

INFLUENCE OF SYSTEM PARAMETERS ON THE DISINFECTION  
CAPABILITY OF WATER DISTRIBUTION NETWORKS

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CAPABILITY OF WATER DISTRIBUTION NETWORKS**

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## **ABSTRACT**

### **INFLUENCE OF SYSTEM PARAMETERS ON THE DISINFECTION CAPABILITY OF WATER DISTRIBUTION NETWORKS**

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Adequate municipal water supply for people, is one of the most important signs of civilization. When water is cleaned from unhealthy compounds, the most important point for consumers is completed. Chlorine is used as the most common disinfectant. Some basic parameters which affect water quality are roughness coefficient, wall decay coefficient and bulk decay coefficient; amount of leakage is also effective. Efficiency of the network and water quality are influenced by changing these parameters in a water distribution network since the system behaviour changes.

This study is prepared for the north part of Ankara Water Distribution Network, which is named N8.3. This area is mainly divided into 6 different DMA's (District Metered Areas). Chlorine is used as employed as the disinfectant by Ankara General Directorate of Water and Sewerage (ASKI) for all regions. The network is analysed for DMA's condition in which all isolation valves are closed and no water transition occurs between subzones. Because of

the mixing process of water while studying whole network, a DMA's based model can perform better than a complex system.

The main aim of this study is to examine the effects of some system parameters and amount of leakage on disinfectant concentrations. In the districts that have no water exchange from the tank, roughness coefficient is not effective on chlorine concentrations. Wall decay and bulk decay coefficients are the main parameters affecting it. However, in the districts that have water exchange with the tank, the higher the roughness coefficient the lower the chlorine concentrations. Another important point is that, increasing leakage amount increases and decreasing leakage amount decreases the chlorine concentrations.

**Key Words:** Water Distribution Networks, Water Quality Modeling, Chlorine Concentration, Leakage, System Parameters

## ÖZ

### SİSTEM DEĞİŞKENLERİNİN SU DAĞITIM ŞEBEKELERİNİN DEZENFEKSİYON KAPASİTESİ ÜZERİNDEKİ ETKİSİ

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İnsanların kullanımı için yeterli kentsel suyun sağlanabilmesi, medeniyetin en önemli simgelerinden biridir. Su sağlığa zararlı bileşenlerden arındırıldığında, tüketicilerin kullanımı için gerekli en önemli aşama tamamlanmış olur. Klor en yaygın kullanılan dezenfektandır. Su kalitesini etkileyen bazı başlıca parametreler; pürüzlülük katsayısı, çeper kaybı ve tüketim kaybı katsayılarıdır; ayrıca sızma miktarı da etkilidir. Bir su dağıtım şebekesinde, şebeke verimliliği ve su kalitesi bu parametrelerin değişiminden etkilenir çünkü sistem farklı şekilde davranır.

Bu çalışma, Ankara su dağıtım şebekesinin N8.3 adı verilen ve kuzeyinde yer alan bir bölümü üzerinde yapılmıştır. Bu alan temel olarak 6 farklı alt bölgeye ayrılmıştır. Su dezenfeksiyonu için, Ankara Su Kanalizasyon İdaresi'nin (ASKI) kullandığı gibi klor kullanılmıştır. Şebeke, tüm izolasyon vanalarının kapalı olduğu ve alt bölgelerin birbirinden etkilenmediği, suyun alt bölgeler arası geçişinin olmadığı alt bölgeler metoduyla incelenmiştir. Tüm şebeke ile

çalıřılırken suyun karıřım srecinin sebep olacađı etkilerden dolayı alt blgeler metoduna dayalı bir model, karmařık bir sistemden daha iyi çalıřacaktır.

Bu çalıřmanın temel amacı, sistem deđiřkenlerinin ve sızma miktarlarının çalıřılan alandaki dezenfektan konsantrasyonları zerindeki etkilerini incelemektir. Tankla su alıřveriři olmayan blgelerde, przllk katsayısının, klor konsantrasyonları zerinde etkisi yoktur. eper kaybı ve tketim kaybı katsayıları klor konsantrasyonlarını etkileyen temel parametrelerdir. Ancak, tankla su alıřveriři olan blgelerde, przllk katsayısı arttıka klor konsantrasyonları dřer. Bir diđer önemli nokta řudur ki, klor konsantrasyonları sızma miktarlarının artırılmasıyla artar, azaltılmasıyla azalır.

Anahtar Kelimeler: Su Dađıtım řebekeleri, Su Kalite Modellemesi, Klor Konsantrasyonu, Sızma, Sistem Deđiřkenleri



Dedicated to all people who work for procurement of clean water

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## LIST OF ABBREVIATIONS

<b>ASKI</b>	Ankara General Directorate of Water and Sewerage
<b>ASPE</b>	American Society of Plumbing Engineers
<b>AWWA</b>	American Water Works Association
<b>DDC</b>	Daily Demand Curve
<b>DMA</b>	District Metered Area
<b>EPA</b>	Environmental Protection Agency
<b>EPS</b>	Extended Period Simulation
<b>FIFO</b>	First In First Out
<b>GIS</b>	Geographical Information System
<b>HAA</b>	Haloacetic Acid
<b>HGL</b>	Hydraulic Grade Line
<b>JC</b>	Junction Number
<b>LIFO</b>	Last In First Out
<b>NOM</b>	Natural Organic Matter
<b>SCADA</b>	Supervisory Control and Data Acquisition
<b>THM</b>	Trihalomethane
<b>WDN</b>	Water Distribution Network

## LIST OF SYMBOLS

- $A_i$  :Cross sectional area of pipe i ( $L^2$ )
- $C_{HW}$  :Hazen-Williams roughness coefficient
- $C_f$  :Unit conversion factor (4.73 English, 10.7 SI)
- $C_i$  :Concentration in pipe i ( $M/L^3$ )
- $C_0$  :Initial concentration (at time zero)
- $C_{i,ni}$  :Concentration entering junction node from pipe i ( $M/L^3$ )
- $C_k$  :Concentration within tank or reservoir ( $M/L^3$ )
- $C_{OUT_j}$  : concentration leaving the junction node j ( $M/L^3$ )
- $C_t$  :Concentration at time t ( $M/L^3$ )
- $C^n$  :Reactant concentration
- $D$  :Pipe diameter (ft, m)
- $D_t$  :Average demand measured at DMA inlet node.
- $D_x$  :Demand per meter pipe
- $d$  :Molecular diffusivity of constituent in bulk fluid ( $L^2/T$ )
- $h_L$  :Head loss due to friction (ft, m)
- $IN_j$  :Set of pipes entering node j
- $K$  :Decay rate constant (reaction rate) ( $1/T$ )
- $k_B$  : Bulk reaction coefficient ( $1/T$ )
- $k_f$  :Mass transfer coefficient, bulk fluid to pipe wall ( $L/T$ )
- $k_w$  :Wall reaction coefficient ( $L/T$ )
- $L$  :Distance between sections 1 and 2 (ft, m)
- $L_i$  :Pipe Length
- $OUT_j$  :Set of pipes leaving node j
- $Q$  :Pipeline flow rate (cfs, m<sup>3</sup>/s)
- $Q_i$  :Flow rate entering the junction node from pipe i ( $L^3/T$ )

$R_H$  :Hydraulic radius of pipeline

$Re$  :Reynolds number

$S$ : Slope of the energy line (head loss per length of pipe or  $h_f/L$ )

$S_H$ : Sherwood number

$t$  :Time

$U_j$  :Concentration source at junction node j (M/T)

$V$  :Velocity

$V_k$  :Volume in tank or reservoir k ( $L^3$ )

$\theta(C_i)$  :Reaction term ( $M/L^3/T$ )

$\theta(C_k)$  :Reaction term ( $M/L^3/T$ )

$\nu$  :Kinematic viscosity of fluid ( $L^2/T$ )

## **CHAPTER 1**

### **INTRODUCTION**

Method of water transportation and putting water into use of people, is one of the most important signs of civilization. First stage of this transportation is taking water from a ground water supply by means of a transmission line, and then storing the water in storage units such as distribution or storage tanks. Before distribution of water to urban areas, it is transferred to a treatment plant. The last stage of the procedure is, potable water transfer to water distribution network (WDN) according to the consumer demands.

Water disinfection means the removal, deactivation or killing of pathogenic microorganisms and this is the most vital step of water procurement for consumers. If unpurified water is drunk or used for an other purpose, it may cause danger in health. Due to all these reasons, many methods are used to obtain, good water quality in the water distribution network (WDN). Chlorine is one of the most common and effective chemical for water disinfection. Despite the fact that chlorine kills harmful organisms, amount of it must not be over a sufficient level. Because, reactions between chlorine and these organisms produce some by-products which causes adverse effects on health.

Water quality models simulate the behaviours of pollutants and the condition of water quality variables in water distribution systems. Some physical, chemical or biological processes are important to understand the transport and transformation of these variables. Water quality is affected by several factors,

such as temperature, solar radiation, organic materials, pH, usage of water, type of construction, etc.

For a water quality model, there are two basic components to analyse; these are disinfectant concentration of the system and how water behaviour differs during the transportation process. In this study, chlorine is used as the disinfectant, chlorine concentration of the system must not be lower than a specific value in order not to endanger human health.

This study was prepared for a small part of Ankara Water Distribution Network (WDN), which is named N8.3 in the north part of the city. This area is mainly divided into six different district metered areas (DMA's). These are; (i) Yayla, (ii) North Sancaktepe, (iii) South Sancaktepe, (iv) Şehit Kubilay, (v) East Çiğdemtepe and (vi) West Çiğdemtepe. Three of these subzones were analysed for water quality studies by changing different system parameters which influence chlorine concentration level in the WDN.

Hydraulic model of N8.3 part of Ankara WDN was taken from Şendil (2013). For this model study, WaterCAD was used (WaterCAD User's Guide,2002). In this model study, this software enables a good water quality modeling solution for engineers to perform constituent, water-age, tank-mixing and source-trace analysis, to develop and simulate the studied WDN.

In the model of N8.3, As disinfectant, chlorine was used as employed by Ankara General Directorate of Water and Sewerage (ASKI) for all regions of water distribution system. Water quality modeling researches may be applied for mainly two conditions. These two conditions can be explained as below.

- 1- For the first condition, all isolation valves between pipes and transmission lines are open. So whole distribution network is interaction between subzones via transmission lines. The network is influenced by properties of

the other parts in the system such as pumps, pipes or reservoirs. In this condition, water is always mixed between adjacent zones and all flows are affected from each other.

2- For the second condition, district metered areas (DMA's) are studied. These areas are divided as containing 1500-2000 consumers approximately and they are separated from each other with isolation valves. Isolation valves between subzones and transmission lines are closed. So whole distribution network is not interaction between subzones via transmission lines or differences in other parts of the network such as pumps, pipes or reservoirs.

In this study, District Metered Areas (DMA's) method is used to analyse water quality model results. In this condition, water reaches each subzones from only a single entrance point and never mixes between adjacent zones. Flows are not affected from each other. Main reason of using DMA's is to detect, to repair and to control leakages from the network (Mac Donald and Yates, 2005). However, while studying the whole network, water may reach subzones from different points during transportation. Therefore, studying whole network may affect the flow field of the system. Because of this reason, studying with DMA's is more practical and efficient for a better estimation of unaccounted water usage, in contrast with studying the whole distribution system. Characteristic behaviour of the system may be understood better with a DMA's study.

The main aim of this study is to examine the differences in chlorine concentrations by changing network properties or some system parameters which affect the water quality such as wall decay coefficient  $k_w$  and bulk decay coefficient  $k_b$ . Also, preparing a study for the most critical cases is aimed to analyse the chlorine concentration levels. All studies were applied for

only DMA's condition with a single pump operation in the subzones, (i) Yayla, (ii) North Sancaktepe and (iii) East Çiğdemtepe.

Yayla is located close to the pump station. This district differs from other subzones because it does not have any neighbouring district so it can be thought as an isolated system from other parts of network. There is no relation with other subzones because Yayla takes water through a single pipeline and water is distributed from this pipe to whole subzone. Because of this isolated behaviour, Yayla is analysed in this study. The water is taken from pump station directly.

North Sancaktepe and South Sancaktepe also take water from the pump station and they are in the middle part of N8.3 WDN. These two subzones behave in a similar way. Therefore; North Sancaktepe is selected to be analysed for this study. Differently from Yayla district, North and South Sancaktepe districts have boundaries between each other. If there is no isolation valves between them, it can not be possible to analyse water quality in these subzones separately. By closing the isolation valves between North Sancaktepe and South Sancaktepe, mixing of water is prevented.

East Çiğdemtepe and West Çiğdemtepe are the most distant subzones from the pump station. Isolation valves are used to isolate these subzones, similarly as in North and South Sancaktepe. Water reaches to these subzones from both the tank and the pump station differently from Yayla or Sancaktepe districts. Therefore, water is taken from the tank only when the pump station is not sufficient. This situation causes different results for chlorine concentrations. Because of these reasons, third subzone is selected as East Çiğdemtepe.

In addition to influence of changing system parameters, leakage effect on water quality is also examined in this study. It is examined that how leakage amounts affect the chlorine concentrations in WDN and how these effects can be



explained depending on water demands by using specific parameters about water quality.

In Chapter 2, general considerations about WDN's and main components of a WDN will be presented in detail. In Chapter 3, general information about water quality modeling and disinfection process in WDN's will be given. In Chapter 4, system parameters that affect water quality will be explained. These are bulk decay coefficient, wall decay coefficient and Hazen Williams roughness coefficient. In Chapter 5, another effective parameter on water quality will be explained in detail, leakage. Influence of water losses on chlorine concentrations will be mentioned. Chapter 6 is the case study part of this thesis. Chlorine concentrations and leakage analysis of N8.3 WDN Model by changing the system parameters, will be presented in this chapter. Finally, the discussion and recommendations about the study will be given in Chapter 7.



## **CHAPTER 2**

### **WATER DISTRIBUTION NETWORKS**

The increasing population, technology development and industrialization increases the water requirement permanently. Depending upon the increasing demand for water resources, clean water depletion causes providing water collection from different sources or supplying water from regions far from the city. This case increases the importance of efficient use of water resources.

A water distribution network is the whole physical equipments that transport water from a natural supply to demanding zones via transmission lines and then put it into service via pipelines. The aim of an ideal WDN is to transport demanded amount of water to consumers with a proper pressure and water quality. Industrialization and rapid urbanization increases the amount of water distribution, so pipe manufacture techniques develops and pipe diameters/types vary. In addition, with the gradual expansion of the network systems of large cities causes water supplying from a large number of sources. Because of the fact that pressure boosting pumps, control valves, etc. equipment is used, to find the optimal solution for a network becomes more complex. There are some important points at planning stage of a WDN. Location, amount and other conditions of sources, the size of the area that receives water, current water requirement and estimated water requirement for future of the region, topographical and geographical properties of the region, an additional water pressurizing component is necessary or not, properties of pipe network and pipe characteristics, water purification methods/facilities, etc.

WDN's are designed for consumer usage in different ways such as, cooking, drinking, cleaning, bathing, gardening (household); commercial, industrial and agricultural use; fire protection, irrigation, etc. For these purposes, WDN's are designed with plenty of equipments such as pipes in particular, pumps, motors, service valves, storage units (reservoirs, tanks) and their specific equipments, fire hydrants, pressure reducing systems, etc.

## **2.1 Main Components of Water Distribution Networks**

### ***2.1.1 Pipes***

Pipes are the main components of a WDN. Water transport is carried out via pipes in different diameters, materials, pressure resistances or lengths. As mentioned before, effect of roughness coefficient on chlorine concentrations are analysed in this study. Pipe material is the most effective property while selecting roughness coefficient of a pipe. Also, wall decay coefficient  $k_w$  is used as an effective system parameter during this study and this parameter is related to the reaction rate at pipe walls. Biofilms which can degrade the water quality by hosting pathogenic organisms, accelerating corrosion or depleting disinfectant residuals; are modeled by this  $k_w$  coefficient. Generally aged pipes have higher  $k_w$  values due to the long term interaction at the pipe wall according to field studies (Li Xin et. al., 2003). Pipes are both used as transmission/distribution lines and service pipes. Pipeline systems can be solved with many computer softwares (KYpipe, WaterCAD, EPANET, CyberNet). WaterCAD is used in this study.

### ***2.1.2 Storage Tanks***

Storage tank is one of the most important equipment in a WDN that stores received water from sources via transmission lines. Its main task is to store excess water that received from the source, when consumer demands decreases to minimum level. Another important aim of a storage tank is to transport water to distribution network when immediate water requirements are necessary.

When the tank is filling, the water entering from upstream pipes mixes with the water that is already in storage. If the concentrations are different, blending occurs. The tank mixing equation mainly accounts for blending but also any reactions related with water quality also occur within the tank volume during any hydraulic step. It is generally assumed that concentrations within the tank or reservoir are completely and instantaneously mixed (Walski et al., 2008).

### **2.1.3 Pumps**

Pump is mechanical equipment that gives energy to the water and by converting the mechanical energy into potential energy, transports it from low pressure regions to high pressure regions.

Planning the proper amount of water for required pressure heads is very important for choosing the true pump combination in a WDN. Accordingly, based on required pressure heads and amount of the water, for every distribution system, pump characteristics and operating management are different. Pump curve properties and pump efficiency play an important role for an ideal system. Usually, more than one pump is used in WDN's, therefore in different conditions, different pump operations can be used for optimum system management.

Specifying the optimum pump schedule is a major problem for water distribution systems which use more than one pump especially. Changing some water quality parameters effects the system scheduling. Because differences in these parameters changes optimum pump scheduling for least-cost design of WDN. For this purpose, the softwares used for water distribution system modeling, may use genetic algorithms to select most appropriate pump schedule. In this study, there are three pumps in the pump station. Pump 1 and Pump 2 have the same capacity while Pump 3 has a larger capacity. During all study only Pump 1 is operating continuously 24 hours.

### **2.1.4 Valves**

A valve is an equipment that regulates, directs or controls the flow of a fluid (gases, liquids or fluidized solids) by opening, closing, or partially obstructing various passageways. In an open valve, fluid flows in a direction from higher pressure to lower pressure. There are many types of valves in a WDN, such as isolation valves, check valves, control valves, air relief valves, pressure reducing valves, drainvalves, etc for different purposes. For example; while check valves are used to prevent the reverse flow in WDN's, control valves are used to arrange the purposes and to balance the high pressures in lower zones. While air release valves are used at top points of pipes to prevent air accumulation there, drain valves are used at the bottom points of pipes to prevent water accumulation there. So that water does not become unusable.

For this study, isolation valves and pressure reducing valves have important roles. Isolation valves influence system behaviour because they are effective on the pathline of water. Pressure reducing valves are also important because of the leakage effect part of the study. Two method may be used in a WDN to decrease water losses. One of them is using a pressure reducing valve in the system. The second one is finding all losses one by one with the help of a device and fixing them all (Bektaş,2010).

- **Isolation Valves**: Isolation valves are special types of valves that completely obstruct the path of flow of a fluid, thereby isolating a portion of the system from fluid flow. Under normal operating conditions, isolation valves remain open. It is typically only under certain special circumstances, such as for safety reasons or for system maintenance or repair, that they are closed. The aim of them is not to be obliged to inactivate the whole distribution system. Flow control capacities of these valves are low, for this reason they must be fully opened or fully closed, or else they damages the system. While studying whole network in a water distribution system, isolation valves between different pressure zones are

fully open and the system works interconnectingly totally. However, while studying with DMA's, all isolation valves must be closed, so water can not be transferred between different pressure zones, so these zones are not affected from each other. Isolation valves prevent interaction.

- **Pressure Reducing Valves:** If there is a considerable amount of pressure head difference through the pipeline, sudden pressure reducing can be provided with pressure reducing valves. Pressure reduction is a very effective means of reducing water loss through leakage particularly at night when demand on the distribution system is lower causing water pressure to rise. Excess pressure minimization in a WDN provides leakage reduction because of preventing failure of pipe connections for example. This type of valves have high flow control capacities. It protects the whole installation from problems due to excess pressure: noises in the pipes, water hammer, splashes, premature wear of household electrical appliances and taps. The pressure reducing valves are completely automatic. Pressure management is now being used not only for leakage control, but also for demand management, water conservation and asset management.

### ***2.1.5 Fire Hydrants***

Normally in the spring, water lines are flushed through the use of fire hydrants as an important preventive maintenance activity. Although it may appear to waste water, the process is part of a routine maintenance program necessary to maintain the integrity of the water system allowing us to continue to deliver the highest quality water possible to our customers. The city's water distribution system is a complex network of pipes and storage reservoirs where sediment or deposits may naturally accumulate over time. If not removed, these materials may cause water quality deterioration, taste and odor problems, or discoloration of the water. Water may also stagnate in lesser used parts of the distribution system. This can result in degraded water quality. Therefore, the systems needs to be routinely flushed. In a water distribution system flushing is

an important tool for helping operators to control distribution system water quality. Flushing stirs up and removes sediments from mains and removes poor quality water from the system, replacing it with fresher water from the source (Walski et al., 2008)

Water contains minerals and these minerals react with the inside of the pipe to produce the by-product. This chemical reaction between the pipe and water is a normal and natural process. This reaction is related to wall decay coefficient  $k_w$  and bulk decay coefficient  $k_b$  also. For example in different seasons because of the temperature differences, chemical reactions occur differently. This process can occur on the inside of the pipe and prevent an adequate volume of water flow. The flushing process removes much of this by-product. Fire hydrants are located close to cross-roads and at 100-150 m intervals. They make water service available for firefighters to assist in extinguishing a fire.

#### ***2.1.6 Junctions and Special Apparatus***

Junctions are pipe connection points in a WDN. To connect pipes and to change pipe diameter or flow direction, special pipe fittings, elbows, tee adapters, flange adapters, or reduction apparatus are used.



## CHAPTER 3

### WATER QUALITY MODELING AND DISINFECTION PROCESS

#### 3.1 Introduction

Water quality models and application of these models, have a very important role in estimation of simulation problems. More specifically, four problem areas are examined in detail: (i) model structure uncertainties, (ii) uncertainties about estimated model parameter values, (iii) the propagation of prediction errors, and (iv) the design of experiments in order to reduce the critical uncertainties associated with a model (Beck, 2010). For analysing the behaviour of real water distribution systems mathematically, model-based simulation is used.

Water quality modeling is a nonignorable process of hydraulic network modeling and it is necessary to perform useful analysis. Mixing due to transportation and decay are the fundamental physical and chemical processes examined by water quality models. Extended Period Simulation (EPS) should be conducted for water quality modeling since transport, mixing and such subjects are time dependent events (Vasconcelos et al., 1997). As a starting point of performing a water quality analysis, the results of an extended period hydraulic simulation can be used (Walski et al., 2008).

#### 3.2 Transport in Pipes

One-dimensional transport equation below is commonly used to predict the changes in constituent concentrations due to transport through a pipe, and to account for formation and decay reactions (Walski et al., 2008).

$$\frac{\partial C_i}{\partial t} = \frac{Q_i \partial C_i}{A_i \partial x} + \theta(C_i), i = 1, \dots, P \quad (3.1)$$

where  $C_i$  = concentration in pipe i (M/L<sup>3</sup>)  
 $Q_i$  = flow rate in pipe i (L<sup>3</sup>/T)  
 $A_i$  = cross sectional area of pipe i (L<sup>2</sup>)  
 $\theta(C_i)$  = reaction term (M/L<sup>3</sup>/T)

Transport in pipes is explained with this equation but also mixing at nodes, mixing in tank and formation reactions are the basic components of a usable water quality model. Bulk reactions in a pipe system affect the water quality model, especially due to the chlorine concentration in the system.

### 3.3 Mixing at Nodes

For the aim of combining concentrations from different pipes a nodal mixing equation is used in a water quality simulation. This equation can be described by the transport equation (Walski et al., 2008).

$$\left( C_{OUT_j} = \frac{\sum_{i \in IN_j} Q_i C_{i,n_i} + U_j}{\sum_{i \in OUT_j} Q_i} \right) \quad (3.2)$$

where,  $C_{OUT_j}$  = concentration leaving the junction node j (M/L<sup>3</sup>)

$OUT_j$  = set of pipes leaving node j

$IN_j$  = set of pipes entering node j

$Q_i$  = flow rate entering the junction node from pipe i (L<sup>3</sup>/T)

$C_{i,n_i}$  = concentration entering junction node from pipe i (M/L<sup>3</sup>)

$U_j$  = concentration source at junction node j (M/T)

This equation uses incoming concentrations to specify flow-weighted average concentrations and explains incoming concentrations to the network or removal from the system. For every pipe junction, this equation assumes that, water reaches to a junction from different pipes and all of them are completely and instantaneously mixed. Turbulent behaviour occurs by this good mixing. Figure 3.1 shows nodal mixing at a junction.

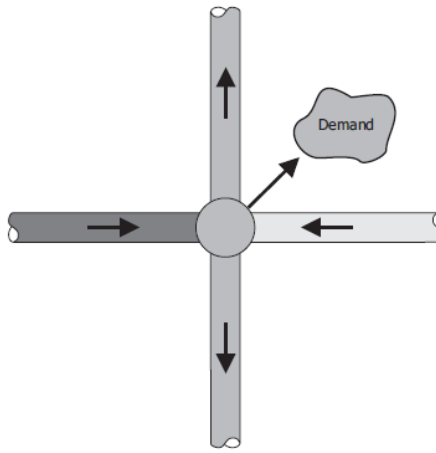


Figure 3.1: Nodal Mixing (Walski et al., 2008)

### 3.4 Mixing in Tanks

In a WDN, pipes are not only connected to junctions but also they may be connected to a tank or a reservoir. For this type of connections the equation below can be performed for water entering or leaving the tank or reservoir.

$$\frac{dC_k}{dt} = \frac{Q_i}{V_k} (C_{i,np}(t) - C_k) + \theta(C_k) \quad (3.3)$$

where,  $C_k$  = concentration within tank or reservoir ( $M/L^3$ )

$Q_i$  = flow entering the tank or reservoir from pipe i ( $L^3/T$ )

$V_k$  = volume in tank or reservoir k ( $L^3$ )

$\theta(C_k)$  = reaction term ( $M/L^3/T$ )

Water comes from upper elevations to a tank or reservoir and mixes with present water there. At this step, blending and any other reactions cause differences in volume of the tank and equation can be simplified.

$$\frac{dC_k}{dt} = \theta(C_k) \quad (3.4)$$

where,  $C_k$  = concentration within tank or reservoir ( $M/L^3$ )

$\theta(C_k)$  = reaction term ( $M/L^3/T$ )

There are different types of mixing models, that can be used for a water quality and these models make differences in process simulation in tanks and reservoirs. For instance, with "First In First Out (FIFO)" model the first volume of water entering the tank as inflow is the first to leave as outflow. On the other hand, with "Last In First Out (LIFO)" model the first volume of water entering the tank during filling is the last to leave while draining. For more complex tank mixing behaviour can "compartment" models may be used. These models help to analyse mixing processes and time delays within tanks more accurately (Walski et al., 2008). Figure 3.2 illustrates the compartment mixing model in tanks.

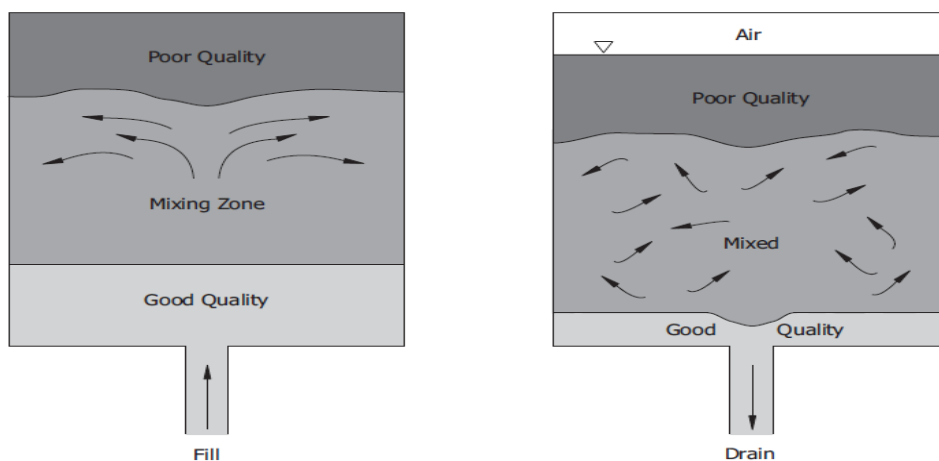


Figure 3.2 Tank Mixing Model in Three-Compartment (Walski et al., 2008)

As it is seen in Figure 3.2, the fresh new water enters the tank from first compartment and it mixes with the older water. When it reaches to oldest water, it mixes with that lowest quality water. The model simulates the exchange of water between different compartments (Walski et al., 2008).

### 3.5 Disinfection In Water Distribution Networks and Water Quality Modeling

Disinfection of water does not mean a sterilized water; but means obtaining safe, good quality, usable and purified from all harmful organisms water. For this purpose, there are many different current methods. These methods can be explained as physical and chemical disinfection methods. Physical methods are based on physical effects as sound, light, temperature differences, etc. For water distribution networks, chemical purification processes are preferred because of the amount of water. Table 3.1 shows these methods and type of water applications.

Table 3.1 Water Disinfection Methods

Disinfection Methods			
Physical Methods	Application	Chemicals Methods	Application
Desalination	Potable water	Biocides	Nondrinking water uses
Distillation	Potable water	Bromine	Nondrinking water uses
Freezing	Potable water	Cations of heavy metals	Nondrinking water uses
Heating/Boiling	Potable water	Chloramine	Potable water
Reverse osmosis	Potable water	Chlorine Dioxide	Potable water
Sedimentation	Potable water	Hydrogen Peroxide	Potable water
Ultrafiltration	Potable water	Iodine	Potable water in emergencies
Ultrasound	Nondrinking water uses	Ozone	Potable water
UV Light	Potable water	Potassium permanganate	Potable water

The most commonly used chemicals include ozone, chlorine and some of its compounds. Ozone is used in the same manner as chlorine. The major difference is that ozone is unstable so cannot be produced and transported to

the point of use. It must be generated at the point of use. A major disadvantage is that due to its instability, ozone must be generated before use and the equipment and operating costs can be quite high. Ozone has an active residual measured in minutes. This lack of long residual is a significant drawback for its use in large distribution systems.

Chlorine is the most widely used chemical for water disinfection and in Turkey, this chemical method is used for potable water disinfection also.

Predicting the behaviour of a real water distribution system by using a model simulation, requires calibration process. Extended Period Simulation (EPS) study must be applied to a water quality model for a certain duration and tank level to compare model results and real field data. Calibration compares the model results with field observations and, if necessary, adjusts the data describing the system until model-predicted values agrees with measured system values (Walski et al., 2008). If the model results and the real case data (obtained from the SCADA records and field observations) match each other, this means that the model is an accurate model or else, this means that the model cannot represent the real case. If they do not match, after the problem is identified, system must be recalibrated. After running the model several times, the system gives better matching results.

Calibration process of the studied network is not involved in the scope of this study. N8.3 WDN was calibrated in previous studies and the model is taken from a previous study which was prepared by Şendil (2013).

### **3.6 Chlorination As A Disinfectant In Water Distribution Networks**

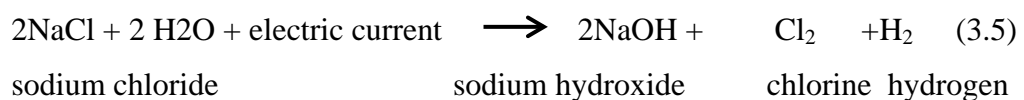
Chlorine (Cl) is a chemical element belonging to halogens and it was first obtained in 1774 by Carl Wilhem Scheele. In 1810, Humphry Davy discovered that it is a new element and named it as Chlorine, from the word “Chloros” which means pale green. It is a greenish yellow poisonous gas. It rages in water

and reacts with it rapidly then produces hypochlorous acid. It was used for water disinfection in 1902, in Belgium firstly. After that, it became a preferable water disinfectant because of its efficiency and economy (AWWA,2006). It is also preferred in Turkey for providing potable water from water distribution networks. There are five important factors that affect chlorination procedure in water distribution network: (i) Chlorine concentration, (ii) Contact time, (iii) Temperature of water, (iv) PH of water and (v) How many foreign substances existing in water (Holben R.J. et al., 2003). If chlorine concentration increases, the contact time may be shortened. However, if chlorine concentration is much more than enough quantity, some by-products occurred in reactions between chlorine and organic matter. This may cause harmful effects on human health. Trihalometane and haloacetic acid are two of them. These byproducts threaten human health substantially causing liver and kidney cancer, chronic health diseases, unconsciousness or death. One research about THM states that, drinking chlorine disinfected water has a 10 – 15 % increased risk of cancer (Hileman, 1993).

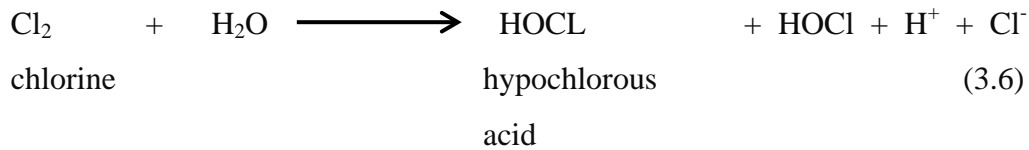
Chlorine gas can be added at different points in the water treatment process, and each step has different effects. This adding point affects contact time or decay rate and influences system behavior.

### ***3.6.1 Chemical Reactions of Chlorine***

Chlorine is stayed in nature only as a component in other chemical compositions and chlorine content in soil changes with distance to sea level. Pure chlorine does not exist naturally. Elemental chlorine is either liquid or gaseous in form. In its liquid form, it must be under extreme pressure. It is commercially produced by the electrolysis of sodium chloride by the following chemical reaction (Wiley, 2010).



While chlorine mixes with water, the reactions below occur;



While chlorine reacts with natural organic matter, many reactions occur and they remove organic matter from the water.

Presence of chlorine in a pipe causes reactions with organic and inorganic matter, microorganisms, or corrosion reactions, etc. Chemical reaction terms were given in Equations 3.2, 3.3 and 3.4. Chlorine concentrations at junctions, in tanks and reservoirs are functions of these reaction terms. Generally these reactions can be explained as two chemical processes. These are; (i) Bulk fluid reactions, (ii) Pipe wall reactions.

Bulk flow reactions totally occur in main flow in a pipe or in a storage tank. They are not influenced by any other processes about the pipe wall. A water quality model simulates these reactions using n-th order kinetics, where the instantaneous rate of reaction (R in unit of mass/volume/time) is assumed to be concentration-dependent.

$$\theta(C) = \pm kC^n \quad (3.7)$$

where,  $\theta(C)$  = reaction term (M/L<sup>3</sup>/T)  
 $k$  = reaction rate coefficient [(L<sup>3</sup>/M)<sup>n-1</sup>/T]  
 $C$  = concentration (M/L<sup>3</sup>)  
 $n$  = reaction rate order constant



The most commonly used reaction model is the first order decay model. A first order decay is equivalent to an exponential decay, represented by Equation 3.8.

$$C_t = C_0 e^{-k \times t} \quad (3.8)$$

where,  $C_t$  = concentration at time  $t$  ( $M/L^3$ )

$C_0$  = initial concentration (at time zero)

$k$  = decay rate constant (reaction rate) ( $1/T$ )

$t$  = time

In addition to bulk flow reactions, constituent reactions occur with material on or near the pipe wall. The rate of this reaction is dependent on the concentration in the bulk flow and pipe wall conditions. As mentioned before, bulk decay and wall decay can not be thought separately. Chlorine reactions occur in the bulk fluid with natural organic matter and also at the pipe wall with biofilms and the pipe material (a cause of corrosion). Figure 3.3 shows this reactions below.

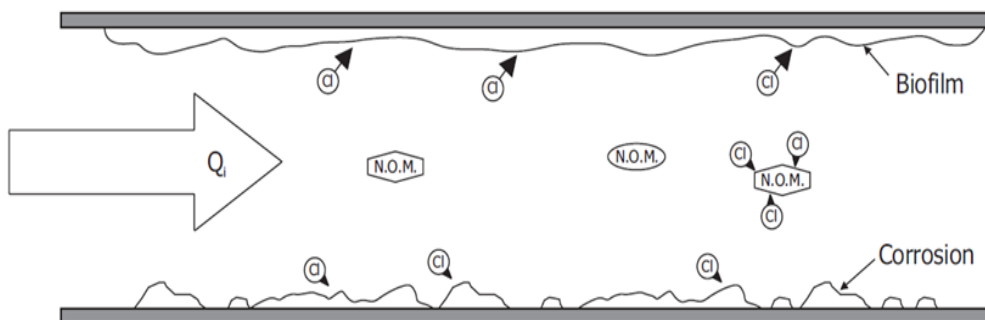


Figure 3.3 Chlorine Reactions in a Typical Distribution System Pipe  
(Walski,2008)

For this reason, a combined physical transport equation can be written. Rossman, Clark, and Grayman (1994) proposed a mathematical framework for this complex reactions occurring within system pipes. This framework explains

physical transport of the disinfectant from the bulk fluid to the pipe wall (mass transfer effects) and the chemical reactions occurring there (Walski et al., 2008).

$$\theta(C) = \pm KC \quad (3.9)$$

where,  $K$  = overall reaction rate (1/T)

If  $K$  is reaction rate coefficient, it is a function of bulk reaction coefficient and wall reaction coefficient as below,

$$K = k_b + \frac{k_w k_f}{R_H (k_w + k_f)} \quad (3.10)$$

where,  $k_b$  = bulk reaction coefficient (1/T)

$k_w$  = wall reaction coefficient (L/T)

$k_f$  = mass transfer coefficient, bulk fluid to pipe wall (L/T)

$R_H$  = hydraulic radius of pipeline (L)

Amount of the disinfectant decay in the system depends on the reaction rate. The mass transfer coefficient is used to determine the rate at which disinfectant is transported using the dimensionless *Sherwood number*, along with the molecular diffusivity coefficient (of the constituent in water) and the pipeline diameter (Walski et al., 2008).

$$k_f = \frac{S_H d}{D} \quad (3.11)$$

where,  $S_H$  = Sherwood number  
 $d$  = molecular diffusivity of constituent in bulk fluid (L<sup>2</sup>/T)  
 $D$  = pipeline diameter (L)

For stagnant flow conditions ( $Re < 1$ ), the Sherwood number is equal to 2.0. For turbulent flow ( $Re > 2,300$ ), the Sherwood number is computed using equation below.

$$S_H = 0.023 Re^{0.83} \left( \frac{v}{d} \right)^{0.333} \quad (3.12)$$

where,  $Re$  = Reynolds number  
 $v$  = Kinematic viscosity of fluid (L<sup>2</sup>/T) (Walski et al., 2008).

### ***3.6.2 Chlorine Dosage and Limits***

Chlorine concentration and dosage is important during chlorination procedure. Disinfection may be confused with sterilization but in sterilization process all living microorganisms are destroyed. However, the aim of disinfection process is to control disease-causing organisms and protect public health.

The standards for drinking water quality are typically set by governments or by international standards. These standards will typically set minimum and maximum concentrations of contaminants for the use that is to be made of the water. Maximum free chlorine concentration level in the water should not be higher than 0.50 mg/l, also the minimum chlorine concentration in the distribution system cannot be less than 0.10 mg/l (Terence J., 1991). According to EPA (Environmental Protection Agency) Standards; maximum free chlorine level for WDN's 1.0 mg / l (as free chlorine). According to Republic of Turkey Ministry of Health, standards for potable water; maximum level of free

chlorine at the point of consumption should not exceed 0.50 mg/l (Republic of Turkey Ministry of Health, 2005). In addition, according to Turkish Drinking Water Standards; minimum level of chlorine at point of consumption is 0.1 mg/l.

Amount of by-products in water is influenced by environmental conditions such as hydraulic conditions, temperature, pH levels and time. For this reason, water quality models have to be set consisting these dynamics for optimizing water quality in water distribution systems.

## CHAPTER 4

### INFLUENCES OF SYSTEM PARAMETERS ON WATER QUALITY

Chlorine concentrations varies depending on some important parameters. These parameters may be the coefficients used in the head loss determination such as roughness coefficient. Differences caused by temperature and amount of organic materials in the system is effective because of the reason that they influence the rate of bulk reactions. Pipe properties and age of construction are also important because of the reason that they influence the rate of wall reactions by corrosion or tuberculation effects, etc. In this chapter, some important parameters will be explained that affect chlorine concentration level.

Chlorine concentration levels can not be lower than a specific value for a WDN not to endanger human health. To analyse chlorine concentrations in a water distribution system, some parameters may be changed and differences of chlorine concentrations can be analysed at specific junctions. Therefore; how these parameters affect the water quality can be interpreted. In this chapter, three basic parameters that affect the water quality will be explained. These are;

- 1) Bulk decay coefficient  $k_b$
- 2) Wall decay coefficient  $k_w$
- 3) Hazen-Williams roughness coefficient  $C_{HW}$

#### **4.1 Bulk Decay Coefficient $k_b$ and Wall Decay Coefficient $k_w$**

Bulk and wall decay coefficients can not be explained in different sections. These parameters are closely related to each other. Chlorine level in WDN pipes is influenced by many concurrent reactions of constituents in the bulk of the water. These reactions may be at or near the pipe walls and sometimes pipe wall material. For this reason, chlorine decay is often modeled using a separate bulk and wall decay model (Jonkergouwa et al., 2009).

The input parameters for a hydraulic model can be classified into two groups; (i) hydraulic model parameters and (ii) water quality model parameters. Hydraulic model parameters can be explained as physical properties of WDN; for example; nodal demands, pipe diameter, pipe length and pipe roughness, etc. However, the input parameters for the water quality model are global bulk and wall decay coefficients basically. Depending on the results, wall decay coefficient and demand in the network are the most effective parameters on the model prediction (Pasha et al., 2011).

Pipe wall reaction rate may differ for all pipes basically depending on pipe material and also bulk reaction coefficient varies from location to location due to mixed water quality from different water sources. The bulk reaction can be in any order (not just 0 or 1) (Wu, 2006).

Determining bulk reaction coefficients involve laboratory analysis. However, determining wall reaction coefficients involve field studies and if necessary a calibration process. It can be said that bulk reaction coefficients are related with individual pipes and storage tanks, they may be applied globally. Wall reaction coefficients are also related with individual pipes, may be applied globally, or assigned to groups of pipes with similar characteristics (Walski et al., 2008).

#### ***4.1.1 Determination of Bulk Reaction Coefficient***

Bulk reaction coefficients can be determined using a simple experimental procedure called a *bottle test*. With the help of a bottle test, the bulk reactions may be explained as only a function of time separating them from other processes that affect water quality. Bottle tests can be performed on any water sample, regardless of where the sample is collected. The volume of water in a bottle can be thought of as a water parcel being transported down a pipe. If samples are taken from treatment plants or other entry points to the distribution system, they have a strong influence on water quality because of the fact that these facilities behave as water sources. If there are different treatment facilities which have different raw water source, the bulk reaction rates for each one would be different depending on different sources (Walski et al., 2008).

In a water distribution system, a separate bulk reaction coefficient may have to be determined for tanks and reservoirs. Because, sometimes although storage tanks are not sources of finished water, the bulk decay rate can be different in tanks and in the distribution system. Mixing process is important when specifying bulk reaction coefficients for pipes. Bottle tests must be performed by taking into consideration representative source mixtures. Mixing level of the system may be specified by a source tracing analysis. Source blending and mixing can change the course of the flow for a pipe. Therefore, the predominant source or mix of sources should be used while specifying bulk reaction coefficient. The bulk reaction coefficient is a function of the water passing through a pipe or stored in a tank, and not a function of the pipe or the tank for single source networks. Therefore, the simplest and the best method for modeling is specifying a global bulk reaction coefficient in such networks (Walski et al., 2008).

#### ***4.1.2 Determination of Wall Reaction Coefficient***

Calibration study is performed to specify wall reaction coefficient. Wall reaction coefficients are more difficult to measure and they are frequently estimated by using disinfectant concentration, field measurements, chlorine residuals and other factors. Therefore, calculated wall reaction coefficient would result in the observed behaviour. Isolating a pipe that has a specific diameter, material, age and flow; and then measuring chlorine residuals and other factors for that pipe is the best method.

Bağcı (2001) studied on wall decay coefficients performing field measurements in N8.3 WDN of Ankara. Sampling program was carried out between 21st of July 2001 and 27th of July 2001 for a period of seven days. At each sampling point the residual chlorine was measured at same hour everyday by means of an equipment called color disc pocket comparator. Storage tank levels were measured from the bottom of the tank also by using hydraulic pattern as input parameters to CyberNet, model was created by the help of demand curves. After 168 hr Extended Period Simulation (EPS) analysis, model results and field observations were compared. By using Root Mean Square Error Method, wall decay coefficient  $k_w$  results were matched to observation results (Bağcı, 2001). Reported values for wall reaction coefficients are in the range of 0 to 1.5 m/day. Estimations can be based upon field concentration measurements and water quality simulation results as part of a calibration analysis (Walski et al., 2008).

#### ***4.1.3 Factors Affecting the Bulk Reaction Coefficient***

The bulk decay is due to chlorine consumption with organic and inorganic matters. Chlorine bulk decay is affected by the initial chlorine concentration, total organic carbon, and temperature in treated water. While initial chlorine concentration increases,  $k_b$  values decrease if the other water quality parameters are fixed. However,  $k_b$  increases with the total organic carbon and temperature (GeorGescu et al., 2012). Temperature plays an important role,



increasing the bulk decay coefficient by a factor of three for every 10 C° rise in temperature (Mutoti et al., 2007).

Booster disinfection (when disinfectant is reapplied to previously disinfected water) is another circumstance in which bulk reaction coefficients are likely to change (Walski et al., 2008).

#### ***4.1.4 Factors Affecting the Wall Reaction Coefficient***

Pipe wall reactions may be explained as water movement inside the pipe and reacting with its material or biofilms on inner pipe walls. These reactions are called pipe wall reactions. Factors which affect wall decay coefficient of the pipes are: Pipe material, the velocity of movements inside the pipe, initial concentration of chlorine and decay (Rezaian et al., 2013).

Corrosion products or biofilm on the wall react with dissolved substances. Therefore, pipe wall area affects total rate of mass transfer between the bulk fluid and the pipe wall (pipe wall reaction) (GeorGescu et al., 2012).

Pipe diameter should also be taken into consideration, because the internal surface- area-to-volume ratio decreases with increasing diameter. As a result, for the same pipe material, age, and flow rates, greater total chlorine consumption is experienced in a smaller diameter pipe than a larger diameter pipe (Mutoti et al., 2007).

In addition to pipe diameter, pipe material is also an effective factor on wall decay coefficient. Pipe material can influence the transport of free chlorine. For example, chlorine residuals dissipate rapidly in unlined ductile-iron pipe but are maintained in PVC pipe.

#### **4.1.5 Water Age**

Chlorine concentration decreases with time in pipes and tanks through bulk decay and wall decay in the system. Water age is influenced by differences in water use habits. Water age is highest in the early morning hours and lowest in the late evening due to storage in the distribution system. Because of the fact that it is a function of water demand, system operation, and system design; when water demand increases, the time that water stays in the system decreases. Demand is influenced by land use patterns, types of commercial-industrial activity present in a community, the weather (i.e., lawn watering), and water use habits of the community. Increased water age cause decreased corrosion control effectiveness, nitrification, and microbial growth/regrowth (AWWA, 2002).

#### **4.2 Hazen-Williams Roughness Coefficient $C_{HW}$**

There are different formulae for different hydraulic conditions to determine frictional losses in a hydraulic system. Chezy, Darcy-Weisbach, Hazen-Williams and Manning formulae are the most commonly used ones. Darcy Weisbach formula is a more physically-based equation, derived from the basic governing equations of Newton's Second Law. With appropriate fluid viscosities and densities, Darcy-Weisbach can be used to find the head loss in a pipe for any Newtonian fluid in any flow regime. The Hazen-Williams and Manning formulas, on the other hand, are empirically-based expressions (meaning that they were developed from experimental data), and generally can only be applied to water under turbulent flow conditions. The Manning formula is not typically used for water distribution modeling, it is generally used for open channel flows.

In 1906, Hazen and Williams provided the formula that was easy to use. The general form of the equation relates the mean velocity of water in a pipe with the geometric properties of the pipe and slope of the energy line.

$$V = kC_{HW}R_H^{0.63} = C_{HW}R^{0.5}S^{0.54} \quad (4.1)$$

where,  $V$ = velocity

$k$ = a conversion factor for the unit system ( $k = 1.318$  for US customary units,  $k = 0.849$  for SI units)

$C_{HW}$ = a roughness coefficient

$R_H$ = hydraulic radius

$S$ = slope of the energy line (head loss per length of pipe or  $h_f/L$ )

The empirical Hazen-Williams equation is widely used in practice to define the discharge-head loss relation for water flows in full pipes.  $C_{HW}$  is Hazen-Williams roughness coefficient which ranges from 150 for smooth-walled pipes to as low as 80 for old, corroded cast iron pipes (Larock et al., 1999). After some arrangements in this formula, the formula below is provided,

$$h_L = \frac{C_f L}{C_{HW}^{1.852} D^{4.87}} Q^{1.852} \quad (4.2)$$

where,  $h_L$ = head loss due to friction (ft, m)

$L$ = distance between sections 1 and 2 (ft, m)

$C_{HW}$ = Hazen-Williams factor

$D$ = diameter (ft, m)

$Q$ = pipeline flow rate (cfs, m<sup>3</sup>/s)

$C_f$ = unit conversion factor (4.73 English, 10.7 SI)

Hazen-Williams formula is unsuitable for accurate prediction of head loss because of the empirical nature of the roughness coefficient  $C_{HW}$ . The Hazen-Williams method is very popular, especially among civil engineers, because of its ease of use and not being a function of velocity or pipe diameter. It is a function of pipe material, pipe linings and pipe age (ASPE, 2008).

$C_{HW}$  roughness coefficient also differs with the condition of pipes. For example for a junction, which way the water follows to reach that junction is important. Because water age in the pipe affects chlorine concentrations and  $C_{HW}$  coefficients for each pipes.

Various researchers and pipe manufacturers have developed tables that provide estimates of pipe roughness as a function of various pipe characteristics, such as pipe material, pipe diameter, and pipe age (Lamont, 1981). For providing a general idea about the effect of pipe material on C roughness coefficient, the Table 4.1 below can be examined.

Table 4.1 A Sample Table for Hazen-Williams Roughness Coefficient (The Engineering ToolBox, 2014)

Material	Hazen-Williams Coefficient	Material	Hazen-Williams Coefficient
ABS - Acrylonite Butadiene Styrene	130	Ductile Iron, cement lined	120
Aluminum	130 - 150	Fiber	140
Asbestos Cement	140	Fiber Glass Pipe - FRP	150
Asphalt Lining	130 - 140	Galvanized iron	120
Brass	130 - 140	Glass	130
Brick sewer	90 - 100	Lead	130 - 140
Cast-Iron - new unlined (CIP)	130	Metal Pipes - Very to extremely smooth	130 - 140
Cast-Iron 10 years old	107 - 113	Plastic	130 - 150
Cast-Iron 20 years old	89 - 100	Polyethylene, PE, PEH	140
Cast-Iron 30 years old	75 - 90	Polyvinyl chloride, PVC, CPVC	150
Cast-Iron 40 years old	64-83	Smooth Pipes	140
Cast-Iron, asphalt coated	100	Steel new unlined	140 - 150
Cast-Iron, cement lined	140	Steel, corrugated	60
Cast-Iron, bituminous lined	140	Steel, welded and seamless	100
Cast-Iron, sea-coated	120	Steel, interior riveted, no projecting rivets	110
Cast-Iron, wrought plain	100	Steel, projecting girth and horizontal rivets	100
Cement lining	130 - 140	Steel, vitrified, spiral-riveted	90 - 110
Concrete	100 - 140	Steel, welded and seamless	100
Concrete lined, steel forms	140	Tin	130
Concrete lined, wooden forms	120	Vitrified Clay	110
Concrete, old	100 - 110	Wrought iron, plain	100
Copper	130 - 140	Wooden or Masonry Pipe - Smooth	120
Corrugated Metal	60	Wood Stave	110 - 120
Ductile Iron Pipe (DIP)	140		

Vasconcelos, Rossman, Grayman, Boulos, and Clark (1997) postulated that the wall reaction coefficient is related to pipe roughness according to the following equation,

$$k_w = \alpha / C_{HW} \quad (4.3)$$

where,  $k_w$  = wall reaction coefficient (ft/day)  
 $\alpha$  = fitting coefficient  
 $C_{HW}$  = Hazen-Williams roughness coefficient

Generally, wall decay coefficient measurement is a difficult process and it is inversely proportional to pipe roughness in literature. It can be said that if the wall reaction constant vary inversely with pipe roughness; older and more deteriorated pipes will have higher reactivity (Vasconcelos, 1996).

The fitting coefficient is determined for a given system by trial and error during calibration. From sufficient number of observation points, concentrations have been measured in the system, initial wall reaction coefficients for each pipe can be estimated and the simulation performed. After that, the observed concentrations are compared to concentrations provided by the computer model. If these values do not match to each other, the wall decay coefficients should be adjusted until a suitable match is obtained (Walski et al., 2008). This application was used by Bağcı (2001) for calibrating wall decay coefficient as mentioned before.

In this study, chlorine concentrations show significant differences by changing bulk decay coefficient  $k_b$  and wall decay coefficient  $k_w$ . However, system is performed for three different Hazen-Williams roughness coefficients, these differences do not affect chlorine concentrations at specified junctions significantly, in subzones that receive water only from the pump station. This means that, system parameters that affect water quality are bulk decay and wall decay coefficients directly. Roughness coefficient  $C_{HW}$  affects the system directly, only when subzones receive water from different water supplies.

## **CHAPTER 5**

### **LEAKAGE**

Leakage is an important parameter about WDN's. It can be defined as the total water loss caused by different parts of the system (such as pipes, joints and fittings) during transport, or overflow in tanks by reason of changing consumer demands. Amount of leakage varies from network to network but it is inevitable. Minimizing the water leakage in a water distribution system is one of the basic part of water demand management because leakage is the biggest part of the losses. This means less drinking water, more energy waste, natural resources waste and more maintenance costs.

In a WDN, relation between amount of leakage and total distributed water, affects the efficiency of the whole system. If the water distribution network is old, leakage amount is higher than newer networks. The amount of actual leakage depends on pressure, burst frequency, leakage control policy and age of pipes. The amount of leakage varies from system to system, but there is a general correlation between the age of a system and the amount of Unaccounted-for water. If water distribution system is new, leakage amount may have 5 percent of total water amount. However for older systems this percentage may increase to 40 percent leakage or higher. If there is no leak detection and repair applications, leakage tends to increase over time. Other factors affecting leakage are system pressure, burst frequency of service pipes. The higher the pressure, the more leakage in a water distribution system and high pressures increase burst frequency of service pipes (Walski et al., 2008).

### **5.1 Reasons of Leakage in a WDN**

Change of consumer water demands causes fluctuations and overflows on the surface of storage tanks. This kind of leakage provides the water balance of WDN. This approach can be expressed as potential losses and it involves water losses from all pipes, fittings, junctions and holes or fractures. Although leakage caused by joints or junctions can be negligible, water loss from fractures or holes may increase the total water loss to nonignorable level depending upon the size of holes or fractures. If damage of the pipes or other equipments is largely effective, it must definitely be planned and reported. Damage failure varies depending on the pressure resistances, materials, or ages of pipes; regional properties, usage of suitable fittings/special parts, etc. Higher supply pressure may cause rupture or burst especially in the systems that have high number of joints, fittings, interconnections and relatively short pipes. Water hammer also causes unexpected pressure differences and so water losses. Excessive loads and vibrations because of road traffic affect leakage amount also.

Corrosion and erosion effects are also important on leakage amount. Corrosion in and out of the pipes or fittings weaken the pipe wall. It shorten the water distribution system's life. Erosions caused by the soil characteristics, change of temperature and moisture amount, affect the system negatively.

To analyse the effect of pipe materials on leakage amount, different pipe materials with the same geometry were used. These pipes were steel and polyethylene pipes. The results confirmed the viscoelastic behaviour of the leaks in these pipes, i.e. the leakage dependence on the functioning conditions time-history. However, the results of steel pipes showed an elastic or elastoplastic behaviour of the leaks (Ferrante et al., 2011).

In a previous study prepared by Bektaş (2010), various factors affecting leakage were explained as; pressure, hole size, leak duration, pipe material,



aging of pipes, workmanship, network design, soil types and water quality (Bektaş, 2010).

## **5.2 Leak Detection Methods**

Solid leak detection equipment or leak noise correlator are some of the leakage measurement techniques. In recent years, for development of water audit procedures and leak detection methods, significant efforts have been made. As a result, water system operators now have several techniques and equipments to control water losses (Hunaidi, 2000). There are several ways of detecting leaks. Some of them are sound detection or vibration methods. Leak noise logging devices are being used for this purpose for more than 20 years. Simply they are mobile microphones that have recording capability. They are generally located at the fittings such as valves, fire hydrants etc. It records the noises of the pipeline for leak sounds. Pinpointing leak spots with ground microphones is one of the most preferred techniques in water leak detection (Bektaş, 2010).

Leaks could also be detected using several non-acoustic methods such as tracer gas, infrared imaging, and ground penetrating radar. The use of these techniques for this purpose, however, is still very limited and their effectiveness is not as well established as that of acoustic methods (Hunaidi, 2000).

## **5.3 Minimizing Leakage in a WDN**

Leakage can be reduced by decreasing hydraulic pressures in a typical WDN because of the fact that leak is a pressure dependent function. A good pressure management must be applied to water distribution systems for minimizing the negative effects of leakage. When pressure drops below normal, less water is used and leakage decreases. When evaluating demands as a function of pressure drop, the model must be adjusted for this change in demand or the simulation results will be conservative. Flow control valves and pressure reducing valves are usable for this purpose. Reducing pressure should reduce

leakage and improve the service life of plumbing fixtures. It is important that the performance of the PRV may be modeled before installation to determine the impacts on existing customers and fire flows (Walski et al., 2008).

Leakage does not only affect the efficiency of WDN negatively, also reduces the water quality. To minimize leakage level, essential data sets can be provided by the help of geographical information systems (GIS). A good planned, designed and optimised WDN can be succeed by using the water supplies effectively to minimize the water losses.

In addition to water pressure management, active leakage control involves the disaggregation of large networks into smaller areas (called District Metered Areas) that can be better monitored (Engelhardt et al., 2000). As it is mentioned before, for this study DMA's method was carried out. Carrying on the studies about leakage with DMA eases to compare results before and after a maintainance. When DMA case is used, the best leakage analysis can be performed at minimum flow level which occurs at night usually. At night, when demands are at minimum levels, frictional losses are at minimum levels also and the pressures are high comparing to peak hours. Therefore, average leakage flow becomes at maximum levels.

The target in a WDN is always zero leakage, but this is impossible for a real case. Minimizing the water leakage provides long life system operation, increases the operation efficiency; decreases the operational expenses, the waste or loss potential, the damages which prevent system operation and water losses. Detection and repair of damages are expensive procedures but it is worth that benefits caused by water savings by preventing leakage.

## CHAPTER 6

### CASE STUDY

This study was carried out in one of the northern pressure zones of Ankara which is named N8.3. This pressure zone is isolated from other zones with definite boundaries and the isolation of this subzone was checked during extensive field tests (Şendil, 2013; Ar, 2011). N8.3 is divided into six subzones (DMA's and subzones are used as similar meaning during this study) and these are named as; Yayla, North Sancaktepe, South Sancaktepe, Şehit Kubilay, East Çiğdemtepe and West Çiğdemtepe. As mentioned before, different chemicals can be used for disinfection of water; however, in this study chlorine is used as a disinfectant for consumers health as employed by Ankara ASKI for all regions of water distribution system.

#### **6.1 Hydraulic Model of the Network and System Characteristics**

A hydraulic model of a WDN is intended to simulate a real case WDN as a mathematical representation. A hydraulic model of the area N8.3 was studied by previous researchers and WaterCAD was used for those studies. The aim of this study is to analyse a water quality model which uses transport, mixing and decay equations; and how the disinfectant chlorine is influenced by changing some system parameters. For this purpose, again WaterCAD was used and EPS was used to analyse the time-dependent changes in WDN.

Water distribution networks and water quality of a network are affected by several different factors, such as temperature, solar radiation, organic materials, pH, usage of water, pipe materials, age of the network, type of construction,

etc. These factors influence some system parameters which were explained in previous chapters. To process and model all these changes, an EPS study is performed by changing these parameters systematically. EPS determines the behaviour of the system in a specific period of time and it performs a series of steady-state simulations in which hydraulic demands and boundary conditions do change with respect to time (Walski et al., 2008).

A subzone is a defined area of the distribution system that is isolated by isolation valves and therefore, quantities of water entering and leaving can be metered. Analysis of flow and pressure enables leakage specialists to calculate the level of leaks in the district at measurement chambers, especially at night, when a high proportion of users are inactive. DMA's can be used to reduce leakage and also to compare levels of leakage in different districts. While using DMA's, basically with the help of isolation valves, water does not mix between subzones and behaviour of the water in a subzone does not affect the others. DMA's consists 1500-2000 sections usually and receive water from a single entrance to the area. DMA's provide better estimation of unaccounted water usage.

All studies were applied for only DMA's condition with a single pump operation in the districts; (i) Yayla, (ii) North Sancaktepe and (iii) East Çiğdemtepe. Yayla is selected because of the fact that it does not have any neighbouring district so it can be thought as isolated from other parts of network. North Sancaktepe and South Sancaktepe are located in the middle of whole N8.3 network and behave in a similar way. Differently from Yayla district, these districts have boundaries between each other and isolation valves are used to separate them while performing a DMA's study. North Sancaktepe is selected to analyse for this study. East Çiğdemtepe and West Çiğdemtepe are the most distant subzones from the pump station. Isolation valves are used to provide them behave separately, similarly as in North and South Sancaktepe. These subzones are feeded from both tank and the pump station, differently

from Yayla or Sancaktepe districts. Because of these reasons, third subzone is selected as East Çiğdemtepe.

N8.3 WDN has a tank named T53 and a pump station named P23. Characteristics of the tank will be explained in Section 6.1.5 and characteristics of the pump station will be explained in Section 6.1.6. Hydraulic model of the network was developed by other researchers and for this study, model was taken from Şendil (2013); valve locations and status were also defined in that study. In Figure 6.1, a sketch for N8.3 WDN is shown below.

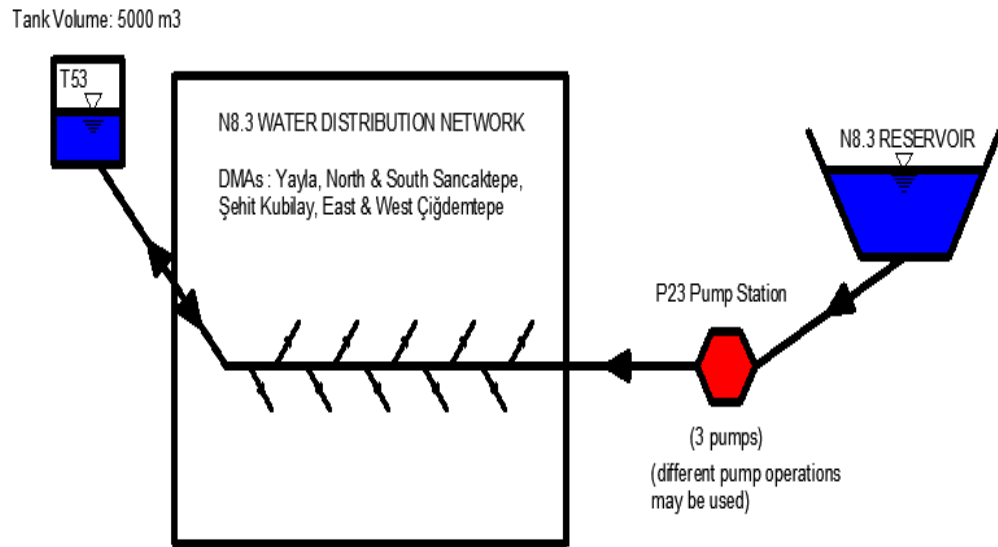


Figure 6.1 Sketch of N8.3 WDN

A satellite view of N8.3 WDN is given in Figure 6.2. In this figure, green pointers show the tank and the pump station. Blue pointers show measurement chambers in subzones and red pointers show test nodes in these subzones. The subzones of N8.3 are shown with different colors in Figure 6.3. Isolation valves, tank and pump station details can be shown in this figure.





Figure 6.2 N8.3 Network Satellite Preview (Google Earth) (Ar, 2011)

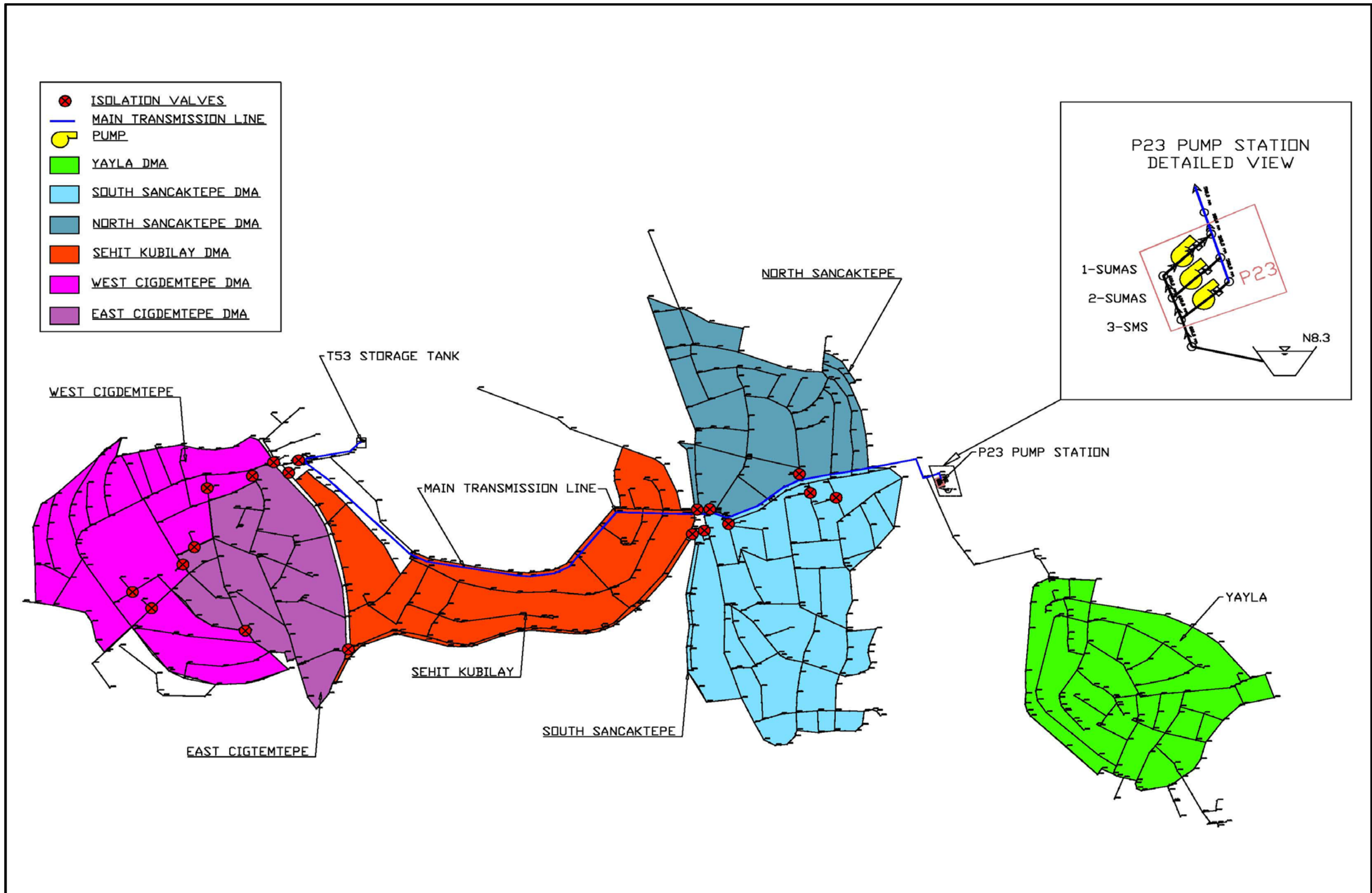


Figure 6.3 View of N8.3 Network and DMA's (Nadiroğlu, 2014)



### **6.1.1 Selection of Hazen-Williams Coefficient $C_{HW}$**

Calibration studies for assigning a valid Hazen-Williams roughness coefficient were conducted by Ar (2012) and Apaydin (2013). However, in this study system was run for three different a Hazen-Williams roughness coefficient value;  $C_{HW}=55$ ,  $C_{HW}=100$  and  $C_{HW}=145$ . The hydraulic model used in this study was taken from the study of Şendil (2013).

### **6.1.2 Selection of Bulk Decay Coefficient $k_b$**

For bulk decay coefficient, two different  $k_b$  values were used as an effective parameter. One of them is for 5°C (winter case), and the other one is for 15°C (summer case). To decide the bulk decay coefficients for these temperatures, some previous studies were considered. One of these is a study prepared by Xin et al. (2003). In this study, for different temperatures,  $k_b$  values were determined in laboratory environment with bottle tests. With this study,  $k_b$  value may be selected as 0.21 1/day for summer season (15 C°) and as 0.06-0.07 1/day for winter season (5-6 C°) depending on the results for iron pipes below,

<b>Temperature C°</b>	<b><math>k_b</math> (1/day)</b>
0	0.0301
5	0.0594
10	0.1140
15	0.2130
20	0.3900
25	0.7000
30	1.2300

Another study was prepared in Japan, by Nagatani et al. (2006). In this study, field studies were done in Sakishima district with 25-35 years mortar lined pipes. For the purpose of determining  $k_b$  value, an equation was proposed as below,

$$k_b = -0,0398 \exp (0,0742T)$$

Therefore, by using this equation, the values below were obtained.

Temperature C°	$k_b$ (1/day)
0	0.0398
5	0.0577
10	0.0836
15	0.1211
20	0.1755
25	0.2544
30	0.3687

With this study,  $k_b$  value may be selected as 0.12-0.14 1/day for summer season (15-17 C°) and as 0.06-0.07 1/day for winter season (5-6 C°). As it is seen, these studies are coherent with each other especially for winter season.

In this study, pipes are ductile iron in N8.3 WDN and age of the system is available to use previous studies as base. Therefore, in this study, system was run for two different chosen  $k_b$  values. One of them is 0.07 1/day for winter case, and the other one is 0.14 1/day for summer case. While selecting for summer case, because of the reason that default value of WaterCAD is also 0.10 1/day, smaller one is selected.

### **6.1.3 Selection of Wall Decay Coefficient $k_w$**

Wall reaction rate coefficients are seen varying between 0 and 1.0 ft/day, as is typical of most distribution systems. (Hudkins et al., 2010). This gives a range between 0-0.305 m/day for typical systems.

In this study system was run for three different wall decay coefficient. For totally new pipes, 0.00 m/day value was used as  $k_w$ . As a second value, the default  $k_w$  value of WaterCAD 0.08 m/day was used. And also as a third value, 0.20 m/day was used for assuming older pipes in N8.3 WDN.

To sum up the studies for these three different Hazen-Williams Coefficient  $C_{HW}$ , three different wall decay coefficient  $k_w$  and two different bulk decay coefficient  $k_b$  values, 18 different scenarios are studied and shown in Table 6.1a below.

Table 6.1a Case Studies For Different System Parameters

Scenario	$C_{HW}$	$k_w$ (m/day)	$k_b$ (1/day) (winter)	Run	$C_{HW}$	$k_w$ (m/day)	$k_b$ (1/day) (summer)
1	55	0.00	0.07	10	55	0.00	0.14
2	55	0.08	0.07	11	55	0.08	0.14
3	55	0.20	0.07	12	55	0.20	0.14
4	100	0.00	0.07	13	100	0.00	0.14
5	100	0.08	0.07	14	100	0.08	0.14
6	100	0.20	0.07	15	100	0.20	0.14
7	145	0.00	0.07	16	145	0.00	0.14
8	145	0.08	0.07	17	145	0.08	0.14
9	145	0.20	0.07	18	145	0.20	0.14

In addition to these 18 scenarios, by using Hazen-Williams roughness coefficient  $C_{HW}$  as 130, wall decay coefficient  $k_w$  as 0.08 m/day and bulk decay coefficient  $k_b$  as 0.10 1/day as used in a previous thesis study prepared by Nadiroğlu (2013); three different scenarios are also analysed. These are without leakage case, normal demand case and increased leakage case. Studied scenarios for analysing leakage effects is shown in Table 6.1b.

Table 6.1b Case Studies For Different Leakage Amounts

Scenario	$C_{HW}$	$k_w$ (m/day)	$k_b$ (1/day)	Leakage
19	130	0.08	0.10	without
20	130	0.08	0.10	normal
21	130	0.08	0.10	increased

At the end of the study, as a most critical case, two scenarios are analysed also. These are studied with two different bulk decay coefficients for winter and summer seasons. These most critical case studies are performed in without leakage case by using Hazen-Williams roughness coefficient  $C_{HW}$  as 145 and wall decay coefficient  $k_w$  as 0.20 m/day. Runs of the most critical cases are shown in Table 6.1c.

Table 6.1c Case Studies For Different Most Critical Conditions

<b>Scenario</b>	<b><math>C_{HW}</math></b>	<b><math>k_w</math> (m/day)</b>	<b><math>k_b</math> (1/day)</b>	<b>Leakage</b>
<b>22</b>	145	0.20	0.07	without
<b>23</b>	145	0.20	0.14	without

Therefore, during this study, 23 different scenarios are studied totally and analysed with regards to water quality.

#### **6.1.4 Pump Operations**

There are three pumps that are connected to reservoir at the entrance point (static hydraulic level: 1106.81) and network receives water from a 5000 m<sup>3</sup> volumed tank in the hydraulic model of N8.3. A sketch for N8.3 was given in Figure 6.1 before.

For this study, a single pump schedule was operated which uses one small pump (Pump 1- SUMAS) working 24 hr. continuously with three different Hazen-William roughness coefficients ( $C_{HW}=55$ ,  $C_{HW}=100$ ,  $C_{HW}=145$ ), three different wall decay coefficients ( $k_w= 0.00$  m/day,  $k_w= 0.08$  m/day,  $k_w= 0.20$  m/day) and two different bulk decay coefficients for winter and summer seasons ( $k_b= 0.07$  1/day,  $k_b= 0.14$  1/day).

### 6.1.5 Tank (T53)

The storage tank of the N8.3 WDN is named "T53". It is located in the north east part of the network. The basic characteristics of the tank is given below in Table 6.2.

Table 6.2 Characteristics of The Tank (T53)

<b>STORAGE TANK</b>	T53
<b>VOLUME</b>	5000 m <sup>3</sup>
<b>CROSS-SECTION TYPE</b>	Rectangular
<b>CROSS-SECTION AREA</b>	800 m <sup>2</sup>
<b>HEIGHT</b>	5 m
<b>ACTIVE STORAGE VOLUME</b>	2600 m <sup>3</sup>
<b>BASE ELEVATION</b>	1149.82 m

Tank has a very important role in water quality modeling, because the chlorine concentration of the tank affects the whole network. Tanks are assumed as completely mixed in water quality modeling and this is called Fully-Mixed Water Quality Model. There are different types of mixing models as mentioned in Chapter 3. For instance, with "First In First Out (FIFO)" model the first volume of water entering the tank as inflow is the first to leave as outflow. On the other hand, with "Last In First Out (LIFO)" model the first volume of water entering the tank during filling is the last to leave while draining. For more complex tank mixing behaviour can "compartment" models may be used. These models help to analyse mixing processes and time delays within tanks more accurately (Walski et al., 2008).

WaterCAD incorporates all these kind of mixing processes; compartment, completely mixed, FIFO and LIFO. In this study for N8.3 WDN, the tank T53 actually fits for Fully-Mixed Water Quality Model. This study is closely related to the amount of leakage in the system and the fluctuations on the surface of

the tank give reliable results about the amount of leakage in the system. The most stable results are provided with using fully-mixed process during the study. As a result of this, the concentrations in overall of the system were affected and to analyse and comment the results reasonably Fully-Mixed Model is used during the process.

#### **6.1.6 Pump Station (P23)**

The pump station of the network is named as "P23". As it is mentioned before the Figure 6.3, there are three pumps at the station. Two of them are manufactured by SUMAS and they have equal capacity (Pump 1 and Pump 2), but Pump 3 is manufactured by SMS (Samsun Makina Sanayi) and it has a larger capacity than Pump 1 and Pump 2. Pump efficiencies and design characteristics of these three pumps are given in Table 6.3.

Table 6.3 Characteristics of The Pumps in P23 Station

<b>PUMP STATION</b>	<b>P23</b>	
<b>PUMP NUMBER</b>	Pump 1, Pump 2	Pump 3
<b>MANUFACTURER</b>	SUMAS	SMS
<b>TYPE</b>	SP 125	SP 150 - 400/493
<b>DESIGN FLOW</b>	188 m <sup>3</sup> /h	350 m <sup>3</sup> /h
<b>DESIGN HEAD</b>	45 m	45 m
<b>PUMP EFFICIENCY</b>	74%	80%

Because of the fact that Pump 1 and Pump 2 have a lower capacity, they are called as SM (Small Pump). In addition, Pump 3 is called as LG (Large Pump) because of the higher capacity. This terminology will be used along all parts of this study.

Pump curves are also added after table; for Pump 1, Pump 2 (same pump curves) and Pump 3. Pump curve is a graphical representation of a pump's

performance characteristics. Each pump has a definite capacity curve for a given impeller diameter and speed. These curves are provided by the manufacturers after several pump tests. The pump curves for SUMAS and SMS is given in Figure 6.4 and 6.5 respectively.

These three pumps receive water from a reservoir which is the water source of the network. The Hydraulic Grade Level (HGL) of the reservoir is 1106.81 m. This water source is the only connection of N8.3 with the Northern Pressure Zone. In real case, there is no reservoir in N8.3 WDN and water sources are out of the city, far from N8.3 and there is a pipe connection with pump station. This point has a hydraulic grade level (HGL) depending on the system's pressure and the average of hydraulic grade level (HGL) readings are used to model the sources as a reservoir.

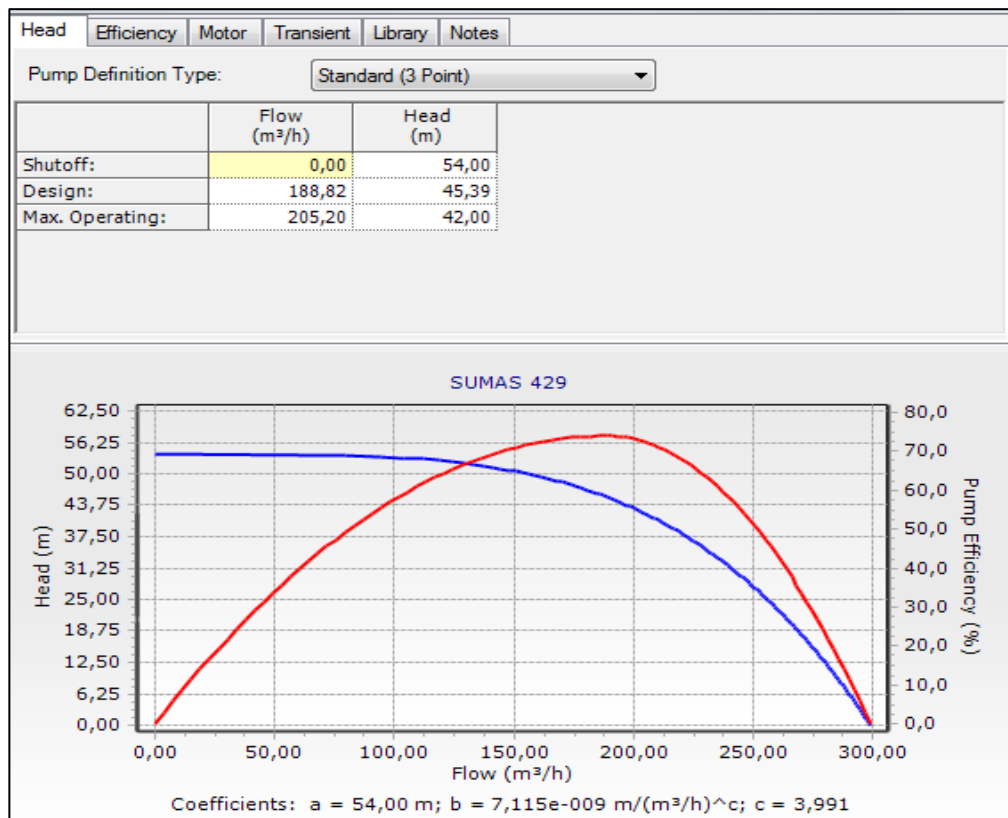


Figure 6.4 Pump Curve of SUMAS (Pump 1 and 2)

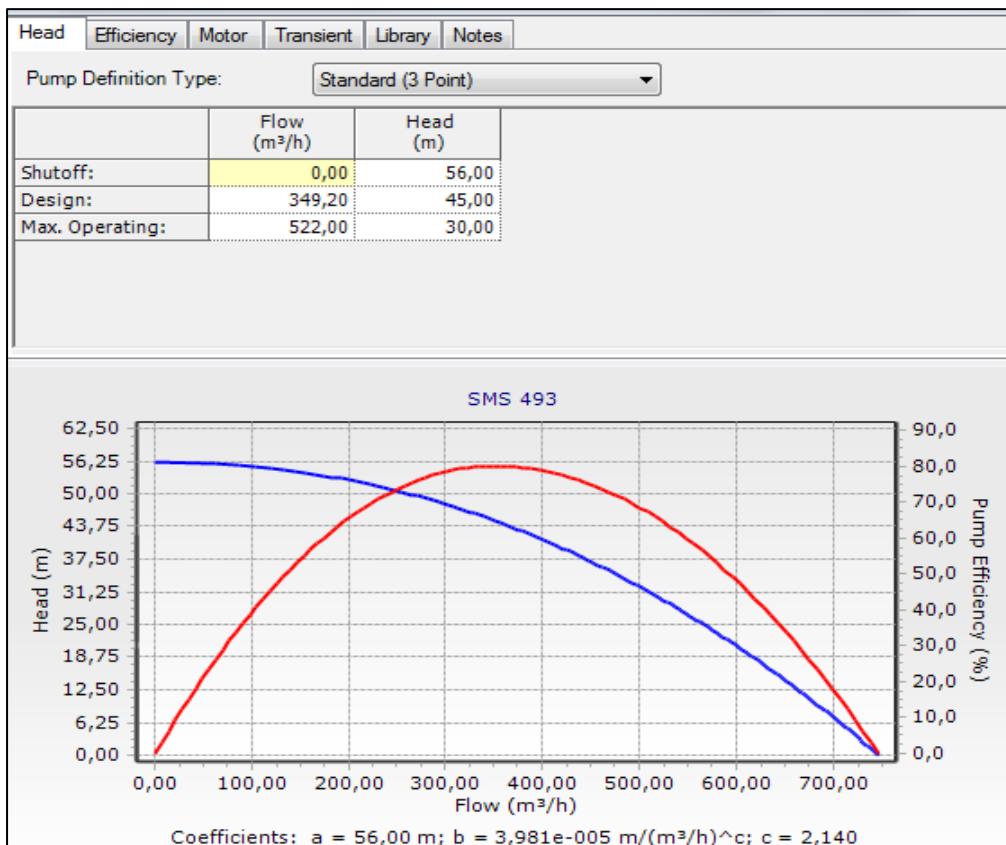


Figure 6.5 Pump Curve of SMS (Pump 3)

### 6.1.7 Demands

Basic equipments of a WDN are pumps, tanks and reservoirs. These equipments are connected to each other with pipes, junctions and fittings. All pipes may be produced by different material, also may have different lengths or diameters depending on the different pressures and demands. For the hydraulic model of N8.3, these parameters were specified in previous studies.

All junctions have different elevations and pressure values. Therefore, for all pipe connections, different demand values are seen at junctions. Demand distribution may be applied as a fixed constant value for all junctions (for instance, fire flow). However, the demands which are distributed to the network with a certain rates differ from subzone to subzone and present the



basic behaviour of the system. This means that, by using a different multiplier for each subzone, demand is distributed properly.

Recorded demand values at every junction for every hour in a day present the flow time-dependent change through all pipes. This change gives daily demand curve (DDC) of that subzone approximately. However, for this study, real daily demand curve (DDC) data were obtained between the dates 07.09.2011 and 22.09.2011 for every 24 hours in a day. On this plots, x axis represents time as hour, and y axis represents demand as  $\text{m}^3/\text{hr}$ . The time period of these records, there is no an extraordinary situation in the network or an overuse because of an unexpected reason. Therefore, it can be said that, these daily demand curves (DDC's) presents an ordinary usage in N8.3 WDN.

For analysing water quality from the point of leakage amount, these demands are readjusted. For without leakage case these demands are decreased and redistributed. Contrary to this, for increased leakage case, demands are increased and redistributed to the system. When performing the most critical case, for summer season, demands are increased by multiplying them by 1.3 and for winter season they are decreased by multiplying them by 0.7. After this adjustment, system is operated with these new demand values in without leakage case.

Figure 6.6 shows DDC's of subzones in N8.3.

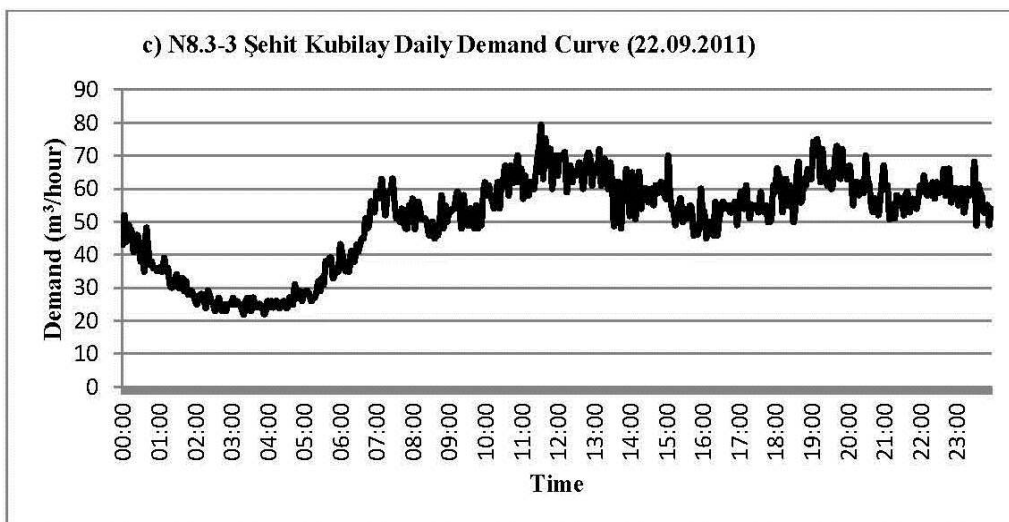
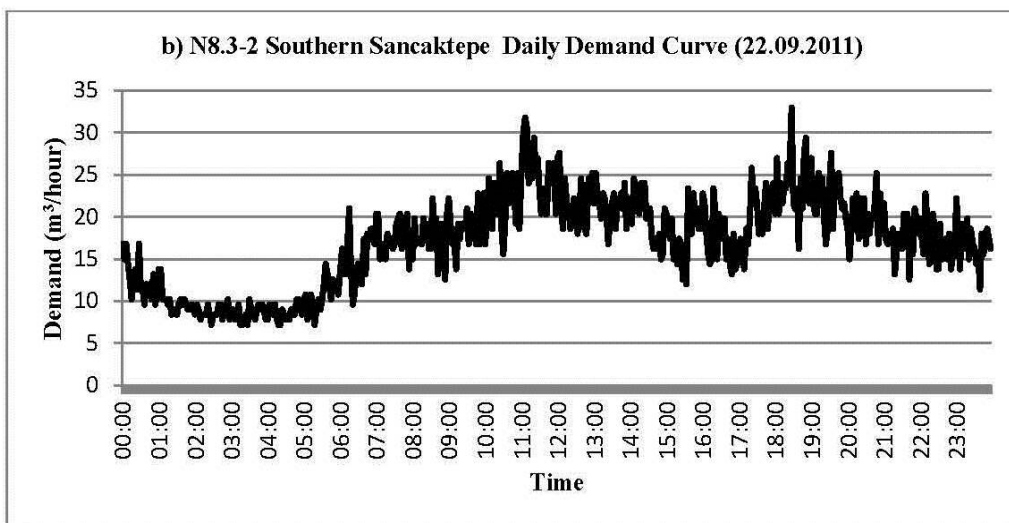
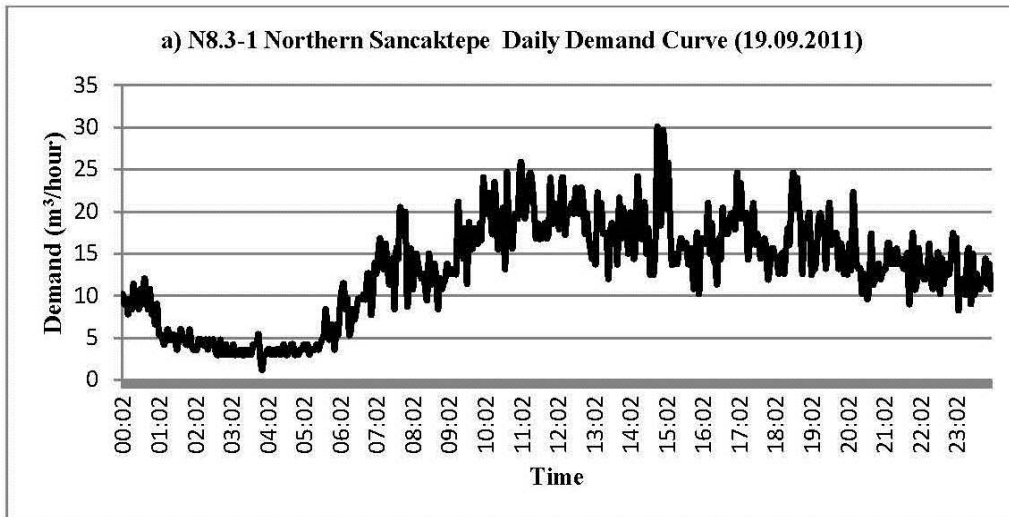


Figure 6.6 Demand Records of N8.3 Subzones

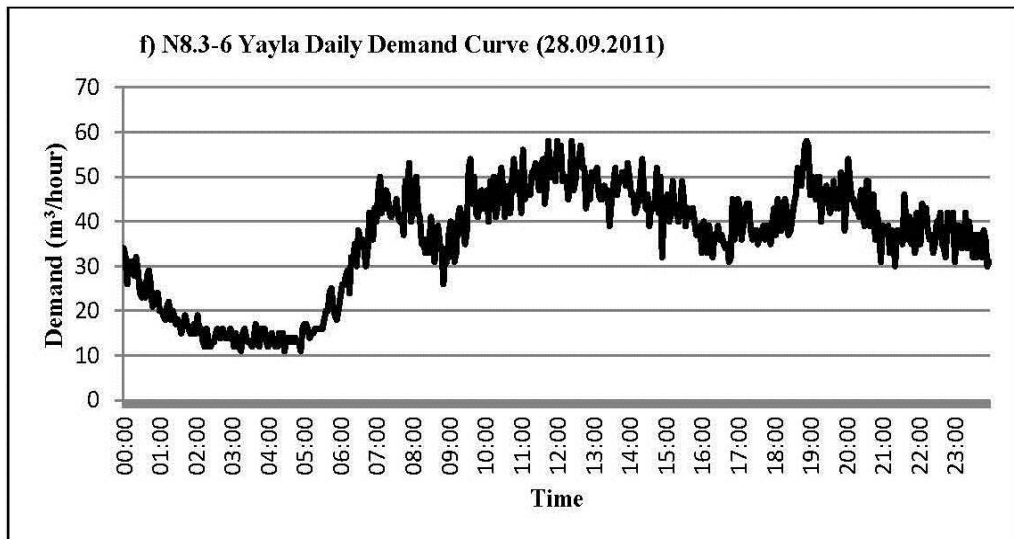
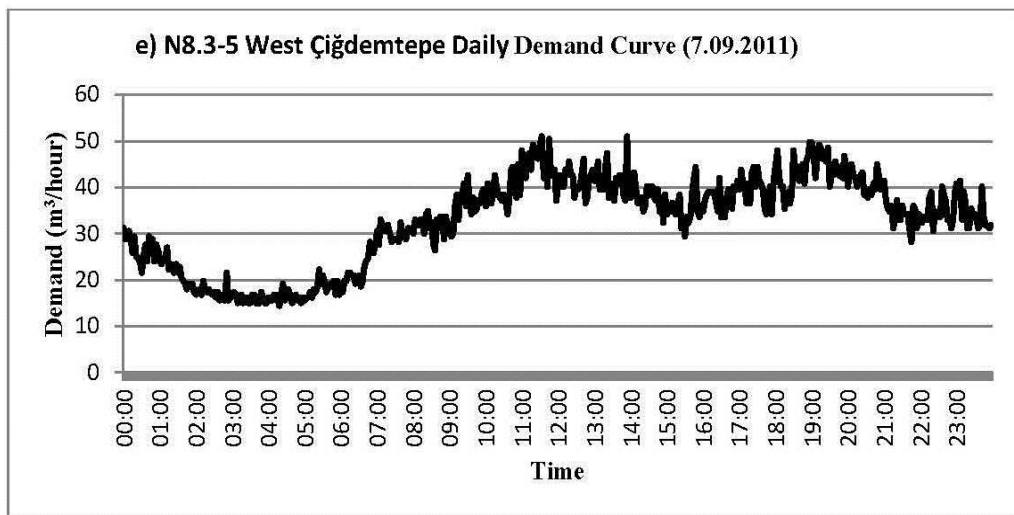
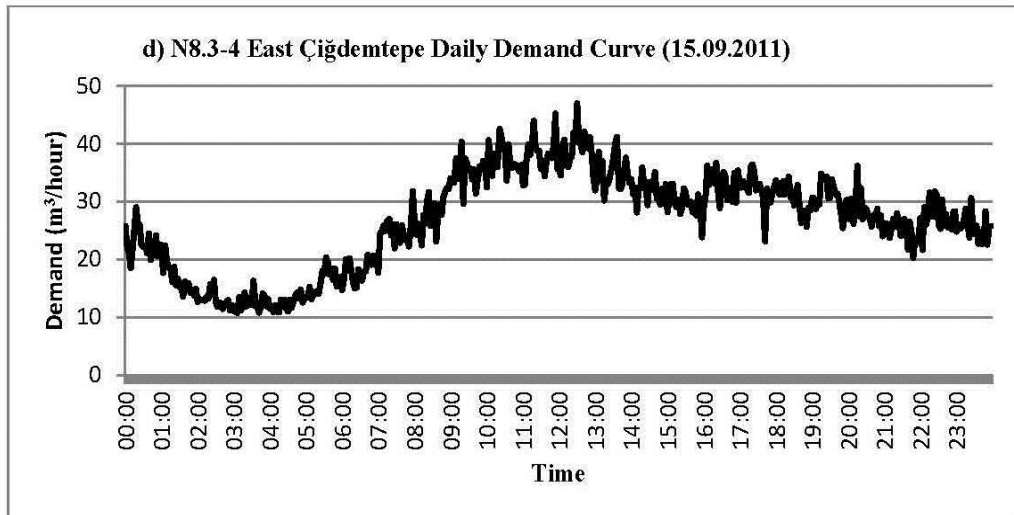


Figure 6.6 (Cont'd) Demand Records of N8.3 Subzones

For distributing demands to each junction, a program is used. The program is called Demand Distributor. The demand at each junction is calculated by multiplication of half of the total length of pipe with the demand. This means that demand at a junction is determined according to the half-length of pipes connected to that node. Every pipe has a start node and a stop node. The demand at both nodes is calculated by multiplication of half of the total length of pipe with demand per meter pipe value which was determined before. This means that demand at one node is determined according to the half-length of pipes connected to that node. A schematic of simple distribution network is shown in Figure 6.7.

$$D_x(\text{Demand per meter pipe}) = D_t / L_i \quad (6.1)$$

$$D_{N5}(\text{Demand at node 5}) = \left( \frac{L_4}{2} + \frac{L_5}{2} + \frac{L_6}{2} + \frac{L_7}{2} \right) \times D_x \quad (6.2)$$

where;  $D_t$  = Average demand measured at DMA inlet node.

$L_i$  = Pipe Length

$D_x$  = Demand per meter pipe

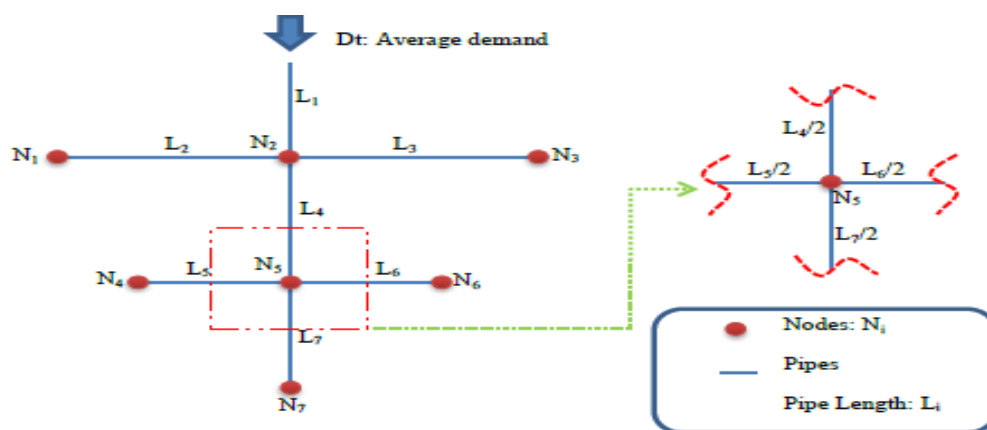


Figure 6.7 Distribution of Demands With Half-Pipe Length Method

### ***6.1.8 Studied Subzones***

In this study, as mentioned before, three subzones were studied. These are; (i) Yayla, (ii) North Sancaktepe and (iii) East Çiğdemtepe

Chlorine concentration of five junctions in a subzone were analysed. One of these junctions are at the entrance of the subzone, three of them in difference parts of the subzone and one of them is at the end point.

Chlorine concentrations change at these five junctions for all subzones will be showed in detail in next pages but it can be said that briefly; at entrance joints concentration differences are very small. However at end joints, this changes can be seen more specifically because of the loss through the pipes until water reaches to the end. Figure 6.8 shows the junctions which were studied in Yayla district.

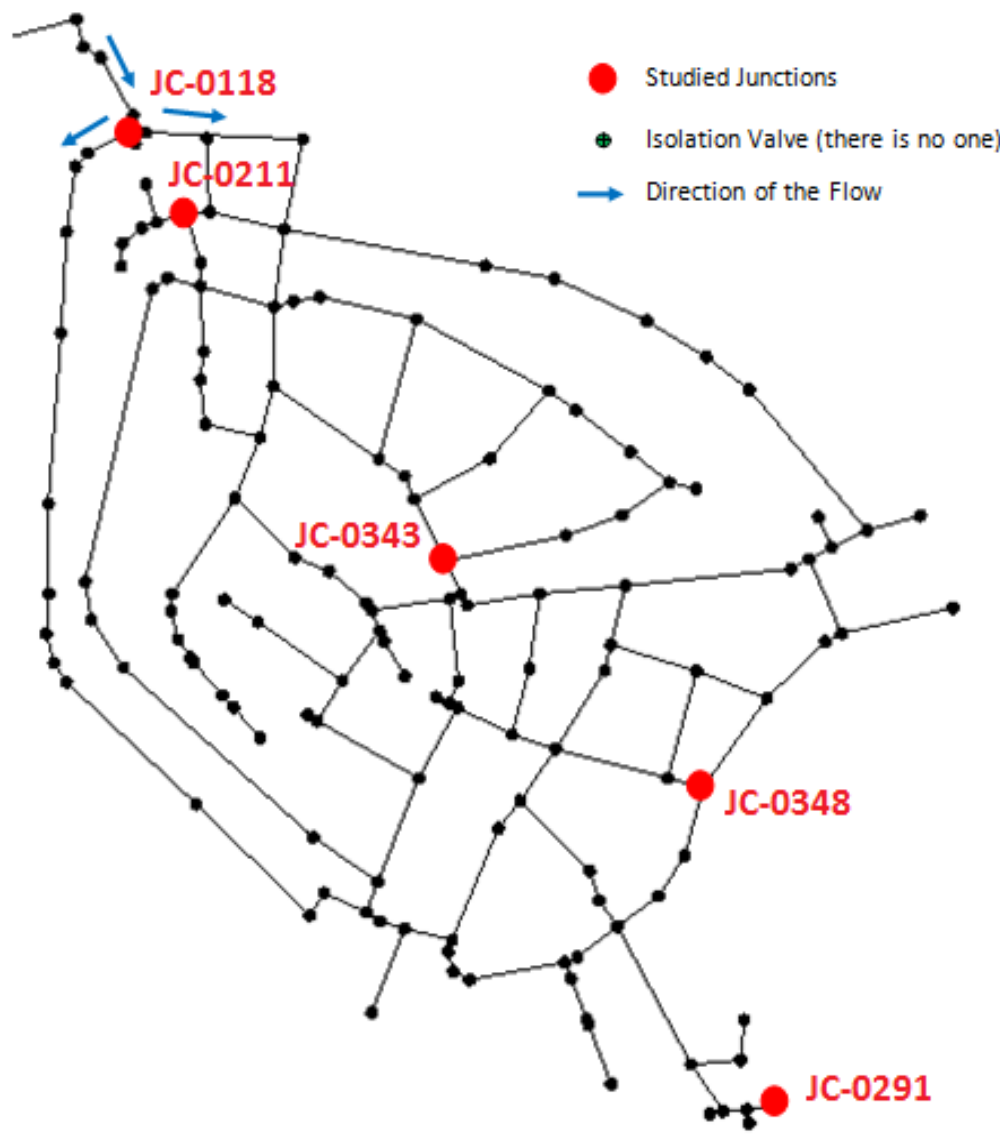


Figure 6.8 Junctions Studied in Yayla District

In Yayla district, JC-0118 was studied as entrance junction. JC-0211, JC-0343 and JC-0348 are the junctions which were studied in different middle parts of the district. JC-0291 was studied as an end point (dead end).

Figure 6.9 shows the junctions which were studied in North Sancaktepe district.

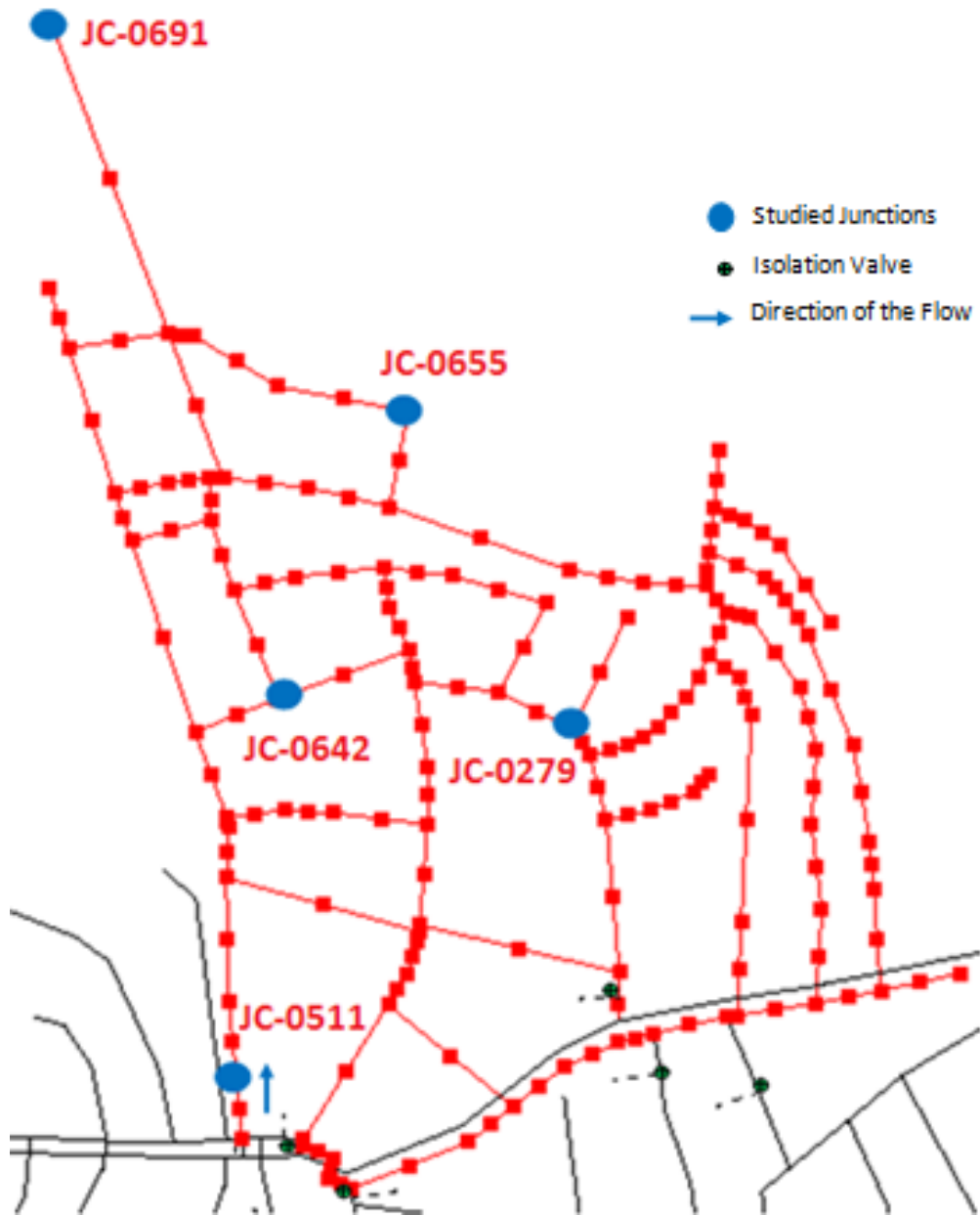


Figure 6.9 Junctions Studied in North Sancaktepe District (To show the area more clearly, district is marked as red.)

In North Sancaktepe district, JC-0511 was studied as an entrance junction. JC-0642, JC-0279 and JC-0655 are the junctions which were studied in different middle parts of the district. JC-0691 was studied as an end point (dead

end). Figure 6.10 shows the junctions which were studied in East Çiğdemtepe district.

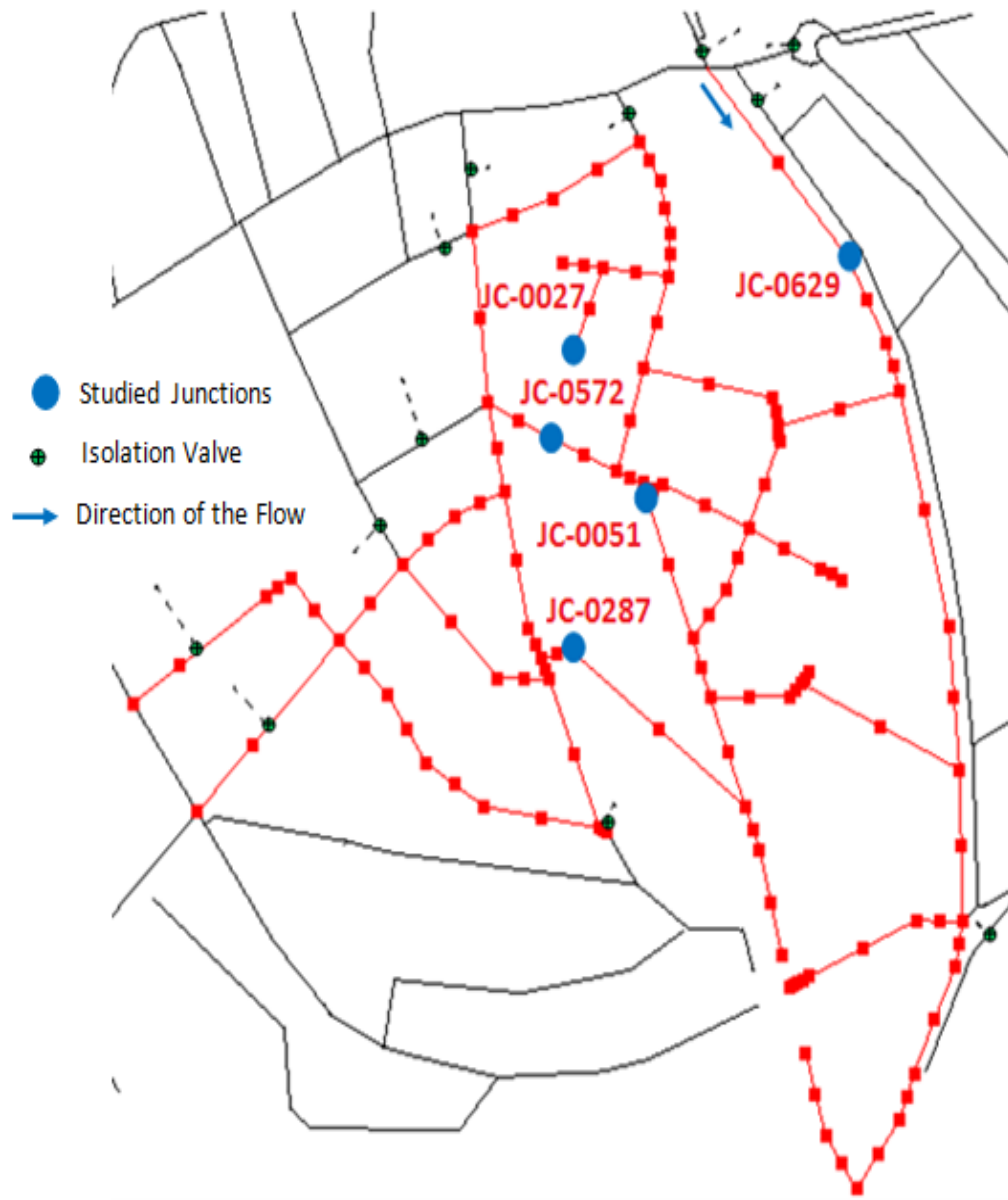


Figure 6.10 Junctions Studied in East Çiğdemtepe District (To show the area more clearly, district is marked as red.)



In East Çiğdemtepe, JC-0629 was studied as an entrance junction, JC-0051, JC-0287 and JC-0572 are the junctions which were studied in different middle parts of the district. JC-0027 was studied as an end point (dead end).

During this study, the junctions pointed in figures above and shown in Table 6.4 below were used to analyse influence of system parameters and leakage effect on chlorine concentration differences in N8.3 WDN.

Table 6.4 Junctions Studied in N8.3 Water Distribution Network

Pattern	Entrance	Middle	Middle	Middle	Dead End
Yayla	JC-0118	JC-0211	JC-0343	JC-0348	JC-0291
North Sancaktepe	JC-0511	JC-0642	JC-0279	JC-0655	JC-0691
East Çiğdemtepe	JC-0629	JC-0051	JC-0287	JC-0572	JC-0027

## 6.2 Water Quality Modeling of N8.3

As it is mentioned in the previous chapters, water quality modeling is affected by chemical and physical properties of water. EPS is applied for the modeling to effort time depending properties of water because of the transport and mixing behaviours. To balance the behaviour of a WDN system EPS study is used. Chlorine is the most commonly used chemical for water disinfection and concentration of it at all junctions are changed by differences in the system.

For this study, chlorine concentration in the pump station was selected as 0.50 mg/l and this concentration level was constant for the source of the system during the calculation time. Calculation was made for 384 hours (16 days) during this study and initial concentration levels of all junctions were selected as 0.20 mg/l. Therefore, it can be said that with a level of 0.5 mg/l chlorine concentration the system was feeded and a level of 0.20 mg/l chlorine concentration at junctions helps the system to reach a stable condition. This

selection is completely preferably. The 0.50 mg/l concentration level was taken from a previous study prepared by Bağcı (2001). In that study actual measurements at the pump station and along the network were present by the help of sampling method. According to these measurements, the average value of the concentrations was obtained as 0.50 mg/l and therefore, this value was taken as the entrance concentration.

Figure 6.11 shows the tank flow, tank level and chlorine concentration levels of the tank for 384 hours EPS study. Figure 6.12 shows the concentration level of a randomly selected junction during the same time period. As it is seen in these figures, reaching the stable condition takes a time period. This duration seems to be 144 hours approximately for chlorine concentration level. However, EPS study was applied for 384 hours (16 days) to provide more realistic and stable chlorine concentration versus time graphs.

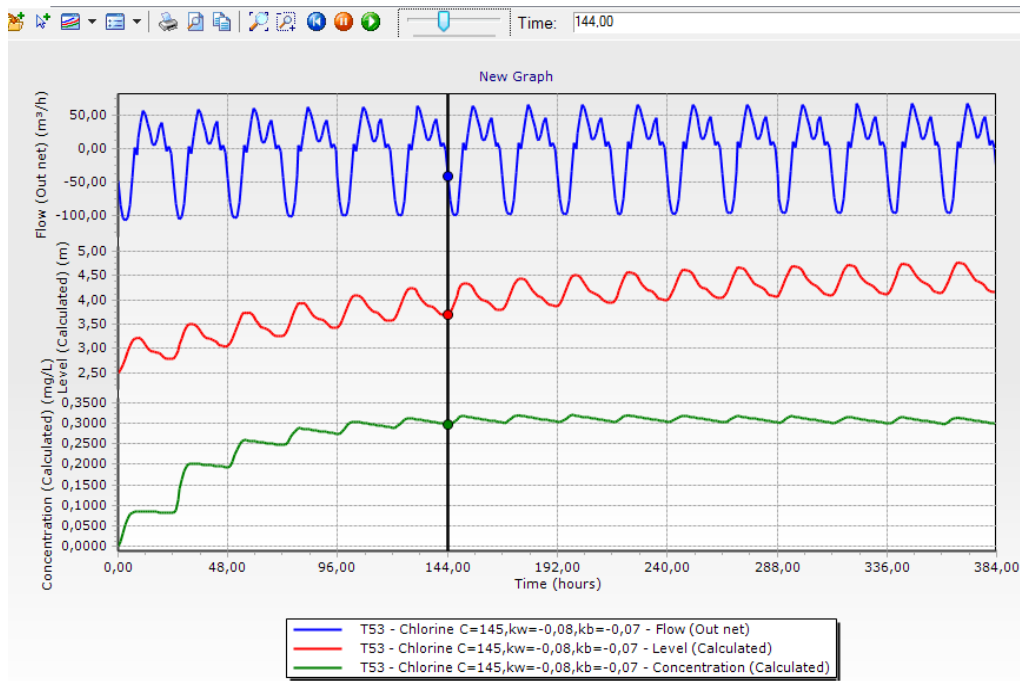


Figure 6.11 Tank Flow, Tank Level and Chlorine Concentrations In The Tank For 384 Hours EPS Study

As it is seen in the graph, in 11th hour the flow gives the peak value and it continues regularly. Initial concentration is 0.00 mg/l in the tank because of being empty at start and after approximately 144 hours, chlorine concentration level reaches a stable condition. Before the stable condition system gives fluctuations during initial hours. After it reaches regularity, approximately at a level of 0.300-0.325 mg/l chlorine concentration is seen during the 384 hours (16 days) time period. It can also be seen that, at peak hours of flow, chlorine concentration of the tank gives maximum values. It starts to decrease until flow gives minimum values. When flow reaches minimum value, chlorine concentration of the tank reaches minimum value and starts to increase with receiving fresh water again. Because of the reason that the pump is continuously operating, demands are satisfied generally from the pump station and tank never becomes empty. Therefore, although tank level starts with 2.50 m, it reaches to a higher level at the end of the study because of the accumulation.

384 hours (16 days) EPS study is performed with a chlorine concentration of 0.20 mg/l at junctions. Figure 6.12 shows that initial junction concentration level is 0.20 mg/l at JC-0437 and before it reaches the stable condition, it drops suddenly under allowed limit. In 14th hour, the concentration level of the junction reaches a stable condition and gives a regular period after that.

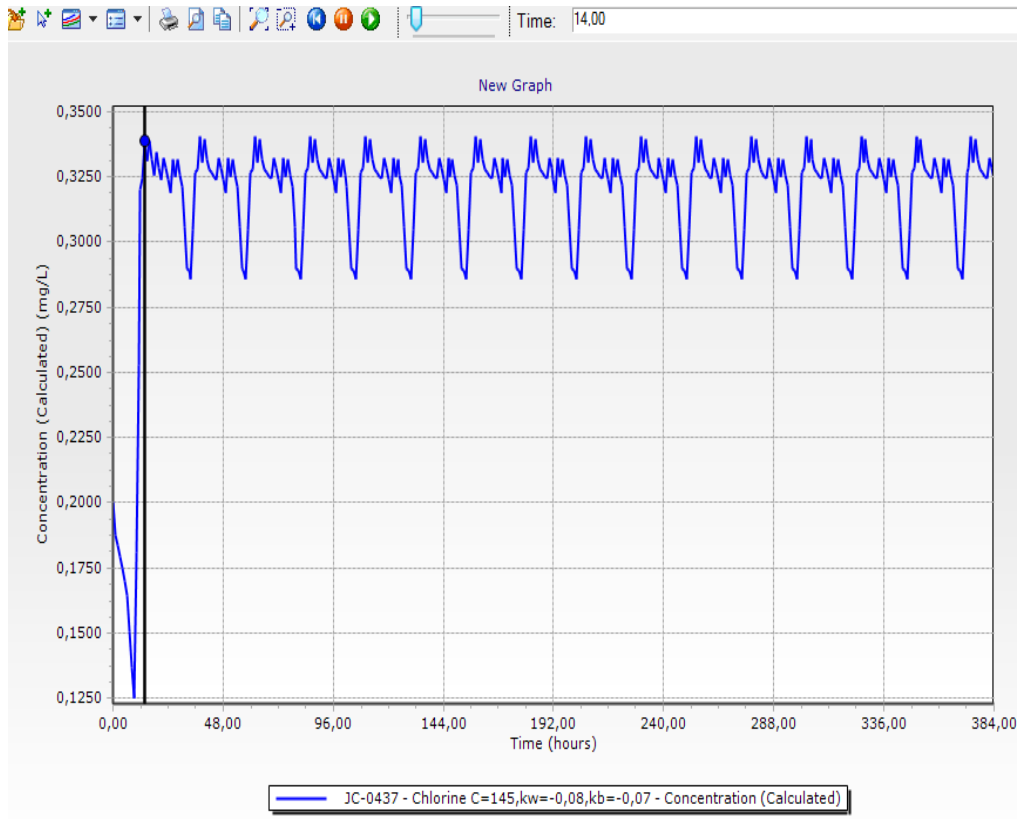


Figure 6.12 Chlorine Concentration Level of JC-0437 for 384 hours EPS Study

If there is no extra unexpected uses causing big demand differences, chlorine concentrations give a regular graph during all time period. Every 24-hours period gives same behaviour as it is seen in figure above.

### 6.3 Single Pump Operation (Continuous for 24 hours)

There are three pumps at the pump station of N8.3 WDN as mentioned before. Pump 1 and Pump 2 are manufactured by SUMAS and they have equal capacity but Pump 3 is manufactured by SMS (Samsun Makina Sanayi) and it has a larger capacity than Pump 1 and Pump 2.

During this study, a single pump operation is performed. While studying this pump operation, one of the three pumps which manufactured by SUMAS is

operating and the other two pumps at the station are set off. 1- SUMAS is in operation during all time, while the operating point of the pump is changing due to the fluctuations in daily demand. For 24 hours period, operation system is shown in the Figure 6.13 below. 1-SUMAS is operating, 2-SUMAS and 3-SMS do not operating.

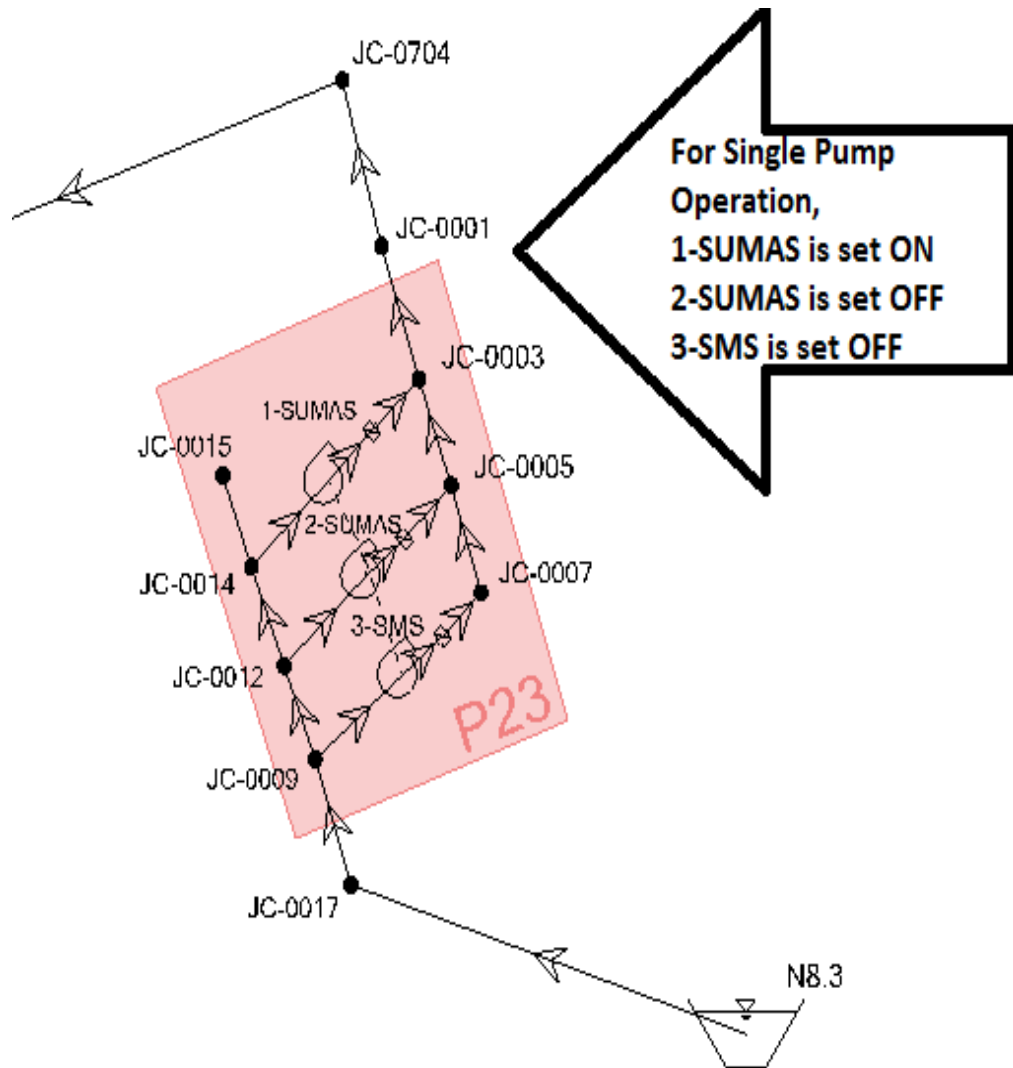


Figure 6.13 Pump On/Off Status of Single Pump Operation

EPS study starts with a 2.50 m tank level. Level of the tank fluctuates depending on the change of demands and pump operation of the system. Pump flow starts with a value of 185.67 m<sup>3</sup>/h. The Figure 6.14 shows the first 24 hours flow versus time graph of 1-SUMAS Pump with tank level in the same duration. This level reaches a stability after 336 hours (2 weeks). For this reason, Figure 6.15 shows last 48 hours tank levels and pump flows in the same graph as the stable condition.

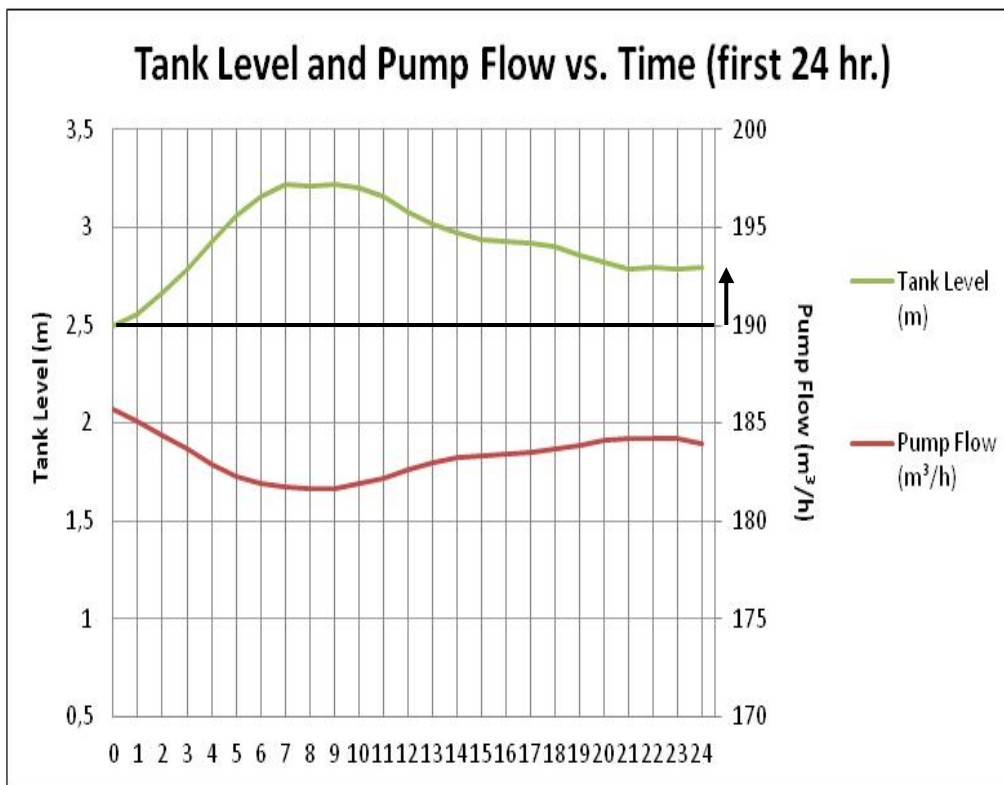


Figure 6.14 Tank Level and Pump Flow versus Time Graph For the First 24 Hours Study (Single Pump Operation)

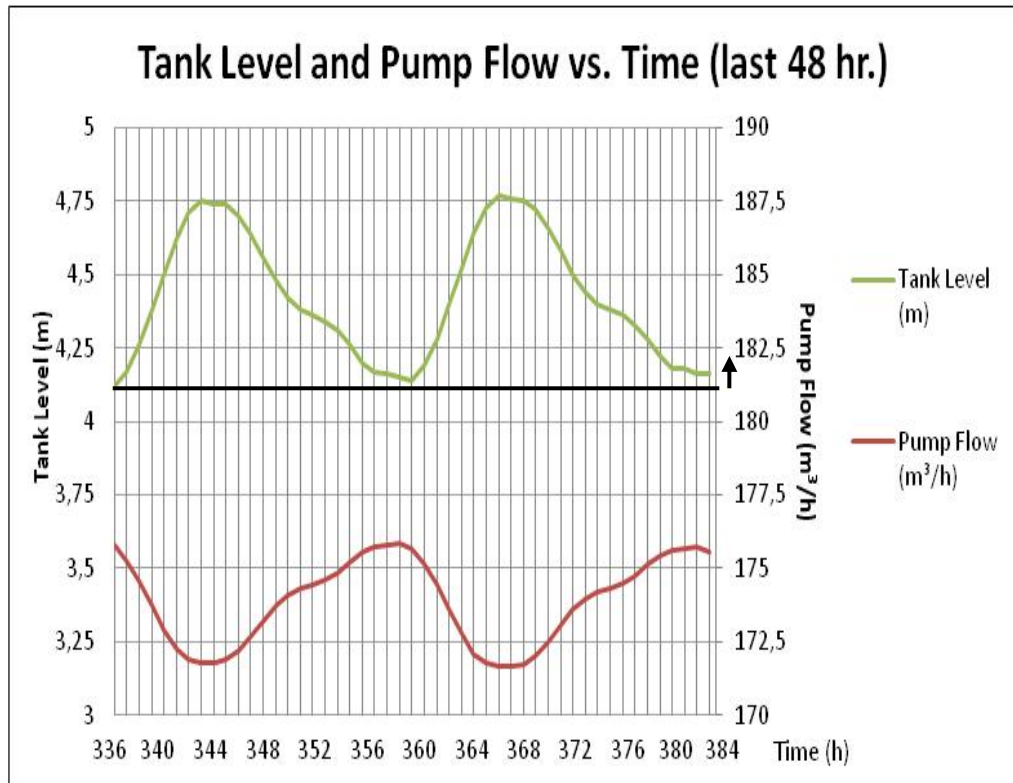


Figure 6.15 Tank Level and Pump Flow versus Time Graph for the Last 48 Hours Study (Single Pump Operation)

### 6.3.1 Yayla District

Yayla is the most distant subzone to the pump station and it has only one entrance for receiving water. The area has no boundaries with other districts which makes it completely insulated. Because of this reason, there is no isolation valves in Yayla. As a result of having only one entrance in Yayla district, insulated and uninsulated system conditions give almost same results, the system will perform similarly.

Hydraulic model of N8.3 Water Distribution System was performed for 384 hours (16 days) for this EPS study. Yayla has 140 junctions totally. As mentioned before, five junctions were chosen to analyse. One of them is near to the entrance of subzone; three of them are middle junctions in the different

parts of the subzone and one of them is at dead end points of the subzone. The junctions chosen for Yayla district is JC-0118 as an entrance junction; JC-0211, JC-0343, JC-0348 at middle points and JC-0291 at dead end point. These junctions were shown in Figure 6.8.

In this part, relation between chlorine concentration and time at chosen junctions are analysed for 18 different cases (for three different  $C_{HW}$ , three different  $k_w$  and two different  $k_b$  values). In all cases, the demands used in previous thesis studies were distributed (Şendil, 2013). After these demands were distributed, it is analysed that how chlorine concentrations are influenced by changing mentioned system parameters.

Figure 6.16a shows chlorine concentrations by using the same bulk decay coefficient  $k_b = 0.07$  1/day and the same wall decay coefficient  $k_w = 0.00$  m/day, assuming the system pipes are totally new. Only Hazen-Williams roughness coefficient  $C_{HW}$  is changed while  $k_w$  and  $k_b$  were constant. Figure 6.16b shows concentrations with  $k_b = 0.07$  1/day and  $k_w = 0.08$  m/day, again changing only roughness coefficient  $C_{HW}$ . Figure 6.16c shows them with  $k_b = 0.07$  1/day and  $k_w = 0.20$  m/day assuming the system has older pipes and changing only the roughness coefficient  $C_{HW}$  similarly in Figure 6.16a and 6.16b. In Figure 6.16d, Figure 6.16e and Figure 6.16f, concentration results of different combinations are shown for  $k_b = 0.14$  1/day this time. All graphs show the results of last 24 hours of the study.

Table 6.5 shows the summary of chlorine concentrations in Yayla district.



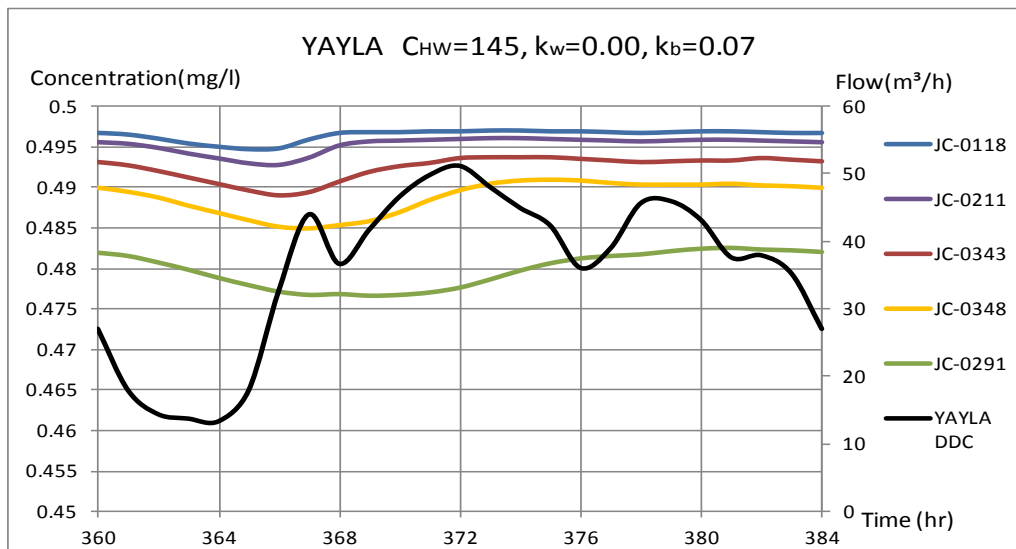
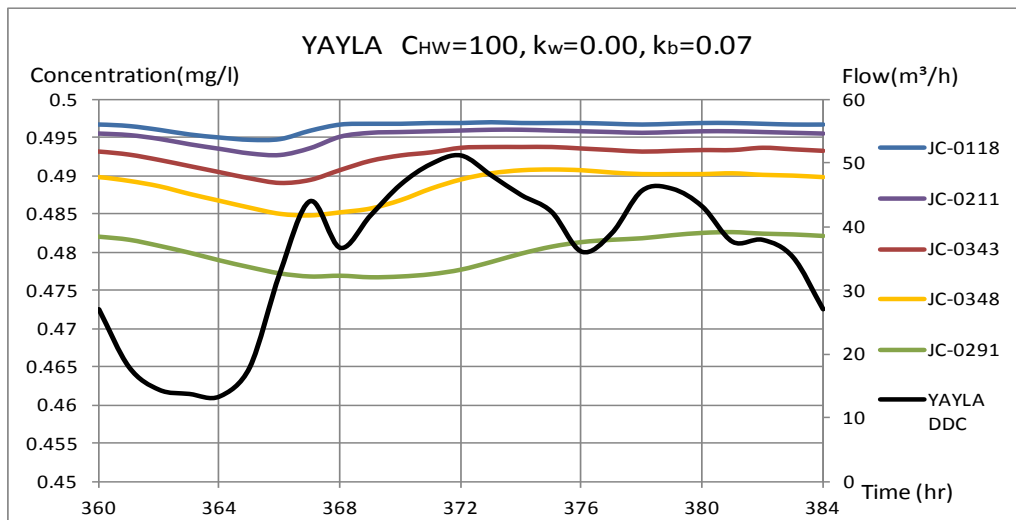
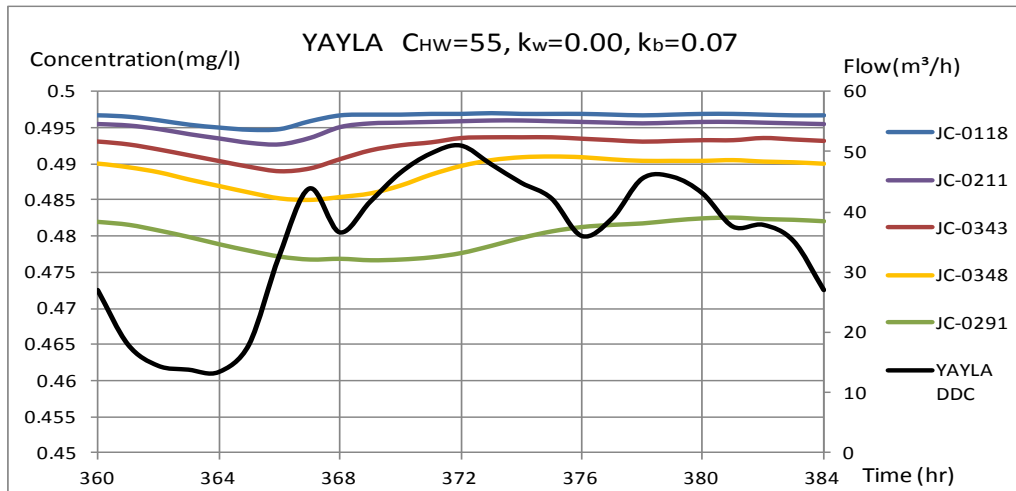


Figure 6.16a Concentration Levels for Different Cases in Yayla District

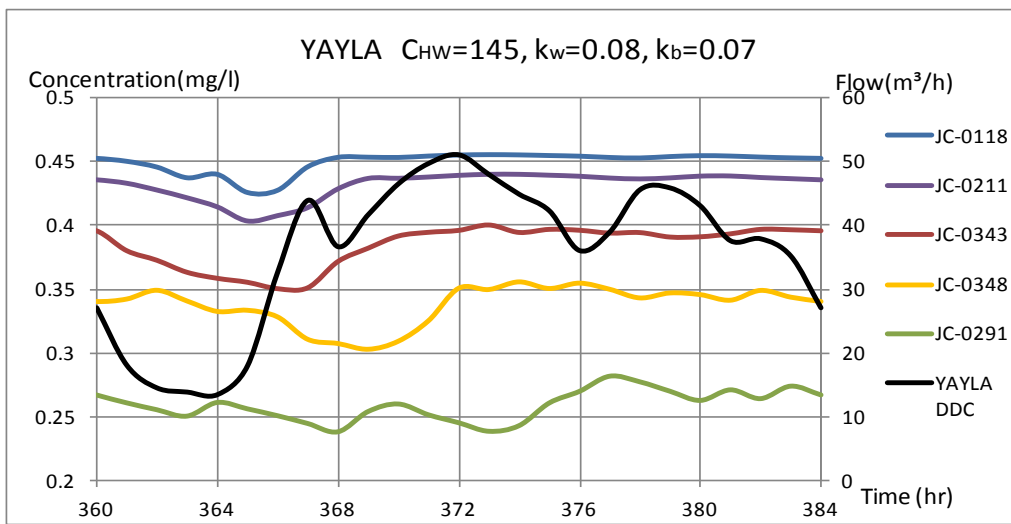
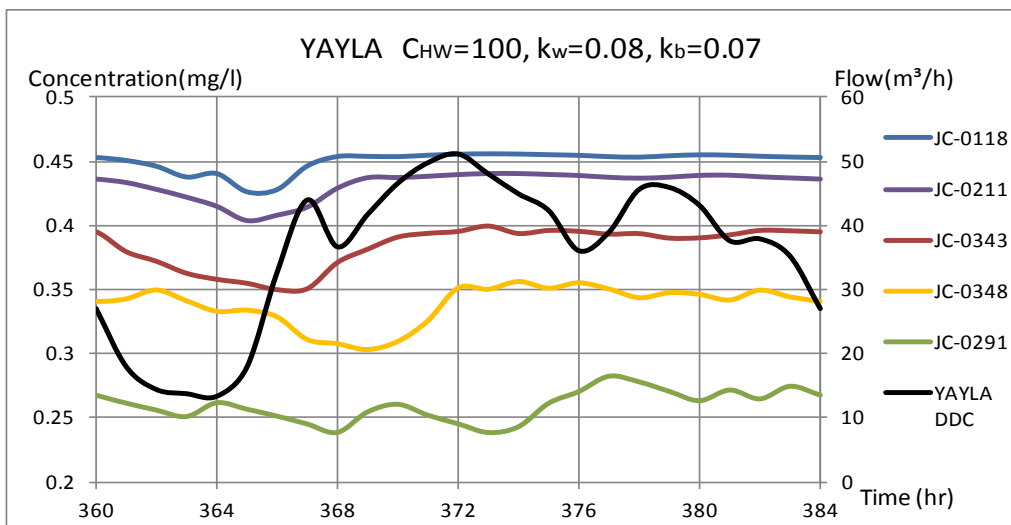
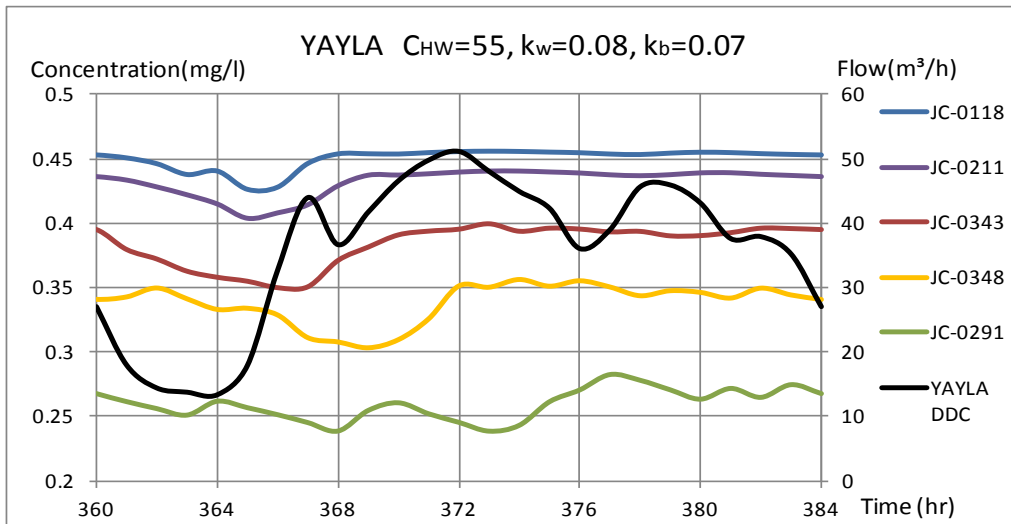


Figure 6.16b Concentration Levels for Different Cases in Yayla District

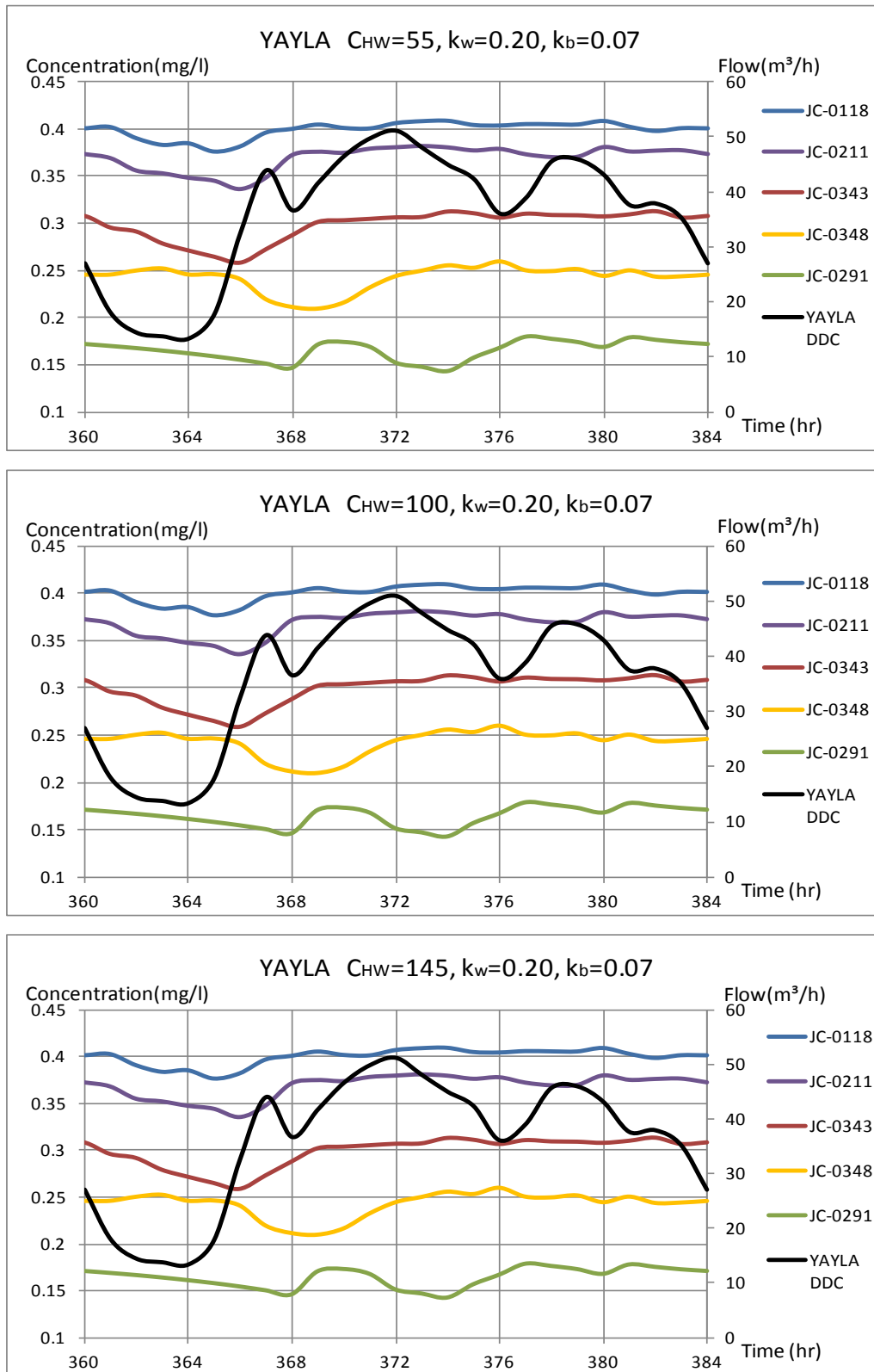


Figure 6.16c Concentration Levels for Different Cases in Yayla District

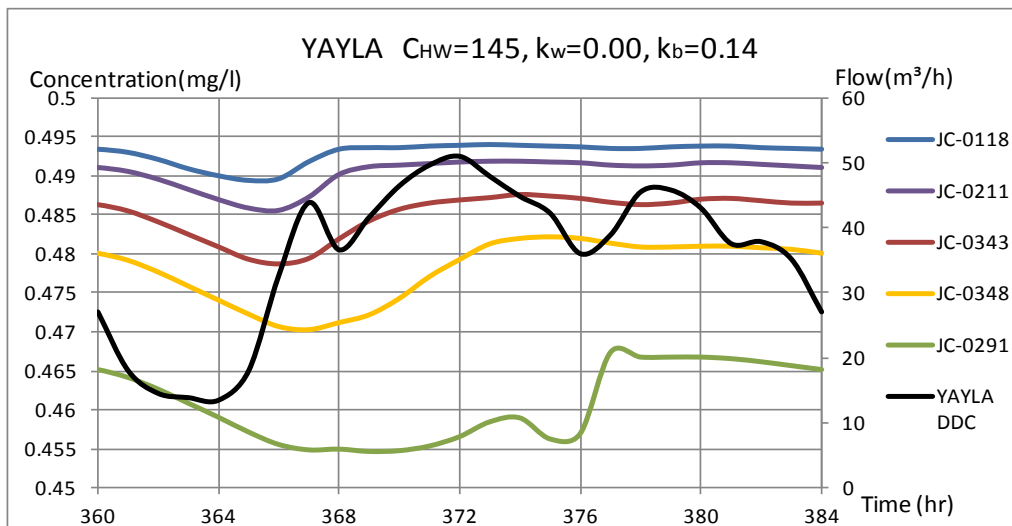
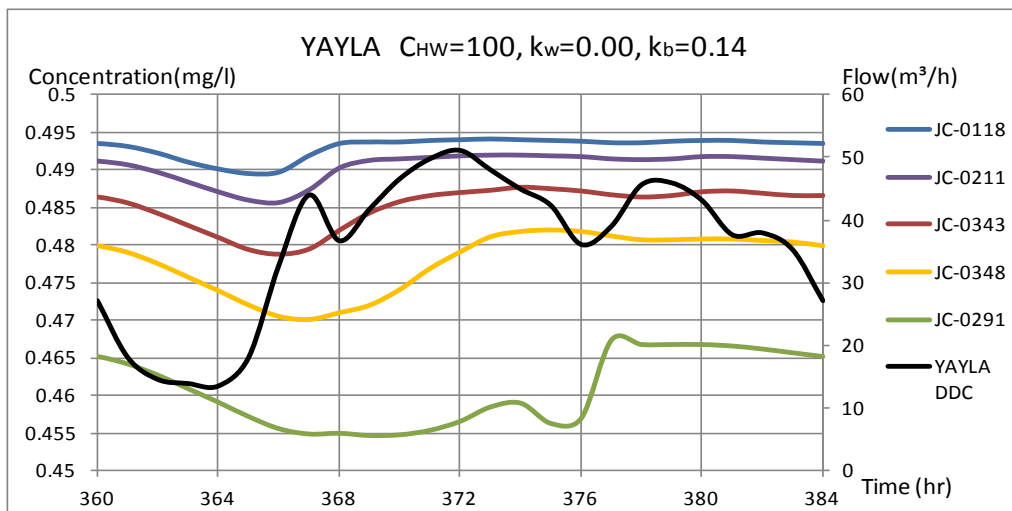
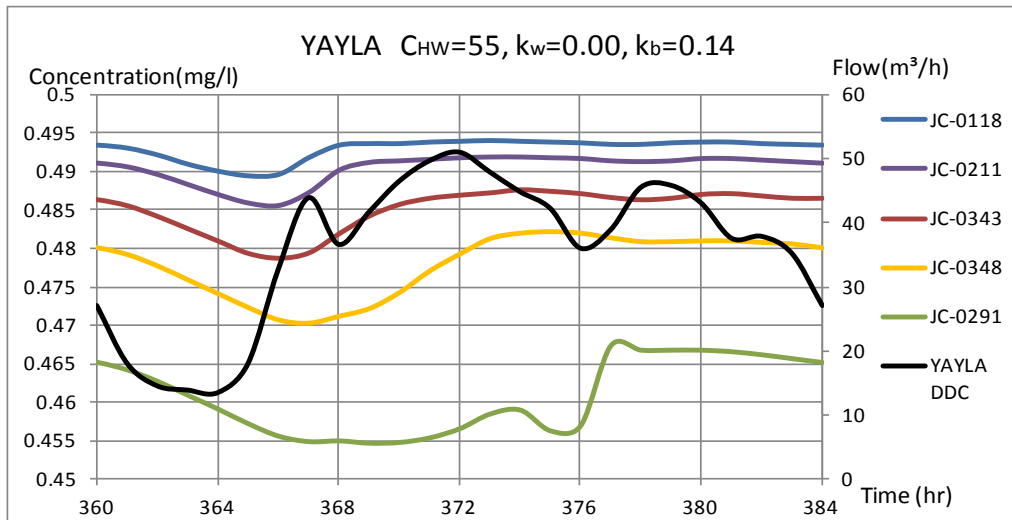


Figure 6.16d Concentration Levels for Different Cases in Yayla District

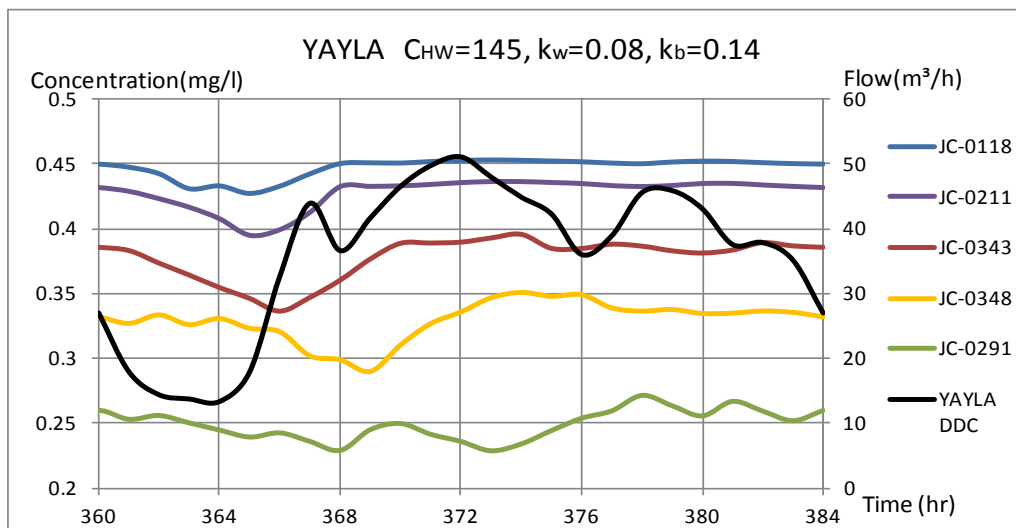
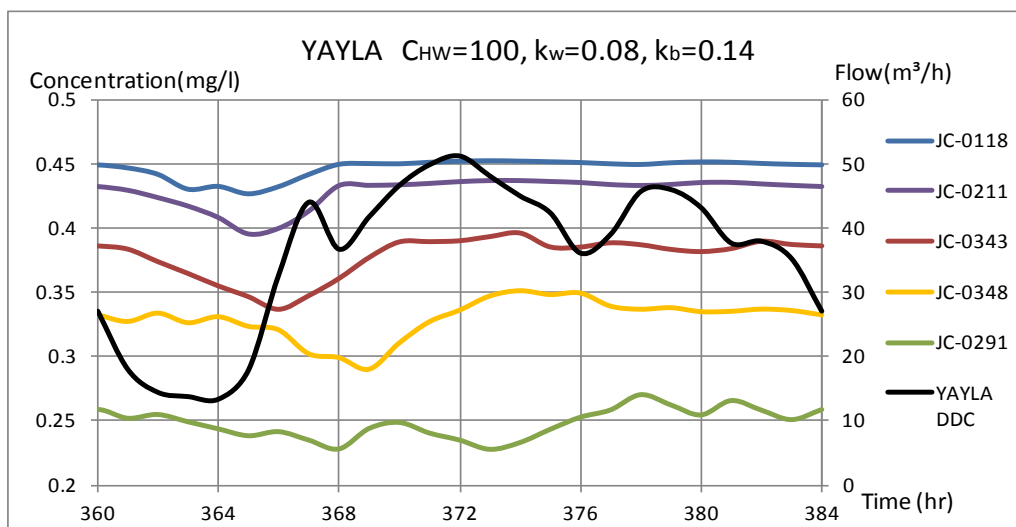
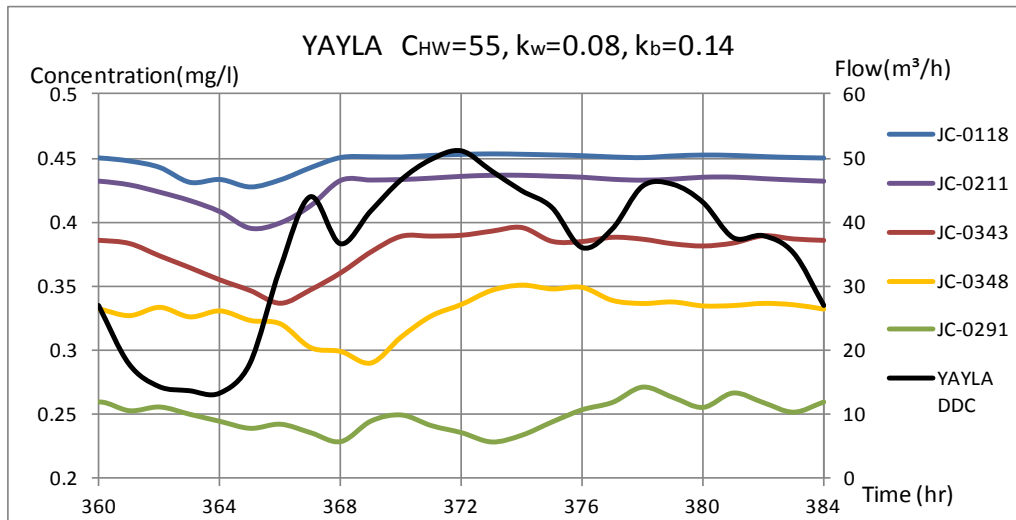


Figure 6.16e Concentration Levels for Different Cases in Yayla District

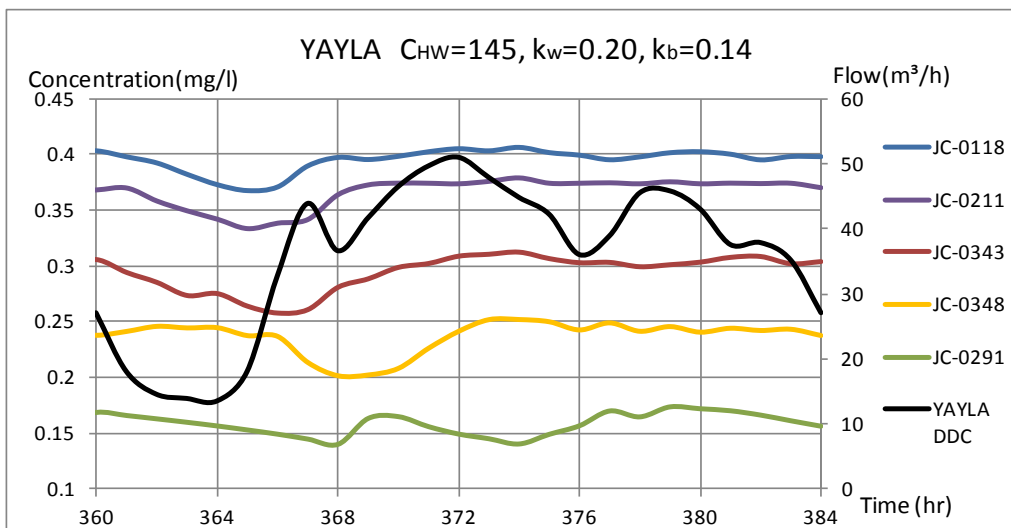
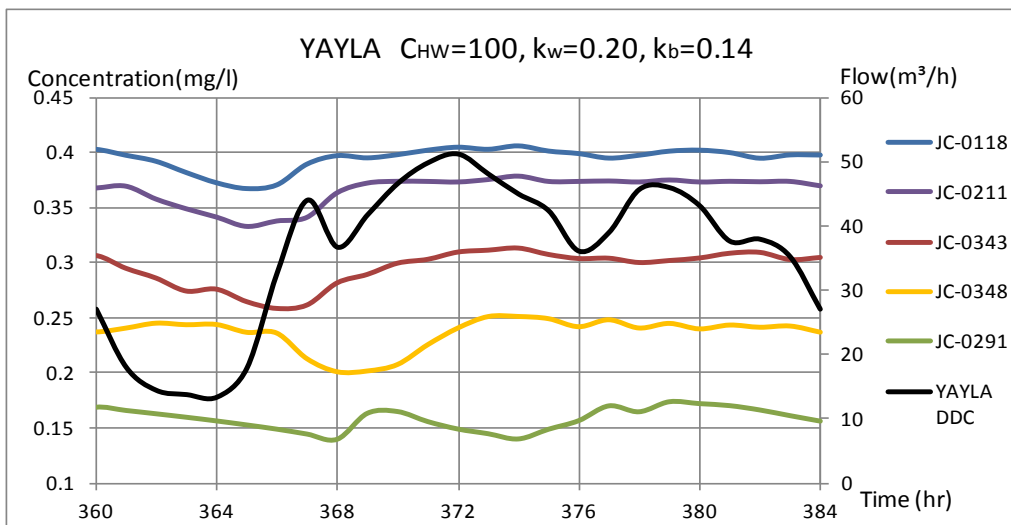
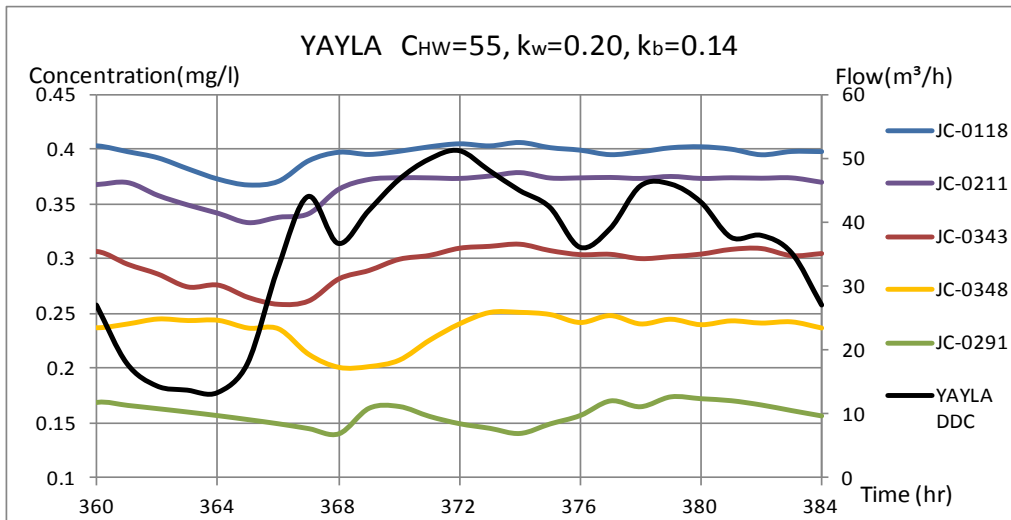


Figure 6.16f Concentration Levels for Different Cases in Yayla District

Table 6.5 Summary of Chlorine Concentrations at Studied Junctions in Yayla

Yayla		CONCENTRATIONS (mg/l)				
C <sub>HW</sub> =55		Entrance	Middle	Middle	Middle	Dead End
		JC-0118	JC-0211	JC-0343	JC-0348	JC-0291
k <sub>w</sub> =0.00 , k <sub>b</sub> =0.07	Maximum	0.49700	0.49600	0.49370	0.49090	0.48260
	Minimum	0.49470	0.49270	0.48900	0.48490	0.47670
	Average	0.49643	0.49517	0.49241	0.48873	0.47985
k <sub>w</sub> =0.08 , k <sub>b</sub> =0.07	Maximum	0.45540	0.44020	0.39980	0.35610	0.28190
	Minimum	0.42580	0.40360	0.35010	0.30320	0.23820
	Average	0.44954	0.43173	0.38326	0.33800	0.25878
k <sub>w</sub> =0.20 , k <sub>b</sub> =0.07	Maximum	0.40890	0.38180	0.31320	0.25970	0.17940
	Minimum	0.37620	0.33620	0.25850	0.20980	0.14300
	Average	0.39924	0.36889	0.29795	0.24220	0.16468
k <sub>w</sub> =0.00 , k <sub>b</sub> =0.14	Maximum	0.49400	0.49190	0.48760	0.48210	0.46730
	Minimum	0.48940	0.48560	0.47840	0.47020	0.45460
	Average	0.49289	0.49038	0.48493	0.47775	0.46046
k <sub>w</sub> =0.08 , k <sub>b</sub> =0.14	Maximum	0.45270	0.43660	0.39570	0.35110	0.27080
	Minimum	0.42690	0.39520	0.33650	0.29010	0.22810
	Average	0.44671	0.42781	0.37721	0.32962	0.24819
k <sub>w</sub> =0.20 , k <sub>b</sub> =0.14	Maximum	0.40650	0.37880	0.31270	0.25160	0.17350
	Minimum	0.36770	0.33120	0.25050	0.20090	0.13970
	Average	0.39596	0.36510	0.29317	0.23636	0.15784
C <sub>HW</sub> =100		Entrance	Middle	Middle	Middle	Dead End
		JC-0118	JC-0211	JC-0343	JC-0348	JC-0291
k <sub>w</sub> =0.00 , k <sub>b</sub> =0.07	Maximum	0.49700	0.49600	0.49370	0.49090	0.48260
	Minimum	0.49470	0.49270	0.48900	0.48490	0.47670
	Average	0.49643	0.49517	0.49241	0.48873	0.47985
k <sub>w</sub> =0.08 , k <sub>b</sub> =0.07	Maximum	0.45540	0.44020	0.39980	0.35610	0.28190
	Minimum	0.42580	0.40360	0.35010	0.30320	0.23820
	Average	0.44954	0.43173	0.38325	0.33800	0.25878
k <sub>w</sub> =0.20 , k <sub>b</sub> =0.07	Maximum	0.40890	0.38180	0.31320	0.25970	0.17940
	Minimum	0.37610	0.33620	0.25850	0.20980	0.14300
	Average	0.39922	0.36889	0.29795	0.24219	0.16467
k <sub>w</sub> =0.00 , k <sub>b</sub> =0.14	Maximum	0.49400	0.49190	0.48760	0.48210	0.46730
	Minimum	0.48940	0.48560	0.47840	0.47020	0.45460
	Average	0.49289	0.49038	0.48493	0.47775	0.46046
k <sub>w</sub> =0.08 , k <sub>b</sub> =0.14	Maximum	0.45270	0.43660	0.39570	0.35110	0.27080
	Minimum	0.42690	0.39520	0.33650	0.29010	0.22810
	Average	0.44671	0.42780	0.37720	0.32950	0.24820
k <sub>w</sub> =0.20 , k <sub>b</sub> =0.14	Maximum	0.40650	0.37880	0.31270	0.25160	0.17350
	Minimum	0.36770	0.33120	0.25050	0.20090	0.13970
	Average	0.39595	0.36510	0.29317	0.23636	0.15784
C <sub>HW</sub> =145		Entrance	Middle	Middle	Middle	Dead End
		JC-0118	JC-0211	JC-0343	JC-0348	JC-0291
k <sub>w</sub> =0.00 , k <sub>b</sub> =0.07	Maximum	0.49700	0.49600	0.49370	0.49090	0.48260
	Minimum	0.49470	0.49270	0.48900	0.48490	0.47670
	Average	0.49644	0.49517	0.49241	0.48873	0.47985
k <sub>w</sub> =0.08 , k <sub>b</sub> =0.07	Maximum	0.45540	0.44020	0.39980	0.35610	0.28190
	Minimum	0.42580	0.40360	0.35010	0.30320	0.23830
	Average	0.44954	0.43173	0.38325	0.33800	0.25883
k <sub>w</sub> =0.20 , k <sub>b</sub> =0.07	Maximum	0.40890	0.38180	0.31320	0.25970	0.17940
	Minimum	0.37610	0.33610	0.25850	0.20980	0.14300
	Average	0.39922	0.36888	0.29795	0.24219	0.16467
k <sub>w</sub> =0.00 , k <sub>b</sub> =0.14	Maximum	0.49400	0.49190	0.48760	0.48210	0.46730
	Minimum	0.48940	0.48560	0.47840	0.47020	0.45460
	Average	0.49289	0.49038	0.48493	0.47775	0.46046
k <sub>w</sub> =0.08 , k <sub>b</sub> =0.14	Maximum	0.45270	0.43660	0.39570	0.35110	0.27080
	Minimum	0.42690	0.39520	0.33650	0.29010	0.22810
	Average	0.44671	0.42780	0.37720	0.32962	0.24819
k <sub>w</sub> =0.20 , k <sub>b</sub> =0.14	Maximum	0.40650	0.37880	0.31270	0.25160	0.17350
	Minimum	0.36770	0.33120	0.25050	0.20090	0.13970
	Average	0.39595	0.36510	0.29317	0.23636	0.15784

Both figures and summary table show the Yayla chlorine concentration results of the last 24 hours and the system is operating with single pump operation. Last 24 hours give more realistic results because of the fact that the system operation does not reach to a stable condition in first six days. After these six days, system continues operation one more week for taking valid results and therefore results of 16th day are analysed. Also, DDC of the subzone is shown with black curve in graphs.

There are three important point to be considered while analysing the figures from 6.16a to 6.16f. Firstly, it can clearly be seen that if a node is far away from the entrance, chlorine concentrations become smaller. Blue curves show the results of entrance junction JC-0118 and green curves show the results of dead end junction JC-0291. At entrance junction, because of the reason that fresh water comes from the pump station continuously, chlorine concentrations are at maximum levels. While analysing the results from entrance to dead ends, travel time until water reaches to a junction affects chlorine concentrations and they give minimum levels at dead ends comparing to other studied junctions. Also, because of locating near to the entrance, chlorine concentration graph of the junction JC-0118 is almost a straight line during the day except peak hours. Because, fresh water reaches to entrance junction firstly and causes lower chlorine decay.

Another point to be considered is that, time of the minimum concentrations are shifting from early morning hours (06.00 am.) at entrance junction, to later morning hours (09.00-12.00 am.) at dead end junctions. When demands increase in morning hours, dead end junctions are the latest influenced junctions again because of the long way of water movement (travel time).

By analysing the Figure 6.16a ( $k_b = 0.07$  1/day and  $k_w = 0.00$  m/day), Figure 6.16b ( $k_b = 0.07$  1/day and  $k_w = 0.08$  m/day), Figure 6.16c ( $k_b = 0.07$  1/day and  $k_w = 0.20$  m/day), Figure 6.16d ( $k_b = 0.14$  1/day and  $k_w = 0.00$  m/day),



Figure 6.16e ( $k_b = 0.14$  1/day and  $k_w = 0.80$  m/day) and Figure 6.16f ( $k_b = 0.14$  1/day and  $k_w = 0.20$  m/day); it can be said that for a fixed  $k_b$  and  $k_w$  combination, changing the roughness coefficient  $C_{HW}$  is not effective on chlorine concentrations. Because Yayla district takes water only from the pump station and changing the roughness coefficient does not affect pump operation or pumping behaviour. Examining the summary of chlorine concentrations in Yayla from Table 6.5, they do not change even in 5th digit by changing  $C_{HW}$  coefficient.

In graphs, DDC of Yayla district is shown with black curve. It can also be clearly seen that, while demands starts to increase in morning hours, chlorine concentrations start to decrease because of the high rate of decay. This demand effect is seen later at dead end junctions comparing to entrance or middle junctions because of the travel time.

In Figure 6.17a, 6.17b, 6.17c, 6.17d and 6.17e show how chlorine concentrations are influenced by changing system parameters Hazen-Williams roughness coefficient  $C_{HW}$ , bulk decay coefficient  $k_b$  and wall decay coefficient  $k_w$  at every studied junction one by one. By analysing this type of graphs, it can be understood more easily that which parameter is the most effective one.

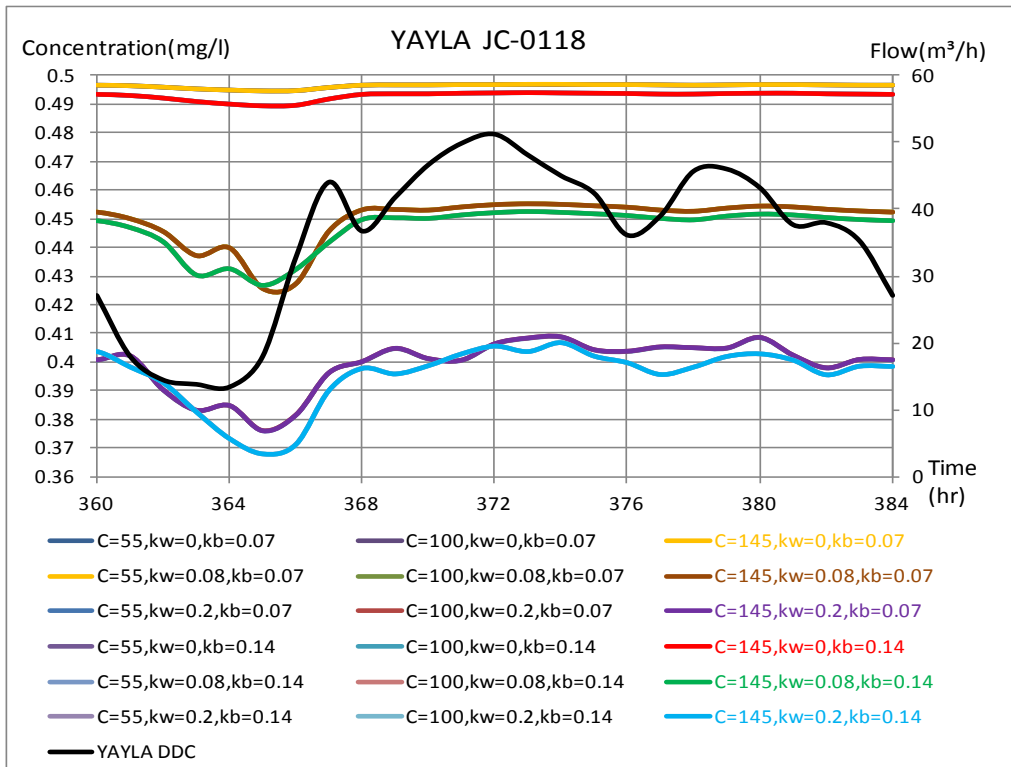


Figure 6.17a Concentration Levels for Different Cases at JC-0118

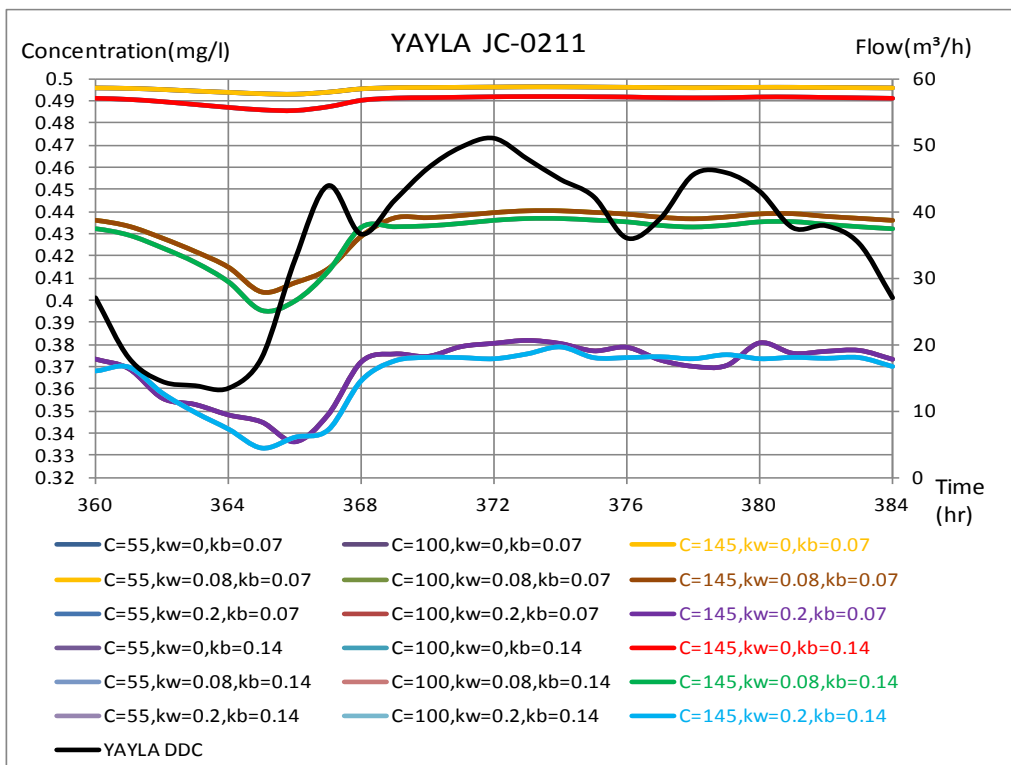


Figure 6.17b Concentration Levels for Different Cases at JC-0211

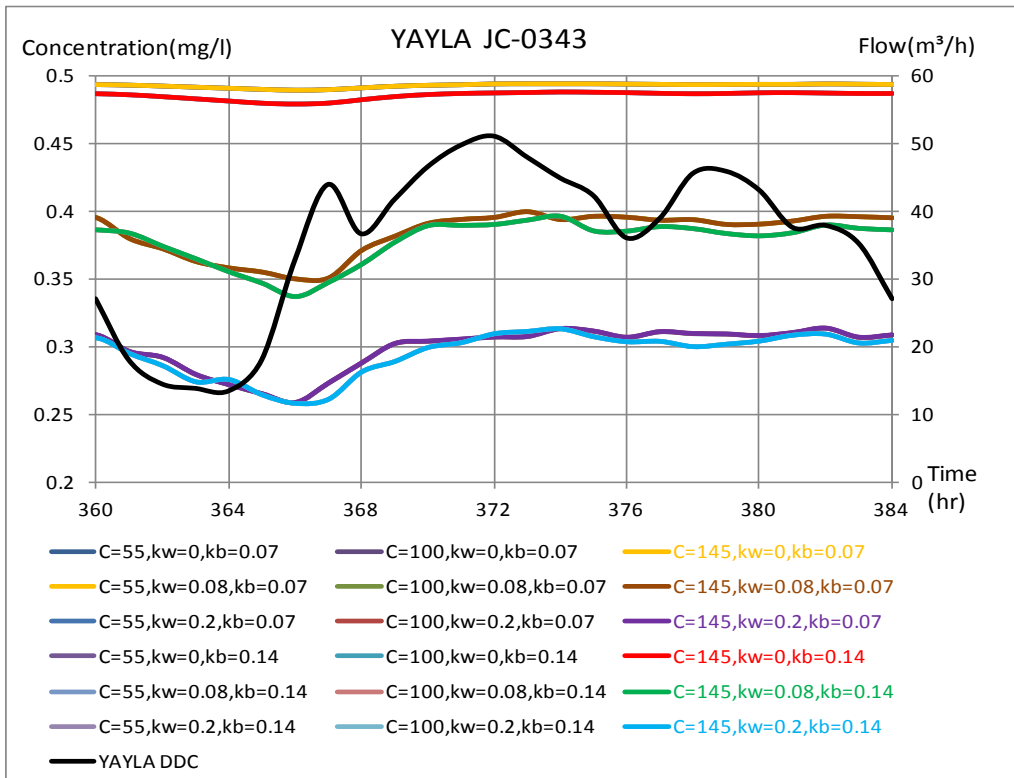


Figure 6.17c Concentration Levels for Different Cases at JC-0343

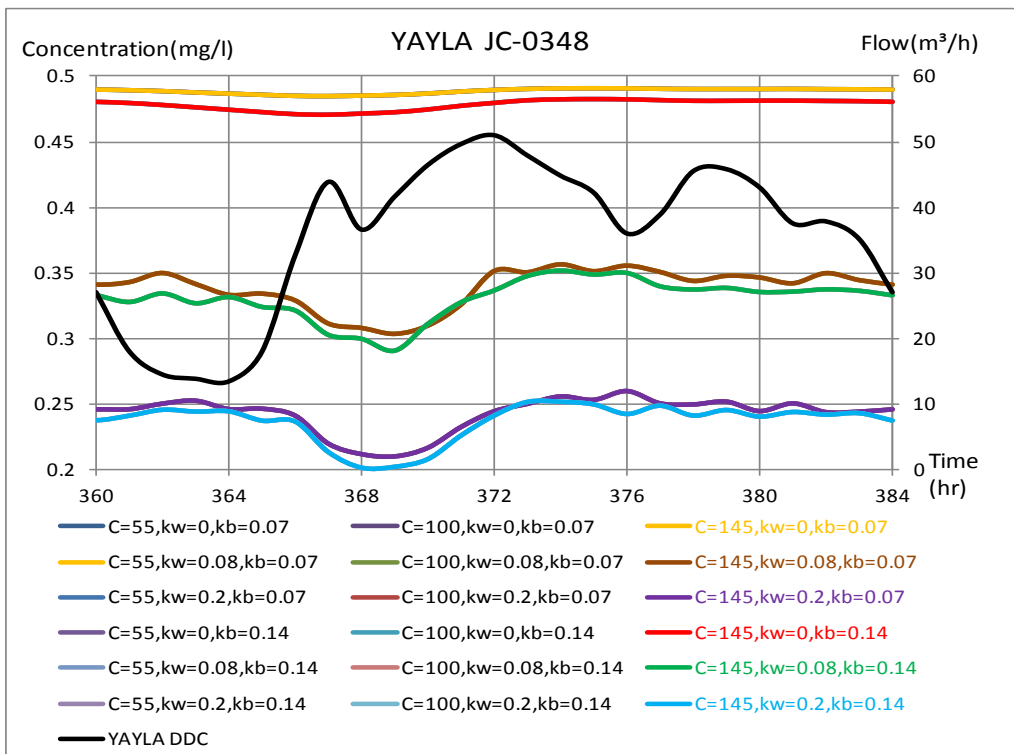


Figure 6.17d Concentration Levels for Different Cases at JC-0348

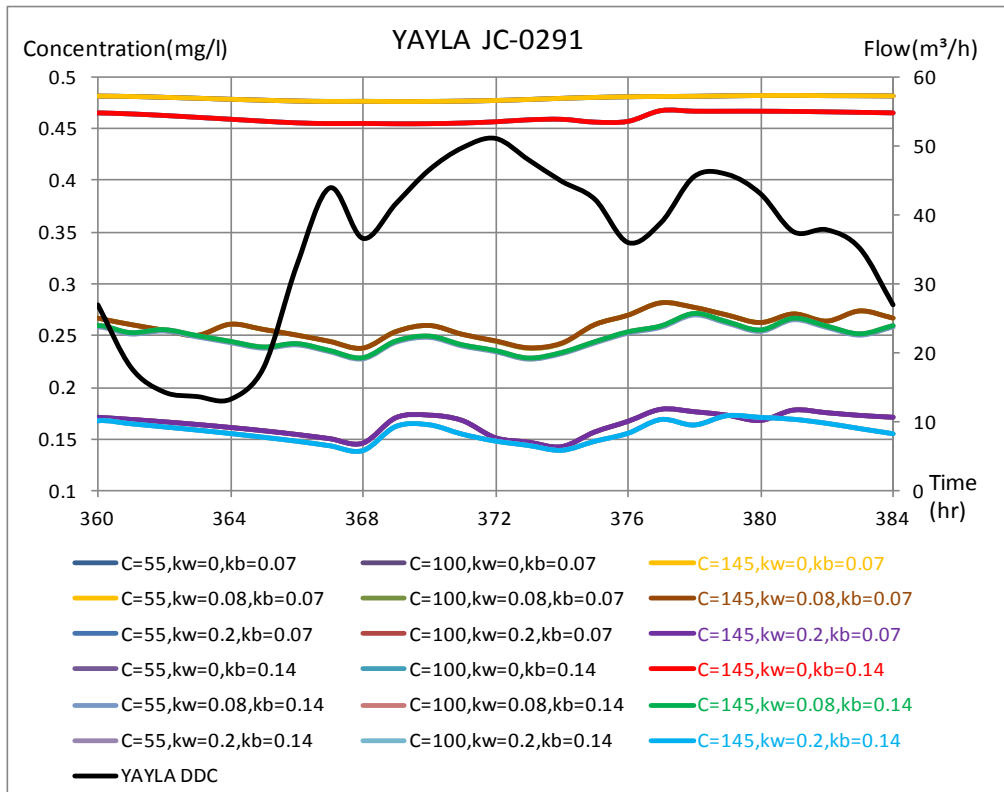


Figure 6.17e Concentration Levels for Different Cases at JC-0291

As it is seen in the figures above, six different curves can be seen and these curves are the results of six scenarios which are written with coloured text. Because of the reason that changing Hazen-Williams roughness coefficient  $C_{HW}$  does not affect chlorine concentrations in Yayla district, results of first twelve scenarios can not be seen, the curves are overlapped. Only  $k_w$  and  $k_b$  differences can be seen clearly. This can be explained by receiving water only from the pump station and not taking water from tank during the procedure while single pump operation is performed 24 hours continuously. Chlorine concentrations are not affected by tank fluctuations and tank concentration. Therefore, velocity does not change and concentrations do not give a difference by changing  $C_{HW}$  coefficients.

All graphs show the results by using Hazen Williams roughness coefficient  $C_{HW}$  as 145. It is clearly seen that, for this specific Hazen-Williams roughness coefficient  $C_{HW}$ ; red and yellow curves show results for  $k_w = 0.00$  m/day, green and brown curves show results for  $k_w = 0.08$  m/day, blue and purple curves show results for  $k_w = 0.20$  m/day. Also, DDC of the subzone is shown with black curve in graphs. As seen in figures, increasing the  $k_w$  value from 0.00 m/day, to 0.080 m/day or to 0.20 m/day decreases chlorine concentrations. Especially at dead end junction JC-0291, concentration levels decrease near to minimum safe limit 0.10 mg/l. For a specific  $k_w$  value changing bulk decay coefficient  $k_b$  is also effective. For example, by using  $k_w$  value as 0.00 m/day, yellow curve shows results for  $k_b = 0.07$  1/day and red curve shows results for  $k_b = 0.14$  1/day. It is clearly seen that, increasing the  $k_b$  value from 0.07 1/day to 0.14 1/day, decreases chlorine concentrations.

As mentioned before in Chapter 4, wall decay coefficient  $k_w$  expresses wall reactions inside the pipe. Due to seeing that while  $k_w$  is increasing, chlorine concentrations are decreasing; it can be said that using older pipes in a WDN causes lower disinfection concentration because of corrosion or tuberculation effects inside the pipes and this affects water quality negatively. Again, as mentioned before in Chapter 4, bulk decay coefficient  $k_b$  expresses the reactions in bulk fluid inside the pipe. It is influenced by the seasonal differences in temperature. Increment in the  $k_b$  value decreases chlorine concentrations when distributing the same demands. It can be clearly seen that, increasing both  $k_w$  and  $k_b$  values give decreasing chlorine concentrations, however changing  $k_w$  is more effective on chlorine concentrations.

### ***6.3.2 North Sancaktepe District***

North Sancaktepe district receives water from the main line passing in the south of the district. It has boundaries with South Sancaktepe district so as a difference from Yayla district, it is expected to see different behaviours and chlorine concentration values for whole network and DMA's cases because of

mixing water and interactions between these subzones. North Sancaktepe has 92 junctions totally.

One of the chosen junctions is near to entrance of district; three of them are at different middle parts and one of them is at a dead end point of district again. The junctions chosen for North Sancaktepe district is JC-0511 at the entrance; JC-0642, JC-0279, JC-0655 at middle points and JC-0691 at dead end point. These junctions were shown in Figure 6.9.

384 hours (16 days) for this EPS study was performed with 18 different cases (for three different  $C_{HW}$ , three different  $k_w$  and two different  $k_b$  values) again and chlorine concentration differences were analysed. All studies are performed with Single Pump Operation (1-SUMAS is set on) continuously for 24 hours.

Figure 6.18a shows chlorine concentrations by using the same bulk decay coefficient  $k_b = 0.07$  1/day and the same wall decay coefficient  $k_w = 0.00$  m/day, assuming the system pipes are totally new. Only Hazen-Williams roughness coefficient  $C_{HW}$  is changed while  $k_w$  and  $k_b$  were constant. Figure 6.18b shows concentrations with  $k_b = 0.07$  1/day and  $k_w = 0.08$  m/day, again changing only roughness coefficient  $C_{HW}$ . Figure 6.18c shows them with  $k_b = 0.07$  1/day and  $k_w = 0.20$  m/day assuming the system has older pipes and changing only the roughness coefficient  $C_{HW}$  similarly in Figure 6.18a and 6.18b. In Figure 6.18d, Figure 6.18e and Figure 6.18f, concentration results of different combinations are shown for  $k_b = 0.14$  1/day this time. All graphs show the results of last 24 hours of the study.

Table 6.6 shows the summary of chlorine concentrations in North Sancaktepe district.

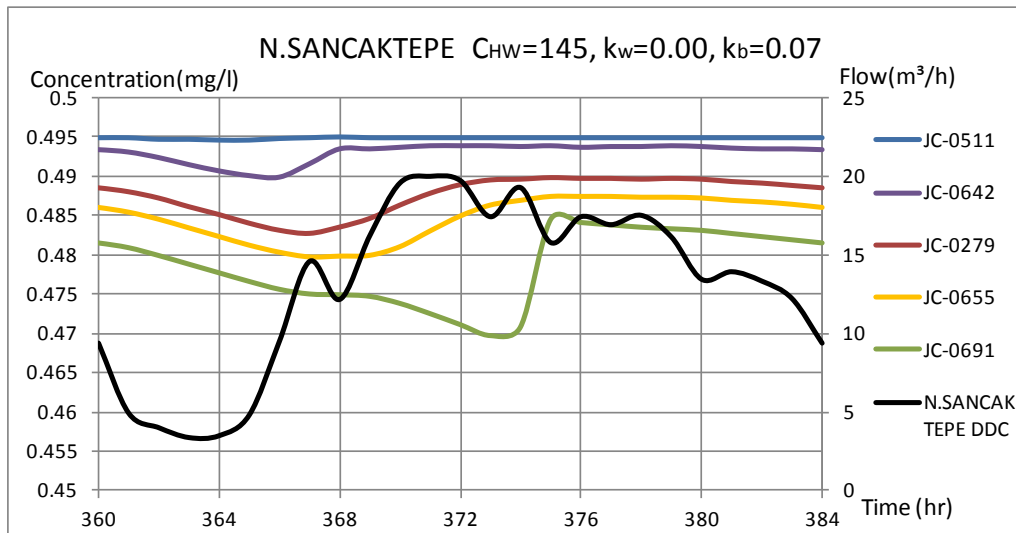
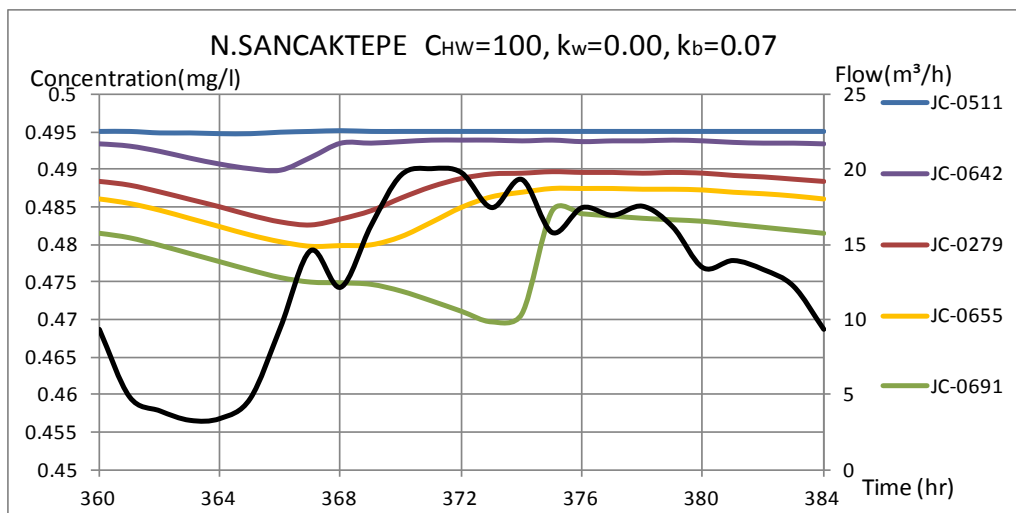
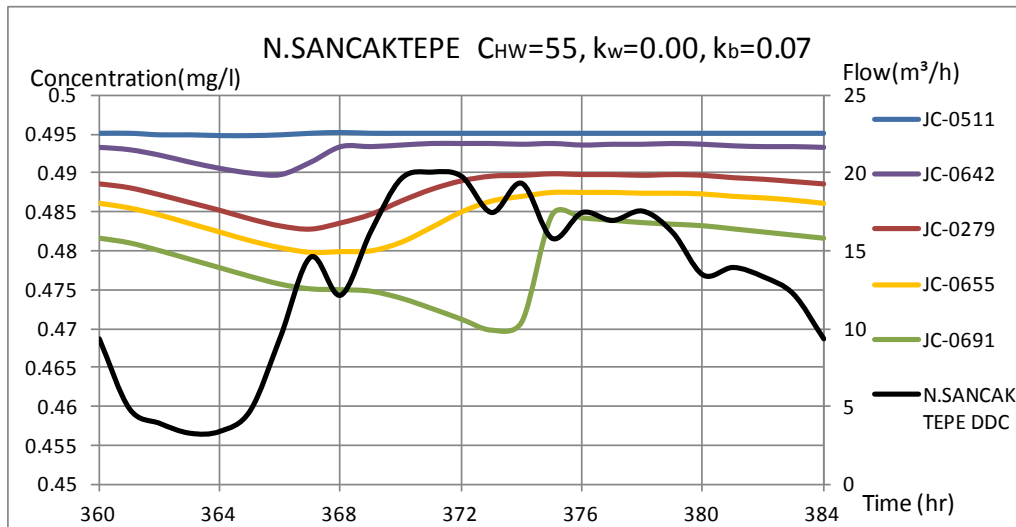


Figure 6.18a Concentration Levels for Different Cases in N.Sancaktepe District

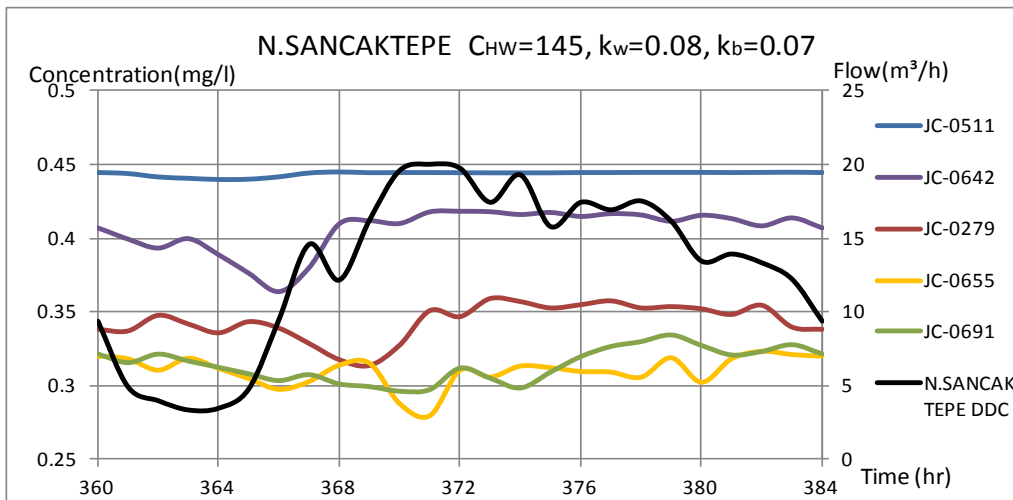
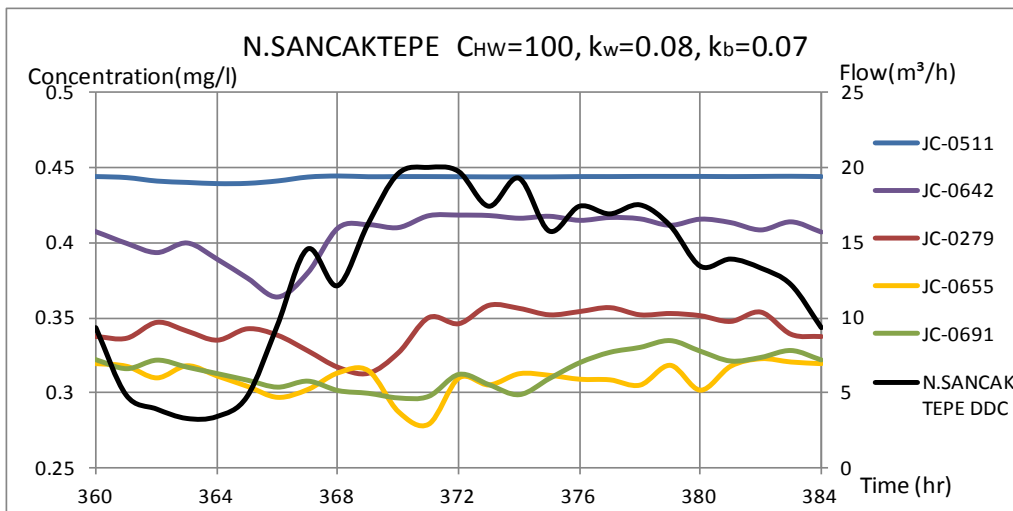
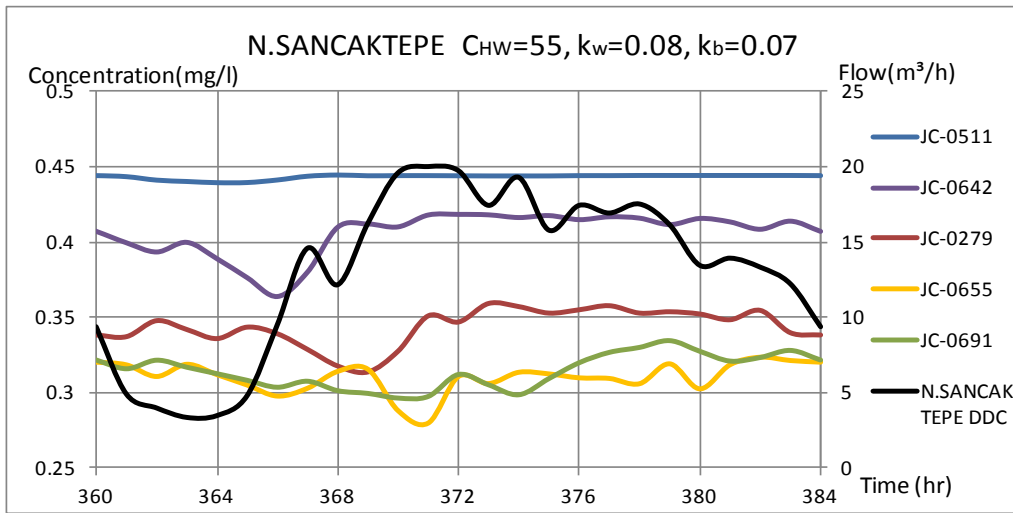


Figure 6.18b Concentration Levels for Different Cases in N.Sancaktepe District



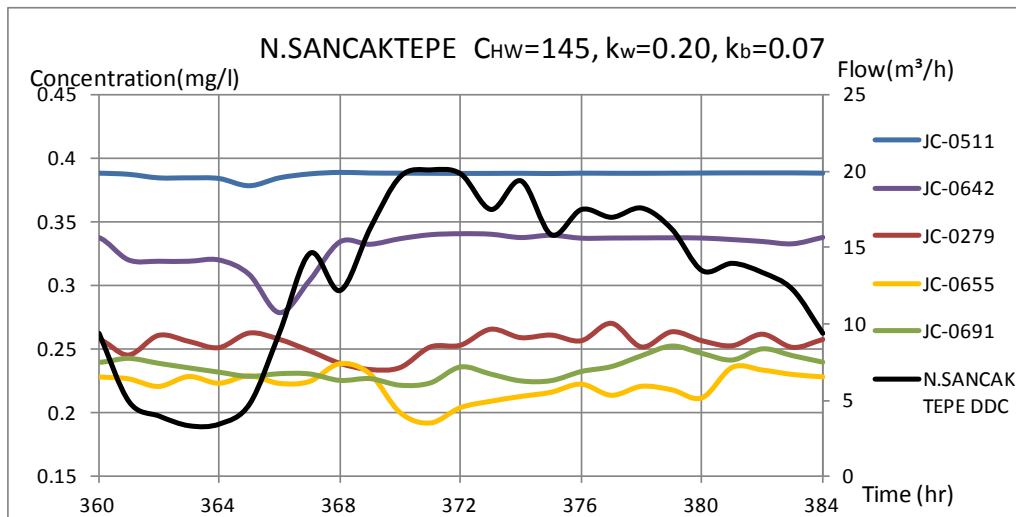
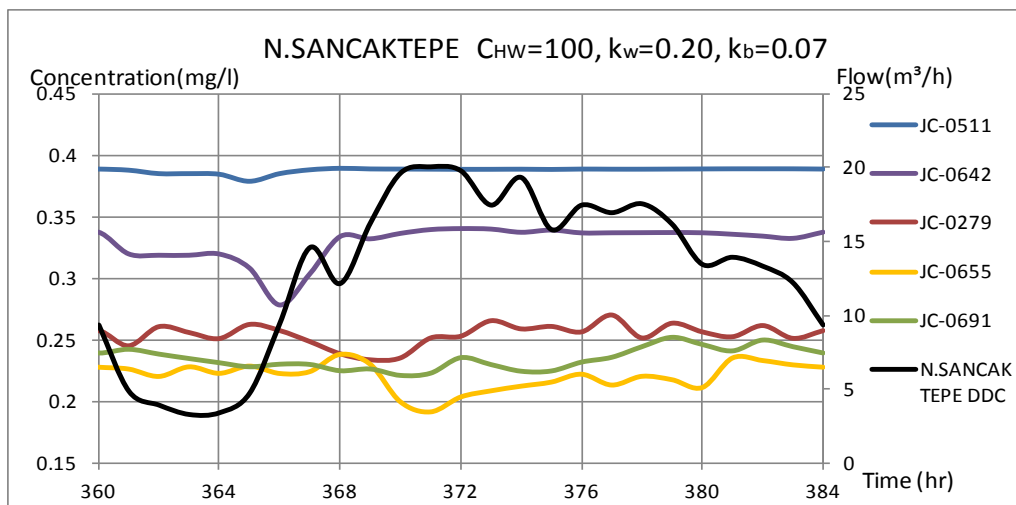
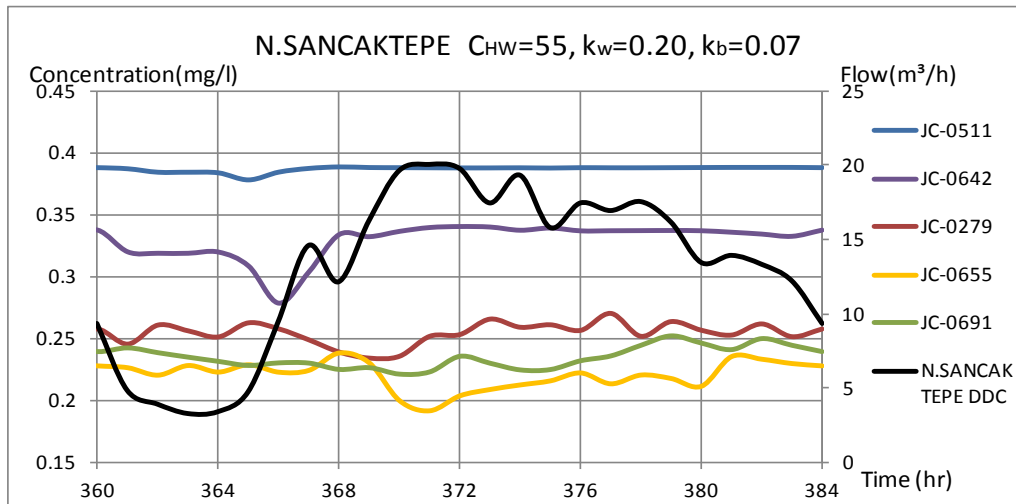


Figure 6.18c Concentration Levels for Different Cases in N.Sancaktepe District

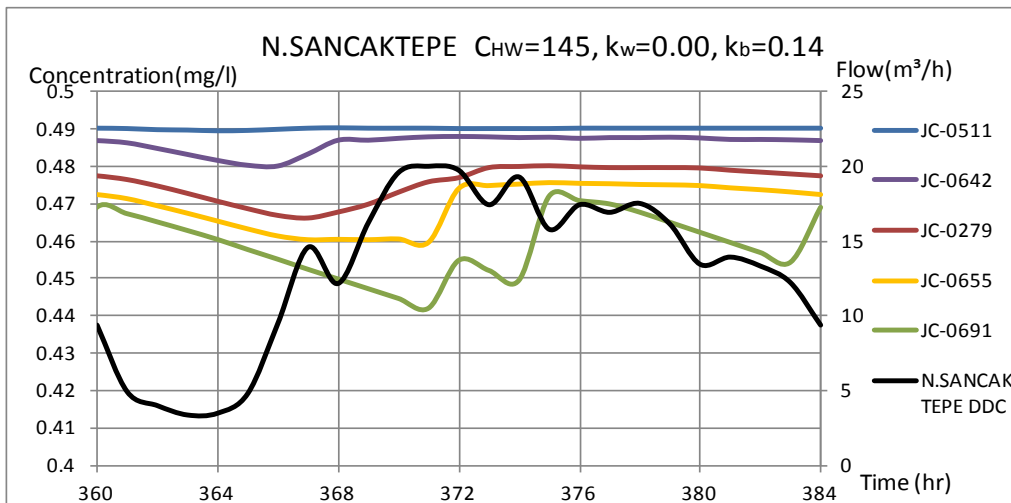
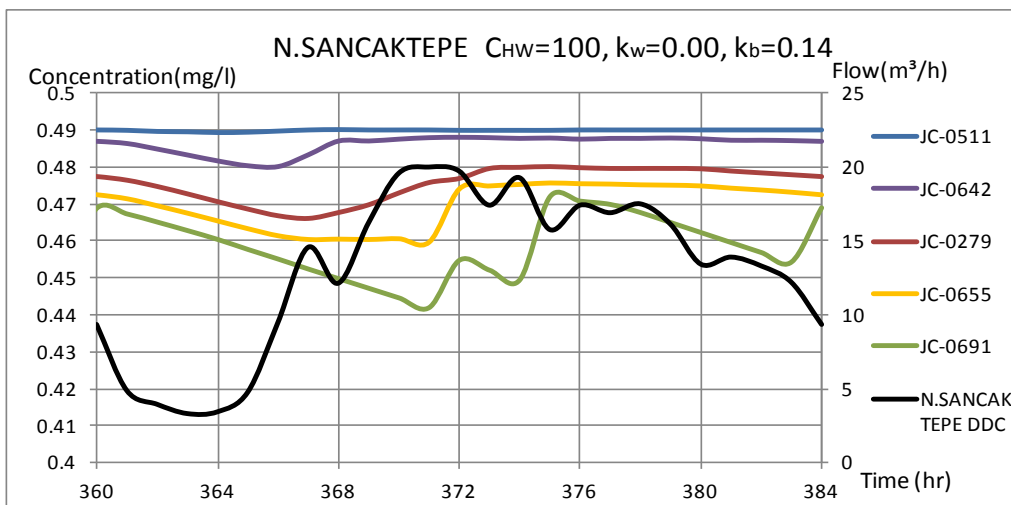
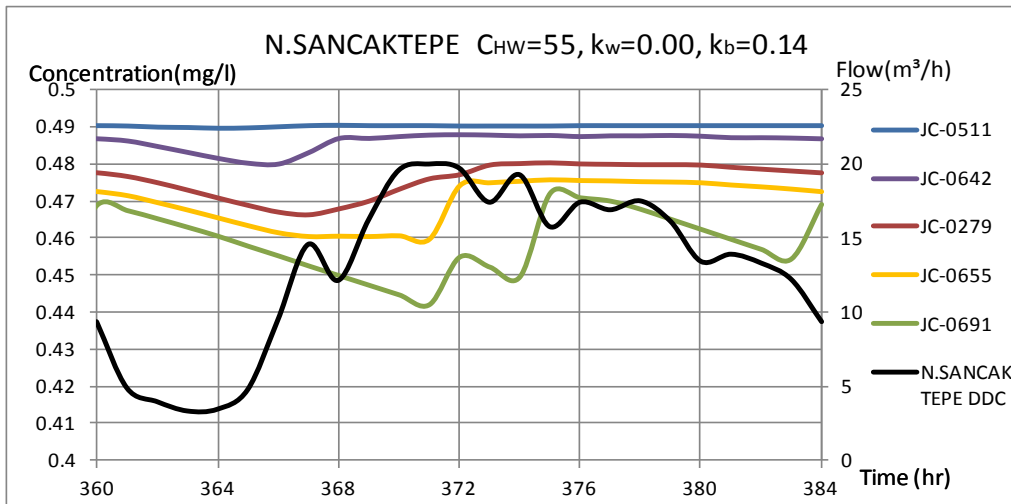


Figure 6.18d Concentration Levels for Different Cases in N.Sancaktepe District

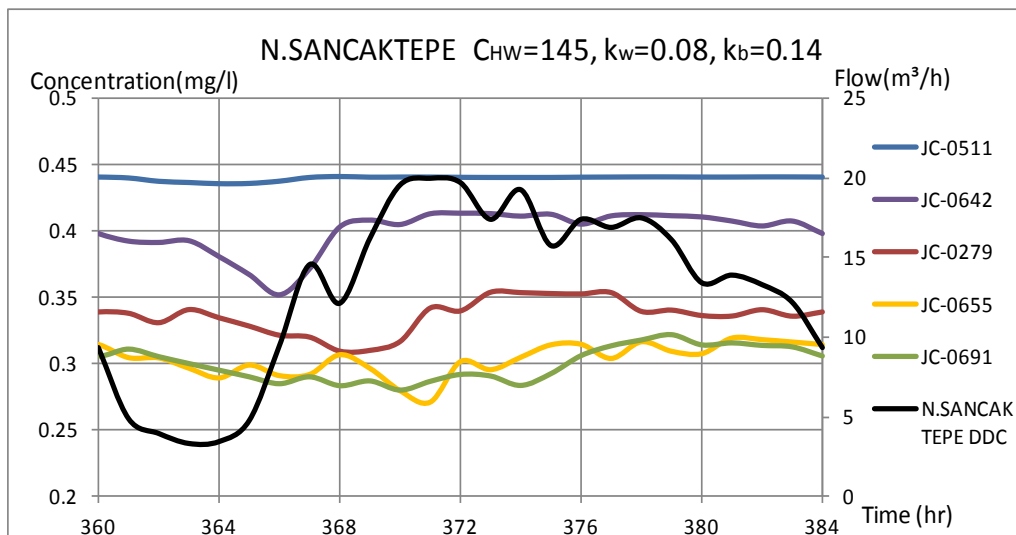
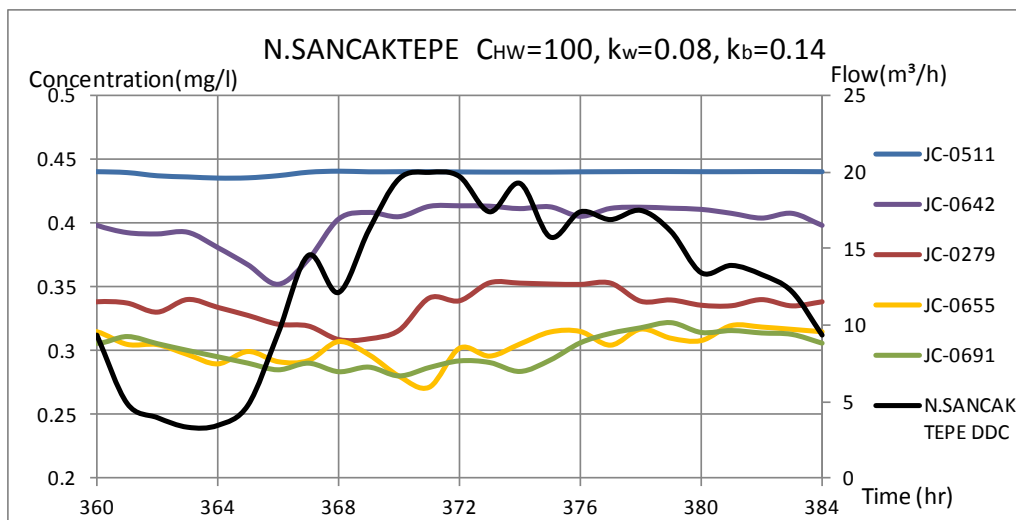
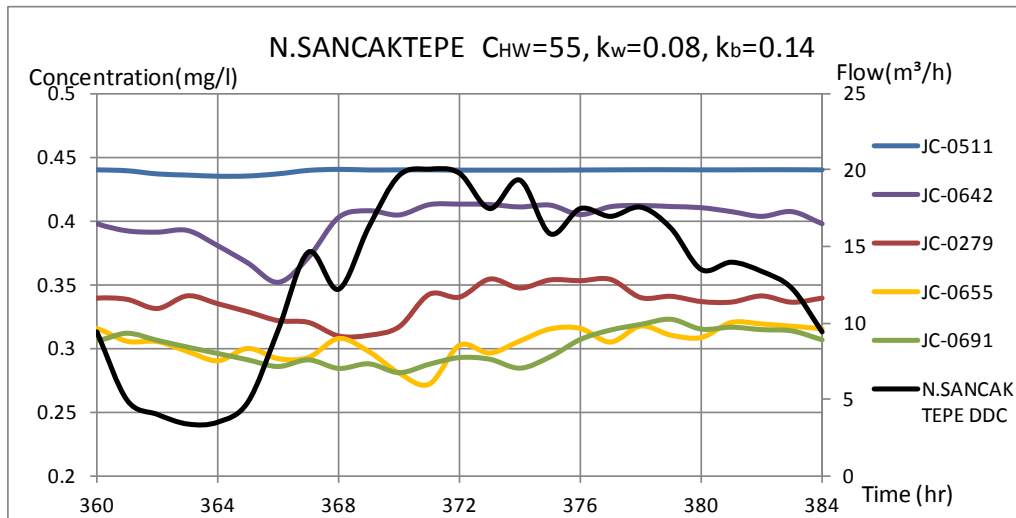


Figure 6.18e Concentration Levels for Different Cases in N.Sancaktepe District

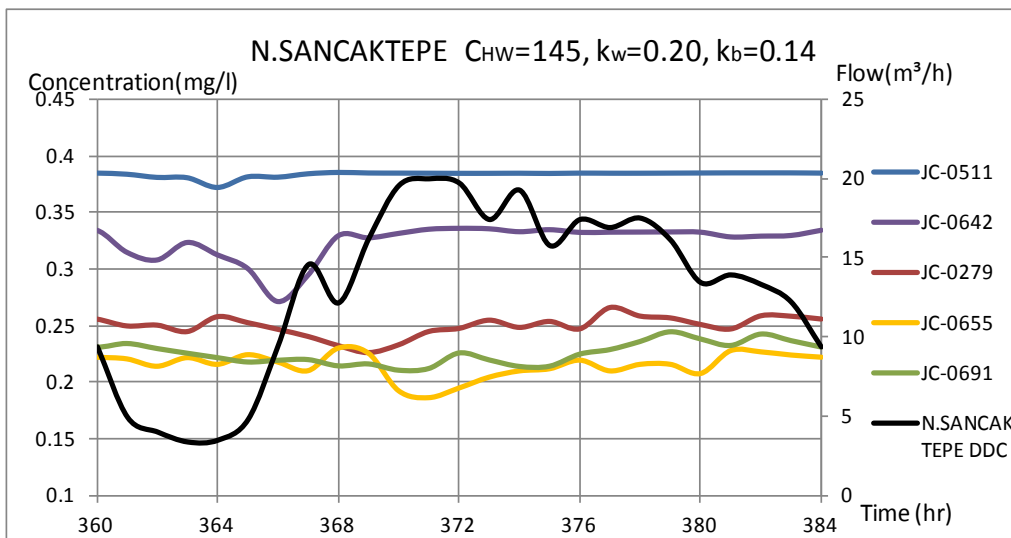
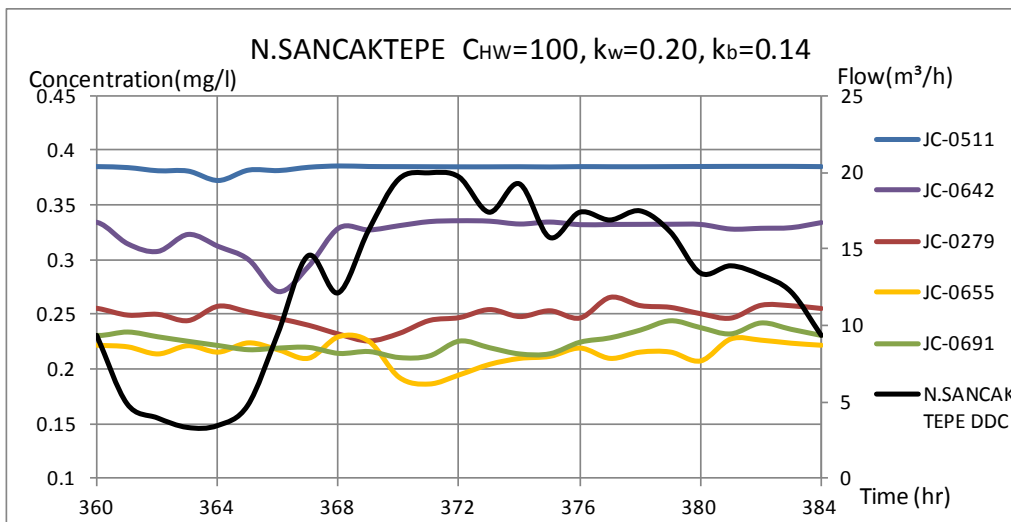
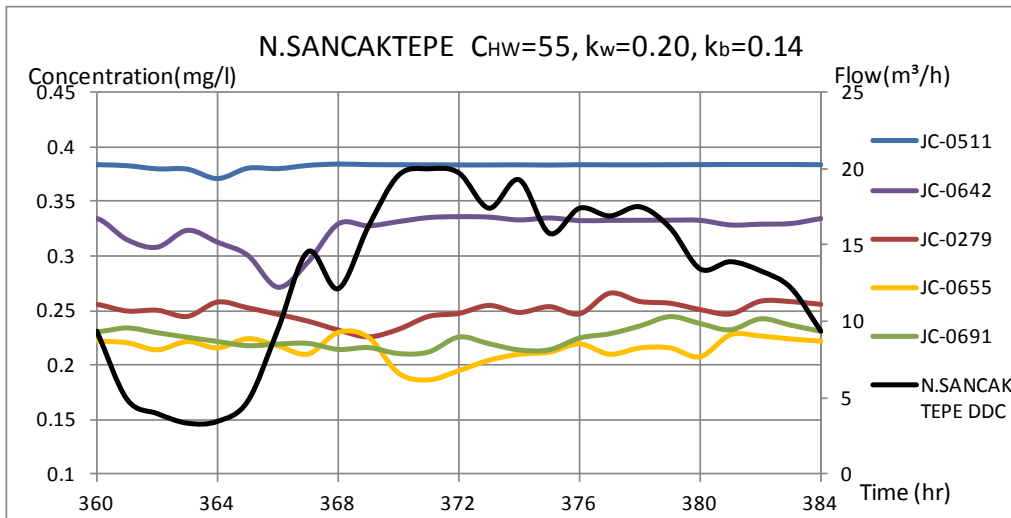


Figure 6.18f Concentration Levels for Different Cases in N.Sancaktepe District

Table 6.6 Summary of Chlorine Concentrations at Studied Junctions in North Sancaktepe

North Sancaktepe		CONCENTRATIONS (mg/l)				
C <sub>HW</sub> =55		Entrance	Middle	Middle	Middle	Dead End
		JC-0511	JC-0642	JC-0279	JC-0655	JC-0691
k <sub>w</sub> =0.00 , k <sub>b</sub> =0.07	Maximum	0.49510	0.49380	0.48980	0.48750	0.48450
	Minimum	0.49470	0.48980	0.48270	0.47980	0.46970
	Average	0.49495	0.49291	0.48750	0.48462	0.47845
k <sub>w</sub> =0.08 , k <sub>b</sub> =0.07	Maximum	0.44480	0.41860	0.35880	0.32300	0.33440
	Minimum	0.43970	0.36400	0.31340	0.27930	0.29620
	Average	0.44357	0.40603	0.34346	0.30932	0.31395
k <sub>w</sub> =0.20 , k <sub>b</sub> =0.07	Maximum	0.38890	0.34050	0.27010	0.23880	0.25270
	Minimum	0.37840	0.27860	0.23400	0.19210	0.22180
	Average	0.38720	0.32896	0.25422	0.22076	0.23528
k <sub>w</sub> =0.00 , k <sub>b</sub> =0.14	Maximum	0.49020	0.48780	0.48010	0.47580	0.47200
	Minimum	0.48940	0.47990	0.46610	0.45980	0.44200
	Average	0.48998	0.48593	0.47539	0.46968	0.45868
k <sub>w</sub> =0.08 , k <sub>b</sub> =0.14	Maximum	0.44040	0.41340	0.35380	0.31980	0.32210
	Minimum	0.43500	0.35200	0.30930	0.27140	0.28030
	Average	0.43910	0.39980	0.33571	0.30322	0.29996
k <sub>w</sub> =0.20 , k <sub>b</sub> =0.14	Maximum	0.38500	0.33620	0.26570	0.23010	0.24380
	Minimum	0.37170	0.27140	0.22580	0.18640	0.21010
	Average	0.38321	0.32406	0.24896	0.21458	0.22483
C <sub>HW</sub> =100		Entrance	Middle	Middle	Middle	Dead End
		JC-0511	JC-0642	JC-0279	JC-0655	JC-0691
k <sub>w</sub> =0.00 , k <sub>b</sub> =0.07	Maximum	0.49510	0.49380	0.48980	0.48750	0.48450
	Minimum	0.49470	0.48980	0.48270	0.47980	0.46970
	Average	0.49496	0.49292	0.48751	0.48462	0.47845
k <sub>w</sub> =0.08 , k <sub>b</sub> =0.07	Maximum	0.44480	0.41860	0.35880	0.32300	0.33440
	Minimum	0.43970	0.36400	0.31340	0.27930	0.29620
	Average	0.44358	0.40603	0.34346	0.30932	0.31394
k <sub>w</sub> =0.20 , k <sub>b</sub> =0.07	Maximum	0.38890	0.34050	0.27010	0.23880	0.25270
	Minimum	0.37840	0.27860	0.23400	0.19210	0.22180
	Average	0.38720	0.32895	0.25422	0.22075	0.23528
k <sub>w</sub> =0.00 , k <sub>b</sub> =0.14	Maximum	0.49020	0.48780	0.48010	0.47580	0.47200
	Minimum	0.48940	0.47990	0.46610	0.45980	0.44200
	Average	0.48998	0.48593	0.47539	0.46968	0.45868
k <sub>w</sub> =0.08 , k <sub>b</sub> =0.14	Maximum	0.44040	0.41340	0.35380	0.31980	0.32210
	Minimum	0.43500	0.35200	0.30930	0.27140	0.28030
	Average	0.43909	0.39981	0.33585	0.30322	0.29996
k <sub>w</sub> =0.20 , k <sub>b</sub> =0.14	Maximum	0.38500	0.33620	0.26570	0.23010	0.24380
	Minimum	0.37170	0.27140	0.22580	0.18640	0.21010
	Average	0.38321	0.32406	0.24895	0.21458	0.22482
C <sub>HW</sub> =145		Entrance	Middle	Middle	Middle	Dead End
		JC-0511	JC-0642	JC-0279	JC-0655	JC-0691
k <sub>w</sub> =0.00 , k <sub>b</sub> =0.07	Maximum	0.49510	0.49380	0.48980	0.48750	0.48450
	Minimum	0.49470	0.48980	0.48270	0.47980	0.46970
	Average	0.49496	0.49292	0.48751	0.48463	0.47845
k <sub>w</sub> =0.08 , k <sub>b</sub> =0.07	Maximum	0.44480	0.41860	0.35880	0.32300	0.33440
	Minimum	0.43970	0.36400	0.31340	0.27930	0.29620
	Average	0.44358	0.40603	0.34346	0.30932	0.31394
k <sub>w</sub> =0.20 , k <sub>b</sub> =0.07	Maximum	0.38890	0.34050	0.27010	0.23880	0.25270
	Minimum	0.37840	0.27860	0.23400	0.19210	0.22180
	Average	0.38720	0.32895	0.25422	0.22075	0.23528
k <sub>w</sub> =0.00 , k <sub>b</sub> =0.14	Maximum	0.49020	0.48780	0.48010	0.47580	0.47200
	Minimum	0.48940	0.47990	0.46610	0.45980	0.44200
	Average	0.48998	0.48593	0.47539	0.46968	0.45868
k <sub>w</sub> =0.08 , k <sub>b</sub> =0.14	Maximum	0.44040	0.41340	0.35380	0.31980	0.32210
	Minimum	0.43500	0.35200	0.30930	0.27140	0.28030
	Average	0.43910	0.39981	0.33598	0.30323	0.29996
k <sub>w</sub> =0.20 , k <sub>b</sub> =0.14	Maximum	0.38500	0.33620	0.26570	0.23010	0.24380
	Minimum	0.37170	0.27140	0.22580	0.18640	0.21010
	Average	0.38320	0.32406	0.24895	0.21458	0.22482

Both figures and summary table show the North Sancaktepe chlorine concentration results of the last 24 hours to provide more realistic stable condition results and the system is operating with single pump operation again. Also, DDC of the subzone is shown with black curve in graphs.

In North Sancaktepe, chlorine concentration levels of the entrance junction JC-0511 and dead end junction JC-0691 again differs depending on the distance to entrance of the subzone. Blue curves show the results of entrance junction JC-0511 and green curves show the results of the dead end junction JC-0691. While analysing the results from entrance to dead ends, travel time until water reaches to a junction affects chlorine concentrations and they give minimum levels at dead ends comparing to other studied junctions similarly in Yayla. Also, because of locating near to the entrance, chlorine concentration graph of the junction JC-0511 is almost a straight line during the day except peak hours. Because, fresh water reaches to entrance junction firstly and causes lower chlorine decay. However, when analysing middle junctions, this stable condition is not seen and fluctuations occur in concentration vs. time graphs.

The shifting in the occurrence time of minimum concentrations can be seen clearly again from entrance junction to dead end junction. At entrance junction, minimum concentration occurs at early morning hours (04.00-05.00 am.), however it occurs at later morning hours (09.00-12.00 am.) while going to middle or dead end junctions. Because of the reason that when demands increase in morning hours, dead end junctions are the latest influenced junctions again because of the long way of water movement (travel time).

By analysing the Figure 6.18a ( $k_b = 0.07$  1/day and  $k_w = 0.00$  m/day), Figure 6.18b ( $k_b = 0.07$  1/day and  $k_w = 0.08$  m/day), Figure 6.18c ( $k_b = 0.07$  1/day and  $k_w = 0.20$  m/day), Figure 6.18d ( $k_b = 0.14$  1/day and  $k_w = 0.00$  m/day), Figure 6.18e ( $k_b = 0.14$  1/day and  $k_w = 0.80$  m/day) and Figure 6.18f ( $k_b = 0.14$  1/day and  $k_w = 0.20$  m/day); it can be said that for a fixed  $k_b$  and

$k_w$  combination, changing the roughness coefficient  $C_{HW}$  is not effective on chlorine concentrations. Because North Sancaktepe district takes water only from the pump station similarly as Yayla and changing the roughness coefficient does not affect pump operation or pumping behaviour. Examining the summary of chlorine concentrations in North Sancaktepe from Table 6.6, they do not change even in 5th digit by changing  $C_{HW}$  coefficient.

In graphs, DDC of North Sancaktepe district is shown with black curve. It can also be clearly seen that, while demands starts to increase in morning hours, chlorine concentrations start to decrease because of the high rate of decay. This demand effect is seen later at dead end junctions comparing to entrance or middle junctions because of the travel time.

At middle junctions, concentrations differ depending on the condition of the junctions as mentioned before. It is important that if studied junction is sink or not. If all water that comes to a junction is extracted, it is a sink junction. However, if only some of the coming water is extracted, and some of it passes away, it is not a sink junction and may give lower concentration results. For example in North Sancaktepe, it must be pointed that, for all different combinations of  $k_w$  and  $k_b$ , although JC-0655 is a middle junction it gives lower chlorine concentrations than dead end junction JC-0691 at some time intervals. This may be caused by the location of the junction and water behaviour caused by the flow field of the system. Amount of nodal demands and the water supplies that feed the subzone are also important points.

It can also be seen in the Figures from 6.18a to 6.18f that, while using  $k_w$  as 0.00 m/day, chlorine concentrations give smooth curves for the shown time interval. However, while increasing  $k_w$  to 0.08 m/day or to 0.20 m/day, concentrations start to give more zigzaggy curves because of the non linear behaviour of the WDN and hydraulic differences caused by changing  $k_w$  value.

In Figure 6.19a, 6.19b, 6.19c, 6.19d and 6.19e show how chlorine concentrations are influenced by changing system parameters Hazen-Williams roughness coefficient  $C_{HW}$ , bulk decay coefficient  $k_b$  and wall decay coefficient  $k_w$  at every studied junction one by one. By analysing this type of graphs, it can be understood more easily that which parameter is the most effective one.

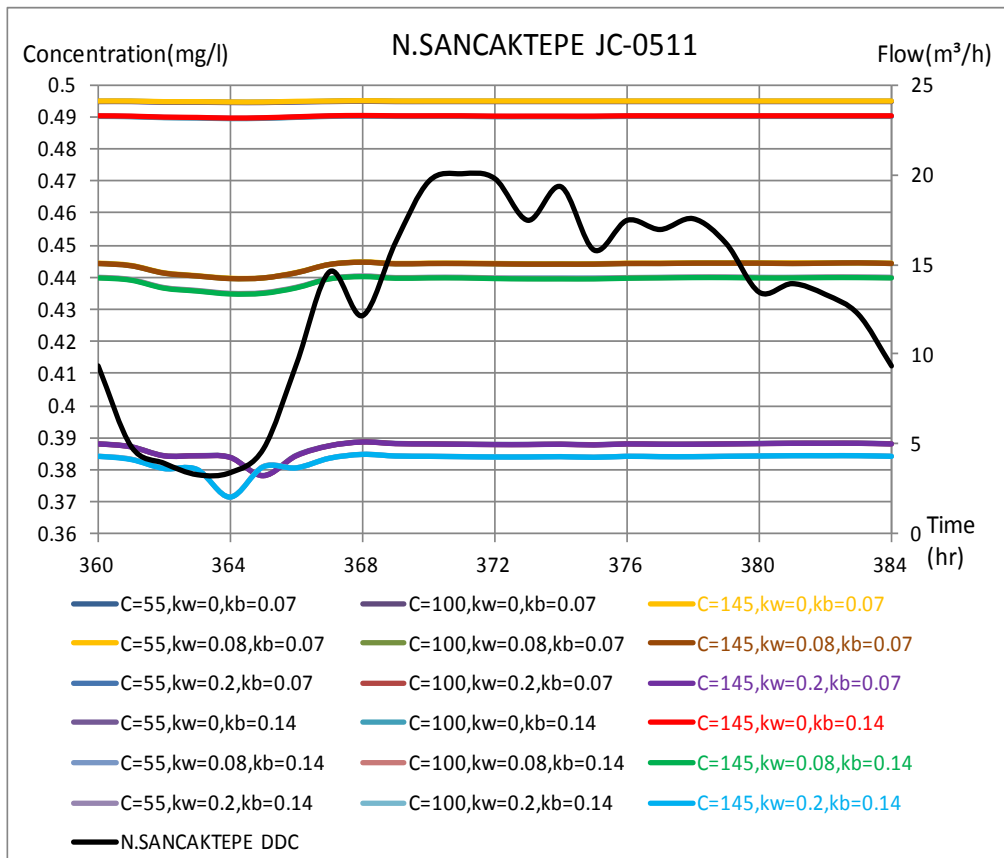


Figure 6.19a Concentration Levels for Different Cases at JC-0511



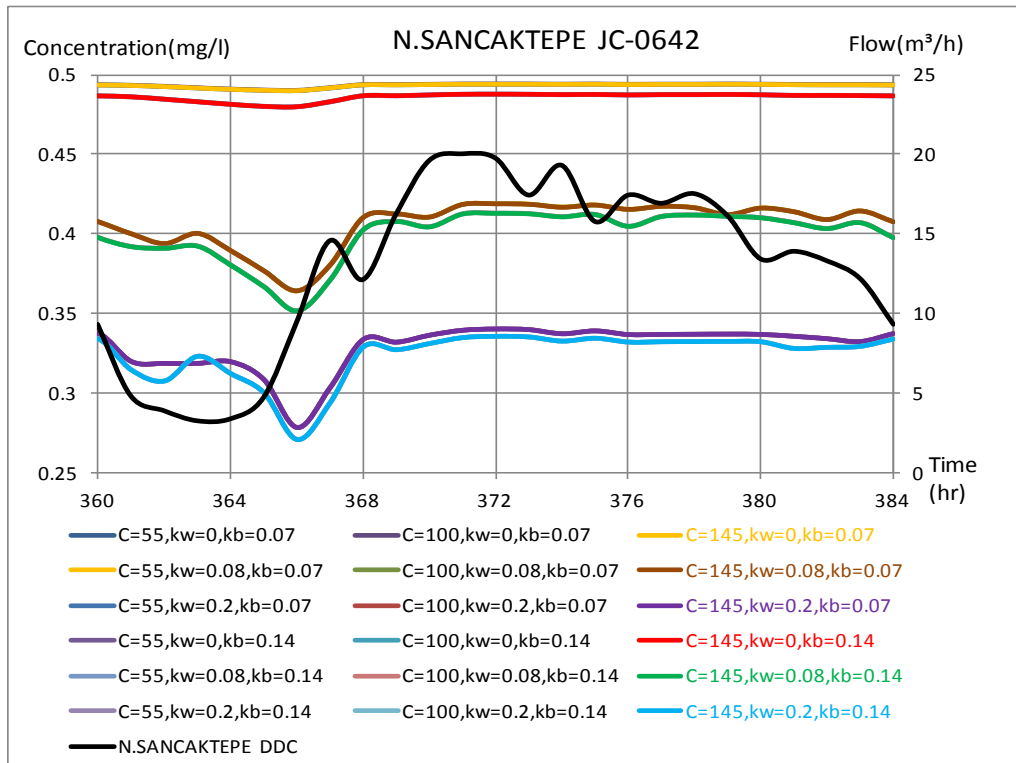


Figure 6.19b Concentration Levels for Different Cases at JC-0642

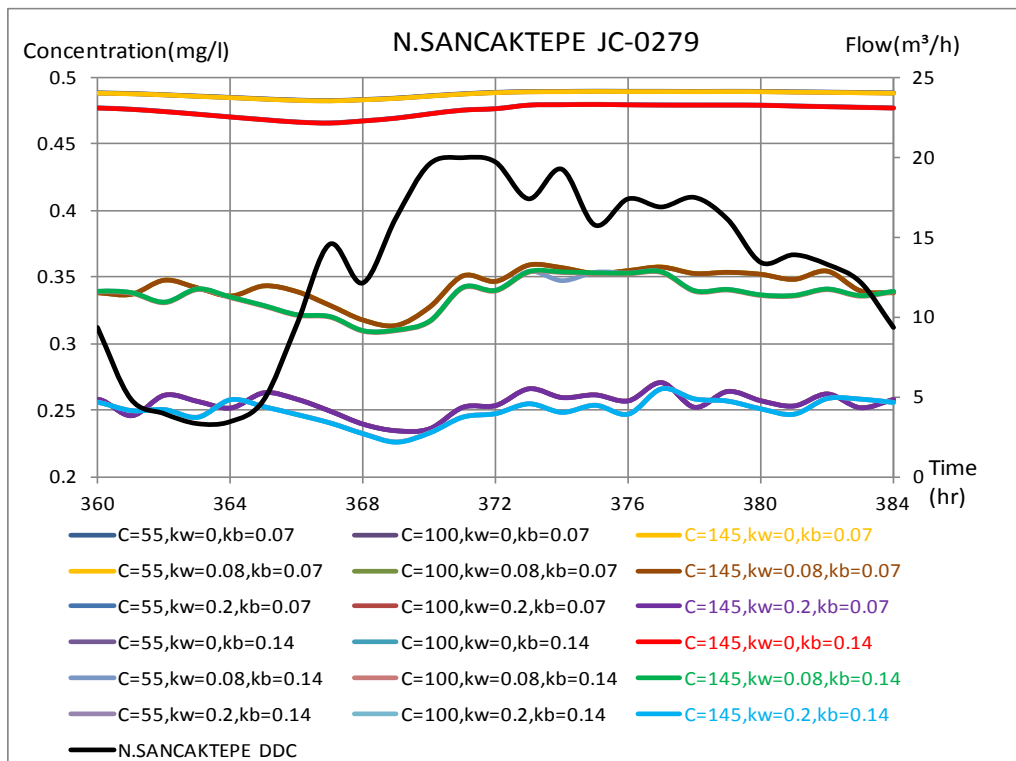


Figure 6.19c Concentration Levels for Different Cases at JC-0279

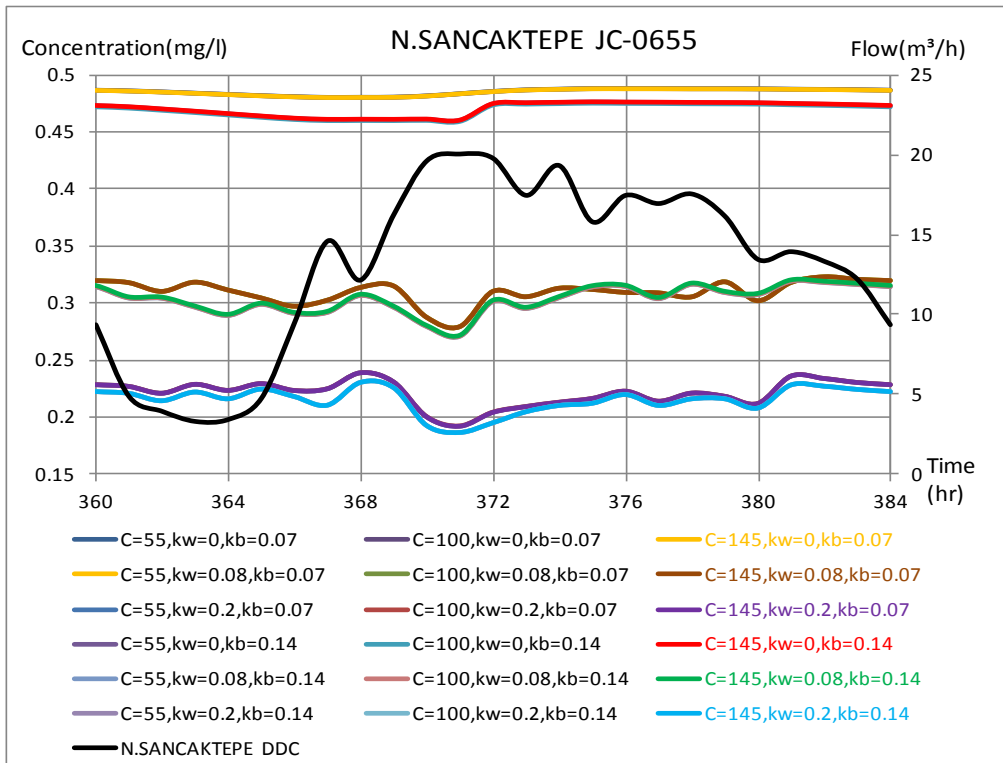


Figure 6.19d Concentration Levels for Different Cases at JC-0655

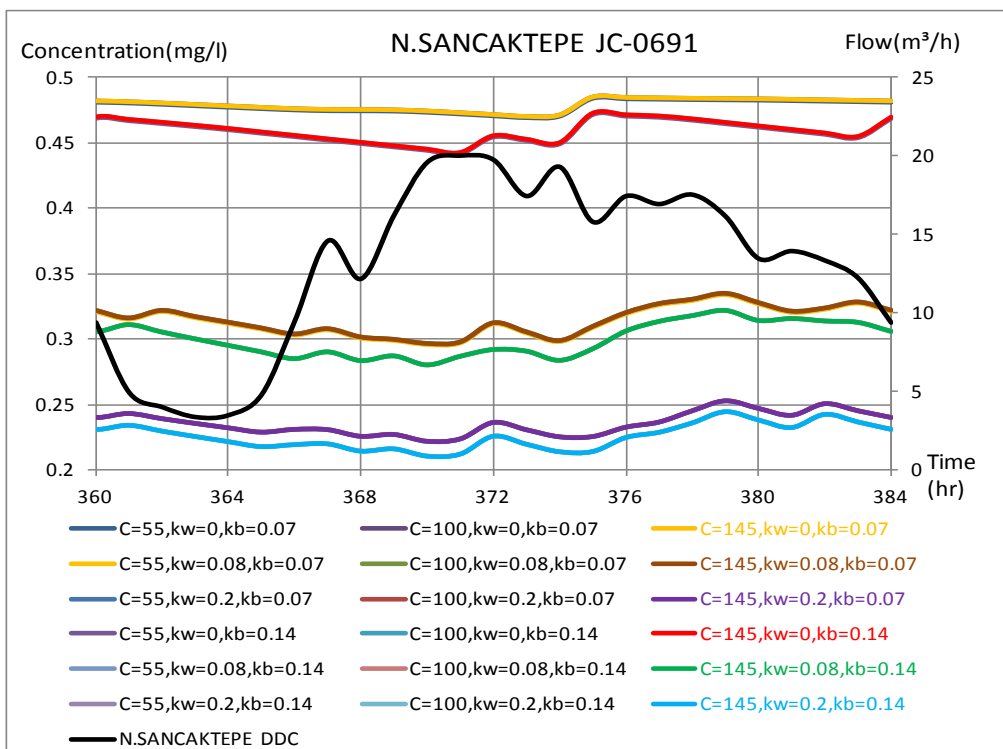


Figure 6.19e Concentration Levels for Different Cases at JC-0691

As it is seen in the figures above, changing Hazen-Williams roughness coefficient does not affect chlorine concentrations in North Sancaktepe in a similar way with Yayla. Because of this reason, six different curves can be seen and these curves are the results of six scenarios which are written with coloured text. Graphs of other twelve scenarios are overlapped and all graphs show the results by using Hazen Williams roughness coefficient  $C_{HW}$  as 145. This is because of the reason that the subzone receives water only from the pump station and it does not receive water from the tank during the procedure. Therefore, chlorine concentrations are not affected by tank fluctuations and the chlorine concentration of the tank because velocity does not change and concentrations do not give a difference by changing  $C_{HW}$  coefficients.

It is clearly seen that, for this specific Hazen-Williams roughness coefficient  $C_{HW}$ ; red and yellow curves show results for  $k_w = 0.00$  m/day, green and brown curves show results for  $k_w = 0.08$  m/day, blue and purple curves show results for  $k_w = 0.20$  m/day. Also, DDC of the subzone is shown with black curve in graphs. As seen in figures, increasing the  $k_w$  value from 0.00 m/day, to 0.080 m/day or to 0.20 m/day decreases chlorine concentrations. For a specific  $k_w$  value changing bulk decay coefficient  $k_b$  is also effective. For example, by using  $k_w$  value as 0.00 m/day, yellow curve shows results for  $k_b = 0.07$  1/day and red curve shows results for  $k_b = 0.14$  1/day. It is clearly seen that, increasing the  $k_b$  value from 0.07 1/day to 0.14 1/day, decreases chlorine concentrations.

Due to seeing that while  $k_w$  is increasing, chlorine concentrations are decreasing; it can be said that using older pipes in a WDN causes lower disinfection concentration Increment in the  $k_b$  value decreases chlorine concentrations when distributing the same demands. It can be clearly seen that, increasing both  $k_w$  and  $k_b$  values give decreasing chlorine concentrations, however changing  $k_w$  is more effective on them.

### 6.3.3 East Çiğdemtepe District

East Çiğdemtepe district receives water from the main line passing in the north-east part of the district. It has boundaries with West Çiğdemtepe and Şehit Kubilay districts so as a difference from Yayla district, it is expected to see different behaviours and chlorine concentration values for whole network and DMA's cases because of mixing water and interactions between these subzones. East Çiğdemtepe has 66 junctions totally.

The junctions chosen for East Çiğdemtepe district is JC-0629 at the entrance; JC-0051, JC-0287, JC-0572 at middle points and JC-0027 at dead end point. These junctions were shown in Figure 6.10. All 18 different scenarios are performed with Single Pump Operation (1-SUMAS is set on) continuously for 24 hours, similarly in Yayla and North Sancaktepe.

In East Çiğdemtepe, differently from other studied subzones, while peak demands are drawn the pump station becomes incapable and the tank starts to feed the subzone as an extra water supply. This causes different results in chlorine concentrations by changing Hazen-Williams roughness coefficient  $C_{HW}$ .

Figure 6.20a shows chlorine concentrations by using the same bulk decay coefficient  $k_b = 0.07$  1/day and the same wall decay coefficient  $k_w = 0.00$  m/day, assuming the system pipes are totally new. Only Hazen-Williams roughness coefficient  $C_{HW}$  is changed while  $k_w$  and  $k_b$  were constant. Figure 6.20b shows concentrations with  $k_b = 0.07$  1/day and  $k_w = 0.08$  m/day, again changing only roughness coefficient  $C_{HW}$ . Figure 6.20c shows them with  $k_b = 0.07$  1/day and  $k_w = 0.20$  m/day assuming the system has older pipes and changing only the roughness coefficient  $C_{HW}$  similarly in Figure 6.20a and 6.20b. In Figure 6.20d, Figure 6.20e and Figure 6.20f, concentration results of different combinations are shown for  $k_b = 0.14$  1/day this time. All graphs show the results of last 24 hours of the study.

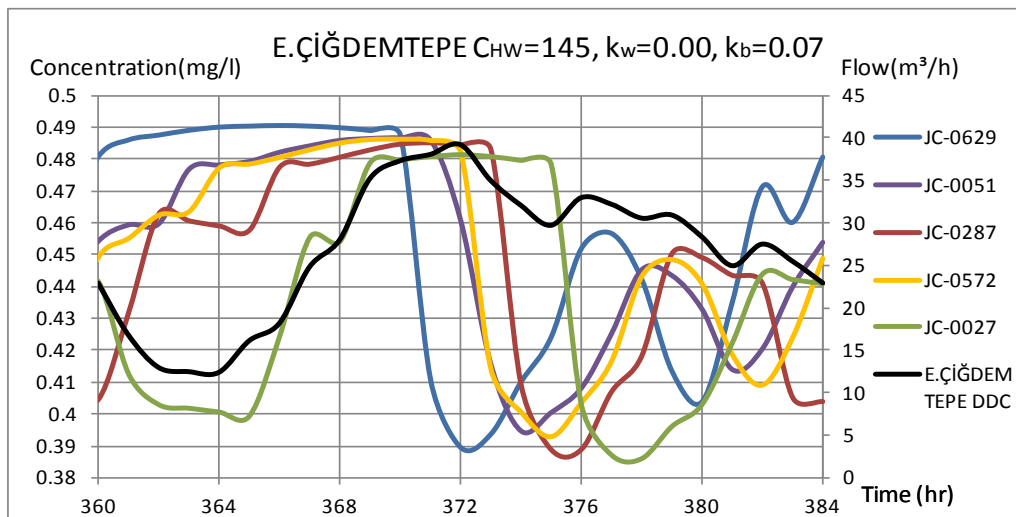
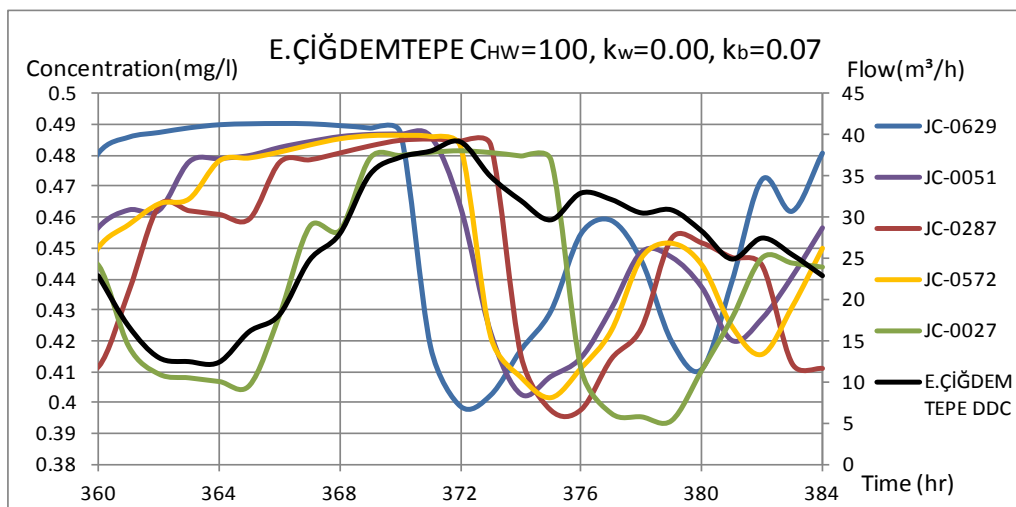
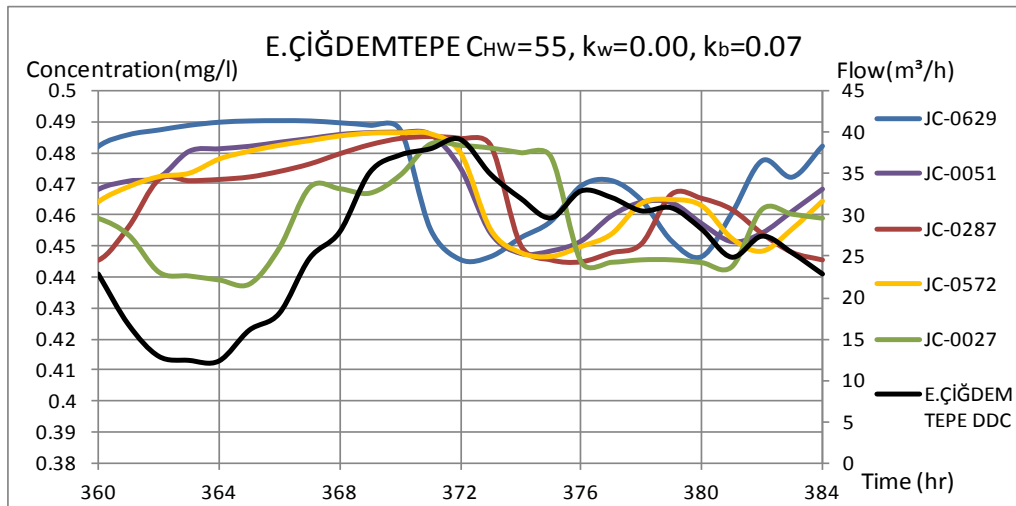


Figure 6.20a Concentration Levels for Different Cases in E.Çiğdemtepe District

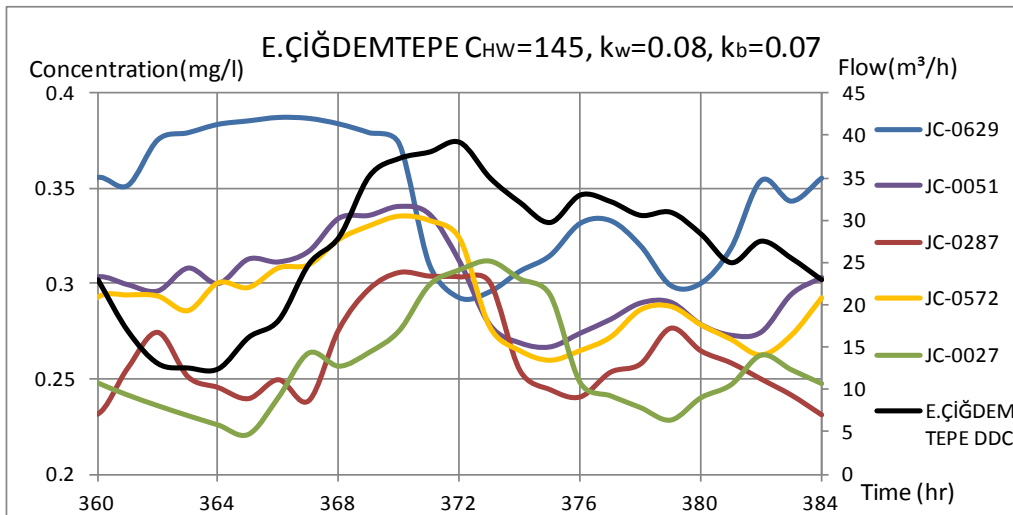
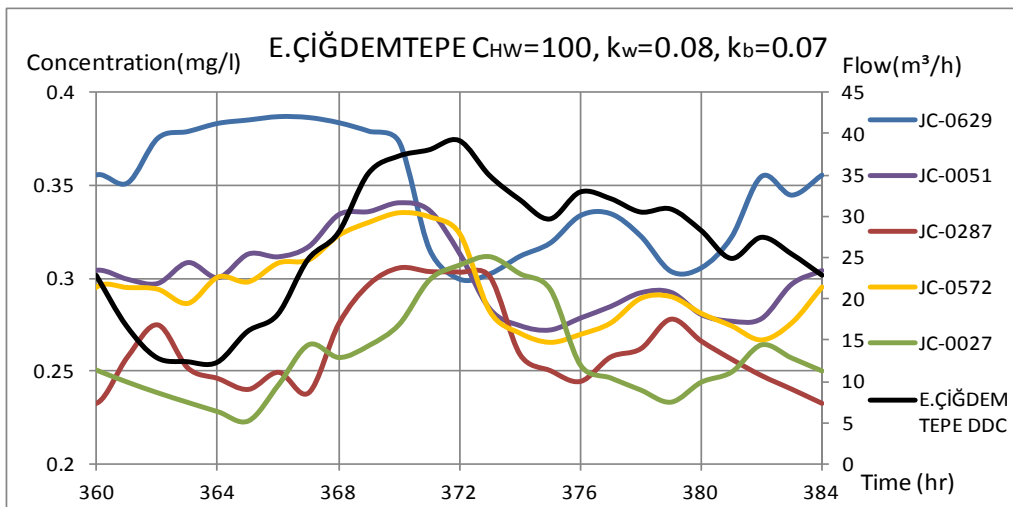
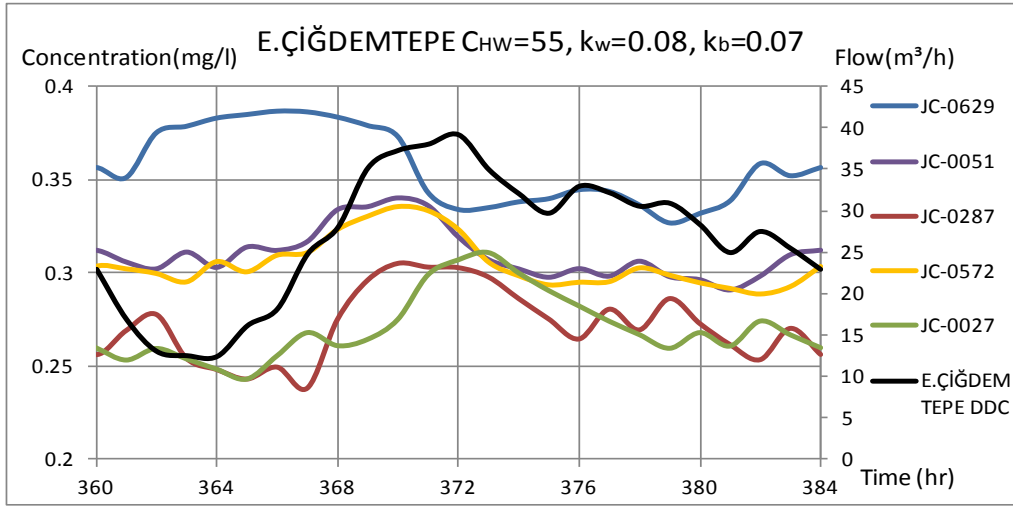


Figure 6.20b Concentration Levels for Different Cases in E.Çiğdemtepe District

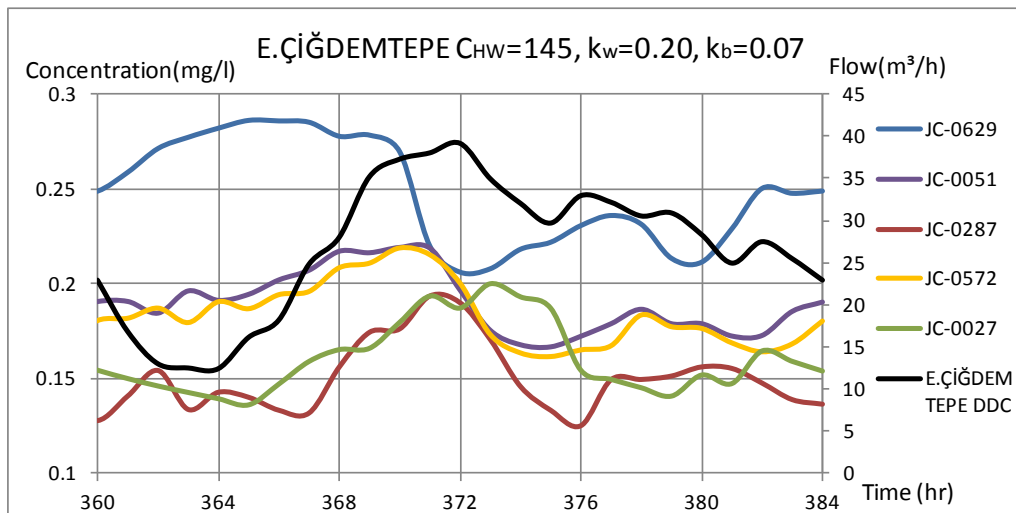
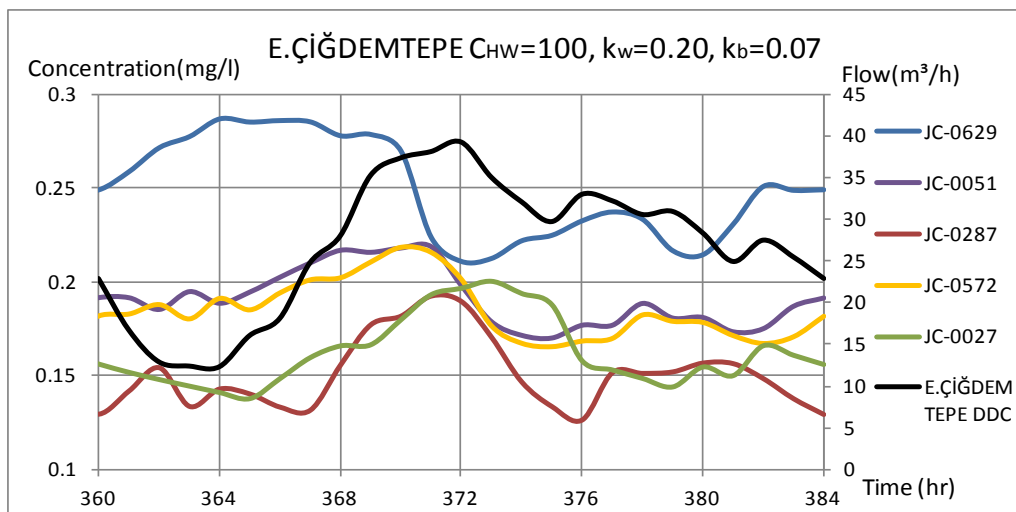
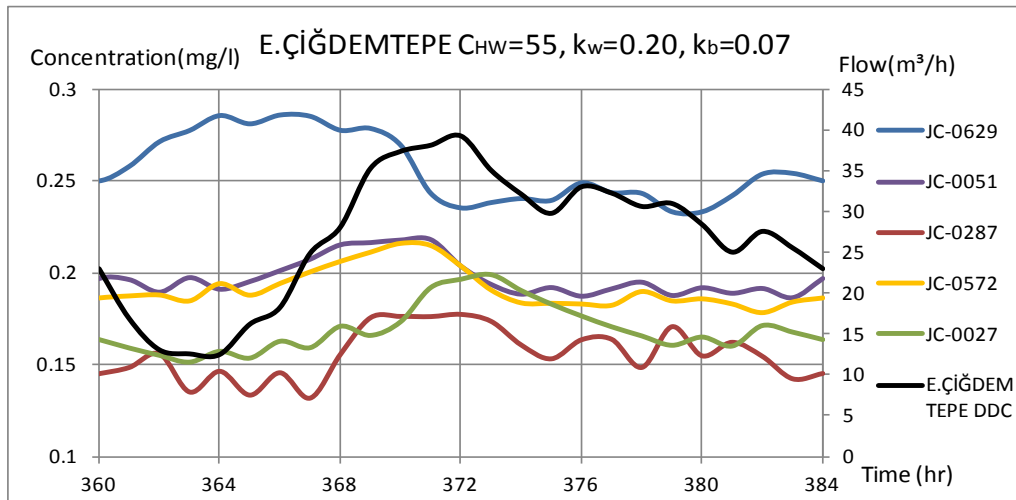


Figure 6.20c Concentration Levels for Different Cases in E.Çiğdemtepe District

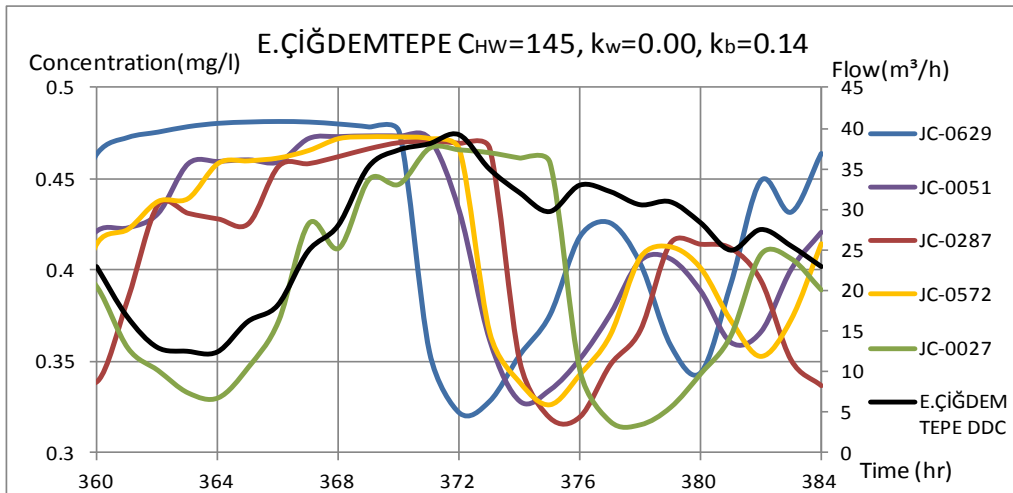
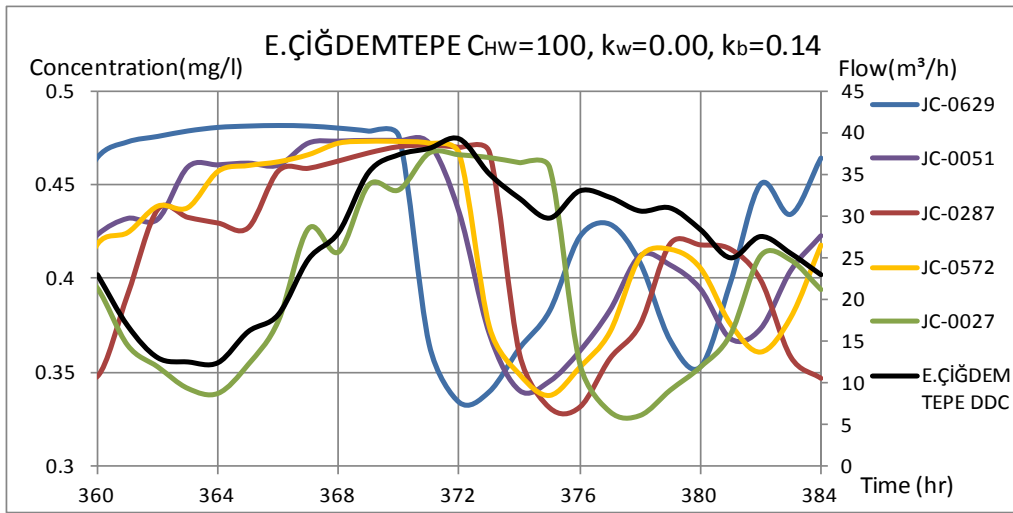
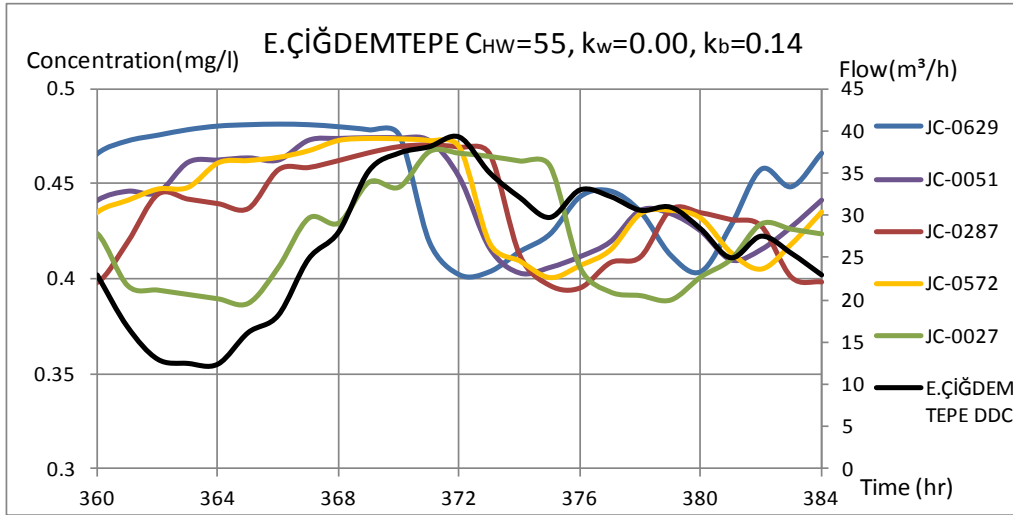


Figure 6.20d Concentration Levels for Different Cases in E.Çiğdemtepe District



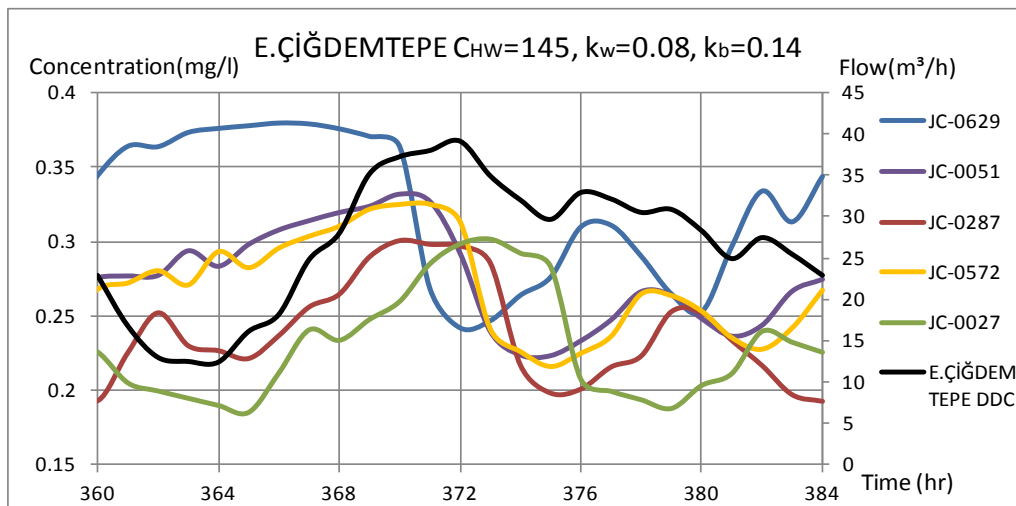
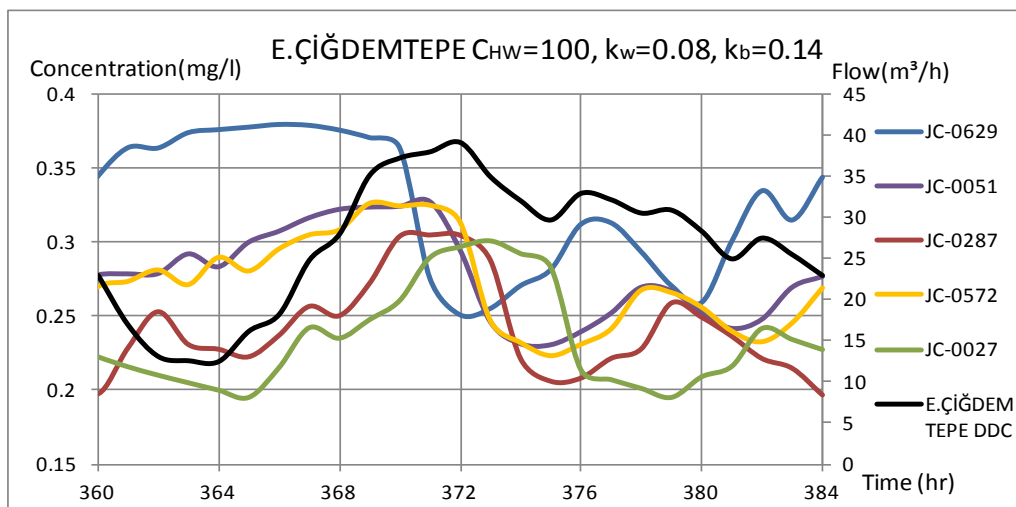
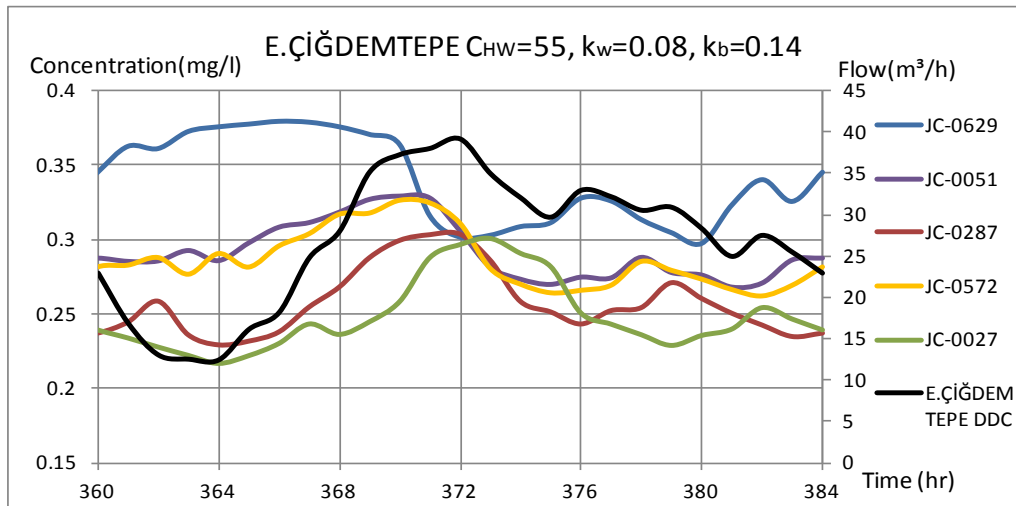


Figure 6.20e Concentration Levels for Different Cases in E.Çiğdemtepe District

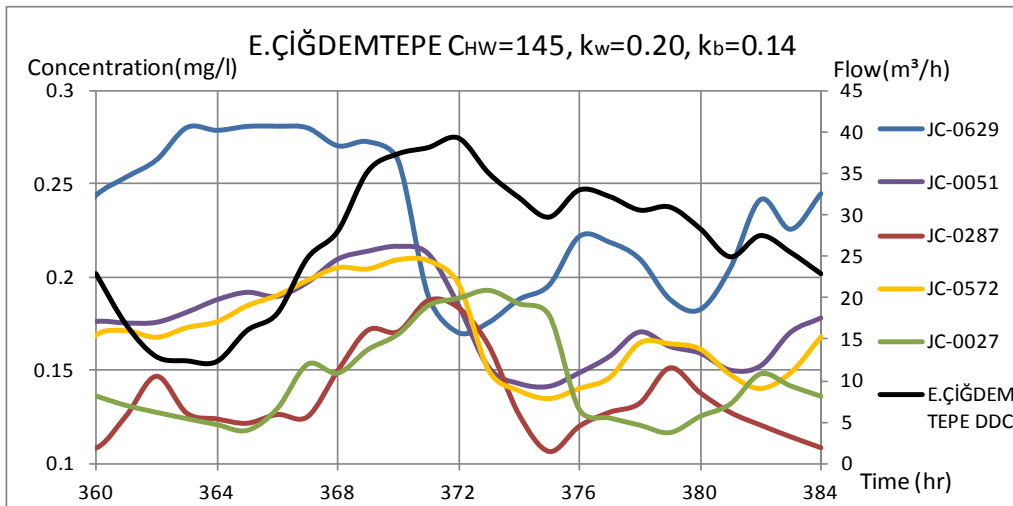
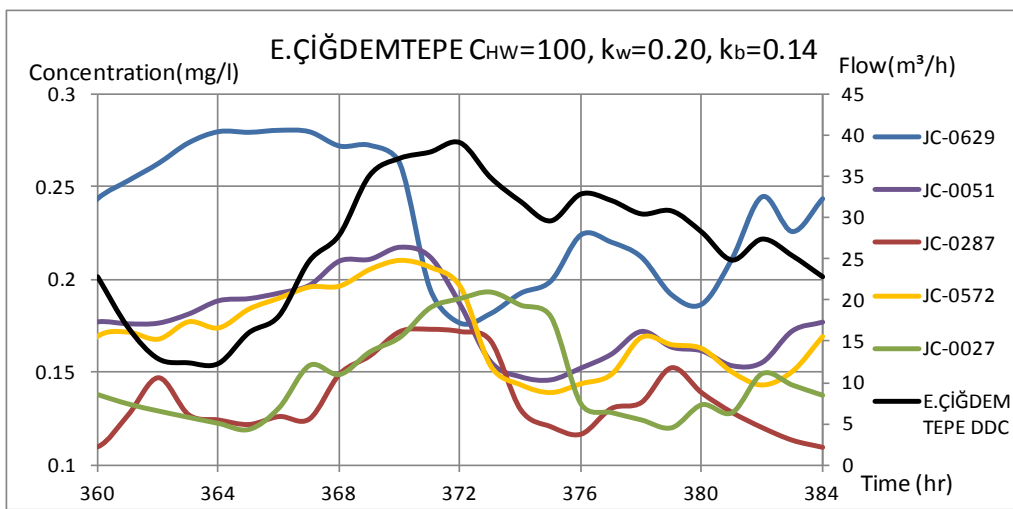
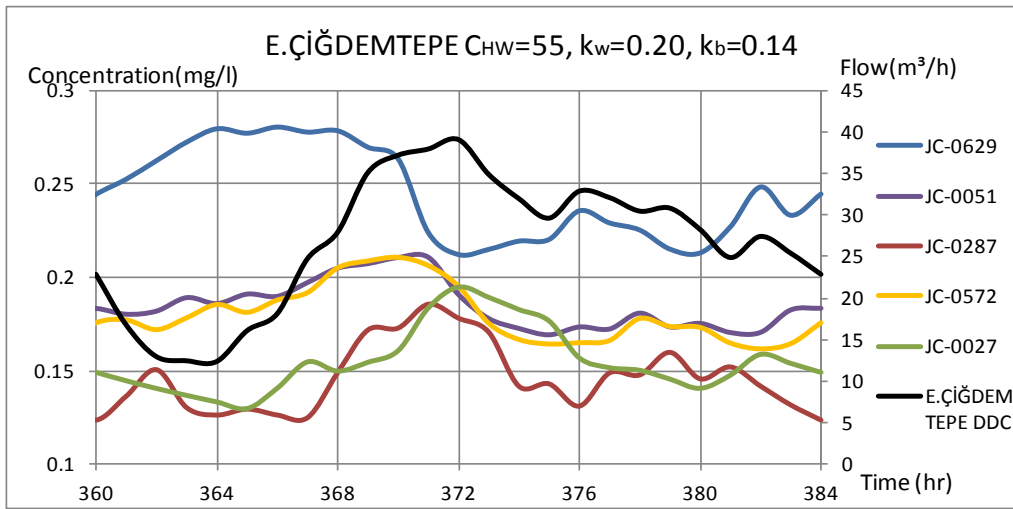


Figure 6.20f Concentration Levels for Different Cases in E.Çiğdemtepe District

Table 6.7 Summary of Chlorine Concentrations at Studied Junctions in East Çiğdemtepe

East Çiğdemtepe		CONCENTRATIONS (mg/l)				
C <sub>HW</sub> =55		Entrance	Middle	Middle	Middle	Dead End
		JC-0629	JC-0051	JC-0287	JC-0572	JC-0027
k <sub>w</sub> =0.00 , k <sub>b</sub> =0.07	Maximum	0.49050	0.48650	0.48510	0.48620	0.48280
	Minimum	0.44560	0.44740	0.44470	0.44630	0.43760
	Average	0.47272	0.46890	0.46537	0.46764	0.45803
k <sub>w</sub> =0.08 , k <sub>b</sub> =0.07	Maximum	0.38670	0.34000	0.30530	0.33510	0.31070
	Minimum	0.32670	0.29070	0.23830	0.28820	0.24280
	Average	0.35667	0.31010	0.27256	0.30499	0.27071
k <sub>w</sub> =0.20 , k <sub>b</sub> =0.07	Maximum	0.28550	0.21750	0.17760	0.21580	0.19920
	Minimum	0.23280	0.18590	0.13200	0.17810	0.15140
	Average	0.25668	0.19700	0.15648	0.19160	0.16968
k <sub>w</sub> =0.00 , k <sub>b</sub> =0.14	Maximum	0.48120	0.47370	0.47070	0.47330	0.46710
	Minimum	0.40180	0.40250	0.39520	0.40000	0.38710
	Average	0.44933	0.44150	0.43578	0.44023	0.42143
k <sub>w</sub> =0.08 , k <sub>b</sub> =0.14	Maximum	0.37940	0.32910	0.30340	0.32620	0.30120
	Minimum	0.29740	0.26780	0.22920	0.26190	0.21710
	Average	0.34005	0.29167	0.25811	0.28658	0.24907
k <sub>w</sub> =0.20 , k <sub>b</sub> =0.14	Maximum	0.28020	0.21100	0.18560	0.21090	0.19480
	Minimum	0.21210	0.16950	0.12350	0.16170	0.13000
	Average	0.24467	0.18533	0.14656	0.18051	0.15543
C <sub>HW</sub> =100		Entrance	Middle	Middle	Middle	Dead End
		JC-0629	JC-0051	JC-0287	JC-0572	JC-0027
k <sub>w</sub> =0.00 , k <sub>b</sub> =0.07	Maximum	0.49060	0.48660	0.48510	0.48630	0.48130
	Minimum	0.39890	0.40250	0.39730	0.40150	0.39410
	Average	0.45862	0.45388	0.45037	0.45281	0.43842
k <sub>w</sub> =0.08 , k <sub>b</sub> =0.07	Maximum	0.38700	0.34020	0.30540	0.33530	0.31160
	Minimum	0.29940	0.27190	0.23250	0.26540	0.22320
	Average	0.34634	0.30073	0.26391	0.29484	0.25934
k <sub>w</sub> =0.20 , k <sub>b</sub> =0.07	Maximum	0.28670	0.21850	0.19270	0.21820	0.20030
	Minimum	0.21080	0.16980	0.12660	0.16520	0.13790
	Average	0.24929	0.19087	0.15171	0.18519	0.16285
k <sub>w</sub> =0.00 , k <sub>b</sub> =0.14	Maximum	0.48130	0.47370	0.47070	0.47330	0.46710
	Minimum	0.33410	0.34070	0.33110	0.33800	0.32700
	Average	0.42917	0.42083	0.41480	0.41944	0.39527
k <sub>w</sub> =0.08 , k <sub>b</sub> =0.14	Maximum	0.37970	0.32740	0.30470	0.32610	0.30060
	Minimum	0.25100	0.23070	0.19670	0.22320	0.19480
	Average	0.32539	0.27836	0.24355	0.27285	0.23447
k <sub>w</sub> =0.20 , k <sub>b</sub> =0.14	Maximum	0.28040	0.21800	0.18870	0.21060	0.19350
	Minimum	0.17630	0.14570	0.10970	0.13910	0.11940
	Average	0.23428	0.17722	0.13800	0.17182	0.14709
C <sub>HW</sub> =145		Entrance	Middle	Middle	Middle	Dead End
		JC-0629	JC-0051	JC-0287	JC-0572	JC-0027
k <sub>w</sub> =0.00 , k <sub>b</sub> =0.07	Maximum	0.49060	0.48660	0.48510	0.48630	0.48120
	Minimum	0.38950	0.39460	0.38880	0.39290	0.38580
	Average	0.45582	0.45094	0.44749	0.44970	0.43493
k <sub>w</sub> =0.08 , k <sub>b</sub> =0.07	Maximum	0.38710	0.34020	0.30550	0.33530	0.31170
	Minimum	0.29250	0.26660	0.23080	0.25980	0.22090
	Average	0.34430	0.29896	0.26280	0.29292	0.25736
k <sub>w</sub> =0.20 , k <sub>b</sub> =0.07	Maximum	0.28620	0.21910	0.19330	0.21890	0.20040
	Minimum	0.20600	0.16660	0.12390	0.16170	0.13640
	Average	0.24771	0.19004	0.15058	0.18430	0.16111
k <sub>w</sub> =0.00 , k <sub>b</sub> =0.14	Maximum	0.48130	0.47380	0.47080	0.47330	0.46710
	Minimum	0.32180	0.32850	0.31920	0.32650	0.31560
	Average	0.42556	0.41681	0.41105	0.41610	0.39029
k <sub>w</sub> =0.08 , k <sub>b</sub> =0.14	Maximum	0.37980	0.33210	0.30060	0.32450	0.30190
	Minimum	0.24190	0.22370	0.19280	0.21580	0.18560
	Average	0.32274	0.27612	0.24142	0.27039	0.23090
k <sub>w</sub> =0.20 , k <sub>b</sub> =0.14	Maximum	0.28050	0.21760	0.18870	0.20900	0.19320
	Minimum	0.16990	0.14120	0.10630	0.13460	0.11650
	Average	0.23210	0.17597	0.13774	0.17016	0.14550

Both figures and summary table show the East Çiğdemtepe chlorine concentration results of the last 24 hours and the system is operating with single pump operation. As mentioned in Section 6.3.1 for Yayla and in Section 6.3.2 for North Sancaktepe, results of last 24 hours are given to give more realistic stable condition results. Also, DDC of the subzone is shown with black curve in graphs.

East Çiğdemtepe differs from other studied subzones because while the pump station becomes incapable, the subzone starts to receive water from the tank. Because of the reason that chlorine concentration in the tank is different from the pumped water (0.50 mg/l), chlorine concentrations at junctions are influenced by this different concentration value. Differently from Yayla and North Sancaktepe, the subzone is feeded by the tank and for every different Hazen-Williams roughness coefficient  $C_{HW}$ , different tank levels occur because of different head losses in the system. This causes concentrations to differ by changing the roughness coefficient  $C_{HW}$ . Because, while tank levels are different for different  $C_{HW}$  values, the feeding velocity becomes different and therefore, rate of the chlorine decay is influenced. By analysing the Figure 6.20a ( $k_b = 0.07$  1/day and  $k_w = 0.00$  m/day), Figure 6.20b ( $k_b = 0.07$  1/day and  $k_w = 0.08$  m/day), Figure 6.20c ( $k_b = 0.07$  1/day and  $k_w = 0.20$  m/day), Figure 6.20d ( $k_b = 0.14$  1/day and  $k_w = 0.00$  m/day), Figure 6.20e ( $k_b = 0.14$  1/day and  $k_w = 0.80$  m/day) and Figure 6.20f ( $k_b = 0.14$  1/day and  $k_w = 0.20$  m/day); it can be said that for a fixed  $k_b$  and  $k_w$  combination, changing the roughness coefficient  $C_{HW}$  is effective on chlorine concentrations this time. For example in Figure 6.20a, a fixed  $k_w$  value was used as 0.00 m/day and a fixed  $k_b$  value was used as 0.07 1/day. By analysing the three different graphs it is seen that three different  $C_{HW}$  values give different chlorine concentrations. Especially while looking at the curve of the entrance junction JC-0629, until tank starts to feed the subzone at 370th hour (10.00 am.) concentration are same for three different  $C_{HW}$  coefficient. However, after the hour 10.00 am., while tank starts to feed the subzone, chlorine concentrations

are influenced by different roughness coefficients. As it is seen, minimum concentrations are different. While using  $C_{HW}$  coefficient as 55, minimum concentration is 0.44560 mg/l; while using  $C_{HW}$  coefficient as 100, minimum concentration is 0.39890 mg/l and while using  $C_{HW}$  coefficient as 145, minimum concentration is 0.38950 mg/l at entrance junction JC-0629. It can be said that for this study specifically, increasing Hazen-Williams roughness coefficient  $C_{HW}$  decreases chlorine concentrations similarly as wall decay coefficient  $k_w$  and bulk decay coefficient  $k_b$ . It must be pointed that this inverse behavior is case specific and may be different for an other WDN model.

The effect of travel time can clearly be seen again in this subzone. At entrance junction, fresh water comes from the pump station and chlorine concentrations are at maximum levels as similarly in Yayla and North Sancaktepe. The shifting in the occurrence time of minimum concentrations can be seen clearly again from entrance junction to dead end junction. At entrance junction, minimum concentration occurs at midday morning hours, however it occurs at later hours while going to middle or dead end junctions. Because of the reason that when demands increase in morning hours, dead end junctions are the latest influenced junctions again because of the long way of water movement. Although demands increase in morning hours, tank comes into play and changes the expected behaviour.

In Figure 6.21a, 6.21b, 6.21c, 6.21d and 6.21e show how chlorine concentrations are influenced by changing system parameters Hazen-Williams roughness coefficient  $C_{HW}$ , bulk decay coefficient  $k_b$  and wall decay coefficient  $k_w$  at every studied junction one by one. By analysing this type of graphs, it can be understood more easily that which parameter is the most effective one.

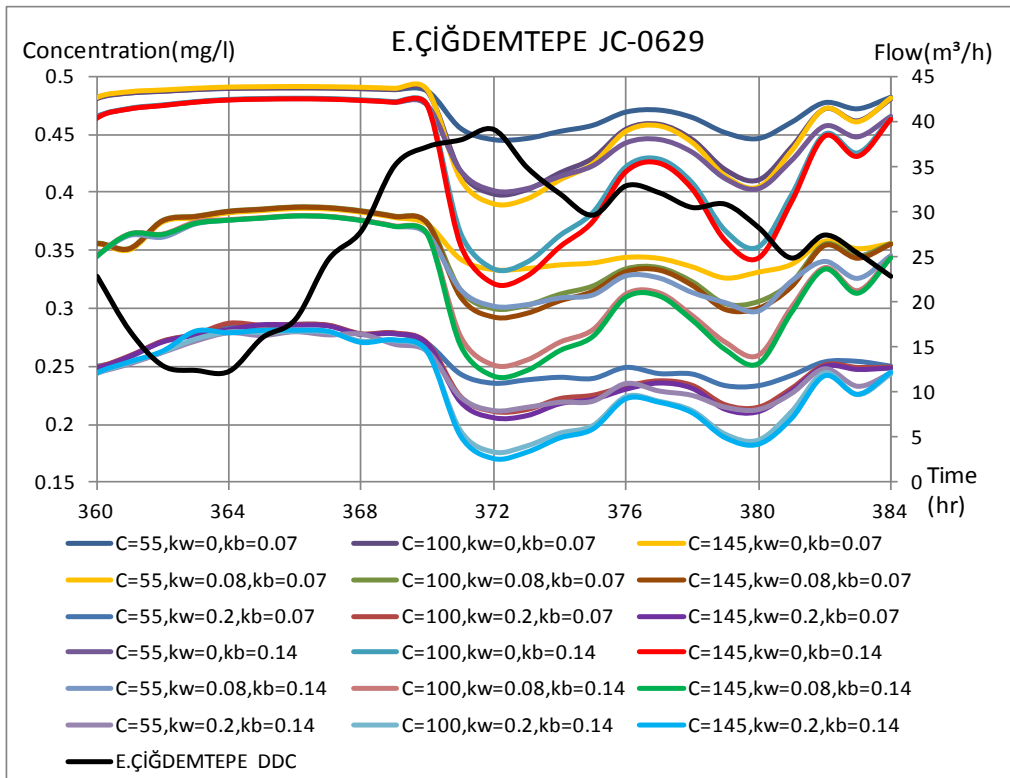


Figure 6.21a Concentration Levels for Different Cases at JC-0629

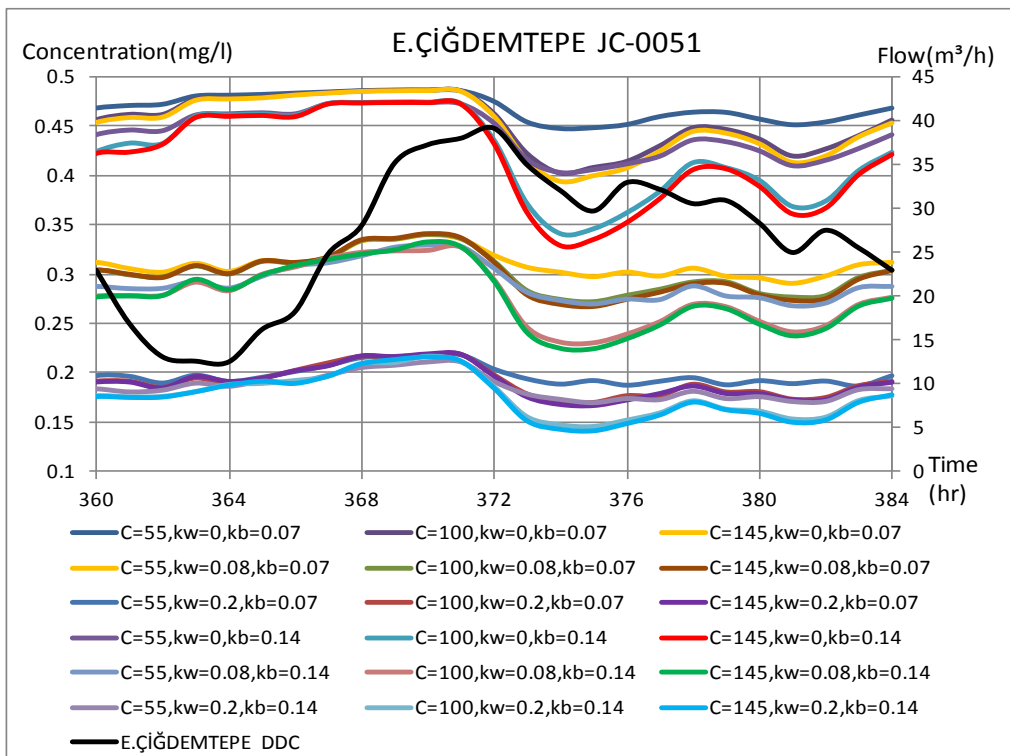


Figure 6.21b Concentration Levels for Different Cases at JC-0051

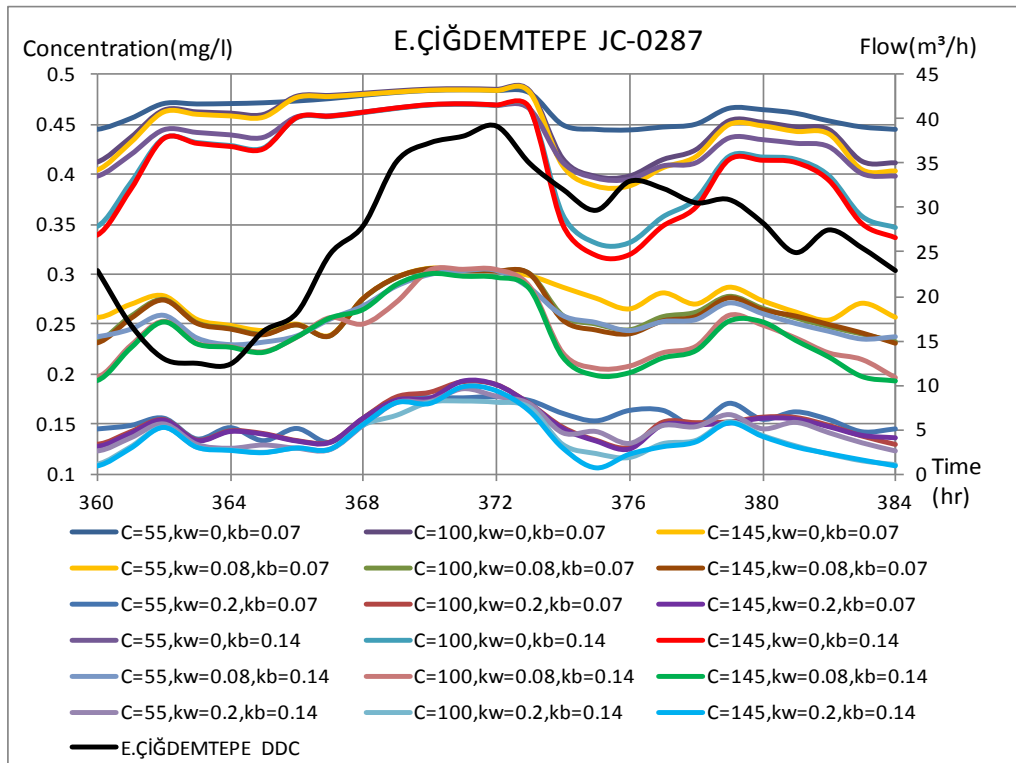


Figure 6.21c Concentration Levels for Different Cases at JC-0287

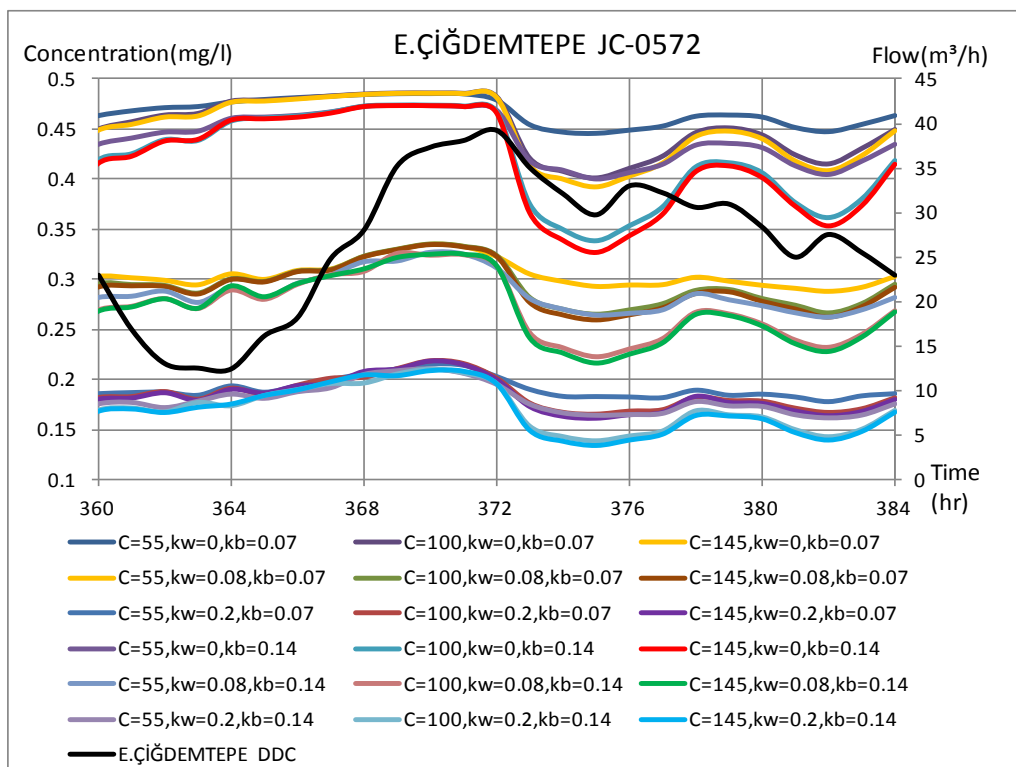


Figure 6.21d Concentration Levels for Different Cases at JC-0572

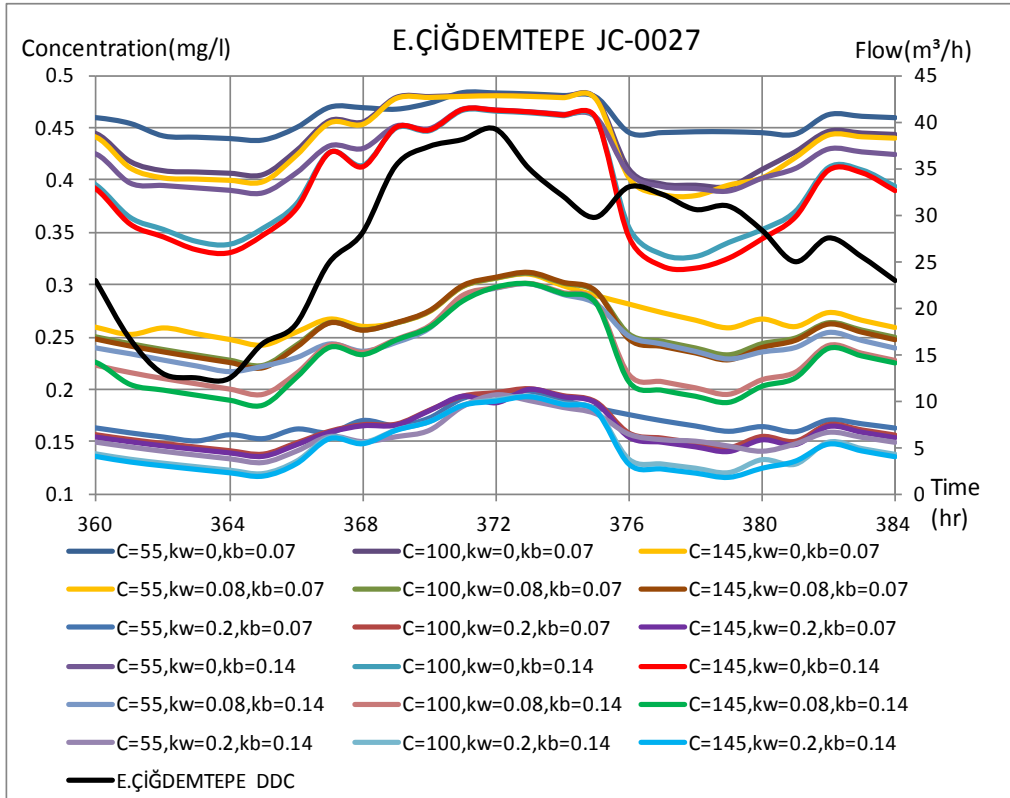


Figure 6.21e Concentration Levels for Different Cases at JC-0027

In the Figures from 6.21a to 6.21e, effects of differences in all system parameters;  $C_{HW}$ ,  $k_w$  and  $k_b$  on chlorine concentrations can be seen clearly differently from Yayla and North Sancaktepe. Especially while looking at the Figure 6.21a, at entrance junction JC-0629, until tank starts to feed the subzone at 370th hour (10.00 am.), curves are overlapped for different  $C_{HW}$  coefficients again similarly in Yayla and North Sancaktepe graphs. However, at 10.00 am., the pump station becomes incapable because of the increased demands and the tank becomes a part of the activity. Different  $C_{HW}$  coefficients give different results because of the different hydraulic behaviours and curves are not overlapped anymore. 18 different scenarios can be analysed clearly during the time that both the pump station and the tank feed the subzone. Also, DDC of the subzone is shown with black curve in graphs. It can be said that briefly, when water is taken only from the pump station continuously, Hazen-Williams



roughness coefficient  $C_{HW}$  does not affect chlorine concentrations at junctions. However, if tank or another source is supplying water, chlorine concentrations are affected from the concentration level and feeding behavior of this water supply.

For a specific Hazen-Williams roughness coefficient  $C_{HW}$  and bulk decay coefficient  $k_b$ , increasing the wall decay coefficient  $k_w$  decreases chlorine concentrations. It is also seen that if Hazen-Williams roughness coefficient  $C_{HW}$  and wall decay coefficient  $k_w$  are fixed as specific values; increasing bulk decay coefficient  $k_b$  decreases chlorine concentrations similarly as  $k_w$ . In this study, similarly as  $k_w$  and  $k_b$ , increasing roughness coefficient  $C_{HW}$  decreases concentrations but this inverse behaviour is case specific and it may be different for a different WDN. Still, the most effective parameter is  $k_w$  comparing to  $k_b$  and  $C_{HW}$ . In the Figures from 6.21a to 6.21e, the minimum concentration values are seen in the scenario that  $k_w$  is used as 0.20 m/day (older pipes),  $k_b$  is used as 0.14 1/day (summer season) and  $C_{HW}$  is used as 145 (case specific). Because of this reason, while trying to specify a critical scenario, these values will be used for this study.

All these studies were performed for the demands that were taken from a previous study of Şendil (2013) and leakage effects were not taken into consideration.

#### **6.4 Effect of Leakage Amount on Water Quality Modeling**

In a WDN, relation between amount of leakage and total distributed water, affects the efficiency of the whole system. Depending on the amount of leakage in a WDN, chlorine concentration changes through pipes and at junctions for the same network. Distribution systems are routinely monitored for several water quality parameters such as Chlorine, Ph, and turbidity. Water loss due to any leaks in the system impacts the flow characteristics and therefore leaks have an impact on the water quality (Kumar et al.,2010).

N8.3 WDN which has six different patterns, was studied for analysing leakage effect with only DMA's method. In this part of the study, values of Hazen-Williams roughness coefficient  $C_{HW}$ , bulk decay coefficient  $k_b$  and wall decay coefficient  $k_w$  were used as studied in a previous thesis study prepared by Nadiroğlu (2013). These values are the default values of WaterCAD. Pump operation was also used as in that study as Single Pump Operation Continuous for 24 hours.

These pump operation was studied for the N8.3 WDN, assuming three different leakage case. These are,

1. Distributing the total demand with no difference in the amount of leakage.
2. Distributing the total demand assuming zero leakage in any patterns.
3. Distributing the total demand by increasing the leakage in all patterns as  $Q_{min}$ .

The aim of this practice was to analyse the system without leakage and with increased leakage and therefore, to see how the chlorine concentrations differ by adding an amount of leakage to the system. To manage this, for three subzones, by using total demand, by a zero leakage DDC and increased leakage DDC were created. By subtracting the minimum demand value from all demand values of 24 hours, a zero leakage DDC is prepared. By adding the minimum demand value to all demand values of 24 hours, an increased leakage DDC was prepared for six different subzones of N8.3 WDN.

Yayla district is used to show this application as an example in Table 6.8 and Figure 6.22.

Table 6.8 DDC Data for Different Leakage Amounts in Yayla

Time (h)	Flow (m <sup>3</sup> /h)	Flow-Qmin (m <sup>3</sup> /h)	Flow+Qmin (m <sup>3</sup> /h)
0	27.050	13.650	40.450
1	18.000	4.600	31.400
2	14.450	1.050	27.850
3	13.800	0.400	27.200
4	<b>13.400</b>	0.000	26.800
5	17.950	4.550	31.350
6	32.700	19.300	46.100
7	43.950	30.550	57.350
8	36.650	23.250	50.050
9	41.700	28.300	55.100
10	46.550	33.150	59.950
11	49.750	36.350	63.150
12	51.100	37.700	64.500
13	48.000	34.600	61.400
14	44.900	31.500	58.300
15	42.300	28.900	55.700
16	36.050	22.650	49.450
17	38.950	25.550	52.350
18	45.600	32.200	59.000
19	45.900	32.500	59.300
20	43.100	29.700	56.500
21	37.600	24.200	51.000
22	37.900	24.500	51.300
23	35.200	21.800	48.600
24	27.050	13.650	40.450
<b>AVERAGE</b>	<b>35.940</b>	<b>22.540</b>	<b>49.340</b>

In this table, blue column shows the data for normal demand condition of Yayla district, green column shows the data for subtracted demand condition of Yayla district for without leakage condition and lastly pink column shows the data for increased demand condition of Yayla district for increased leakage condition. The cell which is shown with red fill is minimum demand of the subzone in 4th hour. When this minimum value is subtracted from all hour's

demand values, zero leakage DDC is obtained and when this minimum value is added to all hour's demand values, increased leakage DDC is obtained as below in the Figure 6.22. Average demand value is distributed to the system, then new demands for without leakage and increased leakage cases are obtained. This process is applied to all six subzones and three DDC's are obtained for these subzones.

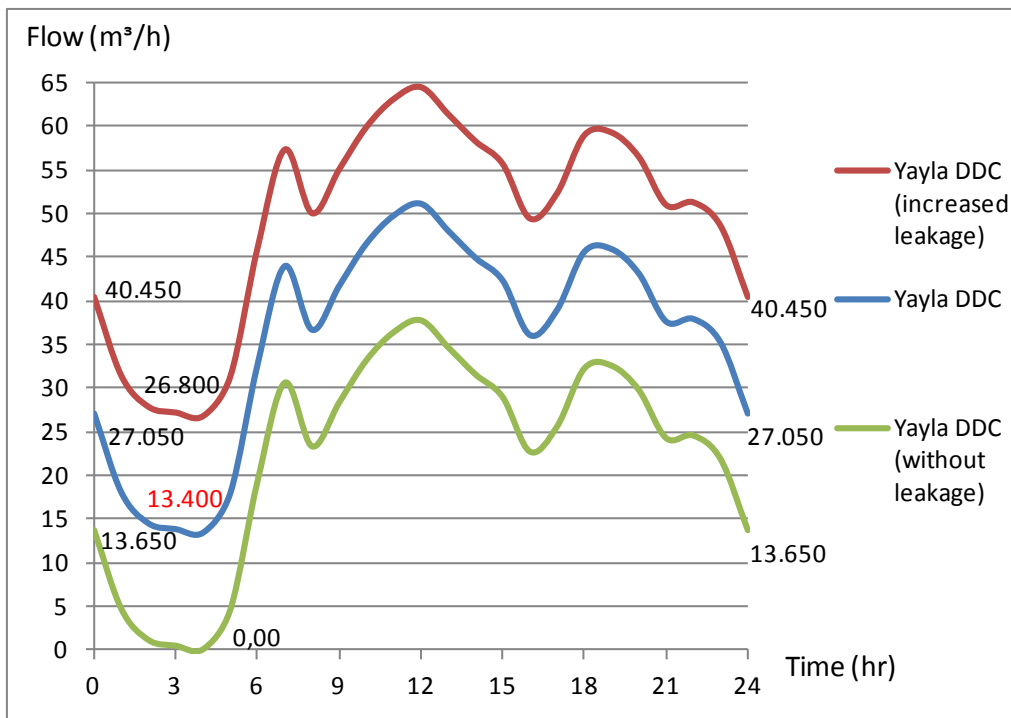


Figure 6.22 DDC's of Yayla District With Normal, Without Leakage and Increased Leakage Cases

Demand of the network were distributed by a demand distributor program written for a previous thesis study. While using this program, average of 24 hr DDC data is used and after that all demands are distributed by multiplying them with a different multiplier for every subzones and every 24 hours.

For normal condition, total demand were put into program as 35.94 m<sup>3</sup>/hr and for the new case, without leakage, 22.54 m<sup>3</sup>/hr were used and for the increased leakage case, 49.34 m<sup>3</sup>/hr were used to distribute new demands to WDN as it is seen in the Table 6.8. Also, new multipliers were used for all subzones by dividing demand values to average demand value to manage the distribution properly. This demand distribution is applied to all six subzones and therefore, water distribution system become totally without leakage condition or comparatively increased leakage condition due to their own demands. Figure 6.23 below shows a view of demand distributor program while applying the without leakage condition again for Yayla district.

	A	B	C	D	E F	G	H	I
1	<b>Label</b>	<b>Length (m)</b>	<b>Start Node</b>	<b>Stop Node</b>		<b>Total Demand =</b>		<b>22.54</b>
2	PP-0171	4.43	JC-0024	JC-0025				
3	PP-0177	4.9	JC-0032	JC-0033				
4	PP-0658	6.47	JC-0053	JC-0054				
5	PP-0647	6.64	JC-0055	JC-0056				
6	PP-0663	7.94	JC-0071	JC-0072				
7	PP-0188	8.48	JC-0084	JC-0085				
8	PP-0193	8.73	JC-0093	JC-0094				
9	PP-0462	8.89	JC-0097	JC-0098				
10	PP-0463	8.96	JC-0099	JC-0094				
11	PP-0480	9.56	JC-0114	JC-0115				
12	PP-0794	9.62	JC-0118	JC-0119				
13	PP-0180	9.83	JC-0120	JC-0121				
14	PP-0179	9.84	JC-0122	JC-0123				
15	PP-0665	10.14	JC-0055	JC-0125				
16	PP-0143	11.45	JC-0140	JC-0141				
17	PP-0786	11.7	JC-0144	JC-0145				
18	PP-0477	12.14	JC-0149	JC-0150				
19	PP-0130	12.42	JC-0151	JC-0152				
20	PP-0137	12.58	JC-0115	JC-0154				
21	PP-0478	12.67	JC-0158	JC-0159				
22	PP-0452	13.06	JC-0162	JC-0163				
23	PP-0443	13.11	JC-0166	JC-0167				
24	PP-0134	13.35	JC-0171	JC-0172				
							<b>Total Length =</b>	
						<b># of Junction =</b>		<b>136</b>

Figure 6.23 View of Demand Distributor For Without Leakage Case – Yayla District

As it is seen in the figure pipes which are included in Yayla District are copied with their names, lengths, start nodes and stop nodes. After that, the program calculates by determining demands from meter pipe. Average demand of DMA is divided by total pipe length of DMA. Every pipe has a start node and a stop node. The demand distribution procedure was mentioned in previous sections of this chapter by using Half Pipe Length Method.

This practice was applied on the same three subzones for this study; Yayla, North Sancaktepe and East Çiğdemtepe. After distribution of demands for new cases (without leakage and increased leakage) to all six subzones, chlorine concentrations of the five junctions which were studied till now, were reanalysed. Explaining reminiscently, one of these junctions are at the entrance of the subzone, three of them in different parts of the subzone and one of them is at an end point.

As mentioned before, in this pump schedule, one of the three pumps at the pump station which manufactured by SUMAS was operating and the other two pumps at the station were closed. 1- SUMAS was in operation during all time, while the operating point of the pump was changing due to the fluctuations in daily demand.

#### ***6.4.1. Chlorine Concentrations for Different Leakage Amounts***

The junctions chosen for Yayla district is JC-0188 at the entrance; JC-0211, JC-0343, JC-0348 at middle points and JC-0291 at dead end point. These junctions were shown in previous chapter with Figure 6.8.

The junctions chosen for North Sancaktepe district is JC-0511 at the entrance; JC-0642, JC-0279, JC-0655 at middle points and JC-0691 at dead end point. These junctions were shown in previous chapter with Figure 6.9.

The junctions chosen for East Çiğdemtepe district is JC-0629 at the entrance; JC-0051, JC-0287, JC-0572 at middle points and JC-0027 at dead end point. These junctions were shown in previous chapter with Figure 6.10.

In the previous pages, how to distribute demands in normal, without leakage and increased leakage conditions were explained in detail. In this part, relation between chlorine concentration and time at chosen junctions are shown for these three conditions in last 24 hours operating with Single Pump Operation in figures below.

Figure 6.24 shows the results for different leakage amounts of Yayla District.

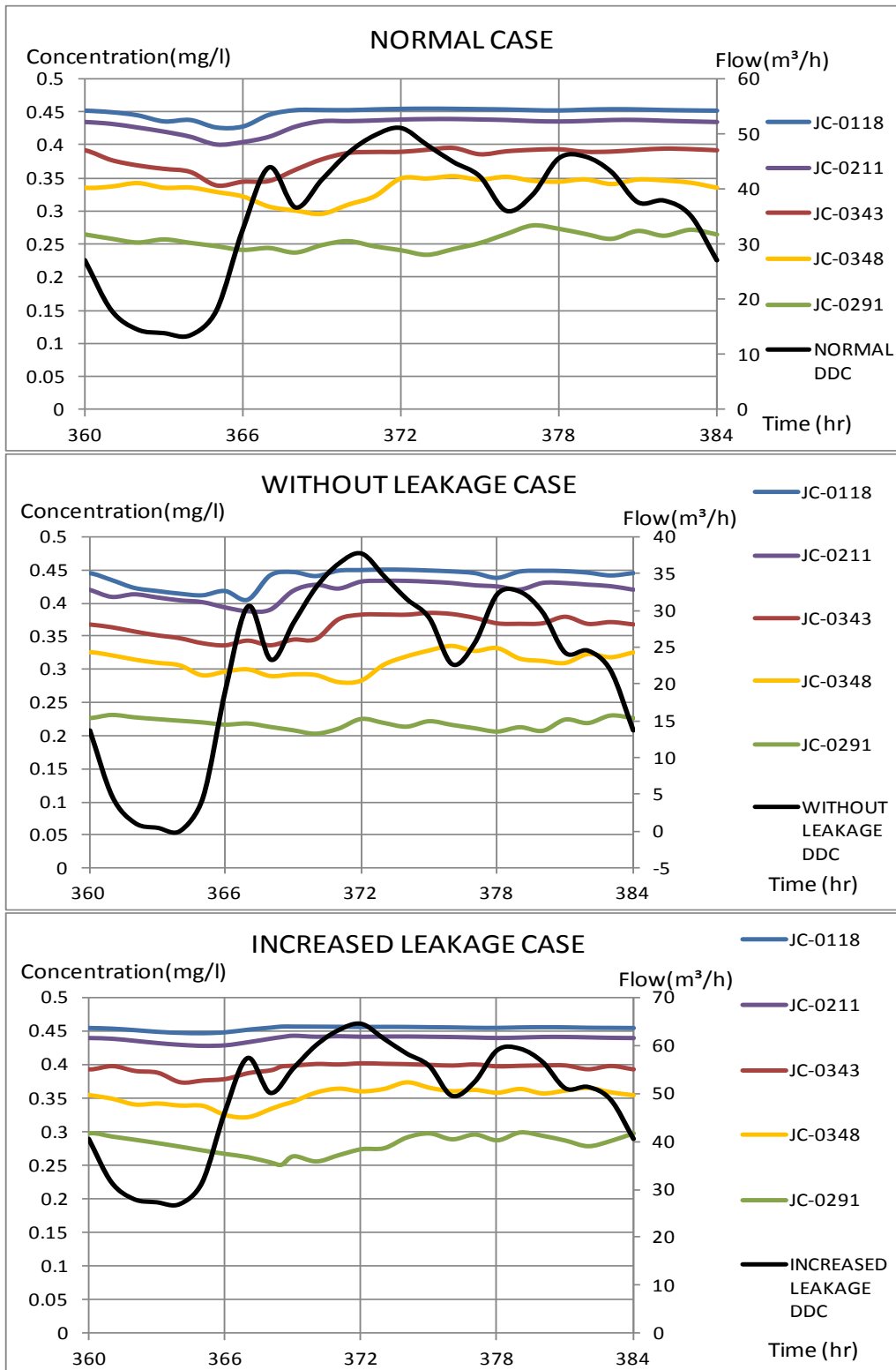


Figure 6.24 Chlorine Concentrations for Different Leakage Amounts in Yayla District (all studied junctions)



Demands are low at night hours and therefore until 06.00 am. At morning, chlorine concentrations decrease. After demands start to increase after 06.00 am. chlorine concentrations start to increase also and follow almost a straight line during the rest of the day at entrance junction JC-0118 for all cases because of the continuously coming fresh water, demand differences do not affect concentrations significantly. At middle junctions, concentrations differ depending on the condition of the junctions and give lower values compared to the entrance junctions. Dead end junction JC-0291 has minimum concentrations because of the fact that it is the most distant junction to the entrance. Concentration results give more zigzaggy graphs because of the time until water reaches to the end. Changing the leakage amount causes a concentration difference in a very small range at the entrance junction JC-0118. Concentration differences differ in a big range at dead junction JC-0291 by changing leakage amount. Table 6.9 shows a summary of the Yayla results.

Table 6.9 Summary of Chlorine Concentrations for Different Leakage Amounts in Yayla District

Yayla		CONCENTRATIONS (mg/l)				
$C_{HW}=130, k_b=0.10, k_w=0.08$		Entrance	Middle	Middle	Middle	Dead End
		JC-0118	JC-0211	JC-0343	JC-0348	JC-0291
Without Leakage	Maximum	0.45140	0.43400	0.38420	0.33600	0.23670
	Minimum	0.40580	0.38810	0.33550	0.28060	0.20000
	Average	0.43881	0.41894	0.36273	0.31048	0.21873
Normal	Maximum	0.45420	0.43870	0.39640	0.35270	0.27830
	Minimum	0.42580	0.40030	0.33930	0.29600	0.23400
	Average	0.44829	0.42987	0.38037	0.33496	0.25496
Increased Leakage	Maximum	0.45600	0.44190	0.40200	0.37320	0.29810
	Minimum	0.44620	0.42690	0.37380	0.32140	0.25020
	Average	0.45335	0.43743	0.39449	0.35163	0.27860

As seen in and Table 6.9, last 48 hours concentration level is approximately at a level of 0.448 mg/l at entrance junction JC-0118. Chlorine concentrations of Yayla are in safe limits. This means that even at dead end junction, concentration level does not drop under the minimum limit 0.10 mg/l and at any junction does not exceed the maximum limit 0.50 mg/l for three different leakage amount cases.

Time of the minimum concentrations are again shifting from early morning hours (06.00 am.) at entrance junction, to later hours (09.00-13.00 am.) at dead end junctions. When demands increase in morning hours, dead end junctions are the latest influenced junctions because of the travel time effect. Because of the same reason, for three different amount of leakage, highest chlorine concentration differences are seen at dead end junction JC-0291.

Increasing leakage causes lower chlorine concentrations. While leakage amount is high, demands are satisfied more difficultly and because of this reason velocities increase to suffice for demands. High velocities cause lower chlorine decay and high chlorine concentrations occur. Inversely, while there is no leakage in the system, velocities decrease and high chlorine decay occurs. Therefore, chlorine concentrations decrease.

Figure 6.25 shows for North Sancaktepe District results for different leakage amounts.

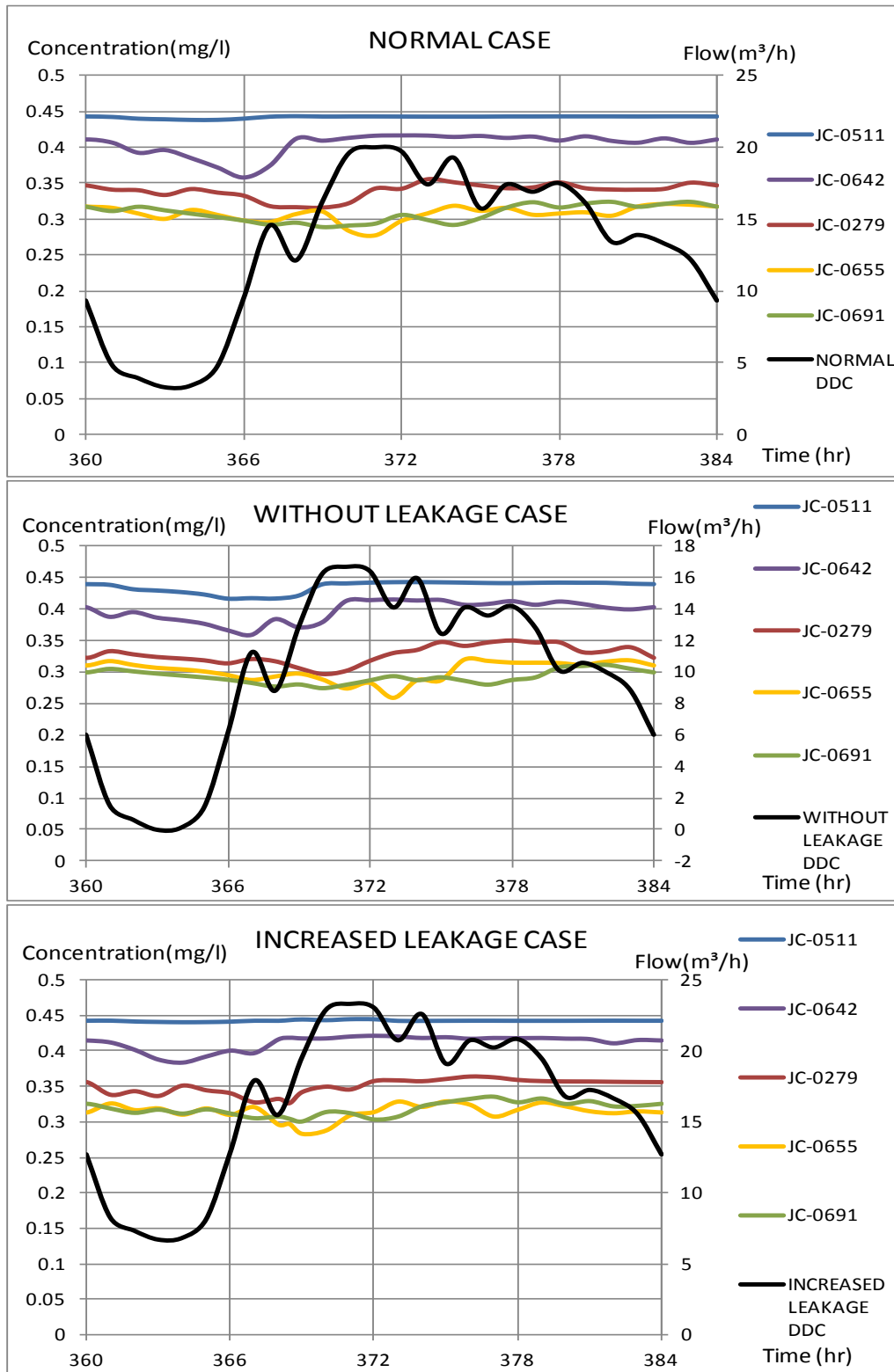


Figure 6.25 Chlorine Concentrations for Different Leakage Amounts in North Sancaktepe District (all studied junctions)

As seen in Figure 6.25 last 24 hours concentration level is approximately at a stable level at entrance junction JC-0511 for normal and increased leakage cases. The effect of differences in demands can be seen significantly for without leakage case only. Demands are low at night hours and therefore until 06.00 am. at morning, chlorine concentrations decrease. After demands start to increase after 06.00 am. chlorine concentrations start to increase also. At middle junctions JC-0642 and JC-0279, concentrations differ depending on the condition of the junctions and give lower values compared to the entrance junctions. At dead end junction JC-0691 minimum concentrations are expected. However, although JC-0655 is a middle junction, chlorine concentration differences of JC-0655 and JC-0691 are seen in the same range. Even higher values are seen at dead end junction JC-0691 than middle junction JC-0655 in some time intervals. The area that receives water to J-0655 has a loop and there are many junctions on the transport area. Therefore, it can be said that several transport and mixing reactions on the path to J-0655 causes unexpected concentrations. Time of the minimum concentrations are again shifting from early morning hours at entrance junction, to later hours at dead end junctions.

For these three cases, highest chlorine concentration differences are seen at JC-0655 and JC-691 but this difference is not as much as in Yayla. Chlorine concentrations are higher in increased leakage case and lower in without leakage case than normal case. They are still in safe limits as Yayla (between limit 0.10 mg/l - 0.50 mg/l). Table 6.10 shows the summary of North Sancaktepe results.

Table 6.10 Summary of Chlorine Concentrations for Different Leakage Amounts in North Sancaktepe District

North Sancaktepe		CONCENTRATIONS (mg/l)				
$C_{HW}=130, k_b=0.10, k_w=0.08$		Entrance	Middle	Middle	Middle	Dead End
		JC-0511	JC-0642	JC-0279	JC-0655	JC-0691
Without Leakage	Maximum	0.44250	0.41340	0.34980	0.32030	0.31080
	Minimum	0.41620	0.35770	0.29650	0.25940	0.27390
	Average	0.43486	0.39492	0.32771	0.30167	0.29148
Normal	Maximum	0.44290	0.41630	0.35630	0.32030	0.32370
	Minimum	0.43770	0.35780	0.31670	0.27670	0.28860
	Average	0.44165	0.40378	0.33991	0.30636	0.30756
Increased Leakage	Maximum	0.44510	0.42030	0.36280	0.32870	0.33600
	Minimum	0.44080	0.38260	0.32530	0.28290	0.30110
	Average	0.44286	0.41058	0.34843	0.31343	0.31836

As seen in and Table 6.10, last 48 hours concentration level is approximately at a level of 0.442 mg/l at entrance junction JC-0511. Chlorine concentrations of North Sancaktepe are in safe limits. This means that even at dead end junction, concentration level does not drop under the minimum limit 0.10 mg/l and at any junction does not exceed the maximum limit 0.50 mg/l for three different leakage amount cases. Time of the minimum concentrations are again shifting to later hours from entrance to dead end junctions because of the travel time effect.

Increasing leakage causes lower chlorine concentrations. While leakage amount is high, demands are satisfied more difficultly and because of this reason velocities increase to suffice for demands. High velocities cause lower chlorine decay and high chlorine concentrations occur. Inversely, while there is no leakage in the system, velocities decrease and high chlorine decay occurs. Therefore, chlorine concentrations decrease. Figure 6.26 shows the concentration vs. time graphs for East Çiğdemtepe District.

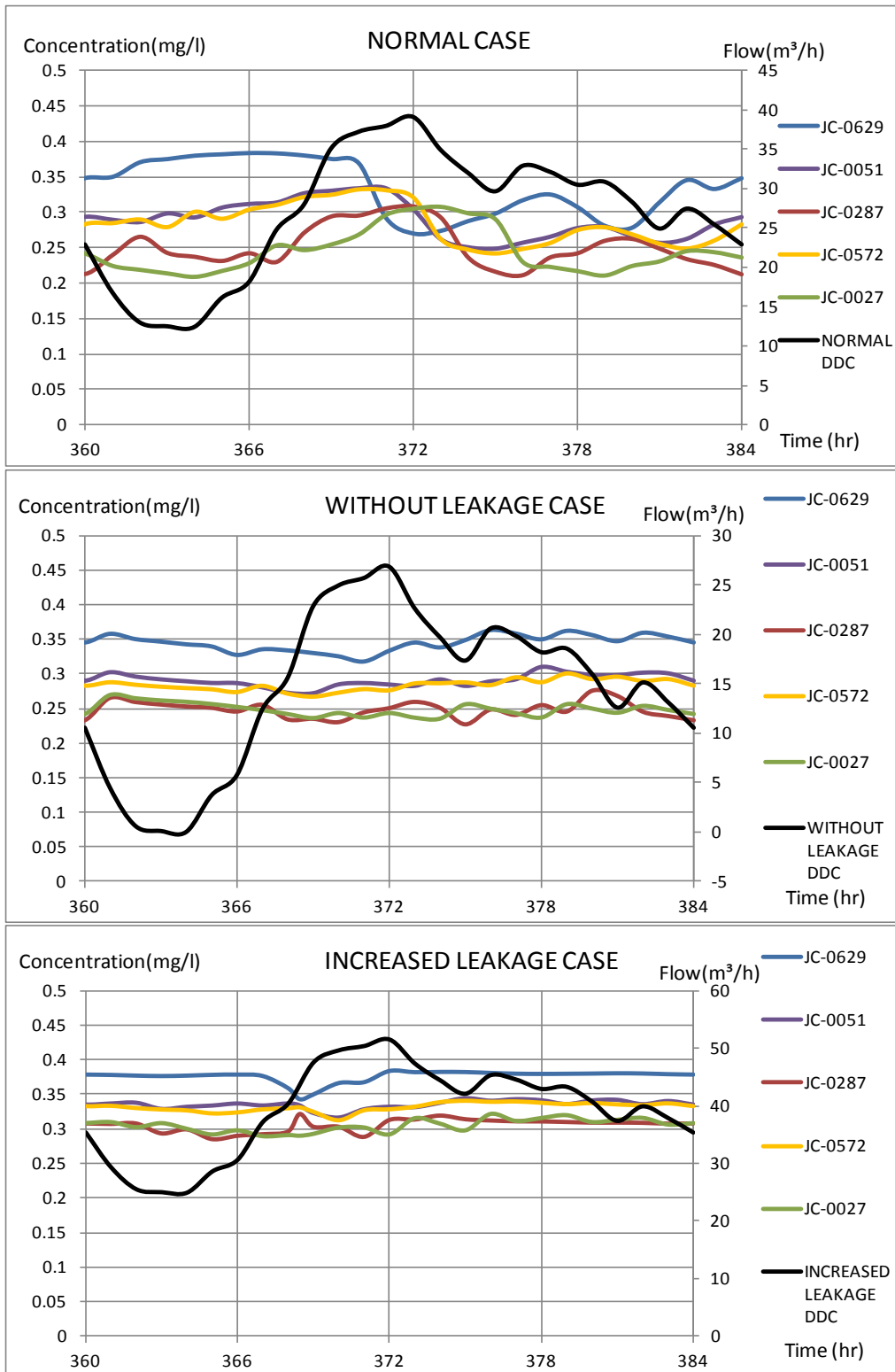


Figure 6.26 Chlorine Concentrations for Different Leakage Amounts in East Çiğdemtepe District (all studied junctions)

As it is seen in Figure 6.26, last 24 hours concentration level is very changeable at all junctions in East Çiğdemtepe. Receiving water from both the pump station and the tank causes these chlorine concentration fluctuations. Although demands are low at night hours, around 06.00 am. at morning, chlorine concentrations are at maximum levels oppositely to Yayla and North Sancaktepe. After demands start to increase after 06.00 am. chlorine concentrations are expected to increase. However, until 12.00 am., they decrease and at 12.00 am. minimum values are seen at the entrance junction JC-0629 for normal case. Examining the without leakage and increased leakage cases, chlorine concentration differences during 24 hours, are not seen as high as in normal case at any junction. While operating increased leakage case, concentrations of the entrance junction JC-0629 follow almost a straight line except the hours between 07.00 am.-12.00 am. Middle junctions and dead end junction behave similarly and a significant difference can not be seen during the process. This is because of the condition of these junctions as mentioned in North Sancaktepe. Chlorine concentrations are also in safe limits although East Çiğdemtepe gave the most irregular results in the first 18 scenarios.

Time of the minimum concentrations are shifting from midday hours (12.00 pm.) at entrance junction, to evening hours (07.00 pm.) at dead end junctions. Dead end junctions are the latest influenced junctions because of the long way of water movement. Therefore, minimum levels are shifted to later hours. Table 6.11 shows the summary of the results for East Çiğdemtepe.

Table 6.11 Summary of Chlorine Concentrations for Different Leakage Amounts in East Çiğdemtepe District

East Çiğdemtepe		CONCENTRATIONS (mg/l)				
$C_{HW}=130, k_b=0.10, k_w=0.08$		Entrance	Middle	Middle	Middle	Dead End
		JC-0629	JC-0051	JC-0287	JC-0572	JC-0027
Without Leakage	Maximum	0.36270	0.30970	0.27530	0.30070	0.26860
	Minimum	0.31750	0.27200	0.22660	0.26730	0.23440
	Average	0.34402	0.29043	0.24816	0.28405	0.24785
Normal	Maximum	0.38390	0.33570	0.31290	0.33130	0.30810
	Minimum	0.27010	0.24810	0.21180	0.24080	0.20450
	Average	0.33454	0.28878	0.25239	0.28319	0.24583
Increased Leakage	Maximum	0.38410	0.34510	0.32180	0.34130	0.32090
	Minimum	0.34320	0.31700	0.28530	0.31300	0.28870
	Average	0.37539	0.33584	0.30555	0.33206	0.30380

#### 6.4.2 Chlorine Concentrations Differences at Studied Junctions One by One

Figure 6.27 shows the chlorine concentration differences at studied junctions by changing the leakage amount one by one junction for Yayla District. Black curves show results for the normal demand distribution case. Red curves show the results for without leakage case in which the demands are distributed by subtracting  $Q_{min}$  value from all hours flow values. Green curves show the results for increased leakage case in which the demands are distributed by adding  $Q_{min}$  value to all hours flow values.

Figure 6.28 shows the chlorine concentration differences at studied junctions by changing the leakage amount one by one junction for North Sancaktepe District.

Figure 6.29 shows the chlorine concentration differences at studied junctions by changing the leakage amount one by one junction for East Çiğdemtepe District.



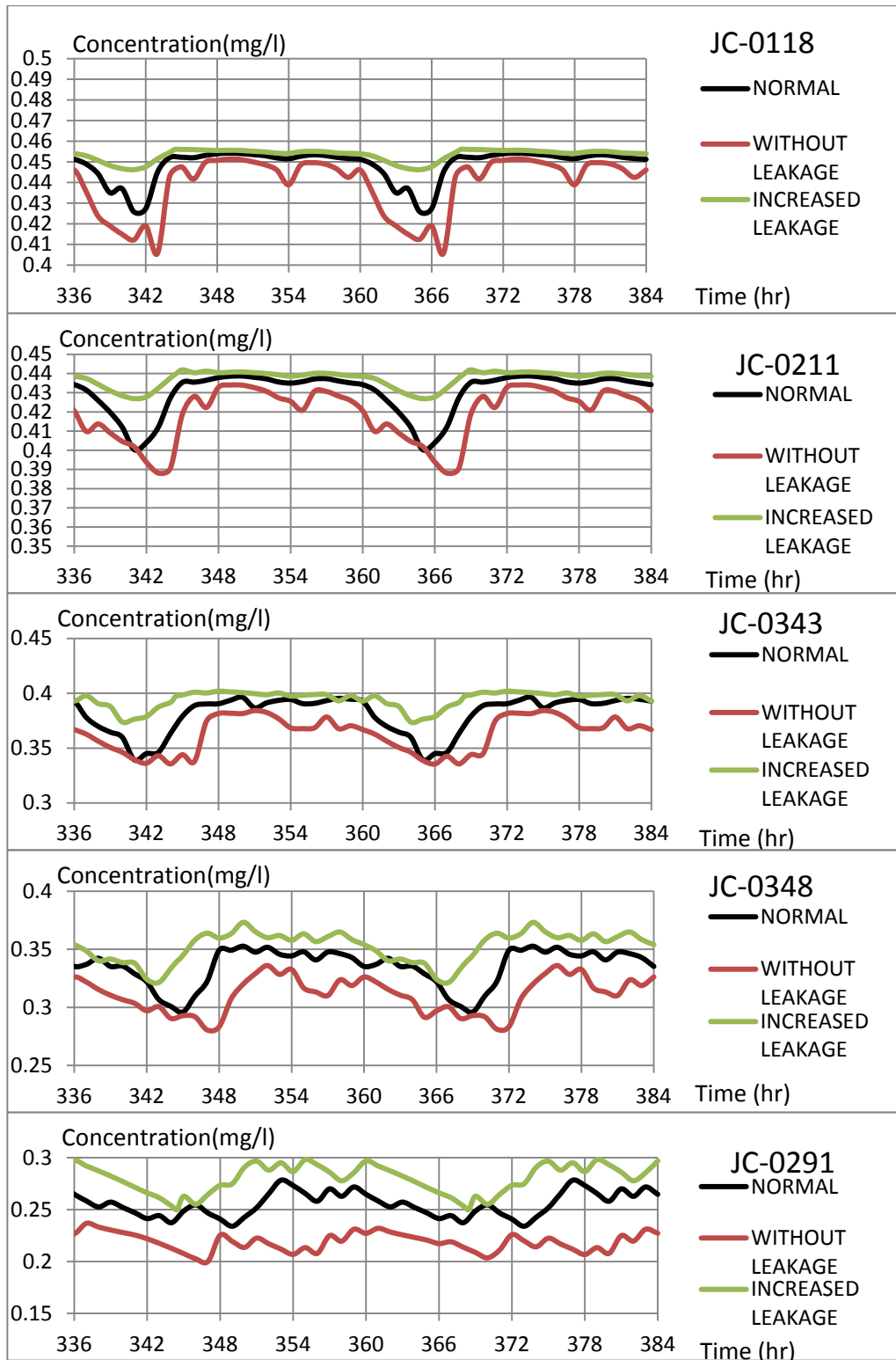


Figure 6.27 Chlorine Concentration Differences for Different Leakage Amounts at Studied Junctions One by One in Yayla District

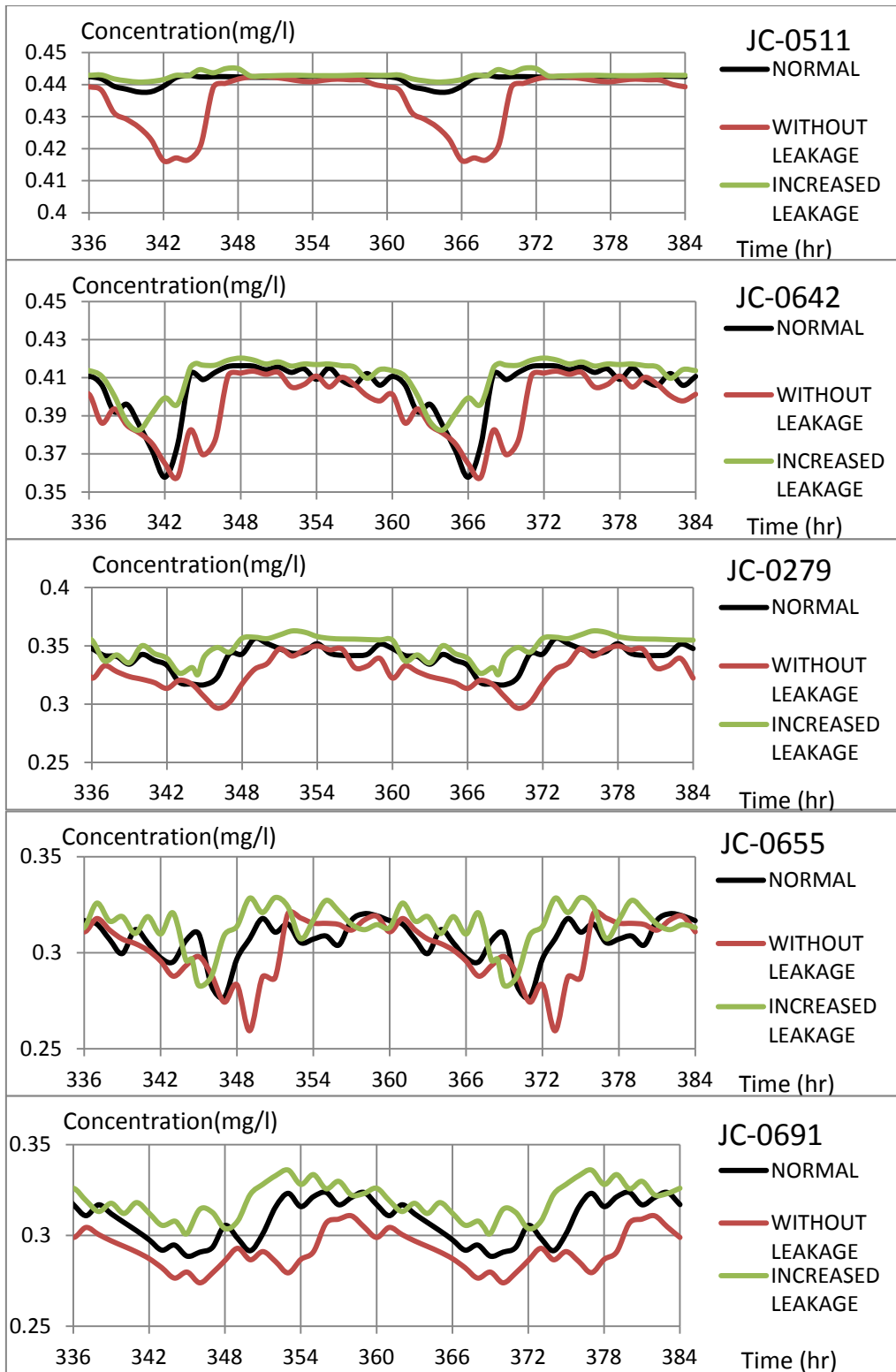


Figure 6.28 Chlorine Concentration Differences for Different Leakage Amounts at Studied Junctions One by One in North Sancaktepe District

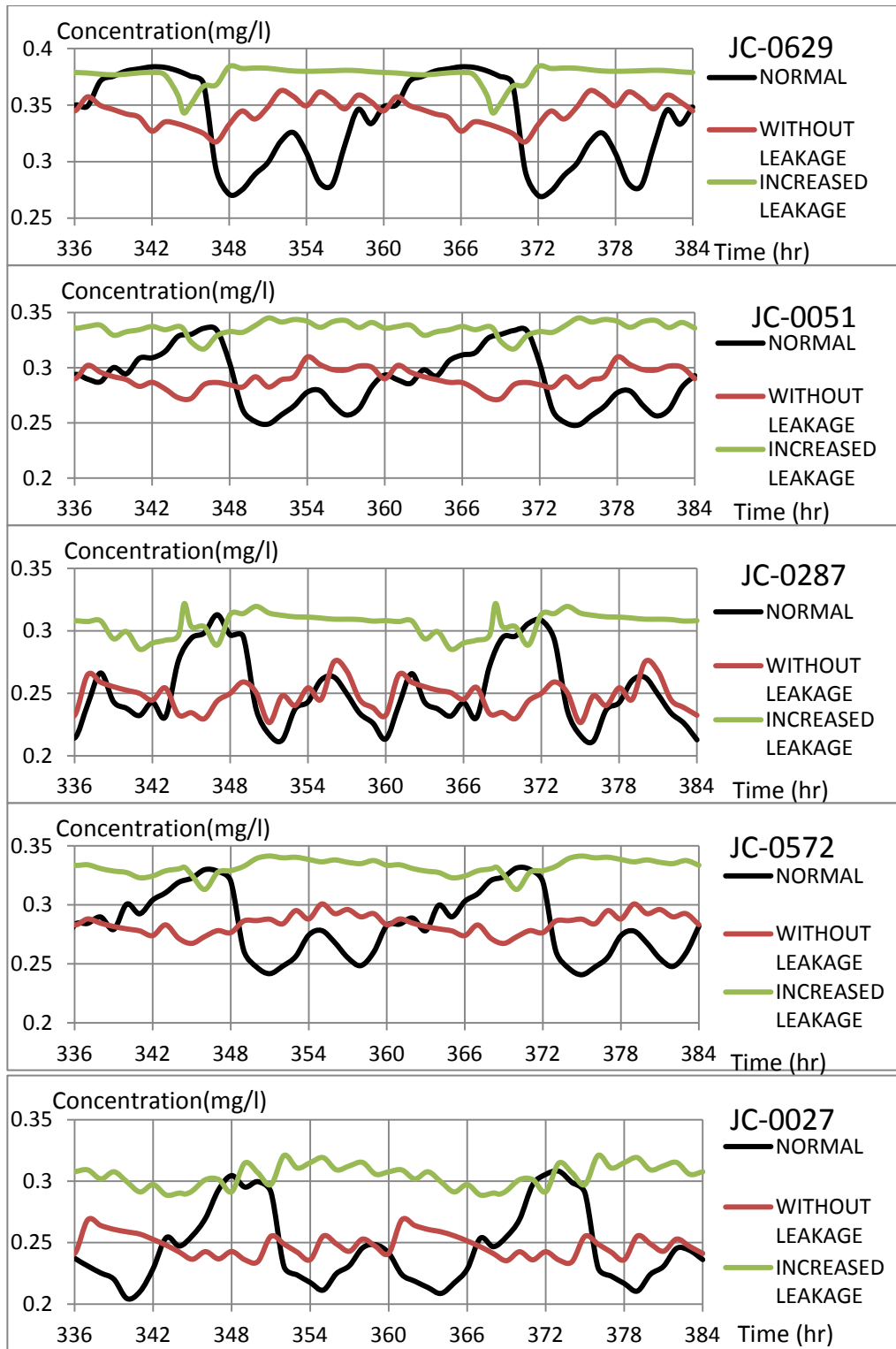


Figure 6.29 Chlorine Concentration Differences for Different Leakage Amounts at Studied Junctions One by One in East Çiğdemtepe District

As it is seen in the three figures below, in Yayla and North Sancaktepe districts, chlorine concentrations of normal case stay between without leakage case and increased leakage case concentrations as expected. This is because of the fact that these districts take water from only the pump station and changing the leakage amount does not change pumping schedule or behaviour. However, in East Çiğdemtepe, chlorine concentrations give irregular plots because, when pump is not sufficient for demands, tank gives water to this district, therefore chlorine concentrations are affected by tank's chlorine concentration also. Curves give irregular results than expected.

Demand distributions were applied for all six subzones of N8.3 and only three of them were analysed in this study. Amount of leakage and total volume of water was determined for all six subzones with Single Pump Operation. Total and leakage amount of water are shown in Figure 6.30 and the results are given in Table 6.12 below.

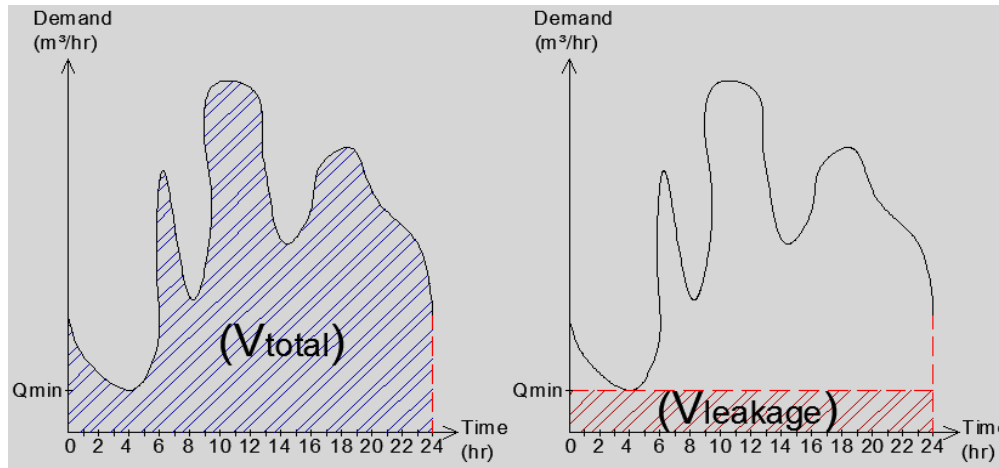


Figure 6.30 Total and Leakage Amount of Water

The area under demand vs. time graph gives the total water amount and Table 6.12 is prepared by using total water amount and leakage amount for all

subzones. Ratio of leakage to total water amount in the subzones are given in this table

Table 6.12 Ratios of Leakage Amounts to Total Water Amounts for Normal and Increased Leakage Cases

DISTRICT	Vleakage (m <sup>3</sup> )	Vtotal (m <sup>3</sup> )	Vleakage/Vtotal (%)
YAYLA	321.599	864.310	37.21
N.SANCAKTEPE	79.902	314.509	25.41
S.SANCAKTEPE	204.480	411.374	49.71
ŞEHİT KUBİLAY	592.800	1223.831	48.44
E.ÇİĞDEMTEPE	297.562	643.234	46.26
W.ÇİĞDEMTEPE	381.599	789.793	48.32
DISTRICT	Vleakage(+) (m <sup>3</sup> )	Vtotal(+) (m <sup>3</sup> )	Vleakage/Vtotal (%)
YAYLA	643.199	1185.911	54.24
N.SANCAKTEPE	159.822	394.426	40.52
S.SANCAKTEPE	408.960	615.854	66.41
ŞEHİT KUBİLAY	1185.600	1816.631	65.26
E.ÇİĞDEMTEPE	595.138	940.810	63.26
W.ÇİĞDEMTEPE	763.199	1171.394	65.15

Examining the Table 6.12, it can be said that while South Sancaktepe, Şehit Kubilay, East Çiğdemtepe and West Çiğdemtepe are influenced by increasing the leakage amount almost at a similar ratio. Yayla is under a high influence than these ones. However, North Sancaktepe is influenced mostly by these increment. The highest ratio is seen in South Sancaktepe district both for normal and increased leakage cases.

### 6.5 The Most Critical Case Study

Until this part of the study, system was performed for three different Hazen-Williams roughness coefficient  $C_{HW}$ , three different wall decay coefficient  $k_w$  and two different bulk decay coefficient  $k_b$  values. In addition to these, using the default Hazen-Williams roughness coefficient  $C_{HW}$ , wall decay coefficient  $k_w$  and bulk decay coefficient  $k_b$  of WaterCAD, it is assumed that there is no

leakage in the system, and then it is assumed that the amount of leakage is bigger than present amount. How water quality is influenced by these differences in leakage amount was analysed. As it is seen in figures and summary tables of these runs, chlorine concentrations are always in safe limits between 0.10 mg/l and 0.50 mg/l.

After all these studies, an other beneficial study is performed to obtain the most critical cases with summer and winter demands by using the system parameter values which give the minimum concentrations during this study. In accordance with this purpose, examining the figures and summaries of chlorine concentrations showed that, using wall decay coefficient  $k_w$  as 0.20 m/day (system pipes are old) gives minimum levels of concentrations. Yayla and North Sancaktepe districts receive water only from the pump station. Changing Hazen-Williams roughness coefficient  $C_{HW}$  did not affect pumping behaviour and it only affected hydraulic properties, not chlorine concentrations. Therefore, to select the most critical roughness coefficient, only results of East Çiğdemtepe may be available. Because of the reason that the minimum chlorine concentrations were seen by using Hazen-Williams coefficient  $C_{HW}$  as 145, this value is used for obtaining the most critical case study. In previous parts of this chapter, East Çiğdemtepe showed the most irregular behaviour. Therefore, for the most critical case, examining the results of East Çiğdemtepe is suitable.

It was also seen that, when studying for three different leakage amount, minimum concentrations were seen for without leakage case. Because of this reason, for the most critical case, assuming the system without leakage is suitable.

To see the most critical chlorine concentration for summer season, demands are increased by multiplying them by 1.3 because of the increased water requirement. These new demands are used to obtain a new DDC for summer,

and subtracting the  $Q_{min}$  value from all hours demand, demands for without leakage case are obtained as explained in previous part. Bulk decay coefficient for summer season is used as 0.14 1/day as known. The same procedure is applied by multiplying demands by 0.7 for winter season. After studying on the new DDC for winter, using bulk decay coefficient as 0.07 1/day the concentration vs. time graphs are obtained for without leakage case. Figure 6.31 shows results for winter and Figure 6.32 shows results for summer. DDC of the subzone for different seasons are shown with black curves in graphs.

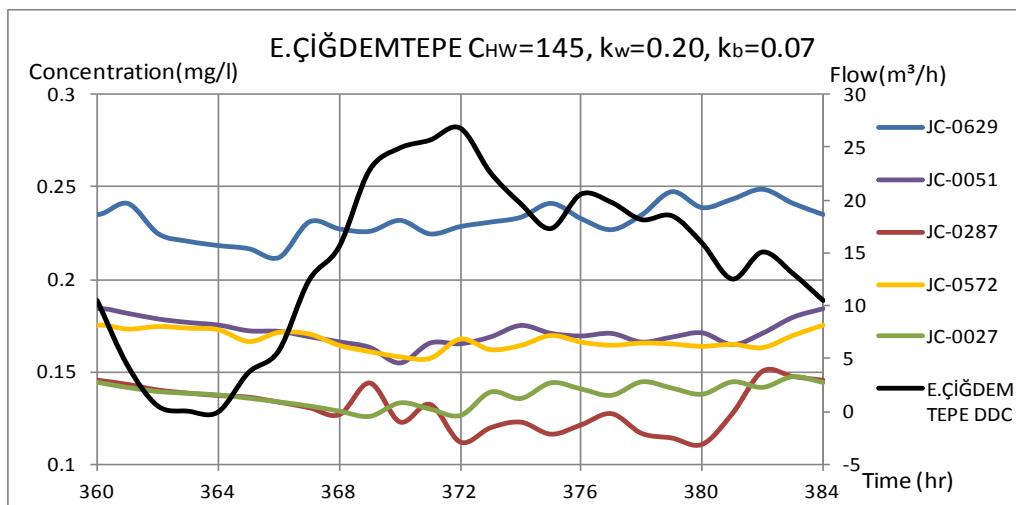


Figure 6.31 The Results of The Most Critical Case for Winter Season

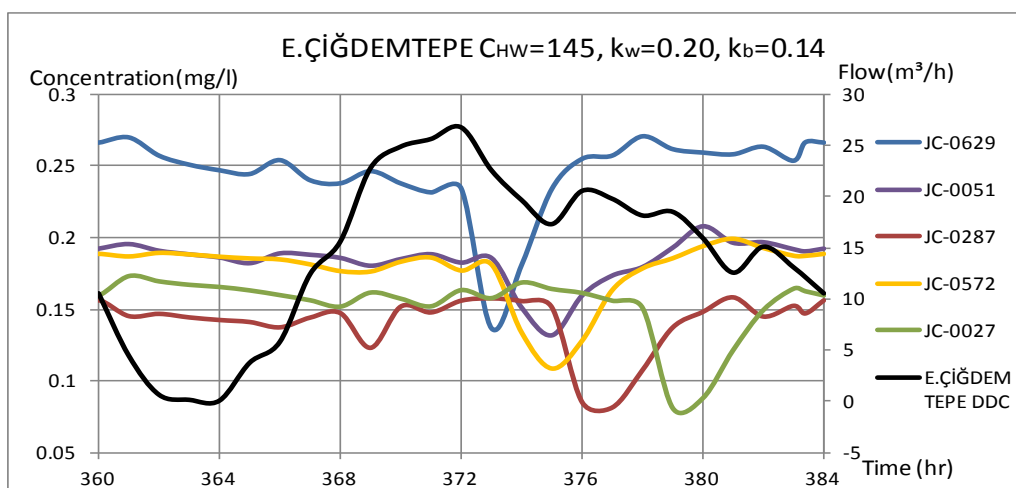


Figure 6.32 The Results of The Most Critical Case for Summer Season

Summary of chlorine concentrations in East Çiğdemtepe for the most critical case is shown in Table 6.13 below.

Table 6.13 Summary of Chlorine Concentrations for The Most Critical Case

Most Critical Cases		CONCENTRATIONS (mg/l)				
		Entrance	Middle	Middle	Middle	Dead End
		JC-0629	JC-0051	JC-0287	JC-0572	JC-0027
C <sub>HW</sub> =145 kw=0.20, k <sub>b</sub> =0.07 (winter)	Maximum	0.24830	0.18380	0.15070	0.17530	0.14770
	Minimum	0.21150	0.15440	0.11110	0.15730	0.12620
	Average	0.23116	0.17063	0.13022	0.16700	0.13795
C <sub>HW</sub> =145 kw=0.20, k <sub>b</sub> =0.14 (summer)	Maximum	0.27100	0.20830	0.15830	0.19940	0.17350
	Minimum	0.13660	0.13180	0.08150	0.10840	0.08130
	Average	0.24484	0.18394	0.14046	0.17727	0.15342

Figure 6.30 and Figure 6.31 show that, chlorine concentrations are in the range of 0.21 mg/l - 0.25 mg/l at entrance junction for winter season and in the range of 0.14 mg/l - 0.27 mg/l in summer. Also at middle junctions average chlorine concentrations in summer are higher than in winter because of the increased water requirement. However, the most critical chlorine concentrations are seen in summer, at dead end junction JC-0027 as 0.0813 mg/l and at middle junction JC-0287 as 0.0815 mg/l. These values are under safe limit 0.10 mg/l. Because of the reason that the lowest chlorine concentration values are seen in summer season, precautions must be taken especially for summer season. For example, using a fire hydrant at JC-0027 or changing isolation valve condition to create a new loop, etc. may be suitable solutions.

It can be conveniently said that; although the demands are decreased and system parameters are selected to give the most critical chlorine concentration values; the system has an acceptable amount of disinfectant concentration for studied junctions and for this WDN. It must be pointed that all results are specific for only this model.



## CHAPTER 7

### CONCLUSION AND RECOMMENDATIONS

Chlorine is the most commonly used chemical for water disinfection in Turkey because of its easy use and economic reasons. The city that this study is performed in, Ankara, also uses chlorine as disinfectant most commonly, as employed by ASKI. Chlorine is firstly injected in İvedik treatment plant and then supplementary chlorination application are used in different locations of Ankara. The pump station which is named as "P23" in the system has a chlorine concentration level of 0.50 mg/l approximately. Concentration measurements are performed at the entrance of the pump station and at different parts of all different pressure zones. Because of blind monitoring, effects of different operations can not be estimated efficiently. It is essential to be able to analyse how the WDN behaves and how chlorine concentrations are influenced by different system conditions.

This study was prepared to investigate if a small part of Ankara WDN named N8.3, evaluate the lowest chlorine concentrations for different conditions, or not. During the study, only one small pump was operating continuously and system was performed for insulated condition. It was analysed that chlorine concentrations are in safe limits or not, while all isolation valves were closed and DMA's study was applied. Main purpose of the study was explaining the effects of some system parameters; Hazen-Williams roughness coefficient  $C_{HW}$ , wall decay coefficient  $k_w$  and bulk decay coefficient  $k_b$ . It was seen that  $k_b$  which is depending mainly on temperature, and  $k_w$  which is depending mainly on pipe material and pipe age; are largely effective on system

performance. Hazen-Williams roughness coefficient is also effective on water quality because of the effects on pressures, flow rates and flow velocity.

Wall decay coefficient  $k_w$  is a parameter which is determined by field study and hydraulic model calibration. However bulk decay coefficient  $k_b$  is determined by laboratory studies. In case, pipe material is the main criteria for selection of Hazen-Williams roughness coefficient  $C_{HW}$ . Despite all, in this study, combinations of three different wall decay coefficient  $k_w$  values, two different bulk decay coefficient  $k_b$  values and three different Hazen-Williams roughness coefficient  $C_{HW}$  values were applied to the water distribution system. For a single pump operation schedule, system were examined in insulated condition.

Apart from these, to comprehend if leakage amounts are effective on concentrations or not, a leakage study was accomplished. For this purpose, by using DDC's of distributed demands, new DDC's were prepared for without leakage and increased leakage cases. New demands were distributed and the system was analysed.

After all these studies, the investigated system parameters were selected to approximate chlorine concentrations to minimum safe limit and two most critical cases were obtained for summer and winter seasons. Because of the increasing water requirement in summer, rapid decay of chlorine is observed and because of decreasing water requirement in winter decay of chlorine slows down conversely. Therefore bulk decay coefficient  $k_b$  is influenced by these differences and affects system performance. In most critical cases  $k_b$  was changed for summer and winter seasons (winter: 0.07 1/day, summer: 0.14 1/day). By taking into consideration all these conditions, system was operated after preparing a DDC without leakage and most critical values were used for wall decay coefficient  $k_w$  (0.20 m/day) and Hazen-Williams roughness coefficient  $C_{HW}$  (145).

Three subzones were analysed in this study. These were (i) Yayla, (ii) North Sancaktepe and (iii) East Çiğdemtepe. Only in one of them, in East Çiğdemtepe, chlorine concentrations dropped below minimum safe limit 0.10 mg/l, at junction JC-0027 as 0.0813 mg/l and at junction JC-0287 as 0.0815 mg/l. Concentrations below 0.10 mg/l are not effective for water disinfection and may cause health diseases due to organisms in potable water. Travel time is an important factor in reducing chlorine concentrations. The more distant from the source, the lower the concentration depending on travel time of the water until reaches that junction. Concentrations are affected by three basic factors at a junction; (i) It is important that if studied junction is sink or not. If all water that comes to a junction is extracted, it is a sink junction. However, if only some of the coming water is extracted, and some of it passes away, it is not a sink junction and may give lower concentration results. (ii) Size of demands have also an important effect on chlorine concentrations. This effect was clearly seen while increasing and decreasing the demands for summer and winter seasons in critical cases study. (iii) Lastly, the source of water is effective on concentrations. In this study, source effect can clearly be seen in East Çiğdemtepe results. Yayla and North Sancaktepe receive water from only the pump station and they give regular results, Hazen-Williams roughness coefficient  $C_{HW}$  does not affect the system. However, in East Çiğdemtepe, by the reason of receiving water from both the pump station and the tank, concentrations are affected by tank concentration, feeding behaviour and water level fluctuations. Velocity changes and concentrations give a difference by changing  $C_{HW}$  coefficients. Water quality is affected by receiving water only from a tank, only from a pump station or from both sources.

Keeping chlorine concentrations at a safe level is essential to use potable water healthfully. For the purpose of preventing concentrations to drop below the minimum safe limit, system may be performed with whole network condition. Because, operating the system as insulated causes lower concentrations. While studying insulated, all isolation valves are closed and it is aimed to keep

leakage amount at minimum level. Separately studying junctions that are not affected by other subzones, cause lower concentrations than a whole network operation. Therefore, opening the isolation valves and giving up to study with DMA's may be good solutions.

Low concentrations caused by the condition of a junction (sink or not) may be prevented by modeling new loops. Thus, extra fresh water is received to junctions and problem may be solved. For example, while studying the most critical cases in East Çiğdemtepe, a chlorine concentration value was seen under safe limit 0.10 mg/l at a middle junction JC-0287 as 0.0815 mg/l. In this junction, changing the conditions of isolation valves or creating new loops may be good solutions.

Injecting extra chlorine to the pump station may be a good solution to increase chlorine concentrations of all system. For example, while studying the most critical cases in East Çiğdemtepe, a chlorine concentration value was seen under safe limit 0.10 mg/l at dead end junction JC-0027 as 0.0813 mg/l. In this junction, a fire hydrant may be used to provide extra fresh water or to change the water amount.

By taking into consideration the effect of wall decay coefficient  $k_w$  and bulk decay coefficient  $k_b$ , new pipes must be used as far as possible and essential maintenance and repair applications must be done. Therefore,  $k_w$  and  $k_b$  values do not increase, bulk and wall reactions decrease and chlorine concentrations do not drop below safe limit. Also,  $k_b$  is a time dependent parameter, especially in summer season, this kind of precautions become more essential and important.

It must be pointed that, all studies were performed by using a continuously operating single pump schedule. However, for different system parameter values different pump scheduling must be applied to the system to provide the

highest efficiency. For this purpose, softwares which are developed for water distribution system modeling, may be used to specify the optimum pump scheduling for different combinations of system parameter values. This provides more realistic and efficient results for water disinfection capabilities.



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