Optimal Design of Water Distribution Network System by Genetic Algorithm in EPANET-MATLAB Toolkit

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ABSTRACT

The emergence of computers in engineering applications increase the demand for improving the already available computer application tools for solving water distribution networks (WDNs). The demand is inevitable for both educational and practical purposes. In this thesis, the well-known computer application software EPANET is used in collaboration with MATLAB toolkit to optimize a water distribution network system given in the literature. In order to focus more on the practical aspects of computer application, and the resultant life cycle cost of a simple pipe network, a selected case study in northern part of Cyprus is analysed using these EPANET and MATLAB toolkits.

The computer application tools are applied for the purpose of improving the methodology of water distribution network systems. This has been achieving by optimizing the design results of any network system. In order to minimize the diameter of the pipes (minimizing the cost) used in the water distribution network system an optimization model is carried out through genetic algorithm process coded in MATLAB. This was achieved while preserving the hydraulic design principles in balance. The model is limited for using the hydraulic design principles that is valid only for water as a liquid and circular cross-sectional shape of the pipes.

The validity is tested through the water distribution network system of Hanoi city in Vietnam. The water distribution of this city was previously tested by other researches to achieve optimized solutions. Therefore, the results of this thesis got the opportunity to compare results with previous findings. As the validity is confirmed, the model is

tested over Karaoğlanoğlu region in North Cyprus in order to select the best result for three different alternative scenarios. This is achieved by comparing the life cycle costs of the three scenarios.

Although the basics of the implementation are sufficiently covered, the provided software and codes can be improved if life cycle cost analyses can be added to the EPANET- MATLAB toolkit.

Keywords: EPANET-MATLAB Toolkit, Genetic Algorithm, Optimization, Water Distribution Network.

ÖZ

Mühendislik uygulamalarında bilgisayar kullanımının artması ile birlikte, su dağıtım şebekeleri analizlerini gerçekleştirmek ve çözümlemek için mevcut bilgisayar uygulamalarının geliştirilmesine olan talebi artırmıştır. Bu talep hem eğitim ve hem de pratik amaçlar için kaçınılmazdır. Bu tez çalışmasında, EPANET adlı tanınmış bilgisayar yazılım uygulaması yardımı ile, literatürde daha önce birkaç kez optimize edilmeye çalışılan bir su dağıtım ağı sistemini optimize etmek için MATLAB araç seti ile birlikte kullanılmıştır. EPANET/MATLAB uygulamasının güvenirliği bu örnek üzerinden test edilmesinden sonra ise Kıbrıs'ın kuzeyindeki seçilmiş bir vaka incelemesi ile yakın zamanda uygulanan basit bir su şebekesinin analizleri optimize edilmiş ve pratikte uygulanan su şebekesinin maliyet analizleri karşılaştırılmıştır.

Bu bağlamda su dağıtım şebeke sistemlerinin metodolojisinin geliştirilmesi amacıyla farklı iki yazılım programının birlikte çalışması sağlanmış ve bilgisayar uygulamaları yardımı ile verimli sonuçlar elde edilmiştir. Amaç uygulanan herhangi bir çalışmada bir ağ sisteminin tasarım sonuçlarını optimize ederek geliştirmektir. Su dağıtım şebekesi optimizasyon modelinde EPANET ile belirlenen boru çaplarını MATLAB'da kodlanmış genetik algoritma işlemi ile en aza indirgemek (maliyeti en aza indirmek) ana çalışma amacını oluşturmuştur. Bu, dengedeki hidrolik tasarım ilkelerini koruyarak başarılmıştır. Geliştirilen model yalnızca dairesel kesitli borular ve akışkan olarak ise “su” içeren durumlarda çalıştırılmak üzere sınırlandırılmıştır.

Üzerinde çalışılan kombine analizin geçerliliği, Vietnam'daki Hanoi şehrinin su dağıtım şebekesi analizleri vasıtasıyla test edilmiştir. Bu şehrin su dağıtım sistemi daha

önceleri optimize edilmiş ve bu çözümler güvenirliği artırmak amacı ile mevcut çalışma ile karşılaştırılmıştır. Bu nedenle, bu tezin sonuçları önceki bulgularla sonuçları karşılaştırma fırsatı yaratmıştır. Geçerlilik doğrulanırken, model üç farklı alternatif senaryonun en iyi sonucunu seçebilmek amacıyla Kıbrıs'taki Karaoğlanoğlu bölgesi üzerinde test edilmiştir. Bu, üç senaryonun yaşam döngüsü maliyetleri de ayrı ayrı incelenmiştir.

Uygulamanın temelleri yeterince kapsanmış olmakla birlikte, yaşam döngüsü maliyet analizleri EPANET-MATLAB araç setine eklenebilirse sağlanan yazılımlar ve kodlar iyileştirilebilir.

Anahtar kelimeler: EPANET-MATLAB Araç Seti, Genetik Algoritma, Optimizasyon, Su Dağıtım Şebekesi.

DEDICATION

TO:

My mother, to her indescribable mercifulness in my life,

My second part, My beloved wife, the source of all successfulness

My gorgeous kids, the best gifts from God.

My lovely sisters and their families.

The pure soul of my father

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“In the Name of God, Most Gracious and Most Merciful’’

I wish to express my deepest appreciations to beautifully and lovely family, to my mother, and all my sisters with their families, who have encouraged and supported me in all my endeavors, especially to continue in graduate studies. Also, I would like to express my deepest gratitude to my true treasures in life, my wife, Mrs. Shaida for her love, kindness, endless wistful, encouragement, and all kind of support during this study. My dear daughter Sidra and my dear son Muhamad, I am so grateful to God for giving them to me such nicest gifts. Also, I hope to see my kids at the highest academic level.

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A	Cross-sectional pipe area (m^2)
C	Hazen-William roughness coefficient (dimensionless)
C_d	Decommissioning and disposal
Ce	Annual cost of energy
C_e	Energy costs
C_{env}	Environmental costs
C_f	Unit conversion factor (dimensionless) for Manning Eq.
C_{ic}	Initial cost, purchase price
C_{in}	Installation and commissioning
C_m	Maintenance costs
C_o	Operating costs
C_s	Downtime costs
CV	Control volume
D	Pipe diameter (m)
D_e	Equivalent pipe diameter (m)
D_h	Hydraulic diameter (m)
e	Spindle depth flow in pipe (m)
f	Coefficient of friction (dimensionless)
F_A	Annual average factor (dimensionless)
F_D	Daily average factor (dimensionless)
GA	Genetic algorithm
h	Pressure head (m)
H	Total energy head (m)
h_f	Major head loss (m)

h_L	Hydraulic head loss (m)
h_m	Minor head loss (m)
HW	Hazen-William coefficient
k_f	Form- loss coefficient (dimensionless)
K_p	Coefficient pump plant cost (dimensionless)
L	Length of the pipe (m)
L_c	Contraction transition length (m)
LCC	Life cycle cost
Le	Expansion transition length (m)
MaxIt	Maximum iteration
m_p	Exponent pump plant cost (dimensionless)
n	Manning's coefficient (dimensionless)
Npop	Population size
P	Pressure with the pipe at specific location (pa)
P_c	Crossover probability (percentage)
P_m	Mutation probability (percentage)
P_o	Power pump (kW)
q_{inlet}	Mass input rate ($kg\ /\ sec$)
q_{outlet}	Mass outlet rate ($kg\ /\ sec$)
Q	Volumetric discharge flow (m^3/sec)
R	Bend radius in pipe (m)
r	Expansion ratio (dimensionless)
R_E	Cost power electricity
R_e	Reynolds number (dimensionless)
R_h	Hydraulic radius (m)
T	Temperature in °C
U	Unit conversion factor (dimensionless) for Hazen-William Eq.
V	Average velocity ($m\ /\ sec$)

ν	Kinematic viscosity (m^2 / sec)
V_{max}	Maximum velocity in the pipe (m / sec)
WDN	Water distribution network
W_p	Wetted perimeter (m)
x	Distance from the transition inlet (m)
Z	Reference elevation (m)
α_c	Contraction Angle in radius
α_e	Expansion Angle in radius
ΔQ	Correction of discharge (m^3/sec)
α	Angle in pipe (degree)
γ	Specific weight (N)
ε	Average height roughness of the wall in pipe (mm)
η	Combined sufficiency of the pump and motor
ρ	Density (kg / m^3)
g	Acceleration due to gravity (m / sec^2)

Chapter 1

INTRODUCTION

1.1 Literature Review

An initial step for any research study is to understand and summarize the previous works and the findings in that area carried out by others. Series of different books, thesis, conference papers and journal articles should be reviewed and the basic principles and fundamental findings of the topic should be thoroughly analysed. In this thesis, such a review is carried out based on pressurized pipe flow theories as will be discussed below.

Water transmission from a source to demanding regions can be performed either through pressurized pipes (closed conduit) or in open channels. Since water transmission is related directly to health issues, it is generally required to transport domestic water through closed conduits. Therefore, all around the world, water distribution network systems have been constructed to achieve the goal of the healthy transportation of water.

Water distribution systems generally consist of four primary components: (1) the actual sources of water and the intake mechanisms; (2) the storage and the treatment mechanisms; (3) the transmission mains, and (4) the actual network for distributing the water. The objective of the distribution system is the provision of water to all residential areas, industrial plants, and the public in general. It is especially necessary that, houses enjoy a consistent water supply at the desired pressure (Lencastre, 1987). The primary goal of a water supply agency is to provide an uninterrupted supply of water at the pressures required to draw a sufficient quantity (Cunha & Sousa, 1999). The components of conventional water distribution systems include valves, reservoirs, tanks, pumps, and pipes. Furthermore, the design of such systems takes into consideration the cost-effective maintenance and quality assurances, the average velocity and the pressure limitations, and the time pattern of the demand (Suribabu, 2006).

The solution process for a pressurized looped pipe network is a quite complex procedure. Modelling or simulation of such a hydraulic behaviour, requires simultaneous solutions of number of non-linear equations, while considering the conservation of mass and the conservation of energy in collaboration with the head loss function due to friction.

The aim of modelling, simulating, and analysing a pipe network is exclusively used to determine either the pipe flow rates (Q) or the nodal point hydraulic heads (h) (Niazkr et.al, 2017a). Therefore, related variables are largely dependent on either the hydraulic heads at nodal points or the demand under steady-state conditions. Once the conventional water distribution network meets the required value of the hydraulic heads at all nodal points, the ensuing inquiry then makes it necessary that the variables h and Q are found. Furthermore, the energy and the continuity equations in such cases determine the network flow conditions and the iteration based methods are used to solve the governing equations. As the iteration method is applied in terms of discharges (Q), the governing equations are collectively called the Q-based method. On the other hand, if the iteration-based method is applied in terms of heads at the nodal points, the governing equations are called the h-based method. The governing equations in both methods constitute the foundation of the non-linear algebraic equations (Niazkr et.al, 2017b). In both methods, initial assumptions are necessary in order to obtain conveying iteration solutions. Generally, even though all the methods can be used in analysing water distribution networks system (WDN) essentially give identical results, they are formulating and detailing the networks differently (Niazkr et.al, 2017b).

A non-linear systems network was simulated through the Newton-Raphson and Hardy-Cross methods using MATLAB software. The nonlinearity of the system is given in the square power of the discharge in head loss equations. In comparing the two methods mentioned above, which are used to analyse the WDN, showed numerically that, the summation in Newton Raphson has high accuracy when compared to Hardy-Cross method (Abdulhamid et.al, 2017). Also, the solution of Newton-Raphson method can be resolved a lesser number of iterations hence faster when compared to Hardy-Cross method.

After analysing the network system, these non-linear equations can be solved using a computer software program known as EPANET. Then, the genetic algorithm in MATLAB could be used to find the minimum diameter needed to get the optimal cost, and ultimately the optimum design as one of the perspective principle in design criteria evaluations is the cost management. As such, choosing between commercially available pipes of different diameters, in order to minimize the cost of water distribution network system is hardly easy (Akdoğan, 2005). Water distribution systems have been designed using a variety of methods. Reaching a consensus on a generally accepted method has, however, proved to be not easy (Cisty & Bajtek, 2009). This can be explained, in part, by the fact that the optimal design of a water distribution network significantly depends on the solution of the necessary optimization problems (Reehuis, 2010).

While later studies utilized non-linear programming (NLP) (Su, et al. 1987; Xu, and Goulter, 1999) or chance-constrained programming (Lansey & Mays, 1989) in solving pipe network optimization problems, linear programming (LP) (Alperovits & Shamir, 1977; Shamir & Howard, 1985) was the preferred choice for most earlier studies. Moreover, researches used for determining the capacity of linear optimization methods the optimal design of a water distribution network is too extensive (Schaake, 1969).

Generally, MATLAB has been used to design the WDNs as a flexible computer program that it is simple enough to use genetic algorithm (GA). In principle, the GA is simulating the mutation, crossover, selection, and reproduction of living creatures using random selection processes to find either the maximum or minimum of an unconstrained function, and has a wide application opportunity through civil engineering project studies (Riazi & Turker, 2017).

Several number of researchers have preferred utilizing genetic algorithms in WDNs as the former has proven its ability to provide better results even in complicated cases. Simpson et.al. (1994) applied the GA to solve the pipe network systems, and compared it with the complete enumeration method and the non-linear programming. Dandy et.al. (1996) improved Simpson's efficiency by simplifying the GA while minimizing the solution time and the error value.

The GA is regarded as a search procedure, applicable to different problems for optimal solutions. Pilar et.al. (1999) proposed convergence optimization to modify the Genetic

Algorithm. They tried to explain how species are selected to change and how they are transformed into different species. To meet the needs of nodes and the layout of hydraulic elements, a cost-effective pipe size is used to determine the optimal design of a looped network for gravity systems.

The literature review motivated us to use the well-known WDN software and EPANET to learn the analysing methodology and to use it in collaboration with ready available GA program written in MATLAB. The performance of the combined use of MATLAB and EPANET is then tested through well-known Hanoi network in Vietnam to confirm the reliability of the results. Later, the combined use of EPANET and MATLAB are used to analyse the three alternative cases in Kyrenia Region of North Cyprus.

1.2 Definition of Observed Problems

Water distribution network systems (WDNS) are expensive investments. Since the design procedure of such projects having alternative solutions, it is necessary to find a way to approach to the optimal design so as to minimize the investment cost.

One of the worldwide accepted readily available free software used for analysing WDN's is EPANET. However, as same as the similar software, EPANET is not capable to optimize the given problems. Therefore, a genetic algorithm code written in MATLAB is used in collaboration with EPANET, in order to optimize the cost of WDN's. In this study, WDN analysis and optimization of the results are achieved.

1.3 Context of this Study

This study was carried out at Eastern Mediterranean University. The tools used include the EPANET software, MATLAB software, and the Civil Engineering Department's computer laboratory. All the theoretical calculations related with the water distribution network are carried out by the help of EPANET software and the optimization procedure through the genetic algorithm existing within MATLAB software. These two software programs are combined in EPANET-MATLAB toolkit.

1.4 Questions at the Initial Stage

The following questions form the basis of this study:

1. What are the fundamental equations of pressurized pipe flow?
2. How can the water distribution network be analysed? By which methods?
3. What is the EPANET software program and how is it used to analyse WDNs?
4. What is genetic algorithm (GA) and how is it used to solve for optimum design in the WDNs?
5. How connections can be done between EPANET and GA in MATLAB writing code so as to design the WDNs?
6. How can we get the optimum design and raise a comment on it?

1.5 Aims and Objectives

The major aim of this study is to improve the methodology of WDS for analysis and optimal design by achieving hydraulic balance and minimum pipe diameters to get the least cost necessary for an optimum design. To perform this task, analyzing a simplified model of WDS and employing results with a code using genetic algorithm is needed. It is expected that; the following objectives would lead to fulfilment of the major aim. These objectives are:

1. To provide a broad introduction to the hydraulics theory necessary for WDS analysis, emphasis on the terminology and mathematical formulations are needed to create a WDS simulation model. This objective is addressed in Chapter 2.

2. To apply the theoretical equations of the pipe flow and hydraulic performance of the simple analyzed WDNs, a software program with a greater degree of accuracy is used, such as EPANET 2 software. This objective is addressed in Chapter 3.
3. To achieve to an optimal design for the analysed WDNs, MATLAB software code capable for optimization through genetic algorithm is used to determine the suitable pipe diameters. This objective is also addressed in Chapter 3.
4. To incorporate objective number 2 and 3, and to transport the analyzed WDNs into the MATLAB code, the EPANET_MATLAB toolkit is used to achieve this objective, which is also addressed in Chapter 3.
5. To test and examine the reliability of EPANET-MATLAB toolkit, a real urban water distribution network system is worked out as a proof of concept. This objective can be addressed in Chapter 4.
6. To benefit from the personal methodology, simple case-study is worked out and alternatives are discussed. Chapter 4 also includes this objective.

1.6 Proposed Methodology

This study requires the extraction of reliable, accurate, and practical results. As such, the study's primary methodology is quantitative. The quantitative analysis composed of approach used in previous studies, the necessary conditions, the references, and the standards, they are all important to determine the physical characteristics of the water distribution network.

The methodology applied began by inputting the initial data of the WDNs for the analysis to determine the hydraulic performance of the system. This will be the primary reason to get discharge values (Q) in the pipes and the pressure values at all nodes based on assumed discharges. Then, the combination of EPANET and MATLAB toolkit is used to perform the analysis using genetic algorithm approach to determine the minimum pipe diameters to be achieved for the optimal design of the network system. Effort to develop an algorithm for the use in the optimal design of water distribution networks, the genetic algorithms and EPANET toolkit are combined with MATLAB. Also, to test this methodology, the Hanoi pressurized pipe distribution network is taken as a case study.

At the end, the results of the design, the final optimum cost, the pressure heads, and the other hydraulics criteria are all tabulated, evaluated, compared and discussed.

1.7 Limitations of this Study

It is evident that finding the optimal cost isn't so easy in the large search spaces. It changes depending on how the program is used to achieve this aim, which is flexible. In addition, there are some hydraulic limitations, such as:

i. All the used equations are valid for water at temperature -℃ used as the liquid in this study.

ii. The cross sectional shapes of the pipes under consideration are all assumed to be circular. Non-circular geometric cross sectional shapes are not included.

1.8 Organization of the Thesis

This thesis is divided into five chapters. Following this first, introductory chapter, which outlines the methodology, objectives, aims, and problem statement of the study, as well as offers a brief introduction of the research, Chapter 2 shortly explains the basic principles of pressurized pipe flow pipe flow, then resistance cases, pipe flow problems, equivalent pipe, and the necessary conditions for any network of pipes to analyze the water distribution networks, methods and conditions for the same purpose.

Chapter 3 is focused on the introduction of the simple model, the pressurized pipe distribution network analysis by EPANET program and the steps of the design by using methods Hardy-Cross. Also, design steps of MATLAB code are for genetic algorithm is discussed. In Chapter 4, the same procedures for the previous chapter was explained in more detail and a case-study is worked out. The final chapter, Chapter 5 contains the conclusions and the recommendations for future studies.

Chapter 2

FUNDAMENTAL PRINCIPLES OF PRESSURIZED PIPE FLOW

2.1 Overview

The most common way of transporting fluids in close conduits with varying discharges is by means of pressurized pipe flow. Due to pressurized flow, no free surface is allowed to form with the pipe flow as the entire area is filled with fluid. In cases when the atmospheric pressure is reasonably greater than the fluid pressure at that specific gravity suction pressure occurs, the resulting the siphon action. This is rarely the case, in general, since the fluid pressure is typically higher than atmospheric pressure. Furthermore, in the case that the pressure in the pipe is less than atmospheric, liquid may change its phase into a gaseous state and blocking the flow.

In general, the conservation of mass simply referred as the continuity equation is used to analyse or determine the weight of variables affecting the flow.

$$Q = A * V \tag{2.1}$$

In a circular pipe with diameter D, the continuity equation for steady flow is given as, where Eq. (2.1) can be written in more detail as,

$$Q = \frac{\pi}{4} D^2 V \tag{2.2}$$

where Q = volumetric rate of flow (L^3T^{-1}), and, V = average velocity of the flow (LT^{-1}). On the other hand, the energy necessary for the fluid to move from one location to another is also an important variable when dealing with the hydraulics of fluids. The primary condition underlying the principle of conservation of energy is that it is impossible either to create energy, or to destroy it. From this, the conclusion can be reached that the difference in energy between two points is constant and it is path-independent. Pipe flow in pressurized usually described in ‘head’ terms. The amount of energy present at any point in a distribution system is the combination of the elevation head, the pressure head, and the velocity head. The amount of energy between two points in a frictionless environment is calculated using the Bernoulli equation shown below:

$$\frac{V_1^2}{2\mathrm{g}} + \frac{P_1}{\gamma} + \mathrm{z}_1 = \frac{V_2^2}{2\mathrm{g}} + \frac{P_2}{\gamma} + \mathrm{z}_2 = \mathrm{H} \qquad (2.3)$$

Where H = Total energy (m)

V = Flow velocity (m/sec)

P = Pressure (N/m^2)

z = Elevation above some fixed level (m)

g = Acceleration due to gravity (9.81 m/sec^2)

γ = Specific weight of water (N/m^3)

2.2 General Classification of Flow

The flow of pressurized water in a conduct is affected by a variety of factors, including gravity, viscosity, space, and time. This section contains a description of how various pipes pressurized and the open channel flows can be classified; with the discussion of the standard pipe hydraulic classifications (M. Hopkins, 2012). The table below outlines the classifications of pressurized pipe and the open channel flows, that are described in the following paragraph.

Table 2.1: Flow Classification

Classification criteria	Flow classification
Time	Steady Flow
	Unsteady Flow
Space	Uniform flow
	Non uniform flow
Reynold's number	Laminar
	Turbulent
	Transitional
Froude number	Subcritical
	Critical
	Super critical

When the velocity is relatively consistent with respect to time – both in terms of direction and magnitude – at all points in the flow, a steady flow is said to exist. Conversely, an unsteady flow exists, when the velocity varies in either direction or magnitude with respect to time at any point within the flow. A flow is said to be uniform when it has a constant velocity and non-uniform when there is any sort of variation in its velocity basically due to cross-sectional changes. A turbulent flow occurs as a result of vigorous mixing caused by different sizes of eddies in the flow. A fully turbulent flow cannot be replicated as it is chaotic and random. Conversely, a laminar flow follows a smooth and regular path. Reynold's number can be used in determining whether a flow is turbulent or laminar. Flows mainly occurring between laminar and turbulent flows are known as transitional flows. Similarly, the Froude number is used to denote open channel flows as either supercritical, critical, or sub-critical. Super-critical flows and sub-critical flows are characterized by a high flow velocity and low depth, and a low flow velocity and high flow depth respectively.

The classification of pipe flows as non-uniform, steady, and laminar or transition or turbulent is due to the fact that they are usually pressurized. A pipe flow is considered to be an open channel flow if the pipe isn't completely filled with the fluid and the fluid flow is only driven by the gravitational forces. Nonetheless, pipe flow usually refers to a pressure-driven full conduit flow.

2.3 Hydraulic Losses

The primary objective here is to generalize the one-dimensional Bernoulli equation for use in relation to viscous flow. The total energy head $H = \frac{V^2}{2g} + \frac{P}{\rho g}$ +z becomes inconsistent if we take into account the viscosity of the fluid. In terms of flow direction, the result of the friction caused by fluid viscosity is a condition whereby $\frac{V_1^2}{2g} + \frac{P_1}{\rho g} + z_1 > \frac{V_2^2}{2g} + \frac{P_2}{\rho g} + z_2$, hence an additional friction loss term has to be inserted. Restoring the equality of these two functions requires that, a scalar quantity is added to the right-hand side of the inequality:

$$H = \frac{V_1^2}{2g} + z_1 + \frac{P_1}{\rho g} - \Delta h_L = \frac{V_2^2}{2g} + z_2 + \frac{P_2}{\rho g} \tag{2.4}$$

The added scalar quantity Δh_L is also known as hydraulic loss. The hydraulic loss of two pipes with unique cross sections measured from the datum is identical to the total energy difference of the cross section:

$$\Delta h_L = H_1 - H_2 \tag{2.5}$$

One should never forget that H_1 is always greater than H_2. When $z_1 = z_2$ in a horizontal pipe and the pipe diameter is constant, so the average velocity at section (1) will be equal to the average velocity at section (2) ($V_1 = V_2$), thus, the hydraulic loss is also identical to the head loss (head of pressure drop):

$$\Delta h_L = \frac{P_1 - P_2}{\rho g} \tag{2.6}$$

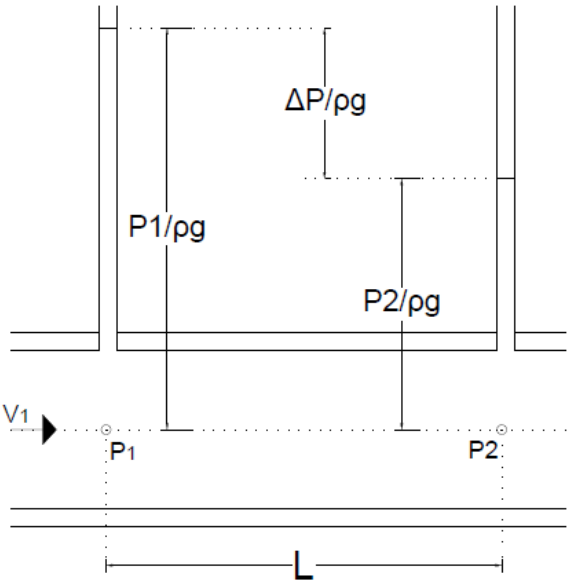

Figure 2.1: Pipe Friction Loss of Constant Diameter in the Horizontal Pipe, this Loss Can be Accounted by the Pressure Drop by Height: ΔP/ρg = h

Equation (2.6) can only be applied to horizontal pipes. Generally, when $V_1 = V_2$ but $z_1 \neq z_2$, the formula for head loss is as follows:

$$\frac{P_1 - P_2}{\rho g} = (z_2 - z_1) + f\frac{L}{D}\frac{V^2}{2g} \tag{2.7}$$

Pressure is typically calculated in relation with the atmospheric pressure, which is taken to be equal to zero, as gage pressure. Negative gage pressure, therefore, occurs when atmospheric pressure is greater than the actual pressure, while gage positive pressure occurs when the pressure is greater than atmospheric pressure. It is imperative that the material used for the pipe is able to withstand the pressure exerted by the fluid. Furthermore, because fluid pressure can dramatically rise under certain conditions, pipes should also be able to withstand to this increased pressure under such conditions.

The state of flow at any point in a pipe system can be determined using the law of conservation of energy given in Fig 2.2.

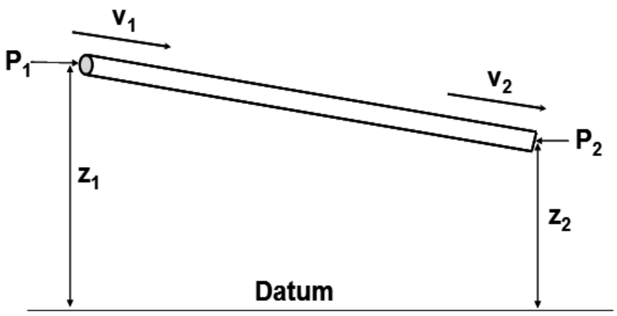

Figure 2.2: Application Conservation of Energy

The energy lost as a result of moving from P_1 to P_2 is represented by h_L. This energy loss results primarily from efforts to counter pipe friction (major losses). It could also result from the energy lost in fittings and valves, or from the turbulence that occurs when switching between pipes of different sizes (minor losses).

2.3.1 Major Losses (Surface Roughness)

Friction loss occurs whenever fluid moves through a pipe due to the viscosity of the fluid. The three formulas commonly used to calculate friction losses are the: Hazen-Williams, Darcy-Weisbach, and Manning's equations.

Pressure loss is proportional from the length of the pipe (L) to the diameter of the pipe (D) (L/D ratio) and the velocity head. In laminar flows with low velocities, the viscous shearing that occurs in streamlines in close proximity to the pipe wall causes friction loss and a clear definition is provided for the friction factor (*f*).

Conversely, in fully turbulent flows with high velocities, the contact between water particles and surface irregularities on the pipe inner wall result in frictional losses; the friction factor in fact is a function of the roughness of the inner wall of the surface (ε).

The velocity in most engineering applications is below the requirement for a fully turbulent flow and *f* is a function of both boundary layer viscosity and pipe surface roughness. We can experimentally determine the values of f and plot these in a dimensionless form against the Reynolds Number (Re) to form a Moody Diagram.

The Darcy–Weisbach equation provides the head loss that results from resistance on the inner wall surface of the pipe:

$$\mathrm{h}_f = \frac{\Delta p}{\gamma} = f\,\frac{\mathrm{L}}{D}\,\frac{\mathrm{V}^2}{2\mathrm{g}} \tag{2.8}$$

where the length of the pipe is represented by L, and f is the friction factor (the coefficient of the surface resistance). The following equation results when we remove V from equations (2.2) and (2.8):

$$\mathrm{h}_f = \frac{8f\mathrm{LQ}^2}{\pi^2\mathrm{gD}^5} \tag{2.9}$$

The average height of the pipe wall's roughness projection, ε, determines the coefficient of a turbulent flow's surface resistance. Table 2.2 outlines the average wall roughness of commercial pipes.

Table 2.2: Average Roughness Surface Heights (Naslihan, 2018)

Pipe materials	$\varepsilon(mm)$	ε(ft)
Brass	0.0015	0.000005
Concrete		
Steel forms, smooth	0.18	0.0006
Good joints, average	0.36	0.0012
Rough, visible form marks	0.6	0.002
Copper	0.0015	0.000005
Corrugated metal (CMP)	45	0.15
Iron (common in order water lines, excepted ductile or DIP-which is welded used today)		
Asphalt lined	0.12	0.0004
Cast	0.26	0.00085
Ductile, DIP-cement mortar lined	0.12	0.0004
Galvanized	0.15	0.0005
Wrought	0.045	0.00015
Polyvinyl chloride (PVC)	0.0015	0.000005
High-density polyethylene (HDPE)	0.0015	0.000005
Steel		
Enamel coated	0.0048	0.000016
Riveted	0.9-9.0	0.003-0.03
seamless	0.004	0.000013
Commercial	0.045	0.00015

The flow's Reynolds number (R_e) is also important when determining the surface resistance coefficient. R_e is mathematically represented as:

$$R_e = \frac{VD}{v} \tag{2.10}$$

Where the kinematic fluid viscosity v can be calculated through the equation provided by Swamee (2004), which is intended specifically for water.

$$v = 1.792 \times 10^{-6} \left[1 + \left(\frac{T}{25}\right)^{1.165}\right]^{-1} \tag{2.11}$$

Where the temperature of the water in ℃ is represented by T. The following equation results if one inserts Q/A given in Eq. (2.1) instead of V in Eq. (2.10) as:

$$R_e = \frac{4Q}{\pi v D} \tag{2.12}$$

Colebrook (1938) found that, for turbulent flows (where $R_e \geq 4000$), f is calculated as follows:

$$f = 1.325 \left[\ln \left(\frac{\varepsilon}{3.7D} + \frac{2.51}{R_e\sqrt{f}} \right) \right]^{-2} \tag{2.13}$$

Where: ε is an average roughness height on the inner wall surface of the pipe (mm).

Using Eq. (2.13), a family of curves between f and R_e was constructed by Moody (1944) for different values of relative roughness ε /D (Figure 2.3). On Moody Chart f depends exclusively on R_e in laminar flows (where $R_e \leq 2000$), and is calculated using the Hagen–Poiseuille equation:

$$f = \frac{64}{R_e} \tag{2.14}$$

No information is provided to estimate f when R_e is a value within the critical range (between 2000 and 4000). The following equation was provided by Swamee (1993) for calculating the value of f in laminar and turbulent flows, and the transition between them:

$$f = \{ (\frac{64}{R_e})^8 + 9.5 \left[\ln \left(\frac{\varepsilon}{3.7D} + \frac{5.74}{R_e^{0.9}} \right) - (\frac{2500}{R_e})^{-16} \right]^{-2} \}^{\ 0.125} \tag{2.15}$$

For transitional turbulent flows, Eq. (2.15) is simplified as

$$f = 1.325 \left[\ln \left(\frac{\varepsilon}{3.7D} + \frac{5.74}{R_e^{0.9}} \right) \right]^{-2} \tag{2.16}$$

Combining the relationship given in Eq. (2.14) with Eq. (2.16) the f –value can be reformulated as:

$$f = 1.325 \left\{ \ln \left[\frac{\varepsilon}{3.7D} + 4.618 \left(\frac{vD}{Q} \right)^{0.9} \right] \right\}^{-2} \tag{2.17}$$

The same formulae can be applied for non-circular pipes if one substitutes the hydraulic diameter D_h, in terms of D in the definition of R_e, and in the ε/D term of the

Colebrook equation. The hydraulic diameter D_h for a pipe with a cross-section area A and perimeter (W_p) is equal to 4 times the hydraulic radius R_h. This hydraulic radius is calculated by this the ratio of water cross sectional area (A) and the wetted perimeter (W_p). A circular pipe, where $A = \pi D^2/4$ and $W_p = \pi D$, has its hydraulic radius calculated as:

$$R_h = \frac{A}{W_p} = \frac{\pi D^2/4}{\pi D} = \frac{D}{4} \tag{2.18}$$

From Eq. (2.18), we obtain the hydraulic diameter:

$$D_h = \frac{4 \times cross\ sectional\ area}{wetted\ perimeter} = \frac{4A}{W_p} \tag{2.19}$$

Where: D_h Hydraulic diameter

W_p Wetted perimeter

The empirically-based Hazen-Williams equation uses variables to those used in the Darcy-Weisbach equation with the exception that its C-factor depends on the type of material used for the pipe, the diameter, and condition of the pipe as opposed to the friction factor. C-factor can be found using a set of dedicated tables; lower C-factors indicate higher degrees of friction loss. Formulated by Gardner Williams and Alan Hazen in the early 1900's, the Hazen-Williams equation (Eq. 2.20) is the most common formula used for friction loss in America (Martorano, 2006).

$$h_L = \frac{UL\, Q^{1.852}}{C^{1.852}\, D^{4.87}} \tag{2.20}$$

where

C Hazen-William roughness coefficient (in Table 2.3)

L Pipe length (m, ft)

Q Flow rate (cms, cfs)

D Pipe diameter (m, ft)

U Unit conversion factor (10.7 for SI, 4.73 for English unit)

Table 2.3: Hazen-Williams Roughness Coefficient (Naslihan, 2018)

Pipe materials	C_{HW}
Brass	130-140
Cast iron (common in order water lines)	
New, unlined	130
10-year-old	107-113
20-year-old	89-100
30-year-old	75-90
40-year-old	64-83
Concrete or concrete unlined	
Smooth	140
Average	120
Rough	100
Copper	130-140
Ductile iron (Cement mortar lined)	140
Glass	140
High-density polyethylene (HDPE)	150
Plastic	130-150
Polyvinyl chloride (PVC)	150
Steel	
Commercial	140-150
Riveted	90-110
Welded (seamless)	100
Vitrified clay	110

An alternative empirically-based formula is Manning's equation, which was developed by Robert Manning in 1889 (Fishenich, 2000). This equation uses the roughness coefficients contained in Table 2.4, where the rougher materials are distinguished by higher coefficients. Manning's equation (Eq. 2.21) is used primarily in relation to open-channel flows and only occasionally for pressurized pipe distribution systems primarily in Australia.

$$h_L = \frac{C_f L (nQ)^2}{D^{5.33}} \tag{2.21}$$

where

n Manning’s coefficient

L Pipe length (m, ft)

Q Flow rate (cms, cfs)

D Pipe diameter (m, ft)

C_f Unit conversion factor (10.29 for SI, 4.66 for English unit)

Table 2.4: Manning Roughness Coefficient for Pipe Flow (Naslihan, 2018)

Type of pipe	Manning's n	
	Min	Max
Brass	0.009	0.013
Cast iron	0.011	0.015
Cement mortar surfaces	0.011	0.015
Cement rubble surfaces	0.017	0.030
Clay drainage tile	0.011	0.017
Concrete, precast	0.011	0.015
Copper	0.009	0.013
Corrugated metal (CMP)	0.020	0.024
Ductile iron (Cement mortar lined)	0.011	0.013
Glass	0.009	0.013
High-density polyethylene (HDPE)	0.009	0.011
Polyvinyl Chloride (PVC)	0.009	0.011
Steel, commercial	0.010	0.012
Steel, riveted	0.017	0.020
Vitrified Sewer pipe	0.010	0.017
Wrought iron	0.012	0.017

In this study Darcy-Weisbach and Hazen-Williams friction loss formula will be used.

2.3.2 Minor Losses (Form Resistance)

Along the flow direction, the presence of any fittings, reducers, enlargers, valves, elbows, or bends, leads to form-resistance (minor) losses. These kinds of loss can also be caused by an uneven interior pipe surface due to poor workmanship. The form loss occurs at the intersection of many pipes, or at the junction between a service connection and the junction of the pipeline. Together, these losses constitute a sizeable portion of the total head loss. Such losses are also significant in water supply networks.

They are, however, of little concern in the case of water transmission lines like pumping or gravity mains, which have no take-offs but are long pipelines. Form loss is mathematically expressed as:

$$h_m = \frac{\Delta P}{\rho g} = k_f \frac{V^2}{2g} \tag{2.22}$$

or the velocity can be expressed based on discharge equation as:

$$h_m = k_f \frac{8Q^2}{\pi^2 g D^4} \tag{2.23}$$

where the minor-loss coefficient is represented by k_f, which may be taken as 1.8 for service connections. Although k_f is dimensionless, the literature does not correlate it with the Reynolds number and roughness ratio but only with the raw pipe sizes. The minor loss coefficient for different conditions (bends, elbows, valves, gradual contraction etc.) can be found in the literature such as Swamee (1990) and Swamee et al. (2005).

2.3.3 Total Form Loss

The aggregate form loss coefficient k_f is derived from the sum of the various loss coefficients $k_{f1}, k_{f2}, k_{f3}, \dots \dots, k_{fn}$ in a pipeline:

$$k_f = k_{f1} + k_{f2} + k_{f3} + \cdots + k_{fn} \tag{2.24}$$

The total energy loss is therefore, is the summation of minor and major losses. Based on the reviewed literature one can simply denote total energy loss as:

$$h_L = (k_f + \frac{fL}{D}) \frac{V^2}{2g} \tag{2.25}$$

Where, Eq. (2.25) can be rewritten in terms of discharge as:

$$h_L = \left(k_f + \frac{fL}{D} \right) \frac{8Q^2}{\pi^2 g D^4} \tag{2.26}$$

2.4 Types of Pressurized Pipe Flow Problems

Three problems are typically encountered when utilizing the Moody chart in the design and analysis of piping systems. These problems concern:

1. How to determine the pressure drop when the diameter and length of the pipe are provided for a specific velocity (flow rate) (Type 1).
2. How to determine the flow rate when the diameter and length of the pipe are given for a particular pressure drop (Type 2).
3. How to determine the diameter of the pipe when the flow rate and pipe length are provided for a particular pressure drop (Type 3).

Problems (1) and (2) are analysis problems, while problem (3) is design type.

2.4.1 The Problem of the Nodal Head

The quantities known in the problems of nodal head are $Q, h_{L,}$ L, D, ε, v, and k_f. Through Eqs. (2.3) and (2.23), one can compute the nodal head pressure at section 2 as:

$$\frac{P_2}{\gamma} = \frac{P_1}{\gamma} + z_1 - z_2 - \left(k_f + \frac{fL}{D} \right) \frac{8Q^2}{\pi^2 g D^4} \qquad (2.27)$$

under the assumption that, the pipe cross section is constant, therefore the velocities at section 1 and 2 are same.

2.4.2 The Discharge Problem

The occurrence of form losses can be ignored in the case of long pipelines. Here, this allows the known quantities L, D, h_f, ε and v. According to Swamee et. al. (2008) the turbulent flow in such a pipeline is calculated as:

$$Q = -0.965 D^2 \sqrt{gDh_f/L} \ln\left(\frac{\varepsilon}{3.7D} + \frac{1.78v}{D\sqrt{gDh_f/L}} \right) \qquad (2.28)$$

Equation (2.28) is exact. The Hagen–Poiseuille equation compute the discharge for a laminar flow as:

$$Q = \frac{8fD^4h_f}{128vL} \tag{2.29}$$

The following equation was provided by Swamee et.al (2008) to calculate pipe discharge. The equation is valid for turbulent, transition, and laminar flows.

$$Q = D^2\sqrt{gDh_f/L}\left\{\left(\frac{128v}{\pi D\sqrt{gDh_f/L}}\right)^4 + 1.153\left[\left(\frac{415v}{D\sqrt{gDh_f/L}}\right)^8 - \ln\left(\frac{\varepsilon}{3.7D} + \frac{1.775v}{D\sqrt{gDh_f/L}}\right)\right]^{-4}\right\}^{-0.25} \tag{2.30}$$

Equation (2.30) is nearly precise in the equation as a maximum error is tested to be equal to maximum 0.1%.

2.4.3 The Diameter Problem

In this problem, these quantities are known as Q, L, h_f, ε, and v. Swamee et.al (2008) proposes the following solution for calculating the pipe diameter for turbulent flows in a long gravity main:

$$D = 0.66\left[\varepsilon^{1.25}\left(\frac{LQ^2}{gh_f}\right)^{4.75} + vQ^{9.4}\left(\frac{L}{gh_f}\right)^{5.2}\right]^{0.04} \tag{2.31}$$

The errors associated with Eq. (2.31) are typically less than 1.5%. On the other hand, the highest error close to the range of the transition, however, that is approximately 3%. The Hagen–Poiseuille equation calculates the diameter for a laminar flow as:

$$D = \left(\frac{128vQL}{\pi gh_f}\right)^{0.25} \tag{2.32}$$

The following equation was provided by Swamee et.al (2008) to calculate the pipe diameter and is valid for turbulent, transition, and laminar flows:

$$D = 0.66\left[\left(214.75\frac{vLQ}{gh_f}\right)^{6.25} + \varepsilon^{1.25}\left(\frac{LQ^2}{gh_f}\right)^{4.75} + vQ^{9.4}\left(\frac{L}{gh_f}\right)^{5.2}\right]^{0.04} \qquad (2.33)$$

The value provided for D by Equation (2.33) is accurate within the range of 2.75%. However, the error increases to about 4% in the transition range.

2.5 Hydraulic Analysis

Water distribution network analysis includes the determination of the following:

- Pipe discharge for all pipes in the network system.
- Head loss (major loss and minor loss).
- Pressure head at all nodes.

A conceptual model of a water distribution network can be presented as an input-output system as depicted in Figure 2.3 (Ulanicka et al., 1998).

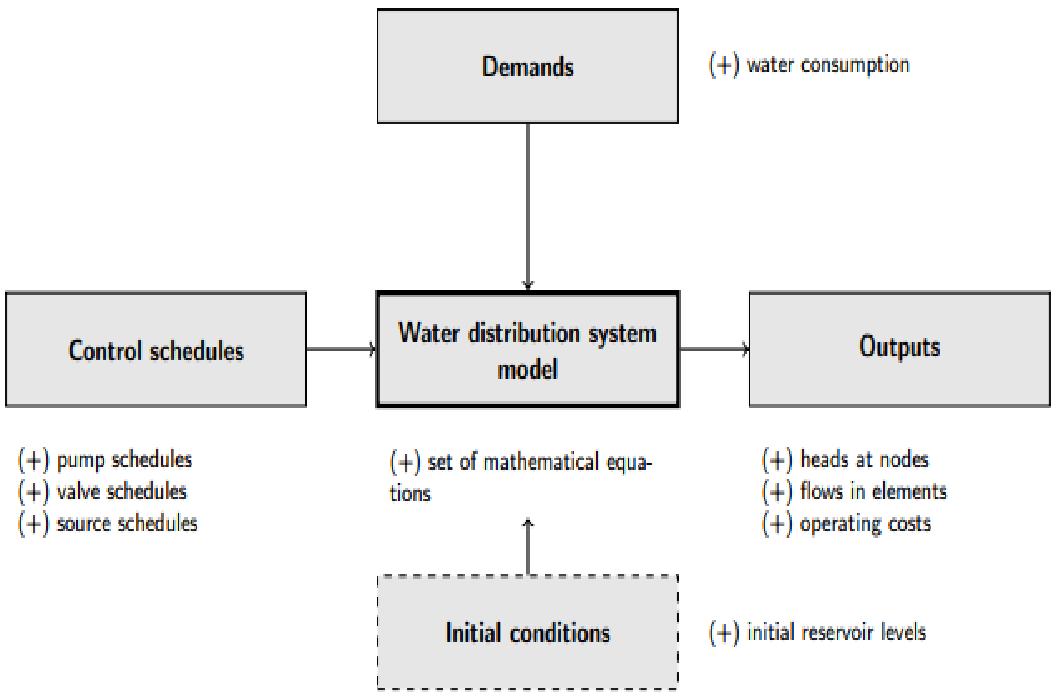

Figure 2.3: A Conceptual Model of Water Distribution System (Ulanicka et al., 1998)

A mathematical model of a WDS can be determined by: (i) its topology, (ii) two conservation laws, namely the mass balance (flow continuity) at nodes and the energy conservation (head loss continuity) around hydraulic loops and the paths, and (iii) equations of its components (Brdys & Ulanicki, 1996).

2.5.1 Water Transmission Lines Analysis

Long pipelines with not withdrawals are known as water transmission lines. Gravity-transported water is known as a gravity main. Any analysis of a gravity main necessitates the calculation of the pipeline discharge.

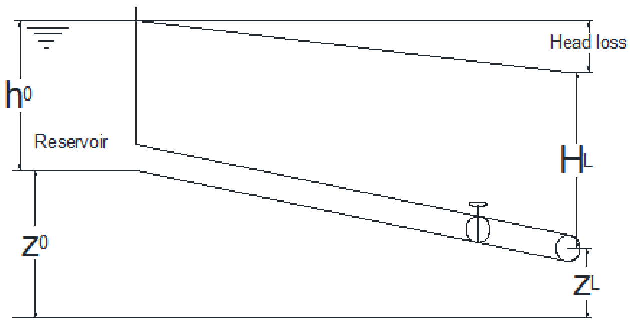

Figure 2.4: A Gravity Transition Main

the discharge can be calculated using Eq. (2.28) as:

$$Q = -965D^2 \left[\frac{gD(h_0 + z_0 - z_L)}{L}\right]^{0.5} \ln\left\{\frac{\varepsilon}{3.7D} + \frac{1.78v}{D}\left[\frac{L}{gD(h_0 + z_0 - z_L)}\right]^{0.5}\right\} \quad (2.34)$$

Pumping water from z_0 to z_L results in a pipeline known as a pumping main (Fig. 2.5). Analysing a pumping main requires a given discharge Q, from which the pumping head H_p is calculated. To do this, Eqs. (2.35) is given as:

$$H_p = H_L + z_L - z_0 + (k_f + \frac{f\text{L}}{D})\frac{8Q^2}{\pi^2 g D^4} \quad (2.35)$$

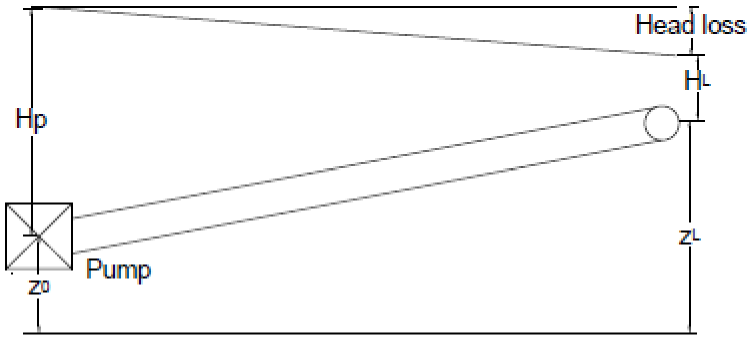

Figure 2.5: A Pumping Transition Main

Where H_L = the terminal head (as $x = L$ for the head). If the minor loss is discounted that experienced in a long pumping main, Eq. (2.35) is then reduced to:

$$H_p = H_L + z_L - z_0 + \frac{8fLQ^2}{\pi^2 gD^5} \tag{2.36}$$

2.5.2 Calculation Mathematics of Pumping Water

In any pumping system, the role of the pump is to provide sufficient pressure to overcome the operating pressure of the system to move fluid at a required flow rate. The operating pressure of the system is a function of the flow through the system and the arrangement of the system in terms of the pipe length, fittings, pipe size, the change in liquid elevation, pressure on the liquid surface, etc. (F. M. White. 2011).

Basic output parameters, assuming steady flow, the pump basically increases the Bernoulli head of the flow between location 1 and location 2, calculating by Appling this equation:

$$H = \left(\frac{P}{\rho g} + \frac{V^2}{2\text{g}} + \text{z}\right)_2 - \left(\frac{P}{\rho g} + \frac{V^2}{2\text{g}} + \text{z}\right)_1 = h_s - h_f \tag{2.37}$$

where h_s is the pump head supplied and h_f the losses. The net head H is a primary output parameter for any turbo machine. Usually V_2 and V_1 are about the same, $z_2 -$

z_1 is no more than a meter or so, and the net pump head is essentially equal to the change in pressure head:

$$H \approx \frac{P_2 - P_1}{\rho g} = \frac{\Delta P}{\rho g} \tag{2.38}$$

The power delivered to the fluid simply equals the specific weight times the discharge times the net head change:

$$P_w = \rho g Q H \tag{2.39}$$

This is traditionally called the water horsepower. The power required to drive the pump is the brake horsepower:

$$bhP = \omega T \tag{2.40}$$

where ω is the shaft angular velocity and T the shaft torque. If there were no losses, P_w and brake horsepower would be equal, but of course P_w is actually less, and the efficiency η of the pump is defined as:

$$\eta = \frac{P_w}{bhP} = \frac{\rho g Q H}{\omega T} \tag{2.41}$$

2.5.3 Analyzing Complex Pipe Network Methods

Simple distribution system analysis procedure delineate the level of demand required by each node:

1. Delineate the level of demand required by each node.
2. Approximate the level of discharge in the pipes.
3. Guess the probable pipe diameters.
4. Calculate the pipes' head loss.
5. At the end of the pipe, determine the residual pressure.
6. Compare the minimum and maximum desired pressures with the terminal pressure.
7. Repeat steps two to six until the required condition has been satisfied.

2.5.3.1 Hardy-Cross Method

The evaluation of a pipe network system is integral if we are to properly analyse a pipe network. In a branched pipe network, Pipe discharges are unique and could be obtained through the application of discharge continuity equations to all nodes. For looped pipe networks, however, simply applying discharge equations is insufficient to calculate the pipe discharges due to the large number of pipes. Consequently, looped networks are analyzed using supplementary equations based on the fact that when navigating a loop, the net head loss as one approached the starting node is zero. The loop equations for a looped network are nonlinear in discharge.

This method finds its basis in the following basic head loss and continuity of flow equations that must be satisfied:

1. The sum of inflow rates and outflow rates at a junction have to be identical:

   $$\sum Q_i = q_j \qquad \text{for all nodes} \quad j = 1,2,3,4,5, \dots \dots \dots, j_L$$

   That the discharge in pipe *i* connecting at junction *j* is represented by Q_i and the nodal withdrawal at node *j* is represented by q_j.

2. The summation of the head loss around each loop must be algebraically equal to zero.

   $$\sum h_f = \sum_{loop\ k} k_i\, Q_i |\, Q_i\,| = 0 \quad \text{for all loops} \ \ k$$

   $$= 1,2,3, \dots, k_L \tag{2.42}$$

where

$$k_i = \frac{8fL}{\pi^2 g D_i^5} \tag{2.43}$$

where *i* is the number of the pipe link is used in loop *k* above.

Generally, if one accepts that the initial assumed pipe discharges satisfy the nodal continuity equation, then it is impossible to satisfy Eq. (2.42). As such, we need to modify these discharges such that Eq. (2.42) is relatively closer to zero. To determine these modified pipe discharges, we apply the correction ΔQ_k to the initially assumed pipe flows. Thus,

$$\Delta Q_k = -\frac{\sum h_f}{n \sum \frac{h_f}{Q_i}} = \frac{\sum k_i Q_i |Q_i|^{n-1}}{n \sum k_i |Q_i|^{n-1}} \tag{2.44}$$

In Darcy-weisbach n=2, knowing ΔQ_k the corrections are performed to obtain the new discharge such as:

$$Q_{i\,new} = Q_{i\,old} + \Delta Q_k \qquad for\ all\ loops\ k$$

This similar procedure is repeated in all loops of the network until the discharge corrections will be equal to zero or relatively very small in loop.

2.5.3.2 Newton-Raphson Method

This method is also used to perform the pipe analysis. In contrast to the Hardy Cross method, this method analyses the network in its entirety. The Newton–Raphson method provides an authoritative numerical way to solve systems of nonlinear equations. Thus, numerically, the summation head losses inside a loop in Newton Raphson, the summation has a high accuracy and approximately zero compared to Hardy-Cross method. Also, the solution in the Newton Raphson method can be obtained using a lesser number of iterations (faster) compared to the Hardy-Cross method (Abdulhamid Saad et.al. 2017). Suppose that there are three non-linear equations, first Eq. $F_1(Q_1, Q_2, Q_3) = 0$, second Eq. $F_2(Q_1, Q_2, Q_3) = 0$, as well as third Eq. $F_3(Q_1, Q_2, Q_3) = 0$ require to be solved and to obtain the three discharge values for Q_1, Q_2 and Q_3, take (Q_1, Q_2, Q_3) to be a starting solution, and consider the solution of the set of equations to be $(Q_1 + \Delta Q_1, Q_2 + \Delta Q_2, Q_3 + \Delta Q_3)$, then

$$F_1(Q_1 + \Delta Q_1, Q_2 + \Delta Q_2, Q_3 + \Delta Q_3) = 0$$

$$F_2(Q_1 + \Delta Q_1, Q_2 + \Delta Q_2, Q_3 + \Delta Q_3) = 0 \qquad 2.45$$

$$F_3(Q_1 + \Delta Q_1, Q_2 + \Delta Q_2, Q_3 + \Delta Q_3) = 0$$

Developing the above equation using Taylor's series expansion,

$$F_1[\partial F_1/\partial Q_1]\Delta Q_1 + [\partial F_1/\partial Q_2]\Delta Q_2 + [\partial F_1/\partial Q_3]\Delta Q_3 = 0$$

$$F_2[\partial F_2/\partial Q_1]\Delta Q_1 + [\partial F_2/\partial Q_2]\Delta Q_2 + [\partial F_2/\partial Q_3]\Delta Q_3 = 0 \qquad 2.46$$

$$F_3[\partial F_3/\partial Q_1]\Delta Q_1 + [\partial F_3/\partial Q_2]\Delta Q_2 + [\partial F_3/\partial Q_3]\Delta Q_3 = 0$$

In matrix from, it is written as;

$$\begin{bmatrix} \partial F_1/\partial Q_1 & \partial F_1/\partial Q_2 & \partial F_1/\partial Q_3 \\ \partial F_2/\partial Q_1 & \partial F_2/\partial Q_2 & \partial F_2/\partial Q_3 \\ \partial F_3/\partial Q_1 & \partial F_3/\partial Q_2 & \partial F_3/\partial Q_3 \end{bmatrix} \begin{bmatrix} \Delta Q_1 \\ \Delta Q_2 \\ \Delta Q_3 \end{bmatrix} = -\begin{bmatrix} F_1 \\ F_2 \\ F_3 \end{bmatrix} \qquad 2.47$$

Solving eq. (2.78),

$$\begin{bmatrix} \Delta Q_1 \\ \Delta Q_2 \\ \Delta Q_3 \end{bmatrix} = -\begin{bmatrix} \partial F_1/\partial Q_1 & \partial F_1/\partial Q_2 & \partial F_1/\partial Q_3 \\ \partial F_2/\partial Q_1 & \partial F_2/\partial Q_2 & \partial F_2/\partial Q_3 \\ \partial F_3/\partial Q_1 & \partial F_3/\partial Q_2 & \partial F_3/\partial Q_3 \end{bmatrix}^{-1} \begin{bmatrix} F_1 \\ F_2 \\ F_3 \end{bmatrix} \qquad 2.48$$

Based on the corrections, we can rewrite the discharges as

$$\begin{bmatrix} Q_1 \\ Q_2 \\ Q_2 \end{bmatrix} = \begin{bmatrix} Q_1 \\ Q_2 \\ Q_2 \end{bmatrix} + \begin{bmatrix} \Delta Q_1 \\ \Delta Q_2 \\ \Delta Q_3 \end{bmatrix} \qquad 2.49$$

It is evident that, repetitively obtaining the inverse of the matrix for large networks is a time-consuming exercise. As such, the initial inverse matrix is conserved and used to obtain the corrections at least three times.

The following steps summarize the entire process of looped network analysis using the Newton–Raphson method:

i. Assign numbers to all loops, pipe links, and nodes.

ii. Use the following formulations to write the equations for nodal discharge:

$$F_j = \sum_{j=1}^{jn} Q_{jn} - q_j \ = 0 \quad \text{for all junctions} - 1 \tag{2.50}$$

Where the discharge in *n*th pipe at node *j*, the total number of pipes at node *j*, and nodal withdrawal are denoted by Q_{jn}, jn, and q_j respectively.

iii. Write the equations for loop head-loss as.

$$F_k = \sum_{k=1}^{kn} k_n Q_n |Q_n| \ = 0 \quad \text{for all the loops } (n = 1, k_n) \tag{2.51}$$

where k_n represents the total number of pipes in the *k*th loop.

iv. Take the initial pipe discharges $Q_1, Q_2, Q_3, \ldots$ ...to satisfy continuity

Equations.

v. Compute friction factors f_i in all pipe links and find the corresponding k_i through Eq (2.74).

vi. Calculate the values of the partial derivatives $\partial F_k / \partial Q_i$ and functions F_n, using the initial pipe discharges Q_i and K_i.

vii. Find ΔQ_i. The generated equations are of the form *Ax* = *b* and thus, can be used to solve for ΔQ_i.

viii. The values found for ΔQ_i are used to modify the pipe discharges.

ix. The process is repeated until the values for ΔQ_i are at their minimum.

2.6 Summary of the Chapter

In this chapter, the hydraulic variables that are effective on calculating and solving flow rate, head loss, effective length, and pipe diameters of the pipe flow in the pipe network system are analysed and discussed. All the variables and constant parameters which are effective in calculating pressure head, velocity direction, and discharge in the system are discussed. Generally, there are three kinds of problems related to the determination of (a) pressure head, (b) the discharge through a pipe link, and (c) the diameter of pipe link – (a) and (b) are solved by deriving the equations. On the other hand, problem (c) is obtained by the application a type of synthesis/design. Most of the equations which are needed to analyse water distribution networks are also theoretically explained. The next chapter will describe and explain the methodology used to obtain an optimal design by getting the least possible cost of pipes in the water distribution network system. All the steps for this aim will be applied based on the two developed software computer programs in this study.

Chapter 3

METHODOLOGY

3.1 Overview

Most of the fundamental information regarding the analysis of water distribution network systems (WDN)s is theoretically explained in the previous chapter. In a manner similar to recent studies, a computer program solver is used to perform the analysis on the water distribution network. In this study, the software program EPANET is used to achieve the same purpose based on driving the equations of pipe flow which is described in previous chapter. The network's information data is exported in EPANET's INP file format. In addition, optimum design of the water distribution system is achieved through genetic algorithm in which minimum cost is worked out while the minimum required pipe diameters are as well considered. For the genetic algorithm analysis MATLAB code is used. To this end, these two programs combined in one toolkit, known as the EPANET-MATLAB TOOLKIT. The WDNs at the result is expected to satisfy the hydraulic principles of fluid mechanics. The methodology is explained briefly in the flowchart given in Figure 3.1.

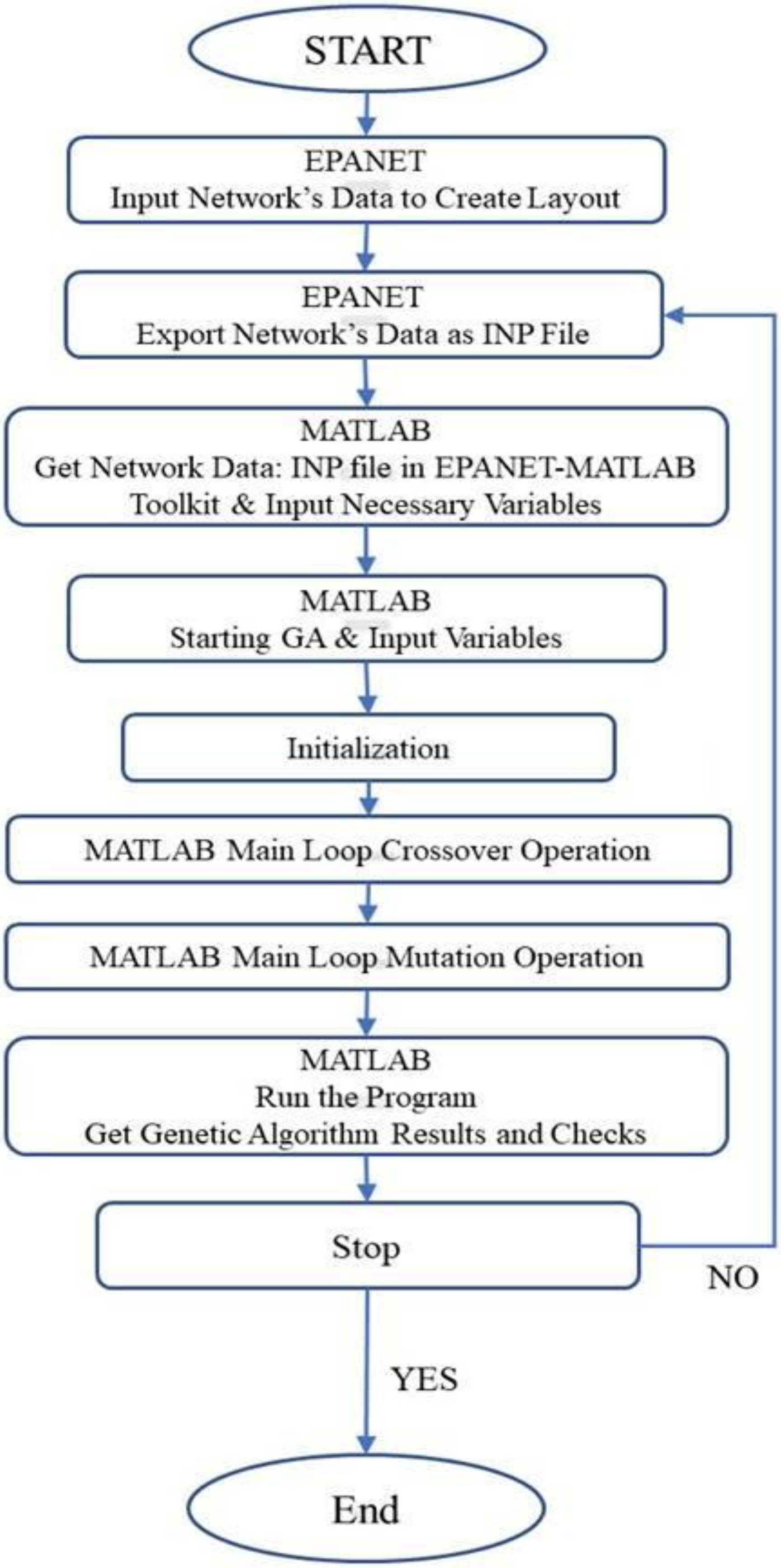

Figure 3.1: Flowchart of the Analysis Performed

3.2 EPANET

3.2.1 Introduction of EPANET

EPANET is used for the analysis of water distribution networks. The EPANET computer model has two components: (1) the computer program itself, and (2) the input data file. The data file delineates the characteristics of the nodes (pipe ends) and pipes, as well as the control elements (valves and pumps) in the pipe network. On the other hand, the computer program is responsible for solving the linear mass and the non-linear energy equations for pipe flow rates and nodal pressures.

Input Data File: The EPANET input data file, describes the physical characteristics of the pipes and nodes, and the connections between the pipes in a pipe network system. It is automatically created in the required format using MIKE NET and allows the user to provide a graphic layout of the whole network. The dialog boxes used to input the values for the pipe network parameters – including the pipe roughness coefficient, the minor loss coefficient, the interior diameter of the pipe, and the length of the pipe– are relatively simple. Every individual pipe has a defined positive flow direction and two nodes. The nodal parameters include the hydraulic grade line, elevation, and water demand or supply.

EPANET Computer Program: The EPANET computer program was development by the American Environmental Protection Agency. The program calculates both the flow rates in pipes and the pressure head at nodes. Calculating the flow rates requires a number of iterations due to the non-linear nature of the energy and the mass equations, regardless, the total number of iterations required to determine the level of accuracy specified by the user and the system of network equations.

For a flow rate solution to be considered satisfactory, it must satisfy a set of user-imposed requirements, including the law of conservation of the energy and the mass in the water distribution system with a particular accuracy level. Calculating the HGL does not require any iterations as the network equations are linear. Once the analysis of the flow rate has been completed, another option which is the computations for determining the quality of the water can be performed.

EPANET is useful for the assessment of alternative strategies for maintaining the quality of water throughout a distribution system (Rossman, 2000) including:

- Changing the utilization of sources in the systems that are using multiple sources.
- Changing the schedules for tank (emptying, filling, and pumping).
- Using satellite treatment methods (such as re-chlorination) at storage tanks.
- Cleaning and replacing specific pipes.

A windows-based program, EPANET contains an integrated environment within which the input data can be edited, the water quality and the hydraulic simulations can be carried out, and the results can be displayed in different formats, such as contour plots, time series graphs, data tables, and color-coded network maps.

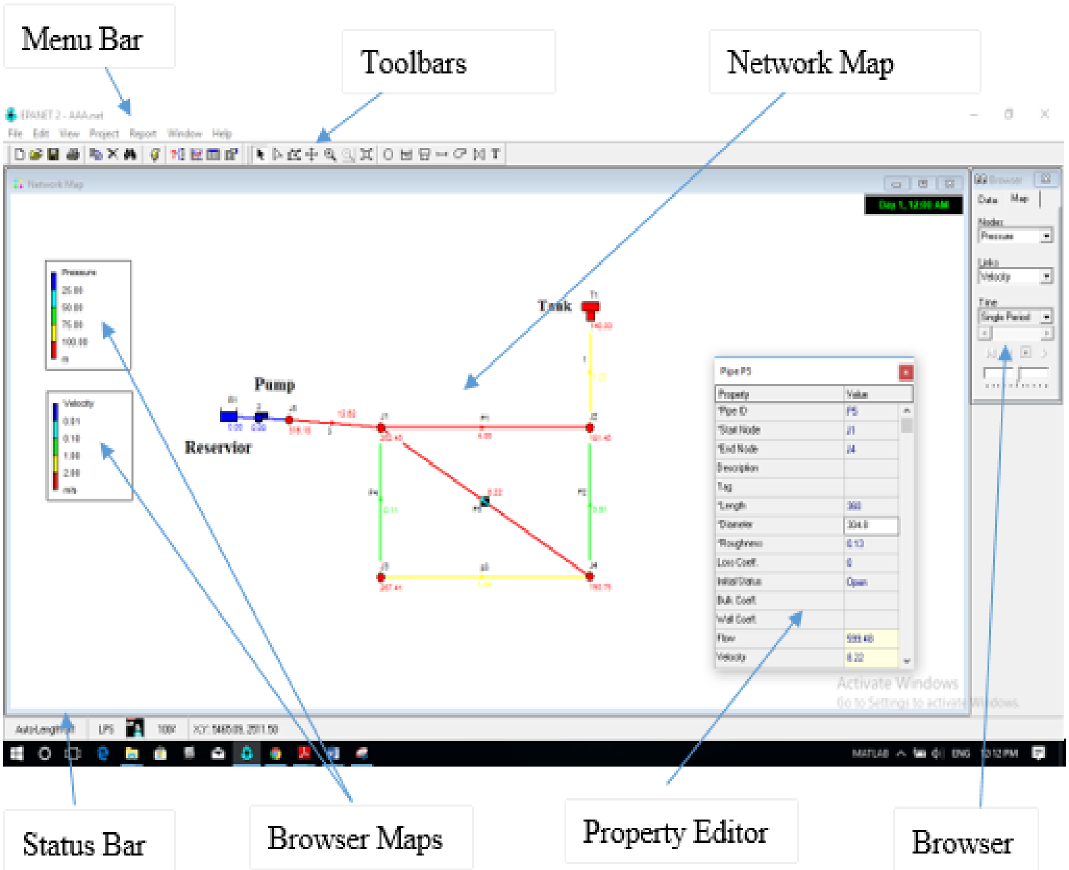

Figure 3.2: EPANET Network Space

3.2.2 Hydraulic Modeling Capabilities

Any effective model for water quality requires a pre-existing accurate and comprehensive hydraulic model. The impressive hydraulic analysis engine contained in EPANET is capable of:

- Analyzing networks of unlimited sizes.
- Using either the Darcy-Weisbach, Hazen-Williams, or Chezy-Manning formulas in calculating the friction head loss.
- Permitting lesser degrees of head loss for bends, fittings, etc.
- Modelling either variable or constant speed pumps.
- Calculating the amount of energy needed and cost of pumping.

- Modelling a variety of valves, including flow control, pressure regulators, check and shutoff valves.
- Allowing various shapes of storage tanks (variations in height and/or diameter).
- Considering multiple nodal demand categories, each with a unique time variation pattern.
- Modelling the pressure-dependent flow projected from emitters (sprinkler heads).
- Basing the operation of the entire system on both simple tank level or timer controls and on complex rule-based controls.

3.2.3 Water Quality Modelling Capabilities

To supplement hydraulic modelling, EPANET also is capable of modelling water quality to derive the equations analyzed in Chapter 2. Its capabilities in this regard include:

- Modelling the movement through the network of a non-reactive tracer material overtime.
- Modelling the movement and outcome of a reactive material over its evolution (e.g., a disinfection by-product) or decays (e.g., chlorine residual).
- Modelling the age of the water in the entire network.
- Tracking the percentage of the flow from a given node that reaches all the other nodes overtime.
- Modelling the reactions both at the pipe wall and in bulk flow.
- Modelling reactions in the bulk flow using n-th order kinetics.
- Modelling reactions at the pipe wall using zero or first order kinetics.
- Accounting for mass transfer limitations when developing the model for pipe wall reactions.

- ➢ Allowing the progression of growth or decay reactions up to a limiting concentration.
- ➢ Employing modifiable global reaction rate coefficients for the individual pipes.
- ➢ Permitting the correlation of wall reaction rate coefficients and pipe roughness.
- ➢ Allowing mass inputs or time-varying concentration to occur at any location in the network.
- ➢ Modelling storage tanks as complete mix, plug flow, or two compartment reactors.

Through a combination of these properties, EPANET is able to study water quality phenomena such as:

- ➢ The combination of water from more than one source.
- ➢ The age of the water in a system.
- ➢ The disappearance of chlorine residuals.
- ➢ The growth of the by-products of disinfection.
- ➢ Tracking events that propagate contaminants.

3.2.4 Steps in Using EPANET

The following steps are usually used to model water distribution systems using EPANET:

1. Import a network description from a text file or draw a network representation of the distribution system.
2. Modify the properties of system objects as required.
3. Describe the operation of the system.
4. Choose from a set of analyses from menu bar > project > defaults.
5. Conduct a hydraulic/water quality analysis.
6. Display the analysis result from toolbars>table shown in Figure 3.2.

3.2.5 Hydraulic Simulation Model

The hydraulic simulation model found in EPANET calculates the link flows and junction heads for a pre-determined set of water demands, and reservoir levels overtime according to the principle equations for pipe flow discharge in the previous chapter. Junction demands and reservoir levels are updated for each successive step based on their suggested time patterns. Current flow solutions are used to update the tank levels. The conservation of flow equation for each junction and the head loss relationship for each of the networks have to be solved simultaneously to find the solutions the heads and flows at specific points in time. Known as “hydraulically balancing” the network, the entire process requires that an iterative technique is used to solve all of the relevant non-linear equations. The user can determine the hydraulic time step used for the extended period simulation (EPS); this value is typically one hour (Rossman, 2000). The time step will automatically be shortened under either of the following conditions:

- ➢ When the subsequent output reporting period begins.
- ➢ The succeeding time pattern period begins.
- ➢ A tank is emptied or filled up.
- ➢ The user activates either a rule-based or simple control.

3.2.6 Forming INP File in EPANET

Beginning the design of a water distribution network with genetic algorithm (GA) requires that the user organizes the input file in the required format. This ‘INP file’ is the EPANET text input file and contains the properties of the water distribution network, including:

- Flow demands and junction topographical elevations.
- Reservoir elevations.

- Pipe diameters, lengths, roughness, and topographical properties.
- Options for hydraulic analysis.

The EPANET command menu is used to create the INP file, which can also be created from the file>export>network pull down menu of EPANET, see Figure 3.3.

After that, the INP file should be saved in the EPANET-MATLAB Toolkit to begin the design process of the water distribution network system with the genetic algorithm. In addition, there is an EPANET program as coded to connect with the MATLAB code (Open Water Analytics, 2018).

3.2.7 Development on an EPANET File

The layout of the distribution network is drawn based on the existing route pattern in an AUTOCAD file. This network is now converted into an EPANET file by using various software, such as "EPACAD". EPACAD converts the AUTOCAD file to EPANET file by considering intersections of the lines as nodes and lines as links. It can be used especially when a huge part of the city is taken from the master plan to design the water distribution network. This procedure is easy to work and has a lot of benefits. Also, other beneficial information about EPACAD can be reached in the EPACAD manual.

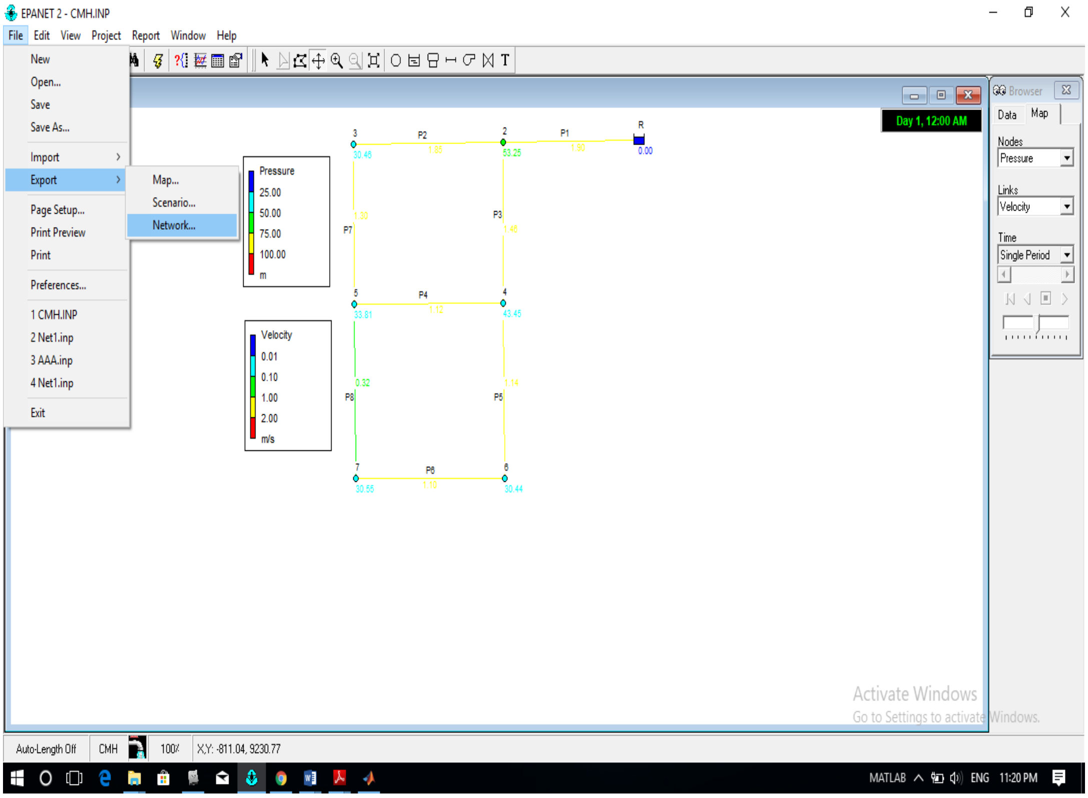

Figure 3.3: Export of Network Pipe

3.3 Genetic Algorithm (GA)

Drawing inspiration from natural evolution, John Holland was the first developed the genetic algorithm (GA) in the 1970s. The GA is a search method that starts with a population of solutions (chromosomes) generated at random. The algorithm generates the best solution through the evolution of the chromosomes in successive iterations. New chromosomes are created in every generation through the application of genetic operators, such as mutation, crossover, and selection. The steps contained in the GA include:

3.3.1 Initialization

The purpose of this step is to determine some fundamental parameters of applied GA, including: the maximum number of iterations (MaxIt), population size (Npop), mutation probability (pm), and crossover probability (Pc).

3.3.2 Encoding

This step is especially important in the development of the GA as it appropriately defined the solutions. Here, each chromosome corresponds to a distinct matrix with a single row and *k* columns; the number of pipes in the network equals the length of the chromosome. Each column (gene) represents the type diameter of the pipe. Fig. 3.4 illustrates a chromosome for one network instance.

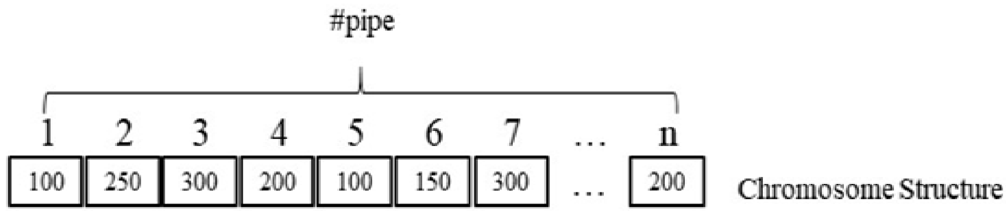

Figure 3.4: Chromosome Encoding

3.3.3 Initial Population

The initial population consists of a set of chromosomes generated at random. Each chromosome consists of diameters types determined for different pipes.

3.3.4 Fitness Evaluation

Each chromosome's fitness value is calculated by multiplying the length of each pipe by the price of the determined pipe type. If a constraint regarding the pressure head and flow is violated, a penalty is applied. In order to compute the pressure head and other specifications, the diameters of the pipes of the hydraulic network are uploaded to EPANET, and so the hydraulic network system, EPANET_MATLAB toolkit is incorporated into the model. The type, position of the pipes, and their length remain without change and the new parameters of the water distribution network are computed by EPANET and the results are returned and used by the algorithm in MATLAB. This process is continued until the best cost for the network according to the program attain.

3.3.5 Parent Selection

The Roulette wheel procedure is used in this study to increase the chance of parents with better fitness values to be selected. The crossover operator is carried out by the selected parents.

3.3.6 Crossover Operator

Generally, there are three types of crossover application: single point, two points, and multiple point crossover operators. In the single point crossover operator which is used in this study, two parental chromosomes are mated to produce two offspring (child chromosomes). The crossover points at which the individual chromosomes are decomposed into two segments is randomly selected. The genes from the first segment and the second segment of the first and second parent respectively, are used to produce the first offspring. Conversely, the second offspring is produced by switching the roles of the parents – that is, using the first segment and the second segment of the second and first parent respectively. Fig. 3.5 graphically illustrates the crossover operation.

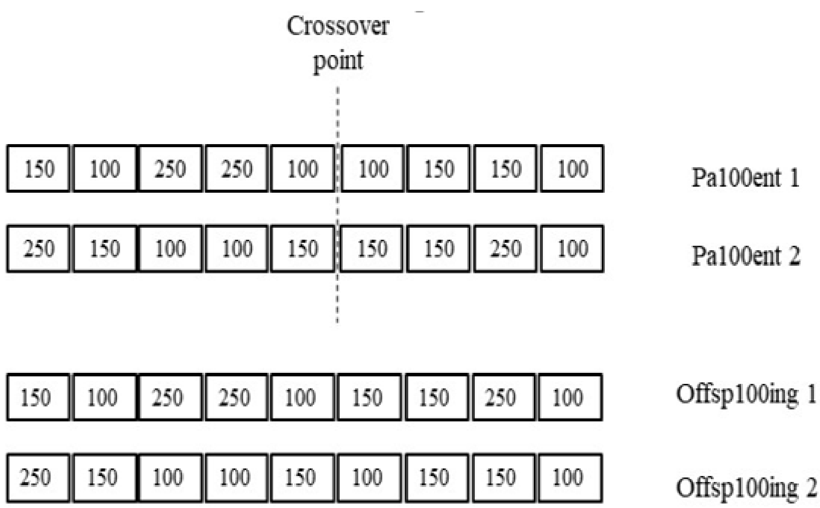

Figure 3.5: An Example of the Crossover Operation

3.3.7 Mutation Operator

In this section, the allele of a gene selected at random from an offspring chromosome is replaced with the index of a pipe diameter also selected at random based on the mutation probability (Pm). In Figure 3.6 the above chromosome represents the parent chromosome that generates one offspring chromosome in mutation operation.

250	150	100	100	150	100	150	150	100

250	150	150	100	150	250	150	100	100

Figure 3.6: An Example of the Mutation Operation

3.3.8 Replacement and Stopping Criteria

In each iteration, the portion of the population with size nPop is selected for the next iteration. The selection is based on fitness values and the chromosomes with better fitness function are in priority.

The algorithm performs the predetermined number of iterations and at the end of the iterations, the best solution is returned. The best solution consists of the least diameters found, velocity in all pipes, flow in all pipes, pressure head in each node and the corresponding cost.

Chapter 4

TESTING, EVALUATION AND DISCUSSION

4.1 Overview

The proposed study that is explained so for in previous chapter had to be tested over a previously solved water distribution network project, thus, the reliability of this study will be confirmed. Therefore, Hanoi water distribution network is analysed, evaluated and discussed as a case.

4.2 Optimization of Hanoi Network

4.2.1 Hanoi Network

Hanoi network, as a real network in Vietnam, was first presented as a case study to obtain the optimum solution by Fujiwara and Khang (1987). Subsequently, numerous researchers (Savic & Walters (1997), Cunha & Sousa (1999), Liong & Atiquazzam (2004) and, Güç (2006) analysed the same project to determine an optimized solution. The Hanoi network is connected to be a moderately sized network.

The network consists of 32 nodes and 34 pipes connected as such that 3 loops are formed and is fed from a single fixed head source ahead by 100m (shown in Figure 4.1). The Hazen-Williams (HW) coefficient (C) 130 for all links are taken constant and the elevation for all nodes are to be zero. The layout of the network includes all the necessary other data shown in Figure 4.1.

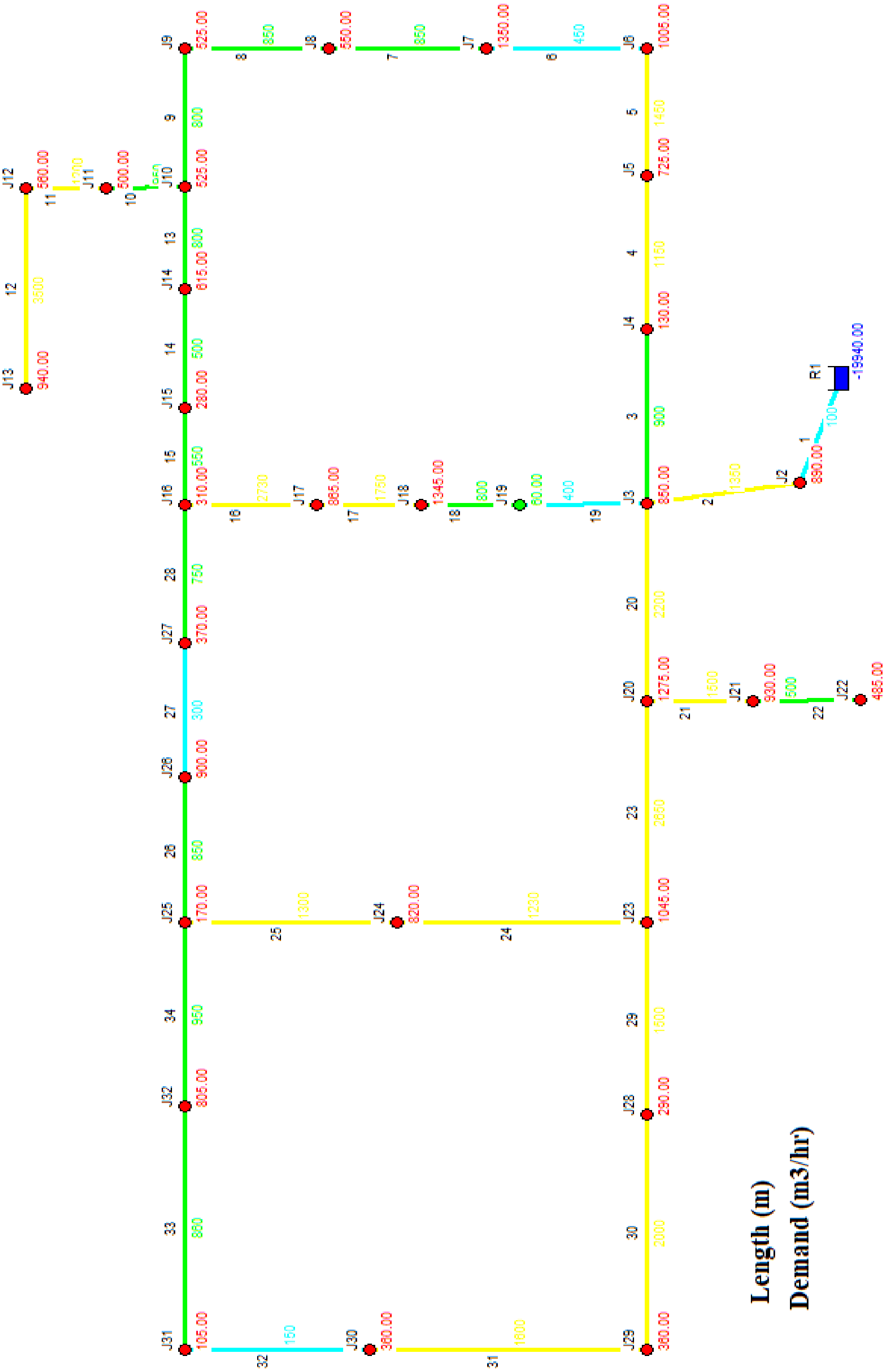

Figure 4.1: Layout of the Hanoi Network

Table 4.1: Hanoi Network, Available Pipe Information

Diameter (Inch)	Diameter (mm)	HW Roughness coefficient	Price (USD/m)
12	304.8	130	45.73
16	406.4	130	70.40
20	508	130	98.38
24	609.6	130	129.33
30	762	130	180.75
40	1016	130	278.28

4.2.2 Solution by Using Genetic Algorithm

As long as the results the EPANET are extracted and the networked is modelled as shown in Figure 4.1, the results are connected into INP file format. The as results are then read by EPANET-MATLAB toolkit and is prepared to be used in MATLAB code (Appendix B) for optimizing the pipe diameters and thus the cost of the project. Later, the input data file for genetic algorithm is prepared and the following input data is given for Hanoi network:

Maximum iteration (MaxIt) : 120

Population size (npop) : 1000

Crossover percentage (pc) : 0.7

Number of off springs (popc) : 2*round(pc*npop/2)

Mutation percentage (pm) : 0.8

Number of mutants (popm) : round(pm*npop)

Beta : 5

The input data screen and the iteration process of MATLAB code is given in Figures 4.2 and 4.3, respectively.

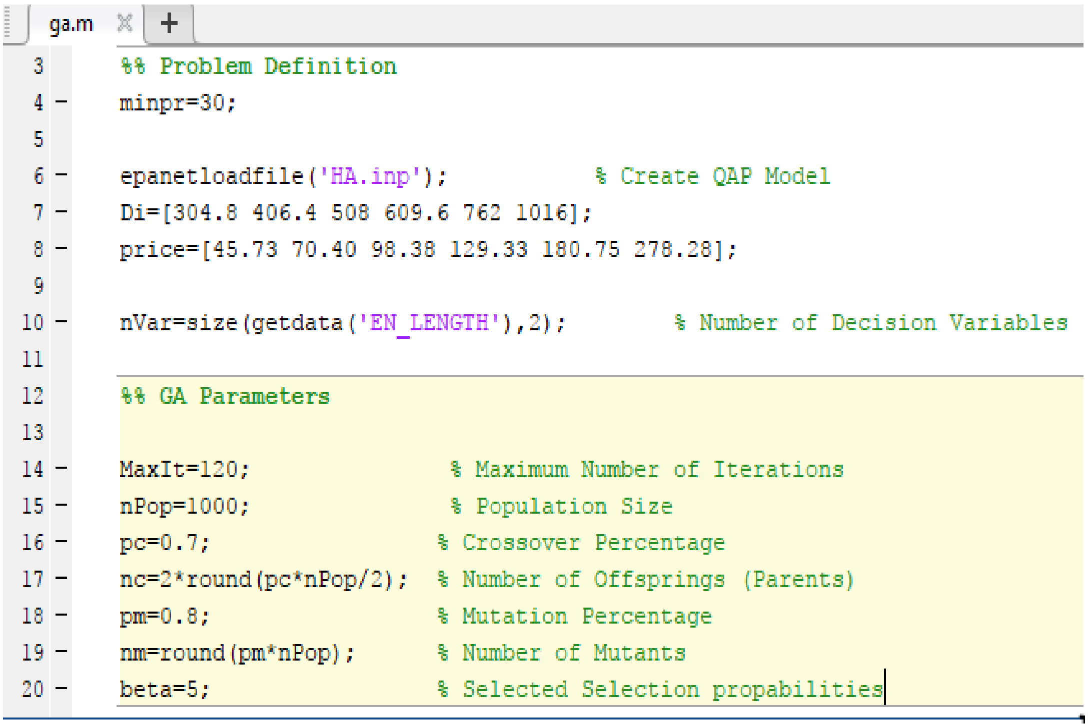

Figure 4.2: Starting Details of Genetic Algorithm

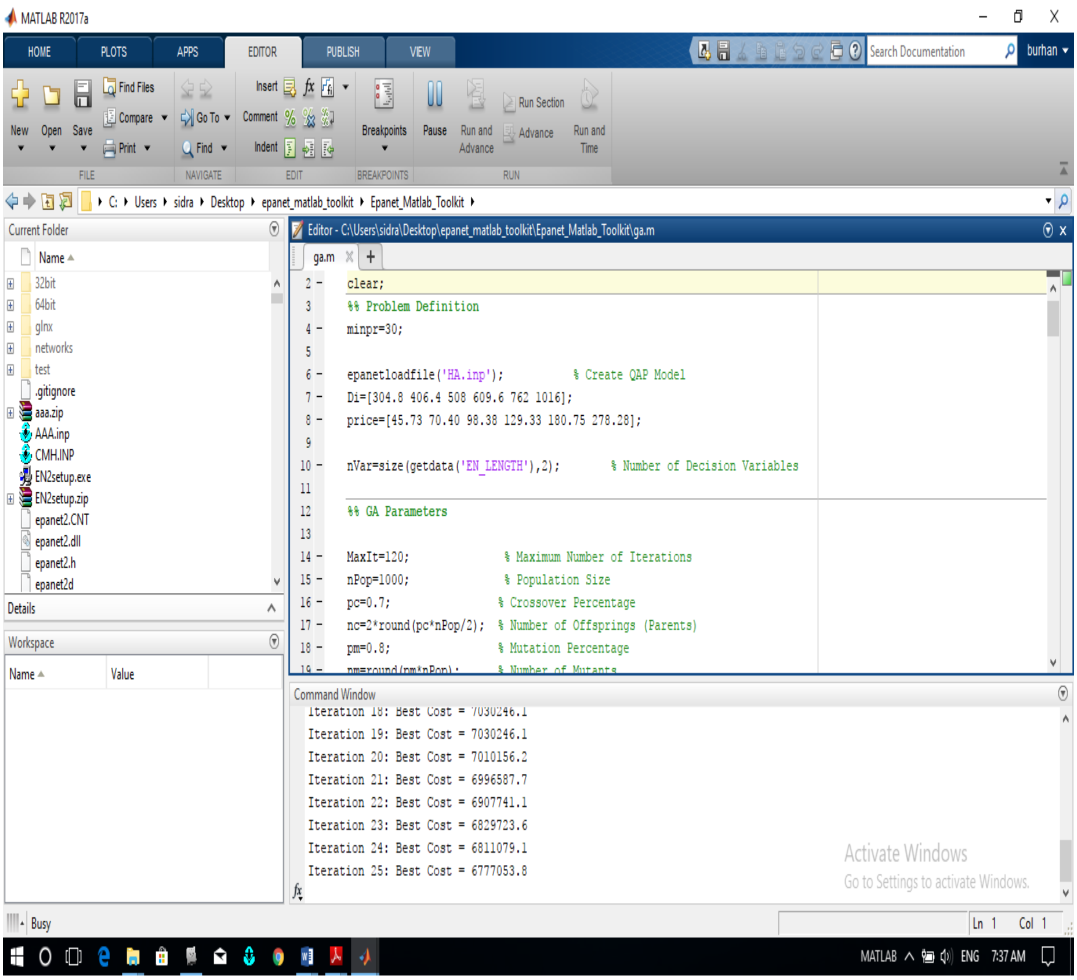

Figure 4.3: Run Process of Hanoi Network to Obtain Best Value

The final optimum results can be seen from the workspace>best solution, which opens the list of the network characteristics desired by user, including best cost, minimum diameters, desired pressure, velocity, and flow in the links. Also, a graph is opened that shows the relationship between the maximum iteration and best cost (Figure 4.4).

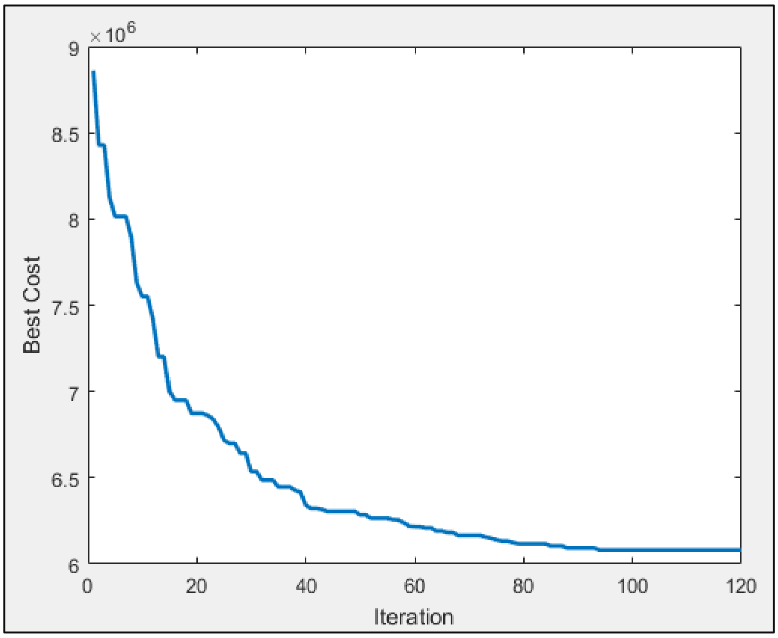

Figure 4.4: Getting Best Cost with Iterations

The final optimal result of diameters is tabulated in Table 4.2, which includes the diameter of all pipes in the Hanoi network. It also includes a comparison of the diameters and optimal costs with those of previous studies.

In the analysis of the Hanoi network, 34 pipes were used that were consisting of six different diameters. Therefore, the optimizing procedure for the EPANET-MATLAB toolkit was operating search space of a size of 6^{34} to select best set of diameters in the analysis. Thus, such iterative solution for above process required a high computer performance with a high quality in CPU and RAM.

Table 4.2: Comparisons for Optimum Pipe Diameters for Hanoi Network

Pipe ID	**Length (m)**	**Studies pipe diameters result (inch)**				
		Savic and Walters (1997)	**Cunha and Sousa (1999)**	**Liong and Atiquazzam (2004)**	**Güç (2006)**	**This study (2018)**
1	100	40	40	40	40	40
2	1350	40	40	40	40	40
3	900	40	40	40	40	40
4	1150	40	40	40	40	40
5	1450	40	40	40	40	40
6	450	40	40	40	40	40
7	850	40	40	40	40	40
8	850	40	40	30	40	40
9	800	40	40	30	40	40
10	950	30	30	30	24	30
11	1200	24	24	30	24	24
12	3500	24	24	24	24	24
13	800	20	20	16	12	20
14	500	16	16	12	12	16
15	550	12	12	12	16	12
16	2730	12	12	24	12	12
17	1750	16	16	30	20	16
18	800	20	20	30	30	24
19	400	20	20	30	20	20
20	2200	40	40	40	40	40
21	1500	20	20	20	20	20
22	500	12	12	12	12	12
23	2650	40	40	30	40	40
24	1230	30	30	30	30	30
25	1300	30	30	24	30	30
26	850	20	20	12	30	20
27	300	12	12	20	20	12
28	750	12	12	24	16	12
29	1500	16	16	16	16	16
30	2000	16	12	16	20	12
31	1600	12	12	12	16	12
32	150	12	16	16	20	16
33	860	16	16	20	16	16
34	950	20	24	24	24	24
Total Cost (Million USD)		6073	6056	6224	6334	6081

Genetic algorithm in network system (GANET) is applied as a methodology in this thesis with some parameters for the Hanoi network in Vietnam to optimally design the water distribution network. An allowable minimum pressure was taken from 30 m for this study and other past studies, as detailed in Table 4.2. This study the optimal cost is $6,081 million, which complies with all pressure constraints. While there are two lower costs in the above Table 4.2, $6,073 and $6,056 for Savic and Walters (1997) and Cunha and Sousa (1999) respectively. However, they don't have any constraint regarding the allowable pressure, which is highlighted by some pressure nodes being below the minimum allowable pressure shown in Table 4.4.

On the other hand, Liong and Atiquazzam (2004) and Güç (2006) concluded that $6,224 and $6,334 million were the least possible cost for the Hanoi network in two different ways, respectively. Although these two cost values more than the optimal cost of this study, they both have convergence pressure at all nodes.

Recently, Vasan and Simonovic (2010) explained and examined the use of the differential evolution of network for pipes (DENET) computer model in the optimal design of water distribution networks by applying an evolutionary optimization technique, EPANET, linked to the hydraulic simulation solver, and differential evolution. Then, they examined the Hanoi network for two different allowable pressures: 30 m and 29.59 m, for which $6,195 and $6,056 million were determined to be the cost (as detailed on Table 4.3). In fact, the optimal cost for 30 m is less than that was obtained by Liong and Atiquazzam (2004) and Güç (2006). But, it is more than the $6,081 which was obtained by this study. In addition, in this study, for a minimum allowable pressure of 29.59 m, it is obtained $6,056 as an optimal cost – similar to Vasan and simonovic (2010). Even as this allowable pressure 29.59 m is very close to 30 m, there is a significant difference of value in the cost of Hanoi network. As a result, it is explained tells that any desired allowable pressure can be obtained at optimal cost by using this present methodology with getting the optimal design of any water distribution network.

Table 4.3: Minimization of Network Cost for Hanoi Network by Decrease Pressure

Pipe ID	**Length (m)**	**Studies pipe diameters result (inch)**			
		DENET (Hmin=29.29 m) (2010)	**DENET (Hmin=30 m) (2010)**	**This study (Hmin=29.59 m) (2018)**	**This study (Hmin=30 m) (2018)**
9	800	40	30	40	40
16	2730	12	16	12	12
17	1750	16	20	16	16
18	800	20	24	20	24
19	400	20	24	20	20
30	2000	12	16	12	12
31	1600	12	12	12	12
32	150	16	12	16	16
33	860	16	16	16	16
34	950	24	20	24	24
Total Cost (Million USD)		**6056**	**6195**	**6056**	**6081**

Finally, the nodal pressures obtained in the present study are compared with the other researcher's results and are presented in Figure 4.5. From nodes 2 to 18, they almost coincide with others but the remaining nodes show different values which may be a cause to achieve optimal cost to the final design.

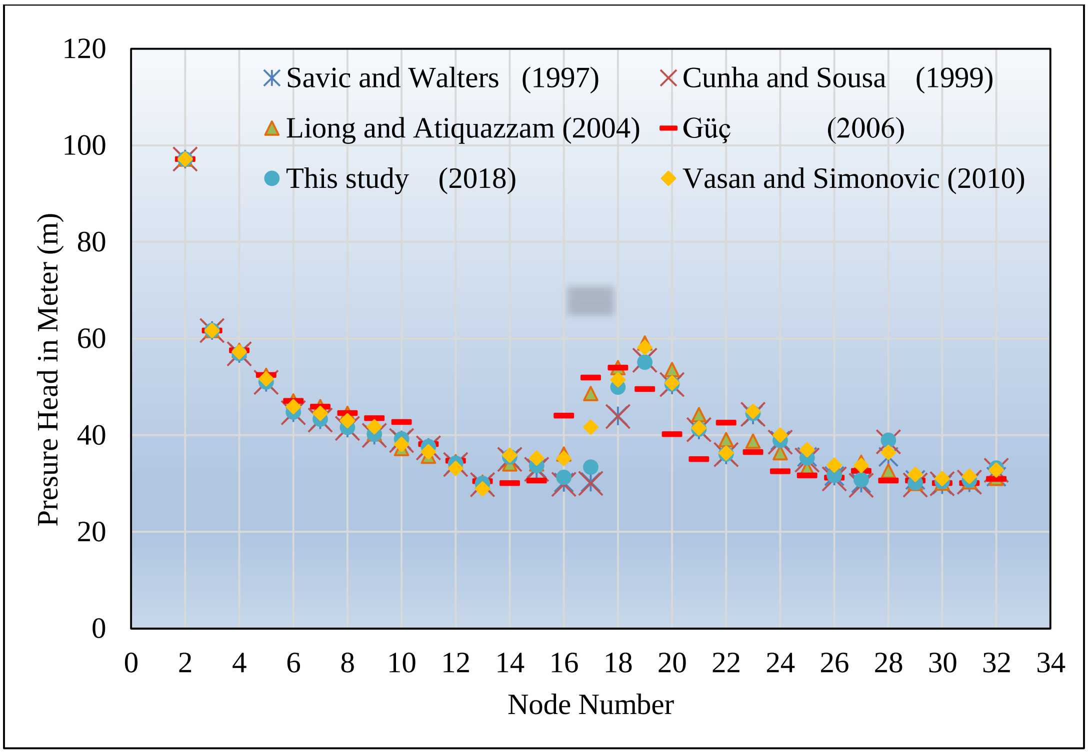

Figure 4.5: Comparison of the Final Pressure Heads of Hanoi Network

In addition, find optimal results of velocity for this present study are compared with other previous studies and tabulated in Table 4.5. However, there isn't generally significant different in velocity especially for maximum and the average velocity, but for minimum velocity, 0.21 m/s is found in pipe 31 being velocity for this study and 0.00 m/s is found in pipe 14 as the lower velocity Liong and Atiquazzam (2004).

Finally, best value of velocity in all pipes generally are obtained for this set of diameters which were achieved in this study as shown in Table 4.5.

Table 4.4: Comparisons of Nodal Pressure Heads of Hanoi Network

Link ID	Pressure head (m) for final diameters at all nodes				
	Savic and Walters (1997)	**Cunha and Sousa (1999)**	**Liong and Atiquazzam (2004)**	**Güç (2006)**	**This study (2018)**
R1	100	100	100	100	100
J2	97.14	97.14	97.14	97.14	97.14
J3	61.67	61.67	61.67	61.67	61.67
J4	56.88	56.87	57.54	57.54	56.92
J5	50.94	50.92	52.43	52.44	51.02
J6	44.68	44.64	47.13	47.14	44.81
J7	43.21	43.16	45.92	45.93	43.35
J8	41.45	41.39	44.55	44.57	41.61
J9	40.04	39.98	40.27	43.51	40.23
J10	39.00	38.93	37.24	42.77	39.20
J11	37.44	37.37	35.68	38.15	37.64
J12	34.01	33.94	34.52	34.72	34.21
J13	29.80	29.74	30.32	30.51	30.01
J14	35.13	35.01	34.08	30.08	35.52
J15	33.14	32.95	34.08	30.59	33.72
J16	30.23	29.85	36.13	44.05	31.30
J17	30.32	30.03	48.64	51.97	33.41
J18	43.97	43.87	54.00	54.00	49.93
J19	55.57	55.54	59.07	49.58	55.09
J20	50.44	50.49	53.62	40.23	50.61
J21	41.09	41.14	44.27	35.07	41.26
J22	35.95	35.97	39.11	42.62	36.10
J23	44.21	44.30	38.79	36.53	44.52
J24	38.90	38.57	36.37	32.52	38.93
J25	35.55	34.86	33.16	31.66	35.34
J26	31.53	30.95	33.44	31.23	31.70
J27	30.11	29.66	34.38	32.62	30.76
J28	35.50	38.66	32.64	30.62	38.94
J29	30.75	29.72	30.05	30.62	30.13
J30	29.73	29.98	30.10	30.06	30.42
J31	30.19	30.26	30.35	30.09	30.70
J32	31.44	32.72	31.09	30.98	33.18

Table 4.5: Comparisons of Flow Velocities for Final Diameters in Hanoi Network

Link ID	Velocity (m/s) for Final diameters				
	Savic and Walters (1997)	Cunha and Sousa (1999)	Liong and Atiquazzam (2004)	Güç (2006)	This study (2018)
1	6.83	6.83	6.83	6.83	6.83
2	6.53	6.53	6.53	6.53	6.53
3	2.76	2.76	2.54	2.54	2.74
4	2.71	2.71	2.50	2.50	2.70
5	2.46	2.47	2.25	2.25	2.45
6	2.12	2.12	1.91	1.91	2.11
7	1.66	1.66	1.44	1.44	1.64
8	1.47	1.47	2.23	1.25	1.46
9	1.29	1.29	1.91	1.07	1.28
10	1.22	1.22	1.22	1.90	1.22
11	1.43	1.43	0.91	1.43	1.43
12	0.89	0.89	0.89	0.89	0.89
13	1.69	1.70	1.32	2.33	1.65
14	1.32	1.35	0.00	0.01	1.25
15	1.29	1.33	1.07	0.61	1.16
16	0.09	0.11	1.84	1.24	0.45
17	1.90	1.92	1.71	1.63	2.11
18	3.06	3.07	2.53	1.54	2.22
19	3.14	3.15	2.56	3.56	3.27
20	2.69	2.69	2.25	2.80	2.67
21	1.94	1.94	1.94	1.94	1.94
22	1.85	1.85	1.85	1.85	1.85
23	1.77	1.77	2.36	1.88	1.75
24	2.05	2.14	1.34	2.21	2.11
25	1.55	1.64	1.32	1.71	1.61
26	1.67	1.65	0.29	0.94	1.58
27	1.21	1.15	1.34	0.87	0.97
28	0.19	0.26	1.28	0.57	0.44
29	1.62	1.28	1.34	1.75	1.28
30	1.00	1.17	0.72	0.72	1.16
31	0.41	0.20	0.09	0.36	0.21
32	0.96	0.88	0.82	0.27	0.89
33	0.77	1.11	0.67	0.64	1.11
34	1.59	1.26	1.23	1.05	1.26
Maximum velocity (m/s)	6.83	6.83	6.83	6.83	6.83
Average velocity (m/s)	1.92	1.91	1.8	1.79	1.88
Minimum velocity (m/s)	0.09	0.11	0.00	0.01	0.21

4.3 Case Study in Northern Part of Cyprus

Different studies that were performed recently has shown that the water shortage in Cyprus is inevitable (Payab and Türker (2017), Jamal and Türker (2015) and Türker et al. (2013)) unless an alternative water resource by means of water transmitting from any other country is implemented. Under these circumstances, water is transmitted from Turkey by means of a submerged suspended pipe line system. The project is aimed to supply 75 Mm^3 potable water per year. The distribution of the transmitted water is to be distributed all around the country with a new design project that is not considering the previous already in use water distribution network system. One part of this new pipeline distribution system is transmitting the potable water along the northern coast of the island approximately at an elevation of 60 to 80 meters. From this main transmission line, there are sub-connections that diverts the water to reservoirs available at higher elevations to store and then to supply water demand of the northern coast of the island by the action of gravitation. One of those sub-connections is the transmission pipeline proposed to divert water to a reservoir located at 355 meters away in Karaoğlanoğlu region. The stored water in Karmi reservoir is supplying the demand of Karaoğlanoğlu region by the gravitational action. In this study, alternative transmission pipeline system is proposed, while including abandoned, already available reservoir tank located at an elevation of 180 meters into the system. The location of the reservoir is same as a pump which is proposed to be installed to regulate the water up to Karmi reservoir. The best alternative is then selected through the optimization technique.

Implemented model consists of 8 nodes and 7 pipes that transfers water with a flow rate of 41.95 lt/s from the main pipeline to the two reservoirs at the end of the pipeline with sizes 136 and 90 cubic meters. The depth of water in the reservoirs are 3 meters. The types of pipes used during the construction is ductile cast iron. Also, the elevation and size of the reservoir that this present study will add to the system is 180 meters and 300 cubic meters heavy a water depth inside the reservoirs to be 3 meters. In addition, the length of the pipe that connects the transmission line to the new reservoir is 260 meters. Installation of new reservoir will help to divide the region into 2 parts: the lower and the upper regions. The proposed reservoir that is located at an elevation of 180 meters will supply water to lower Karaoğlanoğlu region and the reservoir at an elevation of 355 meters will serve for upper Karaoğlanoğlu region. The water demand for the region is estimated to be 150 lt/day/capita. The population of upper Karaoğlanoğlu region is 800 whereas for the lower region 7520. The details of the transmission line and the other information is shown in Figure 4.6.

The analysis of the current transmission pipeline system is the first scenario that is worked out. It is analyzed for the given data and then it is tested with optimization technique to get optimal design by getting minimum pipe diameters. The second alternative as the second scenario, is to consider the abandoned reservoir that has been already used for several years but abandoned with the new transmission line project. For the second alternative same as the first alternative, the procedure is repeated to achieve the best alternative.

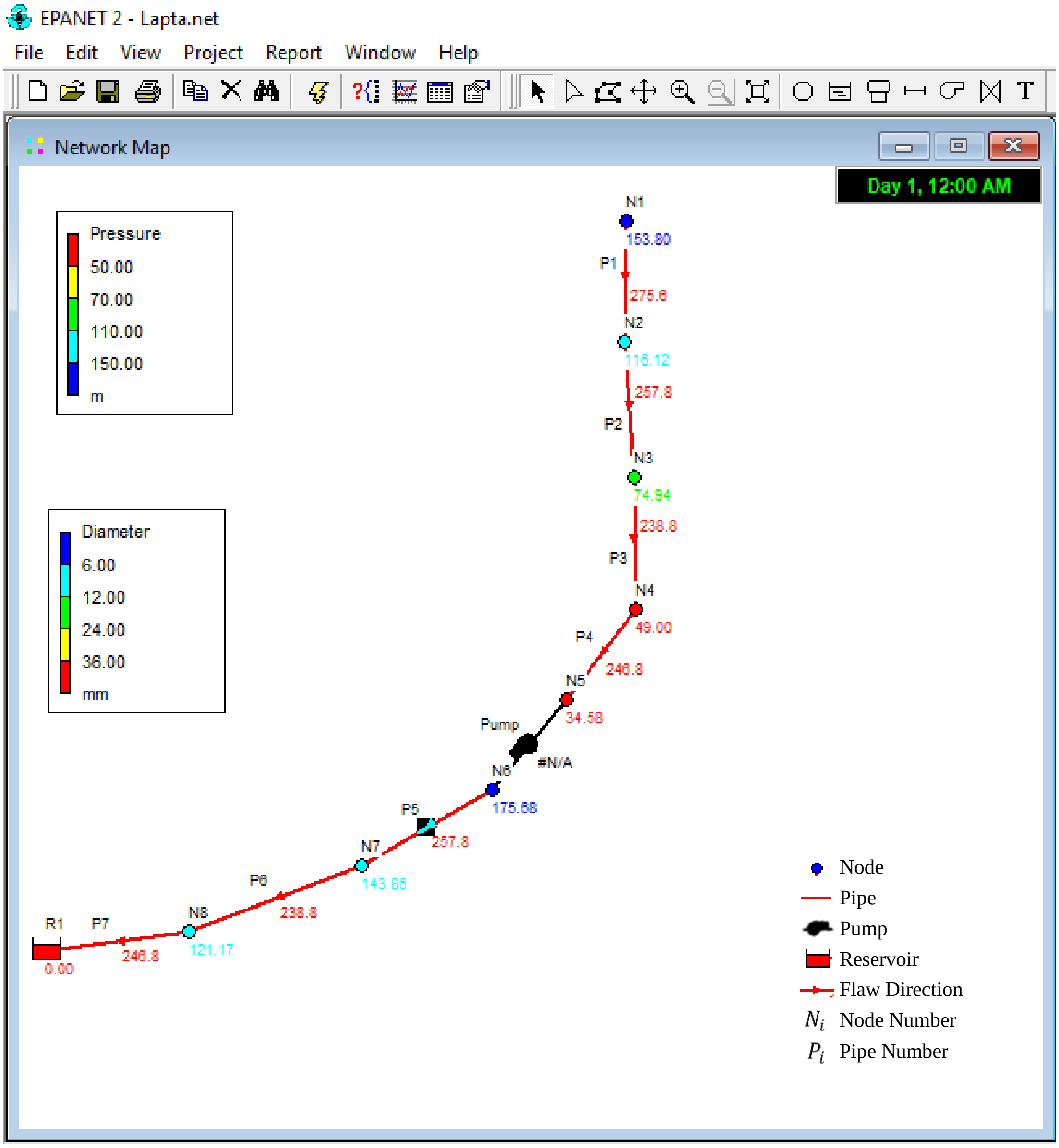

Figure 4.6: Karaoğlanoğlu Region Transmission Pipeline of the Cyprus. All the Lengths and Elevations are Given in Meters

4.3.1 Scenario Analysis

First Scenario: The analysis performed for the first scenario has shown that the required amount of water can't be transferred to Karmi reservoir due to the lack of enough pressure head (Appendix C). The hydraulic parameters like the head loss in the pipes, the pressure variation in the pipes are all given in Table 4.6.

Table 4.6: Output of the Considered Transmission Line for Scenario One

Network components	Z_i (m)	h_i (m)	HGL (m)	L_i (m)	Di (mm)	h_L (m)	V_i (m/s)
Node 1	73.5	173.90	247.40	-----	-----	-----	-----
Pipe 1-2	-----	-----	-----	620	275.6	1.18	0.7
Node 2	110	136.22	246.22	-----	-----	-----	-----
Pipe 2-3	-----	-----	-----	440.44	257.8	1.18	0.8
Node 3	150	95.04	245.04	-----	-----	-----	-----
Pipe 3-4	-----	-----	-----	233.91	238.8	0.94	0.94
Node 4	175	69.10	244.10	-----	-----	-----	-----
Pipe 4-5	-----	-----	-----	127.61	246.8	0.42	0.88
Node 5	189	54.68	243.68	-----	-----	-----	-----
Pump 5-6	-----	-----	121	-----	-----	-----	-----
Node 6	189	175.68	364.68	-----	-----	-----	-----
Pipe 6-7	-----	-----	-----	302.96	257.8	0.82	0.8
Node 7	220	143.86	363.86	-----	-----	-----	-----
Pipe 7-8	-----	-----	-----	675.08	238.8	2.67	0.94
Node 8	240	121.17	361.17	-----	-----	-----	-----
Pipe 8-R1	-----	-----	-----	944.79	246.8	3.17	0.88
Reservoir	358	0	358	-----	-----	-----	-----

The results show that, the designed system will not work efficiently right after the implementation of the project. Therefore, several suggestions can be discussed to release this problem.

Since the pressure head available at node 1 is 153.8 meters, the water in the system will not be able to reach to the end point which is at an elevation of 355 meters. Therefore, an installation of a pump is necessary to add an extra pressure to the system. In its designed form, the pump is located at an elevation of 189 meters. The calculations however have shown that the pressure head just before the pump is 34.58 meters. Therefore, the location of the pump should be reconsidered which will be discussed in Scenario 3. The installed pump head is 121 meters. As shown in Table 4.6, the installed pump head is not enough to deliver the water to the required destination which is the reservoir of Karmi. In order to manage to deliver the water, the required installed pump head should be 141.1 meters as shown in Figure 4.7. This figure also shows the pump curve that is suitable for the designed system. The discharge is given in lt/s and the pump head is given in meters.

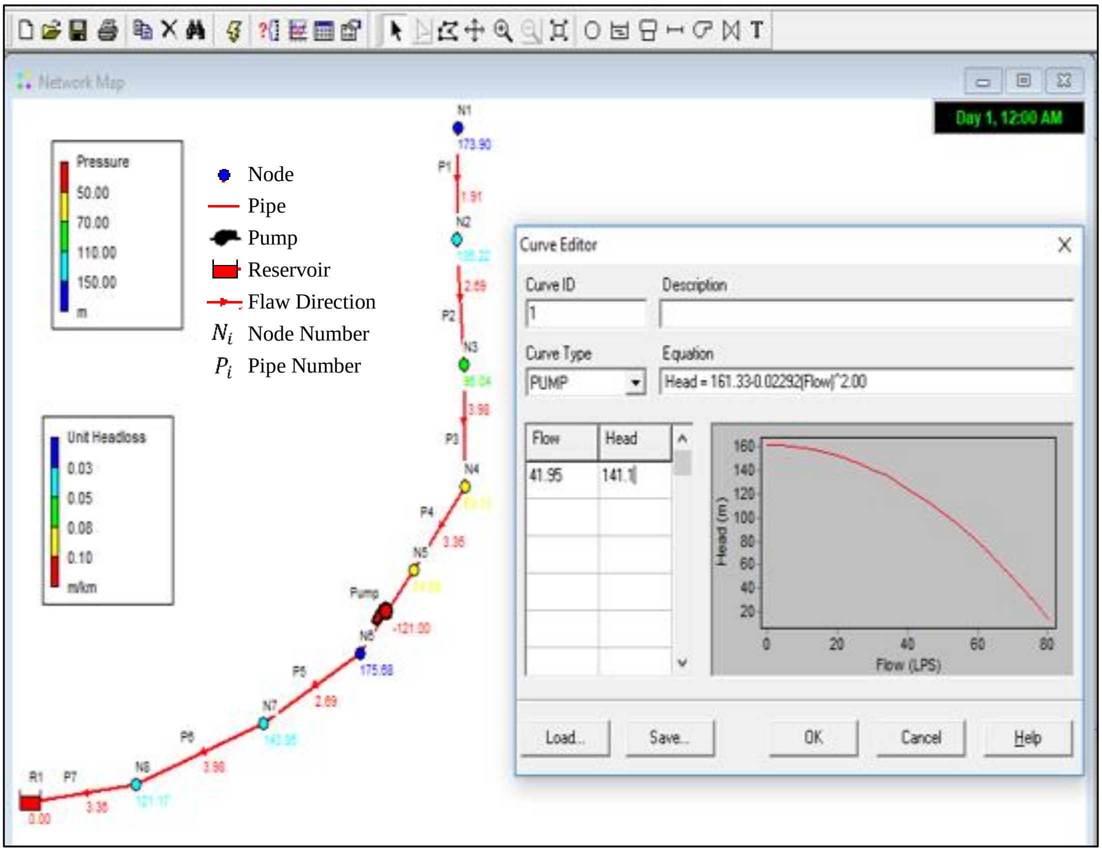

Figure 4.7: Suggestions for the First Scenario before Optimization

Second Scenario: The proposal is to use already available reservoir system at an elevation of 180 meters and divide the water distribution system into two: the upper and the lower region, where the reservoir at an elevation of 180 meters supplies water to the lower region. The required demand for the upper region will be covered by the higher reservoir called Karmi Reservoir. Karmi reservoir will receive the required amount of water through a pump that is capable to pump the water from 180 meters elevation to the 355 meters height. In order to deliver the water from lower reservoir to higher reservoir the pump head should be 185.77 meters. The transmission details of the second scenario are given in Figure 4.8. The outputs of the analysis are detailed in Table 4.7 in which all the information related with the important variables are given. EPANET software is used to perform the analysis starting from the reservoir and moving towards the starting node of the transmission line. The aim is to check whether the pressure heads at nodes are suitable to deliver the water to reservoirs or not. Keeping in mind that at reservoir, the pressure head is atmospheric pressure. This was also carried out for the new reservoir to check whether the available system is capable to carry the water or not. The node pressures were all higher than the obtained results, therefore the design fulfills all the requirements to carry the water from node 1 up to new reservoir. The node pressures for minimum requirement is given in Figure 4.9.

Table 4.7: Outputs of the Considered Transmission Line for Scenario Two

	Node one to new reservoir						
Network components	`Z_i (m)	h_i (m)	HGL (m)	L_i (m)	D_i (mm)	h_L (m)	V_i (m/s)
Node 1	73.5	114.25	187.75	-----	-----	-----	-----
Pipe 1-2	-----	-----	-----	620	275.6	1.2	0.7
Node 2	110	76.55	186.55	-----	-----	-----	-----
Pipe 2-3	-----	-----	-----	440.44	257.8	1.2	0.8
Node 3	150	35.35	185.37	-----	-----	-----	-----
Pipe 3-4	-----	-----	-----	233.91	238.8	0.91	0.94
Node 4	175	9.44	184.45	-----	-----	-----	-----
Pipe 4-5	-----	-----	-----	127.61	246.8	0.42	0.88
Node 5	189	-4.98	184.02	-----	-----	-----	-----
Pipe 5-R2	-----	-----	-----	260	238.8	1.02	0.94
Reservoir 2	183	0.00	183.00	-----	-----	-----	-----
	New reservoir to Karmi reservoir						
Reservoir 2	180	0.00	180	-----	-----	-----	-----
Pump R2-6	-----	-----	185.77	-----	-----	-----	-----
Node 6	180	185.77	365.77	-----	-----	-----	-----
Pipe 6-7	-----	-----	-----	260	238.8	1.04	0.94
Node 7	189	175.73	364.72	-----	-----	-----	-----
Pipe 7-8	-----	-----	-----	302.96	257.8	0.82	0.80
Node 8	220	143.91	363.90	-----	-----	-----	-----
Pipe 8-9	-----	-----	-----	675.08	238.8	2.71	0.94
Node 9	240	121.20	361.20	-----	-----	-----	-----
Pipe 9-R1	-----	-----	-----	944.79	246.8	3.2	0.88
Reservoir 1	358	0.00	358.00	-----	-----	-----	-----

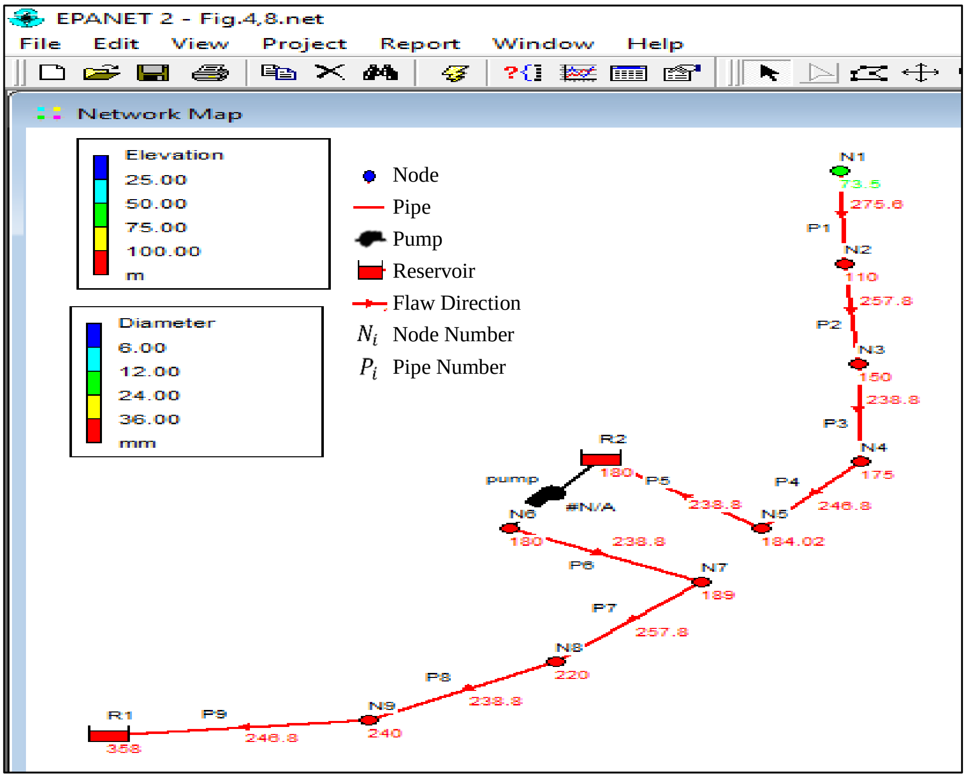

Figure 4.8: Second Scenario for the Design of Karaoğlanoğlu Network System

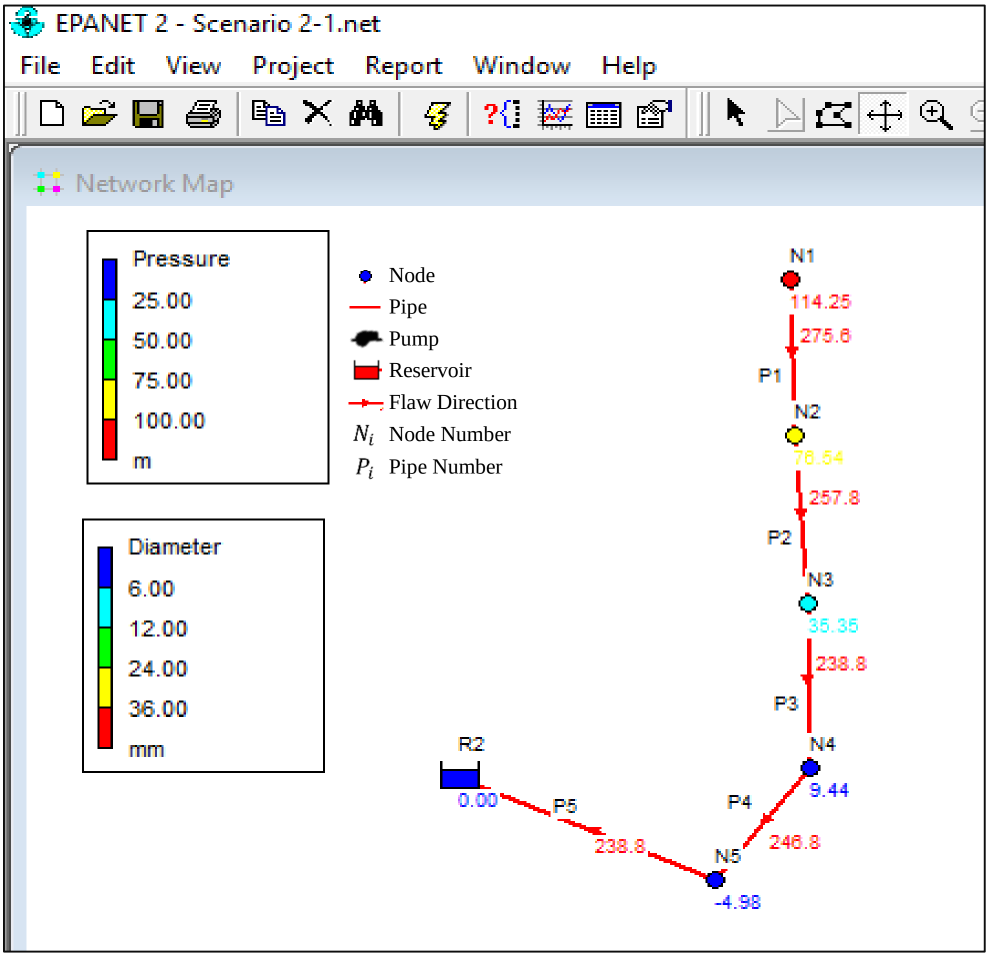

Figure 4.9: Minimum Node Pressure Heads to Carry Water from Node 1 up to the New Reservoir

The same procedure is also applied for the water transmission line between the new reservoir and Karmi reservoir. Keeping the atmospheric pressure at both of the reservoirs help to find the required minimum pressure heads at each node of this project. To keep the same flow rate (41.95 litters/second) between the two reservoirs, it is observed that, the minimum pump head should be 185.77 meters (Figure 4.10). On the other hand, since the population of the upper region (800) is very small when compared with the lower region (7520), 41.95 litters/second pumping rate will require 0.8 hours a day to pump daily requirement to the Karmi reservoir.

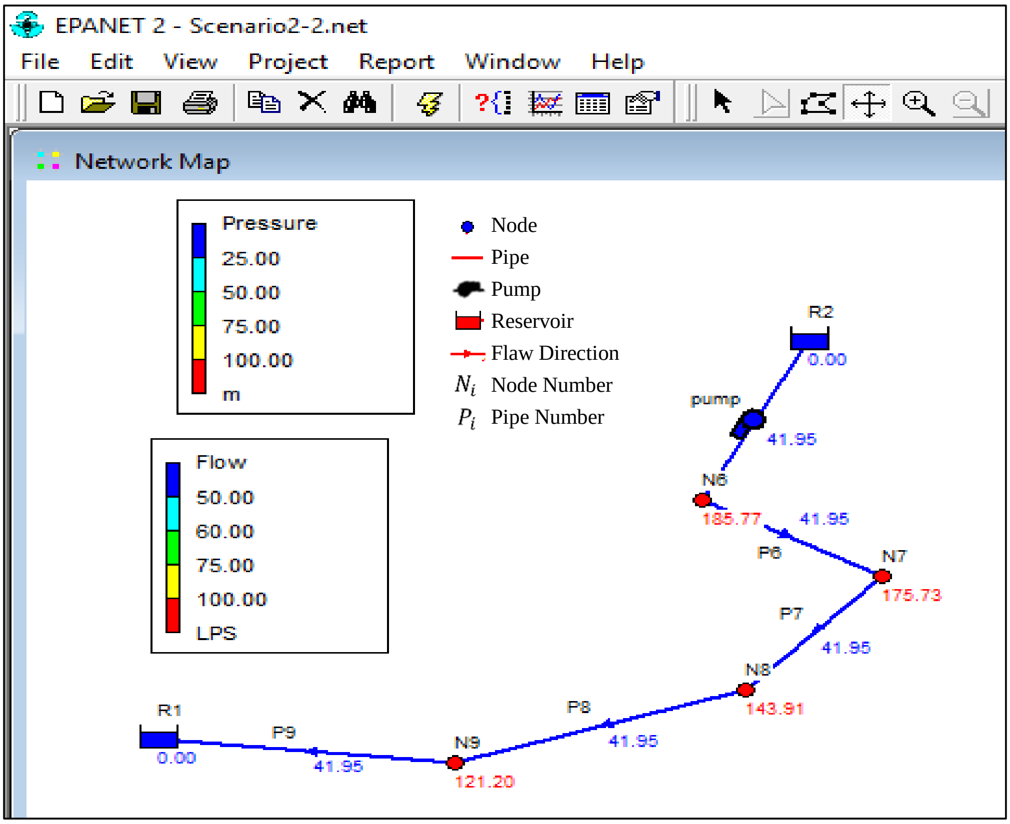

Figure 4.10: Transmission Line Between Two Reservoirs of Scenario Two

Third Scenario: Based on the findings in design project, it was calculated that the pump location and its pressure head was to be changed. Therefore, in the third scenario the research is concentrated on finding a new location for the pump such that the initial pressure head at node 1 will be used efficiently while the pump pressure is optimized for delivering water up to Karmi reservoir. The analyzes show that, the pressure head at node 1 essentially is equal to 153.80 meters which is enough to deliver the water up to node 6 in the absence of pump. In such a case, the pressure head at node 6 will be 2.73 meters (Table 4.8). Therefore, the location of node 6 can be accepted as a suitable location for the installation of a pump with a pump head equivalent to minimum 141.18 meters (Figure 4.11).

Table 4.8: Outputs of the Considered Transmission Line for Scenario Three

Network components	Z_i (m)	h_i (m)	HGL (m)	L_i (m)	D_i (mm)	h_L (m)	V_i (m/s)
Node 1	73.5	153.80	227.30	-----	-----	-----	-----
Pipe 1-2	-----	-----	-----	620	275.6	1.19	0.70
Node 2	110	116.11	226.11	-----	-----	-----	-----
Pipe 2-3	-----	-----	----	440.44	257.8	1.19	0.80
Node 3	150	74.92	224.92	-----	-----	-----	-----
Pipe 3-4	-----	-----	----	233.91	238.8	0.94	0.94
Node 4	175	48.98	223.98	-----	-----	-----	-----
Pipe 4-5	-----	-----	-----	127.61	246.8	0.43	0.88
Node 5	189	34.55	223.55	-----	-----	-----	-----
Pipe 5-6	-----	-----	-----	302,96	257.8	0.82	0.80
Node 6	220	2.73	222.73	-----	-----	-----	-----
Pump 6-7	-----	-----	141.18	----	-----	-----	-----
Node 7	220	143.91	363.91	-----	----	-----	-----
Pipe 7-8	-----	-----	-----	675.08	238.8	2.71	0.94
Node 8	240	121.20	361.20	-----	-----	-----	-----
Pipe 8-R1	-----	------	-----	944.79	246.8	3.20	0.88
Reservoir	358	0.00	358.00	-----	-----	-----	-----

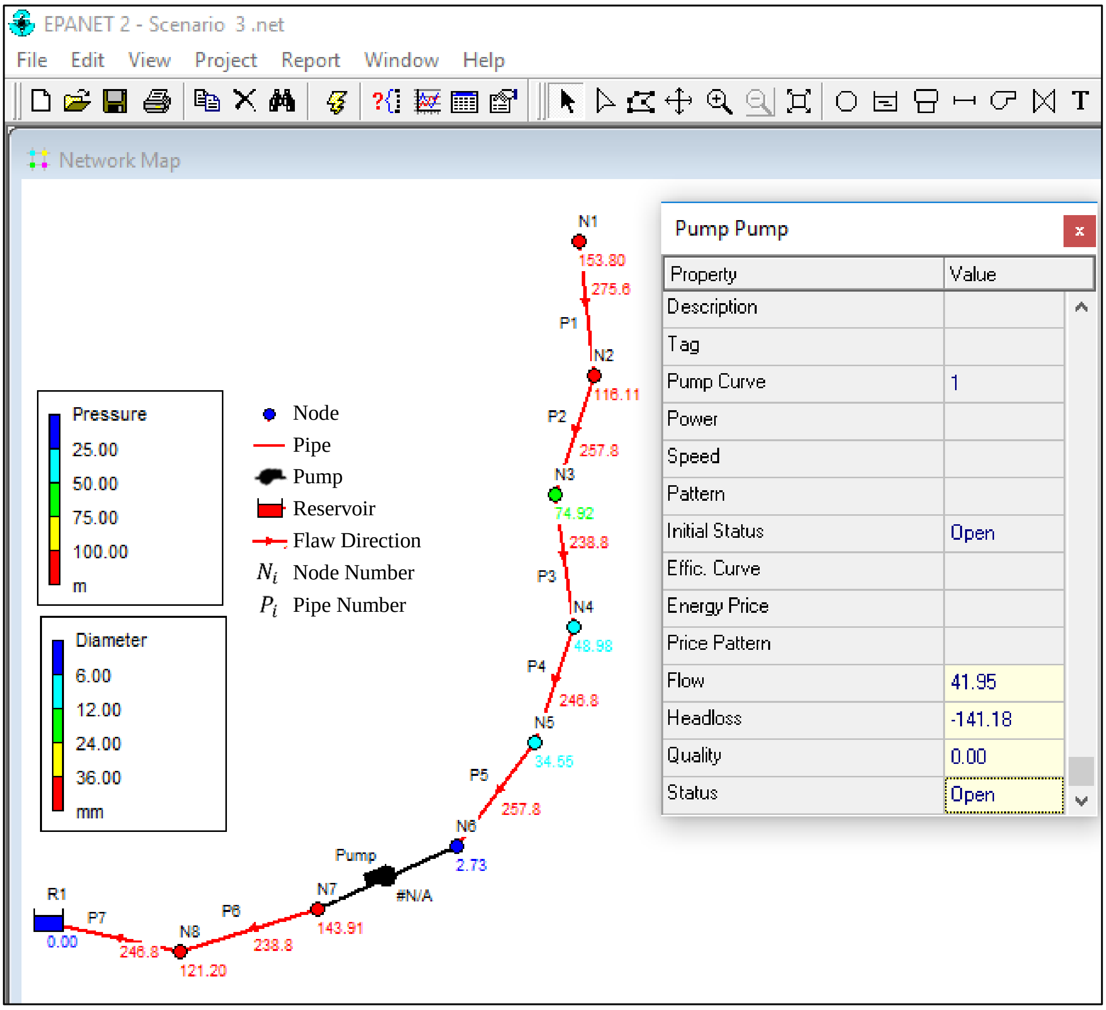

Figure 4.11: Third Scenario Analysis Showing the New Position of the Pump

4.3.2 Optimization Alternatives

The results of the study have shown that the proposed variable diameter pipes in the design process will not generate any problems for design calculations. However, in the case of implementation of the project, it is well known that, using constant diameters will improve the quality of the work and minimize the implementation period. In this study, therefore, the optimization was also aiming to achieve a constant pipe diameter to represent all kind of flows in the system. The optimization is carried out by the help of MATLAB code. The pipes used during the design process were all ductile cast iron with different diameters. These properties are given in Table 4.9.

Table 4.9: Details of Ductile Cast Iron Pipes used in the Project

ID	Nominal diameter (mm)	Pipe diameter (mm)	Mass of pipe per meter (kg)	Unit cost of the pipe (USD/kg)	Price of pipe (USD/m)	Average wall roughness "ε" (mm)
1	245	238.8	45.3	0.75	33.97	0.26
2	255	246.8	47.6	0.75	35.7	0.26
3	265	257.8	50	0.75	37.5	0.26
4	280	275.6	53.6	0.75	40.2	0.26

The input data to the MATLAB code is taken from Table 4.9 for each Scenario and the optimization process is progressed through genetic algorithm analysis. The results of Scenario 1 are given in Table 4.10. The final optimal result includes the diameter of all pipes in the water transmission system. It also includes the comparison of the diameters and the total costs of optimized and before optimized conditions. If the diameters are replaced, the pump head should be increased from 141.1 m to 144.01 m to get the same pressure head at all nodes at the same flow rates.

Table 4.10: Comparison Diameters and Total Cost of the Scenario One

ID	Length (m)	Pipe diameter (mm)	
		Before optimization	After optimization
1	620	275.6	238.8
2	440.44	257.8	238.8
3	233.91	238.8	238.8
4	127.61	246.8	238.8
5	302.96	257.8	238.8
6	675.08	238.8	238.8
7	944.79	246.8	238.8
Total cost (USD)		**121,964.0**	**113,622.0**

The optimization of the second Scenario is also performed based on the information given in Table 4.9. The results of Scenario 2 are given in Table 4.11. The final optimal result includes the diameter of all pipes in the water transmission system. It also includes the comparison of the diameters and total costs of optimized and before optimized conditions. When the diameters are replaced with optimum diameters, the pump head should be increased from 185.77 m to 186.75 m to give same pressure head values at all nodes at the same flow rates.

Table 4.11: Comparison Diameters and Total Cost of the Scenario Two

ID	Length (m)	Pipe diameter (mm)	
		Before optimization	After optimization
1	620	275.6	238.8
2	440.44	257.8	238.8
3	233.91	238.8	238.8
4	127.61	246.8	238.8
5	302.96	257.8	238.8
6	675.08	238.8	238.8
7	944.79	246.8	238.8
8	260	238.8	238.8
10	260	238.8	238.8
Total cost (USD)		**139,629.0**	**131,287.0**

The optimization of the third Scenario is also performed based on the information given in Table 4.9. The results of Scenario 3 are given in Table 4.12. In this Scenario, the position of the pump is different, and when optimum diameters are used the head pump should be increased from 141.18 m to 144.11 m so as to achieve the same flow rate. The results analysis is given in Figure (4.14). Any change on the pump heads definitely affects the power of the pump which is directly related with the energy consumption. Such changes in the pump heads are effective in life cycle cost analysis.

Therefore, such effects are as well worked out in this study and detailed in the next section.

Table 4.12: Comparison Diameters and Total Cost of the Scenario Three

ID	Length (m)	Pipe diameter (mm)	
		Before optimization	After optimization
1	620	275.6	238.8
2	440.44	257.8	238.8
3	233.91	238.8	238.8
4	127.61	246.8	238.8
5	302.96	257.8	238.8
6	675.08	238.8	238.8
7	944.79	246.8	238.8
Total cost (USD)		**121,964.0**	**113,622.0**

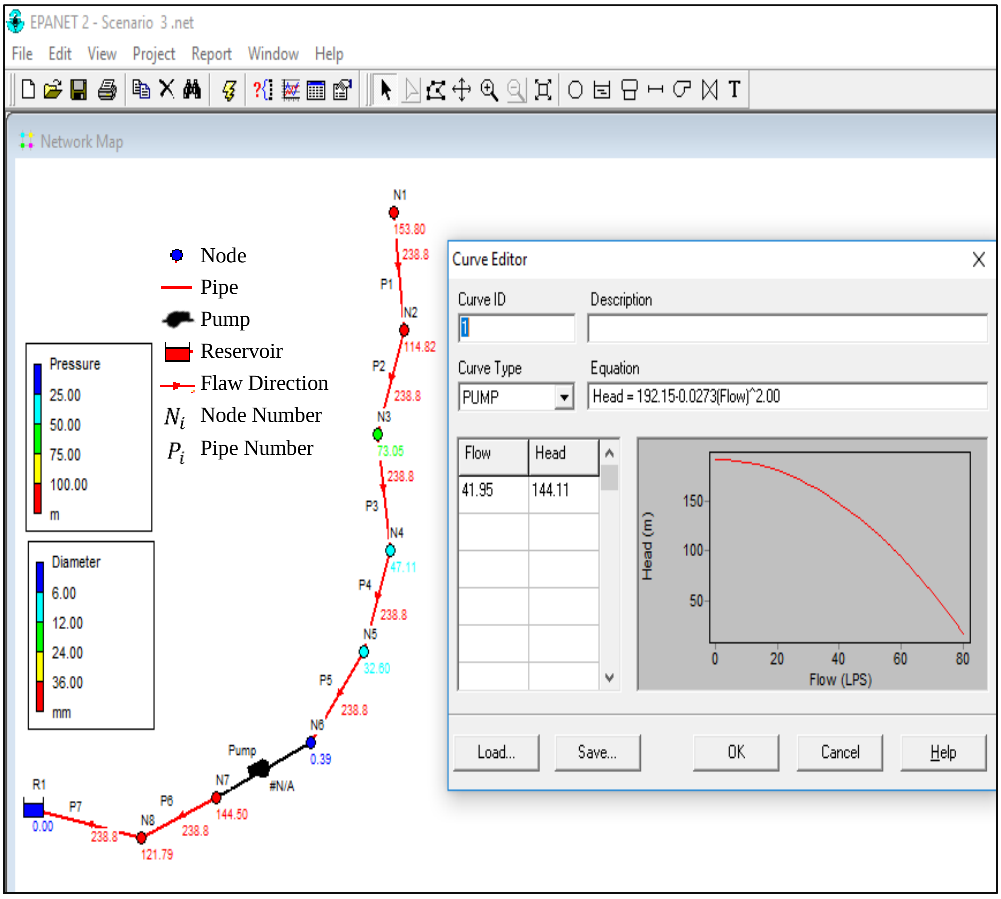

Figure 4.12: Optimum Diameters for the Third Scenario

4.3.3 Life Cycle Cost

The life cycle cost (LCC) of a pump in the network system is the total life time cost of any piece of equipment.

Generally, typical life cycle cost analysis components consist of initial costs, installation and commissioning costs, energy costs, maintenance and repair costs, down time costs, environmental costs, and decommissioning and disposal costs. The LCC provides to the decision maker to select the best alternative configuration system depending on the cost and the curing of the system. Pump systems often have a lifespan of 15 to 20 years.

The elements of the life cycle costs, LCC is presented such as:

$$\text{LCC} = C_{ic} + C_{in} + C_e + C_o + C_m + C_s + C_{env} + C_d \tag{4.1}$$

Where:

C_{ic} Initial cost, purchase price

C_{in} Installation and commissioning

C_e Energy costs

C_o Operating costs

C_m Maintenance costs

C_s Downtime costs

C_{env} Environmental costs

C_d Decommissioning and disposal

Among these elements, only three of them, initial cost, maintenance cost, and energy cost have dominant effect on the life cycle cost of transmission project designed for

Karaoğlanoğlu region. The others don't have significant effect. One of the main reason is that there is a single pump in this transmission system.

4.3.3.1 Initial Cost

The pump plant manager and designer should decide on the outline design of the pumping system. Generally, this element includes purchase price for the pump system, and all accessories service and with some options in the pump and system to increase the life span of the pump. All accessories and election charges for the initial cost, C_{ic} is proportional to its power P_o:

$$C_{ic} = K_p * {P_o}^{mp} \tag{4.2}$$

where P_o is power (kW), mp= an exponent, and K_p= coefficient. The power of the pump can be obtained from:

$$P_o = \frac{\rho\, g\, Q\, H_o}{1000\, \eta} \tag{4.3}$$

where η = combined efficiency of the pump and the motor. To obtain K_p and mp values, Samara et.al, (2003) is used that gives their relationship through the following Table 4.13. The data given in Table 4.13 in fact can be revised for different geographic regions and plotted on a log-log curve.

Table 4.13: Pump and Pumping Station Cost

Pump Power (kW)	Pump and Pumping Station Cost (A$)
10	36,000
20	60,000
30	73,000
50	105,000
100	185,000
200	305,000
400	500,000
600	685,000
800	935,000

When the log-log curve is plotted by the given data, thus, K_p = 5560 and mp = 0.723. Since the Eq. (4.2) can be expressed by the following equation:

$$C_{ic} = 5560 {P_o}^{0.723} \tag{4.4}$$

4.3.3.2 Energy Cost

One of the largest cost effecting criteria is the energy consumption and may cover the life cycle cost particularly if the pumps are operating at a rate more than 2000 hours per year. Average power P_o, developed over a year can be written as:

$$P_o = \frac{8.76\, \rho\ \ g\, Q\, H_o\, F_A\, F_D}{\eta} \tag{4.4}$$

Where: F_A=Daily average factor, F_D=Annual average factor.

Multiplying Eq. (4.4) by the power electricity (kW/h) R_E , the annual energy cost C_e for maintaining the flow rate is consumed, also can be written as:

$$C_e = \frac{8.76\, \rho\ \ g\, Q\, H_o\, F_A\, F_D\, R_E}{\eta} \tag{4.5}$$

4.3.3.3 Maintenance and Repair Cost

The maintenance cost depends on the time and frequency of service and the cost of materials. The cost of routine maintenance can be found by multiplying the cost of per event by the number of events expected during the life cycle of the pump.

$$C_m = \ cost\ per\ event * No.of\ the\ event \tag{4.6}$$

4.3.4 Final Results and Decision

The final results of the three Scenarios are tabulated in Table 4.14. According to the obtained data, the first Scenario and the third Scenario have lower total pipe cost. The initial cost of Scenario 1 and 3 are also very close to each other. The cost difference between them can be accepted as negligible. The third Scenario has higher cost for annual energy consumption for the pump which is almost same as the Scenario 1. The main reason of this is that, both Scenarios have the same length and diameter for the pipes system also they are responsible to supply the water for the both upper and lower region in the Karaoğlanoğlu region by the pump. The only difference is the location of the pump. In addition, the pressure heads at nodes are closed to each other throughout the transmission line.

Since the pressure entering into the pump in scenario 1 is more than the Scenario 3, the maintenance cost of the pump used in scenario 1 will be more than the scenario 3. This will increase the maintenance cost of Scenario 1. When Scenario 1 and 3 are re-evaluated in terms of the maintenance cost, therefore, the Scenario 3 is better than the Scenario 1. One should not forget that at the same time, the design project was offering pump head of 121 m in scenario 1, and in this study, this has been changed to 141.1 m before the optimization is considered. Therefore, the proposed location for the pump at node 6 of the scenario 3 is better than the pump location of the Scenario 1.

In addition, the Table 4.14 has shown that scenario 2 has the lowest cost when it is compared with the scenario 1 and 3 for the annual energy cost of the pump. The reason is the low pumping rates since the Karmi reservoir in scenario 2 is to be used only for serving a population of 800 people. For this the pump will operate only 0.8 hours a day which will minimize the energy cost for the operation of the pump. Also, the expectation of the maintenance cost for scenario 2 is less than scenario 1 and 3. However, total cost of the pipes for scenario 2 is more than the scenario 1 and 3 but at the end Scenario 2 is selected as the best Scenario for the transmission line.

Table 4.14: Average of the Results for Three Scenarios before and after the Optimization

ID Scenario	Total cost of the pipes (USD)		Pump head (m)		Power of the Pump (KW)		Annual energy cost (USD/year)	
	Before opt.	After opt.	Before opt.	After opt.	Before opt.	After opt.	Before opt.	After opt.
1	121,964	113,622	141.10	144.01	77.42	79.02	33,453	34,142
2	139,629	131,287	185.77	186.75	101.93	102.47	4,285	4,307
3	121,964	113,622	141.18	144.11	77.47	79.07	33,472	34,165

In Table 4.14, column 2 and 3 are obtained by running the MATLAB code, column 4 and 5 by EPANET, column 6 and 7 by applying Eq. (4.4), and column 8 and 9 by performing Eq. (4.5) which is explained in Appendix A.

Chapter 5

CONCLUSION

5.1 Conclusion

This study performed the genetic algorithm (AG) analyses on to the design of water distribution network system as known as optimal WDN design which is difficult and complex. The main purpose of this study is an attempt and to improve the methodology of WDS for analysis and optimal design by achieving hydraulic balance and minimum pipe diameters to get the least cost necessary for an optimum design. This process is limited for using the equations for water as a liquid and circular shape of the diameters. The GA was applied by using MATLAB code which is used in collaboration with EPANET to find the minimum best set of diameters in the water distribution system designed by EPANET. Both the programs combined in EPANET-MATLAB toolkit. The Hanoi network in Vietnam was examined as real network system, and Karaoğlanoğlu region main transmission line was tested to find best alternative.

The results show that, the genetic algorithm helps to improve the outcomes of the EPANET analysis. This study has these main conclusion as following:

- Using EPANET, as code program to analysis simulation model into MATLAB code at all iterations has been explained and improved based on organizing the flexible coding program to achieve best solution with regarding satisfaction conditions in analysing the water distribution network.

- MATLAB code calculated the best set optimal diameters for Hanoi network with respecting limited pressure heads from 30 m at all the nodes and the total cost was obtained as 6,081 million dollars for the network system then compared with previous studies.
- This methodology of the present study which is represented as MATLAB code program focuses mostly to find minimum pipe set diameters in the huge search spaces which was made to reduce the total cost of the WDN. Therefore, the influence of this developed coding system mostly appears in WDN system more than water transmission system.
- Three alternative scenarios were taken for transmission pipeline system in the Karaoğlanoğlu region based on optimizing pipe diameters, cost, and with organizing life cycle pump system. After comparison and discussion on the scenarios, second alternative was appointed as the best alternative scenario.

5.2 Recommendations for Future Studies

The major purpose of this study was to optimize the design WDN by getting minimum pipe diameters to achieve best cost with automatically checking the hydraulic conditions by EPANET in the developed MATLAB code. It is recommended for future researches, to apply this methodology with other benchmark problems in other water distribution networks. Also, this study can be developed for other components in WDN systems such as; storage tank, reservoir, and particularly pumping systems.

REFERENCES

Abdulhamid Saad, Hasan Ahweedi, HusamEL-Din Abdulkaleq, Alwaleed Khaleel & Ahmed Abdulsalam (2017). Comparison of Newton Raphson and Hard Darcy methods for gravity main nonlinear water network. *IOSR Journal of Mechanical and Civil Engineering* (IOSR-JMCE); *14* (3) 07:17.

Akdoğan, T. E. V. F. İ. K. (2005). Design of Water distribution system by optimization using Reliability considerations. (Doctoral dissertation, M. Sc. Thesis, The Graduate School of Natural and Applied Sciences, Middle East Technical University).

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

Brdys, M. A., & Ulanicki, B. (1996). Operational Control of Water Systems: Structures, Algorithms and Applications. *Automatica*, *32*(11), 1619-1620.

Cisty, M. & Bajtek, Z. (2009). Hybrid method for optimal design of water distribution system. *International Symposium on Water Management and Hydraulic Engineering* (pp. 235–246), Ohrid, Macedonia. Ss.Cyril and Methodicy University in Skopje, Ohrid, Macedonia: Faculty of Civil Engineering.

Colebrook, C.F. (1938). Turbulent flow in pipes with particular reference to the transition region between smooth and rough pipe laws. *J. Inst. Civ. Engrs. London 11*, 133–156.

Cunha, M. D. C., & 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., & Murphy, L.J. (1996). An Improved Genetic Algorithm for Pipe Network Optimization," *Water Resour*. Res, *32*(2). 449-458.

F.M. White (2011). *Fluid Mechanics.* 7th Edition, McGraw-Hill: New York.

Fishenich, D. C. (2000). Robert Manning (A Historical Perspective). Emlip.

Fujiwara, O., & Khang, D. B. (1990). A two-phase decomposition method for optimal design of looped water distribution networks. *Water Resources Research, 26*(4), 539-549.

Güç, G. (2006). Optimization of Water Distribution Networks with Genetic Algorithm", M.Sc. Thesis, Middle East Technical University.

Hopkins, M. (2012). Critical Node Analysis for Water Distribution System Using Flow Distribution.

Jamal, A. A. A., & Türker, U. (2015). Assessment of the regional water balance of the limestone subaquifers of Cyprus. *Journal of Earth System Science, 124*(3), 541-551.

Lansey, K. E., & Mays, L. W. (1989). Optimization model for water distribution system design. *Journal of Hydraulic Engineering*, *115*(10), 1401-1418.

Lencastre, A. (1987). Hand Book of Hydraulic Engineering " *John Wiley and Sons Ltd*.

Liong, S. Y., & Atiquzzaman, M. (2004). Optimal design of water distribution network using shuffled complex evolution. *Journal of The Institution of Engineers*, Singapore, *44*(1), 93-107.

Martorano, S. (2006). Calculating Friction Loss: Darcy Weisbach Formula vs. Hazen Williams. Viking.

Moody, L.F. (1944). Friction factors for pipe flow. *Trans*. ASME 66, 671–678.

Niazkar, M., & Afzali, S. H. (2017a). Analysis of water distribution networks using MATLAB and Excel spreadsheet: h-based methods. *Computer Applications in Engineering Education*, *25*(1), 129-141.

Niazkar, M., & Afzali, S. H. (2017b). Analysis of water distribution networks using MATLAB and Excel spreadsheet: Q-based methods. *Computer Applications in Engineering Education, 25*(2), 277-289.

Open Water Analytics. (January 06, 2018). Retrieved from https://github.com/OpenWaterAnalytics.

Payab, A. H., & Türker, U. (2017). Analyzing temporal–spatial characteristics of drought events in the northern part of Cyprus. Environment Development and Sustainability. https://doi.org/10.1007/s10668-017-9953-5.

Pilar, M., Adela, G.G., & Jose, L.A. (1999). Pipe Network Systems Optimization Using a Modified Genetic Algorithm, *Water Resour*. Res, *35*(11). 3467-3473.

Pipe Flow Friction and Minor Losses. (January 06, 2018). Retrieved from http://mimoza.marmara.edu.tr/~neslihan.semerci/ENVE204/L1.pdf.

Reehuis, E., (2010). Multi objective Robust Optimization of Water Distribution Networks, Master Thesis. Leiden University, The Netherlands.

Riazi, A., & Türker, U. (2017). A genetic algorithm-based search space splitting pattern and its application in hydraulic and coastal engineering problems. *Neural Computing and Applications*. https://doi.org/10.1007/s00521-017-2945-4.

Rossman, L. A. (2000). *EPANET 2*: User's manual.

Samra, S., & Essery, C. (2003). NSW Reference Rates Manual: For Valuation of Water Supply, Sewerage and Stormwater Assets. Ministry of Energy and Utilities.

Savic, D. A., & Walters, G. A. (1997). Genetic algorithms for least-cost design of water distribution networks. *Journal of water resources planning and management, 123*(2), 67-77.

Schaake, J. C., & Lai, F. H. (1969). Linear programming and dynamic programming application to water distribution network design. MIT Hydrodynamics Laboratory.

Shamir, U. & Howard, C. D. (1985). Reliability and risk assessment for water supply systems. *In Computer applications in water resources* (pp. 1218-1228). ASCE.

Simpson, A. R., Dandy, G. C., & Murphy, L. J. (1994). Genetic algorithms compared to other techniques for pipe optimization. *Journal of water resources planning and management*, *120*(4), 423-443.

Su, Y. C., Mays, L. W., Duan, N., & Lansey, K. E. (1987). Reliability-based optimization model for water distribution systems. *Journal of Hydraulic Engineering*, *113*(12), 1539-1556.

Suribabu, C.R., (2006). Optimal Design of Water Distribution Networks, PhD Thesis. SASTRA Deemed University, India.

Swamee, P. K. (1993). Design of a submarine oil pipeline. *Journal of Transportation Engineering*, *119*(1), 159-170.

Swamee, P. K., & Sharma, A. K. (2008). Design of water supply pipe networks. *John Wiley & Sons.*

Swamee, P.K. (1990). Form resistance equations for pipe flow. Proc. National Symp. on Water Resource Conservation, Recycling and Reuse. *Indian Water Works Association*, Nagpur, India, Feb. 1990, pp. 163–164.

Swamee, P.K. (2004). Improving design guidelines for Class-I circular sedimentation tanks. *Urban Water Journal*. *1*(4), 309–314.

Swamee, P.K., Garg, A., and Saha, S. (2005). Design of straight expansive pipe transitions. *J. Hydraul*. *Eng. 131*(4), 305–311.

Türker, U., Alsalabi, B. S., & Rızza, T. (2013). Water table fluctuation analyses and associated empirical approach to predict spatial distribution of water table at Yeşilköy/AgiosAndronikos aquifer. *Environmental Earth Sciences, 69*(1), 63-75.

Ulanicka, K., Ulanicki, B., Rance, J., Coulbeck, B., Powell, R., & Wang, C. (1998). Benchmarks for water networks modelling. In Hydroinformatics'98: Proceedings of the 3rd International Conference on Hydro informatics, pages 1469-1476, Copenhagen, Denmark. Int. *Association for Hydraulic Research Rotterdam*, The Netherlands.

Vasan, A., & Simonovic, S. P. (2010). Optimization of water distribution network design using differential evolution. *Journal of Water Resources Planning and Management*, *136*(2), 279-287. https://doi.org/10.1061/(ASCE)0733-9496(2010)136:2(279).

Xu, C., & Goulter, I. C. (1999). Reliability-based optimal design of water distribution networks. *Journal of Water Resources Planning and Management*, *125*(6), 352-362. https://doi.org/10.1061/(ASCE)0733-9496(1999)125:6(352).

APPENDICES

Appendix A: Lapta Case Study in North Cyprus

Input flow rate for reservoirs by transmission pipe line

$$Q_{input} = 41.95 * \frac{60\ sec}{1\ min} * \frac{60\ min}{1\ hour} * \frac{24\ hour}{1\ day} * \frac{1\ m^3}{1000\ \ L} = 3{,}624.5\ {m^3}/{day}$$

$$Q_{input/hour} = \frac{3{,}624.5}{24} = 151.02\ {m^3}/{hour}$$

Demand of water for lower region = 0.15 m^3/day*7520 person =1128 m^3/day

This rate of water is supplied without pump from the new reservoir in the scenario 2.

Demand of water for upper region = 0.15 m^3/day*800 person =120 m^3/day

This rate of water is supplied by karmi reservoir in the scenario 2.

For added reservoir (Lower Res.)

Size of added reservoir (R_2) = 300 m^3

If there is a valve in front R_2

Open time per day = (1,128 + 120) / 3,624.5=8.27 hours/day it should be opened

For Karmi Reservoir R1 (upper Res.)

Size two Reservoir 136 m^3 +90 m^3 =226 m^3

Scenario 1 and 3.

Demand of water from Karmi reservoir =0.15 m^3/day*(800 + 7520) =1248 m^3/day

This rate of water is supplied to the both upper and lower region in the scenario 1 and 3.

For Scenario 1 and 3

$$Operating\ Pump\ = \frac{1248\ {m^3}/{day}}{151.02\ {m^3}/{hour}} = 8.27\ {hr}/{day}$$

For Scenario 2

$$Operating\ Pump\ = \frac{120\ ^{m^3}/_{day}}{151.02\ ^{m^3}/_{hour}} = 0.8\ ^{hr}/_{day} = 48\min/day$$

Energy consumption cost

$$C_e = \frac{8.76\,\rho\ \ g\,Q\,H_o\,F_A\,F_D\,R_E}{\eta} + (\text{electricity annual servce cost})$$

Where:

ρ 1000 kg/m^3

g 9.81 m/s^2

Q 41.95 L/s (0.04195 m^3/s)

H_o Is calculated in table 4.14

F_A (0.8 $hr/day\ for\ scenario\ 2\ , Also\ \ 8.27\ hr/day\ for\ scenario\ 1\ and\ 3$

F_A 365 $days/365\ days$

η Pump efficiency (75%)

R_E electricity price= 0.143 $USD/kW - hours$

$$\text{Electricity annual servce cost for one year} = (4.801 * 12\ \text{months})$$

Appendix B: MATLAB Code

```
clc;
clear;
%% Problem Definition
Minpr=30;

epanetloadfile('HA.inp');           % Create QAP Model
Di=[304.8 406.4 508 609.6 762 1016];
price=[45.73 70.40 98.38 129.33 180.75 278.28];

nVar=size(getdata('EN_LENGTH'),2);      % Number of Decision
Variables

%% GA Parameters

MaxIt=120;              % Maximum Number of Iterations
nPop=1000;              % Population Size
pc=0.7;                 % Crossover Percentage
nc=2*round(pc*nPop/2);  % Number of Offsprings (Parents)
pm=0.8;                 % Mutation Percentage
nm=round(pm*nPop);      % Number of Mutants
beta=5;                 % Selected Selection propabilities

%% Initialization
% Create Empty Structure
empty_individual.Position=[];
empty_individual.Cost=[];
empty_individual.Pressure=[];
empty_individual.Velocity=[];
empty_individual.Diameter=[];

% Create Population Matrix (Array)
pop=repmat(empty_individual,nPop,1);

% Initialize Population
for i=1:nPop
    % Initialize Position
    pop(i).Position=randi(size(Di,2),1,nVar);
    if i<=size(Di,2)
        pop(i).Position=i*ones(1,nVar);
    end
    D=Di(pop(i).Position);
    setdata('EN_DIAMETER',D);
    % Evaluation

pop(i).Cost=sum(getdata('EN_LENGTH').*price(pop(i).Position));
    pop(i).Pressure=getdata('EN_PRESSURE');
    if min(pop(i).Pressure(1:end-1))<minpr
        pop(i).Cost=pop(i).Cost+99999999999999;
    end

end

% Sort Population
Costs=[pop.Cost];
```

```
[Costs, SortOrder]=sort(Costs);
pop=pop(SortOrder);

% Update Best Solution Ever Found
BestSol=pop(1);

% Update Worst Cost
WorstCost=max(Costs);

% Array to Hold Best Cost Values
BestCost=zeros(MaxIt,1);


%% GA Main Loop

for it=1:MaxIt

    % Calculate Selection Probabilities
    P=exp(-beta*Costs/WorstCost);
    P=P/sum(P);

    % Crossover
    popc=repmat(empty_individual,nc/2,2);
    for k=1:nc/2

        % Select Parents
        i1=RouletteWheelSelection(P);
        i2=RouletteWheelSelection(P);
        p1=pop(i1);
        p2=pop(i2);

        % Apply Crossover
        c=randi([1 nVar-1]);
        x11=p1.Position(1:c);
        x12=p1.Position(c+1:end);
        x21=p2.Position(1:c);
        x22=p2.Position(c+1:end);
        y1=[x11 x22];
        y2=[x21 x12];
        popc(k,1).Position=y1;
        popc(k,2).Position=y2;
        % Evaluate Offsprings
        D=Di(popc(k,1).Position);

popc(k,1).Cost=sum(getdata('EN_LENGTH').*price(popc(k,1).Position
));
        setdata('EN_DIAMETER',D);
        popc(k,1).Pressure=getdata('EN_PRESSURE');
        if min(popc(k,1).Pressure(1:end-1))<minpr
            popc(k,1).Cost=popc(k,1).Cost+99999999999999;
        end
        D=Di(popc(k,2).Position);

popc(k,2).Cost=sum(getdata('EN_LENGTH').*price(popc(k,2).Position
));
        setdata('EN_DIAMETER',D);
        popc(k,2).Pressure=getdata('EN_PRESSURE');
        if min(popc(k,2).Pressure(1:end-1))<minpr
```

```
            popc(k,2).Cost=popc(k,2).Cost+99999999999999;
        end
    end
    popc=popc(:);

    % Mutation
    popm=repmat(empty_individual,nm,1);
    for k=1:nm
        % Select Parent Index
        i=randi([1 nPop]);
        % Select Parent
        p=pop(i);
        % Apply Mutation
        popm(k).Position=p.Position;
        q=randsample(nVar,ceil(nVar/5))';
        popm(k).Position(q)=randi(size(Di,2),1,ceil(nVar/5));
        % Evaluate Mutant
        D=Di(popm(k).Position);

popm(k).Cost=sum(getdata('EN_LENGTH').*price(popm(k).Position));
        setdata('EN_DIAMETER',D);
        popm(k).Pressure=getdata('EN_PRESSURE');
        if min(popm(k).Pressure(1:end-2))<minpr
            popm(k).Cost=popm(k).Cost+99999999999999;
        end
    end

    % Merge Population
    pop=[pop
         popc
         popm]; %#ok

    % Sort Population
    Costs=[pop.Cost];
    [Costs, SortOrder]=sort(Costs);
    pop=pop(SortOrder);

    % Truancate Extra Memebrs
    pop=pop(1:nPop);
    Costs=Costs(1:nPop);

    % Update Best Solution Ever Found
    BestSol=pop(1);
    D=Di(BestSol.Position);
    BestSol.Velocity=getdata('EN_VELOCITY');
    BestSol.Diameter=Di(BestSol.Position);
    BestSol.Flow=getdata('EN_FLOW');
    % Update Worst Cost
    WorstCost=max(WorstCost,max(Costs));

    % Update Best Cost Ever Found
    BestCost(it)=BestSol.Cost;

    % Show Iteration Information
    disp(['Iteration ' num2str(it) ': Best Cost = '
num2str(BestCost(it))]);

end
```

```
%% Results

figure;
plot(BestCost,'LineWidth',2);
xlabel('Iteration');
ylabel('Best Cost');
```

Appendix C: The Design Project of Karaoğlanoğlu Water Transmission