RELIABILITY BASED
WATER DISTRIBUTION NETWORK DESIGN

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RELIABILITY BASED
WATER DISTRIBUTION NETWORK DESIGN

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This is to certify that we have read this thesis and that in our opinion it is fully adequate, in scope and quality, as a thesis for the degree of Master of Science

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ABSTRACT

RELIABILITY BASED
WATER DISTRIBUTION NETWORK DESIGN

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The need of water and the limited sources, force the researchers to find the most economical and feasible solution in the design of a water distribution network. In this study, reliability and optimization of a water distribution network are taken into account together in the design stage of the network. The relationship between reliability of a water distribution network and its cost is examined during the design of a water distribution network. A methodology for deciding the reliability level of the selected design is proposed by examining the reliability-cost relationship. The design alternatives for the case study area are obtained by the aid of a commercially available software WADISO employing partial enumeration optimization technique. The reliability value for each of the design alternative is calculated according to Mısırdalı (2003)'s adaptation based on the methodology proposed by Bao and Mays (1990) by the aid of a hydraulic network solver program HapMam prepared by Nohutçu (2002). For purposes of illustration, the skeletonized form of Ankara Water Distribution Network subpressure zone (N8-1) is taken as the case study area. The methodology in this study, covering the relation between the reliability and the cost of a water distribution network and the proposed reliability level can be used in the design of new systems.

Keywords: Water Distribution Network Design, Reliability, Optimization, Partial Enumeration Method, WADISO, Ankara.

ÖZ

GÜVENİLİRLİK TEMELİ İLE
SU DAĞITIM ŞEBEKESİ TASARIMI

AKKAŞ, İzzet Saygın
Yüksek Lisans, İnşaat Mühendisliği Bölümü
Tez Yöneticisi : Doç. Dr. Nuri MERZİ

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Suya olan ihtiyaç ve kaynakların kısıtlı olması, araştırmacıları su dağıtım şebekesi tasarımında en ekonomik ve en uygun çözümü bulmaya zorlar. Bu çalışmada, su dağıtım şebekesi güvenilirliği ve optimizasyonu, tasarım aşamasında birlikte ele alınmaktadır. Su dağıtım şebekesi tasarımı aşamasında, güvenilirlik ve optimizasyon ilişkisi bu çalışmada incelenmektedir. Seçilmiş tasarımın güvenilirlik seviyesinin kararlaştırılması için güvenilirlik-maliyet ilişkisi incelenerek bir yöntem önerilmektedir. Kısmi sayım optimizasyon tekniğini uygulayan, ticari program WADISO yardımıyla çalışma bölgesi için çeşitli tasarım seçenekleri elde edilmektedir. Nohutçu (2002) tarafından hazırlanan hidrolik çözüm programı HapMam yardımıyla, Bao ve Mays (1991) tarafından önerilen yöntemin Mısırdalı (2003) uyarlamasına göre her değişik tasarımın güvenilirliği hesaplanmaktadır. Ankara Su Dağıtım Şebekesi alt basınç bölgesinin (N8-1) iskeletleştirilmiş hali gösterim amacıyla uygulama alanı olarak alınmaktadır. Bir su şebekesinin güvenilirliği ve maliyeti arasındaki ilişkiyi ve önerilen güvenilirlik seviyesini kapsayan bu çalışmadaki yöntem yeni sistemlerin tasarımında kullanılabilir.

Anahtar Kelimeler: Su Dağıtım Şebekesi, Güvenilirlik, Optimizasyon, Kısmi Sayım Metodu, WADISO, Ankara.

To My Family

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TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF SYMBOLS

C	:	Hazen-Williams Coefficient
D		Diameter of pipe
H	:	Pressure Head
H_{min}	:	The minimum required head
Hs	:	Supplied head at node
k	:	a coefficient (between 1 and 3)
K and n	:	Constants
L		Length of pipe
N	:	Numbers of node
P	:	Pressure
P_2	:	The known population
P_n	:	The population at the year n
Q	:	Discharge
Q	:	Discharge
Q_a	:	Available flow (consumption) at the node
Q_{max}	:	Maximum Loading
Q_{night}	:	Night Loading
Q_{peak}	:	Peak Loading
Q_r	:	The minimum required demand at node
R_H	:	The reliability of the network based on head approach
$R_n ave$	:	Average nodal reliability
$R_n ave_k$	:	Average nodal reliability of node k^{th}
R_{ni}	:	Nodal reliability of ith simulation
R_Q	:	The reliability of the network based on demand approach
t_1 and t_2	:	Years of the known populations
t_n	:	The year n
W_k	:	Nodal weight of node k^{th}

CHAPTER 1

INTRODUCTION

A water distribution network is designed to supply water to the consumers in the desired quality and quantity by its elements such as pipes, reservoirs, pumps, tanks, valves etc. An entirely adequate water distribution network should be able to provide required amount of potable water at required pressures throughout its economical life. Customers in developed countries wait for service according to this definition whereas customers in developing countries can accept water service on part time basis (Walski, 2000). As an example, it is stated by Chandapillai (1991) that the level of water service may be as low as two-three hours a day in India due to capital and operating costs.

Both quality and the quantity of the water are very important. These characteristics of water depend on several factors including design, construction and operational stages of the network. The construction of a water distribution network generally suggests large capital cost as well as operation, maintenance and repair costs. However, most of the water distribution systems are designed based on traditional trial and error procedure aiming a low cost answer to almost uncertain demands. Optimization techniques are avoided during the design process generally due to unfriendly software (Walski, 2001). Furthermore, at most of the cases, the interaction of different components such as reservoir of the dam, main transmission line, treatment plant, pumps, pipes of different groups of the network, storage tank etc., is simply ignored. That's why, it is difficult to declare that the designed system depicts the minimum cost answer and the resulting reliability is hundred percent.

On the other hand, the evaluation of the complete reliability analysis of a network is truly complex; because reliability depends on various parameters such as availability of the water at the source, rate of power outage, time to failure and time to repair of the supply pumps, roughness characteristics of the original network pipes, the uncertainty of the aging of pipes, uncertainty of the pump performance curves, opening positions of the isolating valves, uncertainty of the design discharge and nodal demands.

Mays (1989) concedes that no universally acceptable definition of the reliability of a water distribution network is available. However, reliability is usually defined as the probability of that system performs within specified limits for a given period of time (Gupta and Bhave, 1994); on the other hand Goulter (1985) describes reliability as the ability of a water distribution system to meet the demands that are placed on it where such demands are specified in terms of the flows to be supplied and the range of pressures at which those flows must be provided.

Bao and Mays (1990) and Mısırdalı (2003) describe ways of determining the reliability of selected water distribution networks. In those studies the hydraulic failure of the water distribution network was taken into consideration. Yıldız (2003) computes the reliability of selected networks using the definitions of Gupta and Bhave (1994) emphasizing importance of valve topology. In that study, the mechanical failure of the water distribution network was examined. Goulter and Coals (1986) and Akdoğan (2005) describe ways of increasing the reliability of a water distribution network examining reliability of individual nodes.

Walski (2001) mentions that, the range of combinations in which failure can occur in water distribution system, establishes the major source of the many theoretical and practical difficulties, which have been encountered in building computationally tractable measures of reliability that can be used in the practical design and operation of water distribution systems.

The design stage of a water distribution network based on optimization of reliability is the main scope and also the title of this study. This study aims to obtain a relation between the cost of a water distribution network and its reliability. This relation between the reliability and its cost will permit the engineer to design a water distribution network at the desired reliability level.

The reason for obtaining a relation between cost of the network and reliability is to examine the possibility of designing a network for a lower price at the cost of some reliability in terms of peak hour loading. This price difference can be saved if following conditions are fulfilled:

(1) The maintenance services should have been carried out correctly by water authority during the economical life of the network; as a result of this activity, aging of the pipe will be minimized through flushing works and original status of the isolation valves will be kept as required; both works will keep in line the original capacity of the network pipes that is no emphasized reliability loss will occur during the economical life of the network.
(2) Each building should have its own storage tank. Anyhow in Greater Cities, almost all the buildings have storage tanks; storage tanks of the buildings, which may suffer during peak hours, may get filled during night.
(3) Water distribution network design should be realized basically based on peak hour loadings since in Greater Cities in this country, fighting occurs through the use of fire trucks with high pumping capacities; since

In this study, the following cases were considered in obtaining the relationship between reliability of a water distribution network and its cost.

- Changing the Minimum Required Pressures (H_{min})
- The effect of Standard Deviation of Hazen-Williams roughness coefficient (C) on the change of reliability
- Changing Pipe Material
- Different Valve Status (Open/Closed)
- Pipe Aging Effect

A very efficient optimization software for engineering applications (WADISO) employs the partial enumeration technique. WADISO (Water Distribution Simulation and Optimization) was produced using the original works of Gessler (1985) and Walski (1985). This software basically designs optimum (capital cost of pipes) water distribution networks satisfying a given required pressure throughout the network.

Besides WADISO, in order to find whether the design alternatives satisfy the hydraulic conditions or to find the order of satisfaction, the reliabilities of the design alternatives are calculated. The hydraulic reliabilities of the design alternatives are calculated by the aid of HapMam (Hydraulic Analysis Program with Mapinfo and Matlab), which was prepared by Nohutçu (2002). Some minor modifications to HapMam were realized by Mısırdalı (2003) and also in this study. HapMam uses the methodology proposed by Bao and Mays (1990) considering Hazen-Williams roughness coefficient of the pipes and the nodal demands as uncertainty parameters. Moreover, HapMam could utilize the pressure dependent theory during the hydraulic analysis in addition to demand-driven theory.

The optimization techniques are simply presented and the employed technique (the partial enumeration technique) is demonstrated in detail in Chapter 2. WADISO is also presented in this chapter. Chapter 3 gives a literature review about the reliability concept and then presents the methodology of reliability computation including HapMam. The case study part is Chapter 4 in which N8-1 sub-pressure zone of Ankara Water Distribution Network is examined. Finally, conclusion and recommendations about this thesis study and some suggestions about further studies were performed in Chapter 5.

CHAPTER 2

OPTIMIZATION OF WATER DISTRIBUTION NETWORKS

Concerning with water distribution network problems, optimization is the process of obtaining the best or optimum solution. Basic examples of optimization problems referring to water distribution are sizing of the pipes and operation of the pumps. Optimization techniques are also used for rehabilitation works in addition to the design and operation studies. In this study, only optimization of the pipes sizes for the lowest cost was worked out.

2.1. THE OPTIMIZATION TECHNIQUES

There are many optimization techniques and each of them has its own algorithm. Traditionally, engineers make use of their experiences in the design of water distribution systems. Engineers perform several design alternatives provided that each alternative satisfies the hydraulic criteria. The design alternative with the minimum cost, comes out to be the optimum design. This technique is nothing but trial and error method. Experiences of the engineers form several rules-of-thumb for this technique and some of these rules are given by Walski (1985).

In this part, the names and definitions of the optimization techniques are shown and the partial enumeration technique is described in detail.

- **Linear Programming Technique:** This technique is used to simplify the solution of nonlinear problem by solving sets of linearized equations. Several researches were performed by this method by Alperovits and Shamir (1977), Goulter and Morgan (1985), Goulter and Coals (1986) and Fujiwara and Khang (1990).

- **Nonlinear Programming Technique:** The partial derivatives of the objective function with respect to decision variables are used in this technique. In the optimization of water distribution network, pipe sizes are assumed to be continues variable. However, this method sometimes gives a local optimum solution.
- **Genetic Algorithm Technique:** This technique is based on the Darwinian Theory, Natural Selection. The combination of alternative solutions of the problem starts with coding the variables. The solution sets are generated by birth, dead, cross-overing, mutation and eliminating of the existing solutions. The determination of the optimum solution is performed by the comparison of the total cost which includes both the cost of the solution and the related penalty cost. The algorithm continues with the survival of the fittest solution until the optimum solution is reached.
- **Enumeration Technique:** The enumeration technique evaluates all the alternative solutions to find the optimum one in the solution set. The main difficulty is the period of time to perform all the trials especially in large systems. The flowchart of the enumeration technique, which is used in the pipe size optimization, is shown on Figure 2.1. To minimize the computation time of enumeration technique, several criteria were developed by Gessler (1985) and this technique is named as partial enumeration technique.

2.1.1. The Partial Enumeration Algorithm

As stated before, the computation time is the most important problem of an enumeration algorithm. The necessary time for the evaluation of all alternative pipe size combinations may require more than the desired and defined amount of time. Even, this problem may cause the researchers and/or the designers to change the optimization algorithm in their studies. In order to minimize the computation time of this algorithm, grouping pipes, test on size range, cost test, size test are performed in the partial enumeration algorithm.

- **Grouping pipes:** In general, pipe diameters do not change from one street to its neighboring streets on a district unless an extra water demand occurs. In other words, the pipe sizes remain constant between two points of an area provided that the area has homogeneous characteristics among the related demand nodes. By this way, all the pipes having the same pipe sizes could be assumed as one link connecting the start and end nodes of the related pipes. As a result, the number of combination of candidate pipe sizes is minimized by forming pipe groups in the network.

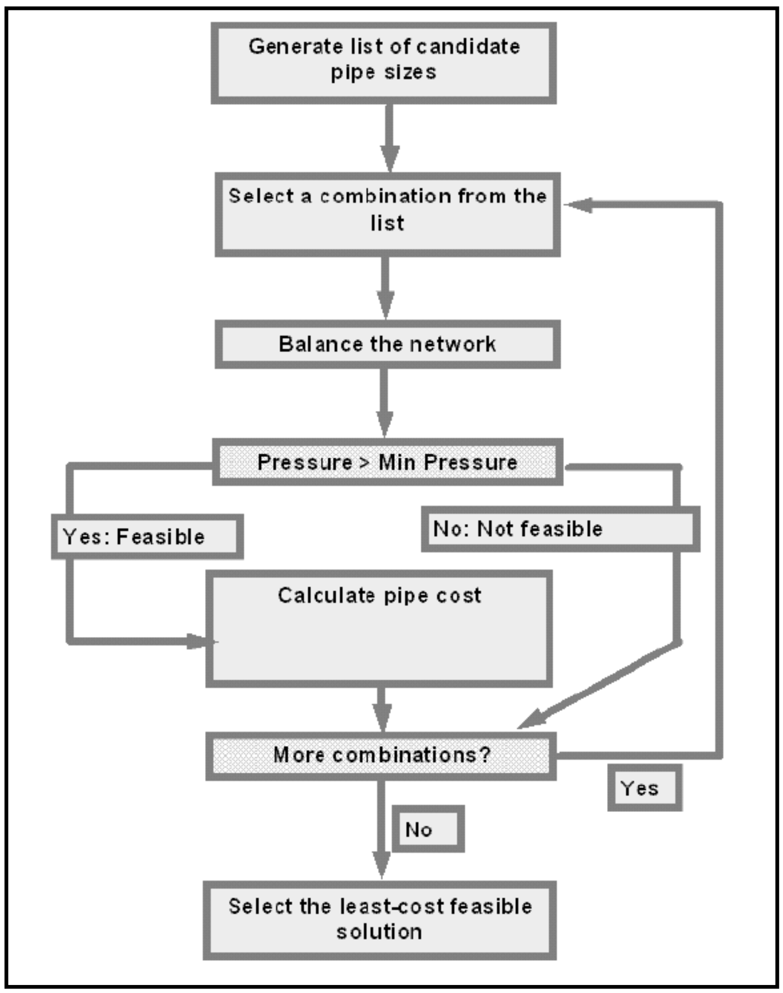

Figure 2.1: Flowchart of the enumeration technique algorithm (WADISO 5 User's Guide)

- **Test on size range:** After grouping of pipes, the candidate pipe sizes should be given to each of these pipe groups. However, the number of alternative pipe size combinations, i.e. trials depend on the number of candidate pipe sizes since the number of trials is nothing but the production of the candidate pipe sizes of each group. As an example, if a network has 5 pipe groups and 4 candidate pipe sizes for each of these groups, the total number of the trials will be (4^5 =4x4x4x4x4) 1024. Even the number of candidate pipe sizes of only one group could be decreased to 3, the number of trials will decrease to (4x4x4x4x3) 768 i.e. %25 less of previous value.

  In order to decrease the number of candidate pipe sizes, the partial enumeration algorithm performs test on size range by controlling the minimum pipe sizes of each group whether they satisfy the pressure requirements. The minimum pipe size of one group is taken together with the maximum pipe sizes of all other groups and this combination is tested. If this combination does not pass the test, it is understood that all the combinations including the minimum pipe size of that group do not satisfy the hydraulic criteria and there is no need to test all of these combinations.

  As a result, the minimum pipe size is removed from the candidate pipe sizes of that group. The test on size range is repeated for the minimum pipe sizes of all pipe groups and then followed by cost test.

- **Cost test:** The cost test also helps in decreasing the number of trials i.e. alternative solutions in the design of network. In the cost test, the cost of any feasible solution, which satisfies the hydraulic criteria, is used as a reference to other alternative solutions. Gessler (1985) stated that there is no need to test any other size combination, which is more expensive than the functional solution. This results in a considerable decrease in the number of alternative solutions.

- **Size test:** Gessler (1985) also stated that there is no need to test any other combination with all pipes equal or smaller than the pipes of a combination, which does not satisfy the pressure requirements.

After these tests, the number of combinations to be tested for pressure requirements is considerably decreased. The order of the mentioned tests could be easily seen on the flowchart of the partial enumeration technique on Figure 2.2. As can be seen on the flowchart after passing the tests, the combination which could not meet the pressure requirements, is entered in as a non-functional combination. Otherwise; the combination is another feasible solution (the new best solution) with the low cost among the tested combinations. The optimization process continues until all the alternatives are tested. As a result, the best solution becomes the global optimum solution.

2.1.2. The employed optimization program (WADISO)

WADISO (Water Distribution Simulation and Optimization) is the software used in this study, which was developed in 1980's. Walski T., Gessler J. and Sjostrom are the main members of the group that produces the first edition of WADISO working on DOS environment. By the improvements in computer technology, the version of WADISO, which has Graphical User Interface and working with databases and Geographical Information Systems was developed by a South African company GLS Software. In this study, the used version of WADISO is WADISO 5 and it was also developed by GLS Software in 2005. The studies on WADISO were being performed by this company since 1996 and it is seen in this study that the problems with the computation time of the program due to number of pipe size combinations (trials) is almost solved in the latest version (used version) of WADISO. The partial enumeration algorithm, which was developed by Gessler (1985), was used in all of the versions of WADISO.

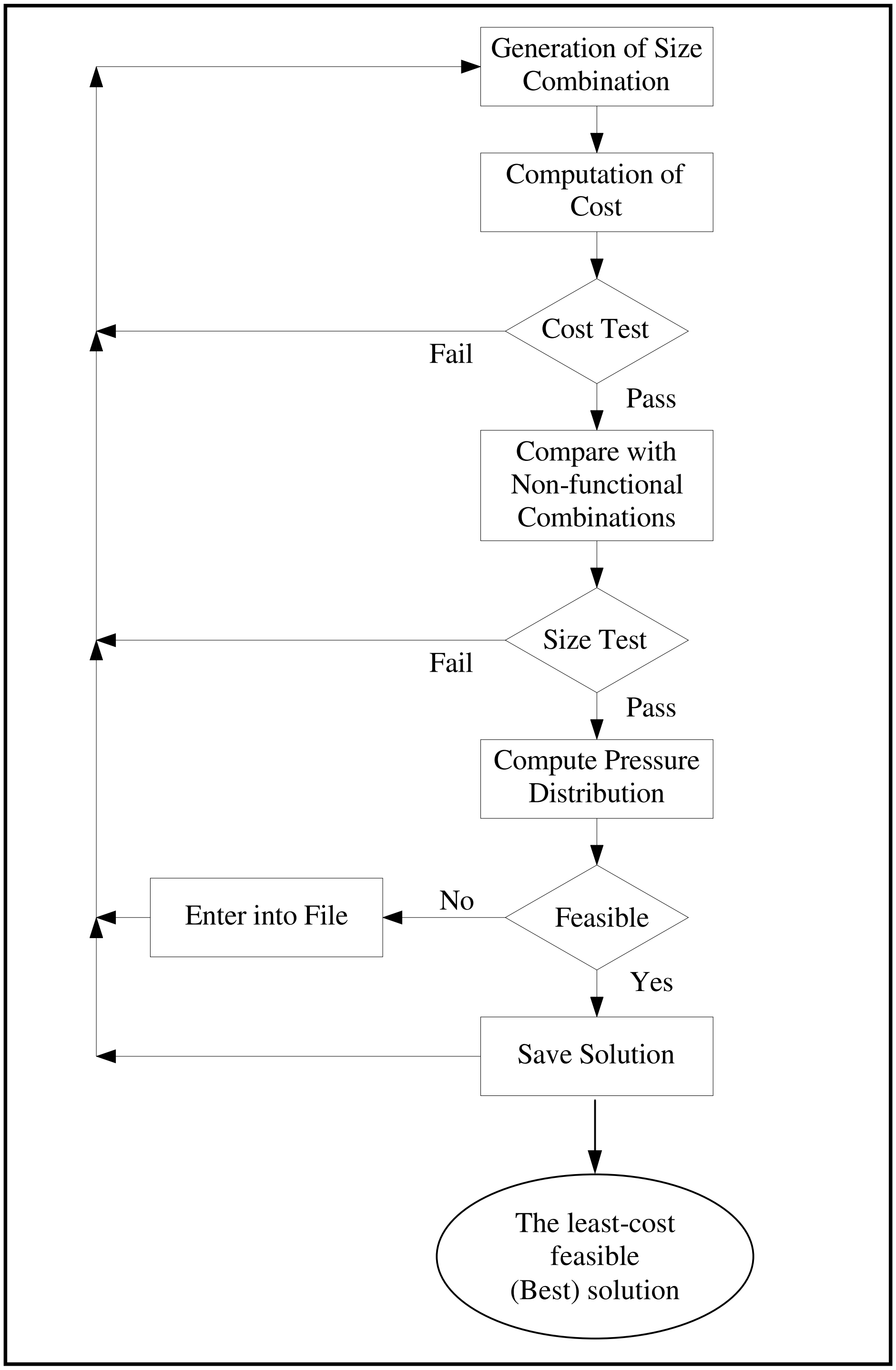

Figure 2.2: Flowchart of the partial enumeration technique algorithm (Keleş, 2005)

2.1.2.1 The data input process in WADISO

Layout of the Network, Loading and Pressure Scenarios: The first step in data input is forming the layout of the network. This could be easily performed by the aid of Graphical User Interface and database functions of WADISO such as import, export etc.. The nodal demands and the required pressures are assigned to the nodes depending on the scenarios given by the designer. In addition, other necessary input parameters of nodes and links (coordinates, elevations, lengths, roughness coefficients etc...) and tank, reservoir and pump characteristics are given in this stage. However, these parameters could also be modified during the design and analysis stage.

Pipe Selection for optimization: After forming the layout of the network and giving the basic data related with the network, the pipes that will be optimized should be defined. In WADISO, the user does not have to select all the pipes of the network for optimization.

Pipe Grouping in WADISO: The pipe grouping is performed among the pipes that will be optimized. In this stage, every pipe that will be optimized should belong to a group and the pipes in the same group will have the same pipe size. In WADISO, the number of pipe groups is limited to 15. However, it is not recommended to form unnecessary pipe groups because of computation time. To decrease the computation time, the number of the pipe groups should be decreased as much as possible. For example, the main transmission lines, the secondary transmission lines, main tributary lines, parallel pipes could be grouped separately. It should be noted that, the characteristics of the network plays an important role in this stage similar as in all data input process. The characteristics such as, the size of the network, the type of the area (residential, agricultural, industrial etc...), the growth rate of the area etc... should be examined with care by the designers in the determination of the pipe groups.

Candidate Pipe Sizes: 10 different pipe sizes could be given to each pipe group in WADISO. As stated before, the minimum pipe sizes are tested by test on size range. Additionally, WADISO performs size test for decreasing the number of pipe size combinations. The range of the pipe size alternatives i.e. the minimum and the maximum pipe sizes, for each pipe group could be achieved after a few runs in WADISO.

It should be noted that, the designer has to make use of his/her engineering judgment and experience in determining candidate pipe sizes similarly in all engineering works. In WADISO, a pipe group could be discarded at no cost by selection of size “0” or “E” which could be assigned to that pipe group by the designer.

In addition, cleaning and relining of existing pipes could be also performed by WADISO in this stage by assigning “C” value as a pipe size to the related pipe group if there exists a parallel pipe (not to be sized) to each pipe in that group. For additional information the User's Guide for Version 5 of WADISO could be examined.

Price Functions: The aim of optimization whichever the technique is used, is to find the least cost design, which satisfies the hydraulic criteria. After all data input stages, the decision stage for the determination of pipe sizes is performed by using the price functions. All the calculations are performed under the given cost data. In WADISO, 8 different cost functions could be assigned. The pipes, which will be optimized, should assign to one cost function. The pipes in the same group may have different cost functions. In addition to the cost functions of pipe, the cost of cleaning and lining of pipes, tank and pump cost functions could also be entered to the program. In this study, only pipe cost function is entered to the program, since only optimization of pipe sizes is performed.

CHAPTER 3

RELIABILITY OF WATER DISTRIBUTION NETWORKS

3.1 THE DEFINITION AND THE MEASURES OF RELIABILITY

Reliability is generally defined as the probability that a system will perform its functions within specified limits for a given period of time. However, it is not simple to put forward a universally accepted definition for reliability of a water distribution network although several definitions have been made by researchers in the past.

It is stated by Goulter (1995) that the reliability of a water distribution network is related with the ability of the network to provide an adequate level of service to consumers, under both normal and abnormal conditions. This service can be explicitly explained by the study (Mays et al. 2000) such that the water distribution network should meet the demands such as flows to be supplied (total volume and flow rate) and the range of pressures at which those flows must be provided.

The abnormal conditions in the water distribution network may occur as a result from any failure in the system and result in the decrease of the reliability. In the previous studies, the failure of a system is mainly divided into two as mechanical and hydraulic failure of the network although they are not independent of each other.

In the summary of major simulation and analytical approaches to assessment of reliability in water distribution networks prepared by Mays et. al., 2000, the following researchers and researches could be given as mechanical and hydraulic failure analysis:

- Kettler and Goulter (1983), Goulter and Coals (1986), Su et.al. (1987), Wagner et al. (1988b), Jowitt and Xu (1993), Gupta and Bhave (1994) were interested in mechanical failure of the components as pipes, pumps etc. in their studies.
- Hydraulic failure may cause from the uncertainties in nodal and/or total system demands, required nodal heads or decrease in the carrying capacities of the pipes etc.. Lansey and Mays (1989), Bao and Mays (1990), Xu and Goulter (1999) were interested in water distribution reliabilities by considering hydraulic failure of the network.

The measure of reliability is another complex subject since there are a lot of reliability measures in this research area and each of these measures can only give reliability results from their own aspects. In other words, there is not a unique reliability measure that could be generalized and could be utilized in the reliability calculations. As an example, in the study of Wagner et al. (1988b), link, system, node and event related reliability measures were presented as follows.

- **Link (Pipe, Pump) related reliability measures:** Number of pipe failures, Percentage of failure time for each pipe, Percentage of time of failure time for each pump, Number of pump failures, Total duration of failure time for each pump.
- **System related reliability measures:** Total system consumption**,** Total number of breaks**,** Maximum number of breaks per event
- **Node related reliability measures:** Total demand during the simulation period**,** Shortfall**,** Average head**,** Number of reduced service events**,** Duration of reduced service events**,** Number of failure events**,** Duration of failure events
- **Event related reliability measures:** Type of event, Total number of events in the simulation period and system status during each event**,** interfailure time and repair duration

3.2 THE METHODOLOGY FOR MEASURING RELIABILITY

In this study, costs of the alternative designs were taken from WADISO. In order to examine the relationship between the initial cost of a network and its reliability, the reliability of each alternative design was measured by the methodology explained in this part.

The probability of non-failure of a network could also be given as a simple definition of the reliability of that network. As stated before, the mechanical and hydraulic failures are accepted as the main failure types of a water distribution network. Mostly hydraulic failure of the network is the scope of this study.

The uncertainties in the water demands and the uncertainties in the roughness coefficients of the pipes are taken as the reasons for hydraulic failure of the network. Mechanical failure of the network, which may arise from the pipe breakages, was also simulated by closing several valves in the network in this study.

In this study, reliability value for each alternative design realized by WADISO, was calculated based on the peak hour demand values. Note that alternative designs were obtained for the same peak hour demand values for different minimum required pressure values. The employed algorithm for measuring reliability is node related and the system reliability is calculated by using the nodal reliabilities.

3.2.1 The pressure dependent theory (The head-driven theory)

In the analysis of water distribution networks, two different theories are used such as demand-driven and head-driven theories. According to the demand driven theory, which was widely used in the past studies, the required demand at any node could be met if and only if pressure (head) at that node is equal or greater than the minimum required pressure (head) at that node, which is generally an unrealistic case. Moreover, during the analyses, which are performed by several softwares using demand-driven theory, it may appear that, the demands are always met at the nodes whereas the pressures may be not enough or even negative pressures may occur at some nodes. This is because; these softwares only consider the nodal demands without examining simultaneously the nodal pressures. This result points out that there is a problem in the network and most of these softwares give error messages.

However, it is clear that there could be temporal insufficiencies in the network frequently, because of several reasons such as pipe breakages, pump failures, increases in nodal demands etc.

In these situations, some of the nodes in the network would not able to deliver the required amount of water at the required pressures, and those nodes would be partially satisfied. The demand-driven theory could not give any assistance to the designer and/or operator who want to get the conditions of this partial satisfied network. The main advantage of the head-driven theory appears here such that it is possible to achieve a relation between the partially satisfied demand and the pressure by the aid of head-driven theory.

The pressure dependent theory is the base of head-driven theory and by using pressure dependent theory; the analysis of the partial satisfied network can be performed.

In this research area various studies were carried out using the pressure dependent theory. Node flow analysis, which is an iterative method by categorizing nodes, for the calculation of partial satisfied flows at insufficient pressures, was formed by Bhave (1981, 1991). Unfortunately, by this method a direct relation between partial flow and head could not be obtained. The relation between partial flow and head, is put forward for the first time by Germanopulos (1985) with three constants. However, two of those three constants were uncertain and could not be described in that study. Wagner et.al (1988b) and Reddy and Elango (1989) also studied and proposed methods about pressure dependent theory.

It is stated by Nohutçu (2002) that the methodology which was built by Chandapillai (1991) was the most efficient and satisfactory method that gives the fundamental equation between partial flow and the related head. The model was later developed by Tanyimboh et. al (2001). Nohutçu (2002) also performed some modifications to this model in his study and named the final model as Modified Chandapillai Model.

3.2.2 Modified Chandapillai Model

In this model the consumed flow is assumed to be from the network to an overhead tank. The related formula is shown in equation (3.1).

$$H = H_{\min} + KQ^{n} \tag{3.1}$$

where,

H (m)	:	head
H_{min} (m)	:	the minimum required head
K and n	:	constants
Q (m^3/s)	:	flow into overhead tank

Equation (3.1) is assigned to a node by replacing the flow rate (Q) with the nodal consumption or nodal flow (c). The consumption at the node could be obtained by equation (3.2).

$$c = \left(\frac{H - H_{\min}}{K}\right)^{1/n} \tag{3.2}$$

It is derived that when the H = H^{req} and c=q^{req} and this relation could be shown in equation (3.3).

$$q^{req} = \left(\frac{H_{req} - H_{\min}}{K}\right)^{1/n} \tag{3.3}$$

Then, K (the constant) is left alone as shown on equation (3.4) and it is subsituted in equation (3.2) in order to find the relation between nodal flow(consumption) and the nodal pressure. This relation is formulated in equation (3.5).

$$\frac{1}{K^{1/n}} = \frac{q^{req}}{(H_{req} - H_{\min})^{1/n}} \tag{3.4}$$

$$c = q^{req} \left(\frac{H - H_{\min}}{H_{req} - H_{\min}}\right)^{1/n} \tag{3.5}$$

Chandapillai (1991) proposes these sets of equation for his model. Nohutçu (2002) performed a modification to equation (3.5) by inserting (P) pressures instead of (H) heads and by setting the minimum pressures to zero. As a result, Modified Chandapillai Model is denoted by equation (3.6).

$$c = q^{req} . \left(\frac{P}{P^{req}}\right)^{1/n}, \quad \text{where} \qquad 0 \le P \le P^{req} \tag{3.6}$$

3.2.3 HapMam

HapMam (Hydraulic Analysis Program with Mapinfo and Matlab) is a hydraulic network simulator prepared by Nohutçu (2002) in order to find the supplied heads and the consumptions at the nodes of a network. Mapinfo Professional software is mainly used for GIS integration and Graphical Interface of HapMam. Matlab is used as the programming language of HapMam. HapMam takes the input data from Mapinfo and performs the hydraulic analysis in Matlab. In addition, the input data could also be given to HapMam as text files without using Mapinfo. In this study, all the input data were given as text files and Mapinfo is not used in HapMam.

HapMam uses linear theory for fixed demand analysis and the pressure dependent theory (head-driven theory) was included into linear theory for partial flow analysis. Additionally, HapMam allows the user to perform the partial flow analysis according to Germanopoulos (1985) , Modified Germanopoulos (2002), Reddy and Elango's (1989) and Modified Chandapillai (2002) Models. In this study, as stated before, the most efficient model (Modified Chandapillai Model), which was proposed by Nohutçu (2002), and also used by Mısırdalı (2003), is selected for partial flow analysis.

Besides the advantage of performing partial flow analysis, HapMam utilizes the methodology based on the study of Bao and Mays (1990). Bao and Mays (1990) considered the probability of head being more than the minimum required head and generated several scenarios from Monte Carlo Simulation.

In that study, reliability was computed by taking demand and the roughness coefficient as the random variables in Monte Carlo Simulation. In this study, demand and the roughness coefficients were assumed to be as uncertainty parameters in a water distribution network. The demands in the water distribution network may change when the nodal consumptions increase, extra demand (new users) occurs, pipe breakages exists.

The roughness coefficient of the pipes, which is a lumped parameter, is used as a display of the interior conditions of pipes. The carrying capacities of the pipes increase with increasing roughness coefficients. Moreover, the roughness coefficients could also refer to the conditions of the network. As an example, ages of the pipe, characteristics of the water, velocity of the flow, homogeneous distribution of flow to the nodes in the network, valve status affect the roughness coefficient. Hazen-Williams roughness coefficient is taken into account in this study since Hazen-Williams flow equation is chosen as the flow equation in the analysis. The values from 20 to 150 could be assigned as the Hazen-Williams roughness coefficient (C) of the related pipes according to their ages, materials and valve status.

3.3 THE MODEL USED IN RELIABILITY COMPUTATION

It should be noted that the reliability of a water distribution network is affected adversely by these uncertainties. The model used in this study for calculation of reliability includes 3 parts. The first step is the random number generation, the second step is the hydraulic simulation, the last and the third step is the reliability computations (Figure 3.1).

3.3.1 Random number generation

It was stated before that, the random variables in this study were demand (Q_{design}) and the Hazen-Williams Coefficients of the pipes (C). Monte Carlo Simulation, the first step for reliability process, is performed for random number generation.

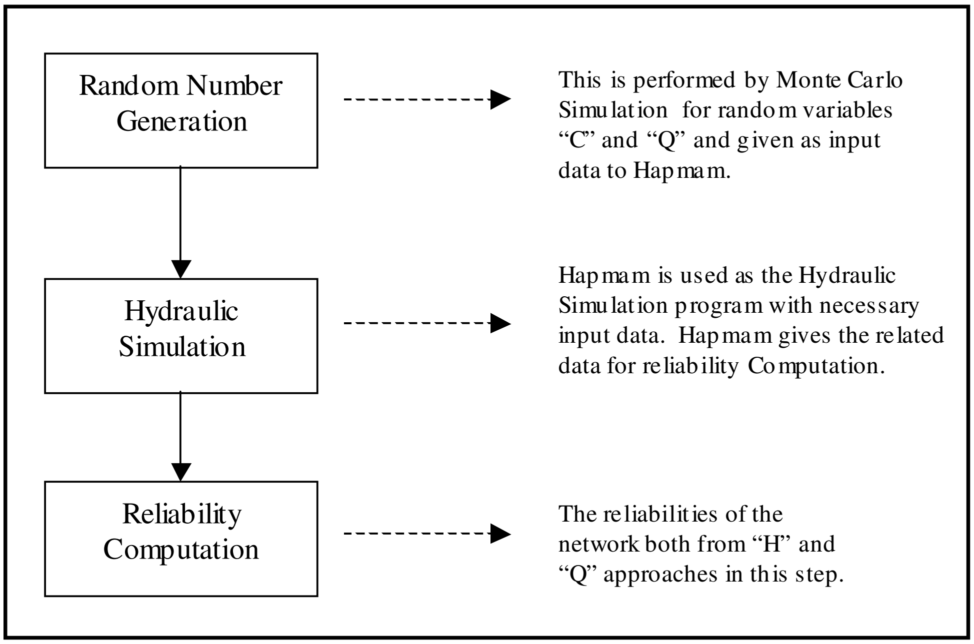

Figure 3.1: The simple flowchart of the model used in reliability process (Mısırdalı, 2003)

The random number generation for Q_{design} and C is performed by using random number generation tool in Microsoft Excel 2000 software. Normal distribution is used in the generation of data sets and 500 different values are generated for Q_{design} and C variables. The mean and the standard deviation of these variables, which could also be seen in detail in the case study part of this study, are shown in Table 3.1.

In this study, 500 different simulations are performed with these data sets (500 Q_{design} values and 500 C values) in each case. Hazen-Williams roughness coefficients (C) are assumed to be same for all pipes in the network. In other words it is assumed that every pipe in the network has same characteristics and a different (C) value is assigned to all pipes in each simulation. The generated Q_{design} values are also assigned to the nodes using their nodal weights in each simulation.

Table 3.1: The data sets generated by Monte Carlo Simulation

The random variable	The reason for random number generation
C	• The data sets are generated for the analysis of pipe aging effect • The data sets are generated for the analysis of standard deviation of "C" effect. • The data set is generated for the analysis of pipe material effect.
Q (m^3/s)	• The data sets are generated both two different economical lifetimes of the design alternatives

.

3.3.2 Hydraulic simulation

It is stated before that the input data for HapMam is given as text files without using Mapinfo. The necessary input files are shown as follows:

- **Pipe data:** From node, To Node, Length (m), Diameter (mm), C and Open/Close
- **Node data:** Nodal demands, Elevation (m), The minimum required head (m)
- **Pump data:** Pump no, From Node, To Node, Shut-off Head (m), Design Head (m), Design discharge (m^3/s), Maximum Operating Head (m), Maximum Operating Discharge (m^3/s)
- **Fixed Grade Node data:** Reservoir and Tank Elevations (m)
- **Element Data:** Number of pipes, nodes, fixed grade nodes, pumps

The generated random variables (Q_{design} and C) are assigned to the input files during the run of the program with minor modifications to HapMam, which were performed by Mısırdalı (2003). These modifications allow the user to perform 500 simulations in only one run.

3.3.3 Reliability Computation

500 hydraulic analyses (simulations) are performed in each run by the aid of HapMam, therefore 500 different numbers of available head (pressure) and demand are achieved in each run. By using these results, the reliability of the network is computed from two different approaches as "Head Approach (From H approach)" and "Consumption (Demand) Approach (From Q approach)" in this study. Some additional minor modifications are performed to the HapMam for reliability computation besides the minor modifications performed by Mısırdalı (2003).

- **Head Approach (From H approach):** In the head approach, the reliability of a node (nodal reliability) is computed by obtaining the probability that available head (supplied head) H_s, is equal or greater than the minimum required head $H_{min.}$ (Equation 3.7). The nodal reliability is computed for 500 times in each run. Then, the final nodal reliability is obtained by taking the average of those values.

$$R_{ni} = \begin{Bmatrix} 1 & if & H_{si} \geq H_{\min} \\ 0 & if & H_{si} < H_{\min} \end{Bmatrix} \tag{3.7}$$

For each run (500 simulations) R_{ni} is computed. Then $R_n$ave. is computed by taking the average of 500 R_{ni} values. (Equation 3.8).

$$R_n ave. = \frac{\sum_{i=1}^{500} R_{ni}}{500} \tag{3.8}$$

where:

H_{si} (m) : Supplied head at node of ith simulation

H_{min} (m) : The minimum required head at node

R_{ni} : Nodal reliability of ith simulation

$R_n$ave. : Average nodal reliability

The reliability of the whole network is computed by taking the weighted average of already calculated nodal reliabilities ($R_n$ave) by equation (3.9).

$$R_H = \frac{\sum_{k=1}^{N} R_n ave_k . W_k}{\sum_{k=1}^{N} W_k} \qquad (3.9)$$

where:

$R_n ave_k$: Average nodal reliability of node k^{th}

W_k : Nodal weight of node k^{th}

N : Numbers of node

R_H : The reliability of the network based on head approach

- **Demand Approach (From Q approach):** In the demand approach, the reliability of a node (nodal reliability) is computed by taking the ratio of consumption (available flow) Q_a to the required flow Q_r (Equation 3.10). The nodal reliability is also computed for 500 times in each run. Then, the final nodal reliability is obtained by taking the average of those values.

$$R_{ni} = \frac{Q_{ai}}{Q_{ri}} \qquad (3.10)$$

For each simulation R_{ni} is computed. Then, $R_n$ave. is computed by taking the average of 500 R_{ni} values similar as head approach. (Equation 3.11).

$$R_n ave. = \frac{\sum_{i=1}^{500} R_{ni}}{500} \qquad (3.11)$$

where:

Q_{ai} (m^3/s) : Avaliable flow (consumption) at the node of i^{th} simulation

Q_{ri} (m^3/s) : The minimum required demand at node of i^{th} simulation

R_{ni} : Nodal reliability of i^{th} simulation

$R_n$ave. : Average nodal reliability

The reliability of the whole network is also computed by taking the weighted average of already calculated nodal reliabilities ($R_n$ave.) by equation (3.12).

$$R_Q = \frac{\sum_{k=1}^{N} R_n ave_k . W_k}{\sum_{k=1}^{N} W_k} \tag{3.12}$$

where:

$R_n ave_k$: Average nodal reliability of node k^{th}

W_k : Nodal weight of node k^{th}

N : Numbers of node

R_Q : The reliability of the network based on demand approach

As a result, the reliabilities of the network from both head and demand approaches could be computed by following the steps, which are explained in this chapter. In order to see the steps clearly and in detail, the flowchart of the reliability process could be examined in Figure 3.2.

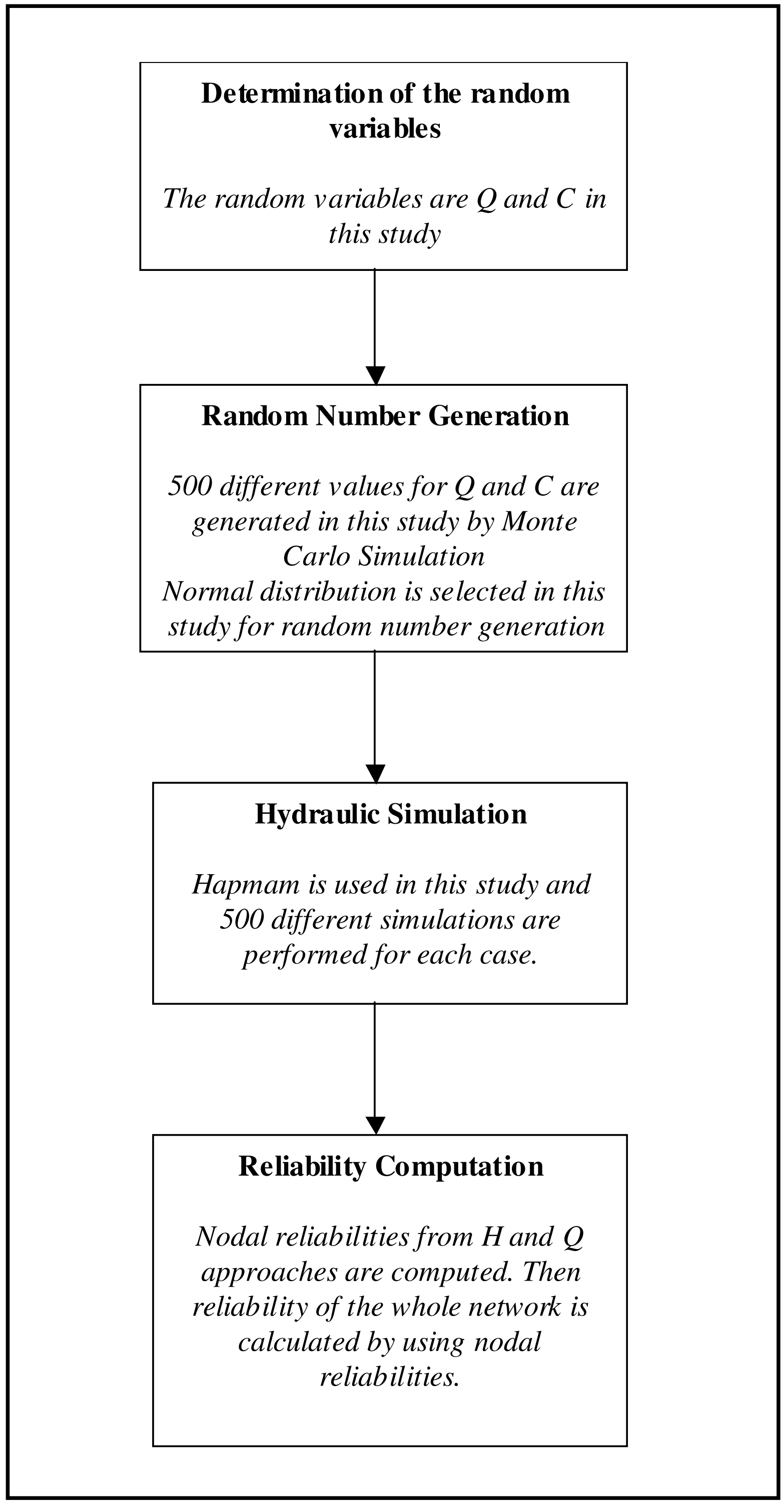

Figure 3.2: The flowchart of methodology of Reliability Process

CHAPTER 4

CASE STUDY

4.1. AIM OF THE STUDY

In this study, the aim was to find a relation between the reliability of a network and the cost of it for the proposed layout by the municipality. A family of networks was designed for the existing layout by WADISO (Chapter 2) which proceeds according to partial enumeration algorithm. Each network was designed satisfying a given minimum required pressure throughout the network. As the minimum required pressure assigned increases, the reliability levels will also increase. Note that, design discharge will be unique for all the networks designed by WADISO having the same economical life; as a larger required pressure value is assigned, diameter of the pipes are expected to come out larger in order to satisfy pressure condition.

The reliability of each network was calculated according to Mısırdalı (2003)'s adaptation (Chapter 3) of the methodology of Bao and Mays (1991). The nodal head values and available nodal demands, which are necessary for reliability calculations, were obtained by using HapMam (Chapter 3) prepared by Nohutçu (2002). Finally, the costs and the related reliabilities of the design alternatives were evaluated for 5 different cases. The relationship and the parameters that affect the relationship between reliability and cost during the design of a water distribution system were also examined in those cases.

4.2. ANKARA WATER DISTRIBUTION NETWORK

Ankara water distribution network serves to the consumers of capital of Turkey, which has the second largest population in the country. Today, it is estimated that this system provides water to about 4 million people and daily consumption is approximately 1,000,000 m^3 (200-250 lt/cap/day). Ankara water distribution network is composed of 2 treatment plants (İvedik and Pursaklar), 36 pump stations and 54 tanks (Figure 4.1). Kurtboğazı, Çamlıdere, Akyar, Eğrekkaya, Bayındır, Çubuk-2 Dams are the water sources of this network.

Ankara Water Distribution System was designed to serve 5 main pressure zones:

- Southern Supply Zone(e.g. Çankaya)
- Southwestern Supply Zone (e.g. Çayyolu, Ümitköy)
- Eastern and Southeastern Supply Zone (e.g. Mamak)
- Central and Western Supply Zone (e.g. Sincan, Etimesgut, Eryaman)
- Northern Supply Zone (e.g Keçören, Yenimahalle)

These main pressure zones are divided into several sub-pressure zones with 40-50 m elevation intervals.

4.2.1 Study Area (N8 Pressure Zone)

The Northern Supply Zone has 10 sub-pressure zones and the study area (N8-1) is one of the sub-zones of Northern Supply Zone in Ankara Water Distribution System (Figure 4.2).

The water is carried to the consumers in Northern Supply Zone from İvedik Treatment Plant starting from the pump station P1. P1 provides water to N-3 and N-4 sub-zones and P2 pump station. N-5 and N-6 are served by both P2 pump station and PN-1 Pump station, which takes water from Pursaklar treatment plant. N-7 zone takes the water from P12 pump station. P12 pump station also provides water to P23 pump station, which serves to N8-3 zone.

The study area, N8-1 zone is served by the pump stations P12 and P19. P19, which is in the study area also takes the water from P12 pump station. This layout is shown on the Figure 4.3.

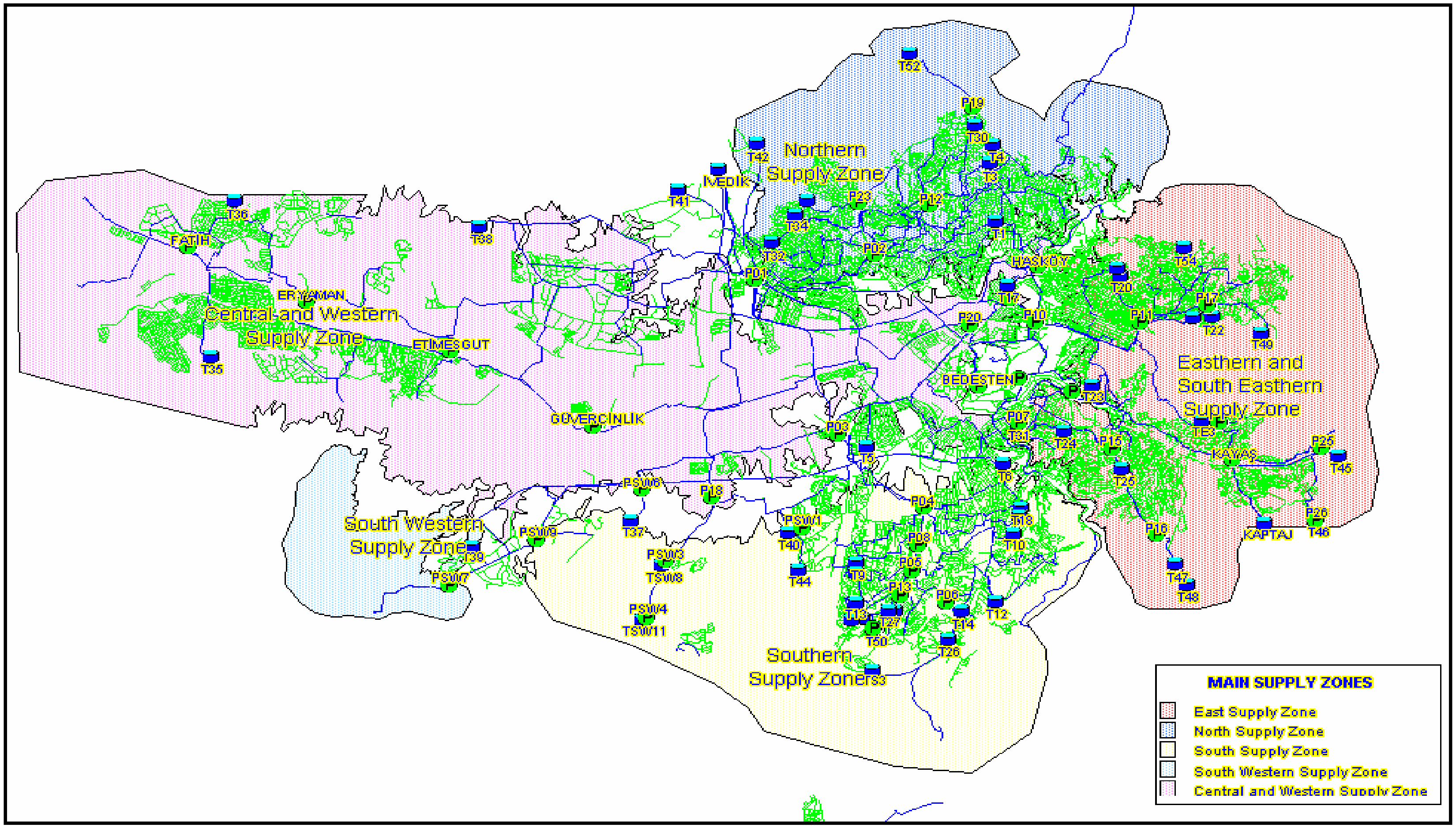

Figure 4.1: Ankara Water Distribution System

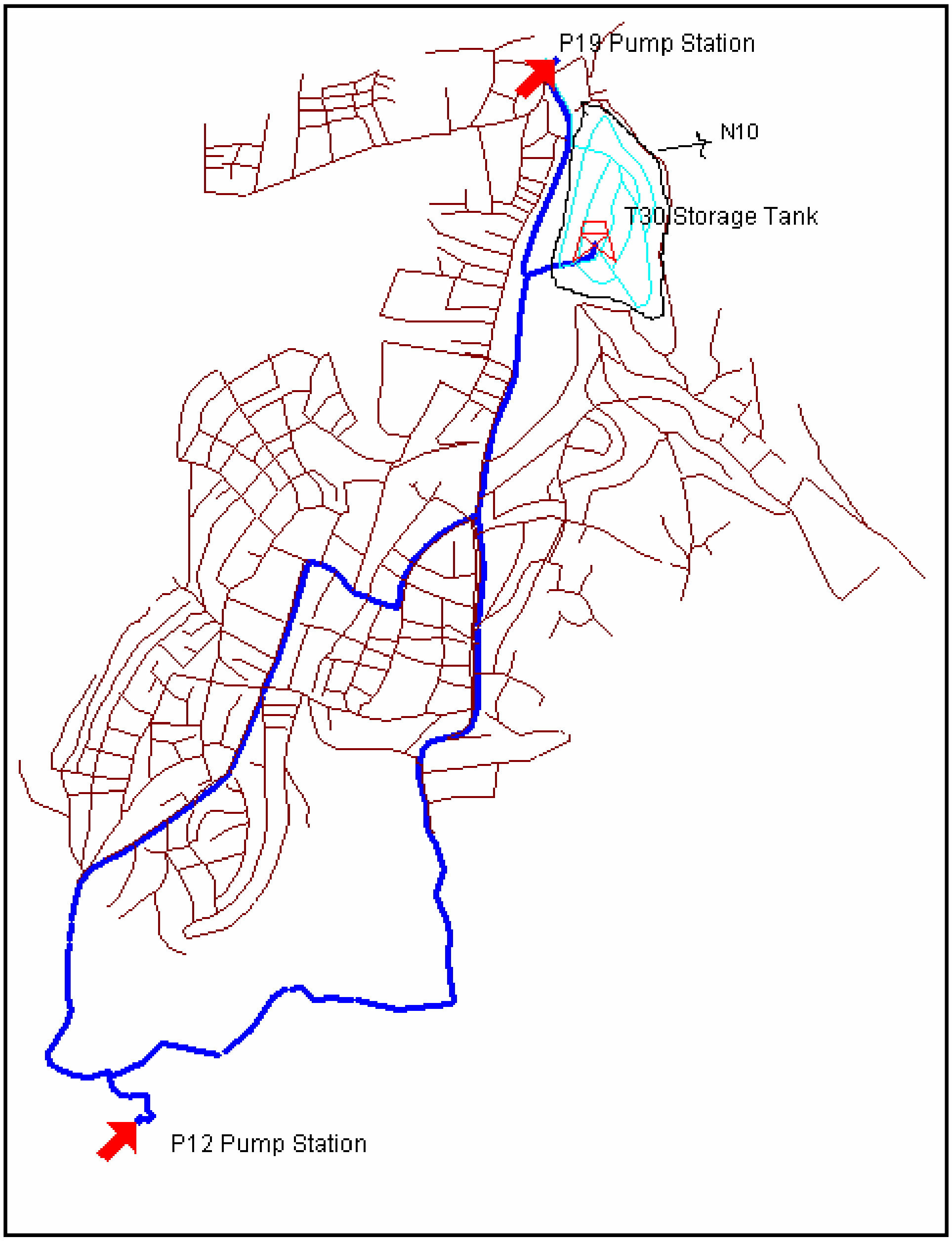

Figure 4.2: N8-1 Sub-pressure Zone (Mısıdalı,2003)

N8-1 Pressure Zone serves about 30.000 people living in Keçiören region. There are 2 pump stations (P12 and P19) and 1 storage tank (T30) in N8-1 Zone.

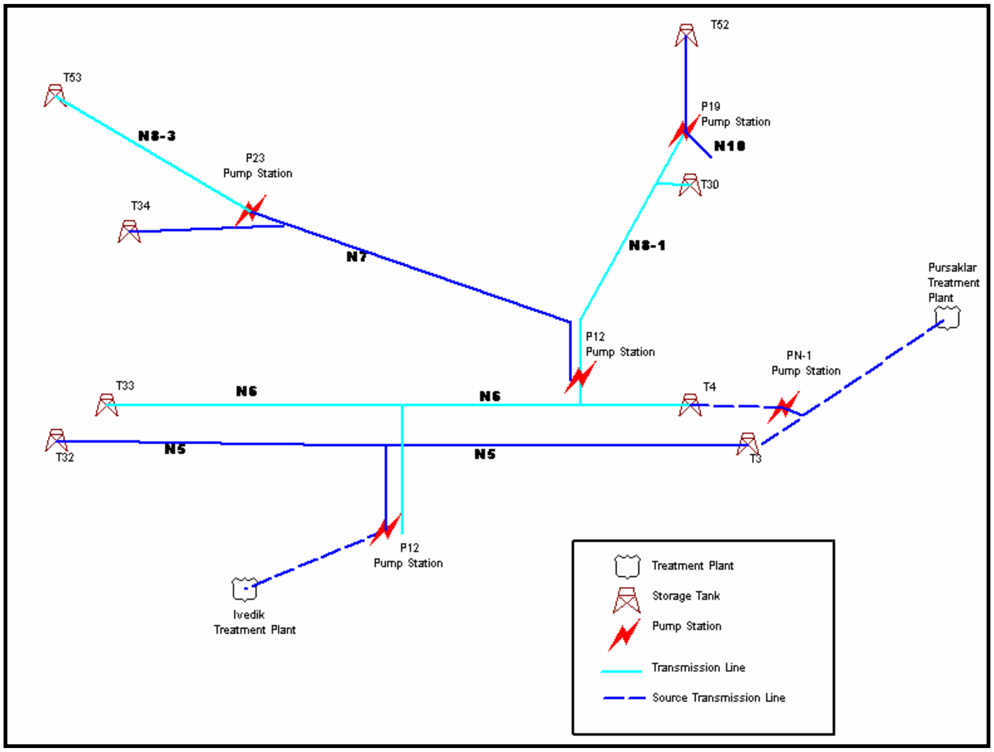

Figure 4.3: The layout of North Supply Zone (Mısırdalı, 2003)

4.3. HYDRAULIC MODEL

P12 pump station has 3 parallel pumps; the volume of T30 rectangular storage tank is 2500 m^3 with a height of 6.5 m. The existing system has ductile iron pipes, which were installed in 1992. The size of the diameter of main transmission line between P12 pump station and storage tank is 500mm.

4.3.1. Skeletonization

A skeletonization was performed for N8-1 Zone by Mısırdalı (2003). In that study, 100 mm diameter pipes and most of the 125 mm diameter pipes were eliminated except the pipes that form an important loop with major diameter pipes. In that study, the final system had 233 pipes and 174 nodes and the pipe diameters were between 100 mm and 500 mm.

Moreover, in this study the remaining dead ends were eliminated from the network for the ease of optimization. As a result, the network became a looped network (Figure 4.4).

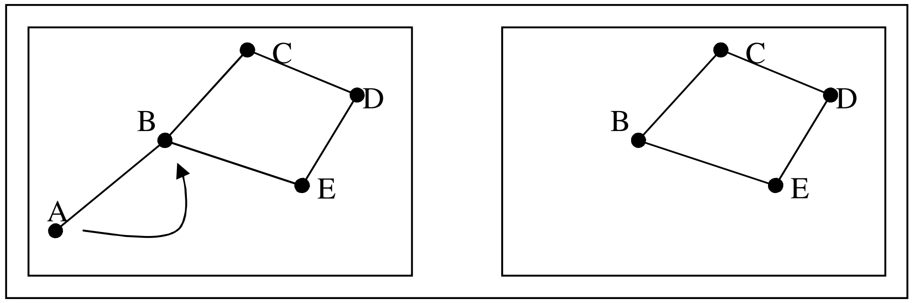

Figure 4.4: Eliminating of dead ends from the network

Furthermore, P12 pump station was replaced a reservoir with a constant water level by for the sake of simplicity. P19 pump station was also replaced by the representative node (closest node). Therefore, the extra flow, which is pumped by P19 to the outside of the study area (Bağlum Region), was assigned as an extra nodal demand to that node.

As a result, there are 215 pipes, 146 nodes, 1 reservoir and 1 storage tank in the final looped network (Figure 4.5).

4.3.2. Nodal Weights and Required Pressures

The nodal weights of the network, which are used in the calculation of nodal discharges, were already calculated by Service Area Method (SAM) (Mısırdalı, 2003). In this study, the already calculated nodal weights were taken into account. However, nodal weights of the dead end nodes were added to the closest node of the loop, while eliminating those dead end nodes. In addition, the effect of the extra water demand, which was assigned to the closest node of P19 is also taken into account in calculating the nodal weighs.

The required pressure for the system was taken as 25 m in the base scenario by Mısırdalı (2003). In this study, the minimum required pressures were also taken as 25 m for each node as one of the design alternative. The nodal weights, elevations of the nodes were shown on Table A.1.

4.3.3. Design Parameters

The pressure zone, N8-1 was established in 1992. However, the values of design parameters such as Q_{max}, Q_{peak} and Q_{night} could not be reached. So that, these design parameters were regenerated for this study. These works were carried out for two different designs having 15 years and 30 years of economical lifes. For this reason, future demand projections were performed for years 2007 and 2022. The necessary data which were obtained from ASKI Data Processing Center and SCADA Center were taken by Mısırdalı (2003).

The daily consumption data were obtained for the days starting from May 2002 to the end of December 2002. However, only 156 different daily demand curves could be used due to the errors in the data. The collected data period contains summer season when the maximum daily demand of the year generally occurs. The maximum daily demand was on 21.07.2002 among the data which could be obtained.

In this study, the future demand projection was performed by taking peak demand of the maximum daily demand curve of the year 2002 as a reference. The hourly peak demand was 652.5 m^3/hr between 13:00-14:00 on 21.07.2002 as shown in Figure 4.6 and Table 4.1.

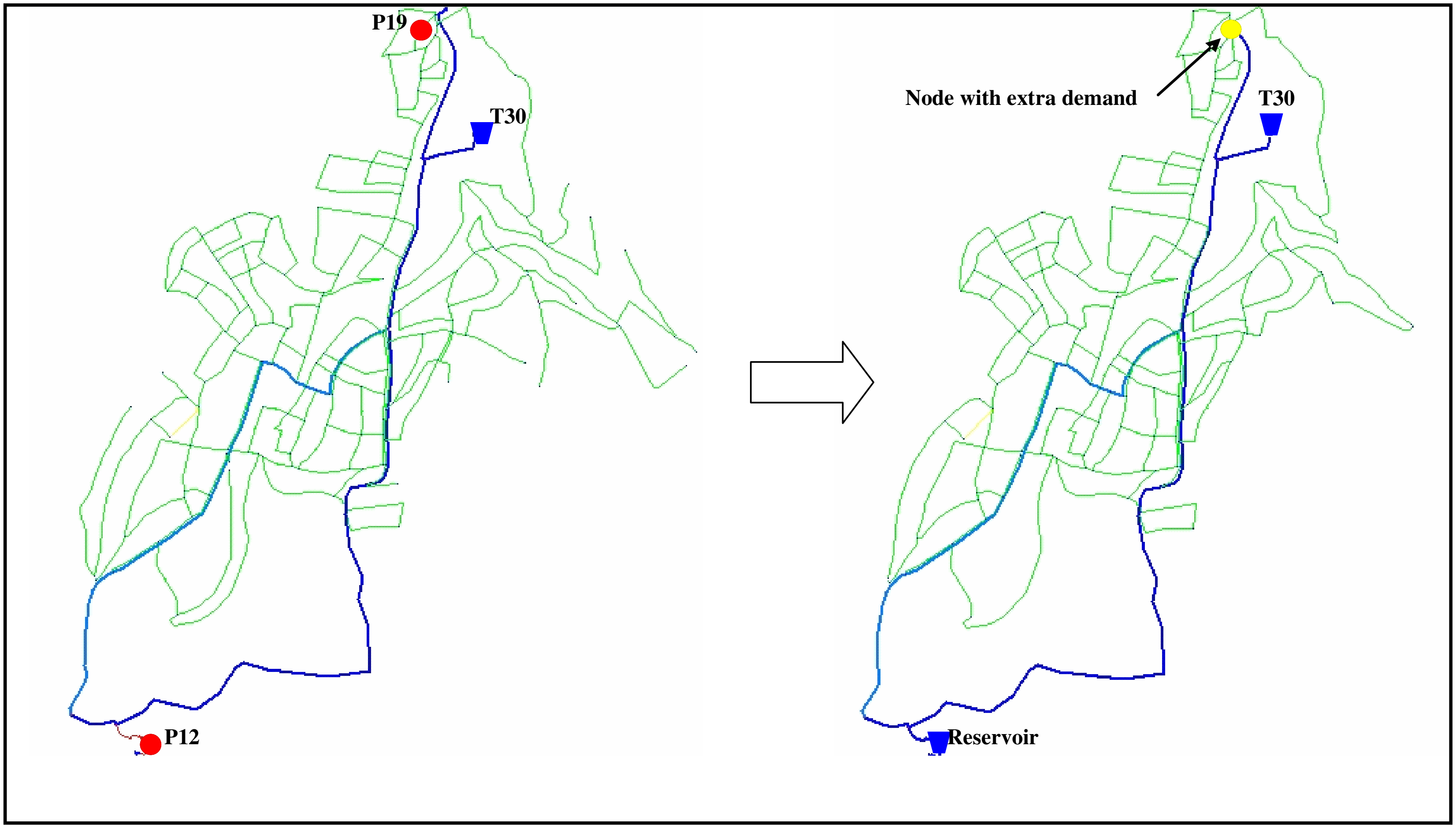

Figure 4.5: Skeletonized N8-1 (Mısrdalı,2003) (left side) and Further Modified N8-1 (right side)

Table 4.1: Daily demand values from Mısırdalı (2003)

Hours	**The maximum daily demand on 21.07.2002 (m^3/hr)**	**Average daily demand of 156 days (m^3/hr)**
0	400.05	307.10
1	342.11	235.90
2	282.55	209.63
3	244.29	201.56
4	295.03	204.45
5	244.86	204.18
6	282.62	234.10
7	341.72	287.49
8	460.46	333.27
9	431.38	388.50
10	585.43	438.36
11	630.70	481.17
12	647.31	489.73
13	**652.54**	477.81
14	588.48	444.61
15	530.10	412.09
16	523.11	394.71
17	536.94	386.85
18	551.80	390.75
19	521.43	381.89
20	514.70	380.40
21	477.42	360.25
22	419.93	336.73
23	410.01	316.91
24	387.69	309.26
Total	**11,302.67 m^3/day**	**8,607.70 m^3/day**

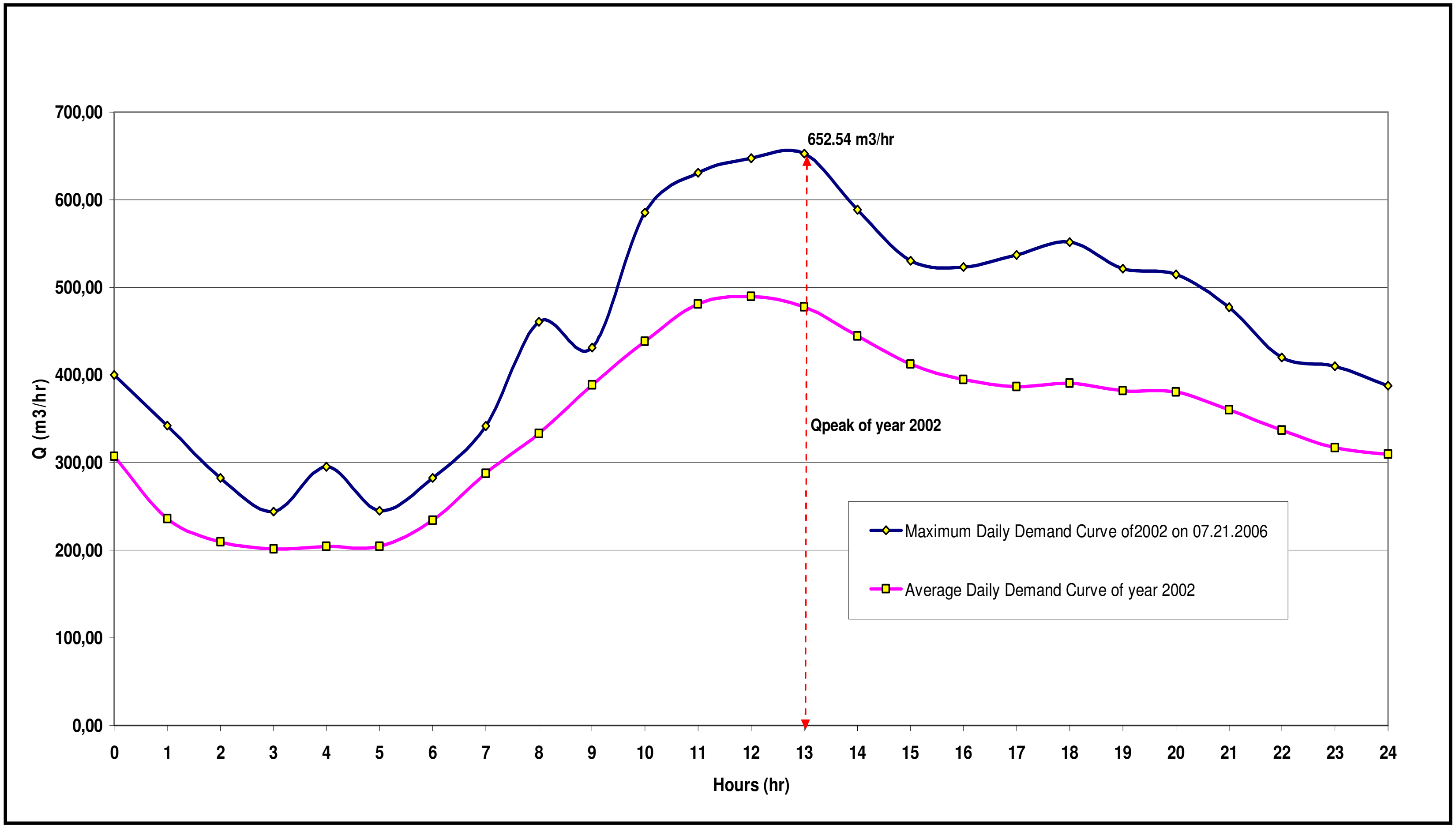

Figure 4.6: Daily Demand Curves

The data in daily demand curve were only the consumption data of N8-1 however, it was stated that an extra demand (100 m^3/hr), which is pumped by P19 pump station to Bağlum Region, was added to the closest node. So, that amount of extra demand was addded to hourly demand data. As a result, the peak demand became (652.5+100) 752.5 m^3/hr. However, this peak demand belongs to the year 2002.

The peak demands of the years 2007 and 2022 were calculated by future demand projection by the aid of Turkish Bank of Provinces method which uses geometric extrapolation as shown on the following equation (4.1).

$$Pn = P_2\left(1+\frac{k}{100}\right)^{tn-t2} \qquad \text{where} \quad k = \left[\sqrt[(t_2-t_1)]{\frac{P_2}{P_1}}-1\right]x100 \tag{4.1}$$

P_n : The population at the year n
P_2 : The known population
t_n : The year n
t_1 and t_2 : Years of the known populations
k : a coefficient (between 1 and 3)

Although population and discharge projections don't obey similar trends, as a rough approximation, population projection formula of Bank of Provinces method is used for discharge projection since demand is a function of population. The "k" coefficient was taken as its maximum value, 3 to obtain the maximum demands. However, according to last census results, a more accurate k coefficient could be taken into account..

In this study, the known peak demand was $Q_{peak(2002)}$ = 752.5 m^3/hr. Then, by using the formula as shown below, the peak demand values for the years 2007 and 2022 were calculated and tabulated on Table 4.2.

$$Q_{peak(2007)} = Q_{peak(2002)} x\left(1+\frac{k}{100}\right)^{2007-2002} = 752.5x(1.03)^5 = 872.4 m^3/hr$$

$$Q_{peak(2022)} = Q_{peak(2002)} x\left(1+\frac{k}{100}\right)^{2022-2002} = 752.5x(1.03)^{20} = 1359.2 m^3/hr$$

Table 4.2: The estimated peak demands of years 2007 and 2022

Life time of the design	$Q_{peak(years)}$	(m^3/hr)	(lt/s)
15 years	$Q_{peak\ (2007)}$	872.4	242.3
30 years	$Q_{peak(2022)}$	1359.2	377.5

4.3.4. The Design of Main Transmission Line

The peak demands were used in optimization part of this study in order to find the most economical pipe sizes of the network except the main transmission line. The main transmission line is composed of the pipes, which connect the water source (reservoir and/or pump) to the storage tank of the network.

The main transmission line was not desired to be designed by the optimization program WADISO, since WADISO normally gave different pipe sizes for the main transmission line when the minimum required pressure values change in the design alternatives. Sometimes the pipe size of the main transmission line would be smaller than the pipe sizes of tributary lines, which was an unacceptable case.

In order to design the main transmission line, night loading (Q_{night}) values should be obtained. This was performed by using the assumed relationship between the loading scenarios; (Q_{peak}, Q_{max} and Q_{night}) as shown below (Table 4.3). It is assumed that 30% of maximum demand is consumed during night (Q_{night}) by the users and the remaining part (70% Q_{max}) is transferred by the main transmission line from the reservoir to the storage tank.

$Q_{peak} = 1.5\ Q_{max}$ and $Q_{night} = 0.3\ Q_{max}$

Table 4.3: Loading scenarios for the years 2007 and 2022

Life time of the design	Q_{night} (lt/s)	Q_{max} (lt/s)	Q_{peak} (lt/s)
15 years (2007)	48.5	161.6	242.3
30 years (2022)	75.5	251.7	377.5

The diameters of the main transmission line were obtained by using Hazen-Williams flow equation (4.2) for designs of economical lifes of 15 and 30 years respectively. The results were shown below.

$$H = \frac{10.583 * L * Q^{1.85}}{C^{1.85} * D^{4.87}} \qquad \text{(for metric units)} \qquad (4.2)$$

where:

H : head Loss in pipe (m)

D : diameter of pipe (m)

L : length of pipe (m)

Q : flow rate in the pipe (m^3/s)

C : Hazen-Williams roughness coefficient

In this study,

H : (ΔH between reservoir and tank)=(1156.5-1137.27)= 19.23 m

L : (Length of the main transmission line) = 4069 m

Q : (0.7 Qmax) = Q=0.113 m^3/s for 15 years design (year 2007)

Q=0.176 m^3/s for 30 years design (year 2022)

C = 130 for new ductile iron pipes

D=0.336 m for 15 years design (year 2007)

D=0.397 m for 30 years design (year 2022)

In order to serve more efficiently in the peak hours, these obtained diameters are increased to be 0.400m and 0.500 m respectively for 15 years and 30 years design.

4.4. OPTIMIZATION PROCESS

In optimization process, as already mentioned, partial enumeration technique was used by the aid of the software, WADISO. The algorithm and the flowchart of the program were given in Chapter 3.

Firstly, the network was presented as a base layer by importing the file from Mapinfo environment into WADISO. The pipes, nodes were redrawn by using this base layer.

The further modified network was obtained by eliminating dead ends as mentioned before. Pipe numbers, lengths, Hazen-Williams roughness coefficients were assigned to the pipe characteristics. Node numbers, elevations and nodal weights were assigned for the nodes. The ground elevation of the tank, water level and static head data were assigned as tank properties.

In this study, several design alternatives were worked out using several loading scenarios by the aid of optimization process. The basic variables were the minimum required heads at the nodes (H_{min}) and the pipe materials.

4.4.1. Pipe Grouping

In the optimization part of this study, the pipe grouping was performed as a first step. The pipes of the network were grouped basically according to their similarities, locations, priorities. The total lengths of the pipe groups were listed on Table 4.4 and the pipes of the groups could be seen on the TableB.1 in detail.

There were 5 pipe groups in the network such as:

- Group 1: The main transmission line between reservoir and tank (dark blue line on Figure 4.7)
- Group 2: The secondary transmission line parallel to main transmission line (red lines on Figure 4.7.)
- Group 3: The line, which connects main transmission line to the node with extra demand (light blue line on Figure 4.7)
- Group 4: The primary distribution lines (the gray lines on Figure 4.7)
- Group 5: The secondary distribution lines (the green lines on Figure 4.7)

Table 4.4: The lengths of the pipes in the groups

Group No.	Length (m)
1	4,069.00
2	2,913.00
3	668.00
4	27,325.99
5	7,343.00
Total Length	**42,318.99**

4.4.2. Candidate Pipe Sizes

The second step in optimization by using WADISO, was the determination of candidate pipe sizes. Although, it is stated that "number of candidate pipe sizes for a group has significant effect on the computation time" by Keleş (2005), it was seen that this problem was almost solved in latest versions of WADISO.

Every pipe groups except group 1 had several candidate pipe sizes as shown on Tables 4.5 and 4.6. Group 1 had only one pipe size alternative for designs having economical lifes of 15 and 30 years. As calculated before, for 15 years design it was taken as 400 mm and 500mm for 30 years design.

4.4.3. Price Functions

The result of optimization is very sensitive to the price functions since the main aim is to get the most economical design, which satisfies the hydraulic conditions. In this study, two different pipe materials were used. These were HDPE (High Density Polyethylene) pipes and ductile iron pipes. The pipe prices of these materials were taken from State Hydraulic Works (Devlet Su İşleri) and Bank of Provinces (İller Bankası) price lists by Keleş (2005).

Table 4.5: Candidate Pipe Sizes for 15 years design

Economical life = 15 years design					
Alternative Sizes	Group 1 (mm)	Group 2 (mm)	Group 3 (mm)	Group 4 (mm)	Group 5 (mm)
1	400	100	125	80	80
2		125	150	100	100
3		150	200	125	125
4		200	250	150	150
5		250	300	200	
6		300	350	250	
7		350	400		
8		400	450		
9		450	500		
10		500			

Table 4.6: Candidate Pipe Sizes for 30 years design

Economical life = 30 years design					
Alternative Sizes	Group 1 (mm)	Group 2 (mm)	Group 3 (mm)	Group 4 (mm)	Group 5 (mm)
1	500	100	125	80	80
2		125	150	100	100
3		150	200	125	125
4		200	250	150	150
5		250	300	200	
6		300	350	250	
7		350	400		
8		400	450		
9		450	500		
10		500			

In the calculation of the price function for HDPE pipes, trench excavation, fill and bedding and compaction unit prices were taken from State Hydraulic Works unit price list. Additionally, unit prices of pipe related works such as pressure test before laying, connection of HDPE pipes with butt welding, laying of HDPE pipes and HDPE pipe resistant to 10 atm were taken from both State Hydraulic Works and Bank of Provinces unit price lists by Keleş (2005).

These unit prices were added to the price of pipe material to obtain the total price function. Moreover, a market search was performed in the calculation of price function of ductile iron pipes, and the price function list was formed by Keleş (2005).

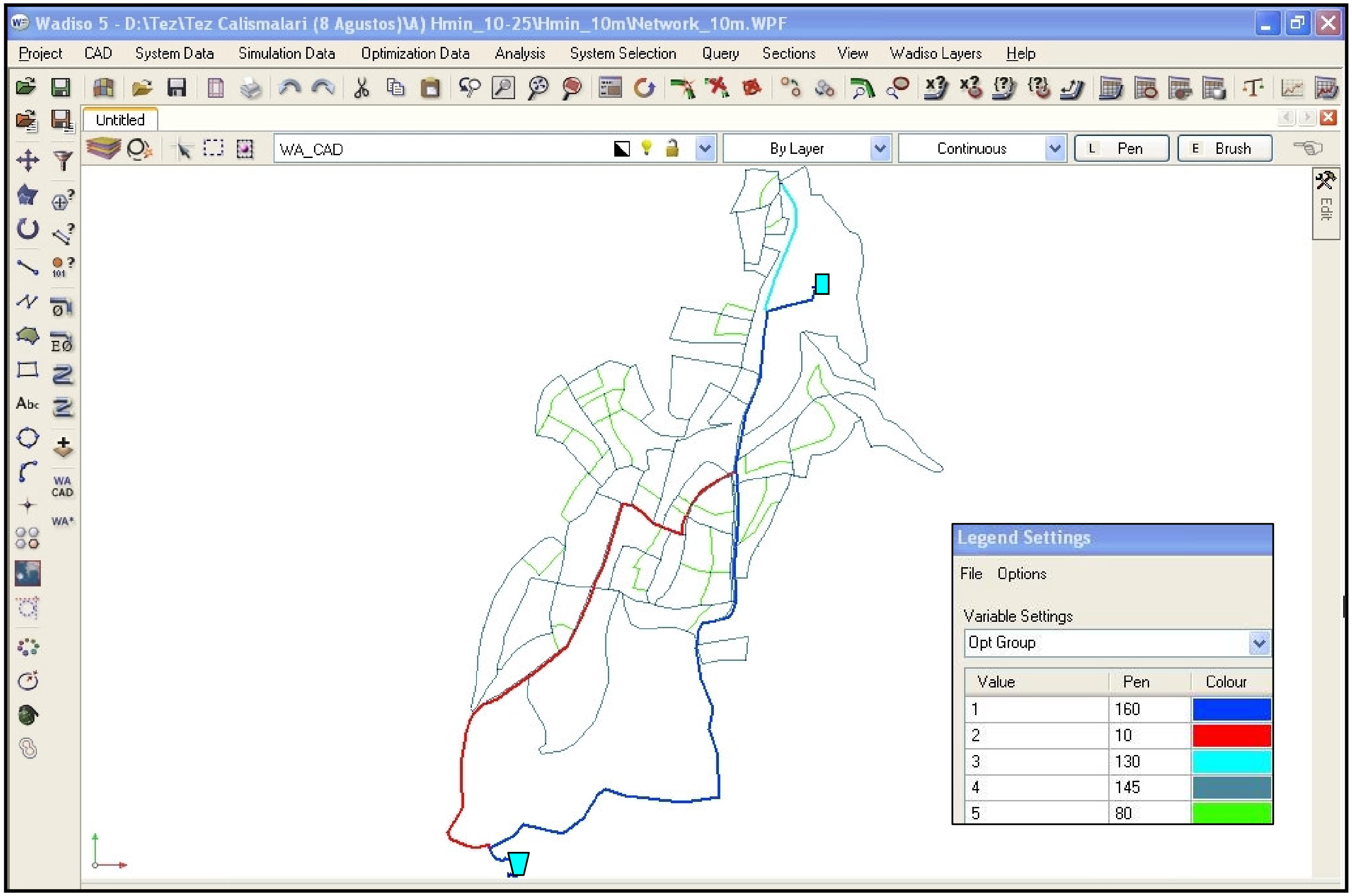

Figure 4.7: The Pipe Groups of N8-1

In the calculation of price function of ductile iron pipes, the cost items such as trench excavation, fill, bedding and compaction, pressure test before laying, welding, inner and outer insulation of welded ends, laying were included additionally to the price of pipe material.

However, the pipe fittings could not be included to the price functions of the pipe materials since the used optimization program WADISO accepts the price functions as price/length although the unit price of the pipe fittings are mostly related to the pipe diameters which they are connected. As stated by Keleş (2005), "This creates a vicious circle: in order to include the price of fitting, diameters have to be known, but the optimization, i.e. inclusion of pipe fitting prices in price function, is performed to determine the diameters. As the result, since there is no mathematical relationship between the fittings and the pipes, it is almost impossible to consider effect of fittings during optimization with WADISO."

The price function of HDPE pipes according to market data which was used by Keleş (2005) was taken as a reference in this study. The range of diameters of HDPE pipes were between 90 mm and 630 mm in that study. By interpolation and extrapolation methods the new price function was obtained for used pipe diameters in this study as can be seen on Table 4.7 and Figure 4.8. The same procedure was followed to obtain the price function of ductile iron pipes.

4.4.4. Alternative Designs

In this study, by the aid of WADISO several design alternatives were obtained. The variables were the minimum required heads at the nodes (H_{min}) and pipe materials as stated before.

Two different economical lifes were selected such as 15 years and 30 years for the network in order to see the effect of this design parameter. Designs were performed by changing the minimum required heads from 10m to 25 m both for economical lifes of 15 years and 30 years designs. Ductile iron pipes were used as the pipe material in both 15 years and 30 years design whereas HDPE pipes were also used for 30 years design for the comparison of pipe materials.

Table 4.7: Price Functions

Diameter (mm)	Price of Ductile Iron Pipes (YTL/m)	Price of HDPE Pipes (YTL/m)
80	15.13	8.43
100	16.19	11.31
125	17.51	15.27
150	19.04	20.28
200	24.98	32.08
250	31.43	47.53
300	37.86	69.52
350	45.96	92.70
400	51.78	118.41
450	65.88	148.12
500	71.27	179.26

The design of main transmission line was already performed in Section 4.4.4 of this study. It was found that the main transmission line had 400 mm and 500 mm of pipe diameters for 15 years and 30 years design respectively. In that design night loading (Q_{night}) was taken into account. In this part, pipe sizes except main transmission line would be optimized by taking peak loading (Q_{peak}) as design discharge (Q_{design}).

The Hazen-Williams roughness coefficients of the ductile iron pipes and HDPE were 130 and 150 respectively. In all scenarios, the water level (static head) in the reservoir was taken as 136 m, which is the rated pump head of P12 pump station. The storage tank had a height of 6.5 m and the operational height, was oscillating between 2.5 m - 5.0 m. It was assumed that in the peak loading the tank level would be in the middle so that the water level at the tank was taken as 3.75 m. These conditions were tabulated on Table 4.8.

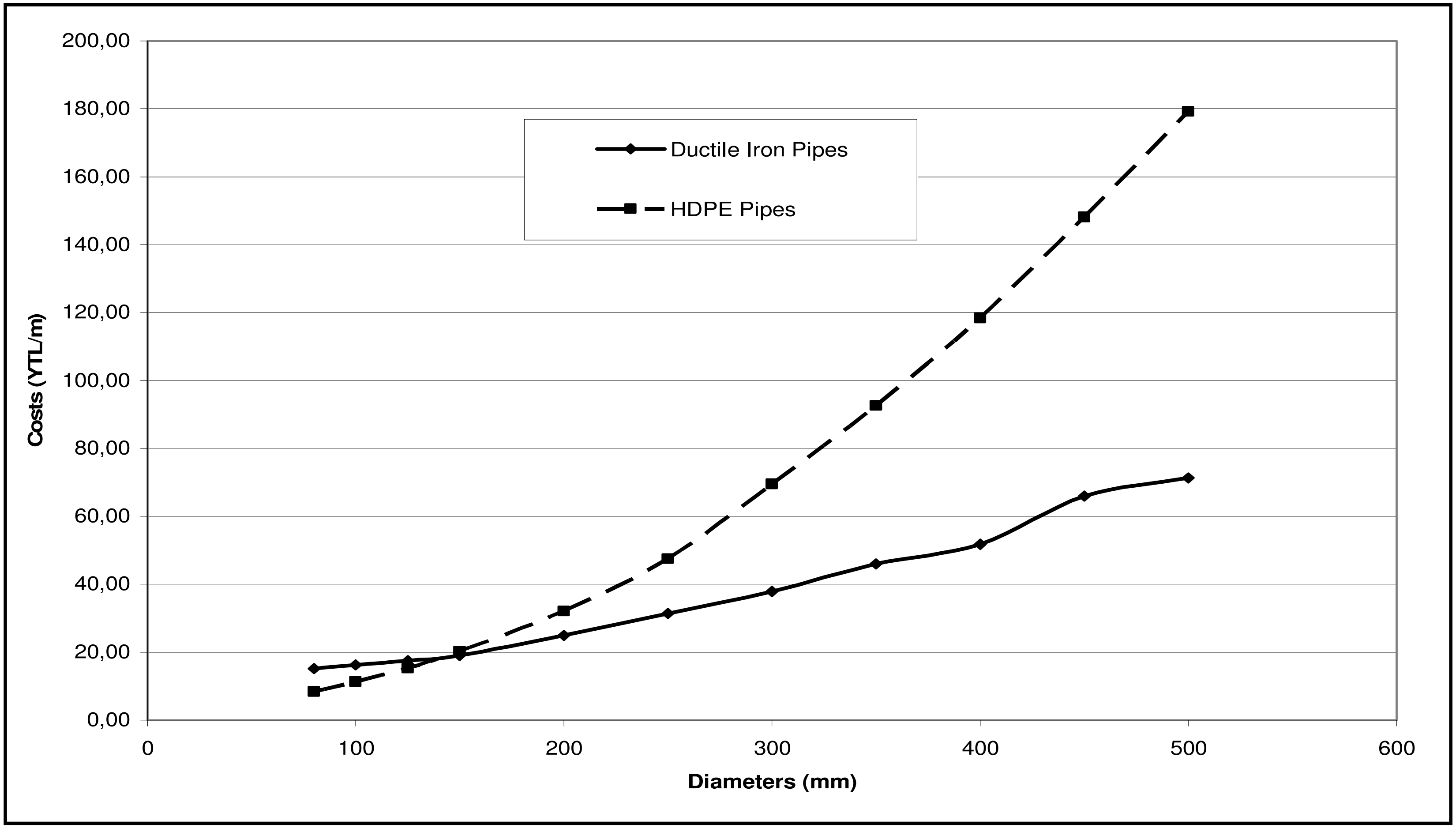

Figure 4.8: Price Functions Graphs

Table 4.8: Design Parameters for 15 years and 30 years design

Life time of the design	15 years	30 years
Q_{design} (m^3/s)	0.242	0.378
H_{min} (m)	10 - 25	10 - 25
HW Coefficient (C)	130	130 and 150
The Water level at Reservoir (m)	136	136
The Water level at Tank (m)	3.75	3.75

4.4.4.1. Changing The Minimum Required Pressures (H_{min})

It is stated that the minimum required pressure in the base scneraio for the case study area was 25m (Mısırdalı, 2003). In this study, the minimum required pressure for the base scenario was also 25 m. However, to achieve the aim of this study, which is to demonstrate the relationship between the reliability and the cost during the design of a water distribution network, several alternative designs (several alternative design costs and related reliabilities) should be obtained. For this reason, minimum required pressures were taken the values from 10 m to 25 m in the design alternatives.

It can be easily seen that, when the minimum required pressure was taken as 10 m (lower limit), the design would not satisfy the real operational hydraulic conditions. The pipe sizes would be smaller, the cost would be less; however the pressures of the most of the nodes would be less than 25 m. As a result the reliabilities would be less as can be seen in the following section.

On the other hand, when the minimum required pressure was taken as 25 m (upper limit, the required pressure) the pipe sizes would be larger, the cost would be more and the pressures of all the nodes would be at least 25 m. As a result the reliability analysis gave the highest reliability values among the other design alternatives as can be seen in the following section.

Finally, several runs were performed by using the already defined pipe grouping, candidate pipe sizes, price functions, nodal weights, design discharges for the 15 years and 30 years designs by WADISO.

(Economical life of) 15 years of design:

In 15 years design, 10 different alternatives (10 different minimum required heads) were obtained from WADISO as shown on Table 4.9 and Figure 4.9.

Table 4.9: The design alternatives of 15 years design with Ductile Iron Pipes

Q_{design} =0.242 m^3/s, HW Coefficient (C) =130							
H_{min} (m)	Cost (YTL)	OPTIMUM DIAMETERS	Group 1 (mm)	Group 2 (mm)	Group 3 (mm)	Group 4 (mm)	Group 5 (mm)
10	894,032.70		400	200	250	125	80
14	898,327.90		400	200	300	125	80
16	907,626.40		400	200	400	125	80
17	922,527.60		400	250	350	125	80
18	934,198.90		400	250	400	125	100
20	954,630.40		400	250	250	150	80
21	973,360.90		400	300	250	150	80
23	996,956.30		400	350	250	150	80
24	1,013,909.90		400	400	250	150	80
25	1,023,615.90		400	400	350	150	80

(Economical life of) 30 years design:

In 30 years design, 14 different alternatives (10 different minimum required heads) were obtained from WADISO as shown on Table 4.10 and Figure 4.9.

The authority should decide on the life time of the design whether 15 years or 30 years. As can be seen on Table 4.11 and Figure 4.9. 15 years design gave more economic results than 30 years design however the pipe sizes may be insufficient after 15 years from construction

Table 4.10: The design alternatives of 30 years design with Ductile Iron Pipes

Q_{design} =0.377 m^3/s, HW Coefficient (C) =130							
H_{min} (m)	Cost (YTL)		Group 1 (mm)	Group 2 (mm)	Group 3 (mm)	Group 4 (mm)	Group 5 (mm)
10	1,019,441.50	OPTIMUM DIAMETERS	500	200	300	150	80
12	1,024,852.30		500	200	350	150	80
13	1,036,523.70		500	200	400	150	100
14	1,038,230.40		500	250	300	150	80
16	1,043,641.20		500	250	350	150	80
17	1,047,528.90		500	250	400	150	80
18	1,066,259.60		500	300	400	150	80
19	1,087,062.40		500	300	500	150	100
20	1,116,227.40		500	400	450	150	80
21	1,163,582.90		500	500	400	150	80
22	1,194,078.50		500	500	500	150	125
23	1,265,237.00		500	400	350	200	80
24	1,307,996.80		500	450	350	200	80
25	1,343,101.40		500	500	450	200	100

Table 4.11: The Comparison of 15 years design with 30 years design

H_{min} (m)	Cost of 15 years design (YTL)	Cost of 30 years design (YTL)	% Change of cost of 15 years design with respect to 30 years design
10	894,032.70	1,019,441.50	14.03%
14	898,327.90	1,038,230.40	15.57%
16	907,626.40	1,043,641.20	14.99%
17	922,527.60	1,047,528.90	13.55%
18	934,198.90	1,066,259.60	14.14%
20	954,630.40	1,116,227.40	16.93%
21	973,360.90	1,163,582.90	19.54%
23	996,956.30	1,265,237.00	26.91%
24	1,013,909.90	1,307,996.80	29.01%
25	1,023,615.90	1,343,101.40	31.21%

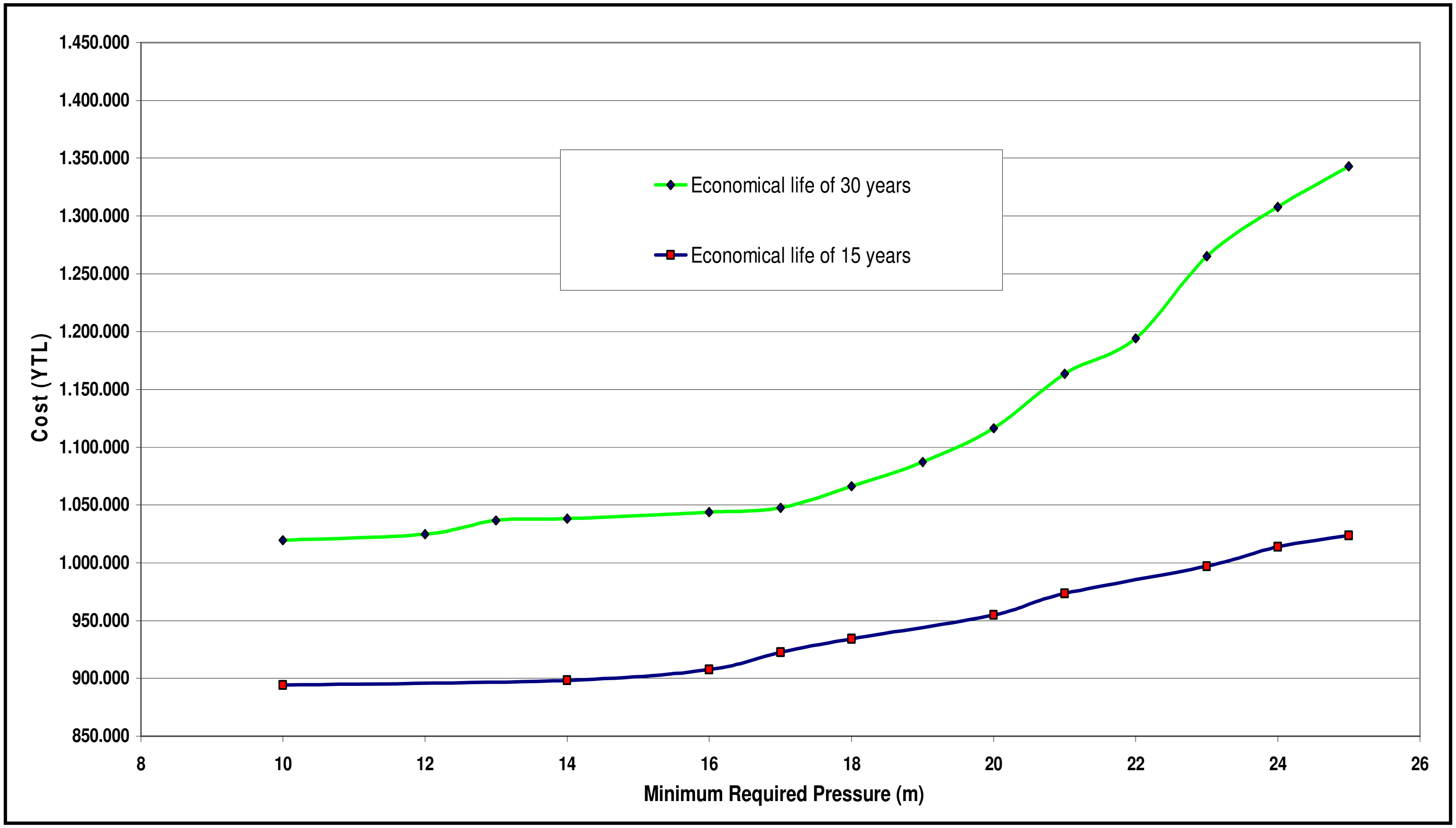

Figure 4.9: The comparison of costs of design alternatives with economical lifes of 15 years and 30 years

In other words, if 15 years design is chosen then the operational cost would be more than 30 years design after 15 years. In order to find the most optimal design, the reliabilities and the overall cost of the network designs should be evaluated carefully.

4.4.4.2. Changing Pipe Materials

There are mainly two effects of pipe materials on the design of the network. The first effect is the difference between price functions i.e. cost effect. The other one is the change of Hazen-Williams roughness coefficient i.e hydraulic effect. The price functions were already examined in the previous section.

In this study, Hazen-Williams flow equation was chosen as the flow equation in the algorithm of WADISO. Hazen-Williams roughness coefficients were given as 130 for the ductile iron pipes and 150 for HDPE pipes.

Since, there is an inverse proportion between head losses and Hazen-Williams roughness coefficient, the friction along the pipes increases when the Hazen-Williams roughness coefficient decreases. As a result, in order to satisfy the hydraulic conditions, the pipe sizes increases in the design of the network where the pipes have less Hazen-William Coefficient.

On Tables 4.12 and 4.13 and in Figure 4.10 it can be seen that the designs with ductile iron pipes are more economic then the designs with HDPE pipes. The change in the cost is about 50% in all designs. However, it should not be forgotten that, the final decision in the selection of pipes would be given by taking the overall cost and the characteristics of the pipe materials into account.

Table 4.12 : The design alternatives of 30 years design with HDPE pipes

Q_{design} =0.377 m^3/s, HW Coefficient (C) =150							
H_{min} (m)	Cost (YTL)	OPTIMUM DIAMETERS	Group 1 (mm)	Group 2 (mm)	Group 3 (mm)	Group 4 (mm)	Group 5 (mm)
10	1,470,680.80		500	200	250	150	80
15	1,485,370.10		500	200	300	150	80
17	1,530,376.00		500	250	300	150	80
18	1,530,376.00		500	250	300	150	80
20	1,594,432.80		500	300	300	150	80
21	1,609,917.00		500	300	350	150	80
22	1,694,614.80		500	350	400	150	80
23	1,789,354.30		500	400	450	150	80
24	1,917,848.60		500	450	500	150	100
25	2,074,426.30		500	500	500	150	150

Table 4.13: Comparison of the design alternatives C=130-150

H_{min} (m)	Cost of design with ductile iron pipes C=130 (YTL)	Cost of design with HDPE pipes C=150 (YTL)	% Change in cost between pipe materials
10	1,019,441.50	1,470,680.80	44.26%
15	1,038,230.40	1,485,370.10	43.07%
17	1,047,528.90	1,530,376.00	46.09%
18	1,066,259.60	1,530,376.00	43.53%
20	1,116,227.40	1,594,432.80	42.84%
21	1,163,582.90	1,609,917.00	38.36%
22	1,194,078.50	1,694,614.80	41.92%
23	1,265,237.00	1,789,354.30	41.42%
24	1,307,996.80	1,917,848.60	46.62%
25	1,343,101.40	2,074,426.30	54.45%

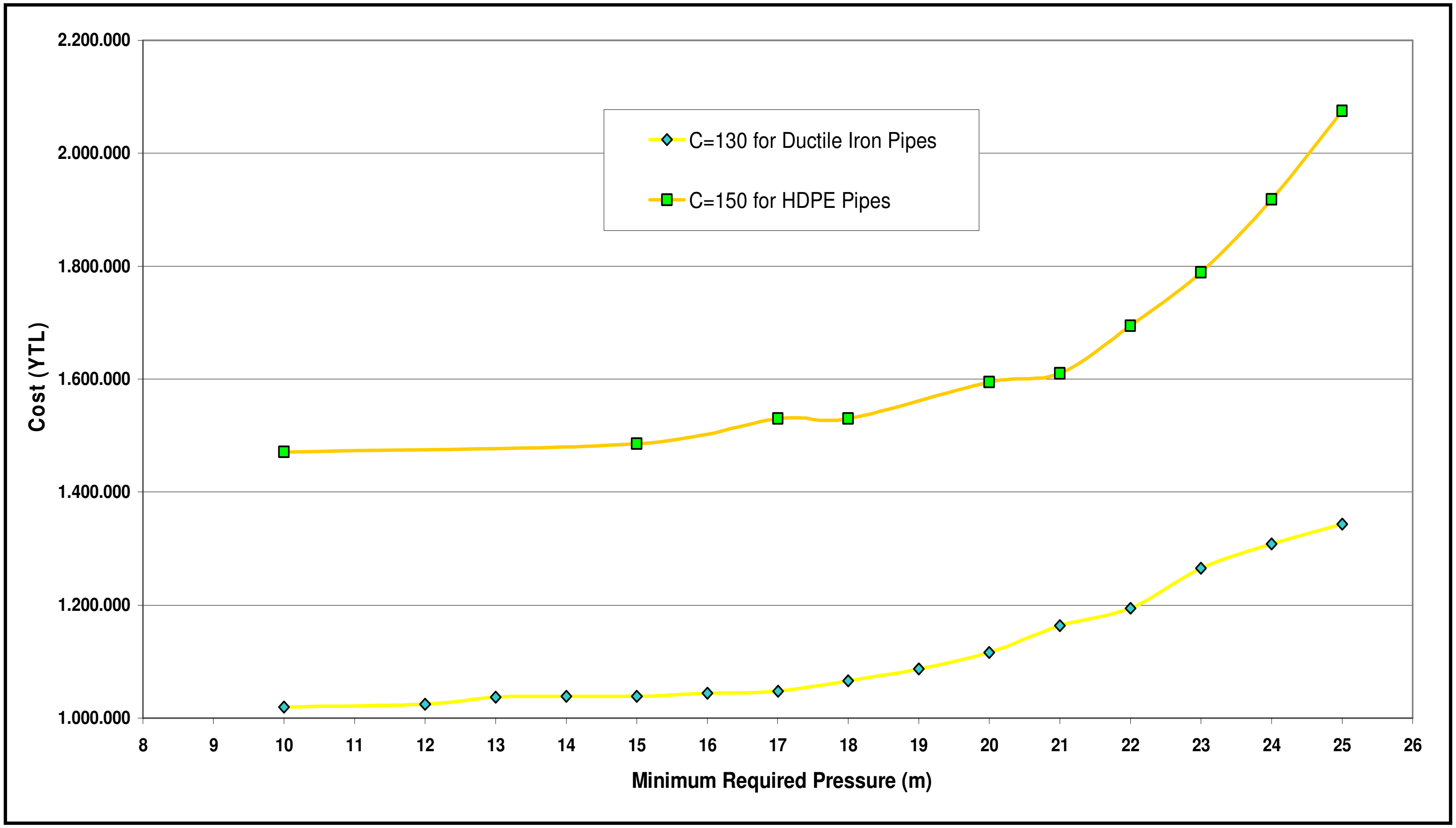

Figure 4.10: The comparison of the design alternatives with ductile iron and HDPE pipes for 30 years design

4.5. RELIABILITY PROCESS

The evaluation of the system reliabilities was the second part of this study. In the optimization part, several alternative designs were obtained by changing the minimum required heads (H_{min}) and pipe materials with different Hazen-Willams roughness coefficient (C) values. In every design, costs, optimum pipe diameter sizes of the groups, the pressures at the nodes, the discharges in the pipes were obtained.

In this part, the system reliabilities of all alternative designs were calculated. Five different cases were taken into account in the reliability calculations. All the cases except first case "Changing The Minimum Required Pressures (H_{min})", are performed only for economical lifetime of 30 years. These cases were:

1. Changing The Minimum Required Pressures (H_{min}) for both economical lifetimes.
2. The effect of Standard Deviation of Hazen-Williams roughness coefficient (C) change on reliability
3. Changing Pipe Material
4. Different Valve Status (Open/Closed)
5. Pipe Aging Effect

The necessary data for reliability analysis are generated by Monte Carlo simulation (Table 4.14). The coefficient of variation for Q and C were taken from the data of Mısırdalı (2003) as 0.185 and 0.154, respectively.

A hydraulic network solver program (HapMam) prepared by Nohutçu (2002) based on modified Chandapillai model, was used as the hydraulic network simulator in this study. HapMam (Hydraulic Analysis Program with Mapinfo and Matlab) finds the supplied heads (H_s) and the consumptions (Q_a) at the nodes of a network.

As stated before, HapMam uses linear theory for fixed demand analysis and the pressure dependent theory (head-driven theory) for partial flow analysis. In this study, the head-driven theory was carried out by Modified Chandapillai model by the aid of HapMam. Additionally, some minor modifications were performed to HapMam in the reliability computations.

Table 4.14: The necessary data for case studies generated by Monte Carlo simulation

The random variable	Mean	Standard Deviation	Note
C	74	11	These data sets are generated for the analysis of pipe aging effect
	83	13	
	95	15	
	110	17	
	130	10, 15, 20, 25, 30, 35, 40, 45, 50, 55, 60	These data sets are generated for the analysis of standard deviation of "C" effect.
	150	23	This data set is generated for the analysis of pipe material effect.
Q (m3/s)	0.242	0.045	This data set is the demand values for the design with lifetime of 15 years.
	0.377	0.070	This data set is the demand values for the design with lifetime of 30 years.

Two different approaches were used such as Head Approach (From H approach) and Demand Approach (From Q Approach) in the reliability computations. In the head approach, the reliability of a node (nodal reliability) was computed by obtaining the probability that available head (supplied head) H_s, is equal or greater than the minimum required head $H_{min.}$ In the demand approach, the reliability of a node (nodal reliability) was computed by taking the ratio of consumption (available flow) Q_a to the required flow $Q_{r.}$

The main difference between these approaches is that, the reliability from H Approach is computed by comparing H_s with H_{min}. If H_s is equal of greater than H_{min} at a node its nodal reliability becomes "1" otherwise "0". However, the reliability from Q approach takes the ratio of consumption (available flow) Q_a to the required flow Q_r into account. The nodal reliability from Q approach could take any value between “0” and “1”. This means although the nodal demand at a node is “0” from H approach; this value could be any value less than “1” from Q approach. As a result, the reliability from Q approach is always equal or greater than the reliability from H approach.

The input files were prepared as text files from the alternative designs which were obtained in optimization part and the reliabilities were calculated by using HapMam. In this study the reliabilities of the design alternatives were calculated both from H and Q approaches as mentioned in the reliability chapter.

4.5.1. Changing The Minimum Required Pressures (H_{min})

The minimum required pressures were changed from 10 m to 25 m in order to get alternative designs in the optimization part. In this part, the reliabilities of the obtained designs were calculated for 15 years and 30 years designs and the following data used in forming 500 Q and C sets (Table 4.15).

Table 4.15: Input Data for Reliability analysis in changing the minimum required Pressures (H_{min})

Life time of the design	15 years	30 years
Q_{design} (m^3/s)	0.242	0.378
Standard deviation of Q_{design} (m^3/s)	0.045	0.070
H_{min} (m)	25	25
HW Coefficient (C)	130	130,150
Standard deviation of HW Coefficient (C)	20	20,23
The Static Head at Reservoir (m)	136	136
The Static Head at Tank (m)	3.75	3.75

The reliability results were shown on Tables 4.16 and 4.17.

(Economical life of) 15 years design:

Table 4.16: The reliabilities of the design alternatives for 15 years design

C	St	H_{min} (m)	Cost (YTL)	Reliability from Q	Reliability from H
150	23	10	1,470,680.80	96.17%	72.41%
150	23	15	1,485,370.10	97.32%	81.46%
150	23	17	1,530,376.00	98.60%	87.15%
150	23	18	1,530,376.00	98.60%	87.15%
150	23	20	1,594,432.80	99.09%	89.95%
150	23	21	1,609,917.00	99.24%	91.15%
150	23	22	1,694,614.80	99.48%	93.09%
150	23	23	1,789,354.30	99.63%	94.67%
150	23	24	1,917,848.60	99.76%	96.50%
150	23	25	2,074,426.30	99.84%	97.88%

(Economical life of) 30 years design:

Table 4.17: The reliabilities of the design alternatives for 30 years design

C	St	H_{min} (m)	Cost (YTL)	Reliability from Q	Reliability from H
130	20	10	1,019,441.50	95.24%	73.07%
130	20	12	1,024,852.30	95.78%	77.63%
130	20	13	1,036,523.70	96.15%	78.89%
130	20	14	1,038,230.40	97.33%	80.83%
130	20	16	1,043,641.20	97.71%	84.23%
130	20	17	1,047,528.90	97.84%	84.76%
130	20	18	1,066,259.60	98.59%	88.26%
130	20	19	1,087,062.40	98.71%	88.81%
130	20	20	1,116,227.40	99.14%	91.93%
130	20	21	1,163,582.90	99.40%	94.84%
130	20	22	1,194,078.50	99.51%	95.46%
130	20	23	1,265,237.00	99.89%	97.24%
130	20	24	1,307,996.80	99.95%	98.30%
130	20	25	1,343,101.40	99.99%	99.49%
ORIGINAL NETWORK (25m)			1,083,028.01	96.38%	81.02%

As stated before, the authority should decide on the life time of the design whether 15 years or 30 years by considering all aspects such as economy, reliability etc.

In the optimization part, it was seen that 15 years design was more economic than 30 years design. The change in the cost was between 14% - 31% for the alternatives. However, this was the construction cost of the network (only including pipe costs), the other cost items may affect these results. Additionally, it is clear that after the life time of the 15 years design, the operational cost would be higher than the 30 years design.

The reliability differences both H and Q approaches were negligible as can be seen on Table 4.18 and on Figure 4.11 and 12. This means that, cost evaluation had more importance in the determination of the life time of the network.

Table 4.18: The comparison of the reliabilities with economical lifes of 15 years and 30 years

% Change of 15 years design values with respect to 30 years design values			
H_{min} (m)	Cost	Reliability from Q	Reliability from H
10	-14.03%	1.21%	5.32%
14	-15.57%	-0.38%	1.18%
16	-14.99%	-0.54%	-1.86%
17	-12.54%	0.97%	7.31%
18	-14.14%	-0.19%	-0.67%
20	-16.93%	0.00%	-3.24%
21	-19.54%	0.06%	-3.45%
23	-26.91%	-0.27%	-3.63%
24	-29.01%	-0.19%	-2.66%
25	-31.21%	-0.09%	-1.56%

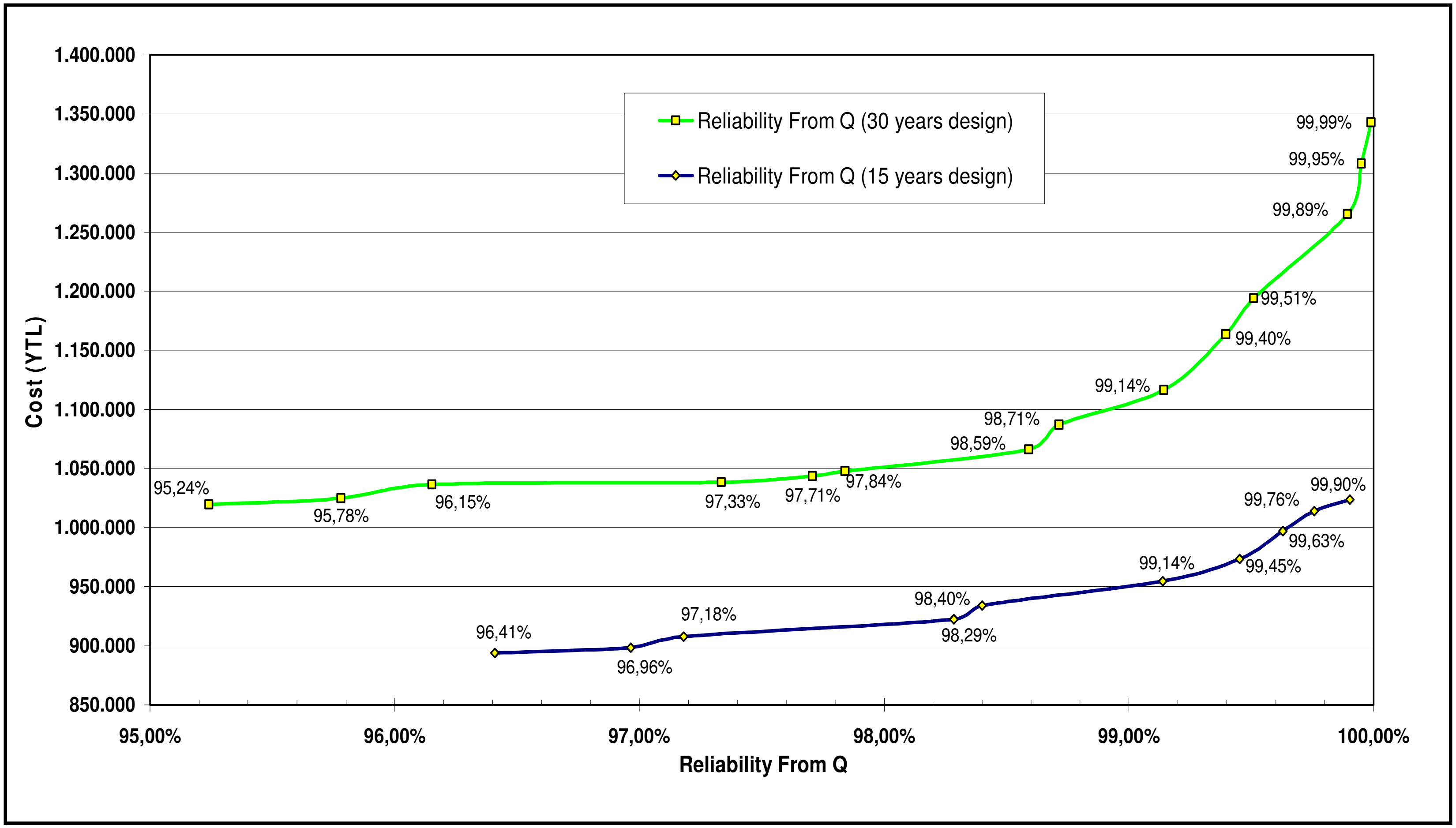

Figure 4.11: The Costs vs. Reliabilities (from Q Approach) for the the design alternatives with ductile iron alternatives with economical lifes of 15 years and 30 years

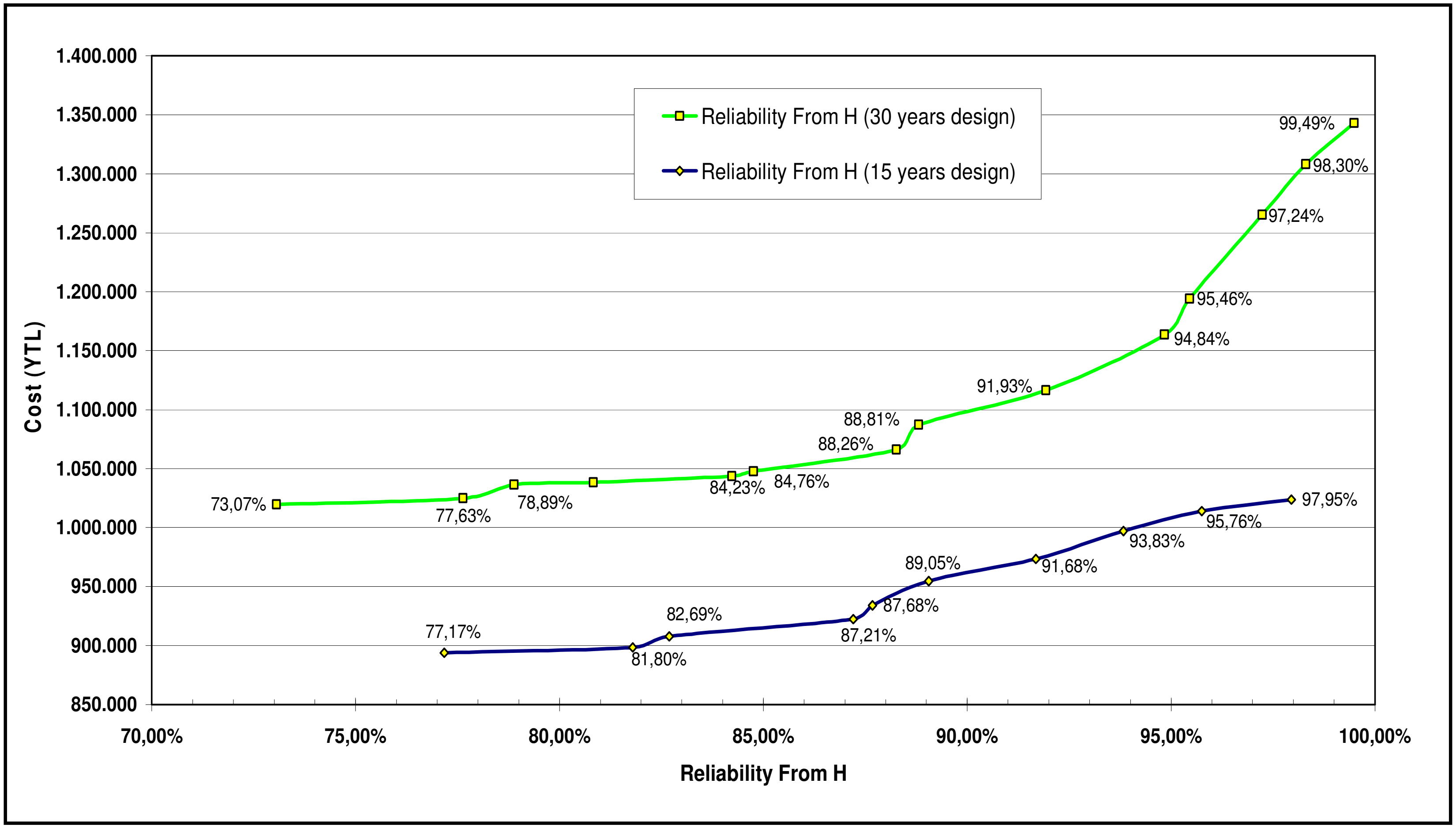

Figure 4.12: The Costs vs Reliabilities (from H Approach) for the the design alternatives with ductile iron alternatives with economical lifes of 15 years and 30 years

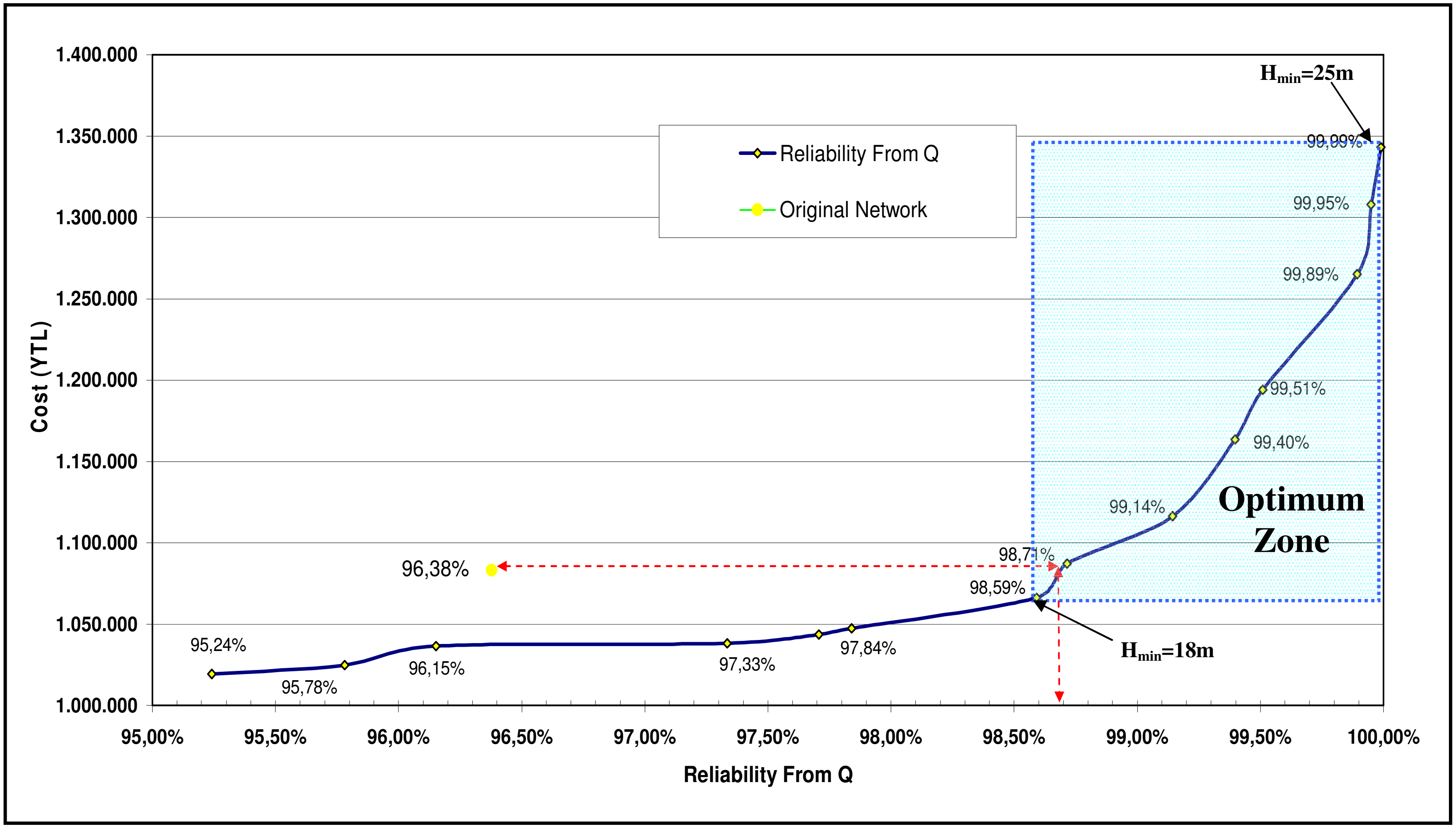

Figure 4.13: The Costs vs. Hydraulic system Reliabilities (from Q Approach) for the design alternatives for 30 years design and original network

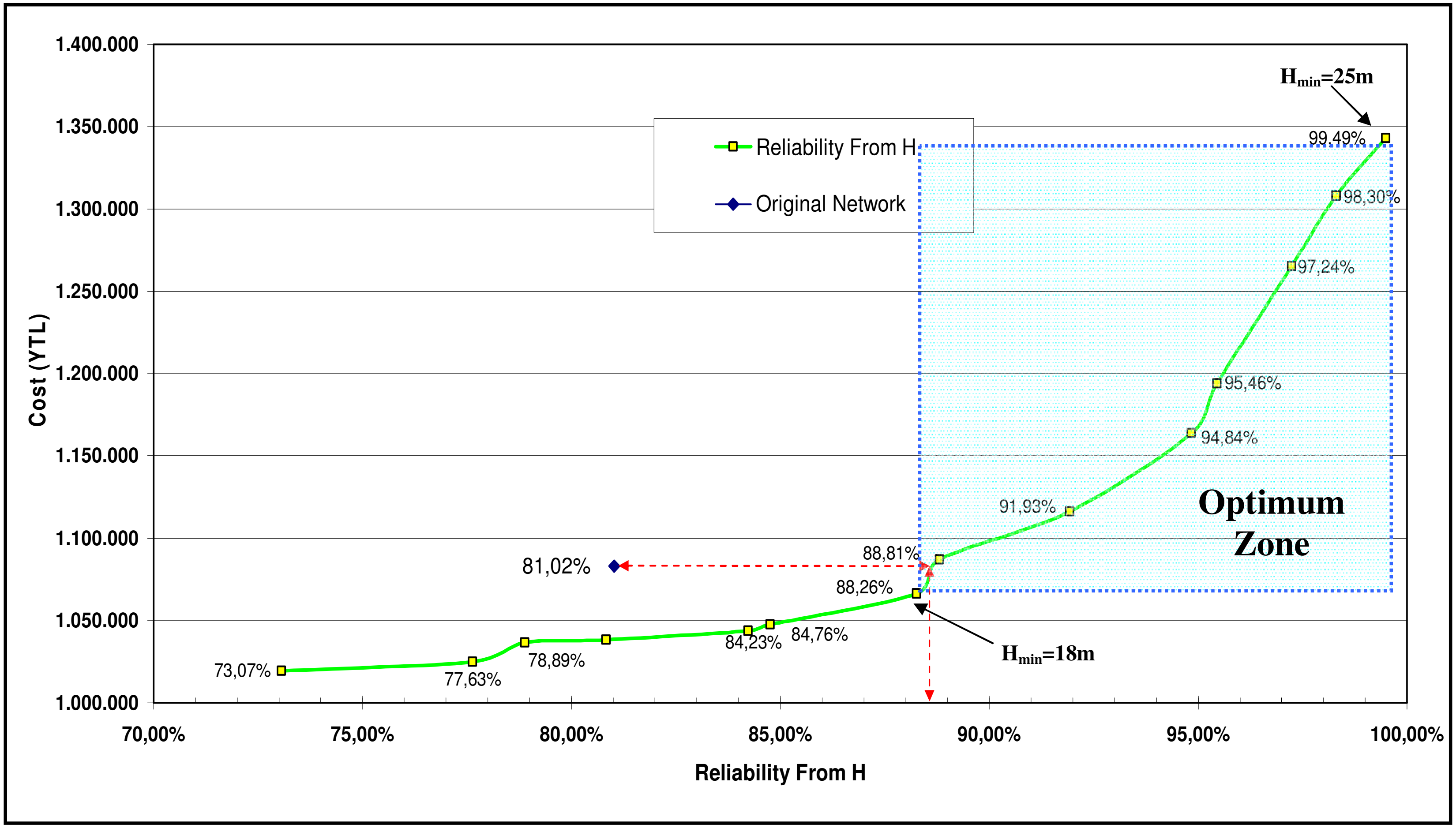

Figure 4.14: The Costs vs. Hydraulic system Reliabilities (from H Approach) for the design alternatives for 30 years design and original network

4.5.1.1 Evaluation of the 30 years design alternative

In this study, the economical life (lifetime) of the network was chosen as 30 years since the existing network was designed to serve for 30 years. The pipe sizes of the existing network were also examined and the cost of the existing network was calculated by using ductile iron pipe price function. Moreover, the reliabilities of the existing network were obtained by the same analysis, which was performed with 30 years design alternatives (Table 4.17).

A new question, which has to be answered by the designer, is that which alternative design was the most appropriate one for the network. In fact, there is more than one answer for this question.

(1) If cost has more priority than reliability, then as can be seen on Table 4.19 and Figures 4.13 and 4.14, with the same reliability values with the existing network, the cost could be decreased up to 4-5% (H_{min}= 13 or 14 m alternatives).

(2) If the cost is determined i.e. the budget is definite, then the reliability of the network could be increased. The reliability from Q approach increases about 2-3 % whereas the reliability from H approach increases almost 9-10%. This case could be clearly seen both in Table 4.19 and in Figures 4.13 and 4.14.

In this study, there was a chance to compare the existing network with the new design alternatives. However, this study also aims to help the engineer/designer in the determination part of a new water distribution network design. In that case, the comparison should be performed between the design alternatives without any existing network (reference). The following comments are proposed for this case:

(1) The first way is the find the minimum slope of the cost vs. reliability curve (Table 4.20).

(2) The second way was the comparison of the alternatives with the alternative that gave the highest reliability result and normally the most expensive design (Table 4.21)

Table 4.19: Comparison of the alternative designs with the original network

% Changes with respect to original network			
H_{min} (m)	Cost (YTL)	Reliability from Q	Reliability from H
10	-5.87%	-1.18%	-9.82%
12	-5.37%	-0.62%	-4.18%
13	-4.29%	-0.23%	-2.64%
14	-4.14%	0.99%	-0.24%
16	-3.64%	1.38%	3.95%
17	-3.28%	1.52%	4.61%
18	-1.55%	2.30%	8.94%
19	0.37%	2.42%	9.61%
20	3.07%	2.87%	13.46%
21	7.44%	3.13%	17.05%
22	10.25%	3.25%	17.81%
23	16.82%	3.65%	20.01%
24	20.77%	3.71%	21.32%
25	24.01%	3.75%	22.78%

By the aid of first way, it was clearly seen that the slopes of the section 3, 4, 6 and 8 on Table 4.20 were less than the neighboring sections. Then, the alternative designs in these sections (H_{min}= 13m, 14m, 16m, 17m, 18m and 19m) were analyzed with night loading conditions. The existence of storage tanks at all buildings in the pressure zone gave us the chance to test the network under night loading conditions since the storage tanks at the buildings would be an extra water source during peak hour loadings besides the network itself . By this way, it was seen that the alternative network designs with $H_{min} \geq 18$m, were able to satisfy pressure heads equal or greater than 25m at all nodes during night loading condition.

This result shows that, there is no need to increase the cost of design after entering the optimum zone part (Figure 4.13 and 14).

The network design which has the smallest reliability and minimum cost in the optimum zone (H_{min}=18m) could be suggested as the design alternative to the authority however all the alternatives in the optimum zone should be evaluated in detail before making this decision.

Table 4.20: Slope of Cost vs. Reliability Curves From H and Q

Section	H_{min} (m)		δ(Cost) /δ (Reliability From Q) (1/YTL)	δ(Cost) /δ (Reliability From H) (1/YTL)
1	10	12	1,003,885	118,438
2	12	13	3,133,718	931,019
3	13	14	**144,377**	**87,780**
4	14	16	**1,454,507**	**159,335**
5	16	17	2,908,242	735,968
6	17	18	**2,494,260**	**533,904**
7	18	19	16,874,498	3,790,869
8	19	20	**6,810,746**	**935,198**
9	20	21	18,612,345	1,627,782
10	21	22	27,025,680	4,955,694
11	22	23	18,517,367	3,988,350
12	23	24	75,960,713	4,029,486
13	24	25	88,773,907	2,966,699

In this case study, section 8 i.e the design with H_{min}=18 m had some advantageous from cost point of view. It had almost the same cost with the existing system and about 20% less than the H_{min}=25m design. However, the reliabilities of this alternative was 1-2% and 11-12% less than the H_{min}=25m design alternative from Q and H approaches respectively. If this difference could be tolerated by the authority, this alternative may be selected.

4.5.2 The effect of Standard Deviation of Hazen-Williams roughness coefficient (C) change on reliability

The standard deviation of the Hazen-Williams roughness coefficient was changed from 10 to 60 for the design with H_{min}=25 m, to get the effects of this parameter. In this part, the reliabilities of the H_{min}=25 m were calculated and the results were shown in the Figures 4.15- 4.16 and on Table 4.22.

Table 4.21: Comparison of the alternative designs with design of H_{min}=25m

% Changes with respect to H_{min}=25m design			
H_{min} (m)	Cost (YTL)	Reliability from Q	Reliability from H
ORIGINAL NETWORK (25m)	-19.36%	-3.61%	-18.56%
10	-24.10%	-4.75%	-26.56%
12	-23.70%	-4.21%	-21.96%
13	-22.83%	-3.84%	-20.70%
14	-22.70%	-2.66%	-18.75%
16	-22.30%	-2.28%	-15.34%
17	-22.01%	-2.15%	-14.81%
18	-20.61%	-1.40%	-11.28%
19	-19.06%	-1.28%	-10.73%
20	-16.89%	-0.85%	-7.59%
21	-13.37%	-0.59%	-4.67%
22	-11.10%	-0.48%	-4.05%
23	-5.80%	-0.10%	-2.26%
24	-2.61%	-0.04%	-1.19%

Table 4.22: The reliabilities of the design alternatives with changing standard deviation of Hazen-Williams Coefficient (C)

C	H_{min}	Cost (YTL)	St	Reliability from Q	Reliability from H
130	25 m	1,343,101.40	10	99.998%	99.753%
			15	99.996%	99.669%
			20	99.990%	99.485%
			25	99.976%	99.364%
			30	99.953%	99.166%
			35	99.899%	98.740%
			40	99.862%	98.378%
			45	99.841%	98.185%
			50	99.832%	98.065%
			55	99.742%	97.472%
			60	99.724%	97.427%

4.5.2.1 Evaluation of the Change in the Standard Deviation of Hazen Williams roughness coefficient (C)

The need for this analysis could be explained such that, the Hazen-Williams roughness coefficient is an indicator of the problems of the network. Besides, its service in the Hazen-Williams flow equation, any increase in the standard deviation of this parameter gave us some clues about the problems of the network.

As an example, when the nodal demands were not appropriate with real operating conditions i.e. some nodes needs more water or some nodes do not need the defined amounts, there would be hydraulic problems. Moreover, when there is a forgotten valve or isolated nodes, the standard deviation of this coefficient again increases. In order to avoid the increase of the standard deviation of this coefficient through the operating years, careful maintenance and operation should be performed. As a result, the standard deviation of Hazen-Williams roughness coefficient gives us chance for interpreting operational conditions and periods of maintenance.

It can be easily seen that the reliabilities from both approaches were decreased with increasing standard deviation of Hazen-Williams roughness coefficient (Table 4.22). In this part, the examined alternative was the most expensive one, which had the highest reliabilities (H_{min}=25m). However, by this study, it could be concluded that, if any other alternative designs was selected as the network design and special care was given to the operational and maintenance process of the network, the final reliabilities would be almost same with this example.

In other words, the authority should consider both constructional and operational costs together in the determination of the design. The difference between the construction costs of alternative designs gives chance to the authority for using that amount as an extra budget for the operational cost of the low-cost design.

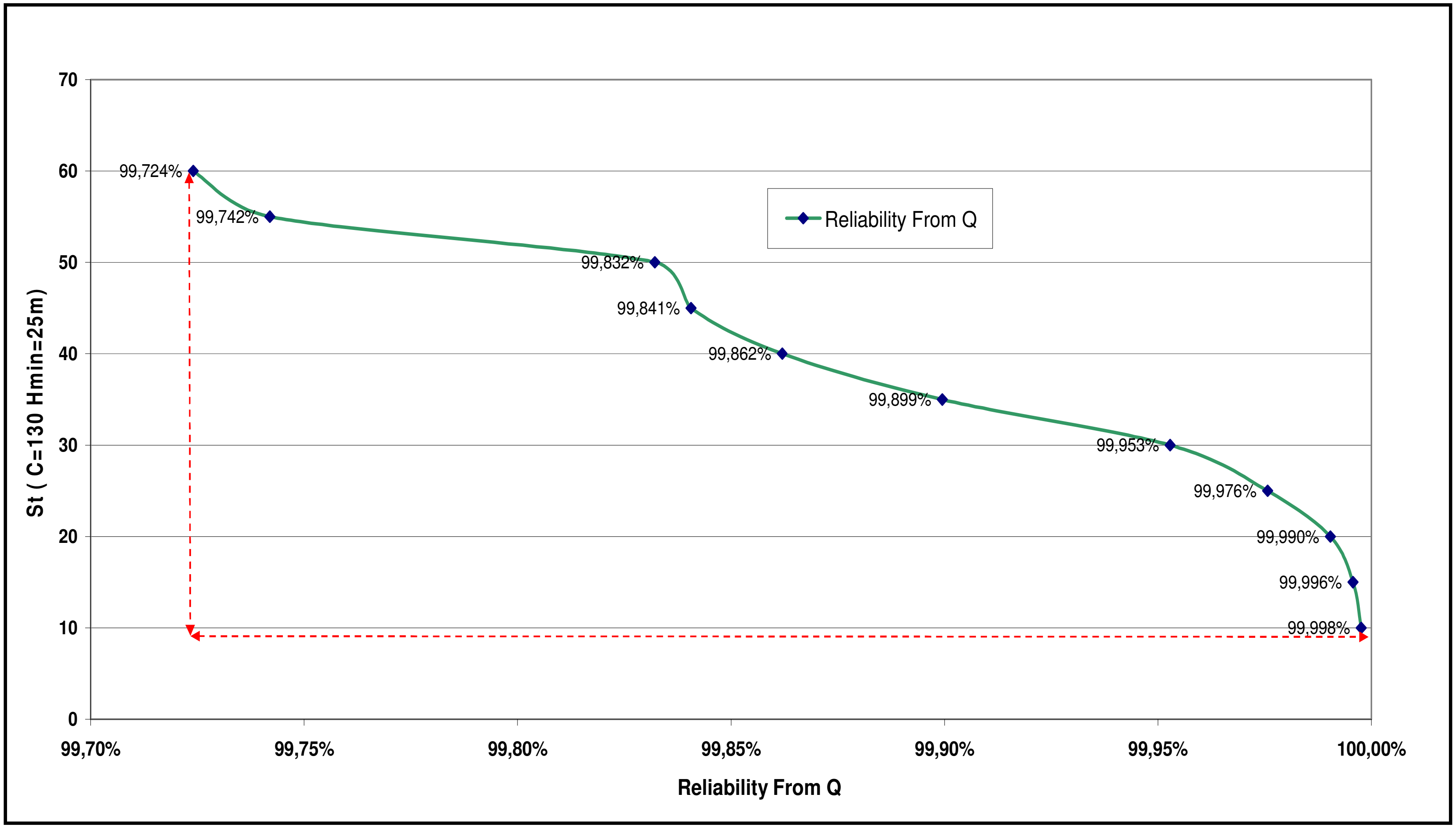

Figure 4.15: The change of reliability (from Q Approach) with changing standard deviation of Hazen-Williams roughness coefficient

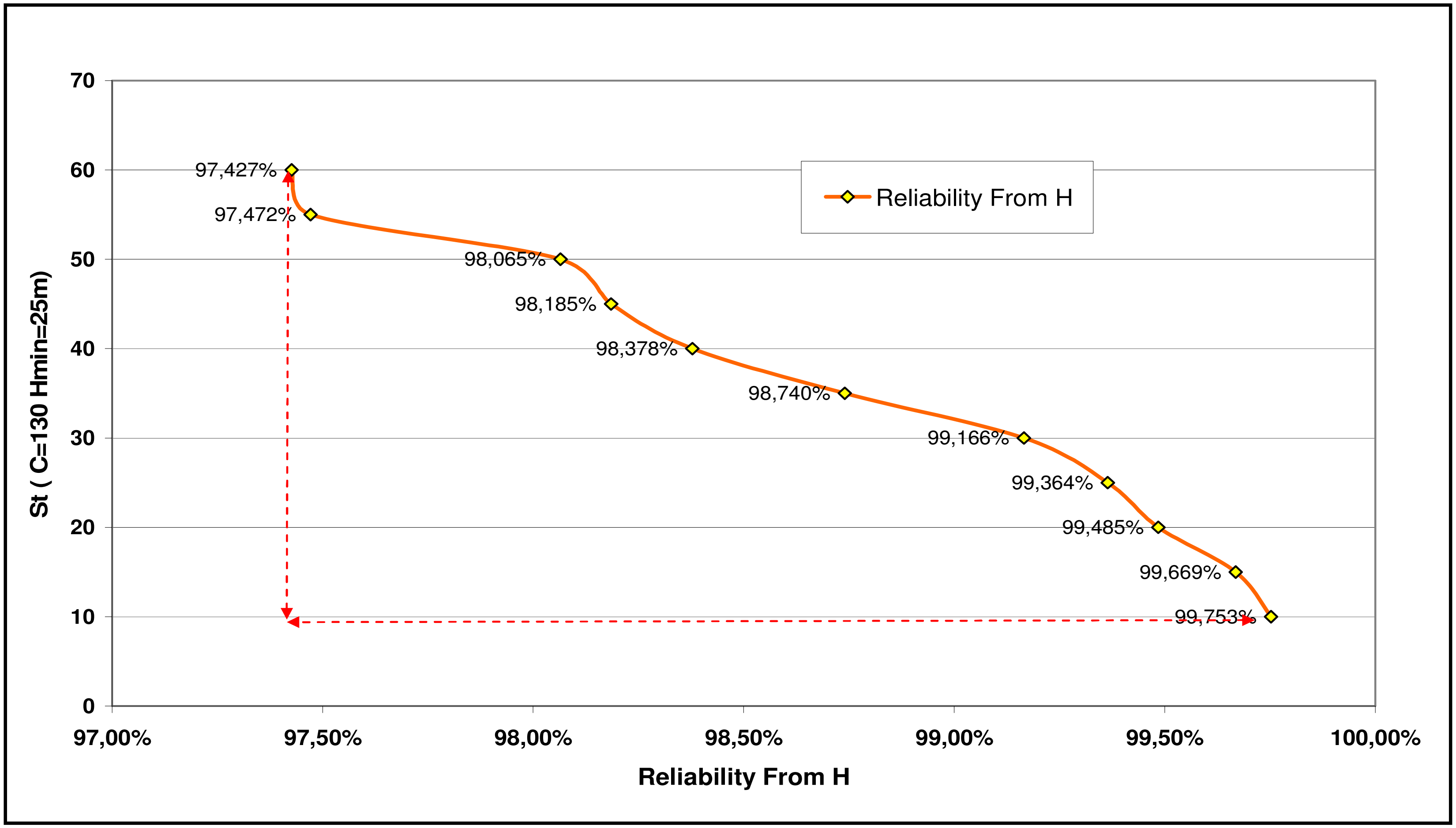

Figure 4.16: The change of reliability (from H Approach) with changing standard deviation of Hazen-Williams roughness coefficient

4.5.3 Changing Pipe Material

In this part, the reliabilities of the networks with alternative pipe materials are calculated (Table 4.23 and Figure 4.17 and 4.18). The comparison of the costs of the alternative designs was performed in the optimization part. Since, the prices of HDPE pipes were generally more than the prices of ductile iron pipes, selection of ductile iron pipes as the pipe material was more economic.

However; HDPE pipes have some advantageous which are given by manufacturers compared with other pipe materials such as:

- Low operating costs
- 50 years of service life guarantee
- High resistance to chemical agents
- High resistance to cracking and impact
- High elasticity and minimum fitting usage
- Light weight, easy transportation etc…

Table 4.23: The reliabilities of the design alternatives with changing standard deviation of Hazen-Williams Coefficient

H_{min} (m)	C=130 DUCTILE IRON PIPES			C=150 HDPE PIPES		
	Cost (YTL)	Reliability from Q	Reliability from H	Cost (YTL)	Reliability from Q	Reliability from H
10	1,019,441.50	95.24%	73.07%	1,470,680.80	96.17%	72.41%
15	1,038,230.40	97.33%	80.83%	1,485,370.10	97.32%	81.46%
17	1,047,528.90	97.84%	84.76%	1,530,376.00	98.60%	87.15%
18	1,066,259.60	98.59%	88.26%	1,530,376.00	98.60%	87.15%
20	1,116,227.40	99.14%	91.93%	1,594,432.80	99.09%	89.95%
21	1,163,582.90	99.40%	94.84%	1,609,917.00	99.24%	91.15%
22	1,194,078.50	99.51%	95.46%	1,694,614.80	99.48%	93.09%
23	1,265,237.00	99.89%	97.24%	1,789,354.30	99.63%	94.67%
24	1,307,996.80	99.95%	98.30%	1,917,848.60	99.76%	96.50%
25	1,343,101.40	99.99%	99.49%	2,074,426.30	99.84%	97.88%

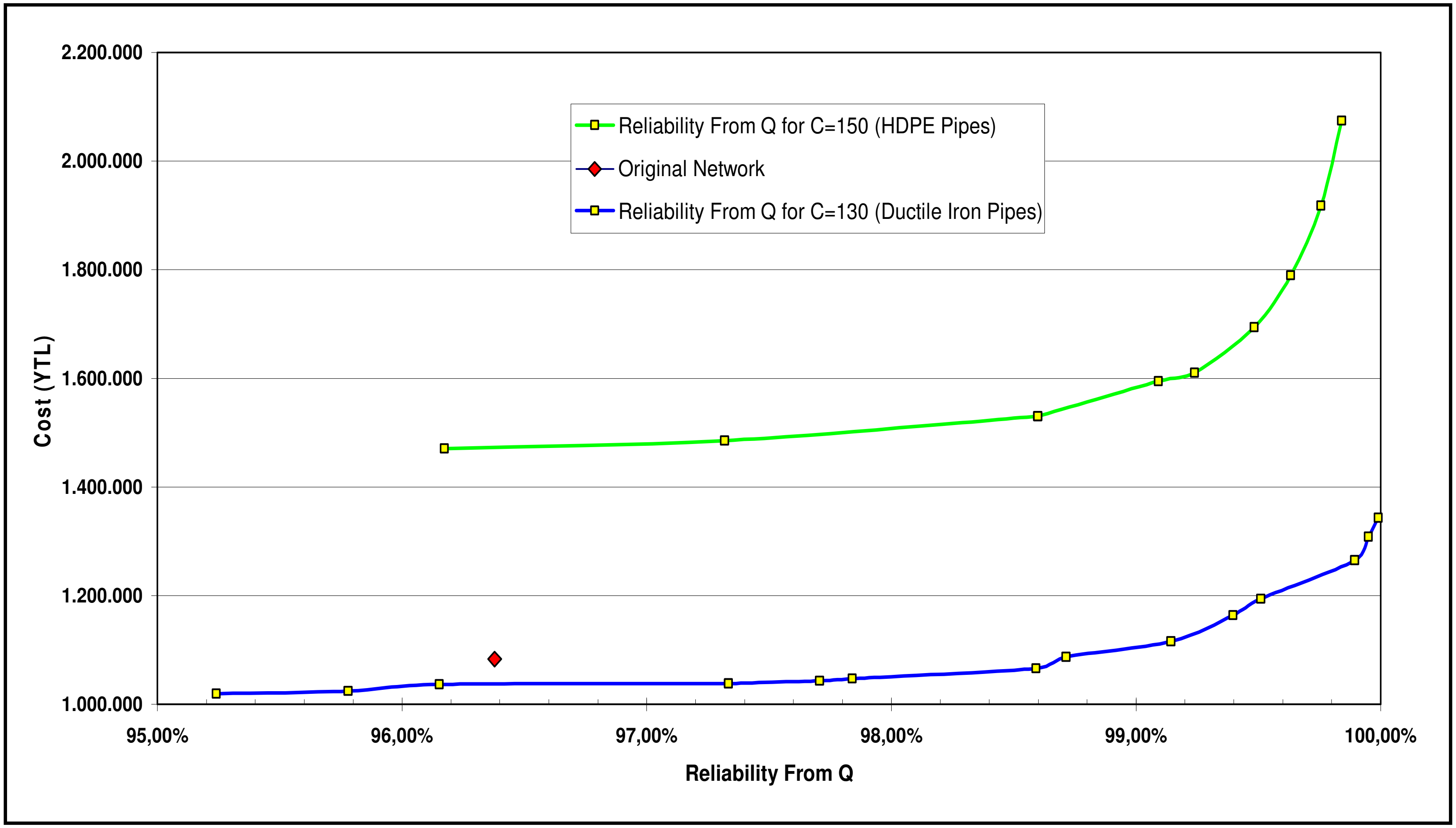

Figure 4.17: The change of reliability (from Q Approach) with changing pipe materials

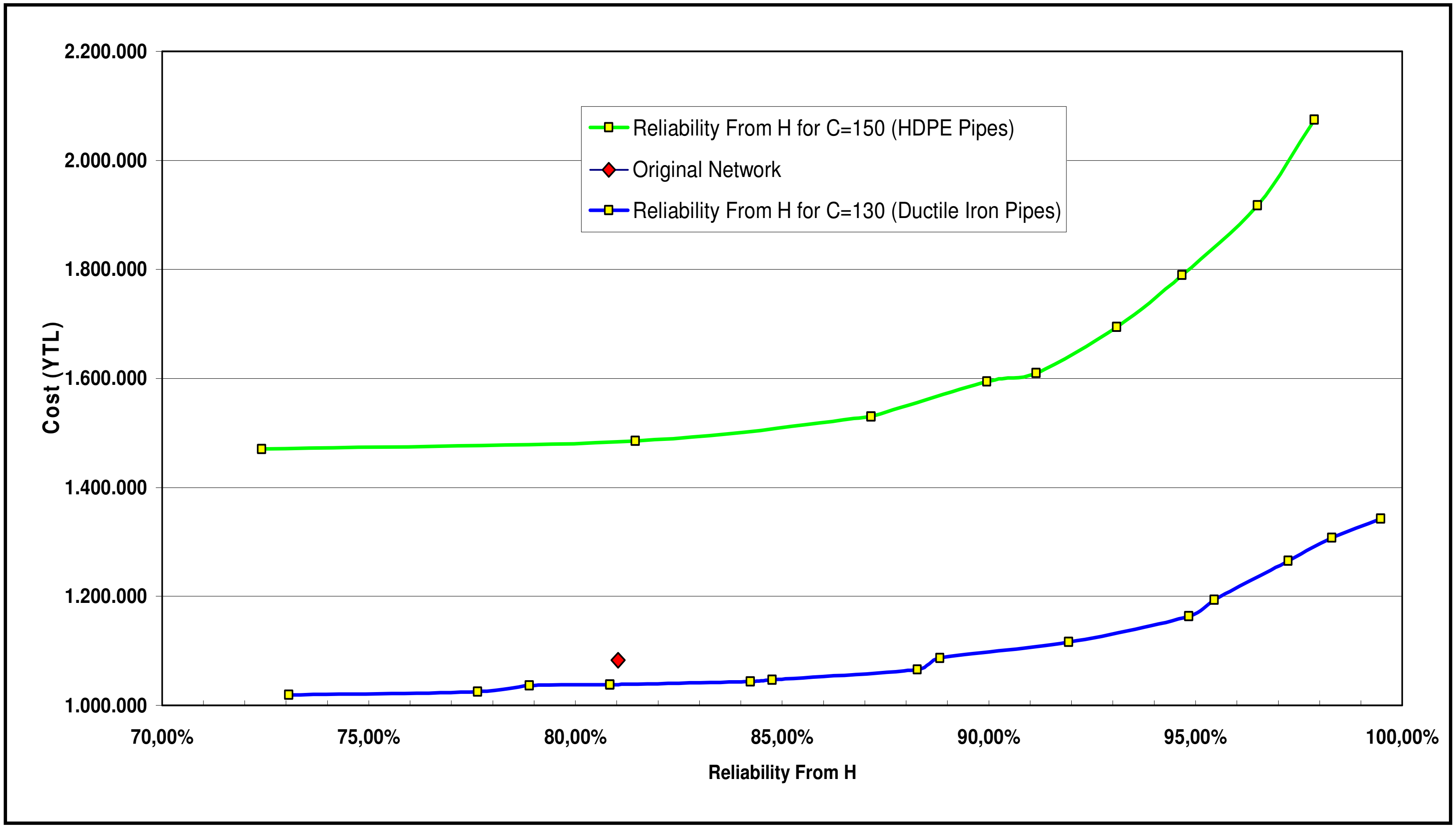

Figure 4.18: The change of reliability (from H Approach) with changing pipe materials

4.5.3.1 Evaluation of changing pipe material

As stated before, optimization of the pipe sizes of a water distribution network is very sensitive to the price functions. It is clear that, the price functions depend on the type of pipe material. So that, in order to obtain an economical design for a water distribution network, price functions should be determined with a very detail work. It depends on many factors such as the sources of the country, the supply and demand ratio for the item etc...

Moreover, the pipe characteristics should be considered in detail for the operational point of view. Pipe aging is a very good example for the effect of pipe characteristics, which was examined on the following section.

The reliabilities of the pipe materials were almost the same as can be seen on Table 4.24 whereas the cost of the network with HDPE pipe was about 40-50% more than the network with ductile iron pipes in all alternatives.

Table 4.24: The reliabilities of the design alternatives with changing pipe material

H_{min} (m)	% Changes between C=130 and C=150		
	Cost (YTL)	Reliability from Q	Reliability from H
10	44.26%	0.98%	-0.90%
15	43.07%	-0.02%	0.77%
17	46.09%	0.77%	2.83%
18	43.53%	0.01%	-1.26%
20	42.84%	-0.05%	-2.15%
21	38.36%	-0.16%	-3.90%
22	41.92%	-0.03%	-2.48%
23	41.42%	-0.26%	-2.64%
24	46.62%	-0.20%	-1.83%
25	54.45%	-0.15%	-1.62%

4.5.4 Different Valve Status (Open/Closed)

As stated before, the mechanical failure was examined in this part by closing control and /or isolating valves of some of the pipes in the network. This is the only case where mechanical failure was taken into consideration in this study. The valves are used for the isolation of the pipes, which are broken. Therefore, maintenance and repair of the network is needed.

There were 215 pipes left in N8-1 after performing skeletonization and further modification. In the study of hydraulic model building process of Ankara Water Distribution Network (N8-3), it was seen that 14 of 42 valves were closed or more than half-closed (Merzi et.al, 1998). By the help of that study and decreasing the ratio of closed or more than half-closed valves to the all valves, 40 valves at 40 pipes were assumed to be closed as can be seen on Table B.2 and Figure 4.19. The pipes were selected randomly provided that every node in the network could obtain water. This is because; HapMam do not allow any node remaining separately at the network. The reliabilities of the networks with 40 valves closed were calculated in each alternative design (Table 4.25 and Figure 4.20 and 4.21). It can be seen that the reliabilities from H Approach was more affected than the reliabilities from Q Approach.

Table 4.25: The reliabilities of the design alternatives
(40 valves closed and no valves closed)

H_{min} (m)	Cost (YTL)	Reliability from Q 40 valves closed	Reliability from H 40 valves closed	Reliability from Q no valve closed	Reliability from H no valve closed
16	1,043,641.20	92.66%	75.10%	97.71%	84.23%
17	1,047,528.90	92.99%	75.91%	97.84%	84.76%
19	1,087,062.40	95.74%	80.61%	98.71%	88.81%
20	1,116,227.40	94.21%	79.13%	99.14%	91.93%
21	1,163,582.90	94.20%	79.19%	99.40%	94.84%
22	1,194,078.50	97.51%	84.46%	99.51%	95.46%
23	1,265,237.00	98.72%	89.03%	99.89%	97.24%
24	1,307,996.80	98.76%	89.28%	99.95%	98.30%
25	1,343,101.40	99.54%	92.95%	99.99%	99.49%
ORIGINAL NETWORK (25m)	1,083,028.01	92.64%	72.13%	96.38%	81.02%

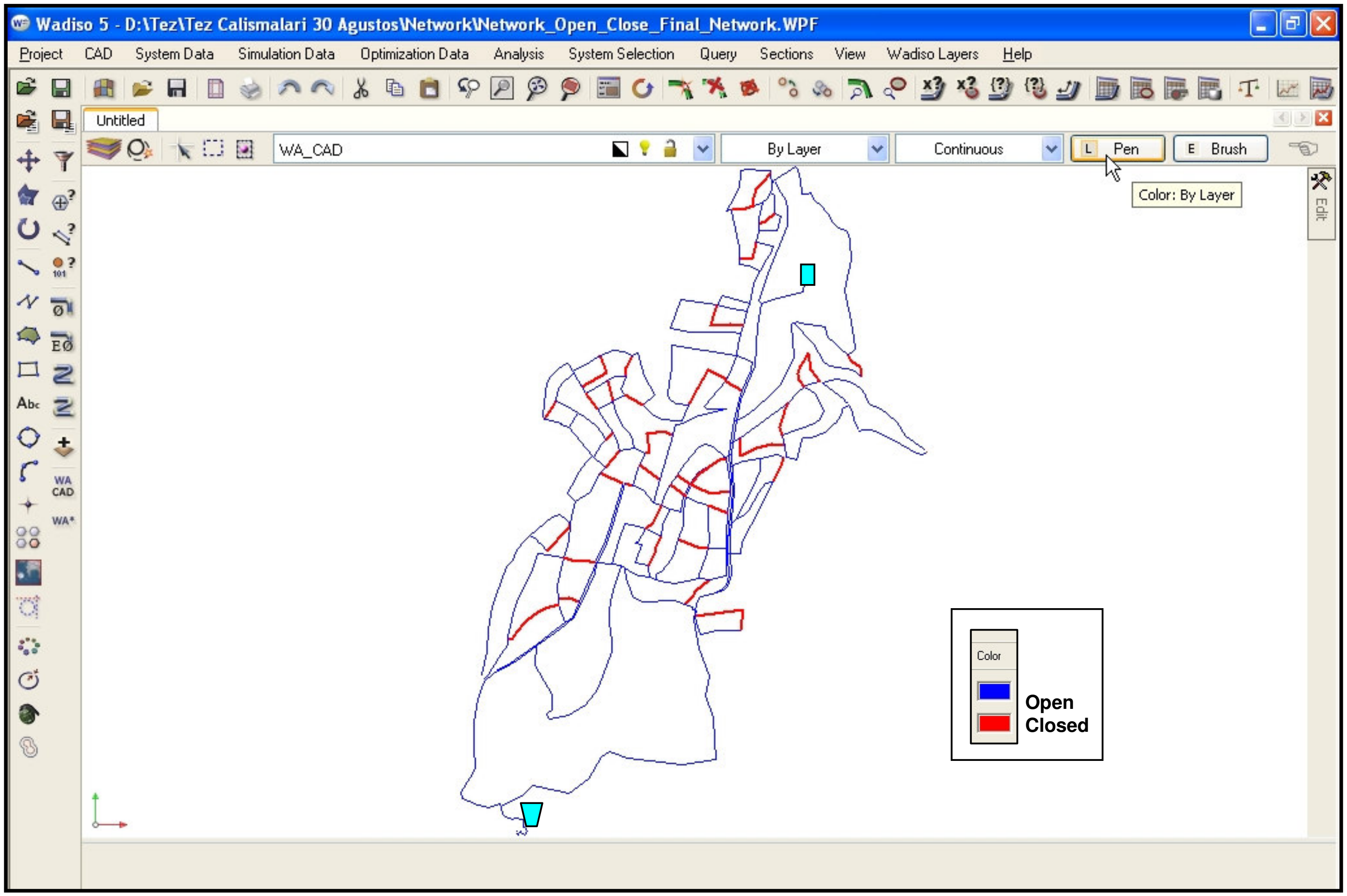

Figure 4.19: The open/closed pipes on the network

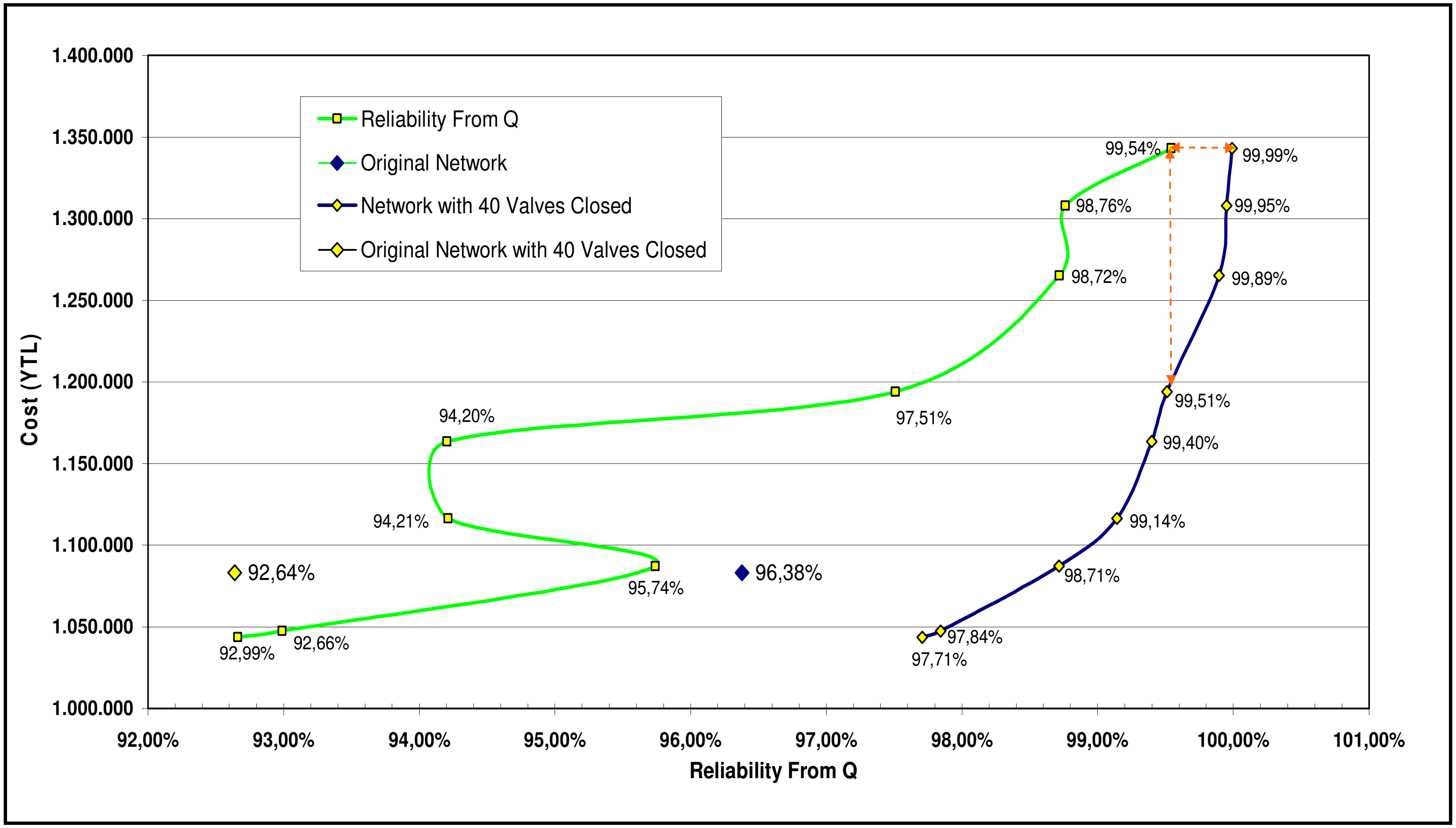

Figure 4.20: The change of reliability (from Q Approach) with changing pipe materials

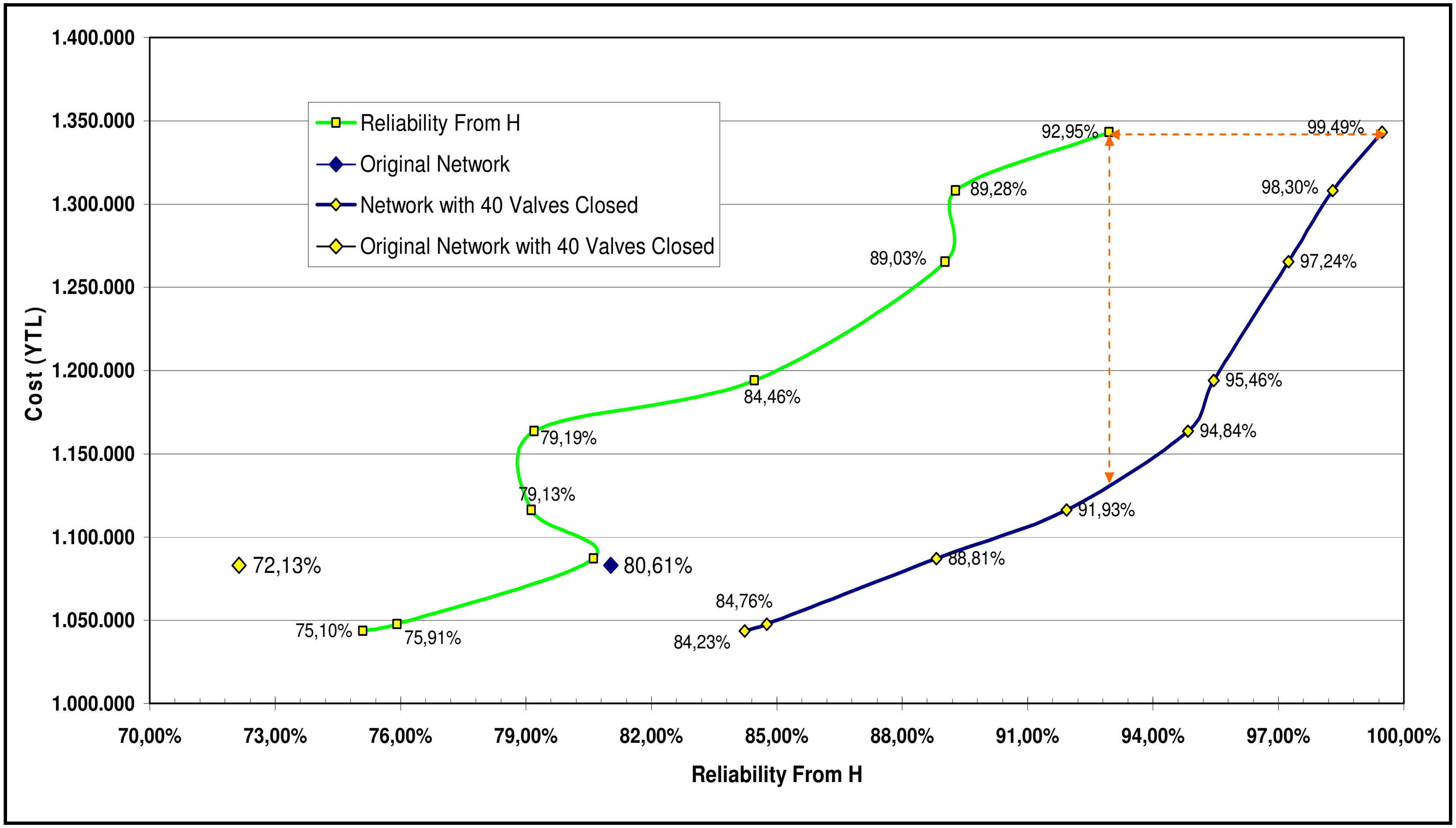

Figure 4.21: The change of reliability (from H Approach) with changing pipe materials

4.5.4.1 Evaluation of different valve status (open/closed)

The installation of valves is very important in the operation of a water distribution network. The area where maintenance and / or repair were needed, could be easily isolated from the network. This could be achieved if and only if the valves are installed to the network properly. The results in this study show that:

(1) As stated before, the reliabilities from H Approach were affected more than the reliabilities calculated from Q Approach. This is normal since the pipes, which were isolated from network, were selected such a way that none of the nodes were separated from network. In other words, all the nodes in the network could obtain water even the pressures were very low at some of the nodes. As a result, the reliabilities from Q approach were greater than the reliabilities from H Approach at the nodes where the supplied pressures were very low.

(2) The reliabilities of all alternative designs were decreased by isolation of 40 pipes among 215 pipes in the network. In addition, the changes in the reliability were generally more in the design alternatives having low minimum required pressures (H_{min}) values compared with the other design alternatives (Table 4.26 and Figures 4.20 and 21). This result comes from that the pipe sizes were larger in the design alternatives with greater H_{min} values. Even the same pipes were isolated in all design alternatives, the remaining pipe sizes, which were also larger in the design alternatives with greater H_{min} values, allowed the water to reach the nodes more efficiently. However, the design alternative with H_{min}=19 m did not obey this statement. This situation could be explained by the inappropriate selection of the pipes, which were isolated, was performed randomly.

(3) In the selection of the design alternatives, not only should the construction cost be considered but also the operational, maintenance and repair costs should be taken into account. As an example, the reliabilities of design

alternative with H_{min}=25m, would decrease 0.45 % and 6.57 % from Q and H approaches respectively (Table 4.26). These amount seems to be unimportant however; if the design alternatives having low initial reliabilities i.e initial reliabilities corresponding to the final reliabilities of H_{min}=25 m alternative, the construction cost (pipe cost) would decrease considerably (11 % and 15% from Q and H Approaches respectively). These savings could be evaluated for the operation, maintenance and repair of the network properly.

Table 4.26: The changes of reliability between 40 valves closed and no valves closed design alternatives

% Changes with respect to no valve closed		
H_{min} (m)	Reliability from Q	Reliability from H
16	-5.16%	-10.84%
17	-4.96%	-10.43%
19	-3.01%	-9.23%
20	-4.97%	-13.93%
21	-5.22%	-16.50%
22	-2.01%	-11.52%
23	-1.18%	-8.44%
24	-1.19%	-9.18%
25	-0.45%	-6.57%
ORIGINAL NETWORK (25m)	-3.88%	-10.97%

In addition, the changes in the reliabilities of the design alternatives with 40 valves were listed on Table 4.27. In this table, it could be seen that the reliabilities of the original network decreased more than the design alternatives. However, this should be evaluated as a unique result because these results depend on the selection of the isolated pipes. In any other selection, the results could be different provided that all the reliabilities decrease.

Table 4.27: The changes of reliability between design alternatives with 40 valves closed with respect to H_{min}=25 design

% Changes with respect to H_{min}=25m design with 40 valves closed		
H_{min} (m)	Reliability from Q with 40 valves closed	Reliability from H with 40 valves closed
16	-6.91%	-19.21%
17	-6.58%	-18.33%
19	-3.82%	-13.27%
20	-5.35%	-14.87%
21	-5.36%	-14.80%
22	-2.04%	-9.13%
23	-0.83%	-4.21%
24	-0.79%	-3.95%
ORIGINAL NETWORK (25m)	-6.94%	-22.40%

4.5.5 Pipe Aging Effect

After deciding on the lifetime of the network, pipe aging analysis should be performed for the design alternatives. The Hazen-Williams Coefficients is taken from the web site of a pipe manufacturer firm (Istec Ingenieria). The same values were used for ductile iron pipes. It is seen hat the coefficient decreases in time however the lower and the upper limits of the intervals shows us the importance of the maintenance. If the network is operated and maintained carefully and periodically, the rate of decrease is also decreases.

In this study, the average values were used and the related standard deviations are listed on Table 4.28. The reliability results are shown on Tables 4.29 and 4.30 and in Figures 4.22 and 4.23.

Table 4.28: Table of Hazen-Williams Coefficients taken from Istec Ingenieria web site
(http://www.istec.com.uy/eng/calculations/HazenWilliamCoefficients.asp)
Last accessed date: November 2006

Table of Hazen-Williams Coefficients			
Time	**Coefficient**	**Average**	**St**
new	130	130	20
10 years	107 - 113	110	17
20 years	89 - 100	95	15
30 years	75 - 90	83	13
40 years	64 - 83	74	11

Table 4.29: The Reliability From Q Change with years

H_{min} (m)	Cost (YTL)	C=74 St=11 40 years later	C=83 St=13 30 years later	C=95 St=15 20 years later	C=110St=17 10 years later	C=130 St=20 New
		Reliability from Q	Reliability from Q	Reliability from Q	Reliability from Q	Reliability from Q
16	1,043,641.20	88.13%	90.61%	93.74%	96.05%	97.71%
17	1,047,528.90	88.67%	91.00%	94.03%	96.23%	97.84%
18	1,066,259.60	91.65%	93.45%	95.83%	97.42%	98.59%
19	1,087,062.40	92.16%	93.90%	96.15%	97.63%	98.71%
20	1,116,227.40	93.94%	95.35%	97.14%	98.33%	99.14%
21	1,163,582.90	94.59%	95.96%	97.63%	98.69%	99.40%
22	1,194,078.50	95.11%	96.66%	97.97%	98.90%	99.51%
23	1,265,237.00	98.57%	99.12%	99.54%	99.77%	99.89%
24	1,307,996.80	98.85%	99.33%	99.67%	99.86%	99.95%
25	1,343,101.40	99.47%	99.71%	99.88%	99.96%	99.99%

Table 4.30: The Reliability From H Change with years

H_{min} (m)	Cost (YTL)	C=74 St=11 40 years later	C=83 St=13 30 years later	C=95 St=15 20 years later	C=110 St=17 10 years later	C=130 St=20 New
		Reliability from H	Reliability from H	Reliability from H	Reliability from H	Reliability from H
16	1,043,641.20	54.86%	62.46%	71.49%	78.63%	84.23%
17	1,047,528.90	60.15%	65.69%	73.14%	79.60%	84.76%
18	1,066,259.60	67.38%	72.06%	78.73%	84.01%	88.26%
19	1,087,062.40	69.04%	73.50%	79.89%	84.79%	88.81%
20	1,116,227.40	74.53%	78.78%	84.52%	88.49%	91.93%
21	1,163,582.90	78.08%	82.25%	87.59%	91.51%	94.84%
22	1,194,078.50	80.13%	84.73%	88.82%	92.35%	95.46%
23	1,265,237.00	84.92%	88.88%	92.79%	95.58%	97.24%
24	1,307,996.80	87.02%	90.71%	94.44%	96.71%	98.30%
25	1,343,101.40	93.18%	95.28%	97.33%	98.66%	99.49%

4.5.5.1 Evaluation of Pipe Aging

The pipes of the network would have some problems in time. However, it could be seen that when the design alternative H_{min}=25 m is chosen, the problems related with pipe aging are minimized. The decreases in the reliabilities are negligibly small (Table 4.31 and 4.32).

However, the decrease in the reliabilities of the low-cost alternatives are really more compared with H_{min}=25 m alternative results. In this time, the importance of the operation and maintenance of the network comes into consideration one more time.

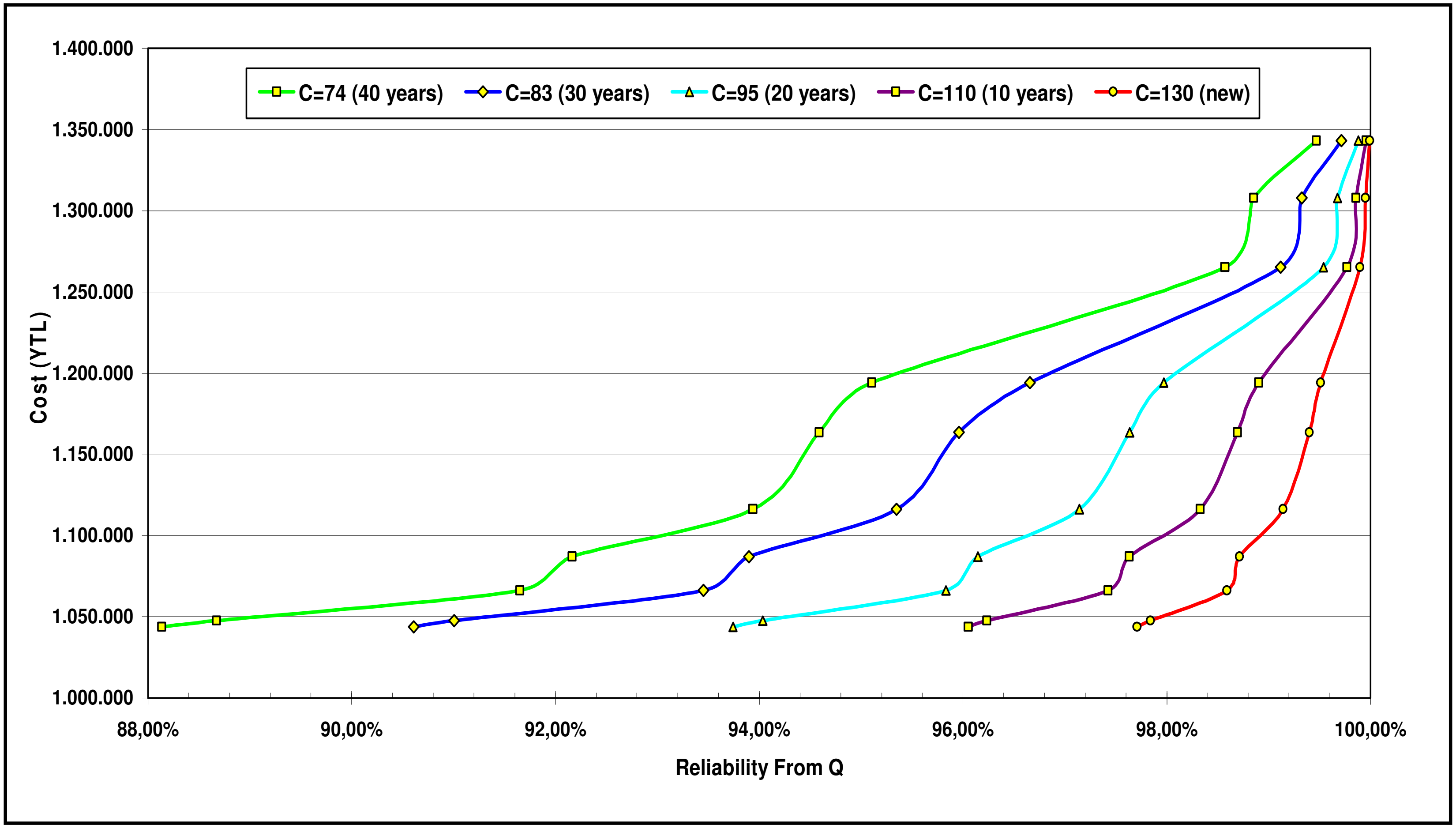

Figure 4.22: The change of reliability (from Q Approach) with pipe aging (time)

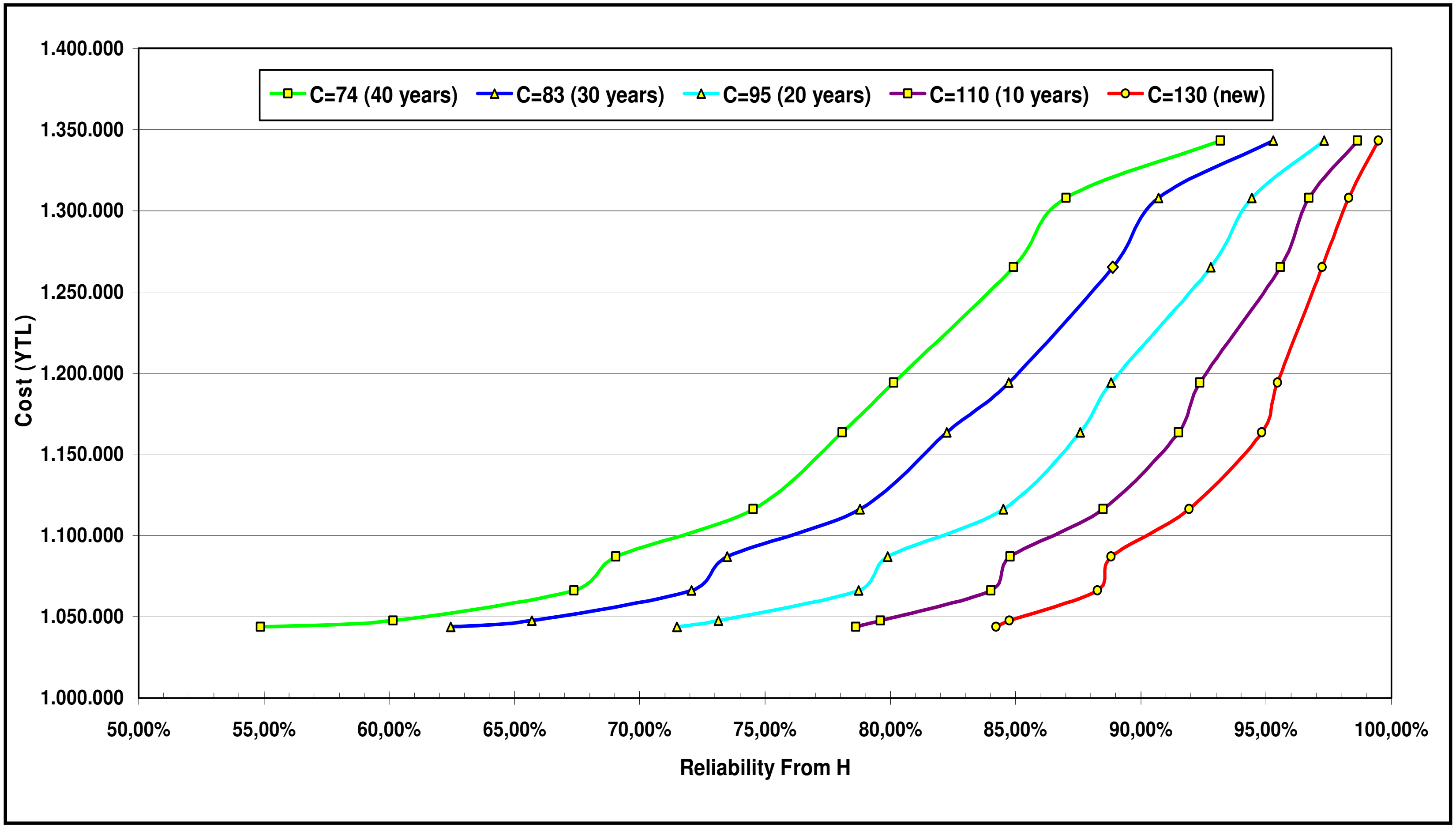

Figure 4.23: The change of reliability (from H Approach) with pipe aging (time)

Table 4.31: Comparison of reliability from Q change with years with respect to new design

% Changes with respect to new design C=130 St=20				
H_{min} (m)	C=74 St=11 40 years later	C=83 St=13 30 years later	C=95 St=15 20 years later	C=110 St=17 10 years later
	Reliability from Q	Reliability from Q	Reliability from Q	Reliability from Q
16	9.80%	7.27%	4.06%	1.69%
17	9.37%	6.99%	3.89%	1.64%
18	7.04%	5.21%	2.80%	1.19%
19	6.64%	4.88%	2.60%	1.10%
20	5.25%	3.83%	2.02%	0.82%
21	4.84%	3.46%	1.77%	0.71%
22	4.43%	2.87%	1.55%	0.61%
23	1.33%	0.78%	0.36%	0.12%
24	1.10%	0.62%	0.28%	0.09%
25	0.52%	0.28%	0.11%	0.03%

Table 4.32: Comparison of reliability from H change with years with respect to new design

% Changes with respect to new design C=130 St=20				
H_{min} (m)	C=74 St=11 40 years later	C=83 St=13 30 years later	C=95 St=15 20 years later	C=110 St=17 10 years later
	Reliability from H	Reliability from H	Reliability from H	Reliability from H
16	34.86%	25.85%	15.12%	6.64%
17	29.03%	22.50%	13.70%	6.08%
18	23.66%	18.36%	10.80%	4.82%
19	22.26%	17.24%	10.05%	4.53%
20	18.93%	14.30%	8.06%	3.74%
21	17.67%	13.28%	7.65%	3.52%
22	16.05%	11.24%	6.95%	3.25%
23	12.67%	8.60%	4.58%	1.71%
24	11.48%	7.72%	3.93%	1.62%
25	6.34%	4.23%	2.17%	0.83%

CHAPTER 5

CONCLUSIONS AND RECOMMENDATION

In this study, a methodology was developed for designing water distribution networks based on reliability. In this context, the first step is to obtain a relationship between the reliability of a water distribution network and its cost (pipe cost); then, the next step is to decide for the reliability level of the proposed design examining the reliability-cost relationship.

This methodology proposes to use a commercially available software, WADISO, to design networks under different design scenarios; these design alternatives were obtained by assuming different minimum required pressures (H_{min}) for the same proposed layout under the same demand conditions. WADISO follows the algorithm of Partial Enumeration Optimization technique proposed by Gessler (1985). Note that these several design alternatives satisfying different minimum required pressures (H_{min}) were obtained under the same peak loading (Q_{peak}) scenario. The reliability value for each of the design alternatives were calculated according to Mısırdalı (2003)'s adaptation based on the methodology proposed by Bao and Mays (1991). The nodal head values and available nodal demands necessary for reliability analysis were obtained employing the hydraulic network solver, HapMam prepared by Nohutçu (2002) and modified later by Mısırdalı (2003).

Some of the alternative network designs having particular reliability values are able to satisfy the minimum required pressure for the network at all nodes (H_{min}≥25m) condition during night loading. The network design which has the smallest reliability and normally with the minimum cost during Q_{peak} could be selected under the condition that it satisfies H_{min}≥25m during night loading. It is obvious that the slope of the cost vs. reliability curve increases for increasing reliability values. In other words, more and more cost is required for a unit increase of reliability. Another fine tuning concerning the ultimate selection of the network design can be realized by choosing a design just before a pronounced increase of the slope of the cost vs. reliability curve. In this study the design with H_{min}=18 m is selected as the ultimate design. It should be emphasized that as seen on all the cost vs. reliability graphs, when the cost of the network increases, the hydraulic reliability also increases. However, after a definite reliability value, the reliability slightly increases even the cost increases considerably. This result shows that, there is no need to increase the cost of design after entering the optimum zone part. The designer should decide on the optimum zone characteristics and then the design should be performed according to these objectives.

The point of designing the network based on a pressure head value smaller than H_{min}=25m, is the existence storage tanks at all buildings in the pressure zone considered. This investment already realized by the owners via developers can be saved by the water authority. It comes out to be that the amount to be saved is roughly around 20%.

This study does not only consider the design of network, which is the skelotinized form of N8-1 sub-pressure zone of Ankara water distribution network, from reliability point of view, but also it examines the influence some mediocre operation practices. One of those two common applications is letting run the network with very low velocities which will favor the aging of pipes. The ultimate remedy against aging of pipes is simply flushing which is unfortunately ignored almost by all the water authorities. The influence of aging of pipes in terms of reliability is fairly well demonstrated in this work. In this study, the reliability of the most expensive design alternative (H_{min}=25m) from H approach decreased in about 1%, 2%, 4% and 7% with respect to 10 years, 20 years, 30 years and 40 years old pipes. Moreover, the decrease in the low cost alternatives was more significant.

Another standard application of low quality operation is employing isolation valves when necessary but then forgets about some of them; because maintenance operations are not recorded on event basis; the consequence of it, is the capacity decrease or in other words reliability loss. As a result, the status of the isolation valves should be effectively maintained. In this study, the reliability of the most expensive design alternative (H_{min}=25m) from H approach decreased in about 7% with respect to 40 valves of 215 valves were closed. It could be seen that the decrease in the low cost alternatives were more significant.

Back to the design of water distribution networks, it should be underlined that the determination of the economic lifetime of the network and the selection of the pipeline material are two important parameters. It was seen that the cost of the design alternatives with 15 years economical life were 20% less than the design alternatives with 30 years economical life in average. There was not any considerable difference between the reliabilities of the designs with 15 and 30 years economical lifes. In addition to the selection of the economical life of the design, the pipe material does not have any significant effect on the reliability of the network. The selection of the pipe material would be performed along with the available budget for the design and the appropriateness of the characteristics of the pipe materials.

Hydraulic modeling (calibrated or not) is a very important tool both for the examination of existing networks and the design of new networks. Considering Hazen-Williams roughness coefficient as a lumped parameter regarding how to express an overall head loss parameter implies that the value of it is of great consequence. Note that unpredicted water demands, significant leakage, will increase Hazen-Williams roughness coefficient whereas, old and/or non existent pipes and pipes with closed valves will decrease Hazen-Williams roughness coefficient. In other words, for a predicted Hazen-Williams coefficient, the standard deviation of it may influence the reliability of a network at a certain extent. The effect of standard deviation of Hazen-Williams roughness coefficient was examined by changing the standard deviation from 10 to 60. It was seen that when the standard deviation of the Hazen-Williams roughness coefficient increases the reliability of the network decreases.

It should be noted that; whichever the design alternative is selected, the engineer should consider short time and long time effects of the selection together. The designer has to realize the importance of the proper operation, maintenance and repair of the water distribution network as well as the construction of the network. Although the reliabilities of the low cost (construction cost) alternatives are less, the cost saving could be used for the proper operation, maintenance and repair of the water distribution network.

In this study, mostly hydraulic reliability was examined. In order to improve this study, the mechanical reliability could also be examined. In addition, different pipe materials could be considered in the design stage of this study. Moreover, in this study only pipe sizes were optimized. Optimization of the other elements of the network such as optimization of the tank size and optimization of the pump stations could also be performed as a future study.

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APPENDIX A

Table A.1: Nodal weights and elevation of the network

Nodes	Final Weights	Elevations (m)
1	0.000000	1037.480
2	0.003987	1065.890
3	0.000191	1089.000
4	0.000000	1105.600
5	0.000130	1119.270
6	0.001405	1118.540
7	0.002505	1116.290
8	0.001176	1115.630
9	0.001046	1115.950
10	0.000107	1116.100
11	0.002375	1092.900
12	0.000000	1109.560
13	0.236395	1105.120
14	0.000000	1044.580
15	0.004017	1103.850
16	0.002215	1093.710
17	0.005262	1098.080
18	0.002612	1096.120
19	0.006889	1056.110
20	0.008073	1091.680
21	0.002803	1105.410
22	0.004330	1112.710
23	0.028029	1101.690
24	0.010494	1092.020
25	0.002818	1099.320
26	0.006683	1099.070
27	0.002161	1107.690
28	0.004659	1109.540
29	0.002261	1114.470
30	0.003284	1074.050
31	0.006133	1085.430
32	0.004002	1079.940
33	0.007897	1059.960
34	0.011135	1100.970
35	0.004346	1112.960
36	0.003956	1109.910
37	0.002475	1085.720
38	0.025432	1084.950
39	0.010318	1094.950
40	0.013090	1107.670
41	0.004391	1061.820
42	0.007378	1108.380
43	0.006003	1091.190
44	0.011250	1026.550
45	0.005636	1106.770
46	0.010440	1101.540

Table A.1 (continued)

47	0.005675	1105.450
48	0.013350	1047.820
49	0.024600	1049.580
50	0.008126	1059.680
51	0.007011	1040.730
52	0.005537	1038.450
53	0.001443	1027.740
54	0.010165	1073.640
55	0.006347	1077.240
56	0.002665	1078.260
57	0.008592	1073.430
58	0.006476	1093.790
59	0.006866	1086.050
60	0.004728	1095.450
61	0.004262	1079.020
62	0.006667	1101.830
63	0.005713	1092.900
64	0.009310	1082.140
65	0.006270	1080.180
66	0.005583	1106.830
67	0.022874	1082.610
68	0.006736	1109.230
69	0.007530	1101.210
70	0.003376	1112.620
71	0.004269	1106.780
72	0.003376	1094.380
73	0.011089	1105.210
74	0.005621	1092.360
75	0.006706	1066.110
76	0.003093	1087.620
77	0.004216	1091.040
78	0.004773	1108.380
79	0.007294	1054.800
80	0.006354	1030.830
81	0.005392	1116.170
82	0.003444	1113.560
83	0.002749	1112.770
84	0.002276	1101.130
85	0.003498	1103.750
86	0.001352	1092.530
87	0.002368	1086.650
88	0.005209	1087.650
89	0.003903	1109.180
90	0.003605	1105.420
91	0.002902	1109.260
92	0.005102	1090.350
93	0.003299	1085.380
94	0.003139	1084.300
95	0.005339	1058.890
96	0.002681	1085.450
97	0.003017	1074.410
98	0.002627	1075.140
99	0.004506	1057.950
100	0.001657	1059.870
101	0.003536	1069.930

Table A.1 (continued)

102	0.003169	1101.390
103	0.006079	1102.580
104	0.004957	1109.690
105	0.003803	1111.440
106	0.002604	1114.950
107	0.003536	1119.170
108	0.003192	1100.780
109	0.002879	1089.280
110	0.006072	1102.200
111	0.005186	1118.170
112	0.002299	1119.270
113	0.002467	1077.410
114	0.002322	1078.570
115	0.004781	1114.380
116	0.005697	1119.350
117	0.004720	1105.760
118	0.007263	1107.050
119	0.005453	1110.030
120	0.003368	1109.470
121	0.007676	1077.930
122	0.007973	1089.080
123	0.006515	1091.530
124	0.006797	1058.350
125	0.007805	1068.670
126	0.003758	1094.100
127	0.003964	1097.840
128	0.004307	1084.110
129	0.005323	1077.790
130	0.005927	1080.630
131	0.003926	1108.940
132	0.003483	1089.280
133	0.002795	1084.390
134	0.003994	1100.950
135	0.002184	1099.920
136	0.002329	1086.400
137	0.001000	1105.590
138	0.006133	1046.330
139	0.004109	1060.410
140	0.006591	1106.020
141	0.003956	1077.460
142	0.005858	1104.260
143	0.005308	1095.950
144	0.005514	1088.300
145	0.002902	1104.700
146	0.001940	1093.560

APPENDIX B

Table B.1: Pipe groups and the pipe numbers of the related groups of the network used in WADISO

Group 1	Group 2	Group 3	Group 4				Group 5
1	2	8	19	78	140	189	37
3	10		20	80	141	190	38
4	11		25	81	142	191	44
5	12		26	82	144	192	48
6	13		27	83	145	193	52
7	14		29	84	146	194	66
	15		30	85	147	195	68
	16		31	86	149	196	79
	17		33	87	151	202	91
			35	88	152	204	93
			36	89	153	206	94
			39	90	154	207	99
			40	92	155	208	109
			41	95	156	209	111
			42	96	157	210	112
			43	97	158	211	113
			45	98	159	212	114
			46	100	160	213	115
			47	101	161	214	116
			49	102	162	215	120
			50	103	163	216	123
			51	104	164	218	127
			53	105	165	219	129
			54	106	166	221	132
			55	107	167	223	133
			56	108	168	224	134
			57	110	169	225	135
			58	117	170	226	143
			59	118	173	229	148
			60	119	174	230	150
			64	121	175	232	171
			65	122	176		172
			67	124	177		181
			69	125	178		185
			70	126	179		197
			71	128	180		198
			72	130	182		199
			73	131	183		200
			74	136	184		201
			75	137	186		217
			76	138	187		222
			77	139	188		231

Table B.2: The pipe numbers of the closed pipes

Pipe Numbers of 40 closed pipes			
1-10	**11- 20**	**21- 30**	**31- 40**
12	84	125	157
20	88	127	160
36	89	129	171
44	91	131	176
49	99	132	181
52	102	135	185
58	109	138	194
68	111	148	197
70	113	150	210
79	120	153	229