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M.Sc. in CIVIL ENGINEERING

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DESIGN OF WATER SUPPLY BY USING EPANET

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IN
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DESIGN OF WATER SUPPLY BY USING EPANET

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Gaziantep University



Supervisor

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by

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DESIGN OF WATER SUPPLY BY USING EPANET

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ABSTRACT

DESIGN OF WATER SUPPLY BY USING EPANET

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In order to fulfill the water demand of the continuously growing population, it is essential to provide the sufficient and uniform quantity of water through the designed network of pipelines by knowing the generic characteristics of the study area such as information related the available water sources, demand of water, the population of the region, requirement of storage tanks, the nature of the area and land profile. This study highlights the procedure achieved to design a gravity potable water distribution system for a village in Shaqlawa, Erbil - Iraq with the assistance of this information and data the water distribution pipelines' design for the study region with the assist of the software "EPANET". The source which has been chosen to provide the system with water is a natural spring named Kani Hangeer. A spring box was used to collect the water, a 90mm, 65mm, and 50mm PVC pipes were used to transmit the water to the community at the distribution area these diameters maintained flow rate ranges between 0.11-2.82 l/sec. As a result, these sizes were appropriate to meet the maximum required demand of water which was calculated as 2.9 l/sec and the head losses kept below 6 m/km in the system, the pressure heads were within the set limits at each outlet node, and fifty two homes participated in the project. The system consisted of 20 pipes and 18 junctions. The water supply system's design for appropriate supply of water is functional and effective to get every day water requirement in the village.

Key Words: Water Distribution System, Network Analysis and Design, EPANET, Population Prediction, Daily Demand and Nodal Demand.

ÖZET
EPANET KULLANILAN SU TEMİNİ TASARIMI

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Yüksek Lisans Tezi, İnşaat Mühendisliği
Danışman: Dr. Öğr. Üyesi Ayşe Yeter GÜNAL
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Sürekli artan nüfusun su ihtiyacını karşılamak için, mevcut su kaynağıyla ilgili bilgiler, su talebi (ihtiyacı), bölge nüfusu, su deposu ihtiyacı, bölgenin doğal yapısı ve arazi profili benzeri bilgilerle, boru hatları ve su dağıtım alanının dizayn edilerek, yeterli ve uniform (düzenli) miktarda su sağlanması gereklidir. Bu çalışma, bu bilgiler ve veriler yardımıyla, EPANET yazılımı kullanılarak Irak, Şaqlawa'daki bir köy için doğal eğimle (cazibe) çalışan içme suyu dağıtım sistemi boru hatlarını tasarlamak için elde edilen prosedürü vurgulamaktadır (göstermektedir). Sistemin su ihtiyacını karşılamak için seçilen kaynak Kani Askı adlı doğal kaynaktır. Suyu toplamak amacıyla kaynağında bir depo ve suyu 0,11-2,82 litre/saniye debi ile dağıtım bölgesine ulaştırmak için 90mm, 65mm ve 50mm PVC borular kullanıldı. Sonuç olarak, bu boyutlar, 2,9 litre/sn olarak hesaplanan maksimum su talebini ve sistemde 6 m/km'nin altında tutulan yük kayıplarını karşılamaya uygundur, her çıkış düğümünde basınç yüklerinin belirlenen sınırlar içindedir, projeye elli iki ev katılmıştır. Sistem 20 boru ve 18 bağlantıdan oluşuyordu. Uygun su temini için su temin sisteminin tasarımı, köyün günlük su ihtiyacını karşılamak için işlevsel ve etkilidir.

Anahtar Kelimeler: Su Dağıtım Sistemi, Şebeke Analizi ve Tasarımı, EPANET, Nüfus tahmini, Günlük talep ve Düğüm talebi.



"Dedicated to my family"

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LIST OF SYMBOLS

α	Alfa
β	Beta, is angle of the weir in radians ²
Q	Flow of water in L/s
C_e	Coefficient (normally $C_e=0.64$)
g	Gravity = 9.81 m/sec ²
h	Height of water in meters
h_f	Head loss due to friction in meters
L	Represents the length of the pipe in meters
D	Internal diameter of pipe in millimeter
C	Friction coefficient
f	Frictional head loss factor
V_i	Velocity
A_i	Area of the pipe
π	Pi
F_{ext}	External forces working on a body
m	Mass
p	Pressure
γ	Specific weight of water (N/m ³)
z	Elevation in meters
P	Present population
I	Average increment for a decade
i	Growth rate%
t	Time
P_F	Future population
P_o	Current population
P_d	Peak hour demand
k	Conversion factor $k = 0.849$ m/sec
R	Hydraulic radius

LIST OF ABBREVIATIONS

EPA	United States Environmental Protection Agency
WDS	Water Distribution System
GRG	Generalized Reduced Gradient
DP	Dynamic Programming
NLP	Non-Linear Programming
GA	Genetic Algorithm
GPS	Global Positioning Systems
AWWA	American Water Works Association
GIS	Geographic Information System
CAD	Computer Aided Design
HGL	Hydraulic Grade Line
IN	Interior Diameter
PVC	Polyvinyl Chloride
L/Ca/D	Liter per capita per day
L/Ca/s	Liter per capita per second
L/s	Liter per second
L/D	Liter per day
m/s	Meter per second

CHAPTER I

INTRODUCTION

1.1 Introduction

Next to air, water is the next life important requirement. It has different forms such as rivers, lake, springs, streams, lakes etc. In addition, all the ancient civilizations were built by the river shores to provide water sufficient to drink, bath, cook and other domestic uses. However, when civilization improved, water use quite raised. Thus, in this phase, with no well systematic public water supply scheme, the present civil life and towns could not improve. The presence of a sufficient clean potable supply could be a fundamental need in way of life and it must be considered as a human right. Access to the potable installation also provides considerable health and economic benefits to the people. Access to water also means the considerable amount of the time people spend getting clean water may be spent more effectively on other tasks, improving their economic productivity.

The water supply systems' main function is to provide sufficient amount of water for the consumers, within the limits of desired pressure with good quality. The water distribution network must provide also a stable hydraulic grade to provide enough pressure and water to serve consumers during emergency conditions.

Gravity flow water supply systems are beneficial where sufficient elevation difference is available to permit the water to flow from an input point which is the entrance of the network to the terminal point of the water supply system with required quantity and pressure head. The most important point in such systems is that no external power is needed to move the water. Thus achieving greater economy and reliability and in a gravity system makes the water flow from an import points which is at the highest elevation to all other points of the system such systems which are generally possible in hilly areas where the source of water is a natural water course which can be tapped by constructing an intake chamber. Normally depending upon

the topography and availability of land the water treatment plants and reserve are also constructed at source points to ensure water quality and quantity. The gravity flow water distribution systems have single or multiple input sources with branched and/or used network configuration depending upon the local conditions topography availability of sources and the water requirements.

Being far away from big cities, there is no easy access to electricity and fossil fuel in many rural villages. Thus, communities with a well hand pump collect rainwater and build systems to store water. They also have systems of gravity flow water by a spring or stream at a position higher than the village, which is the focus of this work utilizing Fatawa village as an instance. The thesis targets engineers with development work in rural areas. It could help social workers and healthcare professionals of these areas in the organizing designing and constructing aqueducts.

Generally in water distribution systems design there are three types of distribution, Gravity distribution systems, pumping systems and combined gravity and pumping systems. In gravity systems the water falls because of their weight from first to the second point and the which is used in this movement is the potential energy it has based on the it's elevation.

The pumping and the gravity systems differ in their functional and constructional requirements. The energy required for operate the pumping system is external energy, while the gravity system does not need any source of energy it runs by the gravitational energy, so the it depends on its own weight. The source of water in pump system may be a tube well, stream or lake while in gravity system it may be a spring, reservoir or lake. In gravity systems the reservoir or the storage tank must be a surface storage tank unlike that in pumping systems which may be surface tank or elevated tank. Pumping station should be provided at the intake of pumping system and a chamber or box should be provided at the intake of a gravity system.

Gravity flow water supply has no costs incurred through pumping. It has a reduced maintenance with no required external energy to operate the system and the system is strong and reliable. The disadvantages of such systems are; usually the quality of the water depends on the quality of the source so if it wasn't within the required limitations it needs physical or chemical treatment, available sources of water may not provide stable amounts of water during the year and water rights cause problems in some areas.

1.2 Objectives of the Study

Design of a Gravity fed water supply system which convey potable water from natural springs to the houses in a rural village for domestic use by the assistant of the software programing (using EPANET program). This work aims at providing the most important information for designing and constructing this rather project. It also helps the engineers and technicians in this field to understand the most important components such a work beginning with studying the location and topography of the area and site overview, computing the population, demand and water supply, land survey, route finding, available springs, collect data like flow rate, elevations and other required information. Then start with design the different parts of the network (spring box, break pressure tank, water storage tank, pipe lines and accessories).

This thesis illustrated the hydraulic design of a gravity flow water supply system in a rural region.

The project aims to deliver clean water to rural communities with very limited financial and technical competence. The choice of purely gravity-driven water distribution systems excluding any adjustable devices, such as pumps and valves, for controlling pressures and flows, faces two main requirements:

1. The system should be as simple and as robust as possible to operate.
2. The operating, maintenance and investment costs must be kept as low as possible, an absolute requirement in rural communities.

1.3 Organization of the Thesis

In this thesis, chapter two provides a literature review of the gravity water distribution systems and describe previous studies, explaining the main components of such a kind of networks, provide a background information on the most famous software used in the designing and analyzing of the water distribution systems. A brief background of gravity water distribution systems is mentioned in chapter three, it presents the main components of this kind of networks, the function of each part of them, necessity and location of those parts. Then, the basic hydraulic principles which achieve the behavior of the water flowing in gravity distribution networks explained. The principles of hydraulic modeling are mentioned as well. Finally the benefits of using gravity water distribution systems and disadvantages of this type networks.

Chapter four provides background information on the study area and an overview of the village including the location, the geography, climate, available water resources, etc. And explaining all the methodologies and materials have been used in this thesis, determination of pipe roughness coefficient, projection of future population, water demands depending on the population and allocating nodal demands, and give a brief summary about the software used in this work which is EPANET program. Chapter five gives the results have been obtained by using certain methods to design the water distribution system in form of tables and graphs. Finally chapter six gives a conclusion reached in this study and recommendations.



CHAPTER II

LITERATURE REVIEW

2.1 Introduction

The gravity water supply systems date back to thousands of years. They transfer water from natural springs, streams, rivers and lakes to domestic, industrial and agricultural uses. Initially, open channels were used to dug into the ground or stone or wood. In addition, it is used above the ground open channel systems in the easiest and cheapest way for conveying water. It also has agriculture uses. However, these systems have their water open to the air. Thus, the water could be contaminated and requires a consistent downhill flow. The current capability of the manufacture to make sealed pipe from metal or plastic energy in the form of pressure makes the transportation of water over hills possible from water source to its final destination. The change in elevations of which the flow can't be raised is dependent on the amount of pressure the pipe can resist and the losses due to friction along the inner walls of the pipe.

Since the mid of the 1500s, the possibility of pump use is real with the aim of conveying water. Pumps could push water from sources such as grounds or rivers to the distribution area. These pumps often reduce the number of pipe of water project and increase the maintenance costs. They need easy accesses to fossil fuel or electricity for their operation. It is important to fully plan and develop a water distribution network in the rural areas. This network could satisfy diverse consumer needs to meet the minimum requirements at one node. Its design phase could benefit to provide cheap solutions to some constraints such as demands and requirements, which are interesting.

The gravity water supply system, a hydraulic structure, is made from different components including source intakes, reservoirs, tanks, sedimentation units and

filtration unit if it is needed, break pressure tank, and pipes. These are important convey suitable water with specific pressure. The systems could be looped or branched. The latter do not close loops and the water transport in one way only unlike the looped system, which allows water to move to different locations, and this point could complicate the looped system design. The branched systems are cheap, easy in design and quick to build. When a pipe is broken, its separation from the looped system is re possible. In this case, it has a small impact on the consumers outside the near area, while second system causes the consumers' downstream lose water supply if these breaks are not repaired.

Colossal sum of cash is apportioned to the plan of an arrangement. So, it is basic to explore and build up a dependable arrange which fulfills the taking after conditions (McGhee and Steel, 1991):

1. Keep up the quality of water at standard level in the dispersion pipes.
2. Build up financial plan and format.
3. Deliver adequate quantity of water.
4. Provide satisfactory amount of water.
5. Keep up required water powered pressure.
6. Guarantee unwavering quality of supply amid any period.
7. Guarantee water for each customer amid repair of any segment of the system.
8. Give great quality channels with least leakage.

2.2 History of Water Distribution

Mesopotamia, which dates back to 3100 BC. It represents the opening of the authentic civilization. In this period, the use of drinking water has been fundamental. This period is the most punctual civilization of individuals in Mesopotamia, where the arrival of the wealthy and prolific is caused by the recurrent surges of the Tigris River and Euphrates. All the old civilizations continuously found close inexhaustible water supplies, and indeed old strolling ways were steered past characteristic springs, waterways, and streams. Old people were intensely thankful for any clear, clean, and inexhaustible water supply. Thus, the individuals could frequently burrow wells to gather water effectively and build protecting structures around those sources. In 1500 B.C., Crete water distribution pipes come into use. In Grete, the Minoan civilization was built. Knossos City developed an aqueduct system with tubular conduits for the

transportation of water. This is in addition to different ancient civilizations which have water canals on the surface which were the possible first used pipes (Walski et al, 2003).

Persia — 700 BC, the designed dispersion framework came within the form of “qanats” in antiquated Persia, presently Armenia. These qanats are canals with open burrows burrowed into slopes. They pushed groundwater from the slopes and cities on mountains. They utilized common qanats and a few of them still stay today. In 1933, qanats supplied water the Iranian capital-Tehran. Additionally, In Roman Domain — 312 BC, the Roman water passages were developed both over and underneath ground between 312 BC and 455 Advertisement. Those water passages connected gravity stream to communicate water from inaccessible sources of water into the cities, and a few of these noteworthy structures are still exist presently. Water Marcia is the longest reservoir conduit on water from a source found 23 miles outer part of Rome. 57 miles was the genuine length of the reservoir conduit because of its bending and bend along the farmland that was vital to preserve the unfaltering downhill stream of water. Underground water Marcia was constructed but seven miles at the conclusion were over the ground, finishing at Rome’s conveyance stores, from which the water was passed on to wellsprings and showers. As it were exceptionally few wealthy families had water channeled straightforwardly into their houses. Stones, clay and lead were used in making the channels (Walski et al, 2003).

Incas — 1450 is most wonderful earliest hydraulic engineering instance which is the Incan “Stairway of Fountains”. It is constructed about 1450 at Machu Picchu. It provided pure spring water from a pair from rain-fed to the entire city. It is located at the top of the mountain. The spring was distant half mile from the first 16 fountains. A 48-foot long spongy wall was designed and built which supply stone-lined canal and gathered water another smaller spring (Walski et al, 2003). There are five inches in the canal in terms of width and same in depth. 3% could convey about 80 gallons in a minute which is twice of the water in peak flow case of springs. Thus, Incan engineers could prevent the overflow and loss. The water coming through the fountains connected by stone channels creating 180-foot long water cascade called “water stairs”. The vertical drop was 65 feet. These fountains could be used by public except and was located within the Temple of the Condor. It not only provided

a secure, reliable public water supply and protected the erosion from the hillsides and slopes. In 1455, the first cast iron pipe was made of casting iron and became practical. Its first installation was made in Siegerland, Germany, which is located at Dillenburg Castle. In Boston, MA, in 1652, in Massachusetts, the nation's initial waterworks was fashioned to produce water for fire-fighting and domestic purposes. In this period, the Fires were a standard hazard because of the wood-framed homes and fireplaces. There were ten-inch thick hemlocks to make bored-out logs which were seven to 9 feet in lengths alongside pitch or tar, or even iron hoops. The road supplies the city district ran from Jamaica pool to the Faneuil Hall space.

There were many problems in wood because of uneven sets causing the logs to sag and hold stagnated water. In addition, it carries insects plagued the pipes and also wood could decay. Also, there was a wood taste in the water. Besides, the increase in pressure for pumping water into quickly increases in cities, which rip the pipes. In the same time, iron turns a lot of quickly in the market. In addition, the cities use it in the distribution systems. In 1664, in the Royal residence of Versailles in Lord Louis XIV in France, a 15-mile cast press water developed was fundamental from Marly-on-Seine to the Royal residence of Versailles. It is usually the longest pipeline at this time, and its parcels benefit into the 21st century. A line part, for benefit, was put in 1960s from France to the Joined together States, which is on display. In 1732, Pitot framed a velocity-measuring tool. Henri Pitot measured the speed of Seine River water (Walski et al, 2003).

Bernoulli distributes *Hydrodynamica* in 1738. The Swiss Bernoullis extends Newton and Leibniz's early arithmetic and material science discoveries to liquid frameworks. *Hydrodynamica* is distributed by Daniel Bernoulli in St. Petersburg and Strasbourg. These requirements had become the foundation for many devices such as the Venturi meter and, most famously, the aircraft wing, as well as the key to vitality standards used in pressure driven models. However, it was Leonard Euler, who in 1752 created the outlines of vitality conditions that would live on for a long time. 1754 — The first water frameworks in the United States are completed. Pennsylvania organized the most punctual consumable water dispersal systems within the United States. The Moravian in Bethlehem, Pennsylvania claimed to have invented the first water framework, which was quickly followed by frameworks in Schaefferstown and

Philadelphia. Within the Philadelphia framework, Steeds operate the pumps, and the canals are fashioned of wood. Afterward they have been supplanted with wood joint channels made with press bands to bear up higher weights. Ten a long time afterward to begin with steam driven pumps has been utilized in Bethlehem.

1770 — Chezy made head loss relationship. While past examiners recognized that imperativeness disappeared when the water was moving, it is Antoine Chezy who caught on that V^2/RS is sensibly relentless circumstances. For ages, this association has acted as a prologue for head loss conditions to be associated. Chime and spout joint are manufactured in 1785. In London, the Chelsea Water Company begins to use the necessary chime and spout joints. In 1804, the essential city inside used cast press channels for water mains worldwide in Philadelphia and was the essential to create a huge-scale water systems. It pulled back from the conduit of Schuylkill. Advanced York City taken after suit with water supply frameworks of 41 miles of canal with a steady in 1842, it had 16 underground passages, 114 culverts, and a bridge as well over the stream Harlem (Walski et al, 2003).

In water plants, pumps were used in which waterwheels and basic water were powered pumps. In 1800s, steam fueled windmills were used since of the ever-extending inhabitation and industrial revolution. For case, Chicago's inhabitation was 350 in 1835. Then, it had shoot up to above 60,000 individuals. Also, in 1869, Chicago grabbed international headlines when it presented its incomparable engineering achievement. Twin under-ground passages were linked in the Chicago Waterpower for providing drinking water. There were 138-foot tallness and three foot in the tunnel with a wide standpipe equalizing a weight within the city's main channels. The standpipe remains from the Extraordinary Chicago Fire at 1871 and it is still currently an important system. Engines which are driven by streams take water from Lake Michigan about 15 million gallons per day to the city's water mains. Today, there six engine pump 72.5 million gallons a day to Chicago with 90% of the U.S. population is supplied water from municipal supply systems. In 1839, the Hagen-Poiseuille equation was developed. Individually and empirically, Gotthilf Hagen and Jean Louis Poiseuille worked on head loss equations for laminar flow in small tubes. The Hagen-Poiseuille equation was not theorized until 1856 by Franz

Neuman and Eduard Hagenbach. In 1843, the St. Venant motion equation was discovered. A group of scientists, including George Stokes, Augustin de Cauchy, Louis Navier, and Simeon Poisson, developed the fundamental differential equations that describe fluid motion, known as the "Navier-Stokes equations." Although the expression St. Venant equations to entail the vertically and laterally averaged (that is, one-dimensional flow) equation form, Jean-Claude Barre de Saint Venant produced the most general version of those equations. Darcy-Weisbach head loss equation is invented in 1845. Julius Weisbach published a three-volume collection on building mechanics, which included the results of his experiments. Because Chezy's C is related to Darcy- f Weisbach's by $C^2 = 8g/f$, the Darcy-Weisbach condition arose from this study, which was essentially an elaboration of Chezy's work. Darcy's name is also linked to Darcy's law for stream across permeable medium, which is widely used in groundwater studies (Walski et al, 2003).

In 1878, the first automated sprinklers were installed. For the first time, Parmelee sprinklers had been installed. These were the first automatic fire sprinklers in the world. In 1879, Lamb's Hydrodynamics was released. The Mathematical Theory of Fluid Motion research of Sir Horace Lamb is published. For following editions, the word Hydrodynamics was used, the last of which was published in 1932. AWWA is founded in 1881. The American Water Works Association was founded by the 22 founding members, with Illinois' Jacob Foster serving as its first president. 1883 — The difference between laminar and turbulent flow is clarified. While earlier engineers like Hagen had noticed the distinctions between laminar and turbulent flow, Osborne Reynolds was the first to undertake the tests that firmly defined the two flow regimes. For defining the conditions under which each form of flow exists, he discovers the dimensionless number, later referred to as the Reynolds number. Osborne Reynolds discharged "An Exploratory look of the circumstances that decide whether the Movement of Water should be coordinate or twisted and the law of resistance in parallel channels". In 1896, Cole designs Pitot tube for pressure pipe. In spite of the various to attempts for the amplification, Henri Pitot's speed devised a measuring gadget to the pressure pipes, Edward Cole creates the primary down to earth device employing a Pitot tube with two manometer tips. The Cole Pitometer is broadly used for a long time, and Cole's company, Pitometer Partners, will perform

stream estimation considers (among numerous other administrations) into the 21st century (Walski et al, 2003).

In 1906, Hazen-Williams formula created. A. Hazen and G.S. Williams made an experimental equation for head losses in water pressurized pipes. In spite of the fact that not as common or exact in unpleasant, turbulent stream as the Darcy-Weisbach condition, the Hazen-Williams condition demonstrates simple to utilize and will be broadly connected in North America. From 1900 to 1930, the Boundary Layer Theory developed. Ludwig Prandtl and his understudies Johan Nikuradse, Theodor von Karman, Thomas Stanton, and Heinrich Blasius investigated the relationship between liquids and solids in depth. They were able to conceptually elucidate and tentatively validate the nature of drag between pipe dividers and a liquid as a result of their inquiry. The tests of Nikuradse, who inserted uniform sand grains inside channels and measured head suffering, led to a far better; a far better; a higher; a stronger; an improved a far better comprehension of the Darcy-Weisbach equation or formula's f coefficient calculation. Stanton created the first graphical representation of the relationship between f , the Reynolds number, and the pipe roughness, which led to the creation of the Moody graph. This work is summarized in H. Schlichting's book, Boundary Layer Theory. The United States' drinking water standards were set in 1914 and before. Drinking water guidelines were announced by the country's Public Health Service, which changed often. The Environmental Protection Agency (EPA) of the United States has finally taken over the job of regulating water quality. Cement-mortar lining of water mains in the 1920s. Water mains were lined with cement mortar to prevent erosion and tuberculation. By the 1930s, procedures for cleaning and lining existing pipes had been devised (Walski et al, 2003).

The first Hydraulic Institute Standards were released in 1921. Trade Standards in the Pump Industry was originally released as a 19-page pamphlet. Pump nomenclature, testing, and rating were all based on these guidelines. The Colebrook-White formula is created in 1938. Imperial College in London's Cyril Colebrook and Cedric White used Prandtl's and his understudies' work to construct the Colebrook-White formula for obtaining the f of Darcy-Weisbach in commercial pipes. Hunter curves are issued in 1940. Roy Seeker of the National Bureau of Measures conducts research on water usage in various types of buildings in the 1920s and 1930s. His "fixture unit

technique" would form the cornerstone for analyzing building water use, despite the fact that plumbing installations may alter over time. Moody's diagram is published in 1944. The Moody diagram, created by Princeton University's Lewis Moody, is a graphical representation of the Colebrook White equation in turbulent flow and the Hagen-Poiseuille equation in laminar flow. This graphic is especially useful because the Colebrook-White equation has no clear solution at the time. Stanton had created a graphic identical to these 30 years before. 1991: GPS technology becomes widely available. The cost of global positioning systems (GPS) has come down to the point that they can be used to define point coordinates in hydraulic models (Walski et al, 2003).

Water quality modeling tool is first introduced in 1993. The USEPA's Lewis Rossman developed the software EPANET, which revolutionized water quality modeling. EPANET is a research tool that serves as the foundation for a number of commercial-grade models. From 1990 to the present Water distribution modeling solutions have been released by a number of commercial software developers. Each iteration provided additional data management enhancements as well as new interoperability possibilities with other computer software. In 2001, there was an automated calibration of distribution models taken from a study tool to the standard by Genetic Algorithms. In the same year, there was Security awareness of water increased and the value of water quality modeling was used as a tool to protect a water system. A year later, Water modeling was integrated with GIS software highly integrated with the WaterGEMS discharge, software combining the operation of both tools (Walski et al, 2003).

2.3 Water Distribution Networks Analysis and Design Methods

Several approaches were used so far to transfer flow rates in pipes. They differ from graphical methods in applying physical analogies and finally mathematical models. After their application and analysis (Ormsbee, 2008), these methods have been developed and used on the software programs as follow:

1. "Hardy Cross Method (s)".
2. "The Simultaneous Node Method".
3. "The Simultaneous Loop Method".
4. "The Linear Method (Simultaneous Pipe Method)".

5. "The Gradient Method (Simultaneous Network Method)".

The previous researches notarized and ported the reliability, suitability, correctness, and computational effectiveness of each of the various approaches. Modern computer methodologies such as simultaneous node, simultaneous loop, linear, and gradient could be applied to matrix formulations of network problems to take full advantage of modern computers' full capacity. The software history is based on the application model in the analysis of the network that could be divided into: the pre-computer period. The advent of the Computer Age follows this. The last period is the advanced methods. Following each time is briefly explained (Ormsbee, 2008).

2.3.1 Age Before Computer

Ormsbee in (2008) concluded that he Professor of structural engineering of the University of Illinois at Urbana Champaign Hardy Cross devised a mathematical method for analyzing moment distributions for statically determinate structures in 1936. Hardy He then realized that the proposed method could also be used to solve for pressures or flows in closed loop water distribution networks, and he published a paper describing how to do so. In this paper, he presented two different methods: one that solved for the flows in each pipe by repeatedly applying a flow adjustment factor to each loop in the system, and another that solved for the hydraulic grades at each node in the system by repeatedly applying a grade adjustment factor to each node in the system. The corresponding nodal grades were derived in the preceding condition by starting with a particular reference point, such as a tank or reservoir, and then adding or removing the associated pipe head loss between consecutive nodes as calculated using the Hazen-Williams equation. In the second scenario, the flows in each pipe were calculated directly using the Hazen- Williams formula for flow, with the differences in the resulting nearby nodal grades serving as a measure of the pipe's head loss. When employing the "loop" method, beginning pipe flow values must be provided for each pipe to ensure flow continuity at each junction node. Initial estimations of nodal hydraulic grade required to be supplied for each junction node when using the "node" approach (Ormsbee, 2008). When using the node adjustment method, convergence was delayed and not very satisfying," Cross said in a comparison of the two methods. This was attributable to "the good initial predictions for the hydraulic grades that were acquired." As a result, the loop adjustment

approach might soon become known as the "Hardy Cross Method" in the engineering world. The Hardy Cross approach was designed to be used with a non-computer based solution methodology at first. In each application, the general method structure was obviously the front-runner. However, the strategy may have to wait decades for the necessary computational hardware and software to be developed (Ormsbee, 2008).

2.3.2 Beginning of the Computer Age

In 1957, Hoag and Steven Weinberg custom-made the Hardy Cross technique to get the network flow downside for a computing machine. This strategy was applied to the water distribution system in California. In the presentation of their application, there were problems of speed and accuracy. Thus, it was economically viable in the application strategy for an outsized category of network issues. Later Rader & Associates, Miami, and Brown & Erskine Preston Caldwell, metropolis appeared as pioneers within the utilization of the pc to examine water distribution systems with their customers. The same year witnessed an Associate in Nursing of electronic computing firm, Datics Corporation of metropolis. It was followed by Texas as one of the primary company to sell pipe network analysis software. This means the arrival of the age of business package for network analysis (Engg. News-Records, 1957).

2.3.3 Advanced Computer Methods

The Hardy Cross method of the analysis of flows and pressures within the water distribution networks. In contrast, applying it to bigger systems as a growing understanding of some of the important method limitations:

- 1) "Depending on the network's size and complexity, the Hardy Cross approach may take a long time to converge to a solution or it may fail to converge at all."
- 2) "The prior method was limited to closed loop systems and did not clearly represent the behavior of network components such as valves, pumps, and other components," These limitations make researchers examine and research novel mathematical network analysis problem formulations fully to exploit the

opportunities of high speed computations. Some of the following techniques which had been developed later:

1. "The simultaneous node method".
2. "The simultaneously loop method".
3. "The simultaneous pipe method".
4. "The composite method".

Martin and Peters (1963) investigated a computer algorithm for simultaneously solving the hydraulic grades in a distribution network junction node. In essence, this method might be a parallel solution methodology for Cross's original "node" (1936). The algorithm is applied to the head loss equations for each pipe (e.g., Hazen Williams Equation) based on the flows in each pipe as a function of the hydraulic grades at the upstream and downstream pipe ends. Substituting these equations into the related conservation of mass equation at each junction provides a set of N nonlinear equations stated in terms of the nodal grades (where N is the total number of junction nodes). The resulting equations are then linearized using a typical Taylor Series expansion, and the Newton Raphson technique is used to solve them iteratively. According to Shamir and Howard (1968), the approach can be used to solve for other unknowns by using networks with pumps or valves. At the University of British Columbia, Alvin Flower and Robert Epp developed a superior way for such analysis in 1969, employing the Newton-Raphson method to tackle the flow adjustment concerns in Hardy Cross's original "loop" method (1936). As a result, the net can take advantage of the original algorithm's considerably better convergence qualities.

In 1972, Wood and Charles used a new formulation to the network's problem (the linear method). They include a mass nodal and energy equation conservation for each loop or route to be solved simultaneously for the direct production of the flow rate in each pipe. The "simultaneous loop" method decides the connected nodal grades needing a secondary head loss routine. Yet, by the feature of combining conservation of mass and conservation of energy equations substitutes the initial flow balance of the nodes. Here, the added vantage is capable to simply regulate other unknown parameters besides the flow rate. In other ways, it is the "linear technique" that is ancient and has resurfaced from Wood and Charles' original method of minimizing

the iterative convergence error of the solution of nonlinear energy equations. The algorithm's commercialization (i.e., WOODNET, KYPIPE, and PIPE 2000) followed a typical Newton Raphson solution process (Wood, 1980).

Following this, Utah State University's Roland Jeppson collaborated with CH2M Hill to produce commercial software for water distribution network analysis using the "simultaneous loop" method (Jeppson, 1976). The nonlinear energy equations for each loop or route in the network are stated in terms of flow adjustment factors when employing this method. The equations are linearized using a typical Taylor Series expansion and then solved repeatedly using the Newton Raphson method, just as the "node" approaches. Individual pipe flows can be computed by multiplying the original pipe flow estimates by the obtained factors once the final adjustment factors have been achieved. The technique, like the original Hardy Cross method, requires initial flow estimates for all pipes that meet flow continuity. In addition, obtaining the corresponding nodal grades requires the use of the Hazen Williams equation for each pipe, just as it did with the original Hardy Cross technique.

Todini and Pilati's Gradient Method is the final analysis (1988). Individual energy equations for each pipe were combined with individual nodal equations in each junction node to offer a simultaneous solution for both nodal heads and individual pipe flows in this formulation. The nonlinear energy equations are first linearized using a Taylor Series expansion, just like the "simultaneous loop" and "linear approach". Nonetheless, the equations are solved using a well-organized recursive technique that inverts the original coefficient matrix in this case. The EPA utilized this technology to create the software EPANET (Rossman, 1994).

2.4 Optimization Mechanisms

As an alternative of the trial-and-error approach, some optimization techniques have been utilized for designing new water distribution systems and expansion of already existed water supply networks. Linear, non-linear, and dynamic programming, for example, are examples of traditional optimization methods. This is in addition to the non-linear programming, dynamic and heuristic optimization techniques. Shamir and Alperovits in (1977) reported a linear programming gradient model for the optimal network design. They linearized mathematical formulation. They applied it to segmental lengths of pipes with variable diameters. The aim of this was minimize the

overall cost of the networks pipes. Although it is not advisable pipes, which constantly change in size along the system, arranging the pipes in this way may cause blockage in the network when the flow of the direction changes (Walski et al., 1990). Researchers developed the linear programming gradient (Quindry et al., 1981; Glulter and Morgan, 1985; Fujiwara et al., 1987; Fujiwara and Khang, 1990, Kessler and Shamir, 1989; Eiger et al., 1994).

The developed methods applied iterative procedures. The pressure heads or the flows are fixed while the sizes of the pipes (diameters) are optimized to suit specified pressure and flow requirements. Morgan and Glulter in (1985) used multiple demand patterns in their study. They created a water distribution network using a Hardy-Cross network solver and a linear programming model. Shamir and Kessler have introduced a two-stage linear programming gradient at 1989, at first step for a specified set of flows. This programming determines many sets of heads. Following that as a second stage, the flow-rates are modified whereby gradient of the objective functions. Eiger et al. in 1994 used the same formula as Shamir and Kessler in (1989). Non-smooth branch and bound algorithms, as well as duality theory, are used to solve the problem. The algorithms integrate primal and dual processes. If the difference between the best solution and the global lower bound was within a defined tolerance, it came to a halt. The LP model suffers a loss as a linearization of non-linear actual conditions. It is not always appropriate to linearize a problem, and in some cases, it may make the original problem more difficult to solve. NLP (Non-Linear Programming) is a form of non-linear programming.

The non-linear programming mechanism was devised by Chiplunkar et al (1986). Su et al. (1987), Mays and Lansey (1989), and Duan et al. (1990) employed non-linear programming to optimize the design of water supply networks. Unlike LP, the NLP model can handle numerous demand patterns and a substantially larger number of design factors. Davidon-Fletcher-Powell techniques were employed by Chiplunkar et al. (1986). It involves graph-theoretic with Newton's method. But, the non-linear programming technique assigned the loop flows as the optimal variable set to minimize the cost and expenses of the systems. One of its defects is that a looped network usually turns a tree network with some zero values in a number of pipe diameters. In addition, the NLP model is frequently linked to the local minima early

on. Su et al. (1987) applied the NLP in conjunction with a steady-state simulation model and the generalized reduced gradient (GRG) approach. This was done in order to come up with the best control variables possible. Mays and Lansey (1989) employ a comparable NLP optimizer with a hydraulic network simulation model, KYPIPE, to decide variables (Wood, 1980).

Non-linear programming algorithms have been one of the most widely used techniques for resolving water supply system problems for many years. Gradient-based algorithms, which require at least the first order derivatives of both the objective and constraint functions to establish the appropriate search direction, are the most effective of these strategies. Gradient-based techniques make it simple to find a relative optimum that is close to the best design. However, if the design space is non-convex, these methods cannot guarantee the global optimal solution. This is also ineffective in situations when the design space is discontinuous and the diameters of the pipes considered in non-linear programming are continuous, which may not correlate to commercially accessible pipe sizes (Savic and Walters, 1997; Gupta et al., 1999; Cunha and Sousa, 1999; Simpson et al., 1994). NLP is also incapable of dealing with large water supply systems. Non-linear programming is no longer widely recognized in the optimization sector.

Dynamic Programming (DP) (Wong and Larson, 1968), a mathematical approach, has been used since the 1960s. They used a number of optimization problems related to water resources in terms of engineering and management. This solves difficult optimization issues by breaking them down into a series of sub-problems known as stages. Each stage's difficulties are addressed using DP in connection to the preceding stage's difficulties. The state of one stage's output is the state of the next stage's input. They used a number of optimization problems related to water resources in terms of engineering and management. It solves difficult optimization issues by breaking them down into a series of sub-problems known as stages. Each stage's difficulties are addressed using DP in connection to the preceding stage's difficulties. The output of one stage becomes the input of the next. The decisions made at each stage result in a return cost, which is then used to compute the network's cost. It can handle a more broad cost function that is dependent on pipe pressure and pipe sizes.

A heuristic algorithm was employed in conjunction with two-stage dynamic programming. The pipe sizes were first determined using a heuristic method. Then, in order to iteratively determine the best answer, dynamic programming was used. At each level, a Newton-Raphson network simulation model has been used to build LOOPT based solutions for looping, branching, and mixed water distribution network challenges (Vamvakieridou-Lyroudia, 1993).

Lall and Percell (1990) developed a dynamic programming based optimizer (GPO) in the gas transportation pipeline networks. It was an optimization tool for determining the feasible steady state strategy for operating the compressor stations overcoming many problems. It aims to reduce the fuel cost of the pipeline. Though, applying dynamic programming is only used in simple network systems. The size of the system and the time it takes to solve the best strategy grow as a result of "the curse of dimensionality."

The enumeration approach was used by Gessler (1985), Loubser and Gessler (1993) in the design and rehabilitation of water delivery systems. The modeler assigns search space in the enumeration process by establishing a range of commercial dimensions for all pipes in the system. This method considers all possible collections of pipe diameters and determines whether or not the pressure constraints are serviceable for each collection. Finally, the algorithms choose the least expensive set of pipes. Of course, the best combination satisfies the constraints.

Simple genetic algorithms were employed by Simpson et al. (1994). These methods encode a feasible answer by encoding each individual population in a set of bits of the same length. The majority binary coded population (chromosomes) subjects in three processes. The first is selection. The second is crossover.

Mutation is the third factor. A roulette wheel is used in the basic GA. For determining the best network design, the first issue to consider is crossover and bit wise mutation. Furthermore, there is an application of the steady state network solver for the computation of the hydraulic performance in any network of the population. With the lack of sufficient network for meeting the head constraints, there is an incorporation of the deficiencies into the network cost as penalties for the calculation of the total cost. Thus, solution quality is assessed by a fitness function- a total

network cost inverse. On the basis of one specific problem, the GA performance was compared to the completion of enumeration and non-linear programming optimization techniques. According to Simpson et al. (1994), In a few function assessments, GA had a worldwide solution. A variable power scaling concept of the fitness function was established by Dandy et al. (1996), the gray codes, and an adjacency mutation operator (Dandy et al., 1996).

The fitness power is an inverse objective function value and increases in steps when GA develops. The exponent (a low value) used at the GA beginning reserves some population variations. It also stores the global exploration of the early generation solution space. Also, there is a need for a high value of exponent for the accentuation of the small differences in the string fitness. The crawling or adjacency mutation operator, which is utilized in typical bitwise mutation operators, is added to this. The latter was chosen at random from the coded series to complete the choice variable.

This administrator moves up or down the list of plan factor options, changing the chosen variable substring to an adjoining choice variable substring. Finally, Gray codes are used to communicate plan elements rather than binary codes. Because of these new qualities, Dandy et al. (1996) concluded that the enhanced GA performed better than the standard GA. They also stated that the new GA beat all existing methods, including linear, non-linear, and dynamic programming, as well as an enumerative search strategy.

Standard GA had been used in conjunction with EPANET network solver by Savic and Walters (1997). A computer model GANET has been developed to design lowest cost pipeline. To improve the capabilities of GANET and the Geographical Information Systems (GIS) (StruMap), the network solver (HARP) combines a relational database and an object-oriented evolutionary algorithm library. The end product would be a package that could be used to solve a variety of optimization problems in water system management, such as network design, reformation, and calibration (Walters et al., 1998).

In Gupta et al's (1999) study, the GA with a hydraulic simulator ANALIS is applied. For the analysis of the hydraulic performance of the network design, it follows the graph theory. Its results were compared to those of the NLP methodology, which

revealed that while NLP converged rapidly, GA produced better results. Yet, the last solution was improved when needing a large amount of considerable computational effort to reduce expenditure (Bassin et al., 1992).

In the study of Wu et al. (2001); Wu and Simpson (2001), the first aim is messy genetic algorithm (fmGA). It is used in designing water supply pipelines. fmGA tackles any problem in the water supply network by two loops, namely the inner and outer loop. The latter modifies big population random variables. The former is a building block of filtration and juxtapositional phases. This block selects the best-fitting short strings from the entire population. In addition, the series' length is reduced. Furthermore, in order to arrive at an ideal solution, the juxtapositional phase builds a new population string using common genetic operators of selection and reproduction.

Eventually, within the typical benchmark issues, fmGA achieved reasonable results. Abebe and Solomatine (1998) employed GLOBE (Solomatine, 1995), which comprises of many look calculations, including GA, instead of a single optimization computation.

They recognized that exceptionally few calculations reach to ideal or close ideal solutions. Cunha and Sousa (1999) presented an arbitrary look calculation (Mimicked Tempering) that's based upon the physical toughening approach's closeness to the Newton look strategy to illuminate the organize equation.

Todini (2000) demonstrated a cutting-edge tool for designing water supply frameworks based on flexibility data in order to improve the unswerving quality and accessibility of water amid drought conditions. In this technique, the versatility concept is employed to construct a heuristic optimization strategy that deals with fetched versus flexibility space, the edge of non-dominated arrangements, or the Pareto set. The proposed approach was put to the test on a huge water transport network.

Vamvakeridou-Lyroudia (2001) designed the network using G FUZNET, a combination of fuzziness and GAs (2001). For each prospective solution, they evaluated the total number of membership functions gathered. Aggregation took place in three stages: (1) pressure constraints for nodes are gathered; (2) partial

membership functions for velocity restrictions for connections are gathered; and (3) pressure aggregator for all nodes-total pressure aggregator is used to evaluate the system's entire membership function .

On the off chance that the punishment figure is high as well, great arrangements that have fair fizzled will be evacuated forever within the look handle (Wu and Simpson, 2002). The conventional techniques use a trial-and-error approach to choose acceptable punishment coefficients, and the fixed punishment compute is used in each run, regardless of the severity of the weight violation or the number of request hubs with weight less than the bare minimum. Variable penalty based on the degree of transgression was proposed by Vairavamoorthy and Ali (2000). They added up the weight deficiency at all hubs where the nodal weight dwindled to meet the bare minimum. Eusuff and Lansey (2003) used the same method to calculate the cost of punishment.

Vamvakeridou-Lyroudia (2001) recommended fluffy thinking in terms of the imperatives care and achievability checks. Rather than punishment, he added up to the amassed enrollment work to the set of doable arrangements and used a non-linear S-shaped enrollment work. He also introduced three imperatives in frame of diameter, velocity and pressure. There were also consolidations of the enrollment capacities for each of these limitations for the inclusion of the genuine expenditure.

2.5 Review of Water Distribution Systems Modeling and Performance Analysis Software

Water distribution networks can be modeled and analyzed using a variety of software programs. There is an open source which is out of charge to use and download for organizations and individuals, while other software programs cannot be used by the public use and other programs are used only commercially by organizations, institutes and individuals. They could be used for the analysis of the evolution of new network systems or for managing the available water distribution systems or use them for research. Each program has its own set of strengths and specialties which can be indicated in the review below.

Water GEMS could be regarded as powerful. It is also inclusive and easy to use water modeling. It could analyze computer programs with the improvements in

systems of optimizations, platform interoperability and model constructing. Water GEMS is a super-set of Water CAD. Water GEMS is an effective and useful modeling program assisting improving understanding of the network behavior, reaction of network to operational strategies and system capability to fulfill the next needs. Its adaptable multi-platform work space permits pipe flow, simulation of water quality, analysis of pressure and energy expense. Its abilities involve managing intelligent system and system reliability planning in addition to system shutdowns, pumping strategies, and preventive maintenance for the reduction of the disruptions for effectiveness of system and asset renewal or innovation supporting sustainability. Its characteristics involve prefixed model structuring, integrating multiple programs, designs and operations. The unique Water GEMS part is the data presentation mode as extremely pleasing and understandable. In addition, the versatility allows presenting different graphical tools. Water GEMS is not an open source or free program but is commercially available. A robust design algorithm constructed the program of Water GEMS to meet the standards of modeling precision, analyzing and designing water distribution systems.

Water CAD is a Water GEMS subtype and can be used friendly for water distribution and analysis program with different assignments assisting the improvement of design productivity. These assignments involve prefixed model structuring, data archiving and bespoke graphical data presentation, water supply system configuration, estimate of water demand, design and running scenarios and finally CAD interoperability i.e. ability to model in any familiar software of choice.

Water CAD construction is based on specific characteristics and abilities that distinguish it from other modeling programs, including the ability to access fire flow capacity, pipe analysis, and valve criticality to construct hydraulic models, manage hydraulic models, and water distribution systems, among other things. It also creates maintenance, flush, and management programs to detect water leaks and manage energy consumption. Water CAD is simple to use and versatile in general. It's also known for its innovative WDS modeling packages.

Pipe Flow Expert is a commercial program available for the design and analysis of complicated water pipe line systems in which the flows and pressures are balance for solving the system. According to pipe flow experts, intuitive user interfaces are easy

to use. They can also model water systems in 2D and 3d isometric presentations. It is a powerful calculation engine solving a complicated pipe network, showing pressure, fluid and pipe data effectively in PDF.

Branch 2014 could be regarded an open source or free of charge water distribution system optimization program structured and progressed for designing branched water distribution systems configurations. It picks accurate input system data like pipe and reservoir elevation, coefficient of friction, length of pipe, and demands at nodes setting expenses as functions. Pipe length and diameters are optimized by the output data to minimize the total system cost.

EPANET is free of charge and an open source hydraulic analysis program which is used for water distribution systems. It comprehensively simulates of hydraulic and water behavior within the system of pressurized water distribution. It aids understanding water travels in these systems. It is a very powerful and multilateral providing workspace for hydraulic analysis in the systems of different sizes. This is through the edition of the system input data. It runs the hydraulic and water quality simulation and views the different choice platform outputs. It is research tool for restoring and developing current and new water distribution systems. It is an EPA invention in the Environmental Protection Agency of the United States of America.

HydrauliCAD is water network design, water modeling, and hydraulic analysis program joined in AutoCAD utilizing industry standard EPANET modeling and reenactment motor. It is an open source computer program and it is amazingly simple for utilization with boundless application bolster. It was built as an AutoCAD expansion computer program and subsequently works with preferences of AutoCAD counting adaptability and recognition. HydrauliCAD was created to calculate pipe pressure, flow, pressure loss and other factors within the supply pipe lines. This program incorporates a pipe properties chronicle which incorporates all the points of interest and data around pipe types, materials and sizes which are utilized in modeling water conveyance frameworks additionally utilized for fire-flow investigation. Hydrauli CAD computer program is outlined and arranged to be solid, aphoristic and quick.

Pipe 2018 is new KYPipe hydraulic modeling software made 45 years ago with nine various engines processing over than 50000 pipes. It is not an accessible source program sold over 5000 licenses. It is an axiomatic and stand-alone workspace which backups a comprehensive package in hydraulic elements. Kypipe could work in third party GIS or CAD workspace with the limits of software benefits. Thus, CAD interoperability could not be associated with this software as it doesn't allow its integration into other familiar workspaces. Yet, Pipe2018 generally is a robust software package as a powerful computational algorithm in fluids such as water. The software could optimize the pump operations and pipe sizing.

Synergi Water is a hydraulic simulation and modeling software package used for modeling and analysis of closed sluiceways framework of channels, valves, controllers, pumps, wells, tanks, stores, and boreholes. A few of synergi water apparatuses include fire stream investigation, the pressure dependent demand, extended period analysis of steady state flow with intense of pumping and other logical controls, dynamic forward tracing and backward tracing, modeling of electric power contract for pump stations and importing and exporting of EPANET models. Synergi Water includes a solid database administration sent for enhancing the adequacy of the water supply pipe lines in presence as well as within the plan and improvement of the unused water supply pipe lines. This program gives multilateral and capable workspace of devices for comprehensive, inclusive and easy water modeling, implements precise and fast analysis, designing of complicated systems with suitable configuration of elements. Its integration with other workspaces is quick and flexible.

H2Onet and H2Omap are commercial essentially having the same work with abilities flexible enough for combination with the GIS and CAD analyzing and designing, modeling and optimization water networks. The deal with leakage detection, cost-optimization and fire-flow analysis, etc. They share preprogrammed and intuitive online SCADA interfaces. H2OMap is a stand-alone GIS software

HYDROFLO3 is a new HYDROFLO series versions, the software is flexible, robust and can used in water distribution system analysis software. It models systems of up to ten sources, nine branches and round about 1000 parts. This means that practically any water distribution system designed by different water distribution system parts

available. It unit conversions between SI and Metric is easy utilizing HYDROFLO3. It has a calibration and effectiveness of water distribution systems, easy changing of hydraulic parameters for all the systems. HYDROFLO3 pump base is used for computing and analyzing pump hydraulic characteristics as a request in pressurized flow systems. HYDROFLO3 is beneficial in employments of many industry, fire-flow analysis etc. It is powerful program for designing water distribution systems with strong capacity of managing data.

InfoWater focuses on huge designing of urban network and modeling of water distribution systems. It designs, calibrates, water quality models, system configuration schedules and system repairs. It is in a full integration with ArcGIS.

It models, analyzes, simulates and optimizes water distribution systems not in the review above including InfoWorks, Eraclito, AFT Fathom 10, WATSYS, Netis, Archimede, Cross, OptiDesigner etc.

The program employment in the water distribution network design relies on the software accuracy and precision. It also involves the total work intense, software - required data, complexity of system, aspect of system to be modeled in terms of demand, location, quality and valve operation. Yet, freeware software EPANET could be used for complex, hypothetical/conceptual on the basis of the activities while WaterGEMS and other licensed software as a Standardized, real and complex work for it flexibility and precision.

2.6 Previous Studies

Jacob in 1991 suggested a new technique to simulate water supply system in the developing countries where the supply does not meet the demand. Walski et al. (2001) studies the issues of water distribution system. He examined the most compelling problems in facing optimization such as the designer striking a balance between cost reduction and net benefits, and water supply network reliability decreases for cost reduction. Yang et al. (2008) solved many of technological problems involving pH regulation, advanced treatment processes, sterilization and pipe selection. Furthermore, published reports the effectiveness and demonstration of multi-parameter water quality sensors and erosion rate sensors eternally located at a U. S. Army installation to discover erosion problems and adjust the chemical

treatment. Water quality and erosion rate sensors had been explained and confirmed in the field (Van and Hock, 2007).

Andrea Bolognesi, Cristiana Bragalli, Angela Marchi and Sandro Artina DISTART at 2009 issued a paper which suggests a new model with name Genetic Heritage Evolution by Stochastic Transmission GHEST, a multi population evolutionary strategy like algorithm used to the design of water supply pipe lines. Al-Hashim and Al-Mansori¹ (2012) displayed in the them study that they issued, the reality of the drinking water in AL-Diwaniya City, it in two essential lines was examined:

Firstly, a study of water distribution in the city trunk network was conducted using software (pipe++) version 1998 to determine the volumes and orientation of water outputs. It also applies head pressure to some network nodes that are connected. Measurements were taken on-site from each zone in order to estimate the head in each trunk and identify the zones that are deprived of water. The distribution network's goal is to provide safe drinking water to every home, public building, and industrial facility. Each residence requires an appropriate supply of water at the desired pressure. [1987, Lencastre] Nilsen (1989) investigated several pipeline network analysis methodologies. The flow equations were reframed in terms of pipe flowrates and energy heads, and the problem was reframed in terms of vectors and matrices by Nilsen. Three separate methods were utilized to tackle the problem: linear theory, Newton-Raphson, and a general-purpose optimization algorithm. The method demonstrates that defining flow resistance in terms of energy heads has two major advantages: it reduces the system size and makes it simple to use computer software to address the problem. The downsides include the need for a strong starting vector and the possibility of slow convergence. Framing the flow resistance in terms of pipe flow rate has the advantage of reducing the number of primary unknowns ($m-n$) to $(n/2)$, where (n) is the number of primary unknowns, but has the disadvantage of necessitating the computation of a basis in order to solve the continuity equations completely. As a basic and reliable starting step, it was confirmed that solving the generalized loop equations using the Newton-Raphson approach in combination with the linear theory method is a useful method for studying pipe networks.

Shibu A. and M. Janga Reddy (2012) presented, potable water supply networks modeling problems linked to massive variables being uncertain because of the

formula used, its coefficients and imprecise knowledge of the values of different factors. In general, nodal demands are unknown, and the uncertainty associated with nodal demands must be taken into account when designing potable water distribution systems. A two-phase methodology based on fuzzy set theory and the cross entropy method has been proposed for the minimum expenditure design of a water distribution system. The pipe diameters for each pipe were chosen using the cross entropy approach from commercially available diameters, and the unknown needs were treated as fuzzy sets. The hydraulic simulation program EPANET tool kit is coupled to the model programmed in the MATLAB software. The proposed methodology was tested on the Hanoi water distribution system, and the results were compared to those of well-known deterministic solutions. It has been proven to be effective in dealing with uncertainties in input elements such as fuzzy sets and discrete diameters, which are dealt with using the cross entropy approach.

To characterize distinct processes and systems, the large majority of the mathematical models in engineering use deterministic methodologies. In one way or another, though, all real-life challenges involve ambiguity. Water distribution network modeling issues are complicated by a large number of factors that are inherently unknown. The uncertainties stem from the formula employed, the coefficients in that formula, and an incomplete understanding of the values of certain parameters.

Abdul-Jabar in 1997 applied a modified finite element method for the solution of pipe networks. This method does not require any initial assumption. Starting point depends upon a selected range of Reynolds's number between "350000 - 550000" depending on the size of the system. Any components can be presented and various pipe properties may be introduced.

At 2000 Al-Samawi presents an approach that is the method of sections in the analysis of potable water distribution system, the program is prepared and written in the "Quick Basic" language in order to perform the analysis operations, and compared this with the Hardy Cross method. The outcomes of that search referred to an agreement improve between the values of the pipe diameters as obtained by the method of sections with those obtained by the Hardy Cross method while the number

of feeding sources into the network raises. It was found that the method of sections produces value of pipe diameters lower than those obtained by Hardy cross method.

Shamrukh in 2005 published a study of applicability of using lateral demands in designing of water distribution networks. He illustrated the design of water distribution systems using demand allocation and traditional methods for friction head loss calculations. A developed methodology for calculating head loss in comparison with both traditional method and uniform withdrawals from the main pipes of water networks was presented. The introduced method assumes a various lateral withdrawals from the water pipes. This assumption was developed to match the reality of urban planning of water distribution systems. More realistic empirical coefficient is introduced for solving the looped or branched water pipes depending on the number of lateral withdrawals. Head loss calculations in water pipes should be modified in case of a limited number of intermediate intersection lateral pipes with the main pipe. Darcy-Weisbach and Hazen-Williams equations for head loss calculations could be reduced by a coefficient equals to 0.5 for intermediate water laterals less than two points. This coefficient is applied in case of inflow to the pipe which is less than two times the withdrawal from the pipe ($Q < 2Q_c$).

In water distribution systems, there are upstream pipes in which the inflow for each pipe is greater than two times the demand from it. For these pipes, the uniform withdrawals equations could be used to calculate head loss due to friction. Moreover, the uniform method can be applied in the cases when the number of lateral withdrawals is more than ten spots regardless the inflow of pipe to demand ratio. The problems of demand allocation to only the nodes at each end of the pipe are believed to be overcome by using this suggested strategy (multiple and relatively uniform lateral withdrawals). Water usage is collected at end nodes rather than at the real locations wherever water is extracted out of the network, resulting in discrepancies between actual field performance and computer-predicted performance. By using this new approach in the design of water networks, there is no need to the process of demand allocation used in the literature design methods. The actual field demands are allocated to the pipe itself instead of its end two nodes. Thus, the developed approach is expected to improve the design and simulation processes of water distribution systems to match its reality (Shamrukh, 2005).

Anisha et al., (2016) published a study of analysis and design of water distribution system using EPANET for Chirala municipality in Prakasam district of Andhra Pradesh. The existed system of potable water supply in Chirala township was intermittent and the network adopted was a dead end system. In Chirala, the system is township which ineffective in the future. As a result, the study looks at the current network and predicts network dependability in the future. It deals with a variety of public demands, as well as the volume of outflows and inflows from elevated reservoirs. It provides wants, losses, and applications. The design of a new supply network will alert the municipality to new demands and the rate at which they are increasing. The design was created with the population growth rate and the developing town in mind. The design improves on the network that already exists. This work included all the main components of the water supply systems and a new design for the water distribution system. The analysis was done with the assistant of the computer program EPANET. EPANET is a program modeling water distribution supply systems. EPANET is public domain software that can be freely downloaded and distributed. EPANET implements extended period simulation of the motion of water and quality attitude within pressurized pipe networks. Pipe networks contain pipes, nodes (junctions), valves, pumps, and reservoirs or storage tanks. EPANET tracks: the pressure at the nodes, the flow rate of water in the pipes, the kind of chemical concentration in the entire network through a simulation period, the height of the water in the tank, water age, source, and tracing. The EPANET input data sheet, which is "produced automatically by MIKE NET," contains descriptions of the physical properties of pipes and nodes, as well as how the pipes in the network system are connected. The water distribution network can be graphically styled by the designer. Easy-to-use dialog boxes are used to enter values for pipe network settings. The EPANET input data file in the format required to execute the analysis is then created by MIKE NET. The interior diameter, length, pipe roughness coefficient, and minor loss coefficient are among the pipe properties. Each pipe has two nodes and a determined positive flow direction. The water demand or supply, as well as the elevation, and pressure or hydraulic grade line are node parameters. As a result they found that the existing system is not suitable for the future demands. The new network was drawn out in accordance with the instructions "In light of the town's planned expansion in the next years, the pattern will be based on a master plan. Two new zones have been designated for the construction of reservoirs to suit

the needs of those zones. The new zones are represented by the overall layout of the municipality (Anisha et al., 2016).

Ibrahim (2015) published a study of design of water distribution system for Kano city in Nigeria by using EPANET software for analysis and design. The demand was met after following studying the population of the study area, and urban settlement area because of the development, the federal ministry standard of water resources manual for water demand was utilized, for the search 180 L/Ca/s was counted. At particular junction, the demand was met through divisions of the total population by the number of junctions and multiplying by 180 L/Ca/D. After that, the following steps were carried out to analyze the state metropolitan water distribution network. The objective and aim of his study was to design a potable water distribution network for Kano Metropolitan. Based upon the population projection, the Based Demand was calculated and the entire network hydraulic was analyzed. The pressure and water head calculated were adequate to withdrawn the actual demand at all the junctions (Nodes) within 24 hours of the Design. The Negative flows in some of the pipes refers to the flow is in opposite direction and all the velocities were within the safe limits as well.

Kumar et al., (2015) published a research article about the design of water distribution networks using EPANET. The study spots the light on the proceedings achieved on water distribution system design for the Kathgarh area with the assistance of the information that they got from the government of Himachal Pradesh the design of the water supply system for the area with the assist of EPANET program. The design of the water distribution system for adequate supply of potable water is effective to meet up with the daily demand of water in the area. After surveying the field and take information about the population and the daily demands, they designed the network according to that. Two 10 horsepower centrifugal pumps are employed in this network. Two overhead tanks are used in storage.

In day time, there are peak hours. In the morning, the demand of water is more as thus, the maximum supply is for 8 hours per day. The various nodes displays variation in the pressures and demand like the demand is more at some nodes comparing to others. Because of their elevation, the pressures at some nodes are higher. For this project, the distribution mechanism is a combination of gravity and a

pumping system. Water is first pushed using centrifugal pumps from an underground water source, such as a well, and then lifted to higher water storage tanks. With the assistance of gravity system is conveyed to the main rising pipe. The distribution layout employed is dead end system or tree system based on the layout of the Kathgarh area (Kumar et al., 2015).

Swamee and Sharma (2000) released an article on gravity flow water distribution system design. A sequential linear programming algorithm Capable of preserving the looped configuration has been involved. The algorithm selects the pipe material and class depending upon available commercial sizes and the working pressure head. In order to reduce the computation time an approximate pipe link diameter equation has been developed for the initial of the algorithm. A criterion for the choice between gravity and a pumping system where elevation difference between supply and demand points is marginal has also been developed.

A Saminu et al., (2013) published an article about design of NDA water distribution network using EPANET. This study presents the use of EPANET software in the design of the Nigerian Defense Academy water distribution network. The software EPANET is research tool to develop the understanding of the motion and fate of potable water elements within supply networks for many various types of applications in distribution networks analysis. This article applied to proceed the hydraulic analysis of the distribution network for of study area. This research analyzes the water distribution network and identifies deficiencies if any in it is analysis, implementation and its usage. At all pipes, the resulting pressures at all the junctions and the flows with their velocities are enough for providing water to the area of the study. The following conditions were noted in the distribution pipe network causing inadequate supply of water to the area in the future are:

1. The pipes which were "connected to the tanks as distribution pipes to the other pipes have smaller diameters".
2. The network on the topographical map have no wider coverage of water distribution in some parts; making water crisis because in future the area could expand.

The results confirmed the pressures at all nodes and at all pipes the flows and velocities are appropriate enough to provide sufficient water to the network the location in this study (Saminu et al., 2013).

Simpson and Elhay (2008) almost defining the water distribution framework conditions of water in terms of heads and velocity. Within the conditions for reframing, the conditions of the pipe organize in terms of head and speed based on the detailing of Todini and Pilati (1988) have been provided, building on the initial induction of the head and stream conditions. Commonly velocities are within the run of 0.5 to 10 m/s with as it were some values higher of lower than this this extend. This work hypothesized that a velocity-head formulation of the conditions depending on the initial Todini and Pilati strategy may lead to a more effective iterative strategy and after that be quicker and illuminate in less cycles than the stream detailing as a result of the the much littler extend of conceivable likely velocity values in differentiate to the conceivable run of streams. Todini and Pilati tested flow-head and the velocity-head definitions for a gather of arbitrarily made systems of various sizes resulting in a number of iterations for the convergence being almost similar. So, the velocity-head formulation provides no improvement in convergence. Also, the results of various starting values of velocity were employed for the investigation of the Todini and Pilati flow-head formulation. It is sensitive to the first velocity estimate for all pipes in the network. A 0.3 m/s is too low because the beginning of the estimate and the results indicate a primary estimate initial between 0.5 and 1.1 m/s as appropriate.

Rushikesh et al. (2020) have published a review paper about design and analysis of water supply networks. The study was based on evaluation of the existing water supply system with the assist of EPANET 2.0 software. The network of pipe and node system could realize its behavior in different inputs with the application of EPANET 2.0. The hydraulic parameters such as pressure, head, and flow rate have been accomplished in the simulation. The findings show that the pressures at each node, as well as the flow rates at all linkages, are sufficient to supply enough water to the network. This work first investigate the water supply system and to monitoring the flaws (if any) in the investigation, establishment and its use. When the process of

analysis ends, then the out putting pressures of the nodes, flows and the velocities became appropriate.



CHAPTER III

A BACKGROUND ON GRAVITY WATER DISTRIBUTION SYSTEM

3.1 Introduction

This chapter presents the main components of this kind of networks, the function of each part of them, necessity and location of those parts. Then, the essential hydraulic basics which achieve the behavior of the water in gravity water distribution networks explained. The principles of hydraulic modeling are mentioned as well. Finally, the benefits of using gravity water distribution systems and disadvantages of this type networks.

3.2 Components of a Gravity Water Distribution System

The most important components in a gravity system are; an elevated water source, intake, storage tank or reservoir, a sedimentation tank if it is necessary, break pressure tank, distribution pipelines, valves and accessories. Below some brief summary of each part is provided.

3.2.1 Water Source

The sources of water that most used in gravity systems are natural springs and small streams. A reservoir can be a source for the mixed or combined systems. Also rivers, ponds and lakes could be used to provide the system with water. The place where underground water comes to the surface called a spring. It may be at the side or edge of a mountain or a hill. The water flows gradually and slowly through the different layers of the soil and filters until reaching an impermeable layer, thus water flows out forming a spring.

The major important point in spring case is that the water is filtered and purified on its journey until blowing out of the ground surface so it doesn't need treatment. In another word, the costs of constructing the system will be less and complex logistics

are avoided, and errors in chlorine dosage is avoided. Unlike the spring water, surface water resources like streams and river branches are considered contaminated and need treatment.

The easier way to measure the flow of the stream is having a known volume container and a timer or watch. For big streams where the flow is big the V-notch weir method is very practical, by placing a V notch weir across the stream and letting water flows through it. The height reached by the water is proportional to the flow, (Arnalich, 2010).

The flow can be determined by applying the following formula:

$$Q = 533 \times C_e \sqrt{2g} \times h^{2.5} \times \tan\left(\frac{\beta}{2}\right) \quad (3.1)$$

Where Q is flow of water in L/s, C_e is a coefficient (normally $C_e=0.64$), g is gravity = 9.81 m/sec^2 , and β is angle of the weir in radians², h is the height of water in meters.

3.2.2 Intake

The first component of the supply system which gathers and enters the water into the system, it depends of the type of the water resource, so each case needs a particular work and design, thus there is no standard design that can be iterated for all cases. In addition to the main function for the intake works which is collecting water, the intakes must perform some other functions like reducing the amount of suspended materials; a well-designed intake can perfectly cancel the need of subsequent treatment. And keep the water out of contamination, protecting it from animals, manipulation and any kind of risks.

For the springs the location of the intake is where the spring is raises, for the situation of the stream there are many potentials, taking in the consideration that the intake should be reachable all the seasons, the location has to be protected from damages of mud slides or floods. The rocky sides of the stream with low velocities are less potential for erosion. The area of the intake must be upstream of the contamination. And have advantage of pools and narrow stretchers so laying foundations being plain and easy.

3.2.3 Storage Tank

Usually the capacity of a storage tank is that enough to provide the village with the required demand of water for 24 hours. Thus, it designed to store water during the night to fulfill the demand at peak hours during the day.

3.2.3.1 Possible Location of Storage Tank in A Gravity Water Supply System

The following possibilities of popular layout for a simple rural water supply gravity system.

A. Locating the storage tank at the use point.

General statements on the layout are as presented in Figure 3.1:

- The inflow to each reservoir can be regulated so that each area receives a set of proportion of water. If the people at each reservoir tend to waste water then they can.
- Only waste their portion and not other consumers' proportions. In a public fountain system having storage tanks at the source, wastage would be maximal if the taps left open.
- The small tank serves out as break pressure tanks in the network. This means that taps have only the head of the tank itself at the point of use, and will last for a longer period of time because of the reduced pressure. Reduced head at the use place reduces wastage as well.
- In this case the main distribution pipe is in use all the time, as a result smaller diameter could be used for these pipes and this reduces the costs. The impact on total cost will be determined by the source's flow compared to typical daily demand, as this will affect storage costs.
- When constructing little scattered reservoirs rather than one huge distant reservoir and fountains, it is sometimes simpler to get community reports, nurture a sense of ownership, and so improve maintenance. Furthermore, the construction of tiny reservoirs allows each component of the community to work at its own pace throughout construction, and the project will be less likely to be hampered by a lack of community engagement.

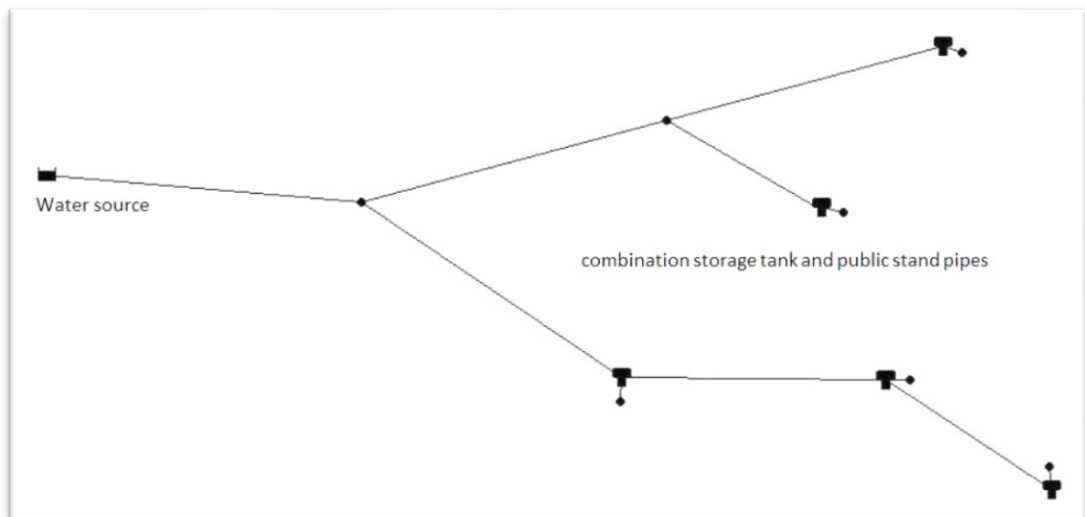


Figure 3.1 Placing the storage tank at the point of use.

B. Placing storage tank between source and distribution points.

This system has many beneficial features, so the general features on the layout are as shown in Figure 3.2:

- The pipes used to the place of storage are small in diameter.
- The tank works like a break pressure tank in the system and could be placed to regulate the pressure at the distribution area.
- Just a single storage tank (larger size) is needed to be built and maintained.
- Treatment and improving the water quality at one center point.
- Public fountains can be easily installed at the wanted distribution point.

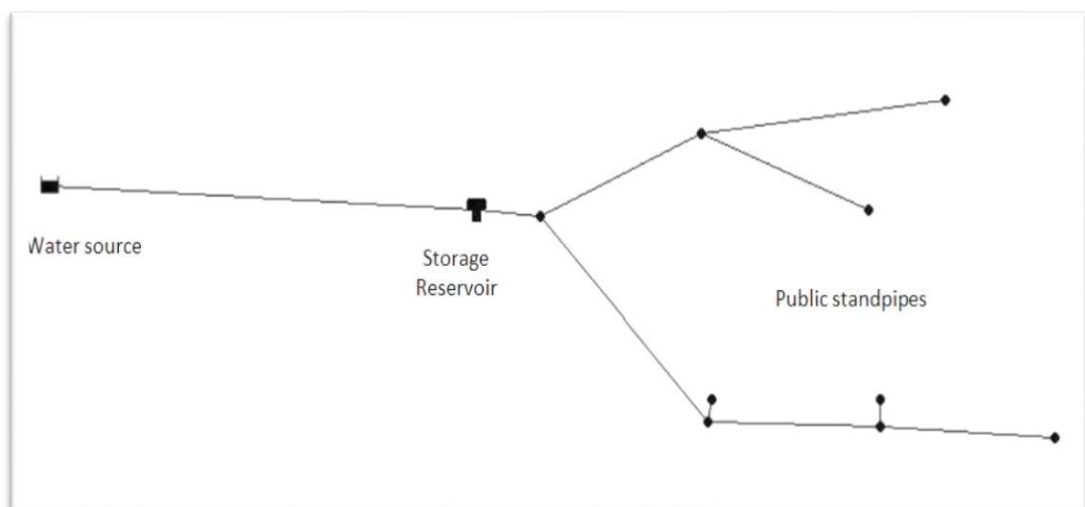


Figure 3.2 Placing the storage tank between the source and distribution.

C. No storage, and distribution is direct to public fountains

General features on the layout are which detailed in Figure 3.3:

- This type of system necessitates a larger diameter pipeline, which raises the overall system cost significantly.
- In order to meet peak demand without requiring storage, this system also requires a source flow. Means the safe yield of the spring should be high so that it can yield the maximum daily requirement of the people within 8 -12 hours.

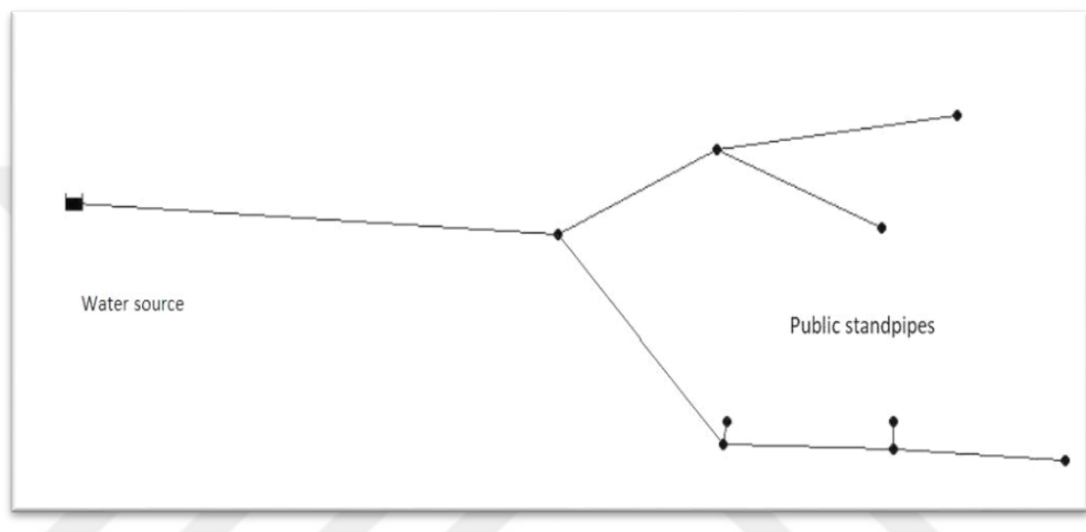


Figure 3.3 No storage tank and distribution direct to the fountains.

D. Storage tank is near or at the water source, then distribution is direct to public fountains

Figure 3.4 presented the general features on the layout are:

- This type of system necessitates a larger diameter pipeline, which is often the most expensive choice; however, storage may be included with the intake construction and prove cost-effective for small systems in some situations.

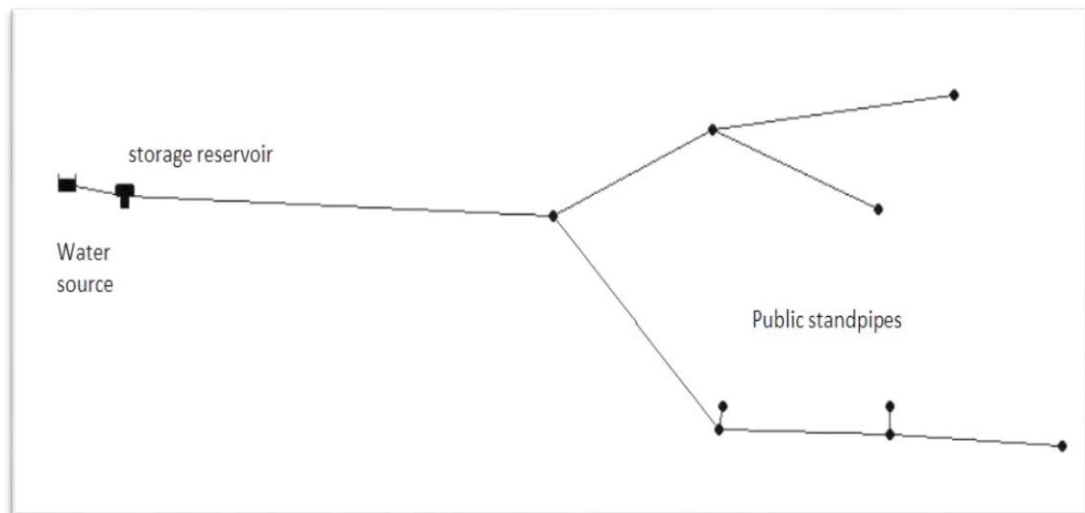


Figure 3.4 The storage tank is near the water source.

3.2.4 Break Pressure Tank

A break pressure tank is an effective technique to break the pressure in the water supply systems. The purpose of these small tanks is for allowing water to discharge into the atmosphere, lowering the piezometer reading to zero, indicating zero hydrostatic pressure, and setting a new static level. It usually applied in rural areas then the maintenance will be minimal, and that is the reason behind preferring break pressure tank rather than pressure reducing valves, in addition to having a higher turn down ratio than pressure reducing valves. Even if the break pressure tank fails, the downstream pipeline will never be exposed to excessive pressures. A break pressure tank has fewer components than a conventional pressure reducing installation. These parts are wanted to be installed when the elevation of the pipe is sufficiently below the water source to exceed the pressure capacity of the pipe. Break pressure tanks could be built from concrete, sheet metal or masonry, with rectangular or circular shape.

3.2.5 Sedimentation Tank

The surface water sources usually contain suspended particles and have turbid appearance. These suspended particles are problematical in terms of the taste as well as the smell. And through time these materials sediment and accumulative in the pipelines in the low points decreases the effective diameter. Sedimentation tanks are required to reduce the amount of suspended materials, it allows the water to settle for

a while then the particles settle down and the water will be clearer and allowed to continue its journey into the system.

3.2.6 Pipelines

The main component in the water distribution system is pipes which convey water from/to different parts in the system and transport the required quantity to the consumers. It's important to understand the hydraulic theory of design the pipelines.

3.2.7 Hydraulic Theory of Pipeline Design

The goal of pipeline design is to determine the appropriate pipe size and, as a result, to effectively address frictional energy losses in order to shift the intended flows through the system by preserving energy at some locations and burning it off (via friction) at others. This is accomplished through careful pipe size (diameter) selection and strategic placement of break pressure tanks, control valves, reservoirs, and tap stands, among other things.

3.2.7.1 Energy

Water transport requires energy, whether moving it uphill, downhill, or horizontally. The energy source in a Gravity Flow Water System is gravity's action on water, as the name implies.

Gravitational energy is used to power a gravity flow system. The relative heights of all sites in the system determine the amount of such energy in the system. All locations in the system are immovably fixed (buried into the ground) and their relative altitudes cannot alter once it is built. As a result, in each given system, there is a definite and exact amount of gravitational energy available to move water. While water moves through pipes from one place to another and via couplings, valves, and tanks, friction dissipates some energy.

The topographic profiles of the systems change, and as a result some points may be a minimum value of energy which means low pressure, however other points there might have too much amounts of energy which means high pressure. A badly designed and built network may not conserve energy fully enough to transport the designed quantities.

3.2.7.2 Head (the measure of energy)

Instead of repeatedly calculating water pressure in hydraulic operations, it is more convenient to just announce the comparable height of the water column. This is known as the head in technical terms, it denotes the quantity of gravitational energy held by the water. Head is always measured in meters in the metric system of measurement. A water pressure of 120 kN/m^2 equals 12 meters of head, 400 kN/m^2 equals 40 meters of head, and 600 kN/m^2 equals 60 meters of head, and so on.

3.2.7.3 Fluid Static (the water at rest)

Anyone who had dived to the bottom of a lake or pool knows that water pressure grows as one descends, but swimming horizontally at a constant depth has no effect on the pressure. This regular event highlights a fundamental concept in hydraulics:

"Water pressure at some depth is directly related to the vertical distance from that depth to the level of the surface, and is not affected by any horizontal distances."

Take a look at the system depicted in Figure 3.5. The depth of the water at point A determines the water pressure there. The pressures at sites C and B are also controlled by the vertical distance between those spots and the water's surface level.

Table 3.1 Water pressure and heads of fluid static at different locations.

Point	Water pressure	Head
A	100 kN/m^2	10 meters
B	200 kN/m^2	20 meters
C	350 kN/m^2	35 meters

Static equilibrium refers to a system in which no water flows through the pipelines. The pressures in this system are communicated as static heads because the surface of the water in this system is known as static level. The control valve is closed and no water is flowing through the pipes in Figure 3.5. If little open ended tubes were put into the pipeline, the water level in all tubes would rise to the static water level. The height of water in such a tube is determined by the pressure head put on the pipeline at that point. No energy is lost due to friction because the water is flowing, and the static level is perfectly horizontal.

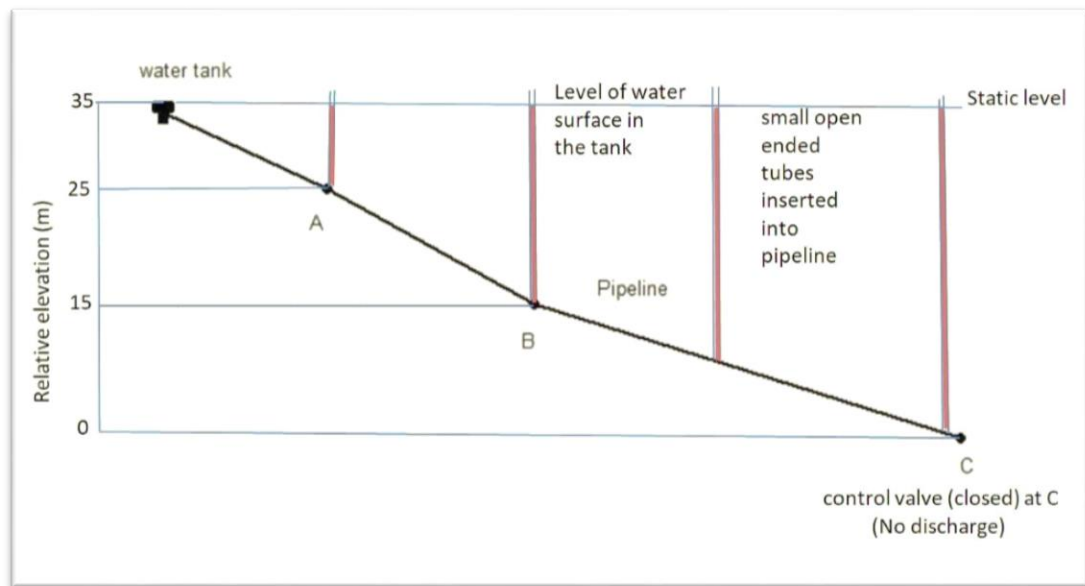
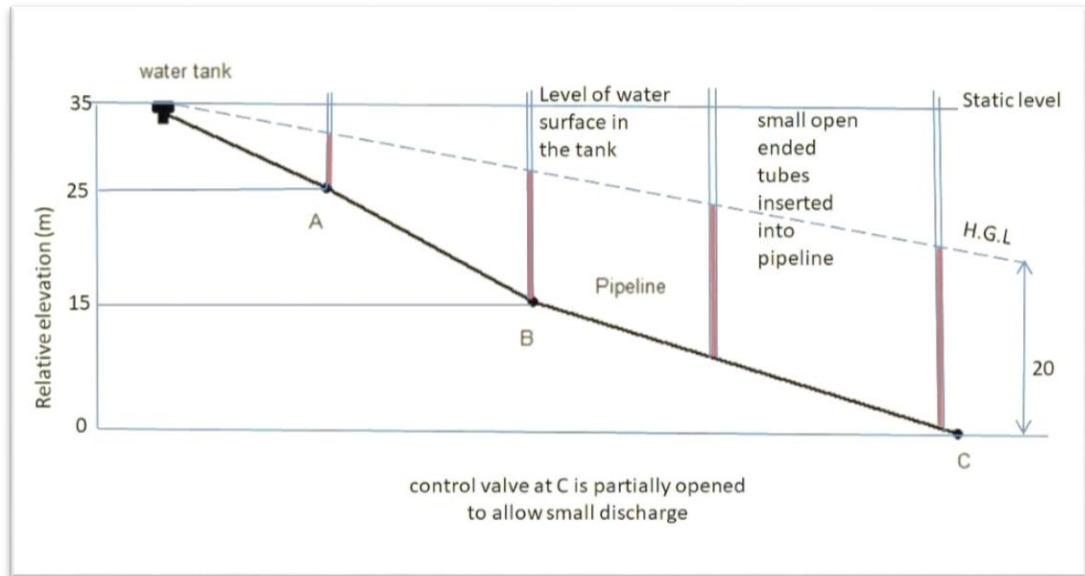


Figure 3.5 Static equilibrium.

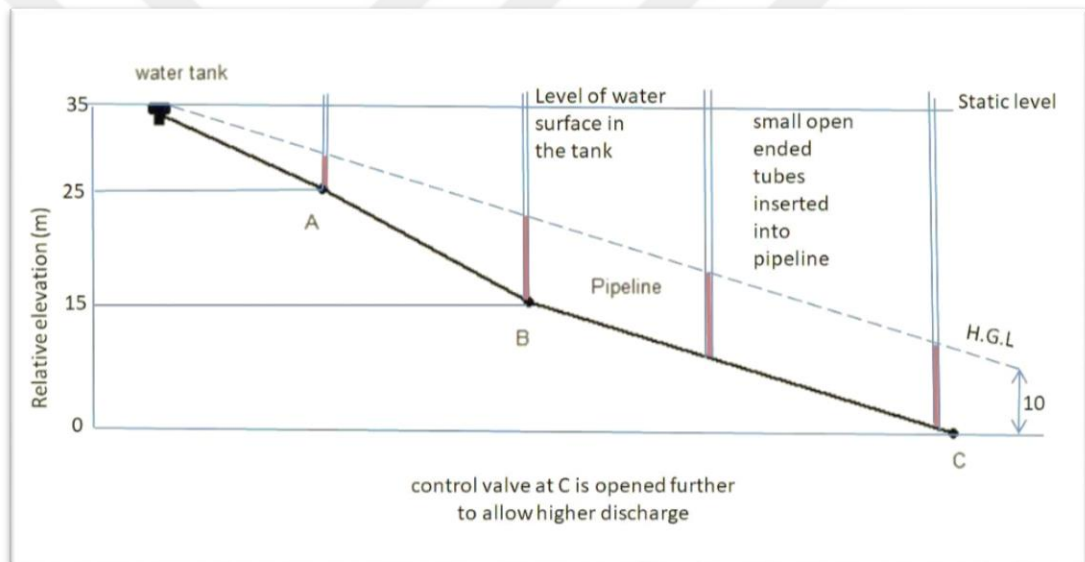
3.2.7.4 Fluid Dynamics (water in motion)

Supposing that the control valve located at point C is partly opened as detailed in Figure 3.6a, and allows a tiny amount of water to flow through the pipes (at the same time assume that the tank refills as fast as it drains, as a result the surface level remains constant). The water levels in each vertical tube decrease a little bit. As the valve is opened more and more to allow greater flows through the pipeline, the water levels in the tubes drop even lower, as shown in Figure 3.6b.

The water levels in these tubes form a new line with each fresh flow through the system, as can be seen. The line produced by the water height will remain constant under a constant flow. The system has reached a state of dynamic equilibrium. The hydraulic grade line, abbreviated as HGL, is the line produced by the water levels in the tubes. The energy level at each location along the pipeline is represented by the line. Distinct flows result in a new HGL and a different dynamic equilibrium.



(a)



(b)

Figure 3.6 Dynamic equilibrium, (a) Partial opened, (b) Fully opened .

3.2.7.5 Hydraulic Grade Line (HGL)

The HGL is a representation of fresh energy levels at various places along a pipeline. There is a specific, constant HGL for any continuous flow through the pipe. The quantity of pressure head (i.e. - energy) is determined by the vertical distance between the pipeline and the HGL, and the difference between the HGL and the static level is determined by the quantity of head lost due to flow friction. At the air/water interface (such as the surfaces of water in tanks or discharges at tap stands),

there is no water pressure. As a result, whenever water comes into contact with the atmosphere, the HGL must always be zero.

The HGL constantly slopes down along the direction of the flow because friction losses are never recovered. The rate at which energy is lost to friction determines the slope's steepness. It should always be above the pipeline, but if the pipe line rises, it is possible for the HGL to fall below the pipeline. This indicates the presence of negative residual head, this means there is not enough gravitational energy to move the desired quantity of water. This situation should be avoided and should be checked for in the design stage. In addition, under static conditions, the HGL is absolutely horizontal, albeit it can be plotted as horizontal for practical purposes in huge pipes with extremely low flows (where the head loss is less than 0.5 meter for 100 meters of pipeline). In practice, the HGL will never incline upwards.

3.2.7.6 Friction (lost energy or head Loss due to friction)

A system has a definite amount of gravitational energy, which is defined by the relative heights of the system's points, as stated at the beginning of the chapter. The friction of the flow against the internal walls of the pipes, or through fittings (such as reducers, elbows, control valves, etc.), or when it goes in or out of the pipes or tanks, causes energy to be wasted when the water is traveling through the pipeline. Friction losses occur when there is a partial obstruction to the flow or another barrier.

A head loss is the reduction of pressure in a pipe which may be due to friction in the pipe and pipe fittings, or valves and can be expressed as a change in head. The amount of energy lost because of the friction against an obstruction is determined by a number of parameters, the most important of which are the roughness of the impediment and the flow velocity. The temperature of the water, suspended particles, and other small parameters are examples of minor influences.

The velocity of the flow is controlled by the pipe sizes and the amount of flow via pipes. When the flow is high, the velocity is higher, resulting in higher frictional losses. Similarly, the rougher the obstacle's surface, leads to greater friction losses. The frictional losses aren't always linear: double the flow doesn't always double the losses; in most cases, losses are tripled, quadrupled, or even larger. The Hazen - William's formula is widely used in designing of the water distribution systems.

The head loss due to friction in the pipe is calculated using the Hazen - Willam's Formula as follows:

$$h_f = \frac{10.7 L Q^{1.852}}{C^{1.852} D^{4.87}} \quad (3.2)$$

Where, h_f = Head loss due to friction in meters, L is represents the length of the pipe in meters, d is strands for the internal diameter of pipe in millimeter, Q = Discharge in L/sec, C = Friction coefficient.

Losses in fittings and valves (Minor losses)

The head losses caused by a few connections and valves used to regulate public fountains and reservoirs are not considerable. However, if a high number are utilized throughout the system to regulate flow, or if the pipeline contains a significant number of bends and joints, head losses will be increased. In general 5% (preferably) to 10% of the head loss in the pipes can be added to account for the minor losses.

Frictional head loss factors

The common method in head loss calculation is to calculate the head loss using the frictional head loss factor (f). The frictional head loss factor is expressed as "meters of head loss per one meters length", or as "m/1m". Nomograms or tables are often used in solving pipe flow problems involving the Hazen - Williams's formula which shows the frictional head loss factor (f) for different flows and pipe diameters.

Residual head (excess energy or free head of available head), (Fh)

Before an appropriate system can be designed, the designer must first understand the meaning of residual heads at tap stands, break pressure tanks, and reservoirs. Residual head is the amount of energy left in the system after the desired flows have reached the discharge destination. It signifies a gravitational energy surplus. The residual head at the discharge point might be positive or negative when charting the HGL for a flow that discharges freely into the atmosphere (such as into a tank or out of a tap), as shown below.

Positive residual head

This indicates that there is an excess of gravitational energy, that is, there is enough energy to move an even greater flow through the pipeline, if allowed to discharge freely. A positive residual head means that gravity will try to increase the flow through the pipe. As flow increase, the frictional head loss will decrease the residual head. The flow will increase until the residual head is reached zero which presented in Figure 3.7.

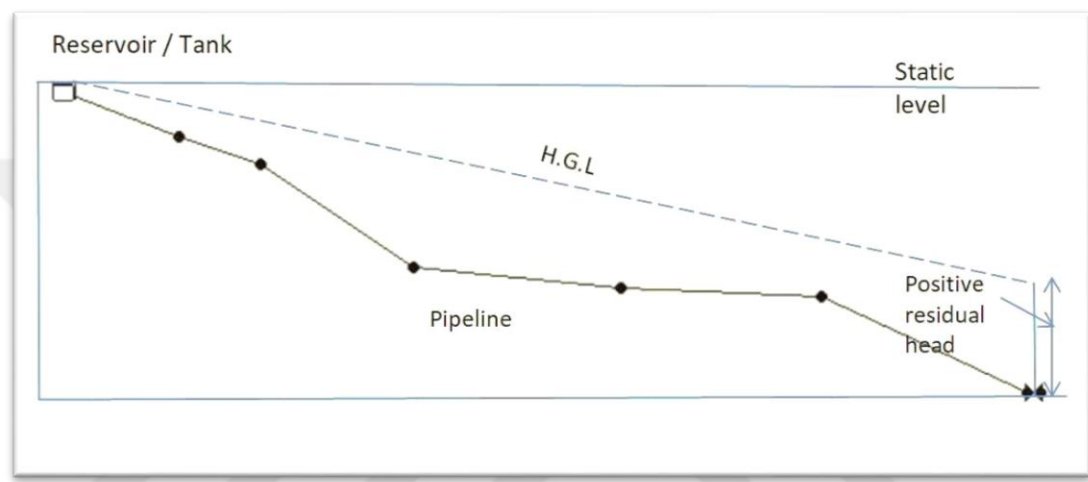


Figure 3.7 Example of positive residual head.

Negative residual head

This one shows that there is insufficient gravitational energy to transport the appropriate amount of water, hence it won't flow. To reduce head loss due to friction, the Hydraulic Grade Line (HGL) should be changed with a smaller flow and/or bigger pipe size. See Figure 3.8 for more information.

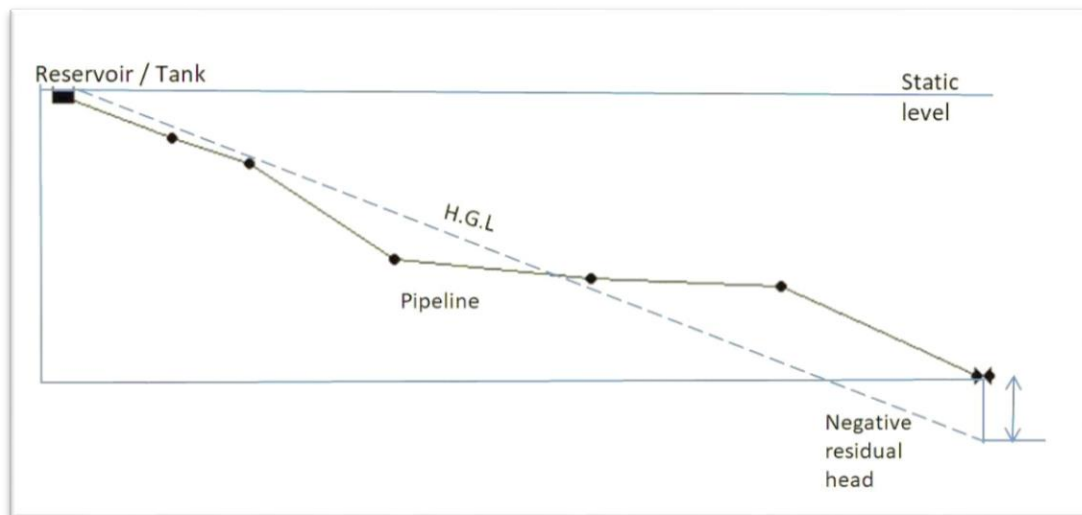


Figure 3.8 Example of negative residual head.

In public fountains or at any point in a pipe line the excess/free/residual head is the difference in meters of water between the reservoir or break pressure tank or source elevation, and the sum of the elevation of the point and head loss in the pipe from the reservoir / break pressure tank/source to the point. Thus, free head at any point, F_h is elevation of reservoir or source minus elevation of the point minus sum of head losses in the pipe from the reservoir or source or BPT to the point.

3.2.7.7 Maximum and Minimum Pressure Limits in Pipes

As previously mentioned, pipe diameters are chosen based on frictional head loss considerations. However, there is another factor to consider when deciding which pipe type and diameter to use. Pressure is a factor to consider, and will dictate whether Polyvinyl Chloride (PVC) or pipes made of galvanized iron (GI) to be used. The maximum pressure that the pipe will be subjected to determines the option (these maximum pressures are always the result of static pressure levels). The suggested maximum pressure limitations for each of these pipes are listed below.

PVC pipes - maximum pressure rating=10 bars = 100 meters of water column.

GI pipes - maximum pressure rating = 25 bars = 250 meters of water column.

3.2.7.8 Velocity Limits

Another matter of consideration in pipe line design is the velocity of the flow via the pipes. Suspended particles in the flow may result in an excessive erosion of the pipe,

in the case of high velocity, unlike the case of low velocity where the same suspended particles might settle down out of the flow and accumulate at low areas in the pipeline, and at the end clogging it if it has been ignored.

3.2.8 Valves

A specific number of valves should be installed on the network to achieve various roles and they should be located in boxes to protect them from damage and easily accessible. To assure a good network operation, it is necessary to install control valves at the entry of each tapstand, break pressure tank and storage tank. The control valves regulate the flow by adjusting the maximum opening. It would be necessary in the case of the spring water flow being higher than the desired flow in the pipeline. Also open/close valves should be installed to separate different sections of the network from each other. This type of valves allows stopping the supply in certain zones for maintenance purposes. Open/close wash out valves installed at storage tank, break pressure tank and lower points in the system.

3.3 Hydraulic Modeling's Basic Principles

The water distribution system is counted as one in which all the parts are connected to each other in the hydraulic simulation modeling. Each component affected by its neighbors, and all components are consistent with the condition of the other components. These conditions are basically commanded by two laws: "Law of Conservation of Mass and Law of Conservation of Energy". "So the total mass of water entering the system should be equal to the total mass of water leaving the system, and the sum of the flows at any given node or junction should be equal to zero. The principle of conservation of energy is essentially dictated by the Bernoulli's equation, which states that the difference in the energy between any two points should be the same regardless of the path taken" (Haestad Methods, 2003).

An ideal network in hydraulic model made of the following elements:

1. Nodes or junctions connecting the pipes.
2. Pipelines.
3. Storage tanks.
4. Water pumps.

5. Reservoirs.

Supplemental accessories like valves. The junctions or nodes symbolize spots with specific base demands. While the tanks may have particular capacity for storing water and can varies with time. The sustainable water source in the system can be presented as reservoirs in the hydraulic modeling. Furthermore, the energy devices in which supply pressure and head to the water represented by pumps. The graph of head vs. flow for a particular pump is called the 'pump curve', which is not included in the gravity water supply systems (Rossman 2000).

3.4 Laws of Conservation

In spite of the fact of the implications of the waters characteristics are massive, there is no list of the water properties will characterize a physical issue entirely. Whether, it is concerned with quality of the water in the tank or transient status in the pipe. Natural events are governed by a set of physical rules, which influence the character and nature of a system's response. If quantitative predictions are to be made, it is necessary to first comprehend the physical problem and the mathematical rules that model its behavior. Fundamental physics laws should be understood and used to a wide range of applications in a variety of settings and habitats, ranging from pump flow to passage conditions in a pressurized channel or a pipe. The derivations of these equations, on the other hand, are not published because they are generally available and require a significant amount of time and effort to complete precisely. Rather, the laws are presented, examined, and summarized in the context of the pipeline. A quantitative description of fluid behavior, more precisely, necessitates the application of three basic relations: a kinematic relation derived from the law of mass conservation in a control volume, and equations of motion derived from Newton's second law and the energy equation.

3.4.1 Mass Conservation

This part introduces one of the most basic and strongest tools. The continuity or mass conservation equation is the central concept of mass conservation, and it is its key formulation. It's vital to realize that not everything changes when it comes to physical systems. The majority of physical laws are conservation of mass assertions: broad statements concerning regularities that occur in the midst of change (Karney, 2000).

A conservation law is a declaration of nature's consistency, particularly constancy throughout change. If a quantity can be established for an isolated system that remains precisely constant independent of changes inside the system, the quantity is said to be perfectly preserved. A number of physical quantities that are conserved in the meaning of Ford's quotation have been discovered. Energy (assuming mass is taken into consideration), momentum, charge, and angular momentum are all examples. Both nuclear and chemical reactions are included in one key generalization of the law of mass conservation (Hatsopoulos and Keenan, 1965).

It is required to account for the pure quantity of fluid flowing into the pipe as fluid storage inside the pipe when applying the concept of conservation of mass to a system with flow, such as a pipeline. Any mass imbalance (in other words, net mass exchange) will result in significant pressure changes in the conduit due to compressibility effects. So the inflow = out flow, since both of the inflow and outflow could happen at many points, as explained in Eq. (3.3).

$$\sum_{inflow} V_i A_i = \sum_{outflow} V_i A_i \quad (3.3)$$

Where A and V are area and velocity respectively.

3.4.2 Second Law of Newton

The concentration is on one element of chemical species, when the mass rates of flow are concerned. However, when a physical law is presented, such as Newton's law of motion, we receive something even more profound: a connection between the seemingly unrelated quantities of force and acceleration. Newton's second law, in particular, links changes in fluid or stiff motion to the forces that create the change. Then, comes the statement that the sum of all external forces, including evolving body forces, operating on a system equals the system's rate of change of momentum over time. This can be stated numerically as:

$$\sum F_{ext} = \frac{d(mv)}{dt} \quad (3.4)$$

Here F_{ext} is the external forces working on a body with mass (m) traveling with velocity and t indicates time (v). When the body's mass remains constant, Eq. (3.4) becomes:

$$\sum F_{ext} = m \frac{dv}{dt} = ma \quad (3.5)$$

Where a denotes the system's acceleration (the time rate of change of velocity). The product of hydrostatic pressure, the weight of the fluid, and friction are the major forces to consider in closed conduits or pressurized pipelines. To produce the net acceleration, these forces operate at all points along the pipe. When these forces and fluid motion are mathematically described, the result is a "dynamic relation" that describes the pipeline's transient reaction.

If flow parameters at a particular place do not change with time for a control volume, the stable form of the momentum equation can be expressed as:

$$\sum F_{ext} = \int_{cs} \rho v(v \cdot n) dA \quad (3.6)$$

The force expression represents the net external force acting on the control volume, while the right hand term represents the net flux of momentum through the control surface. The integral is taken across the entire surface of the control volume, and the integrand is the incremental quantity of momentum exiting the control volume. The flow can normally be considered to be incompressible and uniform and the control surface can be inclined perpendicular to it. The momentum equation can be simplified even more using this assumption, as shown below:

$$\sum F_{ext} = (\rho Av)_{out} - (\rho Av)_{in} = \rho Q(v_{out} - v_{in}) \quad (3.7)$$

Q is the volumetric rate of flow.

3.5 System Capacity

The water supply network or transmission pipeline is an entire system conveys the water due to gravity or by adding pumps to the system conveys water from the sources to general neighboring of the demand. Usually, the system is made of pipes or open channels with the related works, valves, pumps or other elements. A transmission system is generally composed of a single series line, in contrast to the distribution system, which is frequently made up of a complicated network of interconnecting pipes. As previously said, there are numerous questions that the designer of these systems must address, such as whether the system (reservoirs, pipes, or pumps) is appropriate in capacity. Is it possible to manage the flow to

achieve the necessary hydraulic conditions? Is the system financially viable? Is the strength of the connections and pipes sufficient to withstand both fluctuating and constant pressures?

Various classes of modules are used to address the questions above, depending on the nature of the flow and the justified approximations. Hydraulic capacity challenges are normally addressed by projecting demands (water requirements) and then analyzing the system under steady-state conditions. To size and choose the principal pipes in the system, the designer analyzes the best available predictions of future demands. The effective diameter of the pipeline that connects the supply and demand determines the hydraulic capacity of the system. The gradual change in demand over relatively small time periods, such as an average day or a maximum day, is used to answer issues about the operation and sizing of pumps and reservoirs. Because the fluid's acceleration is generally negligible, analysts utilize a quasi-steady approach: That is, they calculate forces and energy balances using steady flow, but the continuity equation is written in the unstable form so that flows can be accumulated and stored. Finally, consider transit conditions, such as the pressure rating of the pipelines and fittings, when determining the needed strength. As a result, the pressures generated by a quick change between flow modes play a role in determining the pipeline's strength. Because high forces and harmful pressures can occasionally be generated during this phase, short-term and quick motions should be considered. Forces and accelerations are balanced here, and mass flow rates and pressure changes are balanced.

Pipeline systems are subjected to a wide range of flow conditions. Those circumstances are frequently grouped according to many criteria to make the analysis easier. The flow can be classified based on channel material features, geometry, dynamic considerations (both kinematic and kinetic), or a few other distinguishing characteristics of the flow. The flow can be classified as open-channel, pressured, or gas depending on the fluid type and channel geometry. The most essential distinctions are mostly certainly dependent on flow dynamics (i.e., hydraulics). Flow can be classified as steady or unstable, uniform or nonuniform, turbulent or laminar, compressible or incompressible, single phase or multiphase in this fashion. For the

analyst, all of these distinctions are critical because they decide which physical laws and material qualities are dominant in any given application.

- Steady flow is defined as a flow in which the conditions at a given point do not change over time. Aside from that, the flow is erratic or temporary. All turbulent flows, and thus most engineering-relevant flows, are technically unsteady according to this definition. As a result, a more narrow definition is frequently used:

If the temporal mean velocity of a flow does not change over short periods of time, it is considered steady. Transient conditions represent a "abnormal," or non-equilibrium, transition from one steady-state flow to another, despite the fact that the assumption is not formally required. The initial conditions in transitory situations are normally believed to be steady unless otherwise stated.

- In a pipe system, steady or equilibrium circumstances refer to a balance of physical laws. Equilibrium is characterized by a constant, uniform flow in both closed and open channels. The rate of fluid input to any portion is equal to the rate of outflow in these applications, the external forces acting on the flow are balanced by changes in momentum, and external work is compensated by mechanical energy losses. As a result, the fluid flows down an energy gradient, which is commonly represented as flow in the direction of decreasing hydraulic grade-line elevations.
- Flow that is almost constant when the flow becomes unstable or unsteady, the model that must be utilized is determined by the rate at which the changes occur. The rate of fluid acceleration is insignificant when the pace of change is generally slow, usually over hours or days. Reservoirs, on the other hand, will collect or deplete fluid, and water demand rates may gradually adapt. As a result, it is possible to employ a quasi-steady or extended-duration simulation model.
- Compressible and incompressible: A flow is considered to be incompressible if the density of the fluid ρ remains constant throughout time and throughout the flow field. In an incompressible flow, ρ is not a function of position or time. The flow is compressible if density changes are permitted or required.

- **Surge:** A surge model is commonly employed when the rate of change in flow is moderate and occurs over a period of minutes. When the following assumptions are made, the expression surge in North America suggests an examination of unsteady flow conditions in pipelines: The fluid is incompressible (i.e., its density remains constant), and the pipe walls are rigid and do not deform or distort. Because of these two assumptions, fluid velocities are not a function of position along a pipe of constant diameter, and the flow is uniform. In other words, as the pressure changes, no more fluid is stored in a length of pipe, the fluid input to the pipe is always equal to the outflow discharge since velocities are uniform. A surge model, on the other hand, accounts for the fluid's acceleration as well as its buildup and depletion from reservoirs.
- **Water hammer** occurs when a rapid, unstable flow occurs in a closed conduit system, and the temporary condition is sometimes accompanied by a pinging or pounding noise. Even if no audible shock waves are produced, this expression is commonly used to apply to all quickly changing flow conditions. In water hammer models, the fluid is commonly assumed to be slightly compressible, and the pipe walls bend as the internal pressure changes. Water hammer waves travel at a finite pace in the pipeline, equivalent to the speed of sound.

3.6 Advantages and Disadvantages of Gravity Water Supply Systems

Every system of water distribution has its advantages which encourage to using the system; also it may have some weak points which have to be noted. Following some advantages and disadvantages of the gravity water distribution system:

In gravity flow water system there is no need for energy source to operate the system and water transports just by the gravity, so there is no need for pumps. Thus the maintenance is very simple and easy to do. The water is delivered to consumer houses or near to them. Sufficient amounts of water can be delivered to cover all the needs of the people in the area with a better quality than the traditional water source. Gravity flow water systems could be constructed by the village people themselves, if trained project supervisor is provided. Systems of different sizes can be built for

different members of people. Small systems can serve single households and large ones will serve many villages or towns.

The disadvantages of gravity systems can be lie in the following points:

In such systems the quality of the provided water depends upon the source water quality, when the sources water is not clean and adequate for humans use then a treatment facility can be constructed with further expense. The accessible water sources in the area may not sufficient to cover the water demand during different seasons. The water flow of the system which should be taking into account is the lowest flow rate recorded during the year. Gravity water supply systems require regular basic maintenance; otherwise the system will fail at the end if the arrangements for it didn't proceed the maintenance. Finally water rights bring out problems in some rural areas like villages near the source may protest to having "their" water piped to villages downstream theirs. Pipelines may be damaged in local disputes.

CHAPTER IV

THE METHODOLOGY AND MATERIALS

4.1 Introduction

In this chapter all the methods used will be illustrated and the materials which have been used. A brief background will be given with all abilities and disability of the program EPANET which used in the modeling, data collection and analysis and presentation, survey of the area, computation of different water demands according to the population of the village and future water demands depending upon the estimated population by the ending of the period of design for the water distribution system. The chapter also covers the description of the study area which is Fatawa village in Shaqlawa, Erbil- Iraq.

The water supply network is a hydraulic structure made of various elements like pipes, pumps, tanks, and valves. These elements are critical in transporting suitable water with particular pressure. The supply pipe lines are looped or branched. The former is usually more desired than branched networks for the following reason for an extra adequate valving, the looped networks are able to provide an additional reliability grade. Furthermore, with a looped system, breaking the pipe is simple to remove and repair, with little bit small effect on users outside of the direct area. Meanwhile, for the branched system, consumers located downstream of the broken feeding pipe will have their water feeding interrupted till the repair process is completed. Looped arrangement, however, has more than one route for water to for users, and the system capability is greater. The advantages of the branched or tree system are easy to design, quick build and economic.

The water supply network delivers water for domestic, industrial, irrigation and commercial usages. Consequently, a large sum of money is assigned to the design of

a system. Along these lines, it is fundamental to explore and set up a dependable system which fulfills the accompanying conditions:

- Maintaining water quality norm in the distribution pipes.
- Establishing financial plan and format.
- Delivering a sufficient amount of water.
- Maintaining required hydraulic pressure.
- Assuring dependability of water provision any time.
- Ensuring water for each customer during fix of any segment of the framework.
- Providing great quality lines with least leakage.

Accomplish the conditions above and the components (pipes, reservoirs, pumps, elevated tanks, and valves) of a common water conveyance network designs and analyzes more complicated. Complex, nonlinear, non-convex runs these systems, with discontinuous hydraulic equations used by Kessler and Shamir (1989); Eiger et al. (1994); Dandy et al. (1996). Solving the pipelines network problems is intricate. It requires defeating the complexities and to realistically models fittings, spatial and temporal variations of water demand, pumps, storage tanks, valves, and differences in water quality urges for the development of simulation model of water supply pipelines. This network is a genuine system that uses mathematical equations to predict work in a wide range of scenarios without interfering with existing systems (Walski et al., 2001). There are two types of simulations: (1) the steady-state simulation and (2) the extended period simulation. The steady-state is defined as a set of conditions that remain consistent across time. In addition, the simulation is used to foresee the response to a specific set of hydraulic circumstances (for example, the peak hour demand). During a specified time, the steady-state simulation determines the flows, pressure, pump locations, valve position, and operating qualities, among other things. The extended period simulation, on the other hand, defines the network's state throughout time (Walski et al., 2001).

Hydraulic behavior simulation is not simple and straightforward. It deals with the solution of a set of nonlinear equations that occur at the same time. The continuity equation (satisfying flow conservation at each specific junction or node), the energy equation, and equations that relate pipe flow with head-losses, such as the Hazen-

Williams formula, Darcy- Weisbach's, and Manning's equations, are examples. Researchers and investigators have been interested in applying soft computing technology in situations involving iterative computations since its introduction. There is a lot of useful and practical computer software obtainable for simulating water supply systems. EPANET is considered as a widespread and significant simulation tool in the format, arrange, design and operation of the network. This model is used to determine the most cost-effective pipe sizes for supplying water to customers or to identify the optimal network parameters, such as nodal demands and pipe roughness coefficients, as well as to improve model dependability (Rossman, 1993).

The trial-and-error method is used in the traditional technique. They use one set of trial pipe sizes (diameters) or network characteristics to test the model's efficiency. Moreover, an inappropriate variable for satisfying hydraulic conditions leads to more adjustments to the variables in search of a better solution. The best design entails evaluating expenses and cost at each conceivable network and making a final decision based on the results. The network parameters, on the other hand, are calculated using some methods for comparing simulation output to actual field values. However, using a trial and error approach in both of these cases is slow and time consuming because defining economic decision alternates necessitates repetitive adjustments of variables (for example, pipe diameters) based on hydraulic outcomes gained from a simulation model (the network solver), until a few pre-defined determinations (least pressure requirement at the nodes) are met. Furthermore, the number of choice factors, proposed adjustments, and identical operating conditions exponentially increases the difficulty of the trial and error technique. An optimization strategy that strives for an ideal solution within the available potential area is one treatment option for dealing with these challenges (Wu et al., 2001).

The design of new water supply systems or the expansion/modification of existing systems is the most important broad concerns in the water distribution system. The main goal is to meet predicted client requests at particular pressure heads, including during pipe failures. In fact, ensuring a financially viable and feasible network system is a difficult mission. The least pressure head criteria may not be met if the network is configured with a sequence of undersized pipes. On the other hand, if the

pipelines were too large, the concept might not be feasible financially. The goal of the research was to develop a design that could determine the least expensive water supply network while meeting all hydraulic restrictions. The chapter depicts the cost-effective design, layout, and detailing of a pipe network for a rural village (Wu et al., 2001).

4.2 EPANET

The United States' Environmental Protection Agency's Water Supply and Water Resources Division devised and advanced EPANET (Rossman, 1993), which is a public scope water supply network modeling software. It simulates a water pipeline structure in both of steady-state and expanded period simulation (EPS). The status of the systems is determined by the steady state. Water system engineers and designers, for example, are concerned with modeling only the maximum and minimum hour conditions. This figures and calculates hydraulic achievement (the pressures, the flows and head losses for the pipes) for a set format and demands at nodes. While, the system performance is evaluated for many sequential time intervals, in the case, of extended period simulation. As a result, the network proposed to be stable and constant in a given time interval (one hour for an example). On an hourly basis, nodal pressures, water levels in tank, pipe flows, and pump rates are determined. Extended period simulation is essential to realize the water usages (demand factor) through time, as well as the water level variety of the tank and the pump operation. EPANET models both network hydraulics and water quality using a coordinated process. The following are a few of the basic hydraulic equations used in EPANET:

1. The equations of flow in the hydraulic model are based on mass and energy conservation. According to the law of mass conservation, "the rate of storage in a system is equal to the difference between the input and outflow to the system."

In every node, the mass conservation is:

$$\Sigma Q_{in} - \Sigma Q_{out} = Q_{ext} \quad (4.1)$$

Where, Q_{in} is the node inflows and Q_{out} is the node outflows; and Q_{ext} is the external demand.

2. “The difference in energy between two points is equal to the losses due to friction and minor losses and the energy added to the flow component between these points”, according to the law of conservation of energy. The total loss for each of the network's fundamental loops is:

$$\sum_{i,j \in I_p} h_{i,j} - \sum_{M_x \in J_p} P_E = 0 \quad (4.2)$$

Where $h_{i,j}$ refers to "the head loss of the pipe linking nodes i and j ; I_p is the set of pipes in the loop P ; M_x is pumps; J_p is for the set of pumps in the loop P ; and P_E is the energy of the pump M_x ".

Simply can be expressed as below:

$$\frac{p_1}{\gamma} + \frac{v_1^2}{2g} + z_1 = \frac{p_2}{\gamma} + \frac{v_2^2}{2g} + z_2 + h_{1-2} \quad (4.3)$$

Where; P is pressure (N/m^2); h_{1-2} is head loss; V presenting the velocity in (m/s); Z stands for the elevation in (m); g is gravitational acceleration constant ($g = 9.81m/s^2$); γ is specific weight of water (N/m^3).

3. The difference between the nodal heads at both ends of the pipe is the head loss. These head losses are calculated using the Hazen Williams technique:

$$h_f = \frac{10.7 L Q^{1.852}}{C^{1.852} D^{4.87}} \quad (4.4)$$

Where units are meters, h_f is the head loss (meters), L : is length, C is the friction coefficient, D is diameter., and Q flow in m^3/sec .

A network connection composed of pipes (links), nodes (junctions of the pipes), valves, pumps, and storage tanks. EPANET follows the water flow of in all of the pipes, the water height in the tanks, the pressure at all junctions, and the concentration of the chemical species everywhere in the network through a simulation time comprised of multiple time steps. In addition to chemical species, source tracing and water age can be simulated too. Running the software under windows, EPANET prepares an integrated circumstance for checking, editing and changing the input information of the system, running hydraulic simulation. The same applied to water quality simulations, and showing the outcomes in different ways and formats. These

formats involve data tables, color-coded network maps, time series graphs, and contour plots. Hydraulic Modeling Capabilities Full- featured and precise hydraulic modeling is a necessary condition for performing efficient water quality modeling. EPANET has a powerful and developed hydraulic analysis engine that contains the below features:

- Sets no limitations on the network size which can be analyzed.
- Calculates friction head loss by applying the Hazen-Williams, Chezy-Manning or Darcy-Weisbach formulas which involves minor head losses for fittings and bends etc.
- Models pumps with constant or variable speed.
- Calculates energy of pumping and its cost.
- Models different kinds of valves including check, shutoff valves, pressure regulating, and flow control valves.
- Allows any shape for storage tanks (i.e., height can vary with the diameter but still circular).
- Considers various demand categories at junctions, each with its own pattern of time variation.
- Models pressure dependent flow issuing from emitters (sprinkler heads).
- Can base system operation on both simple tank level or timer controls and on complex rule based controls.
- Model input data for analysis the water supply network utilizing EPANET software.

The following are required as input data.

- Junction report (NODE)
- Pipe report (LINK)

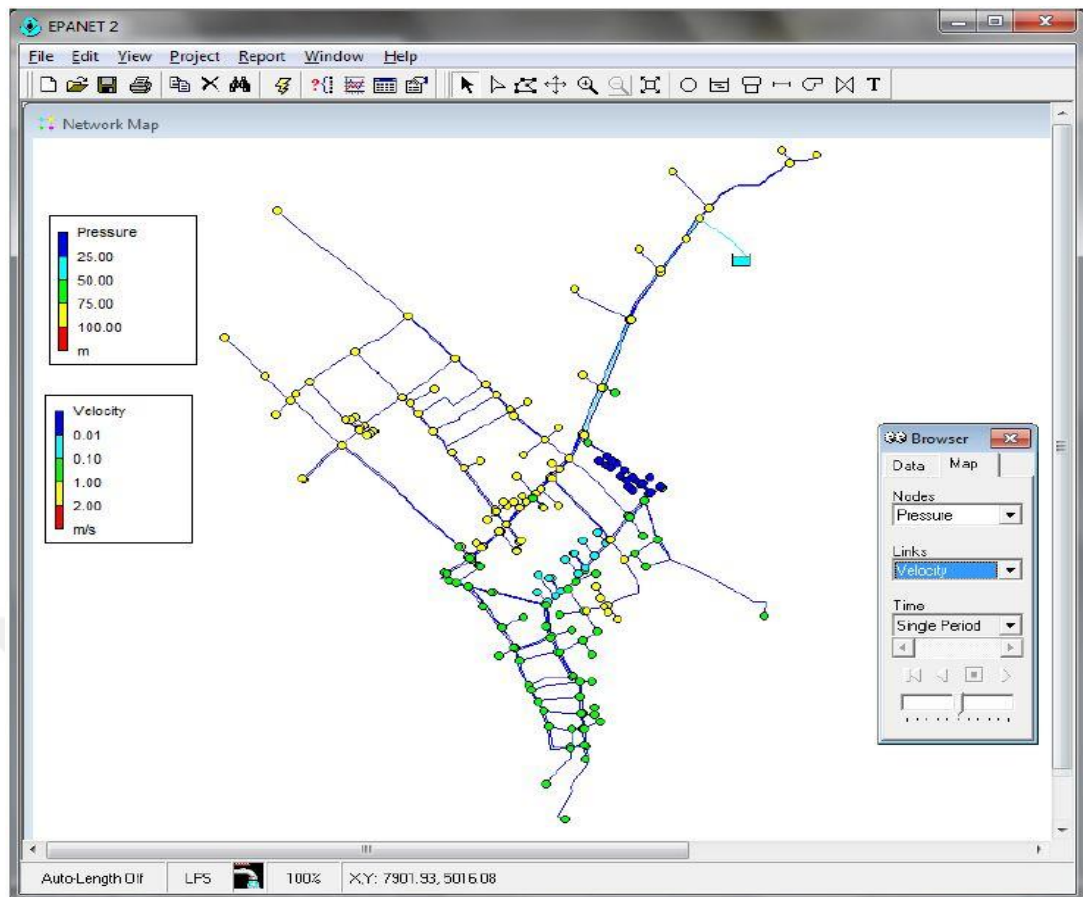


Figure 4.1 Screen shot for EPANET work space

The junction report (NODE)

Junctions or nodes appear in the network which linking together and where the water inflow or outflows the system. The primary input information desired for nodes include:

- Elevation of the node above a reference (in general mean level of the sea).
- The water required amount at nodes (average of dragging from the system).
- Initial water quality.

The outcomes calculated for nodes all time period of simulation include

- "Pressure.
- Hydraulic head (internal energy per unit weight of fluid).

Water quality Junctions can also:

- Have their demand vary with time.
- Have various categories of demands allocated to them.

- Have negative demands referring that water is going into the system.
- Be water quality sources where constituents enter the system.
- Contain emitters (or sprinklers) which make the outflow rate rely upon the pressure".

The pipe report (LINKS)

Pipes are interconnected linkages that convey water from one system site to another. EPANET assumes that all pipelines are always filled. The flow is from the end with the higher hydraulic head (internal energy per weight of water) to the end with the lower hydraulic head. The following are the most important hydraulic input parameters for pipes:

- Start and end nodes.
- Diameter.
- Length.
- Status (closed, open or contains a check valve).
- Roughness coefficient (for determining head loss).

Calculated outputs are:

- Flow rate.
- Head loss.
- Velocity.
- Darcy-Weisbach friction factor.
- Average reaction rate (over the pipe length).
- Average water quality (over the pipe length).

4.3 Modules of EPANET

EPANET is made up of two parts:

1. The network solver which implements hydraulic simulation plus water quality simulation.
2. The network solver's graphical user interface (GUI), which serves as both a front and back end for the network solver.

4.3.1 Network Solver Module

The EPANET network solver module is C-language computer software that includes modules for input processing and conversion, hydraulical analyzing, the water quality analyzing, linear equation analysis, and preparing reports. The input processor module (.INP) which is written utilizing a Problem Description Language (.PDL) receives a readable input data file. The input file characterizes and defines the network arrangement, with each network's information stored in its own isolated part, denoted by a keyword in brackets. Comment lines are lines that begin with a semicolon and can be added anywhere in the file. The hydraulic module runs a comprehensive, extended-period hydraulic simulation and saves the results to an external hydraulic file at each time step (.HYD). If a water quality simulation is given in the input file, the water quality module computes substance materials flow and reactivity throughout the network at each hydraulic time step. It saves the water quality test results to an unformatted output file (.OUT) and sends the results to the model users in the form of a formatted report file (.RPT). This file may contain data such as pipe flows, nodal pressures, and heads, among other things. A further sub-routine is connected to the report recorder in the proposed model to extract the hydraulic heads, which are then supplied to the optimization procedure.

4.3.2 Graphical User Interface (GUI)

The EPANET graphical user interface is compatible with Windows 95/98/NT. It simplifies and accelerates the simulation system's layout. The network can be designed visually by using the mouse to point and click, and the network design depicts the water distribution system graphically.

The GUI is in charge of modifying the network elements' properties and attributes, as well as the simulation choices. It uses the solver module to mimic and simulate network behavior and accesses the solver's results to provide to the user in a variety of ways.

4.4 Description of the Study Area

The area of which this study is designing the water distribution system is village of Fatawa locates in Shaqlawa district, Erbil Province in the northern part of Iraq. It

locates 51 km to the northeast of Erbil, at the lowest down of Safeen Mountain. Shaqlawa is located between Safeen Mountain and Sork Mountain, and sits 1066 m above sea level. The metropolis is well-known for its waterfalls, forests, and greenery.

The nearest natural spring to the village is known as kani Hangeer. The daily discharge of the spring is about 8671 m³ with average flow rate of 115 liters per second (L/s). Many of the springs dried through years, except those with high flow rates like kani Hangeer, which can be considered as a sustainable continues source of water.

Shaqlawa has a Mediterranean climate with very hot summers and relatively cold, wet winters. Subfreezing highs are very common in the winter, which would present frost. Snowfall is not uncommon. The village map is shown below in Figure 4.2.



Figure 4.2 Village of Fatawa, the study area.

4.5 Design Period

All structures and installations do not stay forever or remain with the same efficiency. The design period makes the essential decision. In other words, how long will the structure or system be operational? If the system is designed with the current

population in mind, it will be obsolete before it is even completed. As a result, a system that takes into consideration the size of the population at the end of the design period is required. Typically, a design period of 30 years is used for urban water feeding systems, while in fact that PVC, for example, has a minimum expected life of 50 years. Projecting beyond 30 years comes with a lot more risk and a lot more money up front. Rural communities, on the other hand, have a design term of 15 to 20 years, depending on the area's expected future development. The design duration for this study was set at 15 years.

4.6 Design and Formulation

The primary goal of designing water supply network is determining the appropriate diameter for each pipe in a system for a planned layout, demands under various circumstances, and an operating policy. The model chooses the best network sizes while complying to all implied and explicit restrictions such as mass conservation, energy conservation, design constraints, and head limits. The hydraulic constraints work by requiring a hydraulic head at the junction to be bigger than a specific lowest value.

This chapter talks about the steps and the procedure which taken to design and establish the water supply network in the village of Fatawa in detail .The steps were:

- Preliminary data collection.
- Survey the area of study.
- Population forecasting.
- Determination of water demand.
- Tentative layout of the network.
- Constructing the model in EPANET.
- Appointing water demands to each node.
- Running a hydraulic Modeling using EPANET.
- Calculate :
 - Discharge in each pipe.
 - Diameter of pipes.
 - Pressure at each node.
 - Velocity in pipes.
 - Unit head loss.

The following segments will debate each of the steps mentioned above.

4.7 Data Collection:

The most important phase in any research study is collecting the required data. In constructing the model of the water distribution network, the data were first collected relating all the distribution system parameters. A field visit to the village of Fatawa at Shaqlawa district was conducted on the November, 2020. The information were obtained from official government sources and agencies, ministry of agriculture and water resources, and some others obtained by visiting the village and do site survey.

The village has a school and mosque. Preliminary information, additional inputs were desired for the simulation of the model. The most important was the height dataset. Without the elevations, it is not potential to run the hydraulic simulation. The elevations were obtained by the site survey which had been done.

The second important dataset necessary was the information relating number of houses in the village and how they are distributed on the land to draw map of the area illustrating all details. This information is required to appoint base water demands to each particular node.

4.8 Survey the Area of Study

The extent of land between the water supply source and the distribution region is surveyed to determine the levels needed to align the main conveying pipe. This main will transport water to the distribution area which is surveyed. The detailed maps indicate the locations of roads, streets, lanes, residential areas, commercial areas, farms, and other features are created. To determine the high and low zones, a thorough map of the region is created as shown in Figure 4.3.

The GPS Leica g15, which is connected to the satellite, was used for this survey, and a thorough map was generated by taking coordinates at various spots along the waterway, spring site, roads, and buildings of the hamlet with the help of Autodesk civil 3d software.



Figure 4.3 The study area.

4.9 Population Determination

The population estimate is one of the most significant and crucial aspects of the project's planning and development, thus the water distribution system will serve the community for a specific design period. In this case a design period of 15 years is chosen. The important question here is "what will the population be by the ending of the set period for design", this could be achieved by utilizing different methods for population forecasting. Any underestimated value leads to have inconvenient system for the objective proposed; likewise overestimated value will make it high-cost. Change in the population of the city over the years occurs, and the system has to be designed taking into account of the population at the end of the design period which determined for the system.

Elements that affect changes in population are:

- Increasing because of births.
- Decreasing caused by deaths.
- Migration caused increase or decrease.
- Out of annexation increase.

Academically Inspired:

4.9.1 Population Forecasting Methods

Discovering what the population is by the ending of the design duration is made up of projecting the population based on some parameters, like population density or growth rate. There are various formulas for projection based on growth rate. It is most widely serviceable and functional techniques. In addition, the geometric method considered as the common technics to predict the population which reported in Figure 4.4.

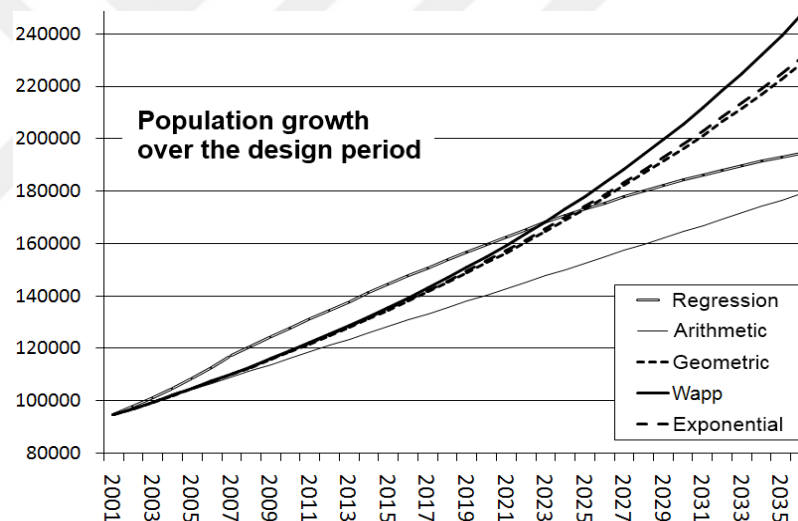


Figure 4.4 Comparison of results of different future population projection methods.

The presence of a census could explain population data and growth rates simply. Otherwise, a site survey must be done. Unluckily, most people do not have an idea about their exact population.

Following the most popular methods of future forecasting population:

- Arithmetical Increase Technique.
- Geometrical Increase Technique.
- The Incremental Increase Technique.
- Iler Bankasi Method.
- Decreasing rate Technique.

- Ratio and correlation.
- The Graphical Technique.
- The Graphical Method Comparison.
- Method of Master Plan.
- The Logistic Curve Technique.

4.9.1.1 Arithmetical Increase Method

It is the simplest approach of population projection, although it produces lower results in most cases. The increase in population from decade to decade in this method is assumed to be constant. This hypothesis can be mathematically expressed as

$$\frac{dP}{dT} = K$$

Where, $\frac{dP}{dT}$ is the rate of change of population, K is a constant.

After the population rise in every decade is obtained, with an average increment added to the next successive future decade. The next generation population (P_n) following n decades is obtained as follows:

$$P_n = P + nI \quad (4.5)$$

Here, P is the Present population and I stand for an average increment for a decade.

4.9.1.2 Geometrical Increase Method or Uniform Percentage Growth Method

In this method, it is supposed that the percentage increase in population from decade to decade is constant. From the population data of previous three or four decades, the future population can be estimated. If I_g is the average percentage increase per decade, or r_g is the increase per decade expressed as ratio, the population P_n after n decades is given by:

$$p_n = p \left(1 + \frac{I_g}{100}\right)^n = p(1 + r_g)^n \quad (4.6)$$

Let P be the present population and P_1 be the population after one decade.

Then,

$$P_1 = P + \frac{I_g}{100} = P(1 + \frac{1}{100})^1$$

Hence,

$$p_n = p(1 + \frac{I_g}{100})^n \quad (4.7)$$

In some references, the same formula is applied with different symbols like,

$$p_F = P_O(1 + \frac{i}{100})^t \quad (4.8)$$

Where P_F is the future population, P_O represents the current population, i : is growth rate%, t : is time in years.

This method yields a better result because the % increase is never constant, but rather declines as the city's expansion hits saturation. The geometric rate approach is the most extensively utilized.

4.9.1.3 Incremental increase method

This method connects the previously stated arithmetic to geometrical average methods. The real rise in each decade can be found using census data from the previous several decades. Then each decade's increment is discovered. The population in the following decade is derived by adding the average increase + the average incremental growth each decade to the existing population. The process is then repeated for the next decade as:

$$P_n = P + nI + n(n+1)/2 \quad (4.9)$$

Where P is the current population and I stands for the average increase in a decade. In addition, r and n are the incremental increase and the decade numbers, respectively.

Let P be the present population P_1 after one decade is:

$$P_1 = P + I + Ir \quad (4.10)$$

4.9.1.4 Iler Bankasi Method

It is a constrained version of the geometric increase method.

The formula for this method as below:

$$P_f = P_o \left[1 + \frac{i}{100} \right]^t \quad (4.11)$$

Where P_F and P_o stands for the population in the future and current population, I is growth rate%, t is time in years, respectively.

4.9.1.5 Decrease Rate of Growth Method

If the entire growth of a too old city is sketched, the curve would be in s-shape, which shows that early growth occurs at an increasing rate, then the growth decreases. This means saturation limit is achieved. Here, the average reduction in the percentage growth is exercised and subtracted from the latest each successive decade increase. This method is applicable where the rate of growth of population showed a down ward trend.

4.9.1.6 Ratio and Correlation Method

In this method, the rate of population growth of a town is related to the rate of population growth of state or nation. Hence it's within reach to estimate the population of a town under consideration by considering the rate of population growth of state or country.

4.9.1.7 Graphical Method

Using this method, the populations of the previous few decades are precisely plotted on the graph at an appropriate scale with regard to decade. The curve is smoothed out to anticipate future population. The graph portraying the current city was drawn from the beginning to show the growth curve.

4.9.1.8 Master Plan Method

The growth of villages, towns, and cities is not permitted at random. Master plans are created for the development of the communities. The city is organized into various

zones or sections, including industrial, commercial, and residential districts, as well as colleges, schools, and parks.

According to the master plan, the future expansion of cities is rigorously governed by various laws of corporations and other local entities. Master plans are typically created to guide the growth of cities over a period of 25 to 30 years. The population densities in various zones of the towns to be created are also set. It is very simple to build a water delivery network for specific zones. The master plan is also used to plan the future development of the various stages of the water works.

4.9.1.9 Logistic Curve Method

When the growth rate of population due to birth death and migration takes place under normal situation and it is not subjected to extraordinary changes to unusual situations like war, epidemic, earthquake, exodus of refugees etc. The population would possibly follow the growth curve characteristics of living things within limited area and economic opportunity. This curve has s-shape and is known as logistic curve.

4.9.2 Applied Methods

As result of the survey and data collection it has been found that the village is consisting of 52 houses, having one small school and a mosque. Current population is about 260 people, with average of 5 persons per house.

The most widely projection method used is the geometrical thus it has been used in the equation (4.8). Thus, P_F and P_o stand for future population and current population, i : growth rate%, t : time in years, respectively. Also, t and P_o are 15, 260 respectively and $i= 3\%$. The future population is 405.

In the same time Iler bankasi method was used and the same result had been obtained. While the result of the arithmetic method was less than the results obtained from the other methods mentioned above, so it has been neglected.

4.10 Water Demand

It is the amount of water consumed by the population for a variety of purposes, including cooking, drinking, washing, and working. There is no easy cure for determining a certain population's demand.

Table 4.1 Water requirements for domestic purposes

S.No.	Description	Amount of water in liters per head per day
1	Bathing	55
2	Washing of clothes	20
3	Flushing	30
4	Washing the house	10
5	Washing the utensils	10
6	Cooking	5
7	Drinking	5
Total		135

Table 4.2 Consumption of water for domestic animals and live-stocks

S.No.	Animal	Water consumption (L/D)
1	Cow & buffalo	40 – 60
2	Horse	40 – 50
3	Dog	8 – 12
4	Sheep & goat	5 - 10

The next guide some minimum data to be used for working:

Table 4.3 Minimum demands per capita per day

Minimum daily demand (l/un.)	
Urban inhabitant	50
Rural inhabitant	30
Student	5

Outpatient	5
Inpatient	60
Ablution	2
Camel(once a week)	250
Goat and sheep	5
Cow	20
Horses, mules, and donkeys	20

The total water demand for the village by the ending of the design period can be obtained from the sum of the per capita demand (domestic demand), commercial demand and loss and waste. If it was necessary animal demand can also be evolved. The demands components are mentioned in the following section.

4.10.1 Per Capita Demand or Domestic Demand

This is the water desired for people of the projected village population. A per capita water demand differs from somewhere to another, for example per capita water demand of at least 15 - 20 liters per capita per day (L/Ca/D) is the present design standard in Ethiopia for rural areas, while in India the minimum allowable per capita for rural areas is 70 liters per capita per day and in other standards is 30 L/Ca/D. In this study the per capita demand used is 140 L/Ca/D (which is the sum of water requirements for domestic purposes). The quantity of water required or demand varies according to the life style of the community, so the minimum demands in some do not suit all communities.

4.10.2 Public Demand

This is the water required for schools, hospitals, hotels, offices, military camps, cottage industries etc. Based on some ideal usage recommended in books, the daily required quantities of water by those facilities are given below.

Table 4.4 Recommended public water demand for different institutions

It.No.	Facility	Demand in liters
1	Day school	2 - 3 liters/student/day
2	health posts and clinics (no bed)	20 liters/staff/day and 2 - 4 liters/patient/day
3	Government offices	2 – 3 liters/worker/day

In the absence of data, the public demands can be estimated as 10 - 20% of the per capita water demand. For our design purpose we can use 15%.

4.10.3 Commercial Demand

This is the amount of water required for commercial centers such as hotels, tea rooms, restaurants, bus station, laundry etc. If there are number of such commercial centers in the village and even if there is a plan to construct hotels in the future, assessment on the water consumption from the existing commercial centers can be made and also projection in demand for the future planned commercial centers can be made and the demand can be included in the study.

4.10.4 Animal Demand

As animals are the basis for the life of the rural community, it is preferable to include animals demand also in the design if the source has adequate yield. It is recommended cattle trough not to be considered if a flowing stream is available at a distance of 3Kms from the village, visual observation and discussion with the beneficiaries are required to be made whether to include in the design or not. To simplify the process of the design, irrespective of the size of the animals usually an average animal demand of 5 liters/animal/day is recommended to be used in the design of rural water supply schemes

In this case the water source is away from the village less than 3 km and there is a channel goes though it which is used for irrigation and animals drinking, so animal demand has no included in the calculations.

4.10.5 Loss and Waste

Water losses include water used at the treatment plant for back washing, leakage in the pipes and unmetered usage, such as overflow from service reservoirs.

Usually loss and waste is estimated to be 10 - 20% of the sum of domestic, commercial, public and animal demand depending on various factors. In our case 10% can be used for loss and waste in the design. The loss and waste will be more if the system components mainly pipes are constructed poorly. The used rate was 20%.

Average daily demand

The average daily demand is the total sum of the domestic demand, commercial demand, public demand, animal demand, and loss and waste.

Maximum daily demand

The water consumption in a year varies from day to day. The maximum day demand is the highest demand in a day of specific year.

Maximum daily flow ($Q_{\max}$)

This is flow in liters per second required to deliver the maximum daily demand of the people at most within 24 hours.

$$Q_{\max} = \text{Maximum daily demand (Md)} / \text{time} \quad (3.12)$$

Where, $Q_{\max}$ is Maximum daily flow in L/s, M_d is maximum daily demand in liters, t is time in seconds in 24 hrs = 86400seconds.

Peak hour demand (Pd):

Water demand in any system is not constant, fluctuating up and down over the course of a 24-hour period. The peak hour demand is the maximum amount of water that is projected to be consumed in a 24-hour period. This is when the most water is consumed, and all taps are frequently left open. Peak demand often occurs during the daytime hours around lunch times, accounting for 20 to 25% of the operating period (4-6 hours per day). The peak hour demand is the peak hourly demand of the

maximum day. This demand is greatly influenced by the size of the town and social activity pattern.

4.11 Factors Affecting the Demand Per Capita

The main affecting factors for per capita demand for a city or town are mentioned below:

- a. Climatic conditions: The amount of water required in dry and hot places is more than in cold countries due to the use of air conditioners, air coolers, gardens, sprinkling of water on lawns, courtyards, washing of rooms, more washing of clothing, and bathing. Due to wastage, the required volume of water in very cold countries may be higher, because of the fear of water freezing in the taps and the use of hot water for housewarming, people in places like those frequently keep the faucets open and water continuously runs.
- b. Community size: As the population of a town grows, more water is necessary for street cleaning, park and garden upkeep, and sewer system operation.
- c. Standard of living: Because of room coolers, air conditioners, flush toilets, lawns, latrines, and automatic household appliances, the town's per capita demand rises in tandem with the people's standard of living.
- d. Commercial and industrial activities: certain industries require far more water than home use; hence their presence in town will significantly raise the town's per capita demand. Actually, the water required by the industry has no direct connection to the town's population.
- e. Pressure in the supply network: Water consumption increases as the building's pressure increases, and even if the required pressure is maintained at the farthest point, the water consumption will rise. The rise in quantity is due to two factors: first, people are using water more freely now than they were previously, and second, there is more water loss owing to wastage, leakage, and thefts, among other things.
- f. Sanitation system: towns with a water carriage system will have a higher per capita demand than towns without one.

- g. Water cost: The cost of water has a direct impact on its demand. When the cost of water is higher, people will use less water than when the cost is lower.

4.12 Temporal Variations in Demand

The average water usage for a year is the town's per capita demand. In actuality, this demand does not remain constant in the year, yet varies according to season, and even according to hour. The amount of water consumed per day is just as important as the time of consumption; demand in the morning is not the same as demand in the afternoon or at midnight; consumption varies by season and week; thus, the design must be for the hour with the highest demand on the most consuming day of the week in the driest and hottest season. In other words the network should be designed for the worst case; if it works it means it can work in other cases as well.

4.12.1 Monthly or Seasonal Variations

The amount of water required varies from season to season. Summer has the highest water demand because people use more water for bathing, lawn watering, cooling, and street sprinkling. In the winter, this demand will be at a minimum because less water will be utilized for bathing and there will be no watering of parks and gardens. The differences might be up to 15% of the year's average demand.

4.12.2 Weekly Variations

There are no significant differences among most populations, but keep a look out for cultural shifts, celebrations, marketplaces, and fairs, among other things. They have the ability to influence everyday demand at various times of the week. If measuring the daily pattern was tough, measuring the weekly pattern (which needs work and logistics over the course of a week) is even more challenging.

You can presume that demand doesn't change greatly through the week unless you have a clear indicator of weekly fluctuations. The calculation of the volume of water that entering a tank over the course of a week could be the easiest approach.

4.12.3 Daily Variations

This varies depending on people's general behaviors, climatic conditions, and the city's character as an industrial, commercial, or residential area. Sundays and holidays will have higher water demand than regular working days due to more comfortable bathing, washing, and other activities. The average daily consumption is usually 180 percent of the maximum daily consumption. The majority of populations follow a similar pattern.

The demand is lowest at night, and it peaks in the morning. Showering, gathering water for cooking, and washing clothes can take 45-65 percent of a person's daily water consumption in just a few hours.

Another smaller surge of around 20-30 percent occurs in the middle of the day. Measuring a population's demand changes can be time-consuming and difficult, and it isn't always attainable. To determine the total demand pattern, so many consumers at the same time must be measured.

4.12.4 Hourly Variations

On weekends and other holidays, peak hours may be around 8 a.m. owing to late awakening, while minimum flow may be between 12 a.m. and 4 a.m. when most people are asleep. However, in a highly industrial city where both day and night shifts operate, night consumption may be higher. The greatest usage might reach 200 times the average daily need.

The determination of this hourly fluctuation is critical since it will be used to modify the pumping rate to fulfill demand at all hours.

If data on daily, weekly, and monthly changes, as well as unaccounted for demand, is not available, an approximation is conducted by multiplying the base demand by a number between 3.5 and 4.5.

4.12.5 Uncounted for Water

Leaks, unauthorized connections, garden watering, water spilled when containers are filled, and other surprises are all part of this group. It is around 20% in a new system.

The daily demand should be multiplied by coefficients for variations, for this study the following values were used in order to find the average daily demand:

- The daily variation coefficient = 2.5
- Weekly variation coefficient = 1
- Monthly variation coefficient = 1.4
- Uncounted for water coefficient = 1.2

After calculating all kinds of water demand and multiply the sum of demands by the factors of variation, and find out the maximum flow needed during the day which denoted by Design flow, compare this flow with minimum daily flow of the water source (the spring).

If it was smaller than the minimum daily flow of the source, then the discharge of the spring is adequate for the water supply system.

4.13 Tentative Layout of the Network

The position of the distribution mains, the spring box, and the network's starting point, nodes, and pipes are then plotted on a rough distribution line plan. The entire area is divided into various distribution districts. House sites are frequently used to indicate population density. Pipeline lengths should be maintained as minimal as possible.

4.14 Design Criteria:

Designing new water distribution networks or expanding/modifying existing ones are the most typical difficulties in water delivery systems. The basic goal is to meet projected demand at predetermined pressure heads, even if there are pipe problems or during pipe failures. Ensuring that a network system is economically viable. The minimum pressure head criterion may not be met when the configuration of the network is conducted using pipes which are undersized. The design, on the other hand, may not be economically feasible if the network pipe is enormous. The goal of the research is to create an appropriate methodology for designing a water network meeting all hydraulic restrictions with the lowest expenditure.

1. Pipe sizes, which is the pipes used are PVC pipes of diameters 90, 63, and 50 mm.
2. Pressure limits in pipes are between 1–3 bars or 10 – 30 m of water.
3. Velocity limits in pipes are between 0.3–2 m/sec. In details, in pipe line design, the velocity of flow through the pipeline is also an important factor to consider. If the velocity is too high, suspended particles in the flow might cause excessive pipe erosion; if the velocity is too low, suspended particles in the flow can settle down and collect at low locations in the pipeline, eventually blocking it if left uncontrolled. In the case of water without suspended solids, the minimum velocity can go down to 0m/sec at all times and where the other parameters are correct. At 0.5 m/sec the pipes are considered self-cleaning. Velocities more than 2 m/sec suggest the pipes are too small and create a risk of damage by water hammer and tear. This project is designed in a way that no velocities exceed 2 m/sec in the system.

Criteria for pipeline sizing are based on keeping velocities low within limits to minimize scouring of interior coatings, and limit head loss in the distribution system. The head loss gradient should be with regard to both velocity and friction.

4. Pipe material, in details, The PVC pipes are the most practical types that can be used in water network. The selection of one type will mainly depend on the economic considerations.
5. Unit head loss is a measurement of energy that is lost by friction inside the pipe and depends firstly on the flow rate and velocity. It is measured by meters of head lost per kilometer of pipe. The less meters of head lost the most efficient the pipeline is.
6. Friction losses through pressure piping are based on Hazen – Williams formula.

4.15 Spring Box Design

The spring box is actually a small transmission box or tank, this tank have two outlets, one over flow spillway which flows into an open canal and a PVC pipe which connected to our water supply system. To direct one of these flows into our system, a small tank can be constructed, or directly connect the pipe to the spring box.

The box designed to be built with concrete walls and ceiling which contains an access door, this door made of sheet metal and used for maintenance. One of the walls has an overflow part allows water to flow out into the existing open canal. Screens or filters are provided at the entrance of the transmission pipe to prevent any debris from entering the transmission line.

4.16 Methods and Formulas

The total demand was calculated using the population of the study region at the end of the design period, which was 405 people and the study location could category of rural settlement. For this study a daily demand considered as 140 L/Ca/D, then this had been multiplied by variation coefficients (daily, weekly, monthly variation coefficients and coefficient of uncounted for water), then multiplied by the population. For school students the demand considered as 8 liters/student/day multiplied by 150 (number of student), after that multiply the result by 4.2 (which is the variation coefficient). The small mosque capacity had been estimated as 150 prayers, and a demand of 15 liters/prayer/day. The average daily demand for the mosque is the result of multiplying number of prayers by the demand of each prayer by the coefficient of variation.

Once the design discharge is known (which is the sum of all demands), pipe diameters are assumed in a way that the velocities of flow in the pipes remain in the range between 0.3 to 2 m/s. smaller velocity is supposed for pipes of smaller diameter and larger velocity for pipes of larger diameter. The friction loss of head in the pipes is then calculated using Hazen – Williams formula as:

$$V = K C R^{0.63} S^{0.54} \quad (4.13)$$

Where, V is velocity of flow in pipe (m/s), R is hydraulic radius (mean depth) in m

$$R = \frac{A}{P} \quad (4.14)$$

Where A is cross-sectional area of flow, P = wetted perimeter, k is conversion factor for unit system k = 0.849 for units of m/sec k = 1.318 for units of ft/sec, S and C are the hydraulic gradient and the coefficient of roughness of pipe.

Thus the formula will be written as below:

$$V = 0.849 C R^{0.63} S^{0.54} \quad (4.15)$$

Where,

$$S = \frac{hf}{L} \quad (4.16)$$

And,

$$Q = VA \quad (4.17)$$

$$A = \pi \frac{D^2}{4} \quad (4.18)$$

Hydraulic radius will have units of length. For circular pipes flowing full, the hydraulic radius is:

$$P = \text{circumference} = \pi D$$

$$R = A/P = (\pi \frac{D^2}{4}) / (\pi D) = \frac{D}{4} \quad (4.19)$$

$$V = \alpha C D^{0.63} S^{0.54} \quad (4.20)$$

Where D is the pipe diameter and α takes on a value of 0.55 for Imperial units and 0.354 for SI units, the formula will be in the following form:

$$V = 0.354 C D^{0.63} S^{0.54} \quad (3.21)$$

D is the pipe's diameter in meters. The Hazen Williams formula looks like this when expressed in terms of head loss hf and pipe length L.:

$$h_f = \frac{10.7 L Q^{1.852}}{C^{1.852} D^{4.87}} \quad (4.22)$$

Where, H and L are the head loss in meters and length in meters, respectively, C and D are the friction coefficient and diameter in meters, respectively, and Q stands for the flow in m³/sec.

The discharge Q (m³/s) is produced as follows:

$$Q = 0.278 C D^{2.63} S^{0.54} \quad (4.23)$$

The Hazen – Williams coefficient C for PVC pipes (135 – 150) depending on the manufacturer, most common is 150. The table below shows different values of C coefficient for different pipe materials.

Table 4.5 C coefficient values in Hazen Williams formula for different pipe material.

Pipe material	C
Asbestos Cement	140
Brass tube	130
Cast-Iron tube	100
Concrete tube	110
Copper tube	130
Corrugated steel tube	60
Galvanized tubing	120
Glass tube	130
Lead piping	130
Plastic pipe	140
PVC pipe	150
General smooth pipes	140
Steel pipe	120
Steel riveted pipes	100
Tar coated cast iron tube	100
Tin tubing	130
Wood Stave	110

Hazen-Williams formula is more intuitive. The coefficient has no decimals, and moreover, this coefficient does not depend so much on the diameter or velocity. Among many types of pipes, the chosen pipe material for this study is PVC pipes with different diameters. So that the value of the coefficient $C = 150$ for this case. Because of its inexpensive cost, ease of installation, and widespread availability, this is the most preferred pipe. It is available in nearly all diameters. The release of dioxins and heavy metals makes it highly hazardous. Different ecological

organizations around the world have started campaigns to replace PVC with other products.

Advantages of PVC pipes:

- Low cost; it is sometimes referred to as the cheapest pipe.
- It is accessible to everyone.
- It's inert, meaning it won't react with chlorine or most other chemicals. • Expertise is readily available; most places have the necessary know-how and abilities for installation.
- Compact size. Even for the biggest diameters, cranes are unnecessary. Pipes can be moved, offloaded, and handled by people, which makes them ideal for difficult-to-access regions.
- When exposed to sunlight, PVC degrades.

Disadvantages:

- Fragile at low temperatures; requires frequent maintenance; mechanical resistance is low; and unions should be better.
- A proclivity for breaking lengthwise. When a pipe breaks like this, it opens up from end to end, creating tremendous leaks.
- Temperature causes a rapid decrease in resistance to pressure. When exposed to the sun at 43 degrees Celsius, resistance is half.
- Long-term toxicity, resulting in environmental issues.
- Adaptors are used to connect to accessories.

The interior diameter of metal pipe is stated, while the external diameter of plastic pipe is specified. The approximate internal diameters (ID) of plastic pipe are shown in this table:

Table 4.6 Internal diameters for PVC pipes

ND	25	32	40	50	63	75	90	110	125	140	160	180	200	250	315	400
ID PVC	21	29	36	45	57	68	81	102	115	129	148	159	185	231	291	369

4.17 Building the Model in EPANET

After lunch the software, the following steps were carried out to create the water distribution network:

- Calibrate the software, e.g. S.I. units, etc.
- Changing the units to LPS, the most comfortable configuration is LPS (L/s). With changing the flow units, the units are:

Flow: L/s.

Pressure: meters (column of water).

Diameter: millimeters.

Pipe length: meters.

Elevation: meters.

Dimensions: meters.

- Choose the Hazen Williams formula to compute the friction head loss. The pipe or roughness friction coefficient for can be selected from table 3.5.
- Importation of the background map, the background map had been brought from AutoCad by changing it into metafile form (WMF). After that the size of the map was modified by multiplying the size by a factor obtained from dividing the actual lengths by lengths which measured on the map in EPANET.
- Turn auto length on in order to measure the lengths directly.
- Place the nodes i.e. reservoir, tanks and junctions.
- Edit the appropriate data in the previous Step, add all needed details including elevation of each node, base demand, coordinates, illustrating if the node is inflow or outflow point by putting the appropriate sign (negative in the case of inflow).
- Draw the links (pipe) to connect all the nodes in previous steps.
- Edit links parameters appropriately, the data required for pipes are diameter, length which can be calculated directly while the auto length option is turned on.
- Run the system and check the pressure drop and check for errors.
- Display the Legend.
- Display the Result in Tabular and Graphical form.

4.18 Assigning Water Demands to Each Node

The way of determining the demand at particular junction can be by dividing the total population by the number of junctions in the system then multiply the result by the per capita demand. However, the area could be divided into zones and compute the demand depending on the population at each zone.

The following procedure used to allocate demands on the nodes:

1. Allocate school demand on the node.
2. Allocate mosque demand on the node.
3. The other nodes are feeding houses for domestic use. Calculate the demand at each node by calculate number of current consumers at the area which surround the node, multiply the number of consumers by a multiplier obtained from dividing future population by the current population as below:

Future population / current population = multiplier

$$405/260 = 1.56$$

Or apply the formula of geometric projection for current population =1

$$P_F = P_O (1+i/100)^t$$

The result will be the same 1.56

The demand at each junction = $140 \text{ L/Ca/D} * 4.2 \text{ (variation factor)} * \text{number of current consumers} * 1.56 \text{ (multiplier)}$

Average demand in L/D then convert it to L/s.

4.19 Hydraulic Modeling Using EPANET

After completing the drawing of the network and entering all required data including elevations of nodes, coordinates of the nodes, base demand, pipe diameters and lengths, run the system in order to check any errors and calculate the pressure at each junction, flow rate, head loss at nodes and velocities in each link.

4.20 Materials

The materials of this work includes:

- Topographical map
- Parameters of water distribution like water demand, the village population

- Distribution network parameters like elevations, diameter and length of pipe and
- EPANET software.

4.21 Laying of Distribution Network

For normal and heavy traffic, the minimum trench depth must be $15 + D + 30 + 145$ cm, and for other circumstances, $15 + D + 30 + 115$ cm. Here, D is the pipe's external diameter in (cm), 15cm is the bedding thickness. In addition, the compacted overburden thickness is 30cm, and the backfilled compacted thickness is the remaining 30cm.

The least filling depth above the pipes has to be greater than 0.8 m (the minimum allowable depth), and ideally one meter, to protect them from traffic loads and to prevent the effects of climatic fluctuations on water in the pipe, rendering it unappealing. The depth of 1m was chosen for our investigation and to make the computations easier.

CHAPTER V

RESULTS AND DISCUSSION

After explaining the methodology that had been used to design and analysis the water supply pipelines, the results have been found are mentioned and discussed in the following parts of this chapter.

5.1 Design Period

The constructions and installations do not last indefinitely or with the same level of efficiency. The crucial decision is made during the design process. To put it another way, to know how long the building or system will be in use. If the system is built with today's population in mind, it will not be sufficient and functional before it is finished. As a result, a system that takes the population size at the conclusion of the design period into account is necessary. The design period for rural communities is normally 15 to 20 years, depending on the area's predicted future development. This study's design period was established at (15) years.

5.2 Future Population Projection

One of the most significant basic tasks in any project to build water distribution networks is to estimate the population of the study area at the conclusion of the design period, because it is based on these statistics that the amount of water necessary for the network is determined. A field survey of the area yielded the current population, which is estimated to be 260 individuals living in approximately fifty-two houses in the village, and there is one school and a small mosque in the village.

There are many methods and techniques to calculate the future population as mentioned before in the previous chapter, the geometrical projection method was used to obtain the population at the end of the design period as below:

$$P_F = P_O (1+i/100)^t$$

T=15, i= 3%, Po=260 person.

The calculated population by the ending of the design period which is 15 years is = 405 person.

The Iler bankasi method was used as well and the result has been found out by applying this method was (405 person) the same as the first method, while the arithmetic method's result was (299 person) which is less than the result of the geometrical increase method so it has been neglected because the design should be for the most critical condition and if the system worked sufficiently in this case it means it will be sufficient in other cases as well.

5.3 Demand

In this network the quantities will be as bellow:

For domestic 140 liter/capita/day

For student 8 liter/student/day

For prayers 15 liter/prayer/day

The daily demand had been multiplied by coefficients for variations as below:

The daily variation coefficient = 2.5

Weekly variation coefficient =1

Monthly variation coefficient=1.4

Uncounted for water coefficient =1.2

The coefficient must be between (3.5 - 4.5).

Results of calculating daily average demand using the multiplier (4.2):

1. Demand for the school

8 lit/student/day × 150 student =1200 L/D.

Multiply it by the factor (4.2) = 5040 L/D.

School demand = 0.05833 L/s.

2. Demand for mosque

15 liter/prayer/day $\times$ 150 prayers = 2250 L/D multiply it by the factor (4.2)
= 9450 L/D.

The mosque demand = 0.109 liter/sec

3. Demand for domestic

$405 \times 140 \times 4.2 = 328140$ L/D.

Domestic demand = 2.756 L/s

$Q_{\text{Total}} = 2.9238$ L/s

The total daily average demand is the design discharge.

The design discharge $Q_{\text{Design}} = 252630$ L/D

The total demand discharge is smaller than the minimum discharge of the source, which means that it could be considered as a sustainable source for feeding the system.

5.4 Allocate Demands on the Nodes:

After allocating school demand and mosque demand on the nodes, the demand has been calculated for the other nodes which are feeding houses for domestic use. The demand at each node had been found out by calculate number of current consumers at the area which surround the node, multiply the number of consumers by a multiplier obtained from dividing future population by the current population as below:

Future population / current population = multiplier

$405/260 = 1.56$

Or apply the formula of geometric projection for current population = 1

It will be the same 1.56

The demand at each junction = $140 \text{ L/Ca/D} \times 4.2$ (variation factor) $\times$ number of current consumers $\times 1.56$ (multiplier)

The average daily demand measured in L/D then covert to L/s.

5.5 Analysis

The volumes of flow and head losses in various pipelines, as well as the resulting residual pressure at various demands in network connections, are all part of the analysis of any water distribution network. The results obtain for the network of Fatawa village water supply systems were described below.

5.6 Summary of Water Supply System Components

The following are the summary of the 24hrs supply Water Distribution Network of study area, which are the number of Junctions, number of reservoir, number of pipes were 18, 1, 20, respectively. In addition, the number of pumps was zero which is mean the water supply system is purely driven by gravitational energy. Table 5.1 presented the dilates of input data for each junctions such as elevations, base demand that has been calculated according to population. Besides that, all details belong to each pipe (link) were reported in Table 5.2 such as starting and ending points, lengths that were measured automatically by software with designed diameters.

Table 5.1 Nodes input data

Node	Elevation	Base Demand	Notes
J1	748.80	-2.91	Network entry
J2	748.02	0	---
J3	742.67	0	---
J4	739.81	0	---
J5	734.32	0	---
J6	736.53	0.15925	---
J7	737.64	0	---
J8	736.49	0	---
J9	736.49	0.15925	---
J10	725.10	0.15925	---
J11	737.22	0.0583	---
J12	736.86	0.37158	---
J13	736.49	0.1094	---
J14	733.40	0.42466	---
J15	731.18	0.212	---

J16	729.81	0.4774	---
J17	725.16	0.10616	---
J18	734.24	0.58391	---
RES1	749.89	0	The spring box

Table 5.2 links input data

Link ID	Start node	End node	Length (m)	Diameter (mm)
P1	RES1	J1	0.91	81.4
P2	J1	J2	27.57	81.4
P3	J2	J3	28.03	81.4
P4	J3	J4	126.13	81.4
P5	J4	J5	220.62	81.4
P6	J5	J6	32.32	81.4
P7	J6	J7	11.38	81.4
P8	J7	J8	55.54	57
P9	J7	J9	45.47	57
P10	J9	J8	29.33	45
P11	J9	J11	32.61	45
P12	J8	J13	35.66	57
P13	J13	J12	20.03	45
P14	J12	J11	28.34	45
P15	J13	J14	26.34	57
P16	J14	J15	8.69	57
P17	J15	J17	17.48	45
P18	J15	J16	10.87	45
P19	J8	J10	45.84	45
P20	J10	J18	124.16	45

5.7 Network Nodes

Table 5.3 presented the output of each NODES (JUNCTION) from EPANET's program after simulation process; such as demand, head and pressure were the desired output.

Table 5.3 Nodes output data

Node ID	Demand	Head (m)	Pressure (m)
J1	-2.91	749.87	1.07
J2	0	749.76	1.74
J3	0	749.66	6.99
J4	0	749.18	9.37
J5	0	748.33	14.01
J6	0.16	748.21	11.68
J7	0	748.17	10.53
J8	0	747.85	11.36
J9	0.16	747.94	11.45
J10	0.16	747.59	22.49
J11	0.06	747.81	10.59
J12	0.37	747.71	10.85
J13	0.11	747.70	11.21
J14	0.42	747.58	13.88
J15	0.21	747.57	16.39
J16	0.48	747.54	17.73
J17	0.11	747.56	22.40
J18	0.58	747.13	12.89
RES1	0.1	749.87	0

5.8 Network Pipes (Links)

The output of Links (Pipe) from EPANET's program after simulation process which detailed in Table 5.4. Therefore, the lengths, the diameters, the flows, the velocities, and Unit Head loss were the output.

Table 5.4 Links output data

Link ID	Length (m)	Diameter (mm)	Flow L/s	Velocity (m/s)	Unit head loss (m/km)	Status
P1	0.91	81.4	-0.10	0.02	0	Open
P2	27.57	81.4	2.82	0.54	3.82	Open
P3	28.03	81.4	2.82	0.54	3.82	Open
P4	126.13	81.4	2.82	0.54	3.82	Open
P5	220.62	81.4	2.82	0.54	3.82	Open
P6	32.32	81.4	2.82	0.54	3.82	Open
P7	11.38	81.4	2.66	0.51	3.43	Open
P8	55.54	57	1.37	0.54	5.73	Open
P9	45.47	57	1.29	0.50	5.07	Open
P10	29.33	45	0.52	0.33	2.98	Open
P11	32.61	45	0.61	0.38	4.02	Open
P12	35.66	57	1.15	0.45	4.12	Open
P13	20.03	45	-0.18	0.11	0.42	Open
P14	28.34	45	-0.55	0.46	3.34	Open
P15	26.34	57	1.22	0.48	4.59	Open
P16	8.69	57	0.8	0.31	2.08	Open
P17	17.48	45	0.11	0.07	0.16	Open
P18	10.87	45	0.48	0.3	2.55	Open
P19	45.84	45	0.74	0.47	5.8	Open
P20	124.16	45	0.58	0.37	3.71	Open

By the utilization of EPANET different connection are being found between the velocities of water, the water flows, the water demands, the heads, the pressures, contours, and so on these relations can be perceived by considering the diagrams plotted. Consequently, Figure 5.1 stated the network layout for this case study, while, the details of area study's layout have been presented in Figure 5.2. which are presenting the pipe and junction's label.

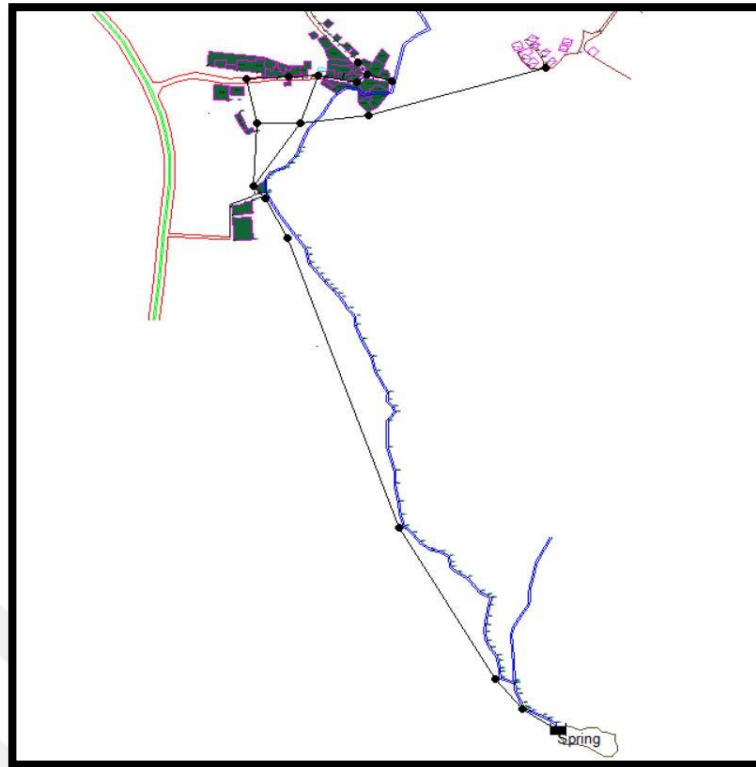


Figure 5.1 Network layout.

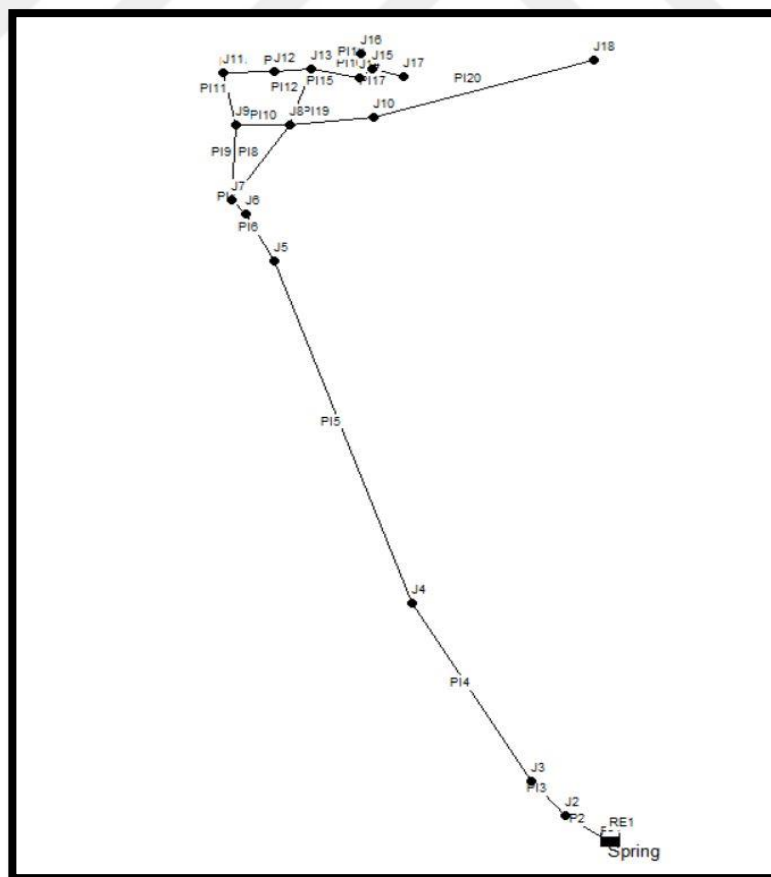


Figure 5.2 The layout of the network with the nodes and pipes ID.

The Pressure Versus Velocity Distribution graph

This graph depicts the variation in velocity in several pipe connections in relation to the pressure provided for each line. The various pressure and velocity numbers are shown in the Figure 5.3.

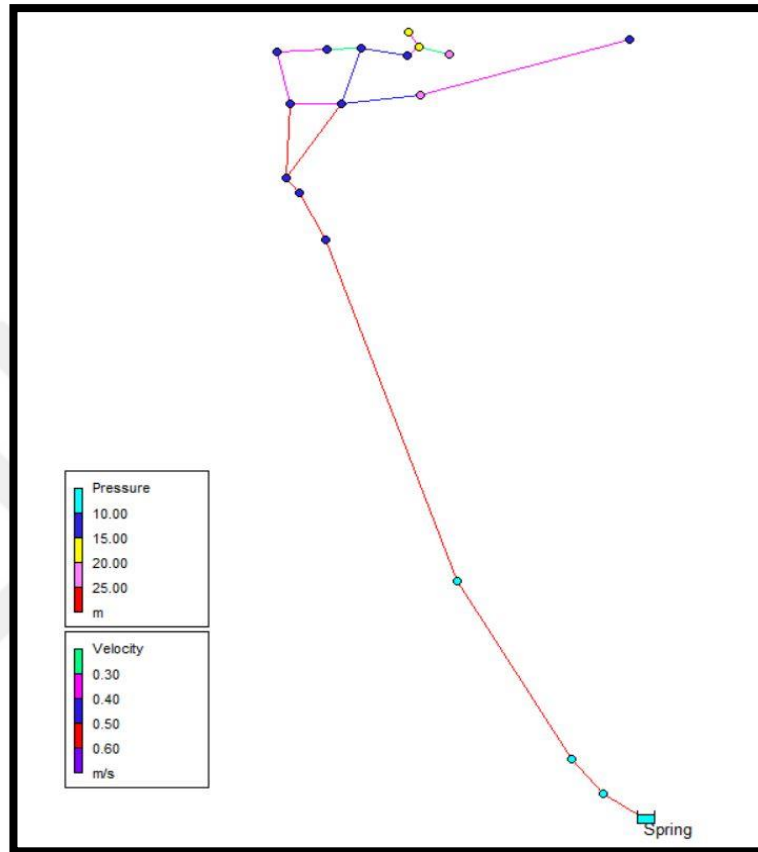


Figure 5.3 Pressure vs. velocity distribution

The Elevation of Contour Plot:

The contour levels are depicted on this map based on the elevation of connecting pipe lines at various elevations. It is distinguished with the assist of various colours as shown in Figure 5.4.



Figure 5.4 Contour plot elevation

The Contour Plot Demand

Figure 5.6 detailed the contour map of area's study regards with demand location and categorized according to different colors regards which started from light blue which refers to demand parameter than 0.1 L/s, then moved to read color that indicated to the demand more than 1.0 L/s.

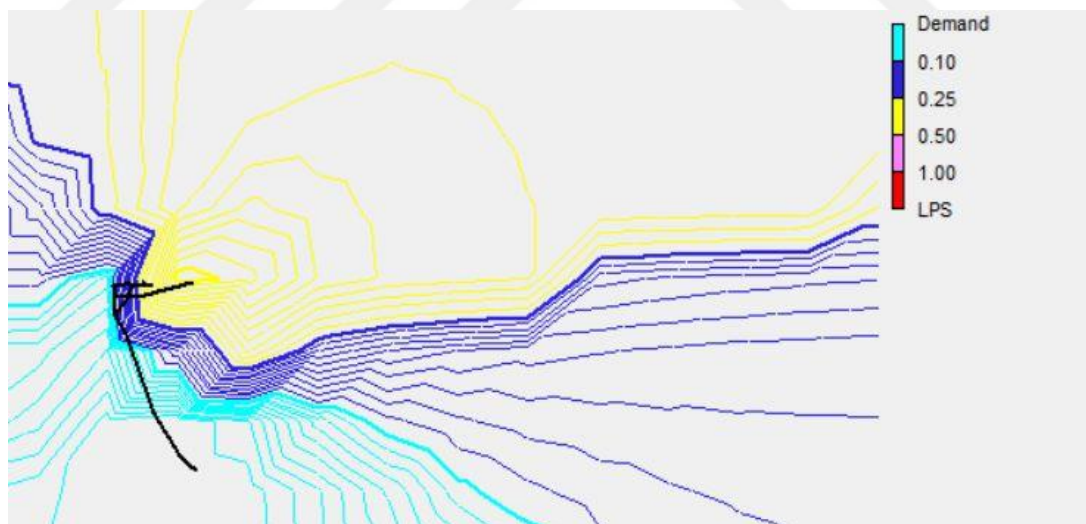


Figure 5.5 Contour map for the demand locations

The Contour Plot Pressure

Figure 5.6 illustrated the contour map of case study and categorized according to different colors regards with pressure zones which started from dark blue which refers to pressures less than 5m. then moved to read color that means the pressures

more than 20m. Moreover, Figure 5.7 presented the variation in pressure and flow rates are represented by various colors in the graph above between pressure and flow.

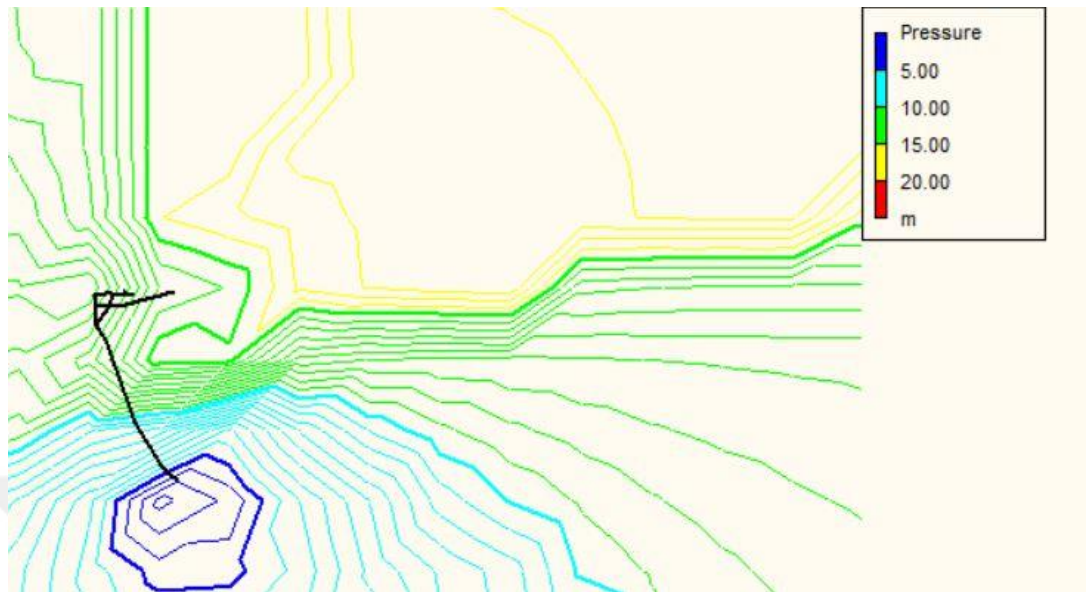


Figure 5.6 Contour map for the pressure zones

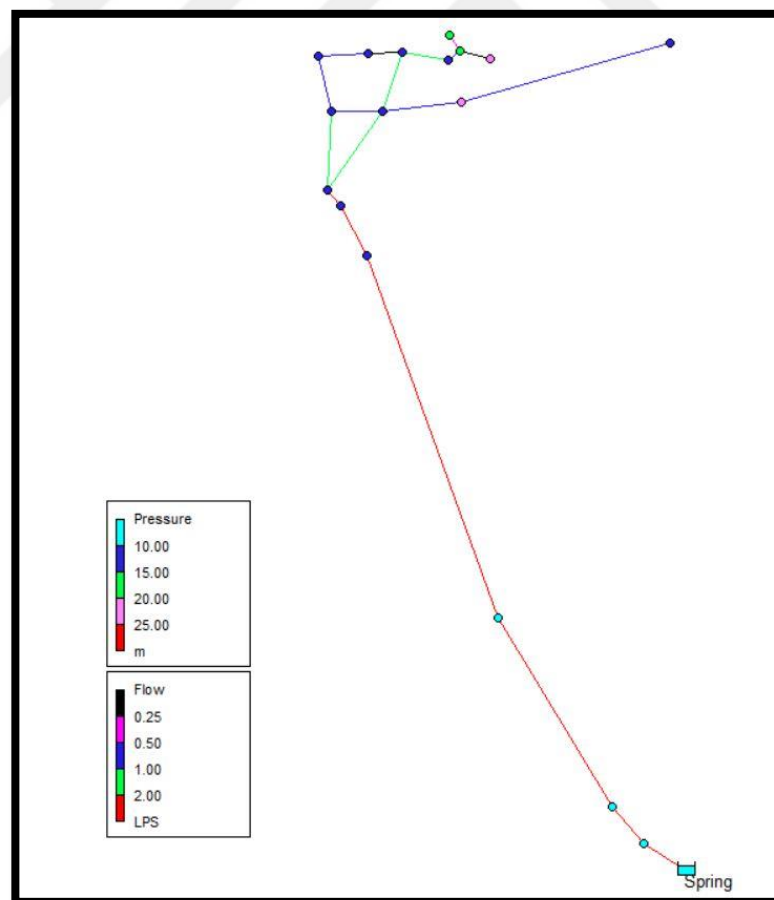


Figure 5.7 Pressure vs. flow

Demand-Flow Distribution

Figure 5.8 illustrated the demand parameters of case study and categorized according to different colors regards with flow parameter which started from light blue which refers to demand less than 0.1 L/s, then moved to read color that means the demand more than 1.0 L/s. On the other hand, the flow in each pipe member start from black color which means the flow rate less the 0.25 L/s, while, the red color represented the flow rate more than 2.0 L/s. Moreover, Figure 5.9 detailed the variation of demand and unit head loss that represented by various colors which is ranged between light blue to red (<0.1 to >1.0) L/s for demand parameter, and green to purple color (<2.0 to >8.0) m/km for unit head loss.

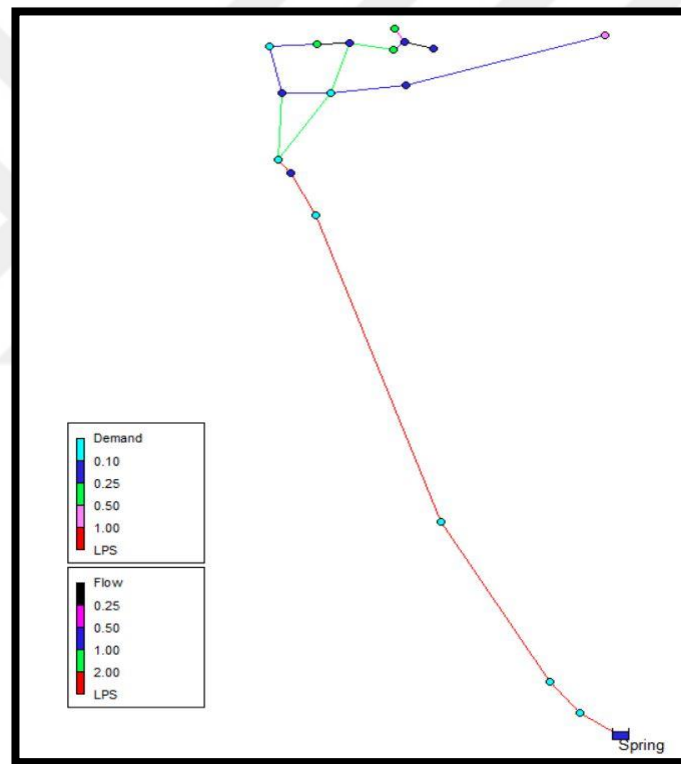


Figure 5.8 Demand vs. Flow

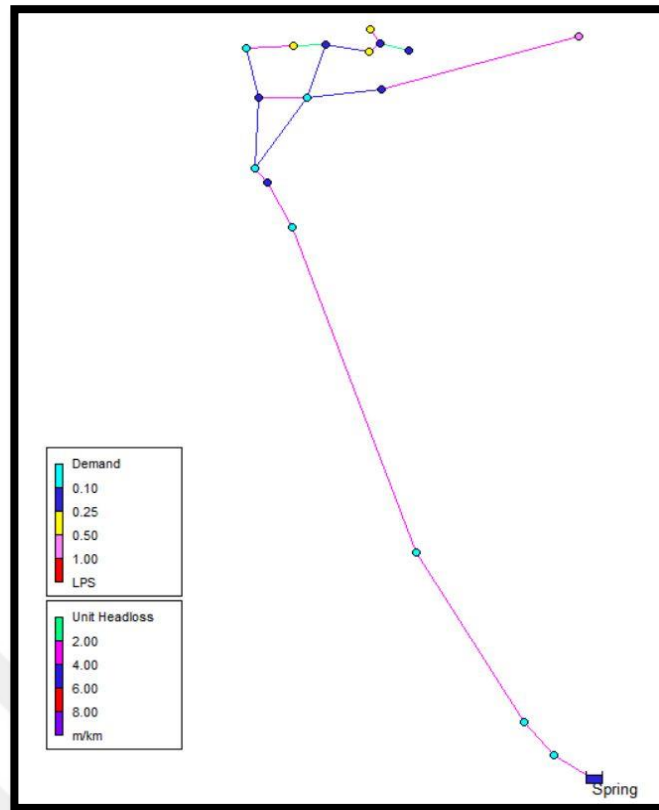


Figure 5.9 Demand vs. unit head loss

The Elevation versus Diameter Distribution

Figure 5.10 presented the elevation each node and categorized according to different colors regards with pipe's diameter which started from light blue which refers to demand less than 720 m, then moved to ping color that means the elevation was limited to 750 m. On the other hand, the same approach has been presented for the pipe diameter, which ranged from green to red color (45 to 81) mm.

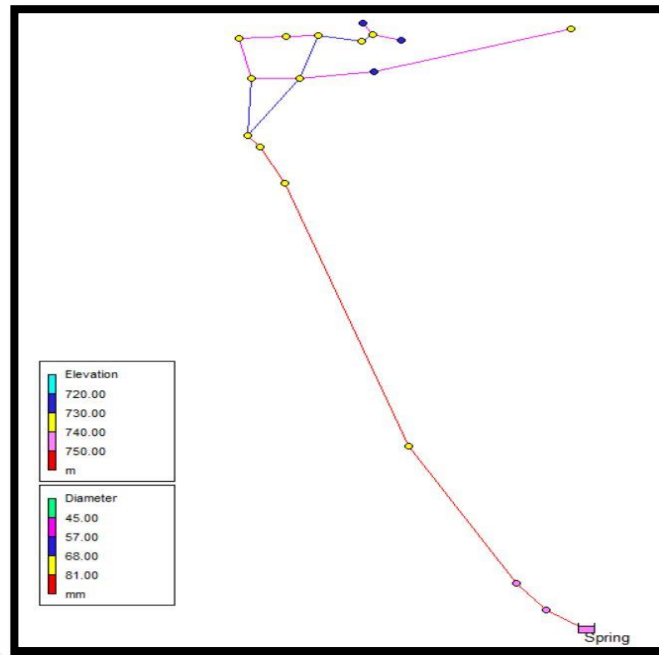


Figure 5.10 Elevation vs. diameter

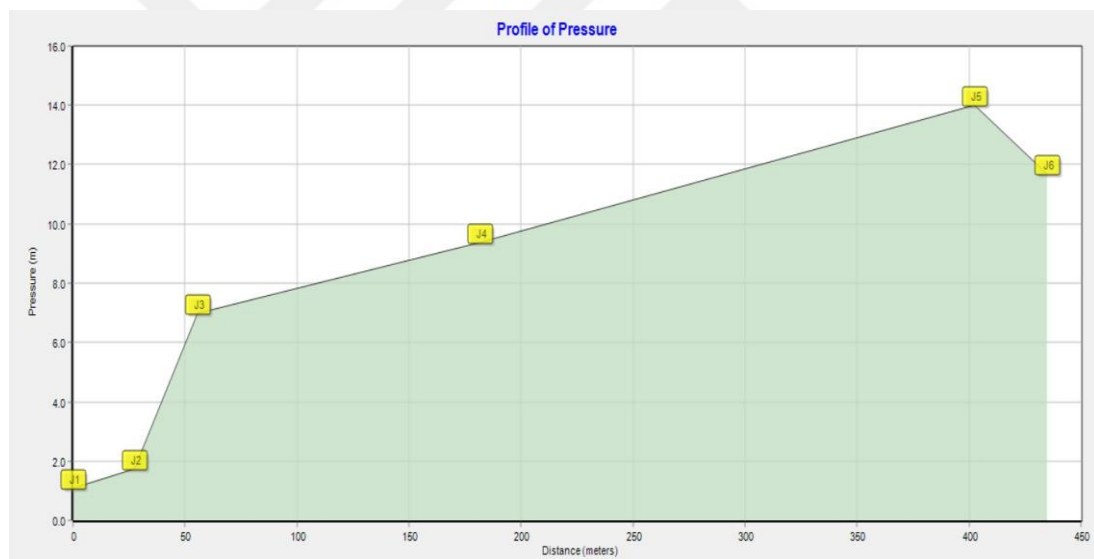


Figure 5.11 The pressure profile between node1 and node 6

Figure 5.11 detailed the variation of pressure with respect distance from the first junction (J1), then passing through nodes 2,3,4,5, and 6. In addition, in this study also investigated that the reversing relationship has been indicated between pressure and elevation as detailed in Table 5.1. Thus, the pressure at J1 recorded the minimum value as inlet point for the system, then the pressure gradually increased with decreasing the elevation till J5. Due to the geometry of the area's study, the pressure at J6 has been decreased compared with J5 because of increasing the elevation.

5.9 Discussion

Based upon the population forecasted, the Based Demand at each node was calculated and the pipelines have been designed then the entire network Hydraulic was analyzed. The values of the diameters of the pipes were assumed in a way that fits with the set limitations of the network, such as velocity inside the pipes, head losses, and pressure at each node or junction. The most suitable diameters were found so that it does not lead to an increase in the velocity in the pipes more than the specified upper limit of the network because this leads to the breakdown of the internal walls of the pipes and an increase in the energy losses resulting from the increase in friction between the flow and the internal walls of the pipes and thus increases the head losses and this what makes the network lose its efficiency, as the pipe size (diameter) controls the velocity for specific flow values. The head loss and energy losses are controlled by the velocity of the flow through pipes and the roughness of the pipe which depends of the pipe material (C factor of the pipe in Hazen-Williams formula). The maximum amount of head loss calculated is (5.8 m/km) in pipe P19. The velocities were adjusted in a way which leads to reduce the unit head loss, so the losses in the system can be reduced to an acceptable value, unit head loss in each node was lower than 6m/km.

The pressure in junctions J1-J2-J3-J4 were less than the minimum set limits, which is estimated at (10 m), as the pressure begins to increase from zero in the spring box gradually towards other points below where the altitude difference begins to increase between the spring and the following points, to become (1.07 m) at node number one until it reaches (14.01m) at node number 5, because of the increase in the difference in height of the water column by having an appropriate diameter of the pipes, some changes in the pressure head can be observed depending on the flow rate in the pipes and the geometry of the path and the elevation of the node. The highest pressure in the system is located at junction J10 which is (22.49m) at elevation of 725.10m.

Some junctions in the network do not have base demand (the base demand is equal to zero), as they are considered as points of connecting the pipes to each other, and there is no exit or consumption of water at these points. The negative demand at junction one refers to the inlet of the system where water enters the pipelines while the negative flows in the pipes (P1, P13, and P14) refer that the flow is in opposite

direction. The velocities were adjusted in a way which leads to reduce the unit head loss, so the losses in the system can be reduced to an acceptable value and this makes the water supply system more functional and effective. The system does not contain a storage tank and that for two main reasons, the first is the difference in height between the source and the distribution area is not sufficient to construct a storage tank providing enough pressure for the network, the second is the minimum discharge of the spring is (100 L/s) and this is greater than the design discharge which is (2.91 L/s), so it is enough to fulfill the daily demand for the village and there is no need for building a storage tank. The Head of the water and pressure calculated were sufficient to withdrawn the actual water demands at all the junctions (Nodes).



CHAPTER VI

SUMMARY AND CONCLUSION

6.1 Conclusion

In this research, the design of drinking water distribution networks was studied based on data on each of the area, population density, average daily demand of water for daily use, and through a topographic and geographic study of the study area, where a site survey was conducted to draw an illustrative map of the area showing the rises and depressions in addition to village layout, houses concentration areas and street layout. Flow of water distribution networks by the gravity are reliable and more economical than pumping system as no external power will be required to maintain and control the water flow. The purpose of this study is to design a water distribution network to supply a rural area with water from a sustainable water spring through a gravity piping network system, and see whether the water demand of consumers can be fulfilled with sufficient amount of water in all seasons through the year taking into account both current and forecasted population growth in 15 years. A village named "Fatawa" located in the northern part of Iraq, Erbil district, have been chosen as a case study for this purpose. The required data and information relevant to the village like population have been requested and collected from the ministry of agriculture and water resources, and also field survey observation have been performed using GPS Leica g15. Eventually, these data have been used as an input to construct the model of piping network, calculate the hydraulic head of water, and finally the water flow.

In this study, "EPANET" software has been utilized in modeling and simulation of piping network of approximately 930 meters, consisted of 20 pipes and 18 junctions. EPANET is free of charge and an open source hydraulic analysis program which is used for water distribution systems. The analysis of the piping system in this study

would have been intractable and time consuming procedure without the use of this software.

Upon the completion of the water network system analysis, the following points have been concluded:

- 1- Supplying water from spring to terminal destination with minimum losses and sufficient required flow rate to select a proper size of the pipe in the network. The assumed PVC pipe diameters in design were as 90, 65 and 50 mm, that maintained flow rate ranges between 0.11~2.82 l/sec. As a result, these sizes were still appropriate to meet the maximum required demand of water which was calculated as 2.9 l/sec and the head loss kept below 6 m/km in the system.
- 2- The velocities of water flow in those pipes were sufficient to convey the required amounts of flow. Velocities were controlled in a way to reduce the head losses in the system. This can be considered as another factor that makes assumed pipe diameters are appropriate to the purpose.
- 3- As a result of small values of water head that imposed by topography, there was no need to construct an intermediate break pressure tank in the system.
- 4- The pressure heads were within the set limits at each outlet node.

6.2 Recommendation

There are a lot of rural areas in northern of Iraq with majority located within the mountain regions. Unfortunately, supplying of water into those areas by network distribution system through the pump might not be in most cases as a simple process, furthermore, it would be economically infeasible. Using gravity water systems supplied from natural sources like water springs which presented in this study would be more reliable and economical. Moreover, the water provided through springs considered so clean as it undergoes an incredible natural filtration process through the layers of the earth that passes through.

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