Republic of Iraq
Ministry of Higher Education
and Scientific Research

ANALYSIS AND EVALUATION OF WATER SUPPLY FACILITIES FOR AL-HILLA CITY

A Thesis

Submitted to the College of Engineering of the University of Babylon in Partial Fulfillment of the Requirements for the Degree of Master of Science in Civil Engineering

By

THULFIKAR RAZZAK AL-HUSSEINI

Supervised by

ASST. PROF. DR. KARIM K. AL-JUMAILY
ASST. PROF. DR. ABDUL-HASSAN K. AI - SHUKUR

February ٢٠٠٤

جمهورية العراق
وزارة التعليم العالي و البحث العلمي

تحليل و تقييم متطلبات تجهيز الماء لمدينة الحلة

رسالة

مقدمة إلى كلية الهندسة في جامعة بابل
كجزء من متطلبات نيل درجة الماجستير
في علوم الهندسة المدنية

من قبل

ذو الفقار رزاق الحسيني

أشراف

أ.م.د. كريم خلف الجميلي
أ.م.د. عبد المحسن خضير الشكر

شباط ٢٠٠٤

ABSTRACT

In the supplementation of drinking water to civilians' communities, the most important think, is that the system should be capable to supply sufficient quantity of water with good quality.

In this research, the process of treated water services in Hilla city has been studied concerning the existed distribution network, optimal strategies for future extension up to year (٢٠٢٥), optimal rehabilitation, operational reliability, and water quality evaluation.

Concerning the water quantity, enough water could be treated and supplied reaching to (٩٩.١٢ %) of actual demand. In evaluating the performance of pipe network low standard operation and pipe constituent has been realized, a case which need careful rehabilitation in the aim of minimizing the required cost. (٣٥.٢٦ %) of cost has been achieved for rehabilitation compared with the cost of new optimum design.

In evaluating the water quality two aspects have been considered, first the quality of treated water in the treatment plant. The analysis was conducted for (turbidity, pH, electrical conductivity, calcium, magnesium, chloride, alkalinity, total hardness, T.D.S, and T.S.S). The second aspect is concerning the study of reliability for turbidity and hardness. Reliability of Hilla Al- Kadeem water treatment plant was found to be (٧٩.٩٥ %) for potable water turbidity, (٤.٤٦ %) for raw water turbidity, (٥١.٠٤٢ %) for potable water hardness, and (٤٧.٤ %) for raw water hardness.

In this research, five computer programs have been used, for analyzing pipe network, optimum design, optimum rehabilitation, reliability analysis, and (STATISTICA) program for statistical analysis of the data.

إلى خاتم الأنبياء (ص) وأئمة الهدى (ع)

إلى الحنان الذي مهد لي طريقي
إلى النور الذي ذابت شمعته ليضيء كتابي
والدتي و والدي
وإلى ذخري في الحياة
اخوتي وأخواتي

اهدي ثمرة جهدي المتواضع هذا ...

الخلاصة

تعتبر نوعية وكمية المياه المجهزة للتجمعات المدنية من المواضيع المهمة لعلاقتهما بالصحة العامة للمواطنين .

في البحث الحالي ، تم دراسة واقع حال خدمات تجهيز الماء الصافي لمدينة الحلة وذلك عبر عدة محاور ، هي ، تقييم كفاءة شبكة التوزيع الموجودة ومتطلبات الأمثلية للشبكة حتى عام (٢٠٢٥) وتحديد التأهيل الأمثل للشبكة واعتمادية التشغيل والتحليل النوعي للماء .

في ما يتعلق بكميات تجهيز المياه ، فقد تم التأكد من كفاية التجهيز بنسبة تصل إلى (% ٩٩.٣) من الطلب الفعلي . وفي مجال تقييم أداء شبكة التوزيع فقد تم التأكد من عدم كفاية المتطلبات التشغيلية و التقنية للشبكة ، الأمر الذي يتطلب إلى تأهيل المنظومة بما يلبي حالة تخفيض الكلفة بمقدار (% ٣٥.٢٦) مقارنة بكلفة تصميم وتنفيذ أمثل لشبكة جديدة .

وفي الجانب النوعي للمياه ، تم دراسة جانبين ، الأول : نوعية المياه المعالجة في المشروع ، حيث تم التركيز على أهم العناصر في هذا الجانب ومنها (العكورة ، الرقم الهيدروجيني ، التوصيل الكهربائي ، الكالسيوم ، المغنيسيوم ، الكلورايد ، القاعدية ، العسرة ، المواد الذائبة الكلية والمواد العالقة الكلية) ، والجانب الثاني يتعلق في دراسة ضوابط الاعتمادية للعكورة و العسرة . وقد وجد بان اعتمادية عكورة الماء لمحطة معالجة ماء الحلة القديم مساوية ل(% ٧٩.٩٥) للماء الصالح للشرب ومساوية ل(% ٤.٤٦)للماء الغير صالح للشرب وكذلك ان اعتمادية عسرة الماء لتلك المحطة مساوية ل(٪٥١.٠٤٢) للماء الصالح للشرب ومساوية ل(٪٤٧.٤) للماء الغير صالح للشرب .

لقد تم استخدام خمسة برامج للحاسوب لتحليل الشبكة وإجراء التصميم الأمثل والتأهيل الأمثل وتحليل الاعتمادية إضافة إلى البرنامج الإحصائي الجاهز (STATISTICA) لتحليل البيانات الخاصة بالموضوع .

Chapter One

Introduction

١.١: General

Water is an essential item for any living, without it, it is not possible to survive, it is a free gift of God to the human race. Much of that earliest activity is subject to speculation. Through out recorded history all ancient civilizations were developed along the riverbanks.

A water supply and distribution system are made up of a raw water supply, treatment facilities, pumping stations, storage facilities, and an integrated pipe network.

The analysis of such a system is a complex process due to the large number of pipes connecting various supply and demand points. The laws of conservation of mass and energy govern the distribution of flow in a pipe network under steady state conditions.

Several methods have been proposed for the solution of the water distribution system problems. The earliest analytical method dealing with the problem was proposed by Hardy-Cross (١٩٣٦) [٤١]. The advent of simple microcomputers has left no excuse for engineers to avoid computer solution to the problem.

After computers became available, there are several methods became easier to be used with a high accuracy for analysis of a water distribution system, such as the linear theory method, Newton-Raphson method, Finite-element method, etc.

Both analog and digital computers were used in analyzing distribution networks by the above methods. The first deals with continuous physical variables while the second is concerned only with numerical values. The analog computer acts as a physical model of the system to be studied and produces results, which are limited in accuracy only by the physical elements of the model. The digital computer is theoretically limited in accuracy only by the reliability of original input data.

The cost of pipe network and its fittings form more than (٥٠ %) from the total cost of project [٩], so pipe network is important in the stage of design and consideration of cost. In the scope of design the study has dealing to find the optimum design for pipe network to satisfy the best balance between the performance of pipe network and its cost.

The subject of performance deficit of pipe networks is very important, and this deficit result is due to the end of design age of pipe network, a case which causes cracks and leakage through pipes once the pressure is increased to supply the amount of water that satisfy the demand.

In general, reliability is defined as the probability that the system performs within specified limits for a given period of time. Traditionally, water distribution systems have been designed to be completely reliable. However, increasingly scarcity of public for construction and maintenance and the advanced age of many systems are causing the reliability of water distribution systems to become an important issue to water system designers and operators. Beyond a general agreement that systems should be “reliable” analysts do not concur on how reliability is defined measured or assessed for existing systems.

Conventionally, systems must conform to a limited set of reliability guidelines. For example, most systems are designed so that each demand point is supplied from two directions.

Contingency analysis may also be performed for a few cases, e.g., to ensure that the system will still perform adequately when one pump has failed.

However, reliability depends on the probabilistic occurrence of pipe and pump or any element failures, whereas these fixed guidelines treat reliability deterministically.

١.٢: Objectives of the research

The main objectives of this research can be summarized as follows:

١. Evaluating the existing water supply system of Hilla city and concerning both capability of supplying the present and future demand, and the water quality.

٢. Obtaining the optimal decision about the solutions required, between, designing a new system, or, rehabilitation and development of the existing network.

Chapter Two

Review of Literature

٢.١ : Introduction

Water distribution network involves the interaction of many processes. Two of these are the analysis of pipes network and the possibility of water pollution.

Many researches concerned with the methods of analysis, methods of design of pipes network, pipes material, location of tanks, reliability analysis, and quality of water. Classification of researches according to their scopes is presented in this Chapter.

٢.٢: Methods of analysis of pipe network

There are many methods for the analysis of pipe networks, among these methods are:

٢.٢.١: Hardy Cross method

This method was considered as the oldest and the most practical method for systematic solution of distribution network [٤١]. This method is well suited for application in design offices and can be easily adapted for computer computation. **Hoag and Weinberg, Graves and Branscome, Adams and**

Dillingham describe the computer programs written to perform the Hardy Cross analysis.

٢.٢.٢: Section method

Hoag and Weinberg (١٩٥٧) [١٨] had explained that section method is not a true analytical technique but it is a very valuable tool since it is capable to give a rapid approximate evaluation of the network systems. Following the determination of demands on a system, arbitrary drawn sections divide the network and the assumption is made that the hydraulic gradient is the same for all pipes crossing the section. With the properties of the pipes and the total flow across the section known, it is easy to calculate the actual hydraulic gradient at the section chosen. It is not, however, satisfactory for evaluating hydraulic conditions in a system with multiple constants –head input such as what is found in the reservoirs.

٢.٢.٣: Equivalent pipe length method

In (١٩٦١) Tong [٥٥] introduced the equivalent pipe length method, in this method all pipes diameters replaced by an equivalent diameter (٢٠٠ mm) and coefficient of Hazen-Williams (C = ١٠٠), and then takes the algebraic summation of equivalent pipes lengths in each loop and equalized to zero. After balancing the network, all pipes diameter (equivalent) return to its real diameters.

Raman and Raman (١٩٦٦) [٤٠] showed mathematically that the summation of equivalent length divided by discharge in each pipe must be equalized zero, not the summation of equivalent length of pipes.

٢.٢.٤: Newton-Raphson method

Shamir and Howard (١٩٦٨) [٤١] studied Newton-Raphson method. Newton-Raphson method solve equations by iterations and iterates on the set of equations simultaneously.

٢.٢.٥: Finite-Element method

Collins and Johnson (١٩٧٥) [٨] provided a method to analyze pipe network by the finite-element method, the two authors showed that the method has many advantages, most of these advantages hold true in a comparison with any loop method. The major advantage is the speed of convergence and the apparent lack of convergence problems of the proposed method over the Hardy Cross balancing flows method (Nodal method). Other important advantages are the ability to include in the analysis all types of hydraulic elements, the choice of flow-head loss relationships and the lack of artificial loops. Inspite of its advantages, this method can be considered more complex for programing and need large computer memory than Hardy Cross method; for example, if the network contains (١٠٠) nodes, the matrix consists of (١٠٠) columns and (١٠٠) rows.

٢.٢.٦ : Matrix form method

In (١٩٩٨) Jeppson [٢١] developed a new method, which depends on matrix form, the governing equation is:

P = J + L + F - ١. …… **(١-١)**

Where:

P: number of pipes in the system, which means there are (P) unknowns (Q-values).

J: number of junctions or nodes.

L: number of primary loops.

F: number of fixed grade nodes (that is, nodes that have known hydraulic grade line such as reservoirs and pumps) (heads).

The matrix formed by writing the continuity equation at each node and energy equation for each loop. These equations are nonlinear because of the nonlinear relationship between head losses and discharges, so, the equations must be linearized to be suitable for matrix solution. Various techniques exist to do this, but all required some types of iteration solution procedure. So, each head loss term is written as a relationship with discharge (Q) and the properties of pipe (K) are as follows [٢١]:

$$\mathbf{K_i Q^n_i = (K_i Q^{n-١}_i) Q_i \qquad , \{i = ١, ٢, ٣, ...\}} \qquad \text{....... (١-٢)}$$

To solve the system, a guess has to be done for the initial value of (Q) in each pipe so that the coefficient matrix ($K Q^{n-١}_i$) can be determined. Next, the matrix is a set of equations solved for new (Q) in each pipe, which are compared with previous (Q). If they are the same (to an acceptable level of accuracy) then the solution is terminated. If they are differ, then the calculated (Q) values are set as guess values and the process is repeated until the system converges.

٢.٣ : Design methods of pipe networks

Jacoby in (١٩٦٨) [٢٠] suggested a method of nonlinear programing for the optimal design of pipe network. This method find the optimum theoretical pipe diameters and then these diameters are approximated to the nearest commercial diameters, which were available.

AL - Ani in (١٩٨٥) [٢] suggested a method for optimum design, which was explained and used in the present study. The author mentioned that this method leads to a cost less obtained by the cost that the other methods with **(٤ %-١٦.٥ %)**. Also, he noticed that no optimum flow velocity can be acheived and the velocity of flow in pipes vary between **(٠.٠١-٢.٠) (m/s)**.

Nakashima and Wenzel in (١٩٨٦) [٣٦] developed an optimization model for designing an economical regional water supply system that consists of water production and water transmission facilities. Water demands are assumed to increase during the planning horizon and to be satisfied from several potentials supply sources. The method consists of two mathematical models. The phase I model selects a system layout, along with capacities in a transmission and source system subject to constrains on water availability at potential source and on increasing water demands. With a given solution from the phase I model as input, the phase II model determined facility components in the transmission system, i.e., pumping locations, capacities, and pipeline diameters.

Mathews in (١٩٩٥) [٢٩] introduced a numerical optimization procedure for complex pipe and duct network design, and shows that the design of optimum pipe and duct networks with available procedures is difficult, if not impossible. The design is formulated as a constrained nonlinear optimization problem. The problem is solved by using a unique numerical optimization algorithm. The solution entails the calculation of the cross sectional dimensions of the duct pipes so that the life cycle cost of the network is minimized. It was shown that the optimized solution would cost (١٤٪) less than the conventioneer design techniques.

In (١٩٩٩) Amnuewattanakul [٥] mentioned that the Hardy Cross method is one of the methods that used in designing the horizontal piping system which uses the mass and energy conservation of the fluid in the system. The author prepared a program for designing piping system. This program requires a sort of plant and an area to fix a desire-piping diagram. After that, the user must provide the basic data such as flow rates or the quantity of water needed in each section as the independent variables such as piping materials, diameter, length and fittings to the program.

٢.٤: Researches on pipes materials and behavior of buried pipes

There are many researches that concerning materials of pipes. **Stafford in (١٩٩٢)** [٤٢] explained that the most commonly used pipes materials are PolyVinyl Chloride, known as PVC and High-Density Polythene, known as HDPE. Under special circumstances, especially when the pipeline has to withstand high pressures, Galvanized Iron (GI) pipes are used. Materials of PVC pipes degrades when exposed to sunlight, losing part of its strength and becoming brittle, care should, therefore, be taken to cover PVC pipes when they are stocked in the open. He, also, concluded that the Asbestos – cement pipes for human water supply should be avoided.

٢.٥ : Location of tanks

Deb and Sharker in (١٩٧١) [١٠] found that the cost of a water distribution system depend on the position of the elevated service tanks. The cost is the maximum when it is located at one corner and is the minimum when it is placed at the center of the network. In the problem considered in his study, the total cost of the system is found to be (١.٣٣) times more when the reservoir position is at one corner of the network than when it is at the center.

While **Steel** [٥١] suggested the locations of tanks at distribution systems. He stated that location of tanks at the opposite side of pumping station would reduce cost. During a high consumption a certain region will be feeded from the two sides and the head losses in the main pipes will reduce approximately to the quarter. When the tanks lie near pumping station they cause a weak hydraulic grade line and low pressure at the remote regions.

٢.٦: Reliability analysis

Su et. al. in (١٩٨٧) [٥٣] evaluated reliability using "minimum cut set". A minimum cut set is a set of system components which, when failed, causes failur of the system. They present the basic framework for a model that can be used to determine the optimal (least-cost) design of a water distribution system subject to continuity, conservation of energy, nodal head bounds, and reliability constraints. The overall model includes three models that are linked: a steady-state simulation model, a reliability model, and an optimization model. The simulation model is used to implicity solves the continuity and energy constraints and is used in the reliability model to define minimum cut sets. The reliability model, which is based on a minimum cut set method, determines the values of the system and nodal reliability. The optimization model is based on a generalized reduced-gradient method.

Fujiwara in (١٩٩٣) [١٣] analyzed the reliability of water supply systems consisting of water source, water treatment plant, storage, pumping station, and a lumped demand node in series. The random fluctuation of the water quality of the source water, the limited water-processing rate of the treatment plant, random pump failure and repair and random demand variation are considered for the assessment of the system reliability. Expected demand shortage is employed as a measurement of system reliability. The decision to be made is the optimal water-processing rate at the treatment plant at each period so that the total expected shortage over a finite time horizon is minimized. Dynamic programming is employed to formulate the model.

Gupta in (١٩٩٤) [١٥] showed that the traditional network analysis (Hardy Cross, Newton-Raphson) presumes that the nodal demands are always satisfied in a water-distribution system and determines the available heads- however, when a pump fails or pipe breaks, the water-distribution system may be unable to supply all nodal demands at required heads. Thus, the traditional network analysis does not correctly describe the partially failed water-distribution system. In reliability analysis of water-distribution system, however, the nodal flows that would be available under deficient conditions should be evaluated and used. Therefore, an approach termed node flow analysis that determines the available nodal flows under deficient conditions by considering the nodal demands and heads, simultaneously, is presented for determining water-distribution system reliability. The reliability is based on a node-reliability factor, volume-reliability factor, and network-reliability factor. Even though water-distribution system reliability depends on several parameters, only the pipe break and pump failure conditions are considered. However, several loading patterns, including fire flow requirements, can be considered as illustrated by a hypothetical example.

Kim (١٩٩٤) [٢٢] introduced a new methodology that can select the pipes to be rehabilitated and / or replaced in an existing water-distribution system. Also, determined the increase in pumping capacities so that the water demand and pressure requirement at all demand nodes are satisfied while the total rehabilitation and energy cost is minimized. Four cost functions are considered: pipe replacement cost, pipe rehabilitation cost, pipe repair cost and pumping cost. The methodology considers the trade-off among decisions regarding each pipe: replace pipes, reline pipes, or leave as it is.

In (١٩٩٨) Al-Shaibani [٤] from his research work on reliability of networks, reported two aspects, first, is to determine the degree of reliability of networks under different states, which in this work considered the daily pattern flow, the demand excess, and the break pipes. Then determined the reliability factors. Second aspect, is to improve the network by using different alternatives based on modification the head at nodal demands by various methods such as increasing diameters, friction coefficient C_{HW}, booster pumps, increase fixed head nodes, or any method of improvement to the head at nodal demands with small different in cost considering the original network is conserved. Then select the best alternate with least cost.

٢.٧: Water quality analysis

Myers (١٩٩٥) [٣٥] mentioned that the copper tubing in potable water systems is highly resistant to corrosion. However, pitting attack will occur in tubing carrying cold water with an aggressive chemistry (typically, pH of ٧.٠ to ٧.٧ and dissolved carbon dioxide of at least ٢٥ mg / ℓ). The most cost – effective method for preventing this pitting is altering the water chemistry by raising the pH and reducing carbon dioxide content.

The relative efficiencies of slow sand filtration and rapid gravity filtration system were discussed by **Lambert (١٩٩٥)** [٢٣]. Relative efficiencies were about the removal of pesticides, organic colour and nutrients, as well as their abilities to deal with pollution incidents and their relative costs. In general, slow sand filtration systems were found to be poorer than coagulant-assisted rapid gravity filtration systems for the removal of organic colour, but more efficient for the removal of several commonly occurring pesticides. Slow sand filtration was also process for the production of biologically stable finished waters and, for small to medium-sized plants, appeared to incur no greater capital costs than comparable rapid sand filtration plants.

Mechi in (٢٠٠١) [٣١] studied the evaluation of water quality in Najaf city in two sides, first related with the study of supplied water quality in the net for the parameters (turbidity, pH, electrical conductivity, calcium, magnesium, chloride, total hardness, alkalinity, T.D.S, and T.S.S) by taking samples from the terminal places of the net and testing them, in Najaf city water services center lab, the results were compared with standard specification and conditions. Second, is the studding of the data information for the period from January to December (٢٠٠١) about the results of raw water test and drinking water test in the treatment plant for the same group of that maintained properties and sketching the relations. Also, he studied the quantity of water, which arrived to inhibition places. From the test results has been found that the quantity of water is in the required level, although there is some needes in some places in the city.

Holton Purification Plant (٢٠٠٢) report [١٩], has given a study about reasons which make water sometimes " Cloudy " looking in the winter. Plant's experience has shown that the cloudiness is simply the result of excess air in the water. Under certain conditions, water is capable of becoming supersaturated with dissolved air. This is a common occurrence during the winter months of the year and is due to the ability of cold water to retain large quantities of dissolved air, which is kept in solution mainly by temperature and pressure. As the water temperature is increased and the pressure released (as in opening the faucet) the dissolved air rapidly comes out of solution, imparting a temporary, cloudy appearance to the water. The "Cloudy " appearance is due to the sudden formation of tiny air bubbles, which slowly rise to the top. This condition usually lasts a minute or two, and then it will be clear. Although it is not a health hazared, entrapped air can impart an aesthetically unpleasant appearance to the water. If the consumer finds this appearance too unappetizing, a simple remedy is to fill a container with cold water and place it on the counter or in the refrigerator. Under normal pressure conditions, the air will quickly dissipate in a few minutes and the water may then be used for drinking and cooking purposes. Also this plant produced a study about using hot water from the tap for cooking, in the same year (٢٠٠٢). The study shows that the using hot water directly from the tap for cooking is generally not recommended. Hot water is more likely to contain dissolved metals such as iron, copper, and lead, picked up from the household plumbing and hot water tank. A better idea is to allow the cold water to run for a few minutes until good and cold, and then use this water for cooking and other consumption purposes. Allowing the water to run its coldest insures adequate flusing of the home’s water service line and the interior household plumbing, which have both been identified as sources of copper and lead contamination in drinking water.

In (٢٠٠٣) Hashim [١٦] drew a conclusion when he used Niku et. al method, that the dependability study of Al – Kerkh wastewater treatment plant showed that the performance of this plant before the war of ١٩٩١ was in an excellent state and within the Iraqi standards. Where the treatment process was stable during that period and the overall reliability was ٩٦.٦٩ % for effluent $BOD_٥$ and ٩٩.٦١ % for effluent S.S. But after the war of ١٩٩١, the plant became unstable and the overall reliability dropped to ٥٢.١٣ % for effluent $BOD_٥$ and to ٦١.٠٠ % for effluent S.S.

Chapter Three

Basic Equations and Methods of Analysis

٣.١: Introduction

Water distribution system are needed to convey the water drawn from the source, through treatment and storage facilities, to the points where it is delivered to the users. Pipe network consists of pipes, which connected with each other to establish nodes (draw off nodes or supply nodes) and loops in pipes distribution system.

There are two types of piped distribution systems:

١- Branched systems are those that convey water from a distribution main to different consumption points, following a tree like pattern, all their branches finish in dead ends. Their design is straightforward but has a main disadvantage in the fact that it causes stagnant water pockets in all dead ends. If repairs are necessary, large areas must be cut off from service. Head losses, due to heavy local demands-or during a fire- may be excessive unless the pipes are quite large.

٢- Looped network systems usually have ring mains to which secondary pipes may be connected. Their design is much more complicated, with them the possibility of stagnant water is reduced. If part of the pipeline needs cleaning or repair, it may be isolated from the rest of the system (with appropriate valves); all watering points outside of it may continue to be supplied.

٣.٢: Analysis methods of pipe networks

Some of the analysis methods of pipe networks- that generally discussed in the previous chapter- are:

١- Hardy Cross method.
٢- Finite-Element method.
٣- Newton-Raphson method.
٤- Section method.
٥- Equivalent pipe length method.
٦- Matrix form method.

٣.٢: General equations for network analysis

The equations to analyze a network of pipe in a water distribution system are derived from conservation of mass and conservation of energy as formed below. The mass conservation is [٢٨]:

$$\frac{M_{t+\Delta t} - M_t}{\Delta t} = \sum_{i=1}^{Nd} M_i \qquad \dots (٣\text{-}١)$$

Where:

M_t: mass of the flow (F $T^٢$/L).
$M_{t+\Delta t}$: mass of flow after (Δt) time (F $T^٢$/L).
Nd: number of pipes, which connected with a certain node.
M_i: mass flow rate (F T/L).

For steady state, incompressible flow in a pipe, the conservation of mass equation becomes the continuity equation, and the left hand side of the equation (٣-١) becomes zero. This implies that the total mass flow rate into a node is equal to the total mass flow rate out of the node, which means $\sum Q = ٠$.

Under the same assumptions, plus assuming there are no pumps or turbines in the system, the conservation of energy equation says that for any primary loops $\Sigma h_f = ٠$. Also the flow-head loss relationship (such as Darcy-Weisbach or Hazen-Williams) must be satisfied for each pipe. When the discharge flowing through the pipe, there are two types of losses; major losses are due to friction between the moving fluid and the inside walls of the pipe; and minor losses are due to fittings such as valves and elbows. The minor losses can be neglected when the ratio between the length and the diameter is large [٢٥].

There are many equations to calculate the major losses. From these equations are:

٣.٣.١: Darcy-Weisbach equation

The equation of Darcy-Weisbach for head losses is: [٢٦].

$$h_f = f \frac{L}{D} \frac{V^2}{2g} \qquad \text{… (٣-٢)}$$

Where:

h_f: head losses in pipe (L)

f: Moody friction factor (dimensionless)

L: length of pipe (L)

D: pipe diameter (L)

V: average velocity of flow in pipe (L/T)

g: acceleration due to gravity = ٣٢.١٧٤ ft/s^٢ = ٩.٨٠٦٦ m/s^٢

In order to solve problems by the equation (٣-٢), it is necessary to have a mathematical formulation for the friction factor, f, in terms of Reynolds number, R_e=VD/υ, and relative roughness, e/D. To calculate friction factor, f, there are three main formulas depend on Reynolds number and relative roughness [٢٦].

If laminar flow {R_e<٤٠٠٠ and any e/D} then:

$$f = \frac{64}{R_e} \qquad \text{… (٣-٣)}$$

If turbulent flow {٤٠٠٠ < R_e< ١٠^٨ and ٠ < e/D < ٠.٠٥} then Colebrook equation is applicable:

$$\frac{1}{\sqrt{f}} = -2.0\log(\frac{e/D}{3.7} + \frac{2.51}{R_e\sqrt{f}}) \qquad \text{… (٣- ٤a)}$$

Equation (٣-٤a) is transcendental, so iteration is needed to evaluate f. Miller suggested that a single iteration would produce a result within ١ percent if the initial estimation were calculated from the following equation: [١١].

$$f_0 = 0.25[\log(\frac{e/D}{3.7} + \frac{5.74}{R_e^{0.9}})]^{-2} \qquad \text{… (٣- ٤b)}$$

If fully turbulent flow (R_e > ١٠^٨ and ٠ < e/D < ٠.٠٥), then:

f = [١.١٤-٠.٨٦٩ Ln (e/D)] ^-٢. … (٣-٥)

Roughness, e, is obtained from table (٣-١) as follows:

Table (٣-١): Roughness for pipes of common engineering materials [١١].

Pipe	Roughness (mm)
Riveted Steel	٠.٩-٩
Concrete	٠.٣-٣
Wood Stave	٠.٢-٠.٩
Cast Iron	٠.٢٦
Galvanized Iron	٠.١٥
Asphalted Cast Iron	٠.١٢
Commercial Steel or Wrought Iron	٠.٠٤٦
Drawn Tubing	٠.٠٠١٥

Also, the friction factor, f, can be calculated from Moody diagram. All of (e) values given in table (٣-١) are for new pipes, in relatively good condition. Over long periods of service, corrosion takes place and, particularly in hard water areas, lime deposits and rust scale form on pipes walls. Corrosion can weaken pipes, eventually leading to failure. Deposit formation increases wall roughness appreciably and also decreases the effective diameter. These factors combine to cause (e/D) to increase by factors of ٥ to ١٠ for old pipes [١١]. Typical multipliers to be applied to friction factor for considering the aging of pipes were shown in the following table.

Table (٣-٢): Typical multipliers to be applied to friction factor [١١].

Pipes Age (Years)	Small Pipes (١٠٠-٢٥٠) Diameter (mm)	Large Pipes (٣٠٠-١٥٠٠) Diameter (mm)
New	١.٠٠	١.٠٠
١٠	٢.٢٠	١.٦٠
٢٠	٥.٠٠	٢.٠٠
٣٠	٧.٢٥	٢.٢٠
٤٠	٨.٧٥	٢.٤٠
٥٠	٩.٦٠	٢.٨٦
٦٠	١٠.٠	٣.٧٠
٧٠	١٠.١	٤.٧٠

٣.٣.٢: Hazen-Williams equation

Equation of Hazen-Williams [٢٧] can be also used to calculate the head losses in pipes. The well-known form is:

$$V = K\ C\ R^{٠.٦٣}\ S^{٠.٥٤} \qquad \ldots (٣\text{-}٦)$$

Where:

S: slope of energy line (h_f /L), dimensionless.

K: unite conversion factor (K = ١.٣١٨ for English units, K = ٠.٨٥ for SI units).

R: hydraulic radius = D/٤ for circular pipe (L).

V: average velocity of flow in pipe (L/T).

C: Hazen-Williams coefficient (dimensionless). Its values may be obtained from table (٣-٣).

Table (٣-٣): Hazen-Williams coefficient [٢٥].

Material	C	Material	C
Asbestos Cement	١٤٠	Copper	١٣٠-١٤٠
Brass	١٣٠-١٤٠	Galvanized Iron	١٢٠
Brick Sewer	١٠٠	Glass	١٤٠
Cast Iron:		Lead	١٣٠-١٤٠
New, unlined	١٣٠	Plastic	١٤٠-١٥٠
١٠ Years old	١٠٧-١١٣	Steel:	
٢٠ Years old	٨٩-١٠٠	Coal-Tar enamel lined	١٤٥-١٥٠
٣٠ Years old	٧٥-٩٠	New unlined	١٤٠-١٥٠
٤٠ Years old	٦٤-٨٣	Riveted	١١٠
Concrete/Concrete lined:			
Steel Forms	١٤٠	Tin	١٣٠
Wooden Forms	١٢٠	Vitrify Clay (good condition)	١١٠-١٤٠

A comparison between Darcy-Weisbach equation and Hazen-Williams equation indicate that, Darcy-Weisbach equation is generally considered more accurate than Hazen-Williams equation. Additionally, Darcy-Weisbach equation is valid for any liquid or gas; Hazen-Williams equation is valid only for water at ordinary temperatures (٤٠ to ٧٥ F^{o}; ٤-٢٥ C^{o}) [٢٥]. Hazen-Williams equation is very popular, especially among civil engineering, since its friction coefficient (C) is not a function of velocity or pipe diameters. Hazen-Williams equation is simpler than Darcy-Weisbach equation for calculating flow rate, velocity, or diameter of pipes.

To solve pipe network (finding head loss and discharge in each pipes), a group of nonlinear equations must be solved, since the relation between head loss and discharge is nonlinear.

In the present research, both Darcy-Weisbach and Hazen-Williams equations are used to find head losses.

٣.٤: Hardy Cross methods to analyze pipe networks

Hardy Cross method may be considered as a very popular method with its two types described below. This method also known as a single path adjustment method and it is a relaxation method.

٣.٤.١: Head balance method

This method is also called the loop method. It can be summarized by assuming an initial discharge in each pipe keeping in mind the continuity equation must be satisfied in each node ($\sum Q = ٠$). The initial assumed discharge could be corrected by adding the correction (ΔQ) derived below until the corrections reached the allowable errors to satisfy the energy equation.

The general relationship between head loss and discharge is ($h_f = K Q^n$). The correction (ΔQ) can be derived by differentiate the previous relationship [١٢].

$dh_f = n K Q^{n-١} dQ.$ …(٣-٧)

Then substituting for ($K = h_f / Q^n$) in equation (٣-٧) to get:

$$dh_f = n \frac{dQ}{Q} h \quad ...(٣-٨)$$

So, the error in (h_f) due to an error in (Q) has already been established as:

$$dh_i = n\frac{dQ_i}{Q_i}h_i \qquad \ldots(٣\text{-}٩)$$

Similar expressions for the other pipes in a loop, the total error round the loop is therefore:

$$\Delta h_f = dh_{f١} + dh_{f٢} + dh_{f٣} + \ldots. + dh_{fi}. \qquad \ldots(٣\text{-}١٠)$$

By substituting for dh_i then:

$$\Delta h_f = [\frac{nh_f1}{Q1} + \frac{nh_f2}{Q2} + \frac{nh_f3}{Q3} + \ldots + \frac{nh_{fi}}{Q_i}]dQ \qquad \ldots(٣\text{-}١١)$$

Where (dQ), correction in each pipe in the loop assumed equal, That is:

$$dQ = \frac{\Delta h_f}{n\sum\frac{h_i}{Q_i}} \qquad \ldots(٣\text{-}١٢)$$

Equation (٣-١٢) can be rewritten in other way since Δh equals to the summation of head losses in loop:

$$\Delta Q = \frac{\sum_{i=1}^{NPL} hfi}{n\sum_{i=1}^{NPL}\frac{hfi}{Qi}} \qquad \ldots(٣\text{-}١٣)$$

NPL: Number of pipes in the loop.

The error (ΔQ) that is be corrected by applying a balancing discharge of the same magnitude but opposite in sign.

٣.٤.٢: Quantity balance method

This method can be also called nodal method. It can be summarized by assuming an initial head in each node keeping in mind the energy equation must be satisfied in each loop (∑hf = ٠). Then by the correction item that derived in equation (٣-٨) the assumed head in each node can be corrected.

Equation (٣-٨) can be rewritten as:

$$\Delta h = \frac{\Delta Q}{\sum_{i=1}^{NPj} \frac{Q_i}{nhfi}} \qquad \ldots(٣\text{-}١٤)$$

Equation (٣-١٤) can be used when there is no supply or drawn off at node. But if there is supply or drawn off at node equation (٣-١٤) can be rewritten as:

$$\Delta h = \frac{\sum_{i=1}^{NPj} Q_i + Q_T}{\sum_{i=1}^{NPj} \frac{Q_i}{nhfi}} \qquad \ldots(٣\text{-}١٥)$$

Where:

Npj: number of pipes connected with node.

Q_T: supply or drawn off water at node, in case of supply (Q_T) take the negative sign, and in case of draw off, Q_T, take the positive sign.

The error (Δh) that is be corrected by applying a balancing head of the same magnitude but opposite in sign. Then by repeating adding the correction to all pipes connected with node until the corrections reach the allowable errors. This involves all nodes.

٣.٤: Comparison between two methods of Hardy Cross

In loop method, the number of equations are equal to the number of pipes in the network, while nodal method has a number of equations equal to the number of nodes in the network, that means that the number of equations in nodal method are less than the equations in loops method.

In loop method any additional loop leads to repeat assuming the initial discharge in all pipes, but in nodal method must be satisfied energy equation in additional loop only.

Dillingham and Cleasby [٨] pointed out that when using loop method, a pipe or pipes with high resistance to flow compared with others in the network can result in calculated flow corrections larger and in the opposite direction to the currently assumed flow. This will often cause a divergence in the computations, and no solution can be obtained. When the nodal method is used, **Dillingham and Cleasby** pointed out that if a large pipe of short length, and relatively low flow exists, much iteration are necessary before an appreciable change in piezometric head is obtained if the values of the assumed piezometric head is incorrect.

Chapter Four

Data Collection and Hydraulic Analysis of Water Scheme for Hilla City

٤.١: Introduction

The water demand depends on the resident populations and type of industry served. With rising standard of living the trend of the domestic consumption is usually increased.

In this Chapter the population, water consumptions, pumping stations, pipe network and other related activeties of water scheme for Hilla city has been analyzed.

٤.٢: Expected population of Hilla city and some villages around

Hilla city populations as based on the census of the years (١٩٨٧) and (١٩٩٧) were (٢١٧٩٠٢) and (٢٥٩٤٩٩) respectively [٤٣], [٤٤]. Making use of these numbers and by using eq. (٤-١) [٤٥], the average yearly increasing rate can be calculated, which is found equal to (١.٩ %).

$$P_t = P_o\, a\, t + P_o. \qquad \text{...... (٤-١)}$$

Where:

P_t: population after (t) years (person).

P_o: current population (person).

a: average yearly increasing rate (%).

Table (٤-١) shows the expected population in Hilla city for the year (٢٠٠٢) and the actual number of population for the year (١٩٩٧).

Some villages that lie around Hilla city fed with water from pumping station at Hilla city. These villages is within the territory of Babylon governorate, but not belongs to Hilla city, so the average yearly increasing rate of its populations is considered the same as the average yearly increasing rate of Babylon governorate except Hilla city. The total number of populations in Babylon governorate at years (١٩٨٧) and (١٩٩٧) were (٨٩٧٨٧٧) and (١١٨١٧٥١) respectively [٤٣], [٤٤]. By using eq. (٤-١), the average yearly increasing rate for these villages can be calculated.

$P_{o\,(١٩٨٧)}$ = ٨٩٧٨٧٧ - ٢١٧٩٠٢ = ٦٧٩٩٧٥

$P_{t\,(١٩٩٧)}$ = ١١٨١٧٥١ - ٢٥٩٤٩٩ = ٩٢٢٢٥٢

The average yearly increasing rate for these villages is (٣.٦ %). Table (٤-٢) shows the expected populations in Hilla city.

Table (٤-١): Expected population in Hilla city for the year (٢٠٠٢).

Quarter Name	**Actual population (١٩٩٧)**	**Expected population (٢٠٠٢)**	**Quarter Name**	**Actual population (١٩٩٧)**	**Expected population (٢٠٠٢)**
Kreta'a	٢٣٣٥	٢٥٥٧	Jubaween	٢٤٣٨	٢٦٧٠
Theila	٢٨٣٠	٣٠٩٩	Ta'aees	٢٨٤٦	٣١١٦
Galage	١٤٩٤	١٦٣٦	Krade	٢٤٢٠	٢٦٥٠
Wardia	١١٧٩	١٢٩١	Tayara	٤٩٨٧	٥٤٦١
Khisirwia	٢٠٥٩	٢٢٥٥	Naseege	٧٠٥٥	٧٧٢٥
Babel	١٢٢١	١٣٣٧	Iskan	٦٩٦٤	٧٦٢٦
Jaza'ir	٢٣٥٢	٢٥٧٥	Jema'aia	٥٥٣٩	٦٠٦٥
Bakarly	٧٢٤٦	٧٩٣٤	Murtadha	٤١١٠	٤٥٠١
J.M.Bakarly	٩٢٧٧	١٠١٥٨	Hussein	٣٦٠٥	٣٩٤٨
Nadir٣	٦٦٥٩	٧٢٩٢	Makhazin	١٧٨	١٩٥
Quarter Name	**Actual population**	**Expected population**	**Quarter Name**	**Actual population**	**Expected population**

	(١٩٩٧)	(٢٠٠٢)		(١٩٩٧)	(٢٠٠٢)
M.Sulaiman	٥٤١٢	٥٩٢٦	Faza'a&Mane'a	٢٥٣٦	٢٧٧٧
Nadir١	٣٣٦٤	٣٦٨٤	Dhu.in Wessia	٩٨١٨	١٠٧٥١
Nadir٢	٩٥٩٤	١٠٥٠٥	Hamza Dally	١٧٠٤	١٨٦٦
Zahraa	١٣٥٢	١٤٨٠	Akrameen	٧٨٨٥	٨٦٣٤
Shawy	٦٥٣٥	٧١٥٦	Shubar	٤٠٨٥	٤٤٧٣
Judeida	٦٨٥٣	٧٥٠٤	Kadhia	٢٨١٨	٣٠٨٦
Jumhury	٨١٥	٨٩٢	Karama	٩٢٨٦	١٠١٦٨
Mashta	١٠٠٢	١٠٩٧	Hukam	١٠٥٢	١١٥٢
Ibrahimia	٢٦١٥	٢٨٦٣	Imam	٧٧٤١	٨٤٧٦
Mustafa R.	٤٥٦٨	٥٠٠٢	Dhu.in Mekruy	٣١٥٥	٣٤٥٥
Ameer	١٥٩٥	١٧٤٧	Shuhedaa in Mekruy	٣٦٧٨	٤٠٢٧
Jama'ein	٣٨٦٩	٤٢٣٧	Askery in Mekruy	٥٣٩٧	٥٩١٠
Take	٣٣٠١	٣٦١٥	Muharbeen	٤٤٨٠	٤٩٠٦
Jubran	٥٩٩	٦٥٦	Asatetha	٦١٧٥	٦٧٦٢
Mehdia	٤٠٣٧	٤٤٢١	Shuhedaa	١٠٠٨٥	١١٠٤٣
Nuwab Dh.	٨٠٣٦	٨٧٩٩	Muhendseen	٧٧٥٥	٨٤٩٢
Nuwab Dh.٢	٣٦٩١	٤٠٤٢	Jelaween	٩٠٥	٩٩١
Thawra	١٤٥٤٢	١٥٩٢٤	١٧-Tamuze	٤٥٨٤	٥٠٢٠
Marana	٢٩٦٦	٣٢٤٨	Afrah	٢٩٩٣	٣٢٧٧
Tineia	١٣٧٧	١٥٠٨	Dur-Babil	٤٥٠	٤٩٣
Total populations in (١٩٩٧) = ٢٥٩٤٩٩			Total populations in (٢٠٠٢) = ٢٨٤١٥٦		

Note: Number of population for the year (١٩٩٧) was taken from reference [٤٤].

Table (٤-٢): Expected populations in some villages around Hilla city for the year (٢٠٠٢).

Village Name	Actual population (١٩٩٧)	Expected population (٢٠٠٢)	Village Name	Actual population (١٩٩٧)	Expected population (٢٠٠٢)
Tuhmazia	٥٨٩٨	٦٩٦٠	Buhamyar	٢١١٦	٢٤٩٧
Luba	٧٣٢	٨٦٤	Wardia Kharege	٢٠٦٧	٢٤٣٩
Jimijma	٢٦٥٠	٣١٢٧	Abu-Ajaje	١٩٣٨	٢٢٨٧
Abu-Ilayan	١٥٠٣	١٧٧٤	Seife Sa’ad	١٨٤٤	٢١٧٦
Banasha	٩٠١	١٠٦٣	Efar	٩١٢	١٠٧٦
Rashedia	٩٢٤	١٠٩٠	Mua’mera	١٧٦٥	٢٠٨٣
Obed Radam	١٢٤٤	١٤٦٨	Ataije	٣٧٧٥	٤٤٥٥
Abu-Gharak	٥٩٣٤٢	٧٠٠٢٤	Anana	٢٤٧٢	٢٩١٧
Sinjar	٢٤٥٥	٢٨٩٧	Kura Muhezim	٣٠٥١	٣٦٠٠
Mia’dan	٨١٩	٩٦٦			

Note: Number of populations for the year (١٩٩٧) was taken from reference [٤٤].

There are (١٤) projects of water supply work in Babylon governorate; some of these projects are perhaps old and need to maintenance, of which are the Methatia and Hilla Al- Kadeem projects, which had already repaired in the year (١٩٩٩) [٤٨]. Also, some of these projects were put out of work such as Kadessia project [٤٩].

٤.٣: Water treatment plants and pumping stations in Hilla city

Hilla city has three water treatment plants: Hilla AL-Kadeem, Tayara AL-Kadeem, and Hilla AL-Jadeed (Abu-Khistawy). Besides, it has four pumping stations: Tayara AL-Jadeed, Tayara AL-Kadeem, Muhezim, and Tuhmazia.

٤.٣.١: Water treatment plants

Table (٤-٣) shows the water treatment plants and the hydraulic properties of the pumps used in each plant.

Table (٤-٣): Pumps properties of the Hilla city water treatment plants [٥٠].

Plant	Pumps type	Total pumps	Working pumps	Capacity of pump (m^{٣}/hr)	Head (m)	Working time (hr)	Total capacity (m^{٣}/day)
Hilla Al-Kadeem	Low lift	٤	٤	٤٠٠	١٢	٢٤	٣٥٨٠٨
				٣٤٢	٢٠		
				٣٥٠	٣٠		
				٤٠٠	١٢		
	High lift	٤	٣	٣٨١	٣٠	٢٤	٢٧٤٣٢
Tayara Al-Kadeem	Low lift	٤	٣	٧٥٠	٣٠	٢٤	٥٤٠٠٠
	High lift	٥	٤	٤٠٠	٣٠	٢٤	٢٨٨٠٠*
							٩٦٠٠**
Hilla Al-Jadeed	Low lift	٦	٦	١٦٣٥	١٣	٢٤	٢٣٥٤٤٠
	High lift	٣	٢	٧٢٥	١٩	٢٤	١٥٢٤٠٠
		٣	٢	١١٥٠	١٦	٢٤	
		٣	٢	١١٥٠	١٠	٢٤	
		٣	٢	١٥٠	٦٠	٢٤	

*: Serving Hilla city

**: Serving Abu- Gharak regions (Abu-Gharak, Sinjar, Kura Muhazim, and Anana)

Hilla Al- Kadeem water treatment plant is the oldest one among the three water treatment plants in Hilla city and it had many technical problems upon which it has subjected to necessary maintinance in (١٩٩٩) [٤٨]. Before this maintenance, it was serving Tayara AL-Jadeed pumping station and the network. After this maintenance, this plant started serving Tayara AL-Jadeed pumping station only.

The capacities which reach to Tayara AL-Jadeed, Muhezim, Tuhmazia pumping stations, and Abu-Garak regions from Hilla Al- Jadeed water treatment plant are: (٣٤٨٠٠, ٥٥٢٠٠, ٥٥٢٠٠, ٧٢٠٠) m^{٣}/day respectively so the total capacity of this plant is (١٥٢٤٠٠) m^{٣}/day.

٤.٣.٢: Pumping stations in Hilla city

There are three pumping stations in Hilla city; there is also Tayara AL-Kadeem pumping station in Tayara AL-Kadeem water treatment plant. Table (٤-٤) shows some properties of the mentioned stations.

Table (٤-٤): Pumping stations in Hilla city [٥٠].

Station	Total pumps	Working pumps	Capacity (m^{٣}/hr)	Head (m)	Working time (hr)	Total capacity (m^{٣}/day)
Tayara Al-Jadeed	٣&	١	٦٠٠	٣٦	٢٢	١٣٢٠٠
	٣&&	١	١٢٠٠	٣٥	٢٢	٢٦٤٠٠
Tuhmazia	٦	١	١٢٠٠	٣٧	٢٤	٢٨٨٠٠
Muhazim	٦	١.٥	١٢٠٠	٥٨	٢٤	٤٣٢٠٠
Tayara Al-Kadeem	٥	٤	٤٠٠	٣٠	٢٤	٢٨٨٠٠*
						٩٦٠٠**

&: Serving Marjan area.

&&: Serving Jibal area.

*: Serving Hilla city

**: Serving Abu- Gharak regions (Abu-Gharak, Sinjar, Kura Muhazim, and Anana).

The total capacity for the four pumping stations is (١٥٠٠٠٠) m٣/day, which equals (١.٧٣٦ m٣/s), if the Abu- Gharak regions are excluded then the remaining capacity shall be equal (١٤٠٤٠٠) m٣/day, equivalent to (١.٦٢٥) m٣/s.

٤.٤: Analysis of water consumption in Babylon University

Babylon university lies out of Hilla city, and fed with potable water from Hilla city by one single pipe of diameter (١٥٠ mm). So it becomes necessary to know how much water the university needs in order to know the required conveyance capacity and pipe characteristic. Table (٤-٥) shows the estimated amount of water that can be applied to the university needs as recommended by **Steel** [٥١].

Table (٤-٥): Water consumption for various purposes [٥١].

Type of consumption	Demand ℓ/c.d
Industrial	١٦٠
Public	٦٠
Losses	٥٠

Service office at Babylon University has estimated the amount of water which is needed for gardens and cooling as (١١.٠) ℓ/s [٣٣]. Table (٤-٦) gives the number of water users and the working time in this university.

Table (٤-٦): Type and number of water users in Babylon University [٣٣].

Type of users	**No. of users (capita)**	**Working time (hr)**
Students	١٣٦٧٠	٧
Public services	١٣٨١	٦
Lecturers	٥٢٨	٦
Industrial services	٢٨	٩
Guests	٢٠	٦

From the information in table (٤-٦), the water consumption for users at university can be calculated. For example, the water consumption for industrial services may be computed as:

amount of water $= ٢٨ * ١٦٠ * \frac{9}{24} = ١٦٨٠$ ℓ/day.

losses $= ٢٨ * ٥٠ * \frac{9}{24} = ٥٢٥$ ℓ/day.

So, the total water consumption for industrial services will equal to the amount of used water plus losses; i.e. (٢٢٠٥ ℓ/day). The matter is the same for each other users. From calculating water demand for all users and the estimated water demand for cooling and gardens; i.e. (١١.٠ ℓ/s), it can be shown that the total water consumption for Babylon university, is equal to (١٤٧٣١٢٠ ℓ/d) ; i.e, (٠.٠١٧٠٥ m^{٣}/s).

٤.٥: Analysis of water consumption in Hilla city and some villages around

Some villages around Hilla city such as: Abu-Gharak, Anana, Sinjar and Kura Muhezim (Abu Gharak regions) fed with water from Hilla city by two individual pipes, one is from Hilla AL-Jadeed water treatment plant, and the other is from Tayara AL-Kadeem pumping station. Except these above villages, the populations of all villages mentioned in table (٤-٢) must be added to the populations of Hilla city, the expected populations in these villages except Abu-Gharak regions equals (٣٤٣٢٥) capita for year (٢٠٠٢).

Approximatly half of the area of Thawra and ١٧-Tamuze quarter fed with water from one of the two pipes that feed Abu-Gharak regions. This (٥٠ %) ratio of water supply is based on that four pipes which feeds Thawra and ١٧-Tamuze quarters. So, (٥٠ %) ratio of populations of these quarters, which equals (١٠٤٧٢) capita, must be excluded from the populations of Hilla city.

The total number of populations except Abu-Gharak regions and half of Thawra and ١٧- Tamuze quarters, which fed with potable water from Hilla city projects is: ٢٨٤١٥٦ – ١٠٤٧٢+٣٤٣٢٥ = ٣٠٨٠٠٩ capita, so the amount of water for each person in Hilla city and some villages around, except Abu-Gharak regions and Babylon University, can be estimated as follows:

Total consumption of water = $\frac{(1.625 - 0.01705) * 24 * 3600 * 1000}{308009} = 451$ ℓ/c.d.

The analysis of water demand according to type of consumption can be discussed as follows:

٤.٥.١: Domestic consumption

The amount of water according to this title is the largest among the other types of water consumptions. Water office in Hilla city estimates this amount, which equal to (٣٠٠ ℓ/c.d) [٥٠].

٤.٥.٢: Public consumption

The amount of water for public consumption depends on the water consumption of public buildings such as schools and companies. Water office in Hilla city estimates this consumption for Hilla city as (٢٠ ℓ/c.d) [٥٠].

٤.٥.٣: Trade and industrial consumption

The amount of water under this title depends on the water consumption of industrial and trading buildings such as factories, resturants, and trading shopes. Water office in Hilla city estimates this type of consumption, as (٣٠ ℓ/c.d) [٥٠].

٤.٥.٤: Losses and waste

The losses and waste in supplied water can be obtained from slippage of pumps, illegal connections of pipes and the leakage in pipes due to cracks. The network of Hilla city suffers from leakage due to aging of a large number of pipes specially under large pressure or load. So, water office in Hilla city estimates the losses and waste of water as (٣٠ %) of the total summation of the amount of consumptions [٥٠].

Table (٤-٧): Estimation of water consumption in Hilla city [٥٠].

Type of consumption	Amount of consumption (ℓ/c.d)
House consumption	٣٠٠
Public consumption	٢٠
Trade and Industrial consumption	٣٠
Summation	٣٥٠
Losses and waste (٣٠٪)	١٠٥
Net summation	٤٥٥

From table (٤-٧), it can be seen that the computed demand per capita (٤٥١ℓ/c.d) is acceptable compared with the standard estimation (٤٥٥ ℓ/c.d) from table (٤-٧), that is, a (٩٩.١٢ %) efficiency of water production is satisfied in Hilla city.

٤.٦: General notes on existing water scheme of Hilla city

In general, potable water supplied to the users by three ways, i.e, three distribution systems.

١- Gravity distribution.

٢- Indirect distribution.

٣- Direct distribution.

Direct distribution is not preferred among the other types because any problem occurs in the source will lead to stop pumping. This type of distribution is in fact applied in Hilla city water supply projects.

Inside pipeline systems in many quarters, the branched system that follows a tree like pattern is used.

Many pipes in Hilla city water supply network suffer from cracks and leakage due to aging of pipes. Moreover, there is no pipeline layout for Hilla city exists before this study. A map was prepared by the author depending on reference [٤٦] and the experience of staff and engineers at water office.

Hilla city ground level vary between (٢٥.٥٣١-٣٠.١٦٢) m above sea level [٣٤]. The difference in elevation will normally effect the pressure distribution, so a contour map for Hilla city is needed in this study for more accurate results.

٤.٧: Hilla city contour map

In analyzing or designing pipe network, the topography of land is very often a controlling factor. The scale of the available contour map is (١:٤٠٠٠٠) at interval between contour lines of (٠.١ m). Table (٤-٨) shows some points of known elevations, these points are shown in Fig. (A-١) in Appendix A.

Table (٤-٨): Ground elevation and spot levels of some points of Hilla city [٣٤].

Point No.	Ground elevation (m)	No.of point	Ground elevation (m)
١	٢٨.٩٠٣	٢٣	٢٦.٦٠٩
٢	٢٨.٦٨٣	٢٤	٢٦.٥٤٥
٣	٢٨.٩٤١	٢٥	٢٧.٢٢٤
٤	٢٨.٠٤٥	٢٦	٢٨.١٣٤
٥	٢٦.٩٥٨	٢٧	٢٨.٢٤
٦	٢٧.٧٢٦	٢٨	٢٨.٢٣٣
٧	٢٨.١٤٩	٢٩	٣٠.١٦٢
٨	٢٨.٩٩٨	٣٠	٢٩.٢٤٦
٩	٢٩.٧٧٠	٣١	٢٨.٦٣٧
١٠	٢٨.٧٣٢	٣٢	٢٨.٨٧٧
١١	٢٨.١٥٦	٣٣	٢٧.٩٩١
١٢	٢٨.٨٢٨	٣٤	٢٧.٧٠٣
١٣	٣٠.١٠١	٣٥	٢٧.٩٧٩
١٤	٢٩.٦٦٩	٣٦	٢٦.٩٦٧
١٥	٢٦.٣٣٤	٣٧	٢٦.٦٢٢
١٦	٢٩.١١٧	٣٨	٢٦.٠١٨
١٧	٢٨.٩٢٠	٣٩	٢٥.٥٥٢
١٨	٢٨.٩٣٧	٤٠	٢٥.٥٣١
١٩	٢٨.٢٣٧	٤١	٢٨.٠٤٥
٢٠	٢٨.٠٠٩	٤٢	٢٧.٤٩١
٢١	٢٧.٤٥٧	٤٣	٢٨.٣٠٦
٢٢	٢٧.٠٢٩		

The contour lines may be drawn mechanically or by using spacing contour method [٣]. Figure (A- ٤) in Appendix A shows the elevation of nodes for Hilla city pipe network. In this work (Surfer program) was used to draw the contour map for Hilla city, Fig. (A-٢) in Appendix A shows the contour map for this city.

٤.٨: Analysis of flow for Hilla city pipe network

Analysis of flow means, finding the amount and direction of water discharge in each pipe, keeping in mind that the laws of mass conservation and energy conservation must be satisfied.

In the field of pipe network analysis the oldest and common method is the well known Hardy Cross method including two procedures, i.e., (Nodal method and Loop method). In this research, the Loop method is used to analyze Hilla city pipe network. This method considers the amount of water discharge as major unknowns, this implies assuming an initial amount of water discharge in each pipe, and then correct these discharges by a correction factor.

In this work Darcy –Weisbach equation is used to calculate head losses in each pipe at the analysis of pipe network, this equation is more accurate than Hazen-Williams equation, since that equation implies variable friction factors as was discussed in Chapter Three.

٤.٨.١: Estimation of water consumption at each node

In estimating water demand at nodes of any pipe network the following data should be followed:

١. The layout of main conveyance pipe network.
٢. The total populations served by the network and the concentration (capita / km٢) for each subarea.
٣. The design water demand per capita including all types of consumption (industrial, puplic, waste, …. etc.).

By multiply the number of populations served by each loop by water consumption for each person (٤٥١ ℓ/c.d), the water consumption in each loop can be obtained. Dividing this amount of water by the number of draw off nodes in each loop to obtain the amount of water consumption in each node. Figure (A-٣) in Appendix A shows pipe network layout, Fig. (A- ٤) and Fig. (A- ٥) in Appendix A, show the layout and analysis data for Hilla city pipe network.

٤.٨.٢: An initial estimation for flow in each

Hardy Cross method (Loop Method) implies estimate an initial disharge in each pipe. For steady state must be noticed that the total amount of water in all supply nodes must equal the total amount of water in all draw off nodes, the steady state case is considered when the flow rate in the supply may be from reservoirs, and /or pumps or specified inflow or outflow at some points in the network [٦]. Figure (A-٦) and Fig. (A- ٧) in Appendix A show the initial estimation for discharges in pipe network for Hilla city.

٤.٨.٣: Heads calculation at supply nodes

Table (٤-٩) shows the known heads for supply nodes which is also shown in Fig.(A-١٠) in Appendix A.

By using Bernoulli’s equation making useful from the ground elevations obtained from the contour map for Hilla city the head of each node can be

calculated. Figure (A- ٩) and Fig. (A- ١٠) in Appendix A show the existing head at each node. For calculating the pressures at nodes, a datum elevation of (٢٥.٥ m) was considered in this study.

٤.٨.٤: Computer program for analyzing pipe network

A computer program was written with (Fortran power station) language to analyze the pipe network. Both Darcy–Weisbach or Hazen-Williams equations can be used within this program to estimate head losses.

The program deals with common pipe between two or more than two loops, in other meaning there are more than two loops connect with one pipe.

The program also deals with reservoirs (system with multiple fixed-pressure – head levels). Pseudo loops are created to account for the unknown outflows and inflows at the reservoirs and to satisfy continuity conditions during balancing [٥٢].

$$\Delta Q = -\frac{EL + \sum_{i=1}^{NP} hfi}{n \sum_{i=1}^{NP} \frac{hfi}{Qi}} \qquad \ldots(٤\text{-}٢)$$

Percentage method [١٤] was used to estimate the flow at pipes no. (٤٩_I, ٤٩_{II}, ٧٥_J, ٧٥_{JJ}) as shown in Fig. (A-١٥) in Appendix A, and then converted to equivalent pipes that carries the same flow and have the same head losses.

The allowable error in this study is taken as (٠.٠٠٠٠١) $m^٣/s$. A flow chart of this program is shown in Fig. (C-١) in Appendix C.

٤.٨.٥: Results of analysis for Hilla city pipe

After using the computer program, final discharges and head losses are obtained. Figure (A- ٨) and Fig. (A- ٩) in Appendix A show the final discharge in each pipe.

Chapter Five

Optimum Design and Rehabilitation of Hilla City Pipe Network

٥.١: Introduction

The purpose of optimization is to find the best possible solution among the many potential solutions satisfying the chosen criteria. Designers often based their designs on the minimum cost as an objective, safety and serviceability .

A general mathematical model of the optimization problem can be represented in the following form:

A certain function (Z), called the objective function,

$Z = f \{X_i\}$ I =١, ٢, … , n … (٥-١)

Which is usually the expected benefit (or the involved cost), involves (n) design variable {X}. Such function is to be maximized (or minimized) subject to certain equality or inequality constraints in their general forms:

$gi \{X_i\} = bi$ i=١, ٢ , ………, I …(٥-٢)

$qi \{X_j\} >= bj$ j =١, ٢, ………., J …(٥-٣)

The constrain reflects the design and functional requirements. The vector {X} of the design variables will have optimum values when the objective function reaches its optimum value. In this chapter, the pipe network for Hilla city is rehabilitated and designed considering the optimum case as a feasible solution to avoid the deficit in heads at some nodes.

٥.٢: The objectives function

The objective function (Z) of the present research involves the cost of transportation, cuting, connecting, filling and repairing road. The following equations are estimated by (STATISTICA) program with a regression coefficient of (R = ١.٠٠٠). There is a difference in cost of carrying out between the paved and unpaved road. So, the objective function of Hilla city pipe network for unpaved road is:

Z = ١٠٠٠٠ . D + ١٠٠٠ ... (٥-٤)

and for paved road is:

Z = ١٠٠٠٠ . D + ٤٥٠٠ … (٥-٥)

Subjected to:

٠.١ m ≤ D ≤ ٠.٨ m.

The methods of optimum design and optimum rehabilitation are considered subjected to the following constraints:

١- Pipes should be plastic.
٢- Head at each node should be ≥ ٢٣.٠ m.
٣- Location of supply nodes considered constant to reduce the required cost.
٤- Elevated storage tanks locations should be neglected since the direct pumping process was used in the existing network.

Where :

Z : final cost of construction per meter (I.D).

D : the commercial diameter of pipe (m).

Figures (٥-١) and (٥-٢) show the relation between the cost of carrying out the unpaved road and diameters and the relation between cost of carrying out the paved road and diameters respectively.

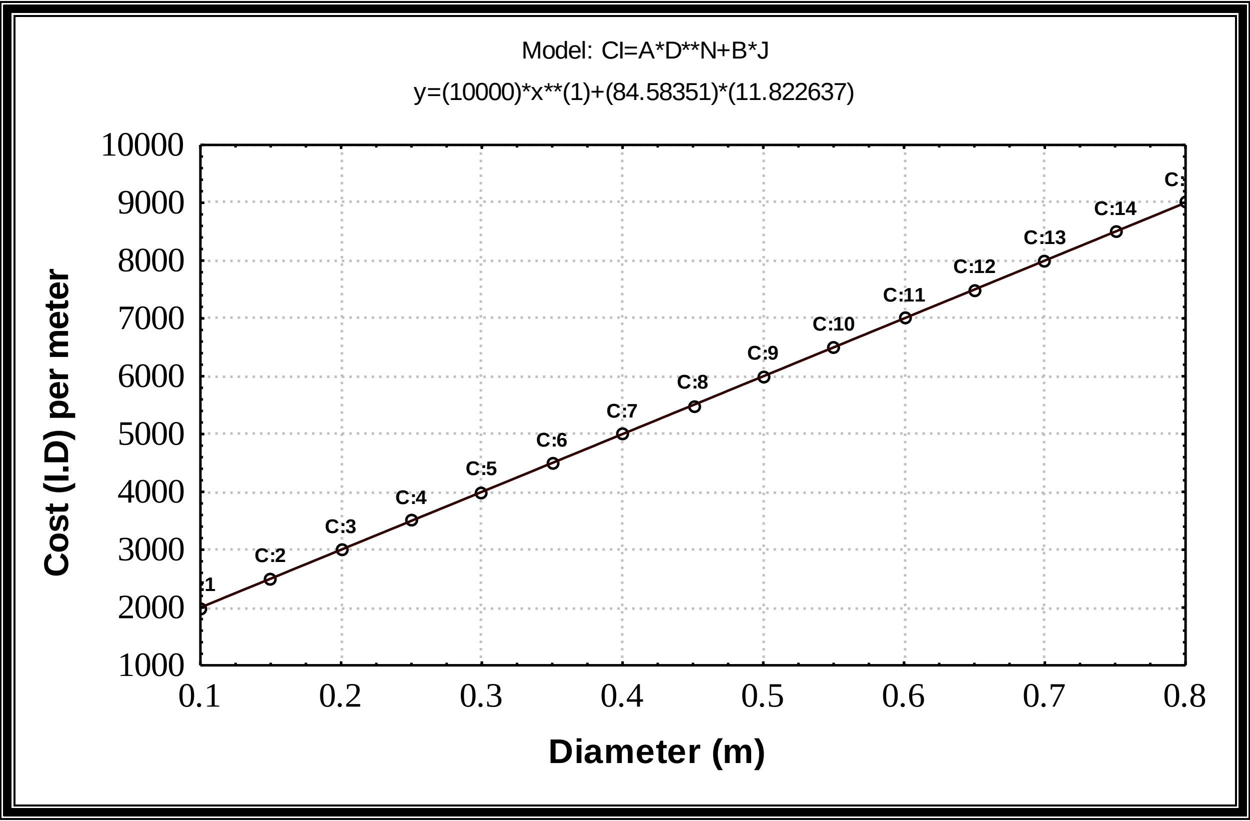


Fig.(٥-١): Cost of carrying out pipes for unpaved road.

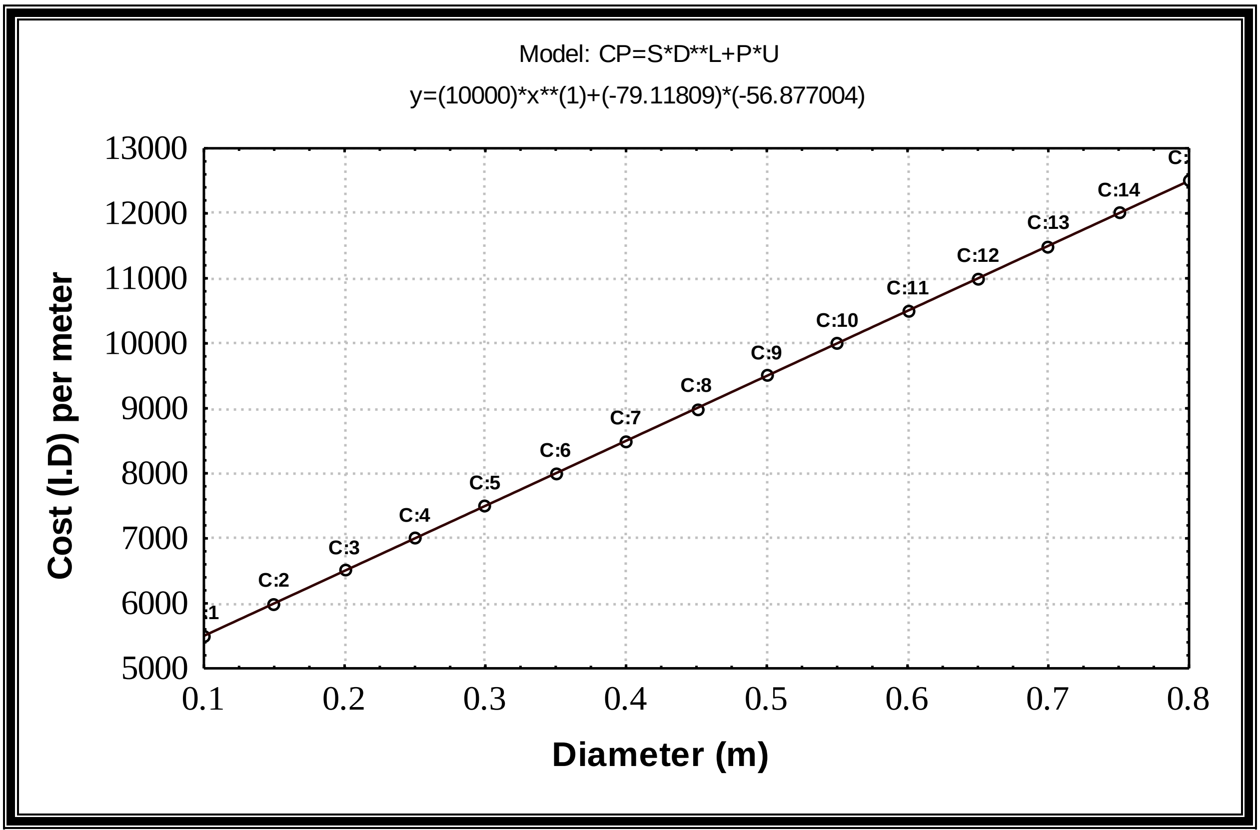


Fig.(٥-٢): Cost of carrying out pipes for paved road.

٥.٣: Methods for optimum design and optimum rehabilitation

The main variables affecting the optimum design and optimum rehabilitation are as follows:

٥.٣.١: Commercial diameters

Commercial diameters are used in this research unlike many methods that deal with theoretical diameters and then approximated to the commercial diameters. Using commercial diameters directly lead to more accurate results for pipe network, since heads at nodes will not change after designing or rehabilitation due to change the theoretical diameters to the commercial diameters.

The following commercial diameters of plastic pipes are used in this research depending on the availability in the water office, since these pipes are the cheapest among the other types. These diameters are: (١٠٠, ١٥٠ , ٢٠٠, ٢٥٠ , ٣٠٠ , ٣٥٠ , ٤٠٠ , ٤٥٠ , ٥٠٠ , ٥٥٠ , ٦٠٠, ٦٥٠, ٧٠٠, ٧٥٠, ٨٠٠) mm [٥٠].

The terms “ the next maximum commercial diameter “ and “ the next minimum commercial diameter “ was used in this research. These terms used to explain choosing a diameter greater than or less than the present diameter with one order. For example, if the diameter that needs to be increased is (٥٠٠) mm, this diameter becomes (٥٥٠) mm after increasing the diameter and becomes (٤٥٠) mm after decreasing it.

٥.٣.٢: Hydraulic gradient

Hydraulic gradient can be defined as the ratio between the head losses and the length of a certain pipe.

The methods of optimum design and optimum rehabilitation depend on hydraulic gradient as a main index for increasing or decreasing the diameters of pipes in the network. When the hydraulic gradient for the certain pipe is large or small, compare with the other hydraulic gradients for other pipes, the two methods will increase or decrease the diameter of this pipe, respectively, to obtain a homogenous hydraulic gradient for pipe network.

Since the hydraulic gradient is important, so the two programs for optimum design and for optimum rehabilitation have a subroutine to arrange pipes according to its hydraulic gradient (from the largest one to the smallest one), that will simplify the procedure of increasing or dicreasing the diameters of pipes.

٥.٣.٣: Design head

After analyzing the pipe network applying Hardy Cross method or any other method, the head at each node becomes known. If the minimum head in the pipe network is less than the design head, some pipes must be increasing to avoid this unacceptable case. The increasing be to the following maximum commercial diameter depending on the largest hydraulic gradient, and vice versa. In this study, the design head considered (٢٣.٠) m (the average value of (١٥.٣١-٣٠.٦١ m /١٥٠-٣٠٠ kpa)) [٥١], for optimum design and optimum rehabilitation.

٥.٤: Standard deviation and coefficient of

Al - Ani in (١٩٨٥) [٢], suggested using standard deviation and coefficient of variation as an index for pressure distribution in pipe network since the standard deviation considered a good measure for variance of data about the arithmetic mean.

The data that used to find the standard deviation is the head at each node in the network. So the unit of standard deviation is the same as the unit of head at node. The coefficient of variation is dimensionless since it is equal to the ratio of the standard deviation to the arithmetic mean. For that reason, the coefficient of variation considered in this study as the index for variation of heads at nodes. Thus [٢]:

$$S.D = \sqrt{\frac{\sum (xi - \bar{x})^2}{NN}} \quad \ldots(٥\text{-}٦)$$

Where :

S.D: standard deviation.

xi : values of head at node.

$\bar{X}$: average of heads at nodes.

NN : number of nodes.

While the coefficient of variation (C.V) is [٢]:

$$C.V = \frac{S.D}{\bar{x}} \quad \ldots(٥\text{-}٧)$$

When the coefficient of variation converge to zero that means that the heads distribution is more constant and vice versa. So, these two variables considered in the optimum design and in the optimum rehabilitation.

٥.٥ : Optimum design

An optimum pipe network design for Hilla city was conducted up to the year (٢٠٢٥) depending on the following items:

٥.٥.١: Estimated population of Hilla city and some villages around

Making use of eq. (٤-١) and the number of populations at the year (١٩٩٧) in table (٤-١) and table (٤-٢), the populations at the year (٢٠٢٥) with the same yearly average increasing in Hilla city and for some villages around can be estimated. Table (٥-١) and table (٥-٢) show the estimated number of populations at the year (٢٠٢٥).

Table (٥-١): Expected population for Hilla city at year (٢٠٢٥).

Quarter Name	Actual populations (١٩٩٧)	Expected populations (٢٠٢٥)	Quarter Name	Actual populations (١٩٩٧)	Expected populations (٢٠٢٥)
Kreta'a	٢٣٣٥	٣٥٧٧	Jubaween	٢٤٣٨	٣٧٣٥
Theila	٢٨٣٠	٤٣٣٦	Ta'aees	٢٨٤٦	٤٣٦٠
Galage	١٤٩٤	٢٢٨٩	Krade	٢٤٢٠	٣٧٠٧
Wardia	١١٧٩	١٨٠٦	Tayara	٤٩٨٧	٧٦٤٠
Khisirwia	٢٠٥٩	٣١٥٤	Naseege	٧٠٥٥	١٠٨٠٨
Babel	١٢٢١	١٨٧١	Iskan	٦٩٦٤	١٠٦٦٩
Jaza'ir	٢٣٥٢	٣٦٠٣	Jema'aia	٥٥٣٩	٨٤٨٦
Bakarly	٧٢٤٦	١١١٠١	Murtadha	٤١١٠	٦٢٩٦

J.M. Bakarly	٩٢٧٧	١٤٢١٢	Hussein	٣٦٠٥	٥٥٢٣
Quarter Name	**Actual populations (١٩٩٧)**	**Expected populations (٢٠٢٥)**	**Quarter Name**	**Actual populations (١٩٩٧)**	**Expected populations (٢٠٢٥)**
Nadir ٣	٦٦٥٩	١٠٢٠٢	Makhazin	١٧٨	٢٧٣
M. Sulaiman	٥٤١٢	٨٢٩١	Faza'a& Mane'a	٢٥٣٦	٣٨٨٥
Nadir ١	٣٣٦٤	٥١٥٤	Dh in Wessia	٩٨١٨	١٥٠٤١
Nadir ٢	٩٥٩٤	١٤٦٩٨	Hamza Dally	١٧٠٤	٢٦١١
Zahraa	١٣٥٢	٢٠٧١	Akrameen	٧٨٨٥	١٢٠٨٠
Shawy	٦٥٣٥	١٠٠١٢	Shubar	٤٠٨٥	٦٢٥٨
Judeida	٦٨٥٣	١٠٤٩٩	Kadhia	٢٨١٨	٤٣١٧
Jumhury	٨١٥	١٢٤٩	Karama	٩٢٨٦	١٤٢٢٦
Mashta	١٠٠٢	١٥٣٥	Hukam	١٠٥٢	١٦١٢
Ibrahimia	٢٦١٥	٤٠٠٦	Imam	٧٧٤١	١١٨٥٩
Mustafa R.	٤٥٦٨	٦٩٩٨	Dhu. in Mekruy	٣١٥٥	٤٨٣٣
Ameer	١٥٩٥	٢٤٤٤	Shuhadaa in Mekruy	٣٦٧٨	٥٦٣٥
Jama'ein	٣٨٦٩	٥٩٢٧	Askary in Mekruy	٥٣٩٧	٨٢٦٨
Tak	٣٣٠١	٥٠٥٧	Muharbeen	٤٤٨٠	٦٨٦٣
Jubran	٥٩٩	٩١٨	Asatetha	٦١٧٥	٩٤٦٠
Mehdia	٤٠٣٧	٦١٨٥	Shuhadaa	١٠٠٨٥	١٥٤٥٠
Nuwab Dh.	٨٠٣٦	١٢٣١١	Muhendseen	٧٧٥٥	١١٨٨١
Nuwb Dh. ٢	٣٦٩١	٥٦٥٥	Jelaween	٩٠٥	١٣٨٦

Thawra	١٤٥٤٢	٢٢٢٧٨	١٧-Tamuze	٤٥٨٤	٧٠٢٣
Marana	٢٩٦٦	٤٥٤٤	Afrah	٢٩٩٣	٤٥٨٥
Tineia	١٣٧٧	٢١١٠	Dur-Babel	٤٥٠	٦٨٩

Table (٥-٢): Expected population for some villages around Hilla city at year (٢٠٢٥).

Village Name	Estimated population (٩٧)	Expected populations (٢٠٢٥)	Village Name	Estimated population (٩٧)	Expected populations (٢٠٢٥)
Tuhmazia	٥٨٩٨	١١٨٤٣	Buhamyar	٢١١٦	٤٢٤٩
Luba	٧٣٢	١٤٧٠	Wardia Kharege	٢٠٦٧	٤١٥١
Jimijma	٢٦٥٠	٥٣٢١	Abu-Ajaje	١٩٣٨	٣٨٩٢
Abu-Ilayan	١٥٠٣	٣٠١٨	Seife Sa'ad	١٨٤٤	٣٧٠٣
Benesha	٩٠١	١٨٠٩	Efar	٩١٢	١٨٣١
Rashedia	٩٢٤	١٨٥٥	Mua'mera	١٧٦٥	٣٥٤٤
Obed Radam	١٢٤٤	٢٤٩٨	Ataije	٣٧٧٥	٧٥٨٠
			Anana	٢٤٧٢	٤٩٦٤
Abu-Gharak	٥٩٣٤٢	١١٩١٥٩	Kura Muhezim	٣٠٥١	٦١٢٦
Sinjar	٢٤٥٥	٤٩٣٠	Mia'dan	٨١٩	١٦٤٥

The total consumption of water can be considered (٥٠٠) ℓ/c.d at the year (٢٠٢٥) according to the reference [٤٧]. By applying the previous method for estimating discharge at each node that used in the analysis of the existing network, the discharge at each node can be estimated for optimum design for Hilla city. Figure (A-١١) shows the plan of the suggested pipeline system, Fig. (A-١٢) shows the information of these pipeline, and Fig.(A-١٣) in Appendix A shows the estimated discharge at each node for Hilla city pipe network.

٥.٥.٢: Expected future expansions

Pipe network may be designed to involve all dimensions of certain cities with probability of expansion, which is expected to occur at the city.

The designer may use one of the following methods for designing pipe network with expansions [٢]:

١- Designing a complete pipe network such as in future with it two parts (the present and the future) as one unit.

٢- Designing the present pipe network keeping in mind the additional discharges at some nodes for future expansions, and the future pipe network will be designed then.

In the first method, the designer will design the original and the future pipe network as one unit. While in the second method, must specify the draw off nodes in the original pipe network, which will feed the future network by trial and error.

Final cost for designing the original and future pipe network when using the first method is less than the final cost of designing when using the second method. So, the first method is used at designing Hilla city pipe network in this study [٢].

٥.٥.٣: Unexpected future expansions

Unexpected future expansion leads to the decreasing in pressure distribution at nodes and shortage in water. Booster pumps, increasing head at pumping station, increasing diameters of pipes or using pipe with large diameter from pumping station to the region directly may be need to avoid this deficit. The important thing, that must specify the connected points between the original and future pipe network with more accurate, to reduce the effect of future network on the pressure distribution of original network. Al - Ani in (١٩٨٥) [٢] suggested using the standard deviation and coefficient of variation as an index for homogenous pressure distribution. For example [٢], if a certain existing pipe network consists of (١٠٠) nodes, and three unexpected future expansions occur at nodes (١, ٢, ٣) for certain region, and these three nodes can feed this region with water. These unexpected expansion were (١٠ %), (٢٠ %), (٣٠ %) from the total consumption of the existing network as presented in table (٥-٣). If the expansion was (٣٠ %) from the total consumption of water, the node, which preferred for feeding the region, is (١) only, since this node has the least value of coefficient of variation. The matter was the same for other expansions (١٠٪) and (٢٠ %) .

Table (٥-٣): Values for coefficient of variation for discharge of different consumption for unexpected expansions [٢].

Expansion (%)	Nodes			S.D (m)	C.V (%)
	١	٢	٣		
	Percentage ratio for expansion (%)				
١٠	------------	------------	١٠	١.٤٤	٥.١٥
	------------	١٠	------------	٠.٧٩	٢.٨
	١٠	------------	------------	٠.٦٩	٢.٤٥
٢٠	------------	١٠	١٠	١.٥٨	٥.٧
	١٠	------------	١٠	١.٤٨	٤.١١
	١٠	١٠	------------	٠.٧٧	٢.٧٨
	------------	------------	٢٠	٣.٢١	١١.٨٣
	------------	٢٠	------------	٠.٩٦	٣.٤
	٢٠	------------	------------	٠.٧	٢.٥١
٣٠	١٠	١٠	١٠	١.٥٨	٥.٧٤
	------------	١٠	٢٠	٣.٣٥	١٢.٤٩
	١٠	------------	٢٠	٣.٢	١١.٨٦
	١٠	٢٠	------------	٠.٩٦٨	٣.٤٨
	------------	٢٠	١٠	١.٨٥	٦.٧٩
	٢٠	------------	١٠	١.٤٥	٥.٢٥
	٢٠	١٠	------------	٠.٨٣٩	٣.٠
	------------	------------	٣٠	٥.٨٢	٢٢.٨

	------------	٣٠	------------	١.٢٥	٤.٥٢
	٣٠	------------	------------	٠.٨١	٢.٥٨

٥.٥.٤: Optimum design method

From the advantages of this method, it deals with any initial assumption for diameters of pipes. Also, one value for all diameters of pipes in the network can be initially considered. This property leads to decrease the number of data that the program needs. When using this method, the cost of designing pipe network is less than the cost of designing it by using other methods with ratio (٤ %-١٦.٥ %) [٢].

After assuming the initial diameters for pipes, pipe network will be analyzed by applying Hardy Cross method or any other method. Then all pipes are arranged according to its hydraulic gradient from the largest to the lower value. The minimum head in the pipe network is found to compare it with design head. If the minimum head is less than the design head, some diameters of pipes that have the largest hydraulic gradients must be increased to the next maximum commercial diameters and repeating the procedure of analysis again. The procedure of increasing the diameters of pipes will repeat until the minimum head in the pipe network becomes greater than the design head. So, the trials for decreasing diameters of pipes to the next minimum commercial diameters that have the smallest value of hydraulic gradient will begin. These trials of decreasing pipes will be repeated until the minimum head in the pipe network becomes less than the design head.

The diameters resulting from trials of increasing and decreasing can be considered in the design, but the cost of the network must be ensured is less as possible. To satisfy that, all diameters of pipes must be inserted the procedure of design. Otherwise, new next minimum commercial diameters are assumed. Repeating the decreasing and the increasing procedure until all pipes inserted the procedure of the design. In each trial of increasing and decreasing, it will change number of pipes proportion with the volume of network for reaching the final results quickly. When choosing a small number of these pipes, the results will be more accurate since the network in some cases does not need to change a large number of pipes for reaching the optimum results. So, in the optimum design and the optimum rehabilitation for Hilla city pipe network, the number of pipes in each trial is taken as one pipe for more accurate.

٥.٥.٥: Program for optimum design

The program was developed with (Fortran power station) language. Subroutine programs are used in the main program to continuate the tasks of the program. There are (١٦) subroutine programs, the first four subroutine programs are used to read data and print it.

Appendix (C-٢) shows the flow chart for this program and Appendix (B-١) shows the results of the new optimum design for Hilla city pipe network. Figure (A-١٤) shows the optimum design for Hilla city pipe network.

٥.٦: Optimum rehabilitation method

The principles of the method for optimum design as mentioned in section (٥.٥.٤) are considered the same for the rehabilitation method. This method deals with existing pipe network (existing pipes and its properties, and existing head at nodes). After analyzing the existing pipe network, the method will find the minimum head in the network and compare it with design head. The procedure of increasing and decreasing diameters of pipes to the next maximum commercial diameters or to the next minimum commercial diameters was also according to its hydraulic gradient. Then the method will find the cost of the network, if the last cost of network becomes greater than the previous cost, the method will decrease the diameters of rehabilitated pipes only to the next minimum commercial diameters and returns to analyze the network again. Otherwise the method will print the results.

٥.٦.١: Program for optimum rehabilitation

This program was developed with (Fortran power station) language. The program has(١٧) subroutine programs, the first four are used to read data and print it. The program will put the variable (MMM) equal to (١) and put the coefficient of Hazen – Williams equal to the value (١٤٥.٠) for all rehabilitated pipes since this value is for plastic pipes according to table (٣-٣). So any pipe that have the value of (MMM) equal to (١) that means this pipe was rehabilitated. Any pipe that has the value of (MMM) equal to zero that means, this pipe was not rehabilitated. Appendix (C-٣) shows the flow chart of this program, and Appendix (B-٢) shows the results of optimum rehabilitation. Figure (A-١٥) in Appendix A shows the optimum rehabilitated pipe network for Hilla city.

٥.٧: Discussion of the results

The total estimated cost for optimum rehabilitation of Hilla city pipe network found (٤٢٥٧٢٢٦٠٠) I.D, while the total estimated cost for the new optimum design of Hilla city pipe network was (٦٥٦٥٦٠٠٠٠) I.D. From this result, it can be noticed that the total cost for optimum rehabilitation is less than the total cost for optimum design of pipe network with percent (٣٥.٢٦ %). In general, when the engineers rehabititate some pipes in the network it may cost less than the new design for the pipe network. This was what occurred in this study.

Some branches pipes were existing under unpaved road, eq. (٥-٤) can be used for this case such as the following pipes. Table (٥-٤) shows the cost of optimum rehabilitation for branches pipes and table (٥-٥) shows the cost of optimum design for branches pipes.

Table (٥- ٤): Costs of optimum rehabilitation for branches pipes.

Pipe No.	Optimum rehabilitation for diameter (mm)	Length (m)	Cost (I.D)
١٥٦	٢٠٠.٠	١٣٠٠	٣٩٠٠٠٠٠
١٧٠	٢٠٠.٠	٢٤٠٠	٧٢٠٠٠٠٠
١٥٢+١٥٣	٣٠٠.٠	٤٢٠٠	١٦٨٠٠٠٠٠

Table (٥- ٥): Cost for optimum design of branches pipes.

Pipe No.	Optimum design for diameter (mm)	Length (m)	Cost (I.D)
١٤٨	٢٠٠.٠	٨٧٠	٢٦١٠٠٠٠
١٤٧	٢٠٠.٠	١٥٠٠	٤٥٠٠٠٠٠٠
١٤٦	١٥٠.٠	٥٠٠	١٢٥٠٠٠٠
١٤٤	١٥٠.٠	١٦٤٠	٤١٠٠٠٠٠
١٤٣	٢٥٠.٠	٥٠٠٠	١٧٥٠٠٠٠٠
١٤٢	٣٥٠.٠	٢٤٠٠	١٠٨٠٠٠٠٠
١٤٥	٢٥٠.٠	١٥٥٠	٥٤٢٥٠٠٠

Optimum diameters of branch pipes for optimum design and optimum rehabilitation in the above tables can be calculated by putting minimum head (١٥.٣١) m at the end point of pipe and the head at the beginning point is known from the results in table (B - ١) and table (B – ٢) in Appendix B, then by using Hazen – Williams equation for head losses, the theoretical diameters of these pipes can be calculated and approximated to the commercial diameters.

Chapter Six

Reliability of Water Distribution Network

٦.١: Introduction

Nothing in this world is ١٠٠ % reliable, and, as water engineers are painfully aware, urban water supply systems has no exception. Streams dry up or become contaminated, pumps break down, pipes rupture, treatment systems fail, and demands overshoot system capacity. The consequences of such failures are often severe.

Economic losses are large and even public health endangered. Water engineers, of course, have long worried about reliability, although they have historically focused mostly on stream flow dependability.

Engineers are, however, becoming increasingly concerned about the many other possible sources of reliability problems and are trying to account for them in the planning process.

How well a water distribution system can supply water in the required quantities at desired residual heads throughout its design period? This goal can be determined from water supply reliability. Reliability is defined as the probability that the system performs within specified limits for a given period of time. In reliability analysis of pipe network for Hilla city, heads of nodes that would be available should be evaluated and used in this chapter, since there is a deficit in head in this pipe network as summarized in Chapter Four.

Three reliability factors: node-reliability factor, head reliability factor, and network – reliability factor are used to describe the performance of pipe network for Hilla city.

٦.٢ : Node - Reliability factor

The node – reliability factor (R_n) is defined as the ratio of the total available outflow volume at a node to the desired outflow volume at that node for all states during the period of analysis. Thus, for node j [٤]:

$$R_{nj} = \frac{\sum_S V_{js}^{avl}}{\sum_S V_{js}^{req}} = \frac{\sum_S Q_{js}^{avl}.ts}{\sum_S Q_{js}^{req}.ts} \quad \text{, For all nodes j} \qquad ...(٦-١)$$

Where:

V^{avl} : available volume, (L^3)

V^{req} : required volume, (L^3)

Q^{avl} : available discharge rate, (L^3/T)

Q^{req} : required discharge rate, (L^3/T)

ts : time duration of state (same for all nodes), (T)

j : subscript denoting demand node .

s: subscript denoting state.

A time interval during which the nodal demands and condition of the network remain constant is termed a “state”.

In this work the parameter that was considered is head at each node. There is one state in this study, since the demand on the required head is considered constant during the time, and the variables in eq. (٦-١) become as in eq. (٦-٢).

$$Rnj = \frac{Hj^{avl}}{Hj^{req}} \quad \text{, For all nodes j.} \qquad \dots (٦-٢)$$

٦.٣: Volume - Reliability factor

The volume- reliability factor (R_V) is defined as the ratio of the total available outflow volume to the required outflow volume for the entire network for all states during the period of analysis. Thus [٤]:

$$R_V = \frac{\sum_S \sum_j V_{js}^{avl}}{\sum_S \sum_j V_{js}^{req}} = \frac{\sum_S \sum_j Q_{js}^{avl} .ts}{\sum_S \sum_j Q_{js}^{req} .ts} \qquad \ldots(٦\text{-}٣)$$

In this work, as mentioned before, the considered parameters are heads at nodes. So, other factor can be called head – reliability factor is considered instead of volume – reliability factor, which is an equal to the average of node reliability factor of the network. Thus:

$$R_h = \frac{\sum_j^{NN} R_{nj}}{NN} = \frac{\sum_j^{NN} H_j^{avl}}{\sum_j^{NN} H_j^{req}} \qquad \ldots(٦\text{-}٤)$$

Where:

NN: number of nodes in the network.

٦.٤: Network - Reliability factor

The node –reliability factor and the volume – reliability factors describe the performance of a distribution network considering the total volume availability at individual nodes and for the whole entire network, respectively. However, these factors do not completely describe the reliability of the network.

For example, consider the following three situations for a network in which all-nodal demands are identical [٤] (quoted in [Gupta and Bhave: ١٩٩٤]).

١. ٩٠٪ of demands are satisfied of ١٠٠ % of time at ١٠٠ % nodes, i.e. there is a uniform shortfall of ١٠ % supply at each node during the entire period of analysis. This situation though not desirable, is tolerable.

٢. ١٠٠ % of demand is satisfied for ٩٠ % of time at ١٠٠ % nodes, i.e. there are no supply at all the nodes during ١٠ % of time of the period of analysis. If this time duration is not concentrated but is distributed throughout the period of analysis, this situation is also tolerable, though less acceptable than situation ١.

٣. ١٠٠ % of demand is satisfied for ١٠٠ % of time at ٩٠ % of nodes, i.e. there is no supply at all at ١٠ % of nodes during the entire period of analysis. This situation is the worst and is unacceptable.

For all three situations, R_V = ٠.٩. For situations ١ and ٢ R_n = ٠.٩ at all nodes, while for situation ٣, R_n = ١ for ٩٠ % nodes and R_n= ٠ for ١٠ % nodes. The R_V and R_n values are the same for situations ١ and ٢ even though their performances are not the same. The value of R_V is also the same for situation ٣. However, it is preferable to have a single reliability factor that can describe situation ٣ and can also properly distinguish between situations ١ and ٢. It is therefore useful to consider network reliability factor (R_{nw}) defined as [٤]:

$R_{nw} = R_V.\ F_t.\ F_n.$ … (٦-٥)

Where:

F_t: time factor

F_n: node factor

In this work the above equation becomes:

$R_{nw} = R_{h.}\ F_{t.}\ F_{n.}$ … (٦-٦)

The time factor is defined as:

$$F_t = \frac{\sum_S \sum_J a_{js}.ts}{NN.Tp}$$ …(٦-٧)

Where:

Tp: period of analysis ($\sum ts$), (T)

The previous equation becomes in this study:

$$F_t = \frac{\sum_j a_{js}}{NN}$$ …(٦-٨)

a_{js} : a dummy variable taking value ١ or ٠ , which a_{js} = ١ , if the head ratio, $H_j\, avl / H_j req$ at a node for a particular state is equal to or more than an acceptable value , and a_{js} = ٠ , otherwise.

The node factor is the geometric mean of the node – reliability factors. Thus [٤]:

$$F_n = \left[\prod_{j1}^{NN} R_{nj} \right]^{1/NN}$$ ... (٦-٩)

If the network is unacceptable when the head available at the node and therefore, R_{nj} is less than a particular value, this R_{nj} is set to zero in eq. (٦-٩). As a result F_n and therefore R_{nw} would be zero and the network would be unacceptable.

The values of (R_V, F_t), F_n (assuming acceptable R_{nj} > = ٠.٩) and R_{nw} for three situations described earlier are shown in table (٦-١). Herein, R_{nw} values can properly depict the reliability for the three situations. Situation ٣, which is unacceptable, has zero network reliability factors.

Table (٦-١): Reliability factors for different situations [٤] [quoted in (Gupta and Bhave: ١٩٩٤)].

Case	R_V	F_t	F_n	R_{nw}
١	٠.٩٠٠٠	١.٠٠٠٠	٠.٩٠٠٠	٠.٨١٠٠
٢	٠.٩٠٠٠	٠.٩٠٠٠	٠.٩٠٠٠	٠.٧٢٩٠
٣	٠.٩٠٠٠	٠.٩٠٠٠	٠.٠٠٠٠	٠.٠٠٠٠

٦.٥: Computer program for reliability analysis

A computer program was written with (Fortran power station) language to evaluate reliability factors for water distribution system. A flow chart of this program was shown in Fig. (C- ٤) in Appendix C.

٦.٦: Application the program of reliability for Hilla city pipe network

The program of reliability analysis was applied for existing, optimum design, and optimum rehabilitation of Hilla city pipe network, by preparing the required data from the results of analysis, rehabilitation, and design, which was obtained in Chapter Four and Chapter Five.

The pressure that considered in this work is (٢٢٥) kpa for optimum design, optimum rehabilitation, and reliability analysis that is equals to (٢٣.٠ m) as head approximately [٥١] and the acceptable (R_{nj} ratio) is considered (٦٦.٥٧ %) i.e (١٥.٣١/ ٢٣.٠), in this study.

The values of R_n, F_t, F_n, and R_{nw} for three situations described earlier are shown in table (٦-٢).

Table (٦-٢): Reliability factors for the three cases of Hilla city pipe Networks.

Description of pipe network	R_h	F_t	F_n	R_{nw}

Existing	١.٠١٩٥٠	٠.٨٩٦٧٧	٠.٠٠٠٠٠	٠.٠٠٠٠٠
Optimum rehabilitation	١.٢١٦٥١	١.٠٠٠٠٠	١.١٩٩٤٩	١.٤٥٩١٩
Optimum Design	١.٢٧٨٠٥	١.٠٠٠٠٠	١.٢٦٨٣٥	١.٦٢١٠١

٦.٧: Discussion of the results

The two main values to evaluate the pipe network are: (F_t) and (R_{nw}). (F_t) in practical mean, means the reliability of network and (R_{nw}) means the index for the best network if the two networks have the same value of (F_t). So, for the existing pipe network for Hilla city, (F_t) for it is the less value among the values of (F_t) for the three pipe networks. That means the existing pipe network is the worst and unacceptable among them since its value of (R_{nw}) equals to zero. The best pipe network is the new optimum design among the three pipe networks since it has the large value of (R_{nw}). But, in general, the optimum rehabilitation of pipe network is preferred than the optimum design, since the last one has cost greater than the first as mentioned in Chapter Five.

Chapter Eight

Conclusions and Recommendations

In this research, an analysis of existing pipe network in Hilla city, a new optimum design, optimum rehabilitation, reliability analysis and evaluation of water quality have been investigated. The following conclusions and recommendations can be withdrawn on the basis of the results obtained.

٨.١: Conclusions

A- Water quantity

١. Enough amount of water consumption, within an average of (٤٥١ ℓ/c.d), and water production efficiency of (٩٩.١٢ %) could be achieved and pumped through the Hilla city pumping stations.

٢. A deficit of head, at some nodes in the existing network, due to leakage and aging of pipes is recognized.

٣. A rehabilitation of Hilla city pipe network has cost less than the cost of designing a new one with ratio (٣٥.٢٦ %).

٤. Reliability of head at nodes was found to be (٨٩.٦٨ %), (١٠٠ %), (١٠٠ %) for: existing, optimum rehabilitation, and for new optimum design respectively.

B- Water quality

١. All parameters, which were studied in this study, are acceptable according to standard values, except turbidity and hardness during the year (٢٠٠٢).

٢. Reliability of Hilla Al-Kadeem water treatment plant was found to be (٧٩.٩٥ %) for potable water turbidity and (٤.٤٦ %) for raw water turbidity.

٣. Reliability of Hilla Al-Kadeem water treatment plant was found to be (٥١.٠٤٢ %) for potable water hardness and (٤٧.٤ %) for raw water hardness.

٨.٢: Recommendations

The following recommendations are suggested for Hilla city water office:

١. Old pipeline inside different quarters should be replaced since these pipelines suffer from cracks, corrosions, and leakage due to its age such as Bakarly quarter.

٢. Elevated storage tanks in the city should subject to suitable repairs to convert the direct pumping to an indirect pumping process in order to obtain a convenient balance in water consumption during the different seasons since there is no control of pumping.

٣. It is recommended to specify all the new pipelines on the related layout map of Hilla city pipe network through maintenance and repair processes.

٤. Carrying out the optimum rehabilitation for pipe network to avoid the deficit in head at nodes.

For further research topics the following recommendations are suggested for Hilla city pipe network:

١. Analysis and evaluation of the quality of water at draw off nodes are to be studied.

٢. Further studies needs to be done on the contingency analysis considering failure of several pipes and several demand excess.

Chapter Seven

Water Quality Evaluation and Reliability of Hilla City Water Treatment Plants

٧.١: Introduction

In this Chapter, the quality of treated water in the treatment plant concentrated on the more important parameters and reliability for water treatment plants in Hilla city.

٧.٢: Water quality evaluation

Hilla city is provided as mentioned before with three water treatment plants. Data available is for two plants only, during the year (٢٠٠٢). These two plants are: Hilla Al-Jadeed and Hilla Al- Kadeem. The physical and chemical parameters that were considered in this research are: Turbidity, PH, Electrical Conductivity, Alkalinity, Hardness, Calcium, Magnesium, Chloride, Total Suspended Solids and Total Dissolved Solids for raw and potable water. Data of each parameter is obtained from the water office at Hilla city. Sketching and its discussion for each parameter as below, table (B- ٤) shows the maximum allowable limits for these parameters according to Iraqi specifications.

٧.٢.١: Physical parameters

Physical parameters that were considered in this study were: Turbidity, Electrical Conductivity, Total Suspended Solids and Total Dissolved Solids for raw and potable water as follows:

A : Turbidity

Turbidity is a measure of the extent to which light is either absorbed or scattered by suspended materials in water. Because absorption and scattering are influenced by both size and surface characteristics of the suspended material, turbidity is not a direct quantitative measurement of suspended solids. For example, one small pebble in a glass of water would produce virtually no turbidity. If this pebble were crushed into thousand of particles of colloidal size, a measurable turbidity would result, even though the mass of solids had not changed [٣٨].

Figures (٧-١) and (٧-٢) represent turbidity of potable and raw water at Hilla Al- Jadeed and Hilla Al-Kadeem water treatment plants, respectively. These Figures show an increasing in turbidity starting from February during year (٢٠٠٢) for raw water, and some increasing in turbidity for potable water. Most turbidity in surface waters results from the erosion of colloidal material such as clay, silt, rock fragments, and metal oxides from the soil. Household and industrial wastewaters may contain a wide variety of turbidity-producing material. So, these sources may be the reason for increasing of turbidity in raw water, and the later affect on potable water. Turbidity was measured with unit (Nephelometric Turbidity Unit) (NTU).

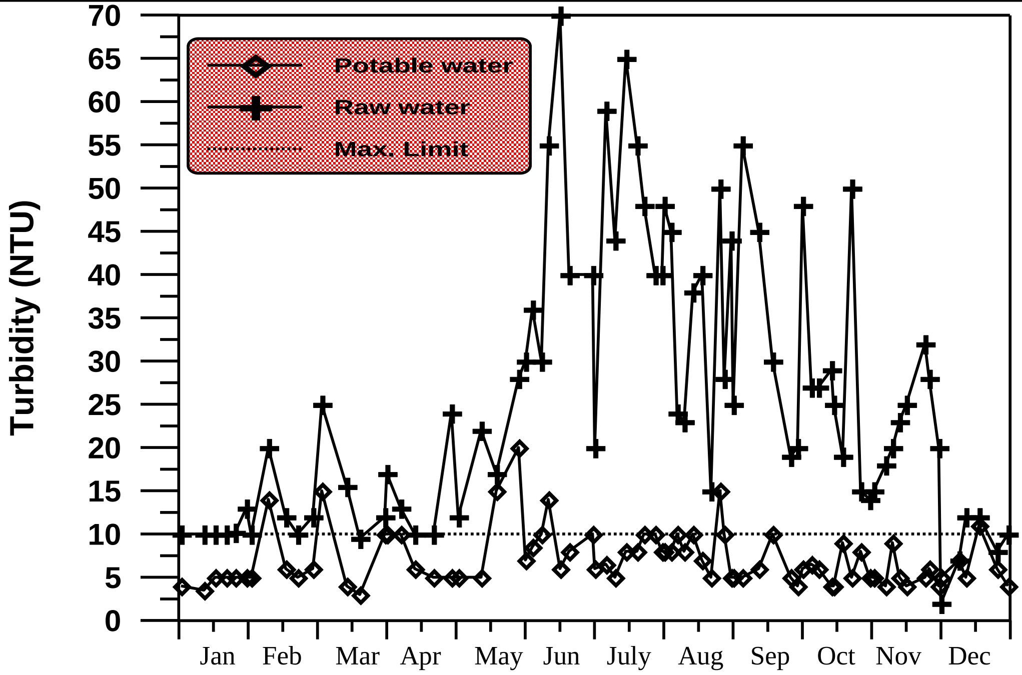


Fig. (٧-١): Turbidity of potable and raw water at (HJWTP) during year (٢٠٠٢).

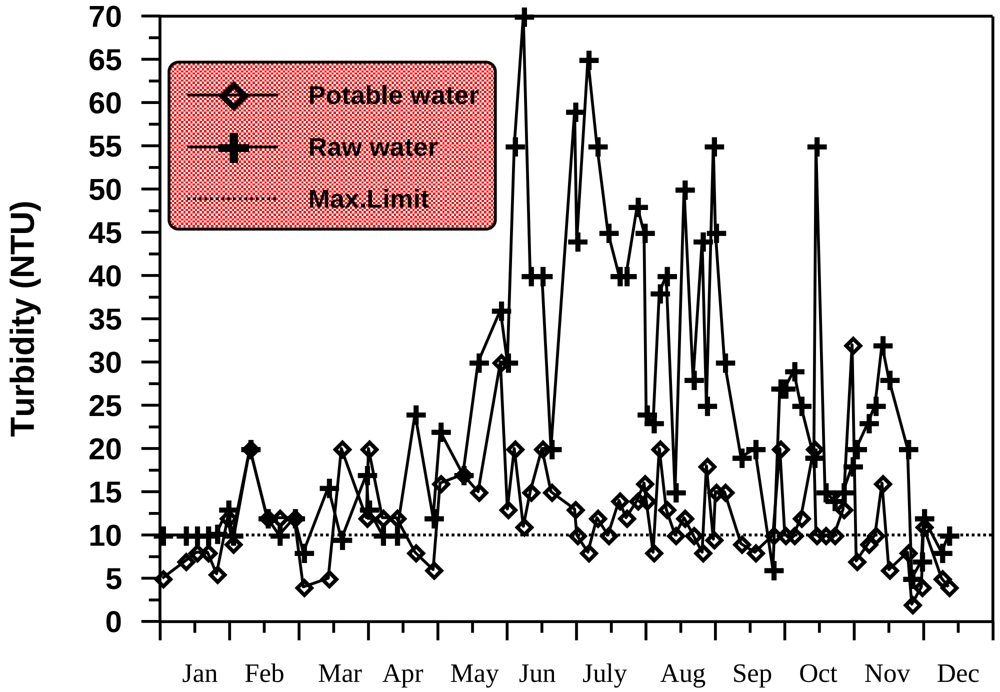


Fig. (٧-٢): Turbidity of potable and raw water at (HKWTP) during year (٢٠٠٢).

B : Electrical conductivity (EC)

Electrical Conductivity (EC) for water can be defined as a numerical value refers to the ability of water to carry the electrical current. This value depends on the concentration of dissolved ions in water, and temperature of water during the measurement, because it affects directly on the movement and direction of ions. So, (EC) increases (٢٪) as temperature increases one percentage degree [١].

Usually, most of acids, alkaline and inorganic dissolved salts carry the electrical current with good matter. While the organic salts and acids carry the current with bad matter. The unit for measuring (EC) is (mhos = ١/ohms) [١]. Figures (٧-٣) and (٧-٤) represent (EC) of potable and raw water for the two plants.

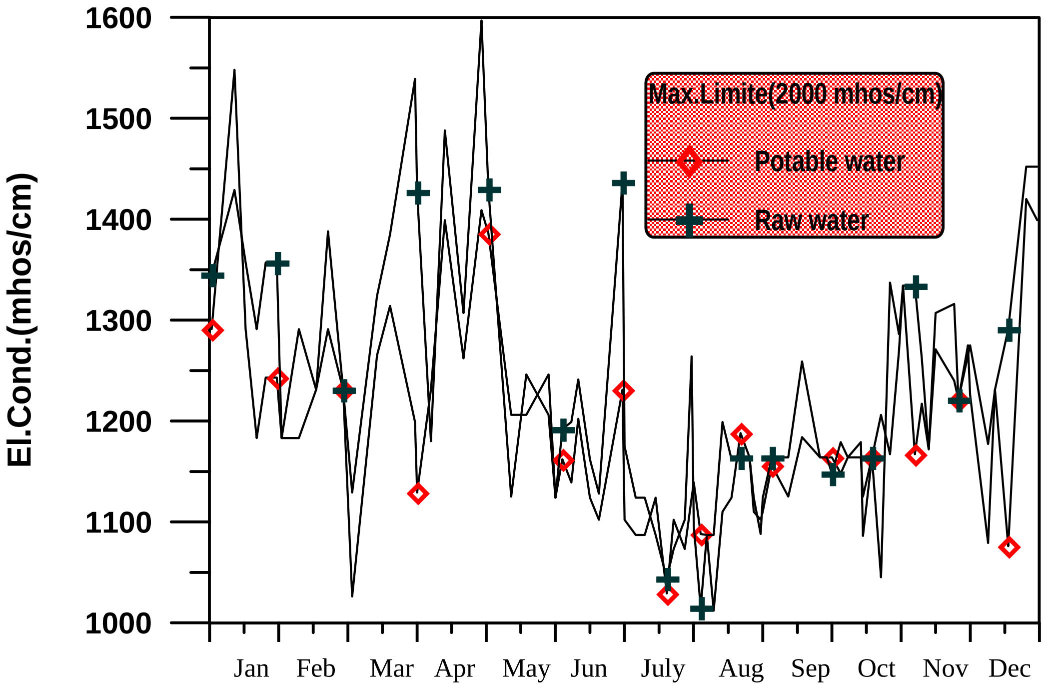


Fig. (٧-٣): (EC) of potable and raw water at (HJWTP) during year (٢٠٠٢).

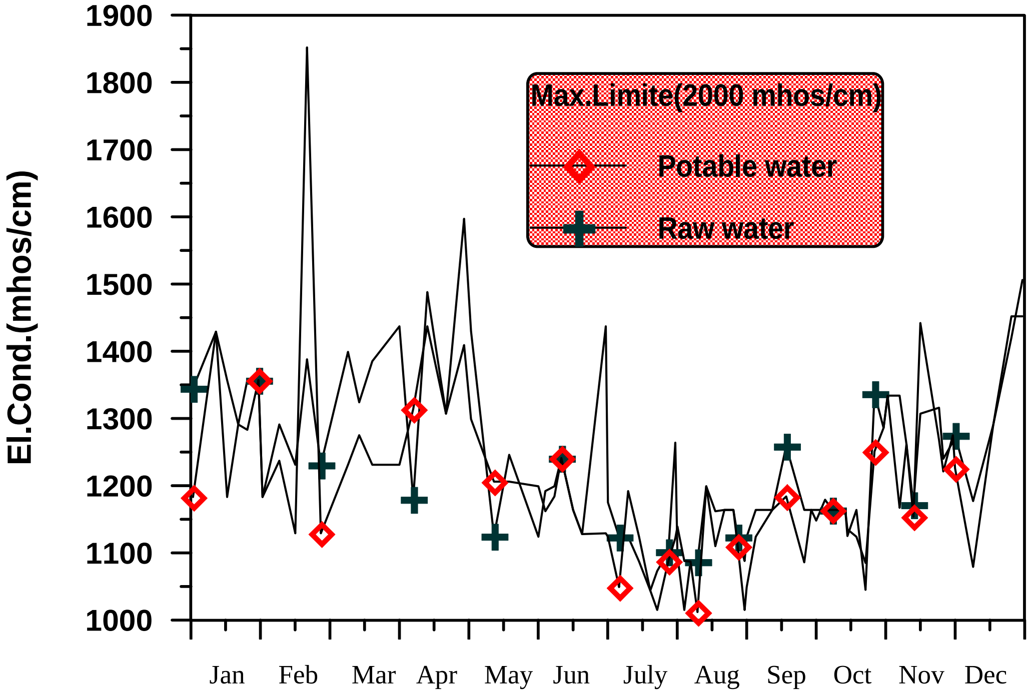


Fig. (٧-٤): (EC) of potable and raw water at (HKWTP) during year (٢٠٠٢).

These two figures show that (EC) at the two plants are less than the maximum allowable limit, accordingly drinking water is safe with respect to this parameter.

C : Total Suspended Solids (T.S.S)

Solids suspended in water may consist of inorganic or organic particles or of immiscible liquids. Inorganic solids such as clay, silts, and other soil constituents are common in surface water. Organic material such as plant fibers and biological solids (algal cells, bacteria, etc.) are also common constituents of surface waters. These materials are often natural contaminants resulting from the erosive action of water flowing over surfaces [٣٩].

Other suspended material may results from human use of the water. Domestic wastewater usually contains large quantities of suspended solids that are mostly organic in nature. Industrial use of water may result in a wide variety of suspended impurities of either organic or inorganic nature.

Figure (٧-٥) represents total suspended solids concentration of raw and potable water at Hilla Al-Kadeem water treatment plant (HKWTS), unfortunately, data for Hilla Al-Jadeed water treatment plant (HJWTP) is not available at Hilla city water office.

Although there is no standard value for this parameter, but if there is large amount of this parameter in water leads to uneconomic filtration for basins and tanks.

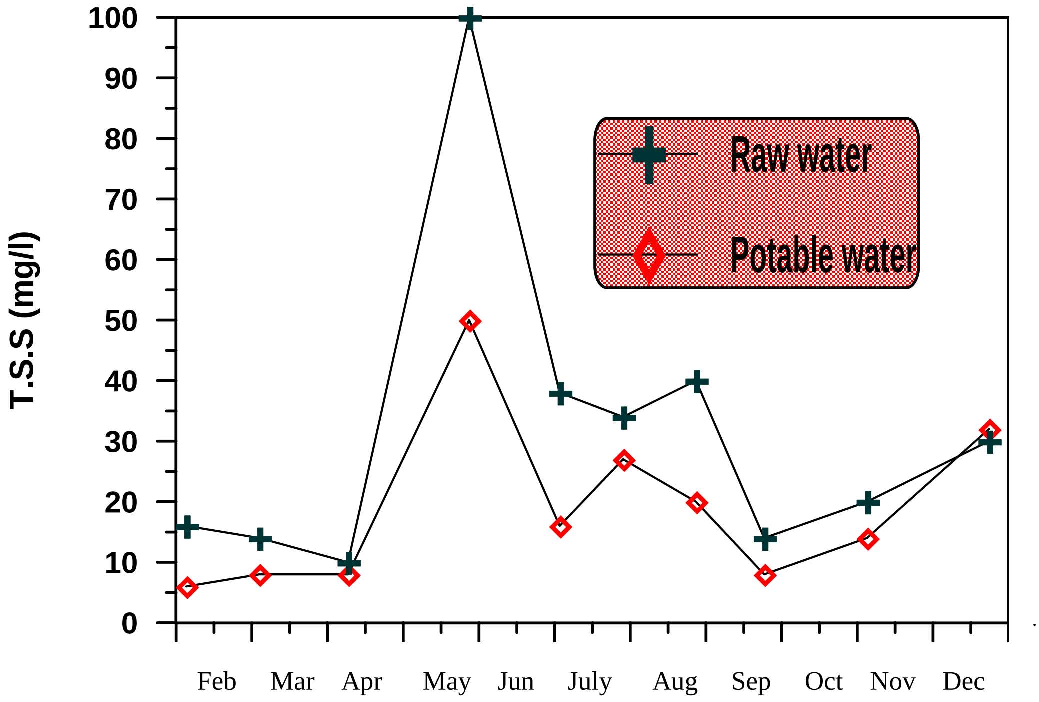


Fig. (٧-٥): Suspended Solids concentration of potable and raw water at (HKWTP) during year (٢٠٠٢).

D : Total Dissolved Solids (T.D.S)

The remaining suspended materials in water after the filtration is usually considered as dissolved materials. Figure (٧-٦) represents the total dissolved solids of raw and potable water for (HKWTS). No data available for this parameter at (HJWTP).

From figure (٧-٦), it can be noticed that the raw and potable water has a concentration of (T.D.S) less than the maximum allowable limit shown in table (B-٤) in Appendix B. So, the water is considered safe for drinking with regard to this parameter.

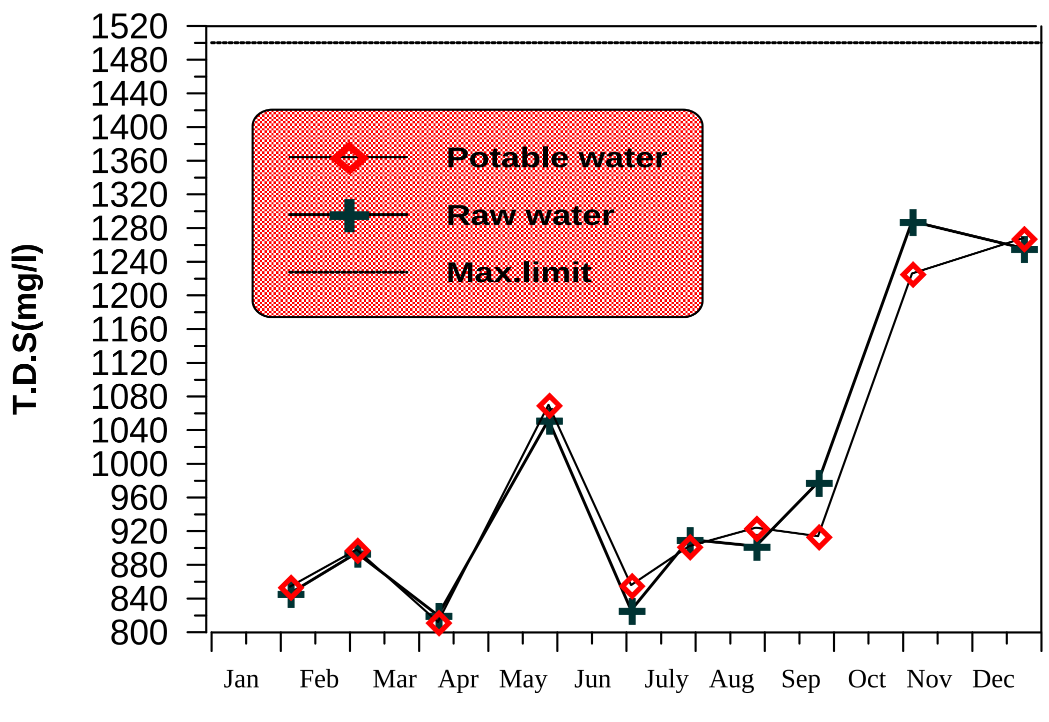


Fig. (٧-٦): Total dissolved solids concentration of potable and raw water at (HKWTP) during year (٢٠٠٢).

٧.٢.٢: Chemical parameters

The chemical parameters considered in this study are as follows:

A : pH-value

pH-value represents the activity of hydrogen ions in water. This value vary between (٠-١٤) for solutions, if the value of it is less than (٧); the solution was from acids, and if this value is greater than (٧) the solution considered from alkaline, otherwise the solution considered from salts.

Figures (٧-٧) and (٧-٨) show the pH-values of raw and potable water at the two plants. These figures show that the values of (pH) are within the allowable limits.

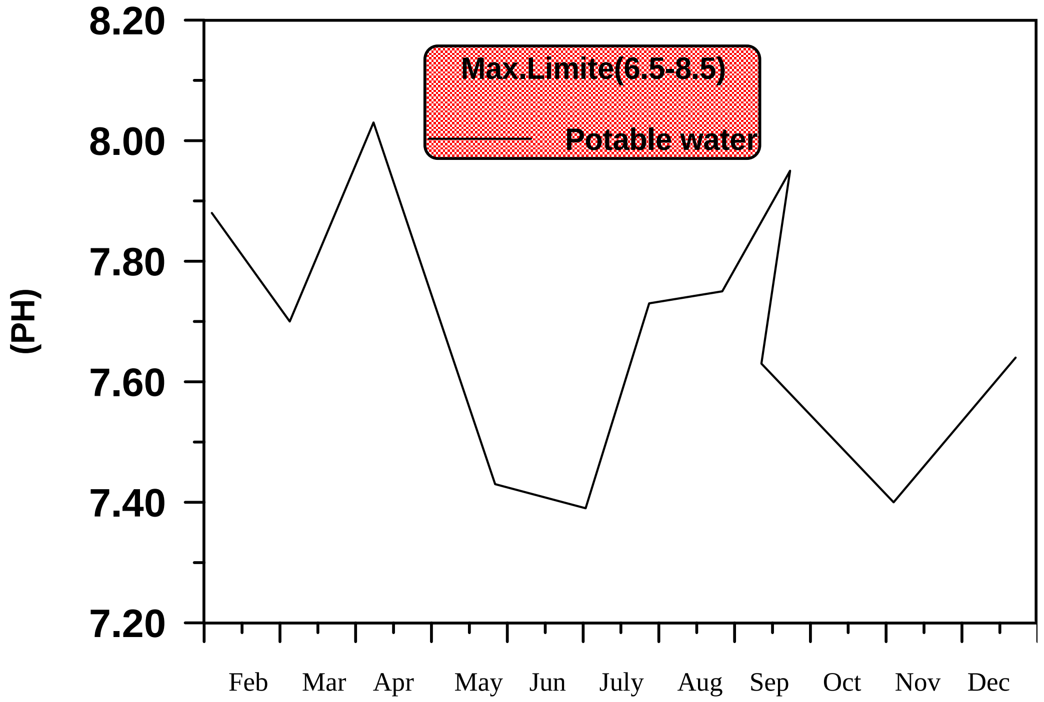


Fig. (٧-٧): pH value of potable water at (HJWTP) during year (٢٠٠٢).

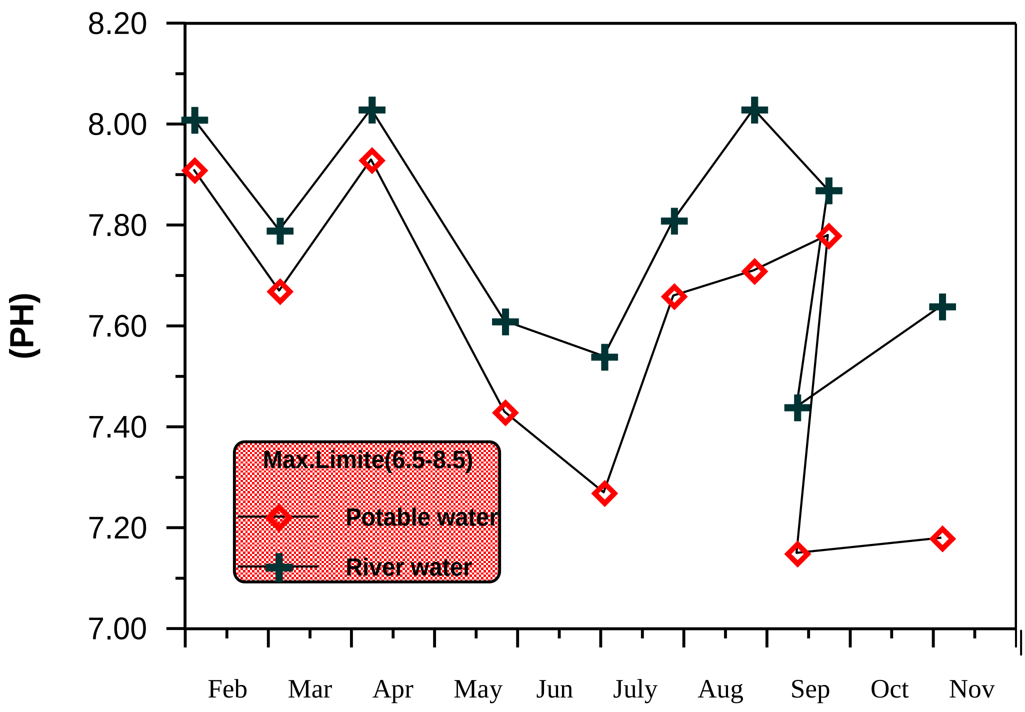


Fig. (٧-٨): pH value of potable and raw water at (HKWTP) during year (٢٠٠٢).

B : Alkalinity

Alkalinity is defined as the quantity of ions in water that will react to neutralize hydrogen ions. Alkalinity is thus a measure of the ability of water to neutralize acids [٣٩]. Figures (٧-٩) and (٧-١٠) represent alkalinity of potable and raw water, respectively, at the two water treatment plants. These figures show that the water may be considered safe for drinking.

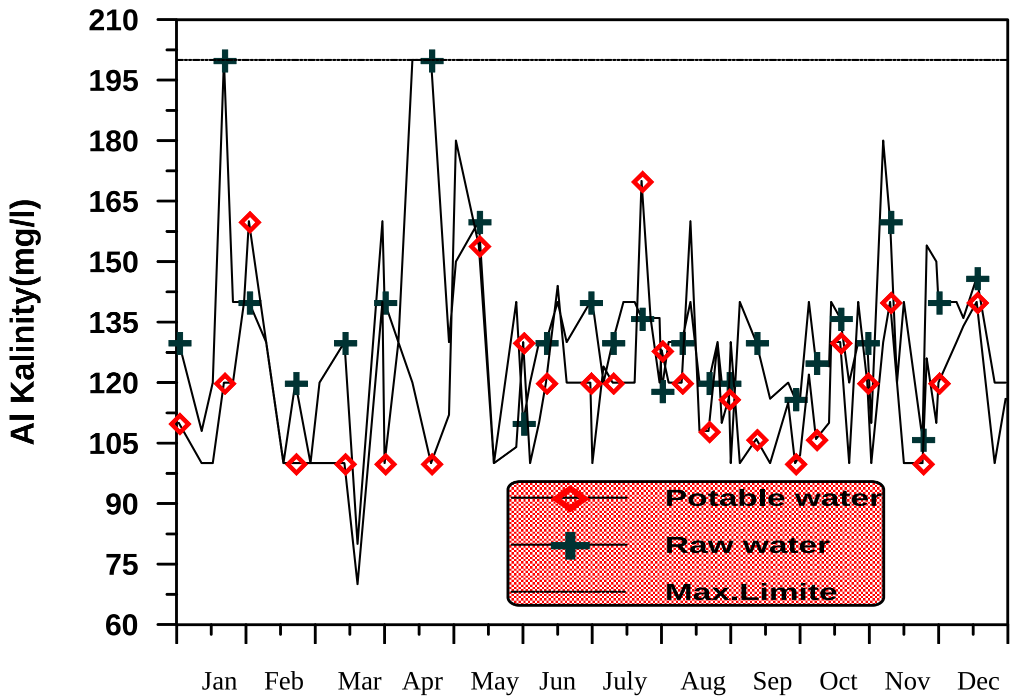


Fig. (٧-٩): Alkalinity concentration of potable and raw water at (HJWTP) during year (٢٠٠٢).

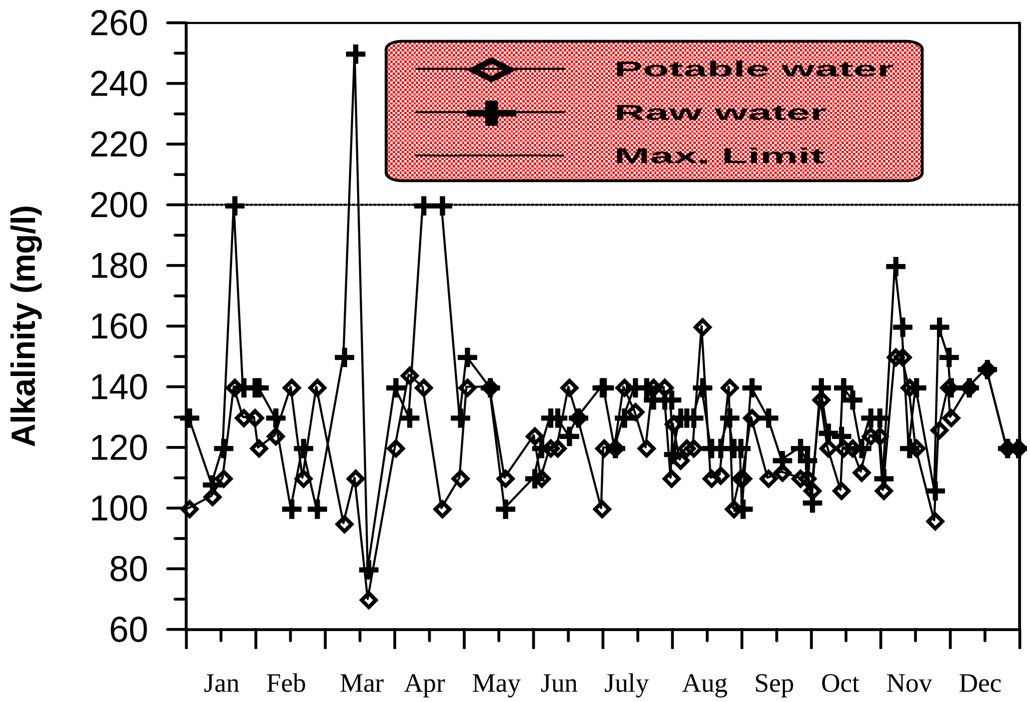


Fig. (٧-١٠): Alkalinity concentration of potable and raw water at (HKWTP) during year (٢٠٠٢).

C : Hardness

Hardness is defined as the concentration of multivalent metallic cations in solution. At supersaturated conditions, the hardness cations will react with anions in the water to form a solid precipitate. Hardness is classified as carbonate hardness and noncarbonate hardness, depending upon the anion with which it associates. The hardness that is equivalent to the alkalinity is termed carbonate hardness, with any remaining hardness being called noncarbonate hardness [٣٩].

From Figs. (٧-١١) and (٧-١٢), it can be noticed that the hardness of two types of water at the two plants is unacceptable. The concentration is above the maximum allowable limit (٥٠٠ mg/l) [٣١]. The increasing of hardness may be occurred due to the increasing of calcium and magnesium ions in soil, especially during the summer months (May – October), due to the increasing in calcium and magnesium ions during the same period as shown in Figs. (٧-١٣) and (٧-١٤).

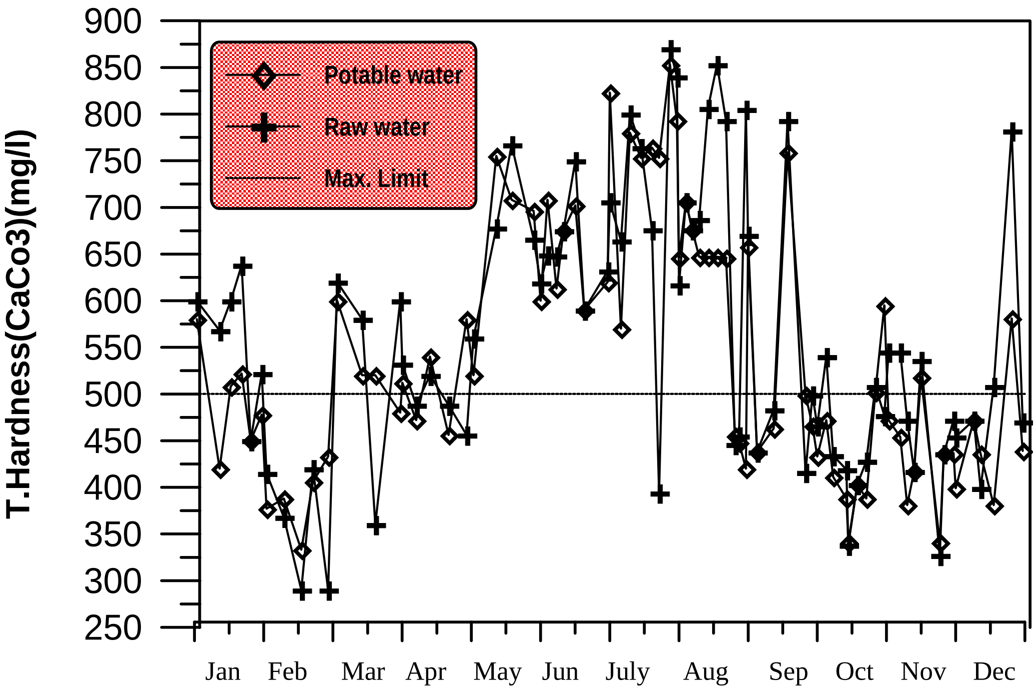


Fig. (٧-١١): Hardness concentration of potable and raw water at (HJWTP) during year (٢٠٠٢).

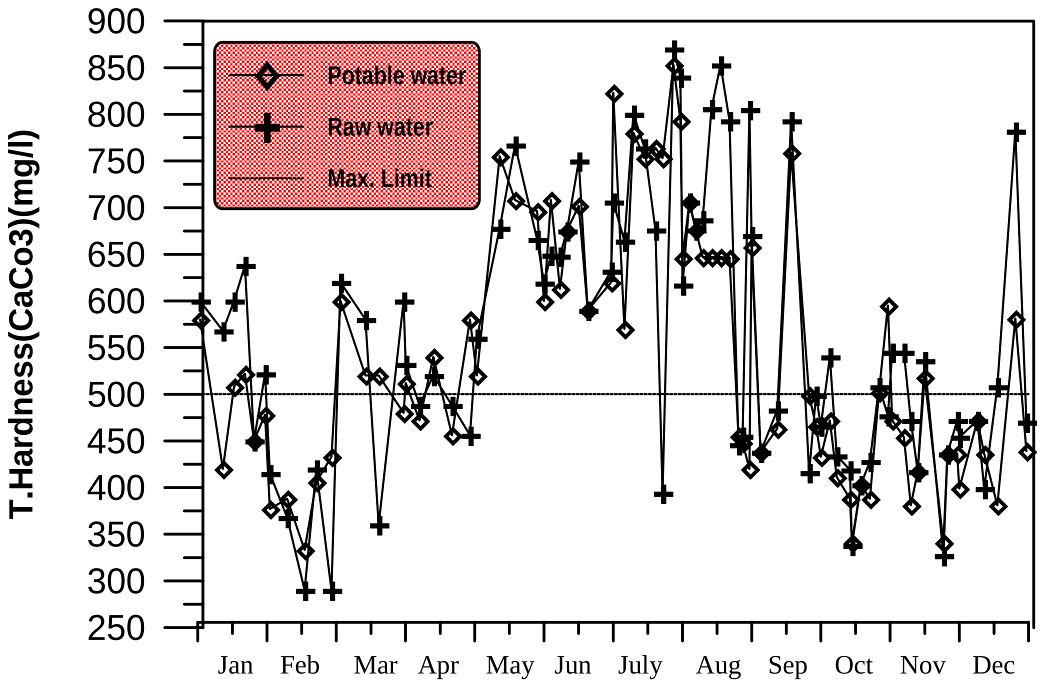


Fig. (٧-١٢): Hardness concentration of potable and raw water at (HKWTP) during year (٢٠٠٢).

D : Calcium and magnesium

Calcium is widely available in natural water due to the dissolution of the calcium component that is available in soil. Calcium considered as the main cause of hardness, as well as, it improves the permeability of the soil and decreases the side effects of sodium. Figure (٧-١٣) and Fig. (٧-١٤) show the availability of calcium in the two types of water at the two plants. Generally, calcium concentration in water can be considered less than the allowable limit. So, the water can be considered safe for drinking according to calcium concentration.

Magnesium is available in natural water, sea water, and metal water. Magnesium also considered as the main cause of hardness. Figures (٧-١٥) and

(٧-١٦) represent the concentration of magnesium in the two types of water at the two plants. Accordingly, water can be considered safe for drinking with regard to the previous mentioned parameters.

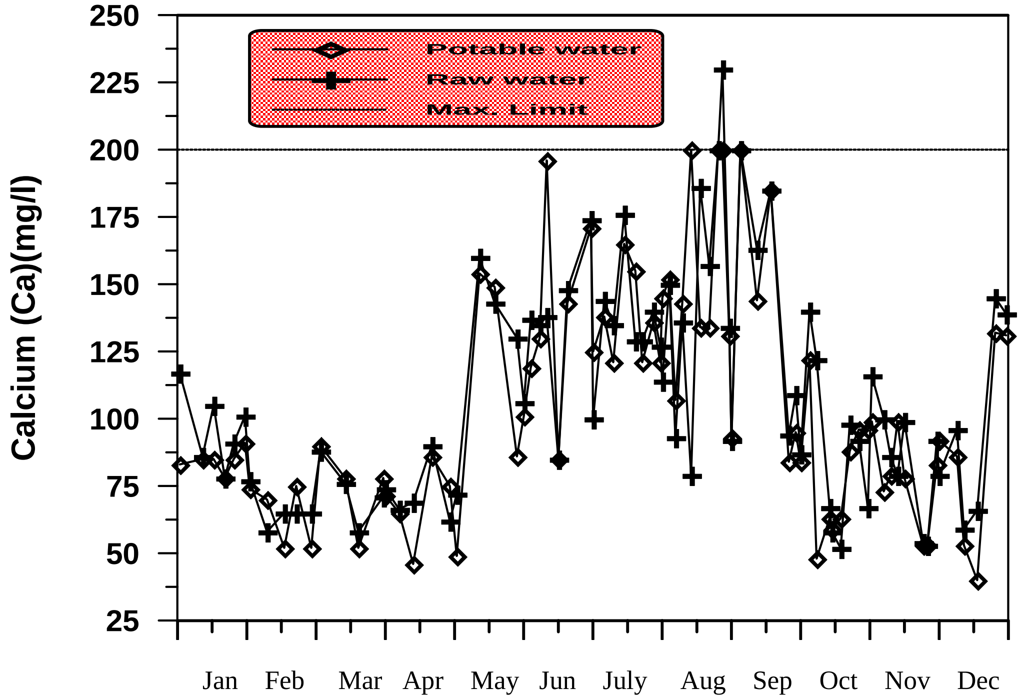


Fig. (٧-١٣): Calcium concentration of potable and raw water at (HJWTP) during year (٢٠٠٢).

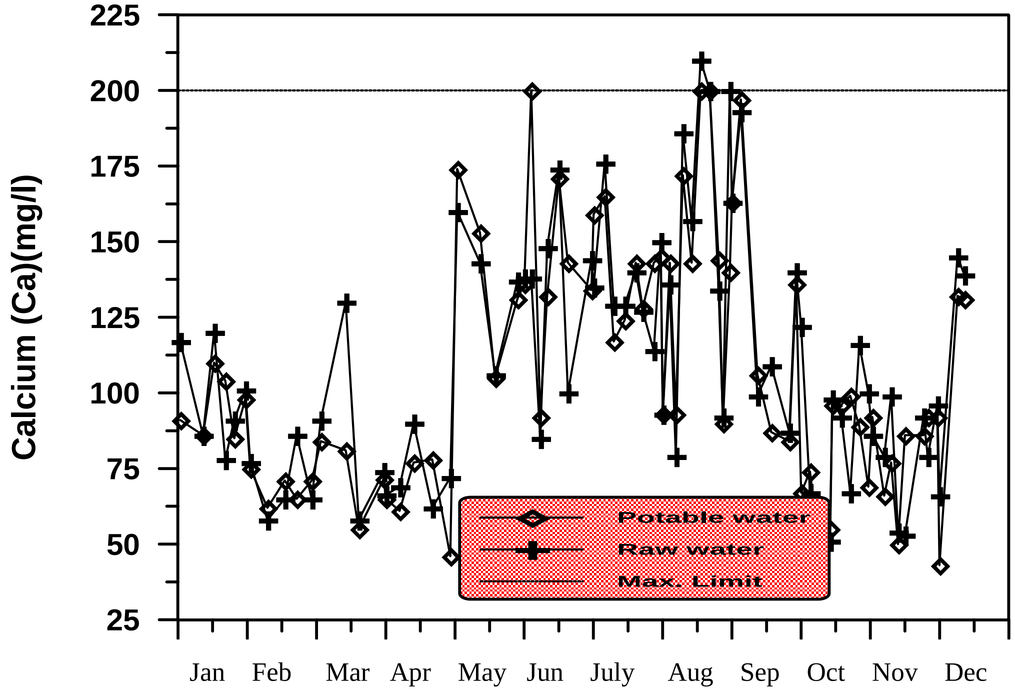


Fig. (٧-١٤): Calcium concentration of potable and raw water at (HKWTP) during year (٢٠٠٢).

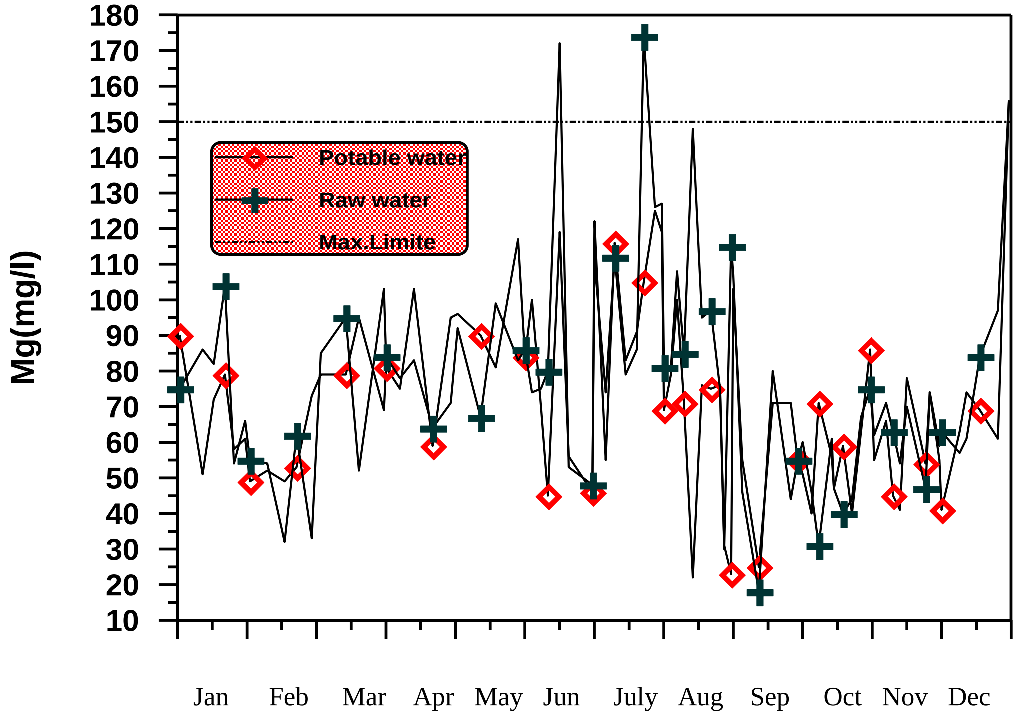


Fig. (٧-١٥): Magnesium concentration of potable and raw water at (HJWTP) during year (٢٠٠٢).

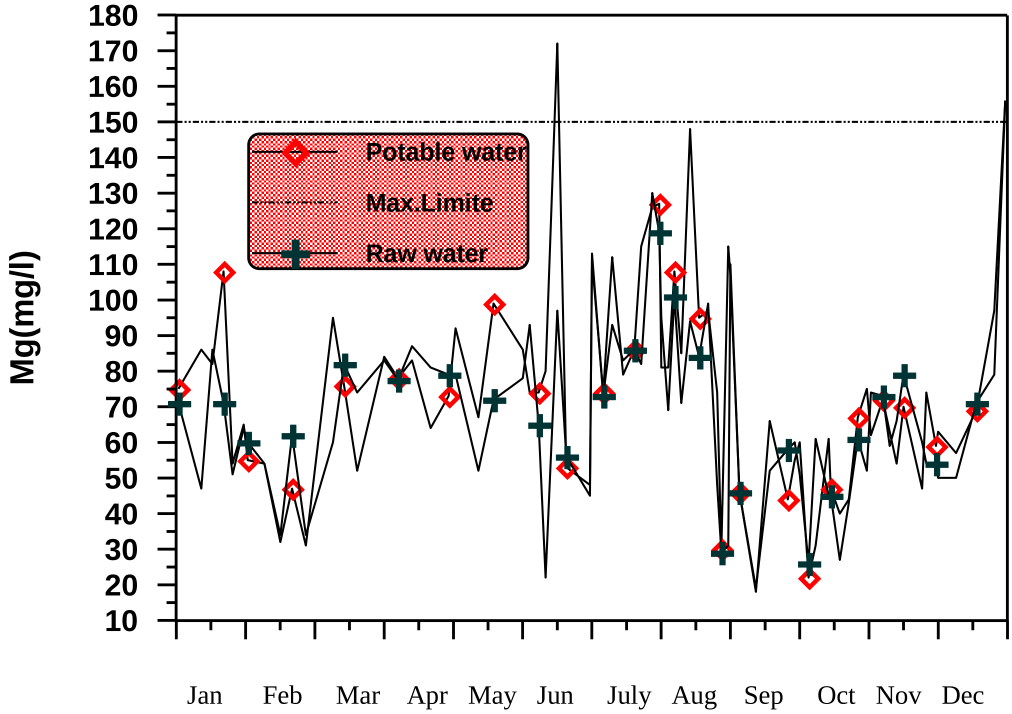


Fig. (٧-١٦): Magnesium concentration of potable and raw water at (HKWTP) during year (٢٠٠٢).

E : Chloride

Chloride is considered as the most important negative ion that is available in natural water. It causes a salt taste for water, especially when it formed with sodium ion to form (Nacl) [٣١]. This ion cannot cause the salt taste when formed with calcium and magnesium. The large concentration of it in water causes the erosion for pipes and metal structures.

Figures (٧-١٧) and (٧-١٨) represent the concentrations of chloride for the two types of water at the two plants. From these figures, it can be clearly seen that the concentration of it is less than the maximum allowable limit.

Water can be considered safe for drinking according to all parameters except turbidity and hardness. So, the following reliability analysis was done for these two parameters for the last (١٦) years.

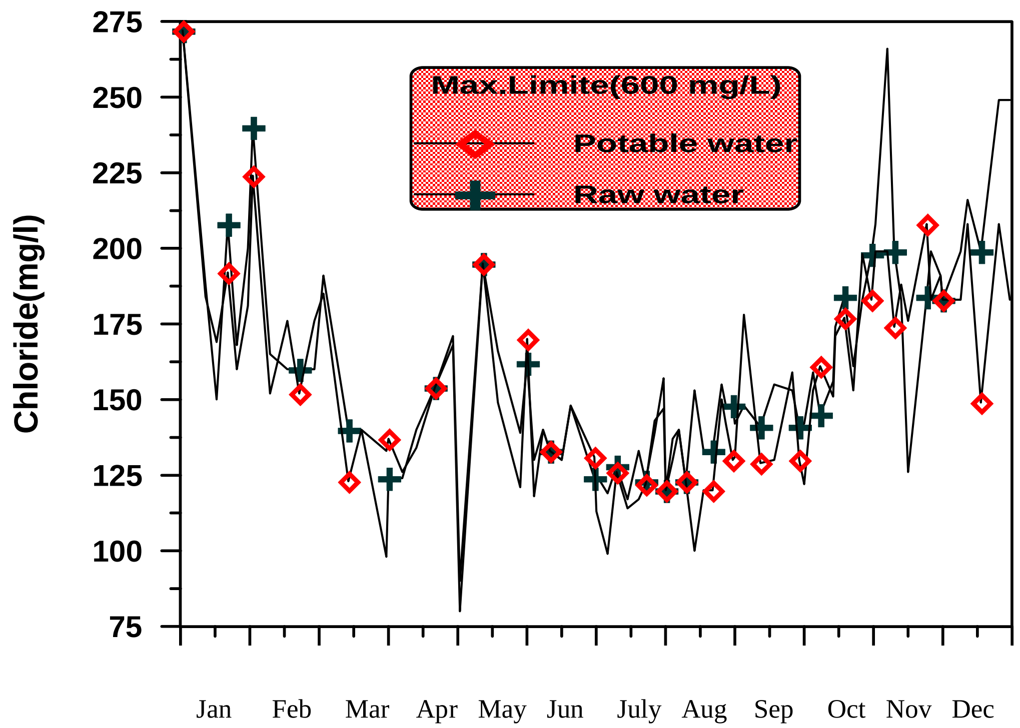


Fig. (٧-١٧): Chloride concentration of potable and raw water at (HJWTP) during year (٢٠٠٢).

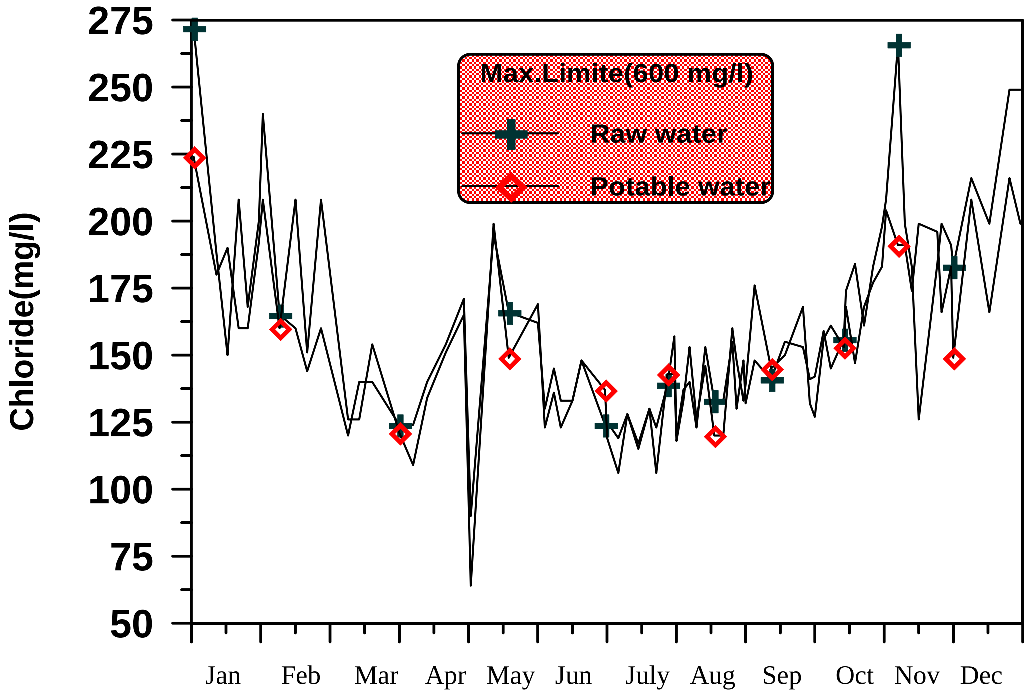


Fig. (٧-١٨): Chloride concentration of raw and potable water at (HKWTP) during year (٢٠٠٢).

٧.٣: Reliability of water treatment plants

The parameters that usually considered for reliability analysis of water as previously mentioned are Turbidity and Hardness of the two types of water, since these two parameters were considered unacceptable, and reliability analysis has to be done with regard to these two parameters. Data obtained was only for (HKWTS). To determine reliability of the turbidity and hardness for water, Behavior method and Niku et al. method were considered in this study. Behavior method has less accuracy than other methods [٣٢]. The two methods can be explained as follows:

٧.٣.١: Behavior method

The probability of reliability can be calculated by dividing the acceptable numbers from the recorded data by the total number of recorded data. However, the reliability is found as [٣٢]:

Rel $=A_i$ / nn. … (٧-١)

Where:

A_i: acceptable numbers from the recorded data.

nn: total number of the recorded data.

For example, if there are (١٠٠) numbers of recorded data and (٩٠) numbers of them are acceptable, then, (١٠) numbers will be considered an unacceptable, in this case the reliability is (٩٠٪). To find reliability of turbidity and hardness for the two types of water for the last (١٦) years a reliability analysis program based on Behavior method as mentioned in Chapter Six has been used. Results of the analysis is presented in Table (٧-١).

Table (٧-١): Reliability analysis for turbidity and hardness using Behavior method for the last (١٦) years.

	Turbidity of raw water	**Turbidity of potable**	**Hardness of raw water**	**Hardness of potable**

		water		water
Reliability (%)	٧.٨١٣	٨٢.٨١٣	٤٧.٣٩٦	٥١.٠٤٢

٧.٣.٢: Niku et al. method

Before explaining this method, it is essential to mentione the condition for using this method. The condition for using this method that the data must follow the lognormal distribution [٣٨]. So, the data must be examined before using this method.

A : Turbidity and hardness concentration distribution

Turbidity and hardness variabilities can be shown and analyzed by determining the histogram and probability density function (p.d.f) of the data.

Selecting the type of distribution is of vital importance as well as the test. In the two following paragraghs the tests used in the present study are introduced.

I : Kolmogorov-Smirnov test

Kolmogorov-Smirnov test is one of the most commonly used tests for selecting the suitable type of distribution for a given histogram. This test involves the examination of random sample from an unknown continuous distribution to test the hypothesis that the unknown distribution function is the specified known continuous distribution function. The conventional null hypothesis (H_o) is that the distribution functions are identical. The test is based on the difference between the observed cumulative density function (C.D.F) (observed cumulative histogram) and the expected cumulative density function at each data point. The largest of these differences, *Dmax*, is used with sample size to test the hypothesis (Ho) [٣٧].

Then (*Dmax*) will be compared with critical valued (Dα) from Table (B-٥) in Appendix B to take decision about the rejection or acceptance of the null hypothesis.

If $D_{\max} \leq D\alpha$ accept H_o.

If $D_{max} > D\alpha$ reject H_o.

II : Chi-Square test ($\chi^{٢}$)

Chi-square test is another test, which can be used to help in using or rejecting a certain distribution toward which it have a certain prediction. This test is based on the difference between actual frequencies, *fi*, and the expected frequencies, $\hat{fi}$, of a known distribution function at mid-interval of each category.

Chi-square statistics for a goodness-of-fit test is given by the following model [١٧].

$$\chi^2 = \sum \frac{\left(fi - \hat{fi}\right)^2}{\hat{fi}} \qquad \ldots(٧\text{-}٢)$$

Where;

fi: actual frequency value at mid-interval of each category.

$\hat{f}_i$: expected frequency value at mid-interval of each category.

i : class interval.

χ^2 Value will be compared with critical value (χ^2_α) from table (B-٦) in Appendix B to take decision about rejection or acceptance of the proposed distribution (null hypothesis, H_o).

If $\chi^2 \le \chi^2_\alpha$ accept H_o.

If $\chi^2 > \chi^2_\alpha$ reject H_o.

B : Lognormal distribution

The lognormal probability law is commonly used in civil engineering practice and seems to adopt originally to produce a better fit to skewed data by using this simple transformation of the familiar normal distribution [٧].

The lognormal distribution has the following probability density function. [٣٨].

$$f_x(x) = \frac{1}{x.\sqrt{2\pi}\,\sigma \ln x} \exp.\left[-\frac{1}{2}\left[\frac{1}{\sigma \ln x}.\ln\left(\frac{x}{m\bar{x}}\right)\right]^2 \right] \quad x \ge 0 \qquad \ldots(\text{٧-٣})$$

Where;

x: represents effluent variable concentration.

$\sigma \ln x$: standard deviation of the logarithm of x.

$m\bar{x}$: median of x.

Figures (٧-١٩), (٧-٢٠), (٧-٢١), and (٧-٢٢) show the histogram of turbidity and hardness for the two types of water for the last (١٦) years at Hilla Al-Kadeem water treatment plants (HKWTS).

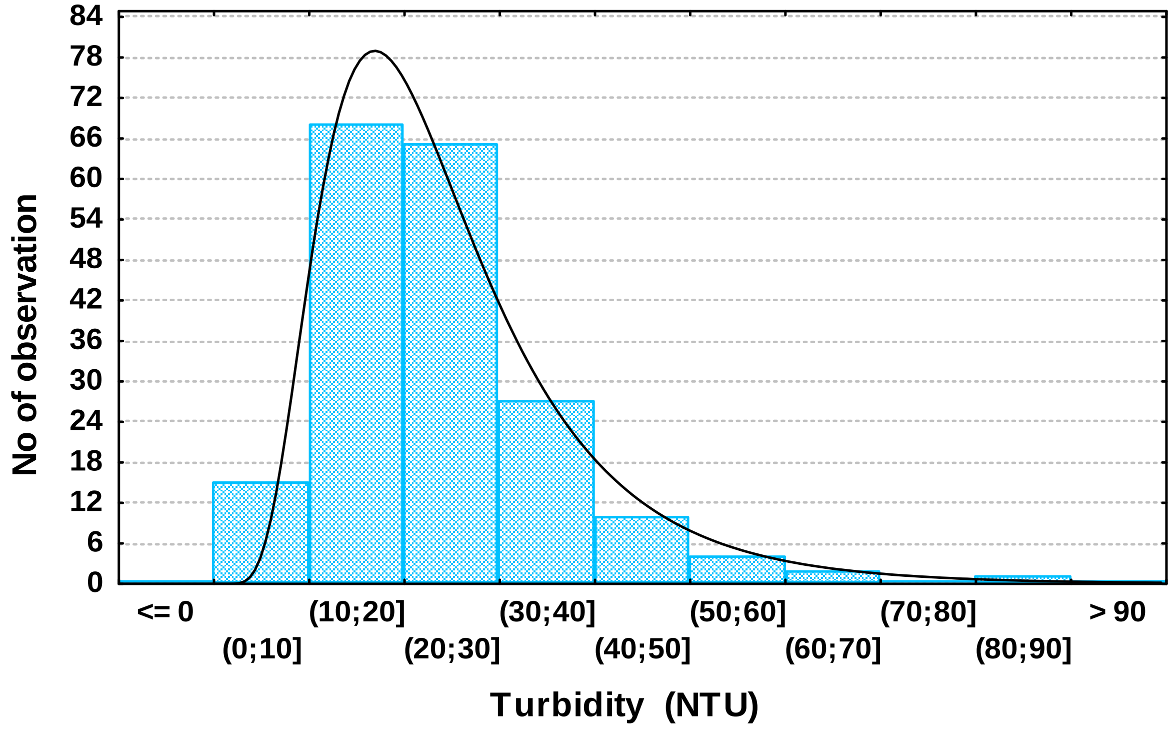


Fig. (٧-١٩): Turbidity histogram of raw water at (HKWTP) in Hilla city.

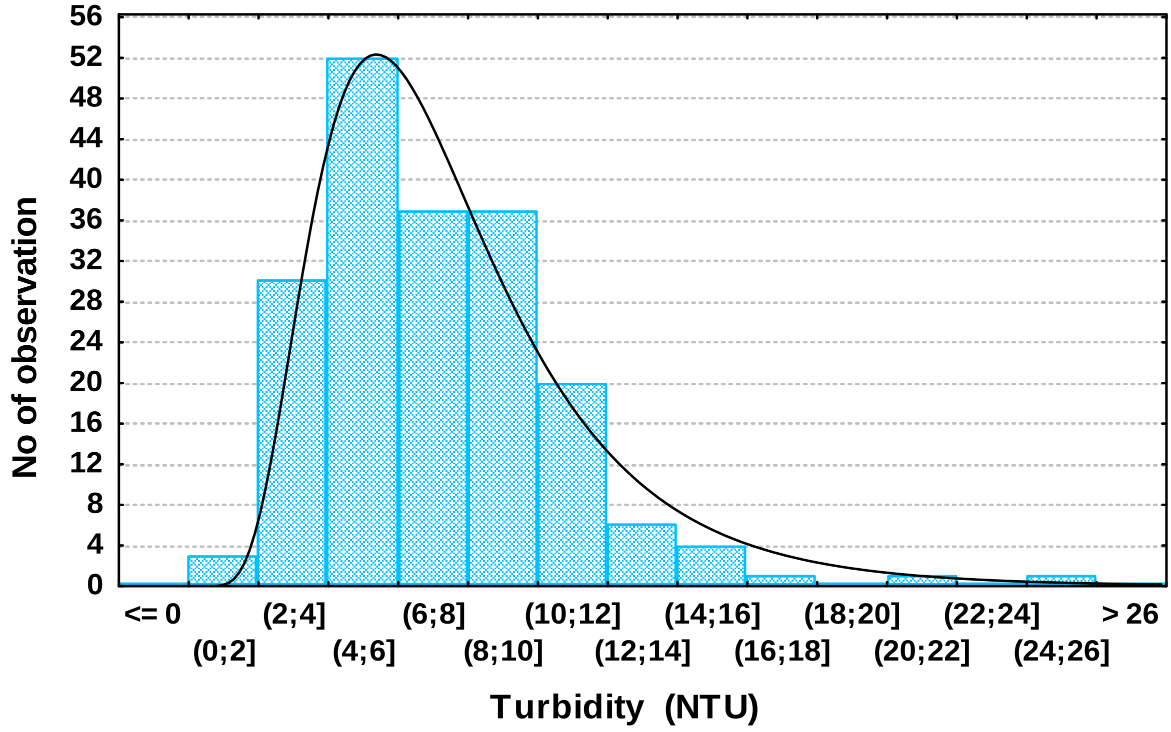


Fig. (٧-٢٠): Turbidity histogram of potable water at (HKWTP) in Hilla city.

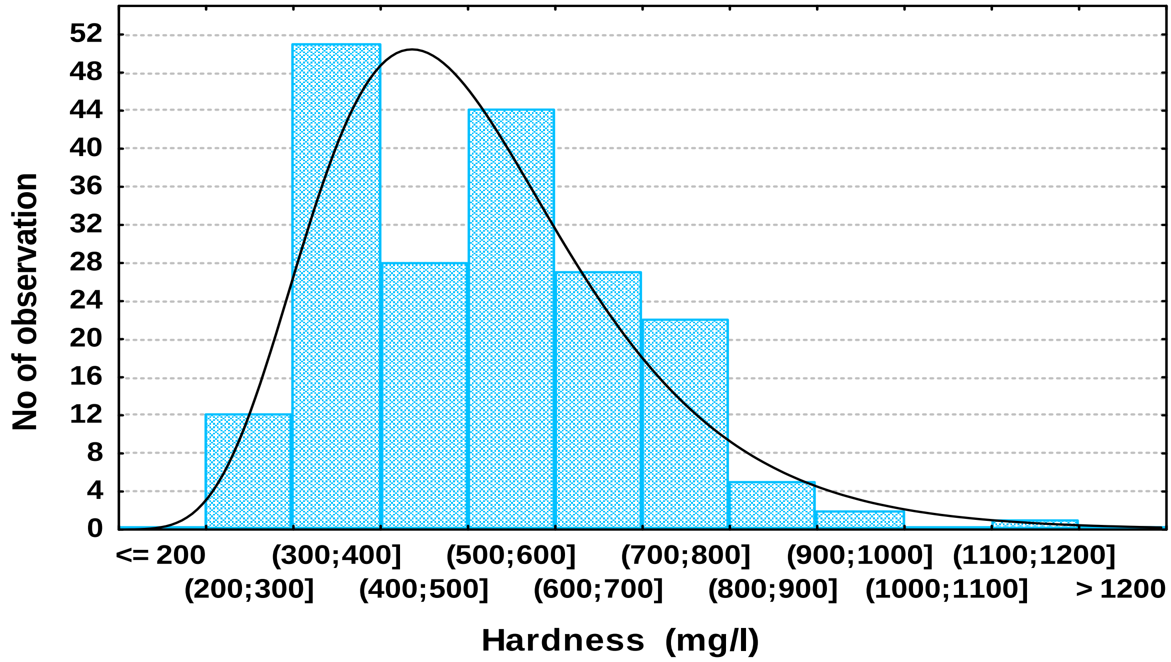


Fig. (٧-٢١): Hardness histogram of potable water at (HKWTP) in Hilla city.

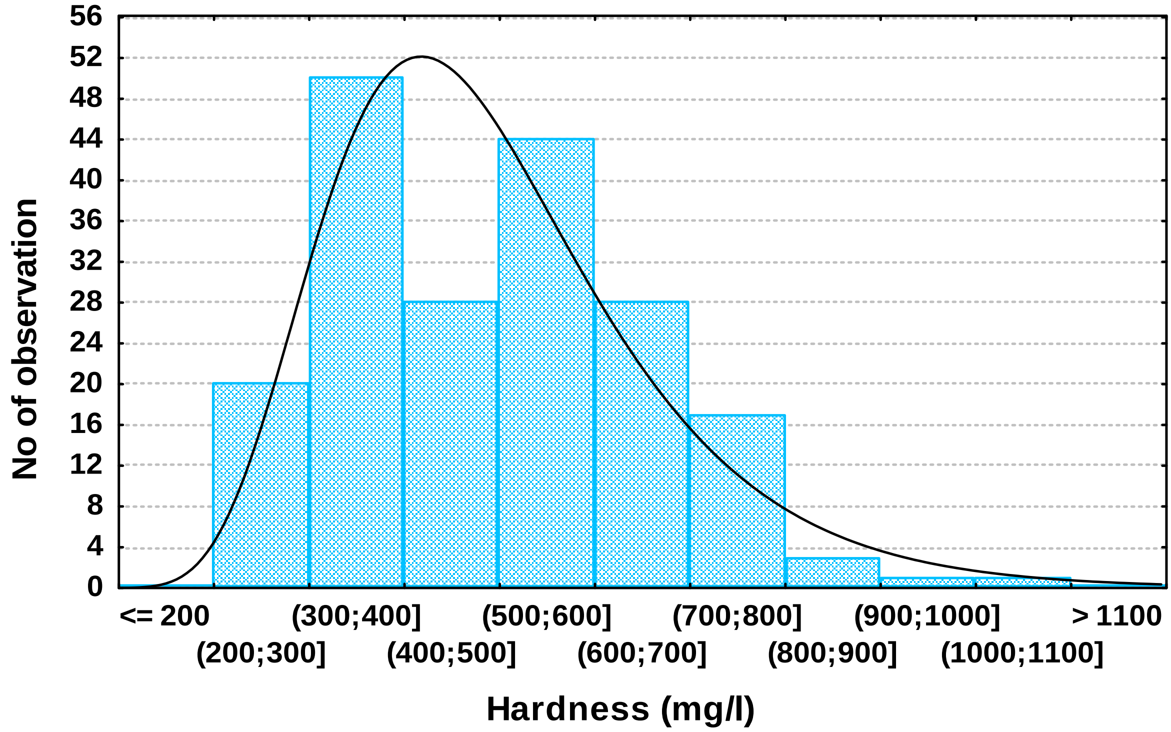


Fig. (٧-٢٢): Hardness histogram of raw water at (HKWTP) in Hilla city.

Based on histograms shown in Figs.(٧-١٩), (٧-٢٠), (٧-٢١), and (٧-٢٢) tables (٧-٢), (٧-٣), (٧-٤), and (٧-٥) have been introduced to show the results for examination. From these tables, it can be noticed that turbidity is following the lognormal distribution while hardness does not.

Table (٧-٢): Turbidity examination of raw water at (HKWTP).

Kolmogorov-Smirnov d = .٠٣٤٣٣٠٠, p = n.s.									
Chi-Square: ٣.٠٠٣٨٢٣, df = ٣, p = .٣٩١٠٥٠٦ (df adjusted)									
	Observed Freq-cy	Cumulatv Observed	Percent Observed	Cumul. % Observed	Expected Freq-cy	Cumulatv Expected	Percent Expected	Cumul. % Expected	Observd-Expected
<=١٠.٠٠٩	١٥	١٥	٧.٨١٢٥	٧.٨١٢٥	١١.٩٠٢٨٥	١١.٩٠٢٨٥	٦.١٩٩٤٠١	٦.١٩٩٤٠١	٣.٠٩٧١٥
٢٠.٠٠٨٠	٦٨	٨٣	٣٥.٤١٦٦٧	٤٣.٢٢٩١٧	٧١.٢٣٢٤٦	٨٣.١٣٥٣١	٣٧.١٠٠٢٤	٤٣.٢٩٩٦٤	-٣.٢٣٢٤٦
٣٠.٠٠٧٠	٦٥	١٤٨	٣٣.٨٥٤١٧	٧٧.٠٨٣٣٤	٥٨.٢٧٣٣٣	١٤١.٤٠٨٦	٣٠.٣٥٠٦٩	٧٣.٦٥٠٣٤	٦.٧٢٦٦٦٩
٤٠.٠٠٦٠	٢٧	١٧٥	١٤.٠٦٢٥	٩١.١٤٥٨٤	٢٨.٥٤٤١٩	١٦٩.٩٥٢٨	١٤.٨٦٦٧٧	٨٨.٥١٧١١	-١.٥٤٤١٩
٥٠.٠٠٥٠	١٠	١٨٥	٥.٢٠٨٣٣٣	٩٦.٣٥٤١٦	١٢.٣٩٧٢٧	١٨٢.٣٥٠١	٦.٤٥٦٩١٢	٩٤.٩٧٤٠١	-٢.٣٩٧٢٧
٦٠.٠٠٤٠	٤	١٨٩	٢.٠٨٣٣٣٣	٩٨.٤٣٧٥	٥.٣١٠٣٨	١٨٧.٦٦٠٥	٢.٧٦٥٨٢٣	٩٧.٧٣٩٨٣	-١.٣١٠٣٨
٧٠.٠٠٣٠	٢	١٩١	١.٠٤١٦٦٧	٩٩.٤٧٩١٦	٢.٣٢٠٩٠٦	١٨٩.٩٨١٤	١.٢٠٨٨٠٥	٩٨.٩٤٨٦٤	-٠.٣٢٠٩١
٨٠.٠٠٢٠	٠	١٩١	٠	٩٩.٤٧٩١٦	١.٠٤٦٤٦	١٩١.٠٢٧٨	٠.٥٤٥٠٣١	٩٩.٤٩٣٦٧	-١.٠٤٦٤٦
٩٠.٠٠١٠	١	١٩٢	٠.٥٢٠٨٣٣	١٠٠	٠.٤٨٨١٠٤	١٩١.٥١٥٩	٠.٢٥٤٢٢١	٩٩.٧٤٧٨٩	٠.٥١١٨٩٦
Infinity	٠	١٩٢	٠	١٠٠	٠.٤٨٤٠٤٧	١٩٢	٠.٢٥٢١٠٨	١٠٠	-٠.٤٨٤٠٥

Table (٧-٣): Turbidity examination of potable water at (HKWTP).

Kolmogorov-Smirnov d = .٠٥٢٢٧٩٤, p = n.s.									
Chi-Square: ٩.٧٣٣١١٢, df = ٥, p = .٠٨٣١٩٢٢ (df adjusted)									
	Observed Freq-cy	Cumulatv Observed	Percent Observed	Cumul. % Observed	Expected Freq-cy	Cumulatv Expected	Percent Expected	Cumul. % Expected	Observd-Expected
<=٢.٠٠٩٣	٣	٣	١.٥٦٢٥	١.٥٦٢٥	١.١٤٠٧٦٧	١.١٤٠٧٦٧	٠.٥٩٤١٤٩	٠.٥٩٤١٤٩	١.٨٥٩٢٣٣
٤.٠٠٨٥٧	٣٠	٣٣	١٥.٦٢٥	١٧.١٨٧٥	٢٥.٣٤٠٩٤	٢٦.٤٨١٧١	١٣.١٩٨٤١	١٣.٧٩٢٥٦	٤.٦٥٩٠٦
٦.٠٠٧٨٦	٥٢	٨٥	٢٧.٠٨٣٣٣	٤٤.٢٧٠٨٣	٥٠.٢٤٨٣٧	٧٦.٧٣٠٠٨	٢٦.١٧١٠٣	٣٩.٩٦٣٥٨	١.٧٥١٦٢٩
٨.٠٠٧١٤	٣٧	١٢٢	١٩.٢٧٠٨٣	٦٣.٥٤١٦٧	٤٤.٧٣٢٤٢	١٢١.٤٦٢٥	٢٣.٢٩٨١٣	٦٣.٢٦١٧١	-٧.٧٣٢٤٢
١٠.٠٠٦٤	٣٧	١٥٩	١٩.٢٧٠٨٣	٨٢.٨١٢٥	٢٩.٨٠٤٢١	١٥١.٢٦٦٧	١٥.٥٢٣٠٣	٧٨.٧٨٤٧٤	٧.١٩٥٧٩١
١٢.٠٠٥٧	٢٠	١٧٩	١٠.٤١٦٦٧	٩٣.٢٢٩١٦	١٧.٦٩٥٦٤	١٦٨.٩٦٢٣	٩.٢١٦٤٧٧	٨٨.٠٠١٢٢	٢.٣٠٤٣٦٣
١٤.٠٠٥٠	٦	١٨٥	٣.١٢٥	٩٦.٣٥٤١٦	١٠.٠٣٧١٥	١٧٨.٩٩٩٥	٥.٢٢٧٦٨٢	٩٣.٢٢٨٩	-٤.٠٣٧١٥
١٦.٠٠٤٣	٤	١٨٩	٢.٠٨٣٣٣	٩٨.٤٣٧٥	٥.٦١١٦٤١	١٨٤.٦١١١	٢.٩٢٢٧٣	٩٦.١٥١٦٣	-١.٦١١٦٤
١٨.٠٠٣٦	١	١٩٠	٠.٥٢٠٨٣٣	٩٨.٩٥٨٣٤	٣.١٣٩٥٨٨	١٨٧.٧٥٠٧	١.٦٣٥٢٠٢	٩٧.٧٨٦٨٣	-٢.١٣٩٥٩
٢٠.٠٠٢٩	٠	١٩٠	٠	٩٨.٩٥٨٣٤	١.٧٧١١٣٦	١٨٩.٥٢١٩	٠.٩٢٢٤٦٦	٩٨.٧٠٩٣	-١.٧٧١١٤
٢٢.٠٠٢١	١	١٩١	٠.٥٢٠٨٣٣	٩٩.٤٧٩١٦	١.٠١١٢٨٧	١٩٠.٥٣٣١	٠.٥٢٦٧١٢	٩٩.٢٣٦٠١	-٠.٠١١٢٩
٢٤.٠٠١٤	٠	١٩١	٠	٩٩.٤٧٩١٦	٠.٥٨٥٤٨٧	١٩١.١١٨٦	٠.٣٠٤٩٤١	٩٩.٥٤٠٩٥	-٠.٥٨٥٤٩
٢٦.٠٠٠٧	١	١٩٢	٠.٥٢٠٨٣٣	١٠٠	٠.٣٤٣٩٣٨	١٩١.٤٦٢٦	٠.١٧٩١٣٤	٩٩.٧٢٠٠٩	٠.٦٥٦٠٦٢
Infinity	٠	١٩٢	٠	١٠٠	٠.٥٣٧٤٣٢	١٩٢	٠.٢٧٩٩١٢	١٠٠	-٠.٥٣٧٤٣

Table (٧-٤): Hardness examination of potable water at (HKWTP).

Kolmogorov-Smirnov d = .٠٥٨٧٧٨٥, p = n.s.									
Chi-Square: ١٩.٤٢١٢٣, df = ٤, p = .٠٠٠٦٥١٠ (df adjusted)									
	Observed Freq-cy	Cumulatv Observed	Percent Observed	Cumul. % Observed	Expected Freq-cy	Cumulatv Expected	Percent Expected	Cumul. % Expected	Observd-Expected
<= ٢١٠	٠	٠	٠	٠	١.٥٨٢٦٤	١.٥٨٢٦٤	٠.٨٢٤٢٩٢	٠.٨٢٤٢٩٢	-١.٥٨٢٦٤
٣٢٠.	٣٦	٣٦	١٨.٧٥	١٨.٧٥	٢٣.١٣١٩	٢٤.٧١٤٥٣	١٢.٠٤٧٨٦	١٢.٨٧٢١٥	١٢.٨٦٨١
٤٣٠.	٤١	٧٧	٢١.٣٥٤١٧	٤٠.١٠٤١٧	٥٢.٧٢٢٣١	٧٧.٤٣٦٨٤	٢٧.٤٥٩٥٤	٤٠.٣٣١٦٩	-١١.٧٢٢٣
٥٤٠.	٤٢	١١٩	٢١.٨٧٥	٦١.٩٧٩١٧	٥١.١٧٩٦١	١٢٨.٦١٦٥	٢٦.٦٥٦٠٥	٦٦.٩٨٧٧٤	-٩.١٧٩٦١
٦٥٠.	٤٠	١٥٩	٢٠.٨٣٣٣٣	٨٢.٨١٢٥	٣٢.٧٦٣١٢	١٦١.٣٧٩٦	١٧.٠٦٤١٣	٨٤.٠٥١٨٦	٧.٢٣٦٨٧٧
٧٦٠.	٢٦	١٨٥	١٣.٥٤١٦٧	٩٦.٣٥٤١٦	١٦.٩٣٤٢٥	١٧٨.٣١٣٨	٨.٨١٩٩٢٢	٩٢.٨٧١٧٩	٩.٠٦٥٧٤٨
٨٧٠.	٣	١٨٨	١.٥٦٢٥	٩٧.٩١٦٦٦	٧.٨١٥٨٤	١٨٦.١٢٩٧	٤.٠٧٠٧٥	٩٦.٩٤٢٥٤	-٤.٨١٥٨٤
٩٨٠.	٢	١٩٠	١.٠٤١٦٦٧	٩٨.٩٥٨٣٤	٣.٣٩٩١٧٧	١٨٩.٥٢٨٩	١.٧٧٠٤٠٥	٩٨.٧١٢٩٤	-١.٣٩٩١٨
١٠٩٠.	٢	١٩٢	١.٠٤١٦٦٧	١٠٠	١.٤٣٦٠٠٦	١٩٠.٩٦٤٩	٠.٧٤٧٩٢	٩٩.٤٦٠٨٦	٠.٥٦٣٩٩٤
Infinity	٠	١٩٢	٠	١٠٠	١.٠٣٥١٤٧	١٩٢	٠.٥٣٩١٣٩	١٠٠	-١.٠٣٥١٥

Table (٧-٥): Hardness examination of raw water at (HKWTP).

Kolmogorov-Smirnov d = .٠٩١٤٥٥٧, p < .١٠									
Chi-Square: ٣١.٢٧٦٠٨, df = ٤, p = .٠٠٠٠٠٢٧ (df adjusted)									
	Observed Freq-cy	Cumulatv Observed	Percent Observed	Cumul. % Observed	Expected Freq-cy	Cumulatv Expected	Percent Expected	Cumul. % Expected	Observd-Expected
<=٢٥٤.٥٥	١	١	٠.٥٢٠٨٣٣	٠.٥٢٠٨٣٣	٤.٧٧٢٢٥٥	٤.٧٧٢٢٥٥	٢.٤٨٥٥٥	٢.٤٨٥٥٥	-٣.٧٧٢٢٦
٣٥٩.٠٩١	٥١	٥٢	٢٦.٥٦٢٥	٢٧.٠٨٣٣٣	٢٩.٦٦٨٢٥	٣٤.٤٤٠٥	١٥.٤٥٢٢١	١٧.٩٣٧٧٦	٢١.٣٣١٧٥
٤٦٣.٦٣٦	٢٣	٧٥	١١.٩٧٩١٧	٣٩.٠٦٢٥	٥٠.٦٣٠٧٦	٨٥.٠٧١٢٦	٢٦.٣٧٠١٩	٤٤.٣٠٧٩٥	-٢٧.٦٣٠٨
٥٦٨.١٨٢	٤٧	١٢٢	٢٤.٤٧٩١٧	٦٣.٥٤١٦٧	٤٥.٨٥٤٩	١٣٠.٩٢٦٢	٢٣.٨٨٢٧٦	٦٨.١٩٠٧١	١.١٤٥١
٦٧٢.٧٢٧	٣٥	١٥٧	١٨.٢٢٩١٧	٨١.٧٧٠٨٤	٢٩.٨٩٦٧٦	١٦٠.٨٢٢٩	١٥.٥٧١٢٣	٨٣.٧٦١٩٤	٥.١٠٣٢٣٥
٧٧٧.٢٧٣	٢٥	١٨٢	١٣.٠٢٠٨٣	٩٤.٧٩١٦٦	١٦.٢٩٧١٦	١٧٧.١٢٠١	٨.٤٨٨١٠٥	٩٢.٢٥٠٠٥	٨.٧٠٢٨٣٩
٨٨١.٨١٨	٦	١٨٨	٣.١٢٥	٩٧.٩١٦٦٦	٨.٠٣٧٢٧	١٨٥.١٥٧٤	٤.١٨٦٠٧٨	٩٦.٤٣٦١٣	-٢.٠٣٧٢٧
٩٨٦.٣٦٤	٣	١٩١	١.٥٦٢٥	٩٩.٤٧٩١٦	٣.٧٤٨٤٢٨	١٨٨.٩٠٥٨	١.٩٥٢٣٠٦	٩٨.٣٨٨٤٤	-٠.٧٤٨٤٣
١٠٩٠.٩١	٠	١٩١	٠	٩٩.٤٧٩١٦	١.٦٩٦٩٩١	١٩٠.٦٠٢٨	٠.٨٨٣٨٥	٩٩.٢٧٢٢٩	-١.٦٩٦٩٩
١١٩٥.٤٥	١	١٩٢	٠.٥٢٠٨٣٣	١٠٠	٠.٧٥٧٧٨٣	١٩١.٣٦٠٦	٠.٣٩٤٦٧٨	٩٩.٦٦٦٩٦	٠.٢٤٢٢١٧
Infinity	٠	١٩٢	٠	١٠٠	٠.٦٣٩٤٤١	١٩٢	٠.٣٣٣٠٤٢	١٠٠	-٠.٦٣٩٤٤

C : Development of the reliability model

The coefficient of reliability may be introduced which relates the mean constituent values (that is, the design values) to the standard that must be achieved on probability basis. The mean constituent value, *mx*, is then obtained from the following equation; [٣٨].

$$mx = (COR) * X_s \qquad \ldots(٧\text{-}٤)$$

Where:

X_s: a fixed standard.

COR: coefficient of reliability.

Two statistical parameters are used in reliability determinations:

** The first parameter is the coefficient of variation, Vx, which is the ratio of the standard deviation, σ_x, and the mean value, mx [٣٨].

$$Vx = \frac{\sigma_x}{mx} \qquad \ldots(٧\text{-}٥)$$

** The second parameter is the percentiles $Z_{١-\alpha}$, of the standard normal distribution; that is the number of standard deviations by which *X* differs from the mean. That is [٣٨].

$$Z_{1-\alpha} = -\frac{Ln[\frac{mx}{X_s}(V_x^2+1)^{-1/2}]}{[Ln(V_x^2+1)]^{1/2}} \qquad \ldots(٧\text{-}٦)$$

In table (B-٧) in Appendix B, the fractional reliability for effluent concentrations is presented as a function of *Vx* and *COR*.

D : Reliability model applications

Table (B-٧) in Appendix B, may be used to estimate the reliability of a treatment plant under operation if the mean and standard deviation for the plant are known. For example, there is (٨٧ %) reliability in a plant operating with COR = ٠.٦ and a Vx = ٠.٧ [٣٨].

Using eq. (٧-٦) to obtain $Z_{١-\alpha}$ and from table (B-٨) in Appendix B, for normal distribution, the reliability of hardness and turbidity can be chosen. Table (٧-٦) shows the variables and results of the reliability analysis for turbidity of the two types of water with confidence interval (٩٥٪) [٥٤]. It is found that hardness parameter does not follows the lognormal distribution.

Table (٧-٦): Reliability analysis for turbidity of water by using Niku et al. model for the last (١٦) years at (HKWTP).

	V_X	mx /Xs	$Z_{١-\alpha}$	Reliability (%)
Turbidity of potable water	٠.٤٧	٠.٧٦	٠.٨٤	٧٩.٩٥
Turbidity of raw water	٠.٤٩١	٢.٤٥	-١.٧	٤.٤٦

بسم الله الرحمن الرحيم

وَهُوَ الَّذِي خَلَقَ مِنْ الْمَاءِ بَشَرًا فَجَعَلَهُ نَسَبًا وَصِهْرًا وَكَانَ رَبُّكَ قَدِيرًا

صدق الله العظيم

الآية (٥٤) سورة الفرقان

Table (B-١) : Program results for optimum design of Hilla city pipe network (٢٠٢٥).

PIPE	Q [M٣/S]	HF [M]	D [MM]	LENGTH [M]	SLOPE *١٠**-٣	VELOCITY [M/S]
١	-٠.٠٣٠٦٨	٧.٢١٤	٢٥٠	١٧٠٠	٤.٢٤٣٥٦٥	-٠.٦٢٤٩
٢	٠.٠٣٩٧٢	١.٥٠٢	٢٠٠	٦٥٠	٢.٣١٠٣٦٨	١.٢٦٤٥
٣	٠.٠٥٣٩	١.٩٩٢	٢٥٠	٤٩٠	٤.٠٦٥٧٢	١.٠٩٨١
٤	٠.٠٣٠٦٤	٥.٤٢١	١٥٠	١٢٨٠	٤.٢٣٥٣٧٨	١.٧٣٤١
٥	٠.٠٧١٦٨	١.٧٠٢	٣٠٠	٦٠٠	٢.٨٣٥٩٦٧	١.٠١٤
٦	٠.١٣٤٨٢	٤.١٤	٣٥٠	٩٦٠	٤.٣١٢٩٦	١.٤٠١٣
٧	-٠.٠٨٦٠٣	٤.٧٧٢	٣٠٠	١٢٠٠	٣.٩٧٦٤٨٧	-١.٢١٧١
٨	٠.٠٥٧٣٦	٤.٧٩	٣٠٠	١٠٥٠	٤.٥٦١٩١٥	٠.٨١١٥
٩	٠.٠٠٣٩٢	٢.٣٤١	١٠٠	٨٥٠	٢.٧٥٤١١٢	٠.٤٩٩٦
١٠	-٠.٠٠٣٨٩	٤.٠٠٣	١٠٠	١٤٨٠	٢.٧٠٤٧٦	-٠.٤٩٤٨
١١	-٠.٠٠٨٩٣	٠.٤٣٩	١٥٠	٢٥٠	١.٧٥٤٢٠٤	-٠.٥٠٥٦
١٢	٠.٠٠٧٠٥	٠.٤٨٧	١٠٠	٤٣٠	١.١٣٢٤٨٦	٠.٨٩٨١
١٣	٠.٠٢٤٢	٠.٦٥٦	١٥٠	٢٤٠	٢.٧٣٥٤	١.٣٦٩٤
١٤	٠.٠٠١٥٧	٠.٧٠٥	١٠٠	١٤٠٠	٠.٥٠٣٧٣٨	٠.١٩٩٦
١٥	٠.٠٣١٠١	٢.١٦٥	٢٠٠	٥٠٠	٤.٣٣٠٩٠٢	٠.٩٨٧٢
١٦	٠.٠٠٣٠٤	٢.٣٢٤	١٠٠	١٣٥٠	١.٧٢١٢٦٦	٠.٣٨٧٦
١٧	٠.٠٠٣٠٢	٠.٧٦٥	١٠٠	٤٥٠	١.٦٩٩٩١٤	٠.٣٨٥
١٨	٠.٠١٤٤٢	٢.٣٤٣	١٥٠	٥٥٠	٤.٢٥٩٨٠٣	٠.٨١٦٣
١٩	٠.٠٠٣٠٤	٠.٩٤٢	١٠٠	٥٥٠	١.٧١٣٥٧٩	٠.٣٨٦٧
٢٠	٠.٠٨٢٢٦	٠.٧٣٢	٣٠٠	٢٠٠	٣.٦٦٠٢٧٨	١.١٦٣٨
٢١	-٠.٠٠٢٩٤	١.٠٤٩	١٠٠	٦٥٠	١.٦١٣١٣٣	-٠.٣٧٤٣
٢٢	٠.٠٤٦٨٤	٠.٦٢٧	٢٥٠	٢٠٠	٣.١٣٤٢٢٢	٠.٩٥٤١
٢٣	٠.٠٢٥٠٢	٠.٧٢٧	٢٠٠	٢٥٠	٢.٩٠٩٥٣١	٠.٧٩٦٤
٢٤	-٠.٠٠٦٧٤	٠.١٨٨	١٠٠	١٨٠	١.٠٤١٩٦٣	-٠.٨٥٨٦
٢٥	٠.٠٠٢٨٨	١.٠٥٥	١٠٠	٦٨٠	١.٥٥١٢٦٣	٠.٣٦٦٥
٢٦	٠.٠٢٨٨١	١.٥١١	٢٠٠	٤٠٠	٣.٧٧٧٢٦٥	٠.٩١٦٩
٢٧	-٠.٠٤٧٨٥	٠.٤٨٩	٢٥٠	١٥٠	٣.٢٦٠٤٧٣	-٠.٩٧٤٧
٢٨	٠.٠٢٩٣٧	٠.٢٦٤	٢٠٠	٢٠٠	١.٣٢٠٧٥٣	٠.٩٣٤٩
٢٩	٠.٠٠٢٠١	١.٣٥٣	١٠٠	١٧٠٠	٠.٧٩٦٠٧	٠.٢٥٥٦
٣٠	٠.٠٣٦٨١	٠.٨٠٢	٢٥٠	٤٠٠	٢.٠٠٦٠٣٥	٠.٧٤٩٨
٣١	٠.٠٥٩٣١	٥.٣٣٨	٢٥٠	١١٠٠	٤.٨٥٢٧٨٢	١.٢٠٨٢
٣٢	٠.٠٨٧٢١	١.٣٠٥	٣٠٠	٣٢٠	٤.٠٧٨٠١٩	١.٢٣٣٧
٣٣	-٠.٠٩٧٦١	١.٠٠٥	٣٥٠	٢٠٠	٥.٠٢٤٠٩	-١.٠١٤٥
٣٤	-٠.٢٣٦٩٥	١.٨٠٢	٤٥٠	٥٠٠	٣.٦٠٣٧٣٧	-١.٤٨٩٩
٣٥	-٠.٠١٨٤٦	١.٠١١	١٥٠	٦١٠	١.٦٥٦٨٠٤	-١.٠٤٤٦
٣٦	-٠.٠٠٣٧٦	١.٦٠٢	١٠٠	٦٣٠	٢.٥٤٢١٣٥	-٠.٤٧٨٥
٣٧	-٠.٠٠١٩٣	١.٥٨٨	١٠٠	٢١٤٠	٠.٧٤٢٠٨٤	-٠.٢٤٦١
٣٨	-٠.٠٠١٨٢	٠.٢٦٧	١٠٠	٤٠٠	٠.٦٦٧٢٦٢	-٠.٢٣٢٤
٣٩	٠.٠٢٦٠٨	١.٣٨٢	٢٠٠	٤٤٠	٣.١٤١٠٣٩	٠.٨٣
٤٠	٠.٠٥٠٥٥	٠.٩٠٣	٢٥٠	٢٥٠	٣.٦١٠٢٤٥	١.٠٢٩٩
٤١	٠.٠٠٩١٦	١.٨٠١	١٠٠	٩٨٠	١.٨٣٧٢٤٦	١.١٦٦٣
٤٢	-٠.١٢٣٣٨	٢.٦٣٥	٣٥٠	٧٢٠	٣.٦٥٩٧٧٣	-١.٢٨٢٤
٤٣	-٠.٣٨٣١٩	٠.٦٦١	٥٥٠	٢٠٠	٣.٣٠٣٠٣٢	-١.٦١٢٩

PIPE	Q [M٣/S]	HF [M]	D [MM]	LENGTH [M]	SLOPE *١٠**-٣	VELOCITY [M/S]
٤٤	٠.٢٣٠٦٦	١.٠٩٧	٤٥٠	٣٢٠	٣.٤٢٨٥٠٧	١.٤٥٠٣
٤٥	٠.١٣٣٧٥	٠.٧٦٥	٣٥٠	١٨٠	٤.٢٤٩٥٣	١.٣٩٠١
٤٦	٠.١١٩٠٤	٠.٦٨٥	٣٥٠	٢٠٠	٣.٤٢٤٨٩٢	١.٢٣٧٣
٤٧	٠.٠٧٦٦٣	١.٤١٢	٣٠٠	٤٤٠	٣.٢٠٩٩٧٧	١.٠٨٤٢
٤٨	-٠.٠٤١٣٩	١.٧٤٥	٢٥٠	٧٠٠	٢.٤٩٣١٩٩	-٠.٨٤٣٢
٤٩	٠.٠٠١٠١	٠.١٣٤	١٠٠	٦٠٠	٠.٢٢٢٨٥٨	٠.١٢٨٥
٥٠	٠.٠٠١٠١	٠.٢٤٥	١٠٠	١١٠٠	٠.٢٢٣١٠٦	٠.١٢٨٦
٥١	٠.٠٤٨٨٩	٤.١٣٩	٢٥٠	١٢٢٠	٣.٣٩٣٠١١	٠.٩٩٥٩
٥٢	-٠.٠٩٦٩١	٠.٩٩١	٣٥٠	٢٠٠	٤.٩٥٧٣١٣	-١.٠٠٧٢
٥٣	-٠.٠٠٣٥٨	٢.٤٣٦	١٠٠	١٠٥٠	٢.٣١٩٧٤١	-٠.٤٥٥٤
٥٤	-٠.٠٥٢٨٨	٤.٧٠٩	٢٥٠	١٢٠٠	٣.٩٢٤٥٣٨	-١.٠٧٧٣
٥٥	-٠.٠٠٣٧١	١.٨٦٦	١٠٠	٧٥٠	٢.٤٨٨٠٣٢	-٠.٤٧٢٩
٥٦	-٠.١١٧٩٥	٠.٦٧٣	٣٥٠	٢٠٠	٣.٣٦٧٠٣٣	-١.٢٢٥٩
٥٧	-٠.١٤١٠٢	٣.٢٨١	٣٥٠	٧٠٠	٤.٦٨٧٦٤٧	-١.٤٦٥٨
٥٨	-٠.٠٢٦٢٢	٢.٢٨٥	٢٠٠	٧٢٠	٣.١٧٣٦٨٧	-٠.٨٣٤٧
٥٩	-٠.٠٠٢٧	٠.٧١٤	١٠٠	٥٢٠	١.٣٧٣٨٨١	-٠.٣٤٣٢
٦٠	-٠.٠١١٧٥	١.٣١١	١٥٠	٤٥٠	٢.٩١٢٣١٤	-٠.٦٦٤٧
٦١	٠.٠١١٥٧	٣.٠٠٢	١٠٠	١٠٦٠	٢.٨٣٢٢٧	١.٤٧٣٣
٦٢	-٠.٠٠١٣٣	٠.١٤٨	١٠٠	٤٠٠	٠.٣٧٠٤٧٩	-٠.١٦٩١
٦٣	٠.٠٠١٩٨	٠.٦٢٦	١٠٠	٨١٠	٠.٧٧٢٣٩٩	٠.٢٥١٥
٦٤	٠.٠٤٧٠٣	٠.٥٦٩	٢٥٠	١٨٠	٣.١٥٨٨٢٤	٠.٩٥٨٢
٦٥	٠.١١٨٠٢	٠.٥٠٦	٣٥٠	١٥٠	٣.٣٧٠٨١٩	١.٢٢٦٧
٦٦	-٠.٠٤٥٥٩	٢.٤١٥	٢٥٠	٨١٠	٢.٩٨١٤١٧	-٠.٩٢٨٧
٦٧	٠.٠١١٣٨	٠.٩٠٧	١٥٠	٣٣٠	٢.٧٤٨٠٦٧	٠.٦٤٤٢
٦٨	٠.٠٤٨٨٧	١.٢٨٨	٢٥٠	٣٨٠	٣.٣٩٠٦٩٨	٠.٩٩٥٥
٦٩	٠.٠٥٤٣٩	٤.٠٩٣	٢٥٠	٩٩٠	٤.١٣٤٤٥	١.١٠٨١
٧٠	-٠.٢٩٩٤١	٠.٨٩٨	٥٠٠	٢٧٠	٣.٣٢٧٢٣٥	-١.٥٢٤٩
٧١	-٠.٢٢١٢	٠.٦٣٥	٤٥٠	٢٠٠	٣.١٧٢٧٧٩	-١.٣٩٠٨
٧٢	٠.١٩٠٤٣	١.٨٣٥	٤٠٠	٤٣٠	٤.٢٦٦٧٢٦	١.٥١٥٤
٧٣	٠.١١٠٤٨	١.٤٩١	٣٥٠	٥٠٠	٢.٩٨٢٧٣٨	١.١٤٨٣
٧٤	-٠.١٦٩٨٦	١.٥٥٤	٤٠٠	٤٥٠	٣.٤٥٢٦١٥	-١.٣٥١٧
٧٥	٠.٠٠٠٣٥	٠.٠٢١	١٠٠	٦٥٠	٠.٠٣١٧٨٢	٠.٠٤٤٩
٧٦	-٠.٠٠٢١٦	٠.٢	١٠٠	٢٢٠	٠.٩١١٠٨٨	-٠.٢٧٤٩
٧٧	-٠.٠٢٥٥٦	١.٢٧١	٢٠٠	٤٢٠	٣.٠٢٦٢٧٧	-٠.٨١٣٥
٧٨	-٠.٠٠٢٦٩	١.٠٠١	١٠٠	٧٣٠	١.٣٧٠٦٦٢	-٠.٣٤٢٨
٧٩	٠.٠٠٩٧٧	٠.٥١٧	١٠٠	٢٥٠	٢.٠٦٨٩٧٧	١.٢٤٣٥
٨٠	٠.٠٠٥١٤	١.٧٢٦	١٠٠	٣٨٠	٤.٥٤٣١٧٩	٠.٦٥٤٧
٨١	٠.٠٠١٧	٠.٤٢٣	١٠٠	٧٢٠	٠.٥٨٧٥٩٣	٠.٢١٦٩
٨٢	-٠.٠٢٦٧٤	١.١٥٢	١٥٠	٣٥٠	٣.٢٩١٤٧٩	-١.٥١٣٣
٨٣	٠.٠٢٤٣٤	٠.٥٥٣	٢٠٠	٢٠٠	٢.٧٦٥٠٩٣	٠.٧٧٤٨
٨٤	-٠.٠٠٣٦٧	١.٠٩٢	١٠٠	٤٥٠	٢.٤٢٧٤٠٦	-٠.٤٦٦٧
٨٥	٠.٠٠٩٩٣	٠.٩٦	١٠٠	٤٥٠	٢.١٣٣٠١٣	١.٢٦٤٢
٨٦	٠.٠٠٠٦٨	٠.٠٢٧	١٠٠	٢٥٠	٠.١٠٧٥٢٩	٠.٠٨٦٧
٨٧	٠.٠٠٤٢٦	٢.٤٠٣	١٠٠	٧٥٠	٣.٢٠٤٣٩٧	٠.٥٤٢٢
٨٨	٠.٠٠٣٥٧	١.٨٤٥	١٥٠	٨٠٠	٢.٣٠٦٥٢١	٠.٢٠١٨
٨٩	٠.٠٣٢٩٣	٦.٧٧٤	٢٥٠	١٤٠٠	٤.٨٣٨٣١٩	٠.٦٧٠٨
٩٠	٠.١٠٨٠٧	٢.١٧٦	٣٥٠	٧٦٠	٢.٨٦٣٦٦٣	١.١٢٣٣

PIPE	Q [M٣/S]	HF [M]	D [MM]	LENGTH [M]	SLOPE *١٠**-٣	VELOCITY [M/S]
٩١	-٠.٠٥٨٦٦	٧.٣٢٣	٢٥٠	١٥٤٠	٤.٧٥٥١١٥	-١.١٩٥
٩٢	٠.٠٠١٦٨	٠.٤٣١	١٠٠	٧٥٠	٠.٥٧٤٢٣٢	٠.٢١٤٣
٩٣	-٠.٠٠٣١٦	١.٨٩٩	١٠٠	١٠٣٠	١.٨٤٣٢٢٣	-٠.٤٠٢٢
٩٤	٠.٠١٤٣٣	٣.٩٩٩	١٥٠	٩٥٠	٤.٢٠٩٥٨٧	٠.٨١١١
٩٥	-٠.٠٠٢٢٦	١.١٨٩	١٠٠	١٢٠٠	٠.٩٩٠٦٣٦	-٠.٢٨٧٦
٩٦	٠.٠٠٢٦١	٢.٤٠٩	١٠٠	١٨٦٠	١.٢٩٥٠٧٩	٠.٣٣٢٤
٩٧	٠.٠١٥٢١	٥.٦٣٨	٢٠٠	١٢٠٠	٤.٦٩٨٢١٦	٠.٤٨٤١
٩٨	٠.٠٠٤٣١	٣.٢٧٤	١٥٠	١٠٠٠	٣.٢٧٤٤٤٤	٠.٢٤٣٨
٩٩	-٠.٣٩٥٣	٢.٤٨٤	٥٥٠	٧١٠	٣.٤٩٨٨٨٣	-١.٦٦٣٨
١٠٠	-٠.١٦١٢	٣.٤٤٧	٤٠٠	١١٠٠	٣.١٣٣٩١٦	-١.٢٨٢٨
١٠١	٠.٠٤٥٩	٣.٦٢٣	٢٥٠	١٢٠٠	٣.٠١٨٨٨١	٠.٩٣٥
١٠٢	-٠.١٩٨١٥	١.٣٧٨	٤٥٠	٣٠٠	٤.٥٩٢٤٢٥	-١.٢٤٥٩
١٠٣	-٠.٠٦٢٣٢	٣.٩٨٩	٣٠٠	٧٥٠	٥.٣١٨٢٤٨	-٠.٨٨١٦
١٠٤	٠.٠٢٥٢٥	١.٦٢٨	٢٥٠	٥٥٠	٢.٩٥٩٩٦٦	٠.٥١٤٤
١٠٥	٠.٠٠٢٠٦	١.٢٩٨	١٠٠	١٥٥٠	٠.٨٣٧٢٠٩	٠.٢٦٢٦
١٠٦	-٠.٠٧٨٨	٤.٧٣٢	٣٠٠	١٤٠٠	٣.٣٨٠٢٨٥	-١.١١٤٨
١٠٧	-٠.١١٥٥٧	١.١٦٧	٣٥٠	٣٦٠	٣.٢٤٢٥٢٤	-١.٢٠١٢
١٠٨	٠.٠٤٦٦٥	٣.١١١	٢٥٠	١٠٠٠	٣.١١٠٦٥١	٠.٩٥٠٣
١٠٩	٠.٠٢٦٧٩	١.٩٨١	٢٠٠	٦٠٠	٣.٣٠٢٠٧٥	٠.٨٥٢٧
١١٠	-٠.٠٤٢١٤	١.٢٨٩	٢٥٠	٥٠٠	٢.٥٧٧٠١٥	-٠.٨٥٨٤
١١١	-٠.٠٥٢٤٦	٢.١٢٧	٢٥٠	٥٥٠	٣.٨٦٦٣٨٣	-١.٠٦٨٧
١١٢	-٠.٠٠٥٠١	٠.٩٠٩	١٠٠	٢١٠	٤.٣٢٦٥٠٢	-٠.٦٣٧٦
١١٣	٠.٠١٠٣٥	١.٤٩٧	١٥٠	٦٥٠	٢.٣٠٢٦٠٤	٠.٥٨٥٦
١١٤	-٠.٠٥٦٨٩	٤.٢٦٨	٢٥٠	٩٥٠	٤.٤٩٢٤٠٣	-١.١٥٨٩
١١٥	-٠.٠٨٤٣	٧.٠٨٦	٣٠٠	١٨٥٠	٣.٨٣٠٠٣٢	-١.١٩٢٦
١١٦	-٠.٠٠٥٠٤	٤.١١٦	١٠٠	٩٤٠	٤.٣٧٨٦٧٦	-٠.٦٤١٨
١١٧	-٠.٤٥٩٥	٠.٦٠١	٥٥٠	١٣٠	٤.٦٢٣٣٥٤	-١.٩٣٤١
١١٨	٠.٠٩١٢٦	٢.٢١٨	٣٠٠	٥٠٠	٤.٤٣٦٠٥	١.٢٩١١
١١٩	٠.٠٠٣٣٢	١.٣١٦	١٠٠	٦٥٠	٢.٠٢٥٣٣٧	٠.٤٢٣٢
١٢٠	-٠.٠٠٣٣٨	١.١٨٨	١٠٠	٥٧٠	٢.٠٨٤٢٩	-٠.٤٢٩٨
١٢١	-٠.٠٠١١٤	٠.١١٢	١٠٠	٤٠٠	٠.٢٨١٢٣٩	-٠.١٤٥٧
١٢٢	-٠.٠٠١٧٦	٠.٧٤٥	١٠٠	١٢٠٠	٠.٦٢١٢٣١	-٠.٢٢٣٦
١٢٣	-٠.٠٢٢١٨	١.٢١	٢٠٠	٥٢٠	٢.٣٢٦٩٠٣	-٠.٧٠٥٩
١٢٤	٠.٠٠٠٦٩	٠.٠٥٥	١٠٠	٥٠٠	٠.١١٠٣١	٠.٠٨٧٩
١٢٥	٠.٠٥١٠٧	١.٥٤٥	٢٥٠	٤٢٠	٣.٦٧٩٣٨٥	١.٠٤٠٥
١٢٦	٠.٠٠٤٢٧	١.٨٦٩	١٠٠	٥٨٠	٣.٢٢٢٩٠٣	٠.٥٤٣٩
١٢٧	-٠.١٢٤٩٤	٤.١٥٨	٣٥٠	١١١٠	٣.٧٤٥٨٥٤	-١.٢٩٨٦
١٢٨	-٠.٠٠١٤٨	٠.٤٦٨	١٠٠	١٠٣٠	٠.٤٥٤٢٨٨	-٠.١٨٨٨
١٢٩	٠.٠٧٢٦٢	٠.٩٠١	٣٠٠	٣١٠	٢.٩٠٥٢	١.٠٢٧٣
١٣٠	-٠.٣٥٠٩١	١.٢٥	٥٠٠	٢٨٠	٤.٤٦٤٠٢٦	-١.٧٨٧٢
١٣١	-٠.٠٧٩٠٥	٠.٧٤٨	٣٠٠	٢٢٠	٣.٣٩٩٧٧١	-١.١١٨٣
١٣٢	-٠.٠٢٦٤٣	٢.٢٥٤	٢٠٠	٧٠٠	٣.٢٢٠١٥٥	-٠.٨٤١٢
١٣٣	-٠.٠٠٤٦٣	٤.٦٧٣	١٠٠	١٢٥٠	٣.٧٣٨٠٥٣	-٠.٥٨٩٢
١٣٤	٠.١٧٢٤٩	٢.٤٨٧	٤٠٠	٧٠٠	٣.٥٥٢٢٤٦	١.٣٧٢٦
١٣٥	-٠.٠٠٣٠٥	١.٢١٢	١٠٠	٧٠٠	١.٧٣١٠٩٣	-٠.٣٨٨٨
١٣٦	-٠.٠٠٣٥٥	٢.٨٥٥	١٠٠	١٢٥٠	٢.٢٨٣٩٨	-٠.٤٥١٦
١٣٧	٠.١٣٥٠٦	٣.٠٢٩	٣٥٠	٧٠٠	٤.٣٢٧٠٠٥	١.٤٠٣٨

PIPE	Q [M٣/S]	HF [M]	D [MM]	LENGTH [M]	SLOPE *١٠**-٣	VELOCITY [M/S]
١٣٨	٠.٠٠٠٨١	٠.١٤١	١٠٠	٩٥٠	٠.١٤٨٢٦٩	٠.١٠٣١
١٣٩	-٠.٠٥٩٢٥	٧.٥٠٨	٣٠٠	١٥٥٠	٤.٨٤٣٥٨	-٠.٨٣٨٢
١٤٠	٠.٠٢٦٢٦	١.٧٥	٢٠٠	٥٥٠	٣.١٨١٥٢٤	٠.٨٣٥٨
١٤١	-٠.٠٠٢٧٤	٢.١٧٢	١٠٠	١٥٣٠	١.٤١٩٥٤٩	-٠.٣٤٩٣

NODE	HEAD [M]	Q-OUT [M٣/S]
١	٢٨.٧٦٦	٠.٠٤١
٢	٢١.٥٥٢	٠.٠٧
٣	٢٣.٠٥٤	٠.٠١٨
٤	٢٥.٠٤٦	٠.٠٣٤
٥	٣٠.٤٦٨	٠.٠٣٢
٦	٣٤.٦٠٨	٠.٠١٦
٧	٢٩.٨٣٦	٠.٠٢١
٨	٢٩.٣٩٨	٠.٠١٢
٩	٢٥.٣٩٥	٠.٠٣٤
١٠	٢٧.٥٦١	٠.٠٢٢
١١	٢٩.٨٨٥	٠.٠١٤
١٢	٣٠.٥٤٢	٠.٠٠٤
١٣	٢٦.١٦١	٠.٠١١
١٤	٢٨.٥٠٤	٠.٠٦٥
١٥	٢٩.٢٣٧	٠.٠٢٦
١٦	٢٨.١٨٩	٠.٠٣٦
١٧	٢٨.٩١٦	٠.٠١٧
١٨	٢٨.٧٢٩	٠.٠٢١
١٩	٢٩.٧٨٤	٠.٠٢٥
٢٠	٣١.٢٩٥	٠.٠٢١
٢١	٣٠.٨٠٦	٠
٢٢	٢٩.٧٩٦	٠.٠١٥
٢٣	٢٨.١٩٤	٠
٢٤	٢٦.٦٠٦	٠.٠١٣
٢٥	٢٧.٩٦	٠.٠٣٥
٢٦	٢٨.٧٦٢	٠.٠٢٣
٢٧	٣٤.١	٠.٠٢٨
٢٨	٣٥.٤٠٥	٠.٠١
٢٩	٢٧.٩٢٨	٠.٠٢٨
٣٠	٢٨.٤٠٧	٠
٣١	٢٩.٣١	٠
٣٢	٣٠.٧٢٢	٠.٠٤٢
٣٣	٣١.٤٠٧	٠.٠١٥
٣٤	٣٢.١٧٢	٠
٣٥	٣٣.٢٦٩	٠.٠١٢
٣٦	٢٩.٠١	٠.٠٢٥
٣٧	٢٩.٦٣٦	٠.٠٣٥
٣٨	٣٠.٢٠٥	٠.٠٢٢
٣٩	٣٠.٧١١	٠.٠٢٧
٤٠	٢٨.٢٩٦	٠.٠٤
٤١	٣٠.١٤٢	٠.٠٣٤

NODE	HEAD [M]	Q-OUT [M٣/S]
٤٢	٣٢.٥٤٥	٠.٠٢٦
٤٣	٣٢.٢٨٢	٠.٠٤٨
٤٤	٣٠.٧٢٨	٠.٠٣٤
٤٥	٢٩.٤٥٨	٠.٠٢١
٤٦	٢٩.٢٥٨	٠.٠٠٢
٤٧	٢٨.٤٥٩	٠.٠١١
٤٨	٢٨.٩٧٨	٠.٠٢١
٤٩	٣٠.٧٠٧	٠.٠٢١
٥٠	٣١.٢٦١	٠.٠١٩
٥١	٣٠.١٧	٠.٠١٣
٥٢	٣١.١٣	٠.٠١٥
٥٣	٢٦.٩٨٦	٠.٠٢٥
٥٤	٢٩.٩٨٨	٠.٠١٢
٥٥	٢٦.٦٦٢	٠.٠٤٢
٥٦	٢٦.٧٩٦	٠
٥٧	٢٧.٠٤١	٠.٠٤٤
٥٨	٣١.١٨١	٠.٠٤٤
٥٩	٢٩.٣١٥	٠.٠٦٦
٦٠	٢٤.٦٠٦	٠.٠٥٤
٦١	٢٣.٤١٧	٠.٠٢١
٦٢	٢٦.٦٩٢	٠.٠١١
٦٣	٣٢.٣٣	٠.٠٣٣
٦٤	٣٤.٧٣٩	٠.٠٤٧
٦٥	٢٧.٤١٧	٠.٠٤٦
٦٦	٣٦.٩١٦	٠.٠٥٦
٦٧	٣٥.٩٥٣	٠.٠٣٦
٦٨	٣٥.٥٣٩	٠.٠٢
٦٩	٣٤.٣٧٣	٠.٠١٦
٧٠	٣٢.٢٤٨	٠.٠١
٧١	٣٠.٩٦١	٠.٠١
٧٢	٣٠.٠٥٤	٠.٠١٥
٧٣	٢٨.٨٦٧	٠.٠٠٥
٧٤	٢٨.٩٢٤	٠.٠٢٨
٧٥	٣٠.٤٧	٠.٠٢٦
٧٦	٣١.٣٧١	٠.٠١٥
٧٧	٣٣.٥٨٩	٠.٠١٧
٧٨	٢٩.٩٢٣	٠.٠٨٤
٧٩	٣١.٥٥١	٠.٠٢٧
٨٠	٣١.٢٢	٠.٠٧٢
٨١	٢٧.١٠٤	٠.٠٨٧
٨٢	٢٤.٩٣٣	٠.٠٢٩
٨٣	٢٦.٦٨٣	٠.٠٣٤
٨٤	٢٦.٨٢٣	٠.١٣١
٨٥	٢٩.٨٥٣	٠.٠٣٣
٨٦	٣٢.٣٣٩	٠.٠٤٩
٨٧	٢٨.١٨٢	٠.٠٤٤
٨٨	٢٧.٧١٥	٠.٠٢٥

NODE	HEAD [M]	Q-OUT [M٣/S]
٨٩	٢٧.٤٣٤	٠.٠٥٣
٩٠	٢٥.١٨	٠.٠٢٨
٩١	٢٣.٩٦٩	٠.٠٠٧
٩٢	٣٦.٤١	٠
٩٣	٣٣.٩٣	٠
٩٤	٣٣.١٨	٠
٩٥	٣٩.٤	٠
٩٦	٣٤.١٩	٠

TOTAL COST OF THIS STAGE = ٦١٠٣٧٥٠٠٠.٠٠٠ I.D

NO.OF CYCLES = ٢١٤٦١

STANDARD DIVEATION = ٢.٩٥٥٠
COEFF.OF VARIATION = ٠.٠٩٨٧

Table (B-٢): Program results for optimum improvement of Hilla city pipe network.

PIPE	Q [M٣/S]	HF [M]	D [MM]	LENGTH[M]	SLOPE *١٠**-٣	VELOCITY[M/S]	C.H.W.	MMM
١	-٠.٠٨٥٨٨	٨.٦٤٢	٣٠٠	٢١٨٠	٣.٩٦٤١١٣	-١.٢١٥	١٤٥	١
٢	-٠.٠١٧٧٨	٣.٥١٥	٢٠٠	٥٦٠	٦.٢٧٦٦	-٠.٥٦٦١	١٤٥	١
٣	٠.٠١٩١٢	١.١٦٧	١٥٠	٦٦٠	١.٧٦٧٨٤١	١.٠٨١٨	١٤٥	١
٤	٠.٠١٩١٢	٠.٠٧١	١٥٠	٤٠	١.٧٦٨٤٩٤	١.٠٨٢	١٤٥	١
٥	٠.٠١٩١٨	٠.١٤٢	١٥٠	٨٠	١.٧٧٩١٩٩	١.٠٨٥٦	١٤٥	١
٦	٠.٠٣٠٦٢	١.٧٧٦	٢٠٠	٤٢٠	٤.٢٢٨٤٤٢	٠.٩٧٤٦	١٤٥	١
٧	٠.٠٣٥٤٧	٨.٧١٩	٢٠٠	١٥٧٠	٥.٥٥٣٥٥٦	١.١٢٩١	١٤٥	١
٨	-٠.٠٩٣٧٤	٠.٢٨	٣٥٠	٦٠	٤.٦٦١٦٢٣	-٠.٩٧٤٣	١٤٥	١
٩	-٠.٠٠١	٠.٢٢٧	١٠٠	١٠٣٠	٠.٢٢٠١٢٧	-٠.١٢٧٧	١٤٥	١
١٠	٠.٠١٤٥	٠.٣٠١	١٥٠	٧٠	٤.٢٩٩٧٩	٠.٨٢٠٤	١٤٥	٠
١١	٠.٠١١٠٧	١.٥٦٥	١٥٠	٦٠٠	٢.٦٠٧٦٦	٠.٦٢٦٢	١٤٥	٠
١٢	٠.٠٢٥٠٧	١.٩٢٦	٢٥٠	٤٣٠	٤.٤٧٨٦٢٨	٠.٥١٠٧	٦٤	٠
١٣	٠.٠٢٧٢٦	٠.٢٣٩	٢٠٠	٧٠	٣.٤١١٣٤٧	٠.٨٦٧٩	١٤٥	١
١٤	-٠.٠٠٣٩٧	١.٩٤١	١٠٠	٦٩٠	٢.٨١٣١٧١	-٠.٥٠٥٤	١٤٥	١
١٥	٠.٠٠٥٥٥	٠.٠٨٧	١٠٠	١٢٠	٠.٧٢٥٨٢٦	٠.٧٠٦٣	١٤٥	١
١٦	٠.٠٠٣٨٩	٣.١٤٢	١٠٠	١١٦٠	٢.٧٠٩٠٤٥	٠.٤٩٥٢	١٤٥	١
١٧	٠.٠٣٣٥٤	١.٢٠٢	٢٠٠	٢٤٠	٥.٠٠٧١٦٤	١.٠٦٧٧	١٤٥	١
١٨	٠.٠٠٣٤٤	١.٨٩٧	١٠٠	٨٨٠	٢.١٥٥٥٨٦	٠.٤٣٧٧	١٤٥	١
١٩	٠.٠٢٠٨٤	٠.٣٣٢	٢٠٠	١٦٠	٢.٠٧٣٦٥٧	٠.٦٦٣٣	١٤٥	١
٢٠	٠.٠٢٠٨٤	٠.٢٢٨	٢٠٠	١١٠	٢.٠٧٣٨٢٦	٠.٦٦٣٣	١٤٥	١
٢١	٠.٠٠٢٢	٠.٤٩٨	١٠٠	٥٣٠	٠.٩٣٩٦٩	٠.٢٧٩٥	١٤٥	١
٢٢	-٠.٠٠٢٢	٠.٥٣٦	١٠٠	٥٧٠	٠.٩٤٠٠١٥	-٠.٢٧٩٦	١٤٥	١
٢٣	٠.٠٠٤٧٦	٠.٨٦٧	١٥٠	٢٢٠	٣.٩٤٣٠٨٨	٠.٢٦٩٥	١٤٥	١
٢٤	٠.٠٠٤٧٦	٤.٦٩٣	١٥٠	١١٩٠	٣.٩٤٣٤٣٧	٠.٢٦٩٦	١٤٥	١

٢٥	٠.٠٣٥٥٦	٤.١٨٥	٢٥٠	٧٥٠	٥.٥٨٠٠٥٣	٠.٧٢٤٥	١٤٥	١
PIPE	**Q [M٣/S]**	**HF [M]**	**D [MM]**	**LENGTH[M]**	**SLOPE *١٠**-٣**	**VELOCITY[M/S]**	**C.H.W.**	**MMM**
٢٦	٠.٠٤٤٨٦	٠.٠٨٧	٢٥٠	٣٠	٢.٨٩٤٠٨٣	٠.٩١٣٩	١٤٥	١
٢٧	٠.٠٥١٥٧	١.٠٨٦	٣٠٠	٢٩٠	٣.٧٤٥٤٩٤	٠.٧٢٩٥	١٤٥	١
٢٨	-٠.٠٥٨٢٧	٢.٤٤٢	٣٠٠	٥٢٠	٤.٦٩٦٢٨١	-٠.٨٢٤٣	١٤٥	١
٢٩	٠.٠٣٩٧٣	٠.٤١٦	٢٥٠	١٨٠	٢.٣١١٣١٥	٠.٨٠٩٤	١٤٥	١
٣٠	-٠.٠٠٥٥٥	٣.١٩٢	١٠٠	٦١٠	٥.٢٣٣٢٥٨	-٠.٧٠٦٦	١٤٥	١
٣١	-٠.٠٠٥٥٥	٣.٢٩٧	١٠٠	٦٣٠	٥.٢٣٣١٠٨	-٠.٧٠٦٦	١٤٥	١
٣٢	٠.٠٠٢٠٧	١.٧٩٨	١٠٠	٢١٤٠	٠.٨٤٠٣٨٣	٠.٢٦٣٢	١٤٥	١
٣٣	٠.٠٣٠٨٨	٠.٦٤٤	٢٠٠	١٥٠	٤.٢٩٥٤٨٩	٠.٩٨٢٩	١٤٥	١
٣٤	-٠.٠٣٠٨٨	٣.٣٩٤	٢٠٠	٧٩٠	٤.٢٩٥٧٦٩	-٠.٩٨٢٩	١٤٥	٠
٣٥	-٠.١٣٤٦٣	١.٠٧٥	٣٥٠	٢٥٠	٤.٣٠١٧٨٨	-١.٣٩٩٣	١٤٥	١
٣٦	-٠.٠٨٤٧٥	٢.٤٣٦	٣٠٠	٦٣٠	٣.٨٦٧٣٠٧	-١.١٩٨٩	١٤٥	٠
٣٧	٠.٠١٩٢٤	٢.٨٨٥	٢٠٠	٨١٠	٣.٥٦١١٣٨	٠.٦١٢٦	١٠٠	٠
٣٨	-٠.٠٠١٤٦	٠.٢٣٤	١٠٠	٥٣٠	٠.٤٤٠٦٠١	-٠.١٨٥٧	١٤٥	١
٣٩	-٠.٠١٥٥٤	٣.٥٢١	١٥٠	٧٢٠	٤.٨٩٠٩٢٢	-٠.٨٧٩٥	١٤٥	١
٤٠	-٠.٠٣٢٨٣	٠.٣٨٥	٢٠٠	٨٠	٤.٨١١٥٠٢	-١.٠٤٥	١٤٥	١
٤١	٠.٠٥٢١٩	٠.٥٧٤	٢٥٠	١٥٠	٣.٨٢٩٤٨٦	١.٠٦٣٢	١٤٥	١
٤٢	٠.٠٢٦١٧	٢.٧٨٣	٢٠٠	٨٨٠	٣.١٦٢٠٦٧	٠.٨٣٣	١٤٥	١
٤٣	٠.٠٠٢١٢	٠.٩٨٤	١٠٠	١١٢٠	٠.٨٧٨٦٦٨	٠.٢٦٩٦	١٤٥	٠
٤٤	٠.٠٣٣٨٩	٢.٣٤٧	٢٠٠	٤٦٠	٥.١٠٢٩٥٤	١.٠٧٨٧	١٤٥	١
٤٥	٠.٠٦١١٩	٢.٠٥٧	٢٥٠	٤٠٠	٥.١٤١٥١٦	١.٢٤٦٥	١٤٥	١
٤٦	٠.٠٨٩٦٩	١.١٨٩	٤٠٠	٤٠٠	٢.٩٧٣٢٧	٠.٧١٣٧	٨٣	٠
٤٧	-٠.١٨٥٦٣	٢.٨٨٤	٤٥٠	٥٢٠	٥.٥٤٦٠٧١	-١.١٦٧١	٩٠	٠
٤٨	-٠.٠٨٥٠٢	٢.٣٩٨	٤٠٠	٧٢٠	٣.٣٣٠٨٥١	-٠.٦٧٦٦	٧٤	٠
٤٩	-٠.٠٠٥٥٨	٧.٤٠٦	١٠٠	١٤٠٠	٥.٢٩٠١٤٧	-٠.٧١٠٨	١٤٥	١
٥٠	-٠.٠٠٢٥٨	٠.٥٩٦	١٠٠	٤٧٠	١.٢٦٨٩٦٣	-٠.٣٢٨٨	١٤٥	١
٥١	٠.٠٠١٤٨	٠.٢١٣	١٠٠	٤٧٠	٠.٤٥٣٧٣٤	٠.١٨٨٧	١٤٥	١
٥٢	٠.٠٨٧٢٨	٣.٠٦٣	٣٠٠	٧٥٠	٤.٠٨٤٤٩٨	١.٢٣٤٨	١٤٥	١
٥٣	٠.١٠١٧٤	٠.٤٨٨	٣٠٠	٩٠	٥.٤٢٥٠٢٩	١.٤٣٩٣	١٤٥	١
٥٤	-٠.٠٩٥٩٣	٤.١٣٦	٣٠٠	٨٥٠	٤.٨٦٥٤٢٨	-١.٣٥٧٢	١٤٥	١
٥٥	-٠.٠٠٣١٣	٠.٨١٥	١٥٠	٤٥٠	١.٨١٠٧١٨	-٠.١٧٧١	١٤٥	١
٥٦	٠.٠١٢٢٤	٥.٥٢٨	١٥٠	١٧٦٠	٣.١٤١٠٥٧	٠.٦٩٢٥	١٤٥	٠
٥٧	٠.٠٠٥٨٤	٥.٩٢٧	١٠٠	١٠٣٠	٥.٧٥٤٤٧٧	٠.٧٤٣٨	١٤٥	١
٥٨	-٠.٠٣٥٤٧	٢.٧٧٧	٢٥٠	٥٠٠	٥.٥٥٤٢٤١	-٠.٧٢٢٧	١٤٥	١
٥٩	-٠.٠٣٥٤٧	١.٣٨٨	٢٥٠	٢٥٠	٥.٥٥٣٣٦٨	-٠.٧٢٢٦	١٤٥	١
٦٠	٠.٠١٣٨٥	١.٣٤٣	٢٠٠	٣٤٠	٣.٩٤٩٠٢٥	٠.٤٤٠٨	١٤٥	١
٦١	٠.٠٣٩٤٥	٤.٧٣٣	٢٥٠	٧٠٠	٦.٧٦١٢٣	٠.٨٠٣٦	١٤٥	١
٦٢	-٠.٠٥١٨٢	٠.٣٤	٣٠٠	٩٠	٣.٧٧٩٧٢٩	-٠.٧٣٣١	١٤٥	١
٦٣	٠.١١٣٢٤	٣.٠٧٤	٤٥٠	١٥٣٠	٢.٠٠٩١٦٤	٠.٧١٢	٩٥	٠
٦٤	-٠.٢٠٥٤٨	٣.٤٨٨	٤٥٠	٧١٠	٤.٩١٢١٨٦	-١.٢٩٢	١٤٥	١
٦٥	-٠.١٢٤٧١	٠.٥٢٣	٣٥٠	١٤٠	٣.٧٣٣٠٠٨	-١.٢٩٦٢	١٤٥	١
٦٦	-٠.٠٢٨٤٩	٢.٥١٦	٢٠٠	٦٨٠	٣.٦٩٩٩٨١	-٠.٩٠٦٨	١٤٥	٠
٦٧	-٠.٠١٤٠٩	٣.٠٩٨	١٥٠	٧٦٠	٤.٠٧٦٤٨١	-٠.٧٩٧١	١٤٥	١
٦٨	٠.٠٠٠٣١	٠.٠٤١	١٠٠	٨١٠	٠.٠٥١٠٩٨	٠.٠٤	١٠٠	٠
٦٩	-٠.٠٢٢٩	٠.١٧٣	٢٠٠	٧٠	٢.٤٦٩٧٩٨	-٠.٧٢٩	١٤٥	١
٧٠	-٠.٠٠٥٢٨	٠.٩٥٣	١٥٠	١٤٤٠	٠.٦٦١٧١٨	-٠.٢٩٨٦	١٤٥	٠
٧١	-٠.٠١٢٣٣	٠.٣١٩	١٠٠	١٠٠	٣.١٨٧١٤١	-١.٥٧٠٣	١٤٥	١

PIPE	Q [M٣/S]	HF [M]	D [MM]	LENGTH[M]	SLOPE *١٠**-٣	VELOCITY[M/S]	C.H.W.	MMM
٧٢	-٠.٠٣٧٦٢	٥.٥٧٢	٢٥٠	٩٠٠	٦.١٩١٥٥٤	-٠.٧٦٦٣	١٤٥	١
٧٣	٠.٠٩٢١٩	٠.٣٦٢	٣٠٠	٨٠	٤.٥١٩٧٤٩	١.٣٠٤٢	١٤٥	١
٧٤	٠.٠٦٤٢٣	٠.٥٦٢	٣٠٠	١٠٠	٥.٦٢٤٨٠٩	٠.٩٠٨٧	١٤٥	١
٧٥	٠.٠٣٦٦٣	٤.١٨٨	٢٥٠	١٩٧٤	٢.١٢١٤٥٩	٠.٧٤٦٢	١٤٠	٠
٧٦	٠.٠٠٤٥٩	٢.٩٤	١٠٠	٨٠٠	٣.٦٧٥٢٦٧	٠.٥٨٣٩	١٤٥	١
٧٧	٠.٠٠٤٤٤	٣.١٥٣	١٠٠	٩١٠	٣.٤٦٤٦٢٨	٠.٥٦٥٥	١٤٥	١
٧٨	-٠.٠٤٩١٦	٣.٦٦٩	٢٥٠	١٠٧٠	٣.٤٢٨٦	-١.٠٠١٥	١٤٥	١
٧٩	-٠.١١٥٩٧	١.٨٦٧	٣٥٠	٢٧٠	٦.٩١٣٢٠٧	-١.٢٠٥٤	١٤٥	١
٨٠	٠.٠٣٣٨٩	٢.٢٩٧	٢٥٠	٤٥٠	٥.١٠٤٦١٢	٠.٦٩٠٥	١٤٥	١
٨١	-٠.٠٠١٢١	٠.١٤٨	١٠٠	٤٧٠	٠.٣١٣٩٩٤	-٠.١٥٤٧	١٤٥	١
٨٢	٠.٠٥٢٧٢	٠.٥٠٧	٢٥٠	١٣٠	٣.٩٠٢٧٢٩	١.٠٧٤١	١٤٥	١
٨٣	٠.٠٦١٧١	١.٩٨٤	٢٥٠	٣٨٠	٥.٢٢٢٢٨٦	١.٢٥٧١	١٤٥	١
٨٤	-٠.٠٧٤٩٥	٠.٤٤٩	٣٠٠	٦٠	٧.٤٨٥٧٧١	-١.٠٦٠٣	١٤٥	١
٨٥	-٠.٠٠١٥٧	٠.٢٢٦	١٠٠	٤٥٠	٠.٥٠٢٥٩٥	-٠.١٩٩٤	١٤٥	١
٨٦	-٠.٠٠٥٨٤	٢.٢١١	١٥٠	٣٨٥	٥.٧٤٢١٣	-٠.٣٣٠٢	١٤٥	١
٨٧	-٠.٠٠٥٧٣	٢.٠٥٦	١٠٠	٣٧٠	٥.٥٥٧٤٤٢	-٠.٧٢٩٩	١٤٥	١
٨٨	٠.٠٠٤٩٧	١.٣٦٤	١٠٠	٣٢٠	٤.٢٦١٧٣٢	٠.٦٣٢٥	١٤٥	١
٨٩	٠.٠١٥٦٧	٠.٦٤٥	١٥٠	١٣٠	٤.٩٦٤٣٨٨	٠.٨٨٦٦	١٤٥	١
٩٠	٠.٠٤٦٧٧	٠.٨٦١	٢٥٠	٢٠٠	٤.٣٠٤٥١٤	٠.٩٥٢٨	١٢٢	٠
٩١	٠.٠٤٦٧٨	٠.٢٨١	٢٥٠	٩٠	٣.١٢٧٤٣٧	٠.٩٥٣	١٤٥	١
٩٢	-٠.٠٢٨٦١	٠.٨٢	٢٠٠	٢٢٠	٣.٧٢٨٦٥٨	-٠.٩١٠٦	١٤٥	١
٩٣	-٠.٠٢٨٥٩	٠.٧٤٥	٢٠٠	٢٠٠	٣.٧٢٥٧	-٠.٩١٠٢	١٤٥	١
٩٤	-٠.٠١٤٦٨	٣.٢١٢	١٥٠	٧٣٠	٤.٤٠٠٢٢٢	-٠.٨٣٠٧	١٤٥	١
٩٥	-٠.٠٠٤٤٨	٤.٢٢١	١٠٠	١٢٠٠	٣.٥١٧١٣	-٠.٥٧٠٢	١٤٥	١
٩٦	٠.٠١٥٩٢	٥.٢٦٨	٢٠٠	١٠٣٠	٥.١١٤٧٦٦	٠.٥٠٦٨	١٤٥	١
٩٧	-٠.٠٣٠٢٣	٤.٥٨٥	٢٠٠	١١١٠	٤.١٣٠٩٨٩	-٠.٩٦٢٤	١٤٥	١
٩٨	-٠.٠٢١٩٣	١.٨٠١	٢٠٠	٧٩٠	٢.٢٧٩٨٤٤	-٠.٦٩٨١	١٤٥	١
٩٩	-٠.٠٠١٥٣	٠.٧١٣	١٠٠	١٤٩٠	٠.٤٧٨٦٤٥	-٠.١٩٤٢	١٤٥	١
١٠٠	-٠.٠٢٤٧٤	٠.٢٨٥	٢٠٠	١٠٠	٢.٨٤٩٢٩٣	-٠.٧٨٧٤	١٤٥	١
١٠١	٠.٠٠١٣٥	٠.٢٩٣	١٠٠	٧٧٠	٠.٣٨١٠٧٣	٠.١٧١٧	١٤٥	١
١٠٢	٠.٠٠٢٤	٠.٨١	١٠٠	٧٣٠	١.١٠٩٧١٩	٠.٣٠٥٨	١٤٥	١
١٠٣	٠.٠٠٣٧٩	٠.٥٤٢	١٠٠	٢١٠	٢.٥٨١٤٧٨	٠.٤٨٢٥	١٤٥	١
١٠٤	٠.٠٤٤٠٩	٠.٩٨١	٢٥٠	٣٥٠	٢.٨٠٢٧٥٦	٠.٨٩٨٣	١٤٥	١
١٠٥	-٠.٠٧٠٧٦	٠.٤٠٤	٣٠٠	٦٠	٦.٧٢٩٠٦٢	-١.٠٠١	١٤٥	١
١٠٦	-٠.٠٠٩٩١	١.٣١٧	١٥٠	٦٢٠	٢.١٢٣٧٢٨	-٠.٥٦٠٥	١٤٥	١
١٠٧	-٠.٠٠٢٨٧	١.٦٠٣	١٠٠	١٠٤٠	١.٥٤١٤٩٥	-٠.٣٦٥٢	١٤٥	١
١٠٨	-٠.٠١٦٤٧	٢.٣٤٣	٢٠٠	٤٣٠	٥.٤٤٧٩٤٢	-٠.٥٢٤٤	١٤٥	١
١٠٩	-٠.٠٢٧٩٤	٢.٨٥٥	٢٠٠	٨٠٠	٣.٥٦٨٧٦٩	-٠.٨٨٩٣	١٤٥	١
١١٠	٠.٠٠١٩٥	٠.٤٥٩	١٠٠	٥٧٠	٠.٨٠٤٩٤٨	٠.٢٤٨٣	١٤٠	٠
١١١	-٠.٠٠٣٧	١.٩٥٤	١٠٠	٧٩٠	٢.٤٧٣٩٦١	-٠.٤٧١٥	١٤٥	٠
١١٢	٠.٠٠١٢٥	٠.٣٥	١٠٠	٩٩٠	٠.٣٥٣٦٤٢	٠.١٥٩٢	١٤٠	٠
١١٣	٠.١١٤٨٧	١.١٥٥	٣٥٠	١٧٠	٦.٧٩٢٨٠٩	١.١٩٤	١٤٥	١
١١٤	-٠.١٢٧١	١.٤٥٧	٤٠٠	٣٣٠	٤.٤١٥٨٧٦	-١.٠١١٤	٩٥	٠
١١٥	-٠.١٨٠٣٨	٠.١٥٤	٤٠٠	٤٠	٣.٨٥٨٩٤٨	-١.٤٣٥٤	١٤٥	١
١١٦	-٠.٢٠١٥١	٠.٥٢١	٤٠٠	١١٠	٤.٧٣٧٨٥٤	-١.٦٠٣٦	١٤٥	١
١١٧	-٠.٠٠١٧٤	٠.٨٦٨	١٠٠	١٤٢٠	٠.٦١١٥٧٤	-٠.٢٢١٧	١٤٥	١
١١٨	-٠.٢٠٣٣	٠.٦٢٦	٤٠٠	١٣٠	٤.٨١٥٧٩١	-١.٦١٧٨	١٤٥	١

١١٩	-٠.٠٣١٦٣	٤.٢٦٧	٢٠٠	٩٥٠	٤.٤٩٢٠٤٥	-١.٠٠٦٩	١٤٥	٠
١٢٠	٠.٠٠١٤٧	٠.٤٤٥	١٠٠	١٠٠٠	٠.٤٤٥٤٧٨	٠.١٨٦٨	١٤٥	١
١٢١	-٠.٠٥٩٦٧	٣.٦٨١	٣٠٠	٧٥٠	٤.٩٠٧٧٥٥	-٠.٨٤٤٢	١٤٥	١
PIPE	**Q [M٣/S]**	**HF [M]**	**D [MM]**	**LENGTH[M]**	**SLOPE *١٠**-٣**	**VELOCITY[M/S]**	**C.H.W.**	**MMM**
١٢٢	-٠.٠٧٠٩٧	٣.٠٤٥	٣٠٠	٤٥٠	٦.٧٦٦٤٠٨	-١.٠٠٤	١٤٥	١
١٢٣	-٠.٠٨٢٢٧	٢.٣٠٦	٣٠٠	٦٣٠	٣.٦٦٠٧٤١	-١.١٦٣٩	١٤٥	١
١٢٤	٠.٠١٤٨٤	٢.٥٦	١٥٠	٥٧٠	٤.٤٩١٦٦٩	٠.٨٤	١٤٥	١
١٢٥	٠.٠٠٦١٦	٠.٢٢	١٠٠	٢٥٠	٠.٨٨٠٩٥٨	٠.٧٨٤٢	١٤٥	١
١٢٦	٠.٠٢٤٧٦	٢.٣٤	٢٠٠	٨٢٠	٢.٨٥٣٦٧٣	٠.٧٨٨١	١٤٥	٠
١٢٧	٠.٠٠٢٤	٢.٠٧٧	١٠٠	٩٤٠	٢.٢٠٩٧١٦	٠.٣٠٥٩	١٠٠	٠
١٢٨	-٠.٠١٦٢	٤.٢٧٦	١٥٠	٨١٠	٥.٢٧٩٣٨	-٠.٩١٦٦	١٤٥	٠
١٢٩	-٠.٠٩٠٩	٤.١٨٣	٣٠٠	٩٥٠	٤.٤٠٣٢١١	-١.٢٨٥٩	١٤٥	١
١٣٠	-٠.٠٦١٦٩	٥.٢٧٢	٢٥٠	١٠١٠	٥.٢١٩٥٥٦	-١.٢٥٦٧	١٤٥	١
١٣١	٠.٠٥٠١٨	٢.٢٤٣	٢٥٠	٦٣٠	٣.٥٦١٠٦٥	١.٠٢٢٣	١٤٥	١
١٣٢	٠.٠٢٠٨٢	٠.٣٧٣	٢٠٠	١٨٠	٢.٠٦٩٥٩	٠.٦٦٢٦	١٤٥	١
١٣٣	٠.٠١٥١	٠.٣٢٤	١٥٠	٧٠	٤.٦٣٤٣٦٧	٠.٨٥٤٣	١٤٥	١
١٣٤	٠.٠٠٤١٩	٠.٦٨٥	١٠٠	٢٢٠	٣.١١٥١١٦	٠.٥٣٤	١٤٥	١
١٣٥	-٠.٠١٧٨٤	٢.٧١٦	١٥٠	٤٣٠	٦.٣١٦٠٥٦	-١.٠٠٩٨	١٤٥	١
١٣٦	٠.٠٠٠٦٤	٠.٠٧	١٠٠	٧٣٠	٠.٠٩٥٢٩٢	٠.٠٨١٢	١٤٥	١
١٣٧	-٠.٠١٦٥٦	٢.٨٦٢	٢٠٠	٥٢٠	٥.٥٠٢٩٩٧	-٠.٥٢٧٣	١٤٥	١
١٣٨	-٠.٠٤١٤٧	٢.٥٩٦	٢٥٠	٣٥٠	٧.٤١٦٥٧٣	-٠.٨٤٤٨	١٤٥	١
١٣٩	-٠.٠٥٢٩٩	١.٠٢٤	٢٥٠	٢٦٠	٣.٩٣٨٦٦٨	-١.٠٧٩٤	١٤٥	١
١٤٠	-٠.٠٥٢٩٩	٢.٥٢١	٢٥٠	٦٤٠	٣.٩٣٩٥٨١	-١.٠٧٩٦	١٤٥	١
١٤١	-٠.٠٢٠٧٥	٣.٠٨٦	٢٠٠	١٥٠٠	٢.٠٥٧٢٠٦	-٠.٦٦٠٤	١٤٥	٠
١٤٢	٠.٠٠٦٠٥	٣.٨٠٢	١٥٠	٦٢٠	٦.١٣٢٨٠٩	٠.٣٤٢١	١٤٥	١
١٤٣	-٠.٠١١٨٥	٥.٢٧٨	١٥٠	١٦٧٠	٣.١٦٠٦٨٩	-٠.٦٧٠٨	١٤٠	٠

NODE	HEAD [M]	Q-OUT [M٣/S]
١	٣٦.٣١	٠
٢	٢٧.٦٦٨	٠.٠٦٨
٣	٢٤.١٥٤	٠.٠٣٧
٤	٢٥.٣٢١	٠
٥	٢٥.٣٩٢	٠
٦	٢٥.٥٣٥	٠.٠١
٧	٢٧.٣١١	٠.٠١
٨	٣٦.٠٣	٠
٩	٢٥.٣٠٨	٠.٠١٥
١٠	٢٥.٦١	٠
١١	٢٧.١٧٥	٠.٠١٤
١٢	٢٩.١	٠
١٣	٢٩.٣٣٩	٠.٠٠٢
١٤	٢٧.٣٩٩	٠.٠٠٢
١٥	٣٠.٥٤١	٠.٠٠٢
١٦	٢٧.٥٠٧	٠.٠١٧
١٧	٢٧.٨٣٨	٠
١٨	٢٨.٠٦٧	٠.٠٠٧

١٩	٢٨.٥٦٥	٠
٢٠	٢٢.٦٧	٠.٠٣٣
٢١	٢٣.٥٣٧	٠
٢٢	٢٨.٢٣	٠.٠٣١
NODE	**HEAD [M]**	**Q-OUT [M۳/S]**
٢٣	٣٢.٤١٥	٠.٠٠٩
٢٤	٣٢.٥٠٢	٠.٠٠٧
٢٥	٣٣.٥٨٨	٠.٠٠٧
٢٦	٣٠.٩٥٧	٠.٠٠٧
٢٧	٢٧.٧٦٥	٠
٢٨	٢٤.٤٦٨	٠.٠٠٦
٢٩	٢٥.٤٥٣	٠.٠٠٦
٣٠	٢٧.٨	٠.٠٢٧
٣١	٢٩.٨٥٧	٠.٠٢٩
٣٢	٣١.٠٤٦	٠
٣٣	٣٣.٩٣	٠
٣٤	٣١.٥٣٢	٠
٣٥	٣١.١٤٧	٠.٠١٢
٣٦	٢٣.٧٤١	٠.٠٠٣
٣٧	٢٣.١٤٥	٠.٠١٦
٣٨	٢٣.٣٥٩	٠.٠٨٦
٣٩	٢٦.٤٢٢	٠.٠٢٨
٤٠	٢٦.٩١١	٠
٤١	٣٢.٨٣٨	٠.٠٢
٤٢	٣٠.٠٦٢	٠
٤٣	٢٨.٦٧٤	٠.٠٢
٤٤	٢٧.٨٥٩	٠.٠٢٥
٤٥	٢٧.٦٢٦	٠.٠١٧
٤٦	٢٧.٧٦٥	٠.٠٢٦
٤٧	٣٢.٤٩٩	٠
٤٨	٣٠.٧٤٤	٠.٠٦٥
٤٩	٣٣.١٨	٠
٥٠	٣٢.١٠٥	٠.٠٢٩
٥١	٢٨.٧١١	٠
٥٢	٢٦.٠١١	٠.٠١١
٥٣	٢٧.٣٧٥	٠.٠١١
٥٤	٢٨.٠٢	٠.٠٣١
٥٥	٢٨.٨٨٢	٠
٥٦	٢٩.١٦٣	٠.٠٠٥
٥٧	٢٩.٠١٦	٠.٠٠٧
٥٨	٣١.٣١٤	٠.٠٣٣
٥٩	٢٩.٦٧١	٠.٠٠٧
٦٠	٣١.٦٥٦	٠.٠٠٧
٦١	٢٩.٤٤٥	٠.٠٠٧
٦٢	٢٧.٦٤٦	٠.٠٢
٦٣	٣٠.٨	٠.٠٢٨
٦٤	٣٤.٩٨٨	٠.٠٢٨
٦٥	٣٥.٥٥١	٠

٦٦	٣٥.٩١٣	٠
٦٧	٣٩.٤	٠.٠١٨
٦٨	٣٨.٨٧٧	٠.٠٥٩
٦٩	٣٦.٣٦١	٠.٠١٤
NODE	**HEAD [M]**	**Q-OUT [M٣/S]**
٧٠	٣٣.٢٦٤	٠.٠١٤
٧١	٣٣.٣٠٥	٠.٠١٤
٧٢	٣٣.١٣٣	٠.٠١٨
٧٣	٣٢.١٨	٠.٠١٨
٧٤	٣٧.٠٩٤	٠.٠١١
٧٥	٣٤.٠٤٩	٠.٠١١
٧٦	٢٩.٩٢٣	٠.٠٣٣
٧٧	٣٤.١٩	٠
٧٨	٣٣.٥٦٤	٠
٧٩	٣٢.٦٩٦	٠.٠١٣
٨٠	٣٠.٣٥٤	٠.٠١
٨١	٢٦.٣٣١	٠.٠١
٨٢	٢٧.٩٤١	٠.٠٠٣
٨٣	٢٨.٤٠١	٠.٠٠٣
٨٤	٢٨.٧٥٢	٠.٠٠٣
٨٥	٢٩.٢٩٦	٠.٠٤
٨٦	٣٠.٢٧٨	٠
٨٧	٢٩.٨٧٥	٠.٠١٩
٨٨	٢٨.٠٧٥	٠.٠٢
٨٩	٢٧.٣٦٣	٠.٠٢٦
٩٠	٢٨.١٩٧	٠
٩١	٢٧.٤٥٢	٠.٠١٤
٩٢	٢٤.٢٤١	٠.٠١
٩٣	٢٠.٠٢١	٠.٠٢
٩٤	٢٥.٢٩	٠.٠١٩
٩٥	٢٥.٩٧٦	٠.٠١١
٩٦	٢٦.٣٠٢	٠.٠٠٦
٩٧	٢٦.٦٧٥	٠.٠١٢
٩٨	٢٨.٩١٩	٠.٠١٢
٩٩	٣٣.٠٤٣	٠
١٠٠	٣٢.٨٩	٠
١٠١	٣١.٤٣٣	٠
١٠٢	٣٠.٣٦٩	٠
١٠٣	٢٩.٣٤٥	٠.٠١٢
١٠٤	٢٦.٧٥	٠.٠٢٥
١٠٥	٢٣.٨٨٩	٠.٠١٧
١٠٦	٢٣.٩٦	٠.٠١٧
١٠٧	٢٦.١٥٥	٠.٠١٨
١٠٨	٢٩.٩٥٨	٠.٠١٥
١٠٩	٣٠.٠٠٧	٠.٠٧٥
١١٠	٢٥.٧٣١	٠.٠١٩
١١١	٢٧.٨٠٨	٠.٠١٩
١١٢	٢٨.٠٢٨	٠.٠١٩

١١٣	٣٠.٣٦٨	٠.٠١٩

NO.OF CYCLES = ٢٥٠٠١

STANDARD DIVEATION = ٣.٦١٣٦

COEFF.OF VARIATION = ٠.١٢٣٤

FINAL COST OF THIS NETWORK = ٣٨٣٦٤٧٦٠٠.٠٠٠ I.D

Table (B-٣): Program results for optimum rehabilitation of Imam quarter pipe network.

PIPE	Q [M٣/S]	HF [M]	D [MM]	LENGTH[M]	SLOPE *١٠**-٣	VELOCITY[M/S]	C.H.W.	MMM
١	-٠.٠٣١٩١	٠.٢٢٨	٢٠٠	٥٠	٤.٥٦٣٩٤١	-١.٠١٥٦	١٤٥	١
٢	-٠.٠٠٤٣٦	٣.٧٧٨	١٠٠	١١٣٠	٣.٣٤٣٤٥٤	-٠.٥٥٤٨	١٤٥	٠
٣	-٠.٠١٣٧	٠.٢٧١	١٥٠	٧٠	٣.٨٦٩٧٦٥	-٠.٧٧٥	١٤٥	٠
٤	٠.٠٠٣٧٢	١.٧٥١	١٠٠	٧٠٠	٢.٥٠٠٨١٥	٠.٤٧٤٣	١٤٥	١
٥	-٠.٠٣٢٣٣	٢.٥٢٥	٢٠٠	٥٤٠	٤.٦٧٥٩٦١	-١.٠٢٩	١٤٥	١
٦	-٠.٠١٨٤٤	١.٠٢٦	٢٠٠	٦٢٠	١.٦٥٤٣٧٩	-٠.٥٨٧١	١٤٥	١
٧	-٠.٠١٨٤٤	٢.٧٥٣	١٥٠	٤١٠	٦.٧١٣٨١٦	-١.٠٤٣٦	١٤٥	١

NODE	HEAD [M]	Q-OUT [M٣/S]
١	٢١.٧٤	٠.٠١١
٢	٢١.٥١٢	٠.٠٠٩
٣	٢٠.٤٨٦	٠
٤	١٧.٧٣٥	٠.٠٠٩
٥	١٧.٤٦٤	٠.٠١٧
٦	١٩.٢١٥	٠.٠٢٩

NO.OF CYCLES = ٧٥٩

STANDARD DIVEATION = ١.٦٩١١

COEFF.OF VARIATION= .٠٨٥٩

FINAL COST OF THIS NETWORK = ١٤١٧٥٠٠٠.٠٠٠ I.D

Table (B-٤): Standard limits [٣١].

Parameters	Max.Limit
Turbidity (NTU)	١٠
PH	٦.٥-٨.٥
Electrical Conductivity (Umhos/cm)	٢٠٠٠
Alkalinity (mg/l)	٢٠٠
Calcium (mg/l)	٢٠٠
Magnesium (mg/l)	١٥٠
Chloride (mg/l)	٦٠٠
Hardness (mg/l)	٥٠٠
Total Dissolved Solid (mg/l)	١٥٠٠

Table (B-٥): Critical statistics for the Kolmogorov-Smirnov test [٣٠].

Sample size	α = ٠.١	α= ٠.٠٥	α= ٠.٠١
٥	٠.٥١	٠.٥٦	٠.٦٧
١٠	٠.٣٧	٠.٤١	٠.٤٩
١٥	٠.٣٠	٠.٣٤	٠.٤٠
٢٠	٠.٢٦	٠.٢٩	٠.٣٥
٢٥	٠.٢٤	٠.٢٦	٠.٣٢
٣٠	٠.٢٢	٠.٢٤	٠.٢٩
٤٠	٠.١٩	٠.٢١	٠.٢٥
Large n	١.٢٢/$\sqrt{n}$	١.٣٦/$\sqrt{n}$	١.٦٣/$\sqrt{n}$

Table (B-٦): Critical statistics for the Chi-square test [١٧].

α	٠.٩	٠.٩	٠.٩	٠.٩	٠.٥	٠.٢	٠.١	٠.٠	٠.٠	٠.٠	٠.٠

	٩٥	٩	٧٥	٥	٠	٠	٠	٥	٢٥	١	٠٥
v											
١	٠.٠ ٠٠	٠.٠ ٠٠٢	٠.٠ ٠١	٠.٠ ٠٣٩	٠.٤ ٥	١.٦ ٤	٢.٧ ١	٣.٤ ٨	٥.٠ ٢	٦.٦ ٣	٧.٨ ٨
٢	٠.٠ ١٠	٠.٠ ٢٠	٠.٠ ٥١	٠.١ ٥٣	١.٣ ٩	٣.٢ ٢	٤.٦ ١	٥.٩ ٩	٧.٣ ٨	٩.٢ ١	١٠. ٦٠
٣	٠.٠ ٧٢	٠.١ ١٥	٠.٢ ١٦	٠.٣ ٥٢	٢.٣ ٧	٤.٦ ٤	٦.٢ ٥	٧.٨ ١	٩.٣ ٥	١١. ٣٤	١٢. ٨٤
٤	٠.٢ ٠٧	٠.٣ ٠	٠.٤ ٨٤	٠.٧ ١	٣.٣ ٦	٥.٩ ٩	٧.٧ ٨	٩.٤ ٩	١١. ١٤	١٣. ٢٨	١٤. ٨٦
٥	٠.٤ ١٢	٠.٥ ٥	٠.٨ ٣١	١.١ ٥	٤.٣ ٥	٧.٢ ٩	٩.٢ ٤	١١. ٧	١٢. ٨٤	١٥. ٩	١٦. ٧٥
٦	٠.٦ ٧٦	٠.٨ ٧	١.٢ ٤	١.٦ ٤	٥.٣ ٥	٨.٥ ٦	١٠. ٨٤	١٢. ٥٩	١٤. ٤٥	١٦. ٨١	١٨. ٥٥
٧	٠.٩ ٨٩	١.٢ ٤	١.٦ ٩	٢.١ ٧	٦.٣ ٥	٩.٨ ٠	١٢. ٠٢	١٤. ٠٧	١٦. ٠١	١٨. ٤٨	٢٠. ٢٨
٨	١.٣ ٤	١.٦ ٥	٢.١ ٨	٢.٧ ٣	٧.٣ ٤	١١. ٠٣	١٣. ٣٦	١٥. ٥١	١٧. ٥٣	٢٠. ٠٩	٢١. ٩٥
٩	١.٧ ٣	٢.٠ ٩	٢.٧ ٠	٣.٣ ٣	٨.٣ ٤	١٢. ٢٤	١٤. ٦٨	١٦. ٩٢	١٩. ٠٢	٢١. ٦٧	٢٣. ٥٩
١٠	٢.١ ٦	٢.٥ ٦	٣.٢ ٥	٣.٩ ٤	٩.٣ ٤	١٣. ٤٤	١٥. ٩٩	١٨. ٣١	٢٠. ٤٨	٢٣. ٢١	٢٥. ١٩
١١	٢.٦ ٠	٣.٠ ٥	٣.٨ ٢	٤.٥ ٧	١٠. ٣٤	١٤. ٦٣	١٧. ٢٨	١٩. ٦٨	٢١. ٩٢	٢٤. ٧٢	٢٦. ٧٦
١٢	٣.٠ ٧	٣.٥ ٧	٤.٤ ٠	٥.٢ ٣	١١. ٣٤	١٥. ٨١	١٨. ٥٥	٢١. ٠٣	٢٣. ٣٤	٢٦. ٢٢	٢٨. ٣٠
١٣	٣.٥ ٧	٤.١ ١	٥.٠ ١	٥.٨ ٩	١٢. ٣٤	١٦. ٩٨	١٩. ٨١	٢٢. ٣٦	٢٤. ٧٤	٢٧. ٦٩	٢٩. ٨٢
١٤	٤.٠ ٧	٤.٦ ٨	٥.٦ ٣	٦.٥ ٧	١٣. ٣٤	١٨. ١٥	٢١. ٠٨	٢٣. ٦٨	٢٦. ١٢	٢٩. ١٤	٣١. ٣٢
١٥	٤.٦ ٠	٥.٢ ٣	٦.٢ ٦	٧.٢ ٦	١٤. ٣٤	١٩. ٣١	٢٢. ٣١	٢٥. ٠٠	٢٧. ٤٩	٣٠. ٥٨	٣٢. ٨٠
١٦	٥.١ ٤	٥.٨ ١	٦.٩ ١	٧.٩ ٦	١٥. ٣٤	٢٠. ٤٧	٢٣. ٥٤	٢٦. ٣٠	٢٨. ٨٥	٣٢. ٠٠	٣٤. ٢٧
١٧	٥.٧ ٠	٦.٤ ١	٧.٥ ٦	٨.٦ ٧	١٦. ٣٤	٢١. ٦١	٢٤. ٧٧	٢٧. ٥٩	٣٠. ١٩	٣٣. ٤١	٣٥. ٧٢
١٨	٦.٢ ٦	٧.٠ ٢	٨.٢ ٣	٩.٣ ٩	١٧.٣ ٤	٢٢. ٧٦	٢٥. ٩٩	٢٨. ٨٧	٣١. ٥٣	٣٤. ٨١	٣٧. ١٦
١٩	٦.٨ ٤	٧.٨ ٣	٨.٩ ١	١٠. ١٢	١٨. ٣٤	٢٣. ٩٠	٢٧. ٢٠	٣٠. ١٤	٣٢. ٨٥	٣٦. ١٩	٣٨. ٥٨
٢٠	٧.٤	٨.٢	٩.٥	١٠.	١٩.	٢٥.	٢٨.	٣١.	٣٤.	٣٧.	٤٠.

	٣	٦	٩	٨٥	٣٤	٠٤	٤١	٤١	١٧	٥٧	٠٠
٢١	٨.٠ ٣	٨.٩ ٠	١٠. ٢٨	١١. ٥٩	٢٠. ٣٤	٢٦. ١٧	٢٩. ٦٢	٣٢. ٦٧	٣٥. ٤٨	٣٨. ٩٣	٤١. ٤٠
٢٢	٨.٦ ٤	٩.٥ ٤	١٠. ٩٨	١٢. ٣٤	٢١. ٣٤	٢٧. ٣٠	٣٠. ٨١	٣٣. ٩٢	٣٦. ٧٨	٤٠. ٢٩	٤٢. ٨٠
٢٣	٩.٢ ٦	١٠. ٢٠	١١. ٦٩	١٣. ٠٩	٢٣. ٣٤	٢٨. ٤٣	٣٢. ٠١	٣٥. ١٧	٣٨. ٠٨	٤١. ٨٤	٤٤. ١٨
٢٤	٩.٨ ٩	١٠. ٨٦	١٢. ٤٠	١٣. ٨٥	٢٣. ٣٤	٢٩. ٥٥	٣٣. ٢٠	٣٦. ٤٢	٣٩. ٣٦	٤٢. ٩٨	٤٥. ٥٨
٢٥	١٠. ٥٢	١١. ٥٢	١٣. ١٢	١٤. ٦١	٢٤. ٣٤	٣٠. ٨٨	٣٤. ٣٨	٣٧. ٨٥	٤٠. ٦٥	٤٤. ٣١	٤٦. ٩٣
٢٦	١١. ١٦	١٢. ٢٠	١٣. ٨٤	١٥. ٣٨	٢٥. ٣٤	٣١. ٧٩	٣٥. ٥٨	٣٨. ٨٩	٤١. ٩٢	٤٥. ٦٤	٤٨. ٢٩
٢٧	١١. ٨١	١٢. ٨٨	١٤. ٥٧	١٦. ١٥	٢٦. ٣٤	٣٢. ٩١	٣٦. ٧٤	٤٠. ١١	٤٣. ١٩	٤٦. ٩٦	٤٩. ٦٤
٢٨	١٢. ٤٦	١٣. ٥٧	١٥. ٣١	١٦. ٩٣	٢٧. ٣٤	٣٤. ٠٣	٣٧. ٩٢	٤١. ٣٤	٤٤. ٤٦	٤٨. ٢٨	٥٠. ٩٩
٢٩	١٣. ١٢	١٤. ٢٦	١٦. ٠٥	١٧. ٧١	٢٨. ٣٤	٣٥. ١٤	٣٩. ٠٩	٤٢. ٥٨	٤٥. ٧٢	٤٩. ٥٩	٥٢. ٣٤
٣٠	١٣. ٧٩	١٤. ٩٥	١٦. ٧٩	١٨. ٤٩	٢٩. ٣٤	٣٦. ٢٥	٤٠. ٢٦	٤٣. ٧٧	٤٦. ٩٨	٥٠. ٨٩	٥٣. ٦٧
٤٠	٢٠. ٧١	٢٢. ١٦	٢٤. ٣٤	٢٦. ٥١	٣٩. ٣٤	٤٢. ٢٧	٥١. ٨١	٥٥. ٧٨	٥٩. ٣٤	٦٣. ٦٩	٦٦. ٧٧
٥٠	٢٧. ٩٩	٢٩. ٧١	٣٢. ٣٦	٣٤. ٧٦	٤٩. ٣٣	٥٨. ١٦	٦٣. ١٧	٦٧. ٥٠	٧١. ٤١	٧٦. ١٥	٧٩. ٤٩
٦٠	٣٥. ٥٣	٣٧. ٤٨	٤٠. ٤٨	٤٣. ١٩	٥٩. ٣٣	٨٨. ٩٧	٧٤. ٤٠	٧٩. ٠٨	٨٣. ٣٠	٨٨. ٣٨	٩١. ٩٥
٧٠	٤٣. ٢٨	٤٥. ٤٤	٤٨. ٧٦	٥١. ٧٤	٦٩. ٣٣	٧٩. ٧١	٨٥. ٥٣	٩٠. ٥٣	٩٥. ٠٢	١٠٠ .٤٣	١٠٤ .٢٠
٨٠	٥١. ١٧	٥٣. ٥٤	٥٧. ١٥	٦٠. ٣٩	٧٩. ٣٣	٩٠. ٤١	٩٦. ٥٨	١٠١ .٨٨	١٠٦ .٦٣	١١٢ .٣٣	١١٦ .٣٠
٩٠	٥٩. ٢٠	٦١. ٧٥	٦٥. ٦٥	٦٩. ١٣	٨٩. ٣٣	١٠١ .٠٥	١٠٧ .٥٧	١١٣ .١٥	١١٨ .١٤	١٢٤ .١٢	١٢٨ .٣٠
١٠٠	٦٧. ٣٣	٧٠. ٠٦	٧٤. ٢٢	٧٧. ٩٣	٩٩. ٣٣	١١١ .٦٧	١١٨ .٥٠	١٢٤ .٣٤	١٢٩ .٥٦	١٣٥ .٨١	١٤٠ .٢٠

Table (B-٨): The normal distribution (Normal Table) [٢٤].

Z	F (z)	Z	F (z)	Z	F (z)
-٣.٠٠	٠.٠٠١٣٤٩٩٠	-٠.٩٠	٠.١٨٤٠٦٠١٣	١.٢٠	٠.٨٨٨٤٩٣٠٣٣
-٠.٢٩	٠.٠٠١٨٦٥٨١	-٠.٨٠	٠.٢١١٨٥٥٤٠	١.٣٠	٠.٩٠٣١٩٩٥٢
-٢.٨٠	٠.٠٠٢٥٥٥١٣	-٠.٧٠	٠.٢٤١٩٦٣٦٥	١.٤٠	٠.٩١٩٢٤٣٣٤
-٢.٧٠	٠.٠٠٣٤٦٦٩٧	-٠.٦٠	٠.٢٧٤٢٥٣١٢	١.٥٠	٠.٩٣٣١٩٢٨٠
-٢.٦٠	٠.٠٠٤٦٦١١٩	-٠.٥٠	٠.٣٠٨٥٣٧٥٤	١.٦٠	٠.٩٤٥٢٠٠٧١
-٢.٥٠	٠.٠٠٦٢٠٩٦٧	-٠.٤٠	٠.٣٤٤٥٧٨٢٦	١.٧٠	٠.٩٥٥٤٣٤٥٤
-٢.٤٠	٠.٠٠٨١٩٧٥٤	-٠.٣٠	٠.٣٨٢٠٨٨٥٨	١.٨٠	٠.٩٦٤٠٦٩٦٨

-٢.٣٠	٠.٠١٠٧٢٤١١	-٠.٢٠	٠.٤٢٠٧٤٠٢٩	١.٩٠	٠.٩٧١٢٨٣٤٤
-٢.٢٠	٠.٠١٣٩٠٣٤٥	-٠.١٠	٠.٤٦٠١٧٢١٦	٢.٠٠	٠.٩٧٧٢٤٩٨٧
-٢.١٠	٠.٠١٧٨٦٤٤٢	٠.٠٠	٠.٥٠٠٠٠٠٠٠	٢.١٠	٠.٩٨٢١٣٥٥٨
-٢.٠٠	٠.٠٢٢٧٥٠١٣	٠.١٠	٠.٥٣٩٨٢٧٨٤	٢.٢٠	٠.٩٨٦٠٩٦٥٥
-١.٩٠	٠.٠٢٨٧١٦٥٦	٠.٢٠	٠.٥٧٩٢٥٩٧١	٢.٣٠	٠.٩٨٩٢٧٥٨٩
-١.٨٠	٠.٠٣٥٩٣٠٣٢	٠.٣٠	٠.٦١٧٩١١٤٢	٢.٤٠	٠.٩٩١٨٠٢٤٦
-١.٧٠	٠.٠٤٤٥٦٥٤٦	٠.٤٠	٠.٦٥٥٤٢١٧٤	٢.٥٠	٠.٩٩٣٧٩٠٣٣
-١.٦٠	٠.٠٥٤٧٩٩٢٩	٠.٥٠	٠.٦٩١٤٦٢٤٦	٢.٦٠	٠.٩٩٥٣٣٨٨١
-١.٥٠	٠.٠٦٦٨٠٧٢٠	٠.٦٠	٠.٧٢٥٧٤٦٨٨	٢.٧٠	٠.٩٩٦٥٣٣٠٣
-١.٤٠	٠.٠٨٠٧٥٦٦٦	٠.٧٠	٠.٧٥٨٠٣٦٣٥	٢.٨٠	٠.٩٩٧٤٤٤٨٧
-١.٣٠	٠.٠٩٦٨٠٠٤٨	٠.٨٠	٠.٧٨٨١٤٤٦٠	٢.٩٠	٠.٩٩٨١٣٤١٩
-١.٢٠	٠.١١٥٠٦٩٦٧	٠.٩٠	٠.٨٢٥٩٣٩٨٧	٣.٠٠	٠.٩٩٨٦٥٠١٠
-١.١٠	٠.١٣٥٦٦٦٠٦	١.٠٠	٠.٨٤١٣٤٤٧٥		
-١.٠٠	٠.١٥٨٦٥٥٢٥	١.١٠	٠.٨٦٤٣٣٣٩٤		

CERTIFICATE

We certify that we have read this thesis, entitled ***"Analysis and evaluation of water supply facilities for Al-Hilla city ",*** and as examining committee, examined the student *"**Thulfikar Razzak Al - Husseini"*** in its contents and in what is connected with it, and that in our opinion it meets the standard of a thesis for the degree of Master of Science in Civil Engineering.

Signature:
Name: Asst. Prof. Dr. Jabbar H. Al- Baidhani
(Member)
Date: / /٢٠٠٤

Signature:
Name: Dr. Faris H. Al- Ani
(Member)
Date: / /٢٠٠٤

Signature:
Name: Prof. Dr. Rafa Hashim S. Al- Suhaili
(Chairman)
Date: / /٢٠٠٤

Signature:
Name: Asst. Prof. Dr. Karim K. Al-Jumaily
(Supervisor)
Date: / /٢٠٠٤

Signature:
Name: Asst. Prof. Dr. Abdul Hassan K. Al-Shukur
(Supervisor)
Date: / /٢٠٠٤

Approval of the Civil Engineering Department
Head of the Civil Engineering Department
Signature:

Name: Asst. Prof. Dr. Ammar Y. Ali
Date: / /٢٠٠٤

Approval of the Deanery of the College of Engineering
Dean of the College of Engineering
Signature:

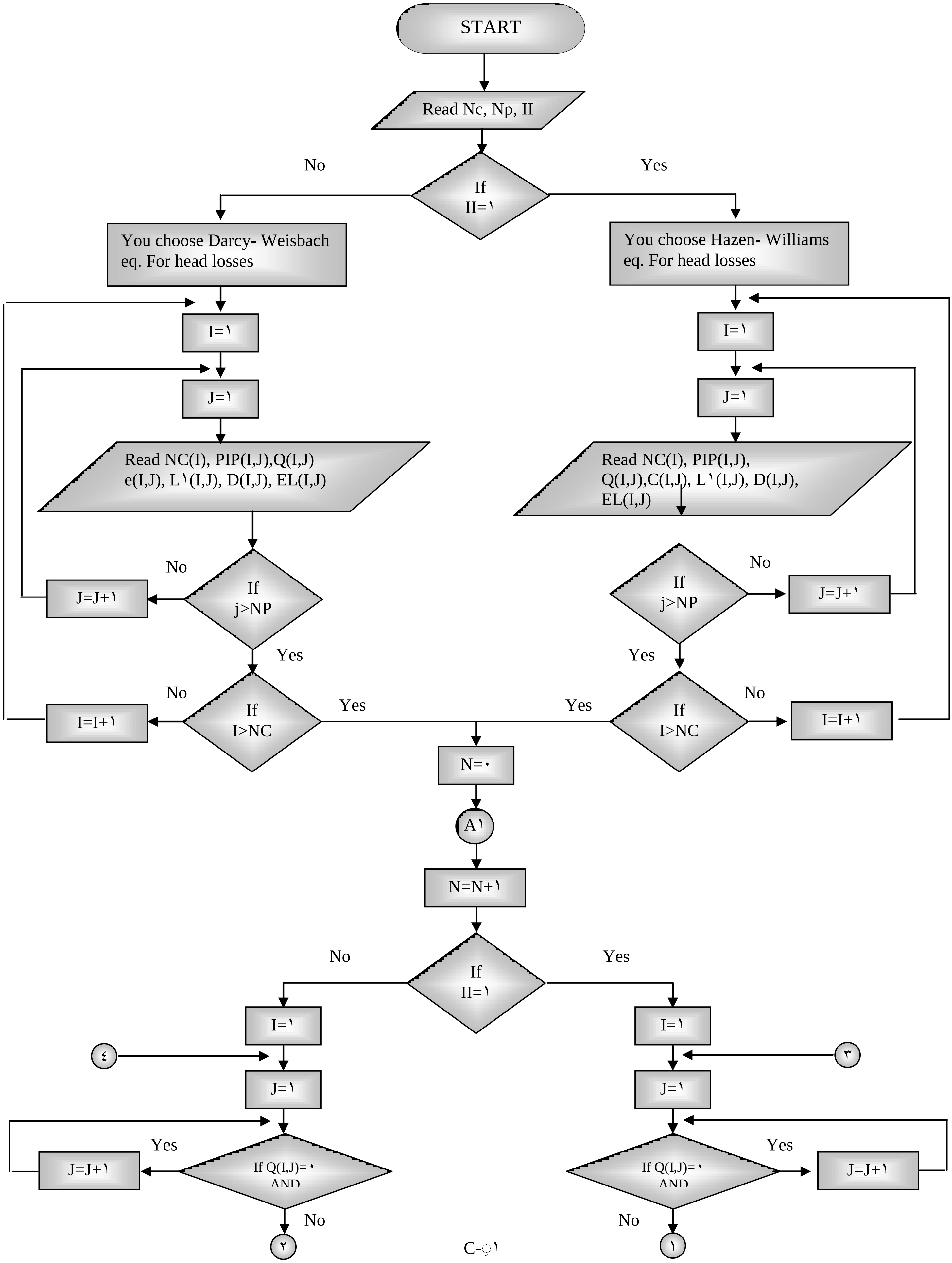
Asst. Prof. Dr. Haroon A. K. Shahad
Dean of the College of Engineering
Date: / /٢٠٠٤

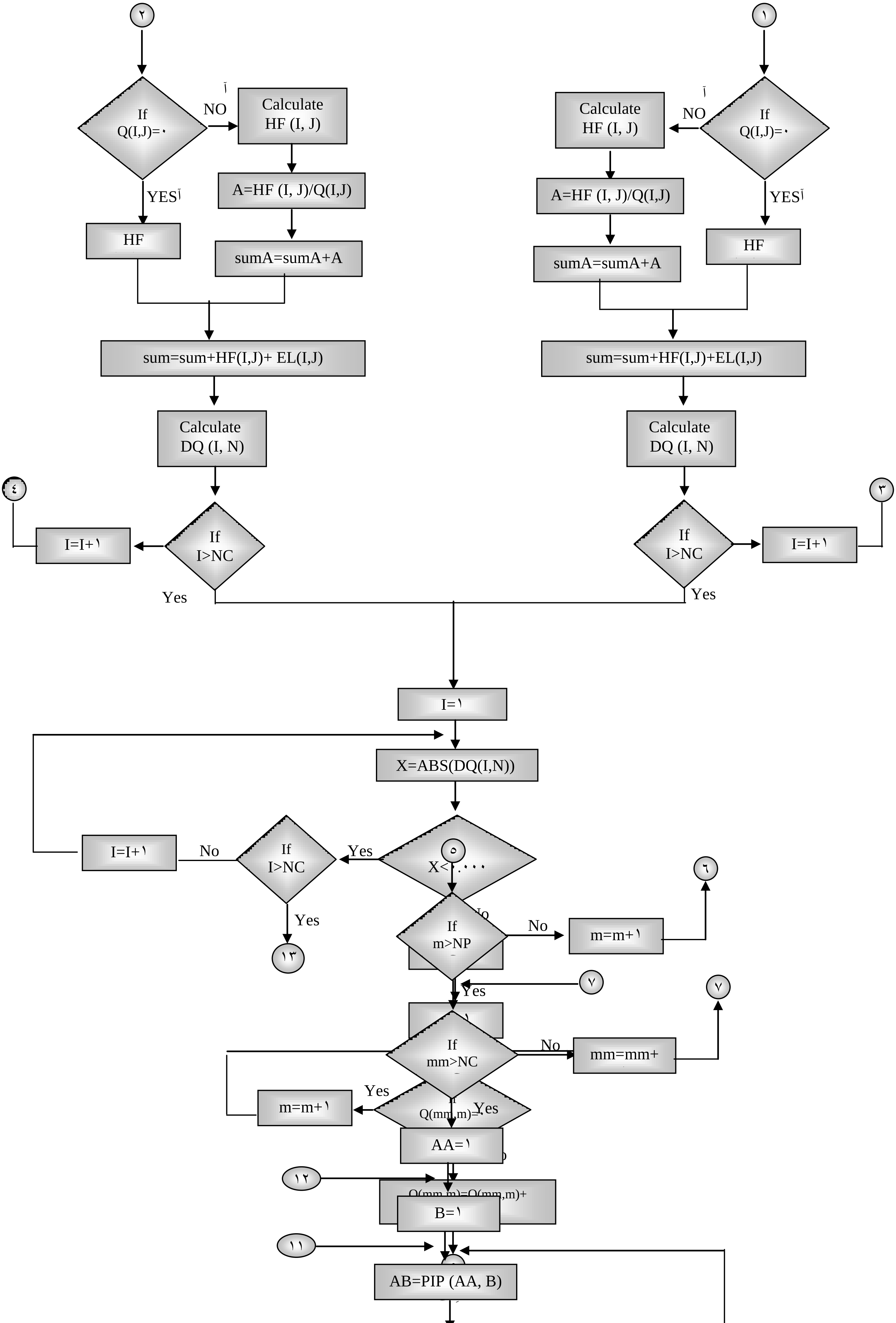
CERTIFICATE

We certify that the preparation of this thesis entitled ***"Analysis and evaluation of water supply facilities for Al-Hilla city "*** was prepared by *"**Thulfikar Razzak Al - Husseini"*** under our supervision at Babylon University, College of Engineering in partial fulfillment of the requirements for the degree of Master of Science in Civil Engineering.

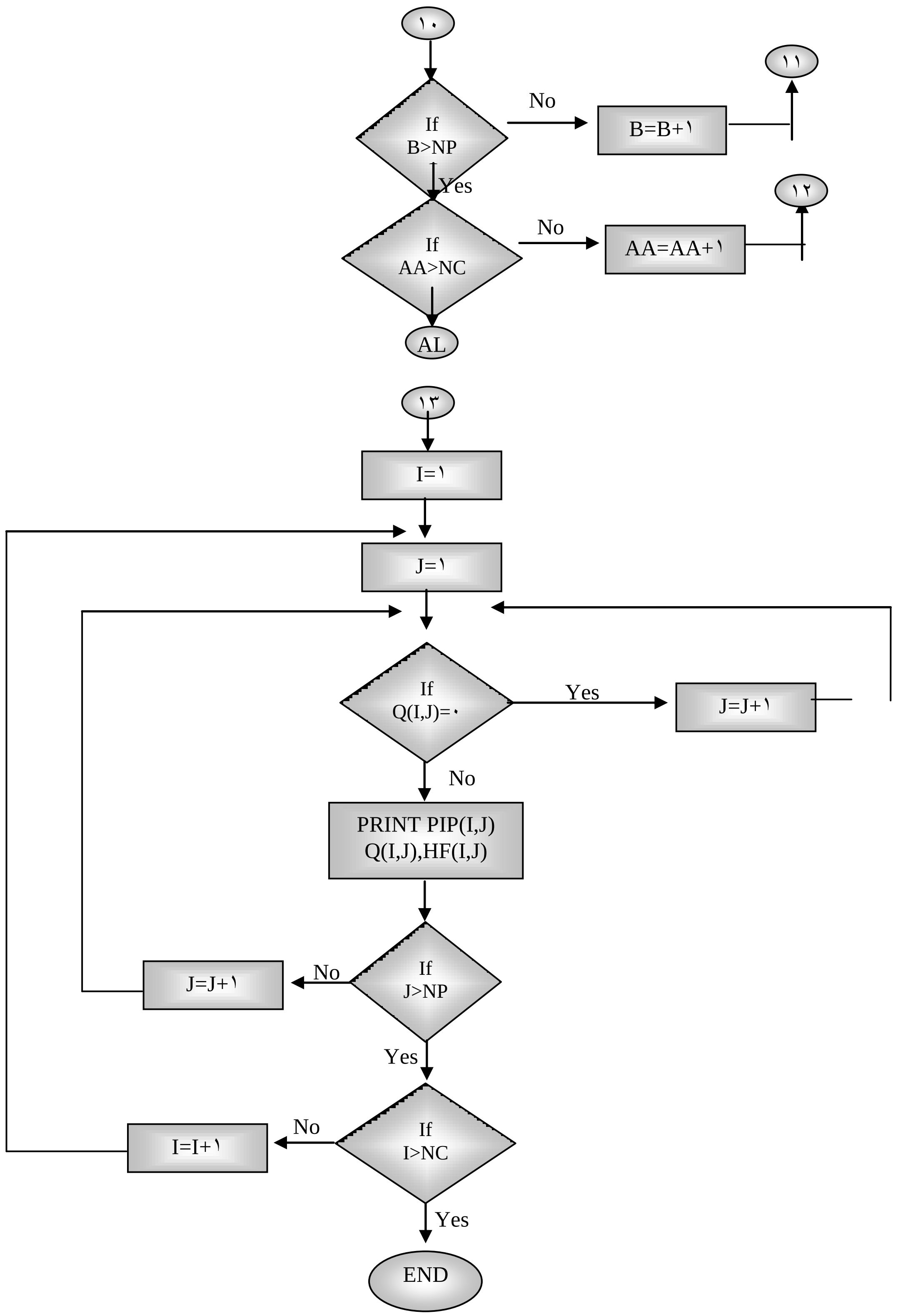
Signature:	Signature:
Name: Asst. Prof.	**Name:** Asst. Prof.
Dr. Karim K. Al- Jumaily	Dr. Abdul Hassan K. Al- Shukur
(Supervisor)	(Supervisor)
Date: / /٢٠٠٤	Date: / /٢٠٠٤

Appendix C
(C-١): Flow chart of computer program for analysis of pipe network.

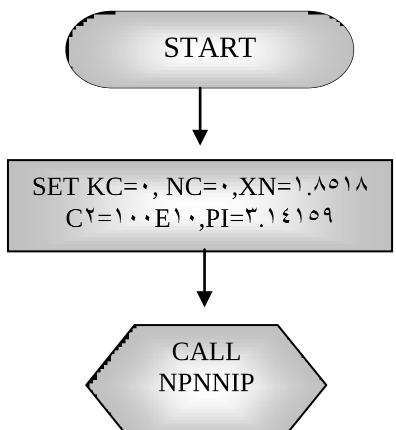


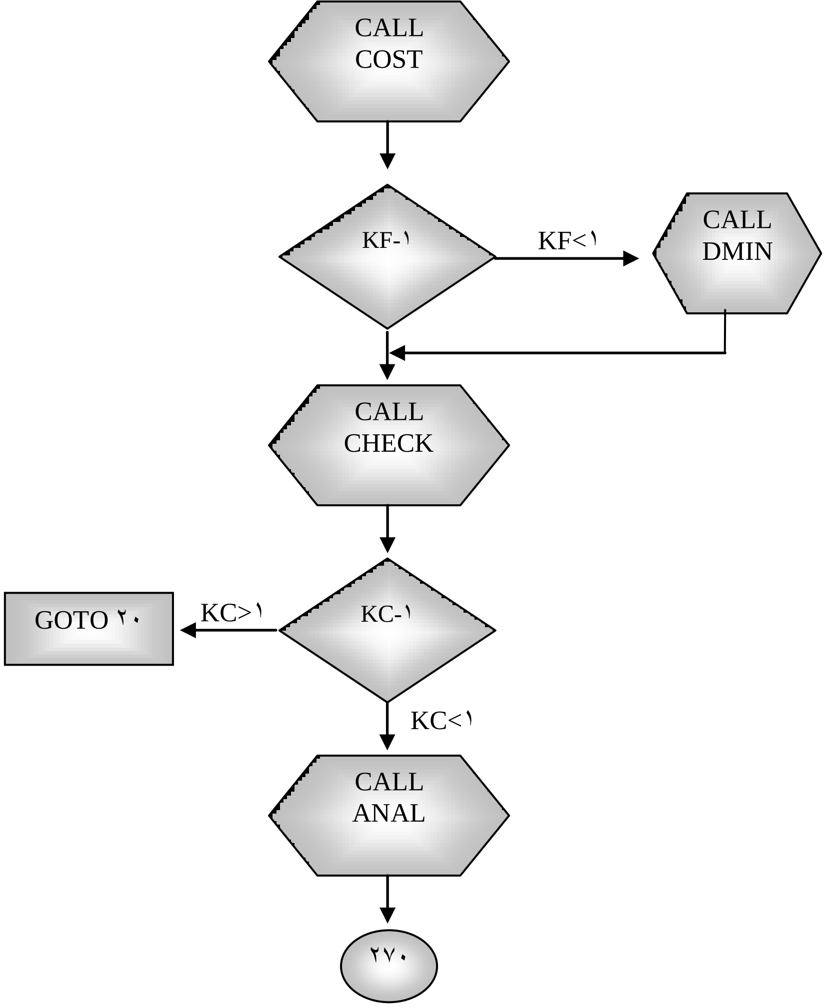


٢
If
Q(I,J)=٠
NO
Calculate
HF (I, J)
A=HF (I, J)/Q(I,J)
YES
HF
sumA=sumA+A
sum=sum+HF(I,J)+ EL(I,J)
Calculate
DQ (I, N)
٤
I=I+١
If
I>NC
Yes
١
If
Q(I,J)=٠
NO
Calculate
HF (I, J)
A=HF (I, J)/Q(I,J)
YES
HF
sumA=sumA+A
sum=sum+HF(I,J)+EL(I,J)
Calculate
DQ (I, N)
٣
If
I>NC
I=I+١
Yes
I=١
X=ABS(DQ(I,N))
I=I+١
No
If
I>NC
Yes
٥
Yes
١٣
٦
If
m>NP
No
m=m+١
٧
Yes
٧
If
mm>NC
No
mm=mm+
m=m+١
Yes
Yes
AA=١
١٢
B=١
١١
AB=PIP (AA, B)

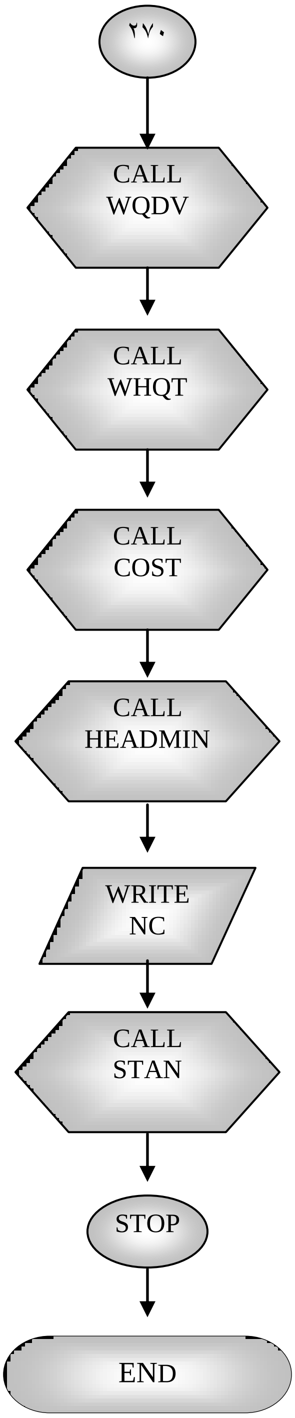


(C-٢): Flow chart of program for optimum design

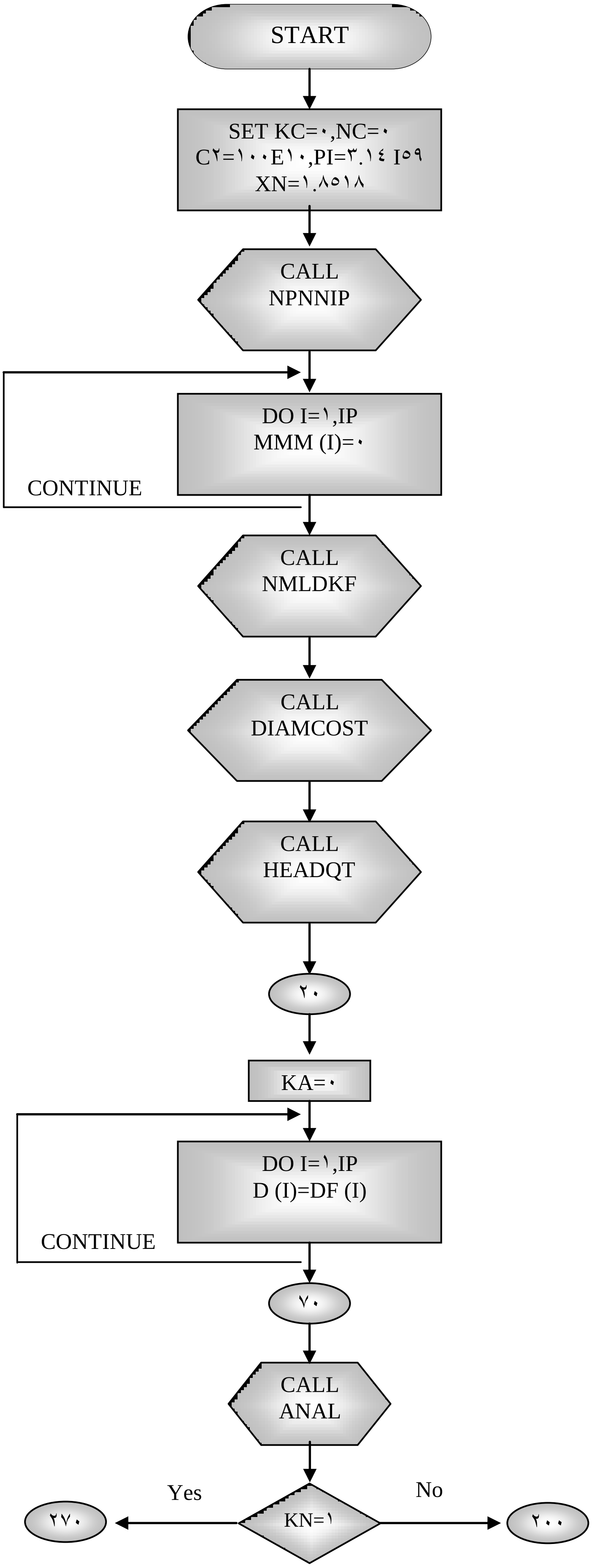


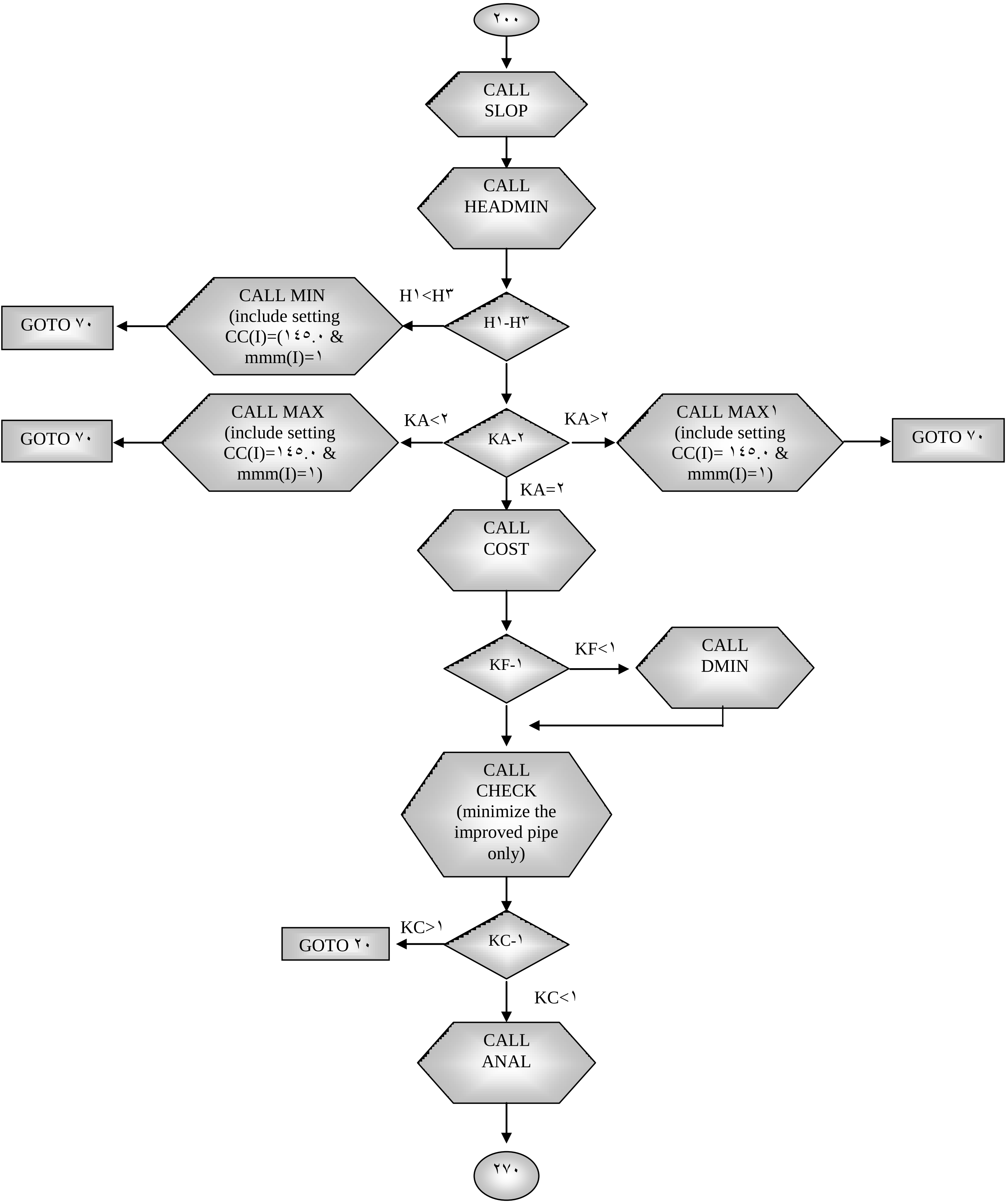


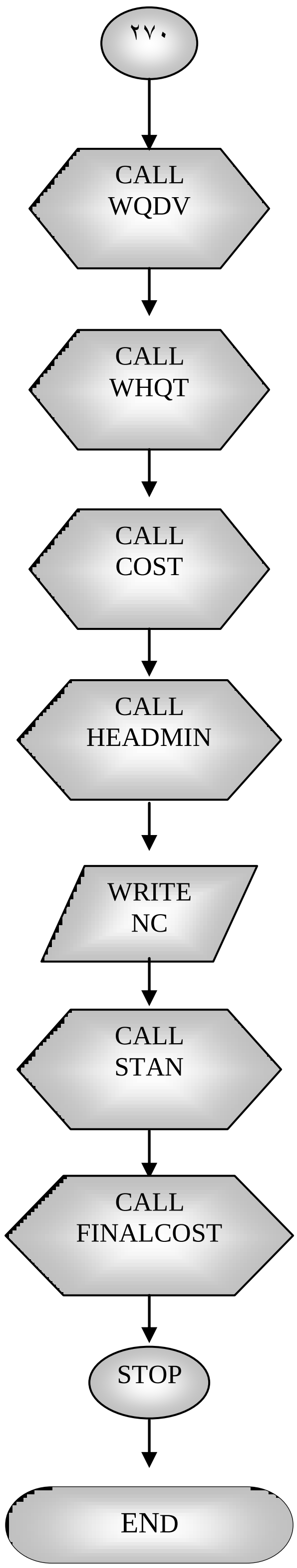
CALL COST
KF-١
KF<١
CALL DMIN
CALL CHECK
KC-١
KC>١
GOTO ٢٠
KC<١
CALL ANAL
٢٧٠


٢٧٠
CALL WQDV
CALL WHQT
CALL COST
CALL HEADMIN
WRITE NC
CALL STAN
STOP
END

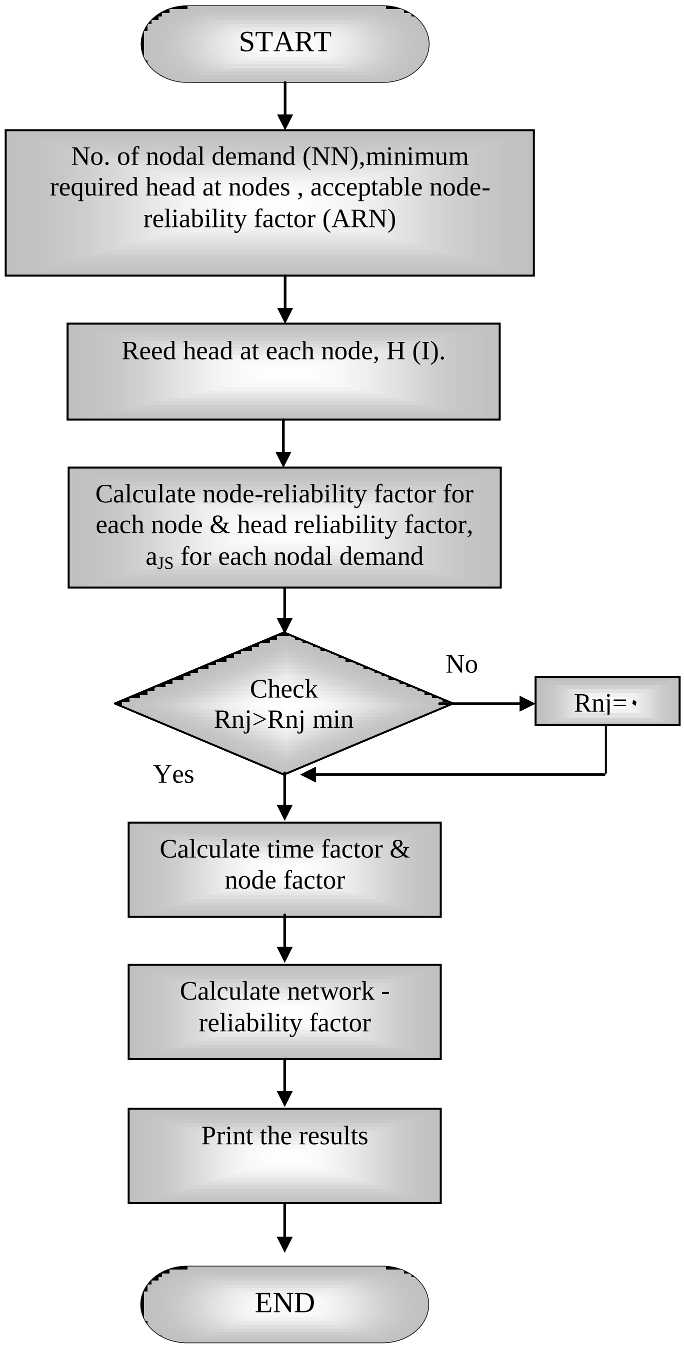
(C-٣): Flow chart of program for optimum rehabilitation




٢٠٠
CALL SLOP
CALL HEADMIN
H١-H٣
H١<H٣
CALL MIN (include setting CC(I)=(١٤٥.٠ & mmm(I)=١
GOTO ٧٠
KA-٢
KA<٢
CALL MAX (include setting CC(I)=١٤٥.٠ & mmm(I)=١)
GOTO ٧٠
KA>٢
CALL MAX١ (include setting CC(I)= ١٤٥.٠ & mmm(I)=١)
GOTO ٧٠
KA=٢
CALL COST
KF-١
KF<١
CALL DMIN
CALL CHECK (minimize the improved pipe only)
KC-١
KC>١
GOTO ٢٠
KC<١
CALL ANAL
٢٧٠


٢٧٠
CALL WQDV
CALL WHQT
CALL COST
CALL HEADMIN
WRITE NC
CALL STAN
CALL FINALCOST
STOP
END

(C-٤): Flow chart of computer program for Reliability Analysis



LIST OF CONTENTS

SUBJEGT **PAGE**

SUBJEGT **PAGE**

List of Symbols

Symbol	Description	Dimension
a	Average yearly increasing	%
Ai	Acceptable numbers from the recorded data.	-
Arn	Acceptable node reliability factor	-
C	Hazen-Williams coefficient	-
C.V	Coefficient of variation	-
COR	Coefficient of reliability	-
CYC	Cycle or loop in pipe network.	-
D	Pipe diameter	L
D_{α}	Critical value of kolmogorov – Smirnov test	-
D_{max}	The largest difference cumulative distribution function	-
DQ	Change in discharge	$L^{٣}/T$
e	Roughness hight.	L
EL	Difference in elevation of the reservoirs	L
F	Number of fixed grade nodes.	-
f	Moody friction factor	-
fi	Actual frequency value at mid – interval of each category	-
F_n	Node factor	-
F_t	Time factor	-
fx(x)	Cumulative distribution function	-
$\hat{f}_i$	Expected frequency value at mid – interval of each category	-
g	Acceleration due to gravity.	$L/T^{٢}$
H١	Design head	L
H_A	Allowable residual head	L
h_f	Head loss in pipe	L
H_o	Null hypothesis	-
HJWTP	Hilla Al- Jadeed water treatment plant	-
HKWTP	Hilla Al- Kadeem water treatment plant	-
II	Dummy variable	-
IP	Number of pipes in the network	-
J	Number of junctions or nodes	-
j	Subscript denoting demand node	-
K	Unite conversion factor	-
L	Number of primary loops	-
L١	Length of pipe.	L
M_t	Mass of the flow	$FT^{٢}/L$
$M_{t+\Delta t}$	Mass of flow after (Δt) time .	$FT^{٢}/L$
mx	Mean of x	-

Symbol **Description** **Dimension**

N	Number of corrections in loops	-
NC	Number of loops	-
Nd	Number of pipes, which connected with a certain node	-
NN	Number of nodes	-
nn	Total number of the recorded data	-
NP	Number of pipes in the loop, which have the maximum no. of pipes.	-
Npj	Number of pipes connected with node	-
NPL	Number of pipes in the loop	-
P	Number of pipes in the system, which means there are (P) unknowns	-
p.d.f	Probability density function	-
PIP	Pipe in the loop	-
P_o	Current population	person
P_t	Population after (t) years	person
Q_A	Allowable residual discharge	L^3/T
Q_T	Supply or drawn off water at node	L^3/T
Q^{avl}	Available discharge	L^3/T
Q^{req}	Required discharge	L^3/T
R	Hydraulic radius	L
Re	Reynolds number	-
Rel	Reliability	-
Rh	Head - reliability factor	-
Rnj	Node - reliability factor	-
Rnw	Network - reliability factor	-
Rv	Volume - reliability factor	-
S	Slope of energy line	-
s	Subscript denoting state	-
S.D	Standard deviation	L
ts	Time duration of state	T
V^{avl}	Available volume	L^3
V^{req}	Required volume	L^3
V	Velocity of flow in pipe	L/T
$\bar{X}$	Average of heads at nodes	L
xi	Values of head at node	L
Xs	A fixed standard	-
χ^2	Chi – square test value	-
Z	Final cost of construction per meter	I.D
$Z_{1-\alpha}$	Percentile of normal distribution	-
$\sigma \ln x$	Standard deviation of the natural logarithm of x	-

References

١. Abawi, S.A. and Hassan, M.S., (١٩٩٠), " **Environmental Engineering (water analysis)** ", Dar Al-Hikma, Mosul, Iraq.

٢. Al- Ani, O.M. (١٩٨٥) " **Analysis and design of water supply networks** ", M.Sc., Thesis, Department of Building and Construction, University of Technology, Baghdad, Iraq.

٣. Al- Sayyab, A.S. and Al – Jassim, J.A., (١٩٨٠) " **Principles of stratigraphy**", College of Science, University of Baghdad, Baghdad, Iraq.

٤. Al- Shaibani, A.H.R., (١٩٩٨)" **Analysis of pipe network and its reliability for water distribution** ", M.Sc., Thesis, Department of Building and Construction, University of Technology, Baghdad, Iraq.

٥. Amnuewattanakul, W., (١٩٩٩)" **Design of piping system by Hardy Cross method and water distribution control by fuzzy logic** ", www.google.com.

٦. Arasu, K., (١٩٩٣) " **Analysis of pipe networks** ", www.google.com.

٧. Benjamin, J.R., and Cornel, C.A, (١٩٧٠) " **Probability, Statistics, and Decision for Civil Engineering** ", McGraw-Hill Book Company.

٨. Collins, A.G. and Robert, L.J., (١٩٧٥) “ **Finite element method for water distribution networks** ”, Journal of AWWA, Vol.٦٧, No.٣٨٥, July.

٩. Deb A.K., (١٩٧٦), “ **Optimization of water distribution network system** ”, Journal of the Environmental Engineering Division, ASCE, Vol.١٠٢, No.EE٤.

١٠. Deb, A.K. and Sarkar, A.K., (١٩٧١) “ **Optimization in design of hydraulic network** ”, Journal of Sanitary Engineering Division, ASCE, Vol.٩٧, No.SA٢, April, PP.١٤١-١٥٩.

١١. Fox, Robert W., and Donald, Alan T.Mc., (١٩٩٨), “ **Introduction to fluid mechanics** ”, John Wiley and Sons, Inc., U.S.A.

١٢. Francis, J.R.D., and Minton, P., (١٩٨٤), “ **Civil Engineering Hydraulics** ”, Thomson Litho Ltd, East Kilbride, Scotland, U.K.

١٣. Fujiwara, O., (١٩٩٣) “ **Reliability analysis of water supply systems integrating with treatment** ”, comstech@isb.comsats.net.pk.

١٤. Giles, Ranald V., (١٩٨١) “ **Fluid mechanics and hydraulics** ”, McGraw – Hill, International Book Company, New York.

١٥. Gupta, R., (١٩٩٤), “ **Reliability analysis of water – distribution systems** ”, comstech@isb.comsats.net.pk.

١٦. H

ashim, K.S., (٢٠٠٣), “ **Dependability study of Al – Kerkh wastewater treatment plant”**, M.Sc., Thesis, College of Engineering, Department of Civil Engineering, University of Babylon, Babylon, Iraq.

١٧. H

ays, W.L., (١٩٦٣) “ **Statistics** ” McGraw-Hill Book Company, New York, U.S.A.

١٨. H

oag, L.N. and Weinberg, G., (١٩٥٧) “ **Pipeline network analysis by electrical computer** ”, Journal of AWWA, Vol.٤٩, PP.٥١٧-٥٢٤.

١٩. H

olton, Philip J. Purification Plant, (٢٠٠٢) “ **Water quality FAQ** ”, www.google.com.

٢٠. J

acoby, S.L.S. and Twigg, D.W., (١٩٦٨) “ **Computer solutions to distribution network problems** ”, Booing Research Report, Renton Washengton.

٢١. J

eppson, J.L., (١٩٩٨) “ **Equations for network analysis** ”, www.google.com

٢٢. K

im, J.H, (١٩٩٤) “ **Optimal rehabilitation model for water distribution systems** ”, comstech@isb.comsats.net.pk.

٢٣. L

ambert, S.D., (١٩٩٥) “ **A comparative evaluation of the**

effectiveness of potable water filtration processes ”, comstech@isb.comsats.net.pk.

٢٤. L

awrence, L.L., (١٩٨٣) “ **Probability, and statistics for modern Engineering.”** Van Nostrand Company.

٢٥. L

MNO Engineering, Research, and Software, Ltd., (٢٠٠٠) “ **Flow under pressure (closed conduits, pipes)** ”, www.dogpile.com.

٢٦. L

MNO Engineering, Research, and Software, Ltd., (٢٠٠١) “ **Major loss calculation for fluid flow using Darcy – Weisbach friction loss equation** ”, www.dogpile.com.

٢٧. L

MNO Engineering, Research, and Software, Ltd., (٢٠٠١) “ **Pipe network calculator Darcy – Weisbach or Hazen – Williams friction losses** ”, www.dogpile.com.

٢٨. M

ajumdar A., Bailey J.W., Schallhorn P., and Steadman T., (١٩٩٨), “ **A generalized fluid system simulation program to model flow distribution in fluid network** ”, www.google.com.

٢٩. M

athews, EH, (١٩٩٥)“ **A numerical optimization procedure for complex pipe and duct network design”**, comstech@isb.comsats.net.pk.

٣٠. M

cCuen, R.H. (١٩٨٥), “ **Statistical methods for engineers”**, Prentice-Hall, Inc., Englewood Cliffs, New Jersey.

٣١. M

echi, Z.A.A., (٢٠٠١) “ **Analysis and evaluation of the refinement water station for Al- Najaf city** ”, Thesis, Department of Building and Construction, University of Technology, Baghdad, Iraq.

٣٢. M

ein, R.G., and McMahon T.A., (١٩٧٨), “ **Reservoir capacity and yield procedures”**, Development in water science ٩, Elsevier, Amsterdam.

٣٣. M

inistry of Higher Education and Scientific Research, University of Babylon (٢٠٠٢), **Annual Report**.

٣٤. M

inistry of Irrigation, office of Hilla – Kifil Irrigation Projects (٢٠٠٠), **Annual Report on Land Topography**.

٣٥. M

yers, JR., (١٩٩٥) “ **Pitting corrosion of copper in cold potable water systems** ”, comstech@isb.comsats.net.pk.

٣٦. N

akashima, M. and Wenzel, H.G., (١٩٨٦) “ **Water supply models with capacity expansion** ”, Journal of Water Resources Planning and Management, ASCE, Vol.١١٢, No.١, January, PP.٨٧-١٠٣.

٣٧. N

ei, N.H. John, C.J., and Ropert, S.W., (١٩٧٥) “ **Statistical**

packages for social science SPSS " Second Ed., McGraw-Hill Book Company.

٣٨. N

iku, S.E., Schroeder, E.D., and Francisco, J.S., (١٩٧٩) " **Performance of activated sludge processes and reliability-based design "** Journal of water pollution control, Vol.٥١, No.١٢, December.

٣٩. P

eavy, H.S. and Rowe, D.R., (١٩٨٦) " **Environmental Engineering "**, McGraw – Hill Book Company, Singapore.

٤٠. R

aman, V. and Raman, S., (١٩٦٦) " **New method of solving distribution system networks based on equivalent pipe lengths "**, Journal of AWWA, Vol.٥٨, No.٥, May, PP.٦١٥-٦٢٦.

٤١. S

hamir, U., and Howard, C.D.D., (١٩٦٨) " **Water distribution systems analysis "**, Journal of the Hydraulics Division, ASCE, Vol.٩٤, No.HY١, January, PP.٢١٩-٢٣٤.

٤٢. S

tafford, M.D., (١٩٩٢) " **Water manual for refugee situations "**, www.google.com.

٤٣. S

tate Commission for Census, Babylon Governorate (١٩٨٧), **Annual Report**.

٤٤. S

tate Commission for Census, Babylon Governorate (١٩٩٧), **Annual Report**.

٤٥. State Commission for Census, Babylon Governorate (٢٠٠٢), **Annual Report**.

٤٦. State Commission for Water and Sewerage Board, (١٩٨٣) **“Hilla Water Supply Project”**.

٤٧. State Commission for Water and Sewerage Board, (١٩٩٨)“ **Adequacy study of drinking water requirement ”**.

٤٨. State Commission for Water and Sewerage Board, Water Office of Hilla City, (١٩٩٩), **Annual Report**.

٤٩. State Commission for Water and Sewerage Board, Water Office of Hilla City, (٢٠٠٠) **Annual Report**.

٥٠. State Commission for Water and Sewerage Board, Water Office of Hilla City, (٢٠٠٢), **Annual Report**.

٥١. Steel, E.W. and McGhee, T., (١٩٨٢) “ **Water supply and sewerage** ”, Fifth Addition, Translated by Dr. Ahmed, F.H., University of Salahaddin , Iraq.

٥٢. Streeter, V.L. and Wylie, E.B., (١٩٧٩) “ **Fluid Mechanics** ”, McGraw – Hill, International Book Company.

٥٣. S

u. Y.G., Mays, L.W., and Lansey, K.E., (١٩٨٧) " **Reliability – Based optimization model for water distribution systems** ", Journal of Hydraulics Engineering, ASCE, Vol. ١١٤, No.١٢ , P.١٥٣٩-١٥٥٦.

٥٤. T

ariq, A.K., (١٩٩٨) **"Reliability study of AL-Rustamiyah wastewater treatment plant."** MSc. thesis, College of Engineering, Department of Civil Engineering, Technology University, Baghdad, Iraq.

٥٥. T

ong, A.L. et al., (١٩٦١)" **Analysis of distribution networks by balancing equivalent pipe length** ", Journal of AWWA, Vol.٥٣, No.٢, February, P.١٩٢.

Table (B-٧): Reliability as a function of normalized mean, and V_X of effluent concentration [٣٨].

COR	٠.١	٠.٢	٠.٣	٠.٤	٠.٥	٠.٦	٠.٧	٠.٨	٠.٩	١.٠	١.٢٥	١.٥	٢.٠
V_X													
٠.٣	٠.٩٩٩٩	٠.٩٩٩٩	٠.٩٩٩٩	٠.٩٩٩٥	٠.٩٩٣٩	٠.٩٧٠٦	٠.٩١٣٤	٠.٨١٧٩	٠.٦٩٣٢	٠.٥٥٨٣	٠..٢٦٩٨	٠.١٠٨٣	٠.٠١٣٦
٠.٤	٠.٩٩٩٩	٠.٩٩٩٩	٠.٩٩٩٥	٠.٩٩٥٠	٠.٩٧٦٧	٠.٩٣٥٣	٠.٨٦٨١	٠.٧٧٩٩	٠.٦٧٩٣	٠.٥٧٦٣	٠.٣٥٠٧	٠.١٩٥٠	٠.٠٥٤٢
٠.٥	٠.٩٩٩٩	٠.٩٩٩٩	٠.٩٩٧٣	٠.٩٨٥٢	٠.٩٥٥٧	٠.٩٠٦٢	٠.٨٣٩١	٠.٧٦٠٧	٠.٦٧٧٠	٠.٥٩٣٤	٠.٤٠٦٤	٠.٢٦٦٩	٠.٠١٠٩
٠.٦	٠.٩٩٩٩	٠.٩٩٩٣	٠.٩٩٢٨	٠.٩٧٣٢	٠.٩٣٦٦	٠.٨٨٤٦	٠.٨٢١٢	٠.٧٥١٦	٠.٦٧٩٨	٠.٦٠٩٢	٠.٤٥٠٢	٠.٣٢٥١	٠.١٦٦٠
٠.٧	٠.٩٩٩٩	٠.٩٩٧٩	٠.٩٨٦٩	٠.٩٦١٠	٠.٩٢١٢	٠.٨٦٧٥	٠.٨١٠٧	٠.٧٤٨٣	٠.٦٨٥٣	٠.٦٢٣٩	٠.٤٨٥٠	٠.٣٧٢١	٠.٢١٧١
٠.٨	٠.٩٩٩٩	٠.٩٩٥٩	٠.٩٨٠٥	٠.٩٥٠٩	٠.٩٠٨٧	٠.٨٥٩٤	٠.٨٠٤٧	٠.٧٤٨٢	٠.٦٩٢٠	٠.٦٣٧٣	٠.٥١٣٧	٠.٤١١٠	٠.٢٦٣٢
٠.٩	٠.٩٩٩٦	٠.٩٩٣٢	٠.٩٧٤٣	٠.٩٤٣٢	٠.٩٠٠٦	٠.٨٢٧٠	٠.٨٠١٨	٠.٧٥٣٩	٠.٦٩٩٢	٠.٦٤٩٩	٠.٥٣٨٠	٠.٤٤٣٩	٠.٣٠٣٣
١.٠	٠.٩٩٩٣	٠.٩٩٠٥	٠.٩٦٨٨	٠.٩٣٥٣	٠.٨٩٤١	٠.٨٤٨٥	٠.٨٠٠٩	٠.٧٥٣٠	٠.٧٠٦٥	٠.٦٦١٤	٠.٥٥٨٩	٠.٤٧١٨	٠.٣٣٩٨
١.٢	٠.٩٩٨٢	٠.٩٨٥١	٠.٩٥٩٧	٠.٩٢٥٤	٠.٨٨٦٠	٠.٨٤٤٥	٠.٨٠٢٣	٠.٧٦٠٤	٠.٧٢٠١	٠.٦٨١٦	٠.٥٩٣٦	٠.٤٨٢٨	٠.٣٩٦٧
١.٥	٠.٩٩٦١	٠.٩٧٨٥	٠.٩٥٠٧	٠.٩١٧٢	٠.٨٨١٢	٠.٨٤٤٦	٠.٧٧٥٤	٠.٧٧٢٨	٠.٧٣٨٩	٠.٧٠٧٥	٠.٥٦١٢	٠.٥٦٧٥	٠.٤٦٢٠

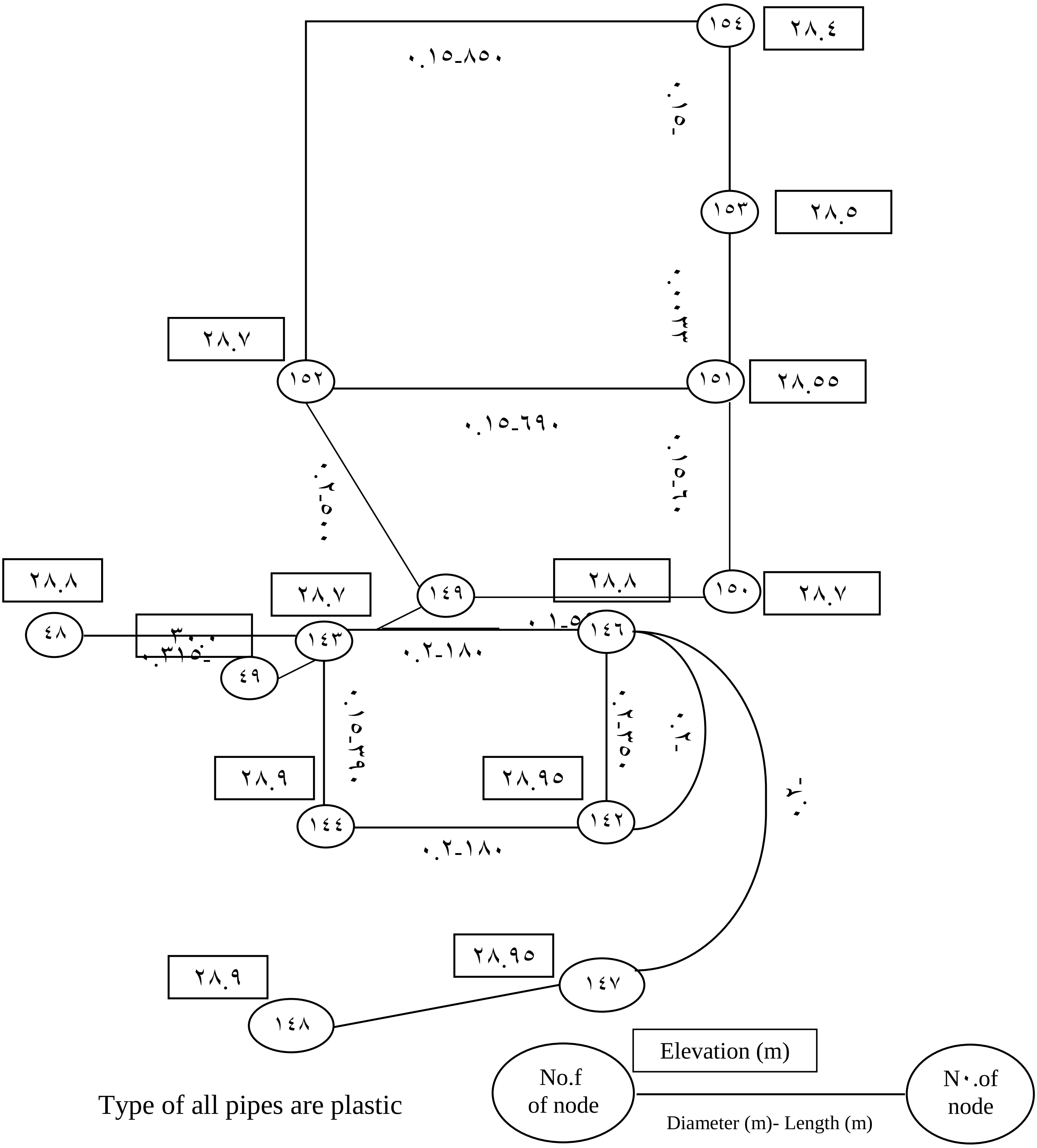
Table (٤-٩): Heads at supply nodes for Hilla city pipe network.

Column No.		**١**	**٢**	**٣**	**٤**	**٥**	**٦**	**٧**	**٨**	**٩ =١-٦+٧-٨**
Node No.	**Working pumps**	**Head of pump (m)**	**Diameter (m)**	**Length (m)**	**$C_{H.W}$ (-)**	**Q ($m^٣/s$)**	**h_f (m)**	***E (m)**	****E (m)**	**Head at node (m)**
١	١	٣٦.٠	٠.٦	٢١٦٠	١١٣	٠.١٥٣	١.٣٤	+٠.٥٥	٢.٠	٣٣.٢١
٣٣	٢	٣٠.٠	٠.٦	٤٠	٩٠	٠.٣٣٣	٠.١٦	+٠.١٧	٠.٠	٣٠.٠١
٤٩	١	٣٥.٠	٠.٨	٢٦٠٠	١١٣	٠.٣٠٦	١.٤٤	-٠.٨٥	٤.٠	٢٨.٧١
٦٧	١.٥^	٥٨.٠	١.٢	٣٤١٠	١١٣	٠.٥٠٠	٠.٦٥	-٠.٧	٢.٠	٥٦.٠٥
٧٧	١	٣٧.٠	١.٢	٣١٦٠	١١٣	٠.٣٣٣	٠.٢٩	-٠.٠٥	٤.٠	٣٢.٦٦

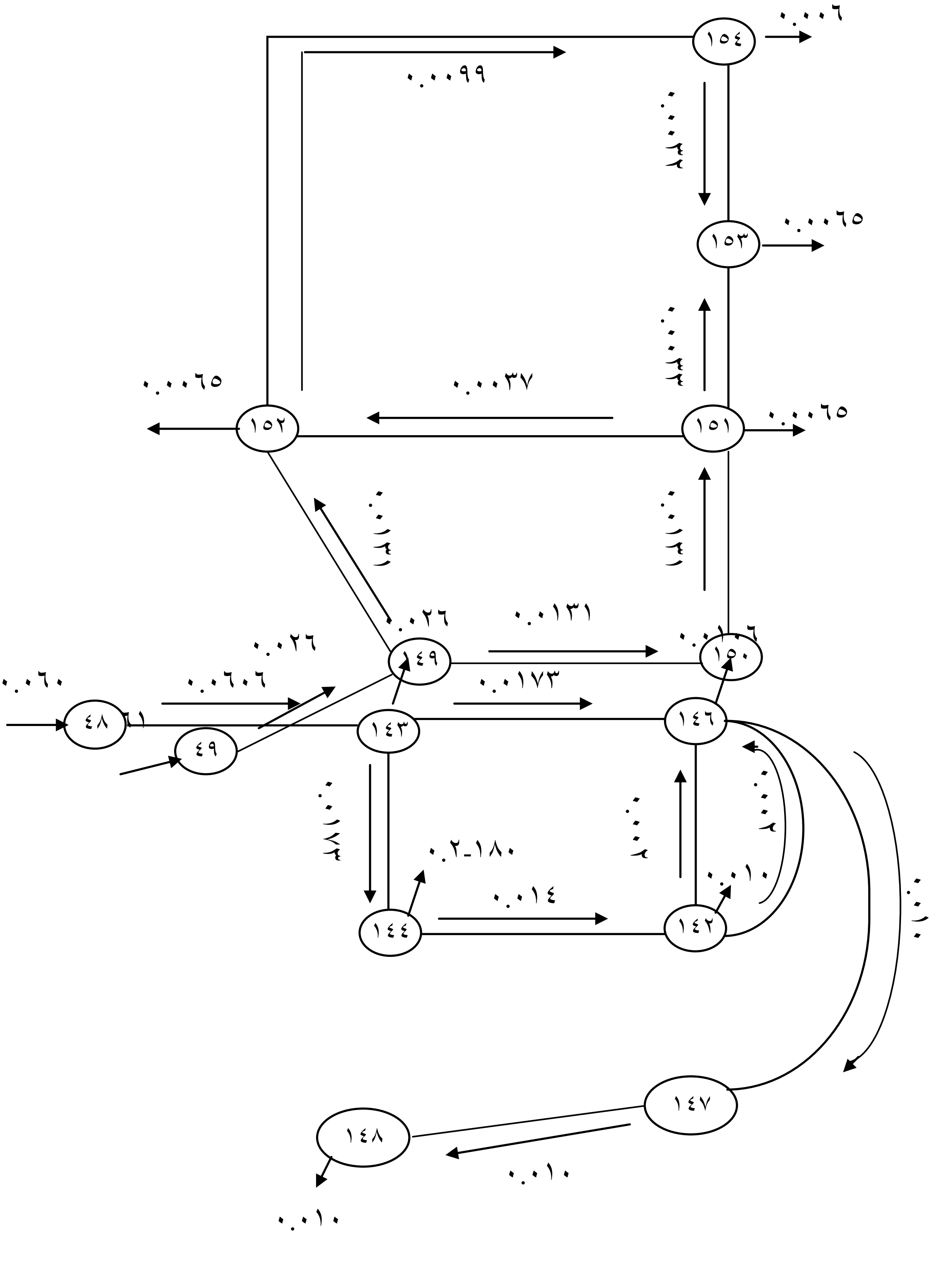
*E: The difference in elevation between the beginning and the end of pipe that carrying flow from pumping station to the supply node.

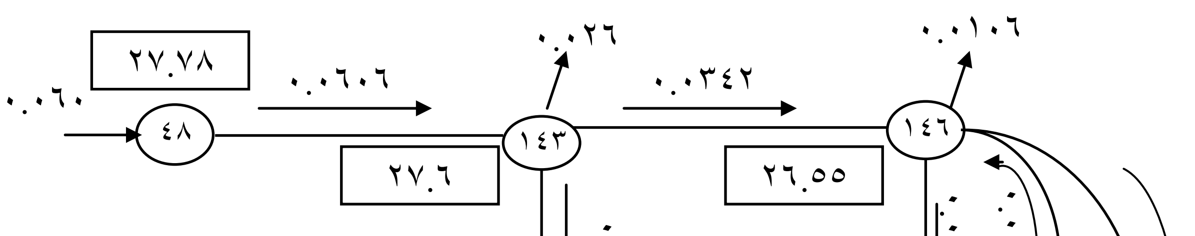
**E: Elevation of pump under natural ground level.

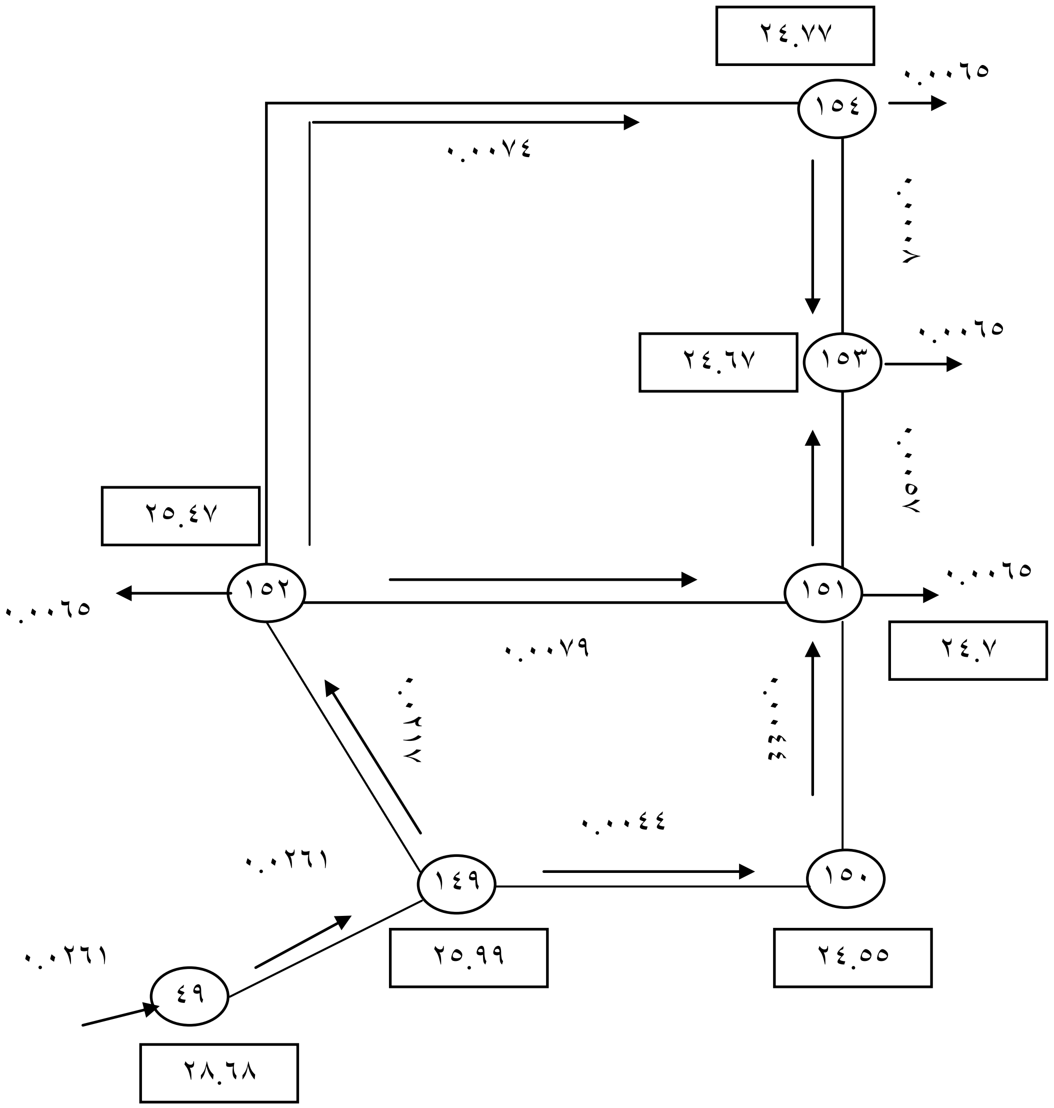
^: Both pumps work during day hours, only one of them during night hours.



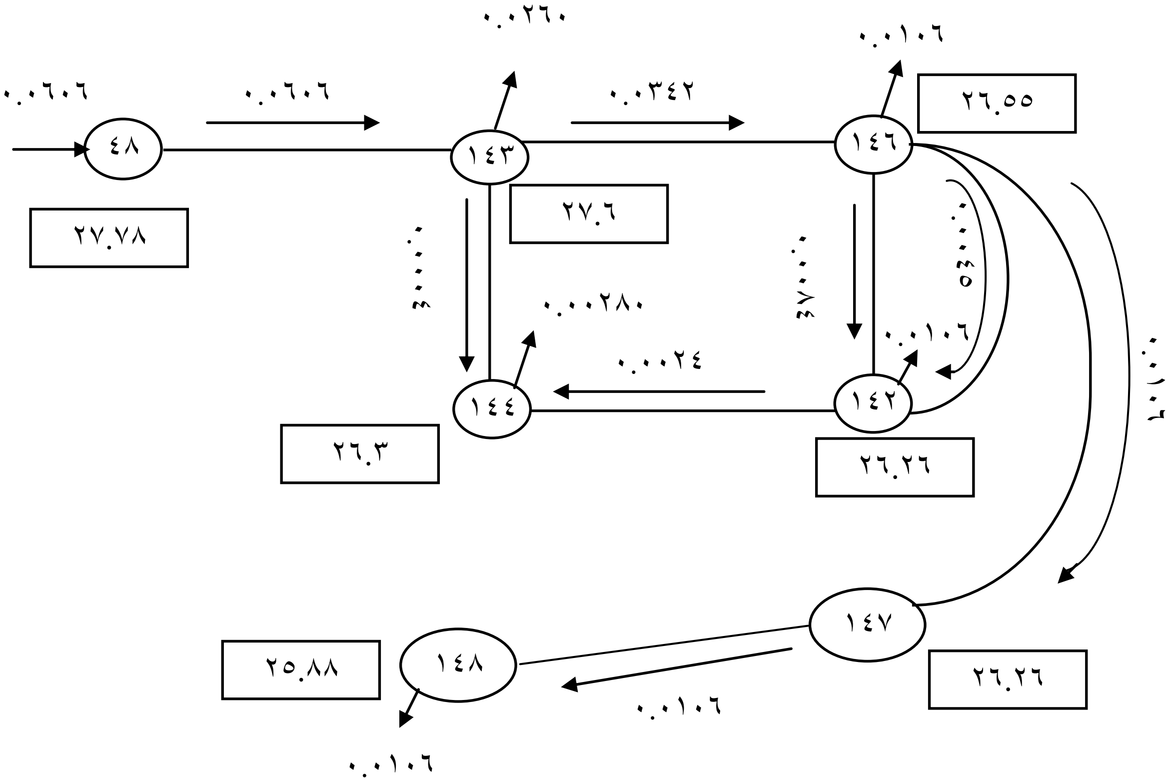
Type of all pipes are plastic


١٥٤
٠.٠٠٦
٠.٠٠٩٩
١٥٣
٠.٠٠٦٥
٠.٠٠٦٥
٠.٠٠٣٧
١٥٢
١٥١
٠.٠٠٦٥
٠.٠٢٦
٠.٠١٣١
٠.٠١٠٦
٠.٠٢٦
١٤٩
١٥٠
٠.٠٦٠
٠.٠٦٠٦
٠.٠١٧٣
٤٨
٤٩
١٤٣
١٤٦
٠.٢-١٨٠
٠.٠١٤
٠.٠١٠
١٤٤
١٤٢
١٤٧
١٤٨
٠.٠١٠
٠.٠١٠


٢٧.٧٨
٠.٠٦٠
٠.٠٦٠٦
٠.٠٢٦
٠.٠٣٤٢
٠.٠١٠٦
٤٨
١٤٣
١٤٦
٢٧.٦
٢٦.٥٥



٢٤.٧٧
٠.٠٠٦٥
١٥٤
٠.٠٠٧٤
١٥٣
٢٤.٦٧
٠.٠٠٦٥
٢٥.٤٧
١٥٢
٠.٠٠٦٥
١٥١
٠.٠٠٦٥
٢٤.٧
٠.٠٠٧٩
٠.٠٢١٧
٠.٠٠٤٤
٠.٠٢٦١
١٤٩
١٥٠
٠.٠٢٦١
٢٥.٩٩
٢٤.٥٥
٤٩
٢٨.٦٨



٠.٠٢٦٠
٠.٠١٠٦
٠.٠٦٠٦
٠.٠٦٠٦
٠.٠٣٤٢
٢٦.٥٥
٤٨
١٤٣
١٤٦
٢٧.٧٨
٢٧.٦
٠.٠٠٢٨٠
٠.٠١٠٦
٠.٠٠٢٤
١٤٤
١٤٢
٢٦.٣
٢٦.٢٦
١٤٧
٢٦.٢٦
١٤٨
٢٥.٨٨
٠.٠١٠٦
٠.٠١٠٦