# A METHODOLOGY FOR CALCULATING HYDRAULIC SYSTEM RELIABILITY OF WATER DISTRIBUTION NETWORKS

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# A METHODOLOGY FOR CALCULATING HYDRAULIC SYSTEM RELIABILITY OF WATER DISTRIBUTION NETWORKS

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# ABSTRACT


**A METHODOLOGY FOR CALCULATING HYDRAULIC SYSTEM RELIABILITY OF WATER DISTRIBUTION NETWORKS**



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A completely satisfactory water distribution network should fulfill its basic requirements such as providing the expected quality and quantity of water with the desired residual pressures during its lifetime.

A water distribution network should accommodate the abnormal conditions caused by failures. These types of failures can be classified into two groups; mechanical failures and hydraulic failures. Mechanical failure is caused due to malfunctioning of the network elements such as pipe breakage, power outage and pump failure. On the other hand, hydraulic failure, considers system failure due to distributed flow and pressure head which are inadequate at one or more demand points.

This study deals with the calculation of the hydraulic system reliability of an existing water distribution network regarding the Modified Chandapillai model while calculating the partially satisfied nodes.

A case study was carried out on a part of Ankara Water Distribution Network, N8-1. After the modeling of the network, skeletonization and determination of nodal service areas were carried out. The daily demand curves for the area were drawn using the data that were taken from SCADA of the water utility. The daily demand curves of different days were joined and one representative mean daily demand curve together with the standard deviation values was obtained. The friction coefficient values of the pipes and storage tank water elevation were taken as other uncertainty parameters for the model. Bao and Mays (1990) approach were carried together with the hydraulic network solver program prepared by Nohutcu (2002) based on Modified Chandapillai model. The sensitivity analysis for the effects of system characteristics and model assumptions were carried out to see the effects of the parameters on the calculations and to investigate the way of improving the hydraulic reliability of the network.

The storage tank should be located at a higher level for improving the reliability of the network. Also having the storage tank water level nearly full level helps in improving the reliability in daily management. Moreover, the hydraulic system reliability is highly dependent on the pumps as the lowest reliability factors were the ones with the no pump scenarios. Determining the required pressures for nodes are very important since they are the dominant factors that effects the reliability calculations. On the other hand, friction coefficient parameters and type of probability distribution function do not have dominant effect on the results.

Results of this study were helpful to see the effects of different parameters on the hydraulic reliability calculations and for assessment of the methods for improving the reliability for the network.

**Keywords:** Hydraulic Modeling of Networks, Hydraulic failure, Partially Satisfied Nodes, SCADA, Reliability, Ankara.

# ÖZ

**SU DAĞITIM ŞEBEKELERİNİN HİROLİK SİSTEM GÜVENİLİRLİĞİNİN HESAPLANMASI METODOLOJİSİ**

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Tam anlamıyla çalışan bir su dağıtım şebekesi temel ihtiyaçları, gerekli nitelik ve nicelikteki suyu, istenilen basınçla birlikte sunma işini, ömrü boyunca yerine getirmelidir.

Bir su dağıtım şebekesi, hataların sonucunda ortaya çıkan anormal durumları telafi edebilmelidir. Hatalar iki grupta sınıflandırılabilir; mekanik hatalar ve hidrolik hatalar. Boru kırılmaları, enerji kesintileri veya pompa bozulmaları gibi şebeke elemanlarının bozulmasıyla mekanik hatalar meydana gelir. Öte yandan, hidrolik hatalar, sistem hatasını, bir veya daha

çok düğüm noktasındaki yetersiz dağıtılan akım ve basınç açısından inceler.

Bu çalışma, varolan bir su dağıtım şebekesinin, Değiştirilmiş Chandapillai modeliyle birlikte kısmi olarak karşılanabilen su talepleri hesaplanarak hidrolik sistem güvenilirliğinin hesaplanmasıyla ilgilenmektedir.

Ankara Su Dağıtım Şebekesinin bir bölümü, N8-1, kullanılarak bir durum çalışması yapılmıştır. Şebeke modellendikten sonra, iskeletleştirme ve düğüm noktaları servis alanları belirlenmiştir. SCADA'dan alınan bilgilerle, bölgenin günlük harcama eğrileri çizilmiştir. Farklı günlere ait olan günlük harcama eğrileri birleştirilerek, bir tane tanımlayıcı günlük harcama eğrisi, standart sapmalarıyla birlikte elde edilmiştir. Boruların sürtünme katsayıları ve su tankının su seviyesi de model için belirsiz parametreler olarak alınmıştır. Bao ve Mays (1990)' ın yaklaşımı, Nohutcu (2002)'nun Değiştirilmiş Chandapillai modelini baz alan hidrolik şebeke çözüm programıyla birlikte kullanılmıştır. Sistem karakteristiklerinin ve model varsayımlarının duyarlılık analizleri, parametrelerin etkilerinin gözlemlenmesi ve şebekenin hidrolik güvenilirliğinin arttırılma yollarının bulunması için yapılmıştır.

N8-1 şebekesinin güvenilirliğinin artırılması için en ideal çözümün , su tankının daha yüksek bir yere taşınması olduğu ortaya çıkmıştır. Ayrıca su tankını doluya yakın bir seviyede tutmak, günlük işletmede güvenilirliğini artırmaktadır. Dahası, en düşük güvenilirlik faktörlerinin, pompanın olmadığı senaryolarda elde edilmesi, hidrolik sistem güvenilirliğinin yüksek derecede pompalara bağlı olduğunu göstermektedir. Düğüm noktalarının gerekli basıncının belirlenmesi, güvenilirlik hesaplarının baskın bir şekilde etkilediği için çok önemlidir. Öte yandan, sürtünme katsayısının ve olasılık dağılım foksiyonunun tipinin, sonuçlar üzerindeki etkisi sınırlıdır.

Bu çalışmanın sonuçları, değişik parametrelerin hidrolik güvenilirlik hesaplarına etkisini gözlemlenmesinde ve şebekenin güvenilirliği arttırma konusunda method geliştirmede kullanılabilir.

**Anahtar Kelimeler:** Şebekelerin Hidrolik Modellenmesi, Hidrolik hata, kısmi olarak karşılanabilen su talepleri, Güvenilirlik, Ankara.

**To My Parents**

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# LIST OF SYMBOLS

$L_{P1}$ = Length of Pipe P1

$L_{P2}$ = Length of Pipe P2

$L_e$ = Length of Equivalent Pipe

$D_{P1}$ = Diameter of Pipe P1

$D_{P1}$ = Diameter of Pipe P1

$D_e$ = Diameter of Equivalent Pipe

$C_{P1}$ =C factor of Pipe P1

$C_{P2}$ =C factor of Pipe P2

$C_e$ =C factor of Equivalent Pipe

$\bar{I}$ : Inflow to the system (e.g. $m^3$/hr)

$\overline{Q}$ : Outflow or demand from the system (e.g. $m^3$/hr)

$dS$ : Storage in the tank or reservoir for a period of $dt$ (e.g. $m^3$)

$H$ :head (m)

$H_{\min}$ : minimum required head (m)

$Q$ : flow into overhead tank

$K, n$ : Constants

$R_n$ : Nodal Reliability

$H_s$ : Available Head at Node

$H_d{}^l$ : Required Head at Node

$f_s(H_s)$ : Probability Density Function of Supplied Pressure Head

$R_n$ : Nodal Reliability

$Q_a$ : Available Flow at Node

$Q_r$ : Required Flow at Node

$R_{sw}$ : Weighted System Reliability.

$R_{ni}$ : Reliability of Node i.

$W_i$ : Weight of Node i.

$N$ : Number of Nodes.

$HR_{sw}$ : Hydraulic Weighted System Reliability.

# CHAPTER 1

# INTRODUCTION

A water distribution network is composed of interconnected elements, such as pipes, pumps, service and storage tanks, control and isolation valves to supply water to the consumers with adequate quantity and quality.

A completely satisfactory water distribution system should fulfill its basic requirements such as providing the expected quality and quantity of water during its entire lifetime for the expected loading conditions with the desired residual pressures; accommodating abnormal conditions such as breaks in pipes, mechanical failure of pumps, valves, and control systems, including malfunctions of storage facilities and inaccurate demand projections.

Reliability is usually defined as the probability that the system performs within specified limits for a given period of time. However, evaluation of water distribution system reliability is extremely complex because reliability depends on a large number of parameters, some of which are quality and quantity of water available at source; failure rates of supply pumps; power outages; flow capacity of transmission mains ; roughness characteristics including the flow capacity of the various links of the distribution network; pipe breaks and valve failures; variation in daily, weekly, and seasonal demands; as well as demand growth over the years ( Gupta and Bhave, 1994).

The reliability of water distribution systems can be examined in terms of two types of failure; mechanical and hydraulic failure. Mechanical failure is defined as the system failure due to pipe breakage, pump failure and power outages, etc. On the other hand, the hydraulic failure considers system failure due to distributed flow and pressure head which are inadequate at one or more demand points.

Hydraulic reliability is a measure of the performance of the water distribution system. It can be defined as the probability that the system can provide the required demands at the required pressure head. In other words, failure occurs when the demand nodes receive either insufficient flow rate and/or inadequate pressure head. Due to the random nature of future water demands, required pressure heads and pipe roughness, the estimation of water distribution system reliability for the future is subject to uncertainty (Bao and Mays, 1990).

Mechanical reliability is the ability of distribution system components to provide continuing and long-term operation with the need for frequent repairs, modifications, or replacement of components.

The objective of this study is to calculate the reliability of a water distribution system from hydraulic point of view. Network reliability based on mechanical failure was investigated by Mays and Cullinane (1986), Wagner et al. (1988a,b ), and Sue et al. (1987); none of these works consider hydraulic reliability. Bao and Mays (1990) considered only hydraulic reliability where as Tanyimboh et al. (2001), Gupta and Bhave (1994) considered both mechanical and hydraulic reliability.

In this work, concerning hydraulic reliability, the work of Bao and Mays (1990) is followed; however, the methodology of Nohutçu (2002) is used for the determination of pressure heads while the system was delivering partial flows to consumers. It provides software necessary concerning the determination of pressure heads which employs data both from SCADA and GIS platforms of the water utility in question. Another progress in this study in regard to the other studies is that both temporal and spatial variations of nodal demands were considered. Mısırdalı and Eker (2002) developed a methodology for the assessment of spatial variation of the nodal demands.

In Chapter 2, general considerations about water supply and distribution networks will be presented. In Chapter 3, information about modeling of water distribution networks together with the GIS usage will be given. In Chapter 4, the required information on reliability analysis will be included together with pressure dependant models and Chandapilla's (1991) partially satisfied networks approach. In Chapter 5 the study area, the case study itself and results will be presented. Finally, the discussion and recommendations about the study will be given in Chapter 6.

# CHAPTER 2

# WATER DISTRIBUTION NETWORKS

A water distribution system's main task is to provide adequate amount of water to the consumers, within the limits of required pressure with desired quality. In a city, the water must be supplied for different kinds of uses such as commercial, industrial, domestic, and public. The water distribution system should provide also a stable hydraulic grade to provide enough pressure and water to serve for emergency conditions; power outage, fire demand; failure of pipe, pump storage tank.

One of the most important design criterion is the required pressure; which must be provided at each node, as the performance is mainly judged by the pressure availability in the system. The water distribution systems should provide enough pressure to meet consumers' demands throughout the usage periods and at peak hours. Although the acceptable limits for main transmission pipelines are between 5m to 80m, for distribution networks these limits have generally lower and upper bound as 20m and 60m, also these values vary for the characteristics of the pressure zone. The ideal way is to provide pressure above the required level. For lower pressures there can not be a water delivery and for higher pressures there can be excessive amount of leakage. To provide this, the service area is divided into different pressure zones. One of the main criteria determining the number of zones is the topology. A system serving to a highly elevated hilly area has more pressure zones than a relatively flat area. Even dividing the pressure zone to the sub-zones might be an appropriate way to provide water efficiently.

Distribution systems are generally classified according to their layout patterns as grid systems, branching systems and combination of these (Özkan, 2001) (Figure 2.1).

Actually the street patterns, topography, construction plans and future plans determine the layout of pipes. The grid system is the best way of arrangement for distribution systems concerning both quality and reliability as all the pipes are looped providing the water circulation by eliminating the dead ends and redundant supply of the water. On the other hand, the branched pipe networks do not permit the water circulation since they contain lots of dead ends. Furthermore, if a pipe repair is needed the whole branch can not deliver water in branch systems; on the other hand, the area out of service can be reduced to a minimum in looped networks with proper valve operation. In real life networks, it is very hard to have a totally grid system. Most of the water distribution systems are a combination of grid and branched systems.

A water distribution system is composed of pipe network, storage facilities, valves, pumps, fire hydrants, service connections and other minor elements. Pipe network consists of transmission lines, arterial pipes, distribution pipes and service lines. The transmission mains are connecting the source and the storage tanks while passing through the service area with relatively bigger diameter. Arterial branches from transmission mains carry water to distribution pipes. The distribution pipes are distributing the water to the consumers. Finally service lines, with the smallest diameter, transmit water to the consumers. (Figure 2.2)

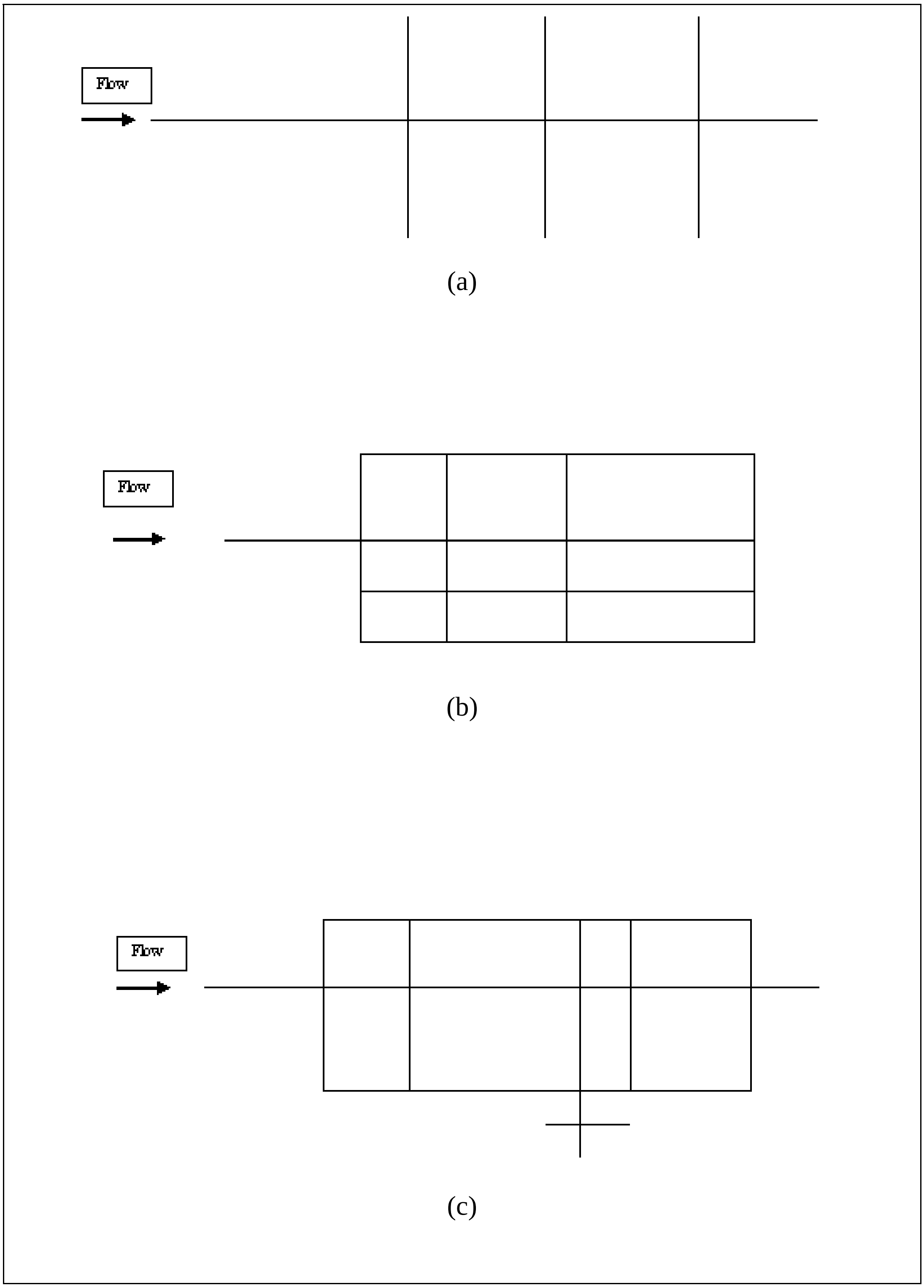


**Figure 2.1 Types of Water Distribution Systems (a) Branching System. (b) Grid System (c) Combination System (Clark et al., 1977)**

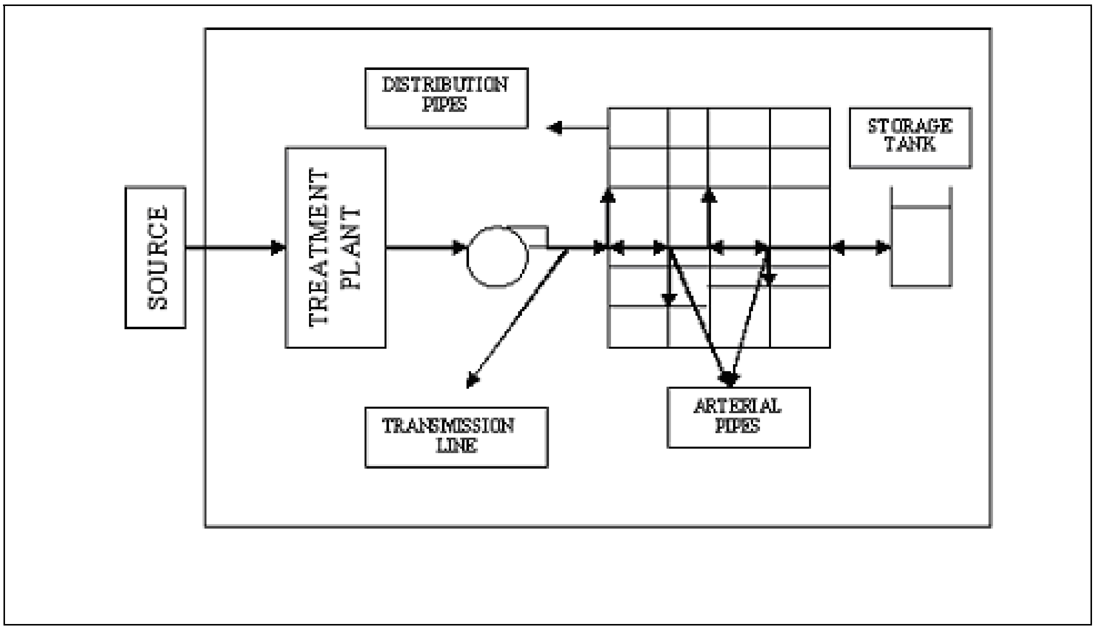


**Figure 2.2 Elements of typical water distribution network**

## 2.1 WATER DISTRIBUTION NETWORK ELEMENTS

### 2.1.1 Pipes

The main components of water distribution systems are the pipes. They can be found in different lengths, materials and diameters laid down in the network. The pipes are mainly grouped into three:

- Transmission lines
- Distribution lines
- Service pipes

The transmission line is the pipe between the source and the storage elements; it carries water from source or pump station to the storage tank while the capacity is enough for both serving the consumers and carrying excess water to the storage tank. Also it delivers water from storage tank when the source or pump is not able to meet the demand.

The distribution lines deliver water to the pressure zone and distribute the water to the service nodes. On the other hand, service pipes are the pipes that mainly send water to the consumers.

### 2.1.2 Pumps

A pump is a hydraulic machine that adds energy to the water flow by converting the mechanical energy into potential energy to overcome the friction loses and hydraulic grade differentiations with in the system.

The pump characteristics are presented by various performance curves such as, power head and efficiency requirements that are developed for the friction rate. These curves are used in the design stage to find out the most suitable pump for the system. In most of the pumping stations two or more pumps are used to ensure reliability, efficiency and flexibility. Pump efficiency plays an important role in water distribution network management as a high percentage of total expenses is used for their electricity bills.

### 2.1.3 Valves

There are different types of valves in water distribution systems with different characteristics and usage conditions. Their locations and characteristics are decisive for the daily management.

#### 2.1.3.1 Check Valves

Check valves are the valves that prevent the water flow backwards from the desired direction. When water flows in the direction of need, check valve status is open; on the other hand, when the flow changes its direction, the check valve’s status is closed in order to permit the flow. They are widely used in front of the pumps in order to prevent reverse water flow through the pumps.

#### 2.1.3.2 Control Valves

Control valves are used to control the amount of water flow in the pipes by reducing the pipe area. Generally butterfly types of valves are used for that purpose. These types of valves generally used for regulating purposes and controlling the overall pressure on the sub-pressure zones.

#### 2.1.3.3 Isolating Valves

When a pipe breaks or if a maintenance work is needed, in order to isolate the pipe or pipe segment from the rest of the network, isolating valves are used. Generally gate valves are chosen as isolating valves. Despite of control valves, their ability to control the flow is very limited. For that purpose, the isolating pipes should be used in the fully close or open position, as partially open valves may end with broken valves in the system.

Furthermore, isolating valves are the mostly used valves in a network. Their locations and working conditions directly affect the distribution systems characteristics and reliability purposes.

#### 2.1.3.4 Air Release Valves

Air in the water distribution system must be taken out from the network in order to have system stable. For that purposes, air release valves are used. They are usually located at the high points of pipes as mostly air is trapped and purged at these locations.

#### 2.1.3.5 Pressure Reducing Valves

Pressure reducing valves are the valves that used to prevent the high inlet pressure pass trough the outlet. As the water flows from pressure reducing valve, the pressure is reduced to the desired level by proper adjustment of the valve. These types of valves are generally used in between the zones with high elevation differences. Furthermore, these valves have the flow controlling abilities.

### 2.1.4 Storage Tanks

A storage tank's main purpose is to store excess water during low demand periods in order to meet widely fluctuating demands such as fire demands and peak hour's demands.

A storage tank's oscillations are directly integrated with the demand and pump working rate. Generally tanks are used as distribution reservoirs to supply coming from the pump and store the excess flow during night. Another usage of storage tank is that they stabilize the excess pressure over the network by opening the system to the atmospheric pressure.

### 2.1.5 Fire Hydrants

Fire Hydrants are used mainly for fire fighting by local fire department which also determines the places and number of them. They are used also for street washing and flushing of water distribution pipes and sanitary sewers if necessary.

As they are important for fire fighting their maintenance should be done properly. The fire hydrants can be used while modeling and calibrating the network; they provide to the modeler high water flows as they were extracted from the related nodes.

# CHAPTER 3

# MODELLING OF WATER DISTRIBUTION NETWORKS USING GIS

The basic aim for modeling is to simulate the real life or field events. In water distribution networks, modeling is generally used for simulating the behavior and characteristics of the system using mathematical and computerized algorithms together with the pressures, demands, friction coefficients and such input parameters in order to increase the efficiency of the management and/or analyze the system.

## 3.1 Steps In Modeling

Today, hydraulic modeling is a necessity especially for the networks of the large cities with rapid growth. Not only for daily system monitoring but also it is a need for future investments.

First of all, in the way of the modeling, the modeler should decide on the goal, in order to have an efficient and reliable model; as the steps of modeling is governed by its aim. For example, in a leak detection model, even the pipes with smallest diameters such as 50 mm, should be included as any pipe can be a source for leakage. On the other hand, for analyzing the pump and storage tank relation, only main transmission pipes may be enough. After having decided for the aim of the modeling further steps can be taken.

### 3.1.1 Data Collection

In order to have a hydraulic model of a water distribution network, extensive amount of information and data should be gathered. Starting from field surveys, the zonal borders, pipe characteristics, materials, diameters, valve locations, tank locations, volumes and elevations to pump locations and characteristics can be included to a model.

Starting from the utility maps, drawings and other records that describe the length, material, age, diameter and location of pipes can be used together with the field survey. Field survey takes an important role as it is not the only source you can get the most accurate information and data, but also it gives opportunity to check the results of your analysis.

Also during the field surveys, the mismanagement like open isolation valves between the zones, closed and forgotten isolation valves in the zones or extra connections and non-drawn pipe segments can be observed and corrected. Correction and searching for such errors also lead to accurate and efficient work with the model.

After the data collection, steps for further studies can be taken. Although, the data collection is not a one time job, it is important to have accurate information about the system.

### 3.1.2 Skeletonization

An ordinary water distribution network contains hundreds of pipes with different diameters that are less than 50 mm to more than 1500 mm. Also a typical municipal water supply system serves thousands of customers. In the past, as early computer systems were unable to solve the networks with huge number of nodes and pipes, because of extensive amount of calculations, reducing the number of pipes and nodes considering the pipe and loop importance should have been done.

Today, with the extensive improvements with the calculation ability of micro-computers, limitation on number of pipes and nodes became flexible. On the other hand, including every service pipe and every user would not be practical both in means of engineering point of view and it is impossible for a modeler to know and control so much information. In practice, pipes smaller than 100 mm or 150 mm are ignored or grouped together and replaced by equivalent pipes. This process is called skeletonization.

Generally omitting the small diameter pipes is satisfactory especially when such pipes are perpendicular to common direction of flow or are near large diameter pipes. On the other hand, the small diameter pipes that are lying near the source of large water users or in the neighborhood of larger diameter pipes should be considered through equivalent pipes (Poyraz, 1998).

In equivalent pipe method, a complex system is replaced by a single hydraulically equivalent pipe segment. For example, for given pipes **P1** and **P2**, the Hazen-Williams coefficient (C- coefficient) can be given by:

$$C_e = C_{P1} + C_{P2} \cdot \left( \left( \frac{D_{P2}}{D_{P1}} \right)^{2.63} \cdot \left( \frac{L_{P1}}{L_{P2}} \right)^{0.54} \right) \quad (3.1)$$

Similarly, two similar pipes having same C- coefficient, pipe **P1** and **P2**, connected in series can be replaced by q equivalent pipe:

$$L_e = D_e \cdot \left( \frac{L_{P1}}{D_{P1}^{4.87}} + \frac{L_{P2}}{D_{P2}^{4.87}} \right) \quad (3.2)$$

| | |
|---|---|

Where:

$L_{P1}$ = Length of Pipe P1

$L_{P2}$ = Length of Pipe P2

$L_e$ = Length of Equivalent Pipe

$D_{P1}$ = Diameter of Pipe P1

$D_{P2}$ = Diameter of Pipe P2

$D_e$ = Diameter of Equivalent Pipe

$C_{P1}$ =C factor of Pipe P1

$C_{P2}$ =C factor of Pipe P2

$C_e$ =C factor of Equivalent Pipe

For different purposes different skeletonization of the same network can be carried out. The degree of skeletonization is governed by the aim of the model. But for reliability analysis, ignoring the pipes with diameters less than 100 mm or 125 mm would be appropriate.

### 3.1.3 Head and Supply at Source Nodes

The water elevation at the reservoirs or service tanks can be measured accurately. Furthermore, the outlet pressure of the pumps can be measured and taken as source head for pumps. Also these values can be estimated by using either past data or pump characteristic curves.

Similarly, the supply of water can be measured or estimated. For a pump, using the pump characteristic curves, outlet and inlet pressure the amount of water supplied by pump can be determined. For a tank, by observing the rate of change in elevation of water in the tank, the supplied water can be measured. For this purposes appropriate meters may also be used.

### 3.1.4 Pipe Roughness

One of the most uncertain parameter in water distribution modeling is pipe roughness coefficient. Not only the material type, but also age of the pipe, average velocity of flow, and characteristics of the water determine the friction coefficient of the pipe. So either good estimation should be done during the modeling or model calibration should be carried out for representative determination of friction coefficients. So putting them as an uncertain parameter, would lead us to get better and realistic results in the way of reliability calculations.

### 3.1.5 Nodal Demands and Nodal Weight Calculations

The other uncertain model parameter is nodal demand. Although in real life there are no nodal demands but the service pipes for every building or consumer, the commercial software accept the water usage is extracted from nodes. This method assumes that these nodes are located at the junction points of links.

The rate of water use at nodes depends on the population served by node; social characteristics of the end users; the time of day; climatic conditions; and type of usage. Not only uncertainty for a single node, but total consumption may differ due to already mentioned reasons.

Although all the usage in the network is metered, it is very hard to place the demands to demand nodes accurately. Because of this, by the use of information gathered from different sources (tank volumes, SCADA, pump curves), the total demand or total water consumption of whole network can be found. Consequently, assigning weights to the nodes the total demand can be distributed to each demand node.

The nodal weights are generally calculated by dividing one half of the length of the pipes connected to that node, to total number of pipes in the network. Although there are several ways for determining the nodal weights, in this study nodal weights were calculated by using Service Area Method (See section 3.1.5.2).

#### 3.1.5.1 Data Collection Using SCADA

SCADA stands for "Supervisory Control and Data Acquisition System". The term supervisory indicates that there is a personal supervising of the operation of the system. Field instruments, Communications Network, Remote Stations and Central Monitoring Station together with the supervisor, compile data concerning the operation of the system and allow the control of some of the elements of the network in a SCADA system.

The inlet and outlet pressures of the pumps, the elevation of the tanks, the discharge through a pipe or pressure at any point can be measured, transmitted and stored in a SCADA system. These data can be used to determine different characteristics of network elements and zonal demands, with appropriate calculations, if needed.

##### 3.1.5.1.1 Daily Demand Curves

Daily demand curves are the curves representing the water consumption of the system in means of time. Using the information gathered from the curves different kinds of information can be obtained such as water usage behavior of the consumers, peak values of water need, and leakage percentage of the system. Generally three points are important in means of analyzing the daily demand curves; minimum demand, maximum demand and average demand (see Figure 3.1).

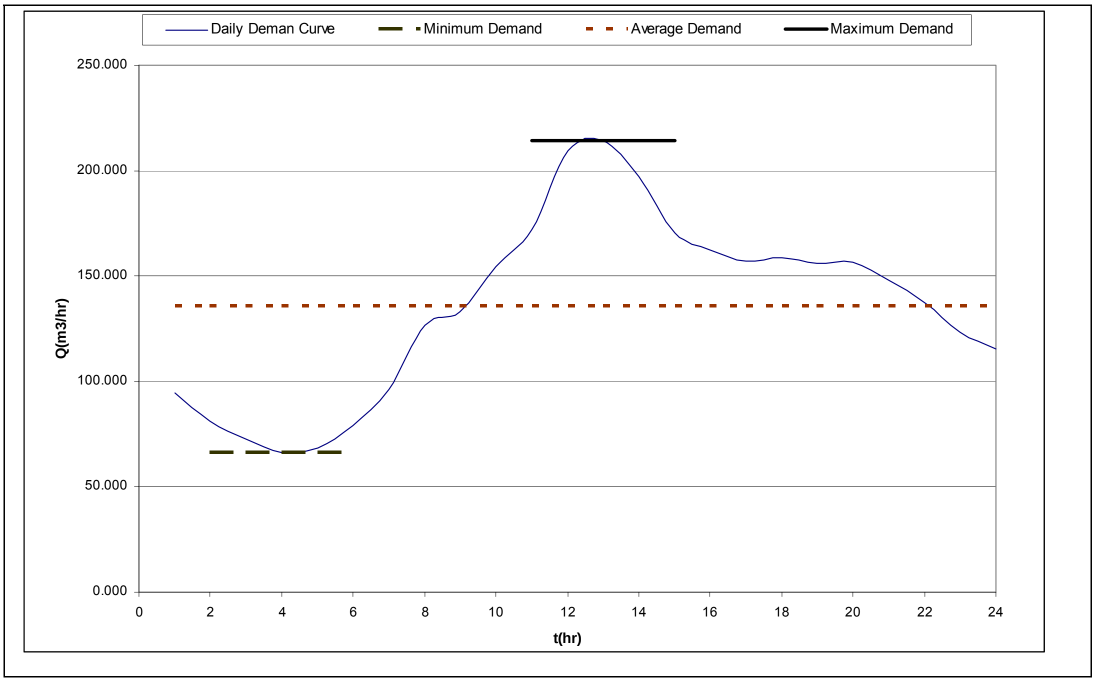


**Figure 3.1 Typical Daily Demand Curve for Part of North Zone of Ankara**

The average demand gives the information of average usage of water in the area that can be helpful to determine the demand projections of future developments of the same or similar areas. Maximum daily demand can be useful for daily management, planning and design of such areas.

Although there may be lots of different techniques to determine the daily demand curve, all techniques uses the same equation that is called continuity equation.

$$\bar{I} - \overline{Q} = \frac{dS}{dt} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3.3)$$

Where;

$\bar{I}$ : Inflow to the system (e.g. $m^3/hr$)

$\overline{Q}$ : Outflow or demand from the system (e.g. $m^3/hr$)

$dS$ : Storage in the tank or reservoir for a period of $dt$ (e.g. $m^3$)

In existing water distribution networks, generally the inflow is generated by a pumping station. Generally, outflow parameter is the demand of the consumers and other pumping stations that serves to other pressure zones. By determining the discharges of the pump stations; either inflow or outflow; and tank water level change in means of time, the daily demand curve of any pressure zone can be determined by using the continuity equation.

#### 3.1.5.2 Service Area Method

In modeling of water distribution network, the general approach is to place nodes on junction points of pipes and assume that these points are the only points that serve water to the consumers. However, in real life, service pipes are used for water transmission to the end users. Most of the buildings are connected to the nearest distribution pipe with a service pipe. For apartment buildings and two or less storey buildings, the diameter of the service pipe is 5/4 inch and 3/4 inch respectively. Moreover, it can be said that every building in the pressure zone has its own node. On the other hand, assumption of nodes at the junctions is an optimal solution to simulate a water distribution network as it may be possible to include every single building node on the model.

The main problem on nodal approach is how to distribute the total demand to the nodes. Assignment of nodal weights to every node and distribute one total demand to node is one of the approaches for solving this problem. On the other hand, determination of nodal weights may be another question for the modelers. To overcome these problems, in this study, Service Area Method (SAM) was used. The working principle of this method is based on finding the areas that are served by every node. By doing this, the number and composition of the consumers can be determined which leads us to have more realistic nodal weights. A program called HYDSAM written in Matlab, for enabling the user to find the respective areas of service of each node together with the program, Vertical Mapper. The HYDSAM contains two parts; first part deals with placing the artificial nodes on pipe segments, and the second part is forming the service area. Before starting to work with the first part, the nodes with zero node weights and the pipes that do not serve the consumers are extracted from the files (see Figure 3.2).

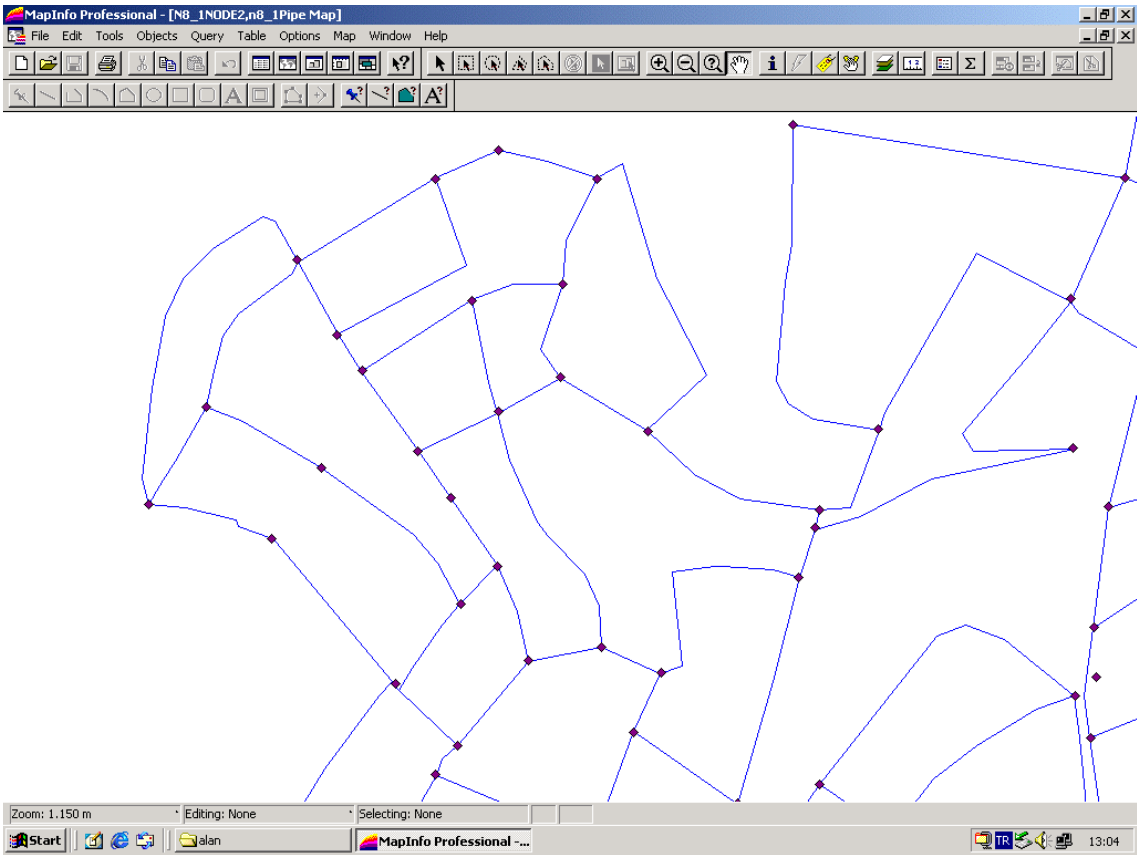


**Figure 3.2 Part of N8-1 Pressure Zone used with HYDSAM**

When starting the first part, the program places artificial nodes on the pipe segments; the number of the artificial nodes depends on the length of the link (see Figure 3.3). The important point on this part, no matter what the numbers of artificial nodes are, the total numbers of them are even on each pipe segment. After that *Create Points from Region (Voroni)*, option of Vertical Mapper is used to generate the areas of every single node. Using this technique, one region is generated around each individual data point.  The resulting network of regions is often referred to as a Voronoi diagram. The Voronoi Options dialogue box provides settings that control the manner in which the Voronoi diagram is created. The most critical area of the Voronoi diagram is the outer margin where no points are present to control the formation of the outermost polygons.

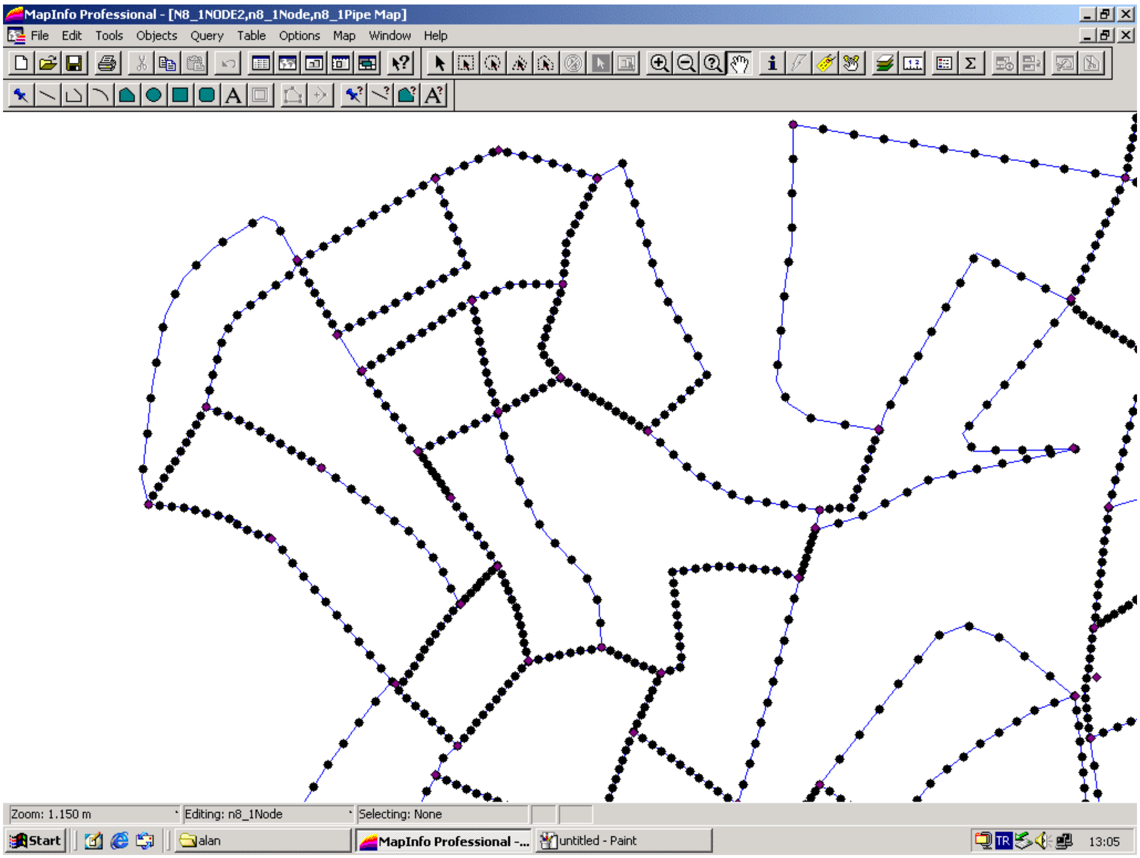


**Figure 3.3 Artificial Node Installation on pipes**

The Boundary Margin Width setting (in map units) controls the distance of the outermost polygon edge from the outer points. Since no points are present beyond the margin to control polygon creation, this setting restricts the construction of polygon sides to a fixed distance from each outermost point (refer to the diagram below). A pre-defined MapInfo region can also be used as the Voronoi boundary by checking the Select Region from Map check box. For a water distribution network, this boundary is generally the boundary of the respective pressure zone (see Figure 3.4). After selecting the Finish button to begin the Voronoi process, the user is prompted to Pick Region From Map Window. The Boundary Smoothness setting determines the number of line segments that are used to construct the corners of the outer hull of the diagram (see Figure 3.5).

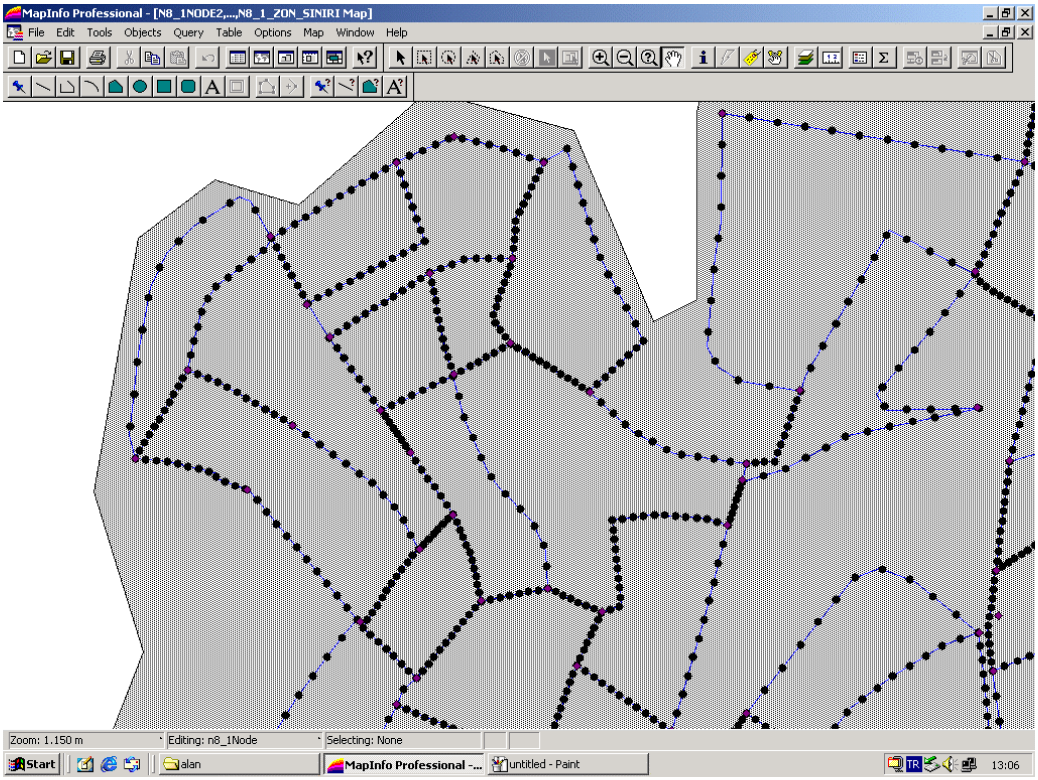


**Figure 3.4 Boundaries of N8-1 Pressure Zone**

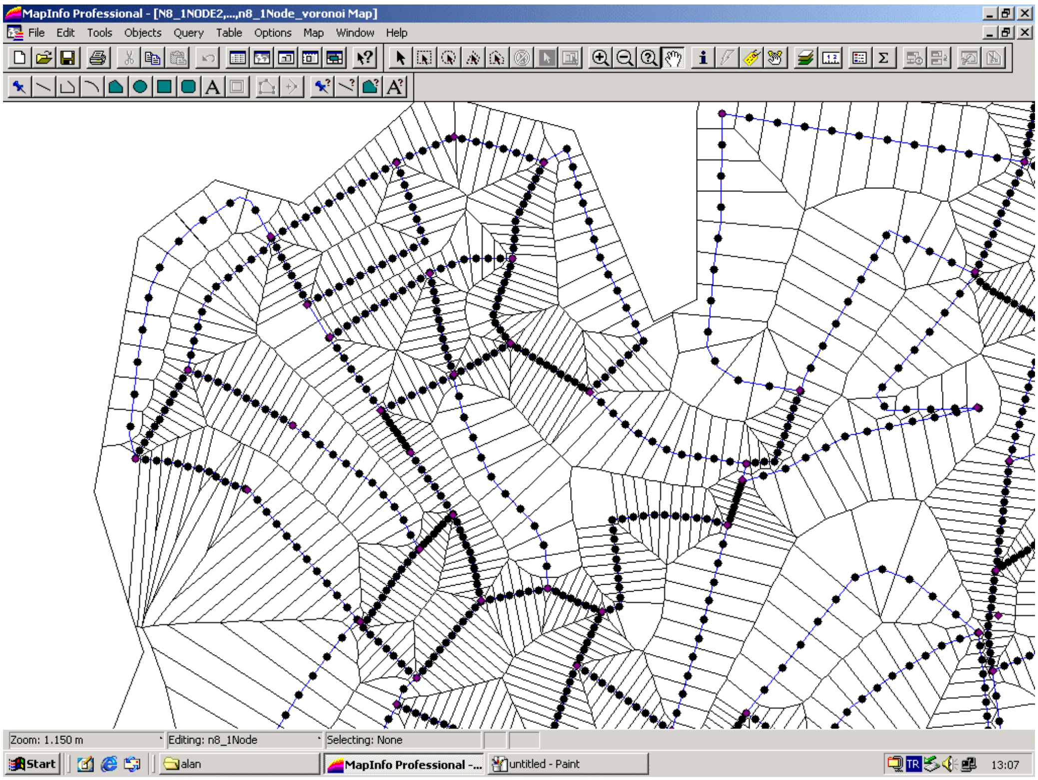


**Figure 3.5 Nodal Areas using Voroni**

Finally, after using the Vertical Mapper, the second part of HYDSAM is used for joining the respective areas to the service nodes, by adding the artificial nodal areas to the neighboring real nodes (see Figure 3.6). By using even number of artificial nodes, the program enables to divide the pipe segment from the middle. The main assumption under this approach is that, every building in the area is getting water from the pipe closest to the building. So, by adding the areas that is generated by the artificial nodes on each pipe segment, service nodal points at the junctions become the main source of extraction for each building.

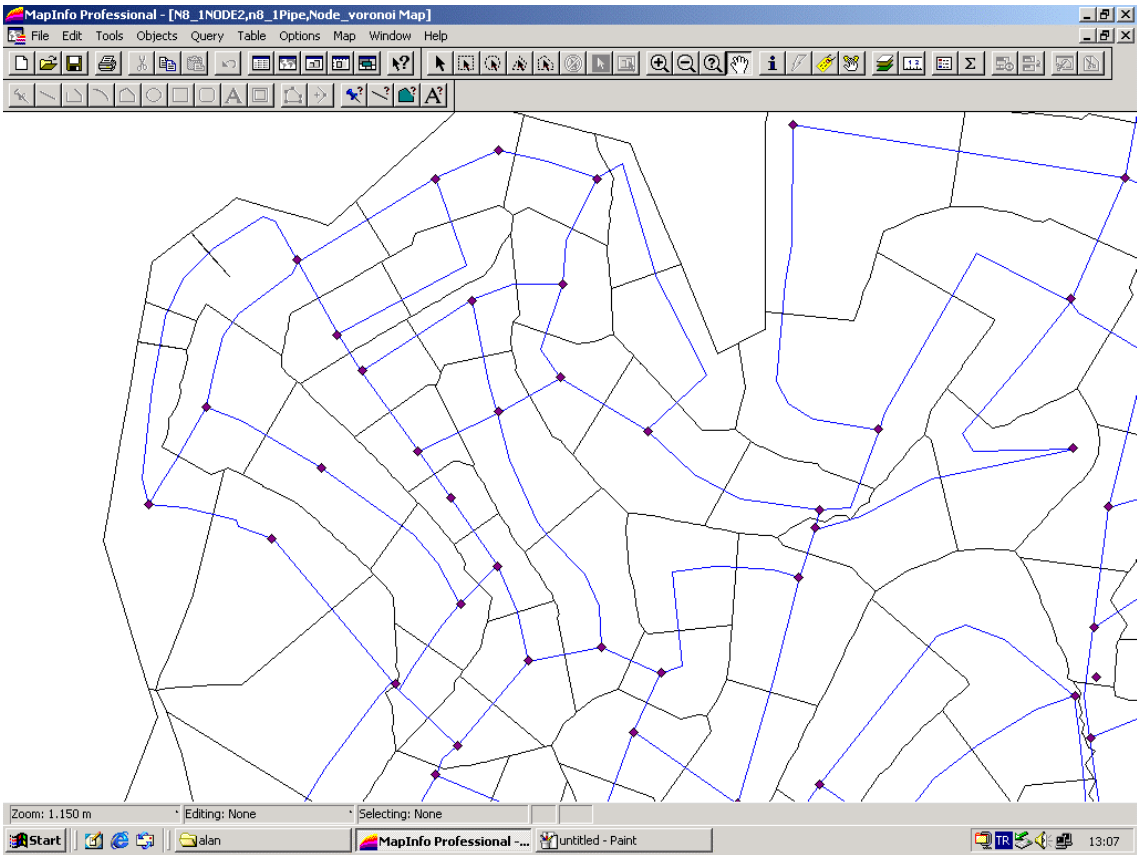


**Figure 3.6 Nodal Areas after Joining Artificial Nodal Areas**

### 3.1.5.3 Consumer Data Integration

After having determined the areas for each node, nodal weights can be calculated using these areas. One approach is direct usage of the areal percentages for the nodal weights, but this approach has disadvantages as some areas may not contain any buildings. To have complete and reliable nodal weights, the best way is to use consumer data together with the areas. The buildings that are inside the area can be determined (see Figure 3.7). After that, by using the consumption data for each building, the average water usage of buildings can be determined. By integrating the consumptions of buildings with the nodal areas, the respective consumption nodal weights can be determined for each node.

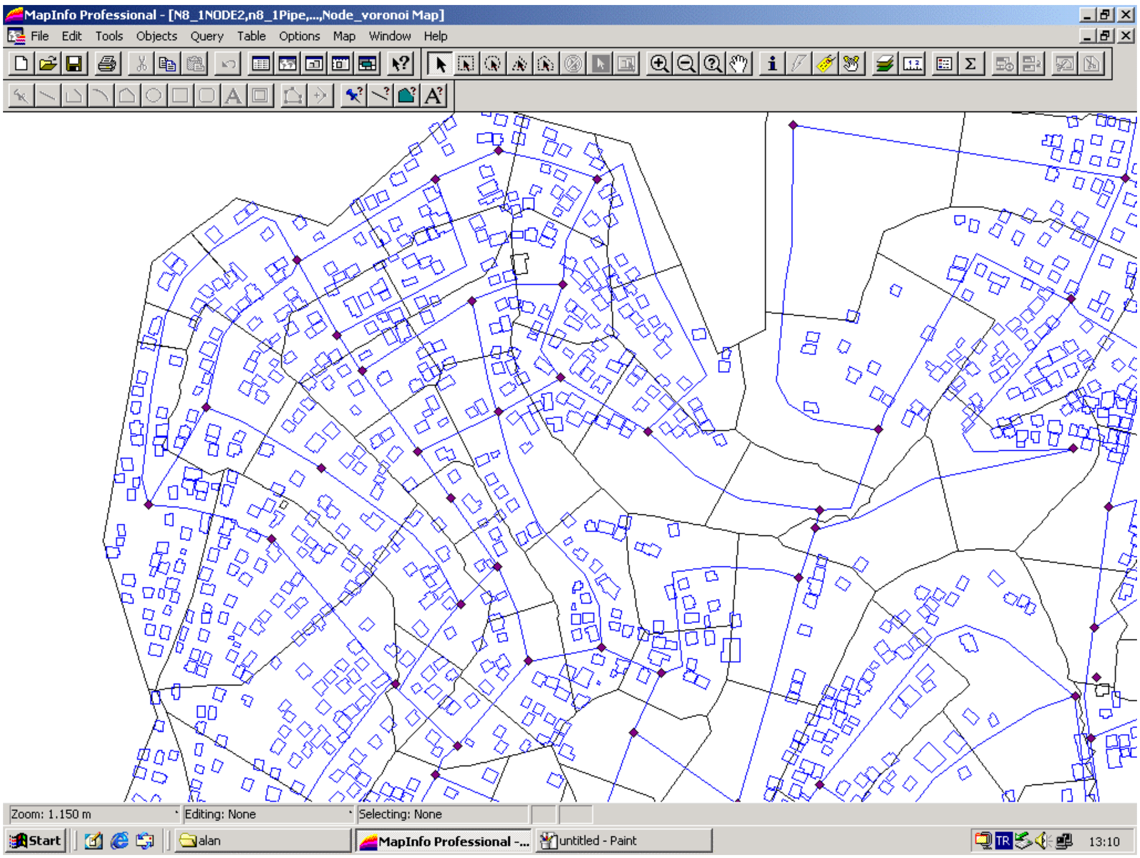


**Figure 3.7 Buildings that are served with the Corresponding Node**

The main advantage of this approach is that, in the way of determining the nodal weights, consumption habits of the consumers can be directly integrated with in the model. Moreover, if the area contains different types of usages, (e.g. commercial, industrial) these different consumption habits can be put into nodal weights without approximation. Furthermore, some nodes may become non-serving nodes as the neighboring pipes may not have buildings nearby. Also, instead of using half of the lengths of neighboring pipes approach, which needs an assumption of totally homogenous users and area, this approach enables the modeler to see different amounts of usages with respect to seasonal changes.

### 3.1.6 Required Head for Nodes

The required pressure for a node is the pressure head needed by the node that enables the extraction of the required demand from the system. Although the definition is simple, determination of the required heads are not that easy. As every node serves in a different location to different types of buildings, it is very hard to determine the required pressure. An assumption of a single value for all nodes can be used, but this approach may fail in determining the reliability of the network.

SAM can be used for determining the required pressures for each node. The elevation of the buildings inside the nodal areas can be used. As the node and the buildings would probably have different elevations, the smallest one can be taken as nodal elevation. Furthermore the difference between the building that has the highest roof elevation and nodal elevation gives the required pressure for that node (see Figure 3.8)

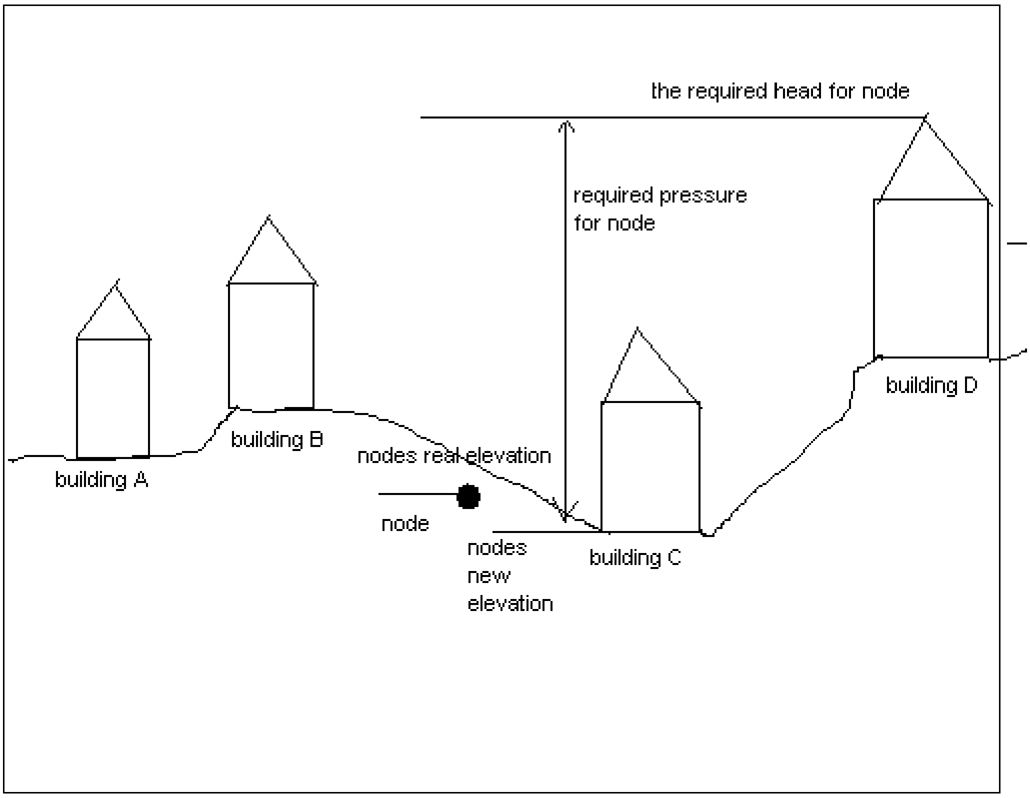


**Figure 3.8 Required Pressure Calculations for a Node**

## 3.2 Calibration

Process of adjusting system input parameters until the output reasonably simulates actual field conditions is called water distribution system calibration. Calibration is an iterative process that requires several executions of model to achieve that desired accuracy. Adjustment of pipe diameters for simulating partially closed valves, changing pipe roughness coefficients to obtain desired flow rates and pressures, adjustment of pump lifts to simulate actual discharge pressures are some of the adjustments that should be made during the calibration.

For improving the reliability and to eliminate the need for trial-error calibration methods, an explicit calibration algorithm is needed for the hydraulic network models. Although different techniques are available, these techniques can be classified under two topics.

- Techniques Adjusting Pipe Head Loss Coefficient (Ormsbee and Wood, 1986)
- Techniques Adjusting Pipe Head Loss Coefficient and Nodal Demands (Boulos and Wood, 1990; Walski, 1983b; Bhave, 1988)

Although the process of calibration is a need for hydraulic analysis, it is generally neglected, that leaves the model with its errors. For different situations and time periods field data should be collected and by the help of the calibration techniques the hydraulic models should be adjusted.

In this study however, no calibration in micro level was carried on. The macro calibration by field studies by determining the zonal leakages were realized on the network. Although, no micro calibration was carried on, the methodology that was carried on determining the reliability, somehow the pipe head loss coefficients were assumed to be adjusted. Furthermore, using SAM, adjusting the nodal demands would be useless with the model.

# CHAPTER 4

# RELIABILITY ANALYSIS BASED ON PRESSURE DEPENDENT MODELS

## 4.1 Reliability Definition

The optimization of the operation and design of the water distribution networks were used to be the main objectives. In these kinds of optimal solutions, the main objective is to minimize the overall cost subject to meeting consumer demands together with satisfying the required pressure for every node. However, in the last decade additional parameters such as reliability and water quality were begun to be considered.

Although there is an increasing interest on assessment of reliability, no universally acceptable definition or measure of reliability is currently available. On the other hand, reliability is usually defined as the probability that a system will perform its missions within specified limits for a given period of time in a specified environment. For large systems that contain many interactive subsystems (such as water distribution systems), it is very difficult to compute the reliability analytically as the accurate calculation of a mathematical reliability requiring knowledge of the precise reliability of the basic subsystems or components.

Reliability of a water distribution system can be defined 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 (1) the flows to be supplied (total volume and flow rate); and (2) the range of pressures at which those flows must be provided (Mays et al., 2000). On the other hand, the measure of reliability that considers both supplied flow and pressure at a node is currently unavailable.

The main source of lack of reliability of a water distribution system is associated with different types of failures. Failure of water distribution networks can be defined as the pressure, flow or both falling below specified values at one or more nodes within the network. However, the failure modes can be classified into two different categories; performance failure and component failure. The performance failure is a result of hydraulic loads being greater than the design loads. For such cases flows together with supply pressures can fall below the desired level. The performance can also be defined as hydraulic failure, which is not to be necessarily catastrophic, as a failure of main transmission pipe line. The pressures at a few nodes may fall below the minimum required one for a certain period of time such as 6 hours. On the other hand, component failure (mechanical failure) can be derived from historical failure records and can be modeled using appropriate probability distribution. For the case of mechanical failure, failure of pipes, tanks and pumps may be analyzed and reliability factors regarding the components can be obtained. The total system reliability can not be calculated without considering both types of failure modes.

## 4.2 Different Approaches of Reliability

Although candidate approaches using concepts of reliability factors such as total number of breaks; economic loss functions; and forced redundancy by adding more pipes into system; in the designs, there is no universally accepted definition or measure of the reliability of water distribution systems. However, these currently available approaches that are available for the assessment of the reliability can be grouped into two different categories; simulation approaches and analytical approaches.

The determination of the reliability by simulation models is usually carried out by a case by case or scenario basis. The corresponding scenarios of component failure and the effects of these components are examined. An important feature of these kinds of “simulation approaches” is the need to generate the time series and to model and simulate the hydraulic performance of the network for each case or condition generated in time series. In a case by case approach, the predetermined scenarios or demand patterns or network combinations prepared. Afterwards, the network is modeled for each case and the simulation is carried out to determine the flows and pressures that would occur in the system as a result of the particular case. The demands that can be used in the analysis can be a combination of demands, like fire and emergency whereas it can be the daily peak demand for that system. The component failure aspects of network reliability performance are handled through modifications to the network configuration such as removing a broken pipe together with the neighboring isolated pipes because of valve configuration.

Bao and Mays (1990) used a Monte Carlo simulation approach to measure the hydraulic reliability of the network. For their cases the time series scenarios were generated by modeling the probability distribution of the demand, pressure head and pipe roughness.

Yıldız (2002) used again a simulation method for measuring the mechanical reliability of the network. For his case, however, the pipe breakages and valve isolation based scenarios were generated and the simulations of the new system were carried out for the assessment of the reliability.

On the other hand, analytical approaches wherein a closed form solution for the reliability is derived directly from the parameters which define the loads, on the network and from the ability of the network to meet those demands (Mays et al., 2000).

The analytical approaches propose together with the features such as reachability, connectivity and cutsets. Reachability is the connection of a specified demand node to at least one source; whereas connectivity is the connection of every demand node to a source node; and finally cutset is a set of links when taken out from the network, disconnects one or more nodes completely from the system.

As a result of examination of a series of reachability and connectivity techniques it was reported by Wagner et al. (1988a) that although the particular techniques are effective for some networks, significant computational problems were encountered when the techniques were applied real-life water distribution networks. Methods for assessment of the reliability can be seen in chronological order. (Table 4.1)

**Table 4.1 Summary of Major Simulation (S) and Analytical (A) Approaches to Assessment of Reliability in Water Distribution Networks (Mays et al., 2000)**

| Study | Approach | S or A | Issues Addressed |
|---|---|---|---|
| Rowell and Barnes (1982) | Minimum cost branched network with cross connections | S | Design branched system -add cross connections to meet demands |
| Morgan and Goulter (1985) | Minimum -cost design model for looped systems | S | Designed for a range of combinations of critical flows and pipe failure |
| Kettler and Goulter (1983) | Minimum-cost design model with constraints on the probability of a pipe failing | A | "Reliability" constrained probability of pipe breakage <= acceptable level "removed" |
| Goulter and Coals (1986) | Minimum-cost design model under constraints on probability of node isolation | A | Probability of a node being disconnected from the network must be < acceptable value - if unacceptable which link should be improved? Be able to meet the demand |
| Su et al. (1987) | Minimum-cost design model with restrictions on the probability of "minimum cut sets" | S to A | Examines the impacts of removal of one ( and two) links on the ability of the network to meet demands in the network- uses probability of pipe breakage Graph theory |
| Germanopoulos et al. (1986) | Assessing reliability of supply and level of service | S | Network performance failure/post failure. Simulation of failure occurrences and repair Times |
| Wagner et al (1988a) | Reliability analysis- analytical | A | Reachability and connectivity. Series and parallel reductions to get trees. Probability of sufficient supply as a reliability measure |

**Table 4.1 Cont'd**

| | | | |
|---|---|---|---|
| Wagner et al (1988b) | Reliability analysis-simulation | S | Models failures of the components. Models repair times for failure. Looks at a range of reliability |
| Lansey et al.(1990) | Minimum-cost design model chance constrained on probability of meeting demands. | A | Uncertainties in:<br>Future demands<br>Pressure Requirements<br>Pipe roughness |
| Bao and Mays (1990) | Reliability of water distribution systems | S | Distribution of Scenarios from Monte Carlo simulation. Probability of head being larger minimum required. |
| Goulter and Bouchart (1990) | Minimum-cost design model with reliability constraints on node performance | A | Probability distribution of demand at each node. Probability of node isolation mechanical failure and failure to meet demand |
| Duan and Mays (1990) | Reliability analysis of pumping systems. Frequency/ duration analyses. | A | Mechanical failure and hydraulic failure of pumps not networks. Eight parameters related to reliability, failure. Probability, failure frequency, cycle time, and expected un-served demand of a failure, expected number of failure, expected total duration of failures and total expected un-served demand. |
| Duan et al. (1990) | Optimal reliability-based design of pumping and distribution systems | A | Extension of work of Duan and Mays (1990) into the design of distribution networks. |
| Kessler et al. (1990) | Least Cost improvements in network reliability | S | Topological redundancy from alternative trees in the network. Level one redundancy. Different levels of acceptable service under component failure. |

**Table 4.1 Cont'd**

| | | | |
|---|---|---|---|
| Fujiwara and De Silva (1990) | Reliability-based optimal design of water distribution networks | A | Ratio of expected maximum demand to total water demand. |
| Awumah et al. (1990) Awumah et al. (1991) | Entropy-based measures of network redundancy | A | Associates reliability with redundancy. |
| Bouchart and Goulter (1990, 1991) | Improving reliability through valve location | A | Demands are not located at nodes. Variation in demand at node. Mechanical failure of links. Expected volume deficit. |
| Quimpo and Shamsi (1991) | Estimation of network reliability- cut sets approaches | S | Component reliability. Enumeration of cut-sets and path sets. Nodal pair reliability. Lumped systems |
| Jacobs and Goulter (1991) | Estimation of network reliability- cut set approaches | A To S | Probability of isolation Node Groups of nodes Probability of m links failing simultaneously. Probability of m simultaneous link failures causing network failures. |
| Cullinane et al. (1992) | Minimum cost model with availability constraints | S/A | Considers pipe, tanks and pumps. Repair Time for failures. |
| Wu et al. (1993) | Capacity weighted reliability | A | Connectivity based. Includes capacity of links. Reduced system by block reduction and path set method. |
| Park and Liebman (1993) | Redundancy Constrained minimum cost-model | S | Expected shortage due to pipe failure. Based on geometry of the network. Reduces system by block reduction and set methods. |

**Table 4.1 Cont'd**

| | | | |
|---|---|---|---|
| Jowitt and Xu (1993) | Predicting pipe failure effects on service | S | Failure of pipes. Simplified prediction of nodal conditions – network performance failure under failed pipes. Expected shortfalls at nodes. |
| Gupta and Bhave (1994) | Reliability analysis considering nodal demands and heads simultaneously | S | Failure of pipes and pumps node reliability, network reliability. |

Simulation approaches evaluate only sample (case by case) conditions identified for the computations; on the other hand, they can generate a broader range of reliability measures and enable a realistic interpretation of reliability. The advantage of the analytical approaches is that they consider the whole network instead of samples; however their main weakness is the interpretation of simplistic reliability measures.

Although both approaches have their own weaknesses and strengths, the simulation approaches' ability to permit the use of any reliability measure gives them a great advantage over the analytical approaches.

The reliability measure that can be derived concerning the hydraulic performance of the network may change regarding the simulation method it was purposed. 20 different reliability measures were listed by Wagner et al. (1988b) on Table 4.2 which can be obtained at the end of different simulation analysis.

**Table 4.2 Different Reliability Measures that can be obtained by Simulation approaches**

| Relation Parameter | Reliability Measure |
|---|---|
| Link- Related | • Number of pipe failures<br>• Percentage of time of failure time for each pump<br>• Percentage of failure time for each pipe<br>• Number of pump failures<br>• Total duration of failure time for each pump |
| System- Related | • Total system Consumption<br>• Total number of breaks<br>• Maximum number of breaks per event |
| Node –Related | • Total demand during the simulation period<br>• Shortfall<br>• Average Head<br>• Number of reduced service events<br>• Duration of reduced service events<br>• Number of failure events<br>• Duration of failure events |
| Event-Related | • Type of event<br>• Total number of events in the simulation period and system status during each event<br>• Interfailure time and repair duration |

Actually Table 4.2 gives an important reason not to have a currently available definition or proposed method for the assessment of the reliability of water distribution networks. Actually the reliability measure that was used for the assessment of the reliability of the network gives us only the reliability of the network for that measure. Therefore the reliability result obtained depends on which measures were purposed and how they are used.

## 4.3 Pressure Dependent Demand Theory

While designing the water distribution networks, the main aim is to supply adequate amount of water with adequate amount of pressure at all nodes under all of the conditions, like maximum demand and fire flow.

Although during the design stage all necessary requirements were met, in real life due to different reasons like accelerated growth, increase demands in some particular nodes, aging and mechanical malfunctioning of network elements, may cause temporary deficiency at some nodes due to decrease of pressure. In order to have required demand at any node, the head at that node must be greater or equal to minimum required residual head. Although traditional demand-driven approaches may be used to determine the nodes which are deficient in heads, these approaches can able to determine the reduced flow due to deficiency.

Temporary deficiencies in water distribution networks are generally caused by malfunctioning elements of that network. Pipe breaks are generally not considered during the design stage of networks. In order to repair a broken pipe it must be isolated from the system, but this may lead isolation of some nodes or pressure head decrease in some other nodes. In a typical water distribution network, although it can be used for different purposes such as pressure reducing and/or sustaining, a valve's main purpose is to isolate a pipe or number of pipes from the rest of the network. In the ideal case, valves are located at the start and end of each junction, so the target pipe or pipes may easily be isolated without disturbing the system. Although some of the junctions contain valves at each end, for most of the system this is not the case and it may need to close more than five or six valves for the isolation of a single pipe. Not only temporary deficiencies but also permanent changes can be done to existing water distribution networks like connecting to another pressure zone or adding new links to the system.

As the major disadvantage of demand-driven approaches is that, they fail to measure a partially deficient network performance. On the other hand the head-driven approaches have ability to fulfil this requirement. In head-driven approaches main emphasis is on the pressures. A node can only be supplied with its full demand if and only if the required pressure is supplied at that node. So in case of a deficiency in the pressure at that node, only a fraction of the demand can be extracted from that node.

The head-driven approaches are relying on pressure dependant demand theory. In this theory, the relation between the demand and available pressure are formulated by an equation and the simulation of the network is carried out using this equation in an iterative method.

There are different approaches to the pressure dependent demand theory. Bhave (1981, 1991) proposed an iterative method called node flow analysis to calculate the available flows at nodes under deficiency. However, it does not give a direct relation between head supplied and demand extracted, but it proposes an iterative solution using the head-driven approaches by categorising the nodes. The first study that directly relates pressure and nodal consumption was carried out by Germanopoulus (1985). However the main disadvantage of this model is that it has three constants, two of which have neither clear meanings nor described. Later on, another approach was purposed by Wagner et al. (1988b). Reddy and Elango (1989), assumes a fixed relation between residual head and corresponding consumption for water distribution networks, however their suggestion is though to be unsuccessful in point of their approach to their subject. Finally, the most satisfying model, that the fundamental equation is suggested by Chandapillai (1991), and then developed by Tanyimboh et al. (2001) (Nohutçu,2002). In this study, this model will be used with the name modified Chandapillai model (Nohutçu, 2002).

### 4.3.1 Modified Chandapillai Model

The model is based on the consideration of a consumer connection that leads flow from the network to an overhead tank and formulated as an equation (4.1).

| | |
|---|---|
| $H = H_{\min} + KQ^n$ ………………………………………………… | (4.1) |

Where,

$H$ :head (m)

$H_{\min}$ : minimum required head (m)

$Q$ : flow into overhead tank

$K, n$ : Constants

With the extension of equation (4.1) to a node by replacing the flow rate (Q) with the nodal consumption (c), node then is able to consume water for the head values higher than $H_{\min}$, it becomes then,

| | |
|---|---|
| $c = (\frac{H - H_{\min}}{K})^{1/n}$ ………………………………………………… | (4.2) |

For the case of $c=q^{req}$ (the demand), $H= H^{req}$ (the required head to consume the demand), it becomes

| | |
|---|---|
| $q^{req} = (\frac{H_{req} - H_{\min}}{K})^{1/n}$ ……………………………………………… | (4.3) |

| | |
|---|---|
| $\frac{1}{K^{1/n}} = \frac{q^{req}}{(H_{req} - H_{\min})^{1/n}}$ ……………………………………………… | (4.4) |

Finally, the substitution of K in to equation 4.2 gives the relation of consumption (c ) for the corresponding head.

| | |
|---|---|
| $c = (\frac{H - H_{\min}}{H_{req} - H_{\min}})^{1/n}$ ……………………………………………… | (4.5) |

As the derivation of the model is satisfactorily provided this method comes out to be better than the other. By replacing the heads (H) with pressures (P), and setting the minimum pressure to zero, our final equation that is called modified Chandapillai model is achieved (equation (4.6))

| | |
|---|---|
| $c = q^{req}.(\frac{P}{P^{req}})^{1/n}, \qquad 0 \le P \le P^{req}$ ……………….. | (4.5) |

## 4.4 Methodology

In this study, the nodal and system hydraulic reliability factors were calculated assuming that some parameters in the hydraulic model have random properties, such as demand, friction factors, and storage tank water level.

The water consumption at the demand nodes, $Q_d$, can not be measured or calculated but can be approximated by using some methods. The rate of water consumption at a node depends on the population served by that node, type of the demand (domestic, public, commercial, etc.), time of the year and the time of the day. In the design of water distribution systems, it is very difficult to predict the future demands for each node. Even for the existing water distribution systems, the nodal demands change due to many factors, such as new users or an increase in the number of existing users. Therefore, the demand values extracted from node showing the consumption is considered as random variable with uncertainity associated with it. The hydraulic uncertainity due to the randomness of water demand can be incorporated by assigning an approriate probability distribution.

The pipe roughness coefficient refers to a value that defines the roughness of the interior of a pipe. Two common roughness coefficients are the Hazen-Williams C-value and the Darcy-Weisbach f-value. Although the Darcy-Weisbach term is generally considered more accurate and flexible by giving information about flow regime, it is also more complicated and difficult to determine. Therefore, the Hazen-Williams C-value is commonly used in network modelling as in this study. The C-values range from 20 to 150. The higher the value, the smoother the interior surface of the pipe and the greater the carrying capacity of the pipe. Since the determination of C-values at the site is very difficult, generally the approximate values in literature are used by knowing the material type and installation year of each pipe. Depending on this reason, the uncertainity in determining the roughness can be accounted for by specifying an appropriate distribution for C.

Water level at storage tanks is another parameter that needed to be put into model to make reliable calculations but as it depends on the management of the system; this value can be taken as another random value for the distribution system.

Although, the list of random natured parameters of a water distribution system can lengthened; such as, pump working hours, fire demands; for this study these three parameters were chosen to simulate the system. The model used has three components: (1) random number generation, (2) hydraulic simulator and (3) computation of nodal and system reliability (Figure 4.1).

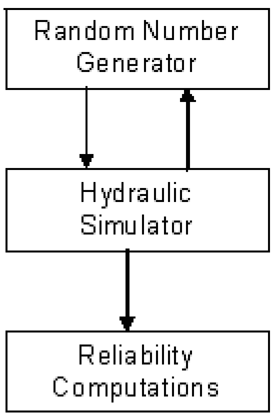


**Figure 4.1 Algorithm of the model**

The first step is the generation of values for water demand, $Q_d$; pipe roughness coefficient, C and water level at tank, $l_t$. For each set of values generated, a hydraulic network simulator (HAPMAM) is used to compute the supplied pressure heads $H_s$ for the nodes throughout the water distribution system. The required pressure head, $H_d$ , at the nodes can be treated as constant with lower and upper bounds or as a random variable . In this study, it is set as a constant value with a lower bound specified for each node seperately. The last part of the method is the computation of nodal and system reliabilities using appropriate equations.

As stated before, the nodal reliability can be defined as the probability that the supplied pressure head at the given node is greater than or equal to the required minimum pressure head. The approach used here is to compute the conditional probability in terms of pressure head, provided that the water demand is fully satisfied; i.e. $Q_s = Q_d$ . Then using the required minimum pressure head $H^l_d$ as the lower bound, the nodal reliability is given as,

| | |
|---|---|
| $R_n = \Pr\left(H_s > H_d^l\right) = \int_{H_d^l}^{\infty} f_s(H_s) d_{H_s}$ ............................. | (4.6) |

where;

$R_n$ : Nodal Reliability

$H_s$ : Available Head at Node

$H_d{}^l$ : Required Head at Node

$f_s(H_s)$ : Probability Density Function of Supplied Pressure Head

Also for calculating the hydraulic system reliability by using the consumption values, formula (4.7) was used for every node to calculate the nodal consumption hydraulic reliability.

| | |
|---|---|
| $R_n = \dfrac{Q_a}{Q_r}$ ............................. | (4.7) |

Where;

$R_n$ : Nodal Reliability

$Q_a$ : Available Flow at Node

$Q_r$ : Required Flow at Node

After calculating the reliabilities of nodes, the system reliability was calculated for both approaches by using the weighted mean of the nodal reliabilities as in formula (4.8).

| | | |
|---|---|---|
| $R_{sw} = \dfrac{\sum_{i=1}^{N} R_{ni}.W_i}{\sum_{i=1}^{N} W_i}$ | ................................................... | (4.8) |

Where;

$R_{sw}$ : Weighted System Reliability.

$R_{ni}$ : Reliability of Node i.

$W_i$ : Weight of Node i.

$N$ : Number of Nodes.

# CHAPTER 5

# CASE STUDY

## 5.1 Aim of the Study

In this study, the main aim is to determine the hydraulic system reliability of water distribution system, together with the implementation of both temporal and spatial variations of nodal demands using GIS and SCADA data integration on an existing part of Ankara Municipality Water Distribution Network, North 8-1 (N8-1) pressure zone.

## 5.2 Ankara Water Distribution Network

Ankara Water Distribution Network serves roughly 3,650,000 people by providing 800,000 $m^3$ of potable water per day from two treatment plants; İvedik and Pursaklar. Water distribution of Ankara has 36 pump stations, 54 tanks serving five different main pressure zones (see Figure 5.1)

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

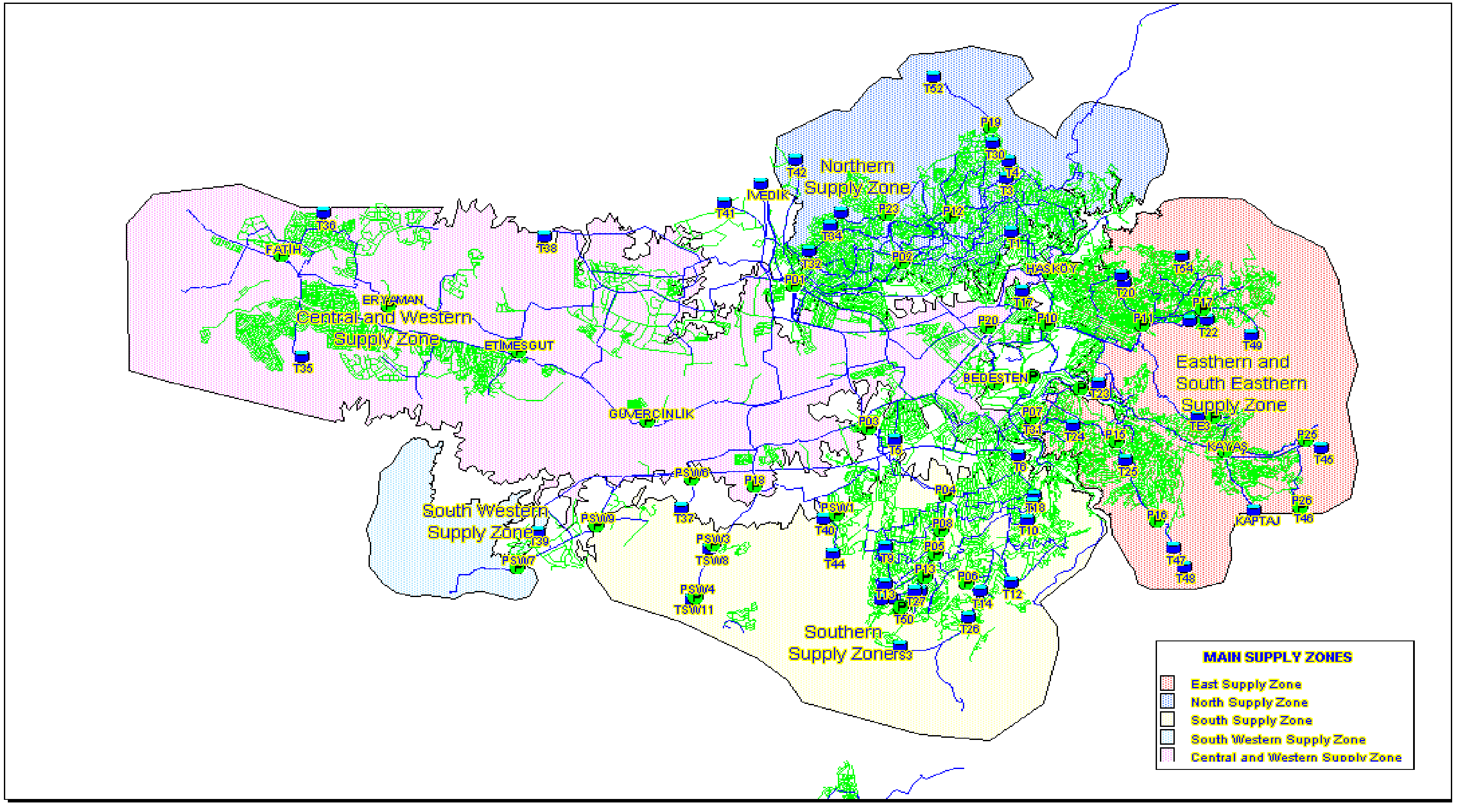


**Figure 5.1 Main pressure zones of Ankara**

Each main pressure zone has its own sub – pressure zones, which have been divided according to the elevations of the concerned areas.

### 5.3 Study Area, N8-1 Pressure Zone

In this study N8-1 (North Supply Zone, Pressure Zone 8-1) zone of the Water Supply System was selected to carry out the reliability analysis. The pressure zone is located at Keçiören County, and serves approximately 30,000 users. All pipes are ductile iron and they were laid down in 1992. In the pressure zone, there are two pump stations and one storage tank. The pump station, P12 composed of three parallel pumps can be used with various combinations. The storage tank, T30 is a rectangular tank with a height of 6.5 m and volume of 2500 $m^3$. The tank and the pump station are connected by a 500 mm diameter, ductile main transmission line (see Figure 5.2).

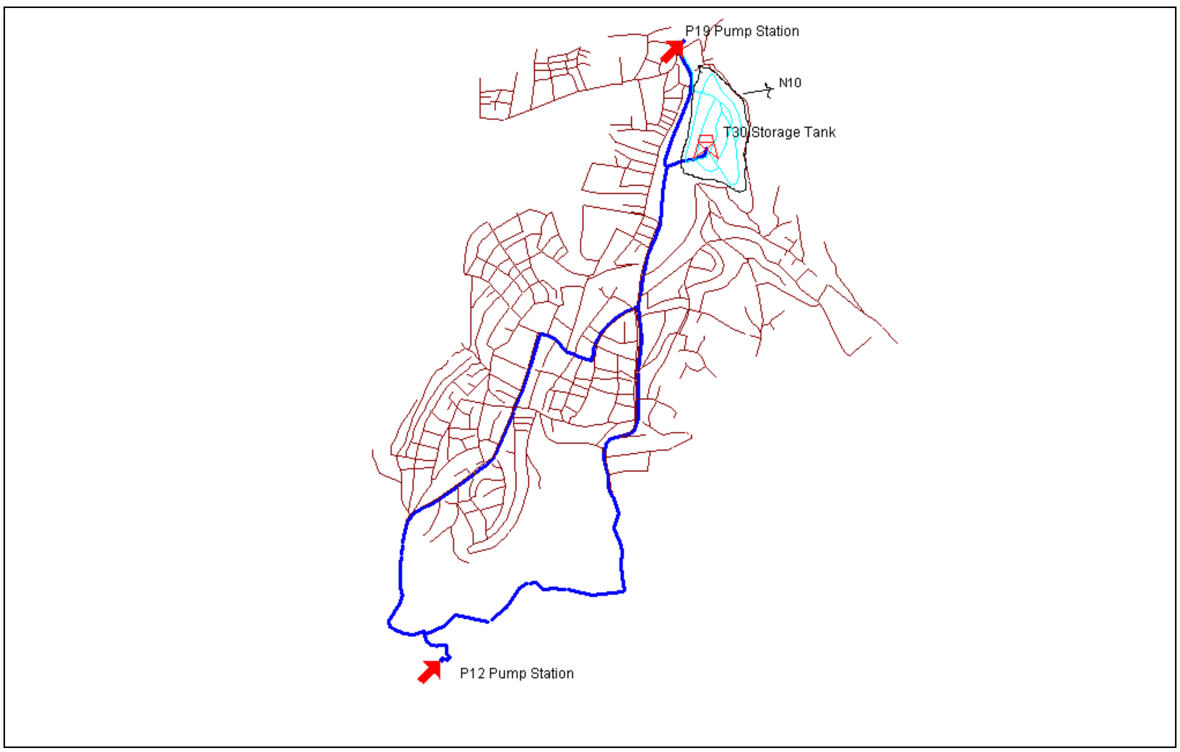


**Figure 5.2 N8-1 Pressure Zone**

N8-1 Pressure Zone is located at the end of the North Supply Zone. The North Supply Zone consists of 10 sub–pressure zones, and mainly takes water from the İvedik Water Treatment Plant by the help of pump station P01. P01 distributes water to the zones N-3 and N-4 while providing water for pump station P02 that is located on N-4 sub pressure zone. P-02 distributes water to N-5 and N-6 pressure zones together. Also pressure zones N-5 and N-6 take water from Pursaklar Treatment Plant, by the use of another pump station PN-1. The pump station P12 transmits water to the pump station P23 which is located on N7 pressure zone. P12 provides water also to N8-1 and P19 pump station (see Figure 5.3).

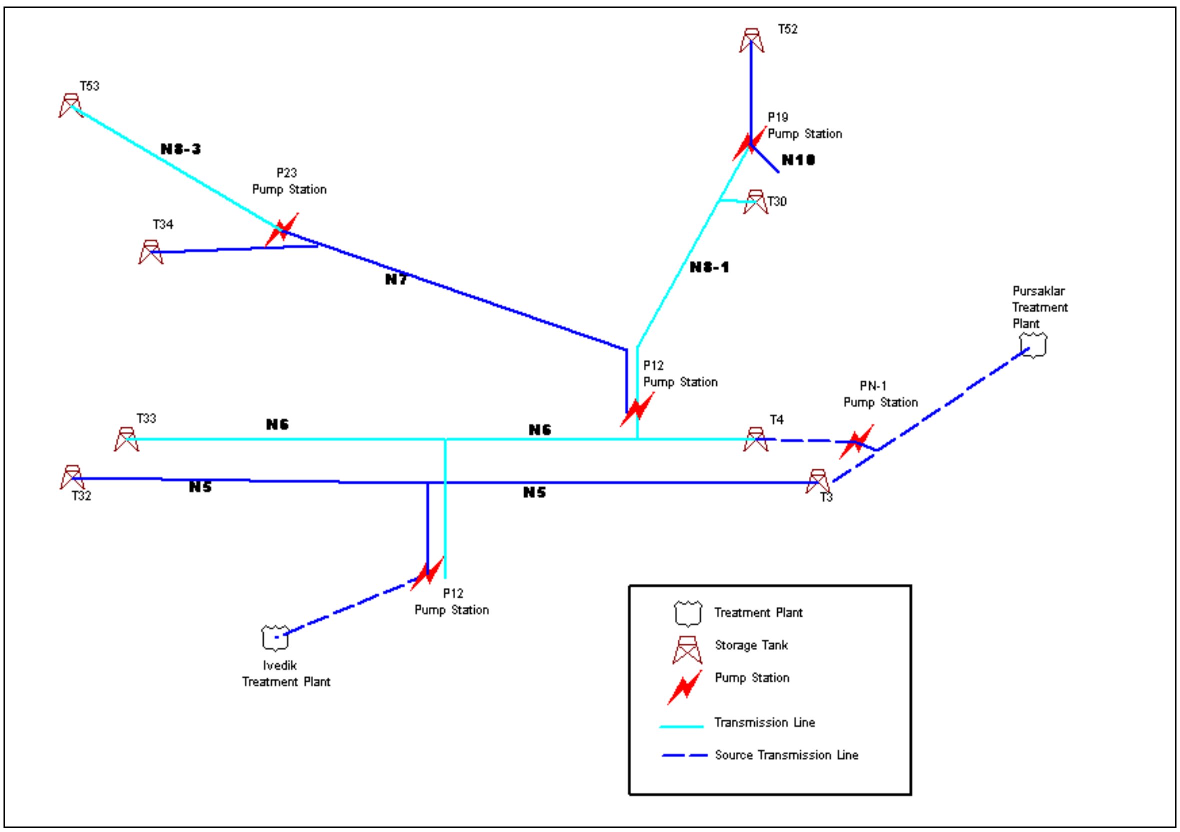


**Figure 5.3 Schematic View of North Pressure Zone**

## 5.4 Hydraulic Modeling of N8-1 Pressure Zone

The N8-1 pressure zone is located at the north edge of the North zone of the Ankara Water Distribution Network. Initially, field studies were carried on the network to determine the connections, borders and characteristics of the network. The inconsistencies between, the drawings and field situation were noted and corrected before the hydraulic model was constructed.

Furthermore, by dividing the pressure zone into the sub-zones, the boundary check was carried on. As a result of this study, not only the control on the network was increased, but also zonal leakages were determined and prevented. After the inner valve connections check, further studies for modeling were carried on.

### 5.4.1 Skeletonization

The N8-1 Pressure zone contains pipes that have diameters ranging from 100 mm to 500 mm. Although all the pipes can be put in to model, skeletonization was carried to optimize the model calculations. In this context, 100 mm diameter pipes were discarded except the only one that forms an important loop with major diameter pipes; furthermore, most of the 125 mm pipes were discarded but the ones that form major loops. Finally, the system was represented with total number of 233 pipes and 174 nodes ( Figure 5.4).

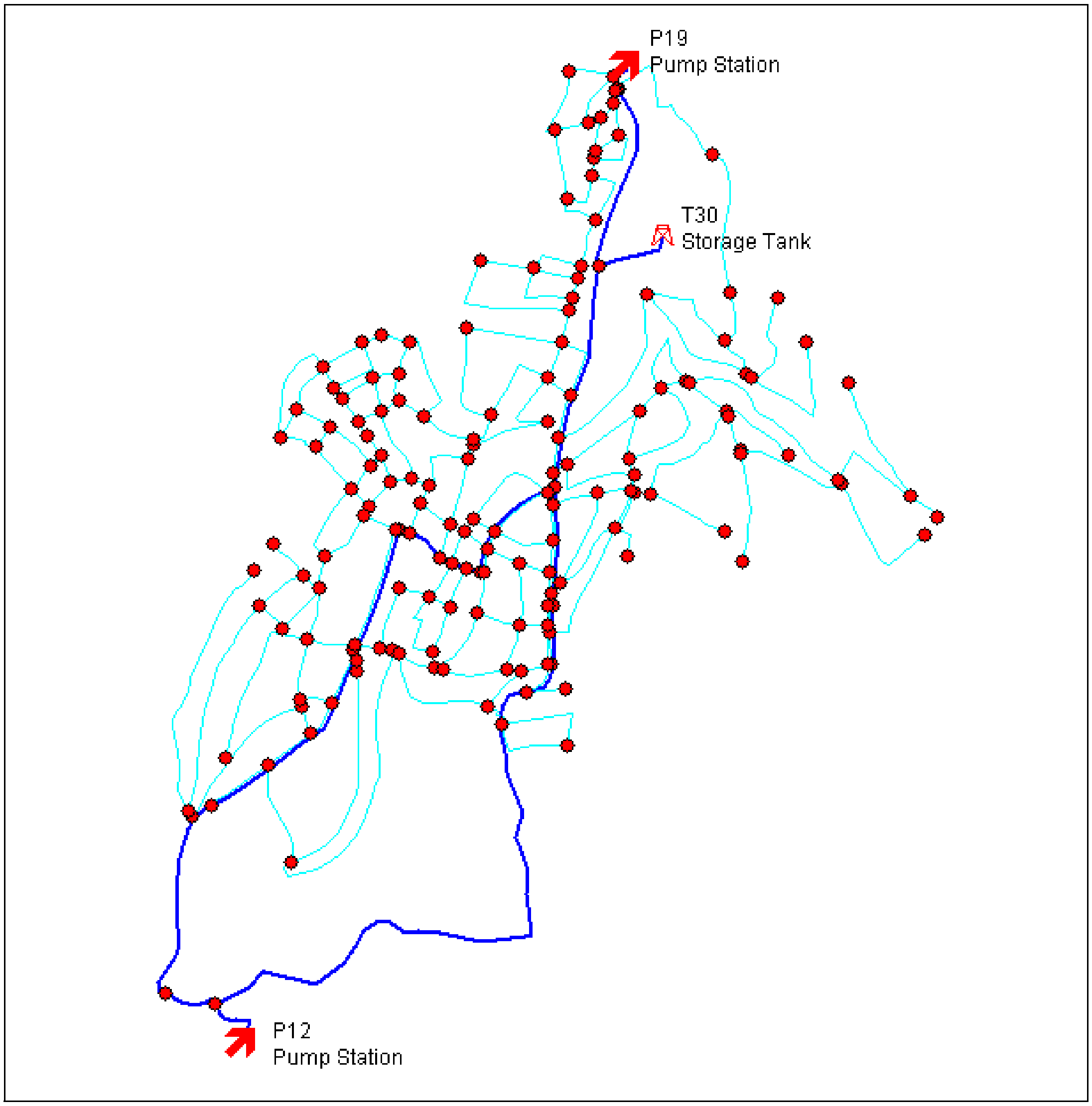


**Figure 5.4 Skeletonized N8-1 Pressure Zone**

### 5.4.2 Nodal Weight and Required Pressure Calculations

Generally the nodal weights are calculated using the pipe lengths of the corresponding neighboring nodes (Eker, 1998). However, in this study Service Area Method (**SAM**) (Mısırdalı and Eker, 2002) was used together with the consumption data to calculate the nodal weights of the system. Initial assumption is that the nodes that are just located on the main transmission line were not serving to the consumers. Only the distribution lines are serving to the consumers. Next assumption is that, the main transmission line and the nodes on the main transmission line were not serving to the consumers so they were extracted before running the software, HYDSAM. Then by using SAM, together with the borders of the pressure zone, service areas for every node were determined (Figure 5.5).

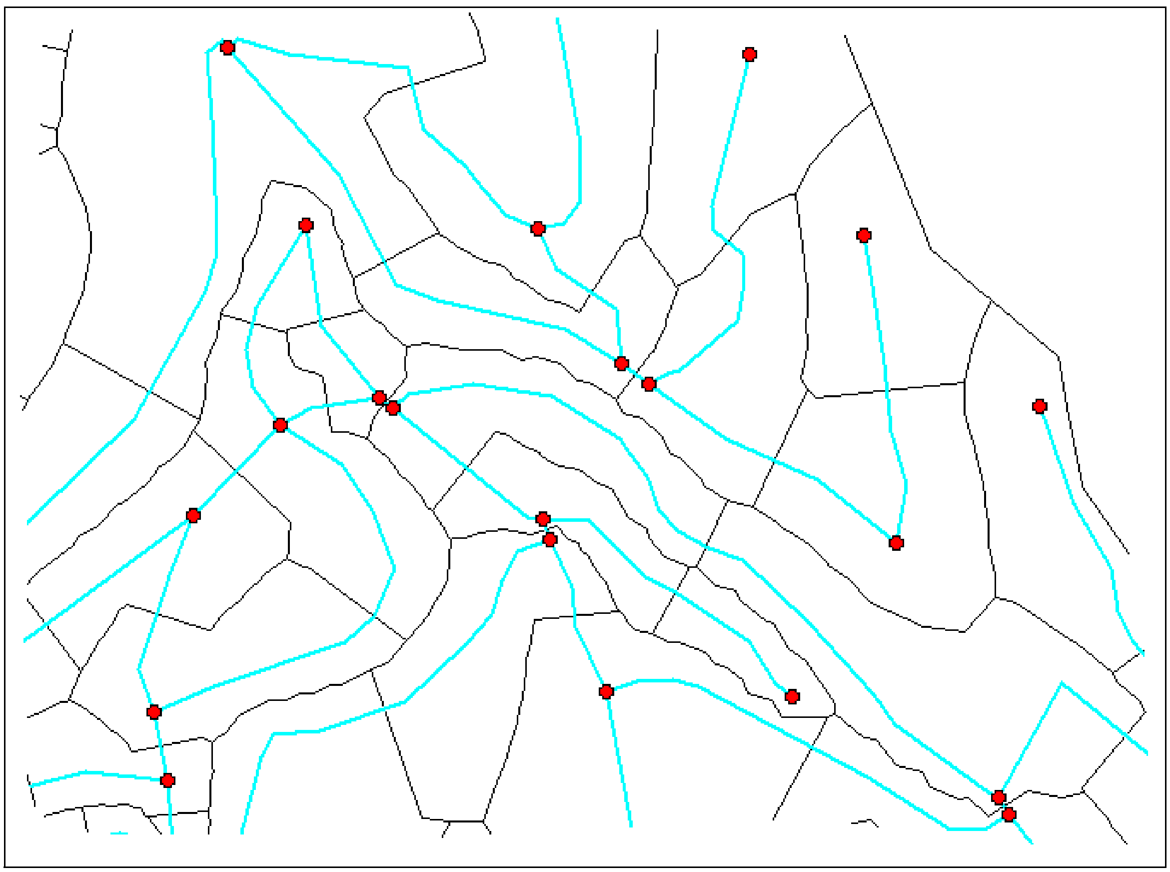

**Figure 5.5 Nodal Service Area Calculation of N8-1 near Kanuni District Using SAM**

After having drawn the service borders for each consumption node, the building layer was overlaid, assuming that the buildings inside the area were being served from the corresponding node (Figure 5.6). In ASKI, consumer information data are being kept in Consumer Information System that enables us to monitor the consumption of every user.  In this study to be consistent with the time period of daily demand curves of the area, the annual consumption data of the whole 2002 year were used. The total yearly consumption of each building inside of each area was calculated and by dividing areal consumptions to the total consumption the nodal weights were found.

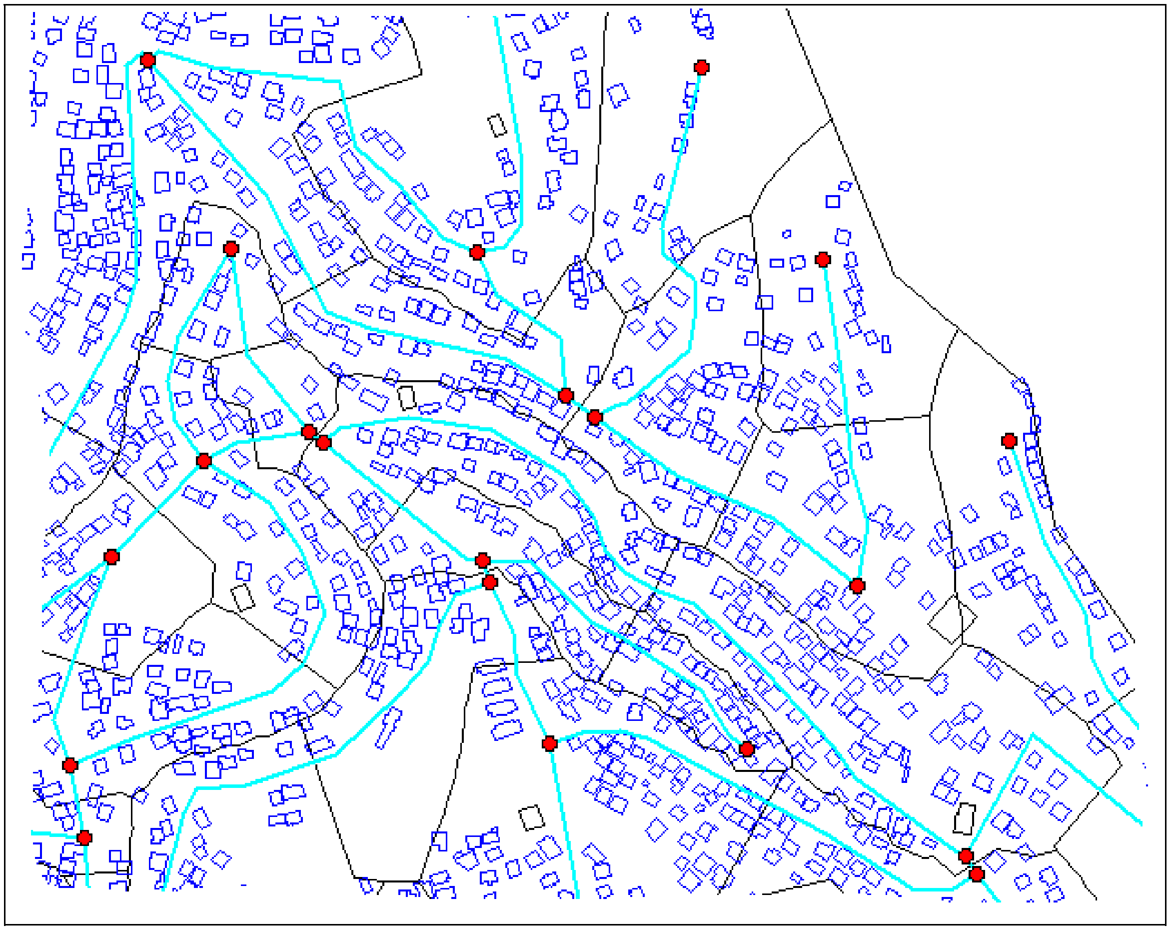

**Figure 5.6 Nodal Areas together with Buildings**

Basic advantage of this approach is that, by locating each single building in the area, the nodal weight can be calculated more realistically. As all of the area can not be homogenous, concerning consumptions, the distribution of the consumption can be observed more clearly. Also, although we assume that all of the nodes in the distribution network are serving as a consumption node, some nodes can come out to be not serving to any building as its area contains none (see Section 3.1.5.2).

Furthermore, the required pressure was also determined according to SAM. Not only the consumption data but also the elevation of each building can be found from the buildings layer. To be consistent with Chandapilla's approach (1991) (see Section 4.3.1) for every building a minimum head and required head was calculated by just adding 5m to elevation for minimum head and 25 m for required head. The maximum of the required head was chosen to be the required head for that node. On the other hand, for determination of the nodal elevation, minimum pressure heads were compared; and the smallest one was chosen as node's elevation.

### 5.5 Data Collection

Data for modeling N8-1 was taken from different departments of ASKI as different departments are responsible for managing and storing the relevant data. However In this study, data were taken from, ASKI Data Processing Center and SCADA Center.

**Table 5.1 Required Pressure calculations for node 3 with node elevation 1078.65 and required head 1107.81**

| Building Elevations (m) | Minimum Head (m) | Maximum Head (m) |
|---|---|---|
| 1076.45 | 1081.45 | 1101.45 |
| 1073.82 | 1078.82 | 1098.82 |
| 1077.98 | 1082.98 | 1102.98 |
| 1075.7 | 1080.7 | 1100.7 |
| 1082.81 | 1087.81 | 1107.81 |
| 1073.65 | 1078.65 | 1098.65 |
| **Node 3 Elev. 1065.81** | **Min. B. Elev. 1078.65** | **Max. B. Elev. 1107.81** |

### 5.5.1 ASKI Data Processing Center

N8-1 Pressure Zone is taken from the digitized maps of ASKI Data Processing Center. These maps contain the necessary information of pipes, pumps and tanks in digital environment. The Center uses Mapinfo Professional as GIS tool for objects. The Center has the whole network data of Ankara, in link/junction representation, together with the characteristics of pipes in schematic base. The center is responsible for new data input and updating process in CAD environment.

The whole Ankara Water Distribution Network Map, together with the buildings, roads etc. is held by the Data Processing Center together with ASKI Computer Center Staff.

### 5.5.2 ASKI Facilities Department SCADA Center

The main purpose of a water distribution SCADA system is to compile data concerning the operation of the water distribution system and to allow automated control of certain components of the water system such as pumps.

ASKI Facilities Department SCADA Center is responsible for collecting transmitting and storing data from various control points of the network. There are 54 storage tanks, 36 pumping stations and 15 additional measuring points monitoring the system.

The information is transmitted from the station to the SCADA Center, when there is a change in the value with a certain percentage change. For example on a storage tank, when the elevation of water level change with 2 cm, this information is transmitted but not for 0.3 cm. Furthermore, the SCADA Center sends transmission to the station in certain time periods to check if there is any change in the value of data via radio waves.

The system of ASKI SCADA center collects data that are necessary for daily implementation and supervisory but these data can be used mainly for leakage detection and daily demand curve generation with the appropriate calculations.

### 5.5.3 Daily Demand Curves of the Pressure Zone Using SCADA

Daily demand curve is a graphical representation of water usage of the consumers in a pressure zone as a function of time. Daily demand curve for a selected day can be extracted using SCADA data; the needed data for calculating the daily demands are storage tank levels and pump discharges.

Daily demand curve is derived using continuity equation. For the pressure zone N8-1, the flow parameter is the flow supplied from the pump station P12. The outflow parameters are consumption and the P19 discharge that delivers water to the N10 pressure zone. The storage tank T30 stores water when there is excess amount of water supplied by the pump station P12 and delivers when more demand is needed.

| | |
|---|---|
| $\bar{I} - \overline{Q} = \dfrac{dS}{dt}$ ………………………………….. | (5.1) |

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

$\overline{Q}$ : average outgoing flowrate from the system for a period of time dt, $m^3$ / hr

dS: Storage in the tank for a period of dt, $m^3$

SCADA Center collects data concerning the amount of the flow supplied by pump stations P12 and P19 in $m^3$ / hr. The discharge values of each pump station can be taken out using the 1 minute queries that show the value of the discharge at every minute. Then, together with the pump working data, the errors inside the data are filtered out as the system may contain errors. After correcting the values the average water pumping discharge were calculated by taking the average of the values for respective hour (see Table 5.2)

The flow supplied or stored by tank T30 was computed by using the volumetric changes of the water in the tank. T30 is composed of two tanks tank1 and tank2 that are connected. Although the water level should be same in both of the tanks, they differ in small values actually. But taking the average of the two levels, the average water level in the tank can be obtained. The storage tank T30, has a height of 6.5 m and volume of 2500 $m^3$, by dividing them, the base area of T30 can be approximated as 384.62 $m^2$. For these calculations again 1 minute queries were used and single values at the start of each hour was taken and used in the calculations (see Table 5.3).

**Table 5.2 Pump Station Discharge for N8-1 Pressure Zone**

| **Date:** | **11/10/2002** | |
|---|---|---|
| **Hour** | **P12 Discharge ($m^3$ / hr)** | **P19 Discharge ($m^3$ / hr)** |
| **0** | 607.9189206 | 130.6125 |
| **1** | 616.192051 | 130.6125 |
| **2** | 620.4323018 | 130.6125 |
| **3** | 577.3842932 | 130.6125 |
| **4** | 563.048099 | 130.6125 |
| **5** | 608.4486146 | 130.6125 |
| **6** | 192.9936576 | 130.6125 |
| **7** | 0 | 130.6125 |
| **8** | 318.4923047 | 130.6125 |
| **9** | 638.2522097 | 130.6125 |
| **10** | 568.2050565 | 130.6125 |
| **11** | 577.5548016 | 130.6125 |
| **12** | 602.7090208 | 130.6125 |
| **13** | 594.9490182 | 130.6125 |
| **14** | 580.9944086 | 130.6125 |
| **15** | 448.282113 | 130.6125 |
| **16** | 0 | 130.6125 |
| **17** | 314.9205273 | 130.6125 |
| **18** | 584.1502792 | 130.6125 |
| **19** | 583.0154992 | 130.6125 |
| **20** | 613.638618 | 130.6125 |
| **21** | 595.737355 | 130.6125 |
| **22** | 590.625 | 130.6125 |
| **23** | 611.4672521 | 130.6125 |
| **24** | 623.1705156 | 130.6125 |

Using hourly volumetric changes of storage tank T30 and discharges provided by the pump station together, the daily consumption of the area can be determined as given in Table 5.4. This process was carried on for approximately every day starting from May 2002 to the end of December 2002 for this pressure zone. Although all the curves representing the daily consumption were drawn, in this study not all of the curves were used. Due to the huge errors in the data of respective days, caused by data transmission and storage, some days were neglected and assumed as they are not representing the consumer demands. Furthermore, due to the maintenance and repairs on that area, daily demand curves of some days were not eligible to use as there were no water served to the consumers.

**Table 5.3 T30 Volumetric Change Calculation**

| **Date:** | **11/10/2002** | | |
|---|---|---|---|
| **Hour** | **T30 Average Water Level (m)** | **T30 Average Volume ($m^3$)** | **T30 Volumetric Change ($m^3$ / hr)** |
| **0** | 2.04 | 784.87 | |
| **1** | 2.70 | 1038.47 | 253.61 |
| **2** | 3.41 | 1311.31 | 272.84 |
| **3** | 4.00 | 1539.68 | 228.37 |
| **4** | 4.55 | 1750.02 | 210.34 |
| **5** | 5.32 | 2045.70 | 295.68 |
| **6** | 4.88 | 1875.02 | -170.68 |
| **7** | 3.88 | 1492.81 | -382.22 |
| **8** | 3.53 | 1359.39 | -133.41 |
| **9** | 3.63 | 1397.85 | 38.46 |
| **10** | 3.52 | 1355.79 | -42.07 |
| **11** | 3.31 | 1271.65 | -84.14 |
| **12** | 3.20 | 1229.58 | -42.07 |
| **13** | 3.09 | 1188.72 | -40.87 |
| **14** | 3.09 | 1188.72 | 0.00 |
| **15** | 2.85 | 1096.17 | -92.55 |
| **16** | 1.75 | 674.29 | -421.88 |
| **17** | 1.41 | 542.07 | -132.21 |
| **18** | 1.57 | 605.78 | 63.70 |
| **19** | 1.85 | 710.34 | 104.57 |
| **20** | 2.18 | 840.15 | 129.81 |
| **21** | 2.62 | 1006.02 | 165.87 |
| **22** | 3.06 | 1175.49 | 169.47 |
| **23** | 3.49 | 1341.36 | 165.87 |
| **24** | 3.90 | 1499.90 | 158.54 |

**Table 5.4 Hourly Consumption Values of N8-1 Pressure Zone on 11.10.2002**

| **Date:** | **11/10/2005** | | | |
|---|---|---|---|---|
| **Hour** | **P12 Discharge ($m^3$ / hr)** | **P19 Discharge ($m^3$ / hr)** | **T30 Volumetric Change ($m^3$ / hr)** | **Consumption ($m^3$ / hr)** |
| **0** | 607.92 | 130.61 | | |
| **1** | 616.19 | 130.61 | 253.61 | 231.97 |
| **2** | 620.43 | 130.61 | 272.84 | 216.98 |
| **3** | 577.38 | 130.61 | 228.37 | 218.40 |
| **4** | 563.05 | 130.61 | 210.34 | 222.09 |
| **5** | 608.45 | 130.61 | 295.68 | 182.16 |
| **6** | 192.99 | 130.61 | -170.68 | 233.06 |
| **7** | 0.00 | 130.61 | -382.22 | 251.61 |
| **8** | 318.49 | 130.61 | -133.42 | 321.30 |
| **9** | 638.25 | 130.61 | 38.46 | 469.18 |
| **10** | 568.21 | 130.61 | -42.07 | 479.66 |
| **11** | 577.55 | 130.61 | -84.14 | 531.08 |
| **12** | 602.71 | 130.61 | -42.07 | 514.16 |
| **13** | 594.95 | 130.61 | -40.87 | 505.20 |
| **14** | 580.99 | 130.61 | 0.00 | 450.38 |
| **15** | 448.28 | 130.61 | -92.55 | 410.22 |
| **16** | 0.00 | 130.61 | -421.89 | 291.27 |
| **17** | 314.92 | 130.61 | -132.22 | 316.52 |
| **18** | 584.15 | 130.61 | 63.70 | 389.83 |
| **19** | 583.02 | 130.61 | 104.57 | 347.83 |
| **20** | 613.64 | 130.61 | 129.81 | 353.22 |
| **21** | 595.74 | 130.61 | 165.87 | 299.26 |
| **22** | 590.63 | 130.61 | 169.48 | 290.54 |
| **23** | 611.47 | 130.61 | 165.87 | 314.99 |
| **24** | 623.17 | 130.61 | 158.54 | 334.01 |

Finally, 156 different daily demand curves were obtained to represent the yearly average daily demand of year 2002. After that, by taking the hourly averages of each day, a representative average daily demand curve was obtained. While doing this, not only the averages but the standard deviations were calculated for every single hour as there may be fluctuations from day to day and month to month in hourly demands. These results can be seen on Table 5.5 with the respective standard deviation values.  Also daily demand curve for 11.10.2002, and average yearly daily demand can be seen on figures 5.7 and 5.8

**Table 5.5 Hourly Demand Values of N8-1 for year 2002**

| Hour | Mean Demand ($m^3$/hr) | Standard Deviation Of Demand ($m^3$/hr) |
|---|---|---|
| **1** | 235.902 | 70.617 |
| **2** | 209.627 | 70.958 |
| **3** | 201.560 | 79.060 |
| **4** | 204.449 | 87.552 |
| **5** | 204.175 | 79.335 |
| **6** | 234.104 | 69.997 |
| **7** | 287.485 | 92.146 |
| **8** | 333.265 | 91.358 |
| **9** | 388.496 | 90.137 |
| **10** | 438.358 | 92.328 |
| **11** | 481.170 | 94.519 |
| **12** | 489.732 | 95.891 |
| **13** | 477.809 | 101.414 |
| **14** | 444.613 | 91.771 |
| **15** | 412.092 | 90.173 |
| **16** | 394.713 | 83.419 |
| **17** | 386.852 | 104.793 |
| **18** | 390.746 | 82.313 |
| **19** | 381.893 | 90.423 |
| **20** | 380.399 | 87.762 |
| **21** | 360.254 | 97.483 |
| **22** | 336.731 | 72.223 |
| **23** | 316.912 | 78.336 |
| **24** | 309.261 | 113.122 |

Not only hourly but by dividing the day to 4 periods, and also taking a day totally, periodically and daily means and standard deviation for demand was calculated (see Table 5.6). The reason for this calculation is to see the sensitivity of the reliability analysis to the daily fluctuation on a day and between the periods (see Figure 5.9).

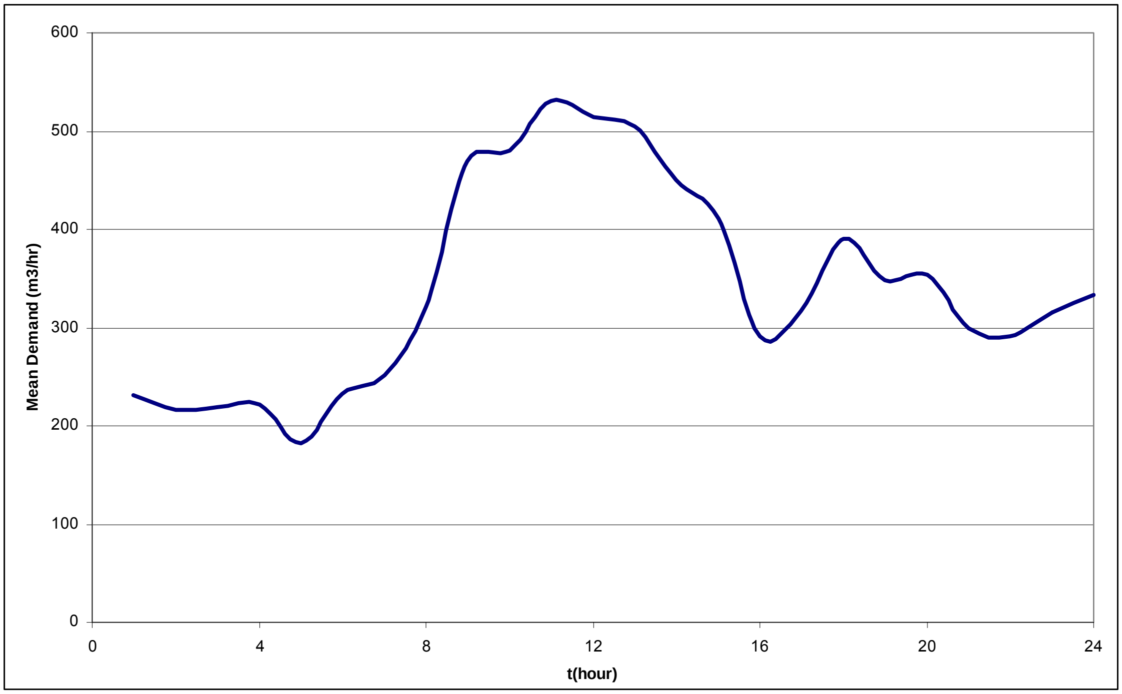


**Figure 5.7 Daily Demand Curve of N8-1 for 11.10. 2002**

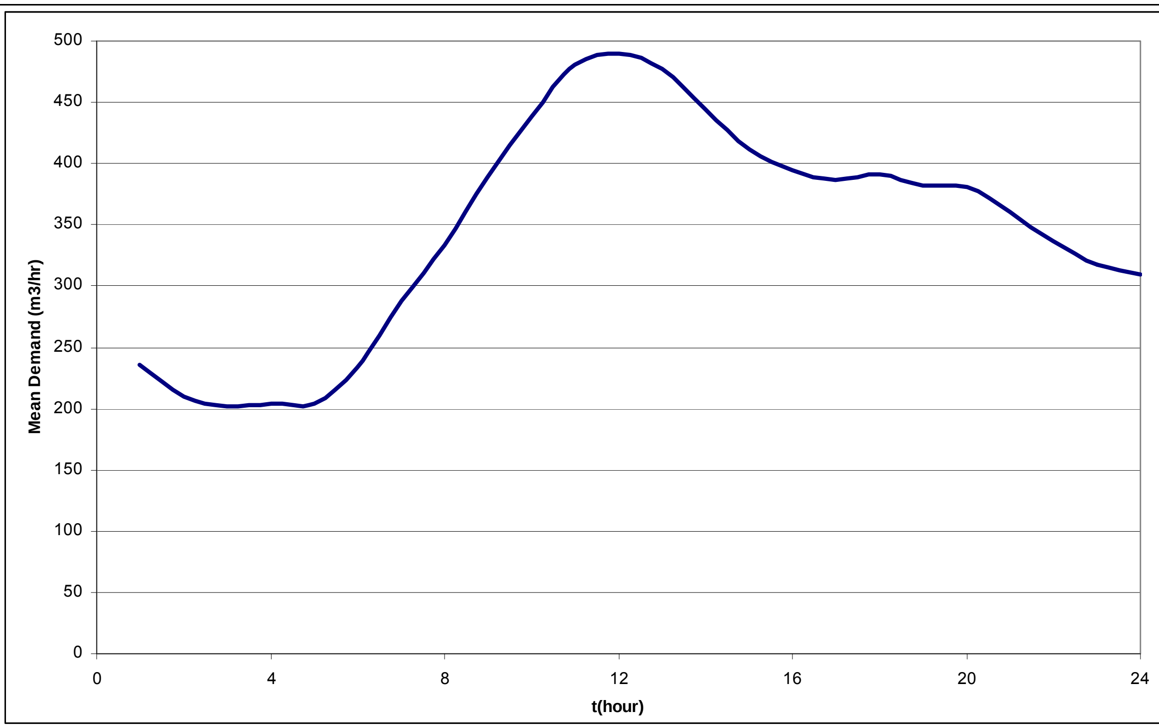


**Figure 5.8 Representative Daily Demand Curve of N8-1 for year 2002**

**Table 5.6 Demand Values for N8-1 in periodic bases**

| Period | Mean Demand ($m^3$/hr) | Standard Deviation ($m^3$/hr) |
|---|---|---|
| **01_06 (P1)** | 214.970 | 77.658 |
| **07_12 (P2)** | 403.085 | 118.796 |
| **13_18 (P3)** | 417.804 | 98.201 |
| **19_24 (P4)** | 347.575 | 95.055 |
| **TOTAL** | 345.858 | 126.870 |

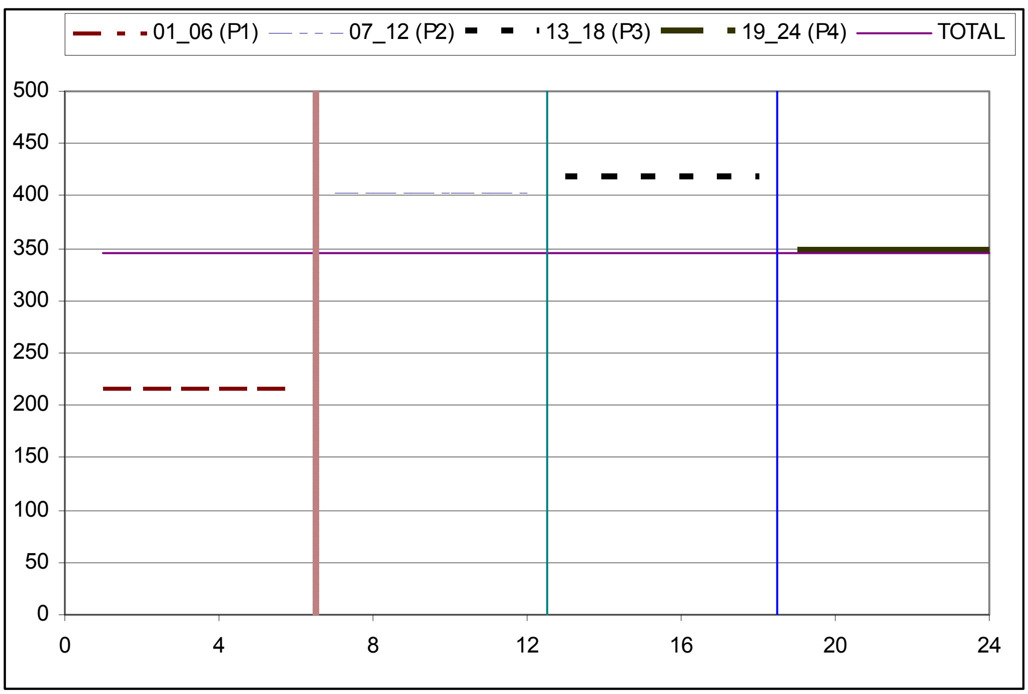


**Figure 5.9 Daily Demand Curve of N8-1 for 6 hour periods**

These values, means and standard deviations, were used during the reliability analysis while generating random demand values. To simulate any period or hour during the analysis, 500 random demand values were generated by the use of random number generator, that lead us with the possible demand values for the whole system for that hour.

### 5.6 Software

Today, for the analysis of water distribution networks, there are lots of available software. The graphical user interface mainly gains importance in developing these kinds of software packages. Although, there are lots of types of analysis programs, these programs have limited capacities under complex conditions and integration with SCADA and GIS. Not only analyzing data, but querying, updating, managing and processing the available data are very useful; because of this, GIS integrated software are in the trend of development.

The available packages work under demand driven method. The demand value of a node is given in the software and it is assumed that the demand assigned at a node is extracted whatever the actual pressure head is. In this study, because of this disadvantage, common software was not used. However, for overcoming this problem, a new network analysis program generated, with the ability to calculate the partial flows at the nodes under deficient pressures was used. The program code was written at Matlab and named as Hydraulic Analysis Program with Mapinfo and Matlab (HAPMAM) (Nohutçu, 2002). Not only calculating the partial flows but highly integrated mode of GIS integration of the program provides flexibility and visual help for the user.

As the N8-1 network configuration provided by ASKI is under Mapinfo environment and HAPMAM is integrated with Mapinfo, Mapinfo Professional was used for both in modeling and analysis part of the study. In modeling process, in order to manage and process the data Mapbasic was used to simplify the processes.

### 5.6.1 Mapinfo Professional

MapInfo Professional is a comprehensive desktop mapping tool that enables performing complex geographic analysis such as redistricting, linking to remote data, dragging and dropping map objects into applications creating thematic maps that emphasizes patterns in data (Mapinfo, 1998); databases can be created and also previously obtained data from databases; CAD packages, GIS applications and spreadsheets can be used in MapInfo. It can be used by decision makers as it enables the users to process databases by SQL queries.

It is designed to integrate easily with existing information systems like, Excel, dBase, delimited ASCII files and 123 with direct link ability to Microsoft office applications. Also there are lots of third party programs that work under Mapinfo to solve particular problems.

### 5.6.2 Map Basic

Mapbasic is a programming environment that is used by MapInfo Professional, used to create mapping applications. Mapbasic is the ideal programming language to create custom mapping applications, extend the functionality of MapInfo Professional, automate repetitive operations and integrate MapInfo Professional. Using Mapbasic enables user to copying, editing, querying the databases. The information can be transferred from other databases and files as well as it has ability to run different applications.

### 5.6.3 Hydraulic Analysis Program with MapInfo and Matlab

Matlab is a powerful, easy to use, comprehensive environment and a well-known package program for technical computing. It provides workspace with ability of computation visualization and programming for different professions such as scientists, engineers and technical staff. High capability of solving specialized matrix operations enables the hydraulic analyzers to simplify their operation time.

HAPMAM is written by Nohutçu (2002) in Matlab environment; it takes the needed information from Mapinfo environment and it returns the output of the analyses to the same program after the analysis. If needed the program can be worked separately form Mapinfo by providing it with the input data in text format.

The data that the program extract form the database can be grouped into four major categories; pipe data, node data, pump data and fixed grade node data. Also an element information data must be provided that informs the system for the number of elements (pumps, pipes, nodes) in the network. Although the fully available package is available for this study, the program was used with minor modifications as the analyses need number of iterations and loop simulations for the calculations (see Figure 5.10).

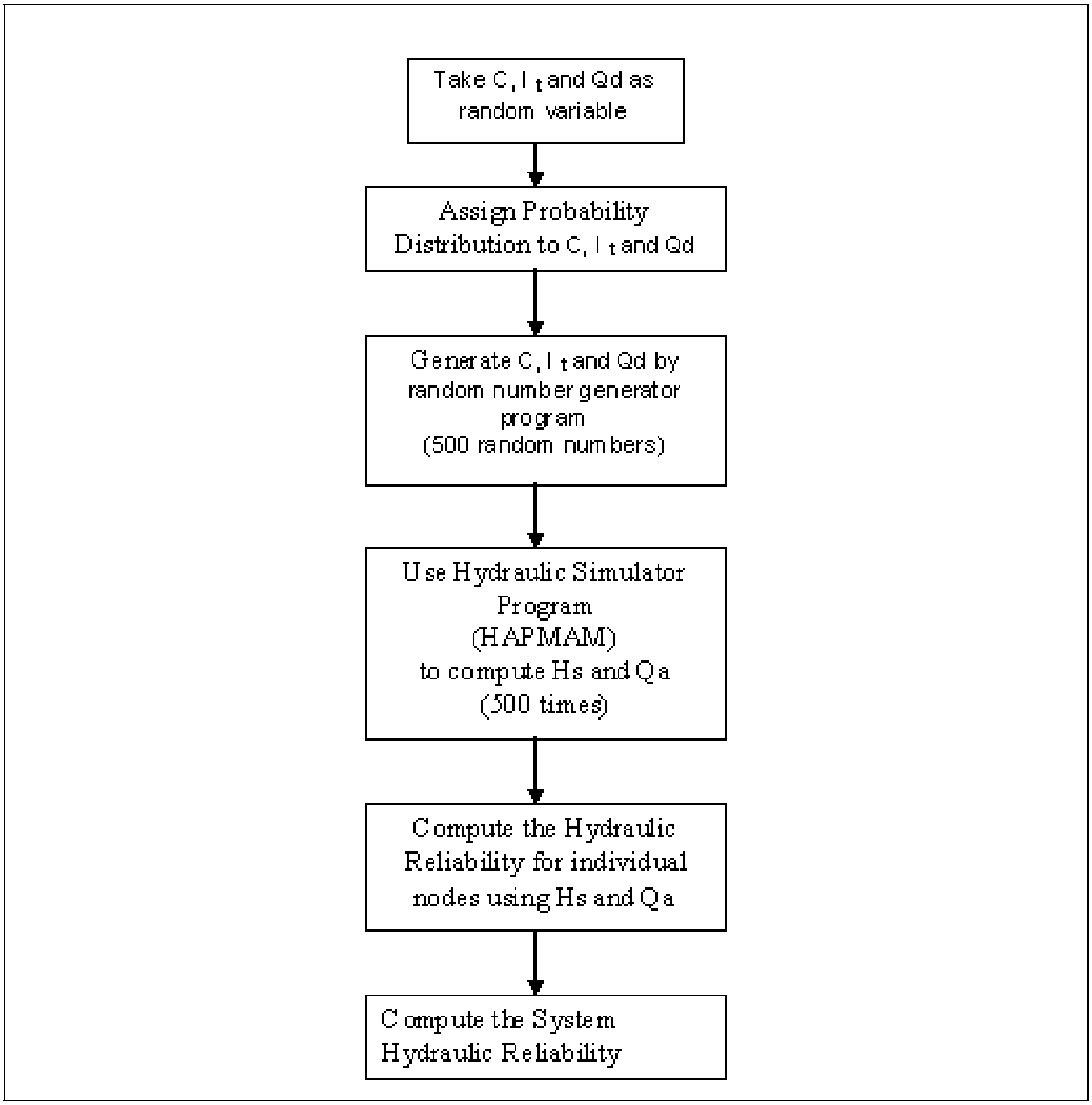


**Figure 5.10 Flowchart of the Evaluation of the Hydraulic System Reliability**

## 5.7 Computation and Results

In previous chapters, modeling of the system, reliability analysis and methodology were explained. In this section, the computation techniques and results will be included.

Reliability calculations will be carried out for three cases:

(i) *every time period starting form 01.00 to 24.00*,
(ii) *4 different periods (01.00-06.00, 07.00-12.00, 13.00-18.00, 19.00-24.00)*
(iii) *one day*.

The random number generator was used to generate 500 total demand values for every period (1 hour, 6 hour and total day) using the mean and standard deviation values that was determined by the analysis of SCADA data (see Section 5.5.3). For example, for analyzing the hydraulic reliability of the system for any time (e.g. 05:00 am), 500 analyses were carried out, by using the randomly generated demand values. The demand values were distributed to the nodes using the node weight of each node.

As one of the most uncertain characteristic parameters, friction coefficient “C” was taken as another random hydraulic parameter. For the case of friction coefficients, C values were generated with the assumption of mean 130.00 and standard variation 20.00. Also, it is assumed that every pipe has same characteristics. Again 500 random “C” values were generated. For every hydraulic simulation same number was assigned to all pipes. This set of generated random numbers was used for every set of analyses without changing or regenerating.

On the other hand, the service tank water level do not fit to any pattern as it is dependent on many things and the oscillation of it would be meaningless by defining it with a mean and standard deviation. To solve this problem the random number generation was used to generate random numbers within the service limits of the tank. In order not to loose the randomness of the storage tank water level, extended period simulation analysis were not used. Instead, again 500 values were generated to be fully consistent with the other parameters. For this application, discrete random generator was used by giving the limits as 0.5 to 6.0 m of water level on the tank.

Although many different probability distribution functions (pdf) may be used, in this study, the normal distribution was used for demand and “C” coefficients to generate random numbers. Also log-normal distribution was used in order to see the sensitivity of the analyses to the distribution type. As these two types of distributions are forming the boundaries of the study carried out by Bao and Mays (1990), these two PDF assumed to be enough for such analysis.After the generation of the random numbers, these values were analyzed 500 times with HAPMAM for every period. A single period with 500 loops of analyses takes approximately 10 minutes.

At the end of 500 hydraulic analyses, 500 numbers of available demand and pressure values were generated. In the way of calculating the hydraulic reliability for the required head, the mean and standard deviation were calculated for every node using their available head values for each simulation. As we know the required pressure for each node, the reliability for every node can be calculated by using the reliability calculation formula (5.2).

| | |
|---|---|
| $R_n = \Pr(H_s > H_r) = \int_{H_r}^{\infty} f_s(H_s) d_{H_s}$ ............................. | (5.2) |

where;

$R_n$ : Nodal Reliability

$H_s$ : Available Head at Node

$H_r$ : Required Head at Node

$f_s(H_s)$ : Probability Density Function of Supplied Pressure Head

Also for calculating the hydraulic system reliability by using the consumption values, formula (5.3) was used for every node to calculate the nodal consumption hydraulic reliability.

| | |
|---|---|
| $R_n = \frac{Q_a}{Q_r}$ .............................. | (5.3) |

Where;

$R_n$ : Nodal Reliability

$Q_a$ : Available Flow at Node

$Q_r$ : Required Flow at Node

After calculating the reliabilities of nodes, the system reliability was calculated for both approaches by using the weighted mean of the nodal reliabilities as in formula (5.4).

| | | |
|---|---|---|
| $R_{sw} = \frac{\sum_{i=1}^{N} R_{ni}.W_i}{\sum_{i=1}^{N} W_i}$ | ................................................... | (5.4) |

Where;

$R_{sw}$ : Weighted System Reliability.

$R_{ni}$ : Reliability of Node i.

$W_i$ : Weight of Node i.

$N$ : Number of Nodes.

As the parameters do not contain any values regarding the mechanical failure, these values can be called as daily hydraulic system reliability ($H_{Rsw}$). To join the mechanical reliability to this hydraulic reliability the scenario based analysis can be carried on (Yıldız, 2002). In the way of calculating the mechanical reliability, the scenarios of pipe failure and valve isolations and their probabilities were prepared. After that, each scenario is carried out by using same parameters for hydraulic system reliability calculations. Then, the probability of each scenario times the system reliability would give us the mechanical reliability of the system. After that, these two parameters can be combined and the system reliability can be calculated.

### 5.7.1 Reliability Results

Initially to simulate the total day, the 24 hour period analyses were carried out for the base scenario. This scenario contains, friction coefficient values with mean of 130 and standard deviation 20 with the assumption of normal probability distribution, 130 meters of pump outflow pressure, 100 $m^3$/hr discharge for P19 discharge to N10 pressure zone, discrete storage tank water level, 25 meters of required pressure for every single apartment building (see Figures 5.11and 5.12)

As it may be expected, the hours with high demand values, the system reliability decreases. Furthermore, the weighted values come out to be lower than the arithmetic mean values that inform us, the nodes with high demand values has low reliability values.

**Table 5.7 Hourly Hydraulic Reliability Results**

| Hour | Consumption $H_{Rsw}$ | Hydraulic Grade Line (HGL) $H_{Rsw}$ |
|---|---|---|
| 1 | 0.993450 | 0.658478 |
| 2 | 0.993542 | 0.661045 |
| 3 | 0.993639 | 0.664876 |
| 4 | 0.993627 | 0.664885 |
| 5 | 0.993582 | 0.663279 |
| 6 | 0.993385 | 0.656970 |
| 7 | 0.993049 | 0.650132 |
| 8 | 0.992577 | 0.641898 |
| 9 | 0.992108 | 0.634825 |
| 10 | 0.991531 | 0.627717 |
| 11 | 0.990816 | 0.619643 |
| 12 | 0.990731 | 0.619017 |
| 13 | 0.990793 | 0.620144 |
| 14 | 0.991435 | 0.626821 |
| 15 | 0.991799 | 0.631276 |
| 16 | 0.991974 | 0.633292 |
| 17 | 0.991972 | 0.634368 |
| 18 | 0.992056 | 0.634589 |
| 19 | 0.992179 | 0.636172 |
| 20 | 0.992223 | 0.636049 |
| 21 | 0.992360 | 0.639315 |
| 22 | 0.992678 | 0.641901 |
| 23 | 0.992816 | 0.645272 |
| 24 | 0.992840 | 0.647510 |

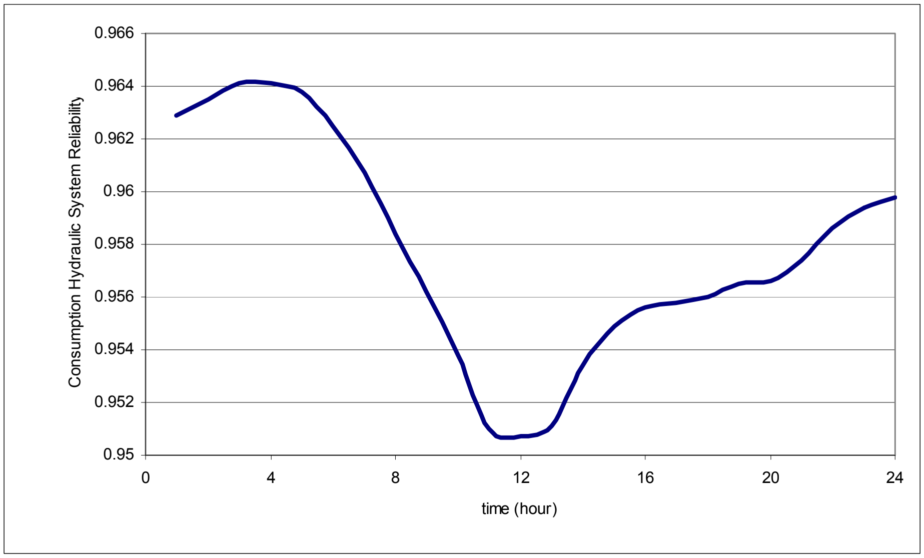


**Figure 5.11 Hourly Consumption Hydraulic Reliability Results**

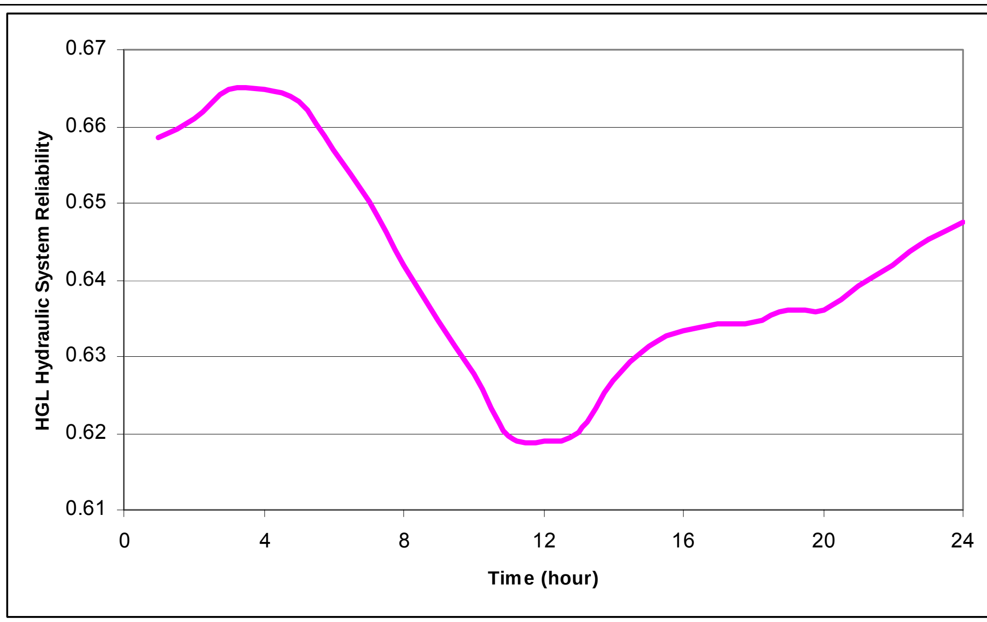


**Figure 5.12 Hourly HGL Hydraulic Reliability Results**

The results that were obtained from two different approaches, HGL and consumption hydraulic system reliabilities, give different information about the system reliability characteristics. While the consumption reliability gives information about the current situation for the systems, whether the flow is served to the nodes or not, HGL reliability gives us the information about the probability of the system to serve the nodes with a fully reliable service with enough required pressure.

For the period of analyses that divides a day to 4 periods, the results are tabulated on Table 5.8. Also analyzing the day as a whole, the hydraulic system reliability of the N8-1 Pressure zone comes out to be **0.678461** in arithmetic calculation and **0.640612** for the weighted calculations.

Although each period gives information about the behavior of the system reliability, dividing the whole day to 6 hours periods seems to be both explanatory on system behavior.

**Table 5.8 Hydraulic Reliability Results of N8-1**

| Period | Cons. System Reliability | HGL System Reliability |
|---|---|---|
| **1** | 0.993532 | 0.660441 |
| **2** | 0.991702 | 0.629434 |
| **3** | 0.991701 | 0.628119 |
| **4** | 0.992498 | 0.639376 |

#### 5.7.1.1 Effects of System Characteristics

In this section the effects of system characteristics on the system reliability are going to be discussed. As the hydraulic model and parameter values contain various assumptions, the effects of changes on the system reliability of these parameters are needed to be examined. Furthermore, while analyzing theses effects, the results can be used for increasing system reliability efficiently. Although using a well calibrated model may give much healthier results, it is not so easy to have well calibrated models for analyses. So this section can be used in understanding of the effects of the uncertain system characteristics of the system.

In this study every single change in any of the model parameters were compared with the base scenario (see Table 5.9). Using this scenario, the effects of different parameter changes were analyzed. In this section, the reliability results are the weighted system hydraulic reliability result that has been obtained by dividing the day into 6 hour periods. Also, the same generated random numbers were used for the parameters that are used in the sensitivity analyses.

**Table 5.9 Parameters of Base Scenario**

| | |
|---|---|
| **Pump 12 Outflow Pressure** | 130 meters |
| **P19 Outflow Discharge** | 100 $m^3$/hr |
| **T30 water level** | Discrete( btw 0.5-6 meters) |
| **Friction coefficients** | Mean 130, Std. 20 |
| **Calculation Method** | Pressure Dependant |
| **Distribution Type** | Normal |
| **Required Pressure for every building** | 25 meters |

#### 5.7.1.1.1 Friction Coefficients

One of the most uncertain parameter in water distribution modeling is pipe roughness coefficient. It is being affected by lots of different parameters such as, aging, velocity of fluid in the pipe, material of the pipe. Although we may find an exact value for the friction coefficient, this value may change in time. Also, every pipe segment may have a different value as a friction coefficient, although they have the same age and material. To overcome this problem, in this study, a mean and standard deviation value was given to pipes friction coefficients. Only a single value was assigned to all of the pipes at each iteration.

Furthermore, the effect of "C" coefficients on the system hydraulic reliability was examined both in consumption and Hydraulic Grade Line (HGL) point of view (see Tables 5.10 and 5.11).

**Table 5.10 Consumption $HR_{sw}$ of N8-1 Pressure Zone for different friction coefficients**

| Mean | Standard Dev | Period 1 | Period 2 | Period 3 | Period 4 | Average % Change |
|---|---|---|---|---|---|---|
| 130 | 10 | 0.993564 | 0.991849 | 0.991892 | 0.992614 | 0.14 |
| ***130*** | ***20*** | ***0.993532*** | ***0.991702*** | ***0.991701*** | ***0.992498*** | ***0.00*** |
| 120 | 20 | 0.993371 | 0.991021 | 0.990961 | 0.992166 | -1.06 |

The effect of the mean on the result is much more dominant than the standard deviation as it can be guessed. So starting with a realistic or well guessed mean and to select a relatively large standard deviation may give much more reliable results. On the other hand, the changes in the C- coefficient factors are relatively small that, in the way of obtaining reliability their effects can be neglected. However in this situation the guess should be appropriate to the system characteristics.

**Table 5.11 HGL $HR_{sw}$ of N8-1 Pressure Zone for different friction coefficients**

| Mean | Standard Dev | Period 1 | Period 2 | Period 3 | Period 4 | Average % Change |
|---|---|---|---|---|---|---|
| 130 | 10 | 0.660460 | 0.630654 | 0.629698 | 0.640217 | 0.01 |
| ***130*** | ***20*** | ***0.660441*** | ***0.629434*** | ***0.628119*** | ***0.639376*** | ***0.00*** |
| 120 | 20 | 0.655986 | 0.620896 | 0.619413 | 0.634197 | -0.05 |

The best way for determining the means for C-coefficients is to make site experiments on different diameter pipes at different locations. Also classifying the pipes and giving different mean and standard deviation to these classes of pipes may give more accurate results. However enormous research on site is needed to classify the pipes and measure their C coefficients.

On the other hand, making an initial single assumption to all pipes and carry out a traditional analysis (demand-driven), and then classifying the pipes according to their flow velocities would be a better approach. However in this case, the main problem is to give to each class an appropriate value that includes the real pipes' characteristics.

##### 5.7.1.1.2 Tank Water level

While determining the level at the storage tank T30, as the way of management directly effects the level of the tank at any time it may take any value. Usually, in models that simulate steady state condition, a single value is given to the tank water level, which may not be the case. Although generally, the operator tries to fill the storage tank at night hours and help the pumps at peak hours, this may not be the case. Also under different conditions and management factors, any water level at any time in a day, can be seen at the storage tank. To simulate this approach the discrete values in between 0.5 – 6.0 meters were generated and used as a random variable.

Furthermore, the effects of the exceptional cases are examined by disconnecting the tank from the system, placing it to a higher location, taking water level as zero and taking the tank as totally full conditions (see Tables 5.12 and 5.13).

**Table 5.12 Consumption $HR_{sw}$ of N8-1 Pressure Zone for different Tank Scenarios**

| SCENARIO | Period 1 | Period 2 | Period 3 | Period 4 | Average % Change |
|---|---|---|---|---|---|
| **No Tank** | 0.999971 | 0.999108 | 0.999184 | 0.999619 | 0.72 |
| **New Location (10 m higher elevation)** | 0.999451 | 0.998310 | 0.998334 | 0.998885 | 0.64 |
| **Full (6.0 meters)** | 0.995589 | 0.993831 | 0.993822 | 0.994577 | **0.21** |
| ***Base*** | ***0.993532*** | ***0.991702*** | ***0.991701*** | ***0.992498*** | **0.00** |
| **Fixed at 3 meters** | 0.993371 | 0.991564 | 0.991570 | 0.992348 | *-0.01* |
| **Empty** | 0.991032 | 0.988854 | 0.988868 | 0.989803 | *-0.27* |

**Table 5.13 HGL $HR_{sw}$ of N8-1 Pressure Zone for different Tank Scenarios**

| SCENARIO | Period 1 | Period 2 | Period 3 | Period 4 | Average % Change |
|---|---|---|---|---|---|
| **No Tank** | 0.905702 | 0.853015 | 0.852959 | 0.875032 | 36.33 |
| **New Location (10 m higher elevation)** | 0.853468 | 0.814581 | 0.814371 | 0.831444 | 29.58 |
| **Full (6 meters)** | 0.720745 | 0.672177 | 0.669321 | 0.684665 | *7.39* |
| ***Base*** | ***0.660441*** | ***0.629434*** | ***0.628119*** | ***0.639376*** | **0.00** |
| **Fixed at 3 meters** | 0.650131 | 0.623848 | 0.625442 | 0.635574 | *-0.87* |
| **Empty** | 0.617974 | 0.590801 | 0.590124 | 0.599882 | *-6.20* |

As the water elevation of the storage tank increase, the reliability in means of HGL and consumption increase. Although no tank condition nearly ties up the system reliability to 1.0, this would not be an ideal case in management. As the pumps outflow pressure and capacity is capable of meeting the required demand and pressure for the area; without a tank, it is not possible to work only with pump in these kinds of systems. Also during the modeling period, it has been observed that whenever the pump starts working on the system, there is an inflow occurs to the tank. There may be several reasons for this; large diameter pipes and excess capacity of pump station. If the general purpose of storage tank is to store excess water during low demand periods in order to meet widely fluctuating demands such as fire demands and peak hours, this is actually not the case for N8-1 pressure zone.

In designing stages generally to project the future demands, large diameter pipes are chosen, as the demand in those areas are lower than the projected value, the large diameter pipes starts acting as a storage tank. Starting from water quality there may be several disadvantages of such conditions. Moreover, even in peak hours, the ability of pump to fill the tank is an indication of high capability of pumps to fulfill the requirements. Under these conditions, the main purpose of the tank in the system is to reduce the excess capacity of the pump station P12 rather than acting as a distribution reservoir.

Furthermore, the poorly located tank reduces the overall pressure on the pressure zone. This case was observed in the field studies that most of the new buildings elevations are higher than the tanks current elevation.

As it can be seen directly form the tables, the water level at tank is directly affecting the reliability of the system. Together with the discrete values, simulation of the tank water level an average value that can be used as system hydraulic reliability may be obtained.

Moreover, for the current solutions of the problem regarding the reliability of the system, having nearly full storage tank would help the system. Furthermore, for future developments and solution of the problem, a new location (10 m higher) would be the optimal solution for the problem; on the other hand investigation of the area for such suitable place should be made.

#### 5.7.1.1.3 Outflow Pressure of Pump Station

The pump outflow pressure of the pump station is observed as it takes more close values to 13 bars. So in these analyses it has been taken as 130 meters. On the other hand the outflow pressure of the pumps may be changed by adding more pumps or working under different conditions. To analyze this by giving different values for outflow pressure of the pump station the reliability analyses were carried on.

**Table 5.14 Consumption $HR_{sw}$ of N8-1 Pressure Zone for different Pump outflow Pressures**

| Pump Station Outflow pressure | Period 1 | Period 2 | Period 3 | Period 4 | Average % Change |
|---|---|---|---|---|---|
| **150 meters** | 0.996057 | 0.994219 | 0.994219 | 0.994878 | 0.68 |
| **145 meters** | 0.995407 | 0.993695 | 0.993648 | 0.994302 | 0.19 |
| **140 meters** | 0.993532 | 0.991702 | 0.991701 | 0.992498 | 0.00 |
| ***130 meters (Base)*** | ***0.993532*** | ***0.991702*** | ***0.991701*** | ***0.992498*** | ***0.00*** |
| **120 meters** | 0.991898 | 0.989124 | 0.989125 | 0.990363 | -0.22 |
| **0 meters (no pump)** | 0.933579 | 0.919827 | 0.919569 | 0.925308 | -6.77 |

The high dependence of the system to the pumps is observed as the lowest reliabilities were observed for no pump working case in the system. Also in daily management, this situation is highly observed. On the other hand, in daily management of the system, the working hours of the pump varies. The operator decides the working hours and number of the pumps. So in a total day there may be several hours that pump is not working that may cause severe reliability problems.

**Table 5.15 HGL $HR_{sw}$ of N8-1 Pressure Zone for different Pump outflow Pressures**

| Pump Station Outflow pressure | Period 1 | Period 2 | Period 3 | Period 4 | Average % Change |
|---|---|---|---|---|---|
| **150 meters** | 0.873928 | 0.817122 | 0.817122 | 0.839505 | 30.88 |
| **145 meters** | 0.83304 | 0.768822 | 0.768822 | 0.790802 | 23.59 |
| **140 meters** | 0.660441 | 0.629437 | 0.628119 | 0.639376 | 0.00 |
| ***130 meters (Base)*** | ***0.660441*** | ***0.629434*** | ***0.628119*** | ***0.639376*** | ***0.00*** |
| **120 meters** | 0.603243 | 0.579551 | 0.579557 | 0.589731 | -8.02 |
| **0 meters (no pump)** | 0.584857 | 0.556239 | 0.556244 | 0.569239 | -11.4 |

#### 5.7.1.1.4 Pump stations P19 discharge value

The N8-1 pressure zone contains two different pump stations, P12 the one that feeds the system, P19 that is fed by the system. The P19 pump station feed N10 pressure zone in the system and send its discharge to Pursaklar

Municipality, for different reasons, the P19 pump station is needed to work simultaneously as it stops working a lot of problems occurs on both N10 and Pursaklar.

Although there is a study regarding the daily demand curve of the area of N10 pressure zone carried out, it comes out to have a single value as 100 $m^3$/ hr seems to be satisfying. On the other hand, the simultaneous changes in the discharge of the pump station and their effects on the reliability are examined in this part.

**Table 5.16 Consumption $HR_{sw}$ of N8-1 Pressure Zone for different P19 Discharges**

| P19 Discharge Values | Period 1 | Period 2 | Period 3 | Period 4 | Average % Change |
|---|---|---|---|---|---|
| **0 $m^3$/ hr ( Not working)** | 0.993747 | 0.992024 | 0.992013 | 0.992744 | 0.03 |
| **80 $m^3$/ hr** | 0.993574 | 0.991776 | 0.991773 | 0.992551 | 0.01 |
| ***100 $m^3$/ hr (Base)*** | ***0.993532*** | ***0.991702*** | ***0.991701*** | ***0.992498*** | ***0.00*** |
| **110 $m^3$/ hr** | 0.993510 | 0.991663 | 0.991663 | 0.992470 | 0.00 |
| **120 $m^3$/ hr** | 0.993489 | 0.991622 | 0.991623 | 0.992442 | -0.01 |

As the discharge value of the P19 decreases, the reliability factors increases as it is expected. On the other hand these changes are very small regarding the other parameters.

**Table 5.17 HGL $HR_{sw}$ of N8-1 Pressure Zone for different P19 Discharges**

| P19 Discharge Values | Period 1 | Period 2 | Period 3 | Period 4 | Average % Change |
|---|---|---|---|---|---|
| **0 $m^3$/ hr ( Not working)** | 0.689307 | 0.646017 | 0.643433 | 0.657243 | 3.06 |
| **80 $m^3$/ hr** | 0.665151 | 0.632684 | 0.631207 | 0.642561 | 0.55 |
| ***100 $m^3$/ hr (Base)*** | ***0.660441*** | ***0.629434*** | ***0.628119*** | ***0.639376*** | ***0.00*** |
| **110 $m^3$/ hr** | 0.658264 | 0.627786 | 0.626542 | 0.637803 | -0.27 |
| **120 $m^3$/ hr** | 0.656199 | 0.626121 | 0.624945 | 0.636237 | -0.54 |

For the not working case, the increase in the reliability is not that significant that leads us the assumption of 100 $m^3$/ hr is a suitable value for the analyses. It shows that other pumps stations (with respective small discharges) would not affect the system reliability so much.

#### 5.7.1.1.5 Required Pressure for Nodes

In this study, the required HGL for nodes was found, by adding the 25 meters to the elevation of the most highly elevated building of the respective nodes. The main assumption beyond this is that every building is approximately has 2 or 3 floors and 25 meters water pressure would be enough for the consumer at the last storey.

On the other hand, this assumption contains lots of weaknesses inside it. Although whole area seems to be homogenous, some buildings has more storey like 5 or 4 where as some part of the area has still one storey buildings. Although by using the suitable data fro each building this problem can be solved, for this study the data contains the roof elevation could not be found.

To see the effects of choosing different required pressures, the analyzes regarding with different values were made.

**Table 5.18 Consumption $HR_{sw}$ of N8-1 Pressure Zone for different Required Pressures**

| Required Pressure | Period 1 | Period 2 | Period 3 | Period 4 | Average % Change |
|---|---|---|---|---|---|
| **15 meters** | 0.999619 | 0.998798 | 0.998859 | 0.999294 | 0.68 |
| **20 meters** | 0.997333 | 0.995498 | 0.995491 | 0.996283 | 0.38 |
| ***25 meters (Base)*** | ***0.993532*** | ***0.991702*** | ***0.991701*** | ***0.992498*** | ***0.00*** |

The decrease in required pressure leads a respective increase in reliability in both consumption and HGL point of views; as it may be guessed. So these results show the main importance of determining the required pressure parameters of each building.  Although taking a single value is not a very bad assumption, determining the required pressure for every single building will increase the effectiveness of the calculations.

**Table 5.19 HGL $HR_{sw}$ of N8-1 Pressure Zone for different Required Pressures**

| Required Pressure | Period 1 | Period 2 | Period 3 | Period 4 | Average % Change |
|---|---|---|---|---|---|
| **15 meters** | 0.918164 | 0.894706 | 0.894727 | 0.905037 | 41.29 |
| **20 meters** | 0.816031 | 0.768926 | 0.767911 | 0.787451 | 30.88 |
| ***25 meters (Base)*** | 0.660441 | 0.629434 | 0.628119 | 0.639376 | ***0.00*** |

#### 5.7.1.1.6 Different Probability Distributions

The distribution of C-coefficients and demand values were assumed to fit to Normal Probability Distribution. The best way of using these parameters, would to use the most suitable PDF for every single value were as this work would be time consuming. On the other hand, by using the log-normal distribution, the same analyses were carried on. As these two different probability distributions are the lower and upper boundaries of the study that carried by Bao and Mays (1990), just using one other distribution function was considered satisfactory.

**Table 5.20 Consumption $HR_{sw}$ of N8-1 Pressure Zone for different PDF**

| PDF | Period 1 | Period 2 | Period 3 | Period 4 |
|---|---|---|---|---|
| **Log-Normal** | 0.993524 | 0.991772 | 0.991636 | 0.992346 |
| ***Normal (Base)*** | ***0.993532*** | ***0.991702*** | ***0.991701*** | ***0.992498*** |

**Table 5.21 HGL $HR_{sw}$ of N8-1 Pressure Zone for different PDF**

| PDF | Period 1 | Period 2 | Period 3 | Period 4 |
|---|---|---|---|---|
| **Log-Normal** | 0.661256 | 0.630116 | 0.627101 | 0.637963 |
| ***Normal (Base)*** | ***0.660441*** | ***0.629434*** | ***0.628119*** | ***0.639376*** |

Although each analysis was given the different results, the difference between them can be considered as unaffected, as they are given the similar results, the analyzer can use any of the PDF for these parameters.

##### 5.7.1.1.7 Solution Method

This study is based on the reliability calculations of water distribution networks by using the pressure dependant approach together with pressure and consumption approaches of calculations of reliability.

On the other hand, the HGL hydraulic reliability can be obtained by using the traditional demand dependant models. To analyze the difference between them a single analysis was made using traditional method.

**Table 5.22 HGL $HR_{sw}$ of N8-1 Pressure Zone for different Methods**

| Method | Period 1 | Period 2 | Period 3 | Period 4 | Average % Change |
|---|---|---|---|---|---|
| **Pressure Dependant (Base)** | ***0.660441*** | ***0.629434*** | ***0.628119*** | ***0.639376*** | ***0.00*** |
| **Demand Dependant** | 0.660327 | 0.628761 | 0.627501 | 0.639067 | -0.07 |

The results regarding pressure dependant approach came out to be a little bigger as it may be expected. As all nodes takes their demand overall pressure of the area drops simultaneously for the traditional method. On the other hand, the difference between these two values is very small. The reason for this may be that the system has a high consumption hydraulic reliability. But as the main one of the source problem having the required demand from each node is the disadvantage of using demand driven approaches.

### 5.7.2 Reliability Improvement

There may be two points of view in looking to the reliability problem of an existing water distribution problem. One of the questions may be that "What is the reliability of this network?" and the second way of looking at the problem is "How can the reliability be improved?"

The first look of the problem is to find the reliability of the network; in the way of solving this problem; the main solution starts from defining the system characteristics and determination of the initial assumptions of the model.

Tables 5.23 and 5.24 show the average percentage of change of reliability for different scenarios. Although the order of average percentage change from the base scenario is different for HGL and consumption results, the most effected parameters are same for both of them. From this, most effected parameters comes out to be, pumps outflow pressure and required pressure for buildings. Although the other parameters affect the results, their effects can be neglected.

So, by finding the most critical parameters and obtaining the most suitable values for these characteristics the ideal, reliable and consistent reliability calculations can be made for the real life networks. On the other hand, the dependence of the results should be taken into account as the percentage rate of change may be different for same parameters on different networks.

**Table 5.23 Average Percentage Change in HGL $HR_{sw}$ with different Scenarios**

| Difference From The Base Scenario | (% of change) |
|---|---|
| **Required Pressure 15 meters** | **41.29** |
| **P12 Outflow Pressure 150 meters** | **30.88** |
| **P12 Outflow Pressure 145 meters** | **23.59** |
| **Required Pressure 20 meters** | **22.78** |
| **Tank Water Level Totally Full** | *7.39* |
| **P19 Discharge 0 $m^3$/ hr ( Not working)** | *3.06* |
| **P19 Discharge 80 $m^3$/ hr** | *0.55* |
| **C-coefficient Mean:130 Std:10** | *0.14* |
| **P12 Outflow Pressure 140 meters** | *0.00* |
| *BASE* | *0.00* |
| **PDF Log-Normal** | *0.00* |
| **Method: Demand Dependant** | *-0.07* |
| **P19 Discharge 110 $m^3$/ hr** | *-0.27* |
| **P19 Discharge 120 $m^3$/ hr** | *-0.54* |
| **C-coefficient Mean:120 Std:20** | *-1.06* |
| **Tank Water Level Fixed at 3 meters** | *-0.87* |
| **Tank Water Level Empty** | *-6.20* |
| **P12 Outflow Pressure 120 meters** | *-8.02* |

Furthermore in the way of improvement the reliability of the current network same results may be used. Currently different approaches are available for improvement the reliability of the network. Also reliability can be included with quantitative approaches in the least cost design with a way of optimization

(Goulter and Coals, 1986). However these approaches are focused on the pipe breakage and node isolation probabilities. The results of the reliability analyses may be used in the way of improving the reliability.

**Table 5.24 Average Percentage Change in Consumption $HR_{sw}$ with different Scenarios**

| Difference From The Base Scenario | (% of change) |
|---|---|
| **Required Pressure 15 meters** | **0.68** |
| **Required Pressure 20 meters** | **0.38** |
| **P12 Outflow Pressure 150 meters** | **0.25** |
| **Tank Water Level Totally Full** | **0.21** |
| **P12 Outflow Pressure 145 meters** | **0.19** |
| **P19 Discharge 0 $m^3$/ hr ( Not working)** | *0.03* |
| **C-coefficient Mean:130 Std:10** | *0.01* |
| **P19 Discharge 80 $m^3$/ hr** | *0.01* |
| **P12 Outflow Pressure 140 meters** | *0.00* |
| *BASE* | *0.00* |
| **P19 Discharge 110 $m^3$/ hr** | *0.00* |
| **P19 Discharge 120 $m^3$/ hr** | *-0.01* |
| **Tank Water Level Fixed at 3 meters** | *-0.01* |
| **PDF Log-Normal** | *-0.04* |
| **C-coefficient Mean:120 Std:20** | *-0.05* |
| **P12 Outflow Pressure 120 meters** | *-0.22* |
| **Tank Water Level Empty** | *-0.27* |

From the data that can be extracted form Tables 5.25 & 5.26, for short period solutions, working with nearly full tank will improve systems reliability. Furthermore, to relocate the tank, would solve systems problems in long range. Moreover adding more pumps or changing the pumps with more efficient ones, may also be another solution but it may not be economical in means of management as their current capacity would satisfy the demands.

**Table 5.25 Average Percentage Change in HGL $HR_{sw}$ with different Scenarios for Reliability Improvement**

| Difference From The Base Scenario | (% of change) |
|---|---|
| **No Tank** | 36.33 |
| **P12 Outflow Pressure 150 meters** | 30.88 |
| **Tank New Location (10 m higher elevation)** | 29.58 |
| **P12 Outflow Pressure 145 meters** | 23.59 |
| **Tank Water Level Totally Full** | 7.391 |
| ***(Base)*** | 0 |
| **Tank Water Level Fixed at 3 meters** | -0.87 |
| **Tank Water Level empty** | -6.2 |
| **No P12 working** | -11.4 |

**Table 5.26 Average Percentage Change in Consumption $HR_{sw}$ with different Scenarios for Reliability Improvement**

| Difference From The Base Scenario | (% of change) |
|---|---|
| **No Tank** | 0.72 |
| **Tank New Location (10 m higher elevation)** | 0.64 |
| **P12 Outflow Pressure 150 meters** | 0.24 |
| **Tank Water Level Totally Full** | 0.21 |
| **P12 Outflow Pressure 145 meters** | 0.18 |
| ***(Base)*** | 0.00 |
| **Tank Water Level Fixed at 3 meters** | -0.02 |
| **Tank Water Level empty** | -0.27 |
| **No P12 working** | -6.77 |

# CHAPTER 6

# CONCLUSIONS AND RECOMMENDATIONS

A fully satisfied water distribution network should supply required amount of water within the specified limits of pressure together with the quality. However, the system can not supply full satisfaction in quality of service throughout its lifetime. Pipe breakage, pump failures, power outages, increase in demands and required pressures may be the reasons for a water distribution system to fail. The system should be designed or improved to meet the minimum standards during the period of failures. Reliability calculations can be used in this respect for determination of the effects of different types of failures.

There is no universally accepted measure of reliability for water distribution networks. Different types of methods and approaches are indicating relative meanings regarding the assessment of reliability. Nevertheless, the reliability parameters obtained by different methods can be used to improve the system capabilities of the water distribution network during the failure periods. By including different parameters like variation of demands, friction coefficients and storage tank water level and simulating the system behavior in a wide perspective would help assessment of a more acceptable value of reliability and show the ways of improving it.

In this study, a methodology was developed for calculating the hydraulic system reliability. Implementation of nodal service areas together with the spatial variation of total demand of the area, friction coefficient characteristics of pipes and different water level at the storage tank were discussed in this study. By using the Service Area Method, using the mid points of the links connecting the nodes the respective service areas for the nodes were drawn. After that, by overlaying the building layer the required pressures for every node were determined. Using the consumption data of the buildings the nodal weights were calculated. Moreover, the daily demand curves and their hourly and seasonal variations were included by obtaining the mean and standard deviation for the respective consumption periods. Implementation of different friction coefficients enabled to see the effects of the pipe characteristics on the system reliability. By using different water levels in the storage tank, the sensitivity of the results to the storage tank water elevation was decreased.

The results of sensitivity analysis give information about both the important parameters that affect the system reliability calculations and the ways of improving the hydraulic reliability of the network. For the N8-1 network of Ankara, the ideal way of improving the hydraulic reliability comes out to be relocating the water distribution tank to a higher location. This approach would increase the pumping costs, so a suggested optimization study can be carried on about this subject. The effect of using log-normal PDF instead of normal PDF is not significant. The mean and standard deviation values for friction coefficients on pipes are not affecting the results severely but using a well guessed mean and a relatively high standard deviation value would help to improve the accuracy. Arithmetic system reliability came out to be higher than weighted system reliability results. This shows that, the nodes with higher nodal weight factors are the ones that have low reliability values. In daily management using pumps and having the storage tank full would help the overall system reliability because these two parameters are dominantly affecting the reliability factors, especially the pump.

As two different reliability measures (consumption and HGL at the node) were used to determine the hydraulic reliability, different results obtained. However, the effect of parameters on these different measures came out to be nearly same for both reliability measures. Moreover, defining the available pressure and available demand curve other than Modified Chandapillai (Nohutçu, 2002) different results using same measures can be obtained.

Finally, as an application of hydraulic reliability on an existing network, this study shows the important system characteristics that affect the reliability result of the network. Moreover, highly integrated GIS and consumption data, during both modeling and analysis of the system, and usage of yearly daily demand curves were enabled a very close simulation of the existing network.

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# APPENDIX A

## Table A1.1 Values of C - Hazen Williams Coefficients

| TYPE OF PIPE | CONDITION | | C |
|---|---|---|---|
| *Cast Iron* | *New* | *All Sizes* | *130* |
| | *5 years old* | *12" and Over* | *120* |
| | | *8"* | *119* |
| | | *4"* | *118* |
| | *10 years old* | *24" and Over* | *113* |
| | | *12"* | *111* |
| | | *4"* | *107* |
| | *20 years old* | *24" and Over* | *100* |
| | | *12"* | *96* |
| | | *4"* | *89* |
| | *30 years old* | *30" and Over* | *90* |
| | | *16"* | *87* |
| | | *4"* | *75* |
| | *40 years old* | *30" and Over* | *83* |
| | | *16"* | *80* |
| | | *4"* | *64* |
| | *50 years old* | *40" and Over* | *77* |
| | | *24"* | *74* |
| | | *4"* | *55* |
| *Welded Steel* | *Values of C the same as for cast-iron pipes, 5 years older* | | |
| *Riveted Steel* | *Values of C the same as for cast-iron pipes, 10 years older* | | |
| *Wood Stave* | *Average value, regardless of age* | | *120* |
| *Concrete* | *Large sizes, good workmanship, steel forms* | | *140* |
| *Concrete* | *Large sizes, good workmanship, wooden forms* | | *120* |
| *Concrete* | *Centrifugally spun* | | *135* |
| *Vitrified* | *In good condition* | | *110* |
| *Plastic or Drawn Tubing* | | | *150* |

# APPENDIX B

## HGL HYDRAULIC RELIABILITY CALCULATIONS FOR TOTAL DAY

Table B.1 includes the calculation details of the nodal reliabilities

**Table B.1: Nodal Reliability Calculations for total day**

| Node No | NODE WEIGHT | Required HGL | Av. HGL Mean | Aval. HGL Std. | Node Rel. | Weighted Node Rel. |
|---|---|---|---|---|---|---|
| 1 | 0.000000 | 1110.00 | 1140.00 | 1.5800 | 1.0000 | 0.0000 |
| 2 | 0.000000 | 1040.00 | 1150.00 | 0.0207 | 1.0000 | 0.0000 |
| 3 | 0.003490 | 1110.00 | 1140.00 | 0.9580 | 1.0000 | 0.0035 |
| 4 | 0.000000 | 1090.00 | 1140.00 | 1.0700 | 1.0000 | 0.0000 |
| 5 | 0.000000 | 1110.00 | 1140.00 | 1.1900 | 1.0000 | 0.0000 |
| 6 | 0.000000 | 1110.00 | 1140.00 | 1.4600 | 1.0000 | 0.0000 |
| 7 | 0.013900 | 1130.00 | 1140.00 | 1.5300 | 1.0000 | 0.0139 |
| 8 | 0.000129 | 1130.00 | 1140.00 | 1.4600 | 1.0000 | 0.0001 |
| 9 | 0.004450 | 1130.00 | 1140.00 | 1.4600 | 1.0000 | 0.0045 |
| 10 | 0.003180 | 1130.00 | 1140.00 | 1.2300 | 1.0000 | 0.0032 |
| 11 | 0.000133 | 1140.00 | 1140.00 | 1.3600 | 0.0166 | 0.0000 |
| 12 | 0.001400 | 1140.00 | 1140.00 | 1.3500 | 0.0009 | 0.0000 |
| 13 | 0.001570 | 1120.00 | 1140.00 | 1.3300 | 1.0000 | 0.0016 |
| 14 | 0.001870 | 1150.00 | 1140.00 | 1.3100 | 0.0000 | 0.0000 |
| 15 | 0.000139 | 1150.00 | 1140.00 | 1.3000 | 0.0000 | 0.0000 |

| | | | | | | |
|---|---|---|---|---|---|---|
| 16 | 0.003350 | 1150.00 | 1140.00 | 1.3200 | 0.0000 | 0.0000 |
| 17 | 0.010100 | 1150.00 | 1140.00 | 1.4100 | 0.0000 | 0.0000 |
| 18 | 0.005240 | 1150.00 | 1140.00 | 1.4700 | 0.0000 | 0.0000 |
| 19 | 0.005660 | 1140.00 | 1140.00 | 1.5000 | 0.9330 | 0.0053 |
| 20 | 0.013500 | 1140.00 | 1140.00 | 1.5200 | 0.0100 | 0.0001 |

**Table B.1 Cont'd**

| | | | | | | |
|---|---|---|---|---|---|---|
| 21 | 0.008890 | 1130.00 | 1140.00 | 1.5300 | 1.0000 | 0.0089 |
| 22 | 0.028200 | 1150.00 | 1140.00 | 1.4800 | 0.0000 | 0.0000 |
| 23 | 0.017400 | 1140.00 | 1140.00 | 1.5900 | 0.0760 | 0.0013 |
| 24 | 0.004670 | 1130.00 | 1140.00 | 1.6100 | 1.0000 | 0.0047 |
| 25 | 0.005320 | 1130.00 | 1140.00 | 1.6100 | 1.0000 | 0.0053 |
| 26 | 0.014900 | 1140.00 | 1140.00 | 1.6100 | 0.9970 | 0.0148 |
| 27 | 0.014800 | 1140.00 | 1140.00 | 1.6000 | 0.0986 | 0.0015 |
| 28 | 0.003310 | 1120.00 | 1140.00 | 1.6100 | 1.0000 | 0.0033 |
| 29 | 0.021900 | 1130.00 | 1140.00 | 1.6100 | 1.0000 | 0.0219 |
| 30 | 0.002030 | 1110.00 | 1140.00 | 1.6100 | 1.0000 | 0.0020 |
| 31 | 0.007710 | 1140.00 | 1140.00 | 1.6200 | 0.0719 | 0.0006 |
| 32 | 0.006270 | 1140.00 | 1140.00 | 1.6300 | 0.1360 | 0.0009 |
| 33 | 0.002530 | 1140.00 | 1140.00 | 1.6300 | 0.7370 | 0.0019 |
| 34 | 0.002940 | 1140.00 | 1140.00 | 1.6200 | 0.0058 | 0.0000 |
| 35 | 0.004560 | 1140.00 | 1140.00 | 1.6100 | 0.0041 | 0.0000 |
| 36 | 0.004090 | 1120.00 | 1140.00 | 1.6600 | 1.0000 | 0.0041 |
| 37 | 0.008170 | 1090.00 | 1140.00 | 1.6700 | 1.0000 | 0.0082 |
| 38 | 0.010500 | 1090.00 | 1140.00 | 1.6800 | 1.0000 | 0.0105 |
| 39 | 0.003480 | 1100.00 | 1140.00 | 1.6900 | 1.0000 | 0.0035 |
| 40 | 0.003500 | 1110.00 | 1140.00 | 1.6900 | 1.0000 | 0.0035 |
| 41 | 0.003080 | 1120.00 | 1140.00 | 1.6900 | 1.0000 | 0.0031 |

| | | | | | | |
|---|---|---|---|---|---|---|
| 42 | 0.001780 | 1120.00 | 1140.00 | 1.6900 | 1.0000 | 0.0018 |
| 43 | 0.001760 | 1100.00 | 1140.00 | 0.9580 | 1.0000 | 0.0018 |
| 44 | 0.003280 | 1120.00 | 1140.00 | 1.1800 | 1.0000 | 0.0033 |
| 45 | 0.008930 | 1130.00 | 1140.00 | 1.4600 | 1.0000 | 0.0089 |
| 46 | 0.005630 | 1100.00 | 1140.00 | 1.4600 | 1.0000 | 0.0056 |
| 47 | 0.011800 | 1110.00 | 1140.00 | 1.6500 | 1.0000 | 0.0118 |
| 48 | 0.005440 | 1100.00 | 1140.00 | 1.6700 | 1.0000 | 0.0054 |
| 49 | 0.004200 | 1120.00 | 1140.00 | 1.6700 | 1.0000 | 0.0042 |

**Table B.1 Cont'd**

| | | | | | | |
|---|---|---|---|---|---|---|
| 50 | 0.004570 | 1140.00 | 1140.00 | 1.6500 | 0.8330 | 0.0038 |
| 51 | 0.007570 | 1140.00 | 1140.00 | 1.4700 | 0.3740 | 0.0028 |
| 52 | 0.001320 | 1140.00 | 1140.00 | 1.4700 | 0.8810 | 0.0012 |
| 53 | 0.002890 | 1130.00 | 1140.00 | 1.4800 | 1.0000 | 0.0029 |
| 54 | 0.002920 | 1120.00 | 1140.00 | 1.4900 | 1.0000 | 0.0029 |
| 55 | 0.004600 | 1130.00 | 1140.00 | 1.4900 | 1.0000 | 0.0046 |
| 56 | 0.005030 | 1110.00 | 1140.00 | 1.4900 | 1.0000 | 0.0050 |
| 57 | 0.003060 | 1120.00 | 1140.00 | 1.4900 | 1.0000 | 0.0031 |
| 58 | 0.002380 | 1120.00 | 1140.00 | 1.5000 | 1.0000 | 0.0024 |
| 59 | 0.005260 | 1150.00 | 1140.00 | 1.4800 | 0.0000 | 0.0000 |
| 60 | 0.016300 | 1160.00 | 1140.00 | 1.6200 | 0.0000 | 0.0000 |
| 61 | 0.015200 | 1160.00 | 1140.00 | 2.2600 | 0.0000 | 0.0000 |
| 62 | 0.018600 | 1160.00 | 1140.00 | 2.3200 | 0.0000 | 0.0000 |
| 63 | 0.008050 | 1130.00 | 1140.00 | 2.3100 | 0.9960 | 0.0080 |
| 64 | 0.011900 | 1140.00 | 1140.00 | 2.5100 | 0.0214 | 0.0003 |
| 65 | 0.007920 | 1110.00 | 1140.00 | 2.3200 | 1.0000 | 0.0079 |
| 66 | 0.004320 | 1110.00 | 1140.00 | 2.3200 | 1.0000 | 0.0043 |
| 67 | 0.006620 | 1110.00 | 1140.00 | 2.3400 | 1.0000 | 0.0066 |

| | | | | | | |
|---|---|---|---|---|---|---|
| 68 | 0.003410 | 1110.00 | 1140.00 | 2.3400 | 1.0000 | 0.0034 |
| 69 | 0.009110 | 1120.00 | 1140.00 | 2.1000 | 1.0000 | 0.0091 |
| 70 | 0.011500 | 1110.00 | 1140.00 | 2.6800 | 1.0000 | 0.0115 |
| 71 | 0.008630 | 1130.00 | 1140.00 | 2.6400 | 1.0000 | 0.0086 |
| 72 | 0.006290 | 1130.00 | 1140.00 | 2.5700 | 1.0000 | 0.0063 |
| 73 | 0.008000 | 1130.00 | 1140.00 | 2.7700 | 0.9990 | 0.0080 |
| 74 | 0.007410 | 1140.00 | 1140.00 | 2.7400 | 0.3560 | 0.0026 |
| 75 | 0.003580 | 1110.00 | 1140.00 | 2.8800 | 1.0000 | 0.0036 |
| 76 | 0.004970 | 1110.00 | 1140.00 | 2.9200 | 1.0000 | 0.0050 |
| 77 | 0.003400 | 1100.00 | 1140.00 | 2.9300 | 1.0000 | 0.0034 |
| 78 | 0.009740 | 1100.00 | 1140.00 | 3.0000 | 1.0000 | 0.0097 |

**Table B.1 Cont'd**

| | | | | | | |
|---|---|---|---|---|---|---|
| 79 | 0.003670 | 1090.00 | 1140.00 | 3.0000 | 1.0000 | 0.0037 |
| 80 | 0.000000 | 1110.00 | 1140.00 | 2.7200 | 1.0000 | 0.0000 |
| 81 | 0.005860 | 1130.00 | 1140.00 | 2.8500 | 0.9300 | 0.0055 |
| 82 | 0.004190 | 1130.00 | 1140.00 | 2.8800 | 0.9990 | 0.0042 |
| 83 | 0.004550 | 1120.00 | 1140.00 | 2.9500 | 1.0000 | 0.0046 |
| 84 | 0.008330 | 1120.00 | 1140.00 | 3.0300 | 1.0000 | 0.0083 |
| 85 | 0.007610 | 1130.00 | 1140.00 | 3.0600 | 0.9940 | 0.0076 |
| 86 | 0.010200 | 1130.00 | 1140.00 | 3.1800 | 0.9990 | 0.0102 |
| 87 | 0.004550 | 1120.00 | 1140.00 | 3.2300 | 1.0000 | 0.0046 |
| 88 | 0.003430 | 1100.00 | 1140.00 | 3.2600 | 1.0000 | 0.0034 |
| 89 | 0.010700 | 1110.00 | 1140.00 | 3.2800 | 1.0000 | 0.0107 |
| 90 | 0.005760 | 1090.00 | 1140.00 | 3.3100 | 1.0000 | 0.0058 |
| 91 | 0.007390 | 1100.00 | 1140.00 | 3.3100 | 1.0000 | 0.0074 |
| 92 | 0.003480 | 1090.00 | 1140.00 | 3.3100 | 1.0000 | 0.0035 |
| 93 | 0.011700 | 1080.00 | 1140.00 | 3.4100 | 1.0000 | 0.0117 |

| | | | | | | |
|---|---|---|---|---|---|---|
| 94 | 0.013400 | 1100.00 | 1140.00 | 3.5000 | 1.0000 | 0.0134 |
| 95 | 0.011900 | 1080.00 | 1140.00 | 3.5800 | 1.0000 | 0.0119 |
| 96 | 0.005830 | 1100.00 | 1140.00 | 3.5900 | 1.0000 | 0.0058 |
| 97 | 0.000000 | 1040.00 | 1140.00 | 3.5800 | 1.0000 | 0.0000 |
| 98 | 0.011400 | 1090.00 | 1140.00 | 3.5000 | 1.0000 | 0.0114 |
| 99 | 0.000000 | 1030.00 | 1140.00 | 3.5000 | 1.0000 | 0.0000 |
| 100 | 0.005080 | 1080.00 | 1140.00 | 3.4200 | 1.0000 | 0.0051 |
| 101 | 0.004540 | 1080.00 | 1140.00 | 1.4700 | 1.0000 | 0.0045 |
| 102 | 0.004070 | 1080.00 | 1140.00 | 1.4700 | 1.0000 | 0.0041 |
| 103 | 0.003890 | 1090.00 | 1140.00 | 1.4700 | 1.0000 | 0.0039 |
| 104 | 0.005320 | 1140.00 | 1140.00 | 2.1300 | 0.0015 | 0.0000 |
| 105 | 0.003130 | 1150.00 | 1140.00 | 2.0600 | 0.0000 | 0.0000 |
| 106 | 0.008580 | 1130.00 | 1140.00 | 2.0200 | 1.0000 | 0.0086 |
| 107 | 0.010700 | 1140.00 | 1140.00 | 2.0400 | 0.7340 | 0.0079 |

**Table B.1 Cont'd**

| | | | | | | |
|---|---|---|---|---|---|---|
| 108 | 0.010300 | 1110.00 | 1140.00 | 1.9500 | 1.0000 | 0.0103 |
| 109 | 0.000000 | 1080.00 | 1140.00 | 1.8000 | 1.0000 | 0.0000 |
| 110 | 0.004430 | 1120.00 | 1140.00 | 1.7900 | 1.0000 | 0.0044 |
| 111 | 0.007270 | 1140.00 | 1140.00 | 1.9400 | 0.0365 | 0.0003 |
| 112 | 0.009150 | 1090.00 | 1140.00 | 2.0100 | 1.0000 | 0.0092 |
| 113 | 0.006660 | 1150.00 | 1140.00 | 1.9000 | 0.0000 | 0.0000 |
| 114 | 0.006630 | 1090.00 | 1140.00 | 1.8800 | 1.0000 | 0.0066 |
| 115 | 0.004380 | 1090.00 | 1140.00 | 1.8700 | 1.0000 | 0.0044 |
| 116 | 0.006450 | 1100.00 | 1140.00 | 1.8500 | 1.0000 | 0.0065 |
| 117 | 0.003500 | 1110.00 | 1140.00 | 1.8600 | 1.0000 | 0.0035 |
| 118 | 0.003600 | 1150.00 | 1140.00 | 1.8700 | 0.0000 | 0.0000 |
| 119 | 0.003450 | 1150.00 | 1140.00 | 1.8400 | 0.0000 | 0.0000 |

| | | | | | | |
|---|---|---|---|---|---|---|
| 120 | 0.002470 | 1110.00 | 1140.00 | 1.8400 | 1.0000 | 0.0025 |
| 121 | 0.004490 | 1150.00 | 1140.00 | 1.8000 | 0.0000 | 0.0000 |
| 122 | 0.005410 | 1160.00 | 1140.00 | 1.5700 | 0.0000 | 0.0000 |
| 123 | 0.001850 | 1110.00 | 1140.00 | 1.5400 | 1.0000 | 0.0019 |
| 124 | 0.004490 | 1110.00 | 1140.00 | 1.5300 | 1.0000 | 0.0045 |
| 125 | 0.002010 | 1120.00 | 1140.00 | 1.5200 | 1.0000 | 0.0020 |
| 126 | 0.003020 | 1120.00 | 1140.00 | 1.5200 | 1.0000 | 0.0030 |
| 127 | 0.002940 | 1160.00 | 1140.00 | 1.5400 | 0.0000 | 0.0000 |
| 128 | 0.002570 | 1160.00 | 1140.00 | 1.5000 | 0.0000 | 0.0000 |
| 129 | 0.004060 | 1140.00 | 1140.00 | 1.4900 | 0.0914 | 0.0004 |
| 130 | 0.005010 | 1140.00 | 1140.00 | 1.4800 | 0.0496 | 0.0002 |
| 131 | 0.003190 | 1120.00 | 1140.00 | 1.4900 | 1.0000 | 0.0032 |
| 132 | 0.005310 | 1130.00 | 1140.00 | 1.7000 | 1.0000 | 0.0053 |
| 133 | 0.003150 | 1130.00 | 1140.00 | 1.6500 | 1.0000 | 0.0032 |
| 134 | 0.003150 | 1140.00 | 1140.00 | 1.6000 | 0.9990 | 0.0032 |
| 135 | 0.003860 | 1140.00 | 1140.00 | 1.5000 | 0.0011 | 0.0000 |
| 136 | 0.006210 | 1140.00 | 1140.00 | 1.4900 | 0.0012 | 0.0000 |

**Table B.1 Cont'd**

| | | | | | | |
|---|---|---|---|---|---|---|
| 137 | 0.004850 | 1120.00 | 1140.00 | 1.7400 | 1.0000 | 0.0049 |
| 138 | 0.003970 | 1100.00 | 1140.00 | 1.7400 | 1.0000 | 0.0040 |
| 139 | 0.003300 | 1090.00 | 1140.00 | 1.7200 | 1.0000 | 0.0033 |
| 140 | 0.004010 | 1100.00 | 1140.00 | 1.7100 | 1.0000 | 0.0040 |
| 141 | 0.003990 | 1110.00 | 1140.00 | 1.7100 | 1.0000 | 0.0040 |
| 142 | 0.003560 | 1120.00 | 1140.00 | 1.7100 | 1.0000 | 0.0036 |
| 143 | 0.006910 | 1130.00 | 1140.00 | 1.6900 | 1.0000 | 0.0069 |
| 144 | 0.004380 | 1110.00 | 1140.00 | 1.7000 | 1.0000 | 0.0044 |
| 145 | 0.003050 | 1100.00 | 1140.00 | 1.7000 | 1.0000 | 0.0031 |

| | | | | | | |
|---|---|---|---|---|---|---|
| 146 | 0.005440 | 1090.00 | 1140.00 | 1.7000 | 1.0000 | 0.0054 |
| 147 | 0.003060 | 1130.00 | 1140.00 | 1.6600 | 1.0000 | 0.0031 |
| 148 | 0.003660 | 1140.00 | 1140.00 | 1.6400 | 0.0082 | 0.0000 |
| 149 | 0.005160 | 1140.00 | 1140.00 | 1.6300 | 0.2490 | 0.0013 |
| 150 | 0.007160 | 1150.00 | 1140.00 | 1.3900 | 0.0000 | 0.0000 |
| 151 | 0.005770 | 1150.00 | 1140.00 | 1.5200 | 0.0008 | 0.0000 |
| 152 | 0.004480 | 1140.00 | 1140.00 | 1.5300 | 0.0399 | 0.0002 |
| 153 | 0.003470 | 1150.00 | 1140.00 | 1.3300 | 0.0000 | 0.0000 |
| 154 | 0.006350 | 1150.00 | 1140.00 | 1.3300 | 0.0000 | 0.0000 |
| 155 | 0.009660 | 1140.00 | 1140.00 | 1.3300 | 0.0012 | 0.0000 |
| 156 | 0.007550 | 1150.00 | 1140.00 | 1.3300 | 0.0000 | 0.0000 |
| 157 | 0.006270 | 1150.00 | 1140.00 | 1.3300 | 0.0000 | 0.0000 |
| 158 | 0.004700 | 1150.00 | 1140.00 | 1.3200 | 0.0000 | 0.0000 |
| 159 | 0.003030 | 1150.00 | 1140.00 | 1.3100 | 0.0000 | 0.0000 |
| 160 | 0.005780 | 1150.00 | 1140.00 | 1.3300 | 0.0000 | 0.0000 |
| 161 | 0.005120 | 1130.00 | 1140.00 | 1.3300 | 1.0000 | 0.0051 |
| 162 | 0.005260 | 1130.00 | 1140.00 | 1.3100 | 1.0000 | 0.0053 |
| 163 | 0.008060 | 1140.00 | 1140.00 | 1.3300 | 0.6360 | 0.0051 |
| 164 | 0.006860 | 1150.00 | 1140.00 | 1.3300 | 0.0000 | 0.0000 |
| 165 | 0.003090 | 1120.00 | 1140.00 | 1.2200 | 1.0000 | 0.0031 |

**Table B.1 Cont'd**

| | | | | | | |
|---|---|---|---|---|---|---|
| 166 | 0.004120 | 1120.00 | 1140.00 | 1.3400 | 1.0000 | 0.0041 |
| 167 | 0.005600 | 1140.00 | 1140.00 | 1.3400 | 0.9600 | 0.0054 |
| 168 | 0.006320 | 1140.00 | 1140.00 | 1.3700 | 0.0300 | 0.0002 |
| 169 | 0.007460 | 1130.00 | 1140.00 | 1.5000 | 1.0000 | 0.0075 |
| 170 | 0.002880 | 1140.00 | 1140.00 | 1.3900 | 0.7780 | 0.0022 |
| 171 | 0.006560 | 1140.00 | 1140.00 | 1.3900 | 0.1050 | 0.0007 |

| | | | | | | |
|---|---|---|---|---|---|---|
| 172 | 0.005070 | 1150.00 | 1140.00 | 1.3700 | 0.0000 | 0.0000 |
| 173 | 0.008070 | 1140.00 | 1140.00 | 1.3800 | 0.7540 | 0.0061 |
| 174 | 0.004190 | 1130.00 | 1140.00 | 1.4500 | 1.0000 | 0.0042 |
| | | | | | 0.6780 | 0.6410 |