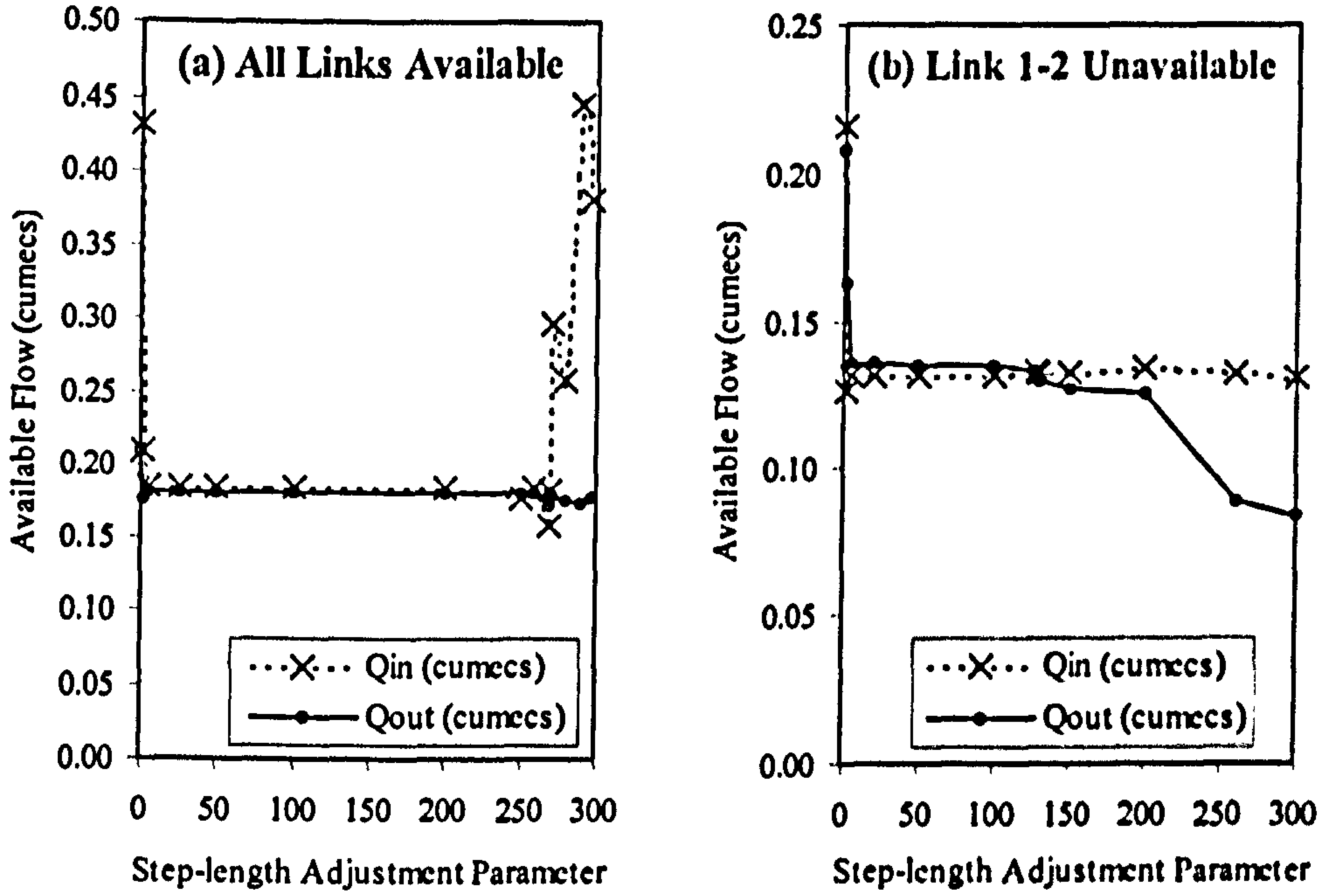

Figure 5.4. Total Inflow, Qin, and Outflow, Qout, vs Step-length Adjustment Parameter, for Design 16.

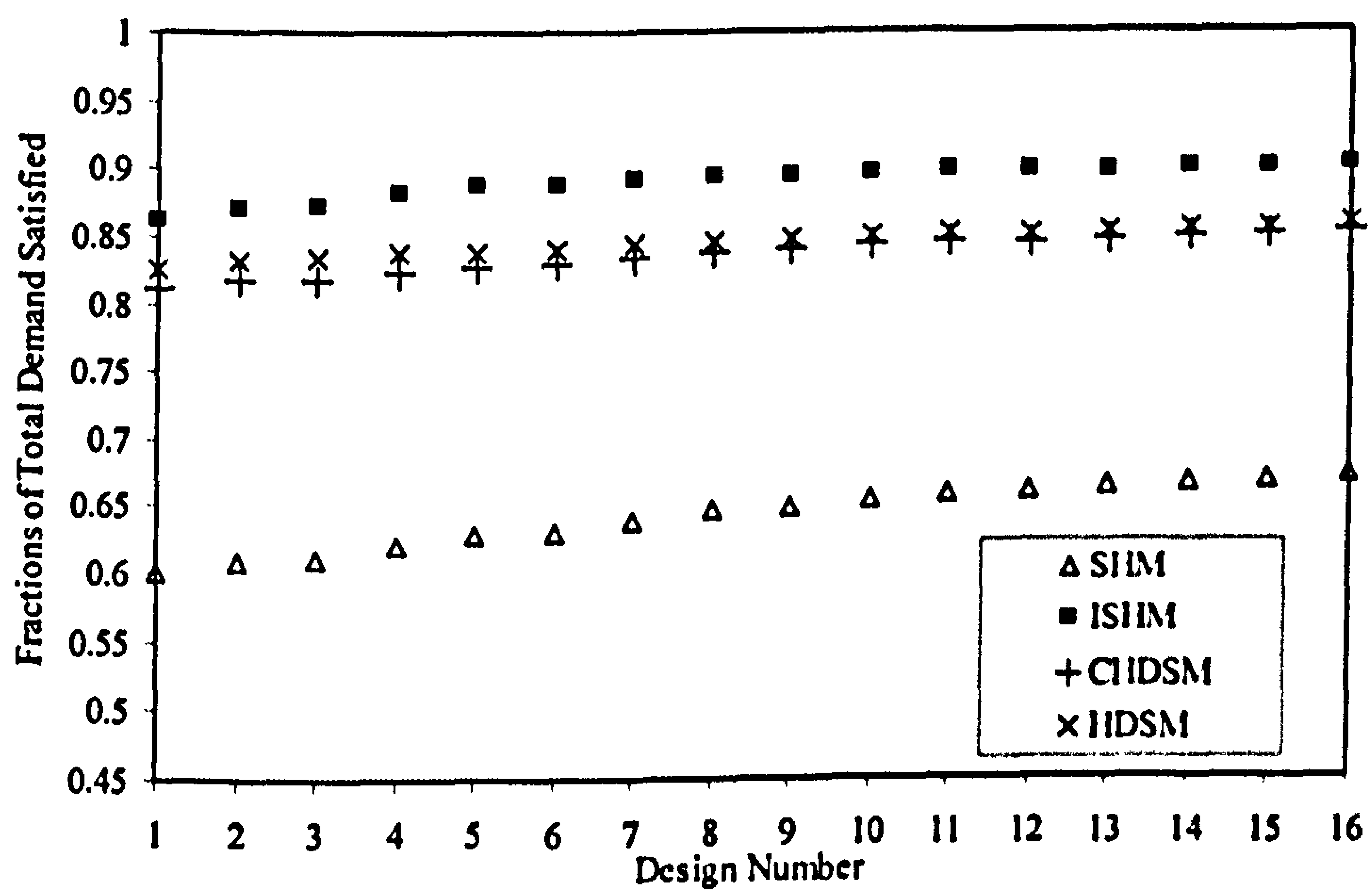

Figure 5.5. Fractions of Total Demand Satisfied by Fully Connected Networks

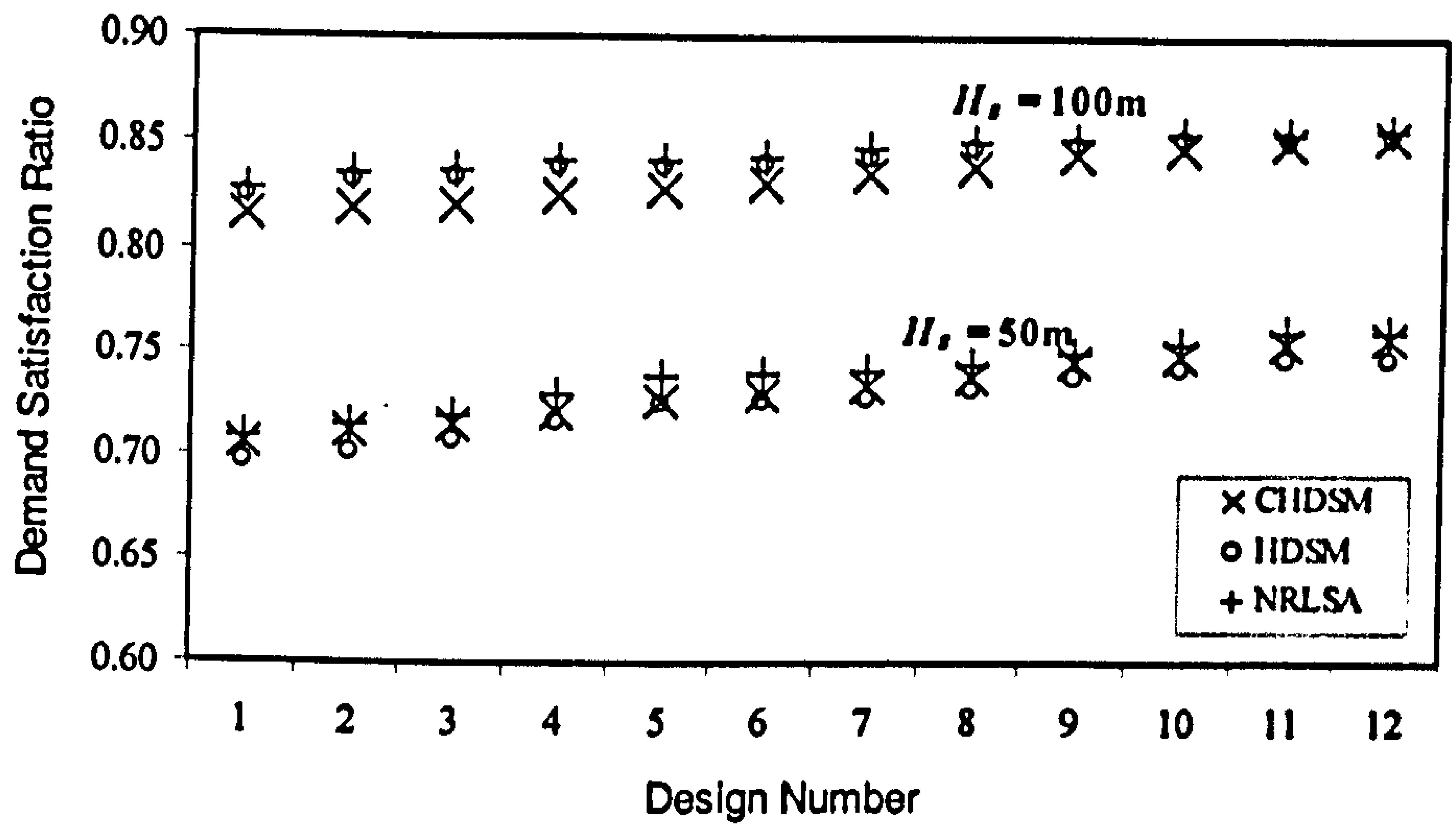

Figure 5.6 Fractions of Total Demand Satisfied for Different Designs and Source Heads

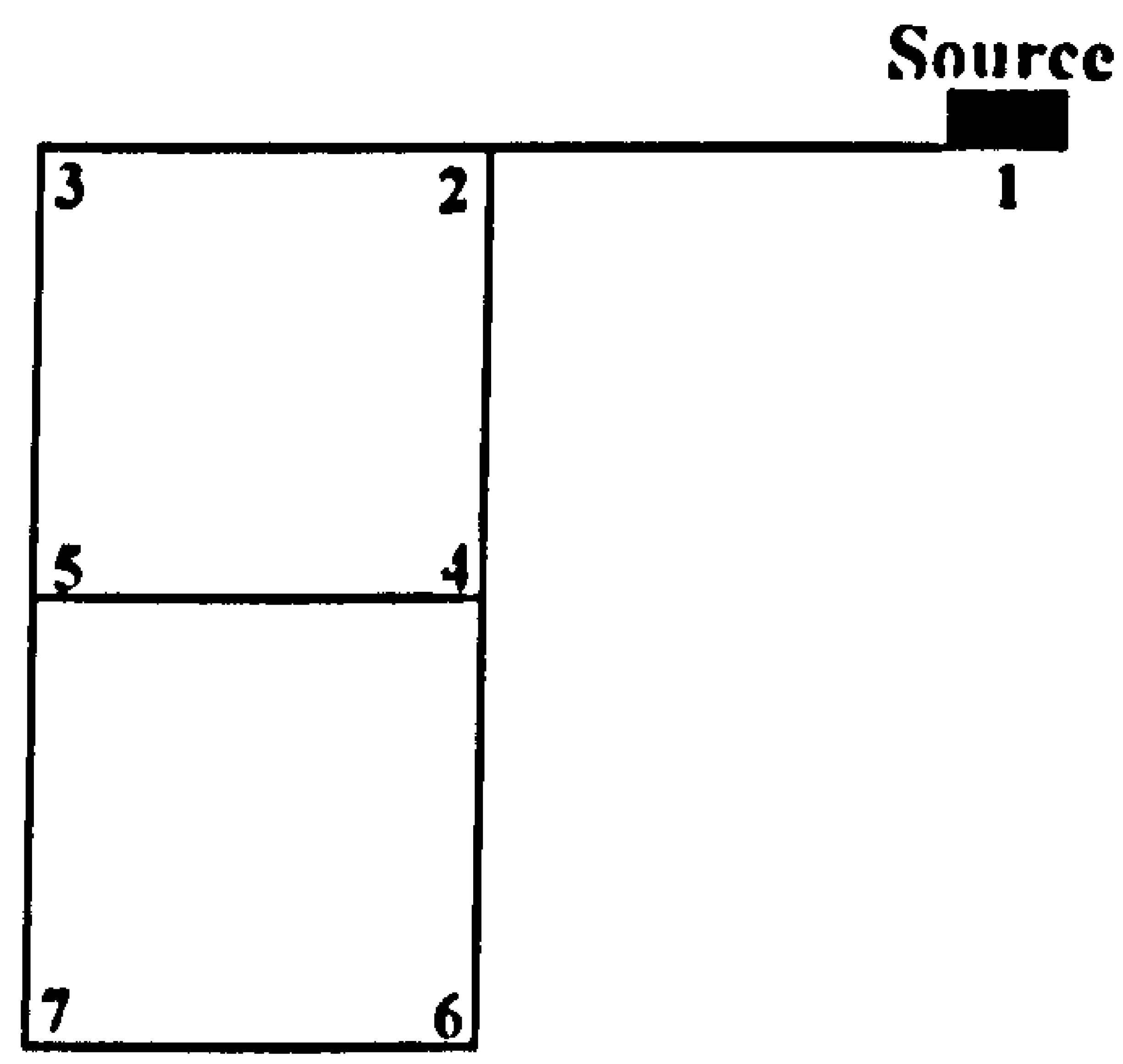

Figure 5.7 Two-loop Sample Network

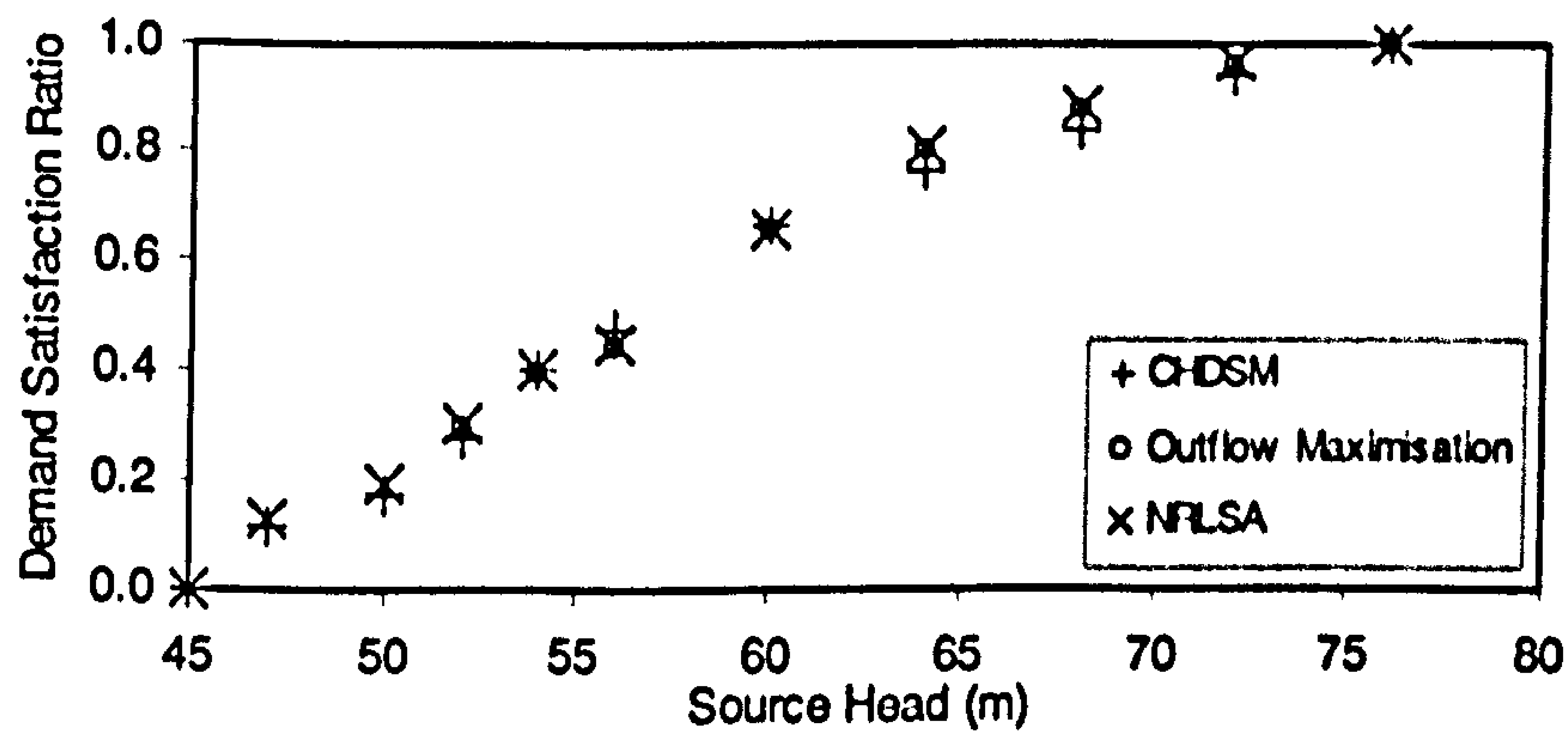

Figure 5.8 Total Demand Satisfaction Ratios for Various Source Heads

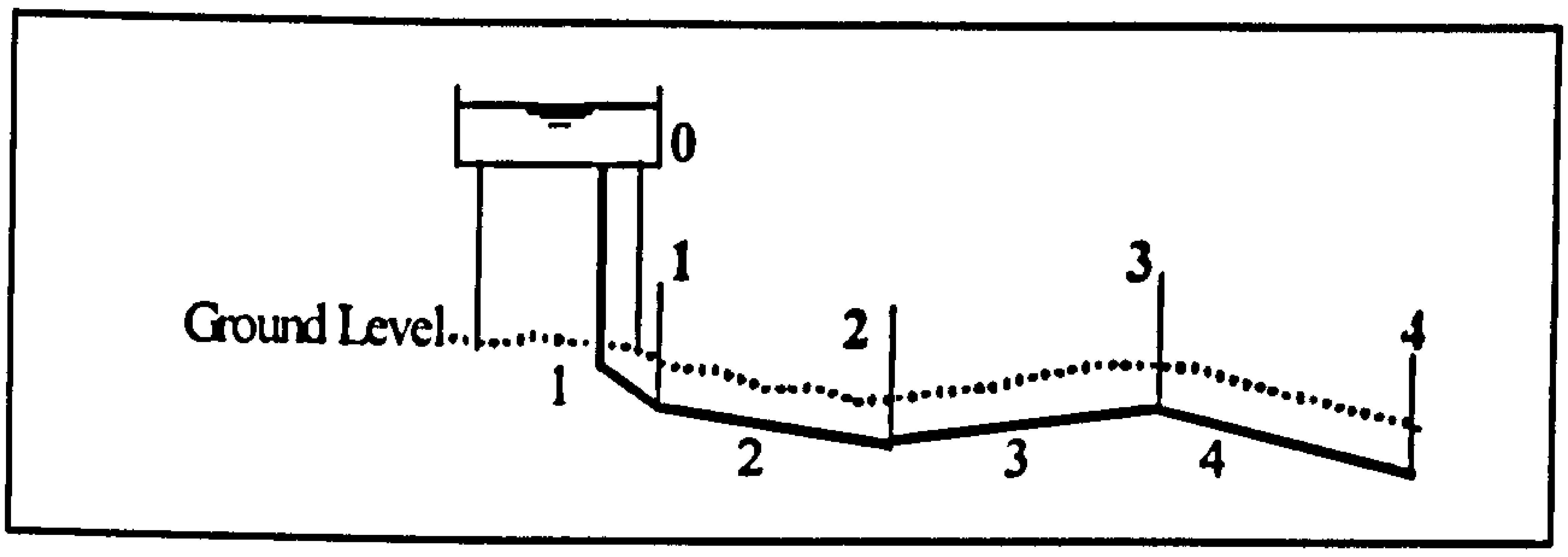

Figure 5.9 Serial Network

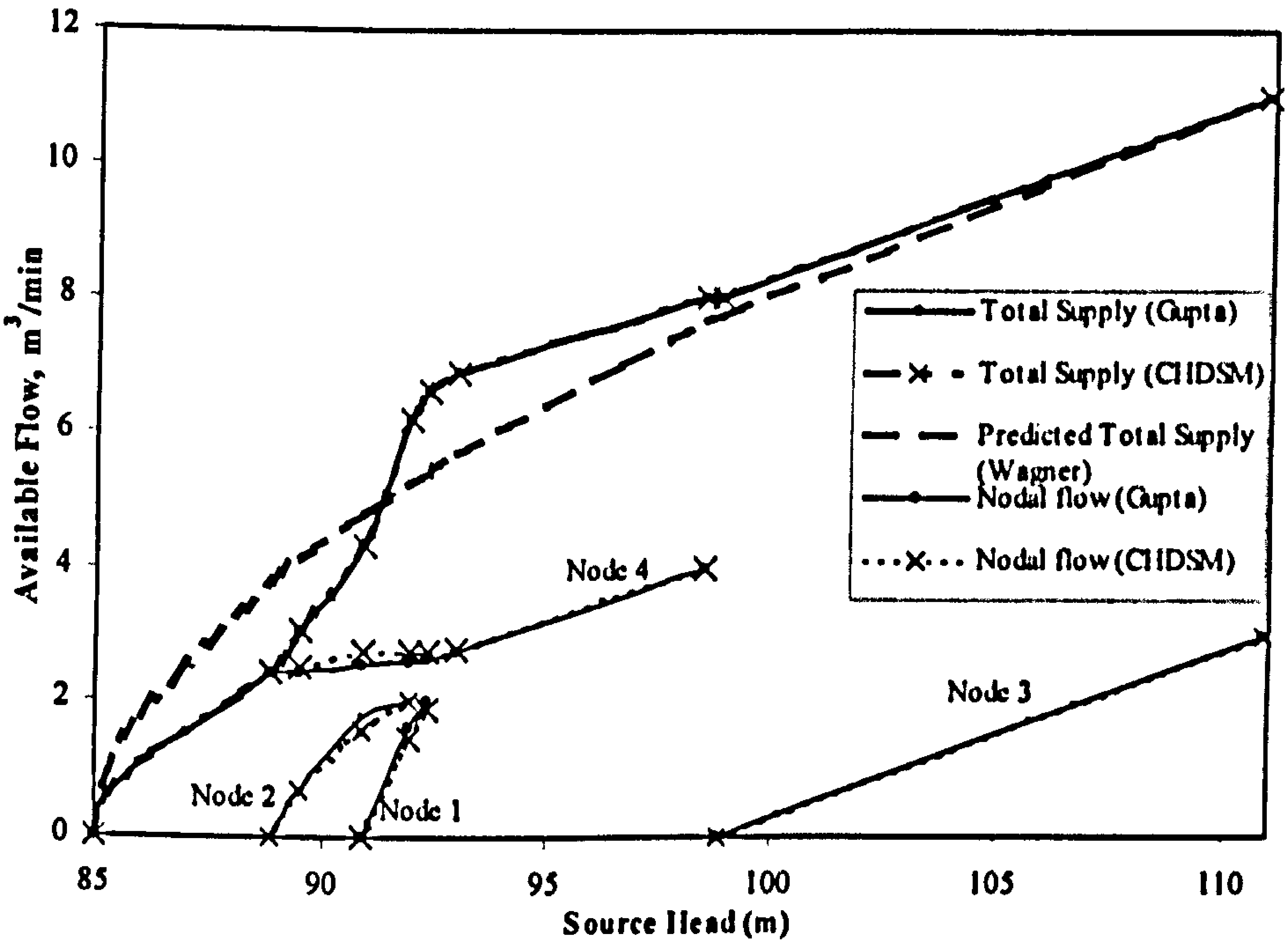

Figure 5.10 Variation of Available Flow for Different Source Heads

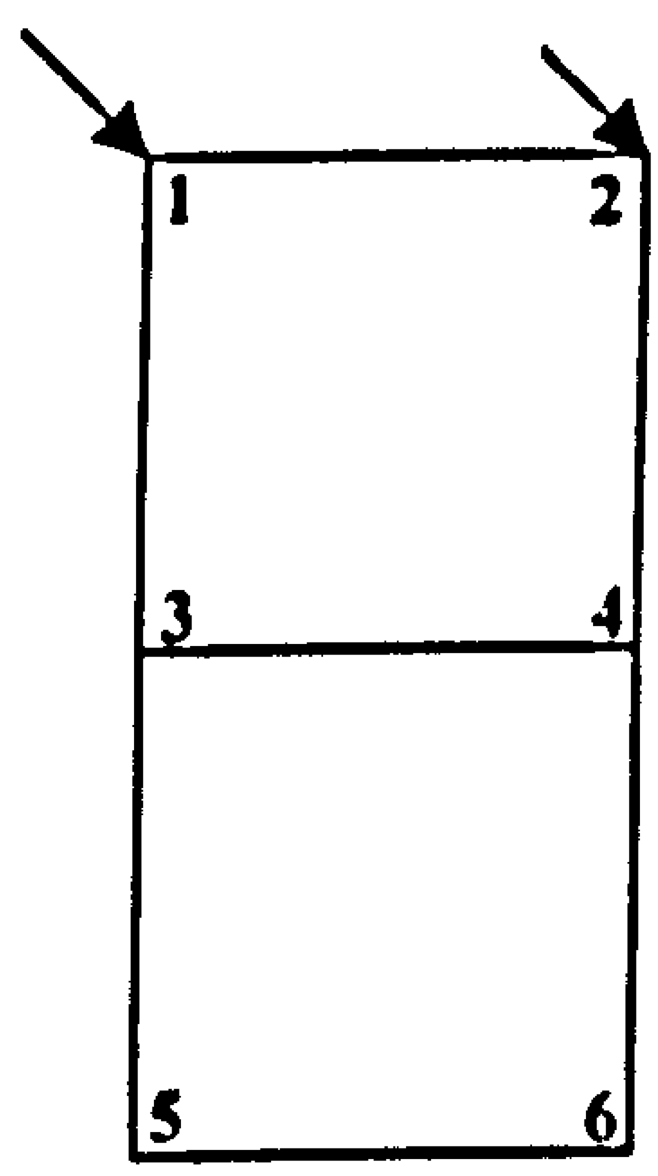

Figure 5.11 Multiple-source Network A

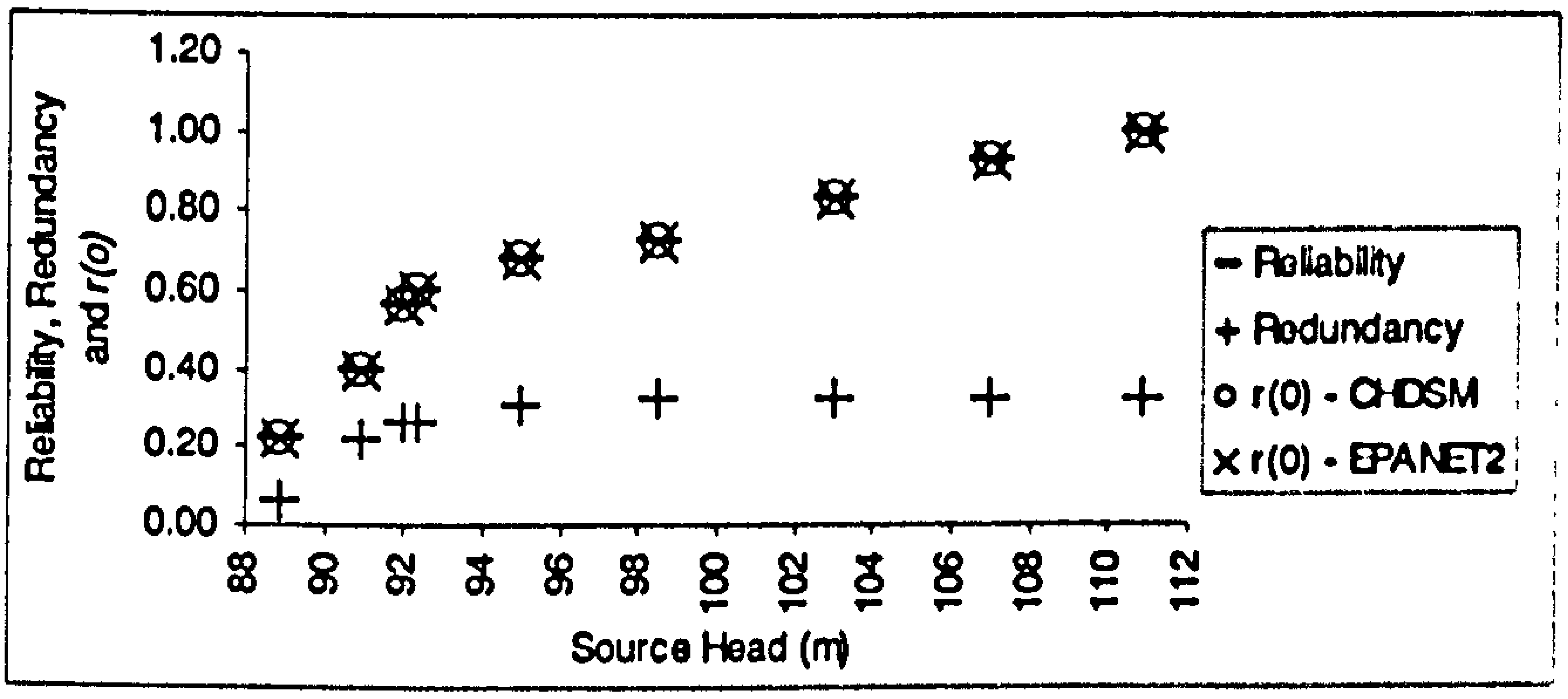

Figure 5.12 System Performance for a Range of Source Heads

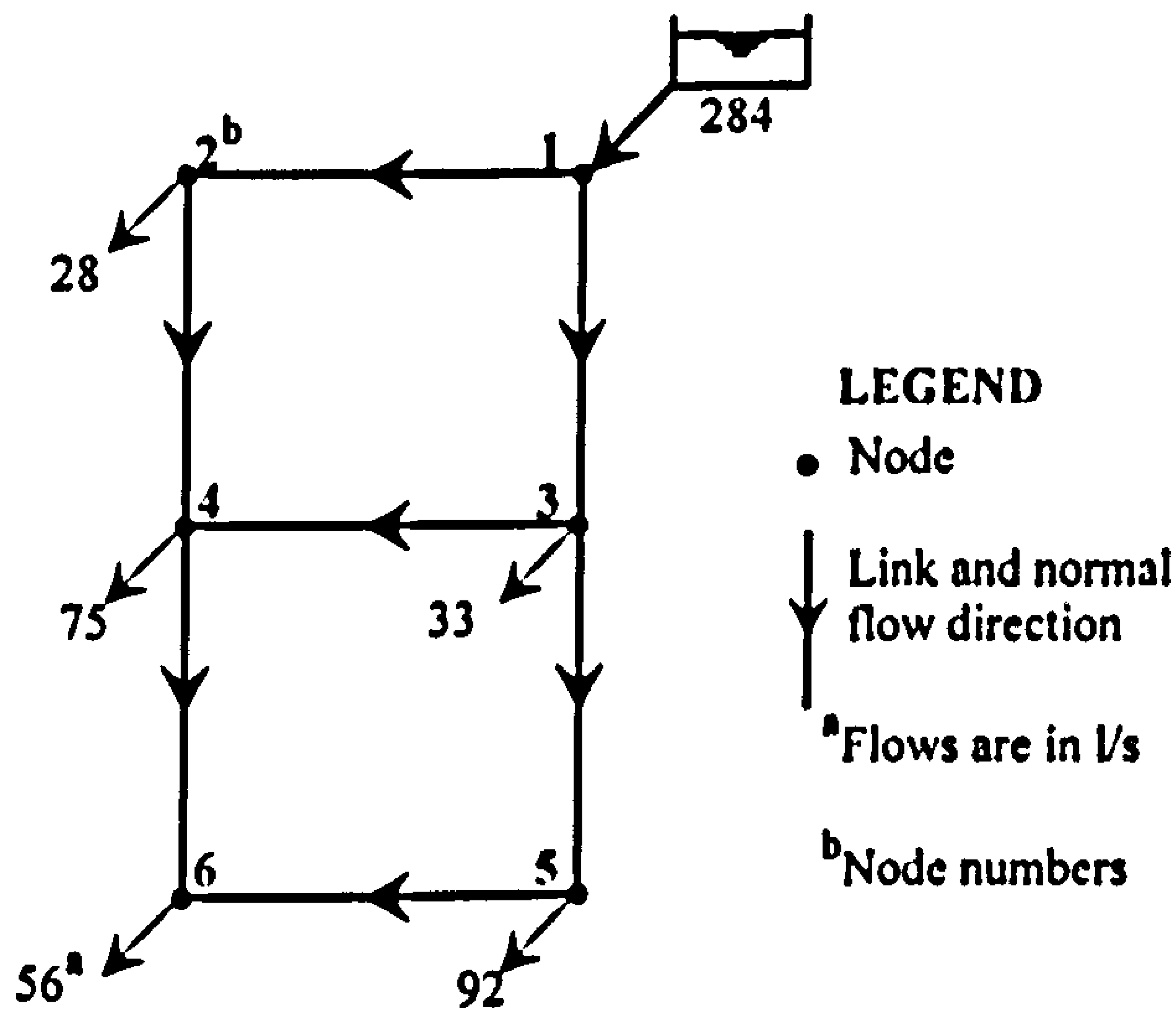

Figure 5.13 Two-loop Network

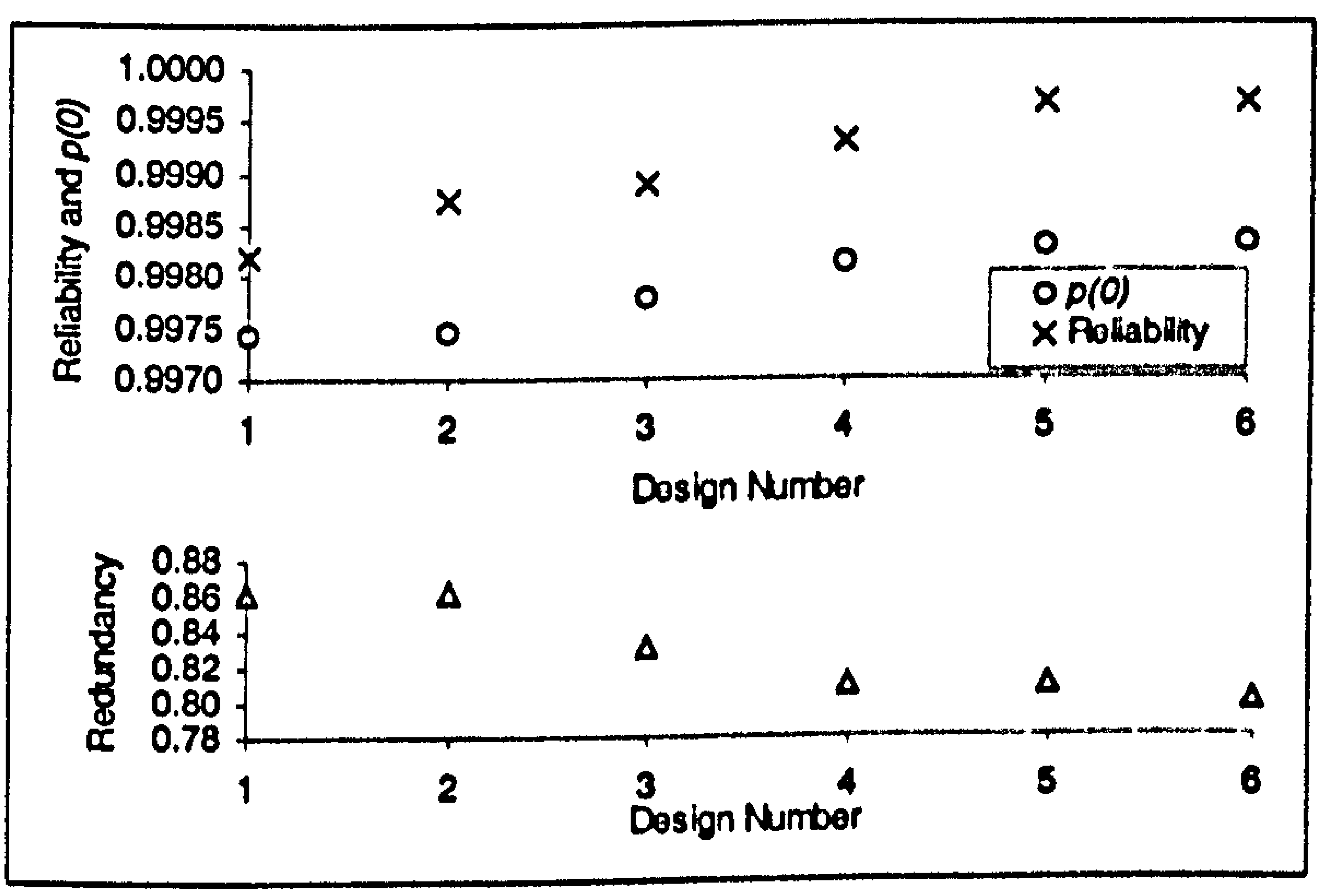

Figure 5.14 Performance Measures for Alternative Designs

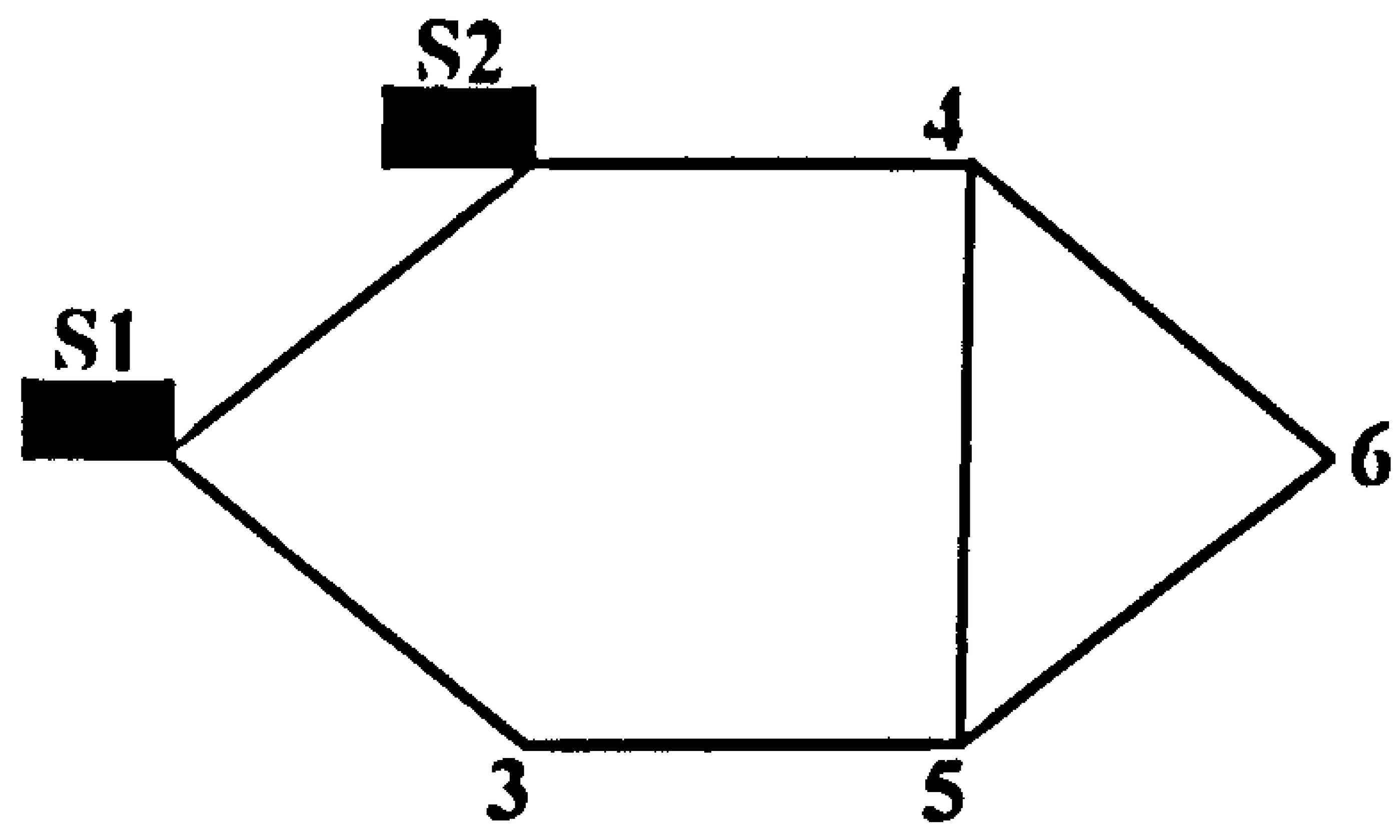

Figure 5.15 Multiple-source Network B

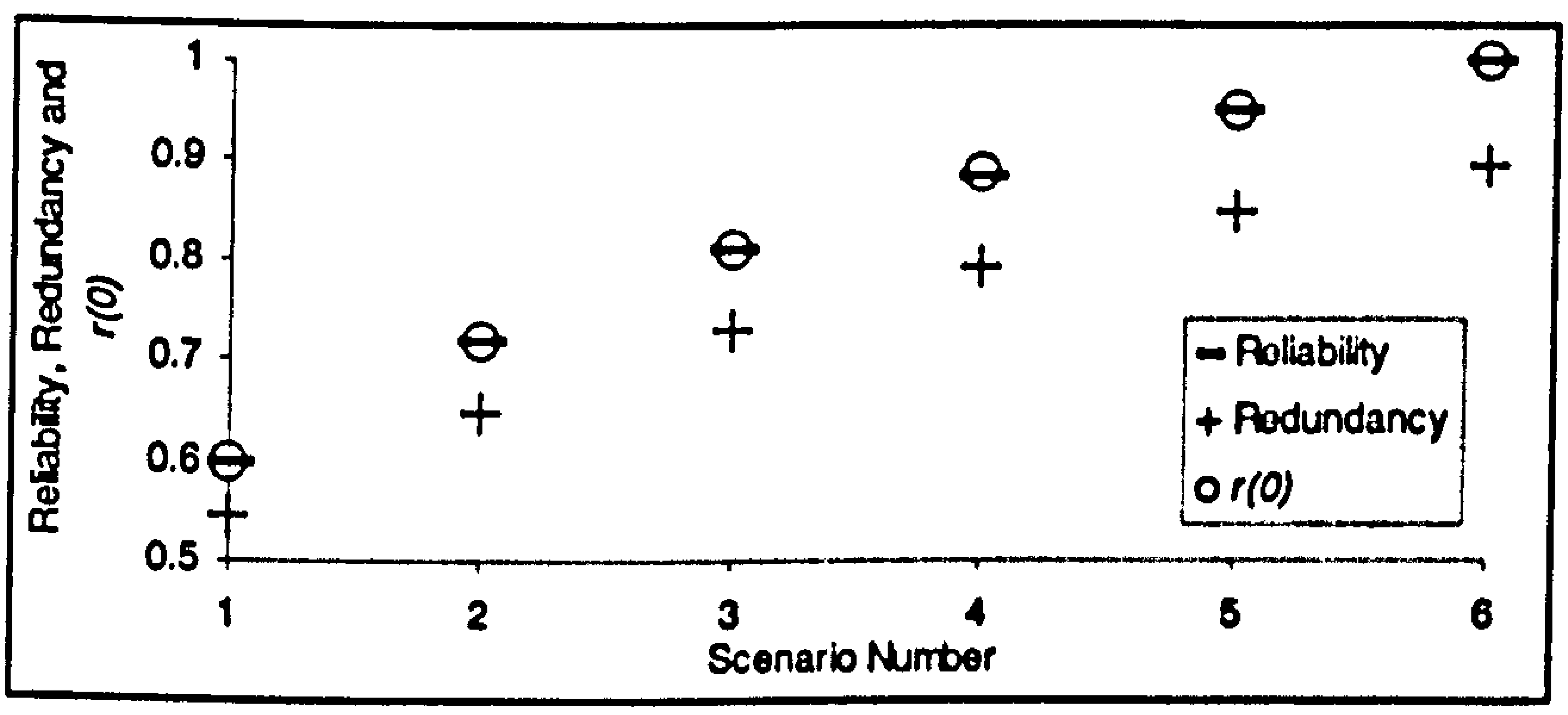

Figure 5.16 System Performance for a Range of Source Heads

Table 5.1. Pipe Data for Designs 1 - 16 (Tanyimboh and Tabesh, 1997a)

Links	Diameters (mm)															
1-2, 1-4	250	250	250	250	250	250	250	250	250	250	250	250	250	255	255	255
2-3, 4-7	175	175	180	180	180	185	185	185	190	190	190	190	190	190	190	190
2-5, 4-5	145	145	145	145	145	145	145	145	145	145	145	150	150	150	155	155
3-6, 7-8	115	115	115	120	125	125	130	135	135	140	140	140	140	140	140	140
5-6, 5-8	100	105	105	105	105	105	105	105	105	105	110	110	115	115	115	120
6-9, 8-9	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100
Designs	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**

Table 5.2. Fractions of Total Demand Satisfied by Fully Connected Network[*]

Design	Fraction of Total Demand satisfied				Error as a % age of HDSM value		
	SHM	ISHM	CHDSM	HDSM	SHM	ISHM	CHDSM
1	0.600156	0.862239	0.816062	0.826290	-27.37	4.35	-1.24
2	0.608066	0.870303	0.817531	0.831543	-26.87	4.66	-1.69
3	0.610132	0.872460	0.819512	0.832883	-26.74	4.75	-1.61
4	0.619686	0.882734	0.823518	0.837405	-26.00	5.41	-1.66
5	0.628385	0.888390	0.828208	0.837808	-25.00	6.04	-1.15
6	0.630685	0.889082	0.829938	0.839274	-24.85	5.93	-1.11
7	0.638796	0.891519	0.834262	0.842946	-24.22	5.76	-1.03
8	0.646091	0.893710	0.838107	0.846175	-23.65	5.62	-0.95
9	0.648594	0.894445	0.839837	0.847785	-23.50	5.50	-0.94
10	0.655285	0.896468	0.843345	0.850711	-22.97	5.38	-0.87
11	0.659220	0.897650	0.845363	0.851211	-22.56	5.46	-0.69
12	0.660846	0.898140	0.846468	0.852287	-22.46	5.38	-0.68
13	0.664712	0.899298	0.848438	0.853969	-22.16	5.31	-0.65
14	0.666996	0.899986	0.850120	0.855560	-22.04	5.19	-0.64
15	0.668625	0.900475	0.851177	0.856622	-21.95	5.12	-0.64
16	0.672434	0.901619	0.853099	0.859308	-21.75	4.92	-0.72
Mean Error					-24.01	5.30	-1.02

[*]*All values apart from the CHDSM results are taken from Tabesh (1998)*

Table 5.3 Four-loop Network Design 1 - Results for a Source Head of 100m[b]

		CONVENTIONAL DEMAND DRIVEN ANALYSIS			
	Demand	Total Heads (m)			
Nodes	(l/s)	NRLSA	HDSM	CHDSM	EPANET
2, 3	20.8	83.17	83.17	83.20	83.19
4, 6	20.8	57.11	57.11	57.18	57.14
5	20.8	56.78	56.78	56.86	56.82
7, 8	20.8	-20.34	-20.34	-20.11	-20.25
9	62.5	-177.63	-177.63	-177.08	-177.46

	HEAD DRIVEN ANALYSIS								
	Outflows (l/s)			Total Heads (m)			EPANET2 Output for Feasibility Check: Total Heads (m)		
Nodes	NRLSA	HDSM	CHDSM	NRLSA	HDSM	CHDSM	NRLSA	HDSM	CHDSM
2, 3	20.8	20.8	20.8	88.20	88.02	88.32	88.21	88.30	88.31
4, 6	20.8	20.8	20.8	71.36	70.94	71.67	71.38	71.63	71.64
5	20.8	20.8	20.8	71.99	71.58	72.31	72.01	72.27	72.28
7, 8	20.8	20.8	20.8	36.68	35.33	37.75	36.71	37.63	37.69
9	26.2	25.5	25.46	5.27	4.28	8.0	5.29	7.75	7.89

[b]*All values apart from the CHDSM results are taken from Tanyimboh et al. (2002)*

Table 5.4 Input Data for Multiple-source Network

LINK DATA						NODE DATA		
LINK No.	FROM NODE	TO NODE	DIAMETER (m)	LENGTH (m)	CHW	NODE No.	DEMAND (l/s)	ELEVATION (m)
1	1	2	0.150	1000	140	1	0.0	76
2	1	3	0.300	1000	140	2	0.0	54
3	2	4	0.250	1000	140	3	-41.7	30
4	3	4	0.250	1000	140	4	-77.8	25
5	3	5	0.250	1000	140	5	-55.6	25
6	4	6	0.200	1000	140	6	-88.9	26
7	5	6	0.200	1000	140			

Table 5.5 Hydraulic Feasibility Verification for Multiple-source Network

	DDA		CHDSM			EPANET2	
NODE No.	TOTAL HEAD (m)	RESIDUAL HEAD (m)	OUTFLOW (l/s)	TOTAL HEAD (m)	RESIDUAL HEAD (m)	TOTAL HEAD (m)	RESIDUAL HEAD (m)
1	76.0	0.00	230.73	76.00	0.00	76.00	0.00
2	54.0	0.00	19.75	54.00	0.00	54.00	0.00
3	55.24	25.24	-41.7	56.37	26.37	56.36	26.36
4	48.54	23.54	-77.8	49.72	24.72	49.72	24.72
5	43.32	18.32	-55.6	45.79	20.79	45.78	20.78
6	36.77	10.77	-75.38	41.00	15.00	41.00	15.00

Table 5.6 Serial Network Performance Results

Design	Source Head (m)	Total Outflow (m^3/min)	Network Reliability	Component Failure Tolerance	Network Mechanical Reliability
1	88.87	2.420	0.219930	0.063723	0.999324
2	90.88	4.340	0.394548	0.222227	0.999324
3	91.96	6.202	0.563763	0.263798	0.999324
4	92.33	6.629	0.602593	0.268471	0.999324
5	98.5	8.000	0.726996	0.324634	0.999324
6	98.84	8.000	0.726996	0.324634	0.999324
7	110.89	11.000	0.999543	0.324634	0.999324

Table 5.7 Candidate Designs for Two-loop Network (Tanyimboh, 1993)

Links	Diameters (mm)					
1-3	401	401	390	384	365	367
2-4	100	100	165	191	238	235
3-5	338	337	337	329	281	294
4-6	100	100	100	151	250	234
5-6	263	262	262	249	152	185
1-2	157	165	203	224	263	261
3-4	237	237	213	215	247	234
Designs	1	2	3	4	5	6

Table 5.8 Two-loop Network Performance Results

Design No.	Outflow for Fully Connected Network (l/s)	*p(0)*	Network Reliability	Component Failure Tolerance
1	283.59	0.997430	0.998188	0.861541
2	283.74	0.997452	0.998750	0.861166
3	283.79	0.997805	0.998923	0.839128
4	283.90	0.998147	0.999322	0.819392
5	284.00	0.998271	0.999662	0.804619
6	284.00	0.998329	0.999666	0.800285

Table 5.9 Performance Results for Multiple-source Network B

Scenario Number	Source Head for S1 (m)	Source Head for S2 (m)	Total Network Outflow (l/s)	Reliability	Redundancy	*r(0)*
1	28.00	20.00	56.85	0.598029	0.546088	0.5984
2	38.00	30.00	68.18	0.717191	0.648008	0.7177
3	48.00	40.00	76.830	0.808169	0.727531	0.8087
4	58.00	50.00	84.160	0.885234	0.792237	0.8859
5	68.00	60.00	90.230	0.949177	0.847578	0.9498
6	78.00	70.00	95.000	0.999309	0.893624	1.0000

CHAPTER SIX

INTRODUCTION TO THE ANALYTIC HIERARCHY PROCESS

6.1 INTRODUCTION

The long-term upgrading of a distribution network should address network economics, technical issues, social and environmental issues as mentioned earlier in Chapters 1, to 5. This is a multi-objective decision-making problem with a high level of complexity, involving both qualitative and quantitative decision factors. This chapter gives an introduction to the module for multiple-criteria decision-making in the integrated model. The chapter starts off with a review of some methods used for multiple-criteria decision-making and then concentrates on a versatile method called the Analytic Hierarchy Process (AHP) (Saaty, 1980). Since most multiple-criteria decision-making methods depend upon the judgment of different individuals, the issue of consensus of judgment and how it is achieved has also been presented in this chapter.

In the AHP, a group of individuals or stakeholders of a project decompose information into a hierarchy of criteria, sub-criteria, and alternatives. The information is then synthesized using pair-wise comparison matrices whose elements show the relative importance of decision factors for each level of the hierarchy, to determine the relative rankings and priority vectors of alternatives (Saaty, 1980). The AHP is popular because it can be used to establish measures in both the physical and social domains. Measurable and subjective factors can be effectively combined in the decision-making process using informed judgments to derive weights and priorities. A notable feature of the AHP is the ability to measure and handle inconsistency of pair-wise comparisons. The AHP allows the models and results to be clearly explained to decision-makers and stakeholders who may have a non-technical background. This provides a more transparent process for project decision-making.

AHP models are applicable to decision-making problems in corporate and governmental settings. The AHP has been applied as a decision support tool and produced good results for examples in fields such as economics, politics, marketing, engineering, etc. (Saaty and Vagras, 1994; 2001). Saaty (1980) has also applied the AHP for examples on infrastructure planning or to decide on the best project or best strategy out of alternatives.

6.2 MULTIPLE-CRITERIA DECISION-MAKING

There are several decision problems that involve a number of objectives. In some cases, these objectives tend to conflict each other. For example, the optimal design of water distribution systems with the goal of minimising cost could end up compromising the reliability of the network (Templeman, 1982b). A road project may have the economic advantage of reducing the transport costs of raw materials, but may end up having an adverse impact on the environment. Realistic models for decision-making should include and measure important quantitative and qualitative factors and to strike a compromise between conflicting decision factors. The tendency is to put monetary values to qualitative factors but it is not easy to cost social and environmental factors.

There are various methods for multiple-criteria decision-making. In general, given various alternatives and decision factors to serve as a basis of selecting the best alternative, there are three main steps involved in the multiple-criteria decision-making process (Canter, 1996). These are

1) Determination of importance weights for each of the decision factors.
2) Ranking or rating of each alternative with respect to each decision factor or criterion.

3) Development of a decision matrix whose elements are the products of the importance weights from Step 1 and the alternative ranks from Step 2. The best alternative is selected on the basis of this decision matrix.

This section presents examples of methods that can be used for multiple-criteria decision-making. A method called the weighted ranking technique (Ahmad, 1985) is presented first. This is followed by a paired comparison technique by Dean and Nishry (1965). Paired-comparison techniques (unranked and ranked) have been extensively used in decision-making efforts especially in environmental impact studies. They involve a series of comparisons between decision factors to form comparison matrices, and a systematic tabulation and manipulation of these matrices to facilitate the decision-making process. (Canter, 1996).

6.2.1 Weighted Ranking Technique

This is a simple method that can facilitate the decision-making process e.g., selection of the best project with due consideration of costs and the environmental impact (Ahmad, 1985). Minimising costs and ensuring that a project has no adverse impact on the environment are criteria that may conflict each other and this method can effectively be used to reconcile these factors. The weighted ranking technique (Ahmad, 1985) involves assigning importance weights to the decision criteria, ranking of alternatives with respect to each of the decision factors and finally obtaining the ranking of project options or alternatives. An example for demonstrating this technique is presented next.

6.2.1.1 Example 1

Assume that Option 1, Option 2 and Option 3 are three design alternatives or options for a project. Let the DF1, DF2 and DF3 be the decision factors e.g. project costs, social impact and environmental impact. The results and data for this example are shown in Table 6.1. The first step is to assign importance weights to the decision factors such that the sum of the weights is 1. DF3 is the most important factor with a weight of 0.5, and DF1 has the lowest importance with a weight of 0.2. The options are then ranked as shown in the table with respect to each criterion; the best or most preferred being assigned a rank of 1, the second a rank of 2 and a rank of 3 to the third most preferred option. If two options have almost equal importance or are equally preferred with respect to a given decision criterion, they are assigned a mid-rank value. For example, if the two options that are equally preferred were to have the second and third ranks, the mid-rank of 2.5 is assigned to each option. Options 1 and 2 have equal importance with respect to DF2 and they are each assigned a value of 1.5.

The weighted ranks of the Options with respect to each of the decision factors are obtained as products of the ranks and the respective importance weights. These products are summed for each option to obtain the total score. The best option is the one with the lowest total score. Option 2 with the lowest total score of 1.55 is therefore the best option.

One of the weaknesses of this method is in effort required by the decision-makers to reach a consensus on the importance weights of the decision factors. These weights have a large influence on the final decision. The Delphi technique (Linstone and Turoff, 1975), which is presented later in Section 6.4, may be used to assist in the process of getting a consensus. However, this technique, though effective, requires a considerable amount of time to achieve a consensus and this time increases as the decision factors and design alternatives increase. Also, the ranking does not reflect the relative difference between two options with respect to a particular criterion, especially if one option is strongly preferred compared to another.

6.2.2 Unranked Paired-comparison Technique

Dean and Nishry (1965) developed a useful technique for multiple-criteria decision-making called the unranked paired-comparison technique. This technique may be used an interdisciplinary group of individuals or by a single individual to make pair-wise comparison of each factor relative to every other factor. A value of 1 is assigned to the factor considered to be more important and 0 to the less important factor in the weighting process. If the decision factors are considered to be equally important, they are each assigned a value of 0.5. This method is demonstrated next in an example.

6.2.2.1 Example 2

Assume that AL1, AL2 and AL3 are three possible design alternatives for a project. Let the DF1, DF2, DF3 and DF4 be the decision factors for the project. The first step is to obtain the factor-importance coefficients (FIC) (Dean and Nishry, 1965) which are the importance weights of each of the factors, by making unranked paired-comparisons of the decision factors relative to each other as shown in Table 6.2. For example, the element in the first row and the third column of the comparison matrix in this figure has a value of 1, meaning that DF1 is more important than DF3. DF5 is a dummy factor that is considered to be less important than each of the factors DF1 to DF4 and always has a value of 0. Thus, the dummy factor is included so as to rule out the net assignment of a value of 0 to any of the basic factors DF1 to DF4. Values in each row are summed and the factor of importance coefficient (FIC) for each decision factor is calculated as the sum of the values in its row, normalised over the total of all the row sums as shown in Table 6.1. The FIC values show that DF1 and DF2 are equally the most important factors followed by DF3 and DF4.

The second step involves the rating of alternatives with respect to each individual decision factor or the assignment of relative desirability. Table 6.3 shows the rating of alternatives AL1, AL2 and AL3 with respect to DF1. For example, the element in the first row and the second column of the matrix of relative desirability in this table has a value of 1, meaning that AL1 is more important than AL2 with respect to DF1. AL4 is the dummy variable and each alternative AL1, AL2, and AL3 is assigned the value of 1 when compared to it. The ranking of alternatives with respect to DF1 or the alternative choice coefficient (ACC) (Dean and Nishry, 1965) is determined in a similar manner as FIC. The ACC value of each alternative is its row sum divided by the sum of all the elements in the matrix of relative desirability. From the ACC values in Table 6.3, alternative AL3 is the most desirable followed by AL1 and AL2 is the least desirable. This process is repeated for DF2 with results as shown in Table 6.4 and the same applies to DF3. The summary of the FIC and ACC values obtained from this process are in Table 6.5.

The final step in the unranked paired comparison technique is to develop a decision matrix comprising of the products of factor-importance coefficients and the alternative choice coefficients (FIC x ACC). The final product matrix is shown in Table 6.6. Summation of these products in each column for each alternative indicates that AL3 with the highest total score of 0.375 would be the best choice, followed by AL1 and AL2.

There is no doubt that this method is a useful and simple approach to multiple-criteria decision-making in that it shows the significant differences between alternatives, and this eases the process of making the best choice. It also employs the binary scale of 1 and 0 for comparison of alternatives that is straightforward to apply in that, for a pair of decision factors to be compared, a factor is more important, less important or equally as important as the other factor. However, the binary scale is rather limited in reflecting the degree of relative importance of one decision factor over another especially when one factor is much more important than another.

The inherent weakness in the process of ranking or rating due to the subjective judgments and the degree of simplification of the above methods leading to a possibility of reducing the accuracy of the results on which the decision is based, are issues that require attention. These issues may be addressed by ensuring that the method selected for multiple-criteria decision-making pays special attention to the process of achieving consensus of judgments for importance weighting (e.g., by use of the Delphi technique) and that a sensitivity analysis is carried out to assess the influence of different perspectives on the overall decision (Canter, 1996).

A method that addresses the above weaknesses is presented shortly in Section 6.3. This method is a ranked paired-comparison technique called the analytic hierarchy process (Saaty, 1980). Ranked paired-comparison techniques involve an initial rank ordering of all decision factors. This is followed by the process of ranking the design options or alternatives and making comparisons between them with respect to each of the decision factors. The results of these comparisons are then used to obtain the overall ranking of the options (Canter, 1996).

Section 6.3 begins with the concept of formation of a hierarchy, followed by priority in a hierarchy and the process of revising judgements to avoid inconsistent comparison matrices. The analytic hierarchy process (AHP) is the method that has been adopted for this research on the optimal long-term upgrading of water distribution networks and there are a number of reasons for this choice. Unlike some of the other expert systems or models for multiple-criteria decision-making, it does not require a lot of time especially during the process of obtaining a consensus on judgments and it can be used to compare non-commensurate objectives effectively. The AHP model can be developed with limited data to evaluate objectives. Considering the subjectivity of judgments in pair-wise comparisons, the main strengths of the AHP lie in its provision for measuring the inconsistency of these comparisons and the revision of judgements to ensure that consistency is maintained. Section 6.4 then follows with the main emphasis being placed on the issue of consensus of judgments of different individuals in the process of determining the importance weighting of the decision factors.

6.3 THE ANALYTIC HIERARCHY PROCESS

The AHP is a basic approach to decision-making that can be used to determine the best outcome when difficulty exists assigning evaluations and weights to decision factors. The process involves the making of pair-wise comparison judgments, which are used to develop overall priorities for ranking project alternatives (Saaty and Vargas, 1994).

The analytic hierarchy process comprises of the following general steps:

1) Break the problem down into decision elements (levels) in the form of a hierarchy.
2) Obtain priority vectors or the ranking of the decision factors and an overall priority matrix that comprises of a combination of the priority vectors for the alternatives with respect to each of the decision factors. The priority vector for the decision factors is obtained by manipulation of a matrix referred to as a comparison matrix that is formed as a result of

pair-wise comparisons of these factors based on a predefined comparison scale. A check for consistency of comparison judgments is made at this stage and non-compliance means that revision of judgments is required. Priority vectors are then obtained for the design options with respect to each decision factor in a similar manner and aggregated into an overall priority matrix.

3) Multiply the overall priority matrix by the priority vector of decision factors to obtain the final ranking of options.

6.3.1 Formation of a Hierarchy

With the AHP, goals, criteria, and alternatives are arranged in gradual steps or a hierarchical structure similar to a family tree (Saaty, 1980). The hierarchical structure for the optimal long term upgrading of water distribution systems that has been adopted for this study comprises of 4 levels and is presented in Figure 6.1. The top level consists of the ultimate goal or decision, the second level is the criteria by which options or alternatives are evaluated, the third level consists of the sub-criteria for each criterion, and the fourth level consists of the various possible design options, scenarios or alternatives. The main advantage of using the hierarchy is that it makes it easier for one to compare options with respect to a given decision criterion if there are finer divisions such as sub-criteria involved. For example, the decision criterion of socio-economic impacts of a project could be sub-divided into sub-criteria such as employment opportunities created, project costs, etc. The cheaper alternative for the project may not necessarily create more job opportunities. Such a detail is best exposed by the hierarchy approach.

6.3.2 Priority in the Hierarchy

6.3.2.1 Comparison Matrix Formation

In the AHP, after the creation of a hierarchy or network to represent a decision a matrix of pair-wise comparison judgments is established for the elements linked under a parent element. Judgments are used to determine the ranking of the sub-objectives and criteria. A comparison scale (Table 6.7) is used for the pair-wise comparisons, to express the relative importance of one sub-objective (or criterion) over another. This scale of 1 – 9 has been conveniently chosen in this research because of the following reasons (Saaty, 1990):

First, the five main attributes of relative importance (equal, weak, strong, very strong, and absolute) coupled with the intermediate attributes as shown in Table 6.7, give a total of 9 attributes that effectively cover one's ability to make qualitative distinctions.

Secondly, these attributes are practical and have an element of accuracy when comparisons are made between items of the same order of magnitude with regard to the property used to make the comparison.

Generally, paired comparisons involve the determination of how many times a dominant element is a multiple of the less dominant one taken as the unit of measurement. The less dominant element therefore has a reciprocal relationship to the more dominant one. The process of developing a matrix of comparisons is presented next in mathematical terms.

For a given set of activities $E_1, E_2, \dots, E_n$; quantified judgment on pairs of activities E_j, E_k may be represented by an n-by-n matrix.

$$\mathrm{A} = (a_{jk}) \qquad (j, k = 1, 2, \dots, n) \tag{6.1}$$

The elements of this matrix are such that

$$a_{jk} = \frac{1}{a_{jk}} \tag{6.2}$$

If E_j is judged to be of equal importance as E_k, then $a_{jk} = a_{kj} = 1$. This matrix is referred to as the matrix of comparisons and it has the form

$$\mathrm{A} = \begin{bmatrix} 1 & a_{12} & \cdots & a_{1n} \\ 1/a_{12} & 1 & \cdots & a_{2n} \\ \vdots & \vdots & \ddots & \vdots \\ 1/a_{1n} & 1/a_{2n} & \cdots & 1 \end{bmatrix} \tag{6.3}$$

Assume the judgments are actual precise physical measurements, for example, weights $w_1, w_2, \dots, w_n$, of n activities $E_1, E_2, \dots, E_n$, respectively. Let E_l weigh w_l =

420kg and E_2 weigh $w_2 = 300$kg, then, taking $w_1/w_2 = 1.4$, it follows that E_1 is 1.4 times as heavy as E_2. Thus in this case of exact measurements, the relations between the weights and the judgments a_{jk} are given by (Saaty, 1980)

$$a_{jk} = \frac{w_j}{w_k} \qquad (\forall j, k = 1,2,\ldots,n) \tag{6.4}$$

and the matrix of comparisons is

$$A = \begin{bmatrix} w_1/w_1 & w_1/w_2 & \cdots & w_1/w_n \\ w_2/w_1 & w_2/w_2 & \cdots & w_2/w_n \\ \vdots & \vdots & \ddots & \vdots \\ w_n/w_1 & w_n/w_2 & \cdots & w_n/w_n \end{bmatrix} \tag{6.5}$$

Saaty, (1980) cautioned against the use of actual quantities obtained from measurement to form the comparison matrix instead of using the comparison scale to form a judgment matrix and compute the eigenvector. The reason is this process can lead to error because relative measurements according to a judge may not necessarily be reflected by their ratios, i.e., 2kg or 3kg may be deemed to have approximately the same weight to a heavyweight lifter yet their ratio shows a significant difference.

Table 6.8 is used to demonstrate the assembling of a paired comparison matrix for four activities *E*, *F*, *G* and *H*. Every element compared to itself is equally important and therefore the leading diagonal elements of the matrix each have a value of 1. Assume *E* has a strong to very strong importance (Table 6.7) compared to *F*, comparison matrix element position (*E*, *F*) takes on the value of 6 and position (*F*, *E*) takes on the reciprocal, 1/6. Similarly, all the other judgments can be made using the comparison scale in Table 6.7 to complete the comparison matrix assembled as shown in Table 6.8.

6.3.2.2 *Computation of the Vector of Priorities*

In comparing sub-objectives and criteria that have non-unique scales, unitless and abstract priorities derived from the pair-wise comparisons are used. The matrix of these pair-wise comparisons is then transformed into a ranking or priority vector of the sub-objectives (or criteria) by using the eigenvector solution (Saaty, 1990).

To compute the vector of priorities, the elements of each column in the comparison matrix are divided by the sum of that column. The average of the elements in each row of the resulting matrix is calculated and averaged to give the column vector of priorities. Thus, applying this method to the comparison matrix (A1) in Table 6.8, every element in each of the columns is divided by the sum of the elements of that column to obtain the normalised comparison matrix (A2) shown in Table 6.9. The elements of each row of Matrix A2 are then summed and averaged to give the column vector of priorities (0.606, 0.241, 0.110, 0.042), referred to herein as Vector V1.

Priority vectors are obtained at different levels in the hierarchy. Starting with level 3 the ranking or priority vectors of the sub-criteria are obtained with respect to the various options or alternatives. These subordinate priority vectors are combined together to form the respective priority matrix for a given sub-criterion. The product of this priority matrix and the importance weighting or the priority vector of the sub-criteria yields the level 2 ranking of options with respect to the criterion. The same calculations are repeated for all the other sub-criteria. Details of these calculations are in Chapter 7.

6.3.2.3 Computation of Consistency

The AHP has a provision for checking that the matrix of comparisons is consistent. In its simplest form, consistence ensures that comparison judgments made are logically correct. For example, if activity F has been judged to be better than G; and E better than F, it follows that E has to be better than G. However, it may not always be possible to maintain this level of consistence when developing the comparison matrices. First of all, from a mathematical point of view physical measurements are never exact; hence, allowance must be made for deviations. Secondly, these deviations increase considerably when dealing with human judgments (Saaty, 1980).

To check the consistence of comparison matrices, the matrix of comparisons is multiplied by the priority vector to give a second column vector. On dividing the first component of the second vector by the first component of the priority vector, the second component of the second vector by the second component of the priority vector, and so on, a third vector is obtained. The average of the components of the third vector gives an approximation to the maximum or principal eigenvalue, λ_{max} (Saaty, 1980). Since small changes in a_{jk} imply a small change in λ_{max}, the deviation of the latter from n is a measure of consistency. The consistency index or the "closeness to consistency", can be written as

$$C.I. = \frac{\lambda_{max} - n}{n - 1} \tag{6.6}$$

Thus, the closer λ_{max} is to n the number of components or order of the matrix, the more consistent the matrix is deemed to be.

The consistence index of a randomly generated reciprocal matrix is referred to as the random index (R.I.) (Saaty, 1980); and Table 6.10 shows average R.I. values that were generated using matrices of the order 1 – 15. Saaty (1980) asserts that for a given matrix of order n, the ratio of C.I. to the average R.I. value is called the consistency ratio (C.R.) and that the comparison judgments may be considered satisfactory if a ratio of 0.10 or less is obtained.

To illustrate these calculations, the consistency ratio of the comparison matrix (A1) in Table 6.8 is obtained as follows. The product of the comparison matrix (A1) and the priority vector (V1) gives another column vector (3.163, 1.189, 0.457, 0.175) referred to herein as Vector V2. Dividing the corresponding components of Vector V2 by the priority vector (V1) gives another column vector (5.216, 4.933, 4.146, 4.124) referred to herein as Vector V3. The average of the elements of Vector V3 gives an approximation to the principal eigenvalue, λ_{max}, as 4.605. The consistency index, C.I. = (4.605 – 4)/(4 – 1) = 0.202.

To determine how good the comparison judgments are, C.I. is divided by the corresponding value of R.I., which is 0.90 for $n = 4$ (Table 6.10). This yields a consistency ratio, C.R. = 0.202/0.90 = 0.224, which is perhaps not as close as required since it is greater than 0.1. Thus, the revision of judgments has to be done. The method of revising judgments shall be presented in Sub-section 6.3.3.

6.3.2.4 Computation of the Overall Priority Vector

The various level 2 option rankings form an overall priority matrix at level 1. The product of this overall priority matrix and the importance weighting or the priority vector of the main decision criteria at level 1 yields the overall priority vector or the overall ranking of options for the project. It is on the basis of this overall ranking of options that the final decision is made. Details of these calculations are in the next chapter.

6.3.3 Revising Judgments

Saaty (1980) states that there are two ways of revising judgments, assuming that the consistency ratio of a matrix of comparisons with elements a_{jk} and a vector of priorities ($w_1, w_2, w_3, \ldots, w_n$), is large enough to warrant revision. The first is to form the matrix of ratios of priorities w_j/w_k just as in Eq. (6.5), form the matrix of absolute differences $\left|a_{jk} - (w_j/w_k)\right|$ and revise the judgment of the element(s) or row sums with the largest such differences. The second and more attractive approach is to form the root mean square deviation using the rows of (a_{jk}) and (w_j/w_k) and revise the judgments for the row with the largest value. The aim is to have an iterative procedure that converges as the difference between corresponding elements a_{jk} and w_j/w_k becomes very small. Thus, the procedure involves replacing all a_{jk} in the row with the largest root mean square deviation value by the corresponding w_j/w_k and recalculating the priority vector. Repetition of this process produces convergence to the consistent case. Saaty (1980) noted that in some instances, w_j/w_k could be greater than the scale's upper limit value of 9 and that this should not cause any alarm.

To illustrate this procedure, the matrix of comparisons (a_{jk}) in Table 6.8 has been used in this study. The consistency ratio for this matrix is 0.224 (Sub-section 6.3.2.3), which is greater than the allowable value of 0.1 and would therefore require revision of judgments. The vector of priorities (w_1, w_2, w_3, w_4) for this matrix is (0.606, 0.241, 0.110, 0.042), and $\lambda_{max} = 4.60$. A matrix of ratios of priorities is formed corresponding to w_j/w_k as shown in Table 6.11. The matrix of absolute differences $\left|a_{jk} - (w_j/w_k)\right|$ is also formed as shown in Table 6.12. The largest absolute difference between corresponding elements a_{jk} and w_j/w_k is between a_{14} and w_1/w_4 = 6.32. Thus, using the first method, element a_{14} is replaced with w_1/w_4 = 14.32. The vector of priorities is recomputed as (0.649, 0.223, 0.097, 0.030), λ_{max} = 4.389 and a consistency ratio, C.R. of 0.14 is obtained. This is the end of the first iteration and the C.R. obtained, though lower than the initial one of 0.224, is still greater than 0.1 and thus another iteration of the above process is required.

Using the second method, the vector of root mean square deviations for the rows is (3.686, 1.554, 1.206, 0.098). The first row has the highest value of 3.686 therefore the judgments of this row should be revised by replacing the elements a_{1k}, for k = 1, 2, 3 and 4, with the corresponding w_1/w_k values. The vector of priorities is recomputed as (0.563, 0.296, 0.108, 0.033), λ_{max} = 4.182 and a consistency ratio, C.R. of 0.067 is obtained. Since this value is less than 0.1, the comparison judgments are deemed to be satisfactory.

The second method gives better results than the first one since the revision of judgments is more spread out and not restricted to just one element. Saaty, (1980) cautioned against excessive use of this process of forcing the values of judgments to improve, noting that it could lead to distortion of the results. He further asserted that improving judgments in the original comparison matrix using the experience of the judge would probably be a better approach.

6.4 CONSENSUS ON JUDGMENTS

The judgment of different individuals is a crucial factor for most multiple-criteria decision-making methods. It is imperative therefore that consensus on the judgments made is achieved in order for the best results to be obtained. This section presents a brief description of a method that is widely used for obtaining consensus on judgments, followed by the approach to consensus used for the AHP.

6.4.1.1 The Delphi Technique

The Delphi technique (Linstone and Turoff, 1975) is a structured approach for achieving group consensus about common issues of concern. It is used to assist in the process of achieving consensus for numerous methods of multiple-criteria decision-making, e.g., the weighted ranking technique (Ahmad, 1985) and the unranked paired-comparison technique (Dean and Nishry, 1965). The technique has broad versatility and the results are easy to understand. Application of the method requires a considerable amount of time ranging from hours to days. Depending on the complexity of the problem, the technique may require a substantial effort in computer data analysis (Canter, 1985).

A Delphi study is conducted by a group of judges (experts and stakeholders) headed by a study director. Questionnaires are prepared and anonymous responses are made and returned to the director by each member of the group of judges to avoid disproportionate influence of strong personalities. The study director designs the questionnaire sets and makes the choice of variables involved. For example, the questionnaire could be designed with the objective of identifying decision factors for a project or determining the relative importance weights of the decision factors. A number of rounds of questionnaires are used in the study. At the end of each round, the study director compiles, edits the information received from the judges and carries out a statistical analysis to determine the median (point of consensus) and the spread of opinion expressed by the inter-quartile range (IQR), which encompasses 25 percent of the responses above and below the median. The judges are given a feedback of the results (median, IQR, and reasons for their distribution) to consider revising their judgments towards the majority of the group. Canter (1985) noted that it may be necessary to repeat the process for a minimum of four rounds to give minority opinions a chance of shifting the median and IQR.

The Delphi technique has many other advantages. The chance of bias due to variation of the questionnaire is minimal, since the same questionnaire is circulated to all panellists. Experts or panellists are not subject to repetition of arguments or dominance by others because they work independently. The anonymity resulting from the statistical analysis removes the pressure to conform and allows individuals to change their minds without embarrassment.

However, the method has some disadvantages too. Helmer (1966) noted that some questions on the questionnaires could be ambiguous and that there was a tendency of some judges to respond to few of the questionnaires and drop out on subsequent rounds. Substitution of these judges may cause some instability of the results. The method requires a lot of time for its implementation and the fact that the study director designs the questionnaires may lead to a possibility of a given design option being favoured by the structure of the questionnaire.

6.4.1.2 The AHP Approach to Achieving Consensus on Judgments

Seeking consensus using the AHP requires a group of individuals and stakeholders for a particular project to assist in selecting the criteria and making the judgments in an interactive forum. Depending upon the subject, a well-informed person can influence the judgment of one who has less information, thus interaction should help bring judgments closer. The approach to consensus adopted by the AHP (Saaty, 1980) involves deriving priority weights for the individual judges according to the soundness of their judgment with respect to the key factors that affect judgment. Examples of such factors include relative intelligence, experience in related projects, past record, depth of knowledge and personal involvement in the issue at stake, to mention but a few. The priority derived is used to weight the final priority result derived from the judgment of each individual and an overall weighted priority for ranking the alternatives of the project is then obtained in the usual way.

There are several advantages of this approach to consensus. It is not time consuming since it uses dynamic discussion in constructing the hierarchy and providing judgments by mutual agreement, and for revision of views. The variables to influence the judgment are chosen by the group and they may ignore some later in the procedure if they have very low priorities assigned to them. The AHP does not require rigorous statistical analyses but instead uses absolute numbers from 1 to 9 reflecting qualitative judgment on pair wise comparison.

6.5 CONCLUSION

This chapter has dealt with the key issues of decision-making involving a combination of various quantitative and qualitative factors. Different techniques of multiple-criteria decision-making have been discussed with emphasis on the analytic hierarchy process that has been selected for use in this research.

Using the AHP model facilitates the task of making the best assignments of importance to all factors and synthesizing this diverse information to make the best decision. The hierarchy approach breaks down the problem into elementary components (criteria and sub-criteria) making it easier for one to compare different options with respect to a specific criterion. This enables the AHP to effectively reflect the degree of relative importance of one decision factor over another especially when one factor is much more important than another.

The approach used for the AHP with respect to consensus of comparison judgments is very reliable in that it reduces disagreements in an open discussion and saves a lot of time in executing the method. The allocation of importance weights to the judges in order to reflect their impact on the overall decision is another attractive feature of the AHP. Further to this, the subjectivity of comparison judgments is well handled by a technique for measurement of inconsistency and revision of judgements to ensure that satisfactory comparison judgments are utilised throughout the process.

The AHP is a highly effective method that can be used even when data is limited and it yields simple results that can easily be explained to decision-makers who may not necessarily have a technical background.

In the next chapter, the application of the AHP model for decision-making in the process of selecting the best design for a hypothetical water distribution network shall be described using a step-by-step approach.

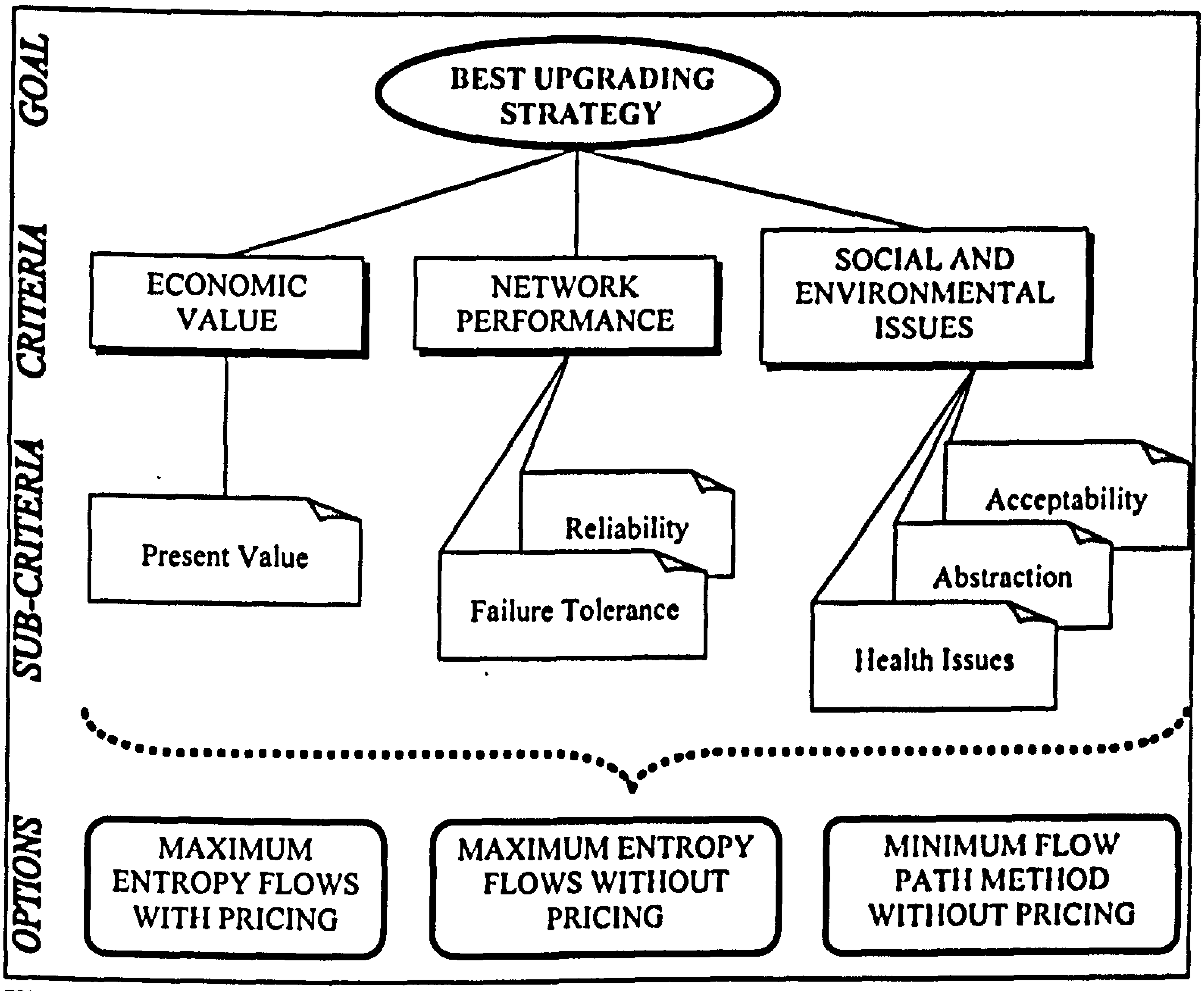

Figure 6.1 Hierarchical Structure

Table 6.1 Weighted Ranking Technique

DESIGN	DF1 (*WEIGHT=0.2*) RANK	DF1 WEIGHTED RANK	DF2 (*WEIGHT=0.3*) RANK	DF2 WEIGHTED RANK	DF3 (*WEIGHT=0.5*) RANK	DF3 WEIGHTED RANK	TOTAL SCORE
OPTION 1	2	0.4	1.5	0.45	3	1.5	2.35
OPTION 2	3	0.6	1.5	0.45	1	0.5	1.55
OPTION 3	1	0.2	3	0.9	2	1	2.1

Table 6.2 Unranked Paired-comparison Technique: Assignment of Importance Weights

DECISION FACTOR	RELATIVE IMPORTANCE WEIGHT ASSIGNMENT DF1	DF2	DF3	DF4	DF5	SUM	FIC
DF1	-	0	1	1	1	3	0.300
DF2	1	-	1	0	1	3	0.300
DF3	0	0	-	0.5	1	1.5	0.150
DF4	0	1	0.5	-	1	2.5	0.250
DF5	0	0	0	0	-	0	0.000
TOTAL						10	1.000

Table 6.3 Unranked Paired-comparison Technique: Rating of Alternatives Relative to DF1

ALTERNATIVE	ASSIGNMENT OF DESIRABILITY AL1	AL2	AL3	AL4	SUM	ACC
AL1	-	1	0	1	2	0.333
AL2	0	-	0.5	1	1.5	0.250
AL3	1	0.5	-	1	2.5	0.417
AL4	0	0	0	-	0	0.000
TOTAL					6	1.000

Table 6.4 Unranked Paired-comparison Technique: Rating of Alternatives Relative to DF2

ALTERNATIVE	ASSIGNMENT OF DESIRABILITY AL1	AL2	AL3	AL4	SUM	ACC
AL1	-	1	1	1	3	0.500
AL2	0	-	1	1	2	0.333
AL3	0	0	-	1	1	0.167
AL4	0	0	0	-	0	0.000
TOTAL					6	1.000

Table 6.5 Unranked Paired-comparison Technique: FIC and ACC Values

DECISION FACTOR	FIC VALUES	ACC VALUES, BY ALTERNATIVE AL1	AL2	AL3
DF1	0.300	0.333	0.250	0.417
DF2	0.300	0.500	0.333	0.167
DF3	0.150	0.250	0.250	0.500
DF4	0.250	0.167	0.333	0.500

Table 6.6 Unranked Paired-comparison Technique: Matrix for Overall Ranking of Alternatives

DECISION FACTOR	FIC x ACC, FOR EACH ALTERNATIVE AL1	AL2	AL3
DF1	0.100	0.075	0.125
DF2	0.150	0.100	0.050
DF3	0.038	0.038	0.075
DF4	0.042	0.083	0.125
TOTAL SCORE	0.329	0.296	0.375

Table 6.7 Comparison Scale for AHP[a]

NUMERICAL RATING	RELATIVE IMPORTANCE	DETAILED EXPLANATION
1	Equal importance	Two activities contribute equally to the objective
3	Weak importance of one over another	Experience and judgement slightly favour one activity over another
5	Essential or strong importance	Experience and judgement strongly favour one activity over another
7	Very strong or demonstrated importance	An activity is strongly favoured over another; its dominance demonstrated in practice
9	Absolute importance	The evidence favouring one activity over another is of highest possible order of affirmation
2, 4, 6, 8	Intermediate values between adjacent scale values	When compromise is needed

[a] *Adopted from Saaty (1980)*

Table 6.8 Comparison Matrix

	E	*F*	*G*	*H*
E	1	6	7	8
F	1/6	1	5	7
G	1/7	1/5	1	5
H	1/8	1/7	1/5	1

Table 6.9 Normalised Comparison Matrix

	E	*F*	*G*	*H*
E	0.70	0.82	0.53	0.38
F	0.12	0.14	0.38	0.33
G	0.10	0.03	0.08	0.24
H	0.09	0.02	0.02	0.05

Table 6.10 Average Random Index Values[b]

Matrix order	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
Average Random Index	0.00	0.00	0.58	0.90	1.12	1.24	1.32	1.41	1.45	1.49	1.51	1.48	1.56	1.57	1.59

[b] *Adopted from Saaty (1980)*

Table 6.11 Matrix of Ratios of Priorities, (w_j/w_k)

	E	*F*	*G*	*H*
E	1.00	2.51	5.50	14.32
F	0.40	1.00	2.19	5.69
G	0.18	0.46	1.00	2.60
H	0.07	0.18	0.38	1.00

Table 6.12 Absolute Differences, $|(a_{jk} - w_j/w_k)|$

	E	*F*	*G*	*H*
E	0.00	3.49	1.50	6.32
F	0.23	0.00	2.81	1.31
G	0.04	0.26	0.00	2.40
H	0.06	0.03	0.18	0.00

CHAPTER SEVEN

APPLICATION OF THE ANALYTIC HIERARCHY PROCESS

7.1 INTRODUCTION

The previous chapter has reviewed methods of multiple-criteria decision-making and presented details of the theory of the Analytic Hierarchy Process (AHP). The AHP is the method that has been used in this thesis for the multiple-criteria decision-making process. In this chapter, the application of the AHP model for decision-making in the process of selecting the best design for a hypothetical water distribution network is described using a step-by-step approach. An EXCEL spreadsheet program has been produced to execute the module for the AHP. This module is primarily for the decision-making process in the integrated model for the long-term upgrading of water distribution networks. The description of the application of the AHP herein is based on extracts of results at different stages of the execution of this EXCEL spreadsheet program.

As seen in Chapter 6, the AHP allows for the use of intangible factors side by side with tangible factors and it can be used to establish measures in both the physical and social domains. The AHP model is well suited for decision-making due to its structure (hierarchical arrangement of factors), which allows it to give evidence of how the problem is perceived. Decision-making in the design of water distribution networks is a multi-objective problem involving economics, technical issues, social and environmental issues.

The AHP has been used for various decision-making problems in the past (Saaty and Vargas, 2001), but it has never been used for decision-making in long-term upgrading of water distribution networks in the manner described in this thesis. The next section dwells on the application of the AHP model for the decision-making process of selecting the best long-term upgrading strategy and design of a hypothetical water distribution network, using a step-by-step approach.

7.2 PROGRAM FOR THE ANALYTIC HIERARCHY PROCESS

An EXCEL spreadsheet computer program has been developed in this study for the implementation of the analytic hierarchy process. The program has the capability of calculating the priority vectors, checking the consistency of comparison judgments, revising judgments where necessary and producing an overall ranking of the alternatives. The program can generally handle any multi-criteria decision-making problem. All that is required is for one to set up the problem in an appropriate manner as described shortly. Note that any computer spreadsheet software may be used to set up a similar program as long as the principle behind the AHP is strictly adhered to. The next sub-sections present an overview of the program's decision framework, followed by the structure plans for the program, the input and output data requirements.

7.2.1 Overview of the Program's Decision Framework

The program is executed in an interactive forum and it requires a group of people headed by a facilitator to participate in the decision making process. The leader or facilitator is referred to herein as the chief decision-maker and is responsible for coordinating all the activities, collecting and evaluating the opinions of the other group members and synthesizing this information into a group decision. The other members are the judges who are each responsible for identifying and agreeing upon the criteria and sub-criteria for the problem; and evaluating the problem in order to establish an individual ranking of options based on the relative importance of the criteria and sub-criteria. Four people have been involved in the decision-making process for the examples in this chapter and these are: the chief decision-maker and three judges who are referred to herein as Judge 1, Judge 2 and Judge 3.

The program has two distinct sections with the first one being that of the process of the judges making individual judgments. Each of the judges performs the steps that are summarised in the structure plan for making individual judgments that is presented in the next sub-section, in order to obtain an individual overall priority vector or overall ranking of the options. The second part of the program covers the process of obtaining the overall group decision by the chief decision-maker using the information from individual judgments. As a means of achieving consensus, the EXCEL spreadsheet program that has been developed for the AHP module includes a section for priority assignment to the judges, reflecting the weight of their contribution to the final decision. The chief decision-maker evaluates judges in terms of three main factors, their technical knowledge, experience and current project knowledge. The chief decision-maker uses these weights together with the individual judgments of the judges to obtain the overall group decision as described shortly in the structure plan for the final group decision. For clarity and ease of reference, the vectors and matrices formed in this section are given labels.

7.2.2 Part I: Structure Plan for Making Individual Judgments

This section is performed by the judges in order to obtain an individual ranking of the options. Steps 1 and 2 are performed in an open group discussion in order to obtain decision factors of the problem. Each judge then performs the rest of the steps in this section to obtain an overall individual judgment.

(1) State the problem and identify the main criteria and sub-criteria that govern it.

(2) Make a decision on the alternatives together with their properties and structure a hierarchy of the decision criteria, sub-criteria, and the alternatives with clear definitions of all elements of the hierarchy to remove ambiguity.

(3) Prioritise the main criteria with respect to their impact on the overall objective or goal and the sub-criteria with respect to their criteria.

(4) For the main criteria, make pair-wise comparison judgments with reference to a pre-defined ratio scale and form a comparison matrix. Calculate the priority vector (**B1**) for this comparison matrix.

(5) Check for consistency of the comparison judgements. If the judgments are not consistent, make the appropriate revision of judgments; repeat this step iteratively until consistency is achieved.

(6) The priority vector (**B2**) for each sub-criterion takes on the value of 1 since there are no further sub-divisions of the sub-criterion. Calculate the relative weight of the options with respect to each sub-criterion by pair-wise comparisons of the options to give another priority vector (**B3**). Perform Step 5. Multiply Vector **B2** by Vector **B3** to give a vector of composite priorities (**B4**) showing the ranking of options with respect to each sub-criterion.

(7) Form a priority vector (**B5**) for each decision criterion by making pair-wise judgments of its sub-criteria if any, else the vector takes on the value of 1. Perform Step 5. Combine the vectors of composite priorities (e.g., Vector **B4**) for the sub-criteria of each decision criterion to form a priority matrix (**M1**) for the criterion. Multiply Vector **B5** by Matrix **M1** to give a vector of composite priorities (**B6**) showing the ranking of options with respect to each of the main criteria.

(8) Combine the vectors of composite priorities (e.g. Vector **B6**) for each decision criterion to form an overall priority matrix (**M2**). Multiply the priority vector of the main criteria (e.g., Vector **B1**) by Matrix **M2** to give an overall ranking of the options (Vector **B7**) by each individual judge.

7.2.3 Part II: Structure Plan for the Final Group Decision

This section is performed by the chief decision-maker to obtain the ranking of the contribution of the judges and the overall group decision.

(1) Perform pair-wise comparisons of the three factors (technical knowledge, experience and current project knowledge) to obtain their priority vector or importance weighting (Vector **C1**). Perform Step 5 in Part I.

(2) The priority vector (**C2**) for each factor takes on the value of 1 since there are no further sub-divisions of the factor. Calculate the relative weight of the judges with respect to each factor by pair-wise comparisons of the judges to give another priority vector (**C3**). Perform Step 5 in Part II. Multiply Vector **C2** by Vector **C3** to give a vector of composite priorities (**C4**) showing the ranking of judges with respect to each factor.

(3) Combine the vectors of composite priorities (e.g. Vector **C4**) for each factor to form an overall priority matrix (**M3**). Multiply the priority vector of the factors (e.g., Vector **C1**) by Matrix **M3** to give an overall ranking of the judges (Vector **C5**) with respect to their contribution towards the final decision.

(4) Combine the vectors of overall ranking of options obtained from each judge in Part I (e.g., Vector **B7**) to form the final priority matrix (**M4**). Multiply the vector (**C5**) of overall ranking of judges by Matrix **M4** to give a vector of the final ranking of options, which represents the final group decision. The best design option is the one with the highest priority value.

7.2.4 Program Input and Output

The process of data input to the program does not necessarily end at the beginning of the program but one may be required to enter more data on comparison judgments especially if the need for their revision arises. In such instances, one may prefer to make the revisions based on personal experience and enter the data manually, or let the program make the revisions automatically using the method described in Chapter 6. The input and output data for Part I followed by that of Part II of the program is presented next.

7.2.4.1 Program Input for Part I

Data input for the first part of the program is done by each of the judges and the following are the requirements:

(1) The structure of the hierarchy, the alternatives together with their properties, decision criteria and sub-criteria.

(2) The quantitative values for the project e.g., present value of project costs, reliability and failure tolerance values.

(3) Pair-wise comparison judgments based on a pre-defined ratio scale.

7.2.4.2 Program Output from Part I

The output from the first part of the program by each of the judges is as follows:

(1) Comparison matrices and priority vectors for decision criteria and sub-criteria.

(2) Overall ranking of options.

7.2.4.3 Program Input for Part II

The chief decision-maker enters the data input for the second part of the program and the following are the requirements:

(1) The individual results of overall ranking of options by each of the judges.

(2) Pair-wise comparison judgments of the factors for the ranking of judges.

7.2.4.4 Program Output from Part II

The output from the second part of the program by the chief decision-maker is as follows:

(1) Vectors of the weight of judges with respect to each of the factors for the ranking of judges.

(2) Vector for the ranking of judges in terms of their contribution to the final decision.

(3) Final ranking of options or the overall group decision.

7.3 MODEL APPLICATION

This section applies decision analysis using the AHP to appraise water distribution network designs for a long-term upgrading strategy, considering the monetary and non-monetary worth measures or criteria. The details of the calculations for the optimal designs, timing and magnitude of the hypothetical networks are in Chapter 4. Saaty (1980) states that the decision-making process is concerned with the ranking of alternatives to fulfil a set of desired objectives such as minimising project costs, maximizing network performance, minimizing environmental damage and maximizing social benefits.

7.3.1 Problem Statement

The problem at hand is to decide amongst three design options for long-term upgrading water distribution networks. The first option is the three-loop Network A of Figure 4.5 that is reproduced as Figure 7.1. This network is designed with maximum entropy flows combined with a pricing policy. The pricing policy involves increasing the price of water as the system capacity is being approached resulting in reduced consumption of water and a delay in the timing of expansion of the network. The second option is the three-loop Network B of Figure 4.6 that is reproduced as Figure 7.2. This network is also designed with maximum entropy flows but without any pricing policy. The third option is the single-loop network of Figure 4.7 that is reproduced as Figure 7.3. This network is designed using the shortest path flow distribution method for link-flow allocation (conventional flow distribution approach). The optimal designs, timing and magnitude of the upgrading for these three options were obtained in Chapter 4 using the module for network design.

7.3.2 Hierarchical Representation of a System

The hierarchical structure for the optimal long-term upgrading of water distribution systems that has been adopted for this study is presented in Figure 6.1 and reproduced as Figure 7.4. The first level of the hierarchy shows that the overall goal is to select the best design option i.e. one that minimizes the cost, minimizes environmental damage, maximises network performance and social benefits. The second level has three main criteria (network performance, economic value, social and environmental issues) and the third level has the sub-criteria, which contribute to the achievement of the overall goal. Finally, the fourth level consists of the three design options.

The economic value is expressed in terms of the Present Value (PV) of the costs for each of the design options. The network performance is defined in terms of the reliability and the failure tolerance of the network as mentioned earlier in Chapter 2. Reliability is considered the network's ability to satisfy customer demands at adequate pressure under normal and abnormal operating conditions (Tanyimboh et.al., 2001). It is a performance assessment measure that tends to focus more on the network hydraulics other than the underlying robustness in terms of its layout. Failure tolerance is the expectation of the proportion of the demand of the network that is satisfied during the periods when some components are unavailable (Tanyimboh and Templeman, 1998). It addresses the robustness or layout of the network more effectively and is concerned about deficient network performance when some components are unavailable.

The social and environmental issues addressed in this research refer to acceptability, abstraction and health issues (Canter, 1996). Acceptability is the beneficiaries' attitude towards a particular upgrading strategy and whether they would tolerate this strategy. For example, the pricing strategy which involves increasing the price of water (tariffs) to reduce consumption and delay capacity expansion is a cost saving option but it may not be favourable to the consumers or acceptable to the regulatory agencies from the social or any other point of view. Abstraction implies the amount of water that is pumped from the source (depletion of the source); the more water that is pumped from the source, the higher the chances of causing environmental damage. The pricing option, which involves a reduction in consumption and thus abstraction, might perhaps be more environmentally friendly. Health issues of concern are whether a particular option or expansion strategy might lead to the health of consumers being compromised through the use of unsafe but cheap alternative water sources or rationing of water usage to a level below the minimum required for essential hygiene. The strategy of pricing could easily tempt consumers to compromise their health, a situation that is not desirable at all.

7.3.3 Establishing Priorities Using the AHP

When using the AHP, each decision-maker has to prioritise judgments i.e. obtain the relative importance of each of the main criteria with regard to the achievement of the overall goal followed by the relative importance of sub-criteria in each criteria. For example the decision-maker could decide that the performance criterion has top priority followed by the economic value and finally the social and environmental issues. At the next level one could decide that the failure tolerance has more priority over reliability. This could be based on the fact that in general, if a network is designed with reliable components, its reliability will be high. On the other hand the failure tolerance is more likely to uncover the vulnerability of the network under stress. It can therefore distinguish clearly between networks with high reliability but with varying capabilities to perform under stress (Kalungi and Tanyimboh, 2001).

7.3.4 Pair-wise Comparisons for the Main Criteria

Pair-wise comparisons are made among all elements at a particular level with respect to each other under the same parent element above them. At level 2 these comparisons are made between the main criteria i.e. economic value, network performance, social and environmental issues, with regard to each judge's contribution to the overall goal of selecting best design option. The comparisons are made in a systematic manner according to the prioritisation of the criteria. For example, network performance is compared with economic value, followed by social and environmental issues. Economic value is then compared with social and environmental issues. The details of these comparisons are shown in Table 7.1. Only the dominant element of any two being compared (those above the matrix's leading diagonal in this particular case) needs to be filled in by the analyst. From the reciprocity assumption, the less dominant elements are set equal to the reciprocals of the corresponding elements above the diagonal. The leading diagonal elements are all equal to 1 by definition of the identity assumption, because each attribute is exactly as important as itself.

Comparisons can be made according to preference, importance or likelihood. In each comparison, the more important criterion must be selected and then expressed in terms of a judgment of how much more important the selected criterion is. To measure how much more important one criterion is compared to another, the AHP uses a comparison scale with values from 1 – 9. Table 6.7, which is reproduced as Table 7.2, shows how the decision-maker's verbal description of the relative importance is converted into a numerical rating.

7.3.4.1 Comparison Matrix for Level 1

The comparison matrix for level 1, which involves the pair-wise comparison between the three main criteria, was obtained as described next. Network performance has equal importance as the economic value thus this comparison takes on the numerical rating of 1 from the AHP comparison scale in Table 7.2; network performance and the economic value criteria each have a weak importance over the social and environmental issues, thus these comparisons each take on the numerical rating of 2 from the AHP comparison. The comparison matrix is shown in Table 7.3.

7.3.4.2 Computation of the Vector of Priorities

Using the pair-wise comparison matrix (Table 7.3), the priority of each criterion in terms of its contribution to the overall goal of selecting the best design option should be calculated. The vector of priorities is computed as described in Chapter 6. The normalised comparison matrix is shown in Table 7.4 and the vector of priorities is (0.4, 0.4, 0.2) as shown in the right hand column of the table. This priority vector for selection of the best option represents the importance of each criterion in terms of its contribution to the overall goal. Thus, the most important criteria are network performance and economic value since they each have a priority of 0.4. This is followed by the criterion for social and environmental issues with a priority of 0.2.

7.3.4.3 Consistency

An important consideration in the AHP is the consistency of the pair-wise judgments. With numerous pair-wise comparisons, perfect consistency is difficult to achieve. The AHP provides a measure of the consistency for the pair-wise comparisons by computing a consistency ratio (C.R.) and the calculations for consistency are in Section 6.3. A consistency ratio greater than 0.10 indicates inconsistency in the pair-wise judgments and would call for a revision of judgements as detailed in Section 6.3.

An illustration of calculations for the consistency ratio of the comparison matrix in Table 7.3 is obtained as described next. From Table 7.4, the product of the comparison matrix (**K1**) and the priority vector (0.4, 0.4, 0.2), Vector **D1**, gives another column vector (1.2, 1.2, 0.6) referred to herein as Vector **D2**. Dividing the corresponding components of this second column vector (**D2**) by the priority vector (**D1**) gives another column vector (3, 3, 3) referred to as Vector **D3**. The average of these values gives the maximum or principal eigenvalue λ_{max} as 3.00. The consistency index, C.I. = (3.00 – 3)/(3 – 1) = 0. To determine how consistent the judgments are, C.I. is divided by the corresponding value of Random Index for n = 3, which is 0.58 (Table 6.12). The consistency ratio, C.R. is 0/ 0.58 = 0, which is lower than 0.10. Therefore the pair-wise comparison judgments are consistent and acceptable.

7.3.5 Pair-wise Comparisons for Sub-criteria and Design Options with Respect to Sub-criteria

The procedure in Sub-section 7.3.4 is repeated for levels 2 and 3. At level 2, pair-wise comparisons of the sub-criteria are carried out. At level 3, pair-wise comparisons are carried out for the options with respect to each of the sub-criteria. The details of the calculations for obtaining the vectors of priorities and assessing whether the matrix of pair-wise judgments are consistent are similar to those illustrated in Sub-section 7.3.4 as detailed below.

7.3.5.1 Pair-wise Comparisons for Level 2

At level 2 of the hierarchy, a priority vector for each of the criteria is obtained by assessing the weight of each sub-criterion in terms of its contribution to the respective criteria. This is done after forming a matrix of pair-wise comparisons of the sub-criterion related to each criterion.

The economic value criterion has only one sub-criterion and therefore its priority vector has one element (Present worth or value) with a priority or weight of 1.

The network performance criterion is subdivided into two sub-criteria, reliability (Rel) and failure tolerance (Ftol). The pair-wise comparison matrix and the priority vector are shown in Table 7.5. There is no need for calculating the maximum eigenvalue, λ_{max}, the consistency index (C.I.) or the consistency ratio (C.R.) since the matrix elements are less than 3 (see Table 6.10).

The criterion for social and environmental issues is subdivided into three sub-criteria, acceptability (Acc), health issues (Hi) and abstraction (Ab). The pair-wise comparison matrix, the priority vector, the maximum eigenvalue, λ_{max}, the consistency index, (C.I.) and the consistency ratio, (C.R.) are shown in Table 7.6.

The economic value criterion has only one sub-criterion and therefore its priority vector has one element (Present worth or value of project costs) with a priority or weight of 1.

7.3.5.2 Pair-wise Comparisons for Level 3

At level 3 of the hierarchy, a priority vector for each of the sub-criteria (reliability, failure tolerance, present value of project costs, acceptability, health issues and abstraction) is obtained. Since none of the sub-criteria is further subdivided into any other parameters, the priority vectors for each sub-criteria has a weight of 1.

Comparison judgments from measurement of quantitative data

Table 7.7 shows a summary of the quantitative data for the three design options. The cost data is obtained from Chapter 4 and the performance data is obtained using methods in Chapter 5. Using this data, the decision-makers make a judgment of the relative importance of the element with reference to the ratio scale. It is important to note that sometimes when the weights are known from measurements or analyses such as the failure tolerance or the present value of project costs for the options, one is inclined to normalise (summing the values and taking each as a fraction of the total) and use them (dividing the resulting higher value by the lower value in the comparison pair) instead of constructing a judgment matrix (from the pre-defined ratio scale of comparisons) and computing the eigenvector. Ideally, this would be correct, however the process can be erroneous especially when the value of relative measurements for the judgments is not reflected in terms of their ratios (Saaty, 1980).

To elaborate on this, assume three elements C_1, C_2 and C_3 of a level in a hierarchy have calculated values for reliability of 0.8, 0.6 and 0.4, respectively. Normalising these values gives (0.4444, 0.3333, 0.2222). Pair-wise comparison of C_1 with C_3 using ratios gives 0.4444/0.2222 = 2. This would mean C_1 is between equally important to weakly more important than C_2 in terms of reliability on the 1 to 9 comparison scale. In this case, the value of relative measurements is not well reflected by the ratios because in terms of reliability, the two values 0.8 and 0.4 are significantly different. In such instances, it is advisable to create a scale of relative intensities as described next to assist in the process of making judgments (Saaty and Vargas, 2001).

Creating a scale of relative intensities

A typical example of a scale of relative intensities is shown in Table 7.8. The percentage difference in compared values is d'. Such a scale is formed by the group of decision-makers or by the chief decision-maker bearing in mind the value of relative measurements between the criteria values and the interpretation on the AHP comparison scale. The scale of relative intensities for quantitative data represents the change in value between a dominant factor and a less dominant factor expressed in terms of the numerical rating on the comparison scale. For example, in terms of failure tolerance, a change from 0.9 down to 0.6 which is (0.9 – 0.6)/0.9 = 0.33 (a reduction of 33.3%) is quite significant. If two elements C_1, and C_2 were to be compared, C_1 with a failure tolerance of 0.9 and C_2 with 0.6, from Table 7.8, this 33.3% change in failure tolerance would imply a numerical rating of 5 for this comparison. This implies that according to the AHP ratio scale, element C_1 is strongly more important than element C_2.

Another example could be in terms of the present value of project costs. Suppose two elements C_1 and C_2 with present value costs of \$3,500,000 and \$4,200,000 are to be compared. Element C_1 is more favourable or important than C_2 since it has the lower cost. The percentage difference in costs between the two elements, $|(3{,}500{,}000 - 4{,}200{,}000)|/\ 3{,}500{,}000 = 0.2$ (an increase of 20 %) corresponds to a numerical rating of 2 from Table 7.8. This implies that according to the AHP ratio scale, element C_1 is weakly more important than element C_2. Such a relative intensity scale can help in reducing the variation in decisions. This scale serves only as a guide to the decision-maker and does not restrict the judgement to the one on the scale of relative intensities, one could opt for a numerical rating value slightly above or below that obtained from the table.

The quantitative data for the three design options (Table 7.7) together with the scale of relative intensities (Table 7.8) are tools that can assist the decision-makers in judging the relative importance of the element with reference to the AHP ratio scale. The relative weight of the options for each of the sub-criteria is obtained by forming a matrix of pair-wise comparisons of the options with reference to each sub-criterion. The priority or weight of each option with respect to the sub-criterion is then calculated. The results of these calculations are shown in Tables 7.9, 7.10, 7.11, 7.12 and 7.14.

7.3.6 Matrix Manipulation to Obtain Composite Vectors of Priorities

Starting from the bottom of the hierarchy (level 3), the weights of the options with respect to the sub-criteria are multiplied by the priority vectors of the respective sub-criteria, in order to obtain vectors of composite priorities for the sub-criteria as shown in Table 7.15. For example, the product of the vector of relative weights of the options with respect to abstraction (0.6, 0.2, 0.2) and the priority vector for abstraction (1.0) is a vector of composite priorities for abstraction (0.6, 0.2, 0.2). Composite vectors are obtained in the same way for all the other sub-criteria.

At level 2 in the hierarchy, the composite vectors of sub-criteria for each criterion are then aggregated to form a priority matrix for the criteria as shown in Table 7.15. For example, the composite vectors for reliability and failure tolerance are aggregated to form a priority matrix for network performance. The composite vectors for acceptability, health issues and abstraction are aggregated to form a priority matrix for social and environmental issues. The present value of project costs, being the only sub-criterion for economic value, has a composite vector of priorities that doubles as the priority matrix for economic value. The products of the priority matrices of the criteria and their respective priority vectors yield composite vectors of priorities for the criteria. The composite vector of priorities for performance is (0.334, 0.490, 0.177); that for economic value is (0.328, 0.261, 0.411) and that for social and environmental issues is (0.249, 0.375, 0.375).

7.3.7 Selection of the Best Alternative

At level 1 in the hierarchy, the composite vectors of the decision criteria are aggregated to form an overall priority matrix as shown in Table 7.15. The product of this priority matrix and the priority vector for selection of the best option (0.4, 0.4, 0.2) gives the overall ranking of the design options according to Judge 1. The best option is the one with the highest priority value. From Table 7.15, the overall ranking of Options 1, 2 and 3 is (0.314, 0.375, 0.310); therefore according to Judge 1, Option 2 is the best.

The summary of the above calculations for Judge 2 and Judge 3 are presented in Tables 7.16 and 7.17, respectively, showing the results of matrix manipulation at different levels in the hierarchy and the overall ranking of options according to each of these judges.

The results of the priority assignment to the judges as compiled by the chief decision-maker are summarised in Table 7.18. The evaluation of the judges was based on three main criteria, their technical knowledge, experience and current project knowledge. Pair-wise comparisons were performed for three criteria to obtain their priority vector or importance weighting shown in Table 7.18. This was followed by pair-wise comparisons of the judges with respect to each of the three criteria resulting in the relative weight of each judge. For example, the weight of Judges 1, 2 and 3 with respect to project knowledge is (0.25, 0.25, 0.5), meaning that Judge 3 has more knowledge about the project than the other two. The product of the weight of the judges with respect to the criteria and the priority vector of each criterion yields a vector of composite priorities. These vectors of composite priorities were aggregated to form an overall priority matrix. The product of this final priority matrix and the priority vector, (0.333, 0.333, 0.333), of factors affecting judgment yields the overall ranking of the judges in terms their contribution to the final decision. In this case the vector of overall ranking of the judges gave (0.333, 0.333, 0.333) meaning that their individual judgments have an equal contribution to the final decision.

The vectors of individual overall ranking of the options by Judge 1 (0.314, 0.375, 0.310), Judge 2 (0.297, 0.356, 0.347) and Judge 3 (0.321, 0.383, 0.295) were aggregated into the final priority matrix in Table 7.19. The product of this final priority matrix and the priority vector for judges (0.333, 0.333, 0.333) yields the vector of final ranking of Options 1, 2 and 3 as (0.311, 0.372, 0.317), and this represents the final group decision. The highest priority value is 0.372 meaning the best design option for long-term upgrading of the network is Option 2, the three-loop network design without any pricing policy.

7.4 SENSITIVITY ANALYSIS

A sensitivity analysis was carried out to check the effect of variation in the priority vector for judges. This particular vector does not benefit from the consensus of the group discussions or debate, because it is generated solely by the chief decision-maker. The results are shown in Tables 7.20, 7.21 and 7.22. Table 7.20 presents Scenario A, in which the priority vector for the judges is (0.5, 0.3, 0.2). This means that Judge 1 has the highest weight and therefore his/her judgments are more influential to the final decision compared to those made by the other two judges. The resulting final option ranking for Options 1, 2 and 3 is (0.311, 0.371, 0.318).

Thus Option 2 is the best design option for this scenario. The worst option remained Option 1.

Scenario B is shown in Table 7.21 and the priority vector for the judges is (0.25, 0.5, 0.25). This means that Judge 2 has the highest weight and therefore his/her judgments are more influential to the final decision compared to those made by the other two judges. The resulting final option ranking for Options 1, 2 and 3 is (0.307, 0.368, 0.325). Thus Option 2 is the best design option for this scenario. This scenario maintains Option 1 as the worst.

In Table 7.22, Scenario C is presented with a priority vector for judges of (0.3, 0.2, 0.5), giving the superiority of judgment to Judge 3. The resulting final option ranking is (0.314, 0.376, 0.310). Once again the best design alternative is Option 2. In this particular instance, the Option 3 is the worst.

7.5 DISCUSSION

From Table 7.7, the reliability values of the Options 1, 2 and 3 are 0.999553, 0.999832 and 0.999484, respectively. These results are arguably very close to each other perhaps due to the fact that the individual component reliabilities of the networks are high. These results would probably not be enough for one to distinguish between the designs using this criterion alone, though reliability is a commonly used performance assessment parameter. On the other hand, Table 7.7 shows that the values of failure tolerance for the Options 1, 2 and 3 are 0.850584, 0.934716 and 0.647552, respectively. The failure tolerance for Option 3 is the lowest and significantly different from that of Options 1 and 2, probably because the layout for Option 3 (Figure 7.3) is partially looped and partially dendritic. This limits the number of alternative paths from the source to the supply nodes and further reduces performance of the network if components become unavailable. On the other hand, the layouts for Options 1 and 2 are purely looped options (Figures 7.1 and 7.2) and are therefore more likely to perform better in times of partial failure. Failure tolerance clearly distinguishes between the designs and stresses the point that reliability and failure tolerance should be used together as performance assessment parameters.

Table 7.7 shows that the present value of project costs for Options 1, 2 and 3 are \$3,810,851.25, \$4,214,803.00 and \$3,529,783.00, respectively. It is clear that the cheapest design, Option 3, also has the lowest value of failure tolerance. Whereas Option 3 is the most favourable option in terms of costs, it is the least favourable in terms of performance when some components are not available. The AHP is very helpful in handling such instances of conflicting decision factors.

The scale of relative intensities is very helpful in facilitating the process of comparing measured or quantitative data. In some cases the percentage variation in the values of the decision factors other than the actual values themselves could be used to judge the importance of one factor over the other. For example, a failure tolerance value of 0.9 can be judged as strongly more important on the AHP ratio scale (Table 7.2) when compared to a value of 0.6, as detailed before in Sub-section 7.3.5.2. Such a judgment is easier to make given the scale of relative intensities in Table 7.8 using the percentage variation of these failure tolerance values. This scale of relative intensities is particularly helpful if the knowledge and experience level of the judge with respect to a particular decision factor is not very high.

In all instances of pair-wise judgements, each of the comparison matrices gave a consistency ratio with a value less than the allowable value of 0.10. This implies that all the judgments were consistent and there was no need for revision or forcing the values of judgments to achieve consistency.

A careful scrutiny of results in Tables 7.15, 7.16 and 7.17 shows that each judge had a different priority vector for selection of best options (importance weights for the main criteria). According to Judge 1, the priorities for performance, economic value, and, social and environmental issues are given by the priority vector, (0.4, 0.4, 0.2), as shown in Table 7.15, meaning that performance and economic value are equally important and that social and environmental issues are of least importance. Judge 2 obtained the priorities for performance, economic value and social and environmental issues as given by the priority vector (0.333, 0.3333 0.333), which is shown in Table 7.16; implying that all the criteria are of equal importance in terms of their contribution to the final goal. The priorities for performance, economic value and social and environmental issues according to Judge 3 are given by the priority vector, (0.5, 0.25, 0.25), as shown in Table 7.17. This means that, according to Judge 3, network performance is the most important criterion and the economic value together with the social and environmental issues are equally less important. Despite the variation in the allocations of importance to the main criteria by the different judges, the overall ranking of the design options by each judge showed that Option 2 is the best.

The results of the overall ranking of the options by each of the judges are also in Tables 7.15, 7.16 and 7.17. According to Judge 1, the ranking of Options 1, 2 and 3 is given by the vector (0.314, 0.375, 0.310), as shown in Table 7.15. This means that Option 2 is the best and Option 3 is the least preferred alternative. Since the AHP is a tool for assisting in the process of decision-making, one could perhaps argue that Options 1 and 3 are not significantly different and are therefore equally less important than Option 2. Judge 2 ranked Options 1, 2 and 3 with the priorities given in the vector (0.297, 0.356, 0.347), as shown in Table 7.16; implying that Option 2 is the best, followed by Option 3 and Option 1 is least desirable. Judge 3 ranked Options 1, 2 and 3 with the priorities given in the vector (0.321, 0.383, 0.295), as shown in Table 7.17. This means that according to this Judge, Option 2 is the best alternative, followed by Option 1 and Option 3 is least desirable.

From the individual ranking of options by the judges, Tables 7.15, 7.16 and 7.17, the second best and third best options vary from judge to judge. This is resolved in Table 7.18 by forming a final priority matrix, which is a combination of the vectors of individual ranking of options by the judges. The product of the final priority matrix and the priority vector for judges (0.333, 0.333, 0.333) yields the vector of final ranking of Options 1, 2 and 3 as (0.311, 0.372, 0.317), and this represents the final group decision. The highest priority value is 0.372 meaning the best design alternative for long-term upgrading of the network is Option 2, the three-loop network design without any pricing policy. This is probably because each individual judge ranked Option 2 as the best. The second best alternative is Option 3 and the least favourable Option 1 because it has the lowest priority value.

The results of the sensitivity analyses in Tables 7.20, 7.21 and 7.22 for Scenarios A, B and C, respectively, show that in the three instances of varying the superiority of judgment, the final group decision remained that of Option 2, the three-loop layout design with maximum entropy flows but without any pricing strategy. This gives an indication that the final group decision in terms of the best option is quite stable and not overly sensitive to the weight of judges with respect to their superiority in making judgments.

7.6 CONCLUSION

The AHP is a versatile and robust tool in handling decisions involving conflicting decision factors, qualitative and quantitative data. The appropriateness of the AHP for multi-criteria decision-making is mainly based on the hierarchical arrangement of the factors. It uses a simple method of pair-wise comparisons for judgments and matrix manipulation to derive a vector of priorities for the decision. Another important feature of the AHP is the measure of consistency for the comparison matrix and provision for revision of judgments in cases of inconsistent comparison matrices.

The application of the AHP model for decision-making in the process of selecting the best design for the long-term upgrading of a hypothetical water distribution network has been detailed in a step-by-step approach. The model execution was done by the EXCEL spreadsheet program that was developed for this purpose. The overall group decision narrowed down to the best design alternative as being Option 2, the three-loop layout of Figure 7.2 designed with maximum entropy flows but without the pricing policy. Sensitivity analysis confirmed that this final group decision is very stable and not too sensitive to varying the weight of judges with reference to their contribution in making judgments. Another observation is that the cheapest option is not necessarily the best because other factors like the performance, social and environmental issues could weigh it down. Thus, it is important to consider all these qualitative and quantitative factors in the decision-making process.

In the next chapter an Integrated Model shall be formulated with separate modules for network design, hydraulic simulation, network performance and the analytic hierarchy process.

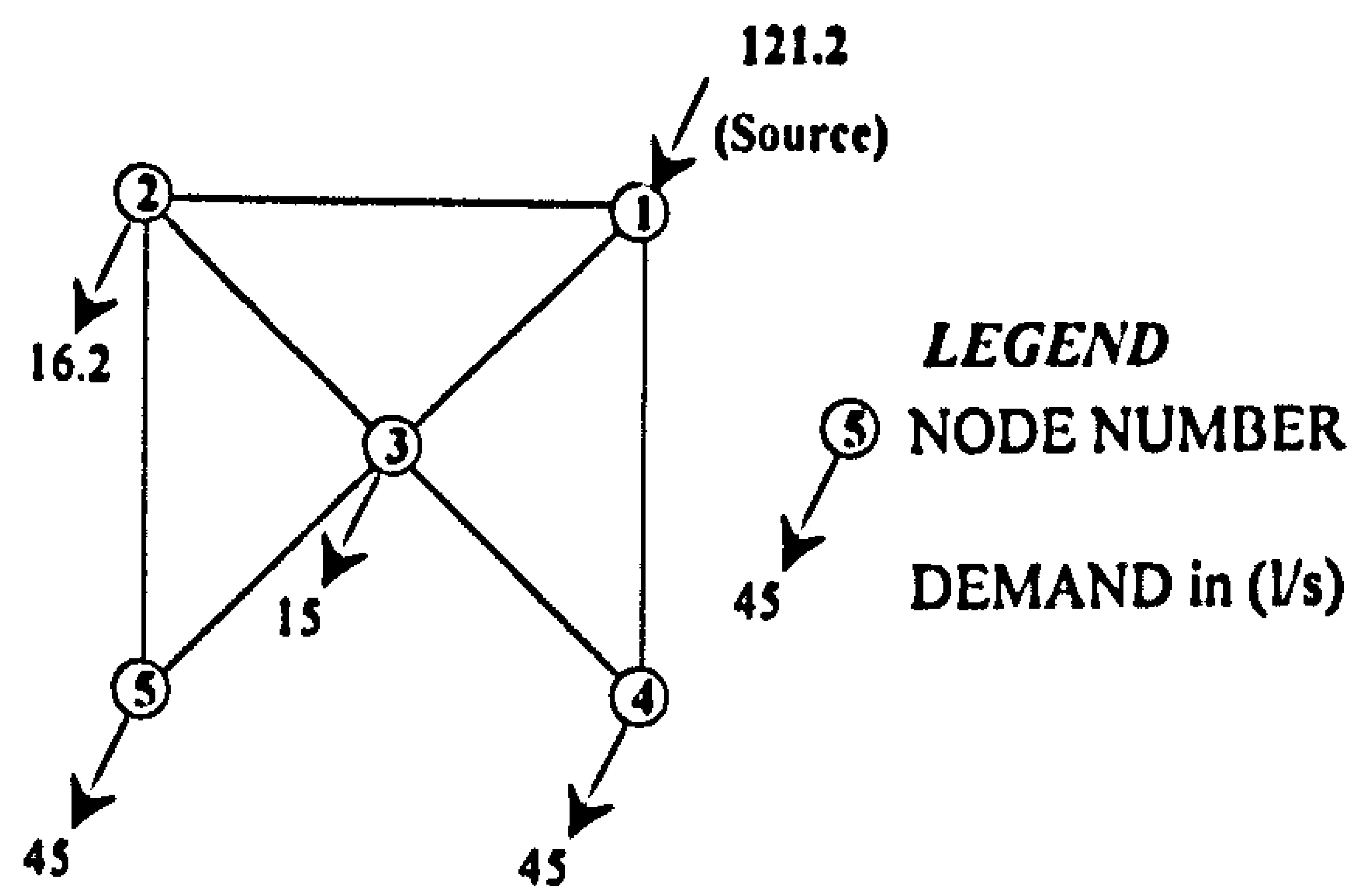

Figure 7.1 Three-loop Network A

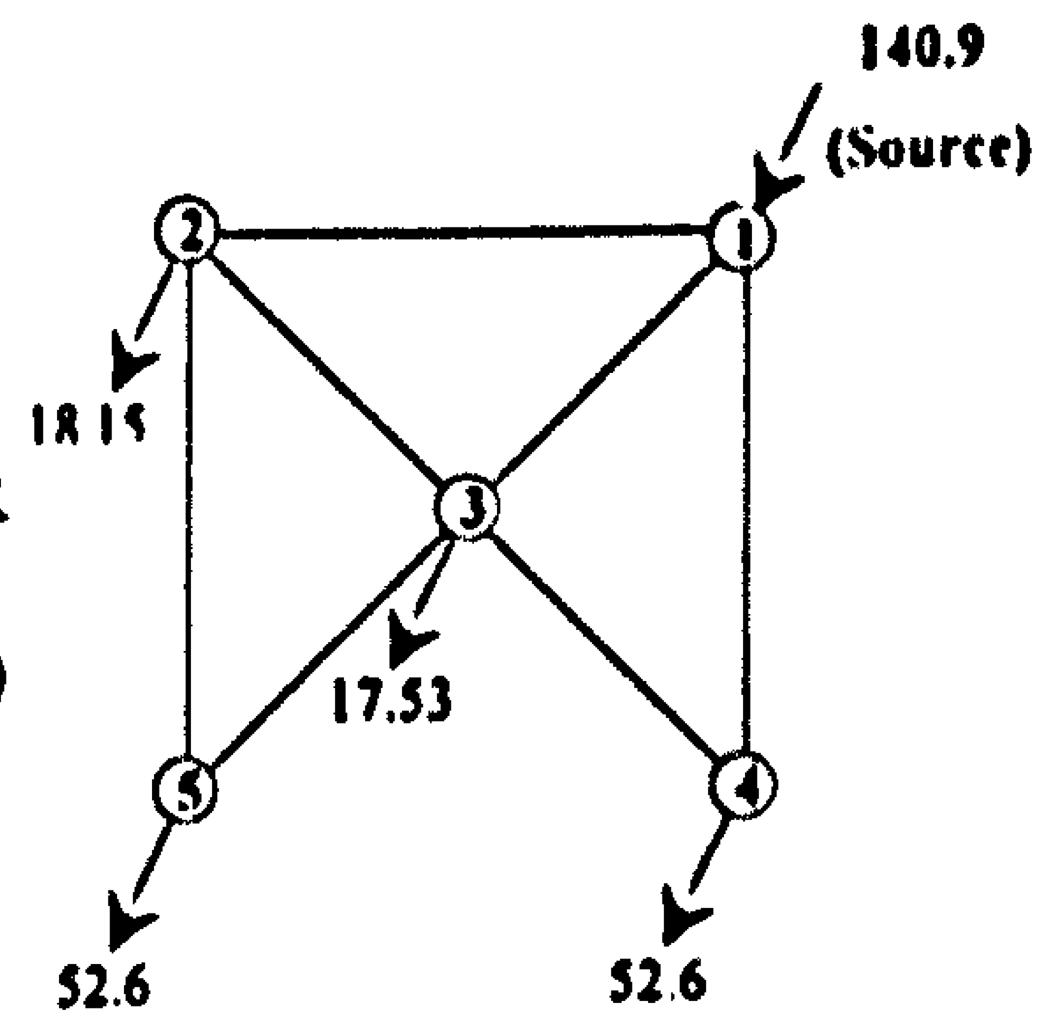

Figure 7.2 Three-loop Network B

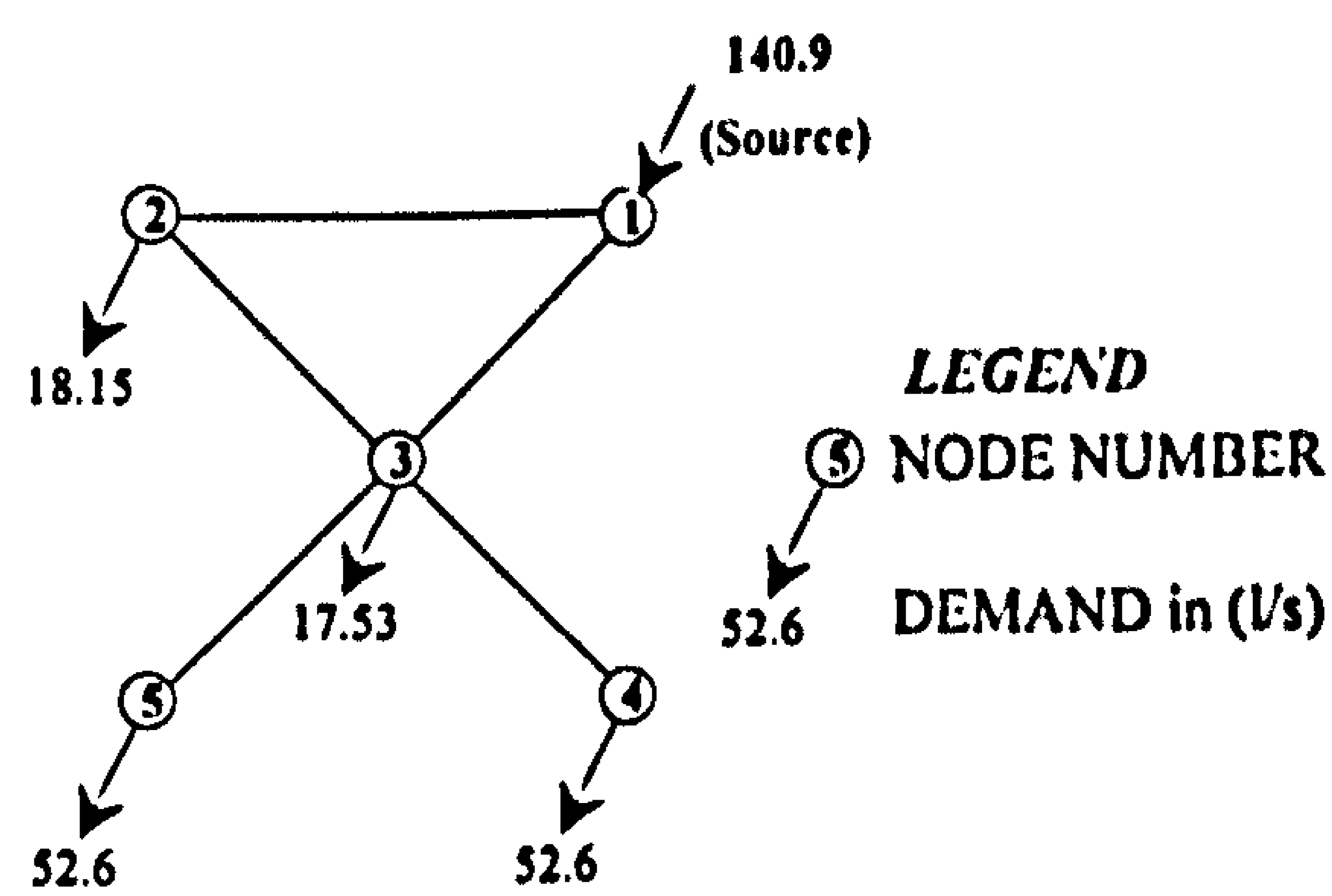

Figure 7.3 Single-loop Network

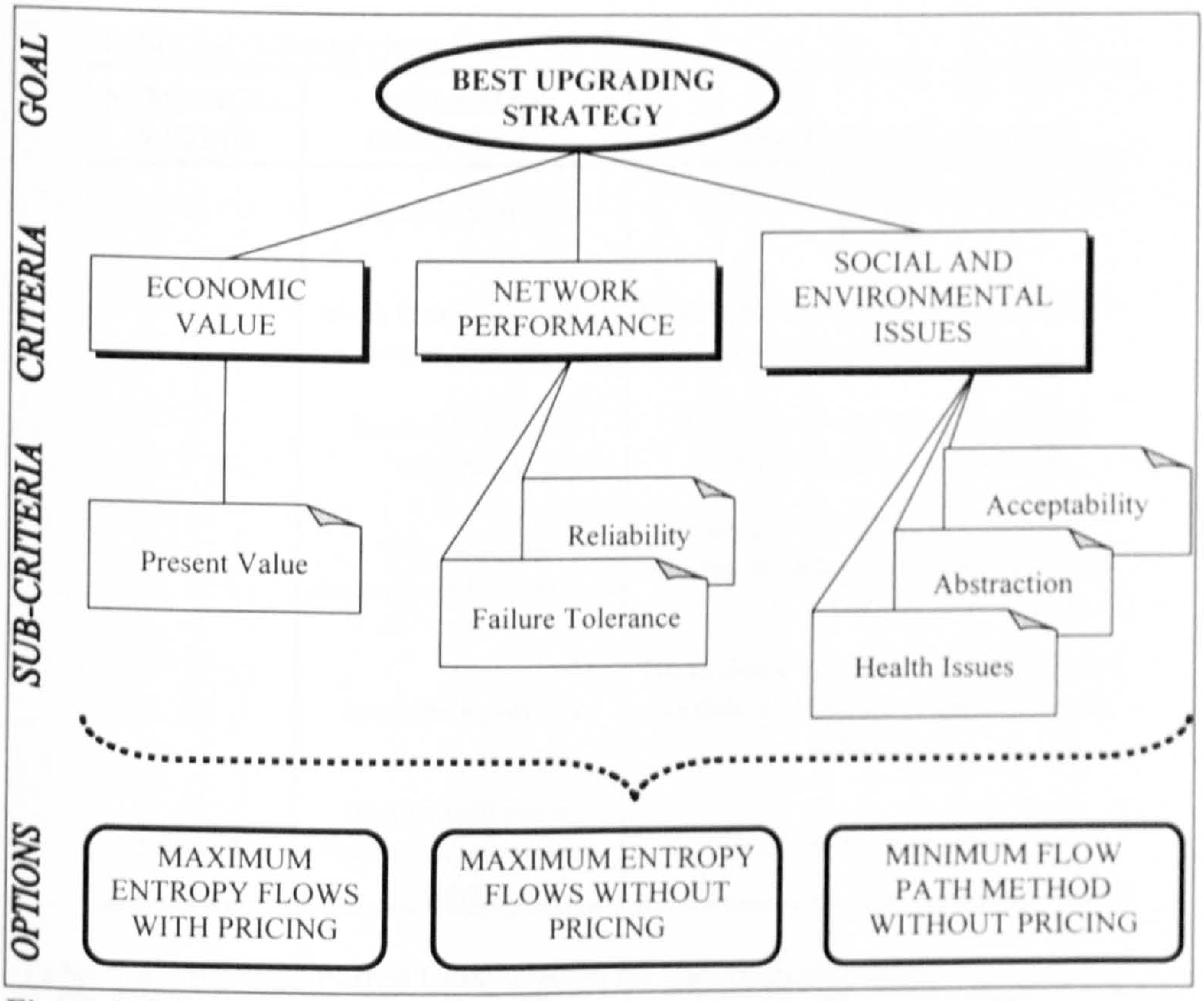

Figure 7.4 Hierarchical Structure

Table 7.1 Details of Matrix of Paired Comparisons (MPC)

	PERFORMANCE	ECONOMIC VALUE	SOCIAL AND ENVIRONMENTAL ISSUES
PERFORMANCE	1	Importance of **Performance** relative to **Economic Value**	Importance of **Performance** relative to **Social and Environmental Issues**
ECONOMIC VALUE	Importance of **Economic Value** relative to **Performance**	1	Importance of **Economic Value** relative to **Social and Environmental Issues**
SOCIAL AND ENVIRONMENTAL ISSUES	Importance of **Social and Environmental Issues** relative to **Performance**	Importance of **Social and Environmental Issues** relative to **Economic Value**	1

Table 7.2 Comparison Scale for AHP

NUMERICAL RATING	RELATIVE IMPORTANCE	DETAILED EXPLANATION
1	Equal importance	Two activities contribute equally to the objective
3	Weak importance of one over another	Experience and judgement slightly favour one activity over another
5	Essential or strong importance	Experience and judgement strongly favour one activity over another
7	Very strong or demonstrated importance	An activity is strongly favoured over another; its dominance demonstrated in practice
9	Absolute importance	The evidence favouring one activity over another is of highest possible order of affirmation
2, 4, 6, 8	Intermediate values between adjacent scale values	When compromise is needed

Table 7.3 Matrix of Paired Comparisons for the Main Criteria

	PERFORMANCE	ECONOMIC VALUE	SOCIAL AND ENVIRONMENTAL ISSUES
PERFORMANCE	1	1	2
ECONOMIC VALUE	1	1	2
SOCIAL AND ENVIRONMENTAL ISSUES	0.5	0.5	1

Table 7.4 Normalised Pair-wise Comparison Matrix and Priority Vector

	PERFORMANCE	ECONOMIC VALUE	SOCIAL AND ENVIRONMENTAL ISSUES	PRIORITY VECTOR
PERFORMANCE	0.4	0.4	0.4	0.4
ECONOMIC VALUE	0.4	0.4	0.4	0.4
SOCIAL AND ENVIRONMENTAL ISSUES	0.2	0.2	0.2	0.2

Table 7.5 Comparison Matrix and Priority Vector for Network Performance

	Ftol	Rel	PRIORITY VECTOR
Ftol	1	2	0.667
Rel	1/2	1	0.333

Table 7.6 Comparison Matrix and Priority Vector for Social and Environmental Issues

	Acc	Ili	Ab	PRIORITY VECTOR
Acc	1	2	4	0.557
Ili	1/2	1	3	0.320
Ab	1/4	1/3	1	0.123

$\lambda_{max} = 3.018$; C.I. = 0.0092; C.R. = 0.016

Table 7.7 Quantitative Data for the Various Design Options

DESIGN OPTION	ECONOMIC VALUE	PERFORMANCE	
	Present Value ($)	Failure Tolerance	Reliability
1. THREE-LOOP DESIGN WITH A PRICING POLICY	3,810,851.25	0.850584	0.999553
2. THREE-LOOP DESIGN WITHOUT A PRICING POLICY	4,214,803.00	0.934716	0.999832
3. SINGLE -LOOP DESIGN	3,529,783.00	0.647552	0.999484

Table 7.8 Scale of Relative Intensities and Measurement Data

Numerical Rating on the AHP Comparison Scale	PERCENTAGE DIFFERENCE, d', OF COMPARED VALUES		
	ECONOMIC VALUE	RELIABILITY	FAILURE TOLERANCE
1	$0 < d' < 10$	$0 < d' < 5$	$0 < d' < 5$
2	$10 < d' < 20$	$5 < d' < 10$	$5 < d' < 10$
3	$20 < d' < 30$	$10 < d' < 15$	$10 < d' < 15$
4	$30 < d' < 40$	$15 < d' < 25$	$15 < d' < 25$
5	$40 < d' < 50$	$25 < d' < 35$	$25 < d' < 35$
6	$50 < d' < 60$	$35 < d' < 40$	$35 < d' < 40$
7	$60 < d' < 70$	$40 < d' < 45$	$40 < d' < 45$
8	$70 < d' < 80$	$45 < d' < 60$	$45 < d' < 60$
9	$d' > 80$	$60 < d' <= 100$	$60 < d' <= 100$

Table 7.9 Comparison Matrix and Priority Vector for Present Value of Project Costs

OPTIONS	1	2	3	PRIORITY VECTOR
1	1	1	1	0.328
2	1	1	1/2	0.261
3	1	2	1	0.411

$\lambda_{max} = 3.054$; C.I. = 0.027; C.R. = 0.046

Table 7.10 Comparison Matrix and Priority Vector for Failure Tolerance

OPTIONS	1	2	3	PRIORITY VECTOR
1	1	1/2	4	0.334
2	2	1	5	0.568
3	1/4	1/5	1	0.098

$\lambda_{max} = 3.025$; C.I. = 0.0123; C.R. = 0.0213

Table 7.11 Comparison Matrix and Priority Vector for Reliability

OPTIONS	1	2	3	PRIORITY VECTOR
1	1	1	1	0.333
2	1	1	1	0.333
3	1	1	1	0.333

$\lambda_{max} = 3.0$; C.I. = 0.0; C.R. = 0.0

Table 7.12 Comparison Matrix and Priority Vector for Acceptability

OPTIONS	1	2	3	PRIORITY VECTOR
1	1	1/2	1/2	0.200
2	2	1	1	0.400
3	2	1	1	0.400

$\lambda_{max} = 3.0$; C.I. = 0.0; C.R. = 0.0

Table 7.13 Comparison Matrix and Priority Vector for Health Issues

OPTIONS	1	2	3	PRIORITY VECTOR
1	1	1/2	1/2	0.200
2	2	1	1	0.400
3	2	1	1	0.400

$\lambda_{max} = 3.0$; C.I. = 0.0; C.R. = 0.0

Table 7.14 Comparison Matrix and Priority Vector for Abstraction

OPTIONS	1	2	3	PRIORITY VECTOR
1	1	3	3	0.600
2	1/3	1	1	0.200
3	1/3	1	1	0.200

$\lambda_{max} = 3.0$; C.I. = 0.0; C.R. = 0.0

Table 7.15 Results of Judge 1 for Overall Ranking of Options by Matrix Manipulation

A matrix		*B* matrix				*C* matrix = *B* x *A*	
Priority Vector for selection of the best Option		Overall Priority Matrix				Overall Ranking	
			PERFORMANCE	*ECONOMIC VALUE*	*SOCIAL & ENV. ISSUES*		
PERFORMANCE	0.400	OPTION 1	0.334	0.328	0.249	OPTION 1	0.314
ECONOMIC VALUE	0.400	OPTION 2	0.490	0.261	0.375	OPTION 2	0.375
SOCIAL & ENV. ISSUES	0.200	OPTION 3	0.177	0.411	0.375	OPTION 3	0.310

Priority Vector for Economic Value		Weight of options with respect to Net Present Value or Priority matrix for Economic Value		Composite Priorities	
			NET PRESENT VALUE		
NET PRESENT VALUE	1.000	OPTION 1	0.328	OPTION 1	0.328
		OPTION 2	0.261	OPTION 2	0.261
		OPTION 3	0.411	OPTION 3	0.411

Priority Vector for Performance		Priority matrix for Performance Sub-criteria			Composite Priorities	
			FAILURE TOLERANCE	*RELIABILITY*		
FAILURE TOLERANCE	0.667	OPTION 1	0.334	0.333	OPTION 1	0.334
RELIABILITY	0.333	OPTION 2	0.568	0.333	OPTION 2	0.490
		OPTION 3	0.098	0.333	OPTION 3	0.177

Priority Vector for Social & Environmental issues		Priority matrix for Social and Environmental issues				Composite Priorities	
			ACCEPTABILITY	*HEALTH ISSUES*	*ABSTRACTION*		
ACCEPTABILITY	0.557	OPTION 1	0.200	0.200	0.600	OPTION 1	0.249
HEALTH ISSUES	0.320	OPTION 2	0.400	0.400	0.200	OPTION 2	0.375
ABSTRACTION	0.123	OPTION 3	0.400	0.400	0.200	OPTION 3	0.375

Priority Vector for Reliability		Weight of options with respect to Reliability		Composite Priorities	
			RELIABILITY		
RELIABILITY	1.000	OPTION 1	0.333	OPTION 1	0.333
		OPTION 2	0.333	OPTION 2	0.333
		OPTION 3	0.333	OPTION 3	0.333

Priority Vector for Failure Tolerance		Weight of options with respect to Failure Tolerance		Composite Priorities	
			FAILURE TOLERANCE		
FAILURE TOLERANCE	1.000	OPTION 1	0.334	OPTION 1	0.334
		OPTION 2	0.568	OPTION 2	0.568
		OPTION 3	0.098	OPTION 3	0.098

Priority Vector for Acceptability		Weight of options with respect to Acceptability		Composite Priorities	
			ACCEPTABILITY		
ACCEPTABILITY	1.000	OPTION 1	0.200	OPTION 1	0.200
		OPTION 2	0.400	OPTION 2	0.400
		OPTION 3	0.400	OPTION 3	0.400

Priority Vector for Health Issues		Weight of options with respect to Health issues		Composite Priorities	
			HEALTH ISSUES		
HEALTH ISSUES	1.000	OPTION 1	0.200	OPTION 1	0.200
		OPTION 2	0.400	OPTION 2	0.400
		OPTION 3	0.400	OPTION 3	0.400

Priority Vector for Abstraction		Weight of options with respect to Abstraction		Composite Priorities	
			ABSTRACTION		
ABSTRACTION	1.000	OPTION 1	0.600	OPTION 1	0.600
		OPTION 2	0.200	OPTION 2	0.200
		OPTION 3	0.200	OPTION 3	0.200

Table 7.16 Results of Judge 2 for Overall Ranking of Options by Matrix Manipulation

A matrix		*B* matrix				*C* matrix = *B* x *A*	
Priority Vector for selection of the best Option		Overall Priority Matrix				Overall Ranking	
			PERFORMANCE	*ECONOMIC VALUE*	*SOCIAL & ENV. ISSUES*		
PERFORMANCE	0.333	OPTION 1	0.334	0.328	0.229	OPTION 1	0.297
ECONOMIC VALUE	0.333	OPTION 2	0.490	0.261	0.318	OPTION 2	0.356
SOCIAL & ENV. ISSUES	0.333	OPTION 3	0.177	0.411	0.453	OPTION 3	0.347
Priority Vector for Economic Value		Weight of options with respect to Present Value or Priority matrix for Economic Value				Composite Priorities	
			PRESENT VALUE				
		OPTION 1	0.328			OPTION 1	0.328
PRESENT VALUE	1.000	OPTION 2	0.261			OPTION 2	0.261
		OPTION 3	0.411			OPTION 3	0.411
Priority Vector for Performance		Priority matrix for Performance				Composite Priorities	
			FAILURE TOLERANCE	*RELIABILITY*			
FAILURE TOLERANCE	0.667	OPTION 1	0.334	0.333		OPTION 1	0.334
RELIABILITY	0.333	OPTION 2	0.568	0.333		OPTION 2	0.490
		OPTION 3	0.098	0.333		OPTION 3	0.177
Priority Vector for Social & Environmental Issues		Priority matrix for Social and Environmental Issues				Composite Priorities	
			ACCEPTABILITY	*HEALTH ISSUES*	*ABSTRACTION*		
ACCEPTABILITY	0.557	OPTION 1	0.164	0.200	0.600	OPTION 1	0.229
HEALTH ISSUES	0.320	OPTION 2	0.297	0.400	0.200	OPTION 2	0.318
ABSTRACTION	0.123	OPTION 3	0.539	0.400	0.200	OPTION 3	0.453
Priority Vector for Reliability		Weight of options with respect to Reliability				Composite Priorities	
			RELIABILITY				
		OPTION 1	0.333			OPTION 1	0.333
RELIABILITY	1.000	OPTION 2	0.333			OPTION 2	0.333
		OPTION 3	0.333			OPTION 3	0.333
Priority Vector for Failure Tolerance		Weight of options with respect to Failure Tolerance				Composite Priorities	
			FAILURE TOLERANCE				
		OPTION 1	0.334			OPTION 1	0.334
FAILURE TOLERANCE	1.000	OPTION 2	0.568			OPTION 2	0.568
		OPTION 3	0.098			OPTION 3	0.098
Priority Vector for Acceptability		Weight of options with respect to Acceptability				Composite Priorities	
			ACCEPTABILITY				
		OPTION 1	0.164			OPTION 1	0.164
ACCEPTABILITY	1.000	OPTION 2	0.297			OPTION 2	0.297
		OPTION 3	0.539			OPTION 3	0.539
Priority Vector for Health Issues		Weight of options with respect to Health Issues				Composite Priorities	
			HEALTH ISSUES				
		OPTION 1	0.200			OPTION 1	0.200
HEALTH ISSUES	1.000	OPTION 2	0.400			OPTION 2	0.400
		OPTION 3	0.400			OPTION 3	0.400
Priority Vector for Abstraction		Weight of options with respect to Abstraction				Composite Priorities	
			ABSTRACTION				
		OPTION 1	0.600			OPTION 1	0.600
ABSTRACTION	1.000	OPTION 2	0.200			OPTION 2	0.200
		OPTION 3	0.200			OPTION 3	0.200

Table 7.17 Results for Judge 3 for Overall Ranking of Options by Matrix Manipulation

A matrix		*B* matrix				*C* matrix = *B* x *A*	
Priority Vector for selection of the best Option		Overall Priority Matrix				Overall Ranking	
			PERFORMANCE	*ECONOMIC VALUE*	*SOCIAL & ENV. ISSUES*		
PERFORMANCE	0.500	OPTION 1	0.334	0.387	0.231	OPTION 1	0.321
ECONOMIC VALUE	0.250	OPTION 2	0.490	0.170	0.385	OPTION 2	0.383
SOCIAL & ENV. ISSUES	0.250	OPTION 3	0.177	0.443	0.385	OPTION 3	0.295
Priority Vector for Economic Value		Weight of options with respect to Present Value or Priority matrix for Economic Value				Composite Priorities	
			PRESENT VALUE				
PRESENT VALUE	1.000	OPTION 1	0.387			OPTION 1	0.387
		OPTION 2	0.170			OPTION 2	0.170
		OPTION 3	0.443			OPTION 3	0.443
Priority Vector for Performance		Priority matrix for Performance				Composite Priorities	
			FAILURE TOLERANCE	*RELIABILITY*			
FAILURE TOLERANCE	0.667	OPTION 1	0.334	0.333		OPTION 1	0.334
RELIABILITY	0.333	OPTION 2	0.568	0.333		OPTION 2	0.490
		OPTION 3	0.098	0.333		OPTION 3	0.177
Priority Vector for Social & Environmental issues		Priority matrix for Social and Environmental issues				Composite Priorities	
			ACCEPTABILITY	*HEALTH ISSUES*	*ABSTRACTION*		
ACCEPTABILITY	0.557	OPTION 1	0.200	0.143	0.600	OPTION 1	0.231
HEALTH ISSUES	0.320	OPTION 2	0.400	0.429	0.200	OPTION 2	0.385
ABSTRACTION	0.123	OPTION 3	0.400	0.429	0.200	OPTION 3	0.385
Priority Vector for Reliability		Weight of options with respect to Reliability				Composite Priorities	
			RELIABILITY				
RELIABILITY	1.000	OPTION 1	0.333			OPTION 1	0.333
		OPTION 2	0.333			OPTION 2	0.333
		OPTION 3	0.333			OPTION 3	0.333
Priority Vector for Failure Tolerance		Weight of options with respect to Failure Tolerance				Composite Priorities	
			FAILURE TOLERANCE				
FAILURE TOLERANCE	1.000	OPTION 1	0.334			OPTION 1	0.334
		OPTION 2	0.568			OPTION 2	0.568
		OPTION 3	0.098			OPTION 3	0.098
Priority Vector for Acceptability		Weight of options with respect to Acceptability				Composite Priorities	
			ACCEPTABILITY				
ACCEPTABILITY	1.000	OPTION 1	0.200			OPTION 1	0.200
		OPTION 2	0.400			OPTION 2	0.400
		OPTION 3	0.400			OPTION 3	0.400
Priority Vector for Health Issues		Weight of options with respect to Health issues				Composite Priorities	
			HEALTH ISSUES				
HEALTH ISSUES	1.000	OPTION 1	0.143			OPTION 1	0.143
		OPTION 2	0.429			OPTION 2	0.429
		OPTION 3	0.429			OPTION 3	0.429
Priority Vector for Abstraction		Weight of options with respect to Abstraction				Composite Priorities	
			ABSTRACTION				
ABSTRACTION	1.000	OPTION 1	0.600			OPTION 1	0.600
		OPTION 2	0.200			OPTION 2	0.200
		OPTION 3	0.200			OPTION 3	0.200

Table 7.18 Results of the Chief Decision-maker for Overall Ranking of Judges by Matrix Manipulation

A matrix		*B* matrix				*C* matrix = *B* x *A*	
Priority Vector for factors affecting judgment		Overall Priority Matrix				Overall Ranking	
			TECHNICAL KNOWLEDGE	EXPERIENCE	PROJECT KNOWLEDGE		
TECHNICAL KNOWLEDGE	0.333	JUDGE 1	0.500	0.250	0.250	JUDGE 1	0.333
EXPERIENCE	0.333	JUDGE 2	0.250	0.500	0.250	JUDGE 2	0.333
PROJECT KNOWLEDGE	0.333	JUDGE 3	0.250	0.250	0.500	JUDGE 3	0.333
Priority Vector for Technical Knowledge		Weight of Judges with respect to Technical Knowledge				Composite Priorities	
			TECHNICAL KNOWLEDGE				
		JUDGE 1	0.500			JUDGE 1	0.500
TECHNICAL KNOWLEDGE	1.000	JUDGE 2	0.250			JUDGE 2	0.250
		JUDGE 3	0.250			JUDGE 3	0.250
Priority Vector for Experience		Weight of Judges with respect to Experience				Composite Priorities	
			EXPERIENCE				
		JUDGE 1	0.250			JUDGE 1	0.250
EXPERIENCE	1.000	JUDGE 2	0.500			JUDGE 2	0.500
		JUDGE 3	0.250			JUDGE 3	0.250
Priority Vector for Project Knowledge		Weight of Judges with respect to Project Knowledge				Composite Priorities	
			PROJECT KNOWLEDGE				
		JUDGE 1	0.250			JUDGE 1	0.250
PROJECT KNOWLEDGE	1.000	JUDGE 2	0.250			JUDGE 2	0.250
		JUDGE 3	0.500			JUDGE 3	0.500

Table 7.19 Final Ranking of Options by Matrix Manipulation

A matrix		*B* matrix				*C* matrix = *B* x *A*	
Priority Vector for Judges		Final Priority Matrix				Final Option Ranking	
			JUDGE 1	*JUDGE 2*	*JUDGE3*		
JUDGE 1	0.333	OPTION 1	0.314	0.297	0.321	OPTION 1	0.311
JUDGE 2	0.333	OPTION 2	0.375	0.356	0.383	OPTION 2	0.372
JUDGE 3	0.333	OPTION 3	0.310	0.347	0.295	OPTION 3	0.317

Table 7.20 Sensitivity Analysis - Scenario A

A matrix		*B* matrix				*C* matrix = *B* x *A*	
Priority Vector for Judges		Final Priority Matrix				Final Option Ranking	
			JUDGE 1	*JUDGE 2*	*JUDGE3*		
JUDGE 1	0.500	OPTION 1	0.314	0.297	0.321	OPTION 1	0.311
JUDGE 2	0.300	OPTION 2	0.375	0.356	0.383	OPTION 2	0.371
JUDGE 3	0.200	OPTION 3	0.310	0.347	0.295	OPTION 3	0.318

Table 7.21 Sensitivity Analysis - Scenario B

A matrix		*B* matrix				*C* matrix = *B* x *A*	
Priority Vector for Judges		Final Priority Matrix				Final Option Ranking	
			JUDGE 1	*JUDGE 2*	*JUDGE3*		
JUDGE 1	0.250	OPTION 1	0.314	0.297	0.321	OPTION 1	0.307
JUDGE 2	0.500	OPTION 2	0.375	0.356	0.383	OPTION 2	0.368
JUDGE 3	0.250	OPTION 3	0.310	0.347	0.295	OPTION 3	0.325

Table 7.22 Sensitivity Analysis - Scenario C

A matrix		*B* matrix				*C* matrix = *B* x *A*	
Priority Vector for Judges		Final Priority Matrix				Final Option Ranking	
			JUDGE 1	*JUDGE 2*	*JUDGE3*		
JUDGE 1	0.300	OPTION 1	0.314	0.297	0.321	OPTION 1	0.314
JUDGE 2	0.200	OPTION 2	0.375	0.356	0.383	OPTION 2	0.376
JUDGE 3	0.500	OPTION 3	0.310	0.347	0.295	OPTION 3	0.310

CHAPTER EIGHT

INTEGRATED MODEL FORMULATION AND APPLICATION

8.1 INTRODUCTION

There are various key issues involved in the upgrading of a water distribution network such as the timing and magnitude of upgrading and the consideration of deterioration of hydraulic and structural capacity of pipes over time within a limited budget. These key issues need to be addressed using a system-wide model that incorporates the performance of the system explicitly. This model should be set in a multi-objective framework to include economic, social and environmental issues together with issues related to the level of service like reliability and failure tolerance.

The primary objective of this chapter therefore, is the formulation of an Integrated Model that can be used for optimal long-term capacity expansion strategies, ensuring system reliability, hydraulic and economic compatibility; socio–environmental issues notwithstanding. The Integrated Model is a combination of individual modules for Network Design, Hydraulic Simulation, Assessment of Network Performance and the Analytic Hierarchy Process. The details of each of the separate modules have been presented earlier in Chapters 4, 5 and 7.

In the previous chapter, the module for the Analytic Hierarchy Process has been applied to a hypothetical network to demonstrate how the process of multi-criteria decision-making has been handled in this thesis. In this chapter, the Integrated Model is applied to a hypothetical water distribution network and a real-life water distribution network to determine the best upgrading strategy, its cost, timing and magnitude; and to show the practicability of the proposed formulation.

As reviewed earlier in Chapter 3, Halhal et al. (1997) have used Genetic Algorithms in a system-wide model to maximise benefits subject to limits on funding. The benefits considered are hydraulic, physical integrity, flexibility and quality. Dandy and Engelhardt (2001) have used a genetic algorithm technique to find a near optimal schedule for the replacement of water supply pipes and included repair and damage costs. The formulation allows for multiple time-steps and an evaluation of the hydraulic performance of the network when replacement of different pipe sizes is done. These system-wide models are very versatile and have been applied successfully to real-life water distribution systems however, they are quite complex and computationally demanding. Furthermore, these models do not explicitly include the assessment of network performance in a multi-objective framework.

These weaknesses have been addressed in the Integrated Model proposed herein. To reduce on the level of complexity and the computational demands, the proposed model involves predefining flows using either the maximum entropy flow distribution algorithm or the shortest path flow distribution algorithm, together with an in-built procedure for reducing the number of segment diameters on the candidate list for each link during optimisation. Using the module for the Analytic Hierarchy Process, the Integrated Model explicitly incorporates the performance of the system, economic, social and environmental issues in a multi-objective decision making framework. As mentioned earlier in Chapter 7, the key performance assessment parameters used are reliability and failure tolerance.

8.2 FORMULATION OF THE INTEGRATED MODEL

8.2.1 Individual Modules of the Integrated Model

8.2.1.1 The Network Design Module

This module is used to obtain least cost network designs for different design horizons and executed using a FORTRAN program (UPSIZE) that has been developed in this research as detailed in Chapter 4. The module is based on linear optimisation and considers issues like deterioration of hydraulic and structural pipe capacity, inflation and time value of money, cost recovery, and pricing policies. It is combined with a dynamic programming section that is used to determine the optimal scheduling, cost and magnitude for the long-term upgrading strategy of each design option.

8.2.1.2 The Hydraulic Simulation Module

The module is executed using a FORTRAN program (CHDSM) that has been developed in this study as detailed in Chapter 5. This module is based on head-dependent network analysis in order to provide a realistic behaviour of water distribution systems, especially in abnormal operating conditions when some components are not available or when there is excessive demand for fire-fighting, etc.

8.2.1.3 The Performance Assessment Module

This module is used to replicate different operating conditions and network configurations that are fed into the hydraulic analysis model for analysis of the network in various stressed states. It utilises the head driven analysis results obtained to calculate key performance assessment measures like reliability and component failure tolerance as detailed in Chapter 5. Unavailability of pipes has been calculated using a formula from Cullinane et al. (1992). The simulation of the water distribution network with the broken pipes isolated has been done using the Critical-node Head Driven Simulation Method (CHDSM) as described in the same chapter. The Performance assessment module automatically carries out network analysis for different combinations of unavailable components up to a point when it is assessed that any further layouts would not make a significant improvement on the reliability value as detailed in Chapter 5.

8.2.1.4 Module for the Analytic Hierarchy Process

The Analytic Hierarchy Process is a multi-criteria decision-making tool used for selecting the best option based on various criteria (Saaty and Vargas, 2001). In the AHP, information is sub-divided into a hierarchy of criteria, sub-criteria, and alternatives. The information is then processed using pair-wise comparison matrices for each level of the hierarchy, to determine the relative ranking and priority vectors of alternatives. Both qualitative and quantitative criteria can be compared using informed judgments to derive weights and priorities.

An EXCEL spreadsheet program has been developed to execute the module for the AHP as detailed in Chapter 7. In this chapter, four people have been involved in the decision-making process. The leader or facilitator is referred to herein as the chief decision-maker and the other three members are the judges who are referred to as Judge 1, Judge 2 and Judge 3. Each judge has been involved in making judgments by use of comparison matrices. The chief decision-maker has been responsible for the evaluation of the judges to assess the weight of their contribution to the overall group decision. This evaluation is based on three main criteria, their technical knowledge, experience and current project knowledge.

The hierarchical structure for the AHP that has been adopted for this research is the same as that in Figure 7.4. The first level of the hierarchy shows that the overall goal is to select the best design option i.e. one that minimizes the cost, minimizes environmental damage, maximises network performance and social benefits. The second level has three main criteria (network performance, economic value, social and environmental issues) and the third level has the sub-criteria, which contribute to the achievement of the overall goal. Finally, the fourth level consists of the three design options. The economic value is expressed in terms of the Present Value (PV) of the costs for each of the design options. The network performance is defined in terms of the reliability and the failure tolerance of the network. The social and environmental issues addressed in this research are sub-divided into acceptability, abstraction and health issues sub-criteria (Canter, 1996) as detailed in Chapter 7.

The comparison matrix elements that are whole numbers have been obtained by assigning a numerical rating from the Comparison Scale of Table 7.2 to reflect the relative importance. For the quantitave data, it has been agreed by the judges to adopt the Scale of Relative Intensities and Measurement Data as shown in Table 7.8 as the basis of comparing the quantitative criteria.

8.2.2 Generalised Integrated Model Linkages

The individual modules described in Section 8.2.1 are linked together to form the Integrated Model as shown in Figure 8.1. For various design options, the Network Design Module is used to obtain least cost designs, timing and magnitude of upgrading by linear optimisation. The Hydraulic Simulation Module is then used to check the each network design is hydraulically feasible and to simulate stressed network conditions. The results are fed into the Performance Assessment Module to calculate the reliability and failure tolerance or redundancy of each network design. The Analytic Hierarchy Process is then used to obtain the overall ranking of the design options by which the choice of the best upgrading strategy is made.

In the next two sections, the Integrated Model is applied to a hypothetical network and a real-life network to determine optimal long-term upgrading strategies. In each of these sections, the results are presented and discussed under appropriate sub-headings that correspond to individual modules of the Integrated Model for ease of reference. Network upgrading is by paralleling and/or replacement of pipes. Execution of the model has been carried out on a personal computer with 128 MB of random access memory (RAM) and a microprocessor speed of 1.2 MHz.

8.3 MODEL APPLICATION TO A HYPOTHETICAL NETWORK

8.3.1 Problem Formulation

8.3.1.1 Description of the Network

The Integrated Model has been used in selecting the best network for serving four demand nodes out of three hypothetical network design options. The execution of each individual module of the model is as detailed in Section 8.2. Design Option 1 is the single-loop layout in Figure 8.2 designed using the shortest path flow distribution method for link-flow allocation (Orth, 1986). This layout is a combined loop and branched network as a compromise design between a highly redundant fully looped network and a branched network with low redundancy. Design Option 2 is a three-loop layout in Figure 8.3 designed with maximum entropy flows, but without any pricing policy. Design Option 3 is a three-loop layout similar to that of Option 2 and it is designed with maximum entropy flows, but with a pricing policy. The main difference between Option 3 and Option 2 is that the pricing policy implemented for Option 3 leads to a reduction in water consumption (nodal demands) and thus a lower total demand for the entire design horizon.

8.3.1.2 Design Data

The design input data is quite similar to that used for the networks in Chapter 4. However, the nodal base demands are assumed to increase at an annual rate (*DGR*) of 3% as opposed to 4% that was used in Chapter 4. This has been done specifically to assess the sensitivity of variation of the rate of growth of demand on the upgrading strategy. The rest of the design data input in this chapter has been briefly reproduced from Chapter 4 for ease of reference. The overall design horizon, *d*, is 20 years and the nodal base demands or the nodal demands in the first year of the design horizon for Nodes 2, 3, 4 and 5 are 3l/s, 4l/s, 12l/s and 12l/s. The network loading that has been adopted for this study is that of a combination of fire demand and the peak hour demands with a peak hour factor of 2.0 (Directorate of Water Development, 1999). Thus, design demands for a given design period are obtained by multiplying the nodal base demands by the peak hour factor to obtain peak hour demands which are then forecast or projected over the design period. Finally, the fire demand is added to obtain the overall nodal design demands. For example, 20-year nodal design demands are shown in Figures 8.2 and 8.3 for Options 1 and 2, respectively.

For Option 3, the design demands are reduced due to enforcement of the pricing policy. For this design option, a tariff increase ratio, (P_t/P_0), has been taken as 1.333 (a 33.33% increase in the water tariff normally implemented in the last one or two years of a given design period). The price elasticity of demand, *PREL*, has been assumed to be –0.2 (Dandy et al. 1985). This combination of tariff increase ratio and price elasticity of demand culminates in reduced nodal design demands. For example, for a 20-year design horizon, the design nodal demand values for Nodes 2, 3, 4, and 5 are 14.63l/s, 12.84l/s, 38.51l/s and 38.51l/s, respectively. (Table B-8.6 in Appendix B). As detailed in Chapters 2 and 4, a proportion (25%) of a fire demand value of 20l/s (at Node 2) has been used to avoid over-designing (Twort et al., 2000).

All links are 1000m long and the pipes used are made of PVC. The source head is 70m while each of the demand nodes have elevations of 0m and a minimum service pressure of 15m. Hazen-Williams coefficient is 130 for all new pipes. All costs in are in US dollars. For Phase II, v varies from 7 to 14 years, i.e. the lower limit for the end of Phase I ($T1$=7 years) and the upper limit for the end of Phase I ($T2$ =14 years). $FCF(LU_i)_{ij}$, the failure cost factors for land use were taken as 4 each of the links. The compound interest rate, b = 8; the discount rate, r = 8%; the initial roughness, $e_{0_{ijm}}$ = 0.0021mm and the roughness growth rate, a_{ijm} = 0.025 (mm/year) (Bhave, 1991). Pipe cost constants (Eqs. 4.4, 4.6 and 4.8) that have been used are $\gamma_p = 32.093$; $c_p = c_r = 3.7$; $\gamma_r = 33.928$ and $\gamma_{br} = 108.87$. The pipe cost exponent $\Phi = 0.6067$.

In order to limit the segment diameters, the minimum velocity used in this study is 0.5m/s, the maximum velocity is 3m/s (Dandy and Engelhardt, 2001) and a maximum hydraulic gradient of 50m/km (Twort et al., 2000). Setting-up costs at the beginning of each design period or phase = \$100,000. VC, the cost coefficient = 130 and VE, the cost exponent = 1.6. These coefficients have been obtained as detailed in Chapter 4.

Option 1 has one loop constraint, two service pressure constraints between the source and Nodes 4 and 5, five summations of link length constraints and non-negativity constraints for the link segment lengths. Options 2 and 3 each have three loop constraints, two service pressure constraints between the source and Nodes 4 and 5, seven summations of link length constraints and non-negativity constraints for the link segment lengths.

8.3.2 Appraisal of the Results

8.3.2.1 Network Design

The Network Design module has been used to obtain the best upgrading strategy and the timing of the upgrading for the three design options. The results are shown in

Tables 8.1 to 8.6 for the first and second phases of the three design options. The overall costs for the three designs are also shown in Tables 8.7, 8.8 and 8.9.

From Table 8.7 the cheapest cost strategy for the hypothetical network design Option 1 has a value of $3,257,143 and it is to design and install a capacity for a 13-year demand in Phase I; then upgrade to the ultimate or 20-year demand capacity in Phase II. This optimal design requires 19 variables in Phase I and 39 variables in Phase II. The overall time required by the computer's central processing unit (CPU-time) for this option is 0.270 seconds. Table 8.8 shows that the cheapest cost strategy for the hypothetical network design Option 2 has a value of $3,732,092.50 and it is to design and install a capacity for a 13-year demand in Phase I; then increase capacity to meet the 20-year demand in Phase II. This optimal design requires 20 variables in Phase I and 46 variables in Phase II. The overall CPU-time required for this option is 0.391 seconds. The overall CPU-time referred to covers the different linear optimisation designs for various Phase I and Phase II design periods, with the Phase I design period varying from 7 years to 14 years as shown in Tables 8.7, 8.8 and 8.9. The time also covers the dynamic programming to determine the timing and magnitude of the upgrading for each design option.

The pricing policy implemented for the hypothetical network design Option 3 results in a two-year delay of expansion. From Table 8.9, the cheapest cost strategy for this option has a value of $3,474,612.50 and it is to design and install a capacity for an 8-year demand in Phase I; delay expansion or upgrading for 2 years and then upgrade the network to a 16-year demand capacity (20 years less the delays due to pricing of 2 years in each design phase), to serve the entire 20-year design horizon. Note that the 8-year demand, is obtained by taking the base demand or the demand of the first year of the 20-year design horizon and projecting or forecasting it over a period of 8 years. Similarly, the 16-year demand is obtained by projecting the demand of the first year of the 20-year design horizon, over a period of 16 years. This optimal design requires 18 variables in Phase I and 45 variables in Phase II. The overall CPU-time required for this option is 0.348 seconds.

8.3.2.2 Network Analysis

Network analysis of the optimal designs of Tables 8.4 to 8.6 has been carried out to check that the results are consistent using the Hydraulic Simulation Module. The link data and results for Options 1, 2 and 3 of the hypothetical network designs are in Appendix B, Tables B-8.1, B-8.3 and B-8.5 respectively. The nodal data and results for Options 1, 2 and 3 are in Tables B-8.2, B-8.4 and B-8.6 respectively.

8.3.2.3 Network Performance Measures

The network performance data for the optimal designs shown in Tables 8.1 to 8.6 has been obtained using the Performance Assessment Module. The hypothetical network performance results for the optimal designs obtained above are presented in Tables 8.10, 8.11 and 8.12 for Options 1, 2 and 3 respectively. From Table 8.10, the first design option has a probability that all components are available of 0.998354 (corresponding to layout No. 1), a network reliability of 0.999478 and a failure tolerance of 0.683046. The overall CPU-time required for execution of the Performance Assessment Module for this option is 0.660 seconds. The overall CPU-time referred to covers the head-dependent analysis of the all the 16 different layouts of the network with the unavailability of components as shown in Table 8.10. The total demand supplied or total outflow for each of the layouts is also shown.

Table 8.11 shows results of Option 2 with a probability that all components are available of 0.996982 (corresponding to layout No. 1), a network reliability of 0.999551 and a failure tolerance of 0.851361. The overall CPU-time required for execution of the Performance Assessment Module for this option is 1.313 seconds. The overall CPU-time referred to covers the head-dependent analysis of the all the 29 different layouts of the network with the unavailability of components as shown in Table 8.11. The total demand supplied or total outflow for each of the layouts is also shown.

The network performance results of Option 3 are presented in Table 8.12. For this option, the probability that all components are available is 0.996816 (corresponding to layout No. 1). The network reliability is 0.999498 and a failure tolerance of 0.842412. The overall CPU-time required for this option is 1.172 seconds. The overall CPU-time referred to covers the head-dependent analysis of the all the 29 different layouts of the network with the unavailability of components as shown in the table.

Table 8.13 shows the summary of the results for the hypothetical network design options. In this table, the probability that all components are available, the network reliability, the component failure tolerance and the present value of costs for each of the three design options are shown.

8.3.2.4 Analytic Hierarchy Process

The results in Table 8.13 have been fed into the Analytic Hierarchy Process Module to assist in the final decision-making process for the optimal long-term upgrading of the hypothetical water distribution network.

Table B-8.7 in Appendix B shows the comparison matrix and priority vector for the network performance criterion. The matrix elements reflect the relative importance of the reliability (Rel) and failure tolerance (Ftol) sub-criteria when compared to each other. For example, a rating of 2 on the Comparison Scale of Table 7.2 means that the superiority of failure tolerance over reliability falls between equal importance and weak importance. The priority vectors are obtained as detailed in Chapter 6. Table B-8.8 shows the comparison matrix and priority vector for the social and environmental issues criterion. The matrix elements reflect the relative importance of the acceptability (Acc) and health issues (Hi) and abstraction (Ab) sub-criteria when compared to each other.

Tables B-8.9 to B-8.11 in Appendix B show the comparison matrices and priority vectors of the options with respect to the quantitative sub-criteria, which have been obtained using the Scale of relative intensities and measurement data (See Table 7.8). Table B-8.9 contains the comparison matrix and priority vector of the three options with respect to the reliability sub-criterion. The elements of the comparison matrix are all unity, meaning that the percentage difference between the reliability values is less than 5% and thus the numerical rating of 1. The priority vector (0.333, 0.333, 0.333) for Options 1, 2 and 3 suggests that all options have equal weight in terms of reliability or that there is no significant difference in the reliability values. Table B-8.10 contains the comparison matrix and priority vector of the three options with respect to the failure tolerance sub-criterion. The priority vector (0.111, 0.444, 0.444) suggests that Options 2 and 3 have equal weight in terms of failure tolerance and about four times more than that Option 1. Table B-8.11 contains the comparison matrix and priority vector of the three options with respect to the sub-criterion for the present value of project costs. The priority vector (0.411, 0.261, 0.368) for Options 1, 2 and 3 suggests that in terms of the present value of costs, Option 1 is the most favourable option followed by Option 3 and then Option 1. In all cases except when the matrix elements are less than 3, the maximum eigenvalue, λ_{max}, the consistency index, C.I. and the consistency ratio, C.R. have been calculated. These consistency ratios are less than 0.1 meaning that the respective comparison matrices are consistent. Therefore there is no need for revision of the pair-wise judgments (see Sub-section 7.3.4.3).

Table B-8.12 in Appendix B shows the main comparison matrix and priority vector according to the judgment of Judge 1. The comparison matrix elements reflect the relative importance of the network performance (Perf), economic value (Ev) and social and environmental issues (S&E) criteria when compared to each other. The priority vector (0.4, 0.4, 0.2) for suggests that Judge 1 considers the network performance and economic criteria as equally important and twice as important as the social and environmental criterion. Tables B-8.13 to B-8.15 present the comparison matrices and priority vectors of the options with respect to the qualitative sub-criteria, acceptability, health issues and abstraction according to the judgement of Judge 1. The comparison matrix elements are obtained in a similar way as those in Table B-8.12.

Table B-8.16 in Appendix B shows the main comparison matrix and priority vector (0.333, 0.333, 0.333) according to Judge 2. Tables B-8.17 to B-8.19 present the comparison matrices and priority vectors of the options with respect to the qualitative sub-criteria, acceptability, health issues and abstraction according to Judge 2.

Table B-8.20 shows the main comparison matrix and priority vector (0.5, 0.25, 0.25) according to Judge 3. Tables B-8.21 to B-8.23 present the comparison matrices and priority vectors of the options with respect to the qualitative sub-criteria, acceptability, health issues and abstraction according to Judge 3.

The overall ranking of the hypothetical network design options by Judge 1 has been obtained by matrix manipulation at different levels of the hierarchy as detailed in Chapter 7 and presented in Table 8.14. According to this decision-maker, the overall ranking of Options 1, 2 and 3 is (0.313, 0.342, 0.346) meaning that the best alternative is Option 3, which has the highest priority value.

The overall ranking of the hypothetical network design options by Judge 2 has been shown in Table 8.15. According to this decision-maker, the overall ranking of Options 1, 2 and 3 is (0.370, 0.325, 0.305) meaning that the best option is Option 1. Similarly, Table 8.16 shows the overall ranking of Options 1, 2, and 3 according to Judge 3 in a vector (0.309, 0.356, 0.334). This means that Judge 3 considers Option 2 to be the best alternative.

The decision-making capability of the judges with respect to the influence of their judgment on the ultimate decision has been ranked and given a priority assignment by the chief decision-maker with results summarised in Table 8.17. From this table, the overall ranking of Judge 1, 2 and 3 is given by the vector (0.333, 0.333, 0.333) meaning that individual judgments of the judges have an equal contribution to the final decision.

The vectors of individual overall ranking of the options by Judge 1 (0.313, 0.342, 0.346), Judge 2 (0.370, 0.325, 0.305) and Judge 3 (0.309, 0.356, 0.334) have been aggregated into the final priority matrix in Table 8.18. The product of this final priority matrix and the priority vector for judges (0.333, 0.333, 0.333) yields the vector of final ranking of Options 1, 2 and 3 as (0.331, 0.341, 0.328), and this represents the final group decision. The highest priority value is 0.341 meaning the best design option for long-term upgrading of the network is Option 2, the three-loop network design without any pricing policy.

8.3.3 Discussion

8.3.3.1 Network Design

The results of the three design options in Tables 8.4 to 8.6 generally show that paralleling is preferred to replacement probably because it is cheaper and that the rate of deterioration of the pipes is quite low. A higher rate of deterioration might perhaps have required a more balanced use of both paralleling and replacement to meet the velocity and maximum hydraulic gradient requirements.

The detailed costs for the three hypothetical network designs shown in Tables 8.7, 8.8 and 8.9 show an increasing trend in costs for Phase I and a decreasing trend in costs for Phase II, as the Phase I design period increases. This is generally due to the fact the as time for the Phase I design period increases, the design demand increases too and thus a larger capacity has to be designed for. A larger capacity implies an increase in installed capacity costs. On the other hand, the decreasing trend in Phase II costs as the Phase I design period increases is due to the fact that the incremental capacity to be designed for in Phase II, decreases as the Phase I design period increases. The lower the capacity designed for in Phase I, the higher the incremental capacity one would have to design for in Phase II to meet the ultimate demand over the 20-year design horizon.

The optimal design costs for Options 1, 2, and 3 are \$3,257,143, \$3,732,092.50 and \$3,474,612.50. Option 1 (the single loop network) has the lowest cost mainly due to the fact that it has fewer links than the other two options. Option 3 is cheaper than Option 2 mainly due to the enforcement of the pricing policy. The increase in the price of water combined with the price elasticity of demand, leads to reduced consumption (Twort et al., 2000) and a delay in the need to upgrade the network. It also means that overall total demand to be designed for is lower as shown in Table B-8.6 (total demand of 104.49l/s) compared to the total demand for Option 2, of 116.99l/s in Table B-8.4.

As mentioned earlier, this hypothetical network was selected with design data similar to that used in Chapter 7, apart from the lower demand growth rate of 3% as opposed

4% that was used earlier. This was done specifically to assess the impact of variation of the rate of growth of demand. The total network demand values shown in Figures 8.2 and 8.3 are lower than those shown in Figures 7.1, 7.2 and 7.3. These values show as expected, that the lower the rate of growth of demand, the lower the design demand for a given design horizon. This also implies lower costs of the designs as seen from Table 8.13 compared to the higher costs in Table 7.7. From these tables, it has been noted that the values of failure tolerance and reliability do not necessary follow a specific trend with respect to variation of the demand growth rate. It may be concluded that whereas the design demands and costs are sensitive to variation of the demand growth rate, this may not be the case with failure tolerance and reliability. Therefore, it may not be easy for one to predict whether these values would increase or decrease due to a specific change in the rate of growth of demand without the appropriate calculations.

The values of overall CPU time for execution of Network Design Module on Options 1, 2 and 3 (0.270, 0.391 and 0.348 seconds, respectively) are quite low considering that they cover different linear optimisation designs for various Phase I and II design periods; together with dynamic programming to determine the timing and magnitude of the upgrading for each design option. This is a good reflection of the efficiency of the Network Design Module. Design Option 1 has the lowest overall CPU time probably because it has only one loop and five links compared to Options 2 and 3 which have three loops and seven links each. On the other hand, Option 2 has the highest CPU time probably because it has the largest number of variables. The CPU time increases with an increase in the total number of variables.

8.3.3.2 Network Performance Measures

The network performance results in Tables 8.10, 8.11 and 8.12 show values of overall CPU time for Options 1, 2 and 3, which are 0.660, 1.313 and 1.172 seconds respectively. These values are quite low considering that they cover the head-dependent analysis of all the 16, 29, and 29 different layouts of the network for Options 1, 2 and 3 respectively, with the unavailability of components as shown in Tables 8.10, 8.11, and 8.12. This shows that the efficiency of the Network

Performance module is considerably high. Design Option 1 has the lowest CPU time probably due to the fact that it has the lowest number of links.

From the summary of network performance results summarised in Table 8.13, the probability, $p(0)$, that all components are available for Options 1, 2 and 3 are 0.998354, 0.996982 and 0.996816. This value for Option 1 is the highest probably due to the fact that $p(0)$ is the product of the individual link reliabilities and that Option 1 has the lowest number of links; thus the lowest product of the individual link reliabilities.

From Table 8.13, the reliability values of the Options 1, 2 and 3 are 0.999478, 0.999551 and 0.999498 respectively. Design Option 2 has the highest reliability perhaps due to the slightly larger components in the network ensuring the highest ability of the network to meet demands at adequate pressure under normal and abnormal conditions. Flows are rerouted through large pipes when a pipe becomes unavailable. The single loop network has the lowest reliability possibly because the large demands on nodes 4 and 5, being terminal nodes served by single links, cannot be fully satisfied by most network simulations of the layouts with unavailable pipes in the process of calculating the network reliability. A further justification of this reliability value being the lowest might be the fact that the shortest path flow distribution method was used for single-loop network design. This method of flow distribution tends to concentrate larger flows on particular supply paths (trunk mains). Thus a lower fraction of demand is likely to be satisfied at desirable pressure when the trunk mains become unavailable, since large flows will tend to get rerouted through the remaining paths with smaller diameters leading to high head losses. Maximum entropy flow distribution attempts to balance the flow by a more even distribution between the alternative supply paths to a particular demand node. The unavailability of one of the paths is less likely to lead to head losses as heavy as those by the shortest path flow distribution method.

The failure tolerance for Option 1 is the lowest and significantly different from that of Options 2 and 3, probably because the layout for Option 1 (Figure 8.2) is partially looped and partially dendritic. This limits the number of alternative paths from the source to the supply nodes and further reduces performance of the network when components become unavailable. On the other hand, the layout for Options 2 and 3 is purely looped options (Figure 8.3) and is therefore more likely to perform better in times of partial failure.

It has to be noted that the network reliability results for the different design options are arguably very close to each other perhaps due to the fact that the individual component reliabilities of the networks are high. These results would probably not be enough for one to distinguish between the designs using this reliability criterion alone, though reliability is a commonly used performance assessment parameter. On the other hand, the values of failure tolerance for the Options 1, 2 and 3 shown in Table 8.13 as 0.683046, 0.951361 and 0.842412 respectively, evidently distinguish between the designs. Failure tolerance is more likely to expose the vulnerability of the network under stress and differentiate clearly between networks with high reliability but with varying capabilities to perform under stress (Kalungi and Tanyimboh, 2001). This emphasizes the point that reliability and failure tolerance should be used together as performance assessment parameters.

It is also noted from Table 8.13 that the costs in terms of present value for Options 1, 2 and 3 are $3,257,143, $3,732,092.50 and $3,474,612.50 respectively. It is clear that the cheapest design, Option 1, also has the lowest value of failure tolerance. Whereas Option 1 is the most favourable option in terms of costs, it is the least favourable in terms of performance when some components are not available. The AHP is very helpful in such instances of conflicting decision factors.

8.3.3.3 Analytic Hierarchy Process

The vectors of individual overall ranking of the options which are (0.313, 0.342, 0.346), (0.370, 0.325, 0.305) and (0.309, 0.356, 0.334) by Judges 1, 2 and 3 show that each individual judge has a different Option as the best Option. For instance, Judge 1 has ranked Option 3 as the best Option whereas Judges 2 and 3 have ranked Options 1 and 2, respectively, as the best options. However this situation is not uncommon and it is resolved easily through the final priority matrix in Table 8.18. In this table, the final priority matrix of individual rankings is multiplied by the priority vector for judges (0.333, 0.333, 0.333) to yield the vector of final ranking of

Options 1, 2 and 3 as (0.331, 0.341, 0.328), and this represents the final group decision. The highest priority value is 0.341 meaning the best design option for long-term upgrading of the network is Option 2, the three-loop network design without any pricing policy.

Since the AHP is a guidance tool for decision-making, one could argue that Options 1 and 3, having almost equal weight, are not significantly different and are therefore equally less favourable than Option 2.

8.4 MODEL APPLICATION TO A REAL-LIFE NETWORK

In order to assess the practicability of the Integrated Model, it has been applied to a real-life network, which is a proposed design for a town called Wobulenzi in Uganda (Associated Consulting Engineers, 1995a).

8.4.1 Background Information

Uganda is located in East Africa and lies astride the Equator. The population is about 20.9 million people with an average national population growth rate of 2.8%. Most of the country is a plateau more than 1,000m above sea level. The average annual rainfall is 1380mm. The total area of the country is about 241,000 square kilometres of which about 44,000 are covered by fresh water bodies (UNCTAD-ICC, 2001). Uganda is the source of the Great River Nile and Africa's largest fresh water lake, the Lake Victoria. The potential for water reserves in terms of surface water and underground water is very high.

The National Water and Sewerage Corporation is a government body responsible for water supply, operation and control of water and sewerage systems for the large towns and cities (with more than 50,000 people) in Uganda. The Directorate of Water Development is responsible for the small towns. Wobulenzi town was selected under the Small Towns Water and Sanitation Project (STWSP) of the Directorate of Water Development for a water supply system. This Small Towns Water and Sanitation Project is a community-based project in which the beneficiaries make a contribution towards the cost of the system (as a sign of their commitment) and the government meets the rest of the costs through a loan facility. Cost recovery is through payment of water tariffs by the consumers. The community participates in the decision-making process from project inception through to the commissioning of the project, and during the operation and maintenance of the system.

Each of the towns benefiting from the STWSP forms Water User Groups (WUGs) comprising of households in the radius of about 250 to 500 metres. Each WUG owns, operates, maintains and collects revenue from a metered communal stand post located in the center of the group. For this purpose, and for the responsibility of the group's sanitation and hygiene education, each WUG elects five members who form the Water and Sanitation Committee (WSC) and head the group. Each WSC is made up of a chairman, treasurer, secretary and two committee members. The levels of service for water supply that are available to the group members are a metered in-house connection, a metered yard tap connection or the communal stand post depending upon affordability.

Wobulenzi town is situated 49km to the North of Kampala, the capital city of Uganda. Associated Consulting Engineers are the consultants that were contracted by the Directorate of Water Development to carry out the design of the system in 1994. The population of the town in 1995 was 10,640 with a population growth rate of 4% (Associated Consulting Engineers, 1995b). The proposed layout of the water distribution network (Associated Consulting Engineers, 1995a) by the consultants is presented in Figure 8.4. This figure shows the different Water User Groups, the business district located along the main road from Kampala to Luweero town and the location of the reservoir on Kigulu hill. The sources of water available at that time in the town were protected and unprotected springs, hand dug wells, boreholes fitted with hand pumps most of which are old with water that has a high iron content and a borehole fitted with a motorised pump to serve the town center. These water sources are not only insufficient for the town, but they are also contaminated. These are the main reasons why the government of Uganda embarked on the project for a water supply system to serve the whole town (Associated Consulting Engineers, 1995a).

8.4.2 Problem Formulation

8.4.2.1 Description of the Network

The Integrated Model has been used in selecting the best network design out of three design options, for serving Wobulenzi town. The layout of the designs used in this case study is based on the network design proposed by the consultants for the Wobulenzi water supply project (Associated Consulting Engineers, 1995a). A brief description of the network they proposed is presented next. The network is supplied by a reservoir at Kigulu Hill. It is made up of PVC pipes with sizes ranging from 150mm down to 40mm. The total length of pipes is about 8.5km. The network has 28 links and 25 demand nodes. A considerable proportion of the network is made of small diameter pipes (40mm and 50mm diameter pipes), located at branched ends of the network.

In this study, the consultant's proposed layout of Figure 8.4 has been reduced by skeletonization to the network layout in Figure 8.5. Skeletonization is the process of creating a reduced model that is hydraulically equivalent to the full network, using selected components that have a significant impact on the behaviour of the system (Walski et al. 2001). Care has to be taken to ensure that the portions of the system that are not modelled during the skeletonization process are accounted for within the reduced model (Walski et al. 2001). For example, in this case study, the skeletonisation process has involved removal of the smaller diameter pipes (of size 40mm and 50mm) including service connections from the network in Figure 8.4, and accounting for them in the model (Figure 8.5) by lumping their demands onto the nearest nodes. The small diameter pipes referred to are those that are branched and at the periphery of the network; and these have been selected specifically to ensure that the resulting model is hydraulically equivalent to the original network. The resulting network layout for the model in Figure 8.5 has 16 demand nodes and 18 links; and this layout is taken as Option 1, which has been designed using the shortest path flow distribution method for link-flow allocation (Orth, 1986).

Considering that the network in Figure 8.5 has only one link between Nodes 9 and 10 for crossing the main road from Kampala to Luwecro (Fig. 8.4), failure of this link would mean isolating Nodes 11 to 16. Similarly, failure of link 8-9 in Figure 8.5 would lead to the isolation of Nodes 9 to 16. To remedy this situation, it has been proposed in this study to introduce another link as an alternative to link 8-9 for supplying Nodes 9 to 16, and another link as an alternative for crossing the main road from Kampala to Luweero. The resulting modified layout is presented in Figure 8.6 and taken as the layouts for Options 2 and 3. These two options are designed using maximum entropy flows and the node numbering of the network in Figure 8.6 is based on the maximum entropy flow distribution algorithm (Chapter 3). The new links that have been introduced are link 8-10 and link 10-13 (link for crossing the main road from Kampala to Luweero) as shown in Figure 8.6. A pricing policy has been implemented for Option 3 but not for Option 2, thus, Option 3 has lower nodal design demands than Option 2.

8.4.2.2 *Design Data*

In this study, the overall design horizon, *d*, is taken as 20 years. The nodal base demands for all options are presented in Appendix B, Table B-8.25 and they include domestic demand, institutional demand and commercial demand (Associated Consulting Engineers, 1995a, Directorate of Water Development, 1999). The network loading that has been adopted for this case study is that of peak hour demands combined with a fire demand and with a peak hour factor of 2.0 (Directorate of Water Development, 1999). The nodal base demands are assumed to increase at the same rate as that of population growth, which is an annual rate (*DGR*) of 4%. Thus, to obtain the design demands for the entire design horizon, the nodal base demands are multiplied by the peak hour factor to obtain peak hour demands. The resulting values are then forecast or projected over the design period. Fire demand is then added to obtain the overall nodal design demands. For example, the 20-year nodal design demands are shown in Appendix B, Table B-8.25 and B-8.27 for Options 1 and 2, respectively.

The design demands for Option 3 are reduced due to enforcement of the pricing policy. For this design option, a tariff increase ratio, (P_1/P_0), has been taken as 1.333 (Associated Consulting Engineers, 1995b). This is a 33.33% increase in the water tariff normally implemented in the last one or two years of a given design period.

This value may seem high at first glance, but it has to be noted that it is a single price increase for a design period. In Wobulenzi, increases in the price of water are not set annually as in some countries e.g., United Kingdom. In the UK, the Office of Water Services (OFWAT) sets the price limits or the maximum annual percentage increase in water tariffs (OFWAT, 1999) for different water companies. A typical annual percentage increase of about 3% per annum (OFWAT, 1999) when compounded over a design period of 9 years for example, results in an increase in the price of water of about 30.5% ($P_t / P_0 = 1.03^9 = 1.305$). This value is quite comparable with the one used for this study. The price elasticity of demand, *PREL*, has been assumed to be −0.3 (Dandy et al. 1985). This combination of tariff increase ratio and price elasticity of demand results in reduced nodal design demands as shown in Table B-8.29, Appendix B. The nodal design demands adopted in this study are a combination of the peak hour design demands and a proportion (25%) of the fire demand (at Node 4) to avoid over-designing the network.

The pipe material for the three design options is PVC and the Hazen-Williams coefficient for all new pipes is 130. The link lengths for the designs are shown in Tables B-8.24, B-8.26 and B-8.28 (Appendix B) for Options 1, 2, and 3 respectively. The nodal elevations and demand data for the designs are presented in Tables B-8.25, B-8.27 and B-8.29 for Options 1, 2, and 3 respectively. The minimum service pressure at all nodes is taken as 15m. Compound interest rate, $b = 8$ and the design horizon $d = 20$ years. For Phase II, v varies from 7 to 14 years, i.e. the lower limit for the end of Phase I ($T1$=7 years) and the upper limit for the end of Phase I ($T2$ =14 years). $FCF(LU_i)_{ij}$, the link-failure cost factors for land use are shown in Tables B-8.24, B-8.26 and B-8.28 for Options 1, 2, and 3 respectively. The discount rate, r = 8%; the initial roughness, $e_{0ijm} = 0.0021$mm and the roughness growth rate, a_{ijm} = 0.025 (mm/year) (Bhave, 1991). Pipe cost constants (Eqs. 4.4, 4.6 and 4.8) that have been used are $\gamma_p = 32.093$; $c_p = c_r = 3.7$; $\gamma_r = 33.928$ and $\gamma_{br} = 108.87$. The pipe cost exponent $\Phi = 0.6067$.

The rest of the input data, minimum and maximum velocity, maximum hydraulic gradient, setting-up costs, *VE*, and *VC* have been taken as those used for the hypothetical network in Section 8.3.

Option 1 has three loop constraints, three service pressure constraints between the source and Nodes 10, 15 and 16, eighteen summations of link length constraints and non-negativity constraints for the link segment lengths. Options 2 and 3 each have five loop constraints, three service pressure constraints between the source and Nodes 12, 16 and 17, twenty-one summations of link length constraints and non-negativity constraints for the link segment lengths.

8.4.3 Appraisal of the Results

8.4.3.1 Network Design

The Network Design module has been used to obtain the best upgrading strategy and the timing of the upgrading for the three design options. The results are shown in Tables 8.19 to 8.24 for the first and second phases of the three design options. The overall costs for the three designs are also shown in Tables 8.25, 8.26 and 8.27.

From Table 8.25, the cheapest cost strategy for design Option 1 for the Wobulenzi network has a value of \$3,909,206 and it is to design and install a capacity for an 11-year demand in Phase I; and then upgrade the network the 20-year demand capacity in Phase II. This optimal design requires 56 variables in Phase I and 118 variables in Phase II and the overall CPU-time required for this option is 4.55 seconds. Table 8.26 shows that the cheapest cost strategy for the Wobulenzi network design Option 2 has a value of \$3,953,663.25. This strategy entails designing and installing a capacity for an 11-year demand in Phase I and then upgrading to the 20-year demand capacity in Phase II. This optimal design requires 57 variables in Phase I and 124 variables in Phase II and the overall CPU-time required for this option is 5.328 seconds. This overall CPU-time referred to covers the different linear optimisation designs for various Phase I and Phase II design periods, with the Phase I design period varying from 7 years to 14 years as shown in Tables 8.25, 8.26 and 8.27. The time also covers the dynamic programming to determine the timing and magnitude of the upgrading for each design option.

The pricing policy implemented for the Wobulenzi network design Option 3 results in a two-year delay of expansion. Table 8.27 shows that the cheapest cost strategy for this option has a value of $3,616,334.75 and it is to design and install a capacity for a 7-year demand in Phase I; delay expansion or upgrading for 2 years and then increase capacity to a 16-year demand capacity (20 years less the delays due to pricing of 2 years in each design phase), to serve the entire 20-year design horizon. This optimal design requires 54 variables in Phase I and 118 variables in Phase II and the overall CPU-time is 5.308 seconds.

8.4.3.2 Network Analysis

The optimal designs of Tables 8.19 to 8.24 have been analysed using the Hydraulic Simulation Module in order to check that they are hydraulically consistent. Tables B-8.24, B-8.26 and B-8.28 in Appendix B show the link data and results for Options 1, 2 and 3, respectively, of the Wobulenzi network designs. The nodal data and results for Options 1, 2 and 3 are in Tables B-8.25, B-8.27 and B-8.29, respectively.

8.4.3.3 Network Performance Measures

Data for network performance with reference to the optimal designs shown in Tables 8.19 to 8.24 has been obtained using the Performance Assessment Module. The Wobulenzi network performance results are presented in Tables 8.28, 8.29 and 8.30 for Options 1, 2 and 3, respectively. Design Option 1 has a probability that all components are available of 0.992277 (corresponding to layout No. 1), a network reliability of 0.996237 and a failure tolerance of 0.880516. The overall CPU-time required for execution of the Performance Assessment Module for this option is 1.843 seconds. The overall CPU-time referred to covers the head-dependent analysis of the all the 44 different layouts of the network with the unavailability of components as shown in Table 8.28. The total demand supplied or total outflow for each of the layouts is also shown.

Results of Option 2 are shown in Table 8.29 with a probability that all components are available of 0.989357 (corresponding to layout No. 1), a network reliability of 0.999197 and a failure tolerance of 0.924534. The overall CPU-time required for this option is 2.703 seconds. The overall CPU-time referred to covers the head-dependent analysis of the all the 50 different layouts of the network with the unavailability of components as shown in Table 8.29.

Table 8.30 presents the network performance results of Option 3. For this option, the probability that all components are available is 0.988749 (corresponding to layout No. 1). The network reliability is 0.999060 and a failure tolerance of 0.916449. The overall CPU-time required for execution of the Performance Assessment Module for this option is 2.152 seconds. The overall CPU-time referred to covers the head-dependent analysis of the all the 50 different layouts of the network with the unavailability of components as shown in the table.

A summary of the results for the Wobulenzi network design options is shown in Table 8.31. The probability that all components are available, the network reliability, the component failure tolerance and the present value of costs for each of the three design options are given in this table.

8.4.3.4 Analytic Hierarchy Process

The results in Table 8.31 have been fed into the Analytic Hierarchy Process Module to assist in the final decision-making process for the optimal long-term upgrading of the Wobulenzi water distribution network. In this study, the judges used for this network example are the same as those for the hypothetical network example of Sub-section 8.3.1. Thus, the comparison matrix and priority vector for the network performance criterion are taken as shown in Table B-8.7. The comparison matrix and priority vector for the social and environmental issues criterion are taken as shown in Table B-8.8.

Tables B-8.30 to B-8.32 in Appendix B show the comparison matrices and priority vectors of the options with respect to the quantitative sub-criteria, which have been obtained using the Scale of relative intensities and measurement data of Table 7.8 (reproduced as Scale A of relative intensities and measurement data in Table 8.32). Table B-8.30 contains the comparison matrix and priority vector (0.333, 0.333, 0.333) of the three options with respect to the reliability sub-criterion. Table B-8.31 contains the comparison matrix and priority vector (0.261, 0.411, 0.328) of the

Options 1, 2 and 3, respectively, with reference to the failure tolerance sub-criterion. Table B-8.32 contains the comparison matrix and priority vector (0.333, 0.333, 0.333) of the Options 1, 2 and 3, respectively, with reference the sub-criterion for the present value of project costs. In all cases, the maximum eigenvalue, λ_{max}, the consistency index, C.I. and the consistency ratio, C.R. have been calculated. These consistency ratios are less than 0.1 meaning that the respective comparison matrices are consistent (see Sub-section 7.3.4.3).

The main comparison matrix and priority vector according to the judgment of the Judge 1 has been taken as shown in Table B-8.12 (Appendix B). The comparison matrices and priority vectors of the options with respect to the qualitative sub-criteria, acceptability, health issues and abstraction according to the judgement of the Judge 1 are taken as those shown in Tables B-8.13 to B-8.15.

The main comparison matrix and priority vector according to Judge 2 has been taken as shown in Table B-8.16. The comparison matrices and priority vectors of the options with respect to the qualitative sub-criteria, acceptability, health issues and abstraction according to Judge 2 are taken as those shown in Tables B-8.17 to B-8.19.

The main comparison matrix and priority vector according to Judge 3 has been taken as shown in Table B-8.20. The comparison matrices and priority vectors of the options with respect to the qualitative sub-criteria, acceptability, health issues and abstraction according to Judge 3 are taken as those shown in Tables B-8.21 to B-8.23.

The overall ranking of the Wobulenzi network design options by Judge 1 has been obtained by matrix manipulation at different levels of the hierarchy as detailed in Chapter 7 and presented in Table 8.33. According to this decision-maker, the overall ranking of Options 1, 2 and 3 is (0.322, 0.362, 0.317) meaning that the best alternative is Option 2.

The overall ranking of the Wobulenzi network design options by Judge 2 is shown in Table 8.34. According to this decision-maker, the overall ranking of Options 1, 2 and 3 is (0.377, 0.341, 0.281) meaning that the best alternative is Option 1. Similarly, Table 8.35 shows the overall ranking of Options 1, 2, and 3 according to Judge 3 in a vector (0.340, 0.363, 0.297). This means that Judge 3 considers Option 2 to be the best alternative.

The decision-making capability of the judges has been ranked and given a priority assignment by the chief decision-maker and the results are the same as those in Table 8.17. The vectors of individual overall ranking of the options by Judge 1 (0.322, 0.362, 0.317), Judge 2 (0.377, 0.341, 0.281) and Judge 3 (0.340, 0.363, 0.297) have been aggregated into the final priority matrix in Table 8.36. The product of this final priority matrix and the priority vector for judges (0.333, 0.333, 0.333) yields the vector of final ranking of Options 1, 2 and 3 as (0.346, 0.355, 0.298), and this represents the final group decision. The highest priority value is 0.355 meaning the best design option for long-term upgrading of the network is Option 2, the modified Wobulenzi network design (Figure 8.6) without any pricing policy and the worst option is Option 3.

8.4.4 Sensitivity Analysis

8.4.4.1 Variation of Priority Vector for Judges

A sensitivity analysis has been carried out to check the effect of variation in the priority vector for judges. This particular vector does not benefit from the consensus of the group discussions or debate, because it is solely generated by the chief decision-maker. The results are presented in Tables 8.37, 8.38 and 8.39. Table 8.37 presents a sensitivity analysis Scenario A in which the priority vector for judges is (0.5, 0.3, 0.2) meaning that Judge 1 has the highest weight and therefore the judgments made by this judge are more influential to the final decision compared to those made by the other two judges. The resulting final option ranking for Options 1, 2 and 3 is (0.342, 0.356, 0.302). Thus, Option 2 is the best design option for this scenario. The worst option has remained Option 3.

In Table 8.38, the sensitivity analysis Scenario B is shown with the priority vector for judges as (0.2, 0.6, 0.2) meaning that Judge 2 has the highest weight and therefore the judgments made by this judge are more influential to the final decision compared to those made by the other two judges. The resulting final option ranking for Options 1, 2 and 3 is (0.359, 0.350, 0.292). Thus, Option 1 is the best design option for this scenario. This scenario maintains the worst option as Option 3.

Sensitivity analysis Scenario C is presented in Table 8.39 in which the priority vector for judges is (0.3, 0.2, 0.5), giving the superiority of judgment to Judge 3. The resulting final option ranking is (0.342, 0.358, 0.300). Once again the best design option is Option 2 and the worst option has remained Option 3.

8.4.4.2 Adjustment of the Scale of Relative Intensities

To assess the impact of the scale of relative intensities on the final decision, the judges have agreed to adjust Scale A of relative intensities that has been presented in Table 8.32. The changes made to this scale are in the column of percentage differences for economic value as shown in Table 8.40 (Scale B of relative intensities and measurement data). These changes directly affect the comparison matrix and priority vector of the three options with respect to the present value sub-criterion. From the unit values shown in Table B-8.32, the elements of this comparison matrix have changed as presented in Table B-8.33 (comparison matrix and priority vector for present value-sensitivity analysis). The rating of 2 in Table B-8.33 means that the superiority of Option 3 over Option 1 or 2 falls between equal importance and weak importance. The priority vector for Options 1, 2 and 3 has also changed from (0.333, 0.333, 0.333) in Table B-8.32, which implies that all options are equally favourable with respect to the sub-criterion for the present value of project costs; to (0.25, 0.25, 0.5) in Table B-8.33, which implies that Option 3 is twice as favourable as Option 1 or 2.

The details of the matrix manipulation for the individual judges is the same as in Tables 8.33, 8.34 and 8.35, apart from substitution of the vector (0.333, 0.333, 0.333) with the new vector (0.25, 0.25, 0.5) as the priority vector for present value of costs or economic value. This yields different individual overall ranking of options by each judge as summarised in Table 8.41. The ranking for Options 1, 2 and 3 is given by the vectors (0.288, 0.328, 0.383), (0.350, 0.314, 0.337) and (0.319, 0.342, 0.339) according to Judges 1, 2 and 3 respectively.

Following these changes in the scale of relative intensities, sensitivity analyses have been carried out to check the effect of variation in the priority vector for judges. The results are presented in Tables 8.41 to 8.44. Table 8.41 presents a sensitivity analysis Scenario D in which the priority vector for judges is (0.3333, 0.333, 0.333). The resulting final option ranking for Options 1, 2 and 3 is (0.319, 0.328, 0.353). Thus, the best design option for this scenario is Option 3 and the worst option is Option 1.

Table 8.42 presents a sensitivity analysis Scenario E in which the priority vector for judges is (0.5, 0.3, 0.2). The resulting final option ranking for Options 1, 2 and 3 is (0.313, 0.327, 0.360). Thus, Option 3 is the best design option for this scenario and the worst is Option 1.

In Table 8.43, the sensitivity analysis Scenario F is shown with the priority vector for judges as (0.2, 0.6, 0.2). The resulting final option ranking for Options 1, 2 and 3 is (0.331, 0.322, 0.347). Thus, Option 3 is the best design option for this scenario and the worst has now become Option 2.

Sensitivity analysis Scenario G is presented in Table 8.44 in which the priority vector for judges is (0.3, 0.2, 0.5). The resulting final option ranking is (0.316, 0.332, 0.352). Once again, Option 3 is the best design option and the worst is Option 1.

8.4.5 Discussion

8.4.5.1 Network Design

From Tables 8.25 to 8.27, the results of the three design options generally show that paralleling is preferred to replacement probably because it is cheaper and that the rate of deterioration of the pipes is quite low. This conclusion does not seem to be case specific because different networks with different layouts and sizes exhibit these characteristics.

Results in Tables 8.25, 8.26 and 8.27 suggest that the detailed costs for the three Wobulenzi network designs exhibit an increasing trend in costs for Phase I and a decreasing trend in costs for Phase II, as the Phase I design period increases. This trend is the same as that for the hypothetical network. Generally, as time for the Phase I design period increases, the design demand increases too and thus a larger capacity has to be designed for in Phase I, followed by a lower incremental capacity to be designed for in Phase II as detailed I, Sub-section 8.3.2.

The optimal design costs for Options 1, 2, and 3 are $3,909,206, $3,953,663.25 and $3,616,334.75. Option 3 (the modified network design with a pricing policy) has the lowest cost mainly due to the enforcement of the pricing policy, which leads a reduced overall consumption, a delay in the need to upgrade the network and thus a lower installed capacity requirement. For Option 3, the overall total demand to be designed for has reduced to 158.24l/s as shown in Table B-8.29 compared to the total demand for Options 1 and 2, of 184.29l/s in Tables B-8.25 and B-8.27. Option 2 has 18 links, which are fewer than the 21 links of Option 3. However, the proportion of this difference in link numbers is small in terms of length and diameter of the links when compared to the overall size of the network and thus has a smaller impact on the difference in cost.

From the values of overall CPU time for execution of Network Design Module on Options 1, 2 and 3, which are 4.55, 5.328 and 5.308 seconds respectively, are quite low considering the network sizes (compared to the sizes of the hypothetical network options) and the fact that they cover different linear optimisation designs for various Phase I and II design periods; and the dynamic programming to determine the timing and magnitude of the upgrading for each design option. This is a good reflection of the efficiency of the Network Design Module. Design Option 1 has the lowest overall CPU time probably because it has only three loops and eighteen links compared to Options 2 and 3 which have five loops and 21 links each. Thus Option 1 has the lowest number of constraints. Compared to the hypothetical network designs, there is a general increase in the number of variables of the Wobulenzi designs and the values for the CPU time. This is probably due to the increase in number of links. Another probable suggestion is that program execution time increases with an increase in the number of variables and therefore highlights the importance of limiting the decision variables to reduce the computational effort as mentioned earlier in Chapter 4.

8.4.5.2 Network Performance Measures

The network performance results in Tables 8.28, 8.29 and 8.30 show values of overall CPU time for Options 1, 2 and 3, which are 1.843, 2.703 and 2.152 seconds respectively. These values are quite low, considering that they cover the head-dependent analysis of all the 44, 50, and 50 different layouts of the network for Options 1, 2 and 3 respectively with the unavailability of components as shown in Tables 8.28, 8.29, and 8.30. This shows that the Network Performance Module has a considerably high efficiency. Design Option 1 has the lowest CPU time probably due to the fact that it has the lowest number of links and it is run for fewer layouts.

From Table 8.31, the probability, $p(0)$, that all components are available for Options 1, 2 and 3 are 0.992277, 0.989357 and 0.988749. This value for Option 1 is the highest probably due to the fact that $p(0)$ is the product of the individual link reliabilities and that Option 1 has the lowest number of links. The reliability values of the Options 1, 2 and 3 (0.996237, 0.999197 and 0.999060, respectively) suggest that Design Option 2 has the highest reliability perhaps due to the slightly larger links in the network ensuring the highest ability of the network to meet demands at adequate pressure under normal and abnormal conditions. The larger links for Option 2 are due to the fact that the network is designed for maximum entropy flows without any pricing policy. Further details of the justification of this reliability being the highest are detailed in Sub-section 8.3.2.

The failure tolerance for Option 1 (Table 8.31) is the lowest probably because the layout for Option 1 (Figure 8.5) has the lowest number of loops (three loops). This limits the number of alternative paths from the source to the supply nodes and further reduces performance of the network when components become unavailable. On the other hand, the layout for Options 2 and 3 (Figure 8.6) has more loops (five loops) and is therefore more likely to perform better in times of partial failure.

Once again, it has to be noted that the Wobulenzi network reliability results for the different design options are quite close to each other probably due to the fact that the individual component reliabilities of the networks are high. With such results distinguishing between the designs using the reliability criterion alone would perhaps be quite difficult. However, the values of failure tolerance for the Options 1, 2 and 3 shown in Table 8.31 as 0.880516, 0.924534 and 0.916449 respectively, clearly distinguish between the designs. These values further demonstrate the ability of failure tolerance to expose the vulnerability of the network under stress and differentiate clearly between networks with high reliability but with varying capabilities of performing under stress (Kalungi and Tanyimboh, 2001). This re-emphasizes the point that reliability and failure tolerance should be used together as performance assessment parameters.

It is also noted from Table 8.31 that the costs in terms of present value for Options 1 and 2 are $3,909,206 and $3,953,663.25. It is evident that the cheaper design, Option 1, also has the lower value of failure tolerance. Whereas Option 1 is the more favourable option in terms of costs, it is the less favourable option in terms of performance under abnormal operating conditions. This further stresses the importance of using the AHP to resolve such instances of conflicting decision factors.

8.4.5.3 Analytic Hierarchy Process

The AHP involves pair-wise judgements and formation of comparison matrices by different decision makers. Each of the comparison matrices for the Wobulenzi network design options shows a consistency ratio with a value less than the allowable value of 0.10. This implies that all the judgments are consistent and that there is no need for revision of judgments to achieve consistency (see Sub-section 7.3.4.3).

The scale of relative intensities is very helpful in facilitating the process of comparing measured or quantitative data based on the percentage variation in the values of the decision factors. Quantitative judgements have been made based on the Scale A of relative intensities and measurement data shown in Table 8.32. The vectors of individual overall ranking of the options (Table 8.36) which are (0.322, 0.362, 0.317), (0.377, 0.341, 0.281) and (0.340, 0.363, 0.297) by Judges 1, 2 and 3, respectively, show that Judges 1 and 3 have ranked Option 2 as the best and Judge 2 has ranked Option 1 as the best. Each of the judges has ranked Option 3 as the worst option. Given a priority vector for judges of priority vector for judges (0.333, 0.333, 0.333), the vector of final ranking of Wobulenzi network design Options 1, 2 and 3 shown in Table 8.36 is (0.346, 0.355, 0.298), and this represents the final group decision. The highest priority value is 0.355 meaning the best design option for long-term upgrading of the network is Option 2, the modified network design (Figure 8.6) without any pricing policy. Option 1 is the second most favourable option and Option 3 is the least favourable option.

8.4.5.4 Sensitivity Analysis

The results of the sensitivity analyses aimed at assessing the effect of variation in the priority vector for judges have been presented in Tables 8.37, 8.38 and 8.39 as Scenarios A, B and C. Scenario A (Table 8.37) favours Judge 1 as the one with the most influential judgement towards the final decision, leading to a final option ranking for Options 1, 2 and 3 of (0.342, 0.356, 0.302) as shown in Table 8.37. Option 2 is the best option for this scenario. Scenario B (Table 8.38) favours Judge 2 as the one with the most influential judgement towards the final decision, leading to a final option ranking for Options 1, 2 and 3 of (0.359, 0.350, 0.292). Thus, Option 1 is the best design option for this scenario. In this case however, it could be argued that the weights for Option 1 and 2 are not significantly different and therefore each of these options is probably equally favourable as the best option. Scenario C (Table 8.39) favours Judge 3 as the one with the most influential judgement towards the final decision, leading to a final option ranking for Options 1, 2 and 3 of (0.342, 0.358, 0.300) implying that Option 2 is the best design option for this scenario. All in all, Option 2 is the best design option and this decision is quite stable even when the superiority of judgement for the Judges is varied.

The results of the analysis to assess the sensitivity of the final decision to the scale of relative intensities have been carefully examined. The changes made to Scale A of relative intensities (Table 8.32) in the column of percentage differences for economic value as shown in Table 8.40 (Scale B of relative intensities and measurement data), imply that a higher significance has been considered for the relative difference in the present value of costs for the different options. This is the reason why the priority vector of Options 1, 2 and 3 has also changed from (0.333, 0.333, 0.333) in Table B-8.32 (implying that all options are equally favourable with respect to present value of project costs) to (0.25, 0.25, 0.5) in Table B-8.33; which implies that Option 3 is twice as favourable as Option 1 or 2. Table 8.41 shows that Judges 1, 2 and 3 have different choices of the best design option, which are Option 3, Option 1 and Option 2, respectively.

Given the alternative scale of relative intensities shown in Table 8.40, the results of the sensitivity analyses aimed at assessing the effect of variation in the priority vector for judges have been presented in Tables 8.41 to 8.44 as Scenarios D, E, F and G. Scenario D (Table 8.41), in which individual judgments of the judges have an equal contribution to the final decision, yields a final option ranking for Options 1, 2 and 3 of (0.319, 0.328, 0.353). Thus, Option 3 is the best design option for this scenario. Scenario E (Table 8.42) favours Judge 1 as the one with the most influential judgement towards the final decision, leading to a final option ranking for Options 1, 2 and 3 of (0.313, 0.327, 0.360). Thus, Option 3 is the best design option for this scenario. Scenario F (Table 8.43) favours Judge 2 as the one with the most influential judgement towards the final decision, leading to a final option ranking for Options 1, 2 and 3 of (0.331, 0.322, 0.347). Thus, Option 3 is the best design option for this scenario. Scenario G (Table 8.44) favours Judge 3 as the one with the most influential judgement towards the final decision, leading to a final option ranking for Options 1, 2 and 3 of (0.316, 0.332, 0.352). Once again, Option 3 is the best design option for this scenario. On the whole, Option 3 is the best design option obtained by using this alternative scale of relative intensities and this decision is consistently stable even when the superiority of judgement for the Judges is varied.

It has to be noted at this point that the decision of selecting Option 3 which is the modified Wobulenzi network design (Figure 8.6) with the enforcement of a pricing policy, has to be taken with caution. Considering that this project is a community-based project in which the community members are stakeholders by virtue of their contribution towards the costs of the project, Option 3 is likely to be resented by the community from an acceptability point of view depending upon the level of increase in water prices. This could easily lead to vandalism of the project components. There is also a possibility of people turning to unhealthy sources of water like unprotected springs, contaminated hand dug wells, and/or existing boreholes fitted with old hand pumps that produce water with a high iron content. This could lead to an outbreak of diseases and reduce the productivity of the town's population with a knock-on effect leading to a reduced micro-economic output. Therefore when deciding on the scale of relative intensities to use in the AHP process, the judges have to consider these issues carefully. However, such an option may still be acceptable if the community is fully sensitised as to what the disadvantages of opting for unsafe water sources are.

8.5 SUMMARY AND CONCLUSION

In this chapter, a holistic approach to the optimal long-term upgrading of water distribution networks has been presented. This approach involves the use of an Integrated Model that has been formulated and applied to hypothetical and real-life networks. The model addresses the key issues involved in the upgrading of a water distribution network in a detailed fashion, such as the timing and magnitude of upgrading while incorporating the deterioration of hydraulic and structural capacity of pipes over time. The Integrated Model is a system-wide model that incorporates the performance of the system explicitly. This model is set in a multi-objective framework that covers hydraulic, economic, social and environmental issues together with issues related to the level of service like reliability and failure tolerance. The Integrated Model's separate modules for network design, hydraulic simulation, network performance and the analytic hierarchy process facilitate the model execution in this multi-objective framework setting.

A notable aspect of the Integrated Model is the efficiency in terms of the low CPU time for execution of its individual modules. For example, determining the timing magnitude and upgrading of a network using the Network Design Module involves designing the network for various design periods with typical CPU time values ranging from 0.5 seconds for a small network of about seven pipes to about 5 seconds for a larger network of about 21 pipes. In general, the relative increase in the time is probably due to the increase in the number of constraints and variables. Determining the reliability and failure tolerance of a network involves head-dependent analysis of the network with numerous simulations of layouts having unavailability components. Typical CPU time values range from about 1.5 seconds for a small network of about seven pipes to about 3 seconds for a larger network of about 21 pipes.

The solution methodology of the Integrated Model is geared towards reducing the computational effort in the process of obtaining the best upgrading strategy, which inevitably involves many decision variables especially during the execution of the Network Design Module. This is achieved by predefining flows using either the maximum entropy flow distribution algorithm or the shortest path flow distribution algorithm, together with an in-built procedure for reducing the number of segment diameters on the candidate list for each link during optimisation.

The importance of using failure tolerance and reliability in the multi-objective decision making framework has been highlighted. Evidence of this has been provided by showing that although reliability is the commonly used parameter for assessing the performance of networks, it may not be a sufficient parameter for one to differentiate between certain design options. However, in these very instances, failure tolerance may assist in differentiating between the design options. It has further been shown that the cheapest network design may not necessarily be the most reliable or have a considerably good level of failure tolerance.

The inclusion of the AHP module which is a versatile and robust tool in handling decisions involving conflicting decision factors, qualitative and quantitative data is a another notable feature of the Integrated Model. The suitability of the AHP for multi-criteria decision-making is mainly based on the hierarchical arrangement of decision factors. This is coupled with its ability to use a simple method of pair-wise comparisons for judgments and matrix manipulation to derive a vector of priorities for the decision. Another advantage of the AHP is the measure for consistency for the comparison matrices and a provision for the revision of judgments in cases of inconsistent comparison matrices. There is also a provision for a sensitivity analysis to test the impact of variation of certain parameters on the overall decision.

Another interesting point to note is that the best options for the hypothetical network and that of the Wobulenzi network together with the sensitivity analysis scenario as described above are designed using maximum entropy flows. The traditional method of network design is to use the shortest path flow distribution. The results of this study probably suggest that designing using the maximum entropy flow distribution approach is quite beneficial considering that in a multi-objective framework, these designs strike a better compromise between economic, social and environmental and performance related issues. The other advantage of using maximum entropy flows is the reduction in the complexity of the problem and computational effort required.

It is important to note that the Integrated Model is a means to an end but not an end in itself in that it can be used for guidance in the decision-making process but the results have to be carefully debated upon to the satisfaction of all those involved in the process. Further work including the application of the Integrated Model to even larger real-life networks is necessary. Essentially, it has been demonstrated in this chapter that the complicated issue of long-term upgrading can be simplified and solved using the Integrated Model according to the methodology presented in this study. The attractive point is that this methodology is adaptable to more powerful optimisation techniques like genetic algorithms, the advantage being that the problem is more simplified and thus the computational effort is likely to be reduced. Some refinements may be necessary but the basic framework has been established.

Table 8.1 Hypothetical Network Design Option 1 - Phase I

	SEGMENT ONE		SEGMENT TWO		EQUIVALENT DIAMETER LINKS		
	DIAMETER	LENGTH	DIAMETER	LENGTH	DIAMETER	LENGTH	CHW
LINK	(m)	(m)	(m)	(m)	(m)	(m)	
1-2	0.200	1000.00	-	-	0.200	1000	121.7
1-3	0.200	1000.00	-	-	0.200	1000	121.7
2-3	0.150	143.96	0.200	856.04	0.186	1000	119.3
3-4	0.150	791.70	0.200	208.3	0.154	1000	119.3
3-5	0.150	1000.00	-	-	0.150	1000	117

Table 8.2 Hypothetical Network Design Option 2 - Phase I

	SEGMENT ONE		SEGMENT TWO		EQUIVALENT DIAMETER LINKS		
	DIAMETER	LENGTH	DIAMETER	LENGTH	DIAMETER	LENGTH	CHW
LINK	(m)	(m)	(m)	(m)	(m)	(m)	
1-2	0.200	1000.00	-	-	0.200	1000	121.7
1-3	0.150	1000.00	-	-	0.150	1000	117
1-4	0.100	1000.00	-	-	0.100	1000	110.5
2-3	0.150	51.95	0.200	948.05	0.195	1000	119.3
2-5	0.100	581.52	0.150	418.48	0.109	1000	113.7
3-4	0.150	621.80	0.200	378.2	0.160	1000	119.3
3-5	0.150	1000.00	-	-	0.150	1000	117

Table 8.3 Hypothetical Network Design Option 3 - Phase I

	SEGMENT ONE		SEGMENT TWO		EQUIVALENT DIAMETER LINKS		
	DIAMETER	LENGTH	DIAMETER	LENGTH	DIAMETER	LENGTH	CHW
LINK	(m)	(m)	(m)	(m)	(m)	(m)	
1-2	0.200	1000.00	-	-	0.200	1000	125.9
1-3	0.150	1000.00	-	-	0.150	1000	121.2
1-4	0.100	1000.00	-	-	0.100	1000	114.7
2-3	0.150	23.23	0.200	976.77	0.198	1000	123.5
2-5	0.100	567.19	0.150	432.81	0.109	1000	117.9
3-4	0.150	610.56	0.200	389.44	0.160	1000	123.5
3-5	0.150	1000.00	-	-	0.150	1000	121.2

Table 8.4 Hypothetical Network Design Option 1 - Phase II

	LINK	(A-B): EXISTING UNPARALLELED LINK			(B-C): PARALLEL LINK			(C-D): REPLACED LINK		
	FLOW	DIAM.	LENGTH		DIAM.	LENGTH		DIAM.	LENGTH	
LINK	(l/s)	(m)	(m)	CHW	(m)	(m)	CHW	(m)	(m)	CHW
1-2	49.55	0.200	1000.00	114.7	-	-	-	-	-	-
1-3	67.43	0.200	962.71	114.7	0.250	37.29	130.0	-	-	-
2-3	33.71	0.186	1000.00	113.5	-	-	-	-	-	-
3-4	43.35	0.154	302.31	110.5	0.200	697.69	130.0	-	-	-
3-5	43.35	-	-	-	0.150	727.38	126.9	0.150	272.62	126.9

Table 8.5 Hypothetical Network Design Option 2 - Phase II

	LINK	(A-B): EXISTING UNPARALLELED LINK			(B-C): PARALLEL LINK			(C-D): REPLACED LINK		
	FLOW	DIAM.	LENGTH		DIAM.	LENGTH		DIAM.	LENGTH	
LINK	(l/s)	(m)	(m)	CHW	(m)	(m)	CHW	(m)	(m)	CHW
1-2	66.41	0.200	645.42	114.7	0.250	354.58	130.0	-	-	-
1-3	36.12	0.150	729.85	110.1	0.200	270.15	130.0	-	-	-
1-4	14.45	-	-	-	0.080	893.57	116.8	-	-	-
					0.100	106.43	120.4			
2-3	36.12	0.195	1000.00	114.3	-	-	-	-	-	-
2-5	14.45	0.109	946.37	104.9	-	-	-	0.150	53.63	126.9
3-4	28.90	0.160	516.13	111.1	0.200	483.87	130.0	-	-	-
3-5	28.90	0.150	1000.00	110.1	-	-	-	-	-	-

Table 8.6 Hypothetical Network Design Option 3 - Phase II

LINK	LINK FLOW (l/s)	(A-B): EXISTING UNPARALLELED LINK DIAM (m)	LENGTH (m)	CHW	(B-C): PARALLEL LINK DIAM (m)	LENGTH (m)	CHW	(C-D): REPLACED LINK DIAM (m)	LENGTH (m)	CHW
1-2	59.56	0.200	1000.00	114.7	-	-	-	-	-	-
1-3	32.09	0.150	950.20	110.1	0.200	49.80	125.9	-	-	-
1-4	12.84	0.100	852.56	103.5	-	-	-	0.150	147.44	121.2
2-3	32.09	0.198	1000.00	114.6	-	-	-	-	-	-
2-5	12.84	0.109	943.53	104.9	-	-	-	0.150	56.47	121.2
3-4	25.68	0.160	566.58	111.2	0.150	433.42	121.2	-	-	-
3-5	25.68	0.150	1000.00	110.1	-	-	-	-	-	-

Table 8.7 Costs for the Hypothetical Network Design Option 1

PHASE I TIME (Yrs)	PHASE I COSTS ($)	PHASE II COSTS ($)	OVERALL COSTS ($)
7	2,529,630.00	1,109,469.38	3,639,099.25
8	2,646,370.25	700,183.69	3,346,554.00
9	2,681,079.00	627,234.75	3,308,313.75
10	2,717,611.50	570,900.81	3,288,512.25
11	2,756,025.00	519,306.38	3,275,331.50
12	2,800,098.25	471,881.00	3,271,979.25
13	2,849,518.00	407,624.91	3,257,143.00
14	2,900,322.00	364,732.97	3,265,055.00

Table 8.8 Costs for the Hypothetical Network Design Option 2

PHASE I TIME (Yrs)	PHASE I COSTS ($)	PHASE II COSTS ($)	OVERALL COSTS ($)
7	3,043,834.25	778,141.25	3,821,975.50
8	3,074,926.00	719,074.75	3,794,000.75
9	3,110,481.50	658,627.25	3,769,108.75
10	3,148,052.50	602,605.25	3,750,657.75
11	3,187,680.75	550,868.06	3,738,548.75
12	3,229,409.50	503,004.34	3,732,413.75
13	3,273,284.25	458,808.31	3,732,092.50
14	3,319,355.25	417,902.63	3,737,258.00

Table 8.9 Costs for the Hypothetical Network Design Option 3

PHASE I TIME (Yrs)	PHASE I COSTS ($)	PHASE II COSTS ($)	OVERALL COSTS ($)
7	3,047,291.50	429,481.38	3,476,773.00
8	3,081,798.00	392,814.38	3,474,612.50
9	3,118,217.25	358,528.56	3,476,745.75
10	3,156,585.50	326,624.84	3,483,210.25
11	3,196,940.75	296,947.72	3,493,888.50
12	3,239,325.50	269,343.63	3,508,669.00
13	3,283,784.25	243,992.94	3,527,777.25
14	3,330,367.75	220,445.69	3,550,813.50

Table 8.10 Hypothetical Network Performance Results for Option 1

LAYOUT No.	UNAVAILABLE COMPONENTS LINK No.	LINK No.	MECHANICAL RELIABILITY (A)	TOTAL OUTFLOW (l/s) (B)	(A) * (B)
1	0	0	0.998354	116.99	116.797
2	1	0	0.000307	89.17	0.027
3	2	0	0.000309	62.24	0.019
4	3	0	0.000334	100.92	0.034
5	4	0	0.000336	73.64	0.025
6	5	0	0.000359	73.64	0.026
7	1	2	0.000000	0.00	0.000
8	1	3	0.000000	85.08	0.000
9	1	4	0.000000	73.64	0.000
10	1	5	0.000000	73.64	0.000
11	2	3	0.000000	15.84	0.000
12	2	4	0.000000	61.51	0.000
13	2	5	0.000000	62.30	0.000
14	3	4	0.000000	73.64	0.000
15	3	5	0.000000	73.64	0.000
16	4	5	0.000000	30.29	0.000
SUMMATION			1.000000		116.929
	NETWORK RELIABILITY = 0.999478 COMPONENT FAILURE TOLERANCE = 0.683046 THE OVERALL CPU-TIME = 0.660 seconds				

Table 8.11 Hypothetical Network Performance Results for Option 2

LAYOUT No.	UNAVAILABLE COMPONENTS LINK No.	LINK No.	MECHANICAL RELIABILITY (A)	TOTAL OUTFLOW (l/s) (B)	(A) * (B)
1	0	0	0.996982	116.99	116.637
2	1	0	0.000284	72.90	0.021
3	2	0	0.000410	102.77	0.042
4	3	0	0.000570	110.72	0.063
5	4	0	0.000316	104.16	0.033
6	5	0	0.000647	105.27	0.068
7	6	0	0.000356	97.04	0.035
8	7	0	0.000430	89.91	0.039
9	1	2	0.000000	23.40	0.000
10	1	3	0.000000	48.31	0.000
11	1	4	0.000000	70.87	0.000
12	1	5	0.000000	72.90	0.000
13	1	6	0.000000	70.82	0.000
14	1	7	0.000000	75.19	0.000
15	2	3	0.000000	81.14	0.000
16	2	4	0.000000	56.69	0.000
17	2	5	0.000000	92.73	0.000
18	2	6	0.000000	93.97	0.000
19	2	7	0.000000	88.05	0.000
20	3	4	0.000000	80.97	0.000
21	3	5	0.000000	100.15	0.000
22	3	6	0.000000	73.64	0.000
23	3	7	0.000000	89.13	0.000
24	4	5	0.000000	86.71	0.000
25	4	6	0.000000	96.34	0.000
26	4	7	0.000000	91.10	0.000
27	5	6	0.000000	90.74	0.000
28	5	7	0.000000	73.64	0.000
29	6	7	0.000000	71.13	0.000
SUMMATION			1.000000		116.938
	NETWORK RELIABILITY = 0.999551 COMPONENT FAILURE TOLERANCE = 0.851361 THE OVERALL CPU-TIME = 1.313 seconds				

Table 8.12 Hypothetical Network Performance Results for Option 3

LAYOUT No.	UNAVAILABLE COMPONENTS LINK No.	UNAVAILABLE COMPONENTS LINK No.	MECHANICAL RELIABILITY (A)	TOTAL OUTFLOW (l/s) (B)	(A) * (B)
1	0	0	0.996816	104.49	104.157
2	1	0	0.000306	59.01	0.018
3	2	0	0.000433	88.63	0.038
4	3	0	0.000693	100.43	0.070
5	4	0	0.000310	89.25	0.028
6	5	0	0.000640	95.80	0.061
7	6	0	0.000368	81.32	0.030
8	7	0	0.000430	81.59	0.035
9	1	2	0.000000	15.34	0.000
10	1	3	0.000000	42.67	0.000
11	1	4	0.000000	57.49	0.000
12	1	5	0.000000	57.92	0.000
13	1	6	0.000000	57.61	0.000
14	1	7	0.000000	58.37	0.000
15	2	3	0.000000	73.30	0.000
16	2	4	0.000000	47.12	0.000
17	2	5	0.000000	82.21	0.000
18	2	6	0.000000	80.26	0.000
19	2	7	0.000000	78.95	0.000
20	3	4	0.000000	73.92	0.000
21	3	5	0.000000	91.61	0.000
22	3	6	0.000000	65.98	0.000
23	3	7	0.000000	80.81	0.000
24	4	5	0.000000	72.12	0.000
25	4	6	0.000000	81.32	0.000
26	4	7	0.000000	83.11	0.000
27	5	6	0.000000	78.95	0.000
28	5	7	0.000000	65.98	0.000
29	6	7	0.000000	60.02	0.000
SUMMATION			1.000000		104.438
	NETWORK RELIABILITY = 0.999498 COMPONENT FAILURE TOLERANCE = 0.842412 THE OVERALL CPU-TIME = 1.172 seconds				

Table 8.13 Summary of Results for the Hypothetical Network Designs

PARAMETER	OPTION 1	OPTION 2	OPTION 3
$p(0)$	0.998354	0.996982	0.996816
NETWORK RELIABILITY	0.999478	0.999551	0.999498
FAILURE TOLERANCE	0.683046	0.951361	0.842412
PRESENT VALUE OF COSTS ($)	3,257,143.00	3,732,092.50	3,474,612.50

Table 8.14 Overall Ranking of Hypothetical Network Options by Judge 1

A matrix		*B* matrix				*C* matrix = *B* x *A*	
Priority Vector for selection of the best Option		**Overall Priority Matrix**				**Overall Ranking**	
			PERFORMANCE	*ECONOMIC VALUE*	*SOCIAL & ENV. ISSUES*		
PERFORMANCE	0.400	OPTION 1	0.185	0.411	0.371	OPTION 1	0.313
ECONOMIC VALUE	0.400	OPTION 2	0.407	0.261	0.371	OPTION 2	0.342
SOCIAL & ENV. ISSUES	0.200	OPTION 3	0.407	0.328	0.257	OPTION 3	0.346
Priority Vector for Economic Value		**Weight of options with respect to Present Value or Priority matrix for Economic Value**				**Composite Priorities**	
				PRESENT VALUE			
		OPTION 1		0.411		OPTION 1	0.411
PRESENT VALUE	1.000	OPTION 2		0.261		OPTION 2	0.261
		OPTION 3		0.328		OPTION 3	0.328
Priority Vector for Performance		**Priority matrix for Performance Sub-criteria**				**Composite Priorities**	
			FAILURE TOLERANCE	*RELIABILITY*			
FAILURE TOLERANCE	0.667	OPTION 1	0.111	0.333		OPTION 1	0.185
RELIABILITY	0.333	OPTION 2	0.444	0.333		OPTION 2	0.407
		OPTION 3	0.444	0.333		OPTION 3	0.407
Priority Vector for Social & Environmental Issues		**Priority matrix for Social and Environmental Issues**				**Composite Priorities**	
			ACCEPTABILITY	*HEALTH ISSUES*	*ABSTRACTION*		
ACCEPTABILITY	0.557	OPTION 1	0.400	0.400	0.167	OPTION 1	0.371
HEALTH ISSUES	0.320	OPTION 2	0.400	0.400	0.167	OPTION 2	0.371
ABSTRACTION	0.123	OPTION 3	0.200	0.200	0.667	OPTION 3	0.257
Priority Vector for Reliability		**Weight of options with respect to Reliability**				**Composite Priorities**	
				RELIABILITY			
		OPTION 1		0.333		OPTION 1	0.333
RELIABILITY	1.000	OPTION 2		0.333		OPTION 2	0.333
		OPTION 3		0.333		OPTION 3	0.333
Priority Vector for Failure Tolerance		**Weight of options with respect to Failure Tolerance**				**Composite Priorities**	
				FAILURE TOLERANCE			
		OPTION 1		0.111		OPTION 1	0.111
FAILURE TOLERANCE	1.000	OPTION 2		0.444		OPTION 2	0.444
		OPTION 3		0.444		OPTION 3	0.444
Priority Vector for Acceptability		**Weight of options with respect to Acceptability**				**Composite Priorities**	
				ACCEPTABILITY			
		OPTION 1		0.400		OPTION 1	0.400
ACCEPTABILITY	1.000	OPTION 2		0.400		OPTION 2	0.400
		OPTION 3		0.200		OPTION 3	0.200
Priority Vector for Health Issues		**Weight of options with respect to Health Issues**				**Composite Priorities**	
				HEALTH ISSUES			
		OPTION 1		0.400		OPTION 1	0.400
HEALTH ISSUES	1.000	OPTION 2		0.400		OPTION 2	0.400
		OPTION 3		0.200		OPTION 3	0.200
Priority Vector for Abstraction		**Weight of options with respect to Abstraction**				**Composite Priorities**	
				ABSTRACTION			
		OPTION 1		0.167		OPTION 1	0.167
ABSTRACTION	1.000	OPTION 2		0.167		OPTION 2	0.167
		OPTION 3		0.667		OPTION 3	0.667

Table 8.15 Overall Ranking of Hypothetical Network Options by Judge 2

A matrix		*B* matrix				*C* matrix = *B* x *A*	
Priority Vector for selection of the best Option		**Overall Priority Matrix**				**Overall Ranking**	
			PERFORMANCE	*ECONOMIC VALUE*	*SOCIAL & ENV. ISSUES*		
PERFORMANCE	0.333	OPTION 1	0.185	0.411	0.513	OPTION 1	0.370
ECONOMIC VALUE	0.333	OPTION 2	0.407	0.261	0.305	OPTION 2	0.325
SOCIAL & ENV. ISSUES	0.333	OPTION 3	0.407	0.328	0.181	OPTION 3	0.305
Priority Vector for Economic Value		**Weight of options with respect to Present Value or Priority matrix for Economic Value**				**Composite Priorities**	
				PRESENT VALUE			
PRESENT VALUE	1.000	OPTION 1		0.411		OPTION 1	0.411
		OPTION 2		0.261		OPTION 2	0.261
		OPTION 3		0.328		OPTION 3	0.328
Priority Vector for Performance		**Priority matrix for Performance**				**Composite Priorities**	
			FAILURE TOLERANCE	*RELIABILITY*			
FAILURE TOLERANCE	0.667	OPTION 1	0.111	0.333		OPTION 1	0.185
RELIABILITY	0.333	OPTION 2	0.444	0.333		OPTION 2	0.407
		OPTION 3	0.444	0.333		OPTION 3	0.407
Priority Vector for Social & Environmental Issues		**Priority matrix for Social and Environmental Issues**				**Composite Priorities**	
			ACCEPTABILITY	*HEALTH ISSUES*	*ABSTRACTION*		
ACCEPTABILITY	0.557	OPTION 1	0.557	0.557	0.200	OPTION 1	0.513
HEALTH ISSUES	0.320	OPTION 2	0.320	0.320	0.200	OPTION 2	0.305
ABSTRACTION	0.123	OPTION 3	0.123	0.123	0.600	OPTION 3	0.181
Priority Vector for Reliability		**Weight of options with respect to Reliability**				**Composite Priorities**	
				RELIABILITY			
		OPTION 1		0.333		OPTION 1	0.333
RELIABILITY	1.000	OPTION 2		0.333		OPTION 2	0.333
		OPTION 3		0.333		OPTION 3	0.333
Priority Vector for Failure Tolerance		**Weight of options with respect to Failure Tolerance**				**Composite Priorities**	
				FAILURE TOLERANCE			
		OPTION 1		0.111		OPTION 1	0.111
FAILURE TOLERANCE	1.000	OPTION 2		0.444		OPTION 2	0.444
		OPTION 3		0.444		OPTION 3	0.444
Priority Vector for Acceptability		**Weight of options with respect to Acceptability**				**Composite Priorities**	
				ACCEPTABILITY			
		OPTION 1		0.557		OPTION 1	0.557
ACCEPTABILITY	1.000	OPTION 2		0.320		OPTION 2	0.320
		OPTION 3		0.123		OPTION 3	0.123
Priority Vector for Health Issues		**Weight of options with respect to Health Issues**				**Composite Priorities**	
				HEALTH ISSUES			
		OPTION 1		0.557		OPTION 1	0.557
HEALTH ISSUES	1.000	OPTION 2		0.320		OPTION 2	0.320
		OPTION 3		0.123		OPTION 3	0.123
Priority Vector for Abstraction		**Weight of options with respect to Abstraction**				**Composite Priorities**	
				ABSTRACTION			
		OPTION 1		0.200		OPTION 1	0.200
ABSTRACTION	1.000	OPTION 2		0.200		OPTION 2	0.200
		OPTION 3		0.600		OPTION 3	0.600

Table 8.16 Overall Ranking of Hypothetical Network Options by Judge 3

A matrix		*B* matrix				*C* matrix = *B* x *A*	
Priority Vector for selection of the best Option		**Overall Priority Matrix**				**Overall Ranking**	
			PERFORMANCE	*ECONOMIC VALUE*	*SOCIAL & ENV. ISSUES*		
PERFORMANCE	0.500	OPTION 1	0.185	0.411	0.456	OPTION 1	0.309
ECONOMIC VALUE	0.250	OPTION 2	0.407	0.261	0.349	OPTION 2	0.356
SOCIAL & ENV. ISSUES	0.250	OPTION 3	0.407	0.328	0.195	OPTION 3	0.334
Priority Vector for Economic Value		**Weight of options with respect to Present Value or Priority matrix for Economic Value**				**Composite Priorities**	
			PRESENT VALUE				
PRESENT VALUE	1.000	OPTION 1	0.411			OPTION 1	0.411
		OPTION 2	0.261			OPTION 2	0.261
		OPTION 3	0.328			OPTION 3	0.328
Priority Vector for Performance		**Priority matrix for Performance**				**Composite Priorities**	
			FAILURE TOLERANCE	*RELIABILITY*			
FAILURE TOLERANCE	0.667	OPTION 1	0.111	0.333		OPTION 1	0.185
RELIABILITY	0.333	OPTION 2	0.444	0.333		OPTION 2	0.407
		OPTION 3	0.444	0.333		OPTION 3	0.407
Priority Vector for Social & Environmental Issues		**Priority matrix for Social and Environmental Issues**				**Composite Priorities**	
			ACCEPTABILITY	*HEALTH ISSUES*	*ABSTRACTION*		
ACCEPTABILITY	0.557	OPTION 1	0.443	0.557	0.250	OPTION 1	0.456
HEALTH ISSUES	0.320	OPTION 2	0.387	0.320	0.250	OPTION 2	0.349
ABSTRACTION	0.123	OPTION 3	0.170	0.123	0.500	OPTION 3	0.195
Priority Vector for Reliability		**Weight of options with respect to Reliability**				**Composite Priorities**	
			RELIABILITY				
RELIABILITY	1.000	OPTION 1	0.333			OPTION 1	0.333
		OPTION 2	0.333			OPTION 2	0.333
		OPTION 3	0.333			OPTION 3	0.333
Priority Vector for Failure Tolerance		**Weight of options with respect to Failure Tolerance**				**Composite Priorities**	
			FAILURE TOLERANCE				
FAILURE TOLERANCE	1.000	OPTION 1	0.111			OPTION 1	0.111
		OPTION 2	0.444			OPTION 2	0.444
		OPTION 3	0.444			OPTION 3	0.444
Priority Vector for Acceptability		**Weight of options with respect to Acceptability**				**Composite Priorities**	
			ACCEPTABILITY				
ACCEPTABILITY	1.000	OPTION 1	0.443			OPTION 1	0.443
		OPTION 2	0.387			OPTION 2	0.387
		OPTION 3	0.170			OPTION 3	0.170
Priority Vector for Health Issues		**Weight of options with respect to Health Issues**				**Composite Priorities**	
			HEALTH ISSUES				
HEALTH ISSUES	1.000	OPTION 1	0.557			OPTION 1	0.557
		OPTION 2	0.320			OPTION 2	0.320
		OPTION 3	0.123			OPTION 3	0.123
Priority Vector for Abstraction		**Weight of options with respect to Abstraction**				**Composite Priorities**	
			ABSTRACTION				
ABSTRACTION	1.000	OPTION 1	0.250			OPTION 1	0.250
		OPTION 2	0.250			OPTION 2	0.250
		OPTION 3	0.500			OPTION 3	0.500

Table 8.17 Overall Ranking of Judges by Matrix Manipulation

A matrix		*B* matrix				*C* matrix = *B* x *A*	
Priority Vector for factors affecting judgment		Overall Priority Matrix				Overall Ranking	
			TECHNICAL KNOWLEDGE	*EXPERIENCE*	*PROJECT KNOWLEDGE*		
TECHNICAL KNOWLEDGE	0.333	JUDGE 1	0.500	0.250	0.250	JUDGE 1	0.333
EXPERIENCE	0.333	JUDGE 2	0.250	0.500	0.250	JUDGE 2	0.333
PROJECT KNOWLEDGE	0.333	JUDGE 3	0.250	0.250	0.500	JUDGE 3	0.333
Priority Vector for Technical Knowledge		Weight of Judges with respect to Technical Knowledge				Composite Priorities	
			TECHNICAL KNOWLEDGE				
TECHNICAL KNOWLEDGE	1.000	JUDGE 1	0.500			JUDGE 1	0.500
		JUDGE 2	0.250			JUDGE 2	0.250
		JUDGE 3	0.250			JUDGE 3	0.250
Priority Vector for Experience		Weight of Judges with respect to Experience				Composite Priorities	
			EXPERIENCE				
EXPERIENCE	1.000	JUDGE 1	0.250			JUDGE 1	0.250
		JUDGE 2	0.500			JUDGE 2	0.500
		JUDGE 3	0.250			JUDGE 3	0.250
Priority Vector for Project Knowledge		Weight of Judges with respect to Project Knowledge				Composite Priorities	
			PROJECT KNOWLEDGE				
PROJECT KNOWLEDGE	1.000	JUDGE 1	0.250			JUDGE 1	0.250
		JUDGE 2	0.250			JUDGE 2	0.250
		JUDGE 3	0.500			JUDGE 3	0.500

Table 8.18 Final Ranking of Hypothetical Network Design Options

A matrix		*B* matrix				*C* matrix = *B* x *A*	
Priority Vector for Judges		Final Priority Matrix				Final Option Ranking	
			JUDGE 1	*JUDGE 2*	*JUDGE3*		
JUDGE 1	0.333	OPTION 1	0.313	0.370	0.309	OPTION 1	0.331
JUDGE 2	0.333	OPTION 2	0.342	0.325	0.356	OPTION 2	0.341
JUDGE 3	0.333	OPTION 3	0.346	0.305	0.334	OPTION 3	0.328

Table 8.19 Wobulenzi Network Design Option 1 - Phase I

	SEGMENT ONE		SEGMENT TWO		EQUIVALENT DIAMETER LINKS		
	DIAMETER	LENGTH	DIAMETER	LENGTH	DIAMETER	LENGTH	CHW
LINK	(m)	(m)	(m)	(m)	(m)	(m)	
1 - 2	0.250	117.00	-	-	0.250	117	127.9
2 - 3	0.250	235.50	-	-	0.250	235.5	127.9
2 - 4	0.150	287.51	0.200	147.49	0.158	435	122
3 - 5	0.150	486.50	-	-	0.150	486.5	119.7
3 - 6	0.200	190.20	-	-	0.200	190.2	124.3
4 - 5	0.150	613.00	-	-	0.150	613	119.7
5 - 7	0.150	241.00	-	-	0.150	241	119.7
6 - 8	0.150	233.00	0.200	227.1	0.164	460.1	122
7 - 8	0.150	30.00	-	-	0.150	30	119.7
8 - 9	0.200	252.90	-	-	0.200	252.9	124.3
9 - 10	0.080	700.00	-	-	0.080	700	109.5
9 - 11	0.150	20.00	-	-	0.150	20	119.7
11 - 12	0.100	216.00	-	-	0.100	216	113.1
11 - 13	0.150	106.00	-	-	0.150	106	119.7
12 - 14	0.080	1.28	0.100	172.22	0.100	172.22	111.3
13 - 14	0.100	273.00	-	-	0.100	273	113.1
13 - 15	0.080	136.80	-	-	0.080	136.8	109.5
14 - 16	0.080	337.00	-	-	0.080	337	109.5

Table 8.20 Wobulenzi Network Design Option 2 - Phase I

	SEGMENT ONE		SEGMENT TWO		EQUIVALENT DIAMETER LINKS		
	DIAMETER	LENGTH	DIAMETER	LENGTH	DIAMETER	LENGTH	CHW
LINK	(m)	(m)	(m)	(m)	(m)	(m)	
1 - 2	0.250	117.00	-	-	0.250	117	127.9
2 - 3	0.250	235.50	-	-	0.250	235.5	127.9
2 - 4	0.150	435.00	-	-	0.150	435	119.7
3 - 5	0.200	190.20	-	-	0.200	190.2	124.3
3 - 6	0.100	320.45	0.150	166.05	0.106	486.5	116.4
4 - 6	0.100	33.78	0.150	579.22	0.142	613	116.4
5 - 7	0.200	460.10	-	-	0.200	460.1	124.3
6 - 8	0.150	241.00	-	-	0.150	241	119.7
7 - 8	0.100	30.00	-	-	0.100	30	113.1
7 - 9	0.200	84.16	0.250	168.74	0.225	252.9	126.1
8 - 10	0.150	250.00	-	-	0.150	250	119.7
9 - 10	0.100	216.00	-	-	0.100	216	113.1
9 - 11	0.150	20.00	-	-	0.150	20	119.7
10 - 12	0.080	484.00	-	-	0.080	484	109.5
10 - 13	0.150	20.00	-	-	0.150	20	119.7
11 - 13	0.080	56.67	0.100	159.33	0.092	216	111.3
11 - 14	0.100	106.00	-	-	0.100	106	113.1
13 - 15	0.100	99.85	0.150	73.65	0.109	173.5	116.4
14 - 15	0.080	273.00	-	-	0.080	273	109.5
14 - 16	0.080	136.80	-	-	0.080	136.8	109.5
15 - 17	0.080	337.00	-	-	0.080	337	109.5

Table 8.21 Wobulenzi Network Design Option 3 - Phase I

LINK	SEGMENT ONE DIAMETER (m)	SEGMENT ONE LENGTH (m)	SEGMENT TWO DIAMETER (m)	SEGMENT TWO LENGTH (m)	EQUIVALENT DIAMETER LINKS DIAMETER (m)	EQUIVALENT DIAMETER LINKS LENGTH (m)	EQUIVALENT DIAMETER LINKS CHW
1 - 2	0.250	117.00	-	-	0.250	117	130
2 - 3	0.200	235.50	-	-	0.200	235.5	127.5
2 - 4	0.150	435.00	-	-	0.150	435	122.9
3 - 5	0.200	190.20	-	-	0.200	190.2	127.5
3 - 6	0.100	322.23	0.150	164.27	0.106	486.5	119.6
4 - 6	0.100	261.42	0.150	351.58	0.114	613	119.6
5 - 7	0.200	460.10	-	-	0.200	460.1	127.5
6 - 8	0.150	241.00	-	-	0.150	241	122.9
7 - 8	0.100	30.00	-	-	0.100	30	116.3
7 - 9	0.150	252.90	-	-	0.150	252.9	122.9
8 - 10	0.100	153.91	0.150	96.09	0.108	250	119.6
9 - 10	0.080	94.58	0.100	121.42	0.088	216	114.5
9 - 11	0.150	20.00	-	-	0.150	20	122.9
10 - 12	0.080	484.00	-	-	0.080	484	112.7
10 - 13	0.150	20.00	-	-	0.150	20	122.9
11 - 13	0.080	216.00	-	-	0.080	216	112.7
11 - 14	0.100	106.00	-	-	0.100	106	116.3
13 - 15	0.100	57.59	0.150	115.91	0.118	173.5	119.6
14 - 15	0.080	273.00	-	-	0.080	273	112.7
14 - 16	0.080	136.80	-	-	0.080	136.8	112.7
15 - 17	0.080	337.00	-	-	0.080	337	112.7

Table 8.22 Wobulenzi Network Design Option 1 - Phase II

LINK	TOTAL LINK FLOW (l/s)	(A-B): EXISTING UNPARALLELED LINK DIAM. (m)	(A-B) LENGTH (m)	(A-B) CHW	(B-C): PARALLEL LINK DIAM. (m)	(B-C) LENGTH (m)	(B-C) CHW	(C-D): REPLACED LINK DIAM. (m)	(C-D) LENGTH (m)	(C-D) CHW
1 - 2	184.20	-	-	-	0.300	117.00	130.0	-	-	-
2 - 3	138.36	0.250	235.50	118.3	-	-	-	-	-	-
2 - 4	45.84	0.158	435.00	111.0	-	-	-	-	-	-
3 - 5	37.25	0.150	454.04	110.1	0.200	32.46	127.6	-	-	-
3 - 6	94.16	-	-	-	0.200	190.20	127.6	-	-	-
4 - 5	18.62	0.150	613.00	110.1	-	-	-	-	-	-
5 - 7	48.91	-	-	-	0.150	223.44	122.9	-	-	-
					0.200	17.56	127.6			
6 - 8	58.69	-	-	-	0.150	460.10	122.9	-	-	-
7 - 8	29.34	0.150	30.00	110.1	-	-	-	-	-	-
8 - 9	69.86	0.200	252.90	114.7	-	-	-	-	-	-
9 - 10	7.40	-	-	-	0.080	638.73	112.7	-	-	-
					0.100	61.27	116.4			
9 - 11	54.05	-	-	-	0.200	20.00	127.6	-	-	-
11 - 12	17.12	-	-	-	0.100	135.15	116.4	-	-	-
					0.150	80.85	122.9			
11 - 13	29.92	0.150	106.00	110.1	-	-	-	-	-	-
12 - 14	7.57	0.100	173.50	103.6	-	-	-	-	-	-
13 - 14	15.15	-	-	-	0.080	47.09	112.7	-	-	-
					0.100	225.91	116.4			
13 - 15	7.89	-	-	-	0.080	5.59	112.7	0.080	131.21	112.7
14 - 16	10.96	-	-	-	0.100	337.00	116.4	-	-	-

Table 8.23 Wobulenzi Network Design Option 2 - Phase II

LINK	TOTAL LINK FLOW (l/s)	(A-B): EXISTING UNPARALLELED LINK DIAM. (m)	LENGTH (m)	CHW	(B-C): PARALLEL LINK DIAM. (m)	LENGTH (m)	CHW	(C-D): REPLACED LINK DIAM. (m)	LENGTH (m)	CHW
1 - 2	184.20	-	-	-	0.300	117.00	130.0	-	-	-
2 - 3	139.44	0.250	235.50	118.3	-	-	-	-	-	-
2 - 4	44.77	-	-	-	0.150	326.81	122.9	0.150	108.19	122.9
3 - 5	114.93	-	-	-	0.250	190.20	130.0	-	-	-
3 - 6	17.55	-	-	-	0.100	406.91	116.4	-	-	-
					0.150	79.59	122.9			
4 - 6	17.55	0.142	613.00	109.2	-	-	-	-	-	-
5 - 7	79.46	0.200	460.10	114.7	-	-	-	-	-	-
6 - 8	28.14	0.150	241.00	110.1	-	-	-	-	-	-
7 - 8	14.07	-	-	-	-	-	-	0.100	30.00	116.4
7 - 9	47.21	0.225	252.90	116.7	-	-	-	-	-	-
8 - 10	22.64	0.150	231.50	110.1	0.100	18.50	116.4	-	-	-
9 - 10	7.55	0.100	214.53	103.5	0.100	1.47	116.4	-	-	-
9 - 11	31.27	0.150	20.00	110.1	-	-	-	-	-	-
10 - 12	7.40	-	-	-	-	-	-	0.080	484.00	112.7
10 - 13	22.79	0.150	20.00	110.1	-	-	-	-	-	-
11 - 13	5.70	0.092	216.00	102.2	-	-	-	-	-	-
11 - 14	18.56	-	-	-	0.100	106.00	116.4	-	-	-
13 - 15	18.94	-	-	-	0.100	123.97	116.4	-	-	-
					0.150	49.53	122.9			
14 - 15	3.79	0.080	273.00	99.9	-	-	-	-	-	-
14 - 16	7.89	-	-	-	-	-	-	0.080	136.80	112.7
15 - 17	10.96	-	-	-	0.080	337.00	112.7	-	-	-

Table 8.24 Wobulenzi Network Design Option 3 - Phase II

LINK	TOTAL LINK FLOW (l/s)	(A-B): EXISTING UNPARALLELED LINK DIAM. (m)	LENGTH (m)	CHW	(B-C): PARALLEL LINK DIAM. (m)	LENGTH (m)	CHW	(C-D): REPLACED LINK DIAM. (m)	LENGTH (m)	CHW
1 - 2	170.68	0.250	117.00	118.3	-	-	-	-	-	-
2 - 3	128.92	-	-	-	0.250	140.59	127.9	0.250	94.91	127.9
2 - 4	41.77	-	-	-	0.200	435.00	124.3	-	-	-
3 - 5	106.26	-	-	-	0.200	190.20	124.3	-	-	-
3 - 6	16.23	0.106	317.63	104.5	0.100	168.87	113.1	-	-	-
4 - 6	16.23	0.114	613.00	105.7	-	-	-	-	-	-
5 - 7	73.47	0.200	460.10	114.7	-	-	-	-	-	-
6 - 8	26.02	0.150	241.00	110.1	-	-	-	-	-	-
7 - 8	13.01	0.100	30.00	103.5	-	-	-	-	-	-
7 - 9	43.65	0.150	147.39	110.1	0.200	105.51	124.3	-	-	-
8 - 10	20.93	-	-	-	0.100	250.00	113.1	-	-	-
9 - 10	6.98	0.088	216.00	101.4	-	-	-	-	-	-
9 - 11	28.91	0.150	20.00	110.1	-	-	-	-	-	-
10 - 12	6.84	0.080	401.78	99.9	0.080	82.22	109.5	-	-	-
10 - 13	21.07	0.150	20.00	110.1	-	-	-	-	-	-
11 - 13	5.27	0.080	216.00	99.9	-	-	-	-	-	-
11 - 14	17.16	-	-	-	0.100	106.00	113.1	-	-	-
13 - 15	17.51	0.118	39.40	106.3	0.150	134.10	119.7	-	-	-
14 - 15	3.50	0.080	273.00	99.9	-	-	-	-	-	-
14 - 16	7.29	0.080	136.80	99.9	-	-	-	-	-	-
15 - 17	10.13	-	-	-	0.080	337.00	109.5	-	-	-

Table 8.25 Costs for the Wobulenzi Network Design Option 1

PHASE I TIME (Yrs)	PHASE I COSTS ($)	PHASE II COSTS ($)	OVERALL COSTS ($)
7	2,884,171.00	1,237,876.88	4,122,047.75
8	2,957,466.75	1,121,718.88	4,079,185.50
9	3,035,500.50	1,013,081.19	4,048,581.75
10	3,131,902.25	874,496.38	4,006,398.50
11	3,248,394.50	660,811.50	3,909,206.00
12	3,347,645.50	567,311.19	3,914,956.75
13	3,484,607.25	439,868.69	3,924,476.00
14	3,590,892.25	390,206.75	3,981,099.00

Table 8.26 Costs for the Wobulenzi Network Design Option 2

PHASE I TIME (Yrs)	PHASE I COSTS ($)	PHASE II COSTS ($)	OVERALL COSTS ($)
7	2,907,129.25	1,385,789.88	4,292,919.00
8	3,006,018.75	1,070,954.50	4,076,973.25
9	3,084,275.25	966,109.13	4,050,384.25
10	3,200,343.75	788,534.38	3,988,878.00
11	3,314,566.00	639,097.19	3,953,663.25
12	3,414,076.25	543,752.69	3,957,829.00
13	3,523,241.00	460,938.06	3,984,179.00
14	3,630,847.00	409,400.34	4,040,247.25

Table 8.27 Costs for the Wobulenzi Network Design Option 3

PHASE I TIME (Yrs)	PHASE I COSTS ($)	PHASE II COSTS ($)	OVERALL COSTS ($)
7	2,936,328.00	680,006.69	3,616,334.75
8	3,010,212.75	612,845.88	3,623,058.75
9	3,121,524.75	500,112.31	3,621,637.00
10	3,205,193.50	453,741.31	3,658,934.75
11	3,319,625.75	329,776.69	3,649,402.50
12	3,419,437.50	262,392.34	3,681,829.75
13	3,529,649.00	220,733.61	3,750,382.50
14	3,719,996.25	157,335.23	3,877,331.50

Table 8.28 Wobulenzi Network Performance Results for Option 1

LAYOUT No.	UNAVAILABLE COMPONENTS LINK No.	UNAVAILABLE COMPONENTS LINK No.	MECHANICAL RELIABILITY (A)	TOTAL OUTFLOW (l/s) (B)	(A) * (B)
1	0	0	0.992277	183.76	182.343
2	1	0	0.000152	0.00	0.000
3	2	0	0.000235	42.45	0.010
4	3	0	0.000402	160.31	0.065
5	4	0	0.000428	172.51	0.074
6	5	0	0.000224	115.00	0.026
7	6	0	0.000428	184.29	0.079
8	7	0	0.000312	181.36	0.057
9	8	0	0.000298	146.77	0.044
10	9	0	0.000428	184.29	0.079
11	10	0	0.000305	114.39	0.035
12	11	0	0.000658	176.88	0.116
13	12	0	0.000253	130.21	0.033
14	13	0	0.000484	179.71	0.087
15	14	0	0.000428	170.37	0.073
16	15	0	0.000690	183.76	0.127
17	16	0	0.000525	180.03	0.095
18	17	0	0.000884	176.40	0.156
19	18	0	0.000562	173.33	0.097
20	1	2	0.000000	0.00	0.000
21	1	3	0.000000	0.00	0.000
22	1	4	0.000000	0.00	0.000
23	1	5	0.000000	0.00	0.000
24	1	6	0.000000	0.00	0.000
25	1	7	0.000000	0.00	0.000
26	1	8	0.000000	0.00	0.000
27	1	9	0.000000	0.00	0.000
28	1	10	0.000000	0.00	0.000
29	1	11	0.000000	0.00	0.000
30	1	12	0.000000	0.00	0.000
31	1	13	0.000000	0.00	0.000
32	1	14	0.000000	0.00	0.000
33	1	15	0.000000	0.00	0.000
34	1	16	0.000000	0.00	0.000
35	1	17	0.000000	0.00	0.000
36	1	18	0.000000	0.00	0.000
37	2	3	0.000000	0.00	0.000
38	2	4	0.000000	56.78	0.000
39	2	5	0.000000	57.78	0.000
40	2	6	0.000000	27.22	0.000
41	2	7	0.000000	42.49	0.000
42	2	8	0.000000	53.78	0.000
43	2	9	0.000000	51.63	0.000
44	2	10	0.000000	42.49	0.000
SUMMATION			0.999974		183.594
	NETWORK RELIABILITY = 0.996237 COMPONENT FAILURE TOLERANCE = 0.880516 THE OVERALL CPU-TIME = 1.843 seconds				

Table 8.29 Wobulenzi Network Performance Results for Option 2

LAYOUT No.	UNAVAILABLE COMPONENTS LINK No.	LINK No.	MECHANICAL RELIABILITY (A)	TOTAL OUTFLOW (l/s) (B)	(A) * (B)
1	0	0	0.989357	184.29	182.329
2	1	0	0.000151	0.00	0.000
3	2	0	0.000234	59.78	0.014
4	3	0	0.000353	157.99	0.056
5	4	0	0.000190	93.35	0.018
6	5	0	0.000479	184.29	0.088
7	6	0	0.000455	184.29	0.084
8	7	0	0.000304	128.17	0.039
9	8	0	0.000427	180.61	0.077
10	9	0	0.000688	184.29	0.127
11	10	0	0.000265	170.83	0.045
12	11	0	0.000427	182.44	0.078
13	12	0	0.000696	184.29	0.128
14	13	0	0.000427	178.23	0.076
15	14	0	0.000894	176.88	0.158
16	15	0	0.000427	177.33	0.076
17	16	0	0.000758	184.29	0.140
18	17	0	0.000505	176.40	0.089
19	18	0	0.000467	171.54	0.080
20	19	0	0.000894	184.29	0.165
21	20	0	0.000894	176.40	0.158
22	21	0	0.000657	173.33	0.114
23	1	2	0.000000	0.00	0.000
24	1	3	0.000000	0.00	0.000
25	1	4	0.000000	0.00	0.000
26	1	5	0.000000	0.00	0.000
27	1	6	0.000000	0.00	0.000
28	1	7	0.000000	0.00	0.000
29	1	8	0.000000	0.00	0.000
30	1	9	0.000000	0.00	0.000
31	1	10	0.000000	0.00	0.000
32	1	11	0.000000	0.00	0.000
33	1	12	0.000000	0.00	0.000
34	1	13	0.000000	0.00	0.000
35	1	14	0.000000	0.00	0.000
36	1	15	0.000000	0.00	0.000
37	1	16	0.000000	0.00	0.000
38	1	17	0.000000	0.00	0.000
39	1	18	0.000000	0.00	0.000
40	1	19	0.000000	0.00	0.000
41	1	20	0.000000	0.00	0.000
42	1	21	0.000000	0.00	0.000
43	2	3	0.000000	0.00	0.000
44	2	4	0.000000	53.78	0.000
45	2	5	0.000000	58.74	0.000
46	2	6	0.000000	27.22	0.000
47	2	7	0.000000	60.28	0.000
48	2	8	0.000000	60.38	0.000
49	2	9	0.000000	60.60	0.000
50	2	10	0.000000	60.58	0.000
SUMMATION			0.999949		184.137

NETWORK RELIABILITY = 0.999197
COMPONENT FAILURE TOLERANCE = 0.924534
THE OVERALL CPU-TIME = 2.703 seconds

Table 8.30 Wobulenzi Network Performance Results for Option 3

LAYOUT No.	UNAVAILABLE COMPONENTS LINK No.	UNAVAILABLE COMPONENTS LINK No.	MECHANICAL RELIABILITY (A)	TOTAL OUTFLOW (l/s) (B)	(A) * (B)
1	0	0	0.988749	158.24	156.460
2	1	0	0.000234	0.00	0.000
3	2	0	0.000211	47.03	0.010
4	3	0	0.000254	134.25	0.034
5	4	0	0.000223	73.34	0.016
6	5	0	0.000601	158.24	0.095
7	6	0	0.000589	158.24	0.093
8	7	0	0.000304	103.76	0.032
9	8	0	0.000426	157.02	0.067
10	9	0	0.000687	158.24	0.109
11	10	0	0.000387	139.73	0.054
12	11	0	0.000483	152.01	0.073
13	12	0	0.000799	158.24	0.126
14	13	0	0.000426	151.03	0.064
15	14	0	0.000868	151.91	0.132
16	15	0	0.000426	150.50	0.064
17	16	0	0.000893	158.24	0.141
18	17	0	0.000504	152.56	0.077
19	18	0	0.000433	148.47	0.064
20	19	0	0.000893	158.24	0.141
21	20	0	0.000893	151.50	0.135
22	21	0	0.000656	148.88	0.098
23	1	2	0.000000	0.00	0.000
24	1	3	0.000000	0.00	0.000
25	1	4	0.000000	0.00	0.000
26	1	5	0.000000	0.00	0.000
27	1	6	0.000000	0.00	0.000
28	1	7	0.000000	0.00	0.000
29	1	8	0.000000	0.00	0.000
30	1	9	0.000000	0.00	0.000
31	1	10	0.000000	0.00	0.000
32	1	11	0.000000	0.00	0.000
33	1	12	0.000000	0.00	0.000
34	1	13	0.000000	0.00	0.000
35	1	14	0.000000	0.00	0.000
36	1	15	0.000000	0.00	0.000
37	1	16	0.000000	0.00	0.000
38	1	17	0.000000	0.00	0.000
39	1	18	0.000000	0.00	0.000
40	1	19	0.000000	0.00	0.000
41	1	20	0.000000	0.00	0.000
42	1	21	0.000000	0.00	0.000
43	2	3	0.000000	0.00	0.000
44	2	4	0.000000	46.49	0.000
45	2	5	0.000000	46.69	0.000
46	2	6	0.000000	23.99	0.000
47	2	7	0.000000	47.09	0.000
48	2	8	0.000000	45.80	0.000
49	2	9	0.000000	47.04	0.000
50	2	10	0.000000	47.05	0.000
SUMMATION			0.999943		158.087

NETWORK RELIABILITY = 0.999060
COMPONENT FAILURE TOLERANCE = 0.916449
THE OVERALL CPU-TIME = 2.152 seconds

Table 8.31 Summary of Results for the Wobulenzi Network Designs

PARAMETER	OPTION 1	OPTION 2	OPTION 3
$p(0)$	0.992277	0.989357	0.988749
NETWORK RELIABILITY	0.996237	0.999197	0.999060
FAILURE TOLERANCE	0.880516	0.924534	0.916449
PRESENT VALUE OF COSTS ($)	3,909,206.00	3,953,663.25	3,616,334.75

Table 8.32 Scale A of Relative Intensities and Measurement Data

Numerical Rating on the AHP Comparison Scale	PERCENTAGE DIFFERENCE, d', OF COMPARED VALUES		
	ECONOMIC VALUE	RELIABILITY	FAILURE TOLERANCE
1	$0 < d' < 10$	$0 < d' < 5$	$0 < d' < 5$
2	$10 < d' < 20$	$5 < d' < 10$	$5 < d' < 10$
3	$20 < d' < 30$	$10 < d' < 15$	$10 < d' < 15$
4	$30 < d' < 40$	$15 < d' < 25$	$15 < d' < 25$
5	$40 < d' < 50$	$25 < d' < 35$	$25 < d' < 35$
6	$50 < d' < 60$	$35 < d' < 40$	$35 < d' < 40$
7	$60 < d' < 70$	$40 < d' < 45$	$40 < d' < 45$
8	$70 < d' < 80$	$45 < d' < 60$	$45 < d' < 60$
9	$d' > 80$	$60 < d' <= 100$	$60 < d' <= 100$

Table 8.33 Overall Ranking of Wobulenzi Network Options by Judge 1

A matrix		*B* matrix				*C* matrix = *B* x *A*	
Priority Vector for selection of the best Option		Overall Priority Matrix				Overall Ranking	
			PERFORMANCE	*ECONOMIC VALUE*	*SOCIAL & ENV. ISSUES*		
PERFORMANCE	0.400	OPTION 1	0.285	0.333	0.371	OPTION 1	0.322
ECONOMIC VALUE	0.400	OPTION 2	0.385	0.333	0.371	OPTION 2	0.362
SOCIAL & ENV. ISSUES	0.200	OPTION 3	0.330	0.333	0.257	OPTION 3	0.317
Priority Vector for Economic Value		Weight of options with respect to Present Value or Priority matrix for Economic Value				Composite Priorities	
			PRESENT VALUE				
		OPTION 1	0.333			OPTION 1	0.333
PRESENT VALUE	1.000	OPTION 2	0.333			OPTION 2	0.333
		OPTION 3	0.333			OPTION 3	0.333
Priority Vector for Performance		Priority matrix for Performance Sub-criteria				Composite Priorities	
			FAILURE TOLERANCE	*RELIABILITY*			
FAILURE TOLERANCE	0.667	OPTION 1	0.261	0.333		OPTION 1	0.285
RELIABILITY	0.333	OPTION 2	0.411	0.333		OPTION 2	0.385
		OPTION 3	0.328	0.333		OPTION 3	0.330
Priority Vector for Social & Environmental Issues		Priority matrix for Social and Environmental Issues				Composite Priorities	
			ACCEPTABILITY	*HEALTH ISSUES*	*ABSTRACTION*		
ACCEPTABILITY	0.557	OPTION 1	0.400	0.400	0.167	OPTION 1	0.371
HEALTH ISSUES	0.320	OPTION 2	0.400	0.400	0.167	OPTION 2	0.371
ABSTRACTION	0.123	OPTION 3	0.200	0.200	0.667	OPTION 3	0.257
Priority Vector for Reliability		Weight of options with respect to Reliability				Composite Priorities	
			RELIABILITY				
		OPTION 1	0.333			OPTION 1	0.333
RELIABILITY	1.000	OPTION 2	0.333			OPTION 2	0.333
		OPTION 3	0.333			OPTION 3	0.333
Priority Vector for Failure Tolerance		Weight of options with respect to Failure Tolerance				Composite Priorities	
			FAILURE TOLERANCE				
		OPTION 1	0.261			OPTION 1	0.261
FAILURE TOLERANCE	1.000	OPTION 2	0.411			OPTION 2	0.411
		OPTION 3	0.328			OPTION 3	0.328
Priority Vector for Acceptability		Weight of options with respect to Acceptability				Composite Priorities	
			ACCEPTABILITY				
		OPTION 1	0.400			OPTION 1	0.400
ACCEPTABILITY	1.000	OPTION 2	0.400			OPTION 2	0.400
		OPTION 3	0.200			OPTION 3	0.200
Priority Vector for Health Issues		Weight of options with respect to Health Issues				Composite Priorities	
			HEALTH ISSUES				
		OPTION 1	0.400			OPTION 1	0.400
HEALTH ISSUES	1.000	OPTION 2	0.400			OPTION 2	0.400
		OPTION 3	0.200			OPTION 3	0.200
Priority Vector for Abstraction		Weight of options with respect to Abstraction				Composite Priorities	
			ABSTRACTION				
		OPTION 1	0.167			OPTION 1	0.167
ABSTRACTION	1.000	OPTION 2	0.167			OPTION 2	0.167
		OPTION 3	0.667			OPTION 3	0.667

Table 8.34 Overall Ranking of Wobulenzi Network Options by Judge 2

A matrix		*B* matrix				*C* matrix = *B* x *A*	
Priority Vector for selection of the best Option		Overall Priority Matrix				Overall Ranking	
			PERFORMANCE	*ECONOMIC VALUE*	*SOCIAL & ENV. ISSUES*		
PERFORMANCE	0.333	OPTION 1	0.285	0.333	0.513	OPTION 1	0.377
ECONOMIC VALUE	0.333	OPTION 2	0.385	0.333	0.305	OPTION 2	0.341
SOCIAL & ENV. ISSUES	0.333	OPTION 3	0.330	0.333	0.181	OPTION 3	0.281
Priority Vector for Economic Value		Weight of options with respect to Present Value or Priority matrix for Economic Value				Composite Priorities	
				PRESENT VALUE			
PRESENT VALUE	1.000	OPTION 1		0.333		OPTION 1	0.333
		OPTION 2		0.333		OPTION 2	0.333
		OPTION 3		0.333		OPTION 3	0.333
Priority Vector for Performance		Priority matrix for Performance				Composite Priorities	
			FAILURE TOLERANCE	*RELIABILITY*			
FAILURE TOLERANCE	0.667	OPTION 1	0.261	0.333		OPTION 1	0.285
RELIABILITY	0.333	OPTION 2	0.411	0.333		OPTION 2	0.385
		OPTION 3	0.328	0.333		OPTION 3	0.330
Priority Vector for Social & Environmental issues		Priority matrix for Social and Environmental issues				Composite Priorities	
			ACCEPTABILITY	*HEALTH ISSUES*	*ABSTRACTION*		
ACCEPTABILITY	0.557	OPTION 1	0.557	0.557	0.200	OPTION 1	0.513
HEALTH ISSUES	0.320	OPTION 2	0.320	0.320	0.200	OPTION 2	0.305
ABSTRACTION	0.123	OPTION 3	0.123	0.123	0.600	OPTION 3	0.181
Priority Vector for Reliability		Weight of options with respect to Reliability				Composite Priorities	
				RELIABILITY			
RELIABILITY	1.000	OPTION 1		0.333		OPTION 1	0.333
		OPTION 2		0.333		OPTION 2	0.333
		OPTION 3		0.333		OPTION 3	0.333
Priority Vector for Failure Tolerance		Weight of options with respect to Failure Tolerance				Composite Priorities	
				FAILURE TOLERANCE			
FAILURE TOLERANCE	1.000	OPTION 1		0.261		OPTION 1	0.261
		OPTION 2		0.411		OPTION 2	0.411
		OPTION 3		0.328		OPTION 3	0.328
Priority Vector for Acceptability		Weight of options with respect to Acceptability				Composite Priorities	
				ACCEPTABILITY			
ACCEPTABILITY	1.000	OPTION 1		0.557		OPTION 1	0.557
		OPTION 2		0.320		OPTION 2	0.320
		OPTION 3		0.123		OPTION 3	0.123
Priority Vector for Health Issues		Weight of options with respect to Health Issues				Composite Priorities	
				HEALTH ISSUES			
HEALTH ISSUES	1.000	OPTION 1		0.557		OPTION 1	0.557
		OPTION 2		0.320		OPTION 2	0.320
		OPTION 3		0.123		OPTION 3	0.123
Priority Vector for Abstraction		Weight of options with respect to Abstraction				Composite Priorities	
				ABSTRACTION			
ABSTRACTION	1.000	OPTION 1		0.200		OPTION 1	0.200
		OPTION 2		0.200		OPTION 2	0.200
		OPTION 3		0.600		OPTION 3	0.600

Table 8.35 Overall Ranking of Wobulenzi Network Options by Judge 3

A matrix		*B* matrix				*C* matrix = *B* x *A*	
Priority Vector for selection of the best Option		Overall Priority Matrix				Overall Ranking	
			PERFORMANCE	*ECONOMIC VALUE*	*SOCIAL & ENV. ISSUES*		
PERFORMANCE	0.500	OPTION 1	0.285	0.333	0.456	OPTION 1	0.340
ECONOMIC VALUE	0.250	OPTION 2	0.385	0.333	0.349	OPTION 2	0.363
SOCIAL & ENV. ISSUES	0.250	OPTION 3	0.330	0.333	0.195	OPTION 3	0.297
Priority Vector for Economic Value		Weight of options with respect to Present Value or Priority matrix for Economic Value				Composite Priorities	
				PRESENT VALUE			
PRESENT VALUE	1.000	OPTION 1		0.333		OPTION 1	0.333
		OPTION 2		0.333		OPTION 2	0.333
		OPTION 3		0.333		OPTION 3	0.333
Priority Vector for Performance		Priority matrix for Performance				Composite Priorities	
			FAILURE TOLERANCE	*RELIABILITY*			
FAILURE TOLERANCE	0.667	OPTION 1	0.261	0.333		OPTION 1	0.285
RELIABILITY	0.333	OPTION 2	0.411	0.333		OPTION 2	0.385
		OPTION 3	0.328	0.333		OPTION 3	0.330
Priority Vector for Social & Environmental Issues		Priority matrix for Social and Environmental Issues				Composite Priorities	
			ACCEPTABILITY	*HEALTH ISSUES*	*ABSTRACTION*		
ACCEPTABILITY	0.557	OPTION 1	0.443	0.557	0.250	OPTION 1	0.456
HEALTH ISSUES	0.320	OPTION 2	0.387	0.320	0.250	OPTION 2	0.349
ABSTRACTION	0.123	OPTION 3	0.170	0.123	0.500	OPTION 3	0.195
Priority Vector for Reliability		Weight of options with respect to Reliability				Composite Priorities	
				RELIABILITY			
RELIABILITY	1.000	OPTION 1		0.333		OPTION 1	0.333
		OPTION 2		0.333		OPTION 2	0.333
		OPTION 3		0.333		OPTION 3	0.333
Priority Vector for Failure Tolerance		Weight of options with respect to Failure Tolerance				Composite Priorities	
				FAILURE TOLERANCE			
FAILURE TOLERANCE	1.000	OPTION 1		0.261		OPTION 1	0.261
		OPTION 2		0.411		OPTION 2	0.411
		OPTION 3		0.328		OPTION 3	0.328
Priority Vector for Acceptability		Weight of options with respect to Acceptability				Composite Priorities	
				ACCEPTABILITY			
ACCEPTABILITY	1.000	OPTION 1		0.443		OPTION 1	0.443
		OPTION 2		0.387		OPTION 2	0.387
		OPTION 3		0.170		OPTION 3	0.170
Priority Vector for Health Issues		Weight of options with respect to Health Issues				Composite Priorities	
				HEALTH ISSUES			
HEALTH ISSUES	1.000	OPTION 1		0.557		OPTION 1	0.557
		OPTION 2		0.320		OPTION 2	0.320
		OPTION 3		0.123		OPTION 3	0.123
Priority Vector for Abstraction		Weight of options with respect to Abstraction				Composite Priorities	
				ABSTRACTION			
ABSTRACTION	1.000	OPTION 1		0.250		OPTION 1	0.250
		OPTION 2		0.250		OPTION 2	0.250
		OPTION 3		0.500		OPTION 3	0.500

Table 8.36 Final Ranking of Wobulenzi Network Design Options

A matrix		*B* matrix				*C* matrix = *B* x *A*	
Priority Vector for Judges		Final Priority Matrix				Final Option Ranking	
			JUDGE 1	*JUDGE 2*	*JUDGE3*		
JUDGE 1	0.333	OPTION 1	0.322	0.377	0.340	OPTION 1	0.346
JUDGE 2	0.333	OPTION 2	0.362	0.341	0.363	OPTION 2	0.355
JUDGE 3	0.333	OPTION 3	0.317	0.281	0.297	OPTION 3	0.298

Table 8.37 Sensitivity Analysis - Scenario A

A matrix		*B* matrix				*C* matrix = *B* x *A*	
Priority Vector for Judges		Final Priority Matrix				Final Option Ranking	
			JUDGE 1	*JUDGE 2*	*JUDGE3*		
JUDGE 1	0.500	OPTION 1	0.322	0.377	0.340	OPTION 1	0.342
JUDGE 2	0.300	OPTION 2	0.362	0.341	0.363	OPTION 2	0.356
JUDGE 3	0.200	OPTION 3	0.317	0.281	0.297	OPTION 3	0.302

Table 8.38 Sensitivity Analysis - Scenario B

A matrix		*B* matrix				*C* matrix = *B* x *A*	
Priority Vector for Judges		Final Priority Matrix				Final Option Ranking	
			JUDGE 1	*JUDGE 2*	*JUDGE3*		
JUDGE 1	0.200	OPTION 1	0.322	0.377	0.340	OPTION 1	0.359
JUDGE 2	0.600	OPTION 2	0.362	0.341	0.363	OPTION 2	0.350
JUDGE 3	0.200	OPTION 3	0.317	0.281	0.297	OPTION 3	0.292

Table 8.39 Sensitivity Analysis - Scenario C

A matrix		*B* matrix				*C* matrix = *B* x *A*	
Priority Vector for Judges		Final Priority Matrix				Final Option Ranking	
			JUDGE 1	*JUDGE 2*	*JUDGE3*		
JUDGE 1	0.300	OPTION 1	0.322	0.377	0.340	OPTION 1	0.342
JUDGE 2	0.200	OPTION 2	0.362	0.341	0.363	OPTION 2	0.358
JUDGE 3	0.500	OPTION 3	0.317	0.281	0.297	OPTION 3	0.300

Table 8.40 Scale B of Relative Intensities and Measurement Data

Numerical Rating on the AHP Comparison Scale	PERCENTAGE DIFFERENCE, d', OF COMPARED VALUES		
	ECONOMIC VALUE	RELIABILITY	FAILURE TOLERANCE
1	$0 < d' < 5$	$0 < d' < 5$	$0 < d' < 5$
2	$5 < d' < 10$	$5 < d' < 10$	$5 < d' < 10$
3	$10 < d' < 15$	$10 < d' < 15$	$10 < d' < 15$
4	$15 < d' < 25$	$15 < d' < 25$	$15 < d' < 25$
5	$25 < d' < 35$	$25 < d' < 35$	$25 < d' < 35$
6	$35 < d' < 40$	$35 < d' < 40$	$35 < d' < 40$
7	$40 < d' < 45$	$40 < d' < 45$	$40 < d' < 45$
8	$45 < d' < 60$	$45 < d' < 60$	$45 < d' < 60$
9	$d' > 60$	$60 < d' <= 100$	$60 < d' <= 100$

Table 8.41 Sensitivity Analysis - Scenario D

A matrix		*B* matrix				*C* matrix = *B* x *A*	
Priority Vector for Judges		Final Priority Matrix				Final Option Ranking	
			JUDGE 1	*JUDGE 2*	*JUDGE3*		
JUDGE 1	0.333	OPTION 1	0.288	0.350	0.319	OPTION 1	0.319
JUDGE 2	0.333	OPTION 2	0.328	0.314	0.342	OPTION 2	0.328
JUDGE 3	0.333	OPTION 3	0.383	0.337	0.339	OPTION 3	0.353

Table 8.42 Sensitivity Analysis - Scenario E

A matrix		*B* matrix				*C* matrix = *B* x *A*	
Priority Vector for Judges		Final Priority Matrix				Final Option Ranking	
			JUDGE 1	*JUDGE 2*	*JUDGE3*		
JUDGE 1	0.500	OPTION 1	0.288	0.350	0.319	OPTION 1	0.313
JUDGE 2	0.300	OPTION 2	0.328	0.314	0.342	OPTION 2	0.327
JUDGE 3	0.200	OPTION 3	0.383	0.337	0.339	OPTION 3	0.360

Table 8.43 Sensitivity Analysis - Scenario F

A matrix		*B* matrix				*C* matrix = *B* x *A*	
Priority Vector for Judges		Final Priority Matrix				Final Option Ranking	
			JUDGE 1	*JUDGE 2*	*JUDGE3*		
JUDGE 1	0.200	OPTION 1	0.288	0.350	0.319	OPTION 1	0.331
JUDGE 2	0.600	OPTION 2	0.328	0.314	0.342	OPTION 2	0.322
JUDGE 3	0.200	OPTION 3	0.383	0.337	0.339	OPTION 3	0.347

Table 8.44 Sensitivity Analysis - Scenario G

A matrix		*B* matrix				*C* matrix = *B* x *A*	
Priority Vector for Judges		Final Priority Matrix				Final Option Ranking	
			JUDGE 1	*JUDGE 2*	*JUDGE3*		
JUDGE 1	0.300	OPTION 1	0.288	0.350	0.319	OPTION 1	0.316
JUDGE 2	0.200	OPTION 2	0.328	0.314	0.342	OPTION 2	0.332
JUDGE 3	0.500	OPTION 3	0.383	0.337	0.339	OPTION 3	0.352

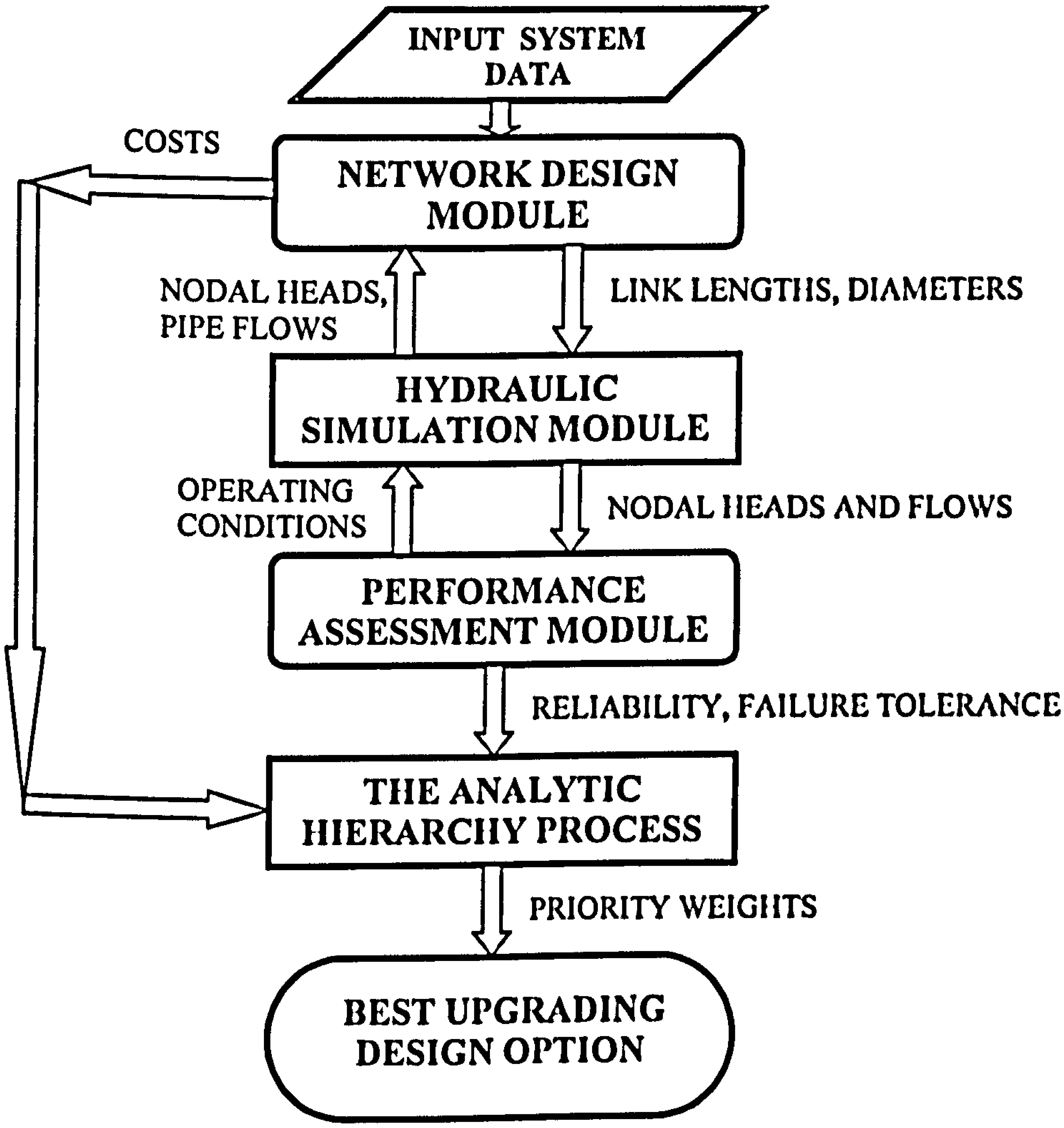

Figure 8.1 The Generalised Integrated Model Linkages

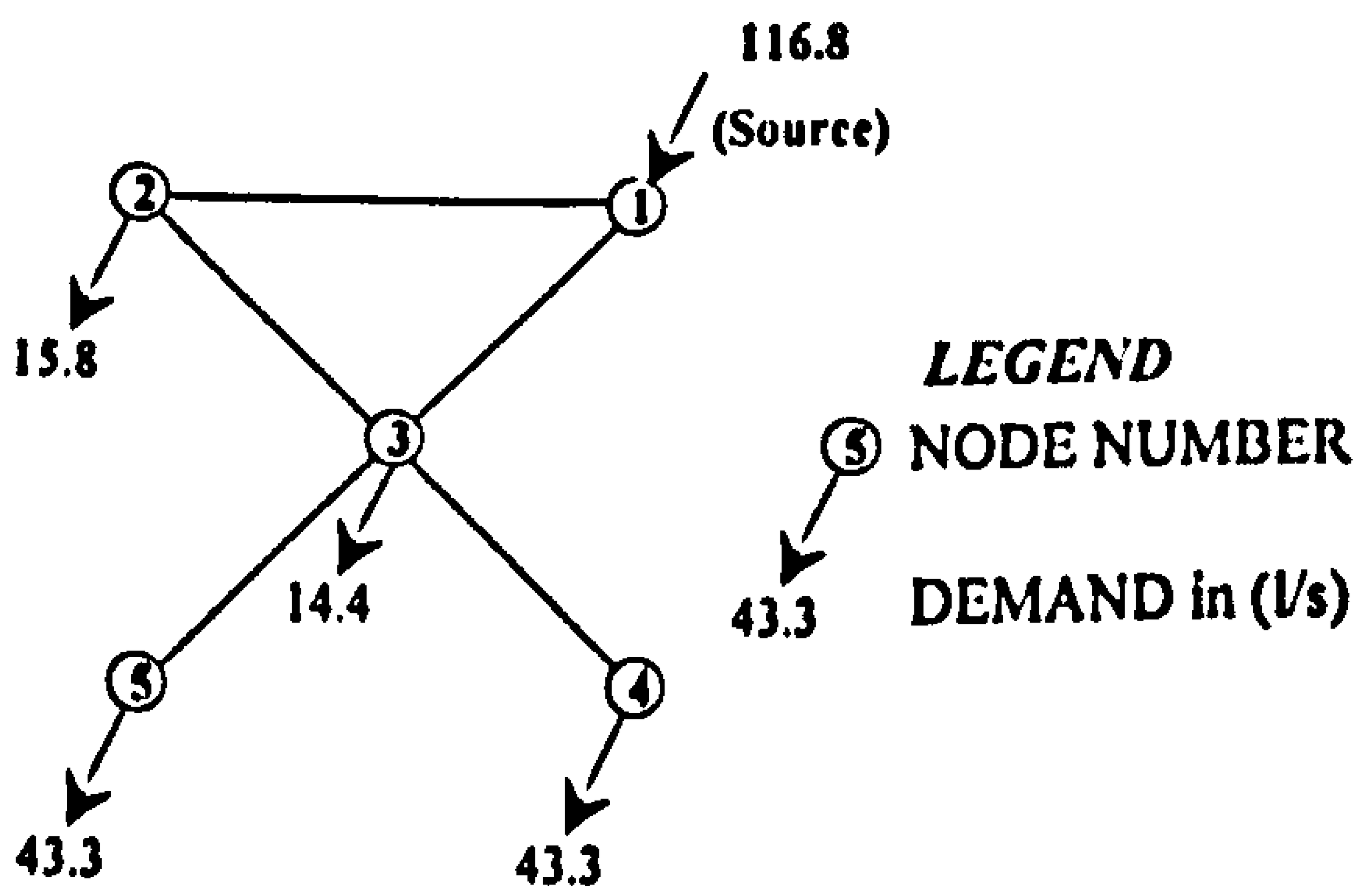

Figure 8.2 Single-loop Network

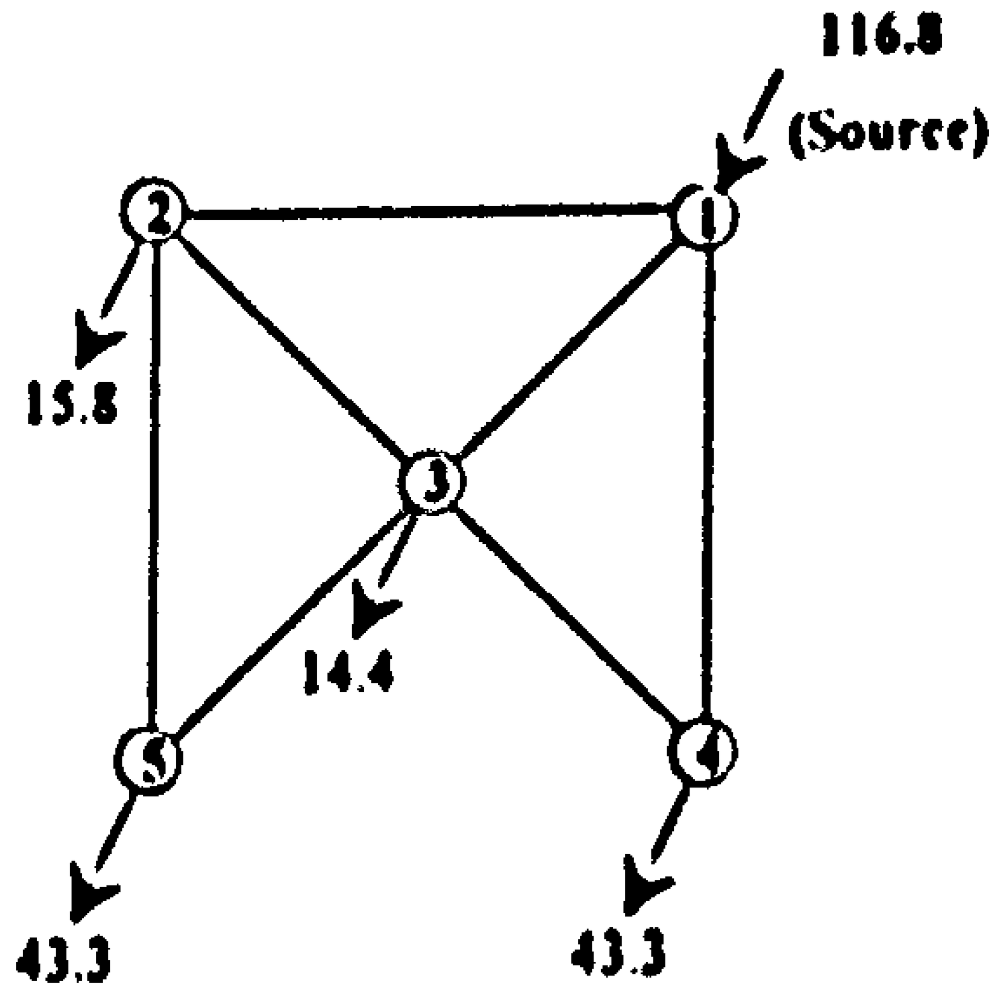

Figure 8.3 Three-loop Network A

Figure 8.4 Map of Wobulenzi Town Showing the Proposed Layout of the Water Distribution Network (*Associated Consulting Engineers, 1995a*)

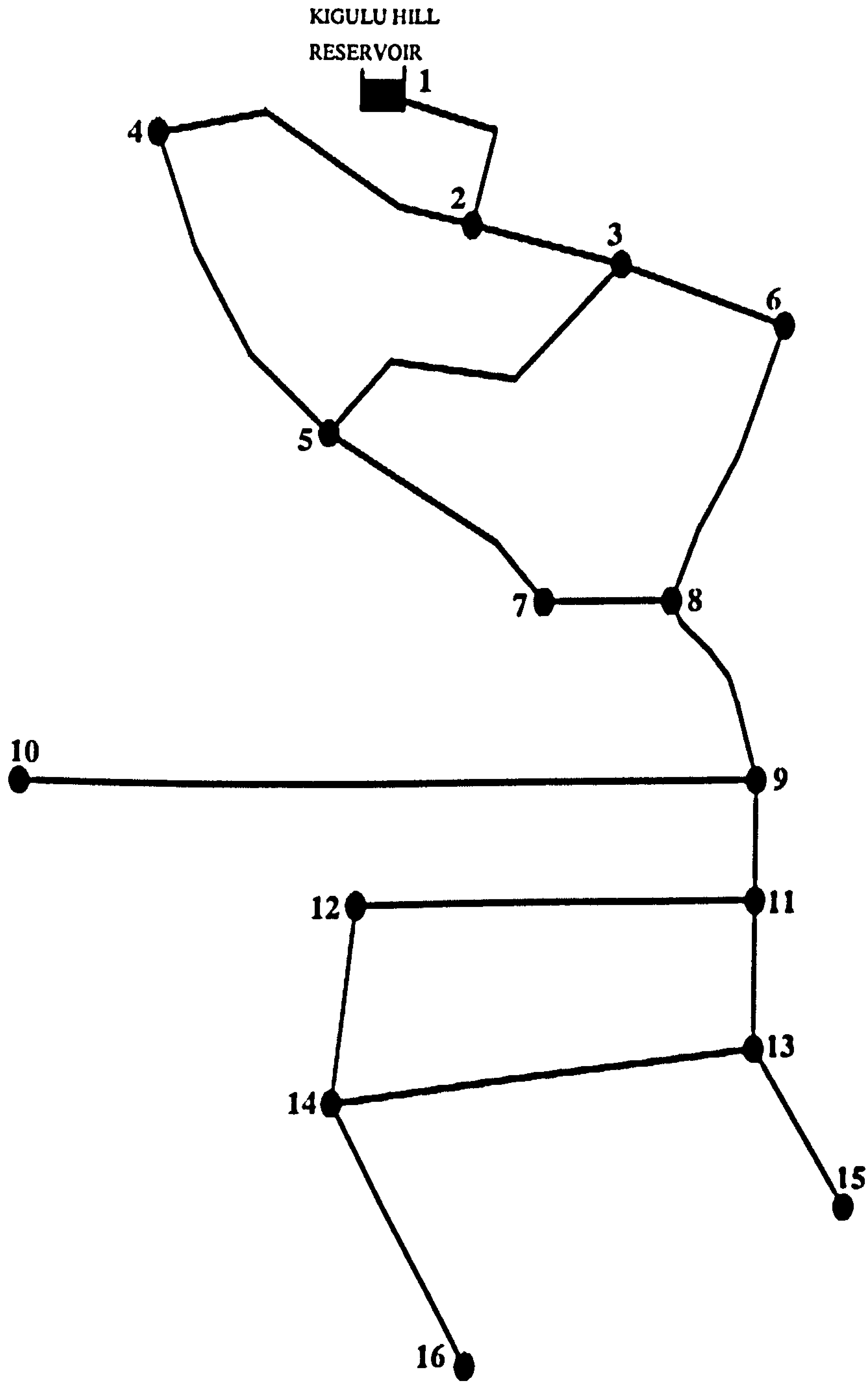

Figure 8.5 Skeletonised Layout of Wobulenzi WDN for Option 1

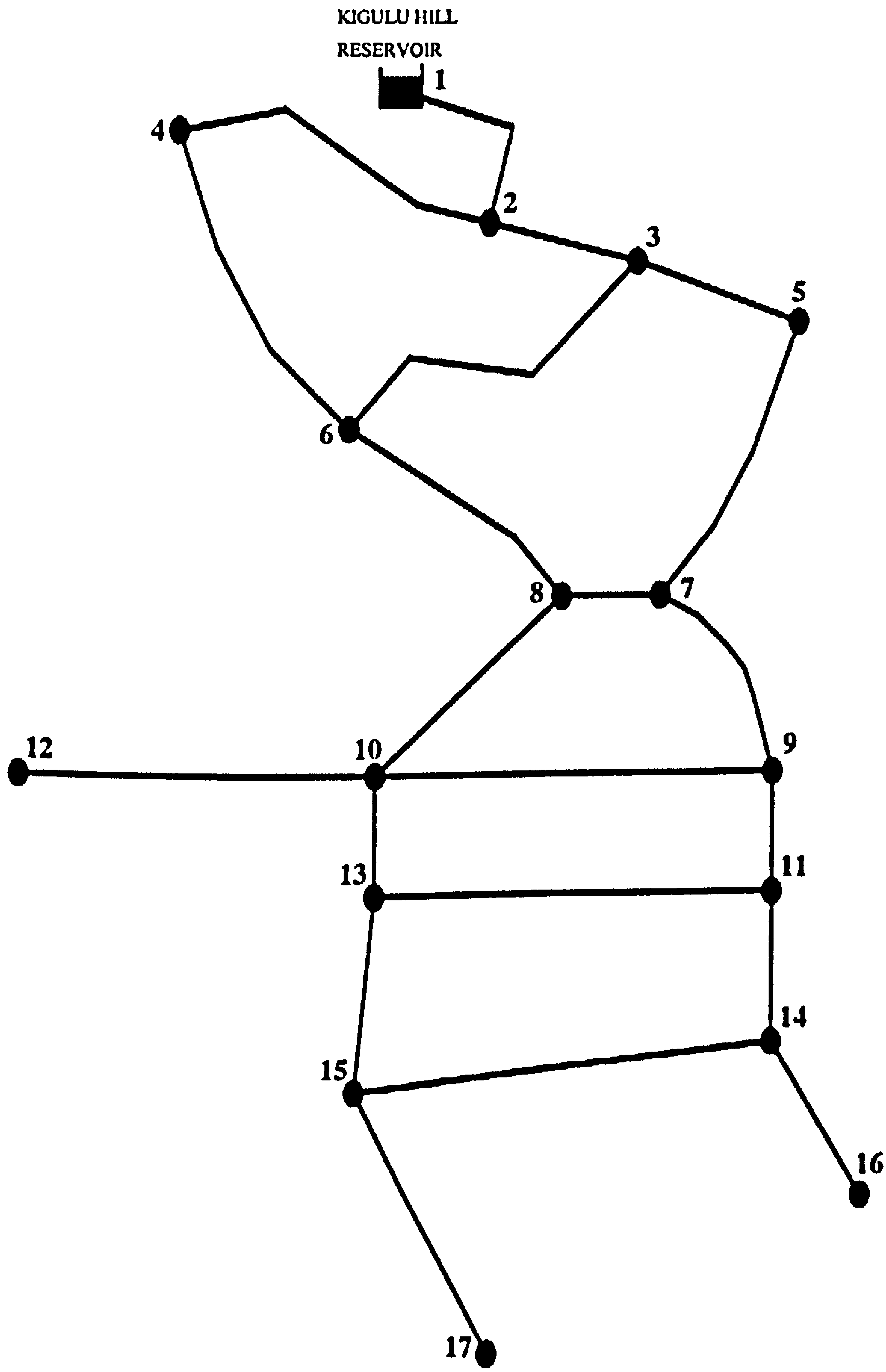

Figure 8.6 Skeletonised Layout of Wobulenzi WDN for Options 2 and 3

CHAPTER NINE

CONCLUSION

9.1 INTRODUCTION

This era of management, operation and maintenance of water distribution networks makes planning for the long-term upgrading of the networks inevitable. Escalation in the demand for water, increase in customer expectation in terms of level of service, tighter regulatory environments coupled with ever-increasing budget constraints are some of the reasons to support this fact. Worse still, the fact that pipe failures increase as the network ages, taking up a considerable proportion of the maintenance and capital expenditure of water companies stresses the need for careful planning for future expansion to minimise these costs.

The main goal of this research was to develop a practical tool that can be used to solve this problem in a comprehensive manner, which involves addressing the main issues involved, and determining the timing and magnitude of network upgrading required. This research has investigated this problem and developed a versatile tool for the optimal long-term upgrading of water distribution networks in a holistic manner to assist decision-makers in dealing with this challenge. The tool developed is referred to as the Integrated Model and it can be used to determine the optimal long-term upgrading requirements of a water distribution network, putting into consideration the network economics, hydraulic performance, social and environmental issues. The optimal timing and magnitude of future upgrading strategies is an integral capability of this tool. Successful application of the model to hypothetical and real-life networks has served as a good test for its practicability.

The Integrated Model is a combination of separate modules for network design, hydraulic simulation, assessment of network performance and the Analytic Hierarchy Process as detailed in Chapters 4, 5, 6, 7 and 8. At the end of each chapter, a summary of the methodologies and conclusions has been presented. This chapter focuses on the overall summary of what has been done in this research, general conclusions and some recommendations for further research.

9.2 OVERALL SUMMARY

Aging water distribution networks are subject to demand increases from year to year, deterioration of structural and hydraulic integrity of the pipes. This is reflected by increased frequency of bursts, decreased hydraulic capacity, reduced pressure in the network coupled with reduced quality and quantity of water. As a consequence, water companies incur network rehabilitation/expansion costs and consumer costs for the degraded system. The challenge to the decision-maker, is to determine the most cost-effective plan of upgrading the network, the magnitude and at what time in the planning horizon, subject to service requirements like system reliability, service pressure, etc. This is a complex non-linear programming problem whose magnitude cannot be underestimated. In an attempt to solve this problem, a novel technique for the long-term optimal upgrading of water distribution networks in a holistic fashion has been presented in this research. This technique has been developed as the Integrated Model, which is an amalgamation of separate modules including FORTRAN programs for network design, hydraulic simulation and assessment of network performance, together with a spreadsheet program for the Analytic Hierarchy Process.

9.2.1 The Network Design Module

In Chapter 4, the Network Design module has been developed. It is used for long-term upgrading strategies that can satisfy hydraulic, network economics and water quality criteria. This model simultaneously considers replacement and paralleling of pipes as upgrading options and can determine exactly which pipes in the network to be upgraded. It explicitly considers loss of both the structural integrity and hydraulic capacity of network pipes over time; and it allows for the direct and indirect failure costs. Linear programming is used to combine distribution network economics and hydraulic performance for various design periods. A dynamic programming approach is used to determine the timing and magnitude of upgrading over the planning horizon. The model is also adaptable for rehabilitation strategies and it can accommodate joint pricing and network upgrading policies. These are policies in which the price of water is increased as demand approaches the system capacity to reduce water consumption.

Outstanding features of the Network Design Module are in its ability to reduce the complexity and dimensionality of the problem. First, the link flows are pre-specified using an entropy-based algorithm for feasible flow distribution (Yassin-Kassab et al., 1999). This is an algorithm for least biased flow distribution as it plays a major role in reducing the complexity of the problem. Designing networks to carry maximum entropy flows considerably simplifies the optimisation problem by eliminating flow variables and flow equilibrium constraints. Thus, the problem is reduced to that of sizing the pipes for pre-specified flows and can therefore be solved by linear programming e.g. the segmental approach of Alperovits and Shamir (1977) that has been used in this research. Secondly, an algorithm that has been developed in this study for limiting the number of link segment diameters on the candidate list is used for reducing the number of variables. In effect, this algorithm reduces the dimensionality of the problem and the computational effort. The algorithm is driven by the limiting values of link flow velocities and hydraulic gradients.

9.2.2 The Hydraulic Simulation Module

In Chapter 5, a new algorithm for pressure driven analysis or head driven analysis (HDA) of water supply networks has been developed in this thesis. This algorithm is referred to as the Critical-node Head Driven Simulation Method (CHDSM). When nodal heads are adequate, the designated demands are fully satisfied and this situation can be satisfactorily modelled using the traditional Demand Driven Analysis (DDA) methods. However, under sub-normal circumstances e.g. when there are component failures or the design demand is exceeded, DDA cannot be used to effectively model this situation, but it can only indicate that a supply problem will arise. On the other hand, CHDSM can model this situation effectively and simulate partial flow or no delivery at nodes with insufficient pressure.

The CHDSM algorithm involves a systematic identification of no-flow nodes, partial-flow nodes and key partial-flow nodes in an iterative scheme. It simultaneously considers nodal heads and flows in the prediction of deficient-network performance. It does not require independent head-discharge relationships in its solution procedure. CHDSM can be used to analyse networks with unavailable components and it can be applied to networks with single and multiple sources. The method is dependable in that it gives results that compare well with other methods of pressure-dependent network analysis.

The Hydraulic Simulation Module has been developed in Chapter 5, based on the CHDSM algorithm. This realistic hydraulic simulator can perform conventional demand driven analysis (DDA) and Head Driven Analysis (HDA). It uses the Newton-Raphson technique enhanced with a line search and backtracking numerical routine to further improve on the efficiency of the solution methodology. Thus, this module has the capability of producing hydraulically more realistic results without any significant loss of computational efficiency compared to DDA. It is capable of simulating networks under abnormal loading conditions, and/or, with random unavailability of components.

9.2.3 The Performance Assessment Module

This module for assessing the performance of networks has also been developed in Chapter 5, based on the CHDSM algorithm as the hydraulic engine. It involves the random simulation of unavailable components and the probability that the network will be in a given full or reduced state in terms of availability of components. The key performance assessment parameters that it evaluates are reliability and failure tolerance. Reliability of the network is its ability to meet customer demands at adequate pressure under normal and abnormal operating conditions (Tanyimboh et al., 2001). Failure tolerance is defined as the expectation of the proportion of nodal or system demand that is satisfied during the periods in which some components are taken out of service for repair or maintenance (Tanyimboh and Templeman, 1998). This module can also calculate other performance assessment parameters such as the probability that all components are available and the proportion of total flow delivered to the total demand.

9.2.4 The Analytic Hierarchy Process

Chapter 6 has introduced a module for the Analytic Hierarchy Process, which has been applied to a sample network in Chapter 7. This is a robust model for multiple criteria decision-making. The AHP is popular because it can facilitate the process of selecting the best option based on measurable and subjective factors or non-commensurate objectives. For each option, decision factors such as economic costs, reliability, failure tolerance, social and environmental issues can all be combined using the AHP to choose the best option. In the AHP, information is sub-divided into a hierarchy of criteria, sub-criteria, and options. The information is then synthesized using pair-wise comparison (weight) matrices for each level of the hierarchy, to determine the relative rankings and priority vectors of options. It is on the basis of these rankings that the best option is selected.

9.2.5 The Integrated Model

The Integrated Model has been formulated in Chapter 8 and it is a combination of the various modules described in Sections 9.2.1 to 9.2.4. It can be used to obtain the best upgrading strategy out of different design options. This model provides a holistic approach to the optimal long-term upgrading of water distribution networks. It is a system-wide model in that it incorporates network reliability, network economics and hydraulics together with socio-environmental issues. The inclusion of the module for the Analytic Hierarchy Process implies that the Integrated Model is set up in a multi-objective framework. This framework enables the model to handle the complex problem of network upgrading in an efficient manner, while including numerous qualitative and quantitative decision factors. There is also a provision for sensitivity analysis to check the effect of variation of some of the parameters on the final decision. This check improves on the level of confidence in the results obtained. The Integrated Model has been successfully applied to a hypothetical network and a real-life water distribution network as a case study to demonstrate its practical capability and efficiency.

9.3 GENERAL CONCLUSIONS

The capability of the Network Design module to consider joint pricing and network upgrading policies is an important feature. These demand management policies involve the increase in the water tariffs combined with the price elasticity of demand to give a potential reduction in demand and a delay in the need to expand or upgrade the network. As a result, a reduction in the overall costs associated with such a strategy is attained. The overall costs of network upgrading are considerably sensitive to the price elasticity of demand. In general, for a given increase in water tariffs, the overall cost of upgrading decreases with a decrease in the price elasticity of demand. Network managers and planners can use this approach as a soft alternative for network upgrading that focuses on varying the price of water to maximise economic efficiency. However, this method may not be as popular to the consumers since it involves increasing the price of water.

Evidence has been provided to show that for a particular design option, the overall cost of the optimal design increases with an increase in the compound interest rate at which borrowed capital for project implementation must be paid back. However, the scheduling of upgrading probably has a very low sensitivity to this interest rate. Failure cost factors that represent the consequential costs of pipe failures such as traffic disruption and damage incurred by third parties, have been shown to have a significant influence on the overall cost of the project. The lower the failure cost factors, the lower the overall costs. These factors that have a significant influence on the overall cost of the project should be carefully examined and a proper assessment through sensitivity analysis made to establish the impact of varying these factors.

The practical application of the hydraulic simulation module that is based on the CHDSM algorithm has been demonstrated. Analysis of network examples has provided evidence that the CHDSM algorithm is capable of producing accurate results that compare favourably with other methods. The robustness of CHDSM has also been tested and shown to be satisfactory, giving hydraulically feasible results even for very low network pressures.

Another practical application of the CHDSM algorithm has been demonstrated in the process of assessing the performance of the network using key performance assessment parameters called reliability and failure tolerance. Using the Performance Assessment Module, which is driven by the CHDSM algorithm, the network performance of a number of networks has been done. It has been shown that despite the fact that the reliability parameter is commonly used; it may not be as effective in differentiating between designs that have components with high individual reliability values, even when the layouts are different. The failure tolerance parameter has been shown to be superior in exposing the vulnerability of a network especially when some components are unavailable. Further to this, it has been shown that higher network reliability does not necessarily mean a higher level of failure tolerance. With these findings, the need to use both the reliability and failure tolerance parameters together in order for a better judgment to be made on the performance of a network design has been highlighted and emphasized.

The effectiveness of the module for the analytic hierarchy process in obtaining a compromise between conflicting decision criteria for different design options has been shown. One of the advantages of this multi-objective decision-making method is in its approach of breaking down the problem into elementary components (criteria and sub-criteria) to facilitate the process of making comparison judgments of different options with respect to a specific criterion. Another advantage is that consensus of comparison judgments is reliably obtained in an open discussion, which leads to savings in the time of executing the method. Since there is an element of subjectivity of judgments in pair-wise comparisons, one of the main strengths of the AHP is the provision for measuring the inconsistency of these comparisons and revision of judgements to ensure consistence. The AHP results can easily be explained to decision-makers and stakeholders who may have a non-technical background.

The results obtained from applying the Integrated Model to hypothetical and real-life water distribution networks have provided evidence of its practical capability and efficiency. For each of the networks, there are different design options and strategies for upgrading. In both cases, the best option, timing and magnitude of upgrading, together with identification of the pipes to be paralleled and replaced, have been successfully achieved with a reasonably low computational effort. This is probably because the individual modules of the model have low computational times. The results obtained stress the importance of selecting the best alternative for network upgrading using a multi-objective framework. This is reflected by the fact that an alternative that is ranked highly with respect to a particular decision factor may not necessarily have a high rank with respect to another decision factor. For example, the cheapest network design may not necessarily be the most reliable or socially acceptable. Also, the importance of different decision factors with respect to the overall decision may not necessarily be the same. It is evident the Integrated Model is a reliable planning tool, that can facilitate the decision-making process for the long-term upgrading of water distribution networks.

The solution methodology of the Integrated Model has shown that the complex problem of network upgrading can be simplified and solved in a multi-objective framework. This should probably set a good foundation for more research to be done in this area.

9.4 SUGGESTIONS FOR FUTURE RESEARCH

The present research has provided answers to various questions related to the problem of planning for future upgrading of the distribution network. However the greatest limitation has been the time allocated for the research. Given more time, a lot more could have been done. This section therefore recommends aspects of individual modules and the Integrated Model as a whole that can be explored further.

The complications involved in the problem of long-term upgrading of water distribution networks have been discussed in this research. Widening the scope of this problem to cover the entire supply system and include other components like pumps, valves, reservoirs and treatment plants only aggravates the situation. Further research needs to go into widening the scope and covering the entire water supply system. This implies that there is need to use more powerful optimisation techniques such as genetic algorithms. Currently, these techniques require a lot of time. However, the level of advancement of computer technology, coupled with the techniques that have been used in this study to reduce the dimensionality of the problem means that this huddle is perhaps short-lived.

Purely from a network analysis point of view, the Hydraulic Simulation Module needs to be developed further to perform extended period simulation and to accommodate all the other network components like pumps, valves and tanks. The suggested approach is to incorporate the proposed algorithm used for pressure dependent network analysis (CHDSM) into existing DDA packages with minor adjustments.

Despite all the advantages of pressure dependent network analysis, the conventional demand driven analysis is still widely applied by practising engineers. Perhaps this is due to the fact that the benefits of using HDA are not obvious to the modellers and practising engineers. More research is required into packaging HDA in terms of the potential advantages and cost savings that can be accrued by using it. The findings should then be presented at meetings or conferences involving practising engineers. Such research work has to be done as a partnership between the academicians and companies in the water industry so that both parties can contribute expertise to the final product.

Since the Integrated Model developed is a tool for decision-makers and operators of water distribution networks, there is a need to develop the model to a level that may attract their attention. To do this, more research is required on the Integrated Model as a whole, proposed extra work on the individual modules notwithstanding. The main suggestions for further work on this model should focus on linking it directly to an electronic database and a Geographic Information System. This is a convenient approach to handling network data since most networks in practice tend to have very many pipes and loops.

The electronic database is a spreadsheet based software package for storing and processing information. Different tasks such as data manipulation, computational and statistical analysis can be performed using databases. The database can be used to store and retrieve information on all the customers such as their addresses, demands, etc. Information of all the network components such as pipe material diameter, connectivity, coordinates, elevations etc., is also stored in the database and can easily be retrieved. A very large water distribution network model can be set up in a very short time, by linking the hydraulic simulation module with the appropriate database. The module for the Analytic Hierarchy Process can also be set up in the database.

A Geographic Information System (GIS) is a computer-based tool for mapping and analysing objects (Walski et al. 2001) and it can be linked to the electronic database and used as a graphical interface for presenting information. GIS technology integrates common database operations such as query and statistical analysis with the unique visualisation and geographic analysis benefit offered by maps. Since GIS stores data on thematic layers linked together geographically, it can be used for displaying and communicating master plans graphically.

With respect to the long-term upgrading problem, GIS and the database can be used to locate links to be paralleled and replaced with respect to existing features on the map like roads buildings, etc. They can also be used to identify or map out a polygon of the customers within a certain distance of mains to be paralleled or replaced. Thus, advance notices can be issued to these customers informing them of the inconvenience. In the same way, the area of influence of low pressure in an area due to deteriorating pipes can be mapped out and the affected households informed about remedial action. Such situations are presented on a thematic map by using different colours and this facilitates the decision-making process. Finally, indirect costs of pipe failure in terms of the inconvenience to third parties can be estimated more accurately, since the area and consumers likely to be affected by a pipe failure can be determined and graphically presented by GIS.

It is hoped that these modifications would make the Integrated Model more attractive to network managers and practicing engineers.

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APPENDIX A

FLOW DISTRIBUTION METHODS

APPENDIX A1 – MAXIMUM ENTROPY FLOW DISTRIBUTION METHOD

A) MAXIMUM ENTROPY FLOWS FOR SINGLE-SOURCE NETWORKS

Introduction

The following is a description of the method proposed by Tanyimboh and Templeman (1993a), for calculating maximum entropy flows for single-source networks. The method consists of a node numbering algorithm, a node weighting algorithm and a flow distribution algorithm.

Initially, the node numbering algorithm is used to number nodes in a convention such that for any node that has been numbered, all nodes connected to it and immediately downstream of it are assigned a larger number than that of the numbered node. The node-weighting algorithm is then used to evaluate the number of paths between the source and each individual node. Finally, the flow distribution algorithm is then used to apportion the total outflow from a given node to the links with inflow to that node. Each of these links receives a fraction of the total outflow at the node equal to the upstream nodal weight divided by the downstream nodal weight of the link.

The details of the method are presented next, followed by an illustration of the method using a two-loop network example shown in Figure A1.1.

Maximum entropy flow distribution method for single-source networks

Node numbering algorithm

1) Assign flow directions to all links and assign number 1 to the source node. Number all nodes that are connected to it and immediately downstream of it in an ascending order.

2) Identify the numbered nodes for which all the immediate downstream nodes have not been numbered and designate them as Set A. Starting from the lowest node number of the nodes with immediate downstream nodes that are not numbered in Set A, number all nodes that are connected to it and immediately downstream of it in ascending order and designate them as Set B. Repeat this process until all Set A nodes have their downstream nodes numbered.

3) Of the numbered nodes in Set B, starting from the lowest node number of the nodes with immediate downstream nodes that are not numbered, number all nodes that are connected to it and downstream of it in ascending order. Repeat this process until all Set B nodes have their downstream nodes numbered.

4) Repeat Step 2 and 3 until all nodes have been numbered.

5) Exit.

Node-weighting algorithm

1) Assign a weight of 1 to the source node.

2) Following the node numbers in ascending order, select the next node that has not been allocated a nodal weight. Allocate a weight to this node equal to the total weight of individual nodal weights that are assigned to all nodes immediately upstream of it and connected to it.

3) Repeat Step 2 until all nodes have been allocated a nodal weight.

4) Exit.

Flow distribution algorithm

1) Select the node with the highest number. For each of the links with inflow to this node, calculate the flow in the link as a fraction of the total outflow at the node equal to the upstream nodal weight divided by the downstream nodal weight of the link.

2) Follow the node numbers in descending order and select the next node for which the inflows in the links connected to it have not been calculated. For each of these links, calculate the flow in the link as a fraction of the total outflow at the node equal to the upstream nodal weight divided by the downstream nodal weight of the link.

3) Repeat Step 2 until all the flows in all the links have been obtained.
4) Exit.

Illustration

The following is an illustration of this method using Figure A1.1.

Node numbering algorithm

1) The flow directions are assigned as shown in the figure. The source node is assigned number 1 and the nodes connected to it are numbered in ascending order, i.e. nodes 2 and 3.
2) Nodes 2 and 3 are designated as Set A. The node immediately downstream of node 2 is assigned number 4 and that immediately downstream of node 3 is assigned number 5. These two nodes 4 and 5 are designated Set B.
3) The node immediately downstream of nodes 4 and 5 is assigned number 6 to complete the node numbering process.
4) Exit.

Node-weighting algorithm

1) The source node is assigned a weight of 1 as shown in the triangle next to it.
2) The source node is the immediate upstream node of nodes 2 and 3; therefore the weight of each of these nodes is equal to that of the source node, i.e., 1.
3) Node 4 has nodes 2 and 3 as its immediate upstream nodes. Thus, its nodal weight is the sum of the weights of nodes 2 and 3 yielding a value of 2. Node 3 is the immediate upstream node of node 5. Therefore the weight of node 5 is equal to that of node 3 and it is 1.
4) Node 6 has nodes 4 and 5 as its immediate upstream nodes. Thus, its nodal weight is the sum of the weights of nodes 4 and 5 yielding a value of 3.
5) Exit.

Flow distribution algorithm

1) The highest node number is 6 and the total outflow at this node is the nodal demand, which is 105 l/s as shown in the figure. The links with inflow to this node are link 4-6 and link 5-6. The flow in each of the links 4-6 and 5-6 is calculated as a proportion of the total outflow at node 6. For link 4-6, the fraction of the total outflow is equal to the nodal weight of node 4 divided by that of node 6 and it is 2/3. Therefore, the flow in link 4-6 is obtained by multiplying 105 l/s by the fraction 2/3 which yields 70 l/s. Similarly, for link 5-6, the fraction of the total outflow is equal to the nodal weight of node 5 divided by that of node 6 and it is 1/3. Therefore, the flow in link 5-6 is obtained by multiplying 105 l/s by the fraction 1/3 which yields 35 l/s.
2) The next node in descending order is node 5. The total outflow at this node is the sum of the flow in link 5-6 (35 l/s) and the demand at the node (30 l/s) yielding a value of 65 l/s. The link with inflow to node 5 is link 3-5 and the flow in this link is obtained by multiplying 65 l/s (total outflow at the node 5) by 1/1 (the fraction equal to the nodal weight of node 3 divided by that of node 5). Thus the flow in link 3-5 is 65 l/s.
3) The total outflow at node 4 is equal to the sum of the nodal demand (50 l/s) and the flow in link 4-6 (70 l/s) i.e., 120 l/s. The links with inflow to this node are link 2-4 and link 3-4. The flow each of the links 2-4 and 3-4 is calculated as a proportion of the total outflow at node 6. For link 2-4, the fraction of the total outflow is equal to the nodal weight of node 2 divided by that of node 4 and it is 1/2. Therefore, the flow in link 2-4 is obtained by multiplying 120 l/s (total outflow at node 4) by the fraction 1/2 which yields 60 l/s. Similarly, for link 3-4, the fraction of the total outflow is equal to the nodal weight of node 3 divided by that of node 4 and it is 1/2. Therefore, the flow in link 3-4 is obtained by multiplying 120 l/s by the fraction 1/2 which yields 60 l/s.
4) The next node in descending order is node 3. The total outflow at this node is the sum of the flow in link 3-4 (60 l/s), link 3-5 (65 l/s) and the demand at the node (45 l/s) yielding a value of 170 l/s. The link with inflow to node 3 is link 1-3 and the flow in this link is obtained by multiplying 170 l/s (total outflow at the node 3) by 1/1 (the fraction equal

to the nodal weight of node 1 divided by that of node 3). Thus the flow in link 1-3 is 170 l/s.

5) The total outflow at node 2 is the sum of the flow in link 2-4 (60 l/s) and the nodal demand (70 l/s) yielding a value of 130 l/s. The link with inflow to node 2 is link 1-2 and the flow in this link is obtained by multiplying 120 l/s (total outflow at the node 2) by 1/1 (the fraction equal to the nodal weight of node 1 divided by that of node 2). Thus the flow in link 1-2 is 130 l/s.

6) Exit.

The flows obtained in the links are the maximum entropy flows for the network that are used for sizing of the pipes based on this method.

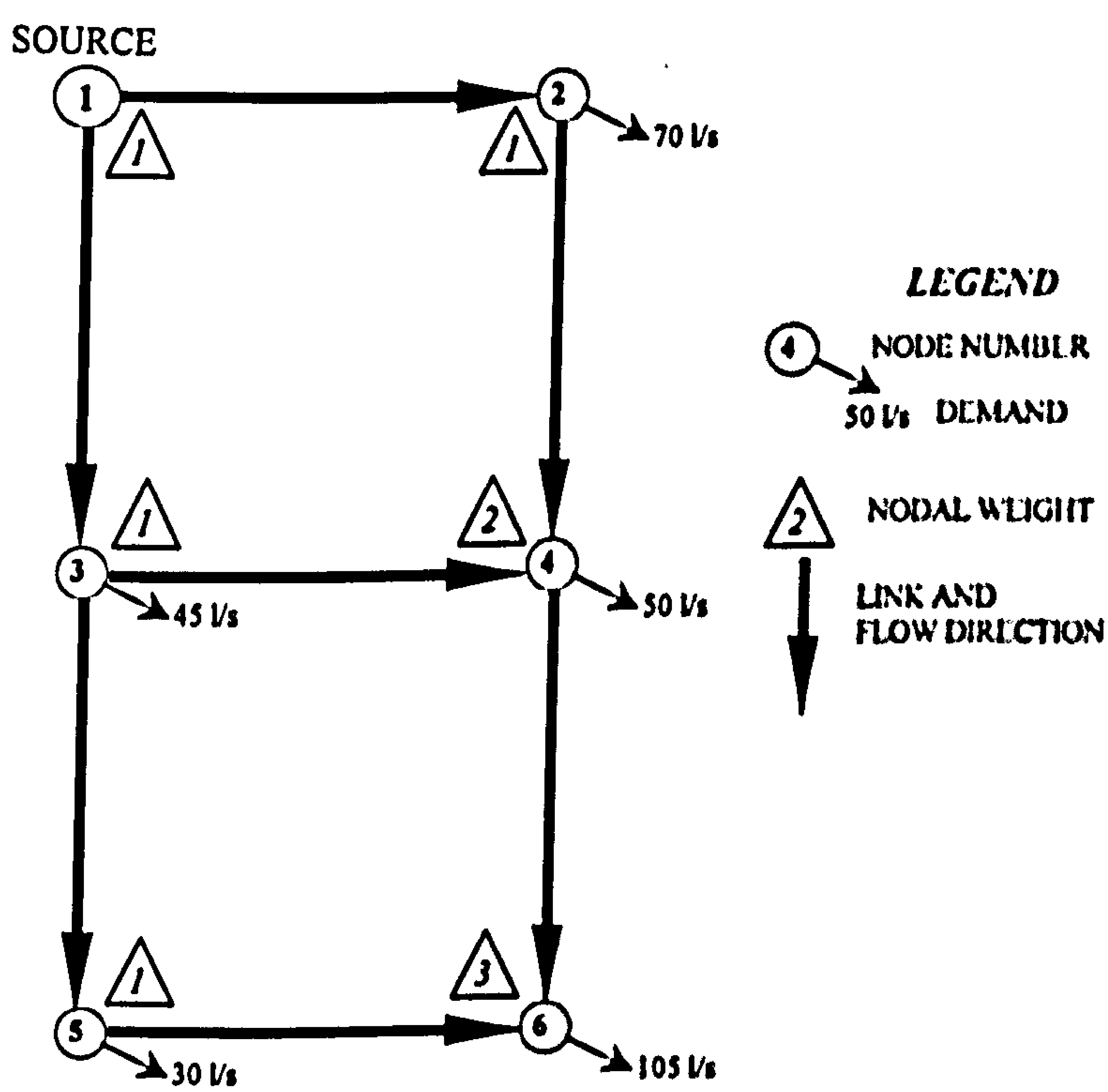

Figure A1.1 Two-loop Network

APPENDIX A2 – SHORTEST PATH FLOW DISTRIBUTION METHOD

Introduction

This appendix presents a description of the shortest path flow distribution algorithm (Orth, 1986) followed by an illustration of this algorithm on a simple network example shown in Figure A2.1.

Shortest path flow distribution algorithm

The algorithm comprises of the following steps:

1) Evaluate a branched trunk system by using either the shortest path or the minimum spanning tree principle (Orth, 1986). This evaluation involves identifying trunk links, which are paths with the shortest lengths that connect individual demand nodes to the source.
2) Determine the flow rates in closing links (additional loop-completing links in the network) by assuming either a proportion of total inflow to a node (e.g. about a third of the total inflow to the node), a minimal flow rate or a slope of the hydraulic grade line. The slope of the hydraulic grade line may be obtained as a result of dividing the difference between the ground elevations of the two end nodes of a closing link, by the length of the link. This slope may then be used with a minimal diameter in an appropriate equation for head loss in a pipe, to determine the flow rate.
3) Use the flow rates in the closing links and the nodal flow continuity or equilibrium equations at all nodes to determine the flow rates in the trunk links.

Illustration

The following is an illustration of this method using Figure A2.1. Node S is the source, and nodes W, X, Y and Z are demand nodes. The lengths of the links are shown in italics. Each of the links S-W, S-Y and Y-Z is 1000m long. The lengths of links S-X, X-Z, X-W, X-Y are 600m, 600m, 800m and 800m respectively. The demands at nodes W, X, Y and Z are 60 l/s, 35 l/s, 95 l/s and 120 l/s, respectively.

1) The shortest path trunk system, which is determined based on the shortest path connecting each demand node to the source, is made up of paths or trunk links S-W, S-X-Z and S-Y. For example, demand node W is connected to the source through path S-W with a length of 1000m and path S-X-W with a total length of 1400m. Thus, the shortest path or the trunk link in this case is S-W.

2) The closing links or loop-completing links are X-W, X-Y and Y-Z. The flow direction is determined from the nodal elevation and assumed to be from the higher to the lower node. It is assumed that the flow rate in a closing link with inflow to a given node is about a third of the total inflow to that node. Thus, the flow rate in link X-W is 20 l/s and that in link Y-Z is 40 l/s. The flow rate in link X-Y is a third of the sum of the demand at node Y (95 l/s) and the flow in link Y-Z (40 l/s), i.e., 45 l/s.

3) Using nodal flow continuity or equilibrium equations at all nodes, the flow rates in the trunk links are then obtained. Thus, the flow rate in link X-Z is 80 l/s and that in link S-W is 40 l/s. The flow rate in link S-Y is the sum of the demand at node Y (95 l/s) and the flow rate in link Y-Z (40 l/s) less that in link X-Y (45 l/s), which yields 90 l/s. The flow rate in link S-X is the sum of the demand at node X (35 l/s) and the flow rates in links X-W, X-Y and X-Z (20 l/s, 45 l/s and 80 l/s respectively), which yields 180 l/s.

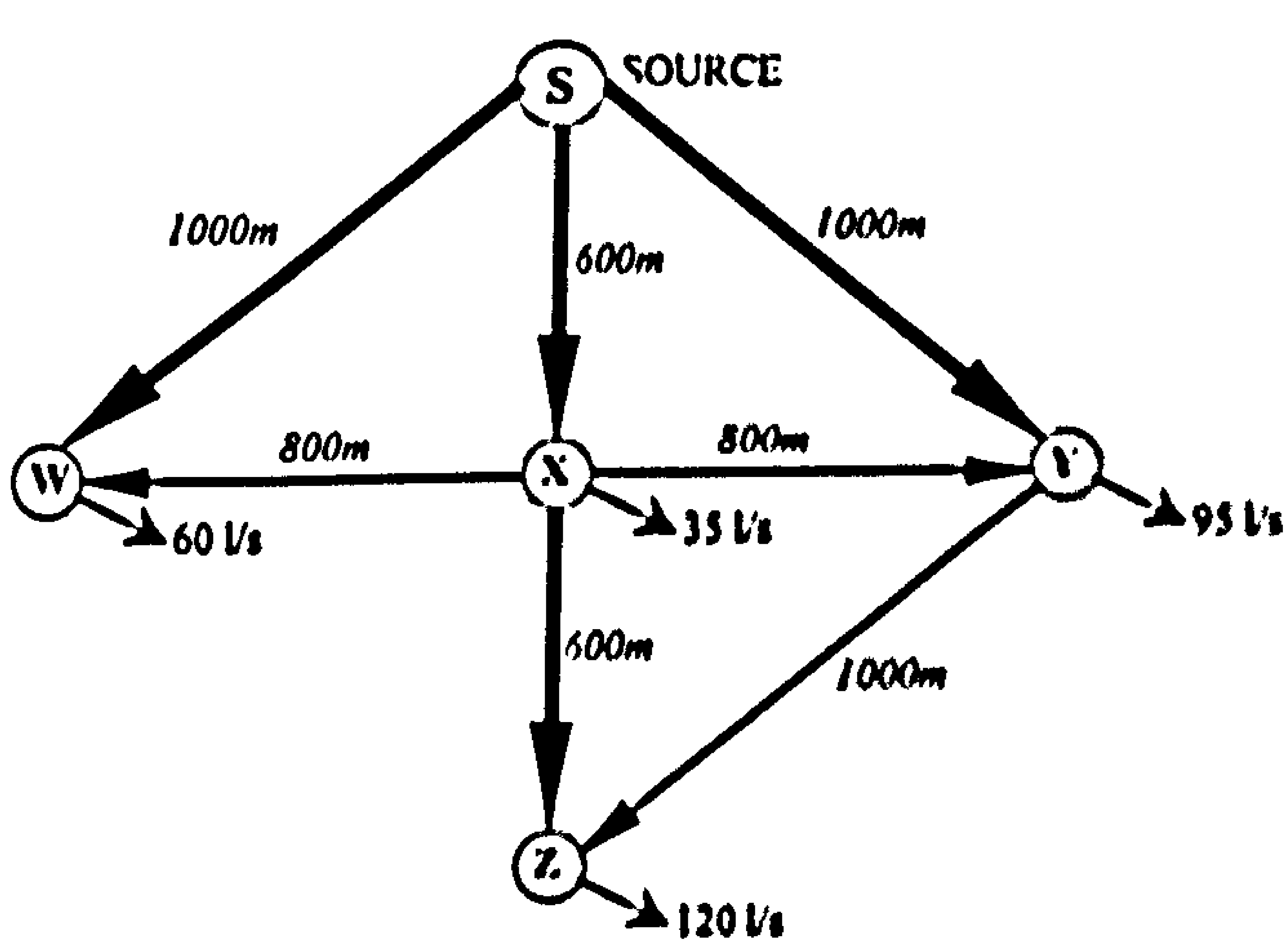

Figure A2.1 Three-loop Network

APPENDIX B

SECTION I – INFORMATION RELATED TO CHAPTER 4

This section presents typical data input and output files for the application of the Network Design Module.

Application of the Network Design Module to the network in Figure 4.6 (Section 4.4), design Option 2

The detailed optimal results shown are for a Phase I period of 14 years (summarised in Table 4.2) and the related incremental capacity to the design horizon of 20 years in Phase II (summarised in Table 4.5). Results for all the other upgrading sequences have the same format. The detailed costs are shown for all the upgrading sequences considered (summarised in Table 4.8). The details of the data input file have been presented in Section 4.3.2 and shown below in the format required by the program (UPSIZE).

Data Input File

```
CAPACITY EXPANSION - THREE-LOOP NETWORK B (MAX. ENTROPY FLOWS)
5, 7, 3, 2, 1
0, 2, 0, 0, 0
7, 15, 20, 0.5, 3, 8
4, 5, 1, 0, 50, 1
0.0021, 0.025, 1.333, -0.3
3, 4, 12, 12
1,2, 1,3, 1,4, 2,3, 2,5, 3,4, 3,5
1000, 1000, 1000, 1000, 1000, 1000, 1000
0, 2, 0, 0, 0, 0, 7,  0, 0, 3, 0, 0, 0, 0
55 40
5, 4
-1, 2, 0, -4, 0, 0, 0,  0, -2, 3, 0, 0, -6, 0,  0, 0, 0, 4, -5, 0, 7
4, 4, 4, 4, 4, 4, 4
130, 130, 130, 130, 130, 130, 130
100000, 8, 130, 1.6, 1.0, 1, 0
```

Data Output File

```
CAPACITY EXPANSION - THREE LOOP NETWORK B (MAXIMUM ENTROPY FLOWS):
PARALLELING AND REPLACEMENT WITHOUT A PRICING POLICY

PHASE I SUMMARY RESULTS

 The Design demands (l/s) for nodes 2, 3, 4 and 5 are:
   15.39005        13.85341        41.56022        41.56022
 Total number of variables in Phase I =  23
```

LINK No.	DIAM.(m)	LENGTH(m)	CHW	DIAM.(m)	LENGTH(m)	CHW
1	0.200	1000.00	120.46	-	-	-
2	0.150	1000.00	115.81	-	-	-
3	0.100	809.01	109.26	0.150	190.99	115.81
4	0.150	76.97	115.81	0.200	923.03	120.46
5	0.100	595.63	109.26	0.150	404.37	115.81
6	0.150	143.12	115.81	0.200	856.88	120.46
7	0.150	1000.00	115.81	-	-	-

****

NETWORK DESIGN DETAILS

****

	SEGMENT ONE			SEGMENT TWO		
LINK No.	DIAM.(m)	LENGTH(m)	CHW	DIAM.(m)	LENGTH(m)	CHW
1	0.20	1000.00	120.5	-	-	-
2	0.15	1000.00	115.8	-	-	-
3	0.10	809.01	109.3	0.15	190.99	115.8
4	0.15	76.97	115.8	0.20	923.03	120.5
5	0.10	595.63	109.3	0.15	404.37	115.8
6	0.15	143.12	115.8	0.20	856.88	120.5
7	0.15	1000.00	115.8	-	-	-

	SEGMENT ONE			SEGMENT TWO		
	VELOCITY		HYD. GRAD.	VELOCITY		HYD GRAD.
LINK No.	MIN.(m/s)	MAX.(m/s)	(m/Km)	MIN.(m/s)	MAX.(m/s)	(m/Km)
1	2.03	2.03	23.246	-	-	-
2	1.96	1.96	32.667	-	-	-
3	1.76	1.76	48.030	0.78	0.78	5.986
4	1.96	1.96	32.667	1.10	1.10	7.482
5	1.76	1.76	48.030	0.78	0.78	5.986
6	1.57	1.57	21.609	0.88	0.88	4.949
7	1.57	1.57	21.609	-	-	-

CRITICAL PATH HEADLOSSES

PATH No.	CRITICAL NODE	CRITICAL-PATH HEAD LOSS TOTAL(m)	ALLOWABLE(m)
1	5	54.28	55.00
2	4	40.00	40.00

PHASE I EQUIVALENT DIAMETER LINKS

LINK No.	DIAMETER(m)	LENGTH(m)	CHW
1	0.200	1000.00	120.5
2	0.150	1000.00	115.8
3	0.103	1000.00	112.5
4	0.192	1000.00	118.1
5	0.108	1000.00	112.5
6	0.186	1000.00	118.1
7	0.150	1000.00	115.8

THE TOTAL NUMBER OF ITERATIONS = 40

THE OPTIMUM COST = $ 416889.53

END OF THE 14-YEAR PHASE ONE PERIOD

&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&
&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&

PHASE II SUMMARY OF RESULTS

The Nodal design demands (l/s) for nodes 2, 3, 4 and 5 are:
18.14673 17.52898 52.58693 52.58693
Total number of variables in Phase II= 48

A) EXISTING UNPARALLELED LINK (A-B)

LINK No.	DIAM.(m)	LENGTH(m)	CHW
1	0.200	93.37	114.73
2	-	-	-
3	-	-	-
4	0.192	1000.00	114.08
5	-	-	-
6	0.186	1000.00	113.54
7	0.150	468.65	110.08

B) PARALLEL LINK (B-C)

LINK No.	DIAM.(m)	LENGTH(m)	CHW	DIAM.(m)	LENGTH(m)	CHW
1	0.250	906.63	130.00	-	-	-
2	0.150	861.88	129.38	-	-	-
3	0.100	1000.00	122.82	-	-	-
4	-	-	-	-	-	-
5	0.100	1000.00	122.82	-	-	-
6	-	-	-	-	-	-
7	0.200	531.35	130.00	-	-	-

C) REPLACED LINK (C-D)

LINK No.	DIAM.(m)	LENGTH(m)	CHW	DIAM.(m)	LENGTH(m)	CHW
1	-	-	-	-	-	-
2	0.150	138.12	129.38	-	-	-
3	-	-	-	-	-	-
4	-	-	-	-	-	-
5	-	-	-	-	-	-
6	-	-	-	-	-	-
7	-	-	-	-	-	-

****
A) PARALLEL LINK SEGMENT DETAILS
****

	SEGMENT ONE			SEGMENT TWO		
LINK No.	DIAM.(m)	LENGTH(m)	CHW	DIAM.(m)	LENGTH(m)	CHW
1	0.25	906.63	130.0	-	-	-
2	0.15	861.88	129.4	-	-	-
3	0.10	1000.00	122.8	-	-	-
4	-	-	-	-	-	-
5	0.10	1000.00	122.8	-	-	-
6	-	-	-	-	-	-
7	0.20	531.35	130.0	-	-	-

	SEGMENT ONE			SEGMENT TWO		
	VELOCITY		HYD. GRAD.	VELOCITY		HYD. RAD.
LINK No.	MIN.(m/s)	MAX.(m/s)	(m/Km)	MIN.(m/s)	MAX.(m/s)	(m/Km)
1	1.24	1.24	6.247	-	-	-
2	1.77	1.77	22.140	-	-	-
3	1.72	1.72	36.969	-	-	-
4	-	-	-	-	-	-
5	1.66	1.66	34.641	-	-	-
6	-	-	-	-	-	-
7	0.80	0.80	3.576	-	-	-

EQUIVALENT PARALLEL LINK DATA

LINK No.	DIAMETER(m)	LENGTH(m)	CHW
1	0.250	906.63	130.0
2	0.150	861.88	129.4
3	0.100	1000.00	122.8
4	-	-	-
5	0.100	1000.00	122.8
6	-	-	-
7	0.200	531.35	130.0

****

B) REPLACED LINK SEGMENT DETAILS

****

	SEGMENT ONE			SEGMENT TWO		
LINK No.	DIAM.(m)	LENGTH(m)	CHW	DIAM.(m)	LENGTH(m)	CHW
1	-	-	-	-	-	-
2	0.15	138.12	129.4	-	-	-
3	-	-	-	-	-	-
4	-	-	-	-	-	-
5	-	-	-	-	-	-
6	-	-	-	-	-	-
7	-	-	-	-	-	-

	SEGMENT ONE			SEGMENT TWO		
	VELOCITY		HYD. GRAD.	VELOCITY		HYD.GRAD.
LINK No.	MIN.(m/s)	MAX.(m/s)	(m/Km)	MIN.(m/s)	MAX.(m/s)	(m/Km)
1	-	-	-	-	-	-
2	2.48	2.48	41.145	-	-	-
3	-	-	-	-	-	-
4	-	-	-	-	-	-
5	-	-	-	-	-	-
6	-	-	-	-	-	-
7	-	-	-	-	-	-

EQUIVALENT REPLACED LINK DATA

LINK No.	DIAMETER(m)	LENGTH(m)	CHW
1	-	-	-
2	0.150	138.12	129.4
3	-	-	-
4	-	-	-
5	-	-	-
6	-	-	-
7	-	-	-

CRITICAL PATH HEADLOSSES

PATH No.	CRITICAL NODE	CRITICAL-PATH HEAD LOSS TOTAL(m)	ALLOWABLE(m)
1	5	43.87	55.00
2	4	36.97	40.00

```
*****************************************************
C)EQUIVALENT LINK DATA FOR PARALLELED SECTION B-C
*****************************************************

LINK No.  DIAMETER(m)  LENGTH(m)   CHW  MAX. HEADLOSS(m/Km)
   1        0.298       906.63    122.4         2.57
   2        0.195       861.88    119.7         5.41
   3        0.132      1000.00    113.4         4.66
   4         -            -         -            -
   5        0.135      1000.00    113.8         4.37
   6         -            -         -            -
   7        0.234       531.35    120.0         3.58


**************************************************************
D) EQUIVALENT LINK DATA FOR PARALLELED & REPLACED SECTIONS
**************************************************************

LINK No.  DIAMETER(m)  LENGTH(m)   CHW
   1        0.298       906.63    122.4
   2        0.182      1000.00    124.6
   3        0.132      1000.00    113.4
   4         -            -         -
   5        0.135      1000.00    113.8
   6         -            -         -
   7        0.234       531.35    120.0


*****************************************************
E)OVERALL EQUIVALENT LINK DATA FOR THE ENTIRE LINK
*****************************************************
LINK No.  DIAMETER(m)  LENGTH(m)   CHW
   1        0.272      1000.00    118.5
   2        0.182      1000.00    124.6
   3        0.132      1000.00    113.4
   4        0.192      1000.00    114.1
   5        0.135      1000.00    113.8
   6        0.186      1000.00    113.5
   7        0.169      1000.00    115.1

THE TOTAL NUMBER OF ITERATIONS =  70

THE OPTIMUM COST =  $ 280952.75

END OF THE SECOND PHASE AFTER  20 YEARS
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%
%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%%

SUMMARY OF PIPE INSTALLATION OPTIMAL COSTS
*********************************************
  TIME     PHASE I COST    PHASE II COST
 (years)       ($)             ($)
    7      404982.28       300214.88
    8      404966.75       306560.91
    9      405544.81       308757.44
   10      406224.28       310428.88
   11      406998.06       309074.44
   12      407858.53       310671.72
   13      412466.47       295820.31
   14      416889.53       280952.75
   15      531576.31        15405.11
```

SUMMARY OF PHASE I COSTS
****

Demand Period (years)	Installed Capacity (l/s)	Variable Costs ($)	Setup Costs ($)	Sub-Total A ($)
7	86.6	762692.00	100000.	3116394.25
8	89.9	809202.94	100000.	3162833.00
9	93.2	858661.63	100000.	3214985.75
10	96.8	911256.88	100000.	3270748.25
11	100.4	967190.19	100000.	3330287.75
12	104.3	1026676.31	100000.	3393784.75
13	108.2	1089943.50	100000.	3478529.25
14	112.4	1157235.25	100000.	3566436.75
15	116.7	1228810.13	100000.	4172562.25

SUMMARY OF PHASE II COSTS
****

Demand Period (years)	Installed Capacity (l/s)	Variable Costs ($)	Setup Costs ($)	Sub-Total B ($)
13	54.3	210689.63	100000.	1299124.00
12	51.0	176651.70	100000.	1200441.63
11	47.6	146500.13	100000.	1099575.25
10	44.1	119916.76	100000.	1006002.19
9	40.4	96611.03	100000.	914353.00
8	36.6	76318.51	100000.	836443.38
7	32.6	58800.18	100000.	737166.81
6	28.5	43841.78	100000.	648366.00
5	24.2	31254.41	100000.	200822.39

OVERALL COSTS FOR DESIGN HORIZON
****

PHASE I TIME (years)	PH1 COSTS ($)	PH2 COSTS ($)	OVERALL COSTS ($)
7	3116394.25	1299124.00	4415518.00
8	3162833.00	1200441.63	4363274.50
9	3214985.75	1099575.25	4314561.00
10	3270748.25	1006002.19	4276750.50
11	3330287.75	914353.00	4244640.50
12	3393784.75	836443.38	4230228.00
13	3478529.25	737166.81	4215696.00
14	3566436.75	648366.00	4214803.00
15	4172562.25	200822.39	4373384.50

The cost of this strategy is: $ 4214803.00.
STRATEGY: Design and Build now for a demand of 14 years in Phase I, and then after 14 years, upgrade by paralleling and replacement to a capacity of the 20 years' demand in Phase II.

SUMMARY OF COSTS FOR CHEAPEST STRATEGY

PHASE I TIME (years)	PH1 COSTS ($)	PH2 COSTS ($)	OVERALL COSTS ($)
14	3566436.75	648366.00	4214803.00

THE OVERALL CPU-TIME = 0.4882813 seconds

SECTION II – INFORMATION RELATED TO CHAPTER 5

This section presents typical data input and output files for the Hydraulic Simulation Module and the Performance Assessment Module of Sections 5.3 and 5.4.

A – Head Driven Analysis of the Two-loop Network in Figure 5.13 Using the Hydraulic Simulation Module, with All Pipes Available

Data Input File

```
Two-loop network, Design 1 - HDA using CHDSM
7 6 1 1 0.0001 1 0
284 1 1 0 0 0
-1 -28 0 1 0 0
-1 0 -33 1 1 0
0 -1 -1 -75 0 1
0 0 -1 0 -92 1
0 0 0 -1 -1 -56
0.157 130 1000
0.401 130 1000
0.100 130 1000
0.237 130 1000
0.338 130 1000
0.100 130 1000
0.263 130 1000
35 35
0 30
0 31
0 29
0 26
0 22
0 15 0
```

Data Output File

```
TWO-LOOP NETWORK, DESIGN No. 1

 CRITICAL-NODE HEAD DRIVEN SIMULATION
 *************************************

 ALL COMPONENTS AVAILABLE
```

PIPE No.	FROM NODE	TO NODE	DIAMETER (m)	LENGTH (m)	CHW	FLOWRATE (l/s)	UNIT-HEADLOSS (m/Km)
1	1	2	0.157	1000.000	130.	31.28	17.48
2	1	3	0.401	1000.000	130.	252.72	8.70
3	2	4	0.100	1000.000	130.	3.28	2.41
4	3	4	0.237	1000.000	130.	72.59	11.19
5	3	5	0.338	1000.000	130.	147.13	7.35
6	4	6	0.100	1000.000	130.	0.87	0.21
7	5	6	0.263	1000.000	130.	55.13	4.05

NODE No.	DEMAND (m)	ELEVATION (m)	TOTAL HEAD(m)	RESIDUAL HEAD(m)	FLOW (l/s)	FLOW (m^3/min)
1	284.00	35.00	35.00	0.00	284.00	17.040
2	-28.00	0.00	17.52	17.52	-28.00	-1.680
3	-33.00	0.00	26.30	26.30	-33.00	-1.980
4	-75.00	0.00	15.11	15.11	-75.00	-4.500
5	-92.00	0.00	18.95	18.95	-92.00	-5.520
6	-56.00	0.00	14.91	14.91	-56.00	-3.360

ACTUAL TOTAL DEMAND-NODE OUTFLOW= 284.00 l/s
ACTUAL TOTAL DEMAND-NODE OUTFLOW= 17.040 m^3/min

ITERATION NO. 5

PIPE No.	FROM NODE	TO NODE	DIAMETER (m)	LENGTH (m)	CHW	FLOWRATE (l/s)	UNIT-HEADLOSS (m/Km)
1	1	2	0.157	1000.000	130.	29.08	15.27
2	1	3	0.401	1000.000	130.	198.92	5.59
3	2	4	0.100	1000.000	130.	1.08	0.31
4	3	4	0.237	1000.000	130.	68.32	10.00
5	3	5	0.338	1000.000	130.	97.60	3.44
6	6	4	0.100	1000.000	130.	5.60	6.50
7	5	6	0.263	1000.000	130.	5.60	0.06

NODE No.	DEMAND (m)	ELEVATION (m)	TOTAL HEAD(m)	RESIDUAL HEAD(m)	FLOW (l/s)	FLOW (m^3/min)
1	284.00	35.00	35.00	0.00	228.00	13.680
2	-28.00	0.00	19.73	19.73	-28.00	-1.680
3	-33.00	0.00	29.41	29.41	-33.00	-1.980
4	-75.00	0.00	19.42	19.42	-75.00	-4.500
5	-92.00	0.00	25.98	25.98	-92.00	-5.520
6	-56.00	0.00	25.92	25.92	0.00	0.000

ACTUAL TOTAL DEMAND-NODE OUTFLOW= 228.00 l/s
ACTUAL TOTAL DEMAND-NODE OUTFLOW= 13.680 m^3/min

ITERATION No. 14

PIPE No.	FROM NODE	TO NODE	DIAMETER (m)	LENGTH (m)	CHW	FLOWRATE (l/s)	UNIT-HEADLOSS (m/Km)
1	1	2	0.157	1000.000	130.	31.26	17.46
2	1	3	0.401	1000.000	130.	252.33	8.68
3	2	4	0.100	1000.000	130.	3.26	2.38
4	3	4	0.237	1000.000	130.	72.51	11.16
5	3	5	0.338	1000.000	130.	146.82	7.32
6	4	6	0.100	1000.000	130.	0.76	0.16
7	5	6	0.263	1000.000	130.	54.82	4.01

NODE No.	DEMAND (m)	ELEVATION (m)	TOTAL HEAD(m)	RESIDUAL HEAD(m)	FLOW (l/s)	FLOW (m^3/min)
1	284.00	35.00	35.00	0.00	283.59	17.015
2	-28.00	0.00	17.54	17.54	-28.00	-1.680
3	-33.00	0.00	26.32	26.32	-33.00	-1.980
4	-75.00	0.00	15.16	15.16	-75.00	-4.500
5	-92.00	0.00	19.01	19.01	-92.00	-5.520
6	-56.00	0.00	15.00	15.00	-55.59	-3.335

```
ACTUAL TOTAL DEMAND-NODE OUTFLOW= 283.59 l/s
ACTUAL TOTAL DEMAND-NODE OUTFLOW= 17.015 m³/min

TOTAL NUMBER OF MAIN ITERATIONS=  19

END OF PART II, EXIT PROGRAM.
****************************************************************************
&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&
```

B – Head Driven Analysis of the Two-loop Network in Figure 5.13 Using the Hydraulic Simulation Module, with Link 2-4 Unavailable

Data Input File

```
Two-loop network, Design 1 - HDA using CHDSM
7 6 1 1 0.0001 1 1
284 1 1 0 0 0
-1 -28 0 1 0 0
-1 0 -33 1 1 0
0 -1 -1 -75 0 1
0 0 -1 0 -92 1
0 0 0 -1 -1 -56
0.157 130 1000
0.401 130 1000
0.100 130 1000
0.237 130 1000
0.338 130 1000
0.100 130 1000
0.263 130 1000
35 35
0 30
0 31
0 29
0 26
0 22
0 15 0
```

Data Output File

CRITICAL-NODE HEAD DRIVEN SIMULATION
****

SINGLE COMPONENT UNAVAILABILITY

UNAVAILABILITY OF COMPONENT No. 3

PIPE No.	FROM NODE	TO NODE	DIAMETER (m)	LENGTH (m)	CHW	FLOWRATE (l/s)	UNIT-HEADLOSS (m/Km)
1	1	2	0.157	1000.000	130.	28.00	14.24
2	1	3	0.401	1000.000	130.	256.00	8.91
3	2	4	0.100	1000.000	130.	Unav.	Unav.
4	3	4	0.237	1000.000	130.	74.56	11.75
5	3	5	0.338	1000.000	130.	148.44	7.47
6	6	4	0.100	1000.000	130.	0.44	0.06
7	5	6	0.263	1000.000	130.	56.44	4.23

NODE No.	DEMAND (m)	ELEVATION (m)	TOTAL HEAD(m)	RESIDUAL HEAD(m)	FLOW (l/s)	FLOW (m^3/min)
1	284.00	35.00	35.00	0.00	284.00	17.040
2	-28.00	0.00	20.76	20.76	-28.00	-1.680
3	-33.00	0.00	26.09	26.09	-33.00	-1.980
4	-75.00	0.00	14.33	14.33	-75.00	-4.500
5	-92.00	0.00	18.62	18.62	-92.00	-5.520
6	-56.00	0.00	14.39	14.39	-56.00	-3.360

ACTUAL TOTAL DEMAND-NODE OUTFLOW= 284.00 l/s
ACTUAL TOTAL DEMAND-NODE OUTFLOW= 17.040 m^3/min

ITERATION No. 7

UNAVAILABILITY OF COMPONENT No. 3

PIPE No.	FROM NODE	TO NODE	DIAMETER (m)	LENGTH (m)	CHW	FLOWRATE (l/s)	UNIT-HEADLOSS (m/Km)
1	1	2	0.157	1000.000	130.	28.00	14.24
2	1	3	0.401	1000.000	130.	181.00	4.69
3	2	4	0.100	1000.000	130.	Unav.	Unav.
4	3	4	0.237	1000.000	130.	7.02	0.15
5	3	5	0.338	1000.000	130.	140.98	6.79
6	4	6	0.100	1000.000	130.	7.02	9.89
7	5	6	0.263	1000.000	130.	48.98	3.25

NODE No.	DEMAND (m)	ELEVATION (m)	TOTAL HEAD(m)	RESIDUAL HEAD(m)	FLOW (l/s)	FLOW (m^3/min)
1	284.00	35.00	35.00	0.00	209.00	12.540
2	-28.00	0.00	20.76	20.76	-28.00	-1.680
3	-33.00	0.00	30.31	30.31	-33.00	-1.980
4	-75.00	0.00	30.16	30.16	0.00	0.000
5	-92.00	0.00	23.52	23.52	-92.00	-5.520
6	-56.00	0.00	20.27	20.27	-56.00	-3.360

ACTUAL TOTAL DEMAND-NODE OUTFLOW= 209.00 l/s

ACTUAL TOTAL DEMAND-NODE OUTFLOW= 12.540 m³/min

ITERATION No. 17

PIPE No.	FROM NODE	TO NODE	DIAMETER (m)	LENGTH (m)	CHW	FLOWRATE (l/s)	UNIT-HEADLOSS (m/Km)
1	1	2	0.157	1000.000	130.	28.00	14.24
2	1	3	0.401	1000.000	130.	253.15	8.73
3	2	4	0.100	1000.000	130.	Unav.	Unav.
4	3	4	0.237	1000.000	130.	72.89	11.27
5	3	5	0.338	1000.000	130.	147.26	7.36
6	4	6	0.100	1000.000	130.	0.74	0.15
7	5	6	0.263	1000.000	130.	55.26	4.07

NODE No.	DEMAND (m)	ELEVATION (m)	TOTAL HEAD(m)	RESIDUAL HEAD(m)	FLOW (l/s)	FLOW (m³/min)
1	284.00	35.00	35.00	0.00	281.15	16.869
2	-28.00	0.00	20.76	20.76	-28.00	-1.680
3	-33.00	0.00	26.27	26.27	-33.00	-1.980
4	-75.00	0.00	15.00	15.00	-72.15	-4.329
5	-92.00	0.00	18.91	18.91	-92.00	-5.520
6	-56.00	0.00	14.85	14.85	-56.00	-3.360

ACTUAL TOTAL DEMAND-NODE OUTFLOW= 281.15 l/s
ACTUAL TOTAL DEMAND-NODE OUTFLOW= 16.869 m³/min

ITERATION No. 22

PIPE No.	FROM NODE	TO NODE	DIAMETER (m)	LENGTH (m)	CHW	FLOWRATE (l/s)	UNIT-HEADLOSS (m/Km)
1	1	2	0.157	1000.000	130.	28.00	14.24
2	1	3	0.401	1000.000	130.	197.15	5.49
3	2	4	0.100	1000.000	130.	Unav.	Unav.
4	3	4	0.237	1000.000	130.	66.74	9.57
5	3	5	0.338	1000.000	130.	97.41	3.42
6	6	4	0.100	1000.000	130.	5.41	6.10
7	5	6	0.263	1000.000	130.	5.41	0.05

NODE No.	DEMAND (m)	ELEVATION (m)	TOTAL HEAD(m)	RESIDUAL HEAD(m)	FLOW (l/s)	FLOW (m³/min)
1	284.00	35.00	35.00	0.00	225.15	13.509
2	-28.00	0.00	20.76	20.76	-28.00	-1.680
3	-33.00	0.00	29.51	29.51	-33.00	-1.980
4	-75.00	0.00	19.93	19.93	-72.15	-4.329
5	-92.00	0.00	26.08	26.08	-92.00	-5.520
6	-56.00	0.00	26.03	26.03	0.00	0.000

ACTUAL TOTAL DEMAND-NODE OUTFLOW= 225.15 l/s
ACTUAL TOTAL DEMAND-NODE OUTFLOW= 13.509 m³/min

ITERATION No. 31

PIPE No.	FROM NODE	TO NODE	DIAMETER (m)	LENGTH (m)	CHW	FLOWRATE (l/s)	UNIT-HEADLOSS (m/Km)
1	1	2	0.157	1000.000	130.	28.00	14.24
2	1	3	0.401	1000.000	130.	252.56	8.69
3	2	4	0.100	1000.000	130.	Unav.	Unav.
4	3	4	0.237	1000.000	130.	73.02	11.31
5	3	5	0.338	1000.000	130.	146.54	7.29
6	6	4	0.100	1000.000	130.	0.40	0.05
7	5	6	0.263	1000.000	130.	54.54	3.97

NODE No.	DEMAND (m)	ELEVATION (m)	TOTAL HEAD(m)	RESIDUAL HEAD(m)	FLOW (l/s)	FLOW (m^3/min)
1	284.00	35.00	35.00	0.00	280.56	16.834
2	-28.00	0.00	20.76	20.76	-28.00	-1.680
3	-33.00	0.00	26.31	26.31	-33.00	-1.980
4	-75.00	0.00	15.00	15.00	-73.43	-4.406
5	-92.00	0.00	19.02	19.02	-92.00	-5.520
6	-56.00	0.00	15.05	15.05	-54.13	-3.248

ACTUAL TOTAL DEMAND-NODE OUTFLOW= 280.56 l/s
ACTUAL TOTAL DEMAND-NODE OUTFLOW= 16.834 m^3/min

TOTAL NUMBER OF MAIN ITERATIONS= 35

END OF PART III, EXIT PROGRAM
****
&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&&

C – Performance Assessment of the Two-loop Network in Figure 5.13 Using the Performance Assessment Module, with Up to Two Unavailable Links

Data Input File

```
Two-loop network, Design 1 - Performance Assessment based on CHDSM
7 6 1 1 0.0001 1 2
284 1 1 0 0 0
-1 -28 0 1 0 0
-1 0 -33 1 1 0
0 -1 -1 -75 0 1
0 0 -1 0 -92 1
0 0 0 -1 -1 -56
0.157 130 1000
0.401 130 1000
0.100 130 1000
0.237 130 1000
0.338 130 1000
0.100 130 1000
0.263 130 1000
35 35
0 30
0 31
0 29
0 26
0 22
0 15 1
```

Data Output File

```
PERFORMANCE ASSESSMENT OF TWO-LOOP NETWORK
 ********************************************

 SINGLE AND DOUBLE COMPONENT UNAVAILABILITY
```

```
SYSTEM PERFORMANCE RESULTS
 **************************
Two-Loop network

PIPE  DIAMETER   AVAILABILITY
 No.    (m)
  1    0.157       0.999592
  2    0.401       0.999865
  3    0.100       0.999306
  4    0.237       0.999748
  5    0.338       0.999834
  6    0.100       0.999306
  7    0.263       0.999777

 PROBABILITY THAT ALL LINKS ARE AVAILABLE =  0.997430
```

CONFIG. No.	UNAVAIL. LINK No. and No.		MECH. RELIAB. (A2)	TOT. OUTFLOW (B) (in l/s)	(A2) * (B)
1	0	0	0.997430	283.59	282.857
2	1	0	0.000408	256.98	0.105
3	2	0	0.000135	31.32	0.004
4	3	0	0.000693	280.56	0.194
5	4	0	0.000251	215.82	0.054
6	5	0	0.000165	140.83	0.023
7	6	0	0.000693	282.97	0.196
8	7	0	0.000222	231.12	0.051
9	1	2	0.000000	0.00	0.000
10	1	3	0.000000	252.56	0.000
11	1	4	0.000000	184.50	0.000
12	1	5	0.000000	121.36	0.000
13	1	6	0.000000	257.82	0.000
14	1	7	0.000000	208.77	0.000
15	2	3	0.000000	28.00	0.000
16	2	4	0.000000	31.32	0.000
17	2	5	0.000000	31.32	0.000
18	2	6	0.000000	31.32	0.000
19	2	7	0.000000	31.32	0.000
20	3	4	0.000000	212.50	0.000
21	3	5	0.000000	140.72	0.000
22	3	6	0.000000	280.56	0.000
23	3	7	0.000000	230.60	0.000
24	4	5	0.000000	64.32	0.000
25	4	6	0.000000	212.32	0.000
26	4	7	0.000000	156.32	0.000
27	5	6	0.000000	136.00	0.000
28	5	7	0.000000	140.83	0.000
29	6	7	0.000000	228.00	0.000
	SUMMATION		1.000000		283.486

SYSTEM RELIABILITY = 0.998188

COMPONENT FAILURE TOLERANCE = 0.861541

OVERALL MAJOR ITERATIONS = 694

THE OVERALL CPU-TIME = 1.122070 seconds

SECTION III: INFORMATION RELATED TO CHAPTER 8

Table B-8.1 Link Data and Results for Option 1 of the Hypothetical Network

PIPE No.	FROM NODE	TO NODE	DIAMETER (m)	LENGTH (m)	CHW	FLOW RATE (l/s)	HYDRAULIC GRADIENT (m/km)
1	1	2	0.2	1000	115	49.52	15.87
2	1	3	0.199	1000	119	67.47	27.16
3	2	3	0.186	1000	114	33.68	11.29
4	3	4	0.185	1000	115	43.35	17.94
5	3	5	0.175	1000	123	43.35	20.99

Table B-8.2 Nodal Data and Results for Option 1 of the Hypothetical Network

NODE No.	BASE DEMAND (l/s)	DESIGN DEMAND (l/s)	ELEVATION (m)	TOTAL HEAD (m)	RESIDUAL HEAD (m)	FLOW (l/s)	FLOW (m^3/min)
1	31.00	116.99	70.0	70.00	0.00	116.99	7.019
2	3.00	-15.84	0.0	54.13	54.13	-15.84	-0.950
3	4.00	-14.45	0.0	42.84	42.84	-14.45	-0.867
4	12.00	-43.35	0.0	24.90	24.90	-43.35	-2.601
16	12.00	-43.35	0.0	21.85	21.85	-43.35	-2.601

Table B-8.3 Link Data and Results for Option 2 of the Hypothetical Network

PIPE No.	FROM NODE	TO NODE	DIAMETER (m)	LENGTH (m)	CHW	FLOWRATE (l/s)	HYDRAULIC GRADIENT (m/km)
1	1	2	0.213	1000	119	64.29	17.83
2	1	3	0.156	1000	115	34.35	26.87
3	1	4	0.118	1000	111	18.35	35.07
4	2	3	0.195	1000	114	34.05	9.04
5	2	5	0.106	1000	116	14.39	34.78
6	3	4	0.176	1000	116	25.00	8.20
7	3	5	0.150	1000	110	28.96	25.40

Table B-8.4 Nodal Data and Results for Option 2 of the Hypothetical Network

NODE No.	DESIGN DEMAND (l/s)	ELEVATION (m)	TOTAL HEAD (m)	RESIDUAL HEAD (m)	FLOW (l/s)	FLOW (m^3/min)
1	116.99	70	70.00	0.00	116.99	7.019
2	-15.84	0	52.17	52.17	-15.84	-0.95
3	-14.45	0	43.13	43.13	-14.45	-0.867
4	-43.35	0	34.93	34.93	-43.35	-2.601
5	-43.35	0	17.39	17.39	-43.35	-2.601

Table B-8.5 Link Data and Results for Option 3 of the Hypothetical Network

PIPE No.	FROM NODE	TO NODE	DIAMETER (m)	LENGTH (m)	CHW	FLOW RATE (l/s)	HYDRAULIC GRADIENT (m/km)
1	1	2	0.200	1000	115	59.62	22.39
2	1	3	0.149	1000	114	31.94	29.90
3	1	4	0.100	1000	112	12.93	40.08
4	2	3	0.198	1000	115	32.16	7.51
5	2	5	0.107	1000	113	12.83	28.11
6	3	4	0.171	1000	114	25.58	10.18
7	3	5	0.150	1000	110	25.68	20.60

Table B-8.6 Nodal Data and Results for Option 3 of the Hypothetical Network

NODE No.	DESIGN DEMAND (l/s)	ELEVATION (m)	TOTAL HEAD (m)	RESIDUAL HEAD (m)	FLOW (l/s)	FLOW (m^3/min)
1	104.49	70	70.00	0.00	104.49	6.269
2	-14.63	0	47.61	47.61	-14.63	-0.878
3	-12.84	0	40.10	40.10	-12.84	-0.77
4	-38.51	0	29.92	29.92	-38.51	-2.311
5	-38.51	0	19.50	19.50	-38.51	-2.311

Table B-8.7 Comparison Matrix and Priority Vector for Network Performance

	Ftol	Rel	PRIORITY VECTOR
Ftol	1	2	0.667
Rel	1/2	1	0.333

Table B-8.8 Comparison Matrix and Priority Vector for Social and Environmental Issues

	Acc	III	Ab	PRIORITY VECTOR
Acc	1	2	4	0.557
III	1/2	1	3	0.320
Ab	1/4	1/3	1	0.123

$\lambda_{max} = 3.018$; C.I. = 0.0092; C.R. = 0.016

Table B-8.9 Comparison Matrix and Priority Vector for Reliability

OPTIONS	1	2	3	PRIORITY VECTOR
1	1	1	1	0.333
2	1	1	1	0.333
3	1	1	1	0.333

$\lambda_{max} = 3.0$; C.I. = 0.0; C.R. = 0.0

Table B-8.10 Comparison Matrix and Priority Vector for Failure Tolerance

OPTIONS	1	2	3	PRIORITY VECTOR
1	1	1/4	1/4	0.111
2	4	1	1	0.444
3	4	1	1	0.444

$\lambda_{max} = 3.00$; C.I. = 0.00; C.R. = 0.00

Table B-8.11 Comparison Matrix and Priority Vector for Present Value of Project Costs

OPTIONS	1	2	3	PRIORITY VECTOR
1	1	2	1	0.411
2	1/2	1	1	0.261
3	1	1	1	0.328

$\lambda_{max} = 3.054$; C.I. = 0.027; C.R. = 0.046

Table B-8.12 Main Comparison Matrix and Priority Vector by Judge 1

	Perf	EV	S&E	PRIORITY VECTOR
Perf	1	1	2	0.400
EV	1	1	2	0.400
S&E	1/2	1/2	1	0.200

$\lambda_{max} = 3.000$; C.I. = 0.00; C.R. = 0.00

Table B-8.13 Comparison Matrix and Priority Vector for Acceptability by Judge 1

OPTIONS	1	2	3	PRIORITY VECTOR
1	1	1	2	0.400
2	1	1	2	0.400
3	1/2	1/2	1	0.200

$\lambda_{max} = 3.0$; C.I. = 0.0; C.R. = 0.0

Table B-8.14 Comparison Matrix and Priority Vector for Health Issues by Judge 1

OPTIONS	1	2	3	PRIORITY VECTOR
1	1	1	2	0.400
2	1	1	2	0.400
3	1/2	1/2	1	0.200

$\lambda_{max} = 3.0$; C.I. = 0.0; C.R. = 0.0

Table B-8.15 Comparison Matrix and Priority Vector for Abstraction by Judge 1

OPTIONS	1	2	3	PRIORITY VECTOR
1	1	1	1/4	0.167
2	1	1	1/4	0.167
3	4	4	1	0.667

$\lambda_{max} = 3.0$; C.I. = 0.0; C.R. = 0.0

Table B-8.16 Main Comparison Matrix and Priority Vector by Judge 2

	Perf	EV	S&E	PRIORITY VECTOR
Perf	1	1	1	0.333
EV	1	1	1	0.333
S&E	1	1	1	0.333

$\lambda_{max} = 3.000$; C.I. = 0.00; C.R. = 0.00

Table B-8.17 Comparison Matrix and Priority Vector for Acceptability by Judge 2

OPTIONS	1	2	3	PRIORITY VECTOR
1	1	2	4	0.557
2	1/2	1	3	0.320
3	1/4	1/3	1	0.123

$\lambda_{max} = 3.018$; C.I. = 0.0092; C.R. = 0.016

Table B-8.18 Comparison Matrix and Priority Vector for Health Issues by Judge 2

OPTIONS	1	2	3	PRIORITY VECTOR
1	1	2	4	0.557
2	1/2	1	3	0.320
3	1/4	1/3	1	0.123

$\lambda_{max} = 3.018$; C.I. = 0.0092; C.R. = 0.016

Table B-8.19 Comparison Matrix and Priority Vector for Abstraction by Judge 2

OPTIONS	1	2	3	PRIORITY VECTOR
1	1	1	1/3	0.200
2	1	1	1/3	0.200
3	3	3	1	0.600

$\lambda_{max} = 3.0$; C.I. = 0.0; C.R. = 0.0

Table B-8.20 Main Comparison Matrix and Priority Vector by Judge 3

	Perf	EV	S&E	PRIORITY VECTOR
Perf	1	2	2	0.500
EV	1/2	1	1	0.250
S&E	1/2	1	1	0.250

$\lambda_{max} = 3.000$; C.I. = 0.00; C.R. = 0.00

Table B-8.21 Comparison Matrix and Priority Vector for Acceptability by Judge 3

OPTIONS	1	2	3	PRIORITY VECTOR
1	1	1	3	0.443
2	1	1	2	0.387
3	1/3	1/2	1	0.170

$\lambda_{max} = 3.018$; C.I. = 0.0092; C.R. = 0.016

Table B-8.22 Comparison Matrix and Priority Vector for Health Issues by Judge 3

OPTIONS	1	2	3	PRIORITY VECTOR
1	1	2	4	0.557
2	1/2	1	3	0.320
3	1/4	1/3	1	0.123

$\lambda_{max} = 3.018$; C.I. = 0.0092; C.R. = 0.016

Table B-8.23 Comparison Matrix and Priority Vector for Abstraction by Judge 3

OPTIONS	1	2	3	PRIORITY VECTOR
1	1	1	1/2	0.250
2	1	1	1/2	0.250
3	2	2	1	0.500

$\lambda_{max} = 3.0$; C.I. = 0.0; C.R. = 0.0

Table B-8.24 Link Data and Results for Option 1 of the Wobulenzi Network

PIPE No.	FROM NODE	TO NODE	DIAMETER (m)	LENGTH (m)	CHW	FLOWRATE (l/s)	HYDRAULIC GRADIENT (m/km)	FAILURE COST FACTORS (m/km)
1	1	2	0.362	117.0	124	184.29	8.68	1.5
2	2	3	0.250	235.5	118	142.74	35.92	1.5
3	2	4	0.158	435.0	111	41.55	38.42	1.5
4	3	5	0.150	486.5	114	30.11	25.78	1.5
5	3	6	0.260	190.2	121	105.66	16.28	1.5
6	4	5	0.150	613.0	110	14.33	7.00	3.0
7	5	7	0.196	241.0	118	37.48	9.97	3.0
8	6	8	0.204	460.1	117	70.20	26.44	3.0
9	7	8	0.150	30.0	110	17.88	10.54	3.0
10	8	9	0.200	252.9	115	69.90	30.06	3.0
11	9	10	0.104	700.0	107	7.41	12.90	3.0
12	9	11	0.234	20.0	119	54.09	8.15	5.0
13	11	12	0.135	216.0	112	17.92	17.24	3.0
14	11	13	0.150	106.0	110	29.15	26.06	3.0
15	12	14	0.100	173.5	104	8.37	20.84	3.0
16	13	14	0.126	273.0	109	14.38	16.77	3.0
17	13	15	0.081	136.8	110	7.89	47.05	3.0
18	14	16	0.119	337.0	108	10.96	13.60	1.0

Table B-8.25 Nodal Data and Results for Option 1 of the Wobulenzi Network

NODE No.	BASE DEMAND (l/s)	DESIGN DEMAND (l/s)	ELEVATION (m)	TOTAL HEAD (m)	RESIDUAL HEAD (m)	FLOW (l/s)	FLOW (m^3/min)
1	40.89	184.29	1064.5	1064.50	0.00	184.29	11.057
2	0.00	0.00	1042.2	1063.48	21.28	0.00	0.000
3	-1.59	-6.97	1020.0	1055.02	35.02	-6.97	-0.418
4	-5.07	-27.22	1032.0	1046.77	14.77	-27.22	-1.633
5	-1.59	-6.97	1003.4	1042.48	39.08	-6.97	-0.418
6	-8.09	-35.45	1002.0	1051.93	49.93	-35.45	-2.127
7	-4.47	-19.59	1000.0	1040.08	40.08	-19.59	-1.176
8	-4.15	-18.19	1000.0	1039.76	39.76	-18.19	-1.091
9	-1.92	-8.41	998.0	1032.16	34.16	-8.40	-0.504
10	-1.69	-7.41	990.0	1023.13	33.13	-7.41	-0.445
11	-1.60	-7.01	998.0	1032.00	34.00	-7.01	-0.421
12	-2.18	-9.55	997.0	1028.27	31.27	-9.55	-0.573
13	-1.57	-6.88	997.0	1029.24	32.24	-6.88	-0.413
14	-2.69	-11.79	995.0	1024.66	29.66	-11.79	-0.707
15	-1.80	-7.89	993.0	1022.80	29.80	-7.89	-0.473
16	-2.50	-10.96	990.0	1020.07	30.07	-10.96	-0.658

Table B-8.26 Link Data and Results for Option 2 of the Wobulenzl Network

PIPE No.	FROM NODE	TO NODE	DIAMETER (m)	LENGTH (m)	CHW	FLOWRATE (l/s)	HYDRAULIC GRADIENT (m/km)	FAILURE COST FACTORS (m/km)
1	1	2	0.362	117.0	124	184.29	8.68	1.5
2	2	3	0.250	235.5	118	138.92	34.16	1.5
3	2	4	0.176	435.0	120	45.37	23.25	1.5
4	3	5	0.298	190.2	122	110.84	8.98	1.5
5	3	6	0.136	486.5	112	21.11	22.35	1.5
6	4	6	0.142	613.0	109	18.15	14.36	3.0
7	5	7	0.200	460.1	115	75.39	34.58	3.0
8	6	8	0.150	241.0	110	32.29	31.49	3.0
9	7	8	0.100	30.0	116	11.06	28.19	3.0
10	7	9	0.225	252.9	117	46.14	7.61	3.0
11	8	10	0.150	250.0	112	23.76	17.40	3.0
12	9	10	0.099	216.0	107	7.07	15.15	3.0
13	9	11	0.150	20.0	110	30.66	28.62	5.0
14	10	12	0.080	484.0	113	7.41	42.19	3.0
15	10	13	0.150	20.0	110	23.42	17.37	5.0
16	11	13	0.092	216.0	102	5.37	14.10	3.0
17	11	14	0.130	106.0	110	18.29	22.14	3.0
18	13	15	0.139	173.5	112	19.23	16.85	3.0
19	14	15	0.080	273.0	100	3.52	13.27	3.0
20	14	16	0.080	136.8	113	7.89	47.39	3.0
21	15	17	0.104	337.0	106	10.96	27.04	1.0

Table B-8.27 Nodal Data and Results for Option 2 of the Wobulenzl Network

NODE No.	DESIGN DEMAND (l/s)	ELEVATION (m)	TOTAL HEAD (m)	RESIDUAL HEAD (m)	FLOW (l/s)	FLOW (m^3/min)
1	184.29	1064.5	1064.50	0.00	184.29	11.057
2	0.00	1042.2	1063.48	21.28	0.00	0.000
3	-6.97	1020.0	1055.44	35.44	-6.97	-0.418
4	-27.22	1032.0	1053.37	21.37	-27.22	-1.633
5	-35.45	1002.0	1053.73	51.73	-35.45	-2.127
6	-6.97	1003.4	1044.57	41.17	-6.97	-0.418
7	-18.19	1000.0	1037.82	37.82	-18.19	-1.091
8	-19.59	1000.0	1036.98	36.98	-19.59	-1.175
9	-8.41	998.0	1035.90	37.90	-8.41	-0.505
10	0.00	997.0	1032.63	35.63	0.00	0.000
11	-7.01	998.0	1035.32	37.32	-7.01	-0.421
12	-7.41	990.0	1012.21	22.21	-7.41	-0.445
13	-9.55	997.0	1032.28	35.28	-9.55	-0.573
14	-6.88	997.0	1032.98	35.98	-6.88	-0.413
15	-11.79	995.0	1029.35	34.35	-11.79	-0.707
16	-7.89	993.0	1026.50	33.50	-7.89	-0.473
17	-10.96	990.0	1020.24	30.24	-10.96	-0.658

Table B-8.28 Link Data and Results for Option 3 of the Wobulenzi Network

PIPE No.	FROM NODE	TO NODE	DIAMETER (m)	LENGTH (m)	CHW	FLOWRATE (l/s)	HYDRAULIC GRADIENT (m/km)	FAILURE COST FACTORS (m/km)
1	1	2	0.250	117.0	118	158.24	43.48	1.5
2	2	3	0.273	235.5	125	120.27	15.48	1.5
3	2	4	0.233	435.0	117	37.97	4.43	1.5
4	3	5	0.260	190.2	120	99.92	15.05	1.5
5	3	6	0.112	486.5	107	14.39	31.01	1.5
6	4	6	0.114	613.0	106	13.98	27.41	3.0
7	5	7	0.200	460.1	115	69.62	29.84	3.0
8	6	8	0.150	241.0	110	22.41	16.01	3.0
9	7	8	0.100	30.0	104	17.11	78.46	3.0
10	7	9	0.163	252.9	114	36.96	25.47	3.0
11	8	10	0.135	250.0	109	22.78	28.13	3.0
12	9	10	0.088	216.0	101	4.66	13.64	3.0
13	9	11	0.150	20.0	110	25.12	19.78	5.0
14	10	12	0.082	484.0	102	6.33	33.43	3.0
15	10	13	0.150	20.0	110	21.10	14.32	5.0
16	11	13	0.080	216.0	100	3.50	13.13	3.0
17	11	14	0.130	106.0	108	15.63	17.01	3.0
18	13	15	0.148	173.5	110	16.43	9.70	3.0
19	14	15	0.080	273.0	100	3.01	9.95	3.0
20	14	16	0.080	136.8	100	6.74	44.25	3.0
21	15	17	0.104	337.0	105	9.36	20.76	1.0

Table B-8.29 Nodal Data and Results for Option 3 of the Wobulenzi Network

NODE No.	DESIGN DEMAND (l/s)	ELEVATION (m)	TOTAL HEAD (m)	RESIDUAL HEAD (m)	FLOW (l/s)	FLOW (m³/min)
1	158.24	1064.5	1064.50	0.00	158.24	9.494
2	0.00	1042.2	1059.41	17.21	0.00	0.000
3	-5.96	1020.0	1055.77	35.77	-5.96	-0.358
4	-23.99	1032.0	1057.48	25.48	-23.99	-1.439
5	-30.30	1002.0	1052.91	50.91	-30.30	-1.818
6	-5.96	1003.4	1040.68	37.28	-5.96	-0.358
7	-15.55	1000.0	1039.18	39.18	-15.55	-0.933
8	-16.74	1000.0	1036.82	36.82	-16.74	-1.004
9	-7.19	998.0	1032.74	34.74	-7.19	-0.431
10	0.00	997.0	1029.79	32.79	0.00	0.000
11	-5.99	998.0	1032.34	34.34	-5.99	-0.359
12	-6.33	990.0	1013.61	23.61	-6.33	-0.380
13	-8.17	997.0	1029.50	32.50	-8.17	-0.490
14	-5.88	997.0	1030.54	33.54	-5.88	-0.353
15	-10.08	995.0	1027.82	32.82	-10.08	-0.605
16	-6.74	993.0	1024.48	31.48	-6.74	-0.404
17	-9.36	990.0	1020.82	30.82	-9.36	-0.562

Table B-8.30 Comparison Matrix and Priority Vector for Reliability

OPTIONS	1	2	3	PRIORITY VECTOR
1	1	1	1	0.333
2	1	1	1	0.333
3	1	1	1	0.333

$\lambda_{max} = 3.0$; C.I. = 0.0; C.R. = 0.0

Table B-8.31 Comparison Matrix and Priority Vector for Failure Tolerance

OPTIONS	1	2	3	PRIORITY VECTOR
1	1	1/2	1	0.261
2	2	1	1	0.411
3	1	1	1	0.328

$\lambda_{max} = 3.054$; C.I. = 0.027; C.R. = 0.046

Table B-8.32 Comparison Matrix and Priority Vector for Present Value of Project Costs

OPTIONS	1	2	3	PRIORITY VECTOR
1	1	1	1	0.333
2	1	1	1	0.333
3	1	1	1	0.333

$\lambda_{max} = 3.000$; C.I. = 0.00; C.R. = 0.00

Table B-8.33 Comparison Matrix and Priority Vector for Present Value of Project Costs - Sensitivity Analysis

OPTIONS	1	2	3	PRIORITY VECTOR
1	1	1	1/2	0.250
2	1	1	1/2	0.250
3	2	2	1	0.500

$\lambda_{max} = 3.000$; C.I. = 0.00; C.R. = 0.00

APPENDIX C

PUBLICATIONS

During the period of this study, a number of papers have been prepared to demonstrate the methodologies and findings of this research. This appendix presents five refereed papers; the first four have been published by the time of submission of this thesis while the fifth has been accepted for publication.

APPENDIX C1

AN EFFICIENT ALGORITHM FOR THE DETERMINATION OF HEAD-DEPENDENT CONSUMPTION FOR WATER DISTRIBUTION SYSTEMS

Proceedings of the 2nd International Conference on Decision Making in Urban and Civil Engineering, Lyon, France, 20-22 Nov. 2000, Vol. 1, 303-315.

DM in UCE – 2000
2nd International conference on Decision Making in Urban and Civil Engineering. Lyon, France, Nov. 20-22, 2000.
2ème conf. internationale sur l'Aide à la Décision dans le domaine Génie Civil et Urbain. Lyon (F), 20-22 nov. 2000.

AN EFFICIENT ALGORITHM FOR THE DETERMINATION OF HEAD - DEPENDENT CONSUMPTION FOR WATER DISTRIBUTION SYSTEMS

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Abstract:

This paper presents an efficient algorithm for simulating the performance of water distribution systems with less than fully satisfactory heads. It is capable of simulating networks under abnormal loading conditions in which the demands exceed the system capacity, and/or systems performing unsatisfactorily because of a partial failure due to some components being unavailable. The superiority of the proposed formulation over the traditional demand-driven analysis approach is demonstrated. Such a realistic approach to network modelling will enhance decision making for the routine operation and management of water distribution systems. The algorithm is reliable, quick, easy-to-implement, and, using examples, it is shown to compare very well with others.

Résumé :

Ce texte présente un algorithme efficient pour simuler la performance d'un système de distribution d'eau dans le cas de pressions inférieures aux valeurs satisfaisantes. Celui-ci permet de simuler le fonctionnement d'un réseau sous des conditions anormales de pression où la demande excède les capacités du système, et/ou en cas de fonctionnement perturbé par l'indisponibilité de certains composants. Nous montrons la supériorité de la formulation proposée sur l'approche traditionnelle fondée sur la demande. Une telle approche réaliste de la modélisation des réseaux devrait constituer une aide à la décision pour l'exploitation et la gestion des systèmes de distribution d'eau. L'algorithme est fiable, rapide, facile à mettre en œuvre, et à partir d'exemples nous montrons qu'il s'avère comparable à d'autres.

Keywords: Water Distribution Systems, Demand Driven Analysis, Head Driven Analysis

1. Introduction

Various mathematical models that are currently available for the analysis of Water Distribution Systems (WDS) are based on Demand Driven Analysis (DDA). The underlying assumption for these models is that nodal demands (quantity of water required at nodes) are fixed and can be fully satisfied regardless of the pressure in the system. This method is satisfactory if the nodal heads are sufficient. There are numerous occasions, however, when all the nodal demands cannot be satisfied due to pressures in the system dropping below a required minimum level. High demands for fire fighting, pump-failure, pipe bursts, and demand in excess of design capacity are some of the possible causes of insufficient pressure in the system, that tend to put the network under abnormal loading conditions. A WDS in this state has deficient hydraulic performance. It does not perform satisfactorily because there is a reduced quality and level of service to the consumers, with the available outflows at some of the nodes being less than the nodal demands. It is imperative therefore, that realistic network simulation be based on Head Driven Analysis (HDA), to take into account the above mentioned aspects. Some researchers in the past have considered this issue (Chandapillai, 1991; Gupta and Bhave, 1996; Wagner, Shamir and al., 1988). However, computer programs for analysing systems with insufficient pressure in a routine manner do not exist.

The aim of this paper is to present a method for analysing WDS with deficient hydraulic performance using an efficient algorithm. Although the method does not use any explicit head-discharge relationship, it systematically identifies no-flow and partial-flow nodes until hydraulic feasibility is achieved. It recognises the spatial performance characteristics of the network, has high computational efficiency and can be used for single and multiple-source networks. It can easily be tailored to carry out extended period simulation with suitable modification. The algorithm automatically carries out DDA or HDA as appropriate, and has been encoded as part of a comprehensive computer analysis program. It is applied to some examples and the results obtained compare very well with other more laborious and time-consuming methods.

2. Demand Driven Analysis

The constitutive equations for flow in water supply networks have to simultaneously satisfy two conditions namely, nodal flow continuity and conservation of energy applied to each loop or path. There are various methods of computation that have been used for solving conventional network analysis problems (namely Linear Theory, Hardy-Cross, Newton Raphson, etc.). The Newton-Raphson method has good convergence characteristics (Lemieux, 1972), and was therefore used in the present formulation.

The continuity equation for each node j, $j=1, \ldots\ldots\ldots, NJ$, may be written as

$$\sum_{i:H_i<H_j} Q_{ij} - \sum_{i:H_i>H_j} Q_{ij} = Q_j^{req} \tag{1}$$

where Q_{ij} is the flow in the link ij, and NJ is the number of nodes in the network. Q_j^{req} is the required nodal outflow (demand). H_i and H_j are piezometric heads at nodes i and j respectively.

The Hazen-Williams equation for headloss in a pipe-link has the form

$$Q_{ij} = K_{ij}^{1.852}\left|H_i - H_j\right|^{-0.46}\left(H_i - H_j\right) \tag{2}$$

in which K_{ij} is a resistance coefficient for link ij and it has the form

$$K_{ij} = \frac{\alpha \cdot L_{ij}}{CHW_{ij}^{1.852} \cdot D_{ij}^{4.87}} \quad (3)$$

where $\alpha = 10.675$ in SI units, L_{ij} = link length, CHW_{ij} = Hazen-Williams coefficient for link ij, and D_{ij} = diameter of link ij.

Eq. (1) can incorporate Eq. (2) to become

$$F_j = \sum_{i:H_i>H_j} \left(\frac{H_i - H_j}{K_{ij}} \right)^{(0.54)} - \sum_{i:H_i<H_j} \left(\frac{H_j - H_i}{K_{ij}} \right)^{(0.54)} - Q_j^{req} = 0 \quad (4)$$

in which F_j represents the continuity equation for node j. Other network components including pumps, non-return valves, flow-control valves and pressure-reducing valves can be included in Eqs. (4) in a similar way (Tabesh, 1998).

Using the Newton-Raphson method and choosing the nodal piezometric heads as the basic unknown parameters, Eqs. (4) would be solved by the following iterative scheme:

$$J_H^m \underline{\Delta H}^m = -\underline{F}\left(\underline{H}^m\right) \quad (5)$$

$$\underline{H}^{m+1} = \underline{H}^m + \underline{\Delta H}^m \quad (6)$$

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

The elements of the Jacobian matrix, J_H, are given by

$$\frac{\partial F_j}{\partial H_i} = 0.54 \left(\frac{|H_i - H_j|^{-0.46}}{K_{ij}^{0.54}} \right) = \frac{\partial F_i}{\partial H_j}; \quad \forall j, \forall i : i \neq j \quad (7)$$

$$\frac{\partial F_j}{\partial H_j} = -0.54 \sum_{\substack{i \in N_j \\ i \neq j}} \left(\frac{|H_i - H_j|^{-0.46}}{K_{ij}^{0.54}} \right) \quad (8)$$

where N_j represents nodes connected to node j.

To use the Newton-Raphson method, nodal demands are assigned values that are assumed to be fixed. The problem then consists of solving the system of equations to determine the pipe flow rates and nodal pressures that are hydraulically consistent with the specified demands. This is the traditional method of analysing water distribution systems and is referred to as demand-driven analysis (DDA).

3. Head Driven Analysis

DDA is sufficient for the analysis of a WDS if the available nodal heads are not less than the desired values. In reality, however, the problem is far more complex. Actual outflow at nodes depends on several factors. Physical characteristics of the components of the WDS like pipe bursts, pump or valve failures, excessive demands e.g. for fire fighting at some nodes, can lead to reduced available pressures in the system (Tanyimboh, Burd and al., 1999). A network in this state is deficient and would not perform satisfactorily. Conventional methods (i.e. DDA) are unable to accurately analyse these deficient WDS, and so it is necessary to simulate them by pressure-dependent network analysis.

Head driven analysis (HDA) differs from DDA in that only the former recognises the primacy of pressures over demands and considers nodal heads and flows simultaneously in the solution procedure. The objective of HDA is to establish the actual supply quantity from each node based on inherent characteristics and available pressures in the WDS.

3.1. Review of Methods for Pressure – Driven Network Analysis

Few studies have addressed the implications of the dependency of nodal outflows on heads. Gupta and Bhave (1996), made a comparison of various formulae for describing the pressure dependency of nodal consumption (outflow) and they concluded that the following parabolic relationship (Wagner, Shamir and al., 1988; Chandapillai, 1991) was sufficiently accurate.

$$H_j^{des} = H_j^{min} + R_j\left(Q_j^{req}\right)^n \qquad (9)$$

where R_j is a resistance constant and n is an exponent. Thus

$$Q_j^{avl} = Q_j^{req}\,; \qquad if \quad H_j \geq H_j^{des} \qquad (10a)$$

$$Q_j^{avl} = Q_j^{req}\left(\frac{H_j - H_j^{min}}{H_j^{des} - H_j^{min}}\right)^{\left(\frac{1}{n}\right)}; \qquad if \quad H_j^{min} \prec H_j \prec H_j^{des} \qquad (10b)$$

$$Q_j^{avl} = 0\,; \qquad if \quad H_j \leq H_j^{min} \qquad (10c)$$

where Q_j^{avl} and Q_j^{req} are the available outflow and demand at node j, respectively. H_j^{des} is the desired head to satisfy the demand. H_j is the available head and H_j^{min} is the minimum required head at node j. The value of the exponent n often varies between 1.5 and 2 (Gupta and Bhave, 1996; Chandapillai, 1991), and is in general both node and network specific.

To approximate WDS performance, Wagner, Shamir and al., 1988, used a two-phase formulation, in which a conventional demand-driven simulation was done to obtain head values at each node. They then used the head – outflow relationship of Eq. (10) to calculate nodal outflows for those nodes with head values less than desired ones. This method however, did not account for the mutual interdependency of the nodal outflows and heads. Bhave (1991) proposed a technique, which involved assigning categories to all demand nodes, and changing them at each iteration according to a predefined scheme of conversion of node categories, until node category compatibility was achieved. It involves violation of constraints in each iteration, though the discrepancy reduces in successive iterations of the procedure, which ends when all constraints are satisfied. Thus, all solutions found by successive iterations are infeasible except the final result. This weakness is avoided in the method proposed herein.

The Source Head Method (SHM) by Tanyimboh and Templeman (1998) does not recognise the spatial performance characteristics of the distribution system and it can not be used for multiple-source networks. The Improved Source Head Method (ISHM) by Tanyimboh, Tabesh and al. (1997) is an improvement of the SHM as it recognises the spatial performance characteristics of the network. However, it is not applicable to multiple-source networks either. The Head Driven Simulation Method (HDSM) by Tabesh (1998), presents a realistic approach to pressure-driven analysis. It is based on the Newton-Raphson technique and explicitly incorporates the head-outflow relationship of Eq. (10) in the continuity equations. The method has a provision for the elimination of oscillations to ensure faster convergence, based on a Step-length Adjustment Parameter (SAP). Unfortunately, the values of this step-length adjustment parameter and the indices R and n of Eq. (9) are difficult to ascertain. Not only are they node and network specific, but also their determination requires a considerable amount of effort in field data collection, analysis and network calibration (Gupta and Bhave, 1997). To address the weaknesses of the above-mentioned approaches, an algorithm for determining head-dependent outflows in WDS is presented in the next section.

3.2. Critical Node Head Driven Simulation

3.2.1. Critical Node Principle: A distribution system in its stressed state (under abnormal loading conditions, and/or, with some components unavailable), experiences a reduction in the expected nodal flow, in a decreasing progression, from the most critically affected nodes to the least critically affected ones.

3.2.2. Overview of the proposed algorithm

The algorithm was developed from a study of various deficient networks to assess the effect of variation of flow at the most critical nodes. The observed pattern was the basis of this algorithm. It involves a systematic identification of no-flow nodes and fixing their flows to zero (this generally raises the heads in the network), followed by partial-flow nodes and nodes whose flows affect flows at other nodes in the network. Partial flows are obtained by converting the head-equations of Eq. (5) to head-flow equations as shown in Eq. (11). Head-flow (H-Q) equations have both nodal flows and heads as the unknown basic variables (Bhave, 1991). Thus

$$J_{HQ}^{m}\begin{bmatrix}\underline{\Delta H}^{m}\\ \cdots\cdots\\ \underline{\Delta Q}^{m}\end{bmatrix}=-\underline{F}\left(\underline{H}^{m},\underline{Q}^{m}\right) \tag{11}$$

and as in Eq. (6), successive values of the nodal outflows for partial-flow nodes are given by

$$\underline{Q}^{m+1}=\underline{Q}^{m}-\underline{\Delta Q}^{m} \tag{12}$$

The respective changes to the elements of the Jacobian, J_H, to give the new Jacobian, J_{HQ}, are as follows:

$$\frac{\partial F_j}{\partial Q_k}=-1;\quad \forall j,\forall k:k=j \tag{13}$$

where k is a partial-flow node

$$\frac{\partial F_j}{\partial Q_k}=0;\quad \forall j,\forall k:k\neq j \tag{14}$$

A brief characterisation of some of the variables used in the algorithm follows. H_j^{min} is the absolute minimum head for flow to be possible at a node. N_{cru} is the most critical node and it has the lowest nodal residual head denoted by H_{cru}. H_{res} is a pre-specified residual head for nodal outflow at a desirable pressure. Nodes said to be in the same pressure contour are those whose residual nodal heads are approximately the same and they are stored and processed together in sets. Based on the above principle, a heuristic algorithm for calculating head-dependent outflows was developed as follows.

3.2.3. Algorithm for calculating head-dependent outflows

Part I: Identification of zero-flow demand nodes

1) Given nodal demands, assume initial heads, H_j for all nodes other than fixed head nodes.
2) Calculate the nodal heads using the Newton-Raphson method of Eq. (5) and (6).
3) Identify all nodes whose static heads are less than their respective minimum heads, H_j^{min}. Fix the demands at these no-flow nodes to zero and repeat step 2.
4) Identify the most critical node, of all non-zero demand nodes. If its head is less than H_{res} then, this node, together with any other nodes in the same pressure contour, should be taken as the critical nodes (stored as set C1). Otherwise, exit.
5) Set the demands of the critical nodes of set C1 to zero, and repeat step 2.
6) If the residual heads of the critical nodes of set C1 are less than H_j^{min}, confirm them as no-flow nodes by fixing their demand values to zero, and return to step 2. Otherwise, they are categorised as partial-flow nodes. Go to Part II.

Part II: Identification of partial-flow nodes

1) The set of partial flow nodes should be stored as P1. If one or more sets of nodes have residual heads between H_j^{min} and H_{res}, they should be categorised as partial-flow nodes with heads, H_{crit}, H^P, H^{P+1}, H^{P+2}, H^{P+3}, etc., and stored in different sets. Fix their heads as $0.5(H_{crit}+H_j^{min})$ for the most critical node (N_{crit}), $H_j^{min}+0.75(H_{crit}-H_j^{min})$ for the set of next most critical nodes, $0.5(H_{crit}+H^P)$, $0.5(H^P+H^{P+1})$, $0.5(H^{P+1}+H^{P+2})$, $0.5(H^{P+2}+H^{P+3})$, etc., up to the least critical and then perform step 2.
2) Convert the system of head-equations in Eq. (5) into a system of head-flow equations as shown in Eq. (11). Solve Eqs. (11) and update the nodal heads and flows using Eqs. (6) and (12) respectively.
3) Or else, if their heads are greater than H_{res}, fix them to H_{res} and then perform step 2. Otherwise proceed to Part III.

Part III: Identification of partial-flow nodes whose outflows affect outflows at other nodes

1) If the most critical node of all demand nodes other than the no-flow nodes and the partial flow nodes in set P1, is less than H_{res}, the demand values of this node, together with any other nodes in the same pressure contour should be set to zero, and stored in set C2. Repeat step 2 in Part I. Otherwise, exit.
2) If the critical nodal residual heads of set C2 are less than H_j^{min}, confirm them as no-flow nodes by fixing their demand values to zero. Repeat steps 1 and 2, in Part II. Otherwise, if the nodal residual heads of set C2 that are not no-flow nodes are greater than or equal to H_{res}, label these nodes as partial-flow nodes of set P2.
3) For the two sets of partial flow nodes in P1 and P2, fix the pressure of nodes in set P1 to H_{res} and that of nodes in set P2 to (H_{res}+0.05). Repeat step 2 in Part II.
4) If another set of nodes has heads less than H_{res}, then these fall into the third set of partial flow nodes, P3. The flows of all partial-flow nodes should be obtained using step 2 in Part II, by fixing the pressure of nodes in the latest set, P3 to (H_{res}+0.05), that of nodes in the first set, P1, to H_{res}, and equally distributing the pressure of any other sets between H_{res} and (H_{res}+0.05). Thus, the pressure of nodes in set P2 should be fixed to (H_{res}+0.025). Repeat step 4 and make appropriate adjustments until there are no more partial-flow nodes.
5) Exit

3.2.4 Main features of the proposed approach

Head driven analysis (HDA) is executed by a comprehensive computer program, using the algorithm for calculating head-dependent outflows described in Section 3.2.3. The method is called the Critical-node Head Driven Simulation Method (CHDSM). The Newton-Raphson technique is used for the solution of the set of simultaneous equations using a refined form of Gaussian Elimination, involving Scaled Column Pivoting (Burden and Faires, 1993). The software developed is capable of simulating the simultaneous unavailability of up to two components, giving results that can be used to calculate the reliability of WDS. The main driving force of CHDSM is a pre-specified nodal residual head, H_{res}, which can be set at a minimum value if flow at outlets is required. H_{res} can also be set at a minimum nodal residual head value according to the terrain, locality served, plumbing arrangements, and the general bylaws regarding residual heads. Under certain circumstances, the absolute minimum desired pressure is suggested to be 7m (US Army Corps of Engineers, 1984). For purposes of establishing the reliability of a WDS, H_{res} can be set at a desirable head below which flow cannot be totally satisfied, with typical values being about 14m to 15m (Insurance Service Office, 1980; Twort, Law and al., 1994). The absolute minimum residual head, H_j^{min}, below which no flow is possible at a node can be set at a bare minimum value or zero.

The introduction of the equations for determining partial-flow in step 2 in Part II of the algorithm, do not alter the basic structure and size of the Jacobian, nor do they lead to an increase in the number of basic unknowns. Moreover, they are introduced at a point when values of all other unknowns have been obtained. This leads to the application of Newton's method in the immediate neighbourhood of the solution, and convergence is attained after very few additional iterations. It can therefore be expected that the basic computational characteristics of the solution methodology will not be highly affected. In identifying critical nodes, nodes that have heads in the same pressure contour are treated together. This speeds up the process of identifying no-flow and partial-flow nodes, and improves the efficiency of the algorithm. The sets of partial-flow nodes tend to be limited to one, two, or three. Therefore, obtaining outflow at partial-flow nodes does not involve many extra iterations. It is worth noting that, unlike Gupta and Bhave (1991), the solutions found in all intermediate iterations are feasible.

4. Appraisal of Performance

4.1. Example 1

A pressure-deficient looped network with 16 designs was chosen to demonstrate some of the aspects that have been mentioned so far. This network (Figure 1), has been used by several researchers to demonstrate several aspects of the design and reliability of water distribution networks (Fujiwara and Tung, 1991; Tanyimboh, Tabesh and al., 1997; etc.). The pipe data for the sixteen different designs are presented in Table 1. All links are 1000m long with CHW=130. The source node has a fixed head of 100m, and all elevations of demand nodes are 0m.

Table 1: Pipe Data

Link	Diameter (mm)															
1-2, 1-4	250	250	250	250	250	250	250	250	250	250	250	250	250	255	255	255
2-3, 4-7	175	175	180	180	180	185	185	185	190	190	190	190	190	190	190	190
2-5, 4-5	145	145	145	145	145	145	145	145	145	145	145	150	150	150	155	155
3-6, 7-8	115	115	115	120	125	125	130	135	135	140	140	140	140	140	140	140
5-6, 5-8	100	105	105	105	105	105	105	105	105	105	110	110	115	115	115	120
6-9, 8-9	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100
Design	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16

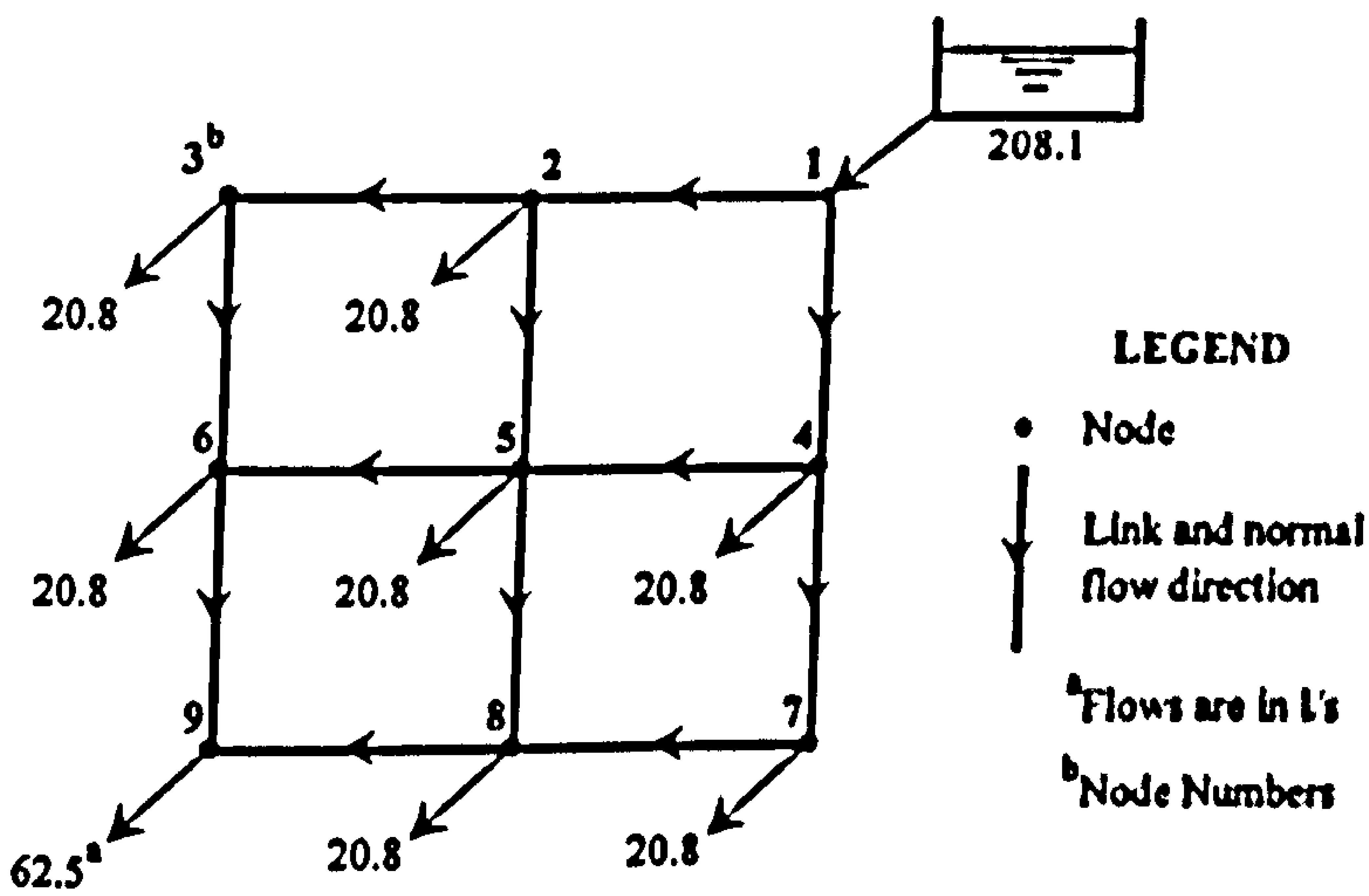

Figure 1. Looped Network

To elaborate on the difficulty encountered in selecting the step-length adjustment parameter when using HDSM, a number of trials were made on the network in Figure 1, for Designs 1 and 16. The results are shown in Figures 2a and 3a, with all links available, and, in Figures 2b and 3b, with link 1-2 unavailable. For each SAP value chosen, the corresponding total inflow and outflow obtained using HDSM were plotted in Figures 2 and 3. The procedures involved in identifying the appropriate SAP value, for each configuration or state of the distribution system, are laborious and time consuming as illustrated in Figures 2 and 3.

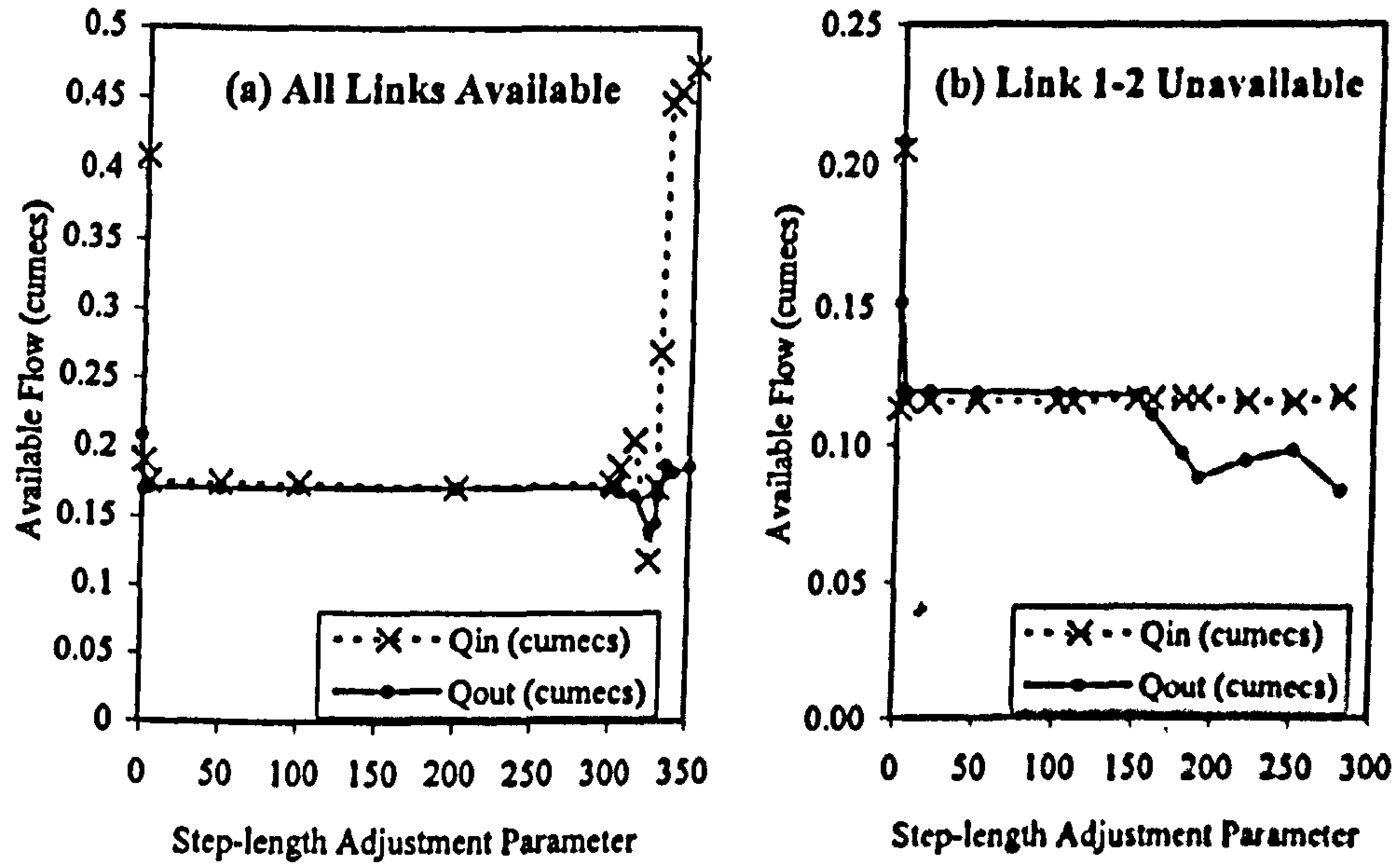

Figure 2. Total Inflow, Qin, and Outflow, Qout, vs Step-length Adjustment Parameter, for Design 1

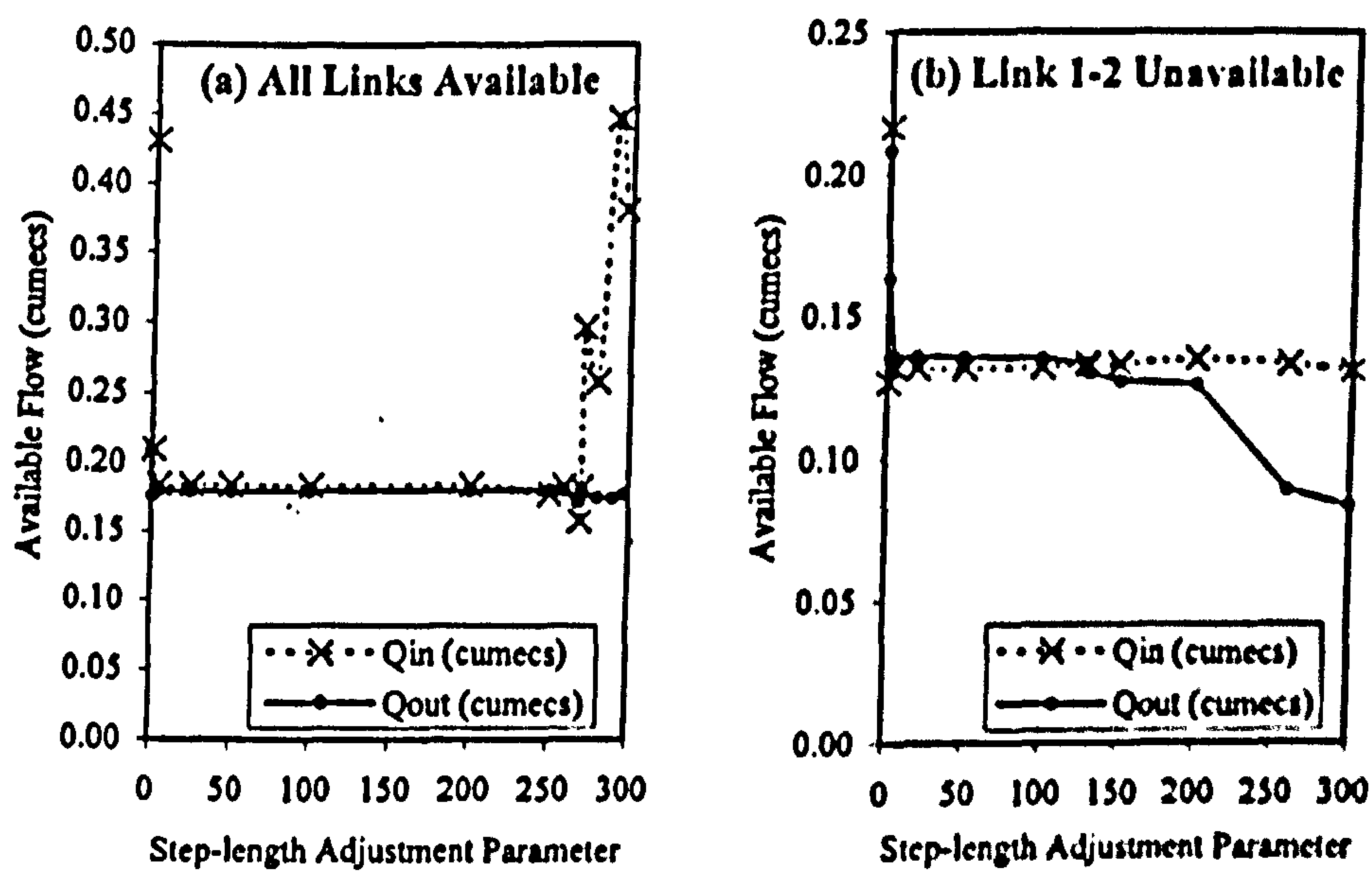

Figure 3. Total Inflow, Qin, and Outflow, Qout, vs Step-length Adjustment Parameter, for Design 16

4.2 Example 2

To demonstrate accuracy of the results obtained using the Critical-node Head Driven Simulation Method (CHDSM), the formulation was applied to the sixteen designs (Table 1) of the sample network in Figure 1. The pre-specified nodal residual head, H_{ns}, was set to a desirable head, H_j^{des}, of 15m, and the absolute minimum residual head, H_j^{min}, was taken as zero. A comparison of the proportions of the total demand satisfied by the fully connected networks using different methods namely Source Head Method (SHM), Improved Source Head Method (ISHM), Head Driven Simulation Method (HDSM) and CHDSM, is presented in Figure 4. The mean errors of the values obtained as compared to HDSM were –24% for SHM, +5% for ISHM, and, +1% for CHDSM. In comparison to the other approaches, the CHDSM results are much more realistic than SHM, which are, strictly speaking, DDA solutions.

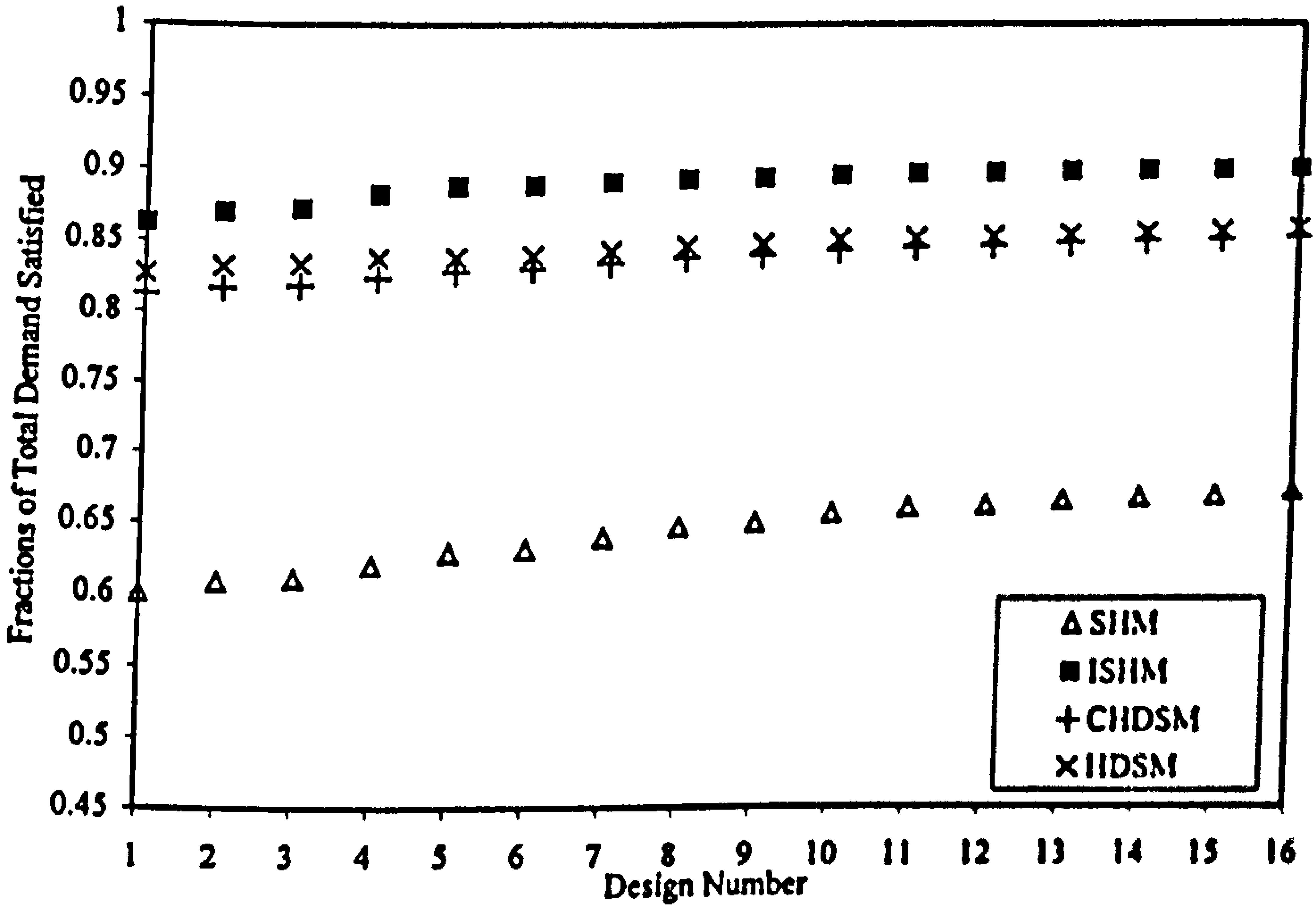

Figure 4. Fractions of Total Demand Satisfied by Fully Connected Networks

4.3. Example 3

CHDSM was also applied to the serial network shown in Figure 5 (Gupta and Bhave, 1996). The lengths and the Hazen-Williams coefficients for all pipes are 1000m and 130 respectively. The diameters of pipes 1 through 4 are 400mm, 350mm, 300mm and 300mm respectively. The nodal outlet elevations of nodes 1 through 4 are 90m, 88m, 90m, and 85m respectively. The demand nodes 1 through 4 have required flows of 2m^3/min, 2m^3/min, 3m^3/min and 4m^3/min, respectively.

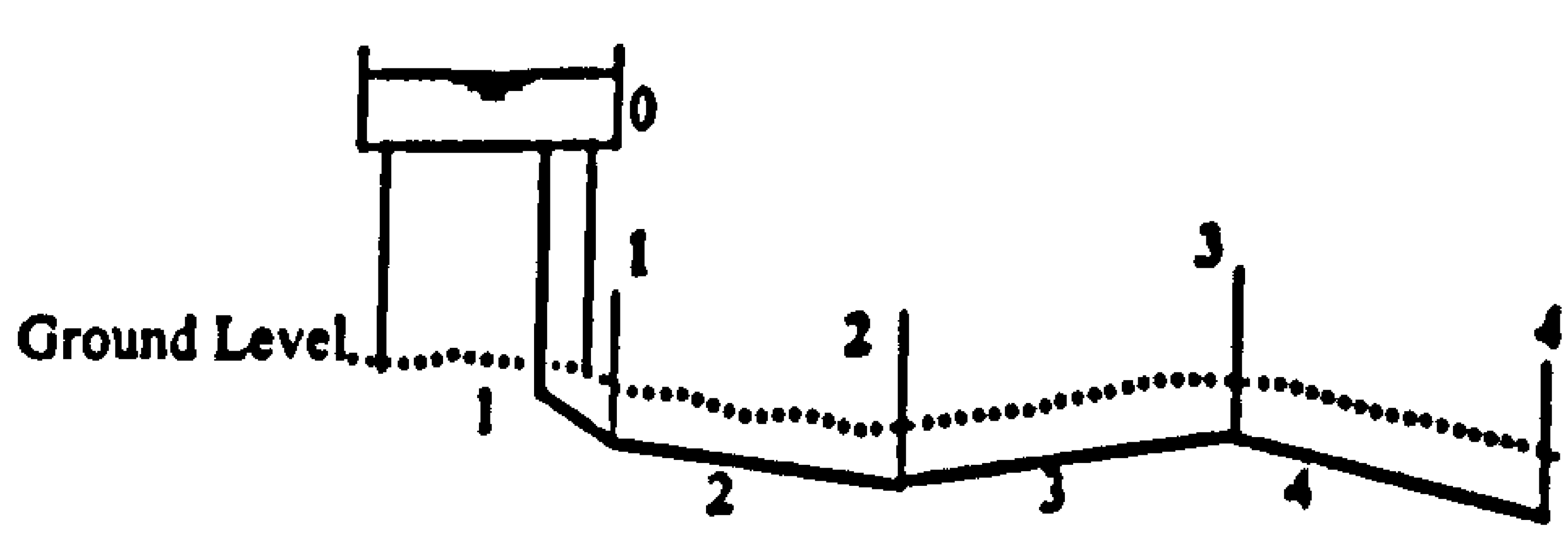

Figure 5. Serial Network

To further confirm the accuracy of the proposed formulation and to demonstrate the effects of variations in the source head on nodal outflows, the source head for the serial network was varied from 85m to 110.89m. The total supply to the network and the available nodal flows obtained by CHDSM are presented in Figure 6 along with the Gupta and Bhave (1996) results for comparison purposes. The head-discharge curves depict the actual quantity of water that would be available to the serial network and individual nodes, for a range of source heads. Using regression analysis, Gupta and Bhave (1996) also fitted the Wagner and al. (1988) head-outflow relation of Eq. (10) as shown in the figure.

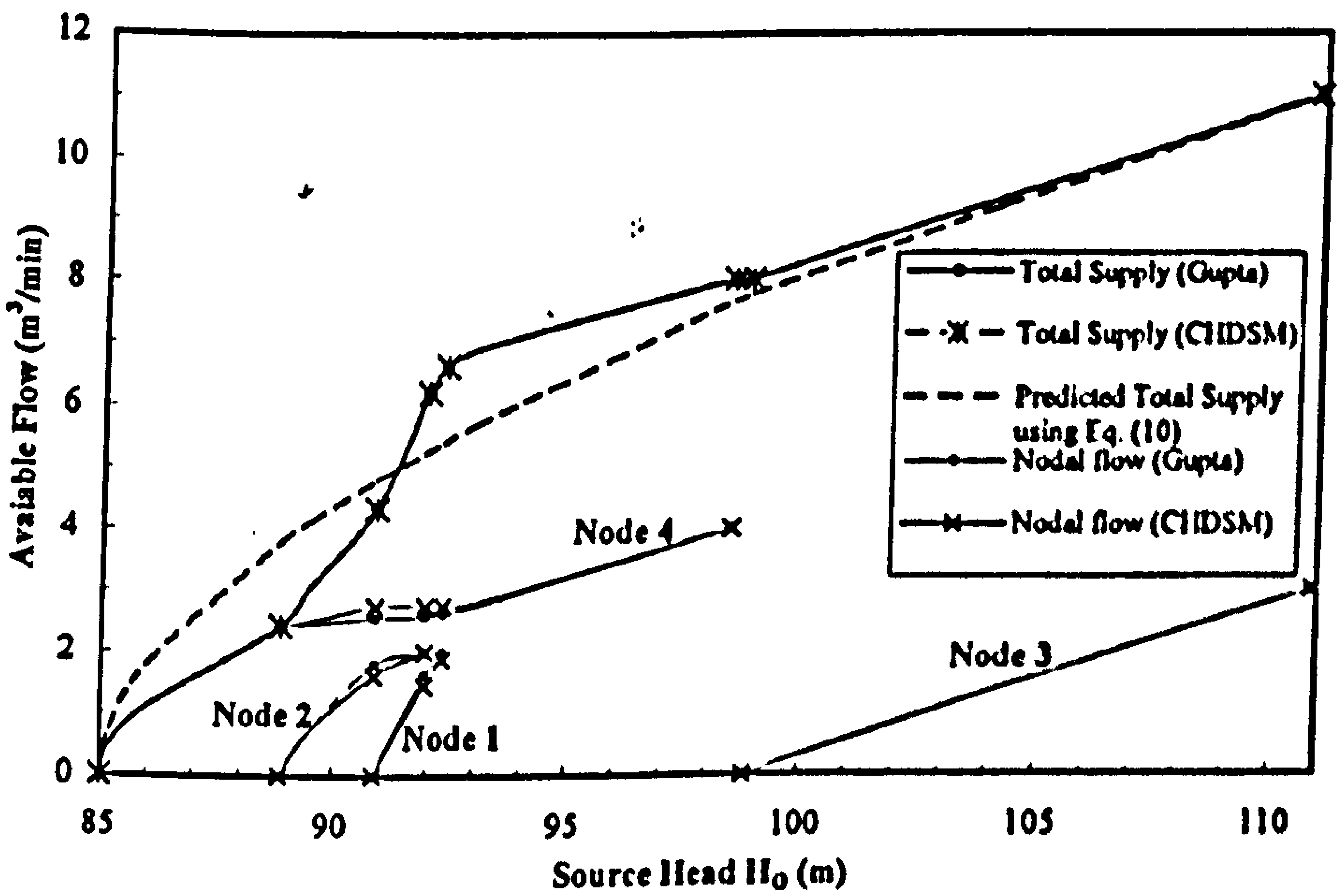

Figure 6. Variation of Available Flow for Different Source-Heads

5. Discussion

From Example 1, it is clear that different choices of the step-length adjustment parameter lead to discrepancies between total inflow and outflow. Figures 2 and 3 confirm that SAP is network specific and quite difficult to obtain. The best choice of the SAP therefore, is one that yields equal values of inflow and outflow. For Design 1, with all links available (Fig. 2a), the best SAP value is about 300. For Design 1, with link 1-2 unavailable (Fig. 2b), the best SAP is about 150. For Design 16, with all links available (Fig. 3a), the best SAP value is about 230. For Design 16, with link 1-2 unavailable (Fig. 3b), the best SAP is about 130. To obtain the best SAP value, many runs of the HDSM model have to be made. Each run is a full HDA simulation and the accumulated run time in the search for the best SAP value, is the basis of the method's computational inefficiency.

In comparison with HDSM, Figure 4 clearly shows that the CHDSM formulation gives more accurate values of flow delivered at adequate pressure, than ISHM or SHM. Whereas SHM underestimates total outflow by about 24% and ISHM overestimates the total outflow by about 5%, CHDSM underestimates the total outflow by only 1%. Therefore the present formulation has an advantage that good estimates of system performance can be obtained in a simple, straightforward way which avoids the difficulties, including calibration and SAP search, associated with HDSM.

Example 3 reveals that by comparison of the CHDSM values to the actual values, there is a very close relationship. The total supply to the network is exactly the same. The curves for the available nodal flows follow the same trend and they are almost identical. Therefore this approach gives a very good representation of the network behaviour, and the accuracy of the present formulation is confirmed. CHDSM gives reasonable results for available nodal outflows, and a good prediction of deficient-condition performance.

This algorithm has many practical applications. It is an appropriate decision-making tool for the day to day operation and management of a water distribution system. It can be used to realistically simulate low-supply situations, determine precisely the nodes with insufficient flow, and the respective magnitudes of the shortfalls in flow. It can effectively be used in the process of determining the best strategy for future upgrading of a WDS, for performance and reliability assessment, pressure-dependent leakage analysis and control, contingency planning, etc.

Other aspects worth emphasising are the computational efficiency and the simplicity of the present formulation. CHDSM results were generated for each of the sixteen designs in Example 1, using a FORTRAN 90 program on a 400MHz Ultra Sparc Sun system. For each of the sixteen designs, the CPU time required was approximately 0.15 seconds, which is about the same time taken by a single run of the HDSM (Tabesh M, 1998). It should however be borne in mind that the initial trial and error phase of the HDSM in which the SAP value is found, is by its nature very time consuming and is not included in the above comparison. Since the proposed technique is an extension of the conventional technique, it can be applied along with conventional methods of network analysis, and other components and fittings like pumps and valves can be handled. It can easily be tailored to carry out extended period simulation.

6. Conclusions

An efficient technique for determining head-dependent outflows in WDS has been presented. It simultaneously considers nodal heads and flows in the prediction of deficient-condition performance. It is applicable to networks with single and multiple sources, and it recognises the spatial performance characteristics of the network. It can perform Head Driven Analysis and is capable of simulating networks under abnormal loading conditions, and/or, with some components unavailable, giving results that compare very well with other methods. It does not require independent head-discharge relationships in its solution procedure. The computer time taken for convergence to a solution is low. The formulation is simple, and forms part of a realistic hydraulic simulation program. It is an appropriate tool for decision making in the routine operation and management of water distribution systems and proper simulation of low-supply situations.

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APPENDIX C2

DISCUSSION OF 'RELIABILITY-BASED OPTIMAL DESIGN OF WATER DISTRIBUTION NETWORKS'

Journal of Water Resources Planning and Management, ASCE, 127(4), 278 – 280.

Reliability-Based Optimal Design of Water Distribution Networks[a]

Discussion by T. T. Tanyimboh[3] and P. Kalungi[4]

A considerable amount of research has been carried out on the reliability analysis and optimal design of water distribution systems, and it has been reported that each of the above problems is very difficult to solve (Eiger et al. 1994; Wagner et al. 1988). The authors are therefore to be commended for their work, which directly incorporated a sophisticated probabilistic reliability model into an optimization routine. The paper had other interesting and useful aspects, which, unfortunately, will not be elaborated upon here.

The proposed reliability model was concerned with the "capacity reliability" of water distribution systems, recognizing the possible random variations in demand and the uncertainties in pipe roughness coefficients. Unfortunately, the method was limited to predefined configurations of the network and so did not incorporate the effects of random component failures. The authors mentioned the idea that this can be remedied using the expected value formulation (Fujiwara and Tung 1991), which is worth pursuing further. Although this may appear in theory to require the hydraulic analysis of a large number of configurations, it is possible to reduce the number of system configurations actually simulated.

Tanyimboh and Tabesh (1997a) have shown that quite accurate estimates of the reliability of water distribution systems can be obtained by averaging the upper and lower bound estimates of the reliability, which can be done in a relatively straightforward way (Tanyimboh and Templeman 2000). Therefore, it might be possible in the above manner to address the concern raised by the authors that such a reliability measure may be inappropriate because it may be incomplete if it does not include all possible failure events.

By contrast, the authors tackled the problem of pipe failures in the cost minimization model by adding an extra capacity reliability constraint for each configuration of the network corresponding to a single critical unavailable link. This approach would appear to be computationally less efficient than the direct inclusion of pipe failure effects in the reliability analysis model. This raises the question of how best to approach failure-tolerant design. Unfortunately, without an exhaustive simulation, there is no straightforward procedure for identifying the critical links of a water distribution system. Furthermore, the greater the number of capacity reliability constraints specified for the critical failures, the more difficult it is to solve the cost minimization problem computationally.

An alternative worth considering might be to define the failure tolerance of the distribution system. If quantified, this measure could provide a single reliability constraint to ensure that the vast majority of the most frequent or severe failure cases are addressed in the optimization. Tanyimboh and Templeman (1998) characterized failure tolerance as the performance of the distribution system, on average, during subnormal conditions ensuing from the unavailability of some components of the system. The formulation has the advantage that it can be obtained while calculating system reliability without any need for additional hydraulic simulations, so its use would not impose a major computational burden. However, the measure did not address possible variations in nodal demands or uncertainties in pipe roughness. The authors' paper underscores the need to investigate this issue along with the possibility of directly incorporating failure tolerance in cost minimization models.

As a possible illustration of the usefulness of realistic failure tolerance measures, the average CPU time of 88 s for the authors' Example 2 (Cases 1–4) with only three critical-failure reliability constraints is 2.5 times the average CPU time of 35 s for Example 1 (Cases 1, 6, 7, and 8) with 12 pipe-failure reliability constraints and identical levels of uncertainty. Example 1 had nine nodes, while Example 2 had 16 nodes. Therefore, the computational demands of the optimization model, including the evaluation of reliability, would appear to be high from a practicability viewpoint, as indicated by the above CPU times for a VAX 6000 mainframe computer. Use of a failure tolerance parameter in the manner suggested herein could limit the number of reliability constraints to two (corresponding to the system reliability and failure tolerance), irrespective of the size of the distribution system, and could help ensure that the CPU times remain manageable.

A potential weakness of the first-order reliability method (FORM), as conceded by the authors, is its very computationally demanding nature; despite being able to estimate the reliability of only one node at a time, it has to be repeated for each node whose reliability is required. It follows that the evaluation of the single-value reliability for a multiplicity of system configurations to address the issue of random component unavailability may render the FORM approach excessively time consuming for real distribution systems, especially if a holistic view of system performance is required. Approximately $NN \times NC$ FORM analyses would be required, where NN = number of nodes and NC = number of configurations considered. The value of NC is commonly equal to the number of components in the system, although it can be a lot larger.

It is worth reiterating a point made by the authors that a prerequisite of the FORM is the determination of the design point, i.e., the most probable failure point on the failure hyperplane. This involves the solution of a constrained nonlinear programming problem, which in general is very difficult to do. Also, it is interesting to note that the authors did not include a definition of the reliability of the system as a whole in their formulation. However, as the FORM is capable of evaluating nodal reliabilities, it may be appropriate in this framework to define the reliability of the system as a whole as the demand weighted average of the nodal reliabilities. Evidence of the equivalence of the above measures can be found in Tabesh (1998). Even though this approach would potentially be time consuming, it has the advantages of providing a reliability value for the entire system and avoiding the complicated task of preidentifying a critical node for each reduced network configuration.

As mentioned above, the authors took the system reliability as the reliability of the most critical node, which was defined as the probability that both the pressure and the outflow at the node in question were fully satisfactory. This is equivalent to estimating system reliability as the probability that all demands are met at an adequate pressure. It is questionable whether this is not too conservative a definition of reliability for water distribution systems. This definition does not recognize the concepts of partial failure, which may cause a reduction in the level of service, and system capacity exceedance, due to excessively high demands. In both situations, the amount of water delivered may be appreciable (Cullinane et al. 1992; Gupta and Bhave 1996).

It has been shown that the requirement that all demands at all nodes including the critical nodes be fully satisfied at an

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adequate pressure does not fully recognize the spatial nature of the hydraulic behavior of water distribution systems (Tanyimboh et al. 1999). Deficiencies in performance have a tendency to be rather localized. In other words, the performance can be unsatisfactory is some areas while being fully satisfactory elsewhere (Gupta and Bhave 1996). If such a system with locally insufficient heads is simulated using the demand-driven network analysis approach (with all demands fully met), the deficiency will appear to be far more serious and widespread than it is in reality (Tanyimboh et al. 1997b). Therefore, the authors' use of demand-driven network simulation in this way compounds the underestimation effect of assuming that system reliability is the same as the reliability of the least reliable node (Tanyimboh et al. 1999).

It is useful to note that many of the issues highlighted above can be addressed if another commonly used definition of reliability is adopted, i.e., the mean value of the ratio of the flow delivered to the flow required. The main difficulty with this approach at present is that it involves the simulation of the system performance using head-driven network analysis, for which software is not readily available. However, it has the following advantages:

1. It is more realistic in that it fully recognizes the concepts of partial failure, reduced service, and system capacity exceedance.
2. The reliability of the system as a whole and that of individual nodes can be calculated in a single operation. For each of the *NC* fully connected or reduced network configurations, a single head-driven hydraulic analysis is required to determine the flow delivered. Therefore, a total of *NC* head-driven network simulations would be performed, as compared with $NN \times NC$ FORM evaluations.
3. There is no need to identify the critical nodes of the network for the various configurations. The identification of the critical nodes in a distribution system is in general not straightforward, as observed by the authors. This is particularly true if nodal demands vary randomly, because the spatial distribution of the demands and their relative magnitudes are important factors also.

To conclude, it is quite clear that more research on easy-to-use methods for the optimal design of reliable water distribution systems is needed. As observed by the authors, one area on which attention should focus is the determination of the appropriate probability distributions for nodal demands. Also, there is an urgent need for clarification of the meaning of the term reliability in this context. Head-driven simulation has the potential to resolve several difficulties associated with the reliability analysis of water distribution systems, some of which have been highlighted herein. This area has received little attention. Finally, it would appear that, though powerful, the proposed FORM-based (optimization) approach is best used for estimating (or optimizing) the reliability of only selected nodes of interest as opposed to entire distribution systems.

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Closure by Chengchao Xu[5] and Ian C. Goulter,[6] Members, ASCE

The writers would like to thank the discussers for their interest in the paper and their thoughtful comments. The major points raised in the discussion relate to: (1) measures of system reliability; (2) use of a head driven hydraulic model; and (3) the computational requirements of FORM.

In respect to the first point, it should be noted that the main thrust of the paper is the development of a probabilistic hydraulic model to compute, with explicit recognition of the uncertainty in demands and pipe capacities, approximate values of the nodal capacity reliability for water distribution networks. The model can be used as a simulation engine (1) to determine hydraulic performance under a variety of mechanical failure scenarios, and (2) to combine the probability of systems residing in each of these scenarios to generate different nodal/system reliability measures. FORM is computationally intensive; hence, a number of approximations are necessary to reduce the computational requirements.

The reliability of the most critical node is used as the reliability constraint in the optimization model. The discussers noted that use of the nodal reliability at the most critical point as a measure of system reliability may lead to an overly conservative estimation of the true system reliability. This is not necessarily true, as it depends on how the system reliability is defined. Consider, for example, a simple system with only two demand nodes. Failure events at each node are defined as A and B, with probability of occurrences P_A and P_B, respectively. The probability of system failure $P(A \cup B)$ is equal to $P_A + P_B - P(A \cap B)$, which will not be less than the maximum of P_A and P_B. This means the system reliability, i.e., $1 - P(A \cup B)$, will not exceed the reliability at the most critical node.

There is still some considerable debate on how system reliability for water distribution networks should be defined. The discussers advocated use of the demand-weighted average of the nodal reliabilities as the system reliability. However, averages can conceal a high degree of variability between nodes. This phenomenon gives rise to the question: Is a design acceptable if it has a high value of averaged reliability but low reliabilities at a few localized nodes? The resolution of this issue is a key factor in determining appropriate reliability measures.

In terms of the appropriateness of head-driven hydraulic models, the inadequacy of traditional demand-driven hydraulic models for simulating network behavior under failed or partially failed conditions has long been recognized (Germanopoulos 1985; Xu and Powell 1992; Xu and Goulter 1997). The writers agree with the discussers that it is more appropriate to use a head-driven hydraulic network model to determine sys-

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tem performance under partially failed conditions. The major advantage of the head-driven model is that it enables the demand actually being supplied to be estimated as a part of the solution. However, incorporation of pressure-dependent demands into the model does require more computational effort, with a few more iterations generally being required for convergence. In other cases, when the nodal heads at extreme points are too low to receive any supply, it may be difficult to achieve convergence. In such cases, these nodes, though physically connected to the rest of the network, may not have appropriate hydraulic connections, leading to rank deficiency in the Jacobian matrix of the linearized system equation. This situation can be avoided if the relevant nodes are removed from the network. However, this will require checking the hydraulic connectivity at each iteration and hence increase computational requirements significantly.

While it is relatively easy to include the pressure-dependent demand into a hydraulic model that is based on a nodal formulation with nodal heads as unknowns, it is not clear how these demands can be incorporated into a loop-based model, in which the flows or loop-corrected flows are defined as unknowns. Even if a head-driven hydraulic model is adopted, there is still a need to develop a probabilistic version to handle the uncertainty in the nodal demands and pipe capacity.

Comparison of computational requirements for the reliability assessment using FORM and the head-driven hydraulic model presented in the discussion is not entirely appropriate, since the latter is not able to address the issues associated with the uncertainty in nodal demands and pipe capacity.

FORM is computationally demanding and the discussers correctly noted that its application to very large-scale hydraulic networks may be limited by current computing power. However, this limitation will be gradually removed by the ongoing increases in computing capacity.

Finally, the writers are in complete agreement with the discussers that more research is needed to investigate the issue of defining and providing reliability in water distribution networks. The writers look forward to seeing further work on this topic.

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APPENDIX C3

FAILURE TOLERANCE AS A QUANTIFIED REDUNDANCY MEASURE FOR WATER DISTRIBUTION SYSTEMS

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FAILURE TOLERANCE AS A QUANTIFIED REDUNDANCY MEASURE FOR WATER DISTRIBUTION SYSTEMS

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Abstract
This paper presents a model, based on head driven simulation, for assessing the hydraulic performance of water distribution systems under a range of normal and subnormal operating conditions. The hydraulic analysis model used recognises the pressure dependency of water consumption in the solution procedure. It determines the actual nodal outflows for subnormal operating conditions when the network is overloaded due to very high demands or unavailability of some components. The performance assessment methodology is probabilistic in that it addresses the randomness of component failure or unavailability. The integrated computer program developed can seamlessly calculate a range of performance indicators for a water distribution network including redundancy, reliability and some connectivity-based measures. The approach interprets redundancy as the equivalent of the expectation of the proportion of the total demand that can be met when some components are out of service. Results are presented which demonstrate that, when quantified, failure tolerance is an adequate measure of redundancy for water distribution systems. The importance of calculating the redundancy along with reliability when assessing system performance is also highlighted.

Keywords:
Water distribution systems, redundancy, head driven analysis, reliability, failure tolerance.

1. INTRODUCTION

Assessment of the performance of water distribution systems is a complex process because many issues have to be taken into consideration. These include possible variations in demands, reliability of individual components, unavailability of components and their locations, fire flow requirements and their locations to mention but a few. Further complications arise from the fact that it is difficult to formulate measures and to define what constitutes acceptable levels for some of the main parameters which can be used in the performance assessment of water distribution systems like reliability and redundancy.

Network reliability is often considered to be the extent to which the network can meet customer demands at adequate pressure under normal and abnormal operating conditions. It is a performance assessment measure that tends to focus more on the hydraulic perspective and less upon the underlying robustness of the network in terms of the shape or layout of the network. Existing methods that realistically depict the network reliability are time-consuming and computationally intensive while simpler measures tend to be inadequate in the representation of the network performance. The situation is further aggravated by the fact that existing commercial software for simulating network performance is based on demand driven analysis. While this may suffice for normal operating conditions, it can not cope with abnormal operating conditions when the pressures at some nodes are less than satisfactory. Such situations may arise due to high demands for fire fighting, pump failure, pipe bursts and the unavailability of components due to maintenance or rehabilitation.

Closely related to reliability, an aspect of overall system performance that is often neglected is redundancy. Redundancy addresses the robustness of the network more effectively. It is concerned with the performance of water distribution networks under conditions of partial system failure, when some components are not available. Under these circumstances, there will often be insufficient pressure at the demand nodes to fully satisfy demands. It follows, therefore, that realistic network performance evaluation should be based on head driven analysis. This paper shows that failure tolerance/ redundancy can be used to enhance the process of assessing overall system performance.

The aim of this paper is to highlight the importance of using redundancy and reliability together in the performance assessment of water distribution networks. Redundancy and reliability are calculated using a method based on head driven simulation, which combines the probability that components of the network are operational at any time and the network's ability to meet consumer demands at adequate pressure. Examples are presented which show the significance of redundancy as a performance assessment parameter and that, when quantified, failure tolerance is an adequate measure of redundancy for water distribution systems. The above-mentioned performance assessment is carried out using an integrated FORTRAN computer program with modules for head-driven network modelling, reliability analysis and the automatic generation of reduced system configurations which simulate the unavailability of network components.

2. RELIABILITY

A network with insufficient pressure due to exceedingly high demands or unavailable components due to failures, bursts, repairs and maintenance experiences reduced outflow from some of the nodes. To be able to assess the performance of a network in a realistic way, these and other conditions of partial failure need to be properly accounted for. This can be done by simulating the performance of the network using head-driven analysis. The results of the simulation can then be combined with the probability that the network will be in a particular full or reduced state in terms of the availability of components. Based on the common assumption that pipe unavailabilities are independent, the probability, $p(0)$, that no pipe is unavailable is

$$p(0) = \prod_{l=1}^{NL} a_l \tag{2.1}$$

in which a_l is the probability that link l is available and NL is the number of links.

Pipe availability can be taken as the ratio of the mean time between failures to the sum of the mean time between failures and the failure duration. For example, this can be calculated using the formula developed by Cullinane et al. [1].

The availability of the components and the hydraulic performance of the network can therefore be combined to obtain an overall measure of the reliability of the system. Taking only one and two unavailable components into consideration, and assuming that the demand is constant, the network reliability, R, is given by [2]

$$R = \frac{1}{Q^{req}} \left(p(0)Q(0) + \sum_{l=1}^{NL} p(l)Q(l) + \sum_{\substack{l=1 \\ m=l+1}}^{NL-1} p(l,m)Q(l,m) \right) \tag{2.2}$$

in which: $p(0)$ is the probability that no link is unavailable, $p(l)$ is the probability that only link l is unavailable and $p(l,m)$ corresponds to the probability that two components l and m are unavailable. $Q(0)$, $Q(l)$, and $Q(l,m)$ are the respective actual total outflows when zero components and components l and, l and m are unavailable while Q^{req} is the total demand for the network. In this study, values of $Q(0)$, $Q(l)$, and $Q(l,m)$ were obtained as described in Section 4.

3. REDUNDANCY

Redundancy is the existence of alternative pathways from the sources to demand nodes or excess capacity in normal operating conditions, for use when components become unavailable. To ensure an uninterrupted albeit reduced supply of water, distribution network designs should include some amount of redundancy. Conventionally, redundancy is assumed to be present if the network is looped rather than branched. The interaction between supply paths, the degree to which various paths contribute to the supply of a node and the multiplicity of paths are factors that complicate the assessment of redundancy. Any parameter used as a measure of redundancy should recognise these factors [3].

This issue was addressed by Wagner et al. [4] using the concepts of reachability and connectivity. Reachability was defined as the connection of a specific demand node to at least one source while connectivity was characterised as a situation in which every demand node is connected to at least one source. These measures, which determine whether paths exist or not, are useful for performing initial screening of the system to identify possibilities of problems resulting from insufficient numbers of alternative paths to some areas of the network. However, a more elaborate analysis is required to determine whether a connected node can also meet its demand. Park and Liebman [5] made an attempt to quantify redundancy through a measure based on the expected shortfall in flow delivered due to the failure of individual pipes. This technique explicitly recognises redundancy as a foundation element of reliability.

The issue of failure tolerance is intimately related to redundancy. Two examples, a tree-type network and a looped network, are used in this paper to show that, when quantified, failure tolerance is indeed an adequate measure of the redundancy of water distribution systems. The formula used herein for redundancy or failure tolerance, T, is [6]

$$T = \frac{R - r(0)p(0)}{1 - p(0)} \tag{3.1}$$

in which $r(0)$ is the ratio of available flow to the required flow when all pipes are available. Failure tolerance is the expectation of the proportion of the demand of the network that is satisfied during the periods in which there are mechanical failures in the system or when some components are taken out of service for repair or maintenance [6]. A key feature of Eq. (3.1) is that it is very easy to compute once R and $p(0)$ have been calculated in the reliability evaluation process, since additional hydraulic simulations are not required.

4. HEAD DRIVEN NETWORK MODELLING

A pre-requisite for the realistic assessment of performance is the recognition of the dependency of nodal outflows on heads. The approach used herein for head driven analysis was developed from a study of various deficient networks to assess the effects of variations of outflows at critical nodes. From this study it was observed that a deficient network had four categories of nodes including, no-flow nodes, partial-flow nodes, partial-flow nodes whose outflows affected outflows at other nodes (like nodes at isolated high points) and nodes whose demands are fully satisfied. No-flow nodes are those nodes with heads below the absolute minimum residual head, H_j^{min}, for outflow to be possible at those nodes. Partial-flow nodes are those with heads between H_j^{min} and a desirable residual head, H_j^{des}, for nodal outflow to occur at a desirable pressure. Nodes whose demands are fully satisfied are those with heads above H_j^{des}. Though partial-flow nodes with outflows that affect outflows at other nodes tend to be nodes at isolated high points in the network, in some cases nodes could fall in this category depending upon the layout of the network, the magnitude of nodal demands and their locations in the network.

In general, a distribution system in a stressed state (under abnormal loading conditions and/or with some components unavailable) experiences a reduction in the nodal outflows, in a decreasing progression from the most critically affected nodes to the least critically affected ones. The model developed in this study involves an iterative network analysis starting with initial demand at all nodes, followed by subsequent substitution of some or all of these demands with nodal outflows of the various categories mentioned above. To begin with, a systematic identification of no-flow nodes is done. This is accomplished by setting the flows of the most critical nodes (nodes with the lowest nodal residual heads) to zero and analysing the network (this generally raises the heads in the network). If the heads at these most critical nodes remain below H_j^{min}, they are confirmed as no-flow nodes and their flows fixed at zero. Partial-flow nodes are then identified and their outflows are obtained by analysing the network as described below. Finally, partial-flow nodes whose outflows affect outflows at other nodes in the network are identified and their outflows are obtained by analysing the network as described below. The procedure ends when the remaining nodes have their heads above H_j^{res}.

The two conditions that have to be satisfied in water supply networks are nodal flow continuity and conservation of energy applied to each loop or path. The Newton-Raphson iterative scheme was used to solve the system of constitutive equations. At the stage of obtaining outflows for the partial-flow nodes, head-flow equations as shown in Eq. (4.1) are required. The heads of the partial-flow nodes are fixed and their outflows become the unknowns. These head-flow equations are also used to obtain heads for the rest of the nodes at this juncture. Thus

$$J_{HQ}^{n}\begin{bmatrix}\underline{\Delta H}^{n}\\ \cdots\cdots\\ \underline{\Delta Q}^{n}\end{bmatrix}=-\underline{F}\left(\underline{H}^{n},\underline{Q}^{n}\right) \tag{4.1}$$

in which $\underline{H}$ is the vector of unknown heads, $\underline{Q}$ is the vector of unknown outflows, J_{HQ} is the Jacobian matrix, $\underline{\Delta H}$ is the vector of the respective changes in nodal heads between iterations, $\underline{\Delta Q}$ is the vector of the respective changes in unknown nodal outflows between iterations and $\underline{F}$ is the vector of respective values of the nodal continuity expressions, i.e. F_j, for j=1, ..., NJ. F_j denotes the sum of flows entering and leaving node j and NJ is the number of nodes. The iteration number is denoted by n.

Herein, nodes said to be in the same pressure contour are those whose residual nodal heads are approximately the same. As such, they are processed together in groups according to their pressure contour zones. Based on the above principles, an algorithm (Critical-node Head Driven Simulation Method or CHDSM) for calculating head-dependent outflows was developed [7].

5. DEMONSTRATIONS USING SAMPLE NETWORKS

To appraise the performance assessment model of Section 4 and demonstrate the redundancy or component-failure tolerance measure, two networks were analysed with up to two components simultaneously unavailable. The pipe availabilities were calculated based on the diameters [1]. The lengths and the Hazen-Williams coefficients for all pipes are 1000m and 130 respectively.

5.1 Example 1

The network is shown in Figure 1 [8]. The diameters of pipes 1 through 4 are 400mm, 350mm, 300mm and 300mm, respectively. The outlet elevations of nodes 1 through 4 are 90m, 88m, 90m, and 85m, respectively. Nodes 1 through 4 have demands of $2m^3$/min, $2m^3$/min, $3m^3$/min and $4m^3$/min, respectively.

The source head was varied to correspond to a range of reservoir conditions and the performance of the system assessed in each case. It is worth noting that below a source head of 110.89m the network is stressed in that the source head is not sufficient to satisfy all the nodal demands. For comparison purposes, another hydraulic analysis software package called EPANET [9] was used with nodal outflows obtained from the proposed performance assessment model, in order to obtain the source heads required to satisfy these outflows. The results are shown in Figure 2. In this figure, the fact that the *r(0)* values of CHDSM coincide with those of EPANET confirms the accuracy of the hydraulic analysis model used.

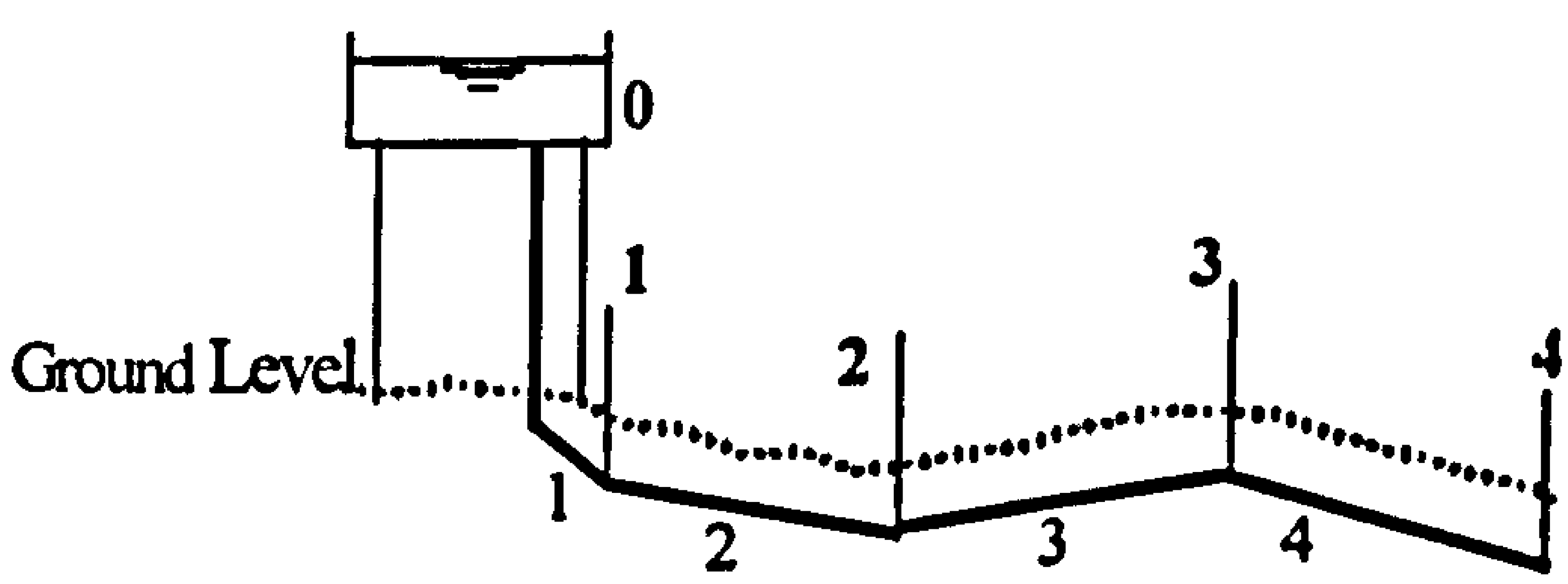

Figure 1. Serial Network

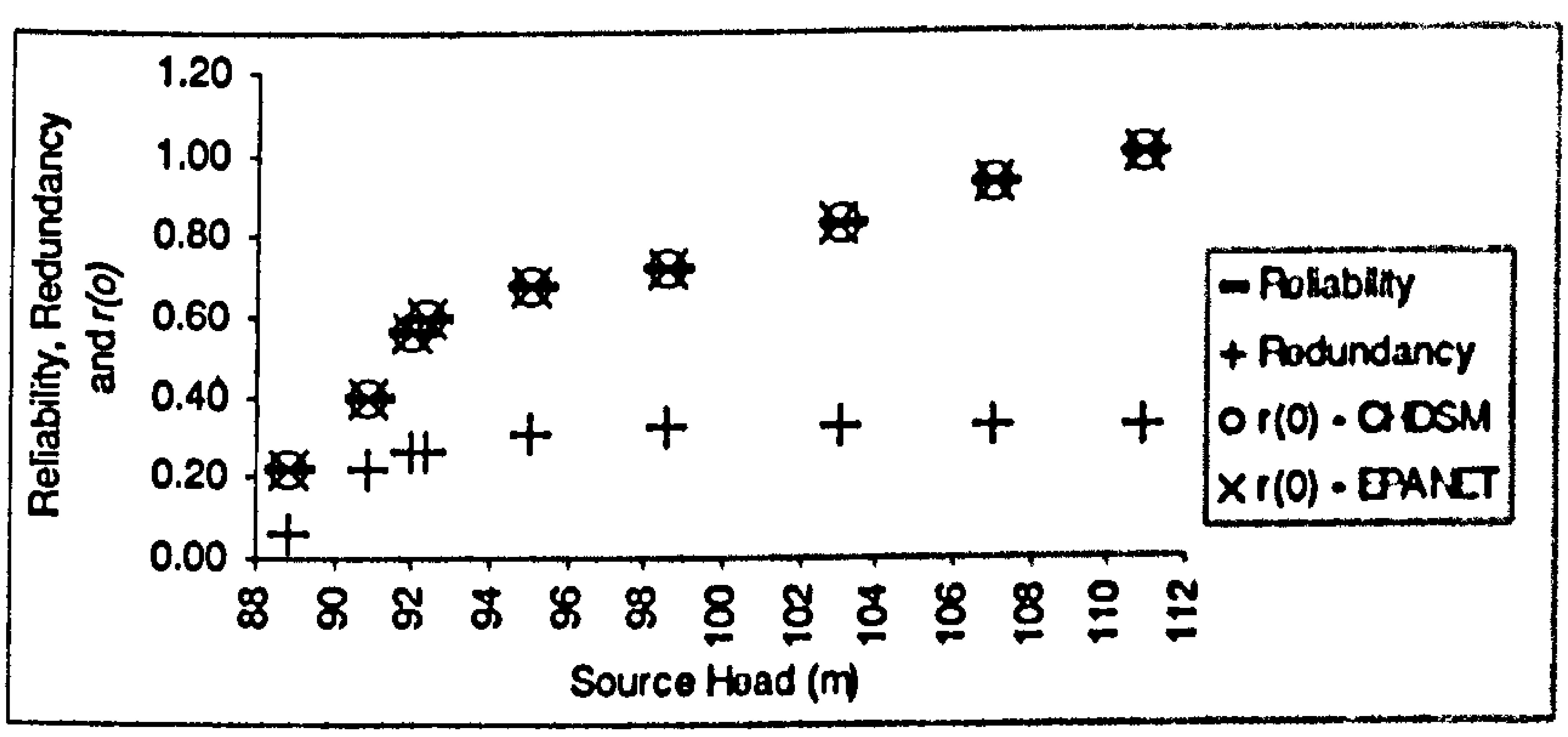

Figure 2. System Performance for a Range of Source Heads

5.2 Example 2

Several researchers (e.g. Fujiwara and Tung [10]) have used the two–loop network in Figure 3. The pipe data for six candidate designs are presented in Table 1. The source node has a fixed head of 35m, and all elevations of demand nodes are 0m. H_j^{min}, the absolute minimum residual head for flow to be possible at a node was taken as 0m, while H_j^{res}, the residual head for nodal outflow at a desirable pressure, was set at 15m [11]. The proposed performance assessment model was used to obtain a range of performance data as shown in Figure 4.

Table 1. **Candidate designs for the two-loop network**

Links	Diameters (mm)					
1-3	401	401	390	384	365	367
2-4	100	100	165	191	238	235
3-5	338	337	337	329	281	294
4-6	100	100	100	151	250	234
5-6	263	262	262	249	152	185
1-2	157	165	203	224	263	261
3-4	237	237	213	215	247	234
Design No.	1	2	3	4	5	6

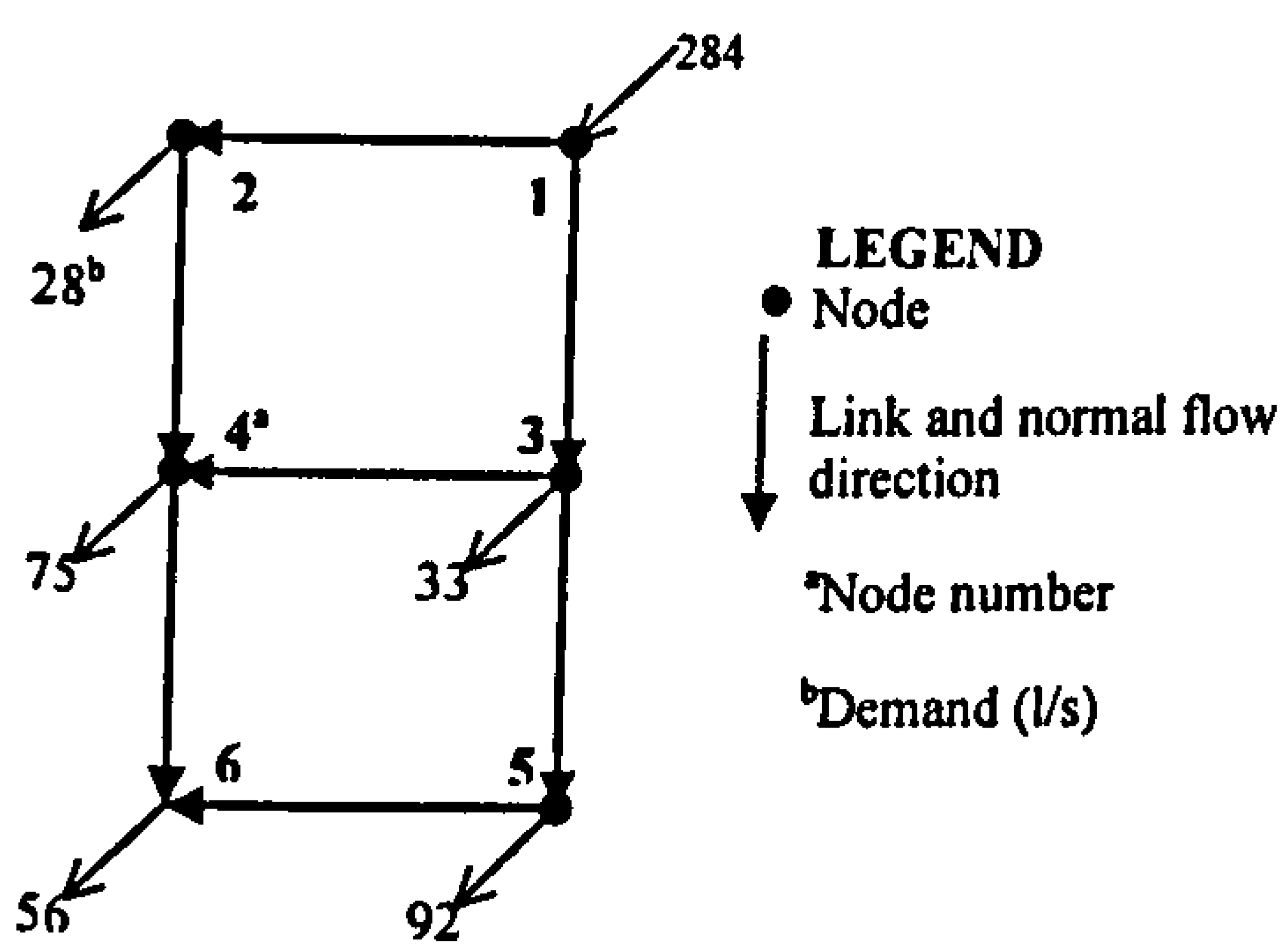

Figure 3. **Two-Loop Network**

6. DISCUSSION

Figure 2 (Example 1) suggests that it may be more difficult to improve failure tolerance or redundancy than reliability in a cost-effective way, especially for branched networks. The figure shows that the network has a low failure tolerance level of less than about 0.35, irrespective of the source head value used, because alternative supply paths are not available. The design therefore has a low degree of redundancy. This vulnerability through lack of redundancy may not necessarily be obvious to an inexperienced designer, if the failure tolerance value is not calculated explicitly. A quantified measure of redundancy such as failure tolerance should therefore, be used alongside reliability for network performance assessment.

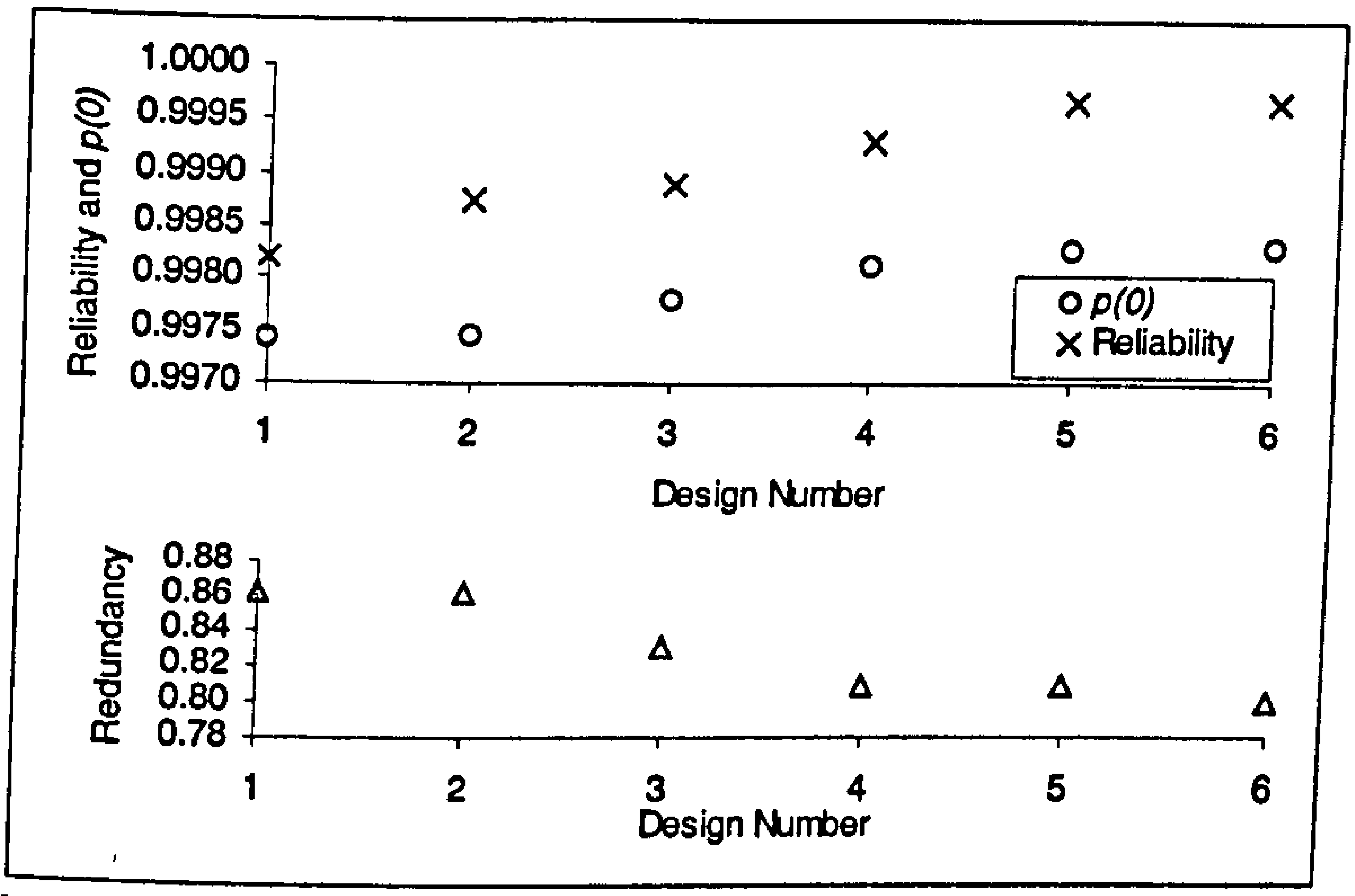

Figure 4. Performance Measures for Alternative Designs

For instance, consider two reservoir conditions when the source head is 98.5m and 110.89m respectively. The associated redundancy value is 0.325 for both conditions and the reliabilities are 0.7270 and 0.9995 respectively. The redundancy or failure tolerance values clearly show that the network is highly vulnerable to component failure in both cases, a fact that could undermine the higher reliability value of the latter case. This would not necessarily be obvious if the reliability values alone were to be considered. The redundancy values are equal due to the nature of the network layout. Being a pipeline or branched network, it depends largely on connectivity such that above a certain source head, the expectation of total flow delivered is the same. On the other hand, the reliabilities vary mainly due to the varying total outflow for the various reservoir conditions as indicated by the trends and closeness of results of reliability and the proportion, *r(0)*, of total flow delivered (Figure 2).

The graphs in Figure 4 (Example 2) suggest, as expected, that reliability or overall performance will improve if mechanically more reliable components are used, to ensure that the frequency of a failure occurring is small. However, if the issue of redundancy is not considered explicitly, then serious problems may result when components are taken out of service for maintenance purposes. As expected, there is a general conformity in the trend of network reliability and the probability, *p(0)*, that all links are available, since the network would be fully connected most of the time (see Eq. (2.1) and the *p(0)* values in Figure 4). However, failure tolerance or redundancy does not follow the same trend. Redundancy values largely depend on the impact of the unavailability of individual components. From Table 1, it is clear that the link diameters for links 2-4, 4-6, and 1-2, generally increase significantly from Design 1 to 6. This probably means that unavailability of these links would lead to total network outflow decreasing from Design 1 to 6, hence the trend in failure tolerance. The lowest failure tolerance value for the six different designs is 0.8, suggesting that all the designs have a reasonable degree of redundancy. It is also clear that the designs with the highest and comparable network reliability values (Designs 5 and 6), which is due to the fact that they have the highest *p(0)* values and the highest total outflow when all links are available, are the most vulnerable to component unavailability. Thus comparing Designs 5 and 6, the former might be deemed to be better on the grounds that its redundancy is marginally superior. Therefore, for a more comprehensive performance appraisal, reliability and redundancy should be used together.

7. CONCLUSIONS

An integrated performance assessment model for water distribution networks has been presented. It is capable of carrying out hydraulic analysis based on the simulation of head-dependent outflows in the solution procedure. It can simulate networks under abnormal loading conditions and/or with some components unavailable. The methodology for system performance assessment automatically considers the randomness of component unavailability in calculating hydraulic performance indices, if required.

A quantified measure, called failure tolerance, for assessing the redundancy of water distribution networks has also been presented. Using examples, the appropriateness of failure tolerance as a redundancy measure for water distribution networks has been demonstrated. The need to adopt both redundancy and reliability as the basis for assessing system performance has also been highlighted.

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APPENDIX C4

MODEL FOR THE OPTIMAL DESIGN AND UPGRADING OF WATER DISTRIBUTION NETWORKS

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MODEL FOR THE OPTIMAL DESIGN AND UPGRADING OF WATER DISTRIBUTION NETWORKS

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ABSTRACT

The choice of the best network upgrading strategy within a limited budget is a complex optimisation problem. A model for assisting in decision making to upgrade a deteriorating network is presented. It explicitly considers deterioration over time of both the structural integrity and hydraulic capacity of every pipe. Water distribution network economics and hydraulic performance are analysed simultaneously over a predefined design horizon using linear programming while the timing of upgrading over the entire planning horizon is based on dynamic programming. Maximum entropy flows are used to reduce the dimensionality of the problem. The aim is to minimise the present value of capital, repair and damage costs. Two design options are used in an example to highlight the importance of incorporating quantified performance measures such as damage tolerance in making network-upgrading decisions.

Key words: Water Distribution Networks, Entropy, Optimal Upgrading, Pipe Deterioration, Reliability.

INTRODUCTION

Water companies are charged with the responsibility of delivering water to meet demand at adequate pressure through water distribution networks. Deterioration of the distribution network as it ages, undoubtedly leads to an increase in operation and maintenance costs and a reduction in the quality of service. Most water companies spend more than half of their total budgets to combat this problem. Therefore, large savings can be gained if a cost-effective long-term upgrading approach is adopted. Any comprehensive upgrading strategy should address the issues of timing and magnitude of the upgrading over the entire planning horizon along with performance and water quality requirements within a limited budget. The performance criterion addresses issues of reliability and damage tolerance. Reliability is the ability of the water distribution system to meet demands at adequate pressure under normal and abnormal operating conditions. Damage tolerance or component failure tolerance is a measure for the existence of alternative pathways from the sources to demand nodes or excess capacity in normal operating conditions, for use when components become unavailable. The water quality criterion is mainly concerned with the fact that aging of pipes is accompanied by tuberculation, microbiologic slime growths and deposits that lead to the deterioration of the quality of water and reduction in carrying capacity of pipes.

Previous work in this area has not addressed all the issues including rehabilitation, replacement and/ or expansion in a holistic fashion. For example, some models did not tackle the issue of timing of the upgrading, whilst others did not address the deterioration of pipes over time. These key issues have to be addressed using systemwide models that incorporate the performance of the system explicitly. Kleiner et al. (1998) proposed a model that considers the life-cycle time for each pipe. The model does not directly identify the pipes that require rehabilitation but rather the funds that are required to be assigned for rehabilitation purposes. Halhal et al. (1997) used Genetic Algorithms in a systemwide model to maximise benefits subject to limits on funding. The benefits they considered are hydraulic, physical integrity, flexibility and quality. Dandy and Engelhardt (2001) used a genetic algorithm technique to find a near optimal schedule for the replacement of water supply pipes and included repair and damage costs. The formulation allowed for multiple time steps and an evaluation of the hydraulic performance of the network when pipes are replaced by pipes of different sizes. These system wide models are quite complex and computationally demanding. Perhaps because of complexity, none of these models addresses a key performance assessment measure called damage tolerance. The importance of damage or failure tolerance has been stressed by Kalungi and Tanyimboh (2001).

This paper presents a model that can assist in decision making to upgrade a deteriorating network. The proposed method explicitly considers deterioration over time of both the structural integrity and hydraulic capacity of every pipe, and allows for the direct and indirect repair costs. It simultaneously considers the options of paralleling and upsizing of pipes. Linear programming is used to combine distribution network economics and hydraulic performance for various design periods. The timing of upgrading over the planning horizon is based on dynamic programming. A new entropy-based approach is used for flow distribution to reduce the dimensionality of the problem. Two design options are compared to highlight the importance of incorporating quantified performance measures such as damage tolerance in the decision making process of network upgrading.

FORMULATION OF UPGRADING PROBLEM

The upgrading strategy over a given planning horizon is subdivided into two design periods. In the first design period, each reach or link has two key design variables, *Ln*, the length and *Dn*, the diameter. In the second design period, each reach is assigned five variables: (i) *Le*, the existing length, (ii) *Lp*, the parallel pipe length, (iii) *Lr*, the length of replaced pipe, (iv) *Dp* the diameter of the parallel pipe, (v) *Dr* the diameter of the replaced pipe.
These variables are depicted in Figure 1.

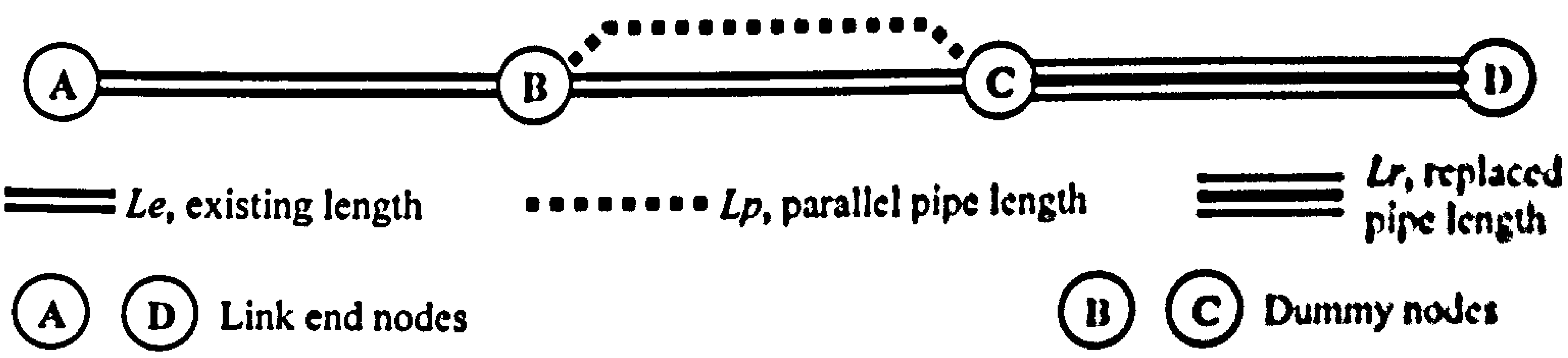

Figure 1. Typical link representation for the second design period

In reality, the upgrading problem is a non-linear problem that would require non-linear optimisation. But given the total number of decision variables that would be involved non-linear optimisation would be limited to small networks. To try and reduce the dimensionality of the problem, a new entropy-based approach is used for feasible flow distribution (Yassin-Kassab et al., 1999), and then linear programming is used directly to obtain the solution. The use of segmental pipes has been used in several optimal design models for looped networks (e.g. Alperovits and Shamir, 1977). The timing or optimal scheduling, cost and magnitude for the long term upgrading strategy of each design option is carried out using dynamic programming.

Mathematical statement

The cost, $\dot{C}_\tau(s_\tau, r_\tau)$, of adding capacity, r_τ, in each design period or phase, τ, is a function of that added capacity, as well as the existing capacity, s_τ, at the beginning of the period. The planning problem is to find that time sequence of capacity expansions that minimises the present value of total future costs (e.g. in US dollars like all costs in this paper) and meets the projected requirements. Hence the planning model can be written as;

Minimise $$Cost = \sum_{\tau=1}^{2} \beta_\tau C_\tau(s_\tau, r_\tau)(1+b)^{(n-v)} \quad (1)$$

where $$C_\tau(s_\tau, r_\tau) = (f1 + f2 + f3) \quad (2)$$

$f1$ represents pipeline costs which include installation, paralleling, replacement, and repair costs. $f2$ is the cost of setting up construction plant and machinery at the beginning of each phase. $f3$ represents costs that vary with the magnitude of installed capacity. β_τ is a product of a discount factor $(1+r)^{-v}$, and a price increase factor $(1+c)^{v}$. The symbol v represents the number of years preceding a design phase. For example, in Phase I, $v = 0$. It is assumed that the discount rate is r% per annum, in each period τ. The price increase factor assumes that there is a general increase in construction costs at a rate of c% per annum. It has also been assumed that the values of c and r are equal, thus β_τ has unit value. Construction costs are incurred at the beginning of each period. Capital is to be raised by borrowing at an annually compounded interest rate of b%. All borrowed capital has to be paid back by the end of the design horizon or the end of the second phase, i.e. after n years.

$f1$: Pipeline costs

The pipeline costs are obtained by linear optimisation as formulated below

Minimise costs, $$f1 = (f1_a + f1_b + f1_c) \quad (3)$$

where $f1_a$ and $f1_b$ represent costs of new and parallel pipelines including supply and installation respectively, which are detailed as

$$f1_a = f1_b = \sum_{ij \in IJ} \sum_{m=1}^{N_{ij}} \left(\gamma_p * \exp(c_p * D_{ijm}) * l_{ijm} + REP_{ijm}\right) \quad (4)$$

in which IJ is the set of all links in a network, D_{ijm} and l_{ijm} are the diameter and length (in meters) of segment m of link ij respectively. N_{ijm} is the number of segments specified for link ij. REP_{ijm} is the failure cost for segment m of link ij consisting of the present value of direct and indirect failure costs for any pipe older than 5 years. γ_p and c_p are parallel pipe cost constants that are specified by the user. The failure cost is given by

$$REP_{ijm} = \sum_{t=tb}^{tr} \frac{J(t)_{ijm} * CB_{ijm} * FCF(LU_i)_{ij} * l_{ijm}}{(1+r)^{t-ts+1}} \quad (5)$$

where r is the discount rate; ts is the first year of a given design period and tr is the last year of a design period. tb = 6 years for replaced and parallel pipes, and pipes in the first design period, e.g. if the second design period starts in the 13th year of a 20-year design horizon, ts = 13 and tr = 20. tb = ts for existing pipes in the second design period. $FCF(LU_i)_{ij}$ is the failure cost factor for land use, LU_i for link ij. These failure cost factors cater for indirect costs caused by pipe failures like disruption to traffic and damage to third parties (Dandy and Engelhardt, 2001). For segment m of link ij, CB_{ijm} is the repair cost per break and $J(t)_{ijm}$ is the break rate (breaks/Km/year) in year t. The repair cost per break is obtained by regression analysis (e.g. based on figures by Dandy and Engelhardt (2001)) as

$$CB_{ijm} = \gamma_{br}(D_{ijm} * 1000)^{\Phi} \quad (6)$$

where γ_{br} and Φ are the break repair cost constant and exponent that are specified by the user. The break rate is

$$J(t) = 0.001974 * \exp(-0.00974 * D_{ijm}) * age_{ijm}^{1.808} \quad (7)$$

where age_{ijm} is the age of segment m of link ij in years, and Eq 7 is valid for asbestos cement pipes. $f1_c$ represents costs of replacing deteriorated pipes including supply and installation detailed as

$$f1_c = \sum_{ij \in IJ} \sum_{m=1}^{N_{ij}} \left(\gamma_r * \exp(c_r * D_{ijm}) * l_{ijm} + REP_{ijm}\right) \quad (8)$$

where γ_r and c_r are replaced pipe cost constants that are specified by the user.

The objective function costs of $f1$ are minimised subject to;

$$h_{ijm} = \alpha L_{ijm}\left(q_{ijm} / C_{ijm}\right)^{1.852} / D_{ijm}^{4.87} \qquad \forall ijm \tag{9}$$

in which C_{ijm} is the Hazen-Williams hydraulic conductivity coefficient in segment m of link ij. C_{ijm} deteriorates over time at a rate that varies according to pipe type, supplied water quality, and the operation and maintenance practices. To model the effect of aging on carrying capacity of pipes, the equation of Sharp and Walski (1988) is used, i.e.

$$C_{ijm}(t) = 18.0 - 37.2\log\left[\frac{e_{0ijm} + a_{ijm}\left(age_{ijm}\right)}{D_{ijm}}\right] \tag{10}$$

in which, for segment m of link ij, $C_{ijm}(t)$ is the Hazen-Williams hydraulic conductivity coefficient at year t that is used to replace C_{ijm} in equation (8), e_{0ijm} is the initial roughness (mm) at time of installation and a_{ijm} is the roughness growth rate (mm/year). The segment diameters should be in millimetres.

$$D_{ijm,\max} \geq D_{ijm} \geq D_{ijm,\min} \qquad \forall ij \tag{11}$$

$$Le_{ij} + Lp_{ij} + Lr_{ij} = L_{ij} \qquad \forall ij \tag{12}$$

$$\sum_{m=1}^{N_{ij}} l_{ijm} = L_{ij} \qquad \forall ij \in IJ \tag{13}$$

$$l_{ijm} \geq 0 \qquad \forall ijm \tag{14}$$

The rest of the constraints which have not been shown explicitly above, include the loop, path, nodal flow continuity and service pressure constraints. The parameters C_{ij}, D_{ij}, h_{ij}, L_{ij} and q_{ij} are the roughness coefficient, diameter, headloss, length and flow rate, respectively, for pipe ij; $D_{ijm,\min}$ = minimum allowable pipe diameter; $D_{ijm,\max}$ = maximum pipe diameter. Le is the existing length, Lp, the parallel pipe length and Lr is the length of replaced pipe.

$f3$: Costs that vary with the magnitude of installed capacity.

$f3$ represents costs that vary with the magnitude of installed capacity in a particular phase. These costs were included on the grounds that the throughput volume of water released from the reservoir into the system, has a proportion of the costs attributed to treatment, transmission of water to the main reservoir, etc. The generalised relationship that was assumed to represent all these costs can be stated as follows:

$$V_{costs} = VC * Q_{inst}^{VE} \tag{15}$$

where, Q_{inst} is the installed capacity in a particular phase in l/s, and V_{costs} is in US dollars, VC and VE which are a cost coefficient and exponent respectively, vary depending on the above factors and are specified by the user.

MODEL APPLICATION AND DISCUSSION

The model was applied to two networks as possible design options for serving five demand nodes. The first option (Fig. 1) is fully looped and the pipe flow rates were obtained using the maximum entropy flow distribution algorithm (Yassin-Kassab et al., 1999). There is a body of evidence which suggests that, for water distribution networks, the association between entropy and reliability is strong (Tanyimboh and Sheahan, in press). Therefore this would ensure that the design has a high reliability and damage tolerance. The second option (Fig. 2) is partially looped and partially dendritic and the pipe flow rates were obtained using the conventional minimum flow path method (Orth, 1986). The second design is a compromise between the fully looped and branch networks and it avoids the possible excessive redundancy of fully looped network. The nodal design demands are a combination of the peak demand pattern and a proportion of fire demand (at Node 2) to avoid over designing and assuming that this combination would stress the system most and give a better representation of the damage tolerance. Demands are assumed to increase at an annual rate of 4% up to the 20-year design demands shown in Figs. 2 and 3.

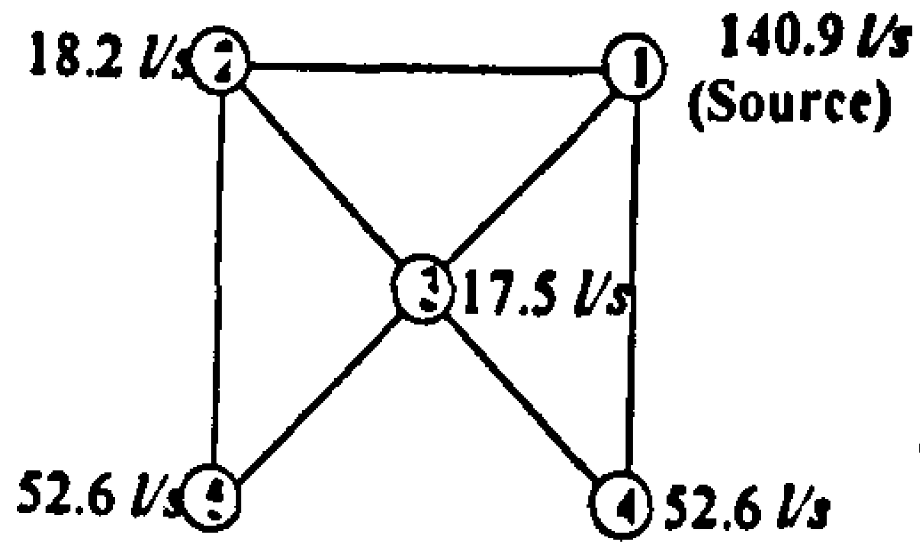

Figure 2. Option 1:
Three-loop network

LEGEND
④ NODE NUMBER

52.6 l/s NODAL DEMAND

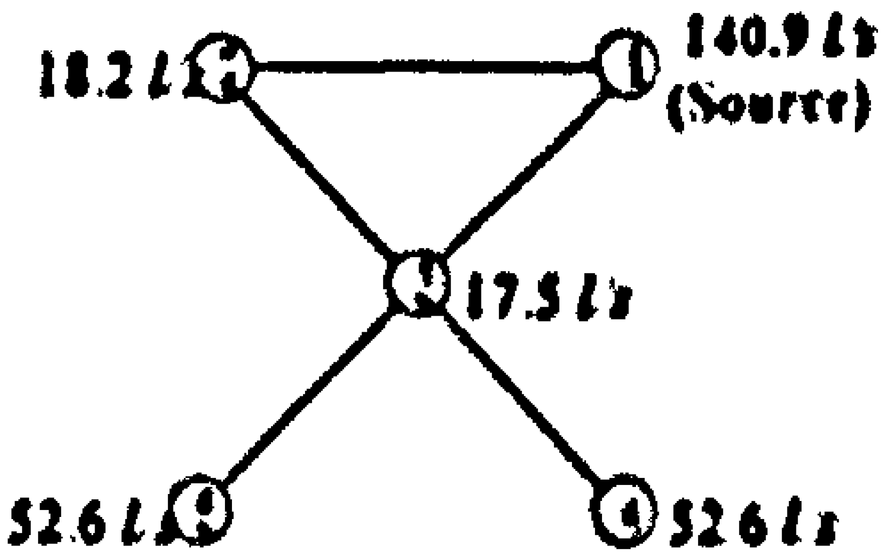

Figure 3. Option 2:
Single-loop network

All pipes are 1000m long. The water level at the source is 70m while demand nodes have elevations of 0m and minimum service heads of 15m. Hazen-Williams coefficient is 130 for all new pipes. Compound interest rate, $b = 8$, design horizon $n = 20$ years, v for Phase II varies from 7 to 15 years. $FCF(LU_l)_{ij}$, the failure cost factors for land use were taken as 4 for all links. r, the discount rate = 8%, e_{0ijm}, the initial roughness = 0.0021mm and a_{ijm}, the roughness growth rate = 0.025 mm/year (Bhave, 1991). Pipe cost constants used were $\gamma_p = 32.093$; $c_p = c_r = 3.7$; $\gamma_r = 33.928$ and $\gamma_{br} = 108.87$. The pipe cost exponent $\phi = 0.6067$. The limiting velocities in pipes are 0.5m/s and 3m/s. The maximum allowable hydraulic gradient is 0.05. Setting up costs, β, per design phase = \$100,000. VC, the cost coefficient = 130 and VE, the cost exponent = 1.6. Unavailability of pipes was calculated using a formula from Cullinane et al. (1992). The performance of the WDS with the broken pipes isolated was simulated and reliability and damage tolerance calculated using the Critical-node Head Driven Simulation Method (CHDSM) as described in Kalungi and Tanyimboh (2001). The equations for the reliability calculations are detailed in Tanyimboh and Sheahan (in press) and for damage tolerance in Tanyimboh and Templeman (1998). The optimal designs are shown in Tables 1, 2 and 3. The overall costs for both designs are also shown in Table 4.

TABLE 1. OPTIMAL DESIGNS FOR DESIGN PERIOD ONE

	OPTION 1: THREE-LOOP DESIGN					OPTION 2: SINGLE-LOOP DESIGN			
	SEGMENT ONE		SEGMENT TWO			SEGMENT ONE		SEGMENT TWO	
LINK	DIAMETER (m)	LENGTH (m)	DIAMETER (m)	LENGTH (m)	LINK	DIAMETER (m)	LENGTH (m)	DIAMETER (m)	LENGTH (m)
1-2	0.200	1000.00	-	-	1-2	0.200	1000.00	-	-
1-3	0.150	1000.00	-	-	1-3	0.200	1000.00	-	-
1-4	0.100	809.01	0.150	190.99	2-3	0.150	150.81	0.200	849.19
2-3	0.150	76.97	0.200	923.03	3-4	0.150	712.23	0.200	287.77
2-5	0.100	595.63	0.150	404.37	3-5	0.150	1000.00	-	-
3-4	0.150	143.12	0.200	856.88					
3-5	0.150	1000.00	-	-					

TABLE 2. OPTIMAL DESIGN FOR THE THREE-LOOP NETWORK IN DESIGN PERIOD TWO

	LINK	(A-B): EXISTING UNPAR LINK			(B-C): PARALLEL LINK			(C-D): REPLACED LINK		
	FLOW	DIAMETER	LENGTH		DIAMETER	LENGTH		DIAMETER	LENGTH	
LINK	(l/s)	(m)	(m)	CHW	(m)	(m)	CHW	(m)	(m)	CHW
1-2	79.50	0.200	93.37	114.7	0.250	906.63	130.0	-	-	-
1-3	43.82	-	-	-	0.150	861.88	129.4	0.150	138.12	129.4
1-4	17.53	-	-	-	0.100	1000.00	122.8	-	-	-
2-3	43.82	0.192	1000.00	114.1	-	-	-	-	-	-
2-5	17.53	-	-	-	0.100	1000.00	122.8	-	-	-
3-4	35.06	0.186	1000.00	113.5	-	-	-	-	-	-
3-5	35.06	0.150	468.65	110.1	0.200	531.35	130.0	-	-	-

TABLE 3. OPTIMAL DESIGN FOR THE SINGLE-LOOP NETWORK IN DESIGN PERIOD TWO

	LINK	(A-B): EXISTING UNPAR. LINK			(B-C): PARALLEL LINK SEG. 1			(B-C): PARALLEL LINK SEG. 2		
	FLOW	DIAMETER	LENGTH		DIAMETER	LENGTH		DIAMETER	LENGTH	
LINK	(l/s)	(m)	(m)	CHW	(m)	(m)	CHW	(m)	(m)	CHW
1-2	59.05	0.200	1000	114.7	-	-	-	-	-	-
1-3	81.80	0.200	948.65	114.7	0.250	51.35	130.0	-	-	-
2-3	40.90	0.185	1000	113.5	-	-	-	-	-	-
3-4	52.59	-	-	-	0.200	513.93	127.6	0.250	486.07	130
3-5	52.59	-	-	-	0.150	189.98	122.0	0.200	810.02	129.88

TABLE 4. SUMMARY OF THE COSTS FOR THE NETWORKS

THREE-LOOP NETWORK COSTS				SINGLE-LOOP NETWORK COSTS			
PHASE 1 TIME (Yrs)	PHASE 1 COSTS ($)	PHASE 2 COSTS ($)	OVERALL COSTS ($)	PHASE 1 TIME (Yrs)	PHASE 1 COSTS ($)	PHASE 2 COSTS ($)	OVERALL COSTS ($)
13	3478529.25	737166.81	4215696.00	10	2847015.25	682930.31	3529945.50
14	3566436.75	648366.06	4214803.00	11	2914191.00	615591.88	3529783.00
15	4172562.25	200822.39	4373384.50	12	2989242.50	551982.13	3541224.50

From Table 4, the cheapest cost strategy for the first option has a value of $4,214,803 and it is to design for a 14 year demand in phase one, and the ultimate design demand in phase two. The cheapest cost strategy for the second option has a value of $3,529,783 and it is to design for an 11 year demand in phase one, and the ultimate design demand in phase two. Design option 1 of the three-loop network has a reliability of 0.999832 and a damage tolerance of 0.934716. Design option 2 has a reliability of 0.999484 and a damage tolerance of 0.647552. The results in Table 2 show that paralleling is preferred to replacement probably because it is cheaper and that the rate of deterioration of the pipes is very low. A higher rate of deterioration might perhaps require a more balanced use of both paralleling and replacement to meet the velocity and maximum hydraulic gradient requirements. None of the links for the single-loop network in design period 2 was replaced (Table 3). It is clear from these results the cheaper option of the single-loop network has a lower damage tolerance, and the more expensive option of the three-loop network has a higher damage tolerance. In order to reconcile these conflicting criteria in the decision making process of selecting the best upgrading option, a simple multi-criteria decision making method referred to as weighted ranking (Ahmad, 1985) has been adopted. It involves assigning importance weights to the decision criteria, present value of costs, reliability, and damage tolerance. The options are ranked with respect to each criterion, the best being assigned 1 and the second 2. If they have almost equal importance, the average of 1 and 2 is assigned (e.g. reliability ranking in Table 5). Weighted ranks are obtained as products of the ranking and the respective importance weights. These products are summed for each option and the best option is the one with the lowest total score. The details are in Table 5, in which the present value of costs is assigned a higher importance weight leading to the cheaper design of option 2 becoming the best option since it has the lowest score.

TABLE 5. WEIGHTED RANKING TO DETERMINE BETTER OPTION

	DAMAGE TOLERANCE (*WEIGHT=0.3*)		RELIABILITY (*WEIGHT=0.2*)		PRESENT VALUE OF COSTS (*WEIGHT=0.5*)		
DESIGN	RANK	WEIGHTED RANK	RANK	WEIGHTED RANK	RANK	WEIGHTED RANK	TOTAL SCORE
OPTION 1	1	0.3	1.5	0.3	2	1	1.6
OPTION 2	2	0.6	1.5	0.3	1	0.5	1.4

TABLE 6. WEIGHTED RANKING SENSITIVITY ANALYSIS

	DAMAGE TOLERANCE (*WEIGHT=0.4*)		RELIABILITY (*WEIGHT=0.3*)		PRESENT VALUE OF COSTS (*WEIGHT=0.3*)		
DESIGN	RANK	WEIGHTED RANK	RANK	WEIGHTED RANK	RANK	WEIGHTED RANK	TOTAL SCORE
OPTION 1	1	0.4	1.5	0.45	2	0.6	1.45
OPTION 2	2	0.8	1.5	0.45	1	0.3	1.55

A sensitivity analysis in Table 6 shows that if the damage tolerance criterion is given a slightly higher importance weight, the best option would become option 1, the three-looped design which is more expensive but more tolerant to damage. On the other hand, the reliability criterion which is more commonly used, shows that an increase in the importance weight gives the same weighted rank. The design choice is quite sensitive to the present value of costs and damage tolerance but it must also be reliable. Thus it is important to include these three criteria in the decision-making process.

Choosing the best network upgrading strategy is not an easy process and a number of compromises have to be made. Explicit consideration of deterioration over time of the pipes, network economics and hydraulic performance must be addressed. The model presented considers all these issues and determines the timing and magnitude of upgrading over the planning period.

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APPENDIX C5

REDUNDANCY MODEL FOR WATER DISTRIBUTION SYSTEMS

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REDUNDANCY MODEL FOR WATER DISTRIBUTION SYSTEMS

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Abstract

This paper presents a model, based on head driven simulation, for assessing the redundancy of water distribution systems. The formulation recognises the pressure dependency of water consumption in the solution procedure. A new algorithm for pressure dependent modelling of water distribution systems has, therefore, been developed. Notable features of the proposed network analysis technique include the introduction of a new subcategory of nodes called key partial-flow nodes and the use of a joint head-flow system of equations. The algorithm is reliable, quick and easy-to-implement.

The redundancy assessment methodology addresses the randomness of component failure or unavailability. Results are presented which demonstrate the suitability and meaning of the redundancy measure. In particular, it is recommended that redundancy be evaluated along with reliability when assessing system performance. The computer program developed can seamlessly calculate several performance indicators including reliability.

Keywords: Water distribution systems, demand driven analysis, head driven analysis, redundancy, reliability

1. INTRODUCTION

Assessment of the performance of water distribution systems is a complex process because many issues have to be taken into consideration. These include possible variations in demands, reliability of individual components and their locations, fire flow requirements and their locations to mention but a few. Further complications arise from the fact that it is difficult to define useful performance measures and establish what acceptable levels for these parameters are. Network reliability is often considered as the extent to which the network can meet customer demands at adequate pressure under normal and abnormal operating conditions. It is a performance assessment measure that tends to focus more on the hydraulic perspective and less upon the underlying robustness of the network in terms of its layout.

Closely related to reliability, an aspect of the overall system performance that is often neglected is redundancy. Redundancy addresses the resilience of the network more effectively. It is concerned with the performance of water distribution networks under conditions of partial system failure. Such situations may arise due to pump failures, pipe bursts and the unavailability of components due to maintenance or rehabilitation. Under these circumstances, there will often be insufficient pressure at the demand nodes to fully satisfy demands. It follows, therefore, that for a realistic evaluation of the redundancy of a water distribution system, head driven analysis should be carried out. Unfortunately, existing commercial software for simulating network performance is based on demand driven analysis. While this may suffice for normal operating conditions, it cannot cope with abnormal operating conditions when the pressures at some nodes are less than satisfactory.

The aim of this paper is to present a quantified measure of redundancy and to show that the process of assessing overall system performance can be enhanced by using redundancy and reliability together. Both performance indices are calculated using a method based on head driven simulation, which combines the probability that components of the network are operational at any given time and the network's ability to meet consumer demands. A new technique is proposed for the head-dependent modelling of water distribution systems. Notable features of the formulation include the use of a joint head-flow system of equations and the definition of an important new subcategory of nodes called key partial-flow nodes.

2. DEMAND DRIVEN ANALYSIS

The constitutive equations for flow in water supply networks have to simultaneously satisfy nodal flow continuity and conservation of energy applied to each loop or path. Various methods have been used for solving conventional network analysis problems (e.g. Hardy-Cross, Newton Raphson, etc.). The Newton-Raphson method has good convergence characteristics [1] and was, therefore, used in the present formulation.

The continuity equation for each node j, $j=1, \ldots\ldots, NJ$, may be written as

$$\sum_{i:H_i<H_j} Q_{ij} - \sum_{i:H_i>H_j} Q_{ij} = Q_j^{req} \tag{1}$$

where Q_{ij} is the flow in the link ij and NJ is the number of nodes in the network. Q_j^{req} is the required nodal outflow (demand). H_i and H_j are piezometric heads at nodes i and j respectively. The Hazen-Williams equation for headloss in a pipe-link has the form

$$Q_{ij} = K_{ij}^{\,0.54}\left|H_i - H_j\right|^{-0.46}\left(H_i - H_j\right) \tag{2}$$

in which K_{ij} is a resistance coefficient for link ij and has the form

$$K_{ij} = \frac{\alpha \cdot L_{ij}}{CHW_{ij}^{1.852} \cdot D_{ij}^{4.87}} \tag{3}$$

where $\alpha = 10.675$ in S.I. units, L_{ij} = link length, CHW_{ij} = Hazen-Williams coefficient and D_{ij} = diameter.

Eq. (1) can incorporate Eq. (2) to become

$$F_j \equiv \sum_{i:H_i>H_j}\left(\frac{H_i - H_j}{K_{ij}}\right)^{(0.54)} - \sum_{i:H_i<H_j}\left(\frac{H_j - H_i}{K_{ij}}\right)^{(0.54)} - Q_j^{req} = 0 \tag{4}$$

in which F_j represents the continuity equation for node j. Other network components including pumps, non-return valves, flow-control valves and pressure-reducing valves can be included in Eqs. (4) in a similar way [2].

Choosing the nodal piezometric heads as the basic unknown parameters, Eqs. (4) may be solved by the following iterative scheme:

$$J_H^m \underline{\Delta H}^m = -\underline{F}\left(\underline{H}^m\right) \tag{5}$$

$$\underline{H}^{m+1} = \underline{H}^m + \underline{\Delta H}^m \tag{6}$$

in which $\underline{H}$ is the vector of unknown heads, J_H is the Jacobian matrix, $\underline{\Delta H}$ is the vector of the respective corrections to nodal heads and $\underline{F}$ is the vector of respective values of the nodal continuity expressions, i.e. F_j, for $j=1, \ldots\ldots, NJ$. The iteration number is denoted by m.

The elements of the Jacobian matrix, J_H, are given by

$$\frac{\partial F_j}{\partial H_i} = 0.54\left(\frac{\left|H_i - H_j\right|^{-0.46}}{K_{ij}^{\,0.54}}\right) = \frac{\partial F_i}{\partial H_j}; \qquad \forall j, \forall i : i \neq j \tag{7}$$

$$\frac{\partial F_j}{\partial H_j} = -0.54\sum_{\substack{i\in N_j \\ i\neq j}}\left(\frac{\left|H_i - H_j\right|^{-0.46}}{K_{ij}^{\,0.54}}\right); \qquad \forall j \tag{8}$$

where N_j represents nodes connected to node j. To solve the problem computationally, nodal demands are assigned values that are assumed to be fixed. The problem then consists of solving the system of equations to determine the pipe flow rates and nodal pressures that are consistent with the specified demands. This is the traditional method of analysing water distribution systems and is referred to as demand-driven analysis (DDA).

3. HEAD DRIVEN ANALYSIS

Mathematical models that are currently available for the analysis of Water Distribution Systems (WDS) are based on DDA. The underlying assumption for these models is that nodal demands are fixed and can be fully satisfied regardless of the pressure in the system. This method is satisfactory if the nodal heads are sufficient. There are numerous occasions, however, when all the nodal demands cannot be fully satisfied due to pressures in the system being too low. High demands for fire fighting, pump-failures, pipe bursts and demands in excess of design capacity are some of the possible causes of insufficient pressure. A WDS in this state has a reduced quality and level of service. Therefore, simulation of networks with low pressure should be based on Head Driven Analysis (HDA), to take into account the above mentioned aspects. Although some researchers have considered this issue in the past [3-7], computer programs for analysing systems with insufficient pressure in a routine manner are not commercially available. The objective of HDA is to establish the actual supply quantity and pressure at each node in the WDS.

3.1. Brief Review of Methods for Pressure-Driven Network Analysis

Gupta and Bhave [6] made a comparison of various formulae for describing the pressure dependency of nodal consumption and they concluded that the following parabolic relationship [5] was sufficiently accurate.

$$H_j = H_j^{\min} + R_j\left(Q_j^{avl}\right)^{n_j} \tag{9}$$

where H_j is the available head at node j and H_j^{min} is the minimum required head at node j, i.e. the value below which outflow is assumed to be zero or the performance is unacceptable. R_j is a resistance constant and Q_j^{avl} is the available outflow at node j. Generally, the exponent n_j is both node and network specific and often varies between 1.5 and 2 [6]. Thus

$$Q_j^{avl} = Q_j^{req}; \qquad if \quad H_j \ge H_j^{des} \tag{10a}$$

$$Q_j^{avl} = Q_j^{req}\left(\frac{H_j - H_j^{\min}}{H_j^{des} - H_j^{\min}}\right)^{\left(\frac{1}{n_j}\right)}; \quad if \quad H_j^{\min} \prec H_j \prec H_j^{des} \tag{10b}$$

$$Q_j^{avl} = 0; \qquad if \quad H_j \le H_j^{\min} \tag{10c}$$

where Q_j^{req} is the demand at node j and H_j^{des} is the desired head to satisfy the demand.

Ackley et al. [7] used an esoteric mathematical programming formulation based on the maximisation of the sum of the nodal outflows. Bhave [8] proposed a technique which involved assigning categories to all demand nodes and changing them after each DDA simulation according to a predefined scheme. It involved violation of constraints in each iteration with the discrepancy reducing in successive iterations until all constraints were satisfied. The Head Driven Simulation Method (HDSM) by Tabesh [2] is based on the Newton-Raphson technique and explicitly incorporates the head-outflow relationship of Eq. (10) in the continuity equations. The method has a provision to ensure faster convergence, based on a step-length adjustment parameter. Unfortunately, the values of this step-length adjustment parameter and the indices R_j and n_j of Eq. (9) are difficult to ascertain. Not only are they node and network specific but also, their determination requires a considerable amount of effort in field data collection and network calibration [9, 10]. To address the weaknesses of the above-mentioned approaches, a new algorithm for determining head-dependent outflows in WDS is presented in the next section.

3.2. A New Head-Dependent Network Analysis Approach

A detailed study of various deficient networks [6, 11] to assess the effect of variation of outflow at the critical nodes of water distribution systems was carried out. The critical nodes are, generally, critical monitoring points in the network (nodes at isolated high points for regular pressure monitoring), nodes with abnormally high demands, nodes at the extreme ends of the network with respect to the distance water travels from a source to the node and nodes representing areas of persistent low pressure problems. From this study it was observed that a deficient network had four categories of nodes including no-flow nodes, partial-flow nodes, key partial-flow nodes and nodes whose demands are fully satisfied. No-flow nodes are those with pressures below the absolute minimum pressure, H_{min} for outflow to be possible. Partial-flow nodes are those with pressures between H_{min} and H_{res}, the latter being a desirable pressure above which nodal outflow can be fully satisfied. Nodes whose demands are fully satisfied are those nodes with pressures above H_{res}. Key partial-flow nodes (nodes whose outflows affect outflows at other nodes) are generally nodes at isolated high points in the network, and in some cases nodes could fall into this category depending upon the magnitude of their nodal demands and their location in the network.

The main conclusion of the study was that a distribution system in a stressed state experiences a reduction in the nodal outflow, in a decreasing progression, from the most critically affected nodes to the least critically affected ones. A model for carrying out pressure-driven network analysis was developed based on a technique involving a systematic identification of no-flow nodes, partial-flow nodes and key partial-flow nodes as shown in Figure 1 and detailed in the algorithm which follows shortly.

3.2.1 Main features of the proposed model

Subnormal nodal flows ($0 < Q_j^{avl} < Q_j^{req}$) are obtained by converting the head-equations of Eq. (5) to head-flow equations as shown in Eq. (11). Head-flow equations have both nodal flows and heads as the unknown basic variables [8]. Thus

$$J_{HQ}^{m}\begin{bmatrix}\underline{\Delta H}^{m}\\ \cdots\cdots\\ \underline{\Delta Q}^{m}\end{bmatrix}=-\underline{F}\left(\underline{H}^{m},\underline{Q}^{m}\right) \tag{11}$$

and, as in Eq. (6), successive values of the nodal outflows for partial-flow nodes are given by

$$\underline{Q}^{m+1}=\underline{Q}^{m}+\underline{\Delta Q}^{m} \tag{12}$$

in which $\underline{Q}$ is the vector of unknown nodal outflows, and $\underline{\Delta Q}$ is the vector of the respective corrections to nodal outflows.

The respective adjustments in the elements of the Jacobian, J_H, to give the new Jacobian, J_{HQ}, are as follows:

$$\frac{\partial F_j}{\partial Q_k}=-1; \qquad \forall j,\forall k:k=j \tag{13}$$

$$\frac{\partial F_j}{\partial Q_k}=0; \qquad \forall j,\forall k:k\neq j \tag{14}$$

where k is a partial-flow node.

The main driving force of the proposed formulation, called Critical-node Head Driven Simulation Method (CHDSM) is a pre-specified residual pressure, H_{res}, for nodal outflow at a desirable pressure. H_{res} can be set at a minimum nodal residual pressure value according to the terrain, locality served, plumbing arrangements and the general bylaws regarding residual heads. Under certain circumstances, the absolute minimum desired pressure is suggested to be 7m [12].

For purposes of calculating the reliability of a WDS, H_{res} can be set at a desirable pressure below which flow cannot be fully satisfied, with typical values being about 14m to 15m [13, 14] or any other standard value set by a regulatory water supply organisation. The absolute minimum residual pressure, H_{min}, below which no flow is possible at a node, can be set at a bare minimum value or zero. Under certain critical operating conditions, for example bursts in trunk mains, H_{res} can be set to the lowest value of those obtained from pressure loggers put at the critical points in the network and H_{min} can be set at a bare minimum value or zero. The actual pressures and actual outflows can then be obtained for the entire network using the model.

A brief characterisation of some of the key concepts and variables used in the algorithm on which the model is based follows. H_{elevj} is the nodal elevation for node j. $H_{stat j}$ is the static head of node j. No-flow nodes are either nodes with no initial base demands or nodes whose outflows are confirmed and fixed as zero during the course of executing the algorithm. The most critical node and the next most critical node are the nodes with the lowest and second lowest nodal residual pressure in a specified category, X, of nodes and their pressures are denoted by H_{critX1} and H_{critX2} respectively. X takes on the value of N, P and K for no-flow nodes, partial-flow nodes and key partial-flow nodes, respectively. To enhance the computational efficiency, critical nodes in a given category X, whose residual pressures are approximately the same are considered to be in the same pressure contour and are processed together in sets. Thus, the set $X1$ of the most critical node(s) refers to one or more nodes with almost equal pressure values of H_{critX1}. H_{critN1} and H_{critN2} (where $H_{critN1} < H_{critN2}$) represent the respective pressure values of critical nodes in the same pressure contours during the stage of identifying no-flow nodes; the sets of nodes are designated sets $N1$ and $N2$ respectively. $H_{critP1}, H_{critP2}, \ldots, H_{critPn}$ (where $H_{critP1} < H_{critP2} \ldots, < H_{critPn}$) represent the respective pressure values of critical nodes in the same pressure contours during the stage of identifying partial-flow nodes and the sets of nodes are designated sets $P1, P2, \ldots, Pn$ respectively. Similarly, $H_{critK1}, H_{critK2}, \ldots, H_{critKn}$ (where $H_{critK1} < H_{critK2} \ldots, < H_{critKn}$) represent the respective pressure values of critical nodes in the same pressure contours during the stage of identifying key partial-flow nodes and the sets of nodes are designated sets $K1, K2, \ldots, Kn$ respectively. No-flow nodes do not belong to any of the $P1, \ldots, Pn$ and $K1, \ldots, Kn$ sets.

3.2.2 Algorithm for head-dependent WDS modelling

Part I: Identification of no-flow demand nodes

1) Given nodal demands, assume initial heads, H_j for all nodes other than fixed head nodes.
2) Calculate the nodal heads using DDA (Eqs. (5) and (6)).
3) Identify all nodes whose static heads are less than their respective minimum heads, ($H_{elevj}+ H_{min}$). Fix the demands at these no-flow nodes to zero and perform step 2.
4) Identify the most critical node of all non-zero demand nodes. If its pressure, H_{critN1} is less than H_{res}, then this node, together with any other nodes in the same pressure contour, should be taken as the critical nodes (designated set $N1$). Otherwise, exit.
5) Set the demands of the node(s) in set $N1$ to zero, and perform step 2.
6) If the pressure H_{critN1} of the node(s) in set $N1$ is less than or equal to H_{min} and the pressure, H_{critN2}, of the next most critical node is less than H_{res}, confirm the nodes in set $N1$ as no-flow nodes by fixing their demand values to zero, and return to step 2. Otherwise, the nodes of set $N1$ are categorised as partial-flow nodes. Go to Part II.

Part II: Identification of partial-flow nodes

1) The set of partial-flow nodes should be designated set $P1$. If H_{critP1} of the node(s) in set $P1$ is less than H_{res}, all nodes with pressures between H_{min} and H_{res} should be categorised as partial-flow nodes with heads, $H_{critP1}, H_{critP2}, H_{critP3}, H_{critP4}, \ldots, H_{critPn}$ and grouped together with nodes of the same pressure contours in sets $P1, P2, P3, P4, \ldots, Pn$ respectively. Set their pressures as $0.5(H_{critP1}+ H_{min})$ for the node(s) in set $P1$, $H_{min}+0.75(H_{critP1}-H_{min})$ for the node(s) in set $P2$, $0.5(H_{critP2}+ H_{critP3})$, $0.5(H_{critP3}+ H_{critP4})$, ..., and $0.5(H_{critPn-1}+ H_{critPn})$, for nodes in sets $P3, P4, \ldots, Pn$, respectively. In effect, these are averages between consecutive pressure values. Otherwise, (i.e. if H_{critP1} is greater than or equal to H_{res}) set their pressures to H_{res}.
2) Convert the system of head-equations in Eq. (5) into a system of head-flow equations as shown in Eq. (11). Solve Eq. (11) and update the nodal heads and flows using Eqs. (6) and (12) respectively (i.e. HDA).
3) Proceed to Part III.

Part III: Identification of key partial-flow nodes

1) The set of partial-flow nodes in $P1$ should be designated set $K1$.
2) H_{critK2} is the lowest nodal residual pressure amongst pressures of nodes that are not no-flow nodes and do not belong to sets $P1, \ldots, Pn$. If H_{critK2} is less than H_{res}, the outflow (Q_{critK2}) of this node together with outflows of any other nodes in the same pressure contour should be set to zero, the nodes designated set $K2$ and then step 2 in Part I performed. Otherwise, exit.
3) If the pressure of the node(s) of set $K2$ is less than H_{min}, confirm them as no-flow nodes by fixing their outflows to zero, otherwise go to step 4. Set the pressure of nodes of set $K1$ to H_{res} and perform step 2 in Part II. Go to step 2.
4) For the two sets of key partial-flow nodes $K1$ and $K2$, set the pressure of nodes in set $K1$ to H_{res} and that of nodes in set $K2$ to ($H_{res}+\varepsilon$), where ε is a small tolerance of about 0.05m. Perform step 2 in Part II.

5) Get the next most critical node of the non-zero demand nodes that do not belong to sets $K1$, ..., Kn-1. If its pressure, H_{critKn}, is less than H_{res}, then this node together with any other nodes in the same pressure contour should be designated the n[th] set of key partial-flow nodes, Kn. The flows of all key partial-flow nodes should be obtained using step 2 in Part II, by setting the pressure of nodes in the latest set, Kn, to $(H_{res} + \varepsilon)$, that of nodes in the first set, $K1$, to H_{res}, and setting the pressures of the remaining sets using a constant increment of $\varepsilon/(n-1)$.
6) Repeat step 5 until there are no more key partial-flow nodes.
7) End program and exit.

3.4 Model Verification

Example 1

A pressure-deficient looped network (Figure 2) with 16 designs [2, 11] was chosen to assess the proposed model. The pipe data are presented in Table 1. All links are 1000m long with CHW = 130. The source node has a fixed head of 100m and all elevations of demand nodes are 0m. The pre-specified nodal residual head, H_{res}, was set to a desirable head of 15m [15] and the absolute minimum residual head, H_{min}, was taken as zero. A comparison of the proportions of the total network demand satisfied by the fully connected networks is presented in Figure 3. The difference in total flow between HDSM and CHDSM is only 1%. Other aspects worth emphasising are the computational efficiency and the simplicity of the present formulation. CHDSM results were generated for each of the sixteen designs in Example 1 using a FORTRAN 90 program on a 400MHz Ultra Sparc Sun system. For each design the CPU time required was approximately 0.26 seconds. A single DDA run for the same network with a higher source head to ensure that all demands are satisfied requires a CPU time of about 0.12 seconds.

Example 2

CHDSM was also applied to the serial network shown in Figure 4 [6]. The lengths and the Hazen-Williams coefficients for all pipes are 1000m and 130 respectively. The diameters of pipes 1 through 4 are 400mm, 350mm, 300mm and 300mm respectively. The outlet elevations for nodes 1 through 4 are 90m, 88m, 90m, and 85m respectively, were taken as the respective minimum nodal heads H_j^{min}. The demand nodes 1 through 4 have required flows Q_j^{req} of 2m³/min, 2m³/min, 3m³/min and 4m³/min, respectively. H_j^{des} values were obtained using Eq. (9) with $H_j = H_j^{des}$, $Q_j^{avl} = Q_j^{req}$, $R_j = 0.1$ and $n_j = 2$ [9]. In order to confirm the robustness and accuracy of the CHDSM algorithm, the source head was varied from 110.89m down to 85m to simulate a range of operating conditions with different pressure regimes. The total supply to the network and the available nodal flows obtained by CHDSM are presented in Figure 5 along with the results of Gupta and Bhave [6] for comparison purposes. Comparing these results, the total supply to the network is exactly the same while the curves for the nodal flows are very similar.

Example 3

CHDSM was applied to the deficient multiple-source network shown in Figure 6. The lengths and the Hazen-Williams coefficients for all pipes are 1000m and 130 respectively. The source nodes are S1 and S2 with heads of 68m and 60m respectively. Pipes S1-S2, 4-5 and 4-6 each have a diameter of 50mm; pipes S1-3 and 3-5 have a diameter of 250mm, pipe S2-4 has a diameter of 80mm and pipe 5-6 has a diameter of 150mm. The ground elevation for all nodes is zero. The demand for each of the nodes 3 and 4 is 10l/s; that for node 5 is 30l/s and for node 6 is 45l/s. H_{min}, the absolute minimum residual head for flow to be possible at a node was taken as 0m, while H_{res}, the residual head for nodal outflow at a desirable pressure, was set at 15m [15]. Table 2 presents the demand driven analysis results generated by both CHDSM and those from the well-known software package EPANET2 [16] for comparison purposes. The values match closely. The head driven analysis results from CHDSM are also shown in the table. EPANET2 was used to check that the CHDSM results were both hydraulically consistent and feasible. In this regard, the CHDSM predictions of nodal outflows were used as the demands in EPANET2 in order to obtain nodal heads corresponding to these outflows. These DDA simulations using EPANET2 gave values of nodal heads that are the same as those of CHDSM. The CPU time required for the head driven analysis was 0.2 seconds. It can be seen that there is insufficient pressure in the network to satisfy the demands in full as the flow that is actually delivered at node 6 is less than the demand.

4. RELIABILITY

A network with insufficient pressure experiences reduced outflow from some of the nodes. To be able to assess the performance of the network in a realistic way, this stressed condition needs properly to be accounted for using head-driven analysis. The results of the simulation can then be combined with the probability that the network will be in a particular full or reduced state in terms of the availability of components. Based on the common assumption that pipe failures or unavailabilities are independent, the probability, $p(0)$, that no pipe is unavailable is

$$p(0)=\prod_{l=1}^{NL} a_l \tag{18}$$

in which a_l is the probability that link l is available and NL is the number of links. Pipe availability can be taken as the ratio of the mean time between failures to the sum of the mean time between failures and the failure duration. For example, this can be calculated using the formula developed by Cullinane et al. [17].

Taking only one and two unavailable components into consideration, and assuming the demand is constant, the network reliability, R, which has been defined in Section 1, is given by [18]

$$R=\frac{1}{Q^{req}}\left(p(0)Q(0)+\sum_{l=1}^{NL}p(l)Q(l)+\sum_{\substack{l=1\\ m=l+1}}^{NL-1}p(l,m)Q(l,m)\right)$$
$$+\frac{1}{2}\left(1-p(0)-\sum_{l=1}^{NL}p(l)-\sum_{\substack{l=1\\ m=l+1}}^{NL-1}p(l,m)\right) \tag{19}$$

in which: $p(0)$ is the probability that no link is unavailable, $p(l)$ is the probability that only link l is unavailable and $p(l, m)$ corresponds to the probability that two components l and m are unavailable. $Q(0)$, $Q(l)$, and $Q(l, m)$ are the respective actual total outflows when zero components, components l and, l and m are unavailable while Q^{req} is the total demand for the network. In this study, values of $Q(0)$, $Q(l)$, and $Q(l, m)$ were obtained as described in Section 3. Herein, the nodal demands are taken as constants. In practice, however, water consumption varies in a random fashion. The incorporation of variations in demands in the reliability assessment of water distribution systems is currently an area of active research.

5. REDUNDANCY

Redundancy is the existence of alternative pathways from the sources to demand nodes or excess capacity in normal operating conditions, for use when components become unavailable. To ensure an uninterrupted albeit reduced supply of water, distribution network designs should include some amount of redundancy. Conventionally, redundancy is assumed to be present if the network has many loops. The interaction between supply paths, the degree to which various paths contribute to the supply of a node and the multiplicity of paths are factors that complicate the assessment of redundancy. Any parameter used as a measure of redundancy should recognise these factors [19].

The formula proposed herein for the above property of redundancy, T, which has been defined in Section 1, is [20]

$$T=\frac{R-r(0)p(0)}{1-p(0)} \tag{20}$$

in which $r(0)$ is the ratio of available flow to the required flow when all network components are available. Redundancy is thus defined as the expectation of the proportion of the demand of the network that is satisfied during the periods in which there are mechanical failures in the system or when some components are taken out of service for repair or maintenance. A key feature of Eq. (20) is that its computation is straightforward once R, $r(0)$ and $p(0)$ have been calculated in the reliability evaluation process, since additional hydraulic simulations are not required. Values of the proposed redundancy measure lie between 0 and 1 [21].

The formulation is based on a rigorous unified probability framework encompassing all components of the distribution system, which either require periodic maintenance or can experience a mechanical or electrical failure. As such, the present redundancy-based performance indicator can be expected to reflect the extent to which a water distribution system is vulnerable to the unavailability of components (pipes, pumps, etc.). At one extreme, the redundancy value should approach 1.0 for invulnerable, highly redundant networks with a lot of spare capacity or oversized pipes and other components. On the other hand dendritic networks, which are generally more vulnerable, can be expected to have much lower redundancy values. Two examples, a tree-type network and a looped network, are used in the next section to show that Eq. (20) is indeed an adequate measure of the redundancy.

It may be noted that the reliability value R is primarily a measure of the performance of the distribution system under normal operating conditions. This is because the availabilities of the individual components are generally high. By contrast the redundancy T is concerned solely with degraded network configurations. It is therefore important that the reliability value be calculated accurately by accounting for as many less-than-fully connected network states as possible. The reliability formulation of Eq. (19) is particularly useful in this regard because it incorporates an element which improves the accuracy of the reliability value significantly by averaging estimates of its upper and lower bounds [18]. It is worth emphasising that, in general, the inclusion of network states corresponding to all the possible combinations of unavailable components in Eq. (19) will be impracticable for the majority of urban systems. Finally, a similar formula to Eq. (20) can be written for each node by replacing R and $r(0)$ with corresponding values for the node. The values of the resulting nodal redundancy parameter would identify the degree of vulnerability of the individual nodes.

6. DEMONSTRATIONS USING SAMPLE NETWORKS

To demonstrate the appropriateness of the redundancy measure, three networks were analysed with up to two components simultaneously unavailable. This implies that each network was analysed with different combinations of unavailable links in order to obtain $Q(0)$, $Q(l)$ and $Q(l, m)$, these being the respective actual total outflows when zero components, component l and components l and m are unavailable. The individual (see Section 4) pipe availabilities were calculated based on their diameters using the formulation of Cullinane et al., [17]. The results were then used to calculate the $p(0)$, $p(l)$ and $p(l, m)$ values. Finally, R and T were calculated using Eqs. (19) and (20). The lengths and the Hazen-Williams coefficients for all pipes are 1000m and 130 respectively.

6.1 Case 1: A pipeline

The network is shown in Figure 4. As observed previously, the source head was varied to correspond to a range of reservoir conditions. It is worth noting that below a source head of 110.89m the network is stressed in that the source head is not sufficient to satisfy all the nodal demands. EPANET2 [16] was used to check that the CHDSM results were both hydraulically consistent and feasible. The CHDSM predictions of nodal outflows were used as the demands in EPANET2 in order to obtain the source heads required to satisfy these outflows. These DDA simulations using EPANET2 gave the same values of the required head at the source node as the actual source head (Figure 7).

Figure 7 suggests that it may be more difficult to improve redundancy than reliability in a cost-effective way, especially for branched networks. The figure shows that the network has a low redundancy level of less than about 0.35, irrespective of the source head value, because alternative supply paths are not available. This vulnerability through lack of redundancy may not necessarily be obvious to an inexperienced designer, if the redundancy value is not calculated explicitly. This quantified measure of redundancy should, therefore, be used alongside reliability for network performance assessment. For instance, consider two reservoir conditions with source heads of 98.5m and 110.89m respectively. The associated redundancy value is 0.325 for both conditions and the reliabilities are 0.7270 and 0.9995, respectively. The redundancy values are equal due to the nature of the network layout. Being a pipeline, it depends largely on connectivity such that above a certain source head, the expectation of total flow delivered is the same. On the other hand, the reliabilities vary mainly due to the varying total outflow for the various reservoir conditions as indicated by the trends and closeness of the reliability and proportion of total flow delivered, $r(0)$ (Figure 7). The redundancy value clearly shows that the network is highly vulnerable to component failure in both cases, a fact that could undermine the higher reliability value of the latter case. This would not necessarily be obvious if the reliability values alone were to be considered.

6.2 Case 2: A Simple Network

Several researchers have used the two–loop network in Figure 8. The pipe data for six candidate designs are presented in Table 3 [20]. The source node has a fixed head of 35m, and all elevations of demand nodes are 0m. H_{min}, the absolute minimum residual head for flow to be possible at a node was taken as 0m, while H_{res}, the residual head for nodal outflow at a desirable pressure, was set at 15m [15]. The proposed model was used to obtain a range of performance data as shown in Figure 9.

The graphs suggest, as expected, that reliability or overall performance will improve if mechanically more reliable components are used to ensure that the frequency of failures is small. However, if redundancy is not considered explicitly, then serious problems may result when components are taken out of service for maintenance purposes. As expected, there is a general conformity in the trend of network reliability and the probability, *p(0)*, that all links are available. Nevertheless, redundancy does not follow the same trend. Redundancy values largely depend on the impact of the unavailability of individual components [22]. Therefore, for a more comprehensive performance appraisal, reliability and redundancy should be used together.

6.3 Case 3: Multiple-source Network

To further confirm the robustness and computational efficiency of CHDSM, it was applied to the multiple-source network in Figure 5 to obtain a range of performance data as shown in Figure 10. The source heads were varied for six different scenarios for S1 and S2 from 28m and 20m respectively, for Scenario 1 when the network had low pressure conditions, increasing each source head by 10m for each operational scenario to improve the pressure conditions. Thus, the respective source heads for S1 and S2 for Scenario 2 were 38m and 30m; for Scenario 3, 48 and 40; and so on up to Scenario 6 when the network could fully satisfy nodal demands with source heads of 78m and 70m for S1 and S2, respectively. The pipe and nodal data together with values for H_{min} and H_{res} are the same as in Example 3.

The graphs suggest that there is a considerably high level of reliability and failure tolerance with values being higher than 0.5 in all cases, probably due to the fact that the network has 2 sources and is looped. This means that there are alternative supply paths, which allow a portion of the demand to be satisfied when pipes become unavailable. Figure 10 shows that there is a general increasing trend in the reliability, redundancy and the proportion of total demand satisfied *r(0)* from Scenario 1 to 6. However, a closer look shows that the rate of increase of redundancy as the source heads increase (from Scenario 1 to 6), is lower than that of reliability. Once again, it is important to use reliability and redundancy together for network performance appraisal. The typical CPU time required to obtain the reliability, which in this case involved a total of 29 full and degraded network configurations, is about 2.5 seconds. This confirms the computational efficiency of the technique and its applicability to multiple-source networks. The technique is robust in that feasible results are obtained even when the source heads are low.

7. CONCLUSIONS

A quantified measure for assessing the redundancy of water distribution networks has been presented. Using examples, its suitability has been demonstrated. The importance of adopting both redundancy and reliability as the basis for assessing system performance has also been demonstrated.

The redundancy model is based on an efficient technique for determining head-dependent outflows in single- and multiple-source normal and pressure-deficient WDSs. The FORTRAN program for system performance assessment automatically considers the randomness of component unavailability in calculating hydraulic performance indices. The introduction of the head-flow equations for determining partial-flow neither alters the basic structure and size of the Jacobian nor leads to an increase in the number of basic unknowns. Thus, convergence is attained after few additional iterations. The computational efficiency and robustness of the technique has been demonstrated.

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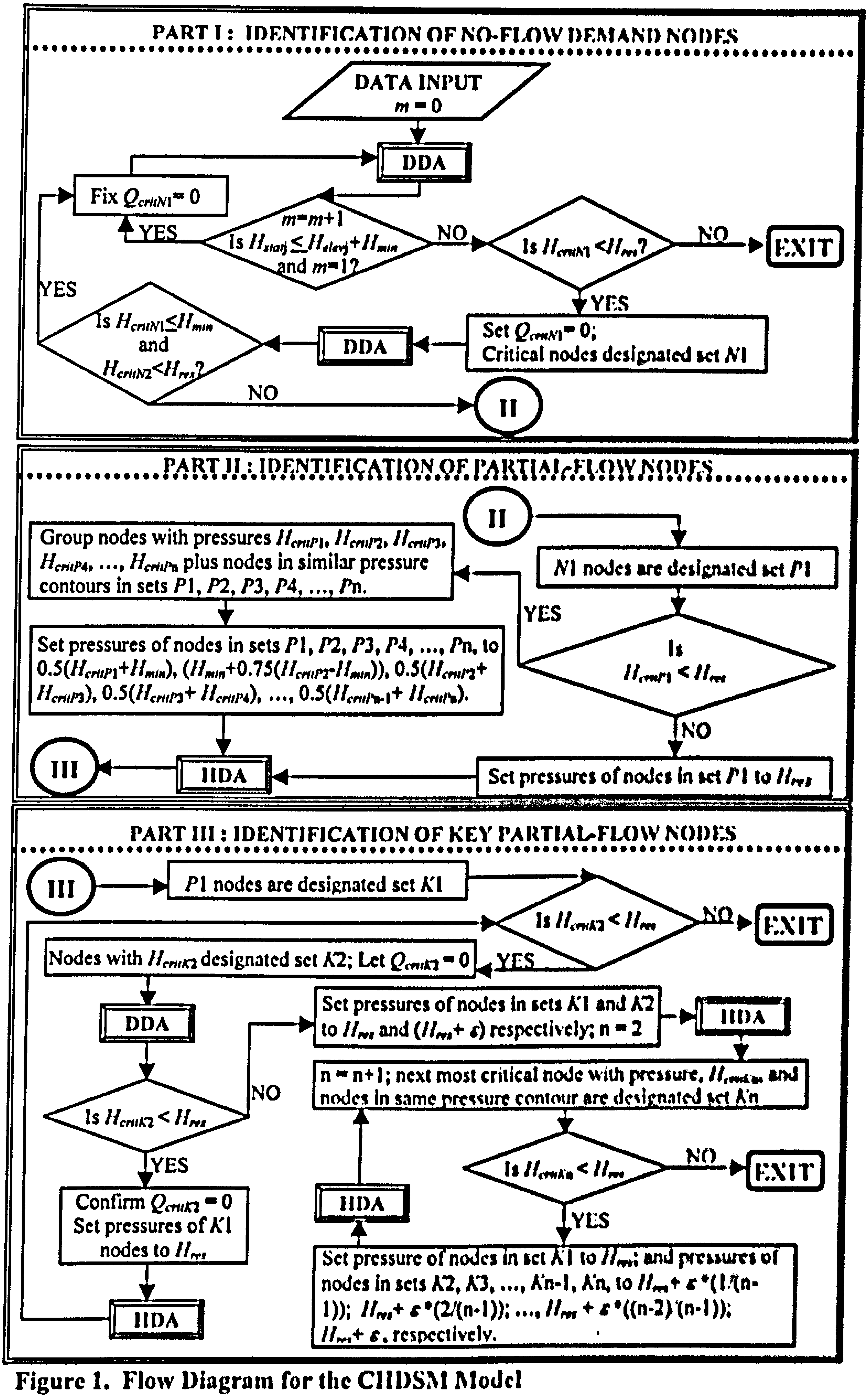

Figure 1. Flow Diagram for the CHDSM Model

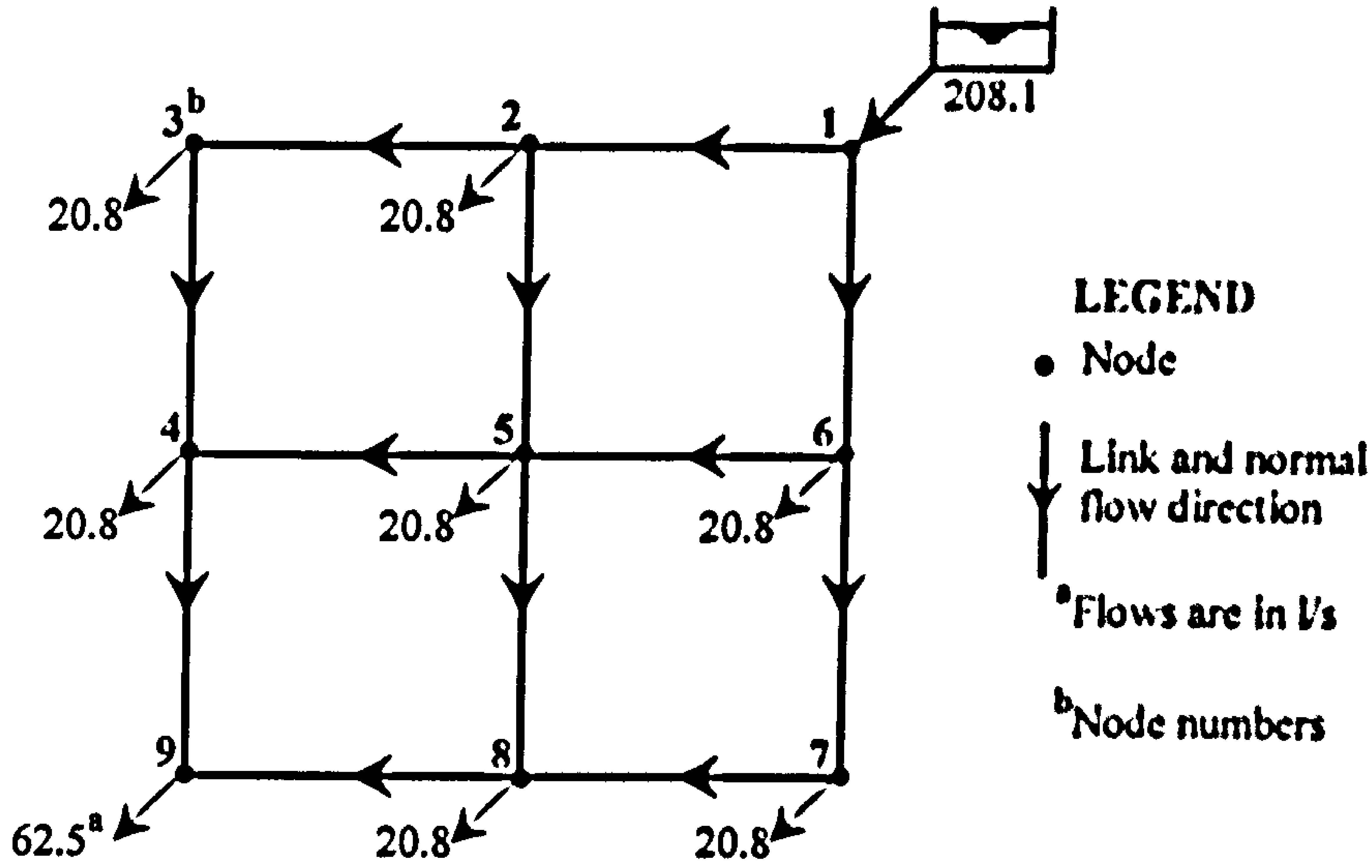

Figure 2. Looped Network

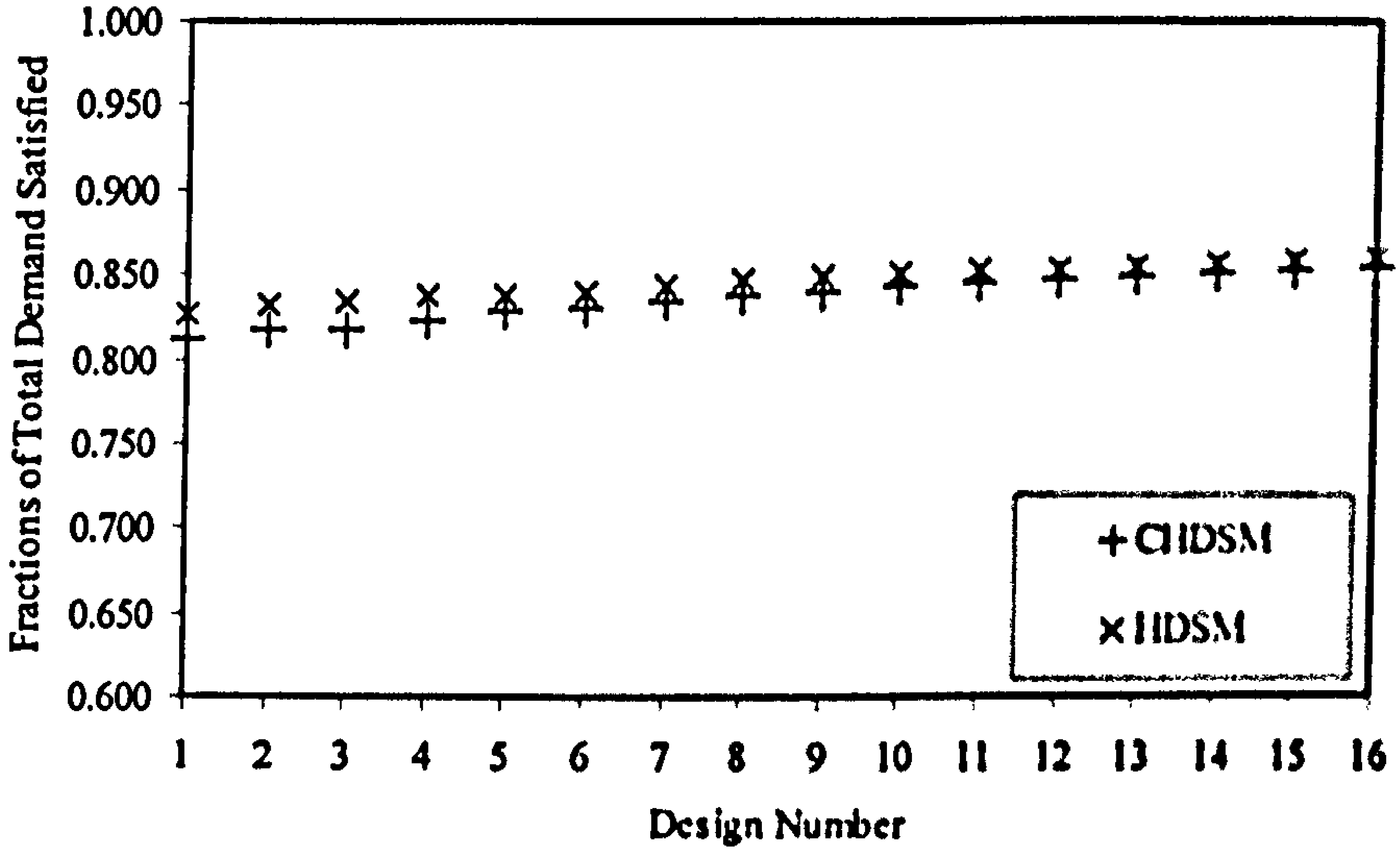

Figure 3. Fractions of Total Demand Satisfied by Fully Connected Networks

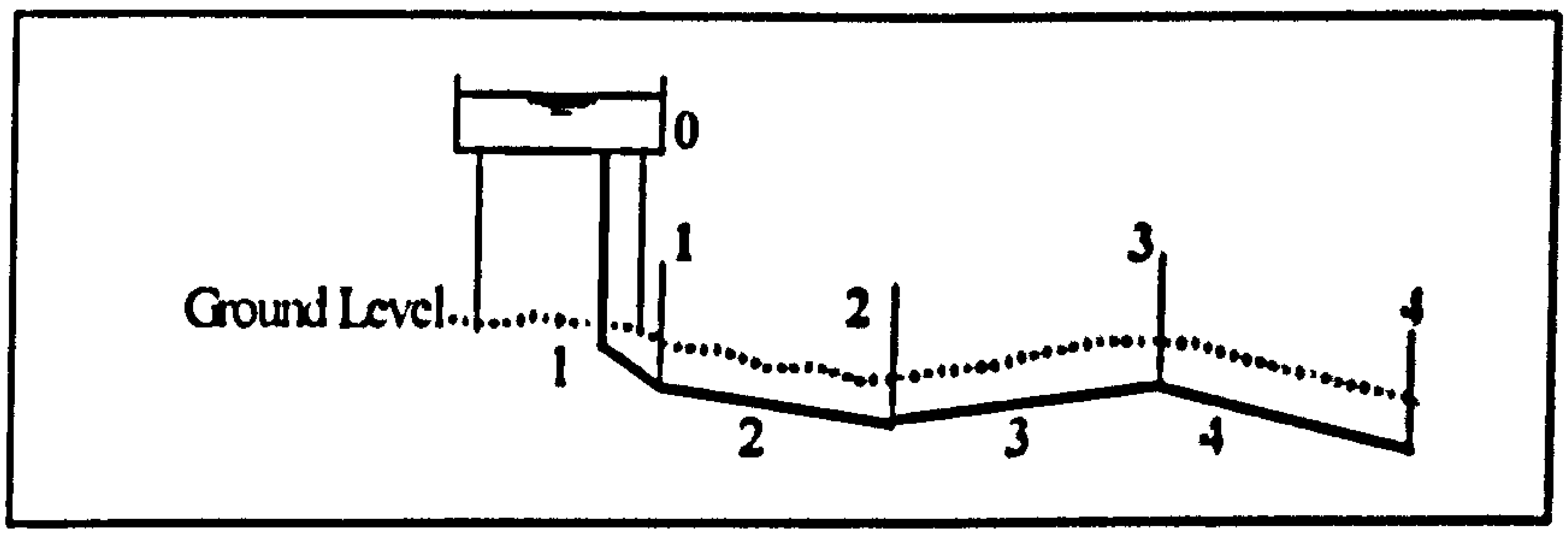

Figure 4. Serial Network

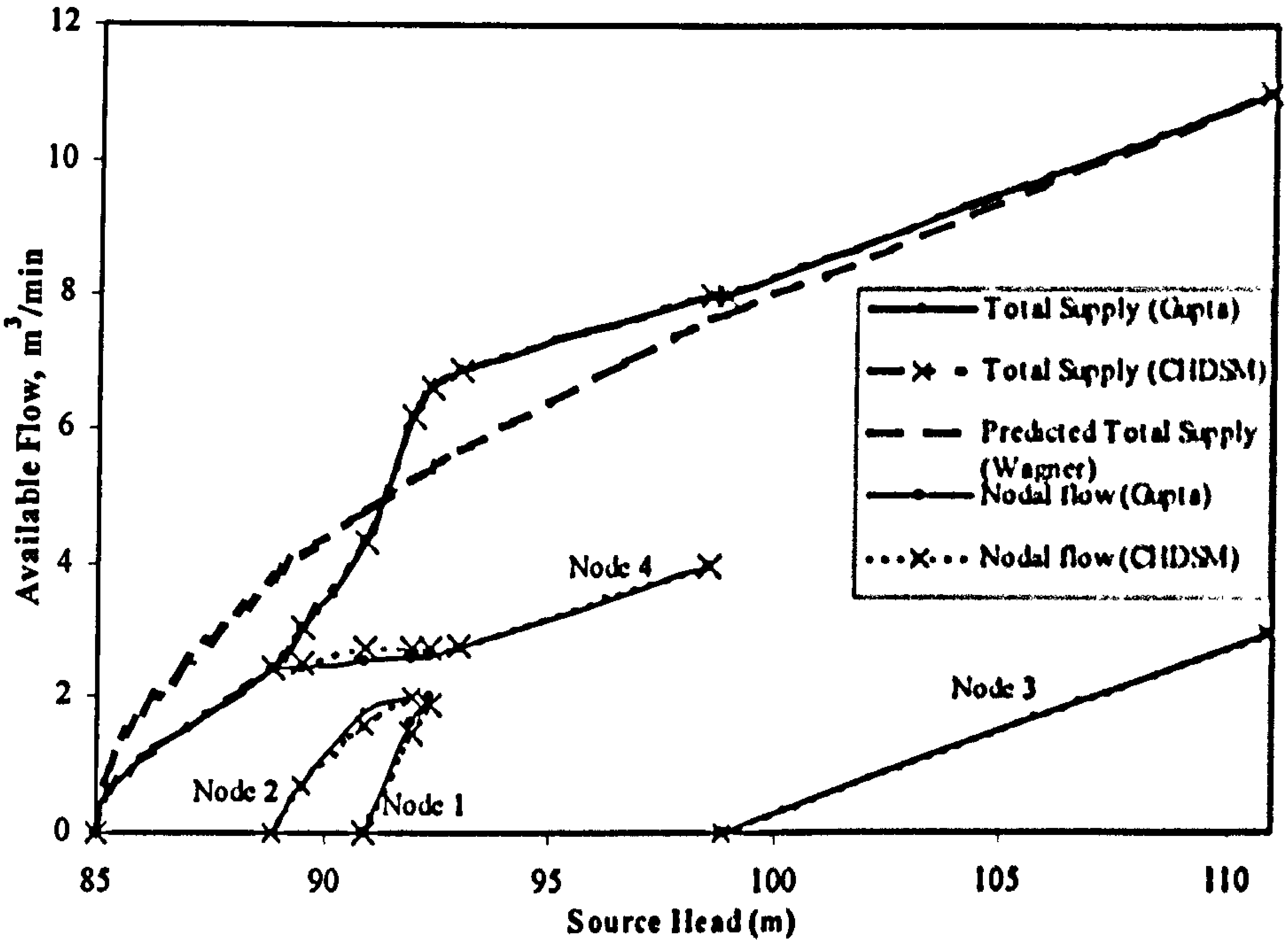

Figure 5. Variation of Available Flow for Different Source Heads

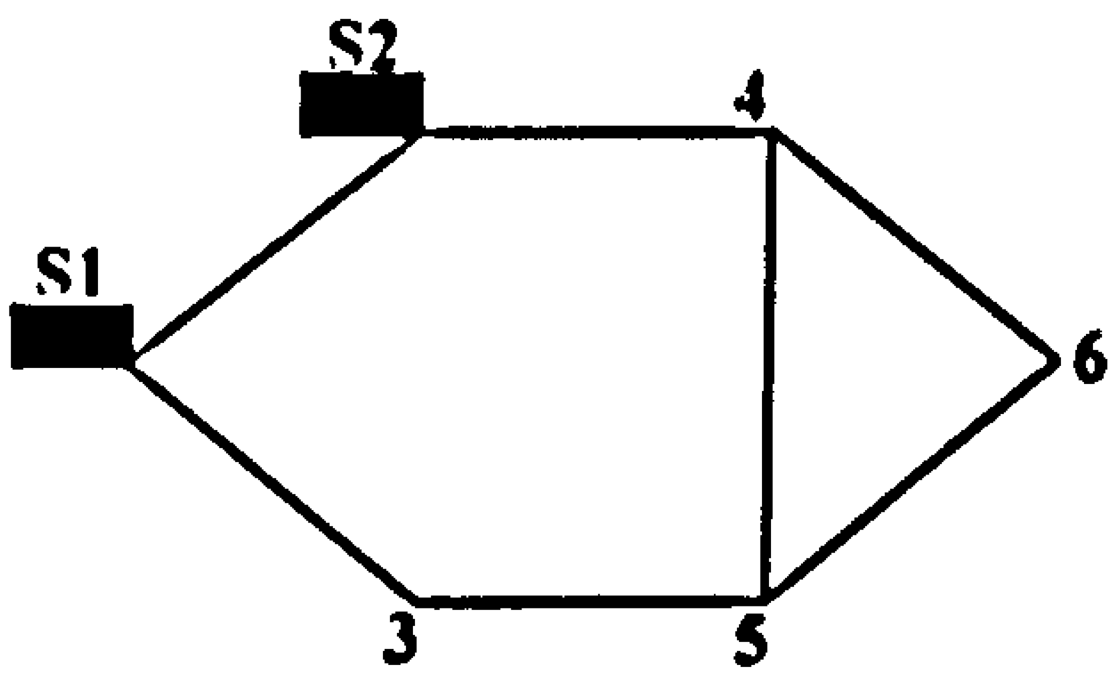

Figure 6. Multiple-source Network

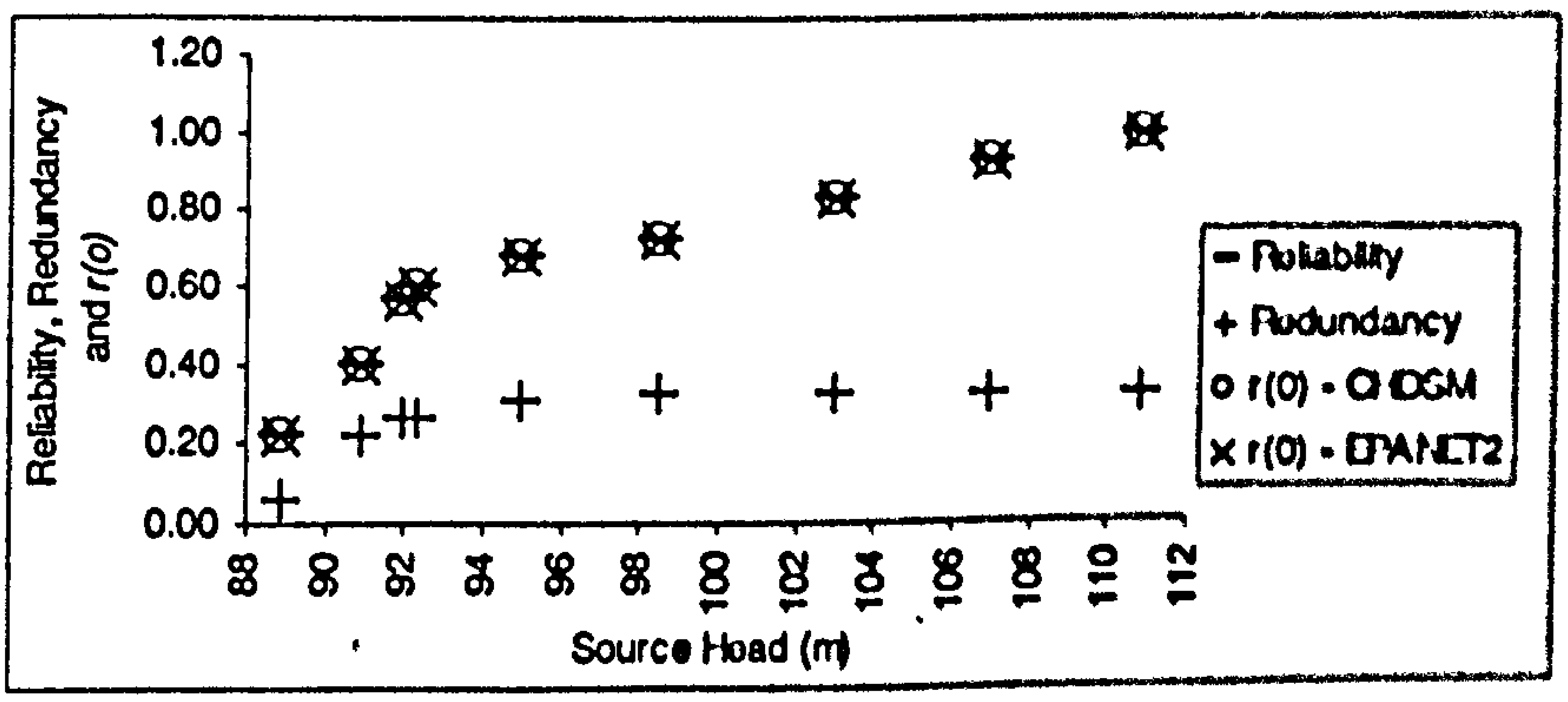

Figure 7. System Performance for a Range of Source Heads

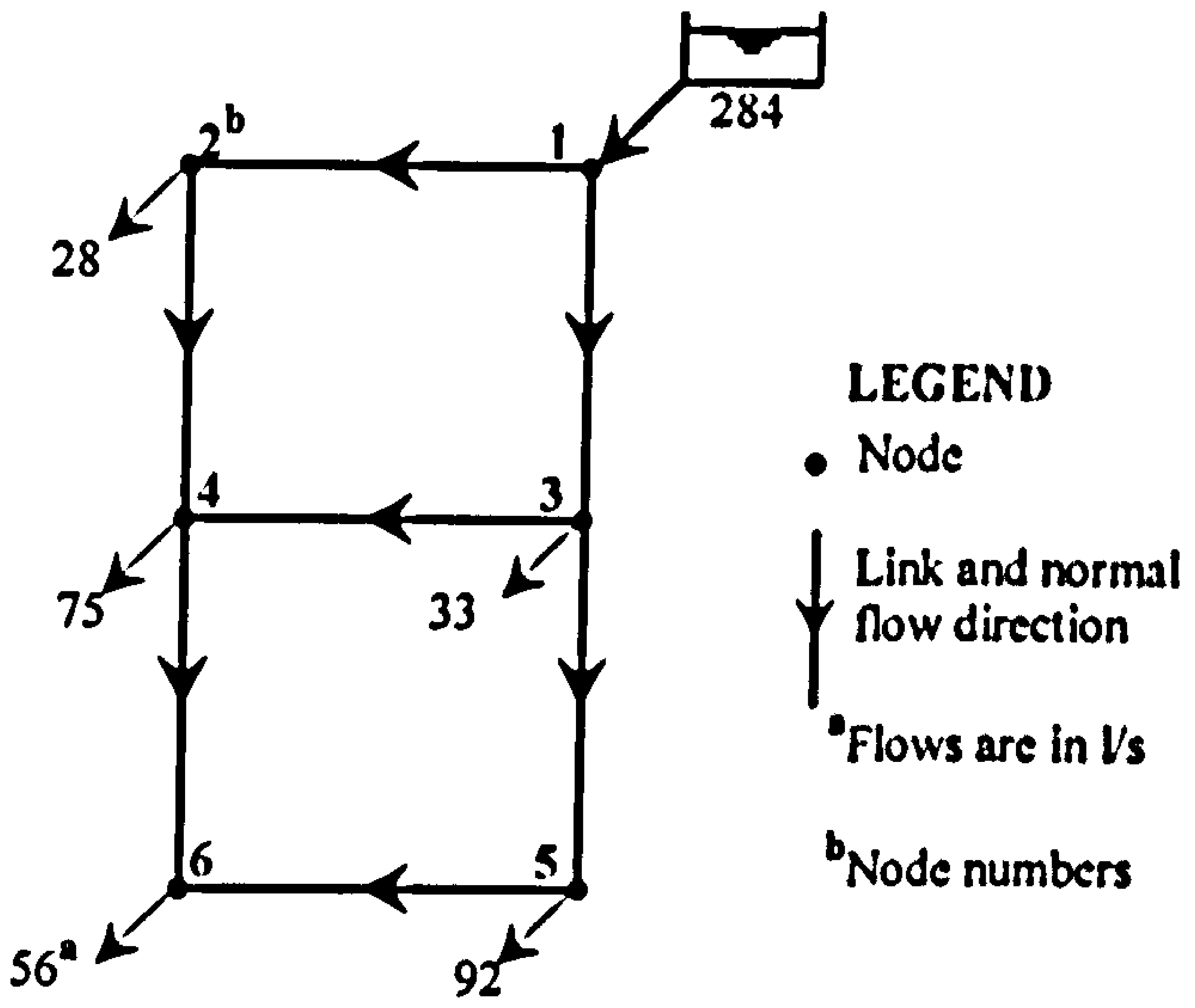

Figure 8 Two-loop Network

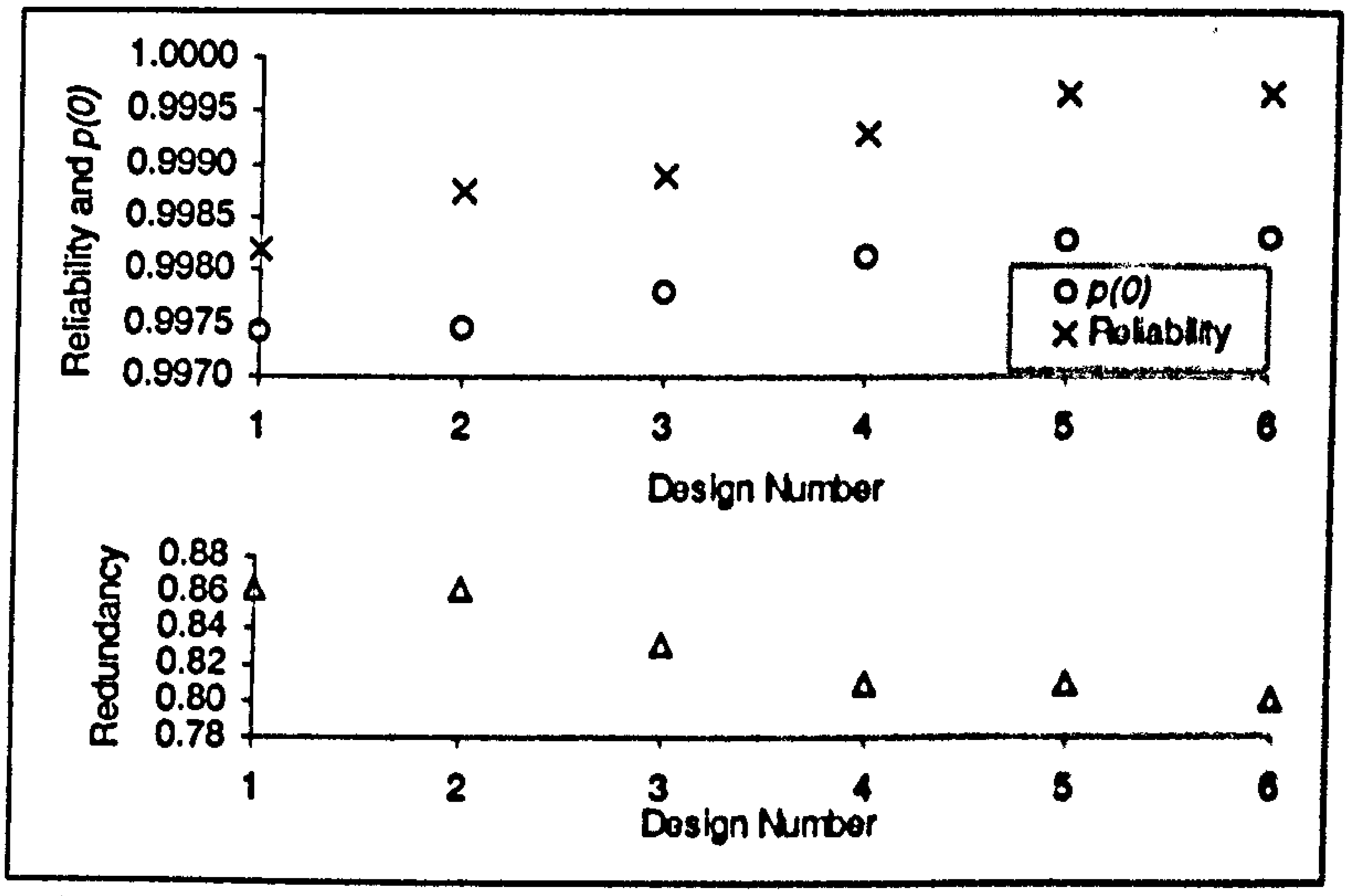

Figure 9 Performance Measures for Alternative Designs

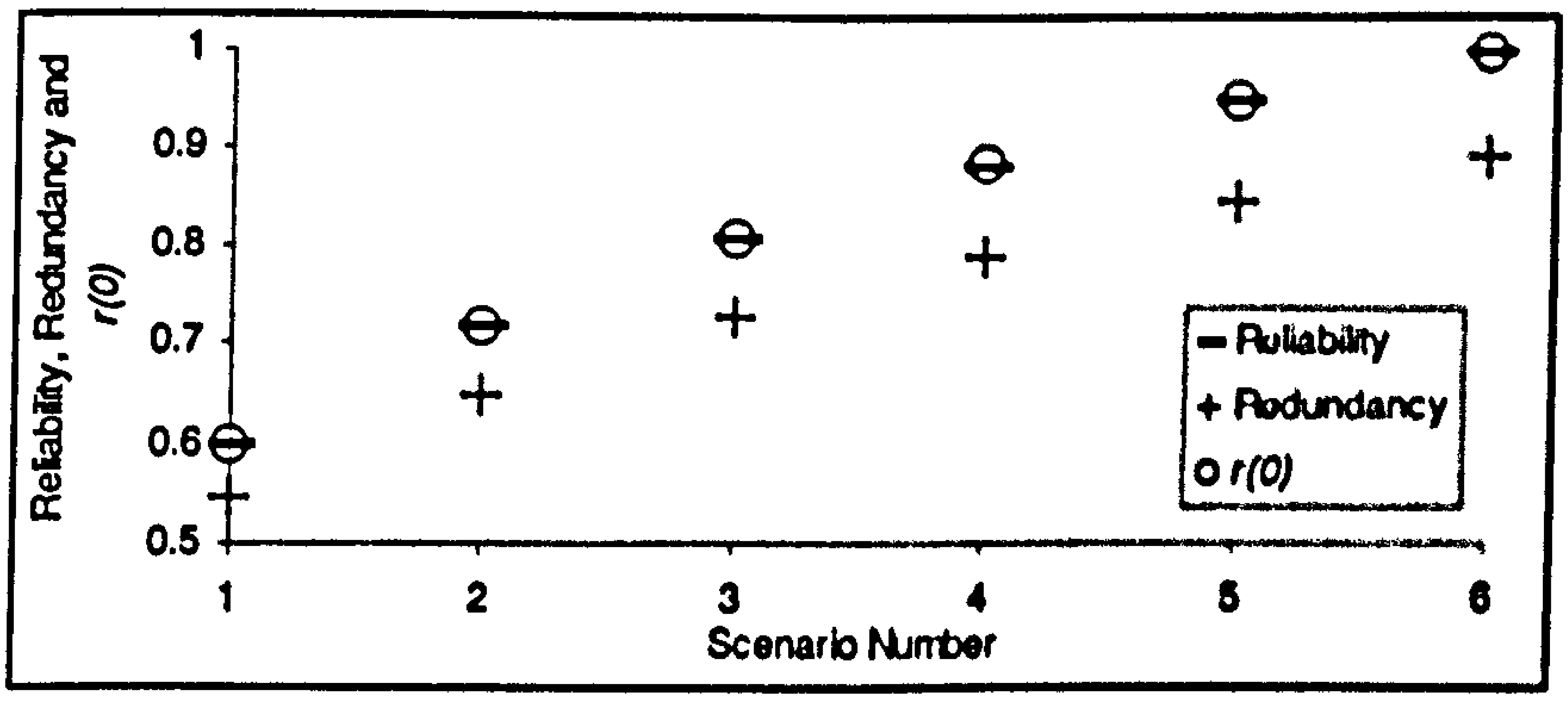

Figure 10. Multiple-source Network: System Performance for a Range of Source Heads

Table 1: Pipe Data

Links	Diameters (mm)															
1-2, 1-4	250	250	250	250	250	250	250	250	250	250	250	250	250	255	255	255
2-3, 4-7	175	175	180	180	180	185	185	185	190	190	190	190	190	190	190	190
2-5, 4-5	145	145	145	145	145	145	145	145	145	145	145	150	150	150	155	155
3-6, 7-8	115	115	115	120	125	125	130	135	135	140	140	140	140	140	140	140
5-6, 5-8	100	105	105	105	105	105	105	105	105	105	110	110	115	115	115	120
6-9, 8-9	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100
Designs	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16

Table 2. Multiple-source Network Results

Nodes	DEMAND DRIVEN ANALYSIS			HEAD DRIVEN ANALYSIS		EPANET2 Output for Feasibility Check: Total Heads (m)
	Demand (l/s)	Total Heads (m)		CHDSM		
		CHDSM	EPANET2	Total Heads (m)	Outflows (l/s)	
S1	-	68.00	68.00	68.00	-82.73	68.00
S2	-	60.00	60.00	60.00	-7.50	60.00
3	10.00	56.17	56.16	57.26	10.00	57.25
4	10.00	14.47	14.41	18.11	10.00	18.06
5	30.00	46.76	46.75	48.83	30.00	48.82
6	45.00	5.80	5.73	15.00	40.23	14.96

Table 3. Candidate Designs for Two-loop Network

Links	Diameters (mm)					
1-3	401	401	390	384	365	367
2-4	100	100	165	191	238	235
3-5	338	337	337	329	281	294
4-6	100	100	100	151	250	234
5-6	263	262	262	249	152	185
1-2	157	165	203	224	263	261
3-4	237	237	213	215	247	234
Designs	1	2	3	4	5	6