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WATER DISTRIBUTION NETWORK ANALYSIS USING NETCAD

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Water Distribution Network Analysis Using Netcad

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

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ABSTRACT

WATER DISTRIBUTION NETWORK ANALYSIS USING NETCAD

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In this thesis, solutions of the drinking water distribution networks are analyzed using Netcad/Water and Netcad/Water 2005 modules of the Netcad software. Most of small size drinking water distribution networks in rural areas are designed by using Netcad / water module, but most of large size drinking water distribution networks in cities are designed by using Netcad/water 2005 module

As an application, drinking water network of Hacıağalar town, which is a small city managed by the Rural Service of Kahramanmaraş city, is solved both as Branched (Open) and Looped (Closed) drinking water network systems. Result of Branched (Open) drinking water network and Looped (Closed) drinking water network are compared. The same network is also analyzed by using Netcad/water 2005 module. It is seen that Netcad/Water 2005 module is superior to Netcad/Water module with respect to optimal network design. The diameter of pipes obtained by using Netcad/Water and Netcad/Water 2005 are compared. It is found that overall pipe costs of the solution obtained by Netcad/water 2005 are more economical than the solution obtained by Netcad/water.

Keywords: Design of Water distribution Networks, Branched network, Looped network, Dead Point method, Hardy Cross method

ÖZ

NETCAD İLE İÇMESUYU ŞEBEKE ANALİZİ

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Bu tezde içmesuyu dağıtım şebekesinin analizi, Netcad programının Netcad/içmesu ve Netsu2005 modülleri kullanılarak şebeke çözümü incelenmiştir. Kırsal bölgelerdeki küçük çaplı içmesuyu şebeke tasarımlarının çoğunda Netcad/içmesu modülü kullanılmasına rağmen şehirlerdeki büyük çaplı içmesuyu şebekeleri Netsu2005 modülü kullanılarak tasarlanmaktadır.

Uygulama olarak Kahramanmaraş İl Özel İdaresine bağlı Hacıağalar beldesinin içmesuyu şebekesi Netcad/içmesu modülü ile açık ve kapalı şebeke tasarımı yapılarak çözülmüştür. Açık ve kapalı şebeke çözüm sonuçları karşılaştırılmıştır. Aynı şebeke netsu 2005 modülü ile de çözümü yapılmıştır. En uygun şebeke tasarımı hakkında Netsu 2005 modülünün Netcad/içmesu modülüne göre daha üstün olduğu görülmüştür. Netcad/içmesu ile çözülen açık şebekedeki boru çapları ile Netsu 2005 ile çözülen boru çapları karşılaştırılmıştır. Karşılaştırma sonucu Netsu 2005 ile çözüm sonucu bulunan boru maliyetleri Netcad/içmesu'ya göre daha ekonomik olduğu görülmüştür.

Anahtar Kelimeler : İçmesuyu dağıtım şebekesi tasarımı, Açık şebeke, Kapalı şebeke, Ölü nokta metodu, Hardy Cros Metodu

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SYMBOLS AND ABBREVATIONS

Q_i: in flow to node in i-th pipe in m³/s

U_i: water used or leaving at the i-th node in m³/s

$\frac{dS}{dt}$: Change in storage

γ: Fluid (water)specific weight, in (N/m³)

Z: elevation in (m)

P: Pressure in (m)

V: Velocity

h_p: pumped head gain in (m)

h_L: head loss in pipes in (m)

h_M: head loss duetominor losses in (m)

h_f: frictional head loss

f: friction factor

L: length of pipe

D: diameter of pipe

g: gravitational accelaration

S_f. Friction slope

R_e : Reynolds number

K_s : Nikuradse equivalent

C: Hazen-Williams Coefficient

n : Manning Coefficient

A: cross-sectional flow area

R: hyraulic radius

Q_j: Discharge in pipe i meeting at node (junction) j

q_i: is nodal withdrawal at node

ΔQ_k: discharge correction factor

Q_u:End of discharge

Q_h: design discharge

P: Relative discharge in any pipe

Q_f: fire discharge

L_e: equivalent Pipe length

H: hydraulic head

N_y : New population enumeration

N_e: Old population enumeration

a: Number of years between new and old years.

N_g: future population enumerations

n:Number of years between future population and last population enumerations

V: Volume of reservoir

V_f : Volume of fire

N_{2042}: population of the year 2042

Q_b:upstream ends of pipe discharge

Q_u: Downstream ends of pipe discharge

Q_h: Design discharge

CHAPTER 1

INTRODUCTIONS

1.1 General Information

Water distribution system models are more commonly being used to replicate the behavior of a real or proposed system for a variety of purposes including: capital investment decisions, development of master plans, estimation of fire protection capacity, design of new systems and extension or rehabilitation of existing systems, energy management, water quality studies, various event simulations and analysis, optimal placement of sensors, and daily operations. The costs associated with constructing and maintaining a distribution system model may be more easily justified if it is used for a variety of applications by water utility (Grayman, 2000).

The demand for water needed to serve agriculture, industry, sanitation, and domestic consumption increases continuously along with population and economic growth. However, with increasing development and urbanization, water flow rates and other hydraulic requirements associated with water distribution systems have been estimated to increase on both national and local scale. Expansion and construction of new water supply areas have made the water distribution network become even more complicated and resulted in numerous problems of water allocation, water supply safety, operation and management. Therefore, managing the water distribution systems in a sustainable and integrated manner is necessary to meet the growing demand of water for drinking, industrial and other necessities. In this regard, there is an urgent need to develop well-designed and optimized solution methods to achieve better control of water distribution systems.

For this purpose, several investigators have conducted studies on the calculation of complex water distribution networks using different solution approaches. However, most have limited use when working with high-dimensional hydraulic data.

The present day water distribution networks are complex and require huge investments in their construction and maintenance. Therefore, in order to develop a continuous strategy for the management of water distribution systems, hydraulic parameters should be attentively controlled routinely for the duration of the testing, and network quality should also be verified under various operating conditions. However, engineers may not have enough time to monitor all hydraulic parameters under different operating conditions. Hence, a number of modification attempts to the Standard solution methods for development of a powerful algorithm may help to assess both steady-state solutions and particularly time-dependent simulations of water distribution systems when the nodal demands change on a daily basis.

Water distribution systems may be laid down in two ways: in loopal networks and in branching structures. From a view point of operational concerns, branching distribution systems, also called dead-end systems, may lead mainly to operational problems in the aspect of system pressure, especially places near the dead-ends. In order to overcome this problem, pipes may be laid down in a loopal manner, which is the most commonly used construction method throughout the world. This way, it is easier to sustain much higher operational pressures all over the distribution system.

1.2 Principal Objectives

In this thesis, drinking water network is analysed and solved as open and closed systems by using programme. Open and closed network solved by netcad is compared with respect to the hydraulic solution and their costs. Netcad includes two modules for drinking water network solutions: Netcad/Water module and Netcad/Water 2005 module. Drinking water network of the Haciagalar Village of the Kahramanmaras province is analysed and solved by using Netcad/water 2005 module.

Open and closed systems solutions obtained by using Netcad/Water module is also compared with each other. Finally, a comparison of the costs of pipe materials obtained by the Netcad/water2005 module and Netcad/water module is presented.

1.3 Layout of Thesis

The purpose of this thesis is to analyze a drinking water distrubution network using Netcad which is a package computer program that has many tool boxes. Nowadays Netcad is widely used by many design engineers and municipilates.

The content of each chapter can be described as follows,

- Literature survey about water distrubution network design and literature review of programming techniques for water distrubution network are given in Chapter 2
- Elements of drinking water distrubution systems are given in Chapter 3
- Chapter 4 deals with analysis of water distrubution network
- In Chapter 5, water distrubution network design methods are given.
- In Chapter 6, prensentation of network analyses by using Netcad/water and Netcad/water2005 are given
- In chapter 7, conclusions and suggestions for future work are discussed.

CHAPTER 2

LITERATURE SURVEY

2.1 History of Water Distribution Network

Hardy Cross first proposed the use of mathematical methods for calculating flows in complex networks (Cross, 1936). Iterative procedure was used throughout the water industry for almost 40 years. With the advent of computers and computer-based modeling, improved solution methods were developed for utilizing the Hardy Cross methodology. The improved implementations of this method were in widespread use by the 1980s (Wood, 1980a).

Also, in the early 1980s, the concept of modeling water quality in distribution system networks was developed based on steady-state formulations (Clark et al., 1986). By the mid-1980s, water quality models were developed that incorporated the dynamic behavior of water networks (Grayman et al., 1988). The usability of these models was greatly improved in the 1990s with the introduction of the public domain EPANET model (Rossman, 2000) and other Windows-based commercial water distribution system models.

Initially, hydraulic models simulated flow and pressures in a distribution system under steady-state conditions where all demands and operations remained constant. Since system demands (and consequently the flows in the water distribution network) vary over the course of a day, Extended Period Simulation (EPS) models were developed to simulate distribution system behavior under time-varying demand and operational conditions.

These models have now become ubiquitous within the water industry and are an integral part of most water system design, master planning, and fire flow analyses. (Epa, 2005)

In Turkey Netcad is started to be developed in 1987. Netcad is a complete software, and has been meeting the requirements of public and private sector for over 20 years, holding various professional solutions within its structure. It has been produced totally with the equity resources of the firm and has no license dependency.

2.2 Programming Techniques For Water Distribution Networks

The invention of digital computers, however, allowed more powerful numerical techniques to be developed. These techniques set up and solve the system of equations describing the hydraulics of the network in matrix form. Because the energy equations are non-linear in terms of flow and head, they can not be solved directly. Instead, these techniques estimate a solution and then iteratively improve it until the difference between solutions fall within a specified tolerance. At this point, the hydraulic equations are solved. Researchers have been investigating cost of effective water distribution network with various approaches such as linear, nonlinear, dynamic and mixed integer programming.

Researchers have been investigating cost of effective water distribution network with various approaches such as linear, nonlinear, dynamic and mixed integer programming. Alperovits and Shamir (1977) presented a linear programming gradient (LPG) in optimizing water distribution network. Segmental length of pipe with differential diameter was used as decision making variable. The LPG method was later further improved by Kessler and Shamir (1989). For example, Kessler and Shamir (1989) presented two stages LPG method. In the first stage, parts of the variables are kept constant while other variables are solved by linear programming (LP). For a given set of flows, the corresponding sets of heads are determined by LP. In the second step, search is conducted based on the gradient of the objective function.

Flows are modified according to gradient of the objective functions. Eiger et al. (1994) used the same formulation of Kessler and Shamir (1989) and solved the problem using a nonsmooth branch and bound algorithms and duality theory. The algorithms are a combination of primal and dual processes and stopped when the difference between the best solution and the global lower bound is within a prescribed tolerance. Although the problem is nonlinear and the gradient information may not be attained in many instances, they nevertheless solved the problem by linearizing the formulation. This results in failure to reach the optimal solution.

Therefore, a nonlinear programming (NLP) technique was developed and applied by Chiplunkar et al. (1986). However, NLP also converges to local minima due to their reliance on the initial solution and derivatives of the unconstrained objective function (Gupta et. al. (1999)). Moreover, nonlinear algorithms perform on the basis of continuous variables, pipe diameter. Available pipe diameters in the market are definitely not continuous. Conversion of the assumed continuous diameters to the market pipe sizes influences the optimal solution (Cunha and Sousa, (1999)). Recently, researchers focus on stochastic optimization methods that deal with a set of points simultaneously in its search for the global optimum. The search strategy is based on the objective function. Simpson et al. (1994) used simple GA in which each individual population is represented in a string of bits with identical length that encodes one possible solution. All binary coded population of points (chromosomes) undergoes three operations: selection, crossover and mutation operators. The simple Genetic Algorithms (GA) was then improved by Dandy et al. (1996) using the concept of variable power scaling of the fitness function, an adjacency mutation operator, and gray codes. Savic and Walters (1997) also used simple GA in conjunction with EPANET network solver. Instead of using a single optimization algorithm, Abebe and Solomatine (1998) applied GLOBE that comprises several search algorithms. They identified that very few algorithms reach to optimal or near optimal solutions. Cunha and Sousa (1999) introduced a random search algorithm (Simulated Annealing) that is based on the analogy with the physical annealing process with Newton search method to solve the network equations. Eusuff and Lansey (2003) proposed Shuffled Frog Leaping Algorithms (SFLA), a new meta-heuristic algorithm works based on memetic evolution (transformation of frogs) and information Exchange among the population. Frogs which are the hosts of memes (consist of memotype like gene in chromosome in GA) search the particle with highest amount of food in a swamp by improving their names.

Although the final solution was improved, Savic and Walters (1997), Cunha and Sousa (1999), and Eusuff and Lansey (2003) required considerable computational effort.

2.3 Review of Modelling Software for Water Distribution Networks

A selection of softwares of water distribution network widely used commercial software packages for designing and analysing piped water distribution systems was presented in a uniform and rapidly accessible format, based on information provided by the respective developers and distributors.

Over recent years, there has been a significant increase in the number of software applications that have been released in this field both in the commercial and in the public domains. As this process continues, it is again becoming difficult for systems managers and designers to select a software package most adapted to local needs and circumstances; some would go further and claim that selection is once again turning into a process based on the principles of a lucky dip. The investigation has been extended towards non-commercial software and the information, covering a revised listing of 5 programs, is categorised according to the sophistication of the applications under review. The facts haven been compiled from information given by the respective software developers and/or distributors as well as by its users.

2.3.1 Branch / Loop

Branch: A computer program for the least-cost design of branched water distribution networks and Branch is used to design pressurised, branched (tree-type, non-looped) water distribution networks by choosing from among a set of candidate diameters for each pipeline so that the total cost of the network is minimised subject to meeting certain design constraints. Both construction costs and the design constraints can be expressed as linear, mathematical statements. Branch formulates the linear programming model for the least cost design, solves the model and outputs the design as well as corresponding hydraulic information.

Loop: A computer program for the hydraulic simulation of looped water distribution Networks. Loop simulates the hydraulic characteristics of a pressurised, looped (closed circuit) water distribution network. The network is characterised by pipes and nodes (points of inputs/demand or pipe junctions). Data required are the description of the elements of the network such as pipe lengths, diameters, friction coefficients, nodal demands and ground elevation, and data describing the geometry of the network. The program outputs include flows and velocities in the links and pressures at the nodes.

2.3.2 Epanet

Epanet was initially developed in 1993 as a distribution system hydraulic-water quality model to support research efforts at EPA (Rossman et al., 1994). Epanet was developed by the Water Supply and Water Resources Division (formerly the Drinking Water Research Division) of the U.S. Environmental Protection Agency's National Risk Management Research Laboratory

Epanet is a Windows based program that performs extended period simulation of hydraulic and water-quality behaviour within pressurised pipe networks. A network can consist of pipes, nodes (pipe junctions), pumps, valves and storage tanks or reservoirs. Epanet tracks the flow of water in each pipe, the pressure at each node, the height of water in each tank, and the concentration of a chemical species throughout the network during a simulation period comprised of multiple time steps. In addition to chemical species, water age and source tracing can also be simulated.

The Windows version of Epanet provides an integrated environment for editing network input data, running hydraulic and water quality simulations, and viewing the results in a variety of formats. These include colour-coded network maps, data tables, time series graphs, and contour plots.

2.3.3 Mike Net

Mıke Net is the most advanced Epanet based water distribution modelling package available. It can analyse an entire water distribution system, or selected portions, under steady state or extended period simulations, with water quality analysis if needed. In a complete graphical modelling environment it considers pipes sizes, flow rates, velocities, head losses, nodal pressures, nodal demands, hydraulics grades, elevations, water age, water quality concentrations, etc.

2.3.4 Pipe2000

Pipe2000 is a general-purpose pipe network hydraulic modelling program, which handles both steady state and transient analysis. Pipe2000 is typically used for steady state and transient modelling municipal and rural water distribution systems. It is also widely used for hot, chilled, and process water systems. It is used for fire protection and irrigation sprinkler systems. It is also used for other liquids (oil, etc.).

2.3.5 WaterCAD

WaterCAD is used to analyse potable water networks, sewage force mains, fire protection systems, well pumps, raw water pumping, and more. It allows calibration of large distribution networks, development of master plans, conducting operational studies and performing cost analysis. Users can perform a steady-state analysis for a “snapshot” view of the system, or perform an extended period simulation to see how the system behaves over time.

The highlights of the most recent version are: advanced graphical editor, animated contouring and colour-coding, graphing and profiling, scenario management, automated fire flows, hydraulic analysis, water quality analysis, customisable tables, GIS and database connectivity.

CHAPTER 3

ELEMENTS OF DRINKING WATER DISTRIBUTION SYSTEMS

3.1 General

Distribution system infrastructure is a major asset of a water utility, even though most of the components are either buried or located inconspicuously. Drinking water distribution systems are designed to deliver water from a source (usually a treatment facility) in the required quantity, quality, and at satisfactory pressure to individual consumers in a utility's service area. In general, to continuously and reliably move water between a source and a customer, the system would require storage reservoirs/tanks, and a network of pipes, pumps, valves, and other appurtenances. This infrastructure is collectively referred to as the drinking water distribution system (Walski et al., 2003).

3.2 Pipe Network

The system of pipes or "mains" that carry water from the source (such as a treatment plant) to the consumer is often categorized as transmission/trunk, distribution, and service mains. Transmission/trunk mains usually convey large amounts of water over long distances, such as from a treatment facility to a storage tank within the distribution system. Distribution mains are typically smaller in diameter than the transmission mains and generally follow city streets. Service mains are pipes that carry water from the distribution main to the building or property being served. Service lines can be of any size, depending on how much water is required to serve a particular customer, and are sized so that the utility's design pressure is maintained at the customer's property for the desired flows.

3.2.1 Pipes

The most commonly used pipes today for water mains are ductile iron, pre-stressed concrete, polyvinyl chloride (PVC), reinforced plastic, and steel. In the past, unlined cast iron pipe and asbestos-cement pipes were frequently used. Even a medium-sized water utility may have thousands of miles of pipes constructed from various types of materials, ranging from new, lined or plastic pipes to unlined pipes that are more than 50 years old. (Clark and Tippen, 1990).

The fast developings in Plastic Technology provide the new findings in raw material producing. The pipes made of PE 32, 40 and 63 are successfully used in systems which do not need high pressure. These pipes because of their technical characteristics find posibility to be used only in systems which require low pressure. PE 100 is the strongest material, which is developed as result of long studyings and searchings, resistant to high pressure upto now. PE pipes' wall thinkness is thinner than PE 32, PE 40, PE 63 and PE 80 in the same operating pressure and outer diameters. In this side for PE 100 pipes thin wall thinkness forms bigger inner diameter and make possible to use short size for the same flow and provide to save the row material.(www.Pilsa.com.tr)

3.2.2 Valves

There are two general types of valves in a distribution system: isolation valves and control valves. Isolation valves are used in the distribution system to isolate sections for maintenance and repair and are typically located in a system so that the areas isolated will cause a minimum of inconvenience to other service areas. Maintenance of the valves is one of the major activities carried out by a utility. Many utilities have a regular valve-turning program in which a percentage of the valves are opened and closed on a regular basis. It is desirable to turn each valve in the system at least once per year. In large systems, this may or may not be practical, but periodic exercise and checking of valve operations should occur. This practice minimizes the likelihood that valves will become inoperable due to corrosion. The implementation of such a program ensures that, especially during an emergency, water can be shut off or diverted and that valves have not been inadvertently closed.

Control valves are used to regulate the flow or pressure in a distribution system. Typical types of control valves include pressure-reducing valves, pressure-sustaining valves, flow-rate control valve throttling valves, and check valves.

3.2.3 Hydrants and Other Appurtenances

Hydrants are primarily a part of the fire-fighting infrastructure of a water system. Although the water utilities usually have no legal responsibility for fire flow, developmental requirements often include fire flows, and thus, distribution systems are designed to support needed fire flows where practical American Water Works Association (AWWA,1998). Proper design, spacing, and maintenance are needed to insure an adequate flow to satisfy fire-fighting requirements. Fire hydrants are typically exercised and tested periodically by water utility or fire department personnel. Fire-flow tests are conducted periodically to satisfy the requirements of the Insurance Services Office (ISO) or as part of a water distribution system calibration program. Other appurtenances in a water distribution system include blow-off valves and air release valves.

3.2.4 Pumps

Pumps are used to impart energy to the water in order to boost it to higher elevations or to increase pressure. Routine maintenance, proper design and operation, and testing are required to insure that they will meet their specific objectives. Pump tests are typically run every five to ten years to check the head-discharge relationship for the pump. Many system designers recommend two smaller pumps instead of one large pump to ensure redundancy.

3.2.5 Reservoirs

Reservoirs are used to provide storage capacity to meet fluctuations in demand, to provide reserves for fire-fighting use and other emergency situations, and to equalize pressures in the distribution system. The most frequently used type of storage facility is the elevated tank, but other types of tanks and reservoirs include in-ground tanks and open or closed reservoirs. Materials of construction include concrete and steel. An issue that has drawn a great deal of interest is the problem of water turnover within storage facilities. Much of the water volume in storage tanks is dedicated to fire protection. Unless utilities make a deliberate effort to exercise (fill and draw) their tanks, or to downsize the tanks when the opportunity presents itself, there can be both water aging and water mixing problems.

CHAPTER 4

ANALYSIS OF WATER DISTRIBUTION NETWORK

4.1 Introduction

The aim of the water distribution network design is to find the optimal pipe diameter for each pipe in the network for a given layout, demand loading conditions, and an operation policy. Real water distribution systems do not consists of a single pipe and can not described by a single set of continuity and energy equations.

A pipe network analysis method selects the optimal pipe sizes in the final network satisfying all implicit constraints (e.g. conservations of mass and energy), and explicit constraints (e.g. pressure head and design constraints). The hydraulic constraints, for example, deal with hydraulic head at certain nodes to meet a specified minimum value. If the hydraulic head constraint is violated, the penalty cost is added to the network cost. However, diameter constraints enforce the evolutionary algorithms to select the trial solution within a pre-defined limit. In order to solve the flow rates in all of the network paths, a series of simultaneous equation must be developed that include every pipeline in the network. The basis for the development of the equations used to balance the flow in the network and the primary assumptions are:

1.Algebraic sum of the mass flow into and out of all nodes must be equal zero

2.The algebraic sum of the pipeline pressure drops (head loss) around a loop must be equal zero.

4.2 Theoretical Concepts :

The theory and application of hydraulic models is thoroughly explained in many available references (Walski et al., 2003; American Water Works Association, 2004; Larock et al., 2000). Essentially, three basic relations are used to calculate fluid flow in a pipe network. These relationships are:

4.2.1 Conservation of Mass:

This principle requires that the sum of the mass flows in all pipes entering a junction must be equal the sum of all mass flows leaving the junction. Because water is essentially an incompressible fluid, conservation of mass is equivalent to conservation of volume.

$$\sum_{i=1}^{n}(Q_i - U_i) - \frac{dS}{dt} = 0 \qquad (4.1)$$

Where ;

Q_i = inflow to node in i-th pipe in m³/sn

U_i = water used or leaving at the i-th node in m³/sn

$\frac{dS}{dt}$ = change in storage (m³/sn)

4.2.2 Conservation of Energy

There are three types of energy in a hydraulic system: kinetic energy associated with the movement of the fluid, potential energy associated with the elevation, and pressure energy. In water distribution networks, energy is referred to as "head" and energy losses (or headlosses) within a network are associated primarily with friction along pipe walls and turbulence.

$$Z_1 + \frac{P_1}{\gamma} + \frac{V_1^2}{2g} + \sum h_p = Z_2 + \frac{P_2}{\gamma} + \frac{V_2^2}{2g} + \sum h_L + \sum h_M \qquad (4.2)$$

where Z_1 and Z_2 = elevation at points 1 and 2, respectively, in (m)

P_1 and P_2 = Pressure at points 1 and 2, respectively, in (N/m)

γ= fluid (water) specific weight, in (N/m³)

V_1 and V_2 = Velocity at points 1 and 2, respectively, in (m/s)

g = acceleration due to gravity, in (m/sec^2)

h_p = pumping head gain, in (m)

h_L = head loss in pipes, in (m)

h_M =head loss due to minor losses, in (m)

4.2.3 Pipe Friction Losses

A key factor in evaluating the flow through pipe networks is the ability to calculate friction headloss (Jeppson, 1976). Three empirical equations commonly used are the Darcy-Weisbach, the Hazen-Williams, and the Manning equations. All three equations relate head or friction loss in pipes to the velocity, length of pipe, pipe diameter, and pipe roughness. A fundamental relationship that is important for hydraulic analysis is the Reynolds number, which is a function of the kinematic viscosity of water (resistance to flow), velocity, and pipe diameter. The most widely used headloss equation in the U.S. is the Hazen-Williams equation. Though the Darcy Weisbach equation is generally considered to be theoretically more rigorous, the differences between the use of these two equations is typically insignificant under most circumstances. Figure 4.1 illustrates friction losses (head losses) during flow in a pipe.

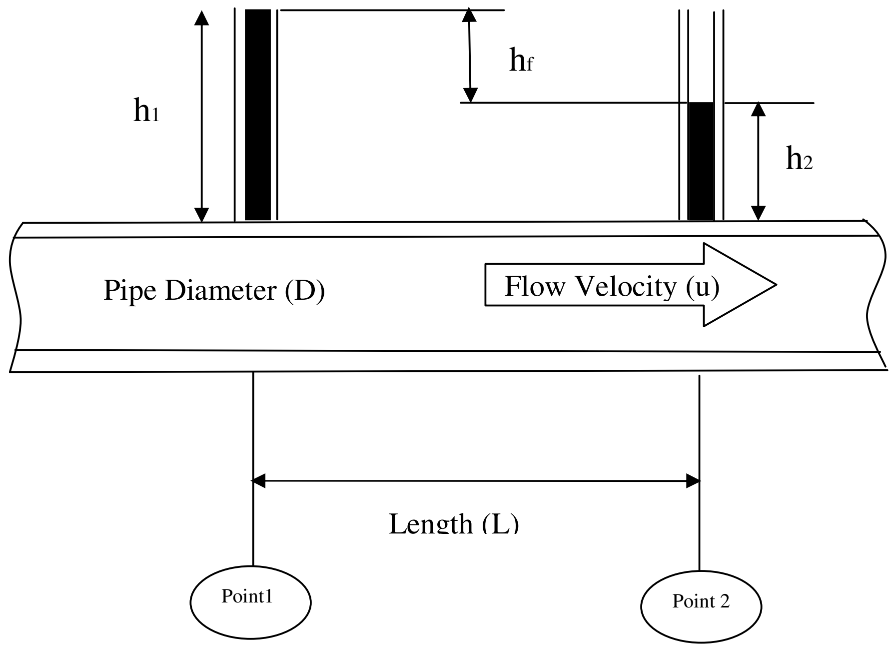


Figure 4.1 Friction losses in pipes

4.2.3.1 Darcy-Weisbach Equation

It is a dimensionally homogeneous equation:

$$h_f = f \frac{L}{D} \frac{u^2}{2g} \tag{4.3}$$

where h_f is the frictional headloss along the pipe in m, f is the friction factor which can be determined from the equations presented below, L is the length of the pipe in m, D is the diameter of the pipe in m, u is the average cross-sectional velocity of the pipe in m/s and g is the gravitational acceleration. Combining with the continuity equation, the Darcy-Weisbach equation can be expressed in terms of discharge:

$$Q = \frac{\Pi}{2} \frac{g^{0,5}}{\sqrt{2f}} D^{2,5} S_f^{\ 0,5} \tag{4.4}$$

where S_f is the friction slope, which is equal to h_f/L. For laminar flow conditions in a circular pipe, f is given by $64/R_e$ where R_e is the Reynolds number, which is uD/υ with υ being the kinematic viscosity.

For a smooth pipe, f is a function of only the Reynolds number:

$$\frac{1}{\sqrt{f}} = 2\log(R_e\sqrt{f}) - 0.8 \tag{4.5}$$

For a rough pipe, f is expressed as a function of the relative roughness, k_s/D, where k_s is the Nikuradse equivalent sand roughness:

$$\frac{1}{\sqrt{f}} = 2\log(\frac{D}{2k_s}) + 1.75 \tag{4.6}$$

For a transitional flow, f is a function of both Re and ks/D:

$$\frac{1}{\sqrt{f}} = 1.14 - 2\log(\frac{k_s}{D} + \frac{21.25}{R_e^{0.9}}) + 1.75 \tag{4.7}$$

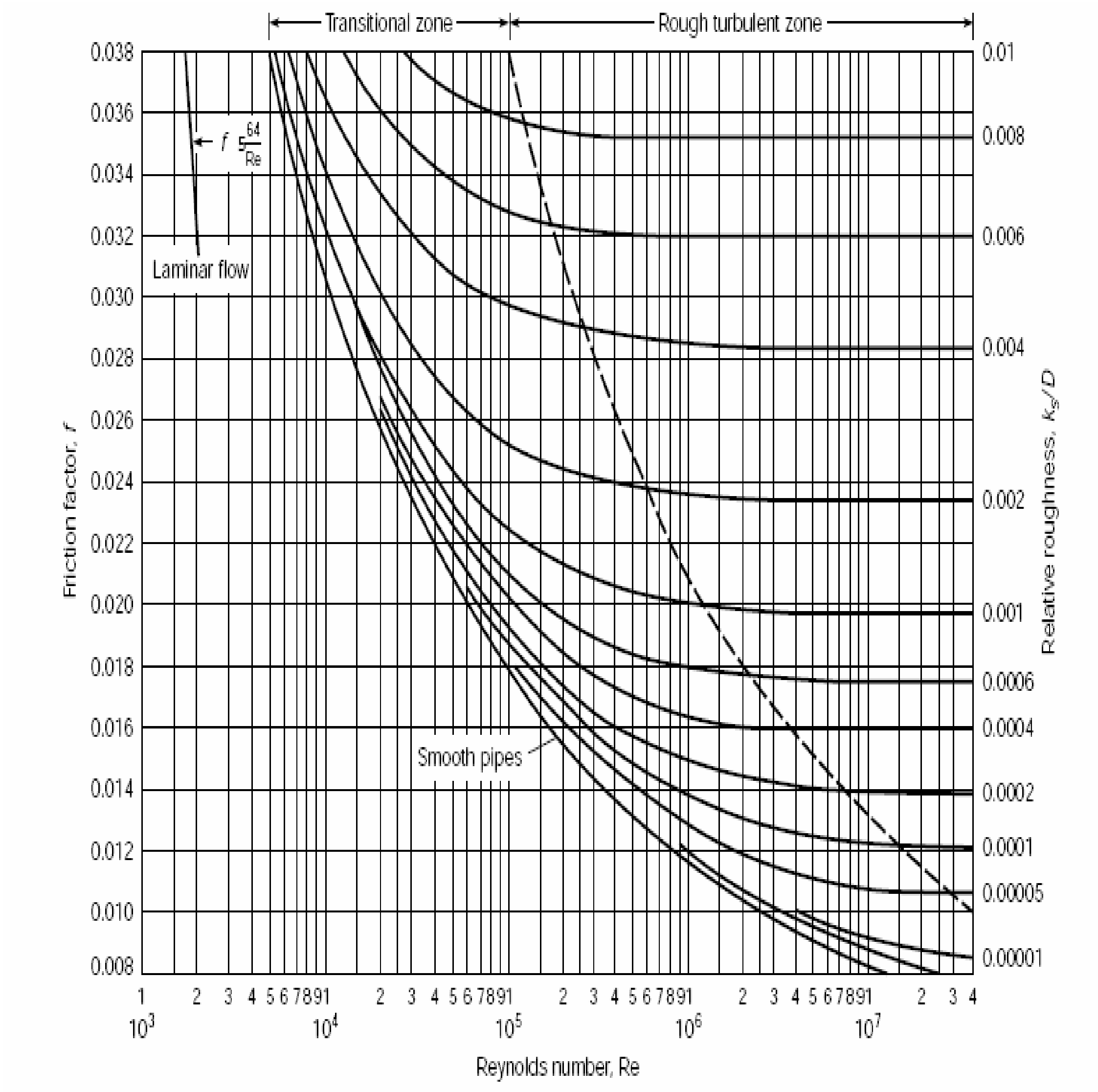


Figure 4.2 Moody Chart

4.2.3.2 Hazen Williams Equation

In SI units it is given as

$$Q = 0.279CD^{2.63}S_f^{0.54} \tag{4.8}$$

where Q is in m^3/s, C is a coefficient (see Table 4.1), and D is in m.

Table 4.1 Hazen-Williams Coefficients

Description of Pipe	Value of C
Cast iron :	
New	130
5 years old	120
10 years old	110
20 years old	90-100
30 years old	75-90
Concrete	120
Cement lined	140
Welded steel, as for cast iron 5 years older	120
Riveted steel, as for cast iron 10 years older	110
Plastic	150
Asbestos cement	140

4.2.3.3 Manning Equation

It is dimensionally non-homogeneous. In SI units it is given as

$$Q = \frac{A}{n} R^{2/3} S_f^{1/2} \tag{4.9}$$

where A is the cross-sectional flow area in m^2, n is the Manning roughness coefficient, and R is the hydraulic radius in m as the flow area divided by the wetted perimeter. For a pressurized flow (full flow) R=D/4 and $A=\Pi D^2/4$. Therefore, Equation (4.9) becomes

$$Q = \frac{0.312}{n} D^{8/3} S_f^{1/2} \tag{4.10}$$

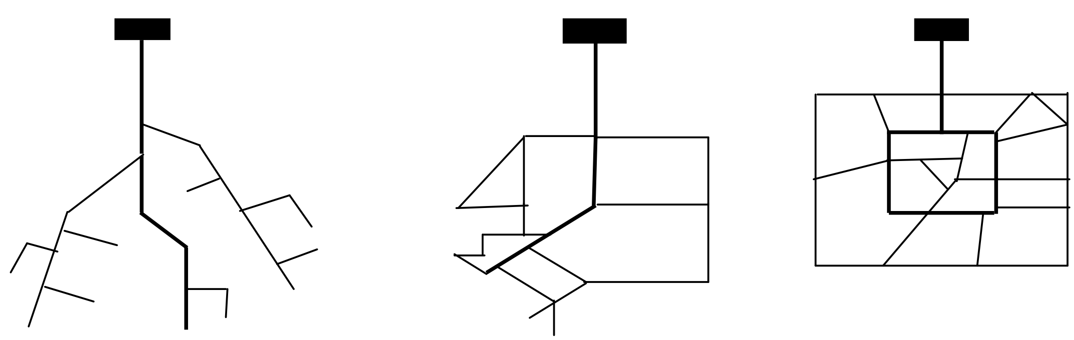
As can be seen from the above equations, the pipe flow formulas are similar to each other that express the discharge as a function of pipe diameter and friction slope having close powers for each equation. However, the use of the Darcy-Weisbach equation is more recommended as its resistance coefficient reflects the effects of both flow conditions and the type of pipe material, whereas the roughness factors in the Hazen-Williams and Manning's equations are expressed only in terms of pipe material.

CHAPTER 5

WATER DISTRIBUTION NETWORK DESIGN METHODS

5.1 Introductıon

A water distribuon network receives water from the main feeder and delivers it to individual consumers. It is normally composed of pipes, valves, hydrants, and pumps. Distribution of water in the city network may be provided by means of gravity or by using pumps with or without some storage. Distribution by gravity is only possible when the storage reservoir is located at a sufficient altitude with respect to the level of the city. Pumping without storage is not desirable since this case does not provide any storage, which may be required during any emergency like fires or repair works. Hence, pumping with storage is the most common way of water distribution. The street plan, topography, location of supply works and level of service dictate the selection of the type of distribution system among a branching pattern with dead ends (Figure 5.1.a), a gridiron pattern (Figure 5.1.b) and a gridiron pattern with central feeder (Figure 5.1.c)



(a) **(b)** **(c)**

Figure 5.1 Types of water distribution networks

5.2 Types of Drinking Water Network System

The water drinking network system is designed according to the state of topography of the city, maximum and minimum pressure, flow rate. Water drinking network system design is made by considering the situation of the city about thirty years future according to Bank of Province of Turkey

The branch (open) and grid/loop (closed) are the two basic configurations for most water distribution systems. A branch system is similar to that of a tree branch with smaller pipes branching off larger pipes throughout the service area. This type of system is most frequently used in rural areas, and the water has only one possible pathway from the source to the consumer. A grid/loop system is the most widely used configuration in large municipal systems and consists of interconnected pipe loops throughout the area to be served. In this type of system, there are several pathways that the water can follow from the source to the consumer. Looped systems provide a high degree of reliability should a line break occur, because the break can be isolated with little impact on consumers outside the immediate area (Clark and Tippen, 1990; Clark et al., 2004).

A branching pattern with dead ends is suitable for strip-shaped districts where water flows in one direction. However, during any repair, the part downstream of the section being repaired cannot take water. A gridiron is preferable for flat and wide terrains where water flows in a loop in converging manner. A gridiron pattern with a central feeder is the best type as the auxiliary main is in a looped pattern (see Figure 5.1.c).
There are two types of the network systems.
1-Branched network systems
2-Grid/Looped network systems.

In Branched network systems, water flows in one directions, in this types of sytems any broken pipes cause lack of water at places after it. In Grid/Looped network systems, water flows in both directions through the pipes so that a failure in a point does not affect the rest of the systems. For this reason in projects closed network systems should be used unless an obligation is present for the the use of open network sytems.

5.2.1 Branched Drinking Water Network Systems

In this systems, pipes are distributed like brunch of trees without joining in any point. Especially this systems is used in beach part of the city, in places connected to main street are not intersected, pipes do not form a network.

Advantages of the the branched drinking water network systems are as follows.

1-Calculatıons are simple and easy

2-It is economic becuse Pipe diameters and lengths are smaller

Disadvantages of the branch drinking water network systems are as follows

1-End points of the pipes are that points at respect to calculations and physically and conditions. Namely at this points water is completely distrubuted so that the flow rate falls to zero. For tis reason velocitys are to small causing foreign substances to sub side. for the same reason water lacks some of it properties.

2-A failure in any point of the system causes lack of water in the whole system

3-System is based on one way flowing. Addition of new places to the network may result in a decrease in system pressure.

5.2.2 Grid/Looped Drinking Water Network Systems

In this systems pipes are connected to each other. Any place in the network can be feeded from several roots.

Advantages of the closed network systems are as follows:

1-As the water flows from many ways, there is no dead points or slow flowing rates.

2- In case of any broken pipe or preparing, the place served by broken pipe can be feeded by another route.

3-When consuming rate of water changes this system is less affected then open systems in another words, this systems has more flexibility.

Disadvantages of the open network systems are as follows:
1-Hydraulic calculations are more complex
2- It requires more pipes and fittings.

5.3. Design of Drinking Water Network Systems

There are several methods presented for the solution of the water drinking network system. Some of them are:
1- Hardy Cross Method
2- Dead point method
3- Equivalent Pipes Method.
4- Newton-Raphson Method
5- Linear Method
6-Turkish Bank of Provinces Method for Designing Drinking Water Networks.

5.3.1 Hardy-Cross Method

Analysis of a pipe network is essential to understand or evaluate a pipe network system. In a branched pipe network, the pipe discharges are unique and can be obtained simply by applying discharge continuity equations at all nodes. However, in case of a looped pipe network, the number of pipes are too large to find the pipe discharges by merely applying discharge continuity equations at nodes. The analysis of looped network is carried out by using additional equations found from the fact that while traversing along a loop, as one reaches at the starting node, the net head loss is zero. The analysis of looped network is involved, as the loop equations are nonlinear in discharge.
Hardy Cross (1885–1951), who was professor of civil engineering at the University of Illinois, Urbana-Champaign, presented in 1936 a method for the analysis of looped pipe network with specified inflow and outflows (Fair et al., 1981). The method is based on the following basic equations of continuity of flow and head loss that should be satisfied:

1. The sum of inflow and outflow at a node should be equal:

$$\sum Q_i = q_j \qquad \text{for all nodes j=1, 2, 3, . . . , } j_L \tag{5.1}$$

where Q_i is the discharge in pipe i meeting at node (junction) j, and q_j is nodal withdrawal at node j.

2. The algebraic sum of the head loss in a loop must be equal to zero:

$$\sum_{loopk} K_i Q_i \left| Q_i \right| = 0 \qquad \text{for all loops k = 1, 2, 3, . . . , } k_L, \tag{5.2}$$

where

$$K_i = \frac{8 f_i L_i}{\Pi^2 g D_i^5} \tag{5.3}$$

where i = pipe link number to be summed up in the loop k.

In general, it is not possible to satisfy Eq. (5.2) with the initially assumed pipe discharges satisfying nodal continuity equation. The discharges are modified so that Eq. (5.2) becomes closer to zero in comparison with initially assumed discharges. The modified pipe discharges are determined by applying a correction ΔQ_k to the initially assumed pipe flows.

Thus,

$$\sum_{loopk} K_i (Q_i + \Delta Q_k) \left| (Q_i + \Delta Q_k) \right| = 0 \tag{5.4}$$

Expanding Eq. (5.4) and neglecting second power of ΔQ_k and simplifying Eq. (5.4),

the following equation is obtained:

$$\Delta Q_k = -\ \frac{\sum_{loopk} K_i Q_i \left| Q_i \right|}{2 \sum_{loopk} K_i \left| Q_i \right|} \tag{5.5}$$

Knowing ΔQ_k the corrections are applied as

$$Q_{inew} = Q_{old} + \Delta Q \qquad \text{for all k} \tag{5.6}$$

5.3.2 Dead point method

This method is used by the Turkish Bank of Provinces. In this method, water is assumed to be withdrawn not only from the joints but also along the pipes. Therefore, if a pipe receives water from both ends, then at an intermediary point the flow is totally depleted. Such a point with zero discharge value is called a dead point. The design procedure is as follows:

1) The maximum discharge, Q_{max}, to be withdrawn by the network is determined

2) A layout is selected for the distribution system and relative densities k_i, are assigned to the pipes according to the quantity of water withdrawn along the pipe. For example, if 5 lt/s and 25 lt/s discharges are withdrawn along two different pipes, then $k_1 = 1$ and $k_2 = 5$ may be assigned to these pipes, respectively.

3) Possible dead points are identified.

4) Relative lengths, $L_i^{'}$ of the pipes are computed from

$$L_i^{'} = k_i L_i \quad (5.7)$$

where L_i is the actual length of a pipe. The maximum discharge withdrawn from unit relative length of pipe is determined from:

$$q = \frac{Q_{max}}{\sum_{i=1}^{n} L_i^{'}} \quad (5.8)$$

5) The discharge in any pipe, P_i, is computed from $P_i = q L_i^{'}$ (5.9)

6) Starting from the dead points and proceeding in the upstream direction, the discharges are determined at the upstream and downstream ends of the pipes as Q_b and Q_u, respectively

$$(P_i = Q_b - Q_u) \quad (5.10)$$

7) The design discharge, Q_i is determined for each pipe from

$$Q_h = Q_u + 0.55P + Q_f \quad (5.11)$$

where Q_f is the fire discharge.

8) The pipe diameter is selected such that the average pipe velocity is between the minimum and maximum allowable values, i.e., 1 m/s<u<2 m/s. The pipe diameter is computed considering the maximum velocity under the design discharge, Q_i and the next available size is selected. Therefore, the average pipe velocity under the design discharge becomes smaller than the maximum velocity.

9) Headlosses along the pipes are computed using a suitable equation.
10) Following each possible path, the total heads at both sides of the dead points are computed. For an acceptable design, the head difference at both sides of the dead point is to be less than 1 m. If this criterion is not satisfied, the pipe diameters are to be changed.
11) The pressure heads at the junctions are checked to observe if they fall in the range between the minimum and maximum allowable values. If this criterion is not satisfied, the pipe diameters are to be changed.
The deficiency of the dead point method lies behind the fact that the discharges on both sides of the dead point may be different from each other. Therefore, pipe sizes may be different on both sides of the dead point.

5.3.3 Equivalent Pipe Methods

A complex network can be replaced by a simple and hydraulically equivalent pipe having the same discharge and headloss. Let us consider the following parallel pipe system shown in Figure 5.2. This system can be replaced by a single pipe having a length and diameter Leq, and Deq, respectively. For a parallel pipe system, since there exists a unique energy level at each junction, headlosses are the same and discharges are additive for the pipes. If the discharge is expressed using the Darcy-Weisbach equation,

$$\sqrt{\frac{g\,\Pi^2\,D_{eq}^5 h_f}{8f_{eq}L_{eq}}} = \sqrt{\frac{g\,\Pi^2\,D_1^5 h_{f1}}{8f_1L_1}} + \sqrt{\frac{g\,\Pi^2\,D_2^5 h_{f2}}{8f_2L_2}} +..+ \sqrt{\frac{g\,\Pi^2\,D_n^5 h_{fn}}{8f_nL_n}} \qquad (5.12)$$

Assuming that the friction factors are almost the same for each pipe and after simplification,

$$\sqrt{\frac{D_{eq}^5}{L_{eq}}} = \sqrt{\frac{D_1^5}{L_1}} + \sqrt{\frac{D_2^5}{L_2}} +....+ \sqrt{\frac{D_n^5}{L_n}} \qquad (5.13)$$

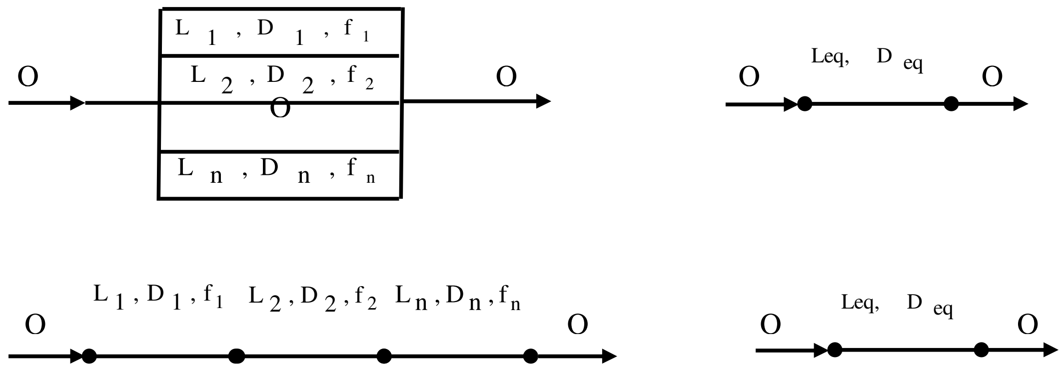


Figure 5.2 Pipes in parelell and series

For the analysis of the pipes in series, Figure 5.2 can be considered. In this case, the discharges are the same, but headlosses are additive:

$$\frac{8f_{eq}L_{eq}}{g\Pi^2 D^5{}_{eq}}Q^2 = \frac{8f_{eq}L_{eq}}{g\Pi^2 D^5{}_{eq}}Q_1{}^2 + \frac{8f_{eq}L_{eq}}{g\Pi^2 D^5{}_{eq}}Q_2{}^2 + \ldots.+ \frac{8f_{eq}L_{eq}}{g\Pi^2 D^5{}_{eq}}Q_n{}^2 \qquad (5.14)$$

Assuming that the friction factors are almost the same for each pipe and after simplification,

$$\frac{L_{eq}}{D^5{}_{eq}} = \frac{L_1}{D^5{}_1} + \frac{L_2}{D^5{}_2} + \ldots.+ \frac{L_n}{D^5{}_n} \qquad (5.15)$$

In Equations (5.13) and (5.15), there are two unknowns, namely Leq and Deq. Therefore, a value is assumed for one of the unknowns and the other one is determined from these equations. The method of equivalent pipes may be applicable in the hydraulic analysis of a complex network. However, it should be noted that this method couldn't be applied in case of loops having a diagonal element and flow takeoff or take in from or into the joints, respectively.

The procedures outlined above consider steady demand pattern. To take into account the variations in tank levels and their effects on pressures, extended period simulation (EPS) is to be applied.

5.3.4 Newton-Raphson Method:

Since the pressure drop equations are non-linear equations they must be linearized before they can be inserted into the matrix to arrive at the final solution. To accomplish this, Newton-Raphson technique is used. Simply stated, the function describing the loop pressure drop is divided by the first derivative of that function. The result is a first order or linear equation that can be used in the matrix calculation. This method needs smallest number of unknowns and equations of formulations. But, this method faces convergences problem, because of poor initial guesses for nodal.

5.3.5 Linear Method

The linear theory method is another looped network analysis method presented by Wood and Charles (1972). The entire network is analyzed altogether like the Newton–Raphson method. The nodal flow continuity equations are obviously linear but the looped head-loss equations are nonlinear. In the method, the looped energy equations are modified to be linear for previously known discharges and solved iteratively. The process is repeated until the two solutions are close to the allowable limits

5.3.6 Turkish Bank of Provinces Method for Designing Drinking Water Networks.

Drinking water study and projects: it is prepared with respect to bank of provinces' city and town drinking water regulations. Reports for projects (hydraulics and explanation reports) are prepared. If the projects are to be carried out by a single stage, then hydraulic report is not necessary; that is explanation report is sufficient. In the reports following subjects are examined:Administrative, geological and historical state, Social economical and cultural state, Maps and cadastral plans (street Plans), Geological state, Energy and drinking water service state.

Future population is projected by taking into account the past population data and projection of the project. Water need is estimated by considering future population as well as special flow rates, number of animal, and fire discharge. Developing regions and end discharge boundaries are determined.

Turkish Bank of Provinces Method

$$Ç=(\sqrt[a]{N_y/N_e}-1)x100 \tag{5.16}$$

At this formula

Ç = Increment Constant

N_y = New population number

N_e = Old population number

a= Number of years between new and old years

$$N_g = N_y * (1+Ç/100)^{30+5+n} \tag{5.17}$$

N_g = Future population number

n= Number of years between future population number and last population number

The town is examined with respect to the climate, flow rate of the near water sources, jeological, physical, chemical, biological analyses, and legal state.

Water supply system is chosen according to the elevation of the town. If the elevation of the source is higher than city, gravitational method is chosen; otherwise water is supplied by pumping. Sometimes one part of the water supply systems is solved by gravity and one part by pumping. The path of the water supply is abstained from passing through the regions under overflowing and landsliding risks. If it is not completely avoided, then protective measures should be taken carefully.

Based on the town's minumum and maximum elevations, and topographical state network elevation flat regions are defined. Volume of the network resorvoirs and their elevations are determined for each flat. For the place of the resorvoirs impressment status is evaluated.

Design of reservoir.

Pumping system

$$V = \frac{Q * 86400}{4000} + V_{fire} \tag{5.18}$$

Gravitional system

$$V = \frac{Q * 86400}{3000} + V_{fire} \tag{5.19}$$

Where V= volume of reservoir, V_{fire} = volume of fire

If there is more than one transmitting and network system available, the economical one is selected due to the fact that the system is to start running by the tecnicians in municipality.

After the land survey, the lenghts of the pipe lines and elevation of the establisment becomes certain. Based on this values, calculations are modified and selected plan is overviewed. The characteristics of the establishment are defined. Standard projects for the source stores, small resorvoirs, reservoirs, and other art buildings are prepared. The place of the reservoirs and drinking water treatment plant are surveyed according to the geological status.

As a source, each spring, drainage, subway, deep well, and small lake are investigated and projected. If the physical, chemical, and bacteriologic properties of the water are not eligible to the Turkish Drinking Water Standards, water is supplied after treatment. Drinking water should be avoided from getting dirty. For this purpose, the place of the sources store is conserved. The size and type of the safe-keeping depends on the state of the reservoir.

Gravity and pumping systems are compared with respect to their technical and economical conditions. Calculations on the comparison of the two systems are presented. In the transmitting systems, air relief valves, air shafts, blow of valve, bridge and road transistors, and fixing elements can be used. In pumping systems; water lifting center, blow which digs central, balance chimney may be used.

The type, diameter, and bar of the pipe is calculated based on the maximum pressure for pumping systems and maximum static pressure for gravity systems. If the economic conditions play a vital role, then bar of the pipes can be levelled. In leveling process, attension should be given because pipe type and operating pressure should only be allowed to change at certain points such as air relief valve, air shaft, etc.

At some points of the transmitting system the pressure may be excessive. At this points, for economical purposes, if the transmitted flow rate is enough, small reservoir or pressure-reduced-valves can be used in order to reduce the pipe pressure. The input pressure is reduced, so that wearing of the elements are prevented.

It is aimed that keeping the maximum static pressure of the system below 80 m. To achieve this, it is economic and safe to use 10-atu-pipes. In some downward and upward sloping places where subscriber's lines are not to be installed and pressures are higher than 100 m., differens pressure (atu) pipes can be used.

If the pipes are passed through the aggressive water and surfaces, precautions required against corrosion. Water supply system is chosen according to the elevation of the town. If the elevation of the source is higher than city, gravitational method is chosen; otherwise water is supplied by pumping. Sometimes one part of the water supply systems is solved by gravity and one part by pumping. The path of the water supply is abstained from passing through the regions under overflowing and landsliding risks. If it is not completely avoided, then protective measures should be taken carefully.

In our country Dead point method is used by Turkish Bank of provincies. Dead point method is easy for small networks. However, application to the large networks requires experiences. Design Criteria of Drinking Water Network System are illustrated on Table 5.1 (İller Bankası Şehir ve Kasaba İçmesuyu Projelerinin Hazırlanmasına ait Yönetmenlik, 1998)

Table 5.1 Design Criteria of Drinking Water Network System

Characterics of drinking water network systems			Fire Discharge			Number of Fire and Volume of Fire at reservoir			Pressures		Difference of pressure at Dead points (m)	Areas of Loops (ha)	Minumum diameter of Base pipe and Minumum diameter of secondary Pipe at drinking water network systems	
(After 35'years) Future Populations N:			At Main Pipe Qfire (lt/sn)	At Base Pipe Qfire (lt/sn)	At secondary Pipe Qfire (lt/sn)	At Simultaneously Number of Fire	Duration of Fire (hour)	Volume of Fire discharge at reservoir (m^3)	Minimum Operating pressure (m)	Maximum Statik Pressure (m)				
	N <=	10,000	5	5	2.5	1	2	36	20	80	1	20	Secondary Pipe	65-70
												30	Base Pipe	80
10,001	<= N <=	50,000	10	5	2.5	2	2	72	20	80	1	20	Secondary Pipe	80
												30	Base Pipe	100
50,001	<= N <=	100,000	20	10	5	2	5	360	30	80	2	20	Secondary Pipe	100
												30	Base Pipe	125
	N >=	100,001	20	10	5	2	5	360	30	80	2	20	Secondary Pipe	100
												30	Base Pipe	150

5.4 Water Distribution Network Design Methods Using Netcad

5.4.1 Netcad

Netcad; is a complete software, started to be developed in 1987, and has been meeting the requirements of public and private sector for over 20 years, holding various professional solutions within its structure. It has been produced totally with the equity resources of the firm and has no license dependency. Netcad produces software and system solutions, compatible with world standards, capable of meeting local requirements as well. Netcad is a company exporting software, carrying its national success to the international fora.

Netcad makes effort to keep its product and service quality understanding always at top level with the ISO 9001 quality certificate it holds. Its Capability maturity model Integration (CMMI 3) level studies are still ongoing. Netcad proves that it has acquired world standards on the subjects of Geographical Information Systems with the Web Map service (WMS), Web Future service (WFS), Simple Future specification (SFS) certificates received from Open Geospatial Consortium (OGC), where Netcad is an associate member of it

Netcad has offered sound and common solutions on many subjects such as almost all Zoning, Land Survey, Eminent Domain, Planning, Highway Project Designing, current cartography etc. carried out in public institutions and enterprises in Turkey for more than 20 years.

5.4.2 Netcad/ Water

Netcad/Water Module is a potable water project design module providing complete solution in open and closed network systems with gravity feed, pumping, air pressure tank, and Hardy Cross Methods. Netcad is especially used by Municipalities, Provincial Administrations, Public Institutions, Universities, Engineering, Mapping, Construction

5.4.2.1. Netcad/Water module by Using dead end point method.(Open and Closed Network Solution)

Solution steps are as follows:

• According to the digital zoning public and topographic plans; the pipes can be drawn on streets and avenues with their stages as main, auxiliary and existing.

• The drawings in graphic display can be transferred to the tree structure with a single process with coordinate and elevation values.

• Stage and open pipes controls and geometrical controls.

• Automatic calculation of pipe diameters and network flow rate.

• Network Calculation Table, Calculation Plan and Construction Plan Drawings.

• User can properly place the simplified principle schema as the basis of the Calculation Plan Profile.

• The dead-end locations in the closed systems are automatically determined and relocated to a user defined place as well.

• Controlling velocity in the pipe and pressure in the nodal points and required notifications are reported to the user, in the network solutions.

• Excavation/fill calculations according to the pipe diameters used. Reports and defining the maximum closed loop area.

• Existing structure change when required. Creating new solutions by adding new pipe and construction works to the existing structure.

• Tank volume calculations according to the population.

• Pipe types can be selected from list or the user can define new pipes.

• Automatic head loss calculation according to the pipe types.

• Pipe inventory reports.

• Automatic Node detail drawings and armature drawings generation.

• By starting from a specific number, systematic number generation to each node.

• Armature Load Reports.

5.4.2.2. Netcad/Water 2005 module by Using Hardy Cross Method (Open and Closed Network Solution)

Solution steps are as follows;

• Hardy Cross module can monitor the flow rate in each pipe, pressure in each node point, height of the water present in the tanks, and the concentration of the chemicals.

• Hydraulic analysis engine for frictional head loss in the network by using Hazen-Williams, Darcy- Weisbach ve Chezy-Manning formulas.

• Hydraulic Simulation Model can calculate the loads and connection flows at the node points for the reservoir levels, tank levels and the fixed state of the water demands on the points and lines.

• Pipe diameters can be calculated automatically.

• Pipe diameter and roughness values are taken automatically from the pipe types selected from the pipe library. State parameters ensure that the pipes include passage valves and that the control for ensuring that the valves pass the flow only one direction.

• Energy consumption and value of the pump are calculated, efficiency curve of each pump and energy cost graphics are generated.

• Pump Curve describing the relation between the water load and water flow rate, Volume Curve describing the way of change of the tank volume in the water level function and Head Loss Curve are drawn.

• Within the defined time criteria, separate demand course can be formed for each node point, and the time dependent change of the demand values at the node points can be modeled. The demand course of the network in each time period can also be displayed, and simulated in the graphic display.

• Before starting the solution, it is possible to make the analysis settings including hydraulic settings, quality settings, reaction settings, duration settings and energy settings.

• A calibration file is generated that ensures the comparison of the measured area data and simulation results.

• Each drawing network element is automatically drawn with digitization menu.

• By digitization menu GIS connection is automatically established, feature tables can be filled for each network element transferred to drawing and stored in the database

• Each network element is classified automatically and graphic data is accessed with table queries.

• Reference manager handles the graphics features of the object based classes used in the potable water project. The image settings in analysis process of this menu perform the Label and Coloring processes according to the pre-determined parameters.

• All analyses are reported on time steps, the average of the values according to time, the lowest values, highest values, difference of the highest and lowest values

• Analyses are reported in Word, Excel and html formats.

• Reports on object classes can be generated for each network element stored in the database.

• Pipe Reports: Pipe Analysis Result, Pipe Information, Pipe Calculation Table, Time-Dependent Change Graph of the Flow in the Pipes, Time-Dependent Change Graph of Flow Velocity in the Pipes.

• Node Point Reports: Node Points Information, Node Points Analysis Results, Time-dependent Change Graph of the Pressure in the Node Points, Time-dependent Change Graph of the Pressure of the Total Head in the Node Points.

• Tank Reports: Tank Information, Tank Analysis Results, Time-dependent Change Graph of the Pressure in the Tanks, Time-dependent Change Graph of the Net Coming Flow in the Tanks.

• Pump Reports: Pump Information, Pump Analysis Results, Time-dependent Change Graph of the Flow in the Pumps, Time-dependent Change Graph of the Unit Head Loss in the Pumps.

• Valve Reports: Valve Information, Valve Analysis Results, Time-dependent Change Graph of the Flow in the Valves, Time-dependent Unit Head Loss in the Valves, and Time-dependent Change Graph of the Flow Velocity in the Valves.

• Reservoir Reports: Reservoir Information, Reservoir Analysis Results.

CHAPTER VI

PRENSENTATION OF DRINKING WATER NETWORK DESIGN USING NETCAD

6.1 Introduction

This study is the solution and comparison of results in drinking water networks at open and closed network systems by using Netcad program. Water module which is one of Netcad module works according to the method of a dead point that is used by the rule of the Turkish Bank of Province. To be able to do a network solution, Netcad/Water module needs a numerical map. Branched (Open) and Looped (closed) networks are solved with Nectad/Water via the numerical map which is taken from Rural service of Kahramanmaraş (Appendix 9)

As the water shortage in our country is becoming more apparant and water resources are coming to end, appropriate and prolific usage of the water becomes more significant. This can be achieved by efficient and successive usage of the Netcad /Water 2005 module.

By using Netcad/Water 2005 module, periodic simulation of the hydraulic and water quality behaviour of the pressurized pipe networks can be performed. This system consists of pipes, tanks, pumps, reservoirs, valves, and objects for nodal points. The characteristic values of these objects are stored in seperate databases. During the quantification, these characteristic values of these objects can also be manualy inputted by using the land model, planch map, or coordinate data. After data generation, many simple or complex queries can be performed using graphs and generated database. The prepared data can effectively be used in GIS analyses. At node points in

areas (industrial zones, hospital districts, residental areas, school campuses) during the realization of hardy cross solutions, designing district demand movements and adjusting water flow with regard to spatial and timing selections are possible, also enabling production and monitoring of simulations of water condition at node points in desired time intervals.
Especially in the world and in our country, the water shortage is in the agenda in these days, so the GIS data production and GIS analysis of water demand management is extremely important.

6.2 Population and Discharge Projection Analysis of the Project

The design of a municipal water supply system is initiated by the estimation of the projected population corresponding to the end of the lifetime of the system. In Turkey, municipal water supply systems of small to medium cities are designed and constructed according to the codes of the Turkish Bank of Provinces, whereas State Hydraulic Works undertakes this duty for large cities. Municipal water demand is the combination of the requirements for domestic, public, commercial and industrial uses, and fire fighting. Domestic use includes the water demand for drinking and sanitary purposes. Population size and living standards affect the domestic use. Therefore, it is a better approach to express the water demand on a per capita basis, which reflects the differences in water uses with respect to districts. Domestic water consumption per capita may reach and exceed about 1000 lt/cap/day in developed countries.
Present population of project issued that;
N_{2007}=850 person
Population projection for the year 2042 (35 years later)
Demand for the year 2042
N_{2042}=2125 person
Total Discharge Q_{total} = 4 lt/sn ($Q_{total} = Q_{people} + Q_{animal} + Q_{specify}$)

6.3 Solution Open And Closed Drinking Water Network Systems By Using Netcad/Water Module

In order to make a solution with Netcad Water module, firstly a numerical map is required. The numerical map of Hacıağalar Town is obtained from the Rural services of Kahramanmaraş city. In this thesis, a network solution projected in single floor is possible. Then, the network system is composed. In a network system, main pipes, base pipes and secondary pipes are orderly represented. If an available pipe is present at the network, it is possible to define and include this pipe in the generated network using Netcad Water module in (Figure 6.1)

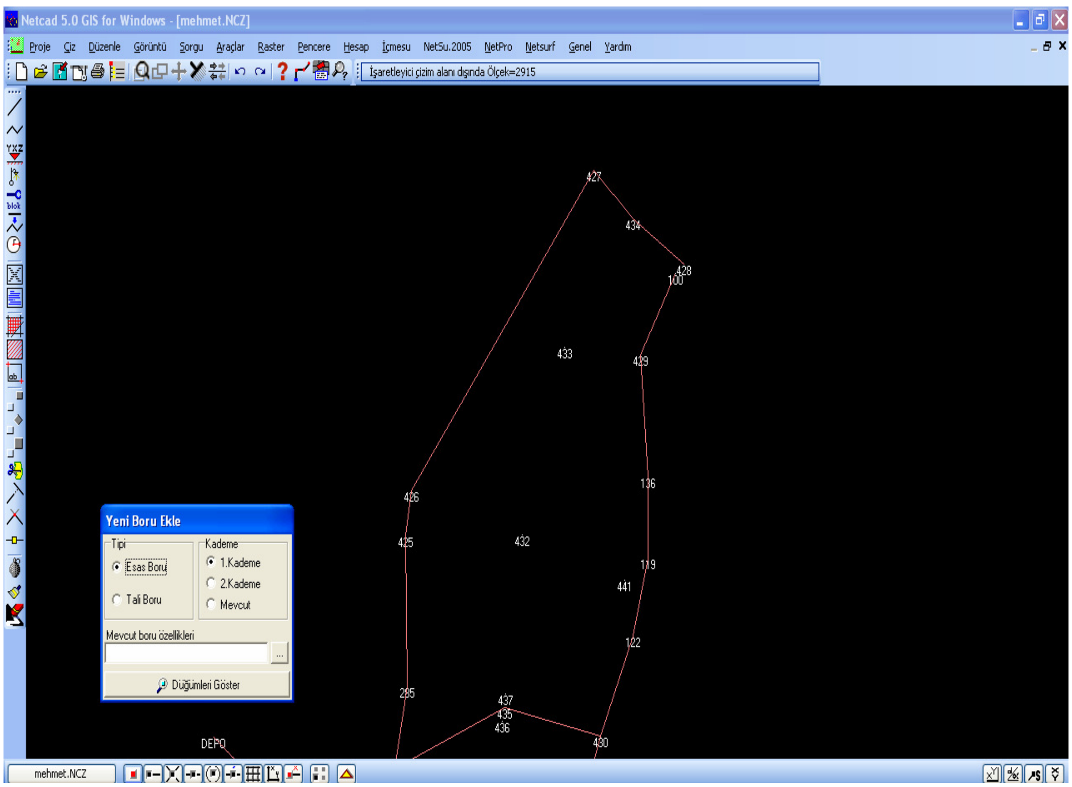


Figure 6.1 Presentation Pipe Network

After introduction of the drinking water network system, the volume of reservoir is calculated.(Figure 6.2)

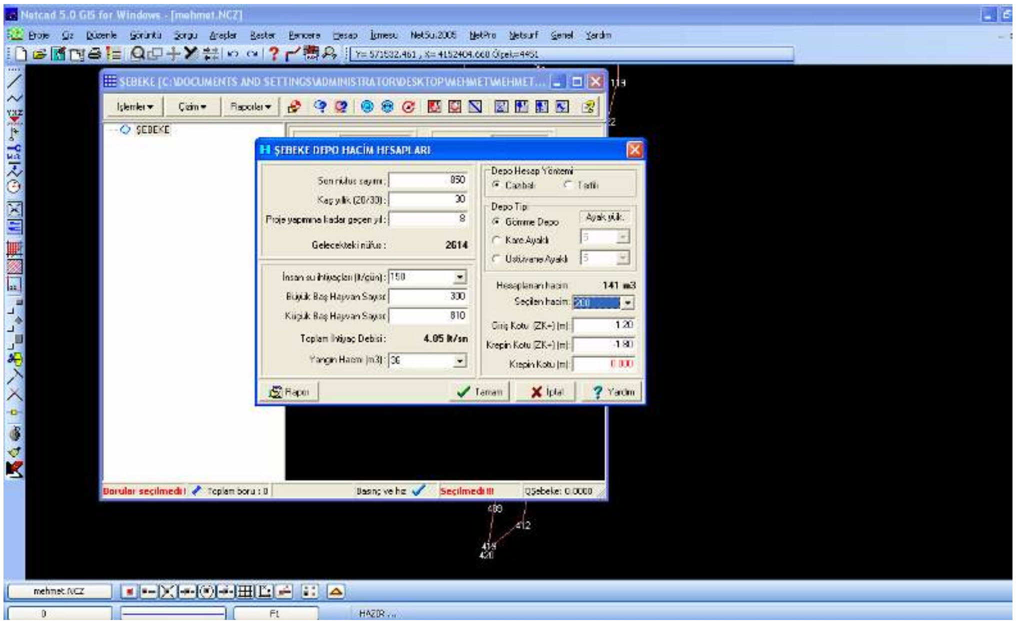


Figure 6.2 Calculation of Reservoir Volume

Network solution is realized according to Turkish Bank of Provinces and Rural Services criteria for pipe selection. (Figure 6.3)

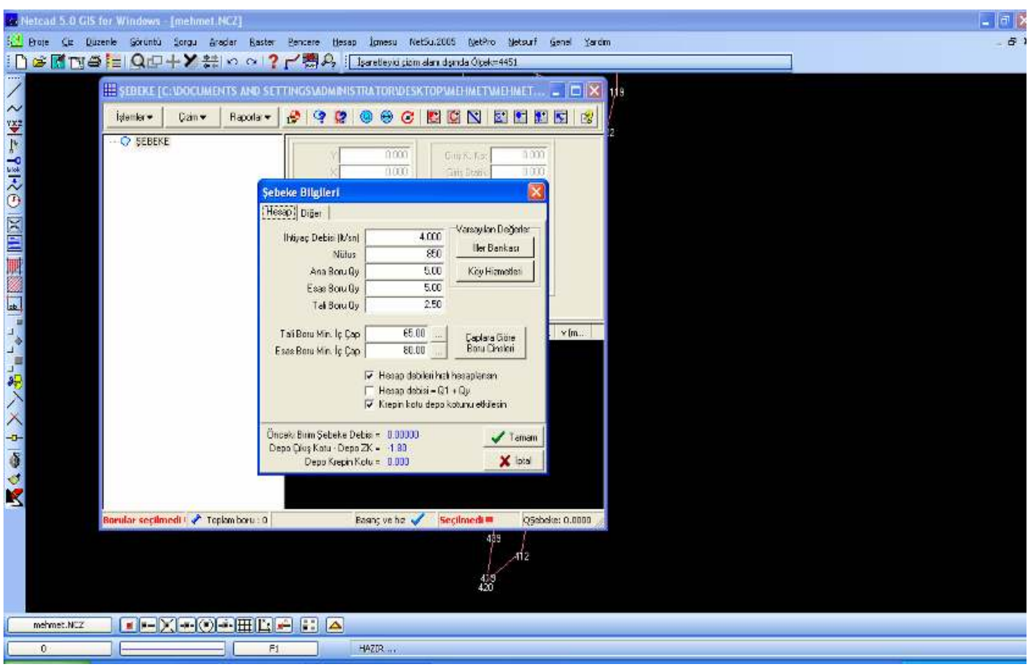


Figure 6.3 Selection of Pipe Network Criterias

Drawing control can be made after network solution with Netcad.(Figure 6.4) During the process, pipe level, pipes on surface and pipe altitudes control can be made.

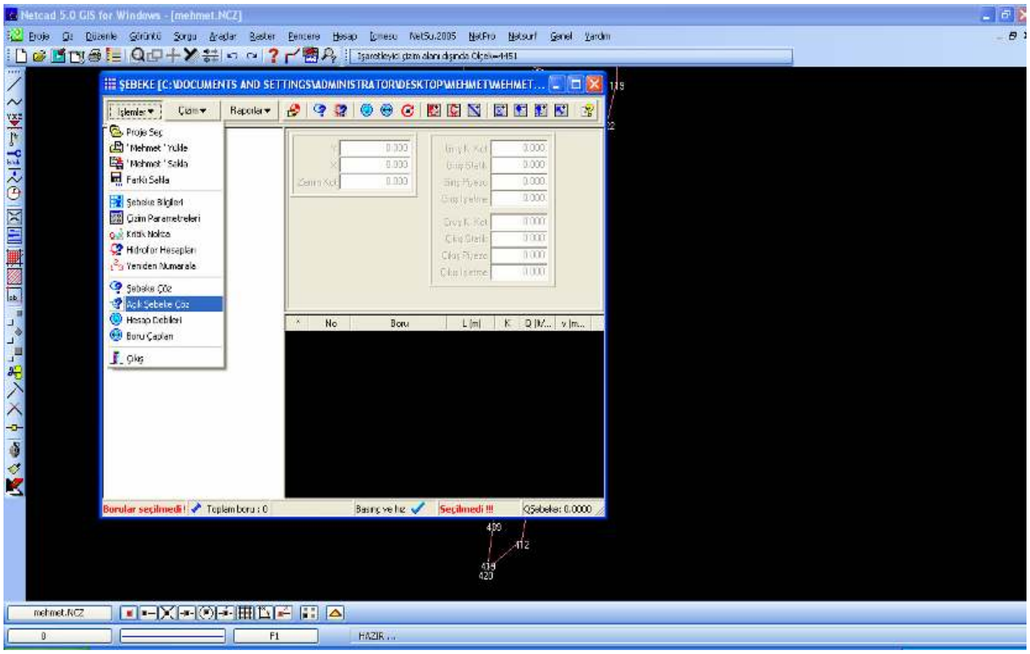


Figure 6.4 Solution Of Branched (Open) Netwok

In the next step, Netcad solution is controlled.

From Netcad menus;

Processes -→ The control of design flow

Processes -→ The control of pipe diameter are done.

Reports -→ Control -→ System -→ Pressure and Velocity Control are done. (see Figure 6.5)

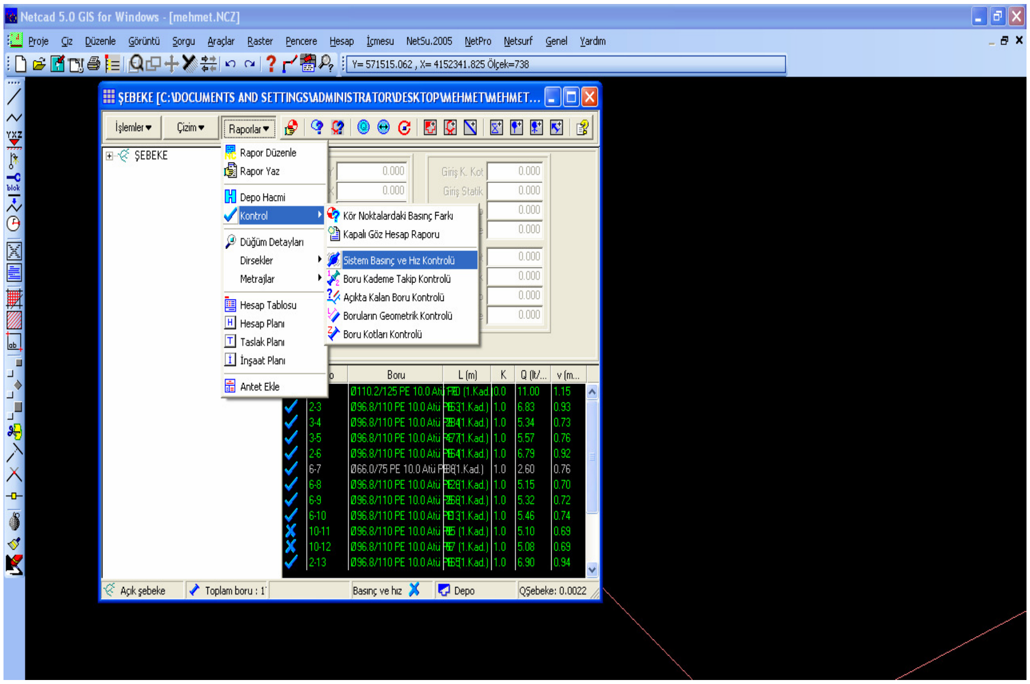


Figure 6.5 Controlling of Water Distribution Network

After such corrections as pressure breaking required for pipe diameter and pressure are done, solution is made, again. In next step, project drawings are done. Netcad/water makes branched and looped network solutions with respect to dead point method. Netcad/water makes solutions in parallel with pipes sold in market, while calculating pipe diameter. Speed is required to be between 0,80-1 m/s. (Appendiks 2 and Appendix 4) (the optimum speed is 1 m/s) in network solutions. In Netcad open and close network solutions, it is seen that only losses and velocity change very little in pipes whose lengths are equal. (Table 6.1)

Table 6.1 Comparison of Open and Closed Network Solutions

Pipe No		Pipe Length L (m)	Diameter. D (mm)		Head Loss (m)		Velocity	
Open Network	Closed Network		Open Network	Closed Network	Open Network	Closed Network	Open Network	Closed Network
17-18	9-10	92	55.4	55.4	2.0198	2.0179	1.08	1.08
17-19	9-11	118	55.4	55.4	2.6471	2.6440	1.1	1.1
13-17	12-9	56	96.8	96.8	0.3917	0.4173	0.83	0.86
14-15	13-14	89	55.4	55.4	1.9477	1.9459	1.07	1.08
14-16	13-15	67	55.4	55.4	1.4387	1.4378	1.07	1.07
13-14	12-13	58	66	66	0.6637	0.6616	0.85	0.85
2-13	2-12	113	96.8	96.8	0.9555	1.0094	0.92	0.95
4-5	4-5	84	55.4	55.4	1.8317	1.8301	1.08	1.08
4-6	4-6	203	55.4	55.4	4.8862	4.8767	1.14	1.14
4-7	4-7	98	55.4	55.4	2.1624	2.1602	1.09	1.09
3-4	3-4	85	66	66	1.3311	1.3230	1.01	1.00
2-3	2-3	166	96.8	96.8	1.4709	1.4469	0.94	0.93
1-2	1-2	155	110.2	110.2	1.7141	1.7028	1.15	1.15

When network solution which is made according to dead point method is done in excell by determining V=1 m/s, it is seen pipe diameters fully change (Appendix 3 and Appendix 5). However diameters found in solutions are not the same as one sold in market, thus leading demand for special production and increase in expense. As diameters in solution are found in market, difficulties are encountered in production process. However, solution of looped drinking water network is preferred more than branched drinking water network and also solution of closed drinking water network has more advantages than open drinking water network.

On the other hand, solution with Netcad/water 2005 module is done in the present network. Solution made in open network by using Netcad/water module is compared.

6.4 Solution of open and closed drinking water network systems by using Netcad/water 2005 module

In Netcad water 2005,

1. Water height, volume of reservoir, entering water levels etc. are defined.

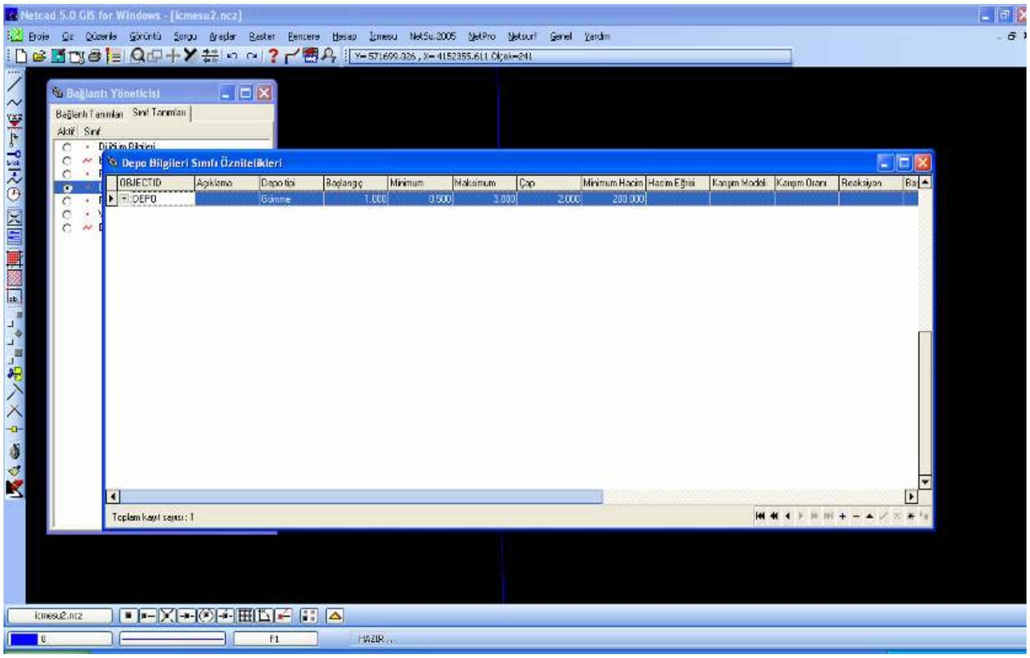


Figure 6.6 Determination of Reservoir Characteristic

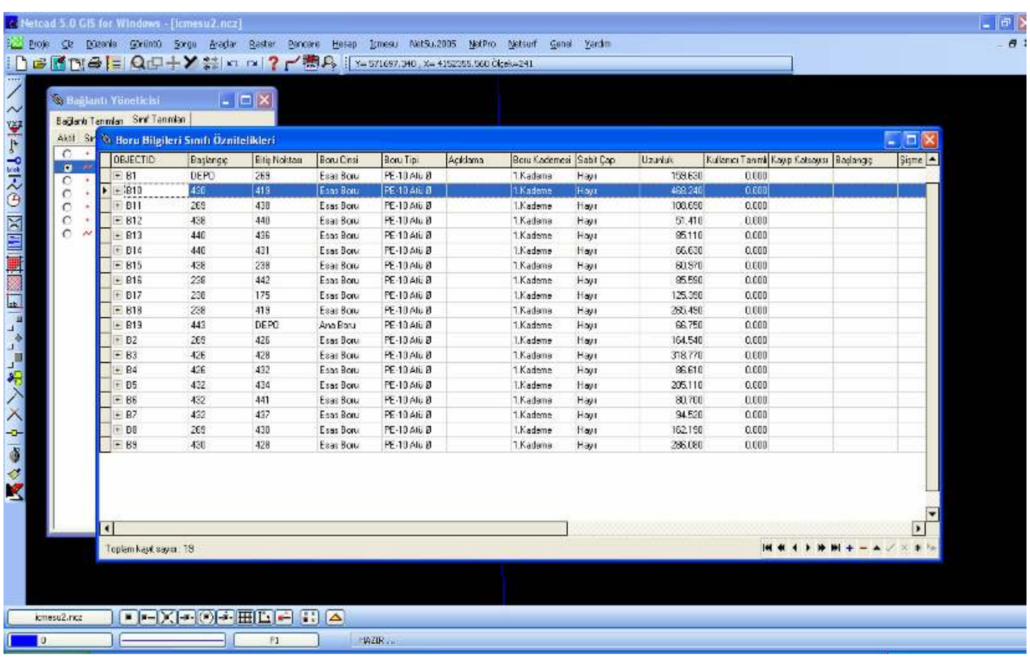


Figure 6.7 Determination of Characteristic of Pipes at the Network

2. Pipes discharges are determined. (Table 6.2.)

Table 6.2 Determination of Pipe Discharge

Pipe No	Pipe is started from Node	Pipe is finished from Node	Pipe Length, L, (m)	qi, lt/m/s/m	Discharge in Pipe, Pi= q_i*L	Fire Dicharge, Qf	Discharge İn pipe + Fire discharge= Pi+Q_f
B1	Reservoir	269	159.63	0.0021	0.3331		0.3331
B2	269	426	164.54	0.0021	0.3433		0.3433
B3	426	428	318.77	0.0021	0.6651		0.6651
B4	426	432	86.61	0.0021	0.1807		0.1807
B5	432	434	205.11	0.0021	0.4280		0.4280
B6	432	441	80.7	0.0021	0.1684		0.1684
B7	432	437	94.52	0.0021	0.1972		0.1972
B8	269	430	162.19	0.0021	0.3384		0.3384
B9	430	100	286.08	0.0021	0.5969		0.5969
B10	430	420	468.24	0.0021	0.9770		0.9770
B11	269	438	108.69	0.0021	0.2268		0.2268
B12	438	440	51.41	0.0021	0.1073		0.1073
B13	440	436	85.11	0.0021	0.1776		0.1776
B14	440	431	66.63	0.0021	0.1390		0.1390
B15	438	238	60.97	0.0021	0.1272		0.1272
B16	238	442	85.59	0.0021	0.1786		0.1786
B17	238	175	125.39	0.0021	0.2616		0.2616
B18	238	419	265.49	0.0021	0.5539		0.5539
Total Drinking water Network Length			2875.67				

3. After calculation of the flow rates of the pipes, for dynamic modeling purposes, flow rate at the nodes are defined by using half-pipe-lenght approach that is flow rate at a node is calculated as the summation of the half of the flow rates of all pipes connected to that node. (see Figure 6.8 and Table 6.3)

Table 6.3 Determination of Discharge at the Node Points

Node No	Pipes at Node	Discharge	Total Discharge at the Node	Fire Discharge	Total discharge
A	B	C	D= 0.5*C		
269	B1+B2+B11+B8	1.24	0.62		0.62
426	B2+B3+B4	1.19	0.59		0.59
432	B4+B5+B6+B7	0.97	0.49	10	10.49
434	B5	0.43	0.21		0.21
428	B3+B10	1.64	0.82		0.82
436	B8	0.34	0.17		0.17
441	B6	0.17	0.08		0.08
430	B8+B9+B10	1.91	0.96		0.96
437	B7	0.20	0.10		0.10
440	B13	0.18	0.09		0.09
431	B14	0.14	0.07		0.07
438	B11+B12+B15	0.46	0.23		0.23
442	B16	0.18	0.09		0.09
175	B17	0.26	0.13		0.13
420	B10+B18	1.53	0.77		0.77
238	B16+B17+B18	0.99	0.50	10	10.50

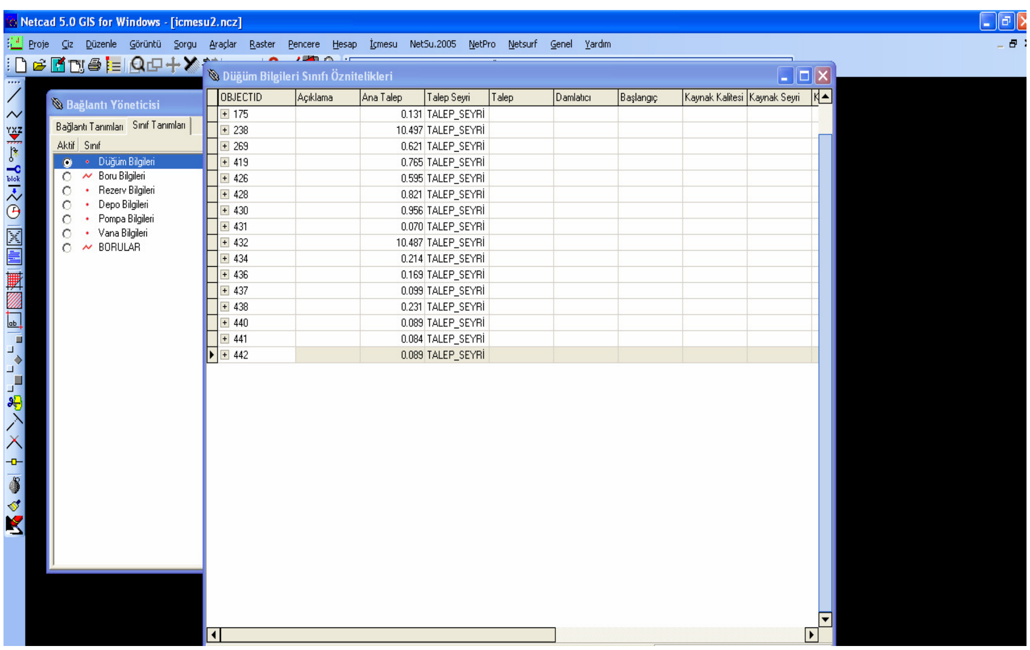


Figure 6.8 Discharges at Node Points.

4. End of pipe discharge definition is not done, assuming there is no end flow in project.

5. Maximum hour in a day is taken into consideration for determination of diameters which will be used in network solutions and pipe diameters are controlled with peak hour coefficient. (Table 6.4)

Table 6.4 Coefficient of Peak Hourly Demand

Time	Coefficient of Peak Hourly of Demand	Time	Coefficient of Peak Hourly of Demand
01:00	0.80	13:00	1.08
02:00	0.60	14:00	1.08
03:00	0.55	15:00	1.05
04:00	0.55	16:00	0.96
05:00	0.60	17:00	1.03
06:00	0.72	18:00	1.01
07:00	1.01	19:00	1.20
08:00	1.06	20:00	1.25
09:00	1.09	21:00	1.20
10:00	1.14	22:00	1.01
11:00	1.13	23:00	0.88
12:00	1.08	24:00	1.00

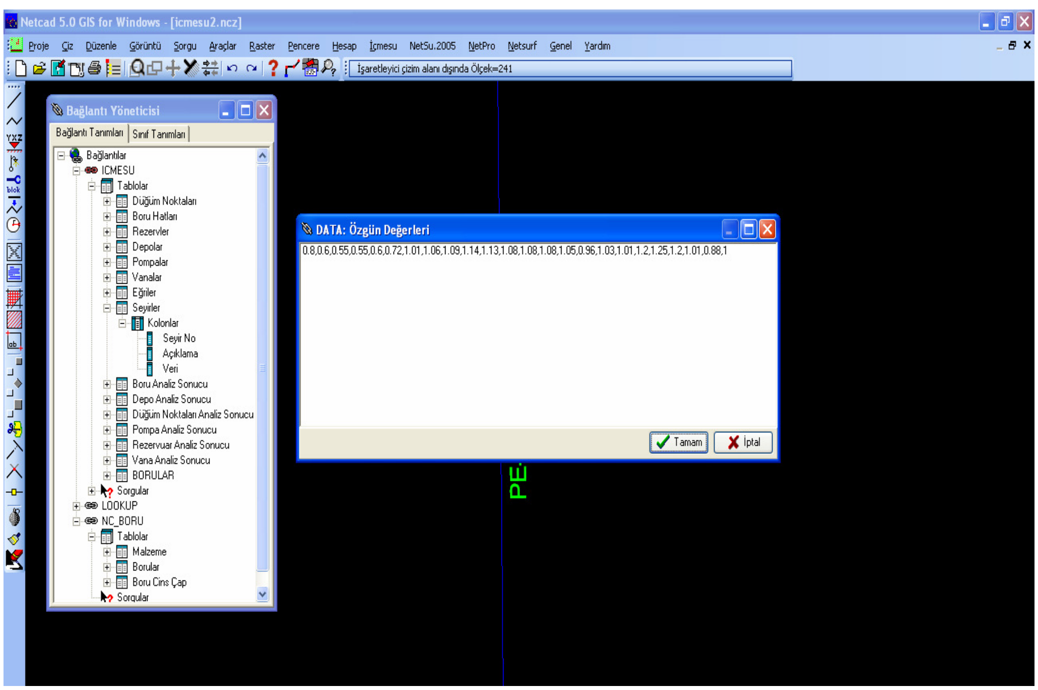


Figure 6.9 Peak Hourly Demand Coefficient

Hourly changes are monitored by taking hour interval in Bank of Provinces General Management Kilis (Merkez) Drinking Water Project. Network's movement is interrogated by adding fire flow supplement to critic points in drinking water network which are 432nd and 238 th node points, regarding the amounts in Turkish Bank of Province's city and town drinking water preparation project regulations. (Kilis (Merkez) İçmesuyu Kesin Projesi 2008)

6. First calculations should be made, regarding coefficients in time intervals.
7. Diameter optimazation is made with respect to velocity criteria, depending on flows that pipes should move. (Table 6.5)
8. Determination of Reynolds number and Flow type are given on Table 6.6
9. Output of pipe discharge and pipe velocity according to Netcad/water2005 are given in Appendix 1

Table 6.5 Determination of Pipe Diameter and Pipe Type

Pipe No	Pipe Length	Pipe Type
B1	159.63	PE-10 Atü Ø 200/11.9
B2	164.54	PE-10 Atü Ø 160/9.50
B3	318.77	PE-10 Atü Ø 40/2.4
B4	86.61	PE-10 Atü Ø 140/8.3
B5	205.11	PE-10 Atü Ø 40/2.4
B6	80.70	PE-10 Atü Ø 40/2.4
B7	94.52	PE-10 Atü Ø 40/2.4
B8	162.19	PE-10 Atü Ø 50/3
B9	286.08	PE-10 Atü Ø 40/2.4
B10	468.24	PE-10 Atü Ø 40/2.4
B11	108.69	PE-10 Atü Ø 140/8.3
B12	51.41	PE-10 Atü Ø 40/2.4
B13	85.11	PE-10 Atü Ø 40/2.4
B14	66.63	PE-10 Atü Ø 40/2.4
B15	60.97	PE-10 Atü Ø 140/8.3
B16	85.59	PE-10 Atü Ø 40/2.4
B17	125.39	PE-10 Atü Ø 40/2.4
B18	265.49	PE-10 Atü Ø 40/2.4

Table 6.6 Determination of Reynolds Number and Flow Type

Pipe	Pipe Type-Pressure Bar	Pipe İnner Diameter (mm)	Pipe Outer Diameter (mm)	Velocity (m/s) at 0:00 pm	υ, Kinematic Viscosity (T=20°C) (m^2/s)	Reynolds Number* $= \frac{V*D}{\upsilon}$	Flow Type
B1	PE-10	176.2	200	0.850	$1.14*10^{-6}$	131,377.19	Turbulence
B2	PE-10	141	160	0.617	$1.14*10^{-6}$	76,313.16	Turbulence
B3	PE-10	35.2	40	0.460	$1.14*10^{-6}$	14,203.51	Turbulence
B4	PE-10	123.4	140	0.728	$1.14*10^{-6}$	78,802.81	Turbulence
B5	PE-10	35.2	40	0.176	$1.14*10^{-6}$	5,434.39	Turbulence
B6	PE-10	35.2	40	0.069	$1.14*10^{-6}$	2,130.53	Turbulence
B7	PE-10	35.2	40	0.081	$1.14*10^{-6}$	2,501.05	Turbulence
B8	PE-10	44	50	0.745	$1.14*10^{-6}$	28,754.39	Turbulence
B9	PE-10	35.2	40	0.215	$1.14*10^{-6}$	6,638.60	Turbulence
B10	PE-10	35.2	40	0.164	$1.14*10^{-6}$	5,063.86	Turbulence
B11	PE-10	123.4	140	0.792	$1.14*10^{-6}$	85,730.53	Turbulence
B12	PE-10	35.2	40	0.270	$1.14*10^{-6}$	8,336.84	Turbulence
B13	PE-10	35.2	40	0.138	$1.14*10^{-6}$	4,261.05	Turbulence
B14	PE-10	35.2	40	0.058	$1.14*10^{-6}$	1,790.88	Laminar
B15	PE-10	123.4	140	0.755	$1.14*10^{-6}$	81,725.44	Turbulence
B16	PE-10	35.2	40	0.073	$1.14*10^{-6}$	2,254.04	Turbulence
B17	PE-10	35.2	40	0.108	$1.14*10^{-6}$	3,334.74	Turbulence
B18	PE-10	35.2	40	0.465	$1.14*10^{-6}$	14,357.89	Turbulence

* ρ =1000 kg/m^3 at 20^0 C

CHAPTER VII

CONCLUSIONS AND FUTURE WORK

7.1 Conclusions

In this thesis, drinking water network, as both branched (open) and looped (closed) system, is solved by using Netcad/Water module. Netcad/Water module generates Network solutions using dead-point method which is still used by the Bank of Provinces and Special Provincial Administrations. In the network solved by Netcad/Water module, it is seen that in the pipes of equal lengths losses and velocity changes are very small for both branched (open) and looped (closed) system solution. City and town drinking water projects preperation regulations which is published in Official newspaper (April 22, 1985; 18733 numbered copy) presents that looped (closed) system solution of the network is more economic and appropriate.

The same network is solved by using Netcad/Water 2005 module which is another module of Netcad and used for drinking water network solutions. Netcad/Water 2005 module generates solution by taking both production and management of the network into consideration. Netcad/Water 2005 module is superior to Netcad/Water module with respect to the optimal network design costs and feasibility. It is excellant in contrast to the Netcad/Water module, since it has options to define different demands over different time periods for specific regions or nodes. Morever, Netcad/Water 2005 module has the ability to solve problems and difficulties faced in data generation and middle stages by using the capabilities of the geographical information systems. Two solutions, generated by Netcad/Water 2005 module and Netcad/Water module, are also compared with respect to the their pipe material costs. As can be seen from Table 7.1 and Table 7.2 the solution generated by Netcad/Water2005 module is more economical.

Table 7.1 Cost analysis of pipe materials solved by Netcad/Water2005

Pipe No	Pipe Length, (m)	Pipe Type	Unit cost, (TL/m)	Total Cost, (TL)
B1	159.63	PE-10 Atü Ø 200/11.9	38.53	6,150.54
B2	164.54	PE-10 Atü Ø 160/9.50	24.61	4,049.33
B3	318.77	PE-10 Atü Ø 40/2.4	1.57	500.47
B4	86.61	PE-10 Atü Ø 140/8.3	18.82	1,630.00
B5	205.11	PE-10 Atü Ø 40/2.4	1.57	322.02
B6	80.70	PE-10 Atü Ø 40/2.4	1.57	126.70
B7	94.52	PE-10 Atü Ø 40/2.4	1.57	148.40
B8	162.19	PE-10 Atü Ø 50/3	2.44	395.74
B9	286.08	PE-10 Atü Ø 40/2.4	1.57	449.15
B10	468.24	PE-10 Atü Ø 40/2.4	1.57	735.14
B11	108.69	PE-10 Atü Ø 140/8.3	18.82	2,045.55
B12	51.41	PE-10 Atü Ø 40/2.4	1.57	80.71
B13	85.11	PE-10 Atü Ø 40/2.4	1.57	133.62
B14	66.63	PE-10 Atü Ø 40/2.4	1.57	104.61
B15	60.97	PE-10 Atü Ø 140/8.3	18.82	1,147.46
B16	85.59	PE-10 Atü Ø 40/2.4	1.57	134.38
B17	125.39	PE-10 Atü Ø 40/2.4	1.57	196.86
B18	265.49	PE-10 Atü Ø 40/2.4	1.57	416.82
Total	**2875.67 m**			**18,767.49 TL**

Table 7.2 Cost analysis of pipe materials solved by Netcad/Water

Pipe No	Pipe Length, (m)	Pipe Type			Unit cost, (TL/m)	Total Cost , (TL)
1-2	155	PE-10	110.2	/125	15.00	2,325.00
2-3	166	PE-10	96.8	/110	11.76	1,952.16
3-8	322	PE-10	79.2	/90	7.85	2,527.70
3-4	85	PE-10	66.0	/75	5.46	464.10
4-6	203	PE-10	55.4	/63	3.86	783.58
4-5	84	PE-10	55.4	/63	3.86	324.24
4-7	98	PE-10	55.4	/63	3.86	378.28
2-9	165	PE-10	96.8	/110	11.76	1,940.40
9-12	291	PE-10	79.2	/90	7.85	2,284.35
9-10 +10-11 (91+374)	465	PE-10	79.2	/90	7.85	3650.25
2-13	113	PE-10	96.8	/110	11.76	1,328.88
13-14	58	PE-10	66.0	/75	5.46	316.68
14-15	89	PE-10	55.4	/63	3.86	343.54
14-16	67	PE-10	55.4	/63	3.86	258.62
13-17	56	PE-10	96.8	/110	11.76	658.56
17-18	92	PE-10	55.4	/63	3.86	355.12
17-19	118	PE-10	55.4	/63	3.86	455.48
17-20	264	PE-10	79.2	/90	7.85	2,072.40
Total	**2891 m**					**22,419.34 TL**

In our country where especially water shortage is getting bigger and bigger, prolific usage of water is very significant. That is why; drinking water network with Netcad/water 2005 has a superior position over network with Netcad /water module in terms of expenses and prolific usage of water.

7.2 Suggestion For Future Work

Netcad/water module which works with dead point method cannot make desired appointment to node points with respect to rural and urban differences. That is why; any solution method cannot be applied with Netcad/water module. Netcad/water 2005 module makes optimum design by using GIS (geographical information system). Moreover; in the projects where this module is applied, if reliability, production speed and easiness in management are regarded, it is suggested that the projects should be made with this module.

Netcad/water 2005 module is a universal product. Netcad/water 2005 module uses Hardy Cross method which is accepted as a universal product with the inclination algorithm. Furthermore; the drinking water Netcad solution with GIS support which is used in every field presents more healthful results. Many advantages like management expense and optimum usage of water are also acquired in drinking water solutions made with Netcad/water 2005.

As a result, it is recommended that Netcad/water 2005 module is more advantageous than Netcad/water module to make optimal designing water distribution network analysis

REFERENCESS

Abebe, A.J. and Solomatine, D.P. (1998). Application of global optimization to the design of pipe Networks. *3rd International Conferences on Hydroinformatics*, Copenhagen, Denmark, pp. 989-996.

Alperovits, E. and Shamir, U. (1977). Design of optimal water distribution systems. *Water Resources Research*, **13** (6), 885-900.

Bulut, M.Serdar (2005). "*Acomputer programming approach to the network of pipes with hardy-cross method*" M.Sc.Thesis submitted to the Graduate school of natural and applied sciences in Civil Engineering, Gaziantep University, Gaziantep, TURKEY

Chiplunkar, A.V., Mehndiratta, S.L. and Khanna, P. (1986) Looped water distribution system optimization for single loading. *Journal of Environmental Engineering,* **112** (2), 265-279.

Clark, R.M., and D. L. Tippen. (1990). Water Supply. *Standard Handbook of Environmental Engineering*. McGraw-Hill, NY. pp 5.173-5.220.

Clark, R.M., W.M. Grayman, S.G. Buchberger, Y. Lee, and D.J. Hartman. (2004). *An Overview in Water Supply Systems Security*. McGraw-Hill, NY

Cross, H. (1936). Analysis of Flow in Networks of Conduits or Conductors, *University of Illinois Engineering Experiment Station Bulletin 286*, Urbana, IL.

Cunha, M.D.C., and Sousa, J. (1999). Water Distribution Network Design Optimization: Simulated Annealing Approach. *Journal of Water Resources Planning and Management*, **125** (4), 215-221.

Dandy, G.C., Simpson A.R., and Murphy L.J. (1996). An improved genetic algorithm for pipe network optimization. *Water Resources Research*, **32** (2), 449-458.

Eiger, G., Shamir, U., and Ben-Tal A. (1994). Optimal design of water distribution networks. *Water Resources Research*, **30** (9), 2637-2646.

Etüd ve Yol Dairesi Başkanlığı, (2008). Kilis (Merkez) içmesuyu Kesin Projesi, İller Bankası Genel Müdürlüğü, Ankara TURKEY

Eusuff, M.M. and Lansey, K.E. (2003). Optimization of Water Distribution Network Design Using the Shuffled Frog Leaping Algorithm. *Journal of Water Resources Planning and Management,* ASCE, **129** (3), 210-225

Fair, G.M., Geyer, J.C., and Okun, D.A. (1981). *Elements of Water Supply and Wastewater Disposal*. John Wiley & Sons, New York, USA

Grayman, W.M. (2000) Put Your Hydraulic Model to Work – How That Costly Model Can Earn its Keep by Making Operations More Effective. Proceedings, *AWWA, Annual Conference*, Denver, CO.

Gupta, I., Gupta, A., and Khanna, P. (1999). Genetic algorithm for optimization of water distribution systems., *Environmental Modelling & software* **4**, 437-446.

http://w3.balıkesir.edu.tr/~ngedik/files/Download/VIIbolum.pdf

http://www.netcad.com.tr/Default_ENG.aspk

http://www.pilsa.com.tr/eng/indextr.html.

İller Bankası Şehir Ve Kasaba İçmesuyu Projelerinin Hazırlanmasına Ait Yönetmenlik (1992) Resmi Gazete (22 Nisan 1985)

Jeppson, R.W. (1976) *Analysis of Flow in Pipe Networks.* Ann Arbor Science, Ann Arbor, MI. USA

Kessler, A. and Shamir, U. (1989). Analysis of the linear programming gradient method for optimal design of water supply networks., *Water Resources Research*, **25** (7), pp. 1469-1480.

Larock, B.E., R.W. Jeppson, and G.Z. Watters. (2000). *Hydraulics of Pipeline Systems.* CRC Press, LLC, Boca Raton, FL.

Mutlu, Özgür Hardy Cross Yöntemiyle İçmesuyu Projlerinin Hazırlanmasında Netcad/Water2005 yazılımının CBS çözümleri, *TMMOB 2.Su Politikaları Kongresi bülteni*

Prabhata K. Swamee Ashok K. Sharma, (2008). *Desıgn Of Water Supply Pıpe Networks*, John Wiley & Sons, Inc., Hoboken, New Jersey USA

Rossman, L.A. (2000). Drinking Water Research Division. *EPANET Version 2 Users Manual*, Environmental Protection Agency (EPA), Cincinnati, Ohio.

Rossman, L.A., R.M. Clark, and W.M. Grayman. (1994) Modeling Chlorine Residuals in Drinking Water Distribution Systems. *Journal Environmental Engineering,* **120** (4), 803-820.

Savic, D.A. and Walters, G.A. (1997). Genetic algorithms for least-cost design of water distribution networks. *Journal of Water Resources Planning and Management*, ASCE, **123** (2)**,** 67-77.

Simpson, A.R., Dandy, G.C., and Murphy, L.J. (1994). Genetic algorithms compared to other techniques for pipe optimization. *Journal of Water Resources Planning and Management*, ASCE, **120** (4), pp. 423-443.

Topacık D., Eroğlu V. (1987). *Su Temini ve atıksu uzaklaştırılması* İstanbul, İstanbul Teknik Üniversitesi

U.S. Environmental Protection Agency (EPA) (2005). *Water Distribution System Analysis: Field Studies, Modeling and Management A Reference Guide for Utilities* Cincinnati, Ohio

Walski, T. M, D.V. Chase, D.A. Savic, W.M. Grayman, S. Beckwith, and E. Koelle.(2003) *Advanced Water Distribution Modeling and Management.* Haestad Press, Waterbury, CT.

Wood, D.J. (1980) *Computer Analysis of Flow in Pipe Networks.* Department of Civil Engineering, University of Kentucky, Lexington, KY.

Wood, D.J., and Charles, C.O.A. (1972). Hydraulic network analysis using Linear Theory. *Journal of Hydraulic Division, ASCE* **98**(7), 1157–1170.

Yanmaz, A.Melih (2006). *Applied Water Resorcues Engineering*. METU Press, ANKARA

Yetilmezsoy K., Demir S.and Manav N. (2008) Development of a Modıfıed Hardy-Cross Algorıthm for Tıme-Dependent Simulations of Water Distribution Networks**.** *Fresenius Environmental Bulletin* by PSP **17** (8), 1045

APPENDIX

Appendix-1 Desıgn Outputs of Pipe Network with Netcad/water 2005

TIME	PIPE DISCHARGES (lt/s)						
	B1	B2	B3	B4	B5	B6	B7
00:00	20.734	9.631	0.448	8.707	0.171	0.067	0.079
01:00	0.000	7.223	0.336	6.530	0.128	0.050	0.059
02:00	0.000	6.621	0.308	5.986	0.118	0.046	0.054
03:00	0.000	6.621	0.308	5.986	0.118	0.046	0.054
04:00	0.000	7.223	0.336	6.530	0.128	0.050	0.059
05:00	0.000	8.668	0.403	7.836	0.154	0.060	0.071
06:00	0.000	12.159	0.556	10.983	0.216	0.085	0.100
07:00	0.000	12.761	0.594	11.537	0.227	0.089	0.105
08:00	0.000	13.122	0.610	11.864	0.233	0.092	0.108
09:00	0.000	13.274	0.638	12.408	0.244	0.096	0.113
10:00	0.000	13.604	0.633	12.299	0.242	0.095	0.112
11:00	0.000	13.002	0.605	11.755	0.231	0.091	0.107
12:00	0.000	13.002	0.605	11.755	0.231	0.091	0.107
13:00	0.000	13.002	0.605	11.755	0.231	0.091	0.107
14:00	0.000	12.641	0.588	11.428	0.225	0.088	0.104
15:00	0.000	11.557	0.538	10.449	0.205	0.081	0.095
16:00	0.000	12.400	0.577	11.211	0.220	0.087	0.102
17:00	0.000	12.159	0.566	10.983	0.216	0.085	0.100
18:00	0.000	14.447	0.672	13.061	0.257	0.101	0.119
19:00	0.000	15.049	0.700	13.605	0.268	0.105	0.124
20:00	0.000	14.447	0.672	13.061	0.257	0.101	0.119
21:00	0.000	12.159	0.566	10.993	0.216	0.085	0.100
22:00	0.000	10.594	0.493	9.578	0.188	0.074	0.087
23:00	0.000	12.039	0.560	10.884	0.214	0.084	0.099
00:00	0.000	9.631	0.448	8.707	0.171	0.067	0.079

Appendix-1 Desıgn Outputs of Pipe Network with Netcad/water 2005 ***(continued)***

TIME (Hour)	PIPE DISCHARGES (lt/s)						
	B8	B9	B10	B11	B12	B13	B14
00:00	1.133	0.209	0.160	9.743	0.262	0.135	0.056
01:00	0.850	0.157	0.120	7.105	0.197	0.101	0.042
02:00	0.779	0.144	0.110	6.513	0.180	0.093	0.038
03:00	0.779	0.144	0.110	6.513	0.180	0.093	0.038
04:00	0.850	0.157	0.120	7.105	0.197	0.101	0.042
05:00	1.020	0.188	0.144	8.526	0.236	0.122	0.050
06:00	1.431	0.264	0.201	11.960	0.331	0.171	0.071
07:00	1.502	0.277	0.211	12.552	0.348	0.179	0.074
08:00	1.544	0.285	0.217	12.907	0.358	0.184	0.076
09:00	1.615	0.298	0.227	13.499	0.374	0.193	0.080
10:00	1.601	0.295	0.225	13.381	0.371	0.191	0.079
11:00	1.530	0.282	0.215	12.789	0.354	0.183	0.076
12:00	1.530	0.282	0.215	12.789	0.354	0.183	0.076
13:00	1.530	0.282	0.215	12.789	0.354	0.183	0.076
14:00	1.487	0.274	0.209	12.434	0.344	0.177	0.073
15:00	1.360	0.251	0.191	11.368	0.315	0.162	0.067
16:00	1.459	0.269	0.205	12.197	0.338	0.174	0.072
17:00	1.431	0.264	0.201	11.960	0.331	0.171	0.071
18:00	1.700	0.313	0.239	14.210	0.394	0.203	0.084
19:00	1.771	0.326	0.249	14.802	0.410	0.211	0.087
20:00	1.700	0.313	0.239	14.210	0.394	0.203	0.084
21:00	1.431	0.264	0.201	11.960	0.331	0.171	0.071
22:00	1.247	0.230	0.176	10.421	0.289	0.149	0.062
23:00	1.417	0.261	0.198	11.842	0.328	0.169	0.070
00:00	1.133	0.209	0.160	9.473	0.262	0.135	0.056

Appendix-1 Desıgn Outputs of Pipe Network with Netcad/water 2005 ***(continued)***

TIME (Hour)	PIPE DISCHARGES (lt/s)			
	B15	**B16**	**B17**	**B18**
00:00	9.026	0.071	0.105	0.452
01:00	6.770	0.053	0.079	0.339
02:00	6.205	0.049	0.072	0.311
03:00	6.205	0.049	0.072	0.311
04:00	6.770	0.053	0.079	0.339
05:00	8.123	0.064	0.094	0.407
06:00	11.395	0.090	0.132	0.571
07:00	11.959	0.094	0.139	0.599
08:00	12.298	0.097	0.143	0.616
09:00	12.862	0.101	0.149	0.645
10:00	12.749	0.101	0.148	0.639
11:00	12.185	0.096	0.141	0.611
12:00	12.185	0.096	0.141	0.611
13:00	12.185	0.096	0.141	0.611
14:00	11.847	0.093	0.138	0.594
15:00	10.831	0.085	0.126	0.543
16:00	11.621	0.092	0.135	0.582
17:00	11.395	0.090	0.132	0.571
18:00	13.539	0.107	0.157	0.679
19:00	14.103	0.111	0.164	0.707
20:00	13.539	0.107	0.157	0.679
21:00	11.395	0.090	0.132	0.571
22:00	9.928	0.078	0.115	0.498
23:00	11.283	0.089	0.131	0.566
00:00	9.026	0.071	0.105	0.452

Appendix-1 Desıgn Outputs of Pipe Network with Netcad/water 2005 ***(continued)***

TIME (Hour)	PIPE VELOCITY (m/s)						
	B1	B2	B3	B4	B5	B6	B7
00:00	0.85	0.617	0.46	0.728	0.176	0.069	0.081
01:00	0	0.463	0.345	0.546	0.132	0.052	0.061
02:00	0	0.424	0.316	0.501	0.121	0.047	0.056
03:00	0	0.424	0.316	0.501	0.121	0.047	0.056
04:00	0	0.463	0.345	0.546	0.132	0.052	0.061
05:00	0	0.555	0.414	0.655	0.158	0.062	0.073
06:00	0	0.779	0.581	0.919	0.222	0.087	0.103
07:00	0	0.817	0.61	0.965	0.233	0.091	0.108
08:00	0	0.84	0.627	0.932	0.24	0.094	0.11
09:00	0	0.879	0.656	1.037	0.251	0.098	0.116
10:00	0	0.871	0.65	1.028	0.248	0.098	0.115
11:00	0	0.833	0.621	0.983	0.237	0.093	0.11
12:00	0	0.833	0.621	0.983	0.237	0.093	0.11
13:00	0	0.833	0.621	0.983	0.237	0.093	0.11
14:00	0	0.81	0.604	0.956	0.231	0.091	0.107
15:00	0	0.74	0.552	0.874	0.211	0.083	0.098
16:00	0	0.794	0.593	0.937	0.227	0.089	0.105
17:00	0	0.779	0.581	0.919	0.222	0.087	0.103
18:00	0	0.925	0.69	1.092	0.264	0.104	0.122
19:00	0	0.964	0.719	1.138	0.275	0.108	0.127
20:00	0	0.925	0.69	1.092	0.264	0.104	0.122
21:00	0	0.779	0.581	0.919	0.222	0.087	0.103
22:00	0	0.678	0.506	0.801	0.194	0.076	0.09
23:00	0	0.771	0.575	0.91	0.22	0.086	0.102
00:00		0.617	0.46	0.728	0.176	0.069	0.081

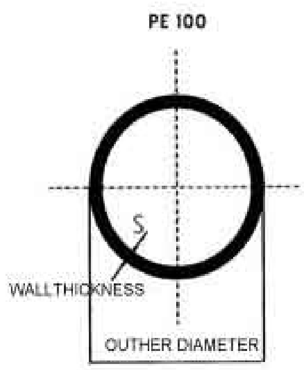
Appendix-1 Desıgn Outputs of Pipe Network with Netcad/water 2005 ***(continued)***

TIME (Hour)	PIPE VELOCITY (m/s)						
	B8	B9	B10	B11	B12	B13	B14
00:00	0.745	0.215	0.164	0.792	0.270	0.138	0.058
01:00	0.559	0.161	0.123	0.594	0.202	0.104	0.043
02:00	0.512	0.148	0.113	0.545	0.185	0.096	0.04
03:00	0.512	0.148	0.113	0.545	0.185	0.096	0.04
04:00	0.559	0.161	0.123	0.594	0.202	0.104	0.043
05:00	0.671	0.193	0.148	0.713	0.243	0.125	0.052
06:00	0.941	0.271	0.207	1	0.34	0.175	0.073
07:00	0.988	0.284	0.217	1.05	0.357	0.184	0.076
08:00	1.015	0.292	0.223	1.079	0.367	0.189	0.078
09:00	1.062	0.306	0.234	1.129	0.384	0.198	0.082
10:00	1.053	0.303	0.232	1.119	0.381	0.196	0.081
11:00	1.006	0.29	0.221	1.069	0.364	0.188	0.078
12:00	1.006	0.29	0.221	1.069	0.364	0.188	0.078
13:00	1.006	0.29	0.221	1.069	0.364	0.188	0.078
14:00	0.978	0.282	0.215	1.04	0.354	0.182	0.076
15:00	0.894	0.258	0.197	0.951	0.324	0.167	0.069
16:00	0.96	0.276	0.211	1.02	0.347	0.179	0.074
17:00	0.941	0.271	0.207	1	0.34	0.175	0.073
18:00	1.118	0.322	0.246	1.188	0.404	0.208	0.086
19:00	1.165	0.335	0.256	1.238	0.421	0.217	0.09
20:00	1.118	0.322	0.246	1.188	0.404	0.208	0.086
21:00	0.941	0.271	0.207	1	0.34	0.175	0.073
22:00	0.82	0.236	0.18	0.871	0.297	0.153	0.063
23:00	0.932	0.268	0.205	0.99	0.337	0.174	0.072
00:00	0.745	0.215	0.164	0.792	0.27	0.139	0.058

Appendix-1 Desıgn Outputs of Pipe Network with Netcad/water 2005 ***(continued)***

TIME (Hour)	PIPE VELOCITY (m/s)			
	B15	B16	B17	B18
00:00	0.755	0.073	0.108	0.465
01:00	0.566	0.055	0.081	0.349
02:00	0.519	0.05	0.074	0.32
03:00	0.519	0.05	0.074	0.32
04:00	0.566	0.055	0.081	0.349
05:00	0.679	0.066	0.097	0.418
06:00	0.953	0.092	0.136	0.587
07:00	1	0.097	0.143	0.616
08:00	1.028	0.1	0.147	0.633
09:00	1.075	0.104	0.153	0.662
10:00	1.066	0.103	0.152	0.657
11:00	1.019	0.099	0.145	0.628
12:00	1.019	0.099	0.145	0.628
13:00	1.019	0.099	0.145	0.628
14:00	0.991	0.096	0.141	0.61
15:00	0.906	0.088	0.129	0.558
16:00	0.972	0.094	0.138	0.599
17:00	0.953	0.092	0.136	0.587
18:00	1.132	0.11	0.162	0.697
19:00	1.179	0.114	0.168	0.726
20:00	1.132	0.11	0.162	0.697
21:00	0.953	0.092	0.136	0.587
22:00	0.83	0.08	0.118	0.511
23:00	0.943	0.091	0.135	0.581
00:00	0.755	0.073	0.108	0.465

Appendix-2 Production Standarts Of Pipe



COIL DETAILS OF PE 100				
Pipe Diameter	**Inner Diameter**	**Outer Diameter**	**Wrapping Width**	**Wrapping Lenght**
(mm)	**(cm)**	**(cm)**	**(cm)**	**(mt)**
Ø20	100	110	21	100
Ø25	100	120	21	100
Ø32	100	130	26	100
Ø40	116	140	36	100
Ø50	116	155	36	50
Ø63	166	190	40	50
Ø75	166	195	40	50
Ø90	225	260	50	50
Ø110	225	270	50	50

Appendix-2 Production Standarts Of Pipe ***(continued)***

Outer Diameter	Pressure 4 Bar (SDR 41/S-20)		Pressure 5 Bar (SDR 33/S-16)		Pressure 6 Bar (SDR 27,6/S-13,3)		Pressure 8 Bar (SDR 21/S-10)		Pressure 10 Bar (SDR 17/S-8)		Pressure 12,5 Bar (SDR 13,6/S-6,3)		Pressure 16 Bar (SDR 11/S-5)		Pressure 20 Bar (SDR 9/S-4)		Pressure 25 Bar (SDR 7,4/S-3,2)		Pressure 32 Bar (SDR 6/S-2,5)	
	Wall thickness		Wall thickness		Wall thickness		Wall thickness		Wall thickness		Wall thickness		Wall thickness		Wall thickness		Wall thickness		Wall thickness	
	min.	max.	min.	max.	min.	max.	min.	max.	min.	max.	min.	max.	min.	max.	min.	max.	min.	max.	min.	max.
16															2,0	2,3	2,3	2,7	3,0	3,4
20													2,0	2,3	2,3	2,7	3,0	3,4	3,4	3,9
25											2,0	2,3	2,3	2,7	3,0	3,4	3,5	4,0	4,2	4,8
32									2,0	2,3	2,41	2,8	3,0	3,4	3,6	4,1	4,4	5,0	5,4	6,1
40							2,0	2,3	2,4	2,8	3,0	3,5	3,7	4,2	4,5	5,1	5,5	6,2	6,7	7,5
50					2,0	2,2	2,4	2,8	3,0	3,4	3,7	4,2	4,6	5,2	5,6	6,3	6,9	7,7	8,3	9,3
63			2,0	2,3	2,4	2,9	3,0	3,4	3,8	4,3	4,7	5,3	5,8	6,5	7,1	8,0	8,6	9,6	10,5	11,7
75			2,3	2,7	2,7	3,2	3,6	4,1	4,5	5,1	5,6	6,3	6,8	7,6	8,4	9,4	1003	11,5	12,5	13,9
90	2,3	2,6	2,8	3,2	3,3	3,9	4,3	4,9	5,4	6,1	6,7	7,5	8,2	9,52	10,1	11,3	12,3	13,7	15,0	16,7
110	2,7	3,1	3,4	3,9	4,0	4,6	5,3	6,0	6,6	7,4	8,1	9,1	10,0	11,1	12,3	13,7	15,1	16,8	18,3	20,3
125	3,1	3,6	3,9	4,4	4,5	5,2	6,0	6,7	7,4	8,3	9,2	10,3	11,4	12,7	14,0	15,6	17,1	19,0	20,8	23,0
140	3,5	4,0	4,3	4,9	5,1	5,9	6,7	7,5	8,3	9,3	10,3	11,5	12,7	14,1	15,7	17,4	19,2	21,3	23,3	25,8
160	4,0	4,5	4,9	5,5	5,8	6,6	7,7	8,6	9,5	10,6	11,8	13,1	14,6	16,2	17,9	19,8	21,9	24,2	26,3	29,4
180	4,4	5,0	5,5	6,2	6,5	7,4	8,6	9,6	10,7	11,9	13,3	14,8	16,4	18,2	20,1	22,3	24,6	27,2	29,9	33,0
200	4,9	5,5	6,2	7,0	7,2	8,2	9,6	10,7	11,9	13,2	14,7	16,3	18,2	20,2	22,4	24,8	27,4	30,3	33,2	36,7
225	5,5	6,2	6,9	7,7	8,2	9,3	10,8	12,0	13,4	14,9	16,6	18,4	20,5	22,7	25,2	27,9	30,8	34,0	37,4	41,3
250	6,2	7,0	7,7	8,6	9,1	10,3	11,9	13,2	14,8	16,4	18,4	20,4	22,7	25,1	27,9	30,8	34,2	37,8	41,5	45,8
280	6,9	7,7	8,6	9,6	10,1	11,4	13,4	14,9	16,6	18,4	20,6	22,8	25,4	28,1	31,3	34,6	38,3	42,3	46,5	51,3
315	7,7	8,6	9,7	10,8	11,4	12,8	15,0	16,6	18,7	20,7	23,2	25,7	28,6	31,6	35,2	38,9	43,1	47,6	52,3	57,7
355	8,7	9,7	10,9	12,1	12,9	14,4	16,9	18,7	21,1	23,4	26,1	28,9	32,2	35,6	39,7	43,8	48,5	53,5	59,0	65,0
400	9,8	10,9	12,3	13,7	14,5	16,2	19,1	51,2	23,7	26,2	29,4	32,5	36,3	40,1	44,7	49,3	21,4	60,3		
450	11,0	12,2	13,8	15,3	16,3	18,2	21,5	23,8	26,7	29,5	33,1	36,6	40,9	45,1	50,3	55,5	31,5	67,8		

Appendix-3 Design Outputs of Open pipe Network with Netcad/water

TABLE OF OF DESIGN OF PIPE NETWORK

Node to Node	LENGTHS: Pipe Lengths	LENGTHS: Relative Density	LENGTHS: Relative Length	DISCHARGES: Relative Pipe discharge P=qx*Li	DISCHARGES: Endof pipe discharge Qu	DISCHARGES: Qb=Qu +P	DISCHARGES: =0,55*P Qo	DISCHARGES: =Qu+Qo Q1	DISCHARGES: Fire discharge	DISCHARGES: Qdesign= Q1+Qy Qh	AT THE PIPES: Pipe Diameter			AT THE PIPES: Head loss/m j	AT THE PIPES: Velocity V	AT THE PIPES: Head loss at the Pipe Lxj	Node	ALTITUDES: Altitude of Piezometric	ALTITUDES: Altitude of Ground	ALTITUDES: Operating Pressure	ALTITUDES: Static pressure	IDEAS
No	m	k	M	Lt/sn	Lt/sn	Lt/sn	Lt/sn	Lt/sn	Lt/sn	Lt/sn	mm				m/sn	m	No	m	m	m	m	
Qtotal demand (Qdemand) =		4.00Lt/sn			q =	(1,5 x Qdemand) Σli		=	6.00 / 2735	=	0.002194			Lt/sn/m								
17-20	264	1	264	0.578	0.00	0.579	0.318	0.318	5.00	5.318	79.2	/90	PE 10	0.014380	1.08	3.79	20	577.81	524.58	53.24	60.08	
17-18	92	1	92	0.202	0.00	0.202	0.111	0.111	2.50	2.611	55.4	/63	PE 10	0.021954	1.08	2.03	18	579.57	522.97	56.60	61.68	
17-19	118	1	118	0.258	0.000	0.258	0.142	0.142	2.50	2.642	55.4	/63	PE 10	0.022433	1.10	2.64	19	578.96	540.61	38.35	44.04	
13-17	56	1	56	0.123	1.039	1.162	0.068	1.107	5.00	6.107	96.8	/110	PE 10	0.006995	0.83	0.39	17	581.60	529.26	52.34	55.39	
14-15	89	1	89	0.194	0.00	0.194	0.107	0.107	2.50	2.607	55.4	/63	PE 10	0.021884	1.08	1.94	15	579.39	523.26	56.14	61.40	
14-16	67	1	67	0.146	0.00	0.146	0.080	0.081	2.50	2.580	55.4	/63	PE 10	0.021473	1.07	1.43	16	579.90	519.81	60.09	64.84	
13-14	58	1	58	0.127	0.340	0.467	0.070	0.410	2.50	2.910	66.0	/75	PE 10	0.011443	0.85	0.66	14	581.33	523.91	57.42	60.74	
2-13	113	1	113	0.248	1.629	1.877	0.137	1.765	5.00	6.765	96.8	/110	PE 10	0.008456	0.92	0.96	13	581.99	527.38	54.61	57.27	
9-12	291	1	291	0.638	0.00	0.638	0.351	0.351	5.00	5.351	79.2	/90	PE 10	0.014546	1.09	4.23	12	577.29	517.04	60.25	67.61	
10-11	91	1	91	0.199	0.00	0.199	0.110	0.110	5.00	5.110	79.2	/90	PE 10	0.013354	1.04	1.21	11	574.29	522.78	51.51	61.87	
9-10	374	1	374	0.821	0.199	1.020	0.452	0.650	5.00	5.651	79.2	/90	PE 10	0.016089	1.15	6.02	10	575.51	523.49	52.02	61.16	

Appendix-3 Design Outputs of Open pipe Network with Netcad/water *(continued)*

TABLE OF OF DESIGN OF PIPE NETWORK																							
	LENGTHS			DISCHARGES							AT THE PIPES								ALTITUDES				
Node to Node	Pipe Lengths	Relative Density	Relative Length	Relative Pipe discharge P=qx*Li	Endof pipe discharge Qu	Qb=Qu +P	=0,55*P Qo	=Qu+Qo Q1	Fire discharge	Qdesign= Q1+Qy Qh	Pipe Diameter			Head loss/m j	Velocity V	Head loss at the Pipe Lxj	Node	Altitude of Piezometric	Altitude of Ground	Operating Pressure	Static pressure	IDEAS
No	m	k	M	Lt/sn	Lt/sn	Lt/sn	Lt/sn	Lt/sn	Lt/sn	Lt/sn	mm				m/sn	M	No	m	m	m	M	
Qtotal demand (Qdemand) =		4.00Lt/sn		q =		(1,5 x Qdemand) / Σli		=	6.00 / 2736	=	0.0021930			Lt/sn/m								
2-9	165	1	165	0.361	1.659	2.020	0.199	1.858	5.00	6.857	96.8	/90	PE 10	0.008669	0.93	1.43	9	581.52	518.10	63.43	66.56	
3-8	322	1	322	0.707	0.00	0.707	0.389	0.388	5.00	5.389	79.2	/110	PE 10	0.014736	1.09	4.75	8	576.73	517.09	59.64	67.56	
4-5	84	1	84	0.185	0.000	0.185	0.102	0.101	2.50	2.602	55.4	/63	PE 10	0.021806	1.08	1.84	5	578.30	520.33	57.97	64.32	
4-6	203	1	203	0.445	0.000	0.445	0.245	0.245	2.50	2.745	55.4	/63	PE 10	0.024070	1.14	4.88	6	575.26	521.59	53.67	63.06	
4-7	98	1	98	0.215	0.00	0.215	0.118	0.118	2.50	2.618	55.4	/63	PE 10	0.022065	1.09	2.17	7	577.97	524.64	53.33	60.01	
3-4	85	1	85	0.186	0.845	1.032	0.103	0.948	2.50	3.448	66.0	/75	PE 10	0.015660	1.01	1.33	4	580.14	532.82	47.33	51.84	
2-3	166	1	166	0.365	1.738	2.103	0.200	1.939	5.00	6.939	96.8	/110	PE 10	0.008861	0.94	1.47	3	581.47	544.51	36.96	40.14	
1-2	155	0	0	0.000	6.00	6.000	0.000	6.000	5.00	11.00	110.2	/125	PE 10	0.011059	1.15	1.71	2	582.94	535.95	46.99	48.70	
			2735																			

Appendix-4 Design Outputs of Open pipe Network with Netcad/water (V=1 m/sc)

TABLE OF OF DESIGN OF PIPE NETWORK																				
Node to Node	LENGTHS			DISCHARGES							AT THE PIPES				Node	ALTITUDES				IDEAS
	Pipe Lengths	Relative Density	Relative Length	Relative Pipe discharge P=qx*Li	Endof pipe discharge Qu	Qb=Qu +P	=0,55*P Qo	=Qu+Qo Q1	Fire discharge	Qdesign= Q1+Qy Qh	Pipe Diameter	Head loss/m j	Velocity V	Head loss at the Pipe Lxj		Altitude of Piezometric	Altitude of Ground	Operating Pressure	Static pressure	
No	m	k	M	Lt/sn	Lt/sn	Lt/sn	Lt/sn	Lt/sn	Lt/sn	Lt/sn	mm		m/sn	m	No	m	m	m	m	
Qtotal demand (Qdemand) =		4.00Lt/sn			q =	(1,5 x Qdemand) / Σli			=	6.00 / 2736	=	0.0021930	Lt/sn/m							
17-20	264	1	264.00	0.579	0.00	0.579	0.318	0.318	5.00	5.318	82	0.011702	1.00	3.09	20	577.81	524.58	53.23	60.08	
17-18	92	1	92.00	0.202	0.00	0.202	0.111	0.111	2.50	2.611	58	0.017722	1.00	1.63	18	579.57	522.97	56.60	61.69	
17-19	118	1	118.00	0.259	0.00	0.259	0.142	0.142	2.50	2.642	58	0.017599	1.00	2.08	19	578.96	540.61	38.35	44.05	
13-17	56	1	56.00	0.123	1.039	1.162	0.068	1.107	5.00	6.107	88	0.010796	1.00	0.60	17	581.60	529.26	52.34	55.40	
14-15	89	1	89.00	0.195	0.000	0.195	0.107	0.107	2.50	2.607	58	0.017736	1.00	1.58	15	579.39	523.26	56.13	61.40	
14-16	67	1	67.00	0.147	0.00	0.147	0.081	0.081	2.50	2.581	57	0.017842	1.00	1.20	16	579.90	519.81	60.09	64.85	
13-14	58	1	58.00	0.127	0.34	0.467	0.070	0.410	2.50	2.910	61	0.016635	1.00	0.96	14	581.33	523.91	57.42	60.75	
2-13	113	1	113.00	0.248	1.629	1.877	0.136	1.765	5.00	6.765	93	0.010170	1.00	1.15	13	581.99	527.38	54.61	57.28	
9-12	291	1	291.00	0.638	0.000	0.638	0.351	0.351	5.00	5.351	83	0.011660	1.00	3.39	12	577.29	517.04	60.25	67.62	
10-11	91	1	91.00	0.200	0.00	0.200	0.110	0.110	5.00	5.110	81	0.011978	1.00	1.09	11	574.29	522.78	51.51	61.88	
9-10	374	1	374.00	0.820	0.20	1.019	0.451	0.650	5.00	5.650	85	0.011296	1.00	4.22	10	575.51	523.49	52.02	61.17	

Appendix-4 Design Outputs of Open pipe Network with Netcad/water (V=1 m/sc) *(continued)*

TABLE OF OF DESIGN OF PIPE NETWORK

	LENGTHS			DISCHARGES							AT THE PIPES					ALTITUDES				
Node to Node	Pipe Lengths	Relative Density	Relative Length	Relative Pipe discharge P=qx*Li	Endof pipe discharge Qu	Qb=Qu +P	=0,55*P Qo	=Qu+Qo Q1	Fire discharge	Qdesign= Q1+Qy Qh	Pipe Diameter	Head loss/m j	Velocity V	Head loss at the Pipe Lxj	Node	Altitude of Piezometric	Altitude of Ground	Operating Pressure	Static pressure	IDEAS
No	m	k	M	Lt/sn	Lt/sn	Lt/sn	Lt/sn	Lt/sn	Lt/sn	Lt/sn	mm		m/sn	m	No	m	m	m	m	
Qtotal demand (Qdemand) =		4.00Lt/sn			q =	(1,5 x Qdemand) / Σli			=	6.00 / 2736	=	0.0021930	Lt/sn/m							
2-9	165	1	165.00	0.362	1.659	2.021	0.199	1.858	5.00	6.858	93	0.010089	1.00	1.66	9	581.52	518.10	63.42	66.56	
3-8	322	1	322.00	0.706	0.000	0.706	0.388	0.388	5.00	5.388	83	0.011613	1.00	3.74	8	576.73	517.09	59.64	67.57	
4-5	84	1	84.00	0.184	0.00	0.184	0.101	0.101	2.50	2.601	58	0.017760	1.00	1.49	5	578.30	520.33	57.97	64.33	
4-6	203	1	203.00	0.445	0.00	0.445	0.245	0.245	2.50	2.745	59	0.017212	1.00	3.49	6	575.26	521.59	53.67	63.07	
4-7	98	1	98.00	0.215	0.000	0.215	0.118	0.118	2.50	2.618	58	0.017693	1.00	1.73	7	577.97	524.64	53.33	60.02	
3-4	85	1	85.00	0.186	0.845	1.031	0.103	0.948	2.50	3.448	66	0.015069	1.00	1.28	4	580.14	532.82	47.32	51.84	
2-3	166	1	166.00	0.364	1.74	2.102	0.200	1.938	5.00	6.938	94	0.010021	1.00	1.66	3	581.47	544.51	36.96	40.15	
1-2	155	0	0.00	0.000	6.00	6.000	0.000	6.000	5.00	11.000	118	0.007659	1.00	1.19	2	582.94	535.95	46.99	48.71	
			2736																	

Appendix-5 Design Outputs of Closed pipe Network with Netcad/water

TABLE OF OF DESIGN OF PIPE NETWORK																						
	LENGTHS			DISCHARGES							AT THE PIPES								ALTITUDES			
Node to Node	Pipe Lengths	Relative Density	Relative Length	Relative Pipe discharge P=qx*Li	Endof pipe discharge Qu	Qb=Qu +P	0,55*P Qo	Qu+Qo Q1	Fire discharge	Qdesign= Q1+Qy Qh	Pipe Diameter			Head loss/m j	Velocity V	Head loss at the Pipe Lxj	Node	Altitude of Piezometric	Altitude of Ground	Operating Pressure	Static pressure	IDEAS
No	m	k	m	Lt/sn	Lt/sn	Lt/sn	Lt/sn	Lt/sn	Lt/sn	Lt/sn	mm				m/sn	m	No	M	m	m	m	
Qtotal demand (Qdemand) =		4.00Lt/sn			q =	(1,5 x Qdemand) / Σli		=	6.00 / 2750	=		0.002167		Lt/sn/m								
9-M1	367	1	367.00	0.796	0.00	0.796	0.438	0.438	5.00	5.438	79.2	/90	PE 10	0.014986	1.10	5.51	M1	575.97	523.49	52.48	61.16	H=0.030
9-10	92	1	92.00	0.200	0.00	0.200	0.110	0.110	2.50	2.610	55.4	/63	PE 10	0.021934	1.08	2.02	10	579.49	522.97	56.52	61.68	
9-11	118	1	118.00	0.255	0.00	0.255	0.140	0.140	2.50	2.640	55.4	/63	PE 10	0.022407	1.10	2.64	11	578.88	540.61	38.27	44.04	
12-9	56	1	56.00	0.122	1.252	1.373	0.067	1.318	5.00	6.318	96.8	/110	PE 10	0.007451	0.86	0.42	9	581.51	529.26	52.25	55.39	
13-14	89	1	89.00	0.192	0.000	0.192	0.106	0.106	2.50	2.606	55.4	/63	PE 10	0.021864	1.08	1.94	14	579.33	523.26	56.08	61.40	
13-15	67	1	67.00	0.144	0.00	0.144	0.079	0.079	2.50	2.579	55.4	/63	PE 10	0.021459	1.07	1.43	15	579.84	519.81	60.03	64.84	
12-13	58	1	58.00	0.125	0.336	0.461	0.069	0.405	2.50	2.905	66.0	/75	PE 10	0.011407	0.85	0.66	13	581.27	523.91	57.36	60.74	
2-12	113	1	113.00	0.245	1.835	2.080	0.135	1.970	5.00	6.970	96.8	/110	PE 10	0.008933	0.95	1.01	12	581.93	527.38	54.55	57.27	
8-M1	374	1	374.00	0.811	0.000	0.811	0.446	0.446	5.00	5.446	79.2	/90	PE 10	0.015027	1.11	5.62	M1	575.97	523.49	52.48	61.16	H=-0.030
8-M2	311	1	311.00	0.674	0.00	0.674	0.3731	0.371	5.00	5.371	79.2	/90	PE 10	0.014644	1.09	4.55	M2	577.06	518.75	58.31	65.90	H=-0.020
2-8	165	1	165.00	0.357	1.485	1.842	0.196	1.681	5.00	6.681	79.2	/90	PE 10	0.008261	0.91	1.36	8	581.59	518.10	63.50	66.56	

Appendix-5 Design Outputs of Closed pipe Network with Netcad/water *(continued)*

TABLE OF OF DESIGN OF PIPE NETWORK																								
	LENGTHS			DISCHARGES							AT THE PIPES								ALTITUDES					
Node to Node	Pipe Lengths	Relative Density	Relative Length	Relative Pipe discharge P=qx*Li	Endof pipe discharge Qu	Qb=Qu +P	0,55*P Qo	Qu+Qo Q1	Fire discharge	Qdesign= Q1+Qy Qh	Pipe Diameter			Head loss/m j	Velocity V	Head loss at the Pipe Lxj	Node	Altitude of Piezometric	Altitude of Ground	Operating Pressure	Static pressure	IDEAS		
No	m	k	m	Lt/sn	Lt/sn	Lt/sn	Lt/sn	Lt/sn	Lt/sn	Lt/sn	mm				m/sn	M	No	M	M	M	M			
Qtotal demand (Qdemand) =		4.00Lt/sn			q =	(1,5 x Qdemand) / Σli		=	6.00 / 2750	=	0.002167			Lt/sn/m										
3-M2	304	1	304.00	0.660	0.00	0.660	0.363	0.363	5.00	5.363	96.8	/90	PE 10	0.014605	1.09	4.45	M2	577.06	518.75	58.31	65.90	H=0.020		
4-5	84	1	84.00	0.183	0.00	0.183	0.101	0.101	2.50	2.601	55.4	/63	PE 10	0.021787	1.08	1.84	5	578.34	520.33	58.01	64.32			
4-6	203	1	203.00	0.439	0.00	0.439	0.242	0.242	2.50	2.742	55.4	/63	PE 10	0.024023	1.14	4.87	6	575.32	521.59	53.73	63.06			
4-7	98	1	98.00	0.213	0.000	0.213	0.117	0.117	2.50	2.617	55.4	/63	PE 10	0.022043	1.09	2.16	7	578.02	524.64	53.38	60.01			
3-4	85	1	85.00	0.184	0.835	1.019	0.101	0.936	2.50	3.436	66.0	/75	PE 10	0.015565	1.00	1.32	4	580.18	532.82	47.36	51.84			
2-3	166	1	166.00	0.360	1.679	2.039	0.198	1.877	5.00	6.877	96.8	/110	PE 10	0.008716	0.93	1.45	3	581.50	544.51	36.99	40.14			
1-2	155	0	0.00	0.000	5.961	5.961	0.000	5.961	5.00	10.961	110.2	/125	PE 10	0.010986	1.15	1.70	2	582.95	535.95	47.00	48.70			
			2750																					

Appendix-6 Design Outputs of Closed pipe Network with Netcad/water (V=1 m/sc)

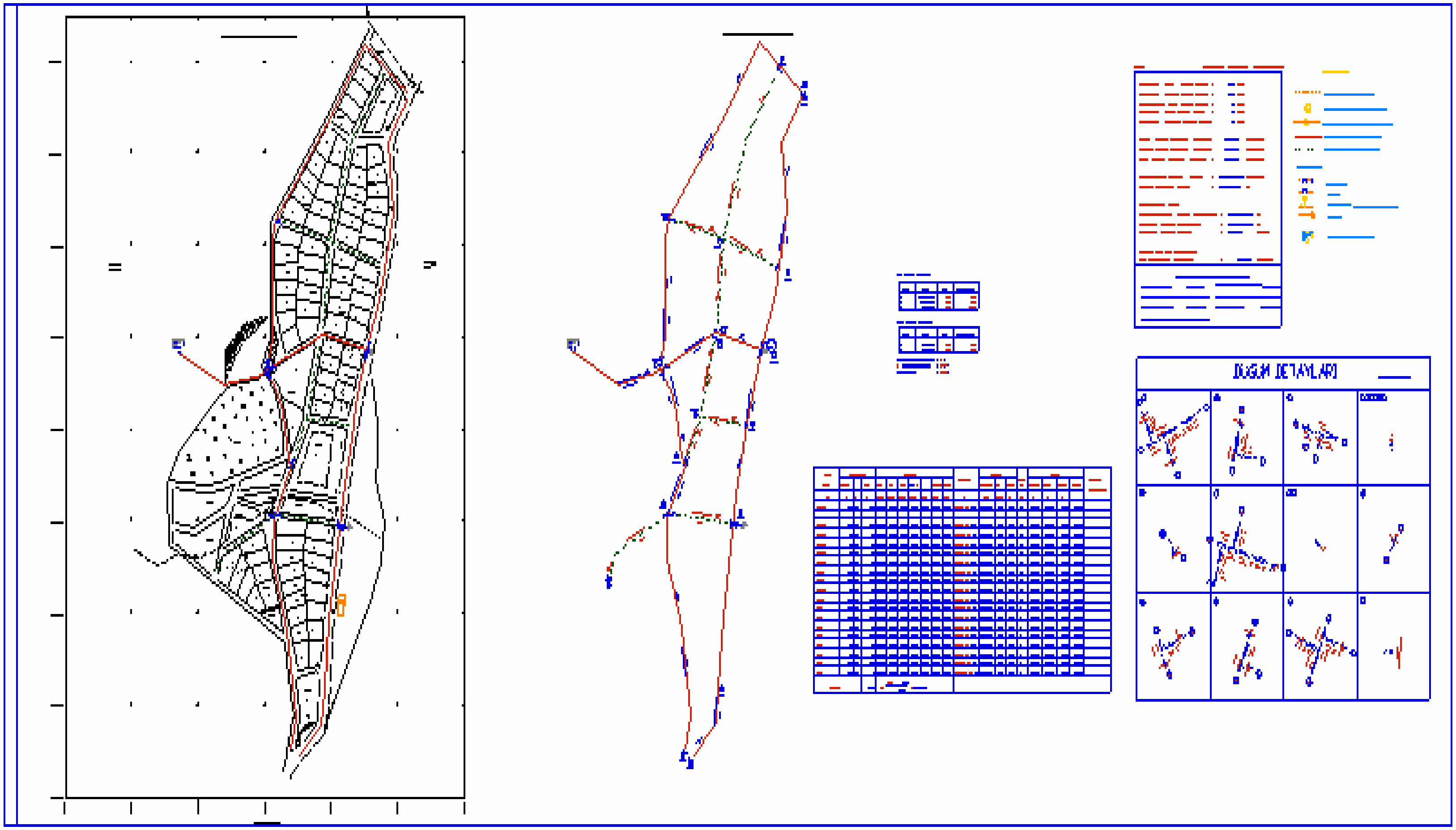
TABLE OF OF DESIGN OF PIPE NETWORK																				
	LENGTHS			DISCHARGES							AT THE PIPES					ALTITUDES				
Node to Node	Pipe Lengths	Relative Density	Relative Length	Relative Pipe discharge $P=q_{x} * L_i$	Endof pipe discharge Qu	Qb=Qu +P	=0,55*P Qo	=Qu+Qo Q1	Fire discharge	Qdesign= Q1+Qy Qh	Pipe Diameter	Head loss/m j	Velocity V	Head loss at the Pipe Lxj	Node	Altitude of Piezometric	Altitude of Ground	Operating Pressure	Static pressure	IDEAS
No	m	k	M	Lt/sn	Lt/sn	Lt/sn	Lt/sn	Lt/sn	Lt/sn	Lt/sn	mm		m/sn	m	No	m	m	m	m	
Qtotal demand (Qdemand) =		4.00Lt/sn		q =		(1,5 x Qdemand) / Σli			=	6.00 / 2750	=	0.0021818	Lt/sn/m							
9-M1	367	1	367.00	0.801	0.00	0.80073	0.440	0.440	5.00	5.440	83.228	0.011548	1.00	4.24	M1	575.97	523.49	52.48	61.16	0.00
9-10	92	1	92.00	0.201	0.00	0.20073	0.110	0.110	2.50	2.610	57.651	0.017724	1.00	1.63	10	579.49	522.97	56.52	61.68	
9-11	118	1	118.00	0.257	0.00	0.25745	0.142	0.142	2.50	2.642	57.995	0.017601	1.00	2.08	11	578.88	540.61	38.27	44.04	
12-9	56	1	56.00	0.122	1.252	1.37418	0.067	1.319	5.00	6.319	89.699	0.010582	1.00	0.59	9	581.51	529.26	52.25	55.39	
13-14	89	1	89.00	0.194	0.000	0.19418	0.107	0.107	2.50	2.607	57.611	0.017738	1.00	1.58	14	579.33	523.26	56.07	61.39	
13-15	67	1	67.00	0.146	0.00	0.14618	0.080	0.080	2.50	2.580	57.319	0.017844	1.00	1.20	15	579.84	519.81	60.03	64.84	
12-13	58	1	58.00	0.127	0.34	0.46255	0.070	0.406	2.50	2.906	60.824	0.016650	1.00	0.97	13	581.27	523.91	57.36	60.74	
2-12	113	1	113.00	0.247	1.835	2.08155	0.136	1.971	5.00	6.971	94.209	0.009994	1.00	1.13	12	581.93	527.38	54.55	57.27	
8-M1	374	1	374.00	0.816	0.000	0.81600	0.449	0.449	5.00	5.449	83.292	0.011538	1.00	4.32	M1	575.97	523.49	52.48	61.16	
8-M2	311	1	311.00	0.679	0.00	0.67855	0.373	0.373	5.00	5.373	82.713	0.011632	1.00	3.62	M2	577.06	518.75	58.31	65.90	

Appendix-6 Design Outputs of Closed pipe Network with Netcad/water (V=1 m/sc) *(continued)*

TABLE OF OF DESIGN OF PIPE NETWORK																				
	LENGTHS			DISCHARGES							AT THE PIPES					ALTITUDES				
Node to Node	Pipe Lengths	Relative Density	Relative Length	Relative Pipe discharge P=q_x*Li	Endof pipe discharge Qu	Qb=Qu +P	=0,55*P Qo	=Qu+Qo Q1	Fire discharge	Qdesign= Q1+Qy Qh	Pipe Diameter	Head loss/m j	Velocity V	Head loss at the Pipe Lxj	Node	Altitude of Piezometric	Altitude of Ground	Operating Pressure	Static pressure	IDEAS
No	m	k	M	Lt/sn	Lt/sn	Lt/sn	Lt/sn	Lt/sn	Lt/sn	Lt/sn	mm		m/sn	m	No	m	m	m	m	
Qtotal demand (Qdemand) =		4.00Lt/sn			q =	(1,5 x Qdemand) / Σli			=	6.00 / 2750	=	0.0021818 Lt/sn/m								
2-8	165	1	165.00	0.360	1.49	1.84500	0.198	1.683	5.00	6.683	92.245	0.010242	1.00	1.69	8	581.59	518.10	63.49	66.55	
3-M2	304	1	304.00	0.663	1.66	2.32227	0.365	2.024	5.00	7.024	94.567	0.009949	1.00	3.02	M2	577.06	518.75	58.31	65.90	0
4-5	84	1	84.00	0.183	0.00	0.18327	0.101	0.101	2.50	2.601	57.545	0.017762	1.00	1.49	5	578.34	520.33	58.01	64.32	
4-6	203	1	203.00	0.443	0.00	0.44291	0.244	0.244	2.50	2.744	59.104	0.017217	1.00	3.49	6	575.32	521.59	53.73	63.06	
4-7	98	1	98.00	0.214	0.000	0.21382	0.118	0.118	2.50	2.618	57.731	0.017695	1.00	1.73	7	578.02	524.64	53.38	60.01	
3-4	85	1	85.00	0.185	0.835	1.02045	0.102	0.937	2.50	3.437	66.152	0.015096	1.00	1.28	4	580.18	532.82	47.36	51.83	
2-3	166	1	166.00	0.362	1.68	2.04118	0.199	1.878	5.00	6.878	93.582	0.010072	1.00	1.67	3	581.50	544.51	36.99	40.14	
1-2	155	0	0.00	0.000	5.96	5.96100	0.000	5.961	5.00	10.961	118.135	0.007675	1.00	1.19	2	582.95	535.95	47.00	48.70	
			2750																	

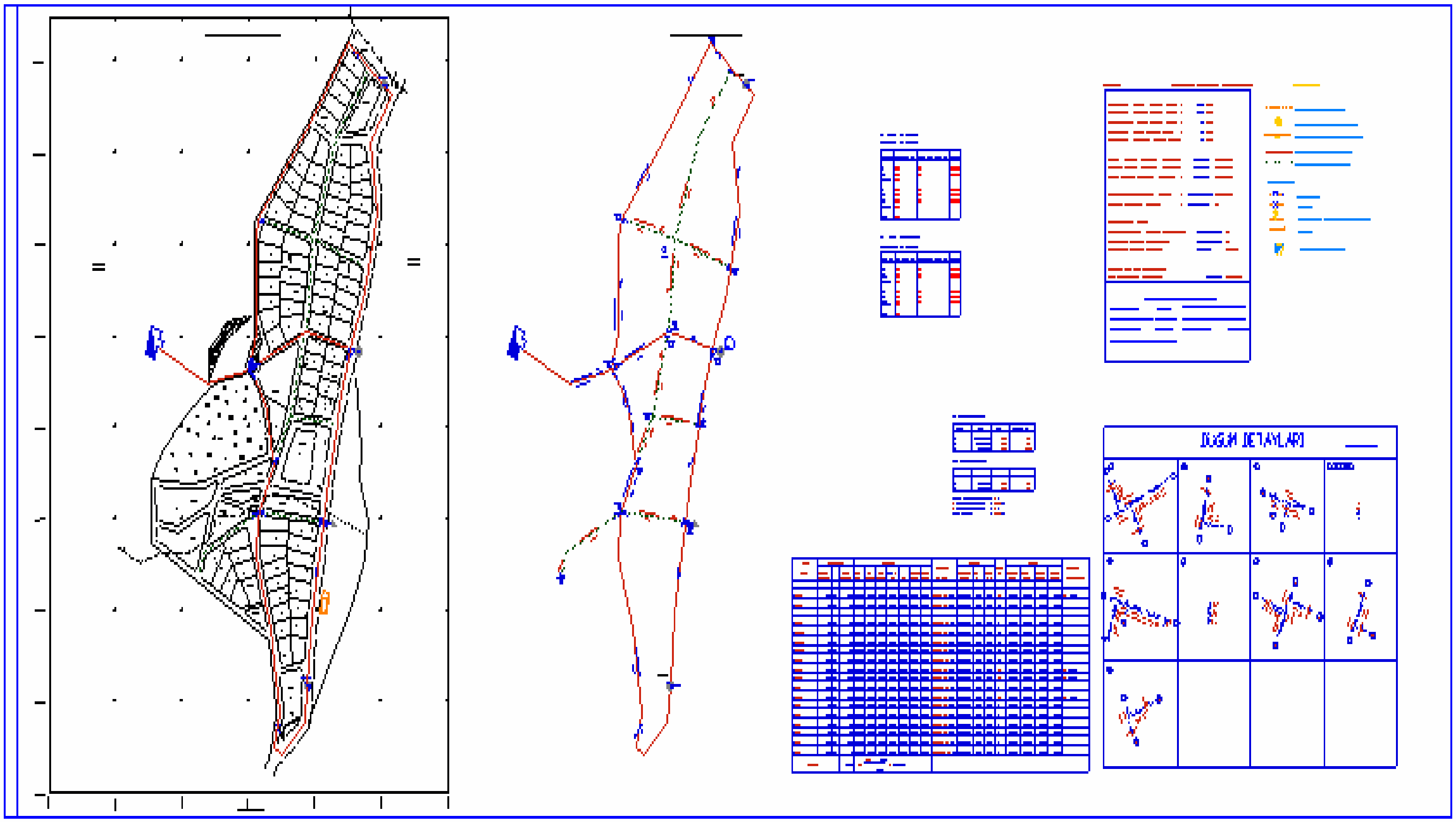
APPENDIX-7

BRANCHED (OPEN) PIPE NETWORK PLAN WITH NETCADWATER



APPENDIX-8

LOOPED (CLOSED) PIPE NETWORK PLAN WITH NETCADWATER



APPENDIX -9

HACIAGALAR TOWN PLAN IN KAHRAMANMARAS CITY