LEAKAGE CONTROL BY OPTIMAL VALVE OPERATION

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Approval of the thesis:

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submitted by **TÜLAY ÖZKAN** in partial fulfillment of the requirements for the degree of **Doctor of Philosophy in Civil Engineering Department, Middle East Technical University** by,

Prof. Dr. Canan Özgen
Dean, Graduate School of **Natural and Applied Sciences** ____________________

Prof. Dr. Güney Özcebe
Head of Department, **Civil Engineering** ____________________

Assoc. Prof. Dr. Nuri Merzi
Supervisor, **Civil Engineering Dept., METU** ____________________

Examining Committee Members:

Prof. Dr. Süha Sevük
Civil Engineering Dept., METU ____________________

Assoc. Prof. Dr. Nuri Merzi
Civil Engineering Dept., METU ____________________

Prof. Dr. Selçuk Soyupak
Civil Engineering Department, AU ____________________

Prof. Dr. Melih Yanmaz
Civil Engineering Dept., METU ____________________

Assist. Prof. Dr. Hürevren Kılıç
Computer Engineering Dept., AU ____________________

Date: : ______________

I hereby declare that all information in this document has been obtained and presented in accordance with academic rules and ethical conduct. I also declare that, as required by these rules and conduct, I have fully cited and referenced all material and results that are not original to this work.

Name, Last Name : Tülay Özkan

Signiture :

ABSTRACT

LEAKAGE CONTROL BY OPTIMAL VALVE OPERATION

Özkan, Tülay
Ph.D., Department of Civil Engineering
Supervisor: Assoc. Prof. Dr. Nuri Merzi

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The main function of a water distribution system is to supply water in sufficient quantity at appropriate pressure with an acceptable quality and as economically as possible. Water leakage in distribution networks may account from 5% to 50% and even larger of the total water delivered. The amount of leakage in a network is directly related to system service pressure. Therefore, reductions in high service pressures will result in considerable reductions in leakage. A methodology for leakage reduction has been presented in context of a developed computer program, LEAKSOL with two sub-programs. The first code, CODE I, provides solution by using optimization techniques with defined pressure-leakage and pressure-demand relations in order to find optimal flow control valve settings minimizing water leakage. The second one, CODE II, makes hydraulic analysis of the network in order to solve the system and to compute the amount of leakage and the amount of water consumed, by using different combinations of isolation valves generated according to the number of valves given and employing the relationships among pressure, leakage and consumption. Computer program application was performed for different scenarios in a sample network previously used in literature and also in N8-3 pressure zone of Ankara Municipal Water Supply System. Leakage reduction up to 10 % has been achieved in N8-3 pressure zone for eight valves located at the entrances of sub-zones, depending on the defined pressure-leakage relationship.

Keywords: Water Leakage, Pressure-Leakage Relationship, Pressure Dependent Demand, Valve Operation, Leakage Reduction.

ÖZ

OPTİMAL VANA OPERASYONLARIYLA SU KAÇAĞI KONTROLÜ

Özkan, Tülay
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Tez Yöneticisi: Doç. Dr. Nuri Merzi

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Su dağıtım şebekelerinin temel fonksiyonu, yeterli miktarda kaliteli suyu belirli bir basınç altında dağıtmaktır. Bir şebekede ana hat ve bağlantılardaki kaçakların oluşturduğu su kayıpları, sağlanan toplam su miktarının %5'i ile %50'si arasında değişebilmekte hatta bazı durumlarda belirtilen oranı da aşmaktadır. Şebekelerdeki kayıp miktarının sistemin servis basıncıyla ilişkili olması, yüksek basıncın kontrol edilmesi suretiyle su kayıplarının da azaltılması imkanını ortaya çıkarmaktadır. Su kaçaklarının azaltılması ile ilgili metodoloji, LEAKSOL adıyla ve iki alt program halinde geliştirilen yazılım vasıtasıyla sunulmaktadır. CODE I adlı ilk yazılım basınç-su kaçağı ile basınç-su tüketimi ilişkilerini tanımlayarak ve optimizasyon tekniklerini kullanarak, su kaçağını azaltacak akım control vana açıklıklarının bulunmasını sağlar. CODE II ise belirlenen vana sayısına bağlı olarak üretilen tüm izolasyon vana kombinasyonları için yapılan hidrolik çözümle su kaçağı ve tüketim miktarlarını hesaplayarak kullanıcıya en iyi alternatifin seçimi için imkan sunar. Geliştirilen program, daha önce literatürde benzer çalışmalarda kullanılan örnek şebeke üzerinde ve Ankara su şebekesine bağlı N8-3 basınç bölgesinde uygulandı. N8-3 basınç bölgesinde alt bölge girişlerine yerleştirilen sekiz vana yardımıyla su kaçaklarında, tanımlanan basınç-kaçak ilişkisine bağlı olarak, % 10'a varan azalma sağlandı.

Anahtar Kelimeler : Su Kaybı, Basınç-Kaçak İlişkisi, Basınca Bağlı Tüketim, Vana Operasyonu, Su Kaybı Azaltılması

To my lovely husband,

ORHAN

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A	cross sectional area
c	consumption (m^3/s or l/s)
coef	coefficient matrix
C'	modified conductance for pipe
C_l	leakage coefficient
C_d	discharge coefficient
C_{HW}	Hazen Williams roughness coefficient
D	pipe diameter (m or mm)
D'	linearization coefficient for pressure-demand relationship
f_q	fictitious pipe flow (m^3/s or l/s)
g	acceleration due to gravity
G	ground elevation (m)
h	head loss (m)
h_f	frictional loss (m)
H	hydraulic grade line elevation (m)
H^{req}	minimum required head (m)
H^{min}	minimum head (m)
$H_{critical}$	critical head
k	composite parameter in pressure-leakage relationship
k'	resistance coefficient for consumer connection
K	pipe resistance constant
L	length of pipe (m)
n	exponent for frictional head loss formula
N	exponent for pressure-leakage relationship
P	pressure head (m)
$\overline{P}$	weighted mean pressure (m)
P^{min}	minimum pressure head (m)
P^{req}	required pressure head (m)
PRES	adequate pressure for desired operating conditions (m)
q	nodal demand (m^3/s or l/s)
q^{req}	required nodal demand (m^3/s or l/s)

Q	flow rate (m^3/s, l/s or m^3/hr)
QS	pipe leakage (m^3/s, l/s or m^3/hr)
R'	hydraulic radius (m)
Re	Reynolds number
rh	right hand side matrix
S	slope of energy line
V	average flow velocity (m/s)
V(k)	valve setting
α	exponent for orifice equation
ε	tolerance (m)
δH	head difference (m)

CHAPTER 1

INTRODUCTION

1.1. General

A water distribution system can be regarded as consisting of a number of source(s) supplying water to various points (nodes) where time varying consumer demands occur through a network of pipes, valves, pump(s) and storage tank(s). The main motive for the movement of water is the extraction of water by consumers located at various nodes of the network.

Water distribution networks are designed and operated to provide sufficient and drinkable water to consumers with adequate pressures. However, in some conditions system performance is called as deficient if it fails to meet pressure requirements. These conditions may result due to leakage, high demand from consumers, fire-fighting demands, pipe breaks, power outages and mechanical failure of physical components (pumps, valves, etc).

Numerous software programs have been written for detailed hydraulic analysis of water distribution networks. However, traditional network analysis (Hardy-Cross, Newton-Raphson, or Linear Theory Methods) and related computer programs written using these methods presume that the nodal demands are always satisfied in a water distribution system and determine the available pressure heads. However, in case of pump failures or pipe breaks, the system may be unable to supply all nodal demands at required heads. In this case, the traditional network analysis may not fully describe the partially failed water distribution systems. However, if the system is simulated for some of the failure conditions, the relationship between the pressure and demand will have to be taken into account if the simulation results are expected to be realistic.

Today, many areas of the world have water shortages and some countries do not have sufficient water to meet future demand and have to look for costly solutions to supply water. This makes the reduction of lost water much more attractive than finding new sources of water.

The American Water Works Association defines basically two categories of losses in water distribution systems (AWWA, 2007). These categories are:

1. Real losses
2. Apparent losses

Real losses are the physical escape of water from the distribution system and include leakage and overflows from storage tanks prior to the point of end use. Those losses inflate the water utility's production costs since they represent water that is extracted and treated but never reached for beneficial use. Real losses typically account for the greatest volume of water lost by suppliers; leakage is the most common form of real losses. In the past decade, considerable researches have been conducted on the nature and impact of leakage and highly effective practices have been implemented around the world. This research will focus primarily on the real losses within a water system and try to minimize them through pressure control.

Apparent losses are the losses that occur in utility operations due to customer meter inaccuracies, billing system data error, unauthorized consumption and authorized un-metered consumption. This is water which is consumed but not properly measured. These losses also preclude obtaining correct data on customer consumption patterns. Authorized un-metered consumption losses include beneficial use by the municipality or utility and are commonly used for flushing water mains and fire fighting (AWWA,2007). Figure 1.1 shows distribution system losses.

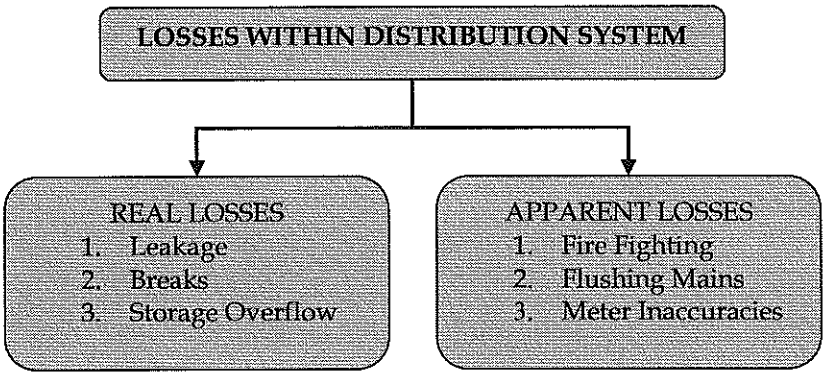

Figure 1.1 Distribution system losses (AWWA, 2007).

Thornton (2002) offers some potential solutions for real losses which can be summarized as follows:

- Establishing pressure zones
- Pressure management to reduce volume of loss and occurrence of new leaks
- Level control to reduce overflows from storage
- Leak detection to locate non-visible leakage
- Rehabilitation and replacement of mains and service pipes

A complete loss control program can be called as a water loss optimization program which means doing everything possible to improve the performance of the water system. System optimization towards a loss control program is in many cases far more cost-effective than finding new sources to satisfy the consumer demands. Many water systems are designed considering the minimum level of pressure required for the demand types, but in many cases no consideration is made for maximum pressure levels requiring a pressure optimization within the system. In fact, pressure management is one of the most basic and cost-effective form of optimizing a water system.

The quantity of water leakage from a water distribution system is related basically to the system pressure. Adequate pressure is essential to maintain a satisfactory service. Excess pressures are undesirable because they can lead to significant levels of leakage. Note also that consumers' hydraulic accessories can be affected from high pressures. Therefore, system pressure should be kept within a certain range. So that leakage can be reduced by reducing pressure during off-peak hours. Pressure management is one way of leakage reduction. Recent studies have shown that both leakage volume and occurrence of new leakages are reduced greatly by the reduction and stabilization of pressure within a distribution system. Obviously, not all systems can tolerate pressure reduction and indeed many systems suffer from lack of pressure. However, there are still many utilities which are operating at pressures in excess of those required and can need a pressure management program. These programs are designed not only to reduce pressure but also to provide a constant supply of water pressure. Some customers experience periods of the day with low pressures caused by high head losses due to high velocities in the system and some of them complain of pressures which are too high and cause damage to equipment. Uncontrolled leakage can also cause lack of supply for customers. Therefore, pressure management can increase customer satisfaction.

Pressure management can be achieved in various forms; one of them is basic sectorization of a gravity system to dynamically controlled automatic control valves (Thornton, 2002). *Sectorization* is one of the most basic and effective forms of pressure management. Subsectors are established either naturally or by using valves. Systems with gravity feeds are usually sectorized by ground level and systems with pumped feeds are usually sectorized depending on the level of elevated tanks or storage. *Pump control* is another method for controlling system pressure. Pumps are activated or deactivated depending on the system demand. This method is effective if reduced level of pumping, especially at night, can still maintain reservoir levels. Another way of pressure control is the usage of *automatic flow control valves.* Flow control valves automatically throttle to limit the rate of flow passing through the valve to a user specified value. This method may be effective for areas with low head losses, demands which do not vary greatly with the seasons, and uniform supply characteristics. With the increase in the use of computerized control systems, the usage of this kind of valves may offer a cost-effective way of reductions in system leakage. A combination of loss control program including valve operation, pump control and tank level regulations can be employed to undertake pressure management.

In this thesis study, the computer program named as LEAKSOL is developed to meet the need to a hydraulic analysis program reducing water leakages in a network by describing the relationship between pressure-leakage and pressure-demand to provide operational results. LEAKSOL is composed of two subprograms, CODE I and CODE II. The first code (CODE I) can be applied for that kind of leakage control scheme presented above. The objective associated with this code is the determination of flow control valve settings by optimization methods to minimize leakage. The inclusion of pressure dependent leakage and pressure dependent demand terms in network analysis allows the application of optimization techniques to identify the most effective means of reducing water losses in distribution systems. The nonlinear network equations are linearized using Linear Theory Method and the optimum flow control valve settings are determined by using the optimization method called as Linear Programming. Flow control valve settings obtained from CODE I, change between 0 and 1. This code is developed for two type of analysis; demand driven analysis in order to verify the results obtained from execution of the code with that of sample network applications in literature and head driven analysis to go a step further with real network application.

The field study performed in N8-3 pressure zone in 2001 (Özkan, 2001) showed that many system operators in local municipalities recognize the need to reduce system pressure and partially close a gate or butterfly valve to create a head loss and reduce pressure. This gave an opinion about the usage of existing valves in the system for reduction of leakage by keeping them closed all the times with suitable combinations not affecting the consumption. Also, the most common type of valves in water distribution systems is the isolation valves, which can be manually closed to block the flow of water. The main purpose of these valves is to turn off a portion of the system to replace a broken pipe or a leaky joint. Well-designed water distribution systems have isolation valves throughout the network so that maintenance and emergencies affect a few customers as possible. However, the usage of these valves as a control device of pressure also may help for reduction of leakage throughout the network. But any operation on isolation valves requires a field crew and setting of these types of valves as partially open is much more difficult than working with fully open or closed position. Also, the usage of isolation valves as partially open can damage the equipment. Since the algorithm which has been described firstly is applicable for flow control valves, a developed new code (CODEII) can be used more effectively with isolation valves. CODE II works on combinations of valves having settings 0 or 1. In other words, valves can only be on or off. A program applicable to these types of valves may be more economical and practical in the existing situations.

The solution obtained from this code may not be the most effective way of leakage minimization but can help to reduce pressure and so leakage. Furthermore, the disadvantage of this solution is that any operation on isolation valves requires a field crew. Therefore it may not be possible to change the valve settings continuously. In this case it is advised that long term analysis of networks with many number of demand curves can give the best combination of valves which will be kept closed always for certain sub-zones or for whole network.

The objective of the second study (CODE II) is to develop an algorithm to determine the isolation valve settings (on or off) through the hydraulic analysis of the network by adaptation of pressure dependent demand terms in order to reduce leakage flow. The potential economic benefits from such a control scheme can be evaluated by comparison of the leakage volumes resulting in the controlled and uncontrolled cases. The nonlinear network equations describing nodal heads and pipe flows are augmented by terms that explicitly account both for pressure-dependent leakage terms and pressure dependent demand terms. Successive linearization of these equations using the linear-theory method allows a program for reduction of leakage through the system. Number of valves selected in the network for this purpose determines the number of valve combinations or number of cases examined at each run. A case defines a combination of valve settings showing that either a valve is fully open or closed. Valve combinations are determined by generating subsets. After the system is solved, the leakage volume and consumptions are computed for each case;

the critical head requirement described for each node is checked and reported. The addition of pressure dependent demand terms provides the computation of consumption for all combinations of isolation valve settings. In the optimization model, the solution where the critical head criterion is not satisfied is excluded although these solutions provide less leakage volume corresponding to others. But, CODE II gives opportunity to the user to see which criterion is not satisfied and also, the amount of leakage and consumption corresponding to this case. Therefore, the usage of pressure driven approach with the addition of pressure dependent leakage terms provides more realistic results and also, expands the group of alternative solutions letting the authorities to select the most appropriate alternative obeying the utility's operating system. The codes are planned to be applied in N8-3 pressure zone of Ankara Municipal Water Supply System.

In this thesis, specific aims that are addressed can be summarized as theoretical understanding of both pressure dependent leakage terms and pressure dependent demand terms in water distribution networks in order to develop a hydraulic analysis program reducing water leakages with proper valve operations and implementation and testing of the related computer program on a large real network by knowing the need and importance of water loss reduction in local municipalities.

Also, an interface was developed in MapInfo Professional in order to input, organize and export data to the core program written for CODE I and to display the results at the end of the execution. However, some of imperfections have to be debugged. Hereafter, main program can combine capabilities of MapInfo and MATLAB for more user-friendly application of algorithm to real networks. MapInfo Professional was selected to develop this interface due to its capable programming editor, MapBasic. Moreover, by knowing that ASKİ Data Processing Center stores all attributes of Ankara water distribution network database in MapInfo environment, it was thought that development of a code with MapInfo interface enables easy extraction of all network related information about zones and working in the same media. AppendixA contains the details of this interface and describes how to use it.

1.2. Thesis Outline

The structure of this thesis closely follows the order in which the work was undertaken in response to the aims they were initially conceived. It consists of seven further chapters.

Chapter 1 gives general information about the subject of this thesis study and indicates the importance of conducting such a scheme of work for leakage reduction.

Chapter 2 presents the literature review on leakage control and on the relationship between leakage and pressure, briefly reviews works on partially satisfied demand for deficient networks and models developed. Also this chapter places the work in the context of previous and contemporary related researches.

Chapter 3 describes briefly the elements of an infrastructure and reviews the formulation of equations and solution techniques in water distribution networks by emphasizing the expressions used in this study.

Chapter 4 outlines the methodology followed for development of codes. The problem is formulated by describing the relation between pressure-leakage and pressure-demand. Usage of Linear Theory Method for terms constituting node flow continuity equations is presented. Arrangement of related equations as an optimization problem to be solved with Linear Programming Method is described. Application of methodology proposed for demand-driven and head-driven analysis in order to minimize leakage on a sample network is illustrated with formed matrices. Chapter 4 describes also the algorithms and the iterative procedure for codes.

Chapter 5 gives information about codes in detail, explains script files used in codes, summarizes the preparation of input files, points out the application of the methodology for a sample network used previously in literature and compares the results for codes. Also, a case study was conducted on N8-3 pressure zone of Ankara Municipal Water Supply System. Chapter 5 describes the application and results of the main code, LEAKSOL on this zone. The reasons for selecting N8-3 as study area are indicated. Model performance is assessed and the implications of the results are discussed.

Chapter 6 summarizes the main results of the previous chapters, discusses the implications of the work within the context of existing leakage and demand modeling literature.

Finally, Chapter 7 presents conclusions and makes recommendations for future work.

CHAPTER 2

LITERATURE REVIEW

This chapter provides the review of the literature corresponding to leakage control and the relationship between leakage and pressure. Then, studies on partially satisfied demands for deficient networks are presented.

2.1. Leakage Control

Available hydraulic network solvers can predict pipe flow, energy losses and nodal pressures in water distribution networks; however, most of the solvers lack the structure needed to deal appropriately with the leakage problem.

Leakage management can be classified as passive and active leakage control (Farley and Trow, 2003). Passive leakage control implies repairing or replacing pipes only for visible and reported leaks by customers or noted by the utility's own staff. Twort et al. (2000) defines active leakage control as a policy of regularly testing all parts of the distribution network for indications of leakage, searching for and repairing leaks by aiming to achieve an economic level of leakage. In addition to reducing leakage, the occurrence of new bursts is reduced, water quality is improved and operating costs are lowered. The most appropriate leakage control policy is dictated by the characteristics of the network and local conditions like financial constraints on equipment and staffing resources (Butler and Memon, 2005). However, the main factor for choosing the most suitable control program is the value of water. Generally, the method of leakage control is usually passive (Butler and Memon, 2005) or low activity, mending only visible leaks. Repairing of visible leaks only may be cost effective in areas where water is plentiful and cheap to produce. But, it is evident that in most of systems, increase in operating costs over time will inevitably justify some amount of active control.

Pressure management is one of the important elements of a leakage control strategy. It can be applied together with leak detection programs. Even if active leakage control is undertaken, it has generally assumed that there is a theoretical minimum level of leakage below which losses can not be minimized which are called as background leakage and it can be minimized through pressure management. Reduction of excess pressure reduces;

-the number of new leaks and bursts,
-flow rates of all existing leaks,
-pressure dependent consumption by establishing more equitable service.

Researches regarding to pressure management have concentrated on the optimized control of valves (usually pressure reducing valves, PRVs and flow control valves, FCVs) in order to minimize leakage (Germanopoulos and Jowitt, 1989; Jowitt and Xu, 1990; Xu and Powell, 1993; Germanopoulos, 1995; Savic and Walters, 1996; Vairavamoorthy and Lumbers, 1998; Alanso et al., 2000; Ulanicki et al., 2000).

Usage of flow control valves in order to reduce leakage was handled by Sterling and Bargiela (1984) first. In their study, the minimization of the sum of the pressures in the network was selected as objective function for the purpose of leakage reduction. The constraints of the problem were the governing equations, but did not include leakage terms. Newton-Raphson method was used for linearization. But, it is obvious that the inclusion of leakage terms affects the flow and pressure distribution in a network and therefore the resulting optimal valve settings.

Germanopoulos (1985) suggested a technique for incorporating pressure dependent demands and pressure dependent leakage terms in network analysis and simulation models. Again Newton-Raphson method was used for model formulation.

Germanopoulos and Jowitt (1989) proposed a method where leakage terms were incorporated explicitly into the governing equations. The objective function was the same as that used by Sterling and Bargiela (1984). Linear Theory Method was used to linearize the governing equations of the optimization problem. But linearization based only on the pressures of the previous iteration and did not include the valve settings of the previous iteration.

Jowitt and Xu (1990) applied the same procedure as Germanopoulos and Jowitt (1989), but modified the objective function and the linearization procedure for elements with flow control valves. The objective function was the minimization of leakage volume, rather than the minimization of the total pressure in the network. The procedure to linearize the head-loss equation for elements with flow control valves was improved to include the valve settings of the previous iteration in its calculation.

In these studies, linear programming method was applied to solve the optimization problem. Linear programming is an iterative procedure that involves linearization of the objective function and constraints and then solving the resulting linear program to obtain new solution. This solution is again used to linearize the objective function and constraints and the procedure is repeated until a termination criterion is met.

The purpose of the study conducted by Reis et. al. (1997) was the appropriate location of control valves in a water supply pipe network and their settings via genetic algorithm to obtain maximum leakage reduction for the given nodal demands and reservoir levels. In their study, for the minimization of leakage, the Jowitt-Xu model was followed. The application of genetic algorithm to the valve location problem was illustrated considering a 25 node network with 37 pipes, used earlier by Germanopoulos and Jowitt (1989) and Jowitt and Xu (1990).

More recently, Vairavamoorthy and Lumbers (1998) formulated the valve setting optimization problem as a nonlinear programming problem (NLP) and solved using a reduced sequential quadratic programming method. Two objective functions were considered and incorporated into the optimization model. The first one was the minimization of the total volume of leakage. The second objective function was selected in their studies as the minimization of the squared differences between actual nodal and target pressure heads where the objective function and pressure constraints together allow minor violations in the pressure objective around the target level. Having both, a nonlinear objective function and constraints, the problem can be called as a NLP formulation and solved as a sequence of quadratic programming subproblems. The performance of the valve setting algorithm was illustrated on a network used by Sterling and Bargiela (1984), Germanopoulos and Jowitt (1989) and Jowitt and Xu (1990). Vairavamoorthy and Lumbers (1998) suggested that minor deviations from target pressures may be acceptable if that kind of deviation provides a more control on excess pressure in the system.

Pressure management can also be achieved in different ways. Wood and Reddy (1995) tried to reduce leakage through optimal variable speed pump schedules. Their goal was to obtain pump speeds as a function of time that would minimize pumping costs and pressures. The nonlinear problem was solved using genetic algorithms (GA) to find the best arrangement of tank levels and pump speeds. Xu and Powell (1993) focused on the combined action of valves and variable speed pumps in order to minimize excess pressures. They were used two sets of decision variables: valve settings and pump speeds.

Then, Tucciarelli et al. (1999) developed an algorithm for the estimation of water losses due to small leaks based on the maximization of likelihood function. Ulanicki et al. (2000) examined the practical implementation of valves for on-line pressure control, discussing issues of control rules and the difference between predictive and feedback strategies.

Araujo et al. (2006) aimed a solution that allows simultaneously optimizing the number of valves and their location, as well as valve opening adjustments for simulation in an extended period, dependently of the system characteristics. EPANET model was used for hydraulic network analysis and two operational models were developed based on GA optimization method for pressure control and consequently leakage reduction. Models were based on demand driven analysis and a case study was conducted on a network used by Sterling and Bargiela (1984), Germanopoulos and Jowitt (1989) and Jowitt and Xu (1990).

The subject of leakage reduction in water distribution systems by optimal valve control is still open to study together with developments in optimization models.

2.2. The Pressure-Leakage Relationship

In the past, approaches related with leakage modeling have involved the application of a multiplier to system demand (Lambert, 2000). Such an approach can not reflect the relationship between leakage and pressure and can lead to inaccurate results. Consequently, representation of leaks by orifice function is accepted as more appropriate. The general orifice function can be described as follows:

$$QS = C_d A(2gP)^{\alpha} \tag{2.1}$$

where QS is the flow through the leak, C_d is a dimensionless discharge coefficient, A is the area of the orifice, g is the acceleration due to gravity, P is the pressure head difference across the orifice and α is 0.5 derived from energy conservation (Bernoulli Equation) between flow through an orifice of fixed dimensions and the pressure gradient across it. However, several researchers have found that the square root relationship is not always appropriate. Goodwin (1980) found the pressure-night flow relationships from longer duration tests on 18 district metered areas in United Kingdom (UK) where all detectable leaks had been located and repaired (Thornton and Lambert, 2005). The study showed that in the lower pressure range (i.e. below about 40 m) changes in net night time flow through the districts were found to be roughly proportional to changes in pressure. But, at higher pressures, the net flow through the district responded more than linearly with changes in pressure (Şendil and Al-Dhowalia, 1992).

The following version of the orifice function, named as power equation, is presented as a general equation for the analysis and prediction of the leakage-pressure relationship for individual pipes or for aggregate leakage from systems (Thornton, 2003):

$$QS_1 / QS_0 = (P_1 / P_0)^N \tag{2.2}$$

Equation shows that if pressure is reduced from P_0 to P_1, flow rates through existing leaks change from QS_0 to QS_1 and the extent of the change depends on the exponent N. The general relationship of Eqn.2.2 is shown in Figure 2.1.

In Japan, Hiki (1981) described several pressure-leakage tests in Japan on metal pipes of diameters ranging from 60 to 180 mm with holes of 1 to 5 mm diameter drilled into them. These pipes were buried in sand or submerged in water and then leakage rates were measured at different pressures. Leakage of 24 to 900 l/hr corresponded to pressure heads between 2 to 60 m. Calculated N values ranged between 0.36 and 0.70 with an average close to 0.50. Lambert (2000) mentioned from pipeline tests conducted by the Tokyo Water Works in 1982 producing N values of 0.51 to 0.54 under the condition of relatively high leakage flow (1500 l/hr). In the UK, segments of leaking service connections were removed and tested in the laboratory for pressures ranging from 10 to 75 m and leakage flows of 0 to 4000 l/hr. For leaky metal pipe segments N values were found to be close to 0.5 while for plastic pipes the value was closer to 1.5.

Ogura (1979) presented the results of 20 short tests, nineteen of which had metal mains, on small isolated sectors of actual distribution network in a Japanese city. The sectors were isolated by closing valves and customer use was prevented by shutting off service connections. Pressures were raised from 5 m to 40 m and changes in sector inflow recorded. Each sector test was lasted in 45 minutes. Values for N were found to be within the range of 0.65 to 2.12 with a resulting weighted average of 1.15 (this value has become the Japanese standard for the last 25 years). Another result of the Ogura study is that sectors comprised of metal mains consistently gave N values above 0.5 which was different from laboratory tests on individual pipes. This unexpected result has been attributed to the sensitivity in C_d for small individual leaks in the laminar flow region (Lambert, 2000). Such leakage is typical for joints and fittings and comprises much of the background leakage.

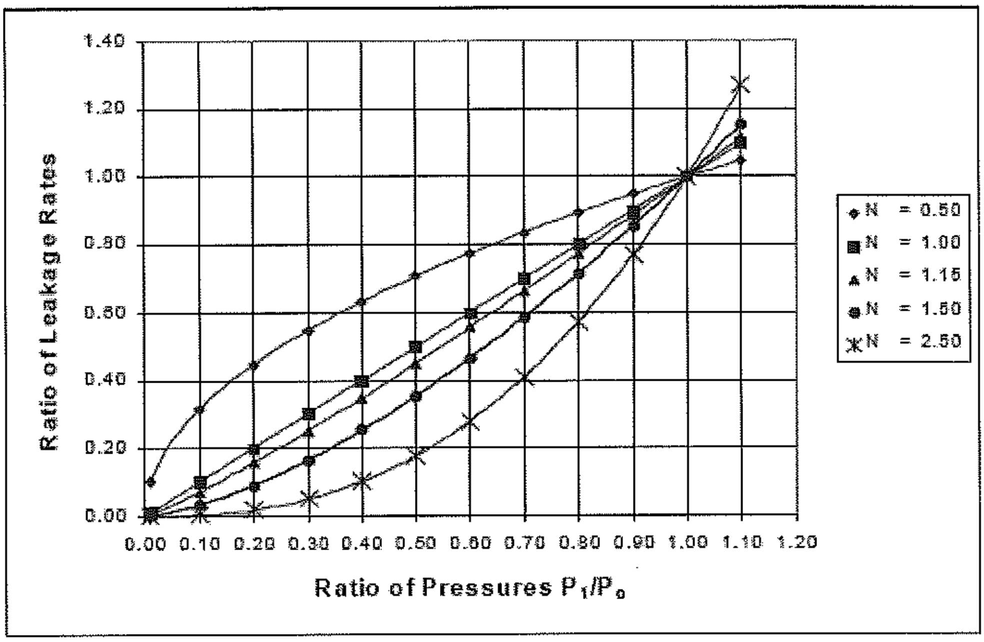

Figure 2.1 General relationship between pressure and leakage rate based on power law equation (Thornton and Lambert, 2005).

Analyses of over 100 field tests on sections of distribution systems in Japan and district metered areas in Australia, Brazil, Canada, Malaysia, New Zealand, U.K. and the U.S.A. have confirmed that N exponent typically lies between 0.5 and 1.5, but may occasionally reach as high as 2.5 (Thornton and Lambert, 2005). Several field experiments have been conducted on sectors of distribution systems with customer night use in the UK. Lambert (2000) reinterpreted the data from tests conducted on 17 distribution system sectors in 1980. Prior to the tests, all detected leaks had been repaired and it was assumed that any remaining leakage was background leakage at joints and fittings. The test produced N values for minimum night flow ranging from 0.70 to 1.68 with an average of 1.13. It was stated that if estimates of customer night use could have been deducted, higher N values from these tests would have been achieved. Unpublished test results in the UK since 1980 on sectors before and after leak detection surveys produced N values for leakage (customer night use subtracted from the night flow) from 0.5 to 1.5 with an average close to 1.0. Tests in Brazil on 7 sectors with a high leakage rate resulted with N values ranging between 0.52 and 0.67 for metal pipe systems and values close to 2.5 for two areas having high leakage rate.

In a Mexican study, Arreguin-Cortes and Ochoa-Alejo (1997) found a moderately weak linear relationship between pressure and leakage for service connections.

Germanopoulos and Jowitt (1989), Jowitt and Xu (1990), Vairavamoorthy and Lumbers (1998) and more recently, Araujo et al. (2006) used the power of 1.18 to describe the relationship between leakage and pressure which was based on the data from a set of field experiments undertaken by the Water Research Centre (Goodwin, 1980).

More recently, Greyvenstein and Van Zyl (2005) discussed the factors which may be responsible for the higher leakage exponent than the theoretical value of 0.5 and they conducted an experimental study with different pipe materials. Van Zyl and Clayton (2005) discussed the effect of pressure on leakage and they proposed four mechanisms that are responsible for the observed range of leakage exponent, N (Greyvenstein and Van Zyl, 2005). There are categorized as leak hydraulics, pipe material behavior, soil hydraulics and water demand. Greyvenstein and Van Zyl (2005) tried to determine the leakage exponents for failed water pipes taken from field and for pipes with artificially induced leaks. Their study included round holes and longitudinal and circumferential cracks in uPVC, steel and asbestos cement pipes where all flows were turbulent and leaks were exposed to the atmosphere. Leakage exponents were found that they vary between 0.42 and 2.30. Also, Greyvenstein and Van Zyl (2005) were concluded that the exponents found in field studies are not unrealistic. In that study, it was observed that round holes had leakage exponents close to the theoretical value of 0.5 with small differences in steel and uPVC pipes. Largest exponents were found in longitudinal cracks. The highest leakage exponents occurred in corroded steel pipes contrary to the perception (Farley and Trow, 2003) that plastic pipes have higher leakage exponents due to their lower modulus of elasticity.

International Water Association (IWA) and several authors (Lambert, 2000; Farley and Trow, 2003; Thornton, 2003) recommend N values according to pipe material and leakage level for both individual pipes and distribution system sectors. An N value of 0.5 is suggested for fixed area leaks in metal pipes. Small leaks at joints and fittings should be modeled with an N value of around 1.5 while leaks in plastic pipes may be modeled with N values of 1.5 or higher. For individual sectors of a distribution system N depends on pipe material and leakage level, but in the absence of such knowledge a linear relationship (i.e., N=1.0) can be assumed.

The experimental work carried out by the Water Research Centre and researchers in Japan and elsewhere has provided an important basis on which to build an understanding of leakage-pressure relationships in distribution systems.

Although it is widely appreciated that leakage is a function of pressure and it is now commonly accepted to model leakage with an orifice function, exact understanding of the pressure-leakage connection in water distribution systems still remains elusive. Results from laboratory experiments on pipe segments and field tests on isolated portions of distribution systems indicate that the orifice function interpreted according to these concepts can be an effective tool for representing the leakage-pressure relationship (Merzi et al., 2003 and Özkan, 2001).

2.3. Partially Satisfied Demands

One of the major considerations in the design and operation of water distribution networks is that the network supplies the water demands of the population being served with adequate pressures at all nodes of the network during the entire design period.

In hydraulic network solvers making demand driven analysis (fixed demand analysis) flows in network links and the corresponding nodal heads are determined for a specified demand pattern for the network. These models search for the answer of the question that what would be the nodal pressures and pipe flows if the prescribed nodal demands are consumed in the network. This formulation is valid only if the hydraulic pressures at all nodes are adequate so that the demand is independent of pressure. Thus, the assumption that nodal consumptions are fixed irrespective the network pressures may be a reasonable assumption under normal operating conditions. On the other hand, in some conditions it is possible to obtain pressures as negative or unacceptably low, since demands are satisfied regardless of the actual pressures at the nodes (Bouchart and Goulter, 2000). This condition of the network is termed pressure deficient. In these situations, fixed demand analysis can provide unrealistic results. Bhave (1991) summarizes some of the situations which may occur during the service life of the network as follows (Nohutçu, 2002):

- The actual nodal demands are more than the predicted design demands due to an unpredicted accelerated growth over the entire area. Thus, the system becomes inadequate even during the design period.

- The withdrawals at some nodes are more than the design ones, due to large diameter withdrawal connections, installation of pumps and illegal usage of water or excessive leakage. This naturally would create a deficiency in some other parts.

- Even though the overall growth is well predicted, it may e unbalanced, causing the system to be deficient in some zones and more than adequate in others.

- The distribution system is used beyond its design period and thus becomes deficient. (Because the cost of replacement of the entire network is usually large, the distribution network is continued in use beyond the design period by suitably strengthening it whenever necessary and replacing those pipes that have excessive corrosion and leakage, thereby creating maintenance problems).

- Some pipes or pumps are closed for cleaning, repairs or replacement. This may cause a temporary deficiency at some nodes (Bhave, 1991).

Since the 1980s, researchers have proposed various methods to compute the actual flows of the network under such pressure deficient conditions. Most of the proposed methods involve an assumption on the relationship between pressure and outflow at the demand nodes. These methods are generally termed head-driven analysis.

Carey and Hendrickson (1984) assumed that the pipe capacities are limited by a maximum energy gradient, transforming the deficient-pressure network problem into a classical minimum cost flow problem (Wagner et al. 1988a). Fujiwara and De Silva (1990) used this method which is based on maximum expected flow of a capacitated network, to calculate the outflow Q_i at each demand node i and the results showed that the outflows tend to be overestimated (Ang and Jowitt, 2006). Also, the regulation of flow to obtain maximum outflow during a temporarily deficient condition is rather difficult to achieve in practice.

Bhave (1981, 1991) developed a method called Node Flow Analysis (NFA) to predict performance when a distribution network is deficient. The method was based on the assumption that when a network deficient and thus unable to satisfy the nodal demands, the network will try to meet the demands, as far as possible, under the given conditions. In other words, the flow in the network will adjust such that the total supply is maximized under deficient conditions. Thus, the network flow analysis problem is considered as an optimization problem subject to certain constraints that relate hydraulic gradeline at each junction to available flow at that junction.

On the other hand, Germanopoulos (1985) was the first directly relating pressure and nodal consumption. He states that the relationship between the pressure and consumption for each node depends on the hydraulic configuration between the node and the consumers downstream. Germanopoulos and Jowitt suggested the use of an empirical pressure-consumption relationship to predict the outflows at various nodal heads

$$c_i = q_i^{req}(1 - a_i e^{-b_i H_i / H_i^*}) \tag{2.3}$$

where c_i is actual outflow or consumption at demand node i; q_i^{req} is desired or required demand at that node; H_i is available head; and H_i^* is the nominal head required to satisfy demand q_i^{req} at node i. The terms a_i and b_i are constants for node i. These constant terms could be calibrated for every demand node, or simply set semi-arbitrarily. It was a device to reflect the fact that local pressure deficiency in the network would result in localized failure to deliver the required demand. Germanopoulos (1985) also described a method for the solution of the modified system with Newton-Raphson method. Jowitt and Xu (1993) used the relationship given in Eqn. 2.3 to predict the demand flows in a pressure-deficient network following network failures.

Wagner et al. (1988b) stated that the quantity of water supplied at a node depends on the head at the node. According to this approach, when nodal heads are above the required heads, nodal demands can be satisfied. If however, at any node, the head is below the required one, it is assumed that at that node the system cannot supply the full demand and nodes with heads below the minimum head will be completely shut off. They proposed the use of a parabolic curve to represent the pressure-outflow relationship at a demand node for the head between $H^{\min}$ and H given in Eqn. 2.4.

Gupta and Bhave (1996) presented a very good review of this method and a summary of other pressure-deficient network predictors. This leads to an interesting discussion by Tanyimboh and Tabesh (1997) and a closure by Gupta and Bhave (1997). The resulting conclusion is that the behavior of a water distribution system under pressure-deficient conditions is complex and that further research was needed.

Then, Reddy and Elango (1989) introduced a fixed relationship between the residual head and the corresponding consumption for water distribution networks. But they oppose to imposing an upper limit on the consumption equal to the estimated demand beyond the required pressure. Thus, consumption continues to increase for pressure values higher than the required one for Reddy and Elango's suggestion.

Chandapillai's (1991) approach, which was presented as the most successful approach by Nohutçu (2002), considers a consumer connection that leads water from the distribution main to an overhead tank (OHT). This tank can be fed only when the pressure at the tapping point is above the static lift (P^{min}), which is the level difference between distribution main and OHT. Figure 2.2 shows the relationship between the head (H) at the tapping point and the quantity of flow (Q) into the OHT. The nonlinear curve between the points 1 and 2 can be represented with the following equation.

$$H = H^{\min} + k'Q^{n} \tag{2.4}$$

where $k'Q^{n}$ denotes head loss at consumer connection.

k' : a constant which can be determined from the length, roughness and diameter of the consumer connection, i.e., resistance coefficient appropriate to the consumer connection pipe.

n : a constant representing the head loss in the consumer demand and can be taken as 1.852 (by Hazen-Williams formula).

When Figure 2.2 is applied to a node and the flow rate (Q) is replaced with nodal consumption (c), it can be said that the node starts to consume water for the head values higher than H^{min}. The consumption reaches to the nodal demand for head values greater or equal to H^{req}. The nonlinear relation given as Eqn 2.4 is valid only for H values between H^{min} and H^{req}.

Gargano and Pianese (2000) and Ostfeld et al. (2002) used Chandapillai's approach in their papers on reliability-based design of water distribution networks. With a traditional demand-driven solver, this requires some iteration, estimating network heads for the nominal demands, correcting the demands at these heads using the parabolic relation described in Figure 2.2, and then re-estimating the heads and so on until sufficient convergence is obtained. This can lead to a high computational requirement and some researchers choose to use much simpler pressure outflow relationships (Ang and Jowitt, 2006). For example, Xu and Goulter (1999) and Khomsi et al. (1996) used a simple zero to one relationship for outflows in their computations of reliability with demands satisfied when the nodal heads were greater than or equal to H^{req} and otherwise zero.

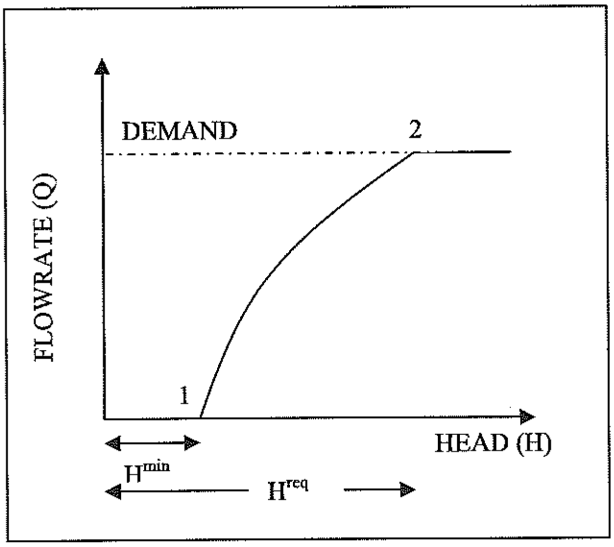

Figure 2.2 Relationship between head and flow rate (Chandapillai, 1991).

Tanyimboh et al. (2001) developed Chandapillai's approach by presenting a new way of interpreting the results of the traditional demand-driven network analysis as an approximation to pressure-driven network simulation in order to calculate the reliability of single-source networks.

Although they did not propose a relationship between pressure and consumption, they have developed Chandapillai's approach. Rearranging Eqn 2.4 to determine the value of c, and for any given nodal head,

$$c=\left(\frac{H-H^{\min}}{k'}\right)^{\frac{1}{n}} \tag{2.5}$$

When $c=q^{req}$ (required demand), and $H=H^{req}$ (the required head to consume the demand), the equation becomes,

$$q^{req}=\left(\frac{H^{req}-H^{\min}}{k'}\right)^{\frac{1}{n}} \tag{2.6}$$

$$\frac{1}{k'^{\frac{1}{n}}}=\frac{q^{req}}{\left(H^{req}-H^{\min}\right)^{\frac{1}{n}}} \tag{2.7}$$

Finally, substituting for k in Eqn 2.5 gives the flow delivered as follows:

$$c=q^{req}\left(\frac{H-H^{\min}}{H^{req}-H^{\min}}\right)^{\frac{1}{n}}, \qquad H^{\min}\le H\le H^{req} \tag{2.8}$$

Note that, for $H\le H^{\min}$, $c=0$, and for $H\ge H^{req}$, $c=q^{req}$.

Tanyimboh et al. (2001) also states that the value of H^{min} can be approximated to the elevation of the node.

More recently, Nohutçu (2002) reviewed all of the studies regarding the performance of the deficient networks. Approaches mentioned above were discussed, compared and some corrections and modifications were suggested. In this study, the model, fundamental equation of which is suggested by Chandapillai (1991) and then developed by Tanyimboh et al. (2001) was accepted as the most successful one among the models discussed. Nohutçu (2002) approximated the minimum required head (H^{min}) to the elevation of a node, so the minimum required pressure (P^{min}) to zero, and after these modifications called the model as Modified Chandapillai Model. Eqn 2.8 becomes;

$$c = q^{req}\left(\frac{P}{P^{req}}\right)^{\frac{1}{n}}, \qquad P^{\min} = 0 \le P \le P^{req} \tag{2.9}$$

Ang and Jowitt (2006) took a modeling approach by adding an artificial reservoir at every pressure deficient node and removing it from fixed demand nodes in an iterative procedure, which requires numerous repetitive solutions of the system equations with no guarantee of convergence. The iterative calculation stops until correct pressure sufficient nodes are identified without negative pressure at a node. This method seems to avoid introducing extra parameters like the method based upon orifice flow equation. But it can be difficult to apply this method to a larger water system, because it is computationally expensive to add a virtual reservoir to each of hundreds of thousands of nodes and then shuffle them around. It can be also programmatically difficult to implement it since each iteration will require changing the solution matrix (Yi Wu et al, 2006).

Yi Wu et al. (2006) developed an approach for pressure dependent demand analysis to simulate a variety of low pressure scenarios. A set of element criticality evaluation criteria was also proposed for quantifying the relative importance of the elements that might be out of the service. Better convergence rates were achieved relative to approach used by Ang and Jowitt (2006).

Yi Wu et al. (2006) also present a typical pressure dependent demand curve as illustrated in Figure 2.3. As can be seen in the figure, the actual demand increases to the full requested demand as pressure increases, but remains constant after the pressure is greater than the pressure threshold. Pressure percentage is the ratio of actual pressure to a nodal threshold pressure while demand percentage is the ratio of the calculated demand to the reference demand (Yi Wu et al, 2006).

Pressure is called as reference pressure which is a pressure supplying 100% of the desired or reference demand. Whenever the pressure is below the reference pressure, nodal demand is certainly dependent on the pressure at the node. In other words, unlike the conventional approach of demand driven analysis, demand is a function of pressure in head driven analysis.

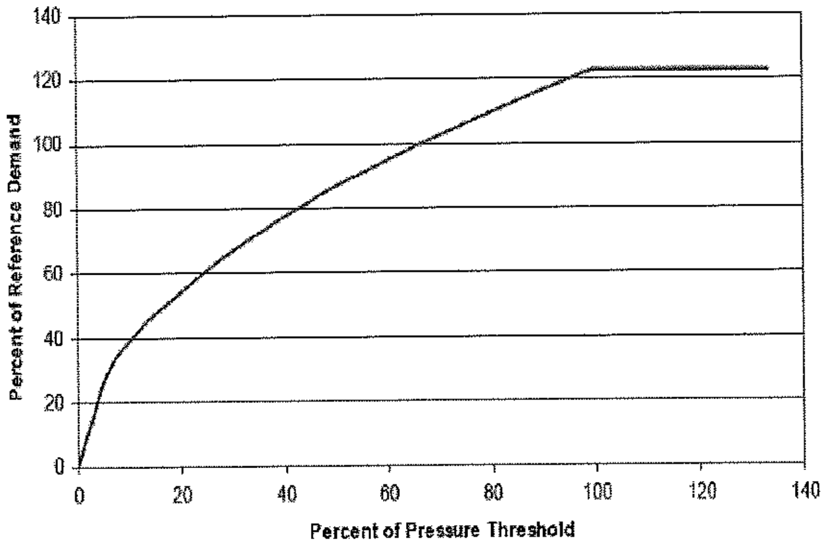

Figure 2.3 Typical pressure dependent demand curves (Yi Wu et al., 2006).

In this thesis study, the computer program, LEAKSOL with two subprograms was developed for distribution networks First one is the optimization program to find optimal flow control valve settings minimizing the leakage volume for described conditions by using pressure dependent leakage and pressure dependent demand terms. The second one is a static analysis program providing solution for all combinations of isolation valve settings defined in the network again by using pressure dependent demand and leakage terms. In both of the codes, for description of the relationship between pressure and demand, the Modified Chandapillai Model, suggested by Chandapillai (1991), then developed by Tanyimboh et al. (2001) and modified by Nohutçu (2002) was used. Application of this model was given in Chapter 4 in detail.

CHAPTER 3

THEORETICAL CONSIDERATIONS

Distribution system infrastructure is generally considered to consist of the pipes, pumps, valves, storage tanks, reservoirs, meters, fittings and other hydraulic appurtenances that connect source to consumers taps.

Pipes are the principle elements in distribution networks. The system of pipes that transport water from the source to the customer are often categorized from largest to smallest as transmission mains, distribution mains and service lines. The three requirements for a pipe include its ability to deliver the quantity of water required, to resist all possible external and internal forces and its durability.

Pumps are used to impart energy to the water in order to boost it to higher elevations or to increase pressure. The cost of power for pumping constitutes one of the major operating costs for a water supply system.

Two types of valves generally utilized in a water distribution system are isolation valves and control valves. Isolation valves are used to isolate sections of pipes for maintenance and repair; they are located so that the service areas isolated will cause a minimum of inconvenience to other service areas. Maintenance of the valves is one of the major concerns carried out by a utility. Many utilities have a regular valve-turning program in which a percentage of the valves are opened and closed on a regular basis. It is desirable to turn each valve in the system at least once per year.

Control valves are used to control the flow or pressure in a distribution system. They are normally sized based on the desired maximum and minimum flow rates, the upstream and downstream pressure differentials, and the flow velocities. Typical types of control valves include pressure-reducing, pressure sustaining and pressure-relief valves, flow-control valves, throttling valves and check valves.

Other appurtenances in a water system include blow-off (drain valves) and air release/vacuum valves, which are used to flush water mains and release entrained air. On transmission mains, blow-off valves are typically located at every low point and an air release/ vacuum valve at every high point on the main. Blow-off valves are sometimes located near dead ends where water can stagnate or where rust and other debris can accumulate.

Hydrants are primarily part of the fire fighting aspect of a water system. Proper design, spacing, and maintenance are needed to insure an adequate flow to satisfy fire-fighting requirements. In addition to being used for fire fighting, hydrants are also for routine flushing programs, emergency flushing and for street cleaning.

Storage tanks provide service storage to meet the widely fluctuating demands often imposed on a distribution system, to provide storage for fire fighting and emergencies, like power outages. During periods of high demand when the supply coming from the pump is less than the demand, tanks supply water to the network and therefore the water stored in the tank is used. During the periods of low demand, at night, when the water supplied by the pump is greater than the demand, the pump supplies water to the tank and the municipality.

3.1. Formulation of Network Equations

A water supply and distribution system consists of a collection of nodes that are interconnected by various elements such as pipes, valves, pumps and reservoirs. The entire system is interrelated in such a way that the condition of one element must be consistent with the condition of all other elements. The system governing equations can be formulated in accordance with the following two rules:

(1) *Node flow continuity relationship*:

For steady incompressible flow in a network, the algebraic sum of the inflows to and outflows from a node must be equal to zero. Thus,

$$\sum_{x\,connected\,j} Q_x + q_j = o \qquad (3.1)$$

Q_x : flow in pipe x

q_j : external flow, i.e., supply (inflow) or demand (outflow) at node j.

(2) *Loop head loss relationship* :

For all loops in a network, the algebraic sum of the head losses in the pipes forming a loop in the network must be equal to zero. Thus,

$$\sum_{x \in y} h_x = 0 \qquad y = 1,2,.......Y \qquad (3.2)$$

h_x : head loss in pipe x

Y : number of loops

3.1.1. Pipe Head Loss Relationship

When a real fluid flows through a pipe, part of the total energy of the fluid is spend in maintaining the flow. This energy is converted to thermal energy due to internal friction and turbulence. This conversion can be named as the loss of energy and termed as head loss, h_f.

The frictional head loss in a pipe can be expressed by a general head loss formula,

$$h_f = KQ^n \qquad (3.3)$$

K : pipe resistance constant

n : an exponent

There are several pipe flow formulas available that relate flow and head losses under various flow regimes. The Darcy-Weisbach, Hazen-Williams and Manning formula are commonly used to express the head loss in a pipe. Even though the Darcy-Weisbach head loss formula is dimensionally homogeneous, some empirical formulas are popular and widely used in practice (Bhave, 1991). In this study, Hazen-Williams formula was used.

Hazen-Williams Formula

An empirical formula widely used in water supply engineering for the flow through pipes is due to G.S. Williams and A. Hazen. This relationship is known as Hazen-Williams formula (Bhave, 1991).

$$V = 0.849 C_{HW} R'^{0.63} S^{0.54} \qquad (3.4)$$

V	:	average velocity of flow in m/s
C_{HW}	:	Hazen-Williams roughness coefficient
R'	:	hydraulic radius (cross-sectional area, A / wetted perimeter)
S	:	slope of the energy line (h_f / L)

Note that for circular pipes $V = 4Q/\pi D^2$ and $R' = D/4$. Substituting these values and taking $S = h_f / L$, eqn. 3.4 becomes,

$$h_f = \frac{10.68\, L Q^{1.852}}{C_{HW}{}^{1.852} D^{4.87}} \qquad (3.5a)$$

$$K = \frac{10.68\, L}{C_{HW}{}^{1.852} D^{4.87}} \qquad (3.5b)$$

in which L and D are length and diameter of pipe in meters and Q is flow through the pipe in cubic meters per second.

Hazen-Williams equation is not really valid for flows throughout the laminar, transition and rough regimes, but it has the advantage of computational simplicity and therefore receives wider application. It is more suitable for smooth pipes and therefore medium to large diameter new pipes. Although it is not quite suitable for old pipes, the formula is used in practice by reducing the Hazen-Williams coefficient values.

3.1.2. Basic Unknown Parameters

Bhave (1991) describes a node with a known head as a *source node*, such as an elevated storage tank and a node with a known flow as a *demand node*. At the source node, the flow is accepted generally as unknown, while at the demand node, the head is generally accepted as unknown. However, occasionally the flow also may be known in addition to the known head at a source node, while the flow may also be unknown in addition to the unknown head at a demand node.

A pipe network is analyzed when all the unknown parameters are determined with help of known parameters and the available interrelating equations. The unknown parameters involved in hydraulic network analysis are;

(1) pipe discharges, Q
(2) nodal heads, H
(3) pipe resistance constants, K
(4) nodal inflows or outflows, q

Some of these parameters are treated as the basic unknown parameters and their values determined initially. Once the values of the basic unknown parameters are known, the values of other unknowns are easily determined. The solution is feasible when the number of equations is equal to the number of unknowns in a network.

When the unknown pipe discharges are taken as the basic unknown parameters, the equations formed are known as Q-equations. When the nodal heads are taken as the basic unknown parameters, the equations formed are known as H-equations. Other basic types of equations are ΔQ and ΔH equations. In this study, H-equations were selected to constitute network equations.

3.1.3. H-Equations

The unknown nodal heads are taken as the basic unknown parameters in formulating H-equations. Nonlinear equations are obtained by applying the node flow continuity relationship to demand nodes;

For all pipes in the network,

$$h_{f_x} = H_i - H_j = K_x Q_x^n \tag{3.6}$$

h_{fx} : head loss in pipe x
Q_x : Discharge in pipe x
H_i, H_j : HGL values at the upstream node i and downstream node j of pipe x

This equation shows nonlinear relationship between h_{fx} and Q_x; eqn. 3.6 can be expressed as;

$$Q_x = \left(\frac{H_i - H_j}{K_x}\right)^{\frac{1}{n}} \tag{3.7a}$$

Some of the elements of eqn 3.7a do not vary with head or flow and therefore they are collected as a composite term K_x.

$$K_x = \frac{10.68L}{C_{HW}^{\;1.852} D^{4.87}} \tag{3.7b}$$

H_i, H_j : HGL level in meters at node i and j.

Using eqn. 3.7a, eqn. 3.1 can be expressed in terms of H_i and H_j, as

$$\sum_{x\,connected\,to\,j}\left(\frac{H_i - H_j}{K_x}\right)^{\frac{1}{n}} + q_j = 0 \qquad (3.8)$$

3.2. Solution Techniques

Since there is no direct method available for the solution of nonlinear pipe network analysis equations, these equations are first linearized and then solved. Naturally, the solution is approximate. Therefore, it should be corrected and an iterative procedure should be continued until satisfactory accuracy is reached. Irrespective of whether a node or loop formulation is used, the problem is one of solving the resulting system of simultaneous non-linear equations for the unknown network flows and nodal heads. A static network solution is thus obtained. Different methods have been proposed for static network solutions and various computer programs have been developed. Three methods are commonly used in practice for iterative solution of nonlinear equations and thereby for the analysis of water distribution networks. These methods, in chronological order of their application to network analysis are (1) Hardy Cross method, (2) Newton-Rapson method and (3) Linear Theory method (Bhave, 1991).

Hardy Cross Method was suggested as a systematic iterative procedure for network analysis in 1936 (Bhave, 1991). This approach is based on loop-flow correction equations, i.e. ΔQ equations. Later, same principles were applied to nodal-head correction equations, i.e. ΔH equations. Both of these approaches are collected under the name of Hardy Cross method. This method attempts to solve the nonlinear equations involved in network analysis by making certain assumptions. The higher power correction terms can be neglected and the iteration number is small for a single loop although the initial guess is poor. However, neglecting adjacent loops and considering only one correction equation at a time can affect the solution and also number of iterations required for convergence increases as the size of the network increases. Modified Hardy Cross method can be applied to improve convergence and reduce the number of iterations. However, this number can be quite large for real networks. Therefore, instead of considering only one correction equation at a time, all the correction equations can be solved by considering the effect of all adjacent loops. So that convergence can be achieved in a smaller number of iterations. Also, some of the equations involved in pipe network analysis are nonlinear. Because there is no direct method for their solution, these equations are linearized and then solved. Since the solution is approximate, it is corrected and the iterative procedure is continued until satisfactory accuracy is reached. The Newton-Raphson method and the Linear Theory method try to solve all the concerned equations simultaneously by applying the iterative procedure (Bhave, 1991).

The Newton-Raphson method expands the nonlinear terms in Taylor's series, considers only the linear terms by neglecting the residues after two terms. Since the nonlinearity in the equations for pipe network analysis is uniform and simple, the nonlinear term can be easily linearized by merging a part of the nonlinear term into the pipe resistance constant. This principle can be used to linearize all types of equations. However, in practice it is applied to pipe-discharge equations and nodal-head equations.

Since this method attempts to solve all network equations simultaneously, convergence criteria using Newton-Raphson approach is quicker than that of obtained using Hardy Cross analysis, which attempts to solve the equations neglecting adjacent loops and considering only one correction at a time. This is especially important when analyzing large networks. However, both methods require initial guesses and bad estimation can lead to slow convergence or in some cases, a situation where the successive trials do not converge and the solution can not be found (Wood and Charles, 1972).

Linear Theory Method

The Newton-Raphson method expands the nonlinear terms in Taylor's series by neglecting the residues after first two terms and considers only the linear terms. This method linearizes the nonlinear equations through partial differentiation while the linear theory method linearizes the nonlinear equations by merging a part of the nonlinear term in the resistance or conductance of the pipes. The Newton-Raphson method is general and works even for the nonlinear equations containing exponential, trigonometric, hyperbolic or logarithmic terms.

However, the nonlinearity in the equations for pipe network analysis is uniform and simple: the variables are raised to the same, nonunity exponent (n=2 in Darcy-Weisbach; n=1.852 for Hazen-Williams). With the help of this feature, the nonlinear terms can be linearized by merging a part of the nonlinear terms into the pipe resistance constants (Bhave, 1991).

In this study, Linear Theory Method was applied to nodal head equations, i.e. H-equations, in order to solve the system equations including relationship among pressure, leakage and demand. The node flow continuity equation written with nonlinear H-equations (eqn. 3.8) for the t^{th} iteration becomes after linearization;

$$\sum_{\substack{i\,connected\,to\,j\\ through\,x}} \left(\frac{\left|{}_tH_i - {}_tH_j\right|^{(1/n)-1}}{K_x^{\frac{1}{n}}} \right) \left(H_i - H_j \right) + q_j = 0 \qquad (3.9)$$

${}_tH_i$, ${}_tH_j$: known or assumed nodal heads for the t^{th} iteration at nodes i,j

H_i, H_j : unknown nodal heads

Eqn (3.9) can be expressed as;

$$\sum_{\substack{i\,connected\,to\\ j\,through\,x}} {}_tC'_x \left(H_i - H_j \right) + q_j = 0 \qquad (3.10)$$

in which ${}_tC'_x$ denotes modified conductance of pipe x for the t^{th} iteration and is given by,

$${}_tC_x' = \left| \frac{{}_tH_i - {}_tH_j}{K_x} \right|^{\frac{1}{n}} * \frac{1}{\left|{}_tH_i - {}_tH_j\right|} \qquad x = 1, \ldots\ldots\ldots X$$

$$ {}_tC_x{}' = \left| \frac{{}_tQ_x}{{}_th_x} \right| \qquad x = 1,\dots\dots\dots X \tag{3.11} $$

in which ${}_tQ_x$, ${}_th_x$ denotes discharge and head loss in pipe x for the t^{th} iteration .

To begin the iterative procedure, it is necessary to initialize the discharge and head loss in a pipe, i.e. to select the values of ${}_1Q_x$ and ${}_1h_x$ for $x=1,\dots.X$.

Issacs and Mills (1980) suggest that initial estimate of flow rates may be set to the same value for all pipes. They also observed that an initial pipe discharge ranging from 0.001 m^3/s to 1 m^3/s, when used in a test network of sixty pipes, had no significant effect on the solution. This means that unlike the Newton-Raphson and Hardy Cross methods, the linear theory method does not have problems with initial estimates (Bhave, 1991).

Alternatively, they also suggest using an initial velocity of 1 m/s in all pipes to initialize the pipe flows. The value of the initial head loss ${}_1h_x$ corresponding to the assumed discharge ${}_1Q_x$ is then evaluated from,

$$ {}_1h_x = K_x {}_1Q_x{}^n \qquad x = 1,\dots\dots\dots X \tag{3.12} $$

Eqn (3.11) now gives, ${}_1C_x{}'$ for all pipes and thus the nonlinear node-flow continuity equations are linearized in the form of eqn. (3.10) for the first iteration. These linearized equations are solved to obtain H_j values at the end of first iteration, i.e. $H_{j(1)}$ values. These $H_{j(1)}$ values are then used to determine $C'_{x(1)}$ from eqn. (3.11). In the iterative procedure, if the values of ${}_tC_x{}'$ are taken equal to ${}_tC_{x(t-1)}{}'$, overcorrection occurs. Therefore it is suggested using of the average of the assumed and obtained values.

$$ {}_tC'_x = \frac{{}_{(t-1)}C'_x + C_{x(t-1)}{}'}{2}, x = 1,\dots\dots\dots\dots X \tag{3.13} $$

The iterative procedure continues until the assumed and obtained C_x' values and therefore the H_j values are sufficiently close. The unknown nodal flows are obtained in the end.

CHAPTER 4

DEVELOPMENT OF METHODOLOGY

LEAKSOL was developed to meet the need for a hydraulic analysis program in order to reduce water leakages in a network by describing a relationship between pressure-leakage and pressure-demand. LEAKSOL was coded in MATLAB programming language as separate script files. It is composed of two subprograms, CODE I and CODE II.

The methodology explained and applied for CODE I was primarily developed in the context of the studies performed by Germanopoulos and Jowitt (1989), Jowitt and Xu (1990), Vairavamoorthy and Lumbers (1998) and Araujo et al. (2006). These studies were aimed to minimize leakage flow by using a similar model with different objective functions and different optimization techniques.

The objective of any pressure control strategy should be to minimize excessive pressure as far as possible, while ensuring that sufficient pressures are maintained throughout the network to make sure that consumer demands are satisfied at all times. The idealized objective of such a strategy would be always to maintain a pressure profile in the network such that the pressure at each node is just sufficient to provide the corresponding demand. This is referred to as a target pressure level. However, target pressure levels can only be achieved by few nodes of the network while in the others the operational pressure remains higher. As the complexity of a distribution system grows, the task of achieving the target pressure level becomes more difficult and the average overpressure tends to increase.

This study has two algorithms. The first algorithm used in CODE I aims to minimize the system leakage through optimal flow control valve settings; the second algorithm used in CODE II determines the isolation valve settings through the hydraulic analysis of the network by adaptation of pressure dependent leakage and demand terms in order to reduce leakage flow. The potential economic benefits from such control schemes can be evaluated by comparison of the leakage volumes resulting in controlled and uncontrolled cases. In both of the algorithms, the nonlinear network equations describing nodal heads and pipe flows are augmented by terms that explicitly account for pressure-dependent leakage and pressure dependent demand terms. Successive linearization of these equations using the linear-theory method allows a program for reduction of leakage through the system. The addition of pressure dependent demand terms provides a solution for all combinations of isolation valve settings through partially satisfied demands.

4.1. Problem Formulation

A water distribution model is created by using a link-node formulation and the understanding of water movement in such a model requires knowledge of hydraulics of pressurized flow. The hydraulics, as mentioned in previous chapter, is subject to two physical laws. Those are the conservation of mass and the conservation of energy, in other words, node flow continuity relationship and loop head loss relationship. The node is a point where water consumption is allocated and defined as demand, which is treated as a known value. This formulation is valid only if the hydraulic pressures at all nodes are adequate so that the demand is independent of pressure. However, in case of pressure driven analysis, the value of consumption is treated as unknown and determined by using the allocated formulation defining the relation between pressure and demand (Chapter 2, eqn.2.9).

Real water distribution systems do not consist of a single pipe and can not be described by a single set of continuity and energy equations. Instead, one continuity equation must be developed for each node in the system, and one energy equation must be developed for each pipe (or loop) depending on the method used. Irrespective of whether a node (H-equations) or loop formulation (Q-equations) is used, the problem is solving the resulting system of simultaneous non-linear equations for the unknown network flows and nodal heads. A static network solution is thus obtained. Different methods have been proposed for static network solutions;

in this study Linear Theory Method was used and node flow continuity equations were written at each node.

The frictional head loss in a pipe can be expressed by a general head loss formula given in eqn.3.3. There are several pipe flow formulas available that relate flow and head losses under various flow regimes (Darcy-Weisbach, Hazen-Williams and Manning Formula). Hazen- Williams equation is not really valid for flows throughout the laminar, transition and rough regimes, but it has the advantage of computational simplicity and therefore receives wider application (Bhave, 1991). Although it is not quite suitable for old pipes, the formula is used in practice by reducing the Hazen-Williams coefficient values. For these reasons, Hazen-Williams equation was chosen to demonstrate the flow-head loss relation (eqn.3.5).

In the case of a valve located between nodes i and j, the relationship between the flow and head loss (Eqn.3.7a) can be expressed as follows:

$$Q_{ij} = V(k)\left(\frac{H_i - H_j}{K_x}\right)^{1/1.852} \tag{4.1}$$

where;

V(k) : a parameter that represents the setting of the k'th flow control valve.

V(k) : 0 means valve is fully closed

V(k) : 1 means valve is fully open

In general, there may be physical limits to V(k) that restrict its value to some range in the interval zero to unity.

Furthermore, the relation between pressure and leakage should be written and inserted into node flow continuity equation. The main difficulty is how to set the leakage coefficients relating pressure to leakage. To determine the leakage coefficients for a system, the leakage flow and the average pressure in the zone should be estimated. Germanopoulos (1985) used the following non-linear function between leakage and pressure by taking the power of N as 1.18 which is based on the results of field experiments performed by the Water Research Center (U.K.) (Goodwin, 1980). Then, Germanopoulos and Jowitt (1989) used the same data to obtain the mathematical representation employed in the Jowitt-Xu model (1990). Khadem et al., (1991) evaluated leakage for six different sectors of the city of Al-Riyadh, divided according to socioeconomic criteria. A comparison between the values reported by these authors and the leakage-pressure model proposed by Germanopoulos and Jowitt (1989) showed that the Al-Riyadh data are scattered around the curve of that model. Thus, this leakage-pressure model was adopted in the present study and was incorporated within the Jowitt-Xu (1990) formulation. Also, in accordance with IWA's and several authors' [Lambert (2000), Farley and Trow (2003) and Thornton (2003)] recommendations who suggested N value of 0.5 for fixed area leaks in metal pipes and N value of 1.0 in the absence of knowledge, different N values were used during the case study. Therefore, in the codes developed to reduce leakage flow, N was defined as variable parameter and its value is assigned depending on the user's choice.

The nonlinear relationship between the leakage and average service pressure can be approximated by the following function:

$$QS_{ij} = C_{l_{ij}}.L_{ij}.(P_{ij})^N \tag{4.2a}$$

$$QS_{ij} = k_{ij}.(P_{ij})^N \tag{4.2b}$$

where;

QS_{ij} : water leakage volume occurring in the pipe element of length Lij spanning nodes i and j.

C_{lij}: : a coefficient that relates the leakage per unit length of pipe to service pressure and depends on the system characteristics

k_{ij} : a composite parameter representing the product of C_{lij} and Lij

P_{ij} : average service pressure head that can be approximated by the average pressure head relative to the ground level at two ends of the element:

$$P_{ij} = 0.5[(H_i - G_i) + (H_j - G_j)] \qquad (4.3)$$

G_i : ground elevation at node i.

Using these equations, the node flow continuity equations with leakage terms included becomes,

$$\sum_{j \in Ri} Q_{ij} + 0.5 \sum_{j \in Ri} QS_{ij} + q_i = 0 \qquad i = 1, \ldots\ldots\ldots NPN \qquad (4.4)$$

q_i : demand at node i (m^3/s)

NPN : total number of nodes with unknown heads

R_i : set of nodes connected to node i

Note that the leakage QS_{ij} from element ij is apportioned equally to the nodes i and j. Substitution of eqn.4.3 into eqn.4.2 and then linearized form of eqn.'s 3.7a, 4.1 and 4.2 into eqn. 4.4 produces a set of NPN simultaneous equations. Hydraulic grade line elevations, flows and leakages can be determined by solving this set of node flow continuity equations, which is usually referred to as network static analysis. In this study, Linear Theory Method was selected for the solution of nonlinear equations.

In water distribution system analysis, if the number of the available independent equations is equal to the number of unknown parameters, a solution would be feasible. However, no method is available that can directly solve nonlinear equations and therefore an iterative procedure is necessary for their solution. Three methods are commonly used in practice for the iterative solution of these equations and thereby for the analysis of water distribution networks. These methods have been already explained in previous chapter briefly.

The linear theory method was initially developed in a loop formulation to determine the set of unknown flows (Wood and Charles 1972). Then the method was developed to solve for the nodal heads (Isaacs and Mills 1980). The extension of this method to accommodate pressure dependent leakage has been presented by Germanopoulos and

Jowitt (1989). The unknown nodal heads were taken as the basic unknown parameters in formulating H-equations. A summary of the linear-theory method for the problem under discussion is presented below:

The pipe flow / head difference relation in eqn.3.7 can be rewritten:

$$Q_{ij}^{m+1} = C_{ij}^{'}(H_i^{m+1} - H_j^{m+1}) \tag{4.5a}$$

where,

$$C_{ij}^{'} = \frac{\left|H_i^{m} - H_j^{m}\right|^{(1/1.852)-1}}{K_{ij}^{1/1.852}} \tag{4.5b}$$

In these equations, the terms with a subscript m are regarded as known variables and those with a subscript of $m+1$ represent the unknown variables.

After eqn. 4.2 is combined with eqn.4.3:

$$Qs_{ij}^{m+1} = \frac{RS_{ij}.\left[0.5(H_i^{m} + H_j^{m} - G_i - G_j)\right]^N}{(H_i^{m} + H_j^{m})}.(H_i^{m+1} + H_j^{m+1}) \tag{4.6a}$$

Let,

$$KS_{ij}^{m} = \frac{k_{ij}.\left[0.5(H_i^{m} + H_j^{m} - G_i - G_j)\right]^N}{(H_i^{m} + H_j^{m})} \tag{4.6b}$$

Then the equation becomes,

$$Qs_{ij}^{m+1} = KS_{ij}^{m}.(H_i^{m+1} + H_j^{m+1}) \tag{4.6c}$$

For the flow through the control valve:

$$Q_{ij}^{m+1} = C_{ij}^{'}(H_i^{m+1} - H_j^{m+1}) \tag{4.7a}$$

where;

$$C_{ij}' = V(k)\frac{\left|H_i^{\,m} - H_j^{\,m}\right|^{(1/1.852)-1}}{K_{ij}^{\;1/1.852}} \tag{4.7b}$$

Instead of embedding the valve setting, V (k), in the equation (eqn.4.7b) of initial conductance values, C'_{ij}, it is possible to show the effect of valve setting in node flow continuity equation directly. In this case, initial conductance value should be computed using eqn. 4.5b.

Eqn's 4.5, 4.6 and 4.7 can be used to linearize the pipe flow and leakage terms within the program. The only remaining nonlinearity is that involving the unknown valve settings. The pipe flow through valve can be defined as:

$$Q_{ij} = V(k).f_{q_{ij}} \tag{4.8}$$

$f_{q_{ij}}$: Fictitious pipe flow

The nominal change in the valve flow can be expressed by forming the total differential,

$$\Delta Q_{ij}^{m+1} = V(k)^m \Delta q_{ij} + f_{q_{ij}}^{\;m} \Delta V(k) \tag{4.9}$$

The valve flow at the current iteration can be considered as,

$$Q_{ij}^{m+1} = Q_{ij}^{m} + \Delta Q_{ij} \tag{4.10}$$

Note that;

$$\Delta V(k) = V(k)^{m+1} - V(k)^m$$
$$\Delta f_{q_{ij}} = f_{q_{ij}}^{\;m+1} - f_{q_{ij}}^{\;m}$$

Substituting eqn.'s. 4.8, 4.9, 4.10, one can obtain the following equation:

$$Q_{ij}^{m+1} = \left[V(k)^m . f_{q_{ij}}{}^{m+1}\right] + \left[f_{q_{ij}}{}^m . V(k)^{m+1}\right] - \left[V(k)^m . f_{q_{ij}}{}^m\right] \tag{4.11}$$

In this case, the fictitious pipe flow,

$$f_{q_{ij}}{}^{m+1} = C_{ij}'(H_i^{m+1} - H_j^{m+1}) \tag{4.12a}$$

$$C_{ij}' = \frac{\left|H_i^m - H_j^m\right|^{(1/1.852)-1}}{K_{ij}^{1/1.852}} \tag{4.12b}$$

Since consumptions are related to pressure, so to HGL, with nonlinear equations, they must be linearized to be involved in the Linear Theory Method. A linearization coefficient, D', which was described first by Nohutçu (2002), is put forward in order to linearly relate the consumptions and HGL values.

$$c_i = D_i' . H_i \qquad \text{i:1,.........NPN} \tag{4.13}$$

In this study, the model, fundamental equation of which is suggested by Chandapillai (1991) and then developed by Tanyimboh et al. (2001) was accepted as the most successful one among the models discussed in Chapter 2. Pressure dependent demand equation based on Chandapillai's approach can be rewritten by selecting Hazen-Williams head loss equation, i.e., n=1.852.

$$c_i = q_i^{req}\left(\frac{H_i - H_i^{\min}}{H_i^{req} - H_i^{\min}}\right)^{1/1.852} \qquad H^{\min} \leq H \leq H^{req} \tag{4.14a}$$

$$c_i^{m+1} = D_i' . H_i^{m+1} = \frac{q_i^{req}\left(\dfrac{H_i^m - H_i^{\min}}{H_i^{req} - H_i^{\min}}\right)^{1/1.852}}{H_i^m} . H_i^{m+1} \tag{4.14b}$$

Nohutçu (2002) approximated the minimum head (H^{min}) to the elevation of a node, so the minimum pressure (P^{min}) to zero, and after these modifications called the model as Modified Chandapillai Model. By applying these changes eqn. 4.14 becomes;

$$c_i = q_i^{req}\left(\frac{P_i}{P_i^{req}}\right)^{1/1.852} \qquad P^{\min} = 0 \le P \le P^{req} \tag{4.15a}$$

$$c_i^{m+1} = D_i'.H_i^{m+1} = \frac{q_i^{req}\left(\frac{P_i^m}{P_i^{req}}\right)^{1/1.852}}{H_i^m}.H_i^{m+1} \tag{4.15b}$$

where, P^{req} is a value above which consumers can take their required demand. This value is defined by operator for each node considering the characteristics of the distribution system.

Linearized form of all terms constituting node flow continuity equations were explained above. Substitution of eqn.'s 4.5, 4.6 and 4.15 into eqn. 4.4 produces a set of NPN simultaneous equations. Static solution of a system is maintained by solving the following equation.

$$\begin{aligned}&0.5.\sum_{j\in R_i} KS_{ij}^{\,m}.(H_i^{m+1}+H_j^{m+1})+c_i^{m+1}+\sum_{j\in R_i^1} C'_{ij}{}^{m}.(H_i^{m+1}-H_j^{m+1})+\\ &\sum_{j\in R_i^2}\left[V(k)^m.C'_{ij}{}^{m}.(H_i^{m+1}-H_j^{m+1})\right]=0 \qquad i=1,\ldots\ldots\ldots NPN\end{aligned} \tag{4.16}$$

where, R_i^1 is subsets of R_i that represents the sets of connections to node i through a pipe and R_i^2 is subsets of R_i that represents the sets of connections to node i through a flow control valve.

The application of Linear Theory Method for fixed demand analysis with addition of pressure dependent leakage terms on a sample network (Figure 4.1) can be summarized as follows where P is pipe number, J is node number, T is tank number and V is valve number:

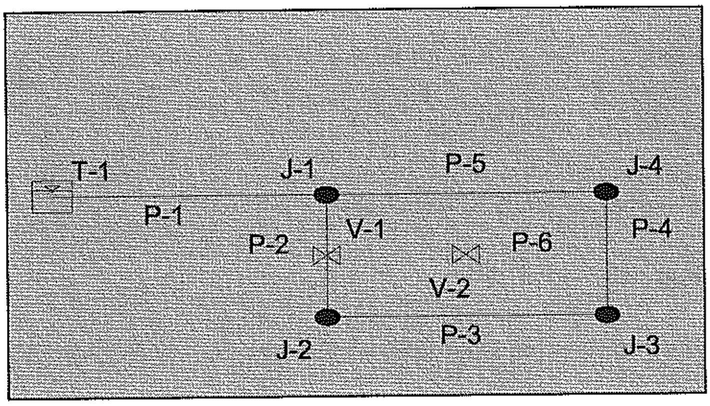

Figure 4.1 Sample network

For fixed demand analysis, $c_i=q_i^{req}$, required demand at node i. By applying eqn. 4.16 for each demand node, i.e., J-1, J-2, J-3 and J-4, and by rearranging them, the problem can be converted to a simple matrix form.

$$coef * H = rh \tag{4.17a}$$

where, *coef* is a symmetrical coefficient matrix involving C_{ij}' and KS_{ij} values, *rh* denotes the right-hand matrix and *H* is the unknown matrix involving the unknown nodal heads for the network.

For *coef* matrix, elements of the diagonal of the matix, *coef(i,i)* consists of the summation of negative of C_{ij}' values, where *i* is connected to *j* through a pipe or through a valve, and half of the negative summation of KS_{ij} values depending on whether a pipe is lying between two demand nodes or between a fixed grade node, i.e, a source node, and a demand node.

For a pipe between two demand nodes, leakage is equally distributed between nodes. However, for a pipe between a fixed grade node (FGN) and demand node, it is assumed that all of the leakage is extracted only from demand node (Figure 4.2).

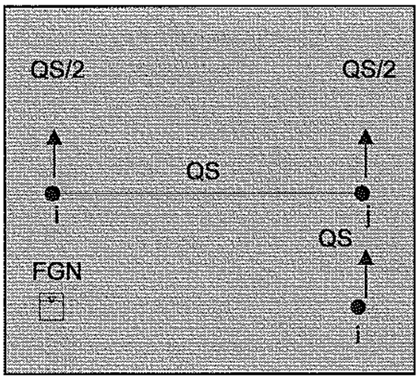

Figure 4.2 Leakage between nodes

Then, *coef (i,i)* can be generalized in he following way;

$$coef(i,i) = -0.5 \sum_{j \in R_i} KS_{ij}{}^{m} - \sum_{FGN \in R_i} KS_{iFGN}^{m} - \sum_{j \in R_i{}^1} C'_{ij}{}^{m} - \sum_{j \in R_i^2} V(k)^{m}.C'_{ij}{}^{m} \qquad (4.17b)$$

Remaining elements of *coef* matrix, i.e., *coef(i,j)*, $i \neq j$, are equal to the summation of negative KS_{ij} values and the modified conductance of pipes connecting nodes i and j where i is connected to j through a pipe or through a valve.

$$coef(i,j) = -0.5.KS_{ij}{}^{m} + C'_{ij}{}^{m} \quad or \quad coef(i,j) = -0.5.KS_{ij}{}^{m} + V(k)^{m} C'_{ij}{}^{m} \qquad (4.17c)$$

The rh matrix generally involves demands for nodes. But, if there is a node i that is connected to a fixed grade node (FGN), modified conductance of the pipe times HGL value of fixed grade node should be subtracted and KS value of pipe times HGL value of FGN should be added to the demand value for the nodes at downstream of the pipe and both of these terms should be added to the demand value for the nodes at upstream of the pipe.

The application of Linear Theory Method for pressure dependent demand analysis with addition of pressure dependent leakage terms on a sample network (Figure 4.1) can be explained as follows:

The difference between the application of methodology for fixed demand analysis and pressure dependent demand analysis is that demand terms in eqn. 4.17 are replaced with linearized consumption terms in eqn. 4.15b and rearranged. Matrix form of this application can be seen in following page. The demand values in *rh* matrix are replaced with zeros since they are not fixed and linearization coefficients of consumptions are subtracted from diagonal elements of the *coef* matrix.

As a summary, the static analysis of any network depending on the type of the analysis chosen, i.e., fixed demand or pressure dependent demand analysis, can be performed mainly by constituting *coef* and *rh* matrices and solving them for unknown nodal heads. Then, linearization coefficients are calculated at the end of each iteration by using nodal heads obtained and they are used for the next iteration until convergence criterion is satisfied.

Static analysis mentioned has to be carried out by assuming flow control valve settings as fully open initially in order to find hydraulically consistent nodal heads prior to the computation of optimum valve settings.

In this optimization problem, the objective function was selected as minimization of total leakage in the system. The constraints of the problem are primarily node flow continuity equations including pressure dependent leakage and pressure dependent demand terms which is optional, i.e., in the optimization code the user has to chance to select the type of analysis as fixed demand or pressure dependent demand. Linearization of node flow continuity equations were performed by applying Linear Theory Method and this procedure was based on not only the pressures of previous iteration but also the valve settings of the previous iteration. Optimization problem was solved by using Linear Programming method. This optimization technique is an iterative procedure that involves linearization of the objective function and constraints about a current point and then solving the resulting linear program to obtain the new

Application of Linear Theory Method for fixed demand analysis on the sample network given in Figure 4.1.

$$\begin{bmatrix} -C_1'-V_1C_2'-C_5'-KS_1-0.5KS_2-0.5KS_5 & V_1C_2'-0.5KS_2 & 0 & C_5'-0.5KS_5 \\ V_1C_2'-0.5KS_2 & -C_3'-V_1C_2'-V_2C_6'-0.5KS_3-0.5KS_2-0.5KS_6 & C_3'-0.5KS_3 & V_2C_6'-0.5KS_6 \\ 0 & C_3'-0.5KS_3 & -C_3'-C_4'-0.5KS_3-0.5KS_4 & C_4'-0.5KS_4 \\ C_5'-0.5KS_5 & V_2C_6'-0.5KS_6 & C_4'-0.5KS_4 & -C_4'-C_5'-V_2C_6'-0.5KS_4-0.5KS_5-0.5KS_6 \end{bmatrix} * \begin{bmatrix} H_1 \\ H_2 \\ H_3 \\ H_4 \end{bmatrix}$$

$$= \begin{bmatrix} q_1^{req}-C_1'H_s-KS_1H_s \\ q_2^{req} \\ q_3^{req} \\ q_4^{req} \end{bmatrix} \qquad (4.17d)$$

Application of Linear Theory Method for variable demand analysis on the same network given in Figure 4.1.

$$\begin{bmatrix} -\dot{C}_1 - V_1\dot{C}_2 - \dot{C}_5 - KS_1 - 0.5KS_2 - 0.5KS_5 - \dot{D}_1 & V_1\dot{C}_2 - 0.5KS_2 & 0 & \dot{C}_5 - 0.5KS_5 \\ V_1\dot{C}_2 - 0.5KS_2 & -\dot{C}_3 - V_1\dot{C}_2 - V_2\dot{C}_6 - 0.5KS_3 - 0.5KS_2 - 0.5KS_6 - \dot{D}_2 & \dot{C}_3 - 0.5KS_3 & V_2\dot{C}_6 - 0.5KS_6 \\ 0 & \dot{C}_3 - 0.5KS_3 & -\dot{C}_3 - \dot{C}_4 - 0.5KS_3 - 0.5KS_4 - \dot{D}_3 & \dot{C}_4 - 0.5KS_4 \\ \dot{C}_5 - 0.5KS_5 & V_2\dot{C}_6 - 0.5KS_6 & \dot{C}_4 - 0.5KS_4 & -\dot{C}_4 - \dot{C}_5 - V_2\dot{C}_6 - 0.5KS_4 - 0.5KS_5 - 0.5KS_6 - \dot{D}_4 \end{bmatrix} * \begin{bmatrix} H_1 \\ H_2 \\ H_3 \\ H_4 \end{bmatrix} = \begin{bmatrix} -\dot{C}_1 H_s - KS_1 H_s \\ 0 \\ 0 \\ 0 \end{bmatrix}$$

solution point. This new solution point is again used to linearize the objective function and constraints. The procedure is repeated until a termination criterion is satisfied.

4.2. Linear Programming

Linear programming (LP) is one of the most widely used optimization techniques. LP models have two basic characteristics, that is, both the objective function and constraints are linear functions of the decision variables. The general form of a LP model can be expressed as:

$$Max\,(or\,Min)\ x_0 = \sum_{j=1}^{w} z_j x_j \tag{4.18}$$

Subject to:

$$\sum a_{ij} x_j = b_i \qquad \text{for i=1,2,..........,g} \tag{4.19}$$

$$x_j \geq 0 \qquad \text{for j=1,2,..........,d} \tag{4.20}$$

where ;

z_j : objective function coefficient

a_{ij} : technological coefficients

b_i : right-hand side (RHS) coefficient

In algebraic form, this LP model can be expressed as:

$$Max\,(or\,Min)\ \ x_0 = z_1x_1 + z_2x_2 + \ldots\ldots\ldots + z_dx_d \tag{4.21}$$

Subject to

$$\begin{array}{l}
a_{11}x_1 + a_{12}x_2 + \ldots\ldots\ldots + a_{1d}x_d \leq b_1 \\
a_{21}x_1 + a_{22}x_2 + \ldots\ldots\ldots + a_{2d}x_d \leq b_2 \\
\quad \vdots \qquad \vdots \qquad\qquad \vdots \qquad\qquad \vdots \\
a_{g1}x_1 + a_{g2}x_2 + \ldots\ldots\ldots + a_{gd}x_d \leq b_g
\end{array}$$

$$x_1 \geq 0,\ \ x_2 \geq 0, \ldots\ldots\ldots, x_d \geq 0$$

Alternatively, in matrix form, an LP model can be expressed as:

$$Max(or\ Min)\ \ x_0 = z^T x \tag{4.22}$$

Subject to:

$$Ax \le 0 \tag{4.23}$$

$$x \ge 0 \tag{4.24}$$

where z is an d by 1 column vector of objective function coefficients, x is an d by 1 column vector of decision variables, A is an g by d matrix of technological coefficients, b is an g by 1 column of the RHS coefficients and the subscript T represents the transpose of a matrix or a vector.

The aim of the optimization of this work is to find the settings of flow reduction valves placed in the network to minimize leakage from the network. The objective function of this linear programming problem is described as function of nodal heads.

The general optimization problem can be stated mathematically in terms of the nodal heads H and the valve settings V(k) as follows:

Minimize $f(H)$ (4.25)

Subject to

$g\ (H,V)$ (4.26)

$H^L \le H \le H^U$ (4.27)

$V^L \le V \le V^U$ (4.28)

where (4.25) represents the objective function; (4.26) are the nodal flow continuity equations including pressure-dependent outflow terms (for demand and leakage); and (4.27) and (4.28) are the bound constraints for pressure heads and valve settings, respectively.

The basic concept of the mathematical formulation of the Linear Programming problem is the incorporation of the nodal pressures and valve settings in pipes where valves are located as continuous variables in the constraint set and the objective function. The constraints are the governing node equations that include the nonlinear node flow continuity relationships.

The objective function incorporated into the optimization model is the minimization of the total volume of leakage,

$$\underset{V(k)}{Min} \sum_{ij \in R} QS_{ij} \tag{4.29}$$

where R is a set of consisting of all pairs of connected nodes.

The general constraints are the node flow continuity equations which include pressure-dependent outflow terms for leakage. In the nodal formulation the state variables are the unknown nodal heads. The number of equations and unknowns is equal to the number of demand nodes in the network.

Since the main duty of a distribution network is to satisfy the consumer demand, nodal heads should be maintained above some specified critical values. Restrictions on some reference nodes will generally dominate the optimization problem. During the execution of linear programming subprogram for CODE I, any solution not satisfying the critical head criterion is disregarded which causes loss of solutions.

However, for determination of the best valve combination reducing the leakage volume using the hydraulic analysis of the system, i.e., CODE II, critical head criterion is considered; any solution not satisfying this criterion is reported giving chance to user selecting the best combination of valves satisfying the situation of the network.

The critical head requirement for both type of analysis can be identified either (1) for some critical nodes such as those with highest elevation, remoteness from the source and maximum load, etc. or (2) for all nodes to obtain a certain pressure level above the ground level.

The first case can be incorporated into the problem as:

$$H_i \geq Hi_{critical} \qquad i = 1..........,NR \tag{4.30}$$

where;

NR : number of the reference nodes selected

$Hi_{critical}$: critical head requirement at selected reference node i.

For the second case;

$$H_i \geq G_i + PRES \tag{4.31}$$

where;

G_i : ground elevation of node i

PRES : adequate pressure head for desired operating conditions

Criteria for assessing acceptable pressures in a distribution network may vary from system to system. Therefore, there are no universally acceptable pressure ranges. For example, according to Walski (2000), the threshold pressure value below which a failure to satisfy the full demands occurs is usually assumed to be below the minimum of 20 m. On the other hand, water companies set minimum acceptable pressure standards such as 15 m while the same standard could be as high as 25m (three story building), to allow for possible increases in demand (Tanyimboh et al. 1999).

For the inclusion of pressure dependent demand terms, the model, fundamental equation of which is suggested by Chandapillai (1991) and then developed by Tanyimboh et al. (2001) was selected (eqn. 4.14). This equation was modified by Nohutçu (2002) by approximating the minimum head (H^{min}) to the elevation of a node so the minimum pressure (P^{min}) to zero and called as Modified Chandapillai Model (eqn. 4.15).

The node flow continuity equations written for all demand nodes with the addition of pressure dependent demand terms create a set of non-linear network equations to be solved. H equations in which the unknown nodal heads are taken as the basic unknown parameters, are selected to constitute network equations. The Linear-Theory method is used for linearization of these equations.

Constraints on valve operational range can be expressed as

$$V^{\min}(k) \le V(k) \le V^{\max}(k) \qquad k = 1,\ldots\ldots NV \qquad (4.32)$$

where, NV is number of flow control valves in the network

Consequently, the minimization of the leakage volume is achieved through the solution of the following mathematical program:

Objective function:

$$Min \sum_{ij\in R} KS_{ij}{}^{m}.(H_i{}^{m+1} + H_j{}^{m+1}) \qquad (4.33a)$$

Subject to:

$$0.5.\sum_{j\in R_i} KS_{ij}{}^{m}.(H_i^{m+1} + H_j^{m+1}) + c_i{}^{m+1} + \sum_{j\in R_i{}^1} C'_{ij}{}^{m}.(H_i{}^{m+1} - H_j{}^{m+1}) +$$
$$\sum_{j\in R_i{}^2}\left[V(k)^m.C'_{ij}{}^{m}.(H_i{}^{m+1} - H_j{}^{m+1}) + f_{q_{ij}}{}^{m}V(k)^{m+1} - f_{q_{ij}}{}^{m}.V(k)^m\right] = 0 \quad i = 1,\ldots.NPN$$

(4.33b)

$$H_i^{m+1} \ge Hi_{critical} \qquad i = 1\ldots\ldots\ldots, NR \qquad (4.33c)$$

$$V^{\min}(k) \le V(k)^{m+1} \le V^{\max}(k); \qquad k = 1,\ldots\ldots NV \qquad (4.33d)$$

$$H_i \ge 0, \qquad i = 1,\ldots\ldots NPN \qquad (4.33e)$$

where R_i^1 and R_i^2 subsets of R_i that represent the set of connections to node i through a pipe and a flow-control valve.

The values in the above equations with a subscript m are constant and those with a subscript of $m+1$ are decision variables.

The application of Linear Programming algorithm for the sample network given in Figure 4.1 can summarized as follows:

The first constraints i.e., node flow continuity equations, are written by using eqn. 4.33b. eqn. 4.34 (next page) shows the linearized node flow continuity equations in the matrix form.

The second constraint is identification of critical head requirements for some selected nodes or for all nodes. For example, it can be assumed for this sample network that nodal heads should be greater than or equal to 30 m above the ground elevation. In this case;

$$\text{Constraint 2:} \begin{cases} H_1 \geq G_1 + 30 \\ H_2 \geq G_2 + 30 \\ H_3 \geq G_3 + 30 \\ H_4 \geq G_4 + 30 \end{cases}$$

Third constraint is the valve control restrictions for two valves in the sample.

$$\text{Constraint 3:} \begin{cases} 0 \leq V_1 \leq 1 \\ 0 \leq V_2 \leq 1 \end{cases}$$

Finally, the objective function is,

$$Min \begin{bmatrix} KS_1(H_1 + H_s) + KS_2(H_1 + H_2) + KS_3(H_2 + H_3) + KS_4(H_3 + H_4) \\ + KS_5(H_1 + H_4) + KS_6(H_2 + H_4) \end{bmatrix}$$

Constraint 1: Matrix form of node flow continuity constraint in Linear Programming problem for the sample network given in Figure 3.1.

$$
\begin{bmatrix}
\begin{array}{c}-C_1' - V_1C_2' - C_5' \\ -KS_1 - 0.5KS_2 - 0.5KS_5 - D_1'\end{array} & V_1C_2' - 0.5KS_2 & 0 & C_5' - 0.5KS_5 & -f_{q_{1-2}} & 0 \\
V_1C_2' - 0.5KS_2 & \begin{array}{c}-C_3' - V_1C_2' - V_2C_6' \\ -0.5KS_3 - 0.5KS_2 - 0.5KS_6 - D_2'\end{array} & C_3' - 0.5KS_3 & V_2C_6' - 0.5KS_6 & -f_{q_{2-1}} & -f_{q_{2-4}} \\
0 & C_3' - 0.5KS_3 & \begin{array}{c}-C_3' - C_4' - 0.5KS_3 \\ -0.5KS_4 - D_3'\end{array} & C_4' - 0.5KS_4 & 0 & 0 \\
C_5' - 0.5KS_5 & V_2C_6' - 0.5KS_6 & C_4' - 0.5KS_4 & \begin{array}{c}-C_4' - C_5' - V_2C_6' - 0.5KS_4 \\ -0.5KS_5 - 0.5KS_6 - D_4'\end{array} & -f_{q_{4-2}} & 0
\end{bmatrix}
* \begin{bmatrix} H_1 \\ H_2 \\ H_3 \\ H_4 \\ V_1 \\ V_2 \end{bmatrix}
= \begin{bmatrix} -C_1'H_s - KS_1H_s - f_{q_{1-2}}V_1 \\ -f_{q_{2-1}}V_1 - f_{q_{2-4}}V_2 \\ 0 \\ -q_{4-2}V_2 \end{bmatrix}
\tag{4.34}
$$

4.3. Algorithm for CODE I

The theory for CODE I can be summarized as follows:

The nonlinear network equations describing nodal heads and pipe flows are augmented by terms that explicitly account for pressure-dependent demand and leakage and by terms that model the effect of valve actions. Successive linearization of these equations using the linear-theory method allows a linear program that minimizes leakage. After the solution of the program (eqn.4.16) the new values for the heads and valve settings are used to relinearize the hydraulic constraint for the next iteration within the linear program. Iterations continue until convergence is obtained. The model applied has two parts: (1) static analysis part which is used like a network solver and (2) optimization part to minimize leakage volume by selected valve openings.

The iterative procedure for the proposed approach can be presented in that way:

i) ***Static Analysis Part:***

1-Initially, node, pipe, valve and source data is taken as text files and pipe resistance coefficients, R_{ij}, are calculated. These values do not change until the end of the program.

2-For the first iteration, initial valve settings, V (k)'s guessed.
(Generally, the valves are initially set as fully open, i.e., V (k) =1.0).

3-The element-flows (Q_{ij}) and leakage terms (QS_{ij}) are initialized and a static analysis is carried out using the linear-theory method to produce a set of hydraulically consistent nodal heads H_i, element flows Q_{ij}, and leakage values, QS_{ij}.

Element flows can be computed by taking flow velocities of about 1 m/s for the first iteration.

$$Q_{ij} = V_{ij} A_{ij} = \frac{\pi D_{ij}^2}{4} \tag{3.48}$$

In the same way, it is possible to initialize leakage values (QS_{ij}) as equal to $0.001 m^3/s$.

4-The resultant nodal heads H_i are checked to ensure that they satisfy the critical head requirement. If not, the valve settings are reset and returned to step (2).

At the end of the static analysis, a set of hydraulically consistent nodal heads, H_i^m satisfying critical head criterion at the selected nodes, element flows Q_{ij}^m, and leakage values QS_{ij}^m are obtained. They are accepted as input for optimization part.

ii) ***Optimization Part:***

5-New conductance values and linearization coefficients for leakage and consumption are constituted by using heads from the first iteration.

For fixed demand analysis, conductance values for consumption are taken as zero.

6-The *coef* and *rh* matrices are constituted.

7-The objective function is linearized using estimates of leakage terms, QS_{ij}^m.

8-The system hydraulic constraints are linearized using the computational flows Q_{ij}^m, leakage QS_{ij}^m and pressure dependent nodal consumptions.

9-The linear program is solved to obtain new nodal heads, H_i^{m+1} and valve settings, $V(k)^{m+1}$.

10-The new element flows Q_{ij}^{m+1} and leakage terms QS_{ij}^{m+1} are computed using heads and valve settings obtained at step 10.

11-The differences in nodal heads between the last two successive iterations are computed. If all the differences are within the specified tolerance which was selected as 0.0001 m, the program terminates. Otherwise update the iteration number, m=m+1, obtain the new computational conductance terms by using the following equation and go to step 5.

Convergence can be accelerated if the estimates of conductance terms take the following form after first iteration:

$$C_{ij}^{\prime m} = 0.5(C_{ij}^{\prime m} + C_{ij}^{\prime m-1}) \qquad (4.36)$$

The iterative procedure is terminated when the maximum head difference $\delta H_i = H_i^m - H_i^{m-1}$ in successive iterations is less than some specified tolerance, ε .

$$Max_{i \in N}\left|\delta H_i\right| \leq \varepsilon \qquad (4.37)$$

12-If maximum head difference in successive iterations is less than the defined tolerance; nodal pressures, consumptions and leaks, pipe discharges, velocities, head losses and leakages are computed and saved in the form of text files.

Elapsed time for execution, total consumption, demand, total pipe leakages and valve openings are showed on the screen and the program is ended.

Figure 4.3 shows the algorithm of the optimization program (CODE I). The linear programming problem defined above is solved by using Revised Simplex Method. Mathematical details of this method will not be mentioned in this thesis study.

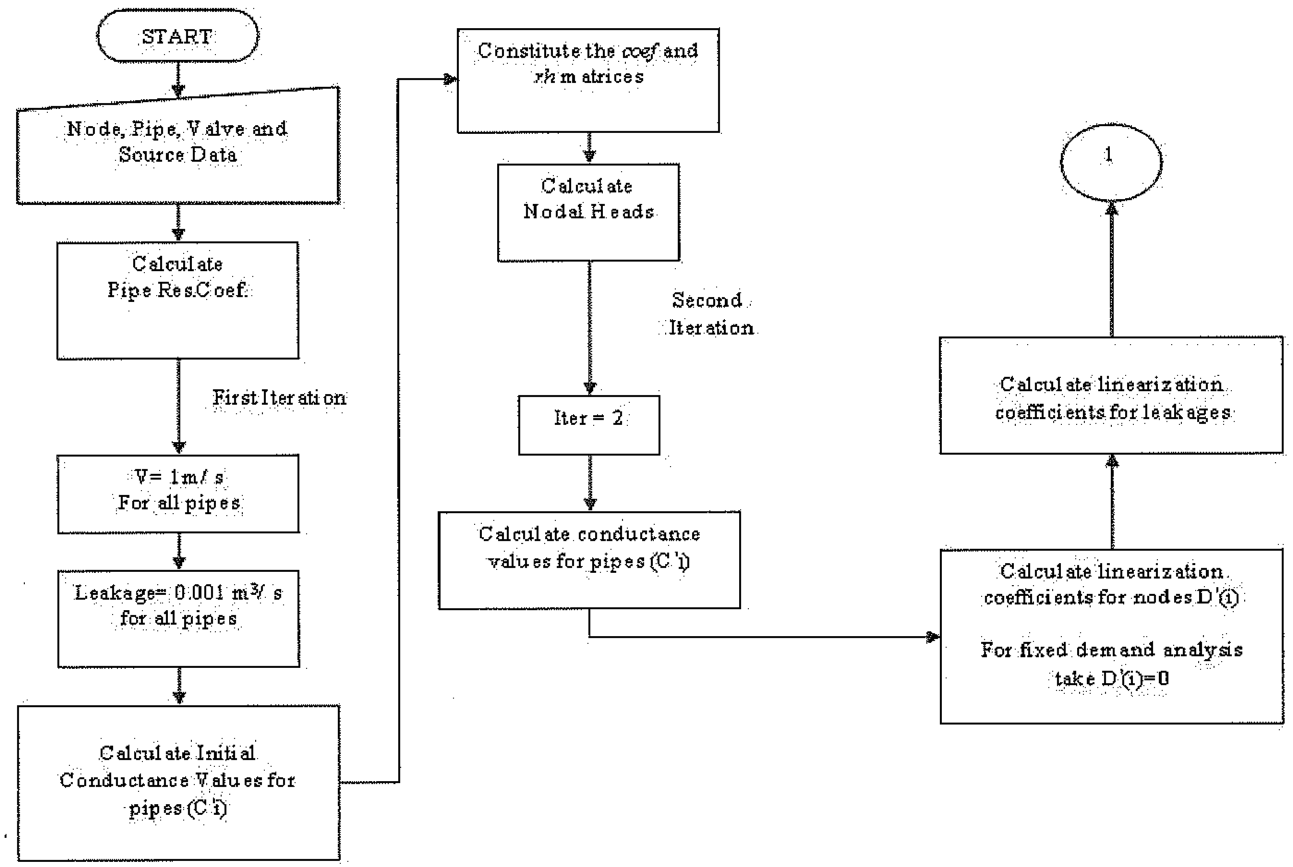

Figure 4.3 Algorithm for optimization program (CODE I)

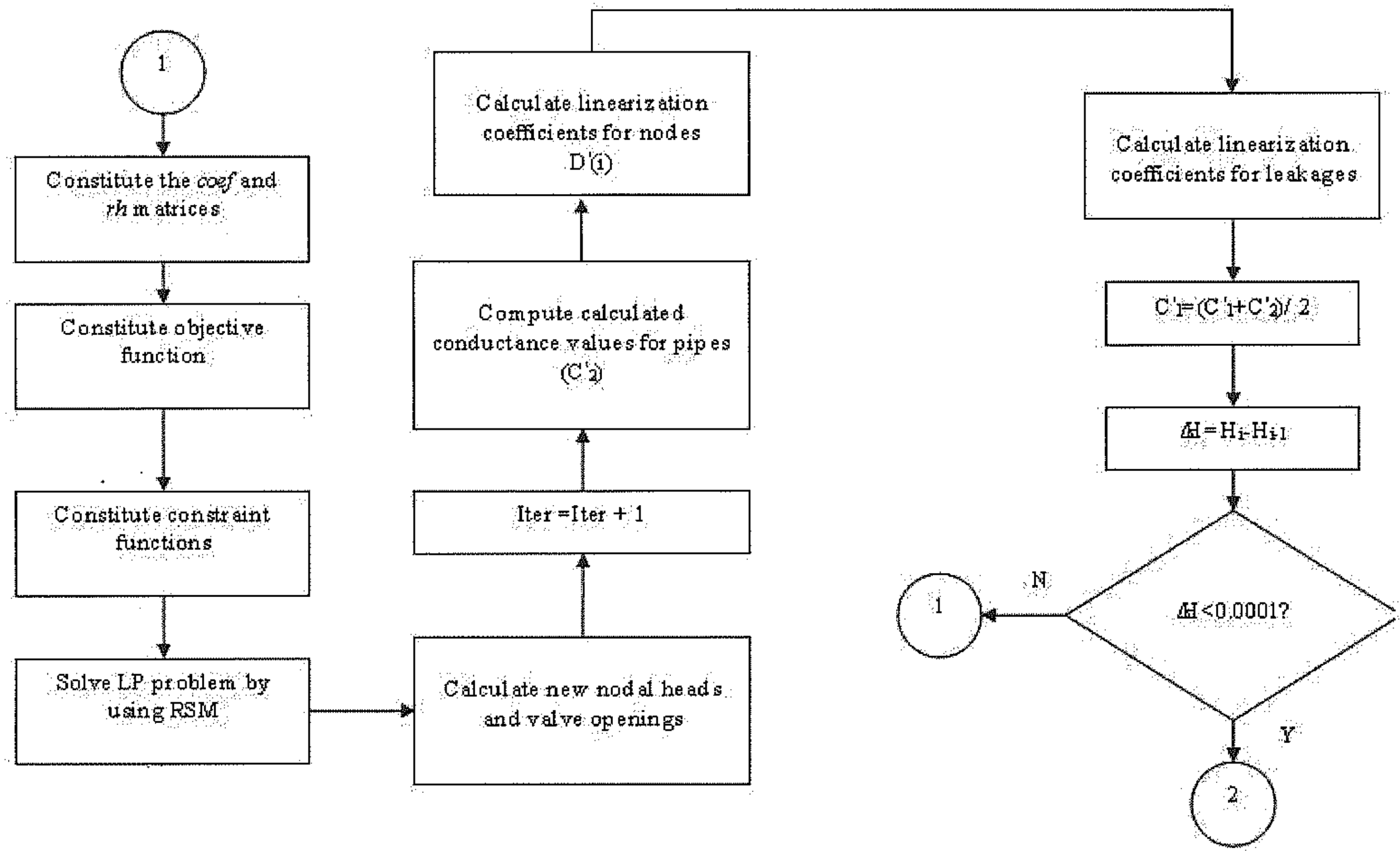

Figure 4.3 (cont'd) Algorithm for optimization program (CODE I)

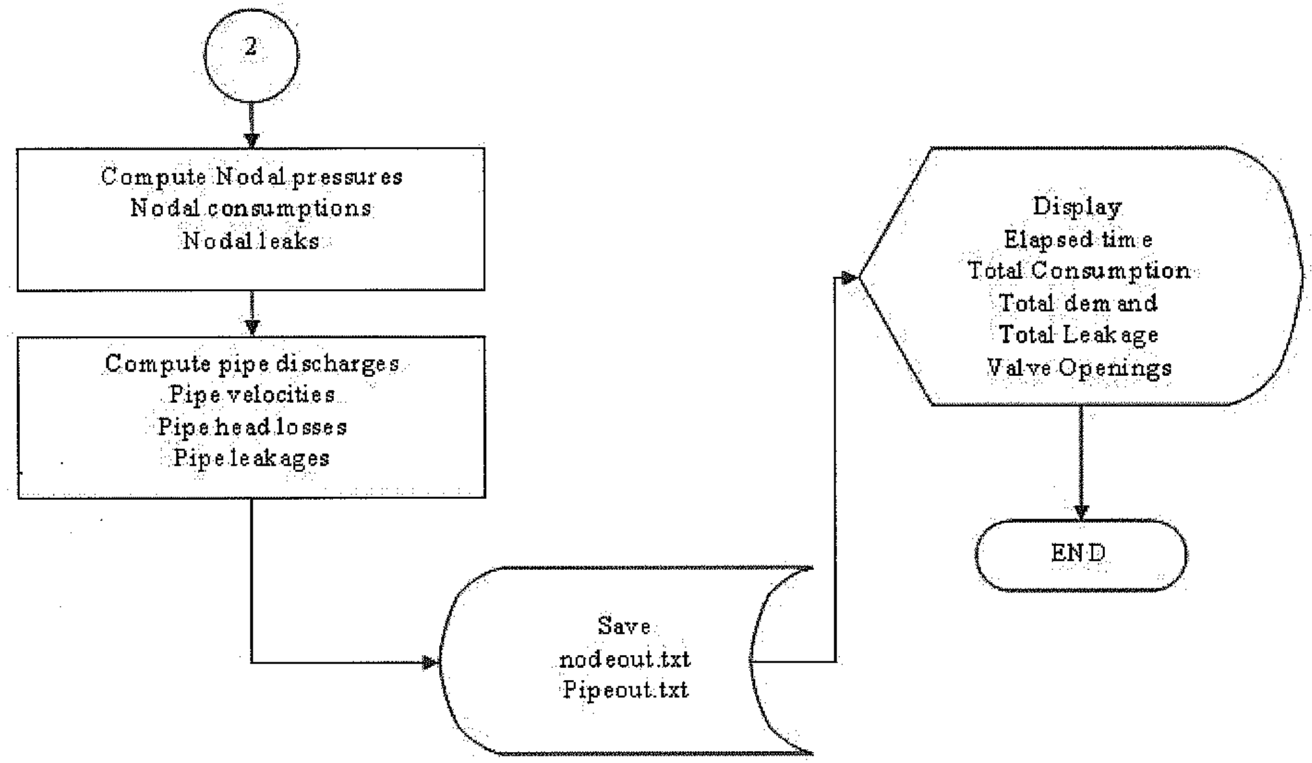

Figure 4.3 (cont'd) Algorithm for optimization program (CODE I)

4.4. Algorithm for CODE II

The objective for CODE II is working on an algorithm to determine the isolation valve settings through the static analysis of the network by adaptation of pressure dependent demand terms in order to reduce leakage flow. This code is a kind of hydraulic network solver making pressure dependent demand analysis by considering the relationships between pressure and leakage for lots of valve combinations generated depending on the number of valves. Generation of valve combinations realized in the following way:

Generating Subsets

A set is a collection of objects and a subset describes a selection of objects, where the order among them does not matter (Skiena, 1997).

The set of subsets of a set S is called the power set of S, and a set of r elements has 2^r subsets (including both the set itself and the empty set). This follows from the fact that the total number of distinct p-subsets on a set of r elements is given by the binomial sum.

$$\sum_{p=0}^{r} \binom{r}{p} = 2^r \tag{4.38}$$

Three most common methods of enumerating the subsets of a set are Lexicographic ordering, Gray codes and Binary Counting. Binary Counting is the simplest approach to subset generation problem in which the subsets are represented by binary string of r bits. In this study, valve combinations to be used in the static analysis were generated in Binary Counting and represented as a set by bit-strings (strings of 0's and 1's).

This binary representation is the key to solving all subset generation problems. To generate all subsets in order, simply it is counted from 0 to 2r-1 where r is the number of elements in the set. For each integer, each of the bits is successively masked off and a subset of exactly the items corresponding to `1' bits is composed. To generate the *next* or *previous* subset, the integer is increased or decreased by one (Skiena, 1997).

Number of valves selected in the network determines the number of case examined at each run of the static program. A case defines a series of valve settings showing that either a valve is fully open or closed.

Table 4.1 Bit-string representation of subsets for 3 valves

Rank	Bit-String Representation	Subset
0	[0,0,0]	ø
1	[0,0,1]	{3}
2	[0,1,0]	{2}
3	[0,1,1]	{2,3}
4	[1,0,0]	{1}
5	[1,0,1]	{1,3}
6	[1,1,0]	{1,2}
7	[1,1,1]	{1,2,3}

For the subprogram used to generate subsets, the only input parameter is the number of valves selected in the system. Output is the equivalent bit-string representation of elements of the subsets. Table 4.1 shows the result of this subprogram for 3 valves (i.e. NV=3). Second column of the table gives the valve settings for each case. The total case number is equal to 2^{NV}.

Valve combinations are determined by generating subsets subprogram. After the system is solved, the leakage volume and consumptions are computed for each case and the critical head requirement described for each node are checked and reported. The addition of pressure dependent demand terms provides the computation of consumption for all combinations of isolation valve settings.

The model applied has two parts:

a-Generation of valve combinations: Depending on the number of valves selected, total case number and the valve settings for each case are determined.

b-Static Analysis Part: This part is used like a hydraulic network solver for each set of valve combinations. The nonlinear network equations describing nodal heads and pipe flows written for all demand nodes are augmented by terms that explicitly account for pressure-dependent leakage and pressure-dependent demand terms. Successive linearization of these equations using the linear-theory method allows a program for reduction of leakage through the system.

Static analysis of the network can be achieved by solving equations 4.5, 4.6, 4.15 and 4.16. The iterative procedure for the static analysis part can be summarized as follows:

1-Initially, node, pipe, valve and source data is taken as text files and pipe resistance coefficients, R_{ij}, are calculated. These values do not change until the end of the program.

2-Then, the code checks for existence of valves. If there is valve in the network, subsets are generated and case number, CASENO is set as 1; otherwise code solves system for no-valve case.

3-For the first iteration, initial valve settings, V (k)'s are taken from the first case of valve combinations.

4-The element-flows (Q_{ij}) and leakage terms (QS_{ij}) are initialized and a static analysis is carried out using the linear-theory method to produce a set of hydraulically consistent nodal heads H_i, element flows Q_{ij}, and leakage values, QS_{ij}.
Element flows are computed by taking flow velocities of about 1 m/s for the first iteration (eqn.4.35).

5-For the next iteration; pipe conductance values, linearization coefficients for consumptions and leakages are computed by using nodal heads from the previous iteration.

6-The *coef* and *rh* matrices are formed and solved to obtain new nodal heads.

7-The differences in nodal heads between the last two successive iterations are computed. If all the differences are within the specified tolerance which was selected as 0.0001 m, the program terminates (eqn. 4.37). Otherwise the iteration number is updated, $m=m+1$, and to accelerate the convergence, the average of the assumed and obtained computational conductance terms are calculated by using eqn. 4.36 and returned to Step 3.

8-If maximum head difference in successive iterations is less than the defined tolerance; the resultant nodal heads are checked to ensure that they satisfy the critical head requirement if identified and reported at the end of the analysis.

Nodal pressures, consumptions and leaks, pipe discharges, velocities, head losses and leakages are computed for this case and saved in the form of text files.

9-Case number is compared to total number of valve combinations. If case number is less than 2^{NV} then returned to Step 3 and algorithm is repeated with the next valve combination. Otherwise, closed pipe scenarios for all cases are summarized in text files and the program is ended.

After the system is solved for each combination of valve settings, the leakage volume and partial demands are computed and reported with nodal heads and pipe discharges for each case. Also, for each case, critical head requirement is checked. If hydraulic grade line at any node is less than the critical head value defined at this node, this case is reported. The comparison of these results with each other by an authority can help for the selection of most suitable valve combination satisfying consumptions fully or partially by knowing whether the critical head criterion is satisfied or not. Selecting whether the case in which demands are fully satisfied or any case in which a percentage of total consumption is provided with a decrease in leakage volume depends on operating policy of the municipality.

In this code, also it is possible to identify open/closed pipes in the input data file. So that, program executes by taking closed pipes as fixed and changing the valve setting of other pipes as defined in each case. This makes possible the hydraulic simulation of networks under pipe failure conditions. Results close to real situation can be obtained with addition of pressure dependent leakage and demand relations.

Figure 4.4 shows the program algorithm for CODE II.

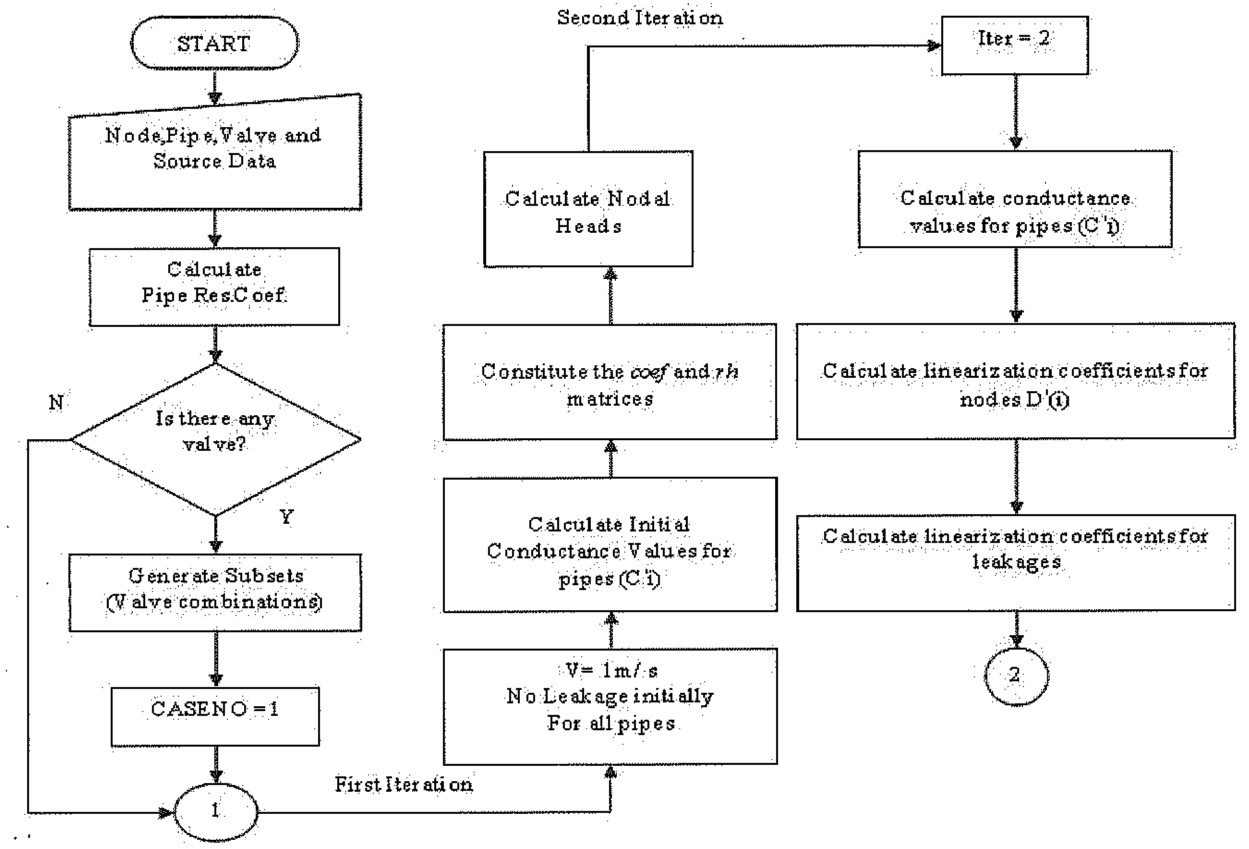

Figure 4.4 Algorithm for CODE II

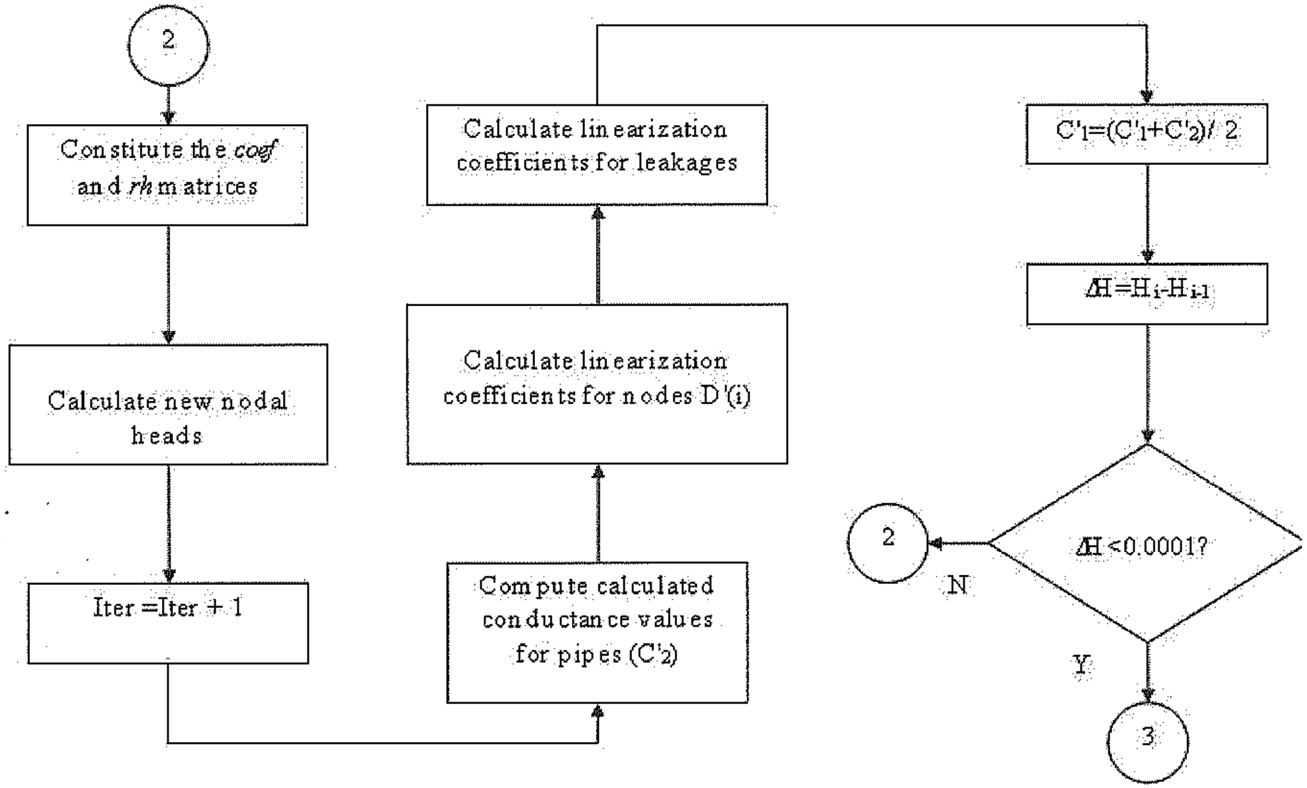

Figure 4.4 (cont'd) Algorithm for CODE II

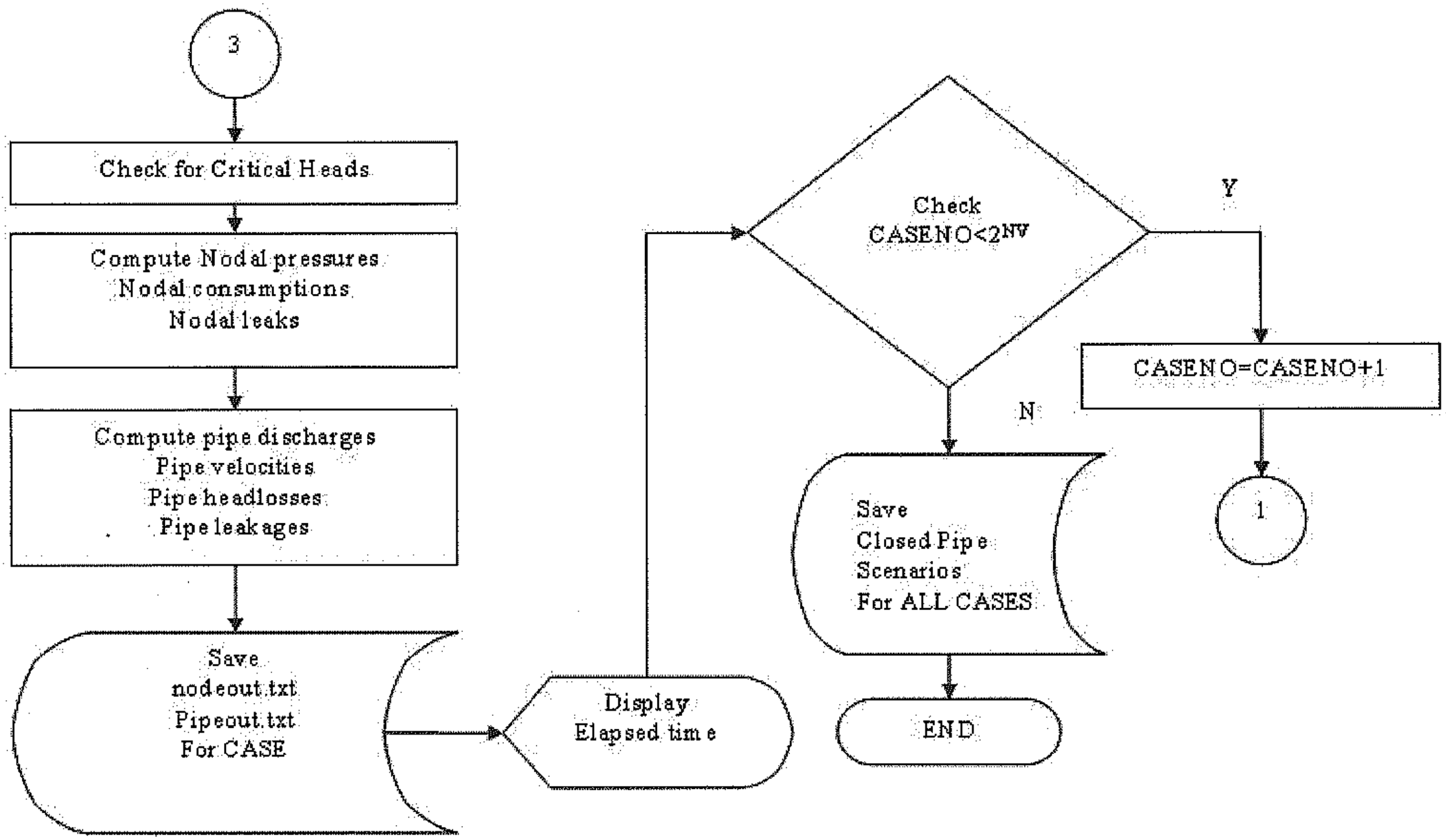

Figure 4.4 (cont'd) Algorithm for CODE II

CHAPTER 5

CASE STUDY

5.1. Introduction

Numerous software programs have been written for detailed hydraulic analysis of complex water distribution networks. However, most standard, commercially available hydraulic network solvers are based on the assumption that nodal demands are fixed and known. There is still no software taking the pressure dependent leakage and demand terms into account directly. Due to this deficiency, a computer program with two sub-programs was developed to reduce leakage in two ways. The first code, CODE I, provides solution by using optimization techniques with defined pressure-leakage and pressure-demand relations in order to find optimal flow control valve settings minimizing water leakage. The second one, CODE II, makes hydraulic analysis of the network in order to solve the system and to compute the amount of leakage and the amount of water consumed, by using different combinations of isolation valves generated according to the number of valves given and employing the relationships among pressure, leakage and consumption.

The codes used in this study were prepared in MATLAB; it is an interactive system for matrix-based computation designed for scientific and engineering use. It is good for many forms of numeric computation and visualization (MATLAB, 1997). MATLAB is a high-performance language for technical computing. It integrates computation, visualization, and programming in an easy-to-use environment where problems and solutions are expressed in familiar mathematical notation.

The program developed for this study is called as LEAKSOL. Figure 5.1 shows the schematic representation of this program.

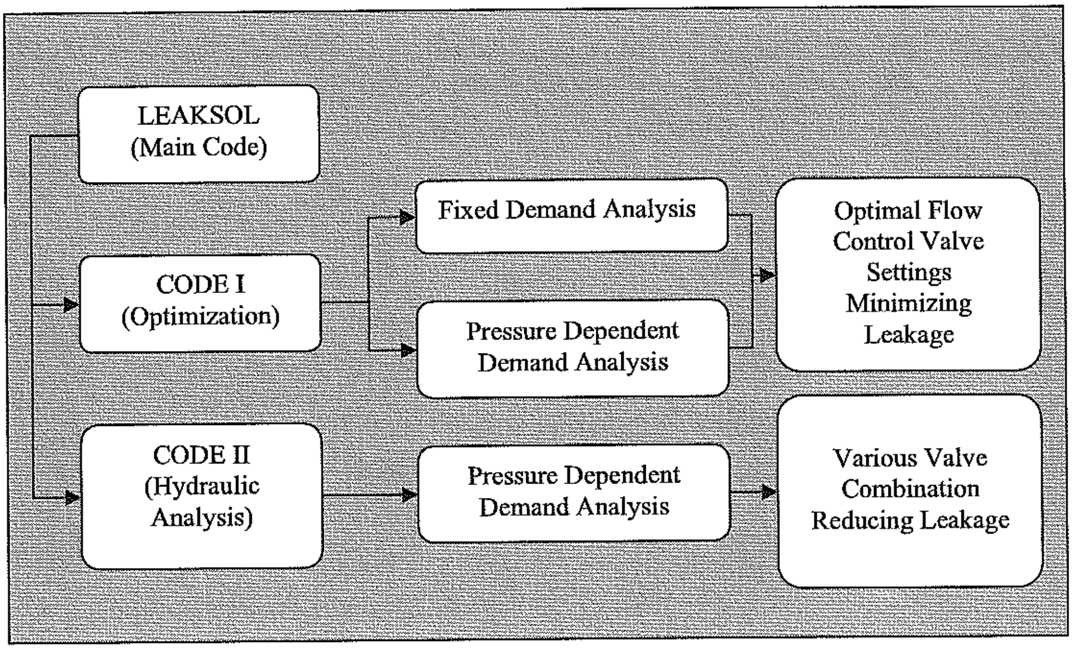

Figure 5.1 Schematic representation of LEAKSOL

LEAKSOL is composed of two parts, CODE I and CODE II. These codes were prepared in the form of separate MATLAB script (m-files) files.

CODE I, as it was stated earlier, finds optimal flow control valve settings minimizing water leakage either by using fixed demands or pressure dependent demands. User defines which method the network will be analyzed during the preparation of input text files. The node flow continuity equations are written by taking the pressure-demand and pressure-leakage relationships into consideration. The objective and the constraint functions are arranged to be solved with revised simplex method. Critical head value, $H_{critical}$, defined for some or for all nodes, controls the optimization problem. CODE I consists of the listed m-files given in Table 5.1. The algorithm for CODEI.m was already given in Figure 4.3.

Table 5.1 Script and text files for codes

CODE I		
Main Code	:	LEAKSOL.m
Subroutines	:	CODEI.m isint.m lprsm.m argmin.m
Text Files	:	element.txt fgn.txt node.txt pipe.txt valves.txt

CODE II		
Main Code	:	LEAKSOL.m
Subroutines	:	CODEII.m Allsubsets.m
Text Files	:	element.txt fgn.txt node.txt pipe.txt valves.txt

Project data is prepared as five text files (Figures 5.2-5.6). Text files are formed in the following column order: demand, ground elevation and minimum required pressure and critical head data for nodes in node.txt; from-node, to-node, length, diameter, Hazen-Williams coefficient and open-or-closed data for pipes in pipe.txt; HGL level and ground elevation for fixed grade nodes in fgn.txt; number of pipes, number of nodes, number of fixed grade nodes, number of flow control valves, demand factor, power used in leakage equation and leakage coefficient obtained for the network, value of variable F defining the type of the analysis in element.txt; pipe numbers on which valves exist, from node and to node information of that pipes in valves.txt.

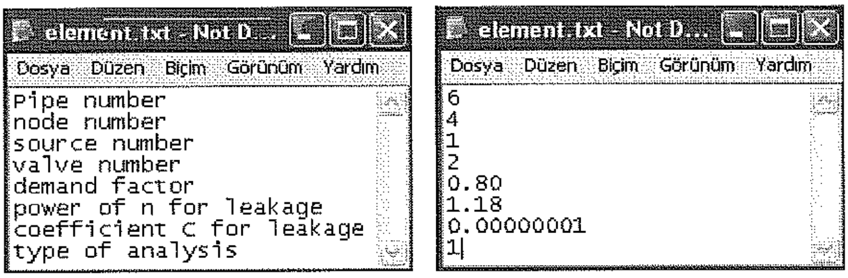

Figure 5.2 Text file for element.txt

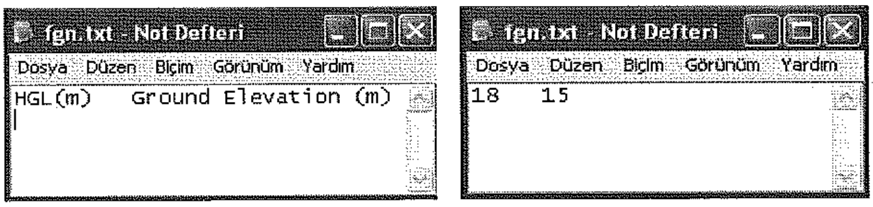

Figure 5.3 Text file for fgn.txt

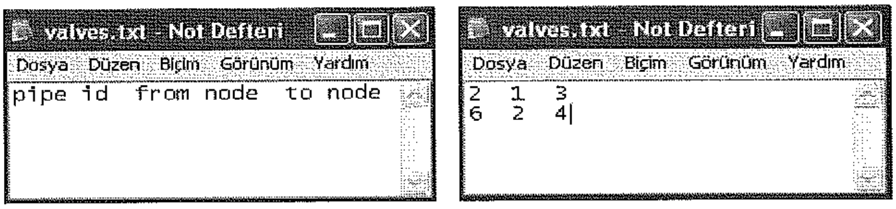

Figure 5.4 Text file for valves.txt

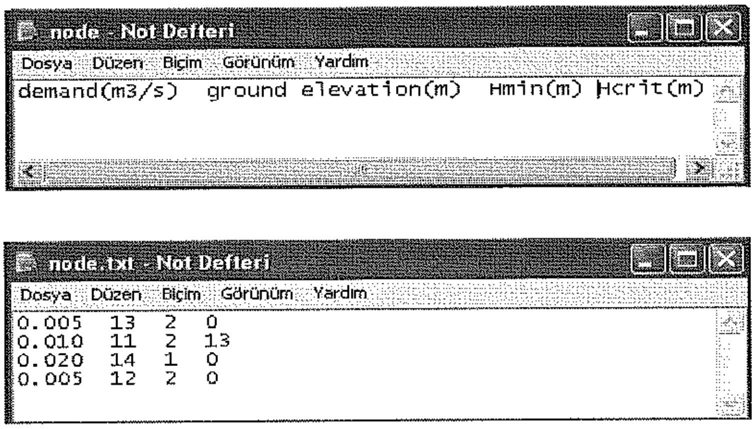

Figure 5.5 Text file for node.txt

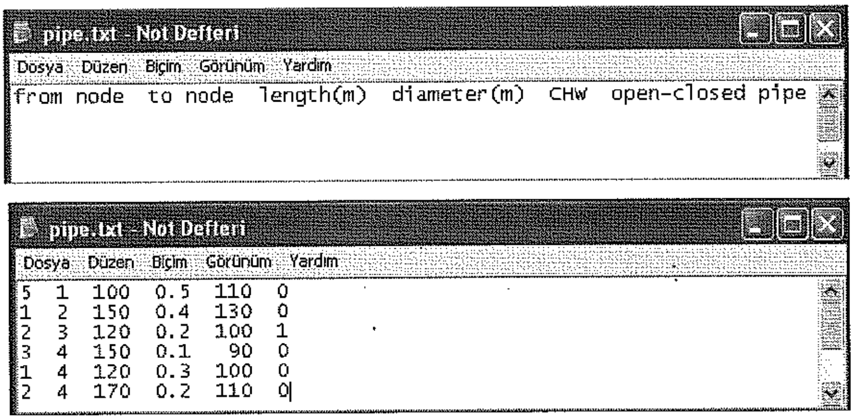

Figure 5.6 Text file for pipe.txt

At the end of the program execution, elapsed time for the operation, total leakage with optimal flow control valve settings, total consumption and total demand are displayed on the screen and outputs are saved as text files in the folder that contains the codes. The output files include nodeout.txt showing node number, HGL, pressure head, consumption and nodal leakage value; pipeout.txt with pipe number, velocity, discharge, head loss and leakage at that pipe; valveout.txt including pipe numbers on which valves exist and resulting valve settings; result.txt showing total leakage, total consumption and demand. Note that, the core program was prepared and tested with MATLAB versions of 7.0 and 7.1.

As it was stated previously, LEAKSOL is composed of two parts, CODE I and CODE II. Code II makes hydraulic analysis of the network for all possible valve combinations generated according to the selected isolation valves by defining the relations between pressure-leakage and pressure-demand in order to reduce water leakage. Minimum required pressure for consumers in order to satisfy their demands and critical head requirement for some selected nodes (or for whole network) are defined. The amount of leakage, consumed water and violations for critical heads are reported for each case at the end of the execution. The computer program was composed of a subset generation part for the determination of valve combinations and a hydraulic analysis part. CODE II consists of the listed m-files given in Table 5.1. The algorithm for CODEII.m was given already in Figure 4.4. As in CODE I, before execution, script files should have been copied to the same folder.

Project data is prepared as five text files. Number of valves selected in the system is used to determine the total case number and the valve combination of each case with an equivalent bit-string representation of elements of the subsets.

Text files are formed in the following column order: demand, elevation, minimum required pressure and critical head data for nodes in node.txt; from-node, to-node, length, diameter, Hazen-Williams coefficient and open-or-closed data for pipes in pipe.txt; HGL level and ground elevation for fixed grade nodes in fgn.txt; number of pipes, number of nodes, number of fixed grade nodes and demand factor of the project, power used in leakage equation and leakage coefficient obtained for the network, value of variable F (which is 2 for CODE II) defining the type of the analysis in element.txt; pipe numbers on which valves exist in valves.txt.

At the end of the program execution, the number of iterations and elapsed time for the operation are displayed on the screen and outputs are saved as text files. For each case, a folder is formed with a name showing the case number which includes nodeout.txt showing node number, HGL, critical head, pressure head, consumption and leakage value extracted from node; pipeout.txt with pipe number, velocity, discharge, head loss and leakage at that pipe. The last text file except the folders arranged for cases is closed_pipe_scenerios.txt which presents scenario number, pipe numbers on which valves exist as closed, total leakage value corresponding to this case, total consumption withdrawn at that scenario, required total demand value and number of nodes in which critical head requirement is not satisfied.

5.2. Application for Sample Network

Two main studies were performed on the sample network to see the performance of the prepared codes.

5.2.1. CODE I Application

LEAKSOL was executed firstly by using the algorithm in CODE I for the network previously used by Sterling and Bargiela, 1984; Germanopoulos and Jowitt 1989; Jowitt and Xu 1990, Vairavamoorthy and Lumbers 1998. Leakage minimization was performed by accepting demands as fixed. A schematic representation of the network is shown in Figure 5.7; the network has 25 nodes, 37 pipes and 3 flow-control valves. The reference nodes were selected as nodes 19, 22, 21 and 13. The critical head criterion at the selected reference nodes were defined as 30 m above ground level. All of the pipes were accepted as open. The node and pipe data are given in Tables 5.2 and 5.3. Daily demand profile and reservoir levels are shown in Figures 5.8-5.11. The locations of the flow control valves are shown in Figure 5.7. The coefficient C_l relating the leakage to service pressure was taken as $10^{-8}$and the power of N as 1.18, same with the values used in Jowitt-Xu study (1990). Note that these values can be estimated through measurement of night flows for a real network.

Results were presented in the form of a graph. Figure 5.12 shows leakage rates in the network when valves are fully open and when valves located at the specified locations are optimally set (i.e. leakage value controlled through optimization study). Obtained valve settings for three flow control valves minimizing leakage are presented in Table 5.4. Results point out that leakage reduction during the peak demand period is relatively small whereas it is maximized during the night when consumer demands are lower and system pressures tends to be higher. Similar results were obtained with literature. The overall reduction in leakage is about 17 %.

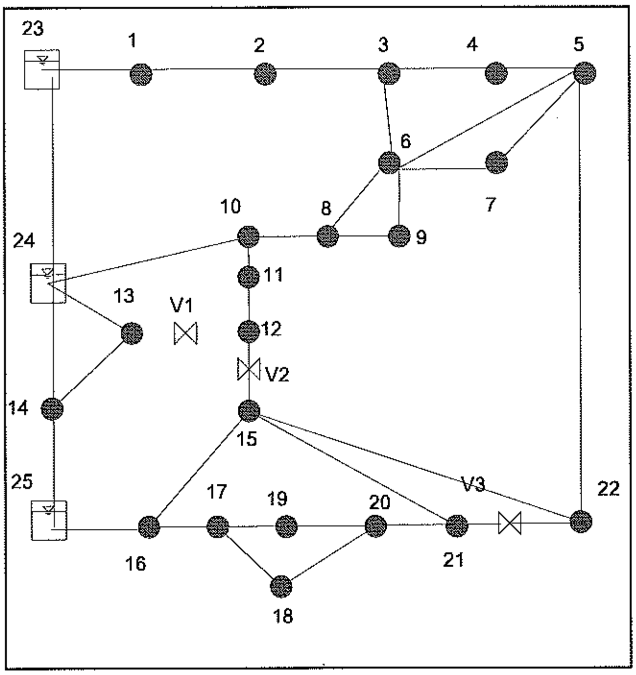

Figure 5.7 Sterling and Bargilea's (1984) network

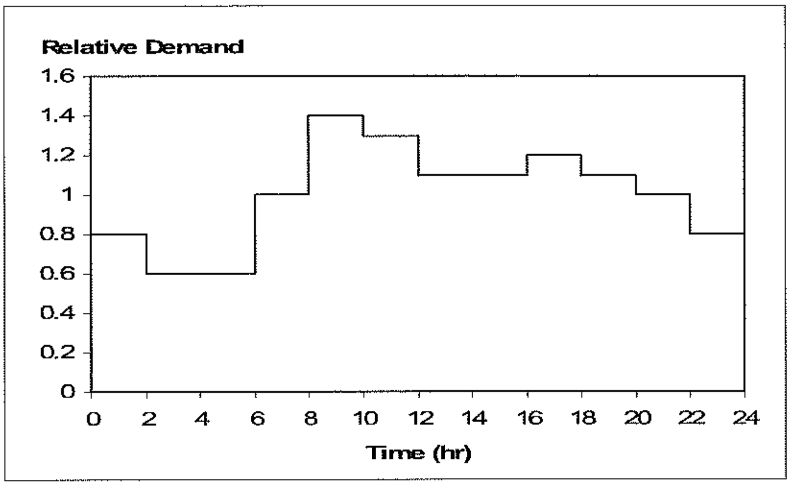

Figure 5.8 Daily demand variations

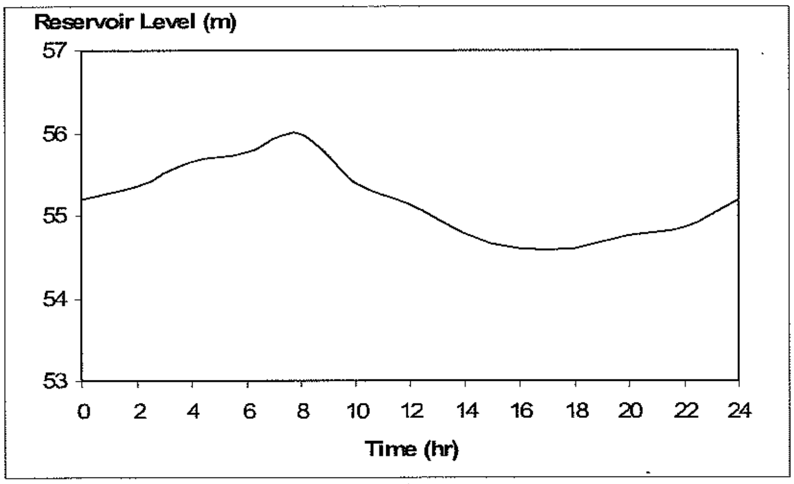

Figure 5.9 Reservoir profile (Node 23)

Table 5.2 Node data for sample network

Node No	Elevation (m)	Demand (l/s)
1	18	5
2	18	10
3	14	0
4	12	5
5	14	30
6	15	10
7	14.5	0
8	14	20
9	14	0
10	15	5
11	12	10
12	15	0
13	23	0
14	20	5
15	8	20
16	10	0
17	7	0
18	8	5
19	10	5
20	7	0
21	10	5
22	15	20
23	Reservoir Nodes	
24		
25		

Table 5.3 Pipe data for sample network

Pipe No	From Node	To Node	Length (m)	Diameter(m)	C_{HW}
1	23	1	606.0	0.457	110.0
2	23	24	454.0	0.457	110.0
3	24	14	2782.0	0.229	105.0
4	25	14	304.0	0.381	135.0
5	10	24	3382.0	0.305	100.0
6	13	24	1767.0	0.475	110.0
7	14	13	1014.0	0.381	135.0
8	16	25	1097.0	0.381	6.0
9	2	1	1930.0	0.457	110.0
10	3	2	5150.0	0.305	10.0
11	12	13	762.0	0.457	110.0
12	15	16	914.0	0.229	125.0
13	17	16	822.0	0.305	140.0
14	18	17	411.0	0.152	100.0
15	20	18	701.0	0.229	110.0
16	19	17	1072.0	0.229	135.0
17	20	19	864.0	0.152	90.0
18	21	20	711.0	0.152	90.0
19	21	15	832.0	0.152	90.0
20	22	15	2334.0	0.152	100.0
21	12	15	1996.0	0.229	95.0
22	11	12	777.0	0.229	90.0
23	10	11	542.0	0.229	90.0
24	8	12	1600.0	0.457	110.0
25	8	10	249.0	0.305	105.0
26	9	8	443.0	0.229	90.0
27	6	8	743.0	0.381	110.0
28	22	8	931.0	0.229	125.0
29	22	21	2689.0	0.152	100.0
30	4	3	326.0	0.152	100.0
31	5	4	844.0	0.229	110.0
32	6	3	1274.0	0.152	100.0
33	5	6	1115.0	0.229	90.0
34	7	6	615.0	0.381	110.0
35	5	22	1408.0	0.152	100.0
36	5	7	500.0	0.381	110.0
37	6	9	300.0	0.229	90.0

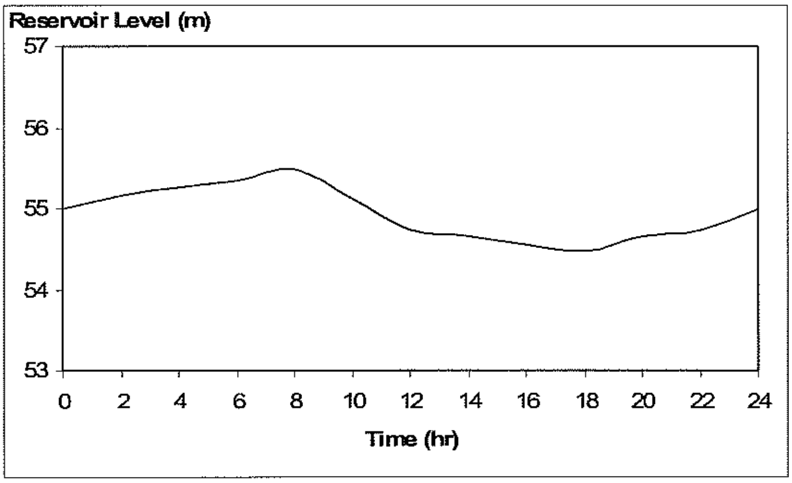

Figure 5.10 Reservoir profile (Node 24)

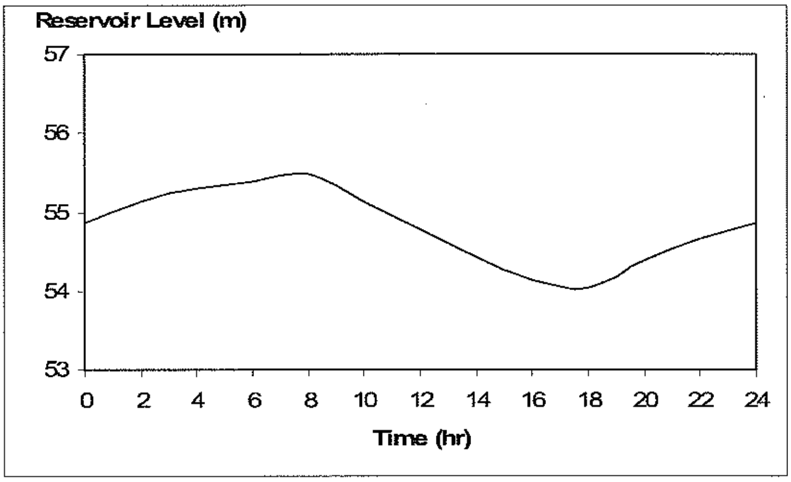

Figure 5.11 Reservoir profile (Node 25)

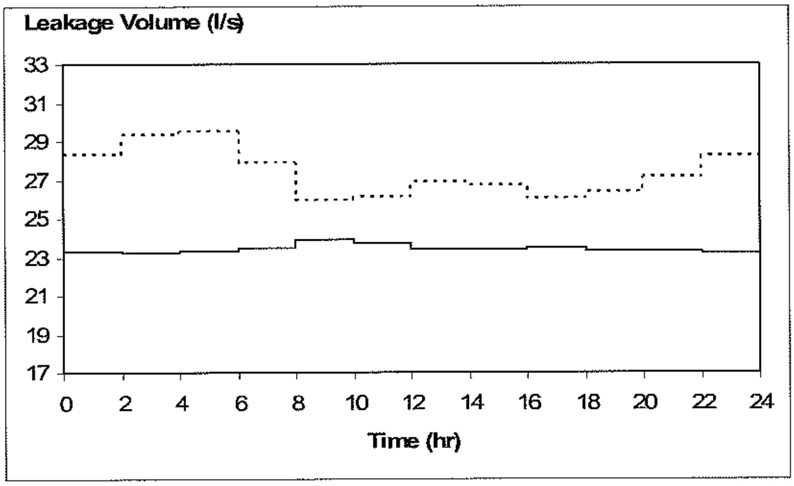

Figure 5.12 Leakage with and without optimized valve control

Table 5.4 Flow control valve settings for Sterling and Bargiela's (1984) network

Time(hr)	0	2	4	6	8	10	12	14	16	18	20	22
V(1)	0.21	0.08	0.08	0.25	0.52	0.46	0.33	0.34	0.42	0.35	0.28	0.17
V(2)	0.36	0.13	0.13	0.58	0.93	0.86	0.69	0.69	0.79	0.70	0.60	0.38
V(3)	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

5.2.2. CODE II Application

Case 1:

CODE II was executed firstly for the sample network given in Figure 5.7 replacing flow control valves by isolation valves. The network characteristics were given previously on Tables 5.2 and 5.3. The reference nodes were selected as nodes 19, 22, 21 and 13. The critical head criterion at the selected reference nodes were defined as 30 m above ground level. The minimum required pressure for consumer to take their required demand was selected as 5 m. The analysis was resulted with leakage and consumption values corresponding to 2^3 cases for each two hours interval of the demand curve. Table 5.5 shows combination number, open/closed valves for which solution was produced, leakage, total consumption and demand for each combination, the column presenting at how many reference nodes the critical head requirement was violated, and finally comments corresponding to examined case.

Table 5.5 CODE II result for Hour_0

Case No:	**V(1)**	**V(2)**	**V(3)**	**Leakage (m^3/s)**	**ΣCons. (m^3/s)**	**ΣDemand (m^3/s)**	**# of Violations**	**Comment**
1	0	0	0	0.0115	0.12	0.12	3	(1)
2	0	0	1	0.0123	0.12	0.12	3	
3	0	1	0	0.0125	0.12	0.12	3	
4	0	1	1	0.0131	0.12	0.12	3	
5	1	0	0	0.0221	0.12	0.12	2	
6	1	0	1	0.0249	0.12	0.12	0	(2)
7	1	1	0	0.0263	0.12	0.12	0	
8	1	1	1	0.0281	0.12	0.12	0	(3)

Row shows the case in which

(1) all valves are close.

(2) minimum leakage satisfying the critical head requirement is obtained

(3) all valves are open.

At the end of the execution, similar tables were obtained for each time interval of the daily demand curve by giving chance to the user to see all results whether satisfying the critical head requirement or not. Leakages for cases where valves are fully open (1), fully closed (2) and minimum leakages satisfying the critical head requirement (3), i.e., leakage values corresponding to rows marked in comment column of Table 5.5, were chosen among the results in order to present them on a graph. Figure 5.13 shows the variation of leakage for these results. Also, combination of valves for cases providing minimum leakages satisfying the critical head requirements were presented in Table5.6. Then, in order to compare outcomes of two codes, minimum leakage values satisfying the critical head requirement obtained from CODE II were presented on the same figure (Figure 5.14) showing leakages for optimally set flow control valves (CODE I results). It was concluded that, clearly the inclusion of optimally set flow control valves reduces leakage more than that of obtained from CODE II. But, it is obvious that application of the best fully open or closed combination of isolation valves satisfying the critical head requirement also reduces the leakage. The overall reduction in leakage corresponding to application of CODEII is about 3.5%. The violation of critical head requirement by taking consumption rate into consideration increases the reduction in leakage. This study shows that for the networks where it is not possible to place flow control valves, searching the best combination of isolation valve settings may be helpful for reduction of leakage up to a certain level.

<u>Case 2:</u>

Second study on the same network through application of CODE II is that the network was examined with same operational data, used in the study presented above, except required demand values. In this one, the demand was exaggerated by 10 % for every interval of daily demand curve and the code was executed. It was aimed to see the effect of demand variations on valve settings. Among the results, the minimum leakages satisfying critical head requirement and their corresponding valve settings for each interval were chosen and presented below. Figure 5.15 shows the leakages resulted from the best combination of valves settings satisfying critical head requirement for the daily demand variations given in Figure 5.8 and for exaggerated demand values. Valve settings for the latter one were presented in Table 5.7. Figure 5.15 shows that leakage decreases when required demand described for nodes increases. Also, if valve settings are compared for two studies, it can be seen that minimum leakage satisfying critical head requirement increases due to difficulty in supplying heads greater than or equal to critical heads at reference nodes which causes more valves to be open at low demand hours. Valve settings of this application were obtained as nearly same with that of previous one.

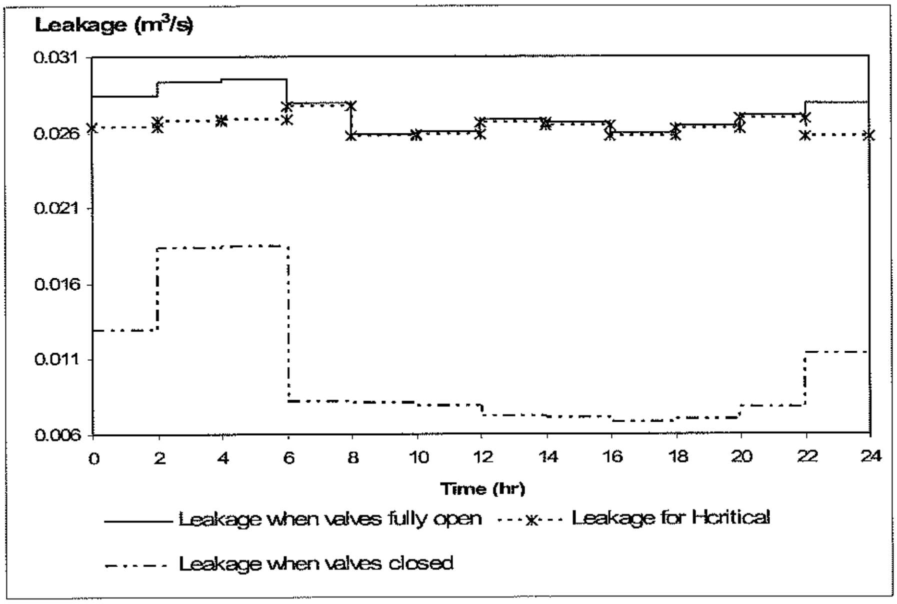

Figure 5.13 Leakage for Sterling and Bargiela's (1984) network _CODE II

Table 5.6 Isolation valve settings for Sterling and Bargiela's (1984) network

Time(hr)	0	2	4	6	8	10	12	14	16	18	20	22
V(1)	1	1	1	1	1	1	1	1	1	1	1	1
V(2)	0	0	0	1	1	1	1	1	1	1	1	0
V(3)	1	0	0	0	0	0	0	0	0	0	0	1

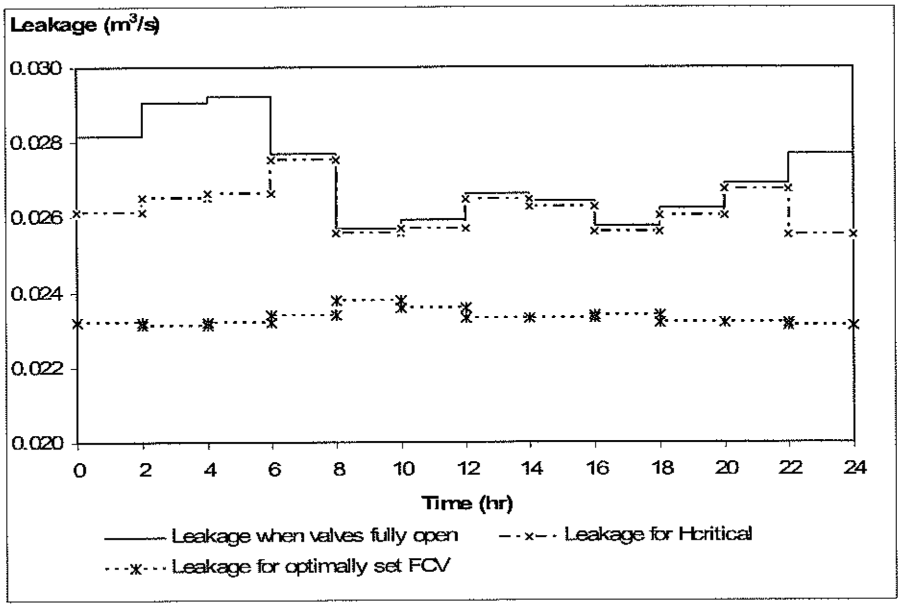

Figure 5.14 Comparison of results for Sterling and Bargiela's (1984) network

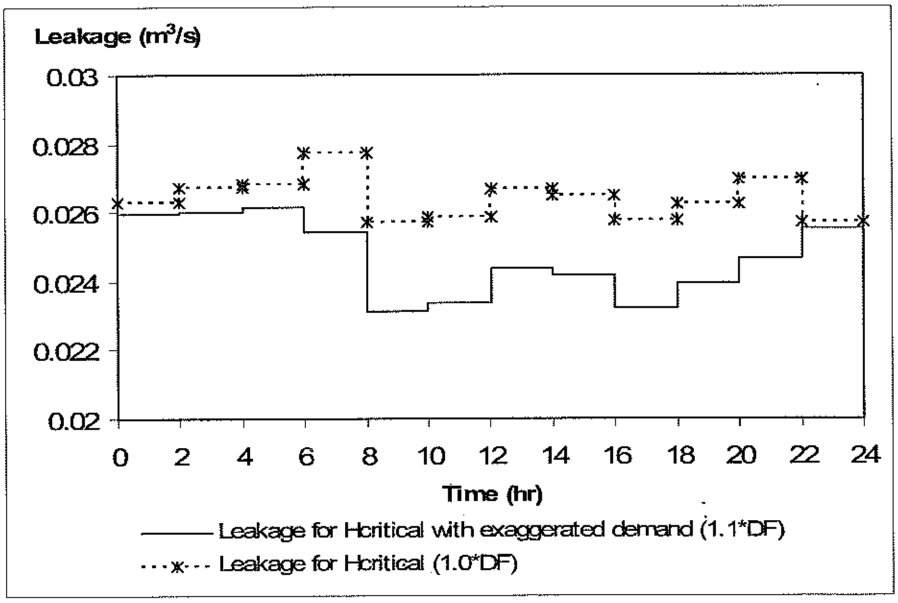

Figure 5.15 Comparison of leakages for exaggerated demand

Table 5.7 Valve settings for exaggerated demand

Time(hr)	0	2	4	6	8	10	12	14	16	18	20	22
V(1)	1	1	1	1	1	1	1	1	1	1	1	1
V(2)	1	0	0	1	1	1	1	1	1	1	1	1
V(3)	0	1	1	0	0	0	0	0	0	0	0	0

Case 3:

For another application on Sterling and Bargelia's (1984) network, the reference nodes remained same, i.e. the critical head criterion at nodes 19, 22, 21 and 13 were defined as 30 m above ground level. The minimum required head for consumer to take their required demand was selected as 20 m above ground level except reference nodes where it was described as 30 m. The network was examined with CODE II and results containing leakage values corresponding to 2^3 cases for each two hours interval of the demand curve were obtained.

At the end of the execution, closed_pipe_scenerios.txt presenting scenario number, pipe numbers on which valves exist as closed, total leakage value corresponding to this case, total consumption withdrawn at that scenario, required total demand value and number of nodes in which critical head requirement was not satisfied, was obtained for each time interval. From this text file, leakage and consumption values corresponding to the valve combination in which V (1) was open, other two valves were closed, [1, 0, 0], were picked and figured in order to demonstrate that if the minimum head criterion is violated taking consumption/demand ratio into consideration, leakage reduction increases.

Figure 5.16 compares the leakage for valve combination selected [1, 0, 0] for a consumption/demand ratio not less than 95 %, minimum leakage satisfying the critical head requirement resulted from CODE II and leakage for optimally set flow control valve settings obtained from CODE I. It can be seen from the figure that especially at peak hours leakage due to preferred combination of isolation valves is less than others. Overall reduction in leakage for this case [1, 0, 0] is about 17.2 %. As it was stated before, this value was 17% for optimally set flow control valves and 3.5 % for the combination satisfying critical head requirement which was obtained from application of CODE II. In this example, the lowest consumption/demand ratio was obtained as 0.96167 for hours 8-10 when demand factor had its maximum with 1.4. Figure 5.17 shows the consumption/demand ratio for that valve combination. The averaged value of this ratio was 0.986 in this application. It can be concluded that the municipality can select the best valve combination regarding the consumer's necessities by using this algorithm so that both amount of leakage and the consumption rate can be kept under control. The advantage of CODE II is that it is possible to see the leakage and consumption rate for any case at the same time.

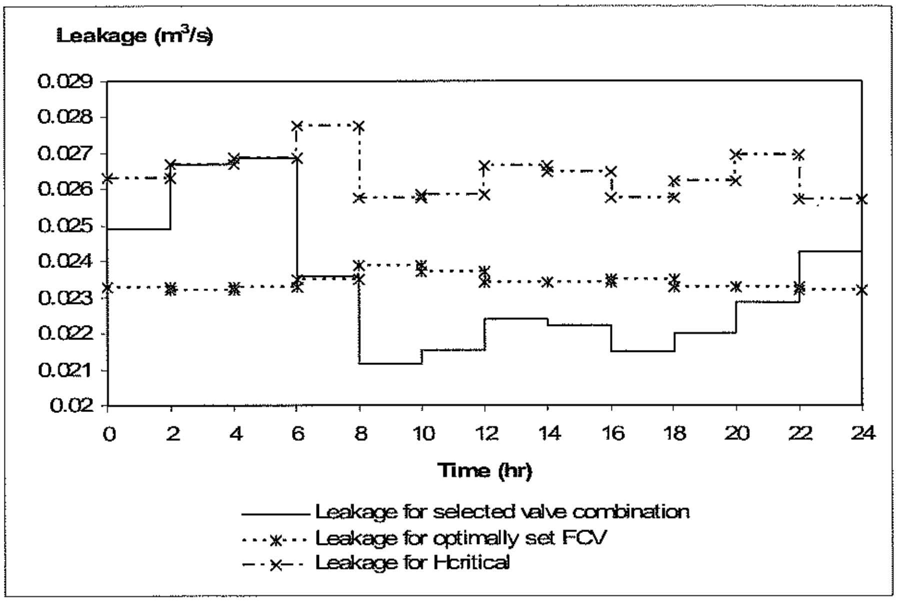

Figure5.16 Leakage for selected valve combination [1,0,0]

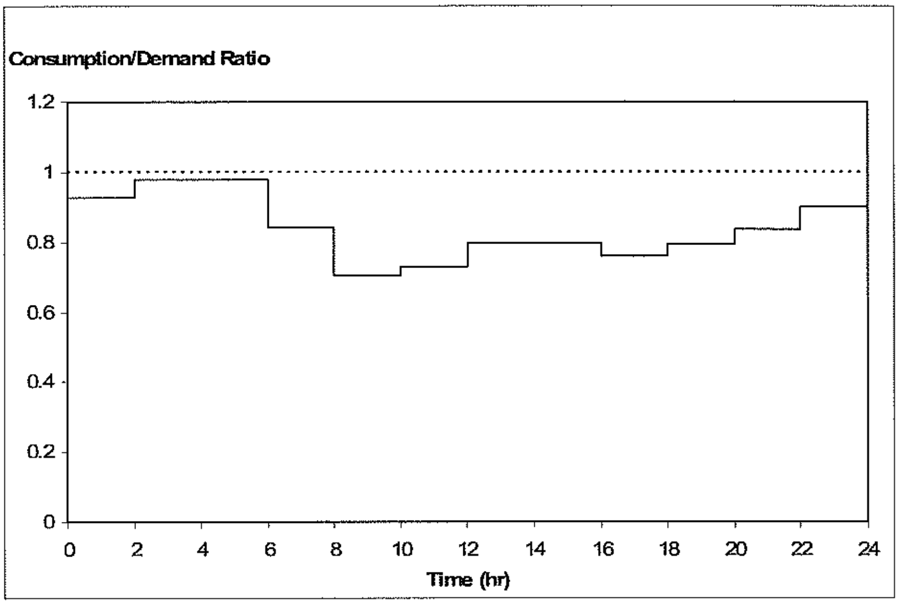

Figure 5.17 Consumption/demand ratio for valve combination [1, 0, 0]

This time, for the analysis, where minimum required head had been selected as 20 m above the ground level, leakage values corresponding to the valve combination, in which V(1) and V(2) were closed and V(3) was open, [0,0,1], were picked and compared in Figure 5.18a with leakage resulted from the combination [1,0,0] and leakage satisfying critical head requirement. Figure shows that leakage related to valve settings [0,0,1] is less than the others with an average consumption/demand ratio of 84 %. This means that leakage reduction increases if consumption decreases. In some situations regarding to operation of a network, not only the amount of leakage but also the ratio of consumption to demand can be accepted as criterias for selection of best combination of valve settings. Figure 5.18b shows the consumption/demand ratio for that valve combination.

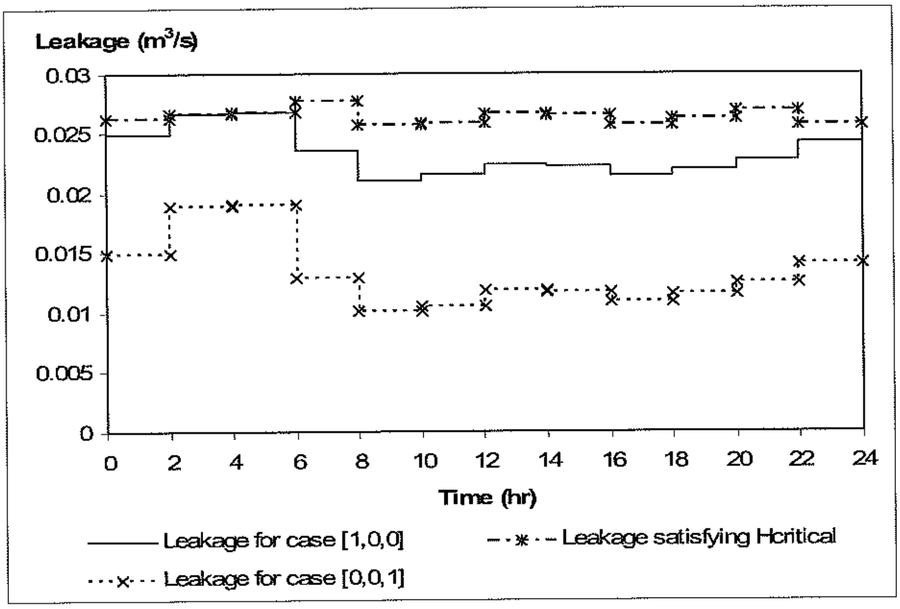

Figure 5.18a Comparison of leakages for selected valve combinations

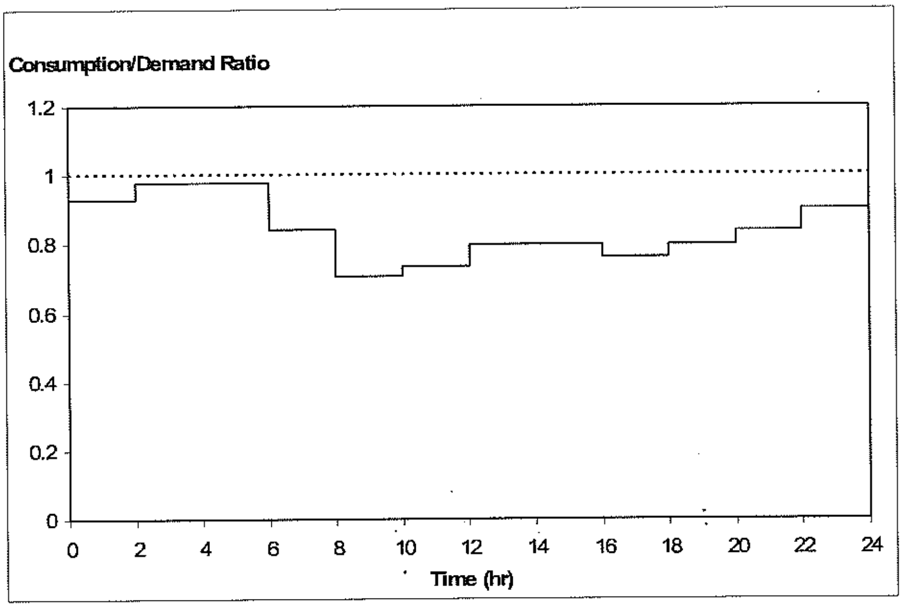

Figure 5.18b Consumption/demand ratio for valve combination [0, 0, 1]

5.3. Application for Study Area

Water supply networks are planned and designed to prevent current and future peak demands. But it is inevitable that excessive pressures will exist and they can lead to significant levels of leakage. To increase economic efficiency, leakage should be reduced to appropriate levels. The method explained above outlines both, a method of minimizing leakage through optimal valve control anda also, selecting best isolation valve combination reducing leakage. The application of this method to N8-3 pressure zone of Ankara municipal water supply system is planned. A previous study performed at this zone showed that 50.14% of the total water used by consumers was lost as leakage (Özkan, 2001). It is a fact that finding the economic balance between cost of repair and cost of lost water is no longer an acceptable solution. Nowadays, water has a perceived value greater than its production cost. Therefore, efficient precautions to avoid leakage should be taken immediately.

The study (Özkan, 2001) conducted to determine the leakage coefficients showed that small leaks and high pressures were encountered mostly in N8-3 pressure zone. High pressures can cause opening up of pipe joints or cracks which will increase water leakage in the system. In such a case, reduction of pressures may be the best solution. Pressure reduction can be achieved in a number of ways, from the optimization of pumping schedules or valve operation or introduction of pressure reducing valves (PRV) or additional storage anywhere in the system by monitoring the system to ensure that pressures do not anywhere drop to a level to prejudice customer supply. The purpose of this study as stated before to make the application of the model, which was primarily based on the reduction of leakage by defining the relation among pressure, leakage and demand.

5.3.1. Study Area

In this study, N8-3 pressure zone, which is the zone at the end of the north main line of Ankara Municipal Water Supply System, was chosen as the study area.

Water Distribution System of Ankara

Ankara is the second largest city of Turkey. Water treatment plant of Ankara (İvedik) supplies roughly 860 000 m^3 of water per day. Table 5.8 shows the growth of population and projected water demand for Ankara. The distribution system of Ankara takes the raw water mainly from Çamlıdere, Kurtboğazı, Akyar, Eğrekkaya, Çubuk II, Kayaş - Bayındır Dams (Figure 5.19). Table 5.9 gives the amount of supply from sources to the city. The water coming to İvedik treatment plant from these dams is transmitted with three primary lines, each is 2200 mm in diameter and of pre-stressed concrete; they carry clean water by gravity to the central supply zone C2 and to five main pump stations connected to the primary pipelines, from which water is pumped to the higher supply zones.

Ankara water distribution network consists of 35 pump stations and 54 storage tanks. The city of Ankara is divided into five main pressure zones to serve water within certain pressure limitations:

- ✓ Central and Western Supply Zone (Sincan, water supplied by gravity)
- ✓ Northern Supply Zone (e.g. Keçiören)
- ✓ Eastern and South-Eastern Supply Zone (e.g. Mamak)
- ✓ Southern Supply Zone (e.g. Çankaya)
- ✓ South-Western Supply Zone (e.g. Çayyolu)

The main pressure zones are divided into several pressure zones by 40-50m elevation intervals. Central-Western supply zone is composed of two pressure zones being C2 and W1. Northern supply zone (Figure 5.20a) has eight pressure zones named N3-N10. Figure 5.20b shows the place of N8-3 zone in Northern supply zone. Eastern-South Eastern zone has seven pressure zones at east being E3-E9 and four at south-east being SE3-SE7. Southern supply zone has ten pressure zones named S3-S12; South-Western supply zone is composed of four zones named SW3-SW6.

Table 5.8 Growth of population and demand for the city of Ankara (DSI, 2008)

Year	Population	Total Gross Water Demand (million m^3/year)
2005	3 690 000	341
2010	4 060 801	384
2020	4 941 669	489
2030	5 899 158	584
2040	6 812 705	674
2050	7 577 934	830

Table 5.9 Amount of supplies for the city of Ankara (DSI, 2008)

Dam Name	Comissioning Date	Yearly Water Supply (million m^3)
Çubuk I	1936	-
Çubuk II	1964	20
Kayaş - Bayındır	1965	7
Kurtboğazı	1973	60
Çamlıdere	1985	142
Eğrekkaya	1993	79
Akyar	2000	45
Sub-Total:		353
Groundwater	-	42
Total:		395

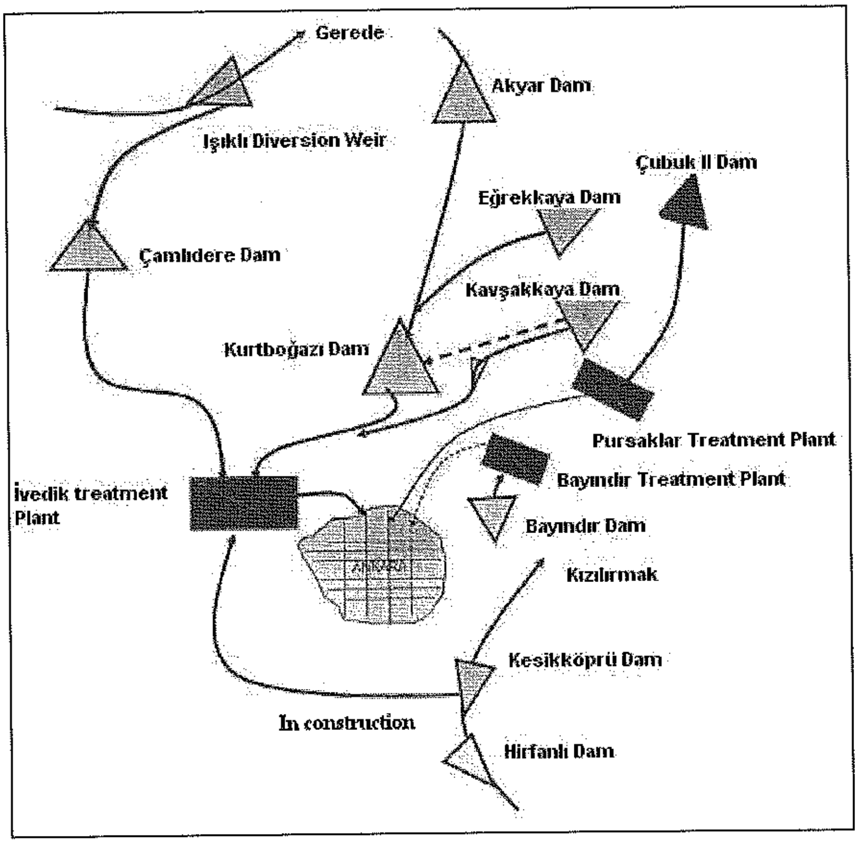

Figure 5.19 Ankara water supply system (DSI, 2008)

The water coming from İvedik treatment plant is transmitted to the first pump station, P1, of the north line. P1 is the main pipe station that gives water to all other pump stations. In the north zone, after P1, network branchs towards different zones. Water is transmitted to the study area (N8-3 pressure zone) through P1, P2 and P12. P23, which is the pumping station of N8-3 pressure zone, pumps water to the distribution network and to the storage tank of this zone, T53.

Kayaş pump station, which is connected to the Kayaş filtration plant in the eastern part of the city, is a small supply for the distribution system. The filtration plant takes water from Kayaş-Bayındır dam and Kusunlar spring intake. This pump station feeds

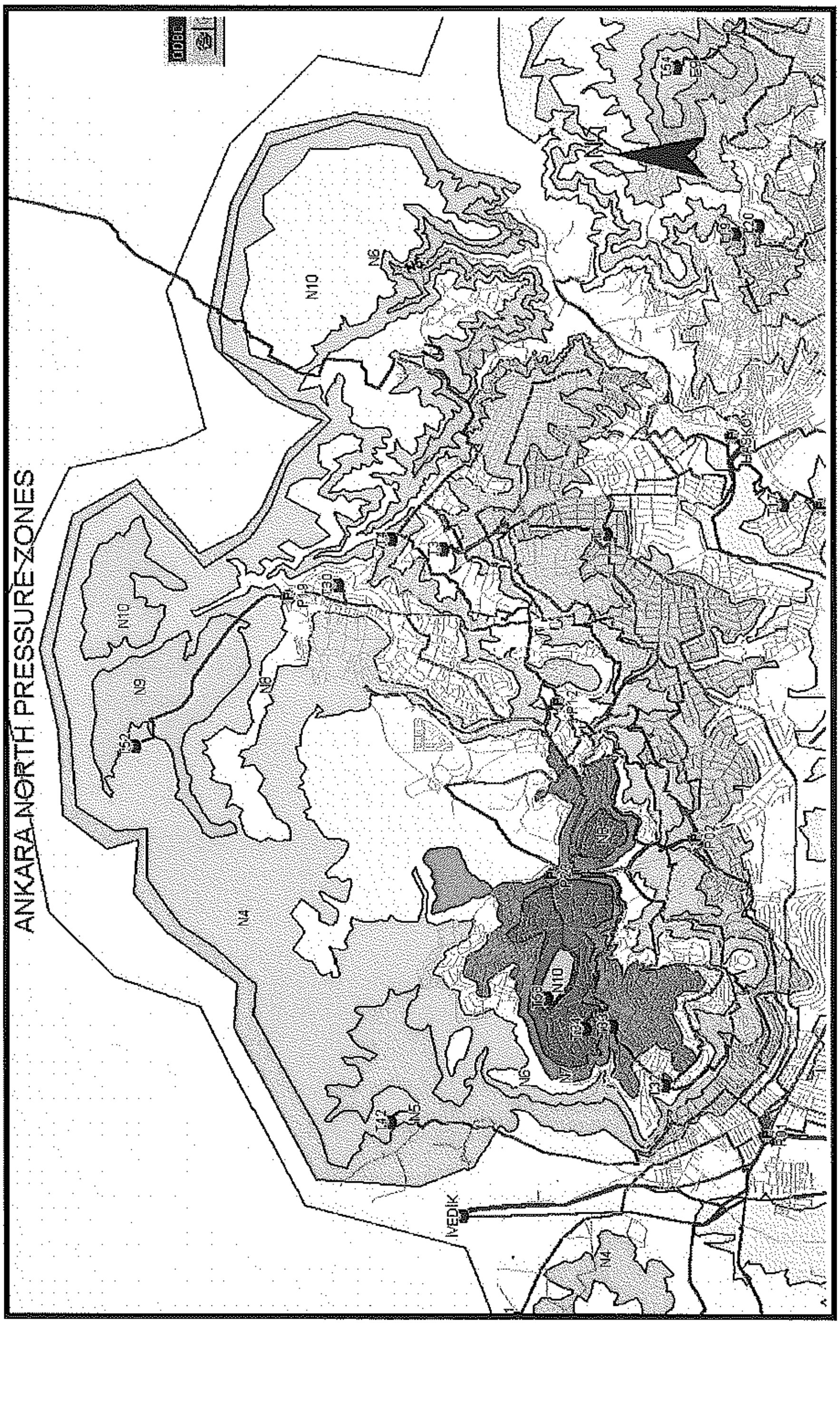

Figure 5.20a Ankara North Pressure Zones

E4 and E5 pressure zones with two separate pumping lines. The water coming from Çubuk II dam is treated in Pursaklar treatment plant and is delivered to the system.

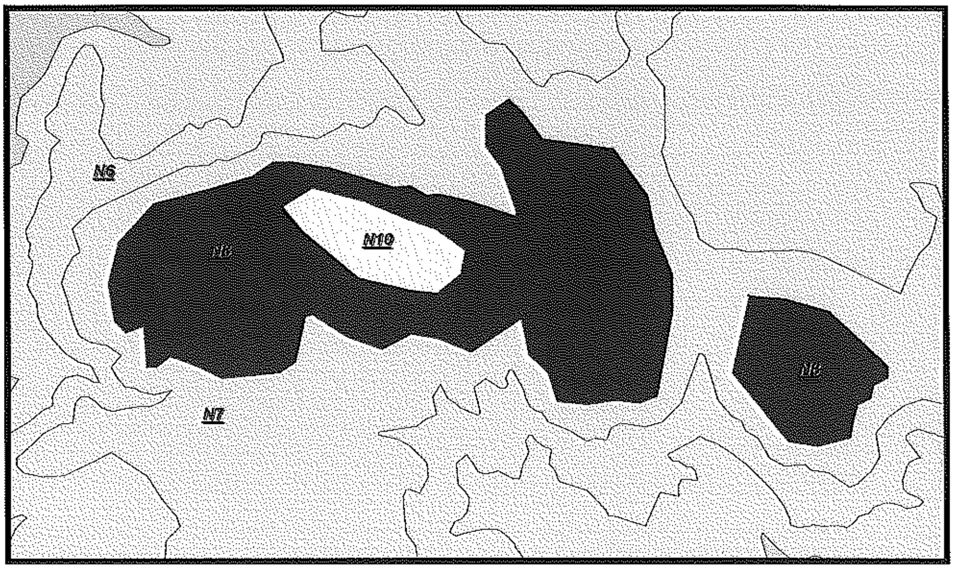

Figure 5.20b N8-3 pressure zone

The study area, i.e. N8-3 pressure zone is laterally distributed on two adjacent hills, including Çiğdemtepe district (Yenimahalle county), Şehit Kubilay, Sancaktepe and Yayla districts (Keçiören county). Schematic view of the zone can be seen in Figure 5.21. Pump, P23 is fed by pump station P12 and the reservoir given in this figure simulates the pressure at the suction side of the pump, P23. Figures 5.22a-5.22c show the network placed map of N8-3 pressure zone on Google Earth. The zone was presented in the form of three figures in order to feel the topography of the region. It can be easily seen that this zone has a very perturbated topography.

There are approximately 25000 people residing in these four districts. The system is composed of 467 links and 371 junction nodes. There is one storage tank, T53, and one pump station, P23. The pump P23 is composed of three parallel pumps which are used with a coordination to provide necessary water to feed the network. The tank T53 is rectangular in cross section with a height of 6.5 m and has a volume of 5000 m^3 which is composed of two tanks. There is a main transmission line with a diameter of 500 mm between the pump and tank. The pipes of the system are ductile iron and the network is a grid type distribution system; pipes are fifteen years old.

There are a lot of reasons to choose the N8-3 pressure zone as the a study area. First of all, existance of one pump station and one storage tank in N8-3 zone makes the simulation of the system easier. The flow rate passing through the pump, input and output pressure head values and tank levels can be easily observed and saved by the help of SCADA system. Another reason is that, the water consumption in this area is nearly homogeneous and except for the mosques and the schools around, there is nothing disturbing this homogeneity. The customers can be accepted as same type having same socioeconomic status. There are very few commercial and industrial customers. The homogeneity of water consumption makes the determination of nodal weights easy and more accurate. Also, according to preliminary studies performed, N8-3 pressure zone was seen as one of the weakest portion among the north pressure zones having a water leakage of 50.14 % of total supply (Özkan,2001). Therefore, it may be a suitable for a study aiming reduction of leakage. The most important reason is that there are important and detailed studies on the area (Merzi et al., 1998a, 1998b; Doruk, 2001; Özkan, 2001; Yıldız, 2002; Nohutçu, 2002).

5.3.2. Data Collection

5.3.2.1. ASKİ Data Processing Center

Network data is the most important part for analysis process. In this study, N8-3 pressure zone is taken from 1/5000 scaled schematic maps. These maps contain all the characteristic information of pipes, pumps, tanks, their locations and they are obtained from ASKİ Data Processing Center. The Center has the whole water network

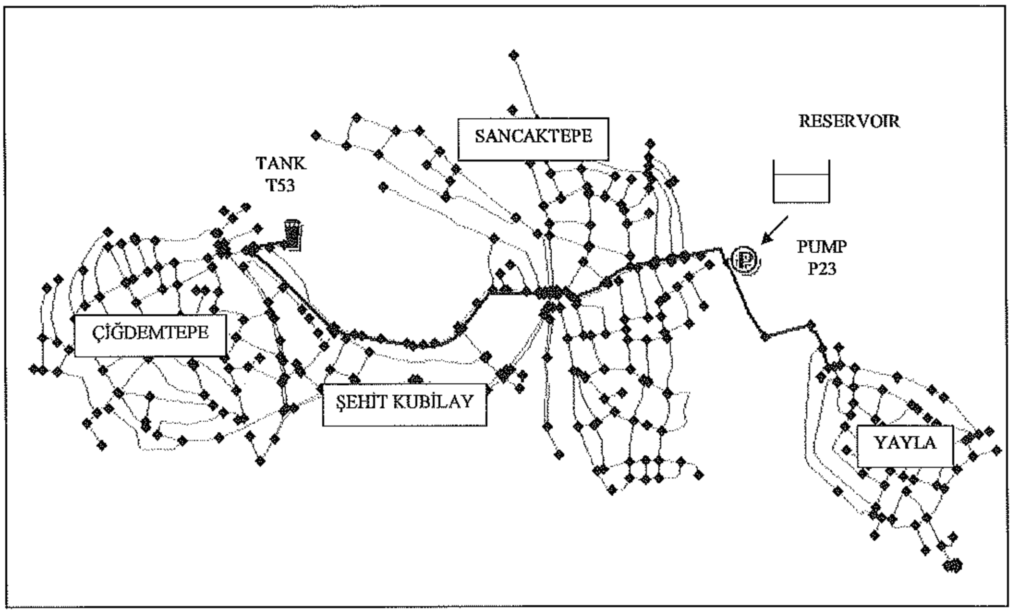

Figure 5.21 Schematic view of N8-3 pressure zone

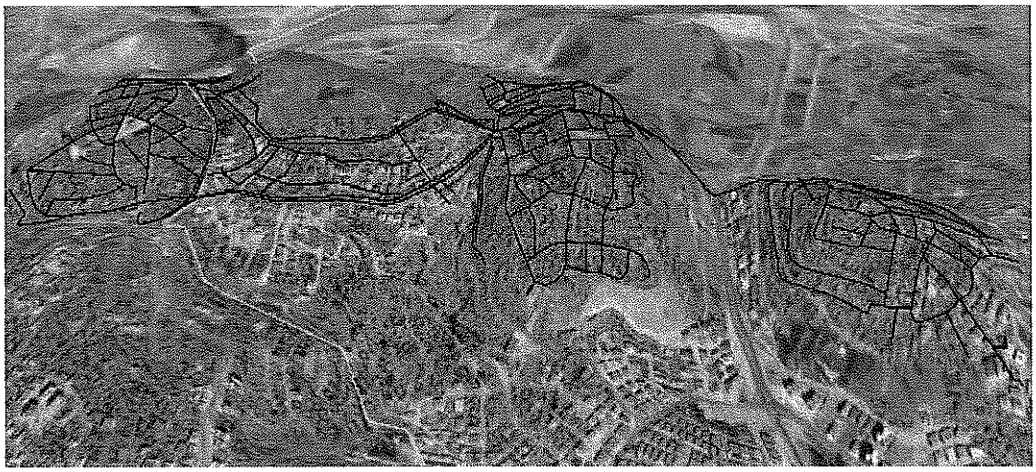

Figure 5.22a View 1 from network placed map of N8-3 pressure zone on Google Earth

Figure 5.22b View 2 from network placed map of N8-3 pressure zone on Google Earth

Figure 5.22c View 3 from network placed map of N8 pressure zone on Google Earth

data of Ankara in 1/5000 scale in node-link database. The center is responsible for new data input and update process in CAD environment. All attributes of Ankara water distribution network database are stored in MapInfo environment. It is used by this center as a Geographic Information Systems (GIS) tool for projects. All network related information about N8-3 zone is gathered in tabular format from ASKİ and they can be used by MapInfo easily.

5.3.2.2. SCADA (Supervisory Control and Data Acquisition)

SCADA systems are the primary source of information about the operation and performance of the water system. A SCADA system which is a combination of hardware, software and communication mechanisms, allow a water system operator to control certain aspects of the operation of that system. The purpose of SCADA is to compile data concerning the operation of the water system and to allow automated control of certain components. As a result, SCADA serves four primary functions including monitoring water system operation, monitoring equipment, controlling the operation of system components and compiling operations data.

ASKİ Facilities Department SCADA Center is responsible for collecting, transmitting and storing data from various control points of the network such as pump stations, storage tans and some critical points on some pipes and then making decisions regarding the operations based on these data. The pressures, flow rates in these facilities are reported on five minute frequency, but are recorded at thirty minutes frequency to the database. Water pressure at the entrance and at the exit of pump stations, flow rate of each pump, water level in the storage facilities, head and pressure values were obtained from that facility's database.

SCADA data are also used for calibrating the network model and investigating and solving operational problems. It provides data to compare modeled and measured values. Values falling outside a desired range can be flagged for investigation. Thus, computerized methods of comparing SCADA data and modeled output data can reduce the amount of time spent for troubleshooting.

5.3.3. Formulation of Leakage Equation in Study Area

As it was already mentined before, leaks in water systems are pressure dependent. If the pressure decreases, then leakage should decrease. The placing the leaks in the system as known demands are not useful. Instead of this, pressure dependent leakage terms should be incorporated. The main difficulty is how to set the leakage coefficients. To determine the leakage coefficients for a system, the leakage flow and the average pressure in the zone should be estimated.

In the past, approaches related with leakage modeling have involved the application of a multiplier to system demand as stated in literature review part. Such an approach can not reflect the relationship between leakage and pressure and can lead to inaccurate results. Consequently, representation of leaks by orifice function is recognized as more appropriate and has more or less become the norm.

The nonlinear relationship given in eqn. 4.2, between the leakage and average pressure, representing the whole study area can be rewritten as:

$$QS = k.(\overline{P})^{N} \tag{5.1a}$$

In this equation, k is a coefficient that relates leakage to pressure. To be representative for whole N8 pressure zone, the k coefficient in the leakage equation was replaced with;

$$k = C_l * L. \tag{5.1b}$$

where, L is total length of pipes for studied part of the network and C_l is a coefficient that relates leakage per unit length of pipe to pressure. $\overline{P}$ denotes the weighted mean pressure representing whole study area in meter. The power of N is taken as 0.5 in general orifice function. However, several researchers have found that the square root relationship is not always appropriate for either individual leaks or aggregate leakage in an entire distribution system or a portion of one.

Germanopoulos (1985) used the non-linear function given in Eqn. 5.2, between leakage and pressure by accepting the power of pressure, N, as 1.18 and the coefficient C_l as 10^{-8} which was based on the results of field experiments performed by the Water Research Center (U.K.). Same leakage-pressure model was adopted by also other authors. Germanopoulos and Jowitt (1989), Jowitt and Xu (1990), Vairavamoorthy and Lumbers (1998) and more recently, Araujo et al. (2006) used the power of 1.18 to describe the relationship between leakage and pressure which was based on the data from a set of field experiments undertaken by the Water Research Centre (Goodwin, 1980).

Since this value had been derived from data collected from a specified set of field experiments, it may not be representative of other circumstances [for example, for data from some networks in Japan (Miyaoka and Funabashi, 1984) water leakage had been found to be a function of pressure to the power of 1.15 instead of 1.18].

A field campaign in 2001 was conducted in N8-3 pressure zone of Ankara water distribution network for the determination of water leakages and to find a relation between pressure and leakage. The study performed to obtain N and k coefficients for whole zone showed that the leakages in this zone are relatively small depending on great N values. That is, small size leaks tend to grow under high pressures. However the correlation coefficient was indicated as very low and the results were presented in the form of a range for both coefficients:

$$N = 1.68 - 3.10$$

$$k = 0.89 - 7.16$$

In fact this is the case for related field studies in the literature. Low coefficient of correlation can be resulted from the following reasons :

- As understood from the results of the field study, the isolation of sub-zones may not be achieved totally. The flow of water from study area to other zones could not be blocked due to inoperate valves or unknown connections between zones.

This case directly affects the amount of leakage estimated for each sub-zone and causes erroneous results of weighted mean pressures in a successive manner.

- The data taken from SCADA and pump station related with operating conditions of the network for predetermined dates can be wrong causing errors in resulting daily demand curves.

- The pump curves used in network analysis may not reflect the actual working conditions of pumps.

Due to these reasons, the range of leakage coefficients determined for N8-3 pressure zone was not preffered to be used in this thesis study. More recently, Lambert (2000) stated that for individual sectors of a distribution system N depends on pipe material and leakage level, but in the absence of such knowledge a linear relationship (i.e., N=1.0) can be assumed. In this thesis study, the relationship between pressure and leakage was defined by using three different values of N; to be representation of the orifice equation (N=0.5); having no exact knowledge of relationship for N8-3 pressure zone (N=1.0) and to follow the studies in literature (N=1.18).

The nonlinear relationship between the leakage and average pressure for a pipe can be rewritten by using equation (5.1) as follows :

$$QS_{ij} = k.(P_{ij})^N \tag{5.2a}$$

$$k = C_l * L_{.ij} \tag{5.2b}$$

where, Qs_{ij} is water leakage volume occurring in the pipe element of length L_{ij} spanning nodes (i) and (j), P_{ij} is average pressure for the pipe element and it is computed as follows:

$$P_{ij} = \frac{(P_i + P_j)}{2} \tag{5.3}$$

The term of average pressure in leakage equation actually expresses the pressure head which reflects the energy resulting from water pressure.

The head (H) at a point in the system is the sum of the pressure and ground elevation (G) for that point. The equation (5.3) can be rewritten as:

$$P_{ij} = 0.5 \left[\left(H_i - G_i \right) + \left(H_j - G_j \right) \right] \tag{5.4}$$

In order to use the leakage equation and determine the coefficients C_l by knowing the N values, it is necessary to know the leak amount of water and the weighted mean pressure corresponding to the related study area. The average water leakage for N8 pressure zone was accepted as 60 m^3/hr from the field work in 2001 (Özkan, 2001).

The other unknown to be found and used in the leakage equation is weighted mean pressure. It is possible to express the overall pressure of an area by the weighted mean pressure. It is calculated by multiplying the length of each pipe with its average pressure, obtained as a result of network analysis, and then dividing it to the total pipe length of the studied area.

5.3.3.1. Network Analysis

Weighted mean pressure is determined through the hydraulic analysis of the network. Network analysis has two components: a hydraulic network solver that performs calculations and a data set that describes the related system. A data set, which should define at least the pipe network, system characteristics and water consumption rates, is required as input to the network solver.

5.3.3.1.1. Developing Hydraulic Model

The computer model is the key for the network analysis part of this study. Efficiency of this study depends on establishment of an accurate model for the distribution system.

In order to apply the codes mentioned previously on a real network, network and operation data should be prepared. Geographic information systems (GIS) contain network description data and SCADA systems contain operations data.

Geographical Information System is a database and mapping system containing information describing the physical components and geographic boundaries of the water distribution system. It is designed to capture, store, update, manipulate and analyze all forms of geographically referenced information within defined restrictions. The purpose of using GIS is to establish a record-keeping database system, containing network and other types of data and to display these data in required formats. Information can be displayed as maps and reports.

Before an actual distribution system is modeled or simulated with a computer program, it must be represented in a form the computer can analyze. All computer programs available for water system analysis require detailed realistic numerical information on the pipe network used to transport the water from the supply points to the demand points. Therefore, collected information must be processed very carefully for all network elements in order to prepare an input data file compatible with the network. In other words, in order to develop a computer model, the collected data should be processed to make it suitable to the requirements of the model. Figure 5.23 shows the flowchart for data collection and analysis.

The base data file is the formulated data-input file that represents the existing water distribution system. It includes both the current physical description of the distribution system like pipe lengths, pipe diameters, roughness coefficients and ground elevations. Pipe lengths, pipes entering and leaving the nodes are taken from base data file in order to compute nodal weights. These values are used to distribute daily demand data to nodes for hydraulic simulation of the network.

Operations data related with the demand are transferred by SCADA where there is also a relationship and compatibility with MapInfo base data file to keep the model of the system accurate and up-to-date.

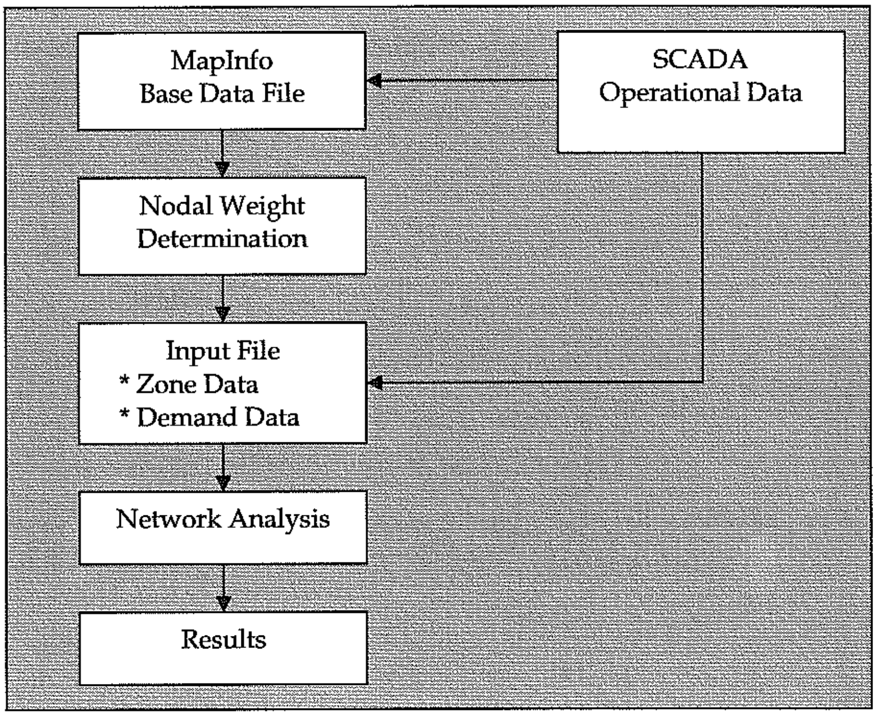

Figure 5.23 Data collection and analysis flowchart

5.3.3.1.2. Nodal Weight Determination

Water consumption at the demand nodes can not be measured or calculated but can be approximated by using some methods. Actually, there is not a real nodal demand in the water distribution networks. The demands are extracted through the links, not from junction nodes. However, this is an approximation method and softwares used for analysis and design of water distribution networks requires nodal demands as the demand values.

Nodal weights are the vital guide for calculating nodal demands. The procedure concerning the calculation of nodal weights in the platform of GIS is described as follows. Nodal weights were calculated using half lengths of the pipes joining related nodes. Each pipe and node was numbered. Lengths of pipes were labeled on them.

The nodes of related sub-zone were selected. This selected layer was saved as a different map. Two columns showing length of pipes entering and leaving each node were added to the browser of this new map under the headings of "from node" and "to node". A table including node number, from node and to node was formed by using SQL query item of MapInfo. Using the pipe information, a new table including pipe number, from node, to node information and lengths of indicated pipe was prepared. Tables were exported as dbf file to excel. Using these tables, total length of pipes in each sub-zone was found. Then length of each pipe was divided into two and one half was added to one side of the node and the other half was added to the node on the other side. Applying this procedure to all nodes, lengths added to nodes were summed up. This value was divided to the total length of pipes in the sub-zone and so, the weight of that node was obtained. If skeletonization is required, the nodal weights of removed junction are carried to adjacent nodes by using a distance-weight technique.

Then, the total demand value obtained from the diurnal curve for a selected day is multiplied by the nodal weights of the junction nodes in order to find the nodal demands to be used in preparation of computer program input file and in network analysis part.

5.3.3.1.3. Determination of Daily Demand Curve

Daily demand curve is derived using the continuity equation (eqn. 5.5). The inputs to the system are the flowrate supplied by the pump, P23 and the outputs from the system can be accepted as the flowrate filling the tank, T53, the nodal demand values and the leaks in the system. To calculate the flowrate supplied by he pump, P23, the difference between the pressures at the exit and at the entrance of the pump is used. This difference is entered as head supplied to the curve of related pump and the value corresponding to this difference is taken as the discharge supplied by the pump. In case of more than one pump working, the discharges corresponding to same head are summed to determine the flowrate supplied by pumps working parallel to each other.

The flowrate supplied by the tank, T53, or the value filling it, is computed by taking the average of water levels in tank 1 and tank 2. The difference between the average levels of a tank corresponding to a desired time interval is found and this value is multiplied by the base area of the tank to compute the volume change during the indicated period of time. The base area of the tank is found by dividing the total volume of tank, 5000 m^3 to the height of it, 6.5 m.

$$\bar{I} - \bar{Q} = \frac{dS}{dt} \tag{5.5}$$

where;

$\bar{I}$: the average flowrate entering to the system for a period of dt, m^3/hr

$\bar{Q}$: the average flowrate going out from the system for a period of dt, m^3/hr

dS : the storage in the tank T53 for a period of dt, m^3

The constructed computer model requires network and operations data. The network description data for N8 pressure zone was taken from ASKİ in GIS environment. It includes both the current physical description of the distribution system like pipe lengths, pipe diameters, roughness coefficients and ground elevations and also geographic boundaries of the network. Operations data related with the demand were transferred from SCADA and then processed to obtain the daily demand curve corresponding to the selected date.

In the application of LEAKSOL, the input file being used in computer program was prepared using the data taken from SCADA belonging to the date of 27.05.2001. A field study, comprising the identification of leakage in N8-3 pressure zone and in its sub-zones, was conducted in 2001 (Özkan, 2001). At this field work, to examine the consumption for entire N8-3 zone, data had been collected belonging to 13 days from 22.05.2001 to 03.06.2001. In N8-3 pressure zone application of codes, the date of 27.05.2001 was selected among 13 days having SCADA records. The reason for selection of that day can be explained as follows:

Field study implemented in N8-3 pressure zone in 2001 resulted in an average leakage of 60 m^3/hr for whole zone. The daily demand curve of the zone (Figure 5.24) belonging to that day gave an average leakage of 56.90 m^3/hr in spite of sharp decrease at night. This is nearly same with the leakage amount estimated for entire network. Therefore, daily demand curve derived for the date of 27.05.2001 can represent the situation of the zone for the time field work was conducted.

Daily demand curve derived for N8-3 pressure zone shows both leakage and daily consumed water. In this analysis it was assumed that the minimum required pressure for consumers to take their required water was 20 m. Only when the nodal pressures are 20 m or more than this value, consumers take their required demand. Since the settling area in this region was composed of one storey buildings at the time when operational data were obtained, minimum required pressure of 20 m was accepted as sufficient to feed the network.

In the future, if the maps corresponding to pressure zones includes data related with number of consumers, storey number in buildings, it will be possible to identify minimum required pressures for the areas having different characteristics in the same pressure zone and more realistic results will be obtained from model applications.

Figure 5.24 shows the daily demand curve of whole N8-3 pressure zone at the date of 27.05.2001.

It can be seen from the above figure that daily demand curve has a sharp decrease at 4 a.m. Data taken from SCADA system and pump stations should have some errors. The SCADA system is expected to keep data for specified durations to obtain daily demand curves. Generally data sent by SCADA are in a period of 7.5 min or 15 min. This data included the water levels in the tanks (tank 1 & tank 2) and the pressures at the entrance and at the exit of the pumps. Since the information about pumping rate was not obtained from SCADA, it was computed by taking the information related with code number of the pumps and their working hours from pump station. Since there are three pumps connected parallel to each other, it is important to know which

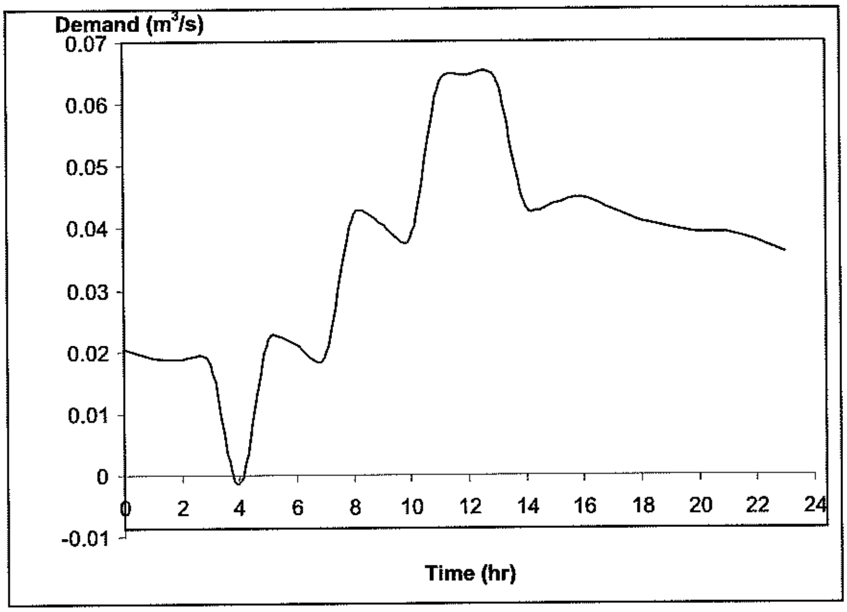

Figure 5.24 Daily demand curve -27.05.2001

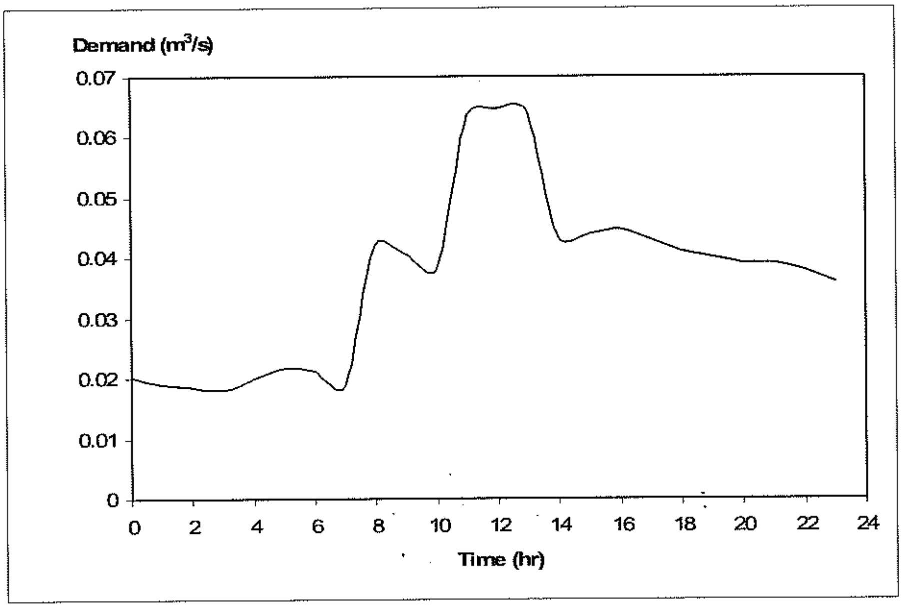

Figure 5.25 Daily demand curve after correction -27.05.2001

pumps are working at which time. Pump station P23 had some difficulties for recording pumped discharges for the study day. That's why pump performance curves were employed to determine the discharge by making use of the head produced by the related pump which was P1 for that day (exit pressure - entrance pressure=head produced by the pump).

In order to eliminate the error coming from the data, new demand value for the time where sharp decrease was encountered, was obtained by averaging the values corresponding to one hour before and after. Figure 5.25 shows the new daily demand curve after correction.

5.3.3.1.4. Parameters Used in Network Analysis

There are three primary types of data for network analysis: (i) network data, (ii)operations data and (iii) consumption data. Network data describe the physical characteristics of various components of the water distribution system. Operations data state the level at which facilities are operating and consumption data describe water demand. Network, operations and consumption data are essential in order to execute the hydraulic network solver and obtain a solution. Some of these data can be taken from existing records, but such data as consumption and C_{HW} values should be measured, or calculated beforehand.

5.3.3.1.4.1. Network Data

Network data include pipe and node data. The minimum pipe data include from node to node information, diameter, length and roughness coefficient of pipe to execute a network analysis program. Nodes represent either sources of supply or points of demand. Node data include node number, topographic elevation and demand data.

In this study, the digital network maps for N8-3 pressure zone were obtained from ASKİ. These maps contain pipe network, pump station and tank locations. Their databases also include attribute data like address code, pipe code, length, diameter, pressure zone from which it takes water, material type, installation year of each pipe, topographic elevations and coordinates of junction nodes. The X and Y coordinates (geographic location) of nodes, which refer to distances away from north-south and west-east, are important to export the network maps from MapInfo to the network solver.

MapInfo Professional, a software for mapping and geographic analysis, allows someone to better visualize and analyze data resulting in more effective business practices and decisions.

In this study, WaterCAD was used as network solver to perform the fixed demand analysis of the network. WaterCAD is an extremely powerful, yet easy-to-use program that helps civil engineers design and analyze water distribution systems by accepting demand as fixed. WaterCAD labels elements such as pipes, tanks and pumps automatically. The data for these elements can be entered individually, or using the built-in tables to enter data for the entire system at once.

There were two sets of data concerning the z-coordinate (topographic elevation) coming from the General Command of Mapping (HGK) and directly from Network Computer Center of ASKİ. During the network analysis, the data belonging to the General Command of Mapping were used.

5.3.3.1.4.2. Determination of Roughness Coefficient

Prediction of pipe roughness cofficients constitutes an important part of hydraulic network analysis. The roughness coefficient refers to a value that defines the roughness of the interior of a pipe. Two common roughness coefficients are the Hazen-Williams coefficient, C_{HW} and the Darcy-Weisbach f-value. Although the Darcy-Weisbach term is generally considered more accurate and flexible by giving information about flow regime, it is also more complicated and difficult to determine; consequently, the Hazen-Williams C-value is more commonly used in network modeling.

C_{HW} values range from 20 to 150. The higher the value, the smoother the interior surface of the pipe and the greater the carrying capacity of the pipe. Since the determination of C_{HW} values at site is very difficult, generally the approximate values in literature are used by knowing the material type and installation year of each pipe. The network was reported to be completely reconstructed in 1992 with ductile iron pipes. Considering the age of pipes and the year when the field study was performed, the C_{HW} values were assumed to be '130' for all pipes during the network analysis.

5.3.3.1.4.3. Operations Data

Operations data describe actual system characteristics at a given time. In this study, the operations data for storage tank include information about tank geometry and initial water levels. Operations data for pumps include data for use in describing the pump-flow characteristics curve and the pressure at the entrance of the pump to simulate the program for certain working conditions. The data taken from SCADA gives information about the water level in the tanks and the pressure at the entrance and at the exit of the pumps; but does not include code number of working pumps. Since the pumping station of P23 is composed of three parallel pumps used at different times or at the same time, it was necessary to know that which pump was in operation. Therefore, the necessary part of data was taken directly from the log books of the pump station, P23.

5.3.3.1.4.4. Consumption Data

Demand is assigned to the nodes in a network model in case of fixed demand analysis. Determining consumption and the distribution of consumption throughout the network is the most important and time-consuming part of analysis. Correct estimation of the rate of water consumption at demand nodes is rather difficult; accurate estimation of nodal demands should be performed by detailed calibration studies (Walski, 1983).

In this work, the consumption data were determined by distributing the average leakage value taken from the field work performed in to the nodes of studied zone based on the length of pipes joining to that node.

5.3.3.2. Determination of Weighted Mean Pressure

The average water leakage for N8-3 pressure zone was accepted as 60 m^3/hr (Özkan, 2001). This is the value obtained from the field work performed at this pressure zone in 2001 by dividing the zone into 6 sub-zones in order to identify the amount of leakage and to determine the relationship between leakage and pressure.

Firstly, nodal weights were calculated for the study area as explained in part 5.3.3.1.2. Then, the average leakage of 60 m^3/hr, i.e., 0.01667 m^3/s, was distributed to the nodes of pressure zone by multiplying with nodal weights. It was assumed that there was no other consumption at the zone for this trial. Network data was imported by the program. Then, WaterCAD was executed for distributed leakage values and operation data given in below scenario assuming that tank, T53 was full and pump, P23 was not working. The initial storage tank elevation was calculated as the summation of the tank bottom elevation and the tank level at a certain hour. The head at the suction side of the pumping station was computed as the summation of topographic or ground elevation and the pressure at the suction side of the pumping station.

In computations (Figure 5.21) :

Initial Tank Elevation = 1148.80 + Tank Level

Reservoir Elevation = 1047.33 + Pressure at the suction side of the pumping station

The hydraulic analysis of the network was performed by using WaterCAD for the scenario given below. Demand was accepted as fixed and leakage was not taken into consideration.

Scenario:

Pump	not working
Tank	full
Leakage	0.01667 m^3/s

At the end of the execution, heads at the start and at the end node of each pipe were taken from result tables of WaterCAD. Average pressures for each pipeline were calculated with these head values by using eqn.5.3.

Then, weighted mean pressure was calculated by multiplying the length of each pipe with its average pressure, obtained from network analysis, and then dividing it to the total pipe length of the studied area.

$$\overline{P} = \frac{\sum \lfloor P_{ij} * L_{ij} \rfloor}{\sum L_{ij}} \tag{5.6}$$

5.3.3.3. Determination of Leakage Coefficients

Average leakage for N8-3 pressure zone, i.e.60 m^3/hr, and the weighted mean pressure computed for the scenario given previously were used to determine the leakage coefficient, C_l for three N values (Table 5.10). The C_l coefficient, that is the only unknown in pressure-leakage relationship, was calculated by using the following equation for given units. The weighted mean pressure was entered to the leakage equation to find the C_l coefficient. Afterwards, the leakage equation with known coefficients, N and C_l was used for remained part of the analysis.

$$C_l = \left[\frac{QS}{\sum L * \overline{P}^N} \right] \tag{5.6}$$

where, QS is the average leakage for the zone in m^3/s, L is total length of pipes in m and $\overline{P}$ is weighted mean pressure in m.

Table 5.10 Leakage coefficients

N	C_l
1.18	3.060E-09
1.00	6.448E-09
0.50	4.972E-08

5.3.4. Determination of Required Demand

Required demand values to be used in further studies in N8-3 pressure zone, should be determined from daily demand curve of the zone corresponding to date of 27.05.2001. The procedure can be explained as follows:

1-The average leakage, i.e.,60 m^3/hr, was subtracted from demand values obtained from the daily demand curve given in Figure 5.25, to find required demand of the system. Figure 5.26 presents the schematic representation of terms used in determination of required demands for leakage coefficients given in Table 5.10. Figure 5.27 shows required demand curve after subtraction of average system leakage from values of daily demand curve.

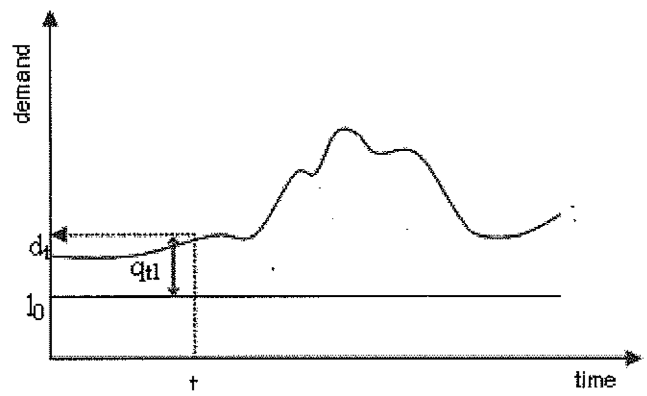

Figure 5.26 Schematic representations of terms

where;

d_t : value taken from daily demand curve for time t

q_{t_1} : required demand at time t for first trial

l_0 : average leakage for the network, i.e., 60 m³/hr for this study

q_{t_1} is found from;

$$q_{t_1} = d_t - l_0 \tag{5.7a}$$

2- q_{t_1} values were distributed to the nodes of N8-3 pressure zone by multiplying them with nodal weights computed previously. Hydraulic analysis of the network for three pairs of leakage parameters, N and C_l, was performed by considering fixed demands and pressure dependent leakage terms in node flow continuity equations. Input files were prepared by using operations data given in Table 5.11. The demand factor can be defined as the ratio of demand at any hour of a day to the maximum demand of the same day. They were calculated to be used in network analysis and demonstrated at column (7) of Table 5.11. Weighted mean pressure and total amount of water leaking from the system for each hour of daily required demand curve were obtained at the end of the analysis.

3-Summation of total leakage, l_{t_1} and required demand, q_{t_1} was compared with the value taken from daily demand curve of N8-3 pressure zone for time t.

Check for;

$$l_{t_1} + q_{t_1} \stackrel{?}{=} d_t \tag{5.7b}$$

where;

l_{t_1} : total leakage obtained from the first hydraulic analysis of the network for time t

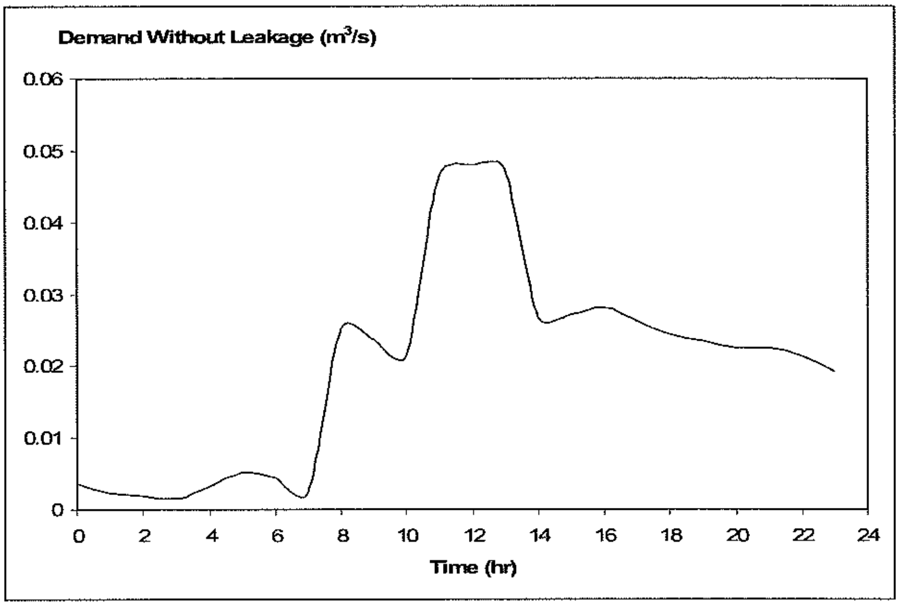

Figure 5.27 Daily required demand curve after leakage subtraction-27.05.2001

Since this criterion could not be satisfied, new values of required demand were computed from eqn. 5.7c to be used in the next hydraulic analysis of the network.

$$q_{t_2} = d_t - l_{t_1} \tag{5.7c}$$

4- q_{t_2} values were distributed again to nodes. Hydraulic analysis of the network for leakage coefficients given in Table 5.10 was performed separately. New leakages, l_{t_2} were obtained. Criterion given in eqn. 5.7b was checked. When summation of leakage and required demands for all hours was equal to the value taken from daily demand curve, execution was stopped.

After having completed the process for calculation of required demands and leakages, related values were tabulated on Table 5.12 for N values of 1.18, 1.0 and 0.5.

Table 5.11 Operation data for N8-3 Pressure Zone_27.05.2001

Time (hr) (1)	Tank Elevation (m) (2)	Head at the discharge side of the pump (m) (3)	Demand (m^3/s) (Figure 5.24) (4)	Demand after correction (m^3/s) (Figure 5.25) (5)	Required demand (m^3/s) (Figure 5.27) (6)	DF (%) (7)
0	1150.77	1152.83	0.0203	0.0203	0.00368	7.67
1	1150.87	1152.83	0.0190	0.0190	0.00234	4.88
2	1150.98	1152.83	0.0187	0.0187	0.00201	4.19
3	1151.09	1152.83	0.0182	0.0182	0.00149	3.10
4	1151.21	1152.83	-0.0014	0.0200	0.00331	6.91
5	1151.43	1154.00	0.0218	0.0218	0.00514	10.71
6	1151.53	1154.00	0.0212	0.0212	0.00453	9.45
7	1151.64	1154.00	0.0189	0.0189	0.00225	4.70
8	1151.75	1154.00	0.0420	0.0420	0.02529	52.72
9	1151.75	1154.00	0.0405	0.0405	0.02383	49.67
10	1151.75	1154.00	0.0381	0.0381	0.02145	44.72
11	1151.75	1154.00	0.0634	0.0634	0.04677	97.51
12	1151.64	1154.00	0.0646	0.0646	0.04797	100.00
13	1151.53	1154.00	0.0641	0.0641	0.04747	98.97
14	1151.43	1154.00	0.0435	0.0435	0.02685	55.97
15	1151.43	1154.00	0.0439	0.0439	0.02727	56.85
16	1151.43	1154.00	0.0448	0.0448	0.02810	58.58
17	1151.43	1154.00	0.0429	0.0429	0.02627	54.75
18	1151.43	1153.80	0.0411	0.0411	0.02443	50.93
19	1151.43	1153.80	0.0401	0.0401	0.02347	48.93
20	1151.43	1153.80	0.0392	0.0392	0.02251	46.93
21	1151.43	1153.80	0.0392	0.0392	0.02251	46.93
22	1151.43	1153.80	0.0380	0.0380	0.02134	44.49
23	1151.43	1153.80	0.0360	0.0360	0.01933	40.30

Table 5.12 Required demands and leakages

	N=1.18		N=1.0		N=0.5	
Time (hr)	**Demand (m^3/s)**	**Leakage (m^3/s)**	**Demand (m^3/s)**	**Leakage (m^3/s)**	**Demand (m^3/s)**	**Leakage (m^3/s)**
0	0.0047	0.0156	0.0046	0.0157	0.0043	0.0160
1	0.0034	0.0156	0.0033	0.0157	0.0030	0.0160
2	0.0031	0.0156	0.0029	0.0158	0.0027	0.0160
3	0.0026	0.0156	0.0024	0.0158	0.0022	0.0160
4	0.0043	0.0157	0.0042	0.0158	0.0040	0.0160
5	0.0059	0.0159	0.0058	0.0160	0.0057	0.0161
6	0.0053	0.0159	0.0052	0.0160	0.0051	0.0161
7	0.0030	0.0159	0.0029	0.0160	0.0028	0.0161
8	0.0262	0.0158	0.0260	0.0160	0.0259	0.0161
9	0.0246	0.0159	0.0245	0.0160	0.0244	0.0161
10	0.0222	0.0159	0.0221	0.0160	0.0220	0.0161
11	0.0477	0.0157	0.0475	0.0159	0.0474	0.0160
12	0.0489	0.0157	0.0488	0.0158	0.0486	0.0160
13	0.0484	0.0157	0.0483	0.0158	0.0481	0.0160
14	0.0277	0.0158	0.0276	0.0159	0.0274	0.0161
15	0.0281	0.0158	0.0280	0.0159	0.0278	0.0161
16	0.0290	0.0158	0.0289	0.0159	0.0287	0.0161
17	0.0271	0.0158	0.0270	0.0159	0.0268	0.0161
18	0.0253	0.0158	0.0252	0.0159	0.0250	0.0161
19	0.0243	0.0158	0.0242	0.0159	0.0240	0.0161
20	0.0234	0.0158	0.0233	0.0159	0.0231	0.0161
21	0.0234	0.0158	0.0233	0.0159	0.0231	0.0161
22	0.0222	0.0158	0.0221	0.0159	0.0219	0.0161
23	0.0202	0.0158	0.0201	0.0159	0.0199	0.0161

5.3.5. Leakage Control through Flow Control Valves

CODE I application of N8-3 pressure zone was performed for three cases:

Case 1: Uncontrolled Network

First case simulates two scenarios: (1) valves are fully open, or, (2) there is no valve in the system. For both of these scenarios, the network is accepted as uncontrolled.

Leakages presented in Table 5.12 also represent values corresponding to the case in which network is uncontrolled. Leakages obtained for uncontrolled situation of the pressure zone will be compared with those resulted from other cases in the following parts.

The daily demand curve derived on 27.05.2001 with leakages computed for the first case can be seen in Figure 5.28.

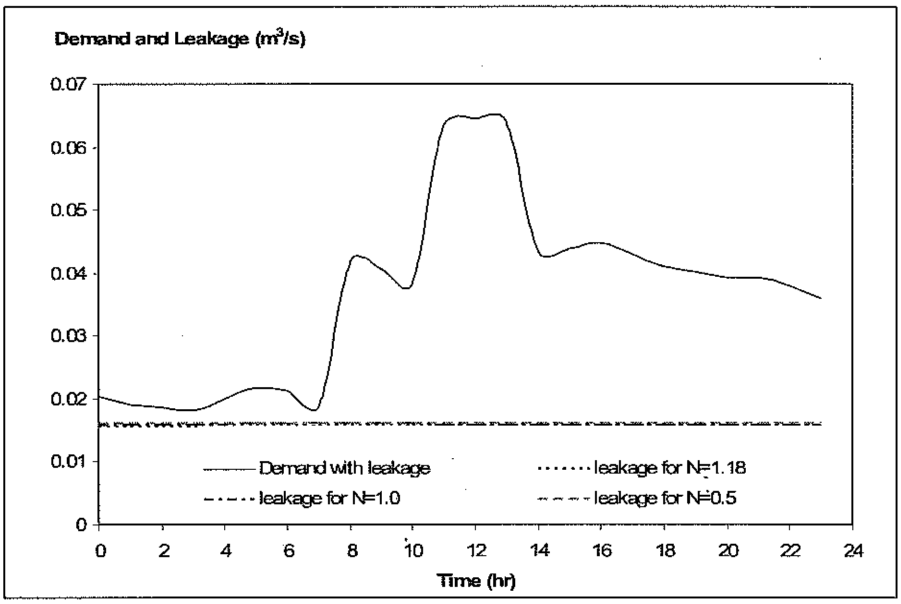

Figure 5.28 Daily demand curve and leakage line for N values-27.05.2001

Figure 5.28 shows that leakages for three N values are very close to each other. Since average system pressure for N8-3 pressure zone is relatively high and changes in a narrow range during the whole day, whatever the relation between leakage and pressure is, nearly same leakage values are obtained for this zone.

Case 2: Network without Critical Head Requirement

The second case simulates the behavior of a valved network for a situation in which no critical head requirement is described.

During the field study conducted in 2001, N8-3 pressure zone was separated into six sub-zones (Özkan, 2001). Number of subzones was limited due to the location and availibility of isolation valves. The isolation was performed firstly by selecting the valves required on the network map of N8-3 pressure zone; the locations of valves which allow the passage of water from main-line to the studied part of the network and all valves which are necessary to isolate the sub-zones were determined. In selecting the valves for isolation, the main idea was that each sub-zone would have a connection with the storage tank during the study. This means that the storage tank would deliver or take water in any case. It was intended that when the valves were closed completely, the sub-zone would be separated from the other part of the network entirely and would work only with main-line connecting pump station and the storage tank. For the field work conducted in 2001, isolation valve locations were selected to isolate sub-zones, but in this case study, valves were tried to be well located in order to achieve a better reduction in leakage.

For the second case, it was aimed to locate valves at entrances of these sub-zones in so far as. Their locations can be seen in Figure 5.29. Additional valves can be incorporated into network in order to reflect the experience of the operator regarding to this network. The minimum required pressure was accepted as 20 m above the ground elevation at each demand node. It means that consumers take their required demand if the pressure at this node reaches to 20 m above its ground level. $H_{critical}$ values, which control the optimization problem, for all demand nodes were accepted as zero above ground elevations. The purpose of this assumption is to show the behaviour of flow control valves if system pressure is not restricted at reference nodes.

Input files were prepared in the form as explained in this chapter previously. The optimization analysis was performed by using CODE I in order to find flow control valve settings minimizing the leakage flow under described conditions. The analysis was repeated for three N values, i.e. N=1.18, N=1.0 and N=0.5, by using data corresponding to date of 27.05.2001

In this case, optimization program gave the minimum leakage by finding all the valves as fully closed for these three N values. Also, total pipe leakage and total consumption in the system for this optimum result were displayed. It can be concluded that if there is no restricting condition for nodal heads in a network, minimum leakage value can be obtained by closing all of the valves. But, this time the amount of water subtracted from the network decreases compared to required demand. In other words, consumers could not get enough water.

Figure 5.30 shows decrease in daily demand curve of the studied date when there is no critical head requirement for mentioned N values.

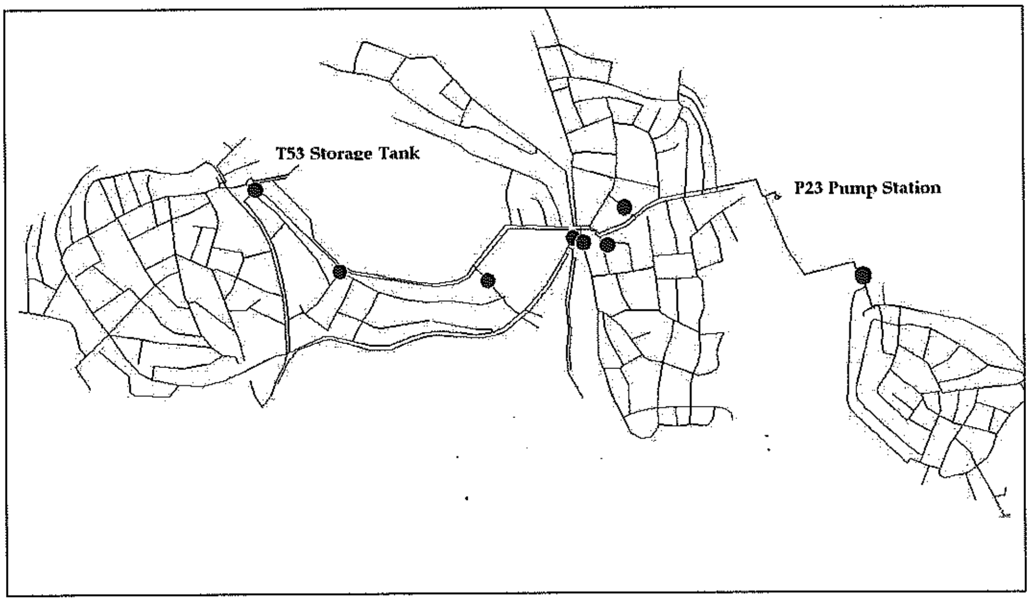

Figure 5.29 Locations of flow control valves

Minimum head requirements described for nodes control the amount of nodal consumptions. In codes, node flow continuity equations including consumption terms change depending on the described minimum required head values, so the valve openings obtained from these codes are affected from this criterion. In other words, in addition to critical head requirement, minimum head criterion also controls the optimization problem.

<u>Case 3: Network with Critical Head Requirement</u>

The third case simulates the behavior of a valved network for a situation in which heads at some nodes or at all nodes are restricted to a certain value. This time, computer program is expected to set valves in order to obtain nodal heads greater than or equal to described critical heads.

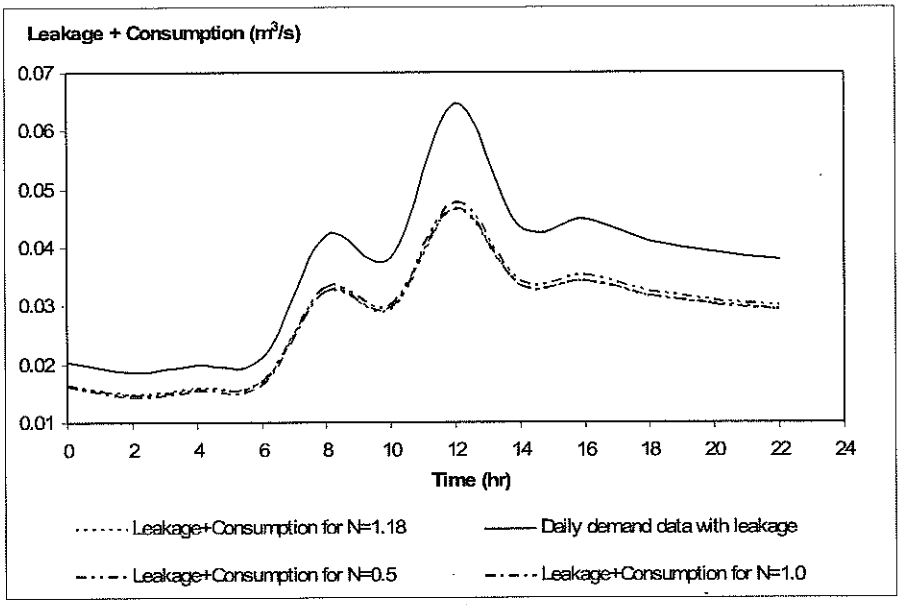

Figure 5.30 Decrease in demand curves

For the third case, $H_{critical}$ values were accepted as 20 m above the ground level for all nodes so that pressure would be sufficient and consumers could take their required demands where minimum required pressure was also 20 m. above the ground. This time since $H_{critical}$ values were defined different than the previous case, different valve settings were obtained. Analysis was performed for whole day by 2 hours intervals. Nearly same valve openings for different N values were obtained (Tables 5.13-5.15).

Figures 5.31-5.33 show the leakage values in N8-3 pressure zone for three cases, i.e., valves were fully open, valves were closed, i.e., valve settings when $H_{critical}$ values were accepted as zero above ground and leakage for optimum valve settings for three N values.

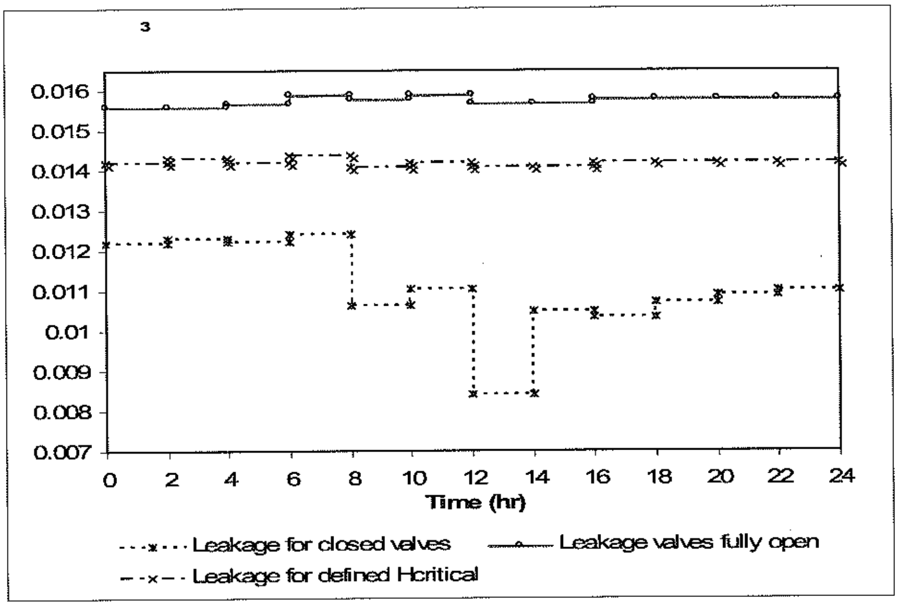

Figure 5.31 Leakage for controlled and uncontrolled flow control valve settings in case of N=1.18.

Table 5.13 Valve settings ($H_{critical}$=20m+GE and N=1.18)

Time (hr)	0	2	4	6	8	10	12	14	16	18	20	22
V(1)	0.000	0.000	0.000	0.000	0.221	0.307	1.000	0.333	0.406	0.159	0.153	0.148
V(2)	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
V(3)	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
V(4)	0.000	0.000	0.000	0.000	0.000	0.000	0.038	0.000	0.000	0.000	0.000	0.000
V(5)	0.746	0.085	0.224	0.010	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
V(6)	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
V(7)	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
V(8)	0.004	0.004	0.004	0.004	0.009	0.008	0.015	0.010	0.010	0.009	0.009	0.008

As can be seen from these figures that leakage is maximum if valves are fully open (Case 1) whereas leakage is minimum when there is no restriction related with nodal heads. In other words, when flow control valves are closed completely, system pressure and so leakage are minimized (Case 2). In case of $H_{critical}$ values defined different than ground elevations (Case 3), leakage is in between that of obtained from other two cases.

Valve settings for valves V(1), V(4), V(5) and V(8) are presented in Figures 5.34-5.37. These settings were very close to each other for all N values. For others, i.e. V(2), V(3), V(6) and V(7), valves were obtained as closed. This means that whatever the relation between pressure and leakage is, for same controlled conditions valves behave similarly to each other with settings nearly compatible especially in a network like N8-3 pressure zone where pressure changes in a narrow range. Since pressure in N8-3 is relatively high during the day, valve settings exhibit an attitude with values very close to each other (mostly closed) for different N values. However, the amount of leakages is different due to the described relations between pressure and leakage, i.e. three N values for which solutions are provided.

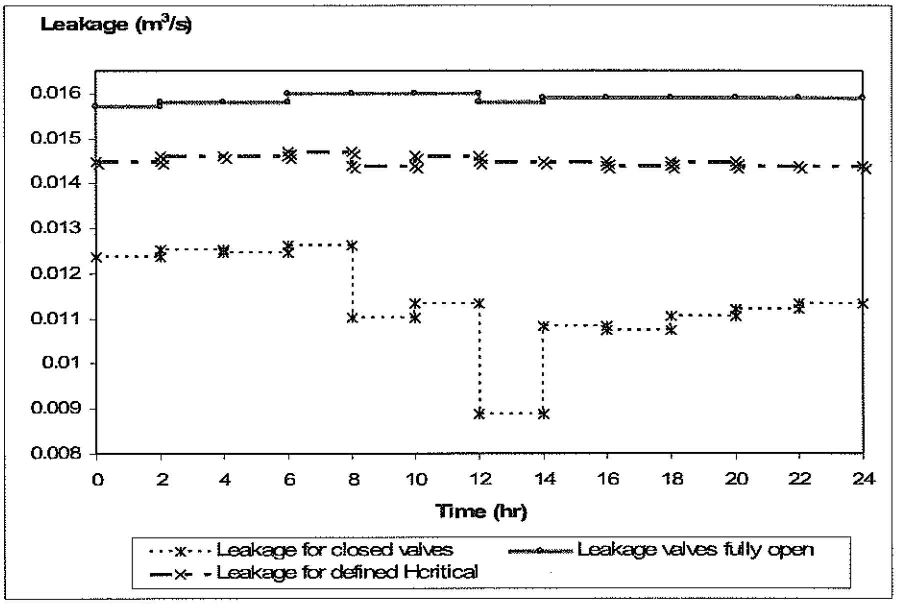

Figure 5.32 Leakage for controlled and uncontrolled flow control valve settings in case of N=1.0.

Table 5.14 Valve settings ($H_{critical}$=20m+GE and N=1.0)

Time (hr)	0	2	4	6	8	10	12	14	16	18	20	22
V(1)	0.031	0.000	0.000	0.000	0.145	0.311	1.000	0.325	0.397	0.256	0.143	0.154
V(2)	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
V(3)	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
V(4)	0.000	0.000	0.000	0.000	0.000	0.000	0.050	0.000	0.000	0.000	0.000	0.000
V(5)	1.000	0.025	0.128	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
V(6)	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
V(7)	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
V(8)	0.004	0.004	0.004	0.004	0.009	0.008	0.015	0.010	0.010	0.009	0.009	0.008

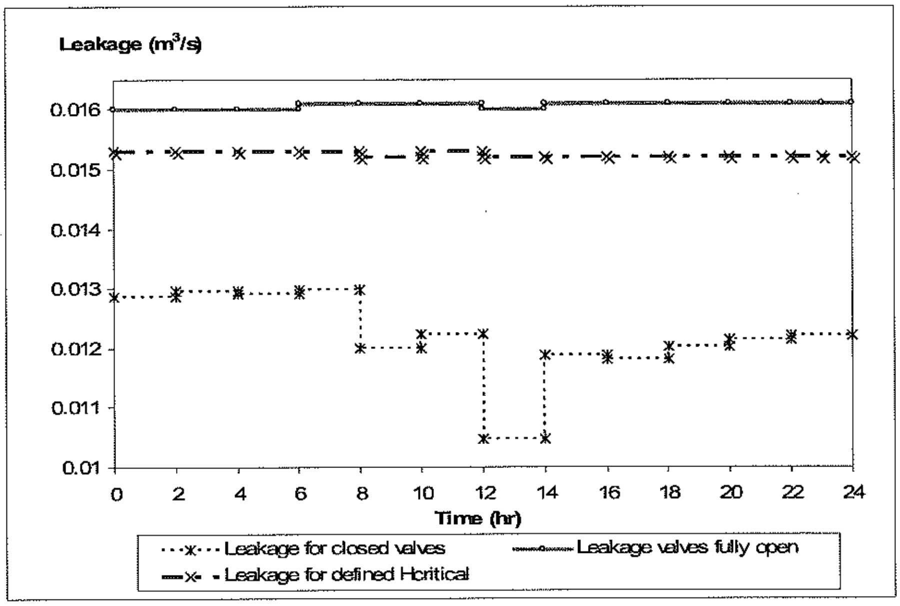

Figure 5.33 Leakage for controlled and uncontrolled flow control valve settings in case of N=0.5.

Table 5.15 Valve settings ($H_{critical}$=20m and N=0.5)

Time (hr)	0	2	4	6	8	10	12	14	16	18	20	22
V(1)	0.042	0.000	0.000	0.000	0.147	0.142	1.000	0.585	0.396	0.509	0.154	0.059
V(2)	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
V(3)	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
V(4)	0.000	0.000	0.000	0.000	0.000	0.000	0.040	0.000	0.000	0.000	0.000	0.000
V(5)	1.000	0.071	0.278	0.019	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
V(6)	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
V(7)	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000
V(8)	0.005	0.004	0.004	0.005	0.010	0.009	0.016	0.010	0.011	0.010	0.009	0.009

Better reduction in leakage may be obtained by changing numbers and locations of valves. Since difference among the ground elevations in N8-3 pressure zone, i.e. the gap between highest and lowest point of the network, is too much, approximately 95 m, it is very difficult to have an effective reduction in leakage. For a network having a relatively flat topology, the application of LEAKSOL can produce favorable variance in result. Also, study of zone by dividing into sub-zones can help to operators having much more control on the network with increased reduction in leakage.

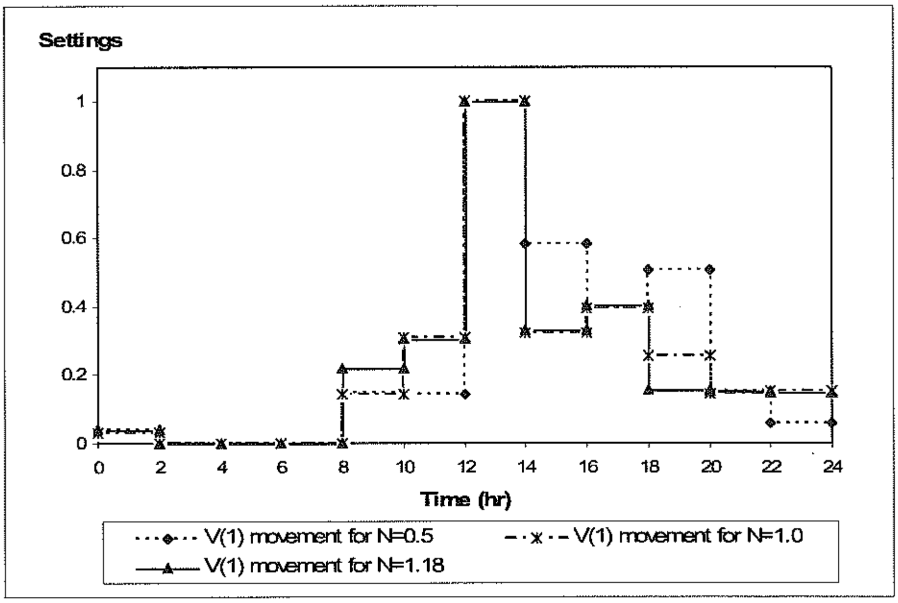

Figure 5.34 V(1) settings for all N values

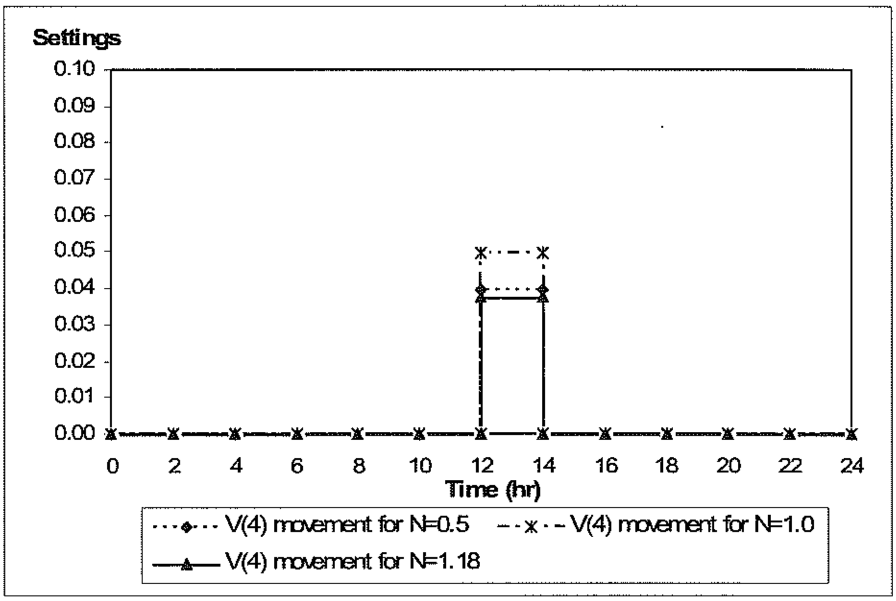

Figure 5.35 V(4) settings for all N values

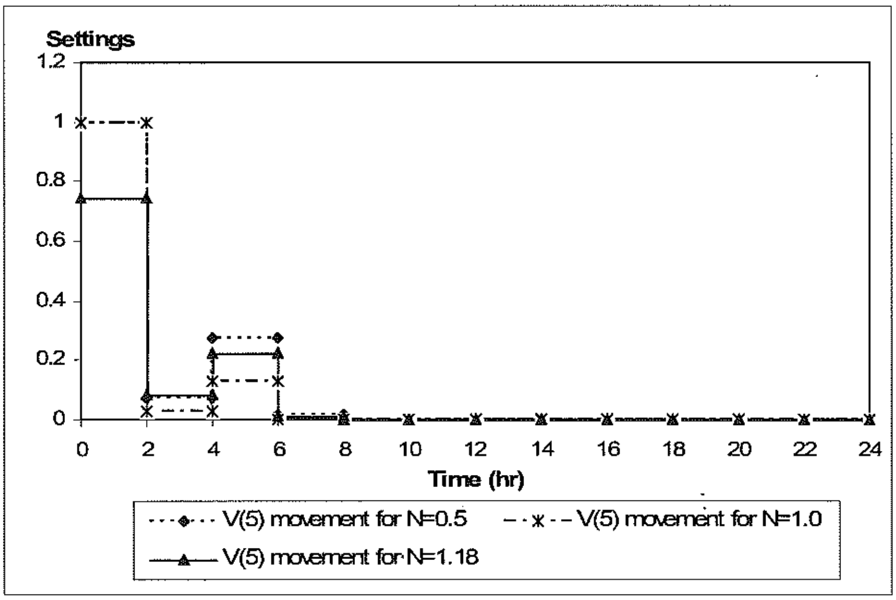

Figure 5.36 V(5) settings for all N values

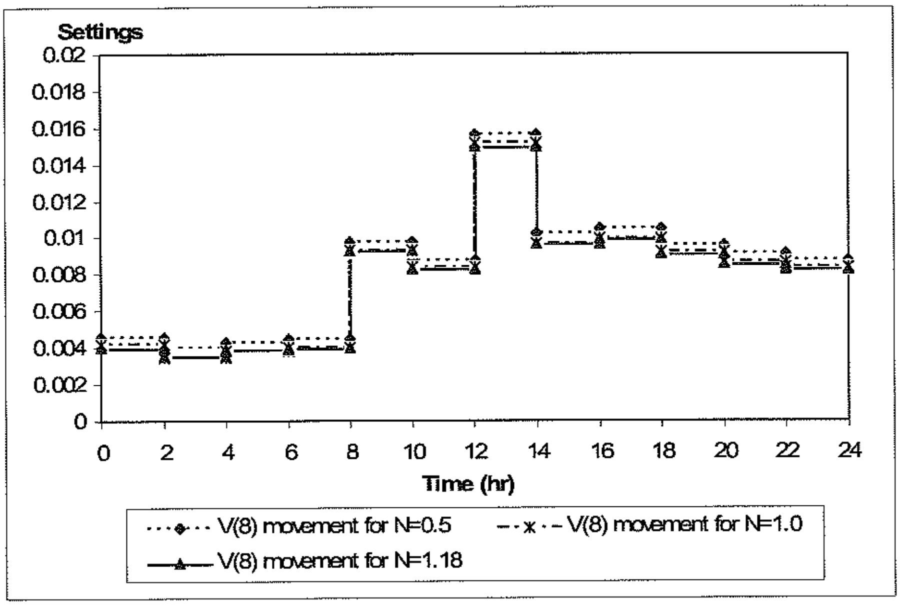

Figure 5.37 V(8) settings for all N values

Case 4: Analysis for Different Valve Locations

In this case, it was aimed to reduce leakage in N8-3 pressure zone by changing the number and locations of flow control valves. Seven valves were located inside the subzone shown in Figures 5.38a and 5.38b.

The minimum required pressure was accepted as 20 m at each demand node. $H_{critical}$ values for all demand nodes were accepted as 20m above the ground elevation. Purpose is to show the behaviour of network for differently placed flow control valves. Input files were prepared in the form as explained in this chapter previously. The optimization analysis was performed by using CODE I in order to find flow control valve settings minimizing the leakage flow under described conditions. The program was executed by assuming a linear relationship between leakage and pressure, i.e., N=1.0, and by using data corresponding to date of 27.05.2001. The results were compared in Figure 5.39 with leakages computed for the valve combination given in Figure 5.29.

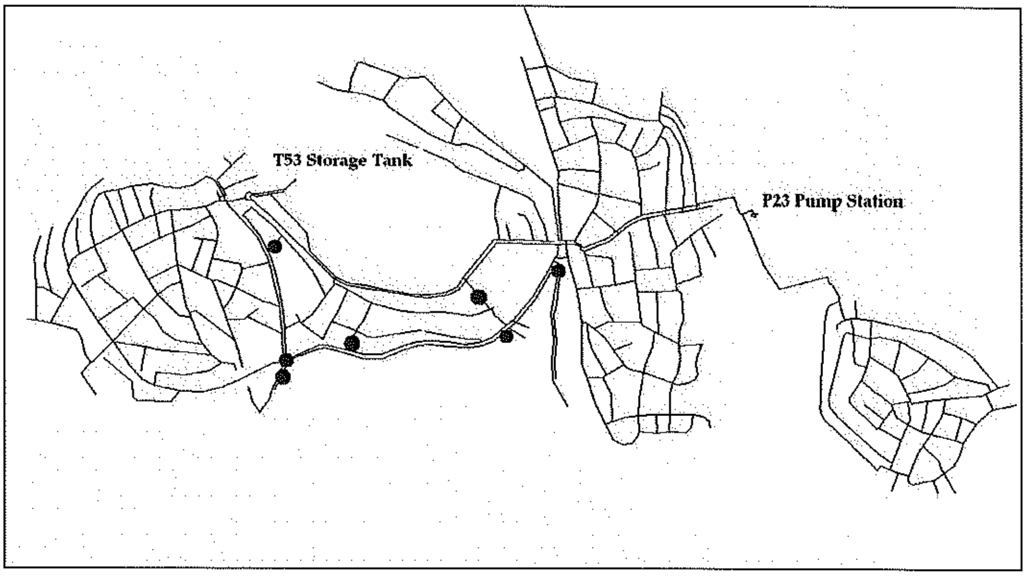

Figure 5.38a Location of valves inside the sub-zone

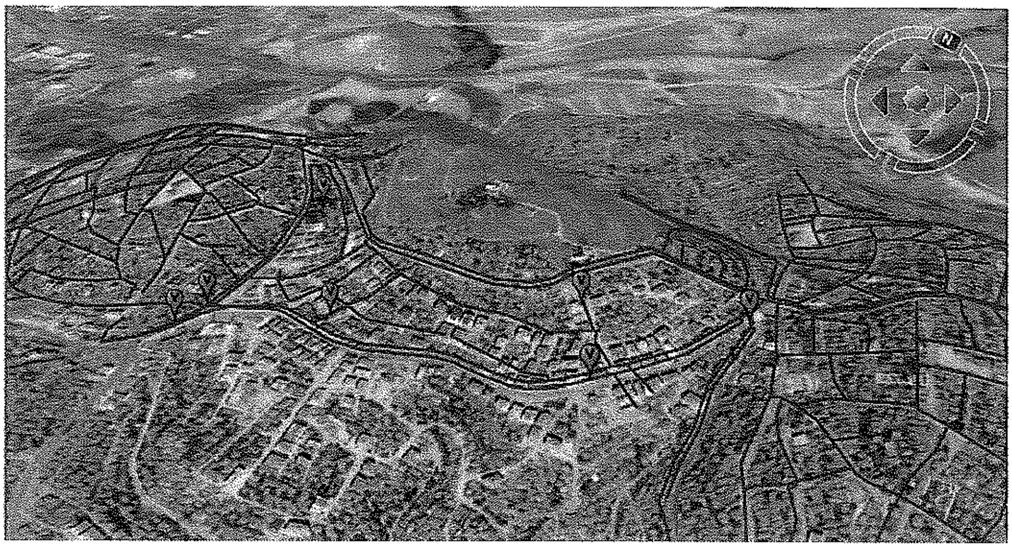

Figure 5.38b Location of valves inside the sub-zone on Google Earth

The analysis was resulted with leakage values greater than that of obtained in Case 3 for N=1.0 where minimum required pressure was 20m and critical head requirement was 20 m above the ground. Further achievement in leakage reduction could not be gained with this valve combination.

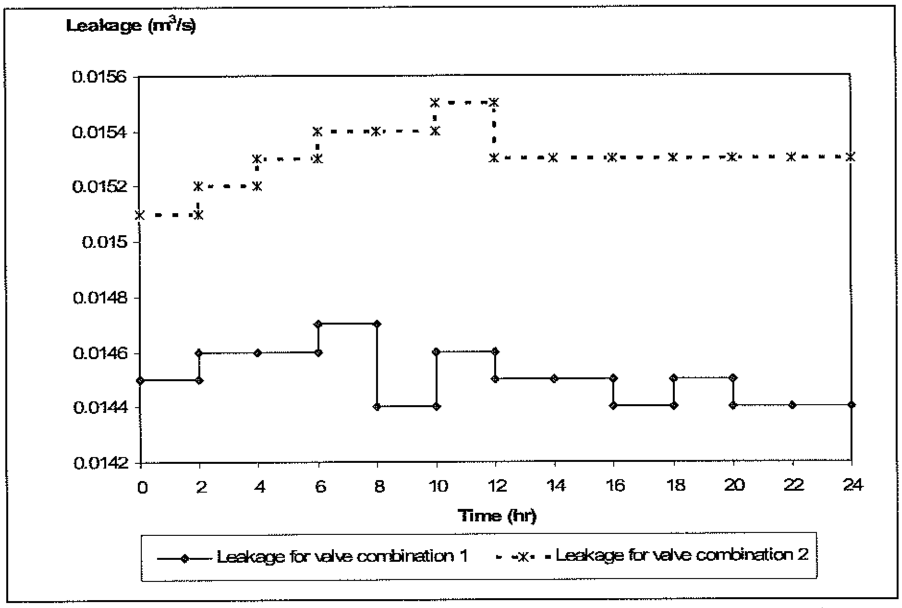

Figure 5.39 Comparison of leakages for N=1.0 in case of different valve locations

5.3.6. Leakage Control through Isolation Valves

Examine of CODE I on N8-3 pressure zone has shown that the flow control valve settings are not obtained as partially open always. Depending on the system characteristics and the critical head requirements described, it is usual to obtain fully closed valve settings. As stated previously, this gave an opinion about the usage of existing valves in the system for reduction of leakage by keeping them closed all the times with suitable combinations not affecting the consumption. Also, the most common type of valves in water distribution systems is the isolation valves. Therefore, a program applicable to these types of valves may be more economical and practical in the existing situations. CODE II works on combinations of isolation valves having settings 0 or 1. In other words, valves can only be on or off.

In this part, it is aimed to reduce leakage in N8-3 pressure zone through hydraulic analysis of isolation valve combinations (valve status: on/off) generated according to the valves considered. Leakages and consumption/demand ratios corresponding to each combination of valve settings are compared and best combination satisfying operating criteria is selected. The input files being used in computer program were prepared using the data taken from SCADA belonging to the date of 27.05.2001. CODE II was executed by taking into account same 8 valves indicated in Figure 5.29 as isolation valves. For three N values, solutions were provided. $H_{critical}$ and minimum required head were taken as 20 m above the ground level. The code performed the hydraulic analysis of 2^8=256 valve combinations for a day. The network was solved for its nodal pressures, pipe discharges, daily consumption, leaked amount of water and the number of violations for critical head requirements for each combination; the operator can see whether or not there is a deviation from intended nodal pressures by examining the outputs.

At the end of the execution of CODE II, leakages for following cases:

(1) valves are fully open,
(2) valves are closed,
(3) valve combination providing minimum leakage and at the same time satisfying the critical head requirement,

were chosen among the results in order to present them on a graph. Figures 5.40, 5.41 and 5.42 show the variation of leakages for these results for three N values. Also, valves settings for cases providing minimum leakages satisfying the critical head requirements were presented in Tables 5.16 – 5.18.

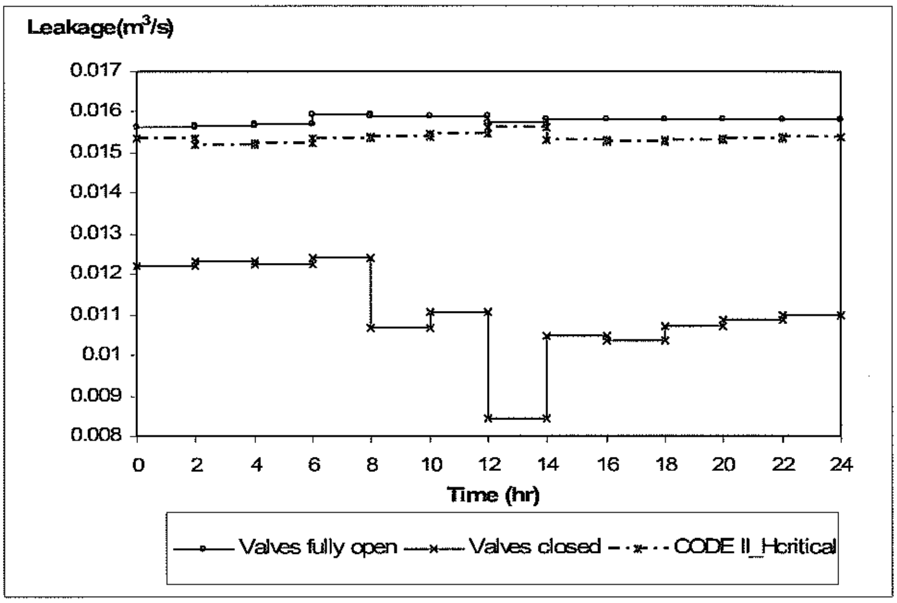

Figure 5.40 CODE II results for N=1.18

Table 5.16 Valve settings for the combination satisfying critical head requirement_N=1.18

Time (hr)	0	2	4	6	8	10	12	14	16	18	20	22
V(1)	1	0	0	0	1	1	0	1	1	1	1	1
V(2)	0	0	0	1	0	0	0	0	0	0	0	0
V(3)	0	0	0	0	0	0	0	0	0	0	0	0
V(4)	0	0	0	0	0	0	1	0	0	0	0	0
V(5)	0	0	1	0	0	0	0	0	0	0	0	0
V(6)	0	0	0	1	0	0	0	0	0	0	0	0
V(7)	0	1	0	0	0	0	0	0	0	0	0	0
V(8)	1	1	1	1	1	1	1	1	1	1	1	1

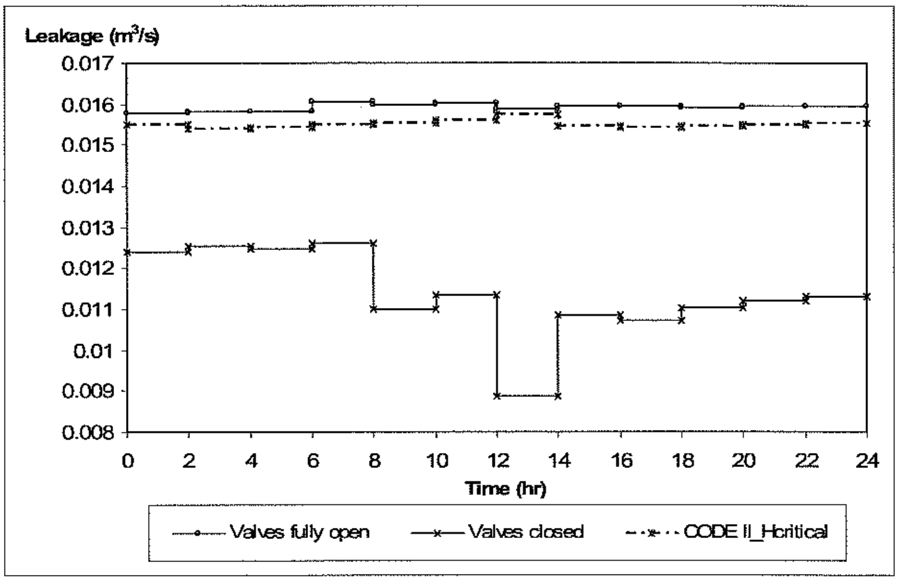

Figure 5.41 CODE II results for N=1.0

Table 5.17 Valve settings for the combination satisfying critical head requirement_N=1.0

Time (hr)	0	2	4	6	8	10	12	14	16	18	20	22
V(1)	1	0	0	0	1	1	0	1	1	1	1	1
V(2)	0	0	0	0	0	0	0	0	0	0	0	0
V(3)	0	0	0	0	0	0	0	0	0	0	0	0
V(4)	0	0	0	0	0	0	1	0	0	0	0	0
V(5)	0	0	1	0	0	0	0	0	0	0	0	0
V(6)	0	0	0	0	0	0	0	0	0	0	0	0
V(7)	0	1	0	0	0	0	0	0	0	0	0	0
V(8)	1	1	1	1	1	1	1	1	1	1	1	1

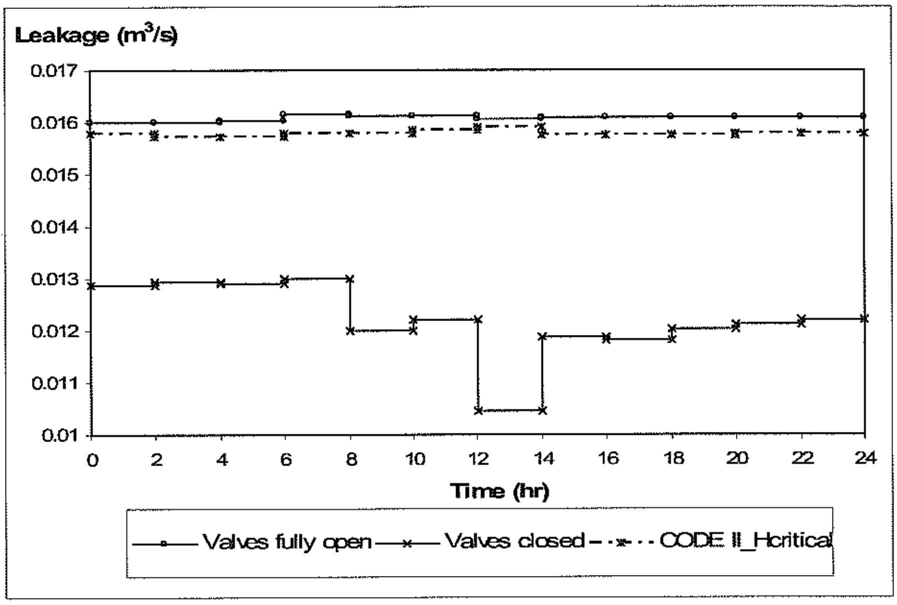

Figure 5.42 CODE II results for N=0.5

Table 5.18 Valve settings for the combination satisfying critical head requirement_N=0.5

Time (hr)	0	2	4	6	8	10	12	14	16	18	20	22
V(1)	1	0	0	0	1	1	0	1	1	1	1	1
V(2)	0	0	0	0	0	0	0	0	0	0	0	0
V(3)	0	0	0	0	0	0	0	0	0	0	0	0
V(4)	0	0	0	0	0	0	0	0	0	0	0	0
V(5)	0	0	1	0	0	0	1	0	0	0	0	0
V(6)	0	0	0	0	0	0	0	0	0	0	0	0
V(7)	0	1	0	1	0	0	0	0	0	0	0	0
V(8)	1	1	1	1	1	1	1	1	1	1	1	1

From the solution sets of three N values, leakage values corresponding to the valve combination [1, 0, 0, 0, 0, 0, 0, 0] with an average consumption/demand ratio of 83% were picked and figured in order to demonstrate that if the minimum head criterion is violated taking consumption/demand ratio into consideration, leakage reduction is increase.

Figures 5.43-5.45 compare the leakage for valve combination [1, 0, 0, 0, 0, 0, 0, 0] selected for a consumption/demand ratio not less than 83 %, minimum leakage satisfying the critical head requirement resulted from CODE II and leakage for optimally set flow control valve settings obtained from CODE I. It can be seen from these figures that leakages due to preferred combination of isolation valves are less than others. Overall reduction in leakage for this case is about 21 % for N=1.18, 20.5 % for N=1.0 and 19% for N=0.5. Another combination of valves having a higher consumption/demand ratio can be selected and applied through the day by the operator.

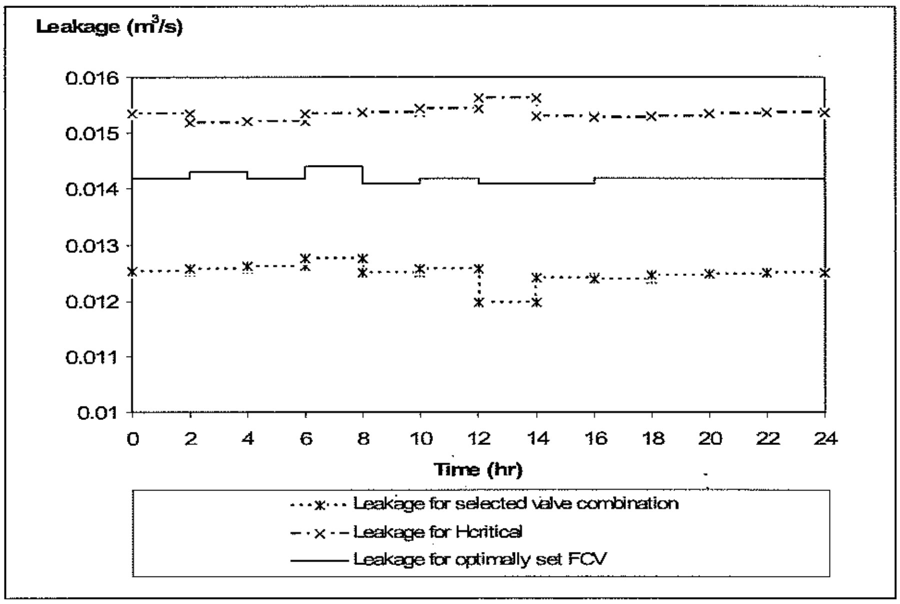

Figure 5.43 Leakage for selected valve combination in case of N=1.18

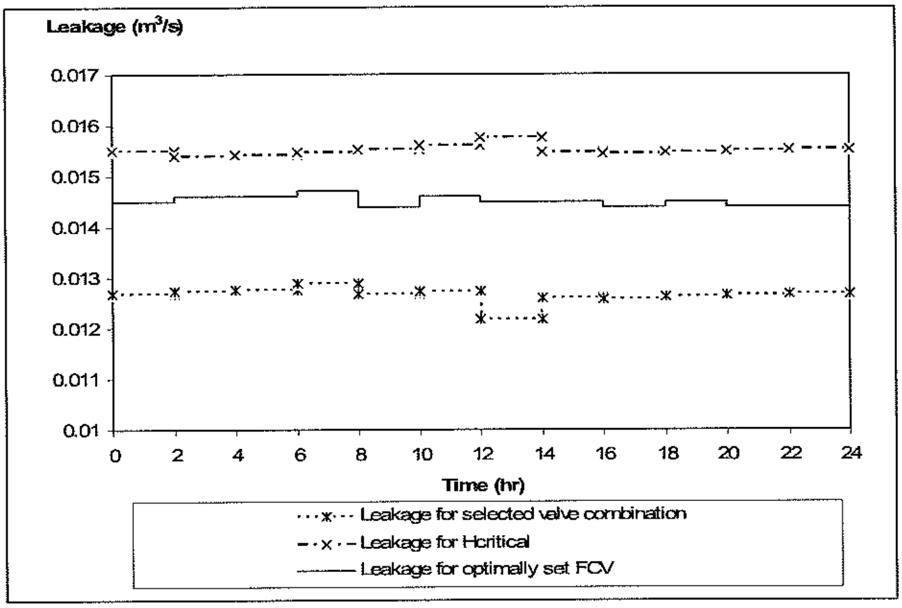

Figure 5.44 Leakage for selected valve combination in case of N=1.0

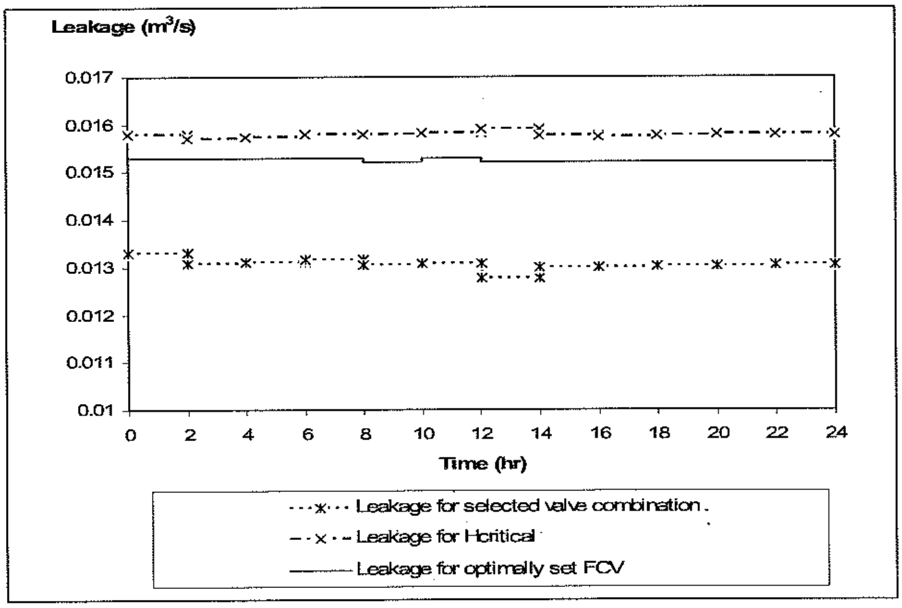

Figure 5.45 Leakage for selected valve combination in case of N=0.5

5.3.7. Comparison of Results

Leakages obtained from the execution of CODE I and CODE II satisfying the critical head requirements were compared in Figures 5.46-5.48 for three N values. Minimum leakages were obtained for the case in which all valves were closed. Reduction in leakage for this case is 25 % for N=0.5, 28.4 % for N=1.0 and 30 % for N=1.18. If optimal valve settings obtained from CODE I are used, reduction in leakage is 5.1 % for N=0.5, 9 % for N=1.0 and 10 % for N=1.18. Leakage can be reduced by 1.9 % for N=0.5, 2.4 % for N=1.0 and 2.62 % for N=1.18 in case of valve combination satisfying the critical head requirement resulted from the execution of CODE II. It is obvious that for defined critical head requirements, optimization code gives the minimum leakage value with optimum flow control valve settings. But, if isolation valves used instead of flow control valves, again a decrease in leakage can be observed as in these figures. Although the obtained leakage volumes are not completely minimized, a reduction throughout the day can be observed.

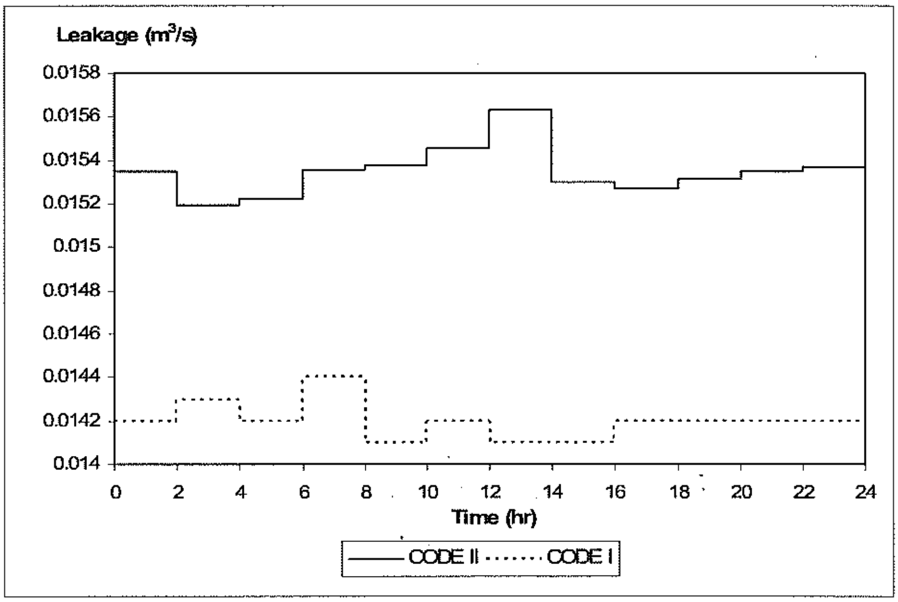

Figure 5.46 Comparison of leakages for N=1.18.

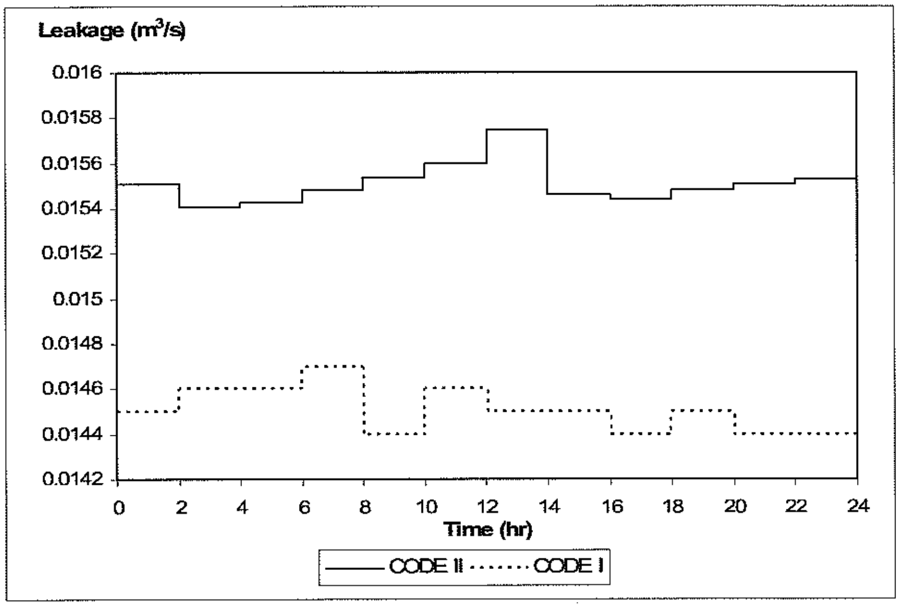

Figure 5.47 Comparison of leakages for N=1.0.

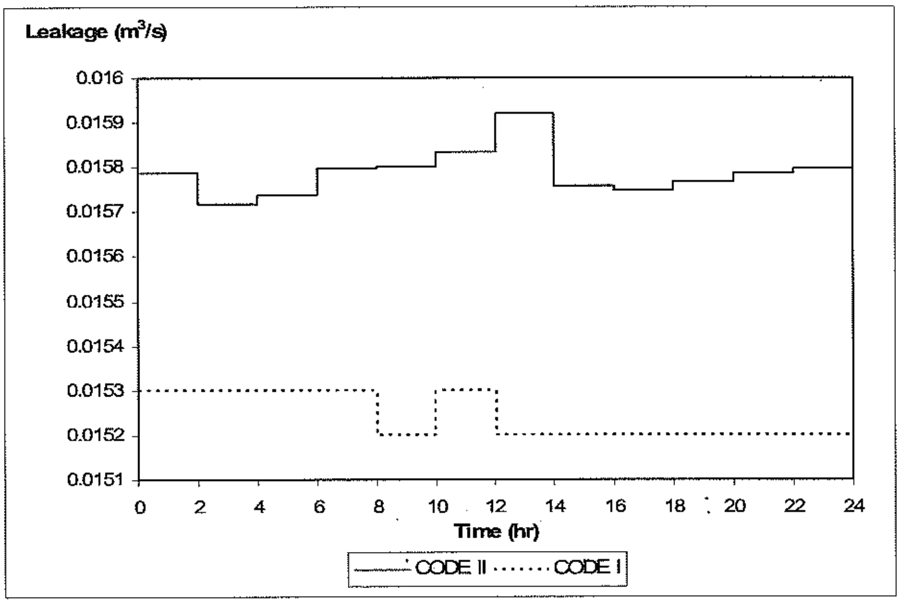

Figure 5.48 Comparison of leakages for N=0.5.

It can be concluded that isolation valve settings can be fixed for sub-zones or whole N8-3 pressure zone by using CODE II through long term analysis of demand curves.

Similar executions can provide realistic results to develop suitable operating policies for distribution networks. It is evident that water distribution networks are designed to meet the future demands and the life of a network is assumed to be 30-35 years (Bank of Provinces, 1998). At the beginning of its life, some of isolation valves are already closed by municipalities which is a real application in our country. However, according to this application of this study, it should be noticed that the closure of valves with an improper combination can cause greater leakages. Also, the usage of valves with an improper combination can cause consumers to take less water from the system. Therefore, the systems should be controlled for better operation at all times. The study of system with different operations data can help to develop a suitable operating policy.

CHAPTER 6

DISCUSSION OF THE RESULTS

6.1. Introduction

This chapter summarizes the main results of the study presented in Chapter 5, furthermore, implications of these results are discussed.

The main objective of this research was to develop a new methodology based on pressure driven approach how to decrease water leakages in real water distribution networks through valve control. This task can be summarized in two subtitles:

a) To use theoretical understanding of pressure dependent leakage and pressure dependent demand terms in water distribution networks in order to develop a hydraulic analysis program reducing water leakages with proper valve operations,

b) To implement and test this computer program on a large real network by knowing the need and importance of water loss reduction in local municipalities.

These subtitles were worked out through a developed modeling approach within the general framework of numerical model applications in water hydraulics. The extent to which these objectives have been achieved and the main results arising from the work presented in this thesis are discussed below.

6.2. Discussion of the Results

The important findings and realizations of this study can be outlined as follows:

1- LEAKSOL was applied to a sample network previously used by Sterling and Bargiela, 1984; Germanopoulos and Jowitt 1989; Jowitt and Xu 1990, Vairavamoorthy and Lumbers 1998 to see the efficiency of the model in CODE I. Leakage minimization was performed by accepting demands as fixed and compatible results were obtained. Leakage rates in the network when valves are fully open and when valves located at the specified locations are optimally set (i.e. leakage value controlled through optimization study) was figured. Results point out that leakage reduction during the peak demand period is relatively small whereas it is maximized during the night when consumer demands are lower and system pressures tends to be higher. The overall reduction in leakage is about 17 % which was a good indication of effectiveness of the code.

2- CODE II was executed for the same sample network used in literature by putting isolation valves instead of flow control valves and generating combinations for three valves. The analysis showed that the inclusion of optimally set flow control valves reduces leakage more than that of obtained from CODE II. But, it is obvious that application of the best fully open or closed combination of isolation valves satisfying the critical head requirement also reduces the leakage. The overall reduction in leakage corresponding to this method is about 3.5% with three valves selected. The violation of critical head requirement by taking consumption rate into consideration increases the reduction in leakage. This study shows that for the networks where it is not possible to place flow control valves, searching the best combination of isolation valve settings may be helpful for reduction of leakage up to a certain level.

3- LEAKSOL was executed for N8-3 pressure zone of Ankara Municipal Water Supply System. The main problem for N8-3 pressure zone application is the difficulty in describing the relationship between pressure and leakage. To find the relation between leakage and pressure and to determine the leakage coefficients in N8-3 pressure zone, an attempt was made through a field work in 2001 (Özkan, 2001). The study performed to obtain the leakage coefficients for whole zone showed that the leakages in this zone are relatively small depending on great N values. That is, small size leaks tend to grow under high pressures. However the correlation coefficient was indicated as very low and the results were presented in the form of a range for both coefficients ($N = 1.68 - 3.10$; $k = 0.89 - 7.16$).

Due to these reasons, the range of leakage coefficients determined for N8-3 pressure zone was not preffered to be used in this thesis study. It was concluded that a calibrated pressure-leakage relationship representing the whole zone should be obtained and used for future studies.

4- In this thesis study, the relationship between pressure and leakage was defined by using three different values of N; to be representation of the orifice equation ($N=0.5$); having no exact knowledge of relationship for N8-3 pressure zone ($N=1.0$) and to follow the studies in literature ($N=1.18$). The resulting leakage values and optimum flow control valve settings were compared for these N values. N8-3 pressure zone application showed that amount of leakage reduction increases for greater N values. However, finally obtained optimum valve settings are close to each other. This means that whatever the relation between pressure and leakage is, for same controlled conditions valves behave similar to each other with settings nearly compatible especially in a network like N8-3 pressure zone where pressure varies in a narrow range. Since pressure in this zone is relatively high during the whole day, valve settings exhibit an attitude with values very close to each other.

5- For N8-3 pressure zone application, minimum required pressure was accepted as 20m. Pressures that are lower than this value may cause a shortfall in supply and failure to supply the full demands. But, in the future, if maps corresponding to pressure zones include data related with number of consumers and storey number in buildings, it might be possible to identify minimum required pressures for the areas having different characteristics in the same pressure zone and more realistic results might be obtained from model applications.

6- The daily demand curve for whole N8-3 pressure zone corresponding to the date of 27.05.2001 was used in computer model application. The sharp decrease in the demand curve showed that the data taken from SCADA system and pump stations should have some errors, i.e., data acquisition system of the network is not healthy. The study performed in 2001 showed that the network should be checked for availability of functional valves to isolate a district and also attention is necessary because existing but old, unutilized valves have tendency to get sticked or remained closed. In order to eliminate the error coming from the data, new demand value for the time where sharp decrease was encountered, was obtained by averaging the values corresponding to one hour before and after.

7- The average water leakage for N8-3 pressure zone was accepted as 60 m^3/hr (Özkan, 2001). Due to the lack of a calibrated pressure-leakage relationship representing the whole zone, the leakage coefficient C_l corresponding to three N values were computed for the scenario presented in Chapter 5 by accepting the 60 m^3/hr as correct leakage from the zone.

8- The number of valves was restricted to eight valves. During the field study conducted in 2001, N8-3 pressure zone was separated into six sub-zones (Özkan, 2001). The valves were located at entrances of these sub-zones. Leakage minimization could be achieved in a noticeable amount. Better reduction in leakage may be obtained by changing numbers and locations of valves. Since difference among the ground elevations in N8-3 pressure zone, i.e. the gap between highest and lowest point of the network, is too much, approximately 95 m, it is very difficult to have an effective reduction in leakage. For a network having a relatively flat topology, the application of LEAKSOL can produce favorable variance in result. Also, study of zone by dividing into sub-zones can help to operators having much more control on the network with increased reduction in leakage.

9- For N8 pressure zone application, solutions were obtained for three cases. In the first case valves were accepted as fully open that is no control on the network in order to compare the system leakage with that of other cases. In the second case, $H_{critical}$ values, which control the optimization problem, for all demand nodes were accepted as equal to ground elevations. In this case, optimization program gave the minimum leakage by finding all the valves as fully closed. It can be concluded that if there is no restricting condition in a network, minimum leakage value can be obtained by closing all of the valves. In other words, when flow control valves are closed completely, system pressure and so leakage is minimized. But, this time the amount of water subtracted from network decreases compared to desired demand, i.e., consumers could not get enough water. For the third case where $H_{critical}$ values are defined different than zero, depending on the type of statement chosen which can be an operator defined certain value for certain nodes or a fixed value for all nodes as in this trial, a leakage value between that of obtained when valves are fully open and that of for closed valves, can be encountered.

10- CODE II was executed for N8-3 pressure zone by accepting the same eight valves used in optimization analysis as isolation valves. The code provided solution for 2^8=256 valve combinations. Minimum leakages were obtained for the case in which all valves were closed. Reduction in leakage for this case is 25 % for N=0.5, 28.4 % for N=1.0 and 30 % for N=1.18. If optimal valve settings obtained from CODE I are used, reduction in leakage is 5.1 % for N=0.5, 9 % for N=1.0 and 10 % for N=1.18. Leakage can be reduced by 1.9 % for N=0.5, 2.4 % for N=1.0 and 2.62 % for N=1.18 in case of valve combination satisfying the critical head requirement resulted from the execution of CODE II. It is obvious that for defined critical head requirements, optimization code gives the minimum leakage value with optimum flow control valve settings. But, if isolation valves used instead of flow control valves, again a decrease in leakage can be observed. Although, the leakage volumes are not completely minimized, a reduction throughout the day can be observed.

CHAPTER 7

CONCLUSIONS AND RECOMMENDATIONS

The thesis study can be concluded with following statements:

1- Leakage minimization through optimum flow control valve settings can be achieved for real networks based on the proposed methodology.

2- The reduction of leakage through the use of isolation valves may not be the best way of water loss minimization, but it can help to reduce pressure and so leakage. The advantage of such an approach can be the usage of existing valves in a network providing economical solutions. For the networks where it is not possible to place flow control valves, searching the best combination of isolation valve settings may be helpful for reduction of leakage up to a certain level.

3- Methodology regarding to best combination of isolation valves can give chance to the operator for selecting the most appropriate valve combinations considering the consumer's necessities. In some situations concerning to operation of a network, not only the amount of leakage but also the consumption/demand ratio can be accepted as criterias for determination of best combination of valve settings.

4- It is known that, 50 % of the total water used by consumers is lost as leakage at N8-3 pressure zone (Özkan, 2001). The application of methodology can reduce leakages up to 10 % for valves located at the entrances of sub-zones, depending on the defined pressure-leakage relationship. The high variation in topography can complicate the reduction of leakage. Control of any pressure zone through valve operations can be achieved initially by placing valves at connections to the main feeding line. Then, a better reduction can be provided by increasing the number of valves and dividing the zone into sub-zones to be studied separately. Determination of valve locations for sub-zones requires going to a step further.

5- The leakage reduction can be increased not only by determining the optimum valve settings but also by fixing the optimum locations of control valves. A future study can be arranged to address the problem of appropriate location of control valves in a water supply network with their optimum settings.

6- Similar executions can provide realistic results to develop suitable operating policies for distribution networks. It is evident that water distribution networks are designed to meet the future demands and the life of a network is assumed to be 30-35 years (Bank of Provinces, 1998). At the beginning of its life, some of isolation valves can be closed by municipalities. However, it should be noticed that the closure of valves with an improper combination can cause greater leakages. Also, the usage of valves with an improper combination can cause consumers to take less water from the system. Therefore, the systems should be controlled for better operation at all times. The study of system with different operations data can help to develop a suitable operating policy.

7- Demand driven analysis can be effective for hydraulic analysis of networks in case of design purposes. But, for operation oriented studies, head driven analysis can produce more meaningful results. It is the author's opinion that future versions of many commercial softwares working for hydraulic analysis of water distribution networks should take the pressure dependent demand terms into account in order to identify the consumption.

8- The effective usage of control valves requires the establishment of a remotely control system and also prediction of daily demand curves. Demand can vary significantly within each day, from day to day, from weekday to weekend and from season to season. The identification of future demand forms a basis for application of presented methodology.

9- In case of absence of remotely controlled flow control valves, it would be better to apply the methodology derived for isolation valves through their manual control. But any repetitious operation on isolation valves requires intensive field work. In that case, utilization of existing valves by keeping them closed all the times with suitable combinations regarding to consumption/demand ratio can constitute an alternative solution.

10- The future work for this study will mainly deal with the improvements of the computer program, LEAKSOL. A more user-friendly interface, enabling easy extraction of all network related information about zones, is aimed for future versions of the program.

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APPENDIX A

DEVELOPMENT OF INTERFACE

An interface was developed in MapInfo Professional in order to input, organize and export data to the core program written for CODE I and to display the results at the end of the execution. However, some of imperfections have to be debugged. Hereafter, main program can combine capabilities of MapInfo and MATLAB for more user friendly application of algorithm to real networks. This appendix contains the details of this interface and describes how to use it.

The interface is used as a graphical editor in order to extend the visual capabilities of main code. MapInfo Professional was selected to prepare such an interface for the program. MapInfo Professional is widely regarded as the easiest to use and most powerful PC-based mapping software that enables business analysts and GIS (Geographic Information System) professionals to easily visualize the relationships between data and geography (Daniel et. al., 2002). GIS is a computer system for capturing, storing, checking, integrating, manipulating, analyzing and displaying data related to positions on the Earth's surface.

One of the reasons for selecting MapInfo Professional as an interface is availability and capabilities of the programming editor, MapBasic. It is an advanced GIS application and development language which has been formed by adding GIS abilities into Basic language. It has got a wide function library. With MapBasic, users can alter the MapInfo Menus, can create and add new menus.

Another reason to use MapInfo Professional as the interface for the program is that it will give the opportunity to benefit from the capabilities of GIS software to manage and manipulate the network data. Also, by knowing that ASKİ Data Processing Center stores all attributes of Ankara water distribution network database in MapInfo environment, development of a code with MapInfo interface enables easy extraction of all network related information about zones and working in the same media. In the future, it may be possible to use LEAKSOL as a MapInfo tool in municipalities.

The main functions of interface for hydraulic analysis of any network in order to minimize leakage by using flow control valves are to input, organize and export data to MATLAB for analysis and to retrieve and display the results of the execution. A patch file with a name of LEAKSOL was prepared by using MapBasic to automate the functions mentioned above. The executable MapBasic file, LEAKSOL.mbx, can be run by double clicking the file or by selecting the file from "MapInfo Professional's Run Map Basic Command" which creates a menu named as "Hydraulic Analysis" including commands required for data acquisition, organization, exporting data and importing the results; "Send to MATLAB", "Get Data from MATLAB", "New Project", "Open Project" and "Exit"; and a button pad named as "LEAKSOL" containing buttons of "Add Pipe", "Add Node", "Add Pump", "Add Fixed Grade Node", "Add FCV" and "Default Pipe Properties" in MapInfo Professional window.

Main code will be developed also for analysis of pumped systems; thereafter the button placed for inclusion of pumps will be used. Figure A.1 shows MapInfo Professional Window after the run of LEAKSOL.mbx.

A.1. Data Input

New project is introduced by selecting "New Project" command from "Hydraulic Analysis" menu which results in a "New Project" dialog box (Figure A.2). From this dialog, the file name and the directory in which the new project will be placed are identified. Following this step, LEAKSOL forms five map layers as *file name*_node, *file name*_pipe and *file name*_valve, *file name*_result and *file name*_pump. Associated to these layers, a map window and a table with appropriate fields are constituted for each one. These tables have following fields:

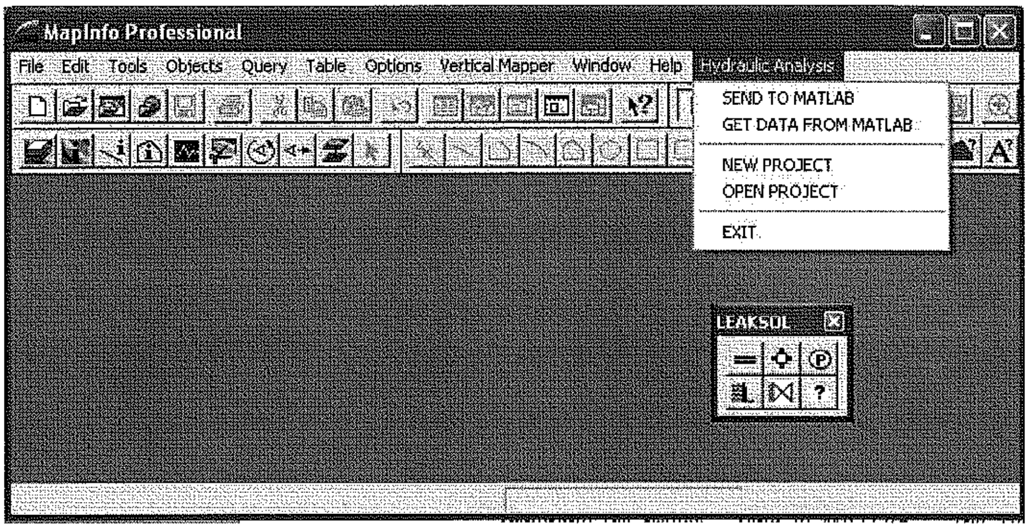

Figure A.1 MapInfo Professional window with LEAKSOL.mbx patch

Node table:

Label, NodeNo, Demand, Elevation, P_Req (required pressure), Node_or_FGN (fixed grade node), HGL, Pressure_Head, Consumption, Node_Leak.

Pipe table:

Label, PipeNo, FromNode, ToNode, Length, Diameter, C_{HW} (Hazen-Williams Coefficient), OpenOrClosed, Velocity, Dicharge, HeadLoss, Pipe_Leak.

Valve table:

Label, ValveNo, FromNode, ToNode, Setting.

Pump table:

Label, PipeNo, FromNode,ToNode, ShutOffHead, Design Head, Design Dischrg (Design Discharge), MaxOperHead (Maximum Operating Head), MaxOperDischrg (Maximum Operating Discharge), Pump_Head and Pump_Discharge.

Result table: Total Leakage, Total Consumption, Total Demand.

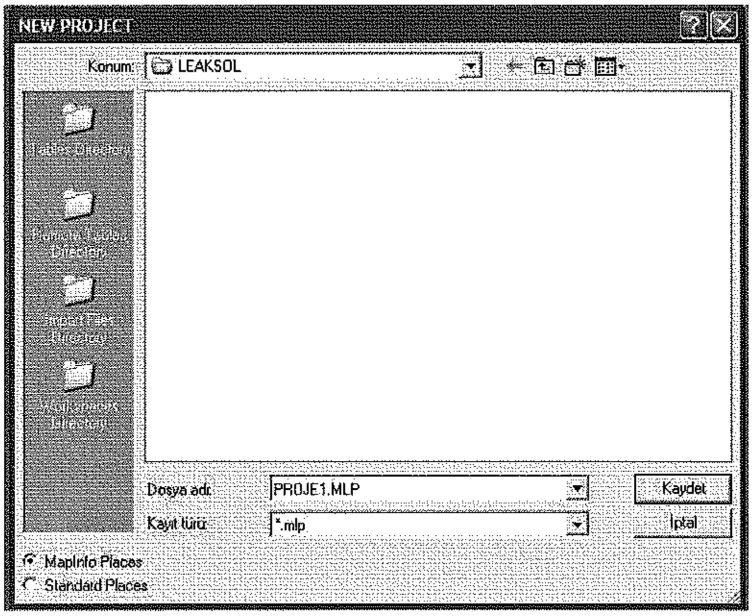

Figure A.2 New project dialog box

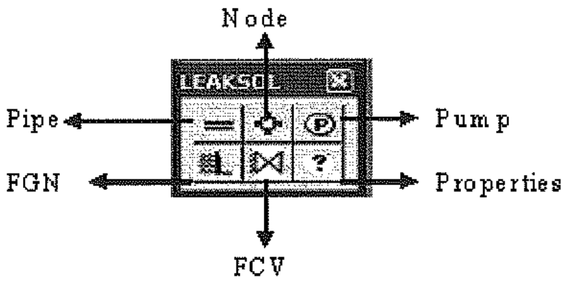

Figure A.3 LEAKSOL button pad

Network elements can be added to the map window by selecting the appropriate button from "LEAKSOL" button pad (Figure A.3).

Node and fixed grade nodes are added to the map window as point objects. Nodes are labeled as J_*node number* and fixed grade nodes as FGN_*fixed grade node number.* Pipes are link elements that connect nodes or fixed grade nodes and can be added as lines or polylines. They are labeled as P-*pipe number* and can be changed later by user. Pumps and flow control valves are inserted on pipes and are labeled as Pump_*pump number* and FCV_*flow control valve number.* Adding of an element to the map window results in an added row to the corresponding table for this element. Map window for the sample network given in Figure 4.1 with elements and labels is demonstrated in Figure A.4. The topology data consists of pipe numbers on which pumps and valves are located and from node to node information for all pipes. Network elements are numbered as follows: nodes from 1 to J, pipes from 1 to P, pumps from 1 to PMP, valves from 1 to FCV, fixed grade nodes from J to J+FGN, where J, P, PMP, FCV and FGN denotes for number of nodes, pipes, pumps, flow control valves and fixed grade nodes. Program constitutes the topology data by using these element numbers automatically.

Data input can be achieved by means of info tool by selecting a single element or by means of table browser window selecting a layer. Sample info tool and table browser window can be seen in Figure A.5a and A.5b. Demand, elevation, required pressure and critical head requirement data for nodes; HGL level and ground elevation for fixed grade nodes, length, diameter, Hazen-Williams coefficient data for pipes; leakage coefficients N and Cl to define the relationship between pressure and leakage for the study area; and demand factor, i.e., the ratio of demand at the analysis hour to the maximum demand during that day and type of the analysis should be defined as input for the network by the user before exporting data to MATLAB. Leakage coefficients, demand factor and type of analysis are defined by means of dialog given in Figure A.6. Analysis method can be the fixed demand analysis or pressure dependent demand analysis. Pipes are taken as open by default in the program. In order to close pipes, zero value in OpenOrClosed column should be replaced with one. Also, the user should be aware of data units used as follows; demand, consumption and discharge in m^3/s; elevation, length, required pressure, HGL level, critical head, head loss and diameter in *m*.

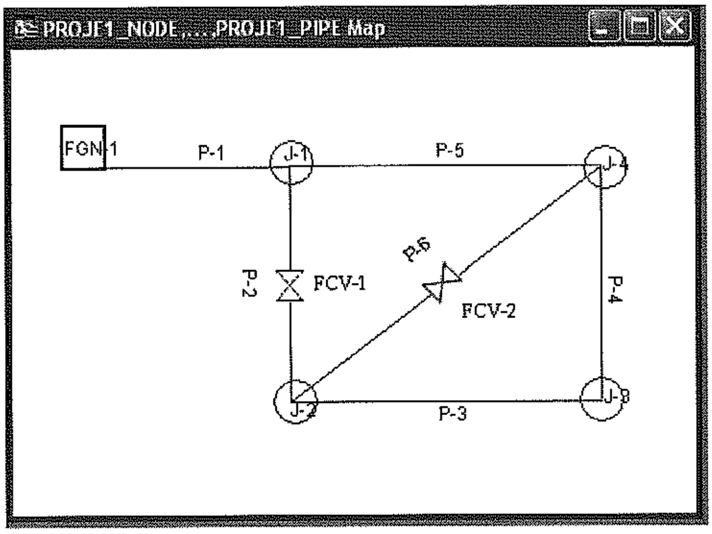

Figure A.4 Sample network demonstration on LEAKSOL window

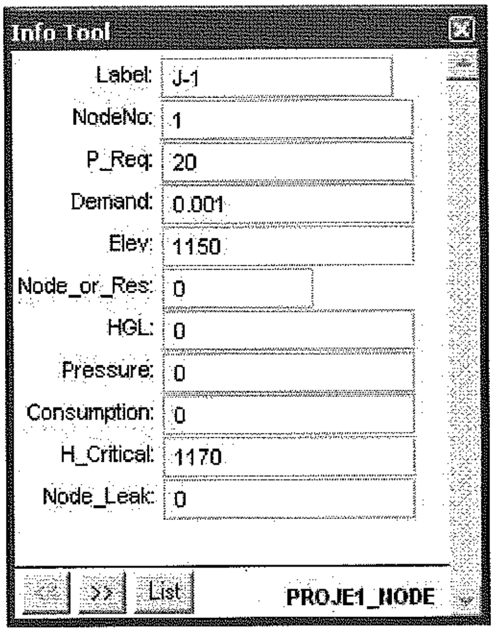

Figure A.5a Info tool for node

PROJE1_NODE Browser

Label	NodeNo	P_Req	Demand	Elev	Node_or_Res	HGL	Pressure	Consumptioi	H_Critical	Node_Leak
FGN-1	0	0	0	1,190	1	1,195	0	0	0	0
J-1	1	20	0.001	1,150	0	0	0	0	1,170	0
J-4	4	20	0.002	1,153	0	0	0	0	1,153	0
J-2	2	20	0.001	1,140	0	0	0	0	1,160	0
J-3	3	20	0.003	1,145	0	0	0	0	1,145	0

Figure A.5b. Data input from table browser window for nodes

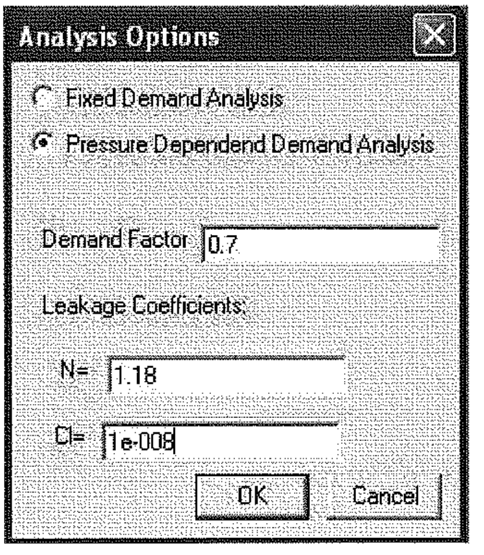

Figure A.6 Dialog box to define leakage coefficients, demand factor and type of the analysis

Data input part is completed after program performs checks for nodes, pipes and pumps. For nodes, the elements with zero elevation and/or required pressure are checked by the program. For the nodes with zero elevation, user is asked for whether the program will continue to run with zero elevation or not. A warning message seems for the nodes having zero required pressure. Checks for pipes having zero diameter and/or Hazen-Williams coefficient are performed by means of dialog boxes. For the pipes with zero length, user has alternative to automatically update the lengths convenient to the map scale.

A.2. Data Export to MATLAB

At the end of data input part for elements of a network, data defined from screen is sent to MATLAB for analysis by selecting "Sent to MATLAB" command from "Hydraulic Analysis" menu.

LEAKSOL exports the project data within five text files in folder C:\LEAKSOL. If the folder does not exist, program creates the folder in its first run. Text files are formed in the column order given in Chapter 5.

Program execution is started by locating the executable MATLAB file, matlab.exe in in folder C:\LEAKSOL. Location of matlab.exe is read from the file, *matlab.prg* which keeps the folder information. If this file does not exist in folder C:\LEAKSOL, user is again asked to locate *matlab.exe.* Locating this file, program runs MATLAB by using script files LEAKSOL.m, CODEI.m, isint.m, lprsm.m and argmin.m. Note that, before execution, script files should have been copied to the folder, C:\LEAKSOL. During the run, LEAKSOL gives information about the running file in a message window (Figure A.7) and waits MATLAB to constitute a text file, end.txt in order to import results to MapInfo Professional. Message windows for successful and unsuccessful operations are given in Figures A.8a and A.8b.

A.3. Data Import

At the end of the program execution, elapsed time for the operation, total leakage with optimal flow control valve settings, total consumption and total demand are displayed on the screen and outputs are saved and imported in the form of five text files. The output files include nodeout.txt showing node number, HGL, pressure head, consumption and nodal leakage value; pipeout.txt with pipe number, velocity, discharge, head loss and leakage at that pipe; valveout.txt involving pipe number on which the valve exist and settings obtained from the analysis and result.txt constituting total leakage due to application of optimum flow control valve settings, total consumption and demand. These results are imported and inserted to the appropriate columns of tables formed following the initiation of a new project from "Hydraulic Analysis" menu. The output text files are deleted by the program at the beginning of each analysis.

Figure A.7 Message windows giving information about the running file

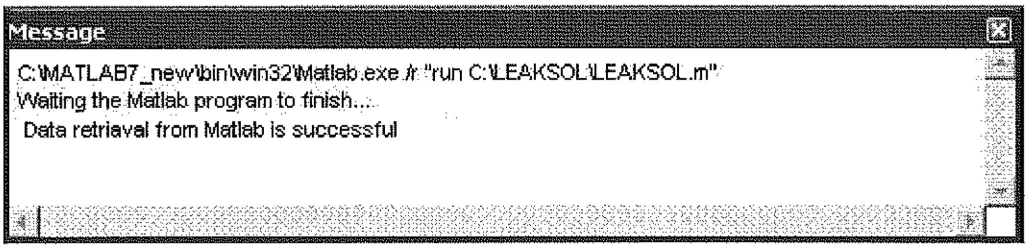

Figure A.8a MATLAB command window for successful operation

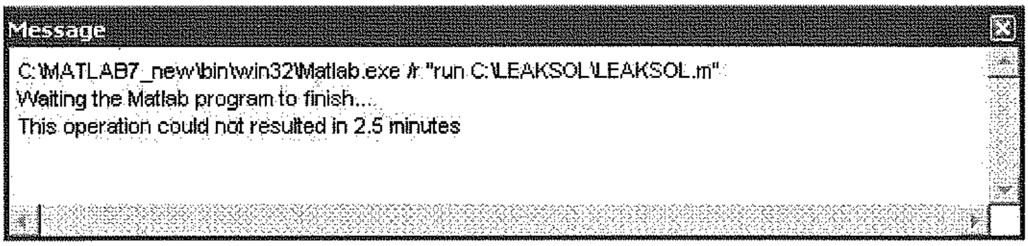

Figure A.8b MATLAB command window for unsuccessful operation

CURRICULUM VITAE

PERSONAL INFORMATION

Surname, Name: Özkan, Tülay

Nationality: Turkish (TC)

Date and Place of Birth: 26 January 1976, Ankara

Email: tozkan@gmail.com

EDUCATION

Degree	Institution	Year of Graduation
MS	METU Civil Engineering	2001
BS	METU Civil Engineering	1998
High School	Atatürk Lisesi, Ankara	1993

WORK EXPERIENCE

Year	Place	Enrollment
Dec 2005 – Present	General Directorate of State Hydraulic Works (DSI)	Civil Engineer
Sept 1998 - Dec 2005	METU Civil Engineering	Research Assistant
May - Sept 1998	Private	Civil Engineer

FOREIGN LANGUAGE

English

PUBLICATION

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