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(Water Engineering)

Presented by

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USER FRIENDLY NUMERICAL HYDRAULIC MODEL FOR COMPLEX PIPE NETWORKS

CASE STUDY: MBALE SCHOOL ZONE, UGANDA

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Abstract

Whenever there are substantial variations in quantity of demands within a metropolitan water network, it is necessary to assess the pipe network. Variations in demand exist every time new industries or residences are connected to the network. In cases where no analyses are done prior to making new connections, unnecessarily huge funds are incurred and use of unreasonably bigger pipes is inevitable, some of which may stay redundant. The present study aims at developing a user friendly numerical hydraulics model for analysing compound pipe networks: The case of Mbale School zone in Uganda. The model was developed using the V-Model approach, written in visual basic language, to resolve the elementary pipe system equations using improved Hardy Cross method. This program examines steady state flows, head losses, flow velocities, and pressures for single, two, three, and four loop water distribution network. The four loop example represents the entire network of the case study area in consideration. The comparative study conducted on results from the program and EPANET indicated consistency in the results as coefficient of determinant, R^2, for all the computed variables was approximately unity(1). The Root Mean Square Error (RMSE) and Mean Bias Error (MBE) were found to be reasonably so small. Therefore, it can be concluded from the statistical analysis that the model is reliable.

Résumé

Chaque fois qu'il existe des variations substantielles de la quantité de demandes au sein d'un réseau d'eau métropolitain, il est nécessaire d'évaluer le réseau de canalisations. Des variations de la demande existent chaque fois que de nouvelles industries ou résidences sont connectées au réseau. Dans les cas où aucune analyse n'est effectuée avant d'établir de nouvelles connexions, des fonds inutilement énormes sont engagés et l'utilisation de conduites déraisonnablement plus grandes est inévitable, dont certaines peuvent rester redondantes. La présente étude vise à développer un modèle hydraulique numérique convivial pour l'analyse des réseaux de canalisations composites: le cas de la zone d'école de Mbale en Ouganda. Le modèle a été développé à l'aide de l'approche V-Model, écrite en langage visuel de base, afin de résoudre les équations du système de conduite élémentaire à l'aide de la méthode améliorée de Hardy Cross. Ce programme examine les débits en régime permanent, les pertes de charge, les vitesses d'écoulement et les pressions d'un réseau de distribution d'eau à une, deux, trois et quatre boucles. L'exemple à quatre boucles représente l'ensemble du réseau de la zone d'étude de cas considérée. L'étude comparative menée sur les résultats du programme et sur EPANET a montré que la cohérence des résultats était satisfaisante car le coefficient du déterminant, R^2, pour toutes les variables calculées était approximativement égal à l'unité (1). L'erreur quadratique moyenne (RMSE) et l'erreur de biais moyenne (MBE) se sont avérées raisonnablement faibles. L'analyse statistique permet donc de conclure que le modèle est fiable.

Originality Statement

This is to ratify that the material presented in this thesis report is utterly my own work, apart from where specific references have been made to the works of others, and no portion of this work has been submitted in for award of a degree, diploma or certificate to any university or institution.

Transitory citations from this study are admissible without special authorization, on condition that accurate acknowledgment of source is done.

Signature: ……………………….

Denis Obura

Date: August 17th 2019

Certification

I undersigned, Prof. Abdelkrim Khaldi, Guest Lecturer at the Pan African University Institute of Water and Energy Sciences including Climate Change (PAUWES), and permanent lecturer at the University of Sciences and Technology Mohamed Boudiaf, Oran, Algeria/ Department of Hydraulics; certify that Mr. Obura Denis conducted his Master Thesis research under my supervision. Certified further, that this master thesis entitled *“User friendly Numerical Hydraulics Model for Complex Pipe Networks: A case of Mbale School Zone, Uganda”* is an authentic work of Mr. Obura Denis who carried out the research under my guidance.

Signature:

Pr Abdelkrim Khaldi

Date: August 17th 2019

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Abbreviations

ADLC	Application Development Life Cycle
GDP	Gross Domestic Product
GI	Galvanized Iron
GMS	Galvanized Mild Steel
GUI	Graphical User Interface
HC	Hardy Cross
LTM	Linear Theory Method
MWE	Ministry of Water and Environment
NR	Newton-Raphson
NRW	Non-Revenue Water
NWSC	National Water and Sewerage Cooperation
SDGs	Sustainable Development Goals
UBOS	Uganda Bureau of Statistics
UNICEF	United Nations International Children's Emergency Fund
UPVC	Unplasticized Polyvinyl Chloride
VB	Visual Basic
WHO	World Health Organisation

Chapter 1 : Introduction

1.0 Background Information

Worldwide, all languages have a word for “water”. For instance, in Swahili, a language widely spoken by the natives of East Africa, water is called “maji”. However we say it, water is a very essential resource for the existence of all life forms on earth (Sonaje & Joshi, 2015). It plays voluminous central roles such as navigation, irrigation, power production, recreation, machine cooling and raw material cleaning in factories and receiving wastewater (Ahmed, 1997). Today, this resource is delivered to communities either through looped, branched or combined pipe networks which are one of the principal infrastructure assets of the general public (Poulakis, et al., 2003). These networks are interconnections of various components such as transmission pipes, distribution pipes, service connection pipes, pumps, joints, valves, and fire hydrants supplying water to consumers in recommended quantities with adequate pressure (Esiefarienrhe & Effiong, 2014). A study conducted by WHO/UNICEF (joint Water Supply and Sanitation Monitoring programme) in 2015 exposed that the percentage of the world’s population with access to developed drinking water source propagated from 76% to 91% from 1990 to 2015, and the population share with access to piped water on their buildings grew from 44% to 58%.

Uganda has undergone policy reforms that have brought about increased investments and a faster economic growth at an average annual GDP growth rate of 6.9%. This has led to expansion of cities, industrial and economic development[1]. Besides that, the population of Uganda has continually enlarged at an average annual growth rate of 3.0 percent. By mid-year 2017, Uganda Bureau of Statistics (UBOS) projected the population to be 37,730,300. All these factors have imposed more pressure on the scarce fresh water resources in the country. In the past few years, levels of access to piped water in urban areas like Mbale district in Eastern Uganda have improved slightly from 69% in 2011/12 to 71% in 2016/17 (MWE, 2013). Mbale being the main center of trade in Eastern Uganda houses a big population that tends to outpace benefits in infrastructure development. There has been an influx of students in schools located

[1] Uganda National Water Development Report - 2005

in Mbale School Zone. This has led to growth in the number of schools (nursery, primary, secondary and higher institution) within the area. As more schools are being constructed, the number of service connections shoots up due to high demand for portable water. Service connection tasks in populated urban areas require prior accurate calculation of required flow rates and pressures needed to deliver sufficient quantity of water. This entreats unrivalled opportunities for both engineers and scientists in Mbale National Water and Sewerage Cooperation (NWSC)[2] to put their knowledge and skills to effectively design, rehabilitate and expand the existing pipelines with the capacity to provide water to consumers with adequate quantity at sufficient pressure (Harry, 2008). However, the designers involved in design, construction and maintenance of public water distribution systems have been faced with a great trial when it comes to calculation of flows and pressure losses in a composite network. It is highly recommended for designers to maximize water supply of satisfactory quantity while minimizing pressure drops along the pipe network. Pressure drops are generally a result of two mechanisms, (a) friction along the pipe walls and (b) the turbulence due to sudden pipe expansion and contraction, bend in pipe, and pipe fittings causing changes in streamlines. Pressure losses due to friction are commonly referred to as major losses, while losses due to turbulence are known as minor losses (Elojali, 2011). Assessment of pressure drops due to friction is very paramount in hydraulic study of pipe networks.

The development of adequate engineering decision support tools such as a reliable user friendly water network model can help engineers and scientists quantify the head losses and flow in the water distribution system. A number of algorithms have been established in past few eras since the inventive work of Hardy Cross, a structural engineering professor at the University of Illinois (Cross, 1936). The three (3) widely used methods in solving water distribution networks are; Newton-Raphson (Shamir & Howard, 1968), Linear Theory (Wood & Charles, 1972) and Hardy-Cross (Cross, 1936). These methods require initialization of flow rates (Lee, 1983) for all pipes that satisfy flow continuity followed by a precise step-by-step computational procedure to yield an output value in a finite number of steps. The drudgery of manual iterative computation associated with these algorithms is time consuming.

[2] National Water and Sewerage cooperation (NWSC) is the body responsible for service connections in Large towns

Therefore, development of a complex pipe network, stand-alone, user friendly hydraulic model for the efficient calculation of steady state flows, pressures and network pipe costs with great precision while shortening iterative process, can be very useful for both academic and operational purposes.

Problem Statement

Despite the fact that piped water supply is regularly regarded as the criterion of improved water supply (Erickson, 2016), its reliability is at the mercy of pressure needed to provide sufficient quantity of water to the end users. According to UBOS[3], Mbale district population census in 2002 was estimated at 332,571 persons, with the population projection figures of 568,800 in 2019. The ever increasing population has led to perpetual growth in water demand and low pressures in pipe networks. As more people get connected to the network, the water authorities are tasked to transit from branching arrangement with dead ends to grid configuration with loops to increase the pressure heads in quotas of a municipality (including institutional, industrial, business and commercial areas). This invokes rigorous and precise analysis of the required flow rates and sustainable pressures to deliver adequate quantity of water at lower cost. Simple branched network problems can be resolved by hand calculation. Conversely, compound networks with multipart loops need additional effort even for steady-state flow situations (Lansey & Mays, 1999). Today, reliable commercial hydraulic network software suites available are unaffordable particularly in under developed countries such as Uganda. Besides, their usage has been a real test as it calls for cutting-edge computer knowledge and skills or acquaintance with a particular software package (Tigkas, et al., 2015). For the past few years, Engineers engrossed in software development have focused more on the numerical code (the computation engine), ignoring the ease of use of the final product (Khezzar, et al., 2000). Consequently, most firms involved in design, construction and operation of water distribution networks resort to manual calculation. Manual calculation is susceptible to mistakes and is time wasting. Using a computer model to calculate and analyse hydraulic networks will help save much time. In calculation process, computers are less vulnerable to

[3] https://www.ubos.org

errors (Kurniawan, 2009). Therefore, this study aims to develop a user friendly numerical hydraulics model to efficiently analyze and evaluate the cost of pipes in complex network.

1.1 General Objective

To design and implement a user friendly numerical hydraulic model for complex pipe networks.

1.2 Specific Objectives

1) To develop a complex pipe network model for solving up to a maximum of four (4) closed loops.
2) To check pipeline flow rates, velocities, head losses and node pressure heads of the looped network.
3) To evaluate the cost of pipes in closed loop networks.
4) To test and validate the model.

1.3 Significance of the Study

Analysis of complex networks is obligatory to the water utilities to supply satisfactory quantity of water to customers at adequate pressure and reduced cost. Design and management of a water supply system is reliant on steady state pipe network analysis (Lee, 1983). However, analysing complex network requires more effort since the solutions are not straight forward as compared to simple branched networks. Hence, implementing a user friendly numerical hydraulic model for intricate pipe network analysis can enable water authorities establish new operational strategies and policies to not only cut the operating expenses but also improve reliability and lessen wastage of water (Brock, 1970; Hudson, 1974; Shamir, 1974; Lee, 1983). Additionally, this hydraulic model will mark the beginning of further research in this field, on the continent of Africa, since the source code will herein be presented.

1.4 Scope and Limitation of the Study

This study has been limited to developing a user friendly numerical hydraulic model for analyzing and costing closed loop complex pipe network configuration at Mbale school zone case study area. The numerical model was limited to the analysis of a maximum of four (4) loops. In this study, the head losses considered are due to friction and turbulence due to the presence of valves. The flow rates in the distribution network were determined using modified Hardy Cross method (Epp & Fowler, 1970) under steady-state-conditions. The choice for improved Hardy Cross method was based on three basic criteria; simultaneous solution to flow corrections, reduced iterative procedure, and self-rectification. Improved Hard Cross algorithm solves a classical network problem by simultaneously solving for flow correction factors and iteratively rectifying the errors made in the first guess (Cross, 1936; Brkic, 2011). The developed numerical model solves for head losses due to friction using two commonly applied head loss formulae; Darcy-Weisbach and Hazen-Williams methods. Testing of the model on the case study area network, was performed using Darcy-Weisbach scheme. This was chosen over Hazen-Williams formula because it can compute energy losses for all fluid types with much accuracy. The program also implements Barr (1981) modified equation to calculate friction factor (f). Visual Basic (VB) language was chosen as the programming language since it offers a convenient mode for rapid building of user friendly interfaces. The fast, easy and intuitive codes and tools of VB have turned it into the 'de facto engineering model' for quick programming and development of applications.

1.5 Description of Study Area

The study area is about $2km^2$ traced at about $1^0 04'49.49"N$ and $34^0 10'28"E$ with a pipe network of about 3km within Mbale town water service area along Pallisa Road in Eastern Region of Uganda. The overall land area coverage of Mbale district is $519\ \mathrm{km}^2$ with a pipe network of around $305\mathrm{km}$ extending over a radius of 25km within the water service area and a consumer base of 11,824 connections. Presently, $4{,}200\ \mathrm{m}^3$of water per day is the average demand from the served places. Piped water is supplied to the clients through a delivery network consisting of 300 mm GMS trunk mains which is reduced into 9 inch and 6inch GI pipes along Pallisa road.

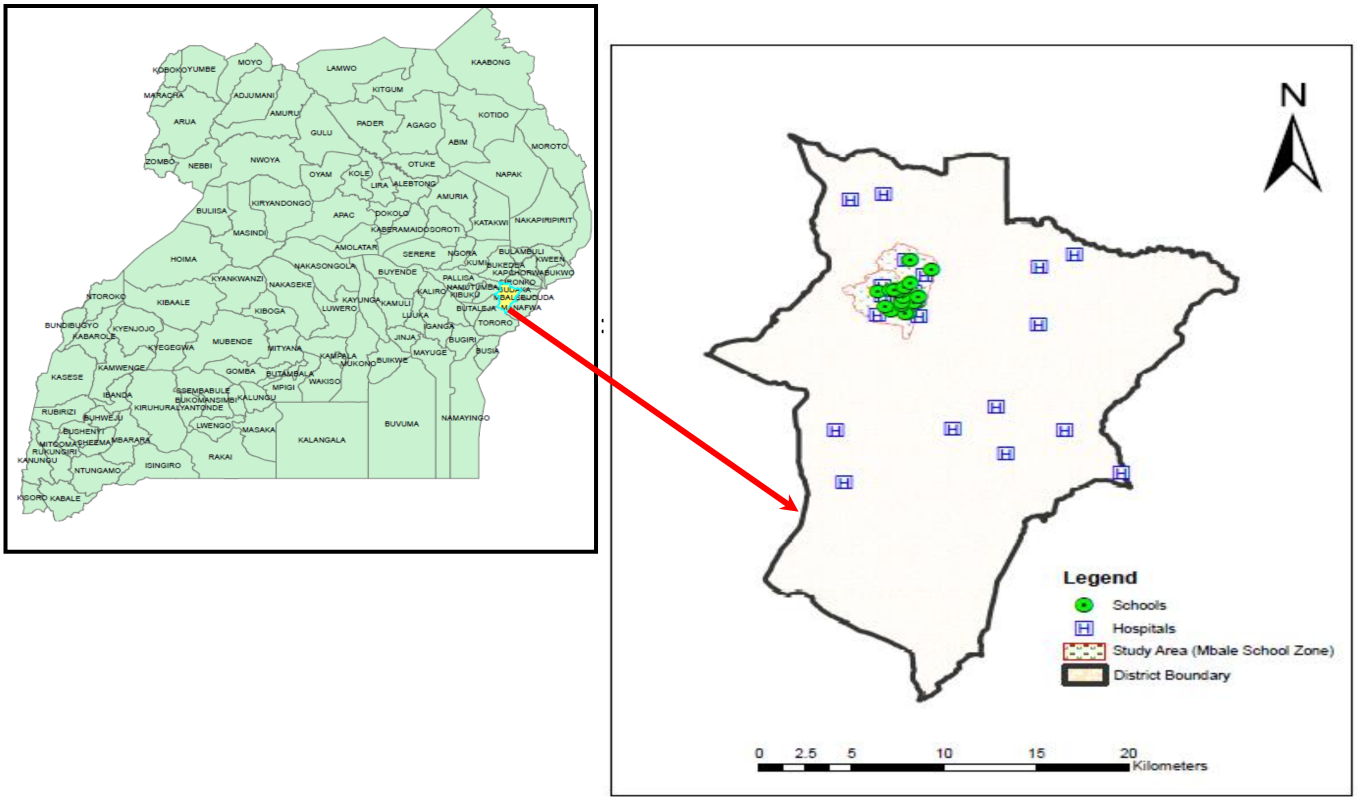


Figure 1-2: Study Area Map

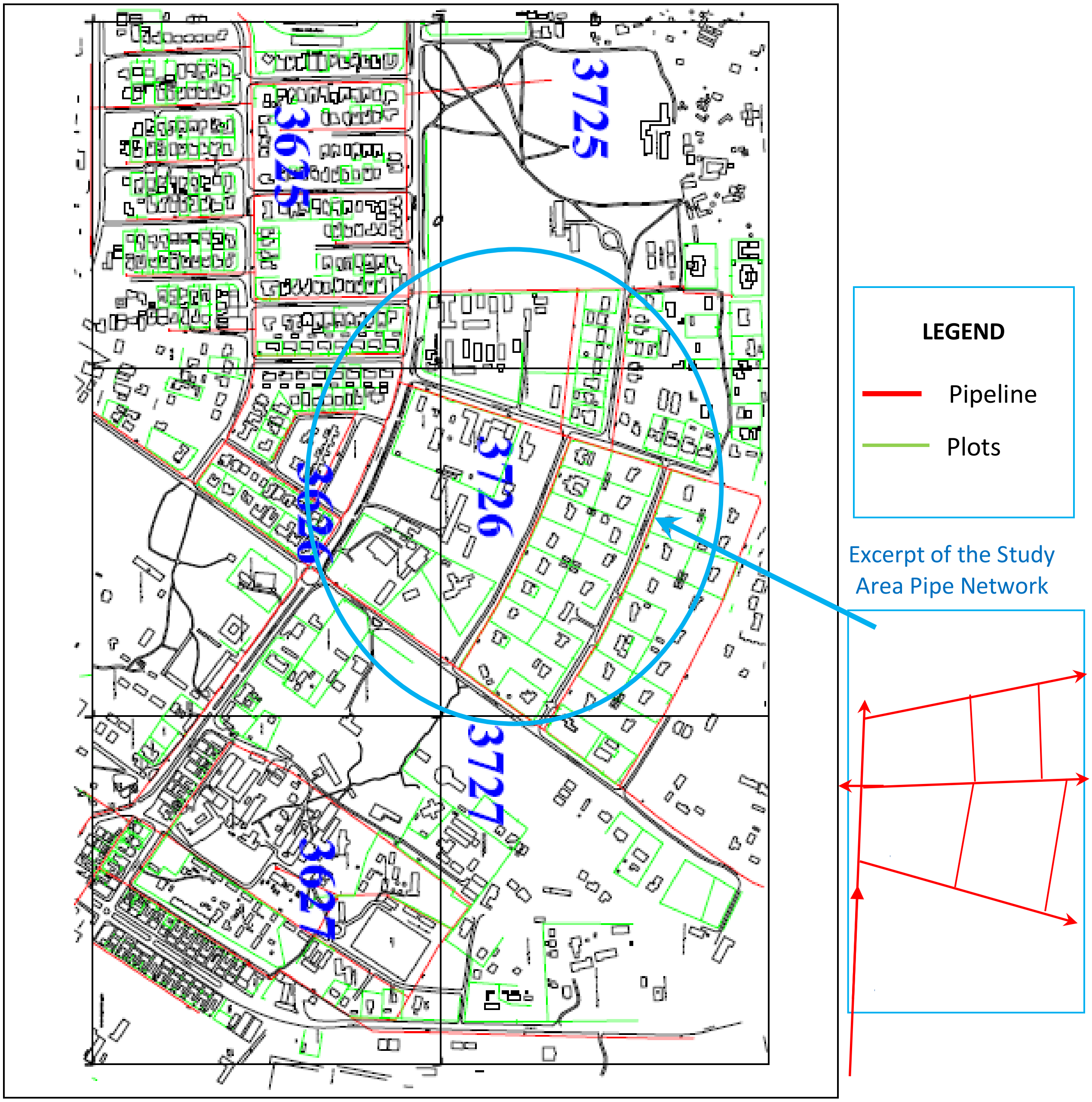


Figure 1-3: Mbale Town Water Network Layout

1.6 Study Approach

This study was undertaken in three (3) phases as follows;

1.6.1 Pre-field work

The activities conducted before the field work itself; problem definition, literature review on which algorithm to implement, the exact data required by the model, Appropriate methodology for Application Development Life Cycle (ADLC), and which validation approach to use. Related works, journal works, and previous studies including discussions with supervisors were all done in this phase.

1.6.2 Field work

This involved collecting field observation data and all the model input data type that were not acquired during the pre-field work but relevant for the study was acquired from Mbale water authorities.

1.6.3 Post-field work

This phase of the research comprises thesis preparation through model development, results analysis and presentation.

1.7 Thesis Outline

This study was set out in six chapters and table (1-1) synopses what is involved in each chapter.

Table 1-1:Thesis Report Outline

Chapter one	Represents the research background information, problem statement, objectives, significance and scope of the study, outline and general structure of the research.
Chapter two	Detailed comprehensive review of the literature related to this study and research gaps identified that justified the research study at hand.
Chapter three	Presents a detailed methodology applied in the model development.
Chapter four	Presents the model description and testing
Chapter Five	Covers a chorological discussion of the results and findings from model testing and validation.
Chapter Six	Represents conclusions and recommendations for future advancement of the model.

Chapter 2 : Literature Review

2.0 Introduction

Hydraulics network analysis provides the basis for the design of new systems and the extension of existing systems (Featherstone & Nalluri, 1995). Design, construction and operation of water distribution networks entail an understanding of the flow in compound systems with the accompanying head losses (Lansey & Mays, 1999). For the past few decades, water services in developing countries have been unreliable due to the use of a tree-type network. One of the reasons for the implementation of tree-like network in most developing countries has always been due to low rate of urbanization. However, today, with the increasing urbanization and the effort to achieve Sustainable Development Goal No. 6 (SDG 6)[4], developing countries like Uganda have shifted from tree-like to looped (Grid) networks especially in densely populated urban areas. Even though looped pipe networks increase the intricacy of determining flows and pressures in the pipe network, it substantially upturns the steadfastness of a water utility. Due to the non-linear correlation between flows and energy losses in pipelines, the analysis of looped pipe networks requires iterative algorithms (Ellis, 2001).

2.1 Reflection on Previous Studies

Earlier analyses of water networks were carried out manually. However, with the arrival of computers, a tremendous change in the way more complex pipelines are analyzed has been realized (Ormsbee, 2006). The research pioneers to publish a computer algorithm that implements Newton-Raphson method in analyzing water network were Martin and Peters (1963). Their algorithm offered a simultaneous solution technique for the original "node" method of Hardy Cross (1936). Later on, Alvin Fowler and Robert Epp in 1970 proposed a new technique commonly known today as "improved Hardy Cross (HC) method" to simultaneously solve for the flow adjustment factors associated with the original (classic) "loop" method of Cross (Brkic, 2011). The modified HC technique was an idea conceived and produced by Epp and Fowler (1970) which is a kind of Newton– Raphson (NR) approach used to simultaneously

[4] SDG 6 emphasizes access to water and sanitation by all.

solve for unknown corrective flows (ΔQ). In 1972, Wood and Charles proposed "the linear method" which first linearizes non-linear energy equations using a Taylor Series expansion. The problem with the earlier developed computer algorithms is that most of them were written in FORTRAN[5] , a non-user friendly program in which the data is keyboarded into the command prompt window (Ahmed, 1997). This requires advanced computer knowledge and skills to correctly use the model. Data input and output is best handled by a Graphical User Interface (GUI). FORTRAN cannot be used to build GUI applications due to absence of GUI development toolkit. (Demir, et al., 2008) built a computer model in Microsoft Excel to run both steady state and time-dependent analyses using the classic HC Algorithm. In their study, they used an improved methodology of obtaining steady state solutions at various instants using the original HC Algorithm and then combining them to produce a time-dependent results. However much accurate their model was proven to be, there are still shortcomings with it. In their model, the user has to assign the assumed initial flow rates for each pipe in the network. There are a number of draw backs with these computer models where the user is required to guess and assign the initial pipe flow rates: Firstly, should the user forget the sign convention while assigning the initial flows, then the whole analysis results would be misleading. Secondly, the user may fail to obey the continuity law while estimating the initial pipe flow rates. Thirdly, it is time wasting and a cumbersome process to estimate the initial flow rates with a hand calculator. (Yengale, et al., 2012) and (Sadafule, et al., 2013), developed optimal loop water distribution system models using the original HC method. However, they focused more on determining discharges and ignored the node pressure heads in their models. Therefore, this study extends the implementation of improved HC Algorithm in VB6.0 to solve for pressure heads at the nodes and flows in a looped network under steady-state conditions. More importantly, the program automatically initializes the pipe flow rates and calculates the base demand once the population or base demand (external flow) data is assigned. The developed model is also capable of evaluating the cost of network pipelines.

Understanding the concepts of flow distribution networks is very vital in the analysis of pipe networks and in development of hydraulic simulation tools. Therefore, the following

[5] FORTRAN means Formula Translation developed by John W. Backus in 1957

subheadings present a review on hydraulic network components and configurations, water demand concept, basic equations for steady-state flow conditions, three (3) key broadly used iterative schemes in hydraulic analyses with cost analysis equation, model validation techniques, brief review of the existing hydraulic simulation tools, computer programing, and lastly, the different program development methodologies.

2.2 Components of Hydraulic Networks

A pipe network is perceived as a network consisting of many elements. Table 2-1 summarizes the various components that make up a water network system.

Table 2-1:Hydraulic Network Components

Components	**Description**
Pipe	A pipe is the principal network element, viewed as a circular closed conduit for supplying water under pressure to the end users.
Pump	A hydraulic device used to increase water pressure within the system.
Storage tank	Used to maintain continuous water supply by storing water during low demand periods and releasing at peak demands.
Node	A connection point where pipes join together within the network.
Valve	Pressure and flow regulator within the network.

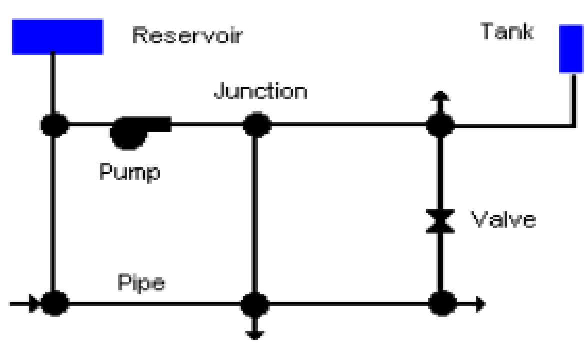


Figure 2-1: Illustrating Water Network Components (Rossman, 2000)

2.3 Water Network Configurations

While designing and/or analysing a water distribution system for a given community, the designer could be faced with the following distinct network problems: branching (tree-like) pattern, grid (looped) arrangement and combined (branching plus grid) system.

2.3.1 Branching/Tree-like pattern

It comprises the main (trunk) line, sub-mains with branches (figure 2-2). In this type of arrangement, no consumer receives water from the trunk. In other words, the consumers receive water from the branches. There are merits and demerits of tree-system which are presented in table 2-2 below.

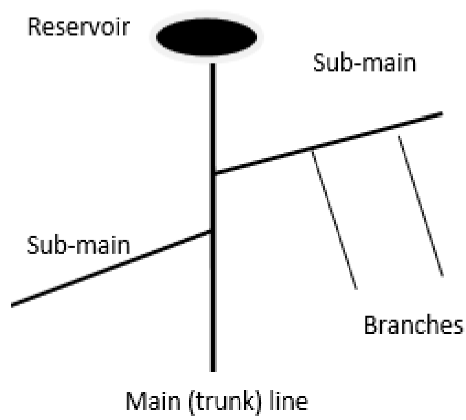


Figure 2-2: Tree System

Table 2-2: Merits and Demerits of Tree system

Advantages	Disadvantages
i. Very simple method of water supply. ii. Comparatively easy computations are done. iii. Relatively few cut-off valves are needed. iv. The needed pipe sizes are economical.	i. Solid deposition and bacterial growth at dead ends. ii. Consumers on a line under repair are without water until completion. iii. Low pressure at the end of the line as more users get connected.

2.3.2 Grid (Looped) Arrangement

In looped arrangements, there are no dead ends. Water reaches a particular user from different directions. Therefore, it is the most reliable and used especially in developed cities.

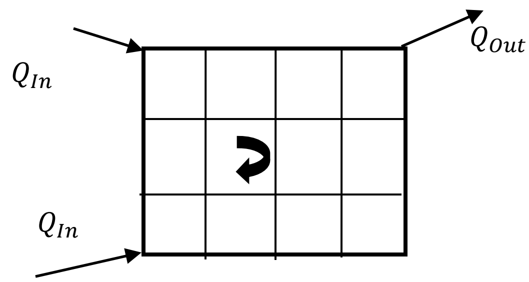


Figure 2-3: Grid (Looped) Pattern

Table 2-3: Merits and Demerits of Grid System

Merits	Demerits
✓ No dead ends ✓ Water reaches a given point from all directions ✓ Reliable pressure for fire-fighting	✓ Comparatively more expensive. ✓ It requires more number of valves. ✓ Determining the pipe sizes is more complex.

2.3.3 Combined System

It is an arrangement of both looped and tree systems (figure 2-4). This kind is extensively used worldwide.

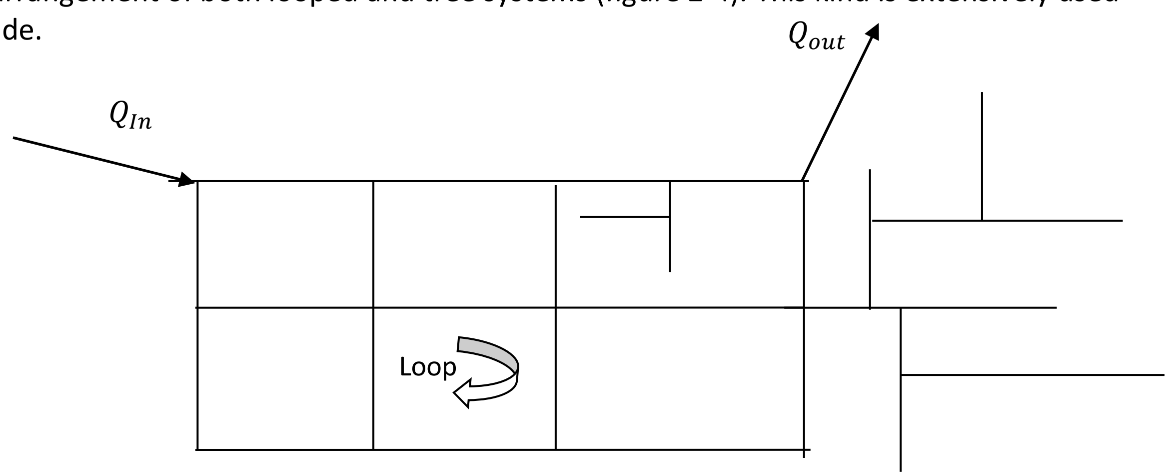


Figure 2-4: Combined (looped and Tree) System

2.4 Water Demand

The Uganda Water Resources Development Authority known as Ministry of Water and Environment (MWE, 2013) stresses that before performing water network design process, the water demand estimation for a considered community must be the number one activity. Indeed, understanding the water demand concept is paramount to designing sustainable water supply systems. As the demand is expected to grow continuously with the years to come, any water supply scheme must be able to accommodate the ever increasing demand (MWE, 2013).

2.4.1 Water Demand Estimation

MWE (2013) maintains that approximating the water demand for a considered proposed area involves:

(i) Estimating the numbers of water users belonging in the various consumer classes at different phases of the design period.

(ii) Defining the average day unit water demand statistics for the given different user groups.

There are generally three classes of consumption considered in designing a water supply system in Uganda (MWE, 2013):

- ✓ domestic (subdivided into high, medium and low income groups);
- ✓ commercial and
- ✓ institutional types.

For each class, demand estimation and projection ought to be separately performed (MWE, 2013). According to the MWE (2013), for the case of the country Uganda, the per capita demand for smaller towns of up to 5, 000 persons is 20 l/day, 35 l/day for intermediate towns up to 20,000 persons and 50 l/day for the bigger towns. From the above given statistical figures, it can clearly be seen that per capita consumption increases along with town size. This is due to higher number of institutions and commercial undertakings in bigger towns as opposed to smaller towns. Table 2-4 presents the per capita demand per consumer class.

Table 2-4: Domestic Water Demand (MWE, 2013)

CONSUMER CLASS	**Rural Area** $(l/ca/d)$	**Urban Area** $(l/ca/d)$	**Comments**
Low Income using kiosks or pubic taps	20	20	Most squatter areas, to be taken as the minimum
Low income multiple household with Yard Tap	40	40	Low income housing with no inside installation.
Low income, single household with Yard Tap	50	50	Low income housing with no inside installation.
Medium Income Household		100	Medium income group housing, with sewer or septic tank.
High Income Household		200	High income group housing, with sewer or septic tank.

Table 2-5: Institutional Water Demand (MWE, 2013)

CONSUMER CLASS	UNIT	RURAL (l/d)	URBAN (l/d)	COMMENT
Day Schools	l/std/d	10	10	Using pit latrine
			20	Using WC
Boarding Schools	l/std/d	50	100	Using WC
Health Care Dispensaries	l/visitor/d	10	50	Out patients only
Health Center 1	l/bed/d	20	50	No modern facilities
Health Center 2	l/bed/d	50	70	With maternity Using pit latrine
Health Center 3	l/bed/d	70	100	With maternity Using pit latrine
Health Center 4	l/bed/d	100	150	With maternity Using WC
Hospital, District	l/bed/d		200	With surgery unit
Hospital Regional Referral	l/bed/d		400	With surgery unit
Administrative Offices	l/worker/d	10	70	Using pit latrine Using WC

Estimation of industrial water demand is a complex process especially in less developed nations where the precise industry type is antecedently unknown within the municipal designated industrial park. When faced with that scenario, MWE (2013) emphasizes that the design engineer should estimate the diurnal water demand per unit area as shown in the table below.

Table 2-6: Industry Water Demand (MWE, 2013)

Industry Type	Water Demand m3/ha/d
Medium Scale (water intensive)	40
Medium Scale (medium water intensive)	15
Small Scale (dry)	5

2.4.2 Water Supply System Design Period

According to MWE (2013) water supply design manual, the design period is defined as "the period within which the long term projected demands for a least cost project are estimated". In simple words, it is that time-period within which water supply is expected to be higher than the demand. The information provided by MWE (2013) states that the optimal design period for a water supply structure is within the range of 5-10 years. It hardly ever goes above 20 years. Generally, the demand forecasts ought to be done for the "initial Year (5 years)", the "Future Year (15 years)" and the "Ultimate Year (25 years)" (MWE, 2013). This can be done by firstly obtaining the future population statistics. The expression used to obtain future population is in form:

$$P_n = P_o(1 + r)^n \qquad \dots (2.1)$$

Where
P_n = population after n years,

P_o = present population, and

r = annual growth rate (%)

The Uganda water supply systems are usually designed for the "Ultimate Year (25)" water demand period. After obtaining the predicted population numerals and determining the per capita demand[6] (L/ca/d) for every consumer group, the average and the maximum daily demand (L/d) are estimated as follows respectively (MWE, 2014):

$$Q_{Av} = P_n * Per\ Capita\ Demand\ (L/ca/d) \qquad + Non - Revenue\ Water\ \ (NRW) \qquad \dots (2.2)$$

Where; Q_{Av} =Average day water demand (L/d) and P_n = Projected population.

[6] Per capita demand refers to amount of water allocated to each individual per day units of measurement being liters per capita per day (L/ca/d)

To compute the maximum demand (Q_{Max}), the peak factor must be incorporated into the formula. Therefore, equation (2.2) becomes:

$$Q_{Max} = Q_{Av} * Peak\ Factor(PF) \qquad \dots (2.3)$$

Where the peak factor is influenced by the population.

2.4.3 The Non-Revenue Water Concept

Designing a sustainable water supply network requires engineers and scientists to capture within the design the Non-Revenue Water (NRW). NRW refers to the water volume introduced into the distribution network pipelines in a year minus (-) the billed authorised consumption (Shilehwa, 2013; Juan, 2008). The NRW is made up of the subsequent elements (MWE, 2013; Shilehwa, 2013; Allan, 2010):

- ✓ Unbilled authorized consumption (for instance water used for firefighting and community/religious functions)
- ✓ Apparent losses (these include water theft, meter errors, unmetered public use, and unbilled metered water),
- ✓ Real/physical losses (the annual amount of water lost from transmission mains, storage facilities, distribution mains or service connections) through leaks.

In the year 2003, the International Water Association (IWA) published a broadly accepted practical methodology for assessing NRW and its constituents (MWE, 2013; Allan, 2010; Farley & Trow, 2003). According to IWA, NRW is mathematically described as:

$$NRW = System\ Input\ Volume\ (SIV) - Billed\ Authorized\ Consumption\ \ (BAC) \quad \dots (2.4)$$

This water balance is broadly accepted to determine the amount of water lost in a water distribution scheme. However, Alan (2010) disagreed with this water balance rationale, asserting that its applicability is only more logical in the developed nations. His reasoning was that in technologically advanced world, a bigger portion of revenue for all billed water is collected unlike in most underdeveloped nations. He therefore maintains that in less developed countries, considering only the paid volume of water is very significant. Eventually, he proposed some modification to the IWA (2003) water balance to give a correct representation in the less developed nations:

$$NRW = SIV - \textit{Paid For Billed Authorized Consumption } (PBAC). \quad \dots (2.5)$$

The most widely used yardstick to measure NRW is the percentage (%) of NRW as a segment of system input volume (MWE, 2013).

$$NRW(\%) = (\textit{NRW Volume}/\textit{System Input Volume})\ x\ 100 \quad \dots (2.6)$$

A plethora of water utilities use this percentage for performance benchmarking although, Alan (2010) affirms that this may be deceptive when adopted to describe actual performance trends over a long period. This is logically applicable for a constant consumption pattern which is normally not the case. Consumption will always vary occasionally depending on a number of reasons such as augmentation in water tariff. In the year 2005, the average NRW (%) in Uganda was 31% (MWE, 2013).

2.4.4 Peak Factor Calculation

(Harmon, 1918) and (Babbitt, 1928) were the first to propose techniques that describe the relationship between peak factor and population as follows respectively (Balacco, et al., 2017):

$$PF = \frac{18 + \sqrt{P/_{1000}}}{4 + \sqrt{P/_{1000}}} \quad \dots (2.7)$$

$$PF = 20. P^{-0.2} \quad \dots (2.8)$$

In 2005, the Australian ENVC (Environment and Conservation Department) applied the Harmon formula to estimate peak flow rates for communities that do not have adequate water consumption pattern needed to compute average and maximum flow despite being an ancient formula (Balacco, et al., 2017).

2.5 Pipe Network Analysis

A hydraulic network analysis model is a very important decision making tool for assessing the sufficiency of a pipe network. The solution to the steady-state flow network problem is directed by two basic hydraulic principles: (1) the conservation of mass at nodes; and (2) the conservation of energy around the loops (Lee, 1983). The conservation of mass at nodes uses

linear algebraic equations while the energy conservation around the closed loops is based on non-linear equations written in terms of flow rate. The non-linear equations require special solution techniques with rigorous iterative steps.
Studying complex pipe network involves undertaking the method of approach stated below.

Step 1: Defining pipe properties (length, diameter, roughness coefficient) and node elevation.
Step 2: Devising of non-linear solution equations.
Step 3: Identifying iterative method of analysis.
Step 4: Convergence criteria.

2.6 Basic Equations for Steady-State Flow Conditions

Analysis of steady-state flow network problem has been of great significance in water engineering. Fluid flow in a pipeline is said to be steady-state when velocity at any given point does not change in magnitude or direction with time. The steady-state flow network problem is solved for pressure at nodes and flow distributions in pipe. In order for the pressure heads at nodes to be exceptionally established, a fixed hydraulic energy must exist at one of the locations.

2.6.1 The Bernoulli's (Energy) Equation

The energy equation states that; "for an ideal incompressible fluid flow under steady –state condition, summation of pressure, kinetic, and potential (elevation) heads is constant along a streamline". To achieve Bernoulli's equation, Euler's equation for steady-state flow is integrated (Rajput, 2008) as shown below:

From Euler's differential equation of flow;

$$\frac{dP}{\rho} + V.dV + g.dZ = 0 \qquad \dots (2.9)$$

By integrating equation (1), we shall have;

$$\frac{1}{\rho}\int dp + \int V.dV + \int g.dZ = Constant \qquad \dots(2.10)$$

$$\frac{P}{\rho} + \frac{V^2}{2} + gz = Constant \qquad \dots(2.11)$$

Divide by g, we get;

$$\frac{P}{\rho g} + \frac{V^2}{2g} + z = Constant \qquad \dots(2.12)$$

Or

$$\frac{P}{w} + \frac{V^2}{2g} + z = Constant \qquad \dots(2.13)$$

Where;

$$\frac{P}{\rho g} = Pressure\ head$$

$$\frac{V^2}{2g} = Velocity\ head$$

$$z = Elevation\ (Potential) head$$

2.6.1.1 Pressure

(ACF, 2008) define water pressure as *"the force exerted by water against the container walls it occupies such as pipe's walls, reservoir's wall etc."*

The pressure at a particular point is equal to the weight of water column above the given point. The weight of water column above a considered point is commonly expressed as (ACF, 2008):

Water column weight $(\mathrm{w}) = water\ density(\rho)\ x\ water\ column\ height(h)$ ... (2.14)

$= 1\mathrm{g/cm^3}$ x water column height (cm)

$=$ pressure at the considered point $(\mathrm{g/cm^2})$

So, we get:

$$Pressure\ \left(\frac{g}{cm^2}\right) = 1\frac{g}{cm^3}\ x\ water\ column\ height\ (cm) \qquad \dots(2.15)$$

= water column height (cm)

The pressure exerted by water at the bottom of a water column depends only on the height of water column.

The SI units of pressure are: $\mathrm{kg/cm^2}$, bar or “metres water gauge”:

$$1\ \mathrm{kg/cm}^{\ 2} = 1\ \mathrm{bar} = 1\ \mathrm{mWG}$$

Technically, the pressure unit, mWG, is commonly used for the hydraulic calculations applied in sizing a water distribution network

2.6.1.2 Static Pressure Vs Dynamic Pressure

Distinguishing between static pressure and dynamic pressure is very paramount to the vivid understanding of pressure dynamics in water supply systems.

“The static pressure is the force exerted by water on the pipes walls when all taps are turned off (that is to say when water is at rest in the conduits) whereas the dynamic pressure is the force exerted by water on the pipe walls when one (1) or several taps are open (that is to say when water is flowing in the duct) “ (ACF, 2008).

2.6.1.2.1 Static Pressure

Static pressure literally means the maximum pressure that occurs within the pipelines. It is equivalent to the height difference between the uppermost point of the pipe and the considered point, with the uppermost point being the water’s free surface in either the break pressure tank (BPT) or the reservoir. For illustrative purposes, let’s consider figure 2-5 below.

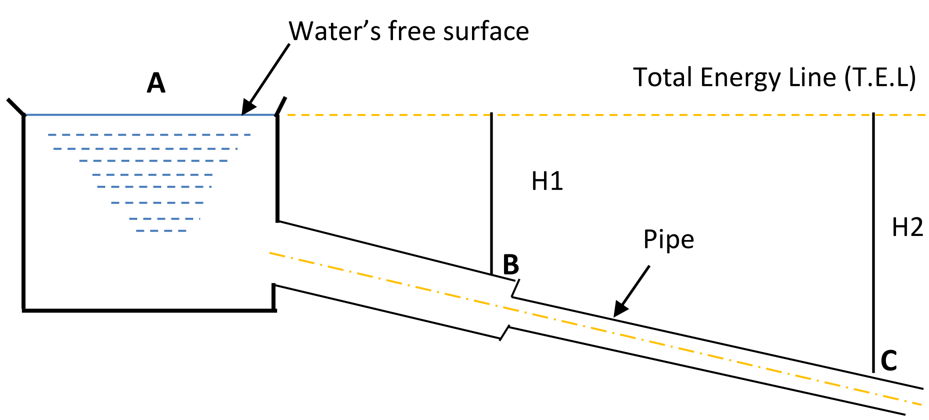


Figure 2-5: Illustrating Determination of Pipeline Static Pressure (Bansal, 2010)

$P_{static}\ (mWG) = \ H\ (m)$ ... (2.16)

- The pressure exerted by water in the pipe at the point B = the height H1 (in meters).
- The pressure exerted by water in the pipe at the point C = the height H2 (in meters).

Static pressure is used as the basis for defining the pressure the pipe must resist. It as well helps to determine whether or not to mount pressure breaking appurtenances to safeguard the pipe. A certain pressure, namely Nominal Pressure (NP), must be maintained within the water distribution pipelines. If the pressure in the pipe exceed NP, then the risk of rupture is inevitable. The different pipe sizes plus the empirical NP range for pipes commonly used in distribution networks are presented in table 2-5 and table 2-6 respectively.

Table 2-7: Different Pipe Sizes Commonly Used in Uganda (MWE, 2013)

Pipe Type	**Size (mm)**
uPVC	63, 90, 110, 160, 200, 250, 315,400
PE	20, 25, 32, 40, 63, 75, 90, 110, 125, 140
GS and Steel	15, 20, 25, 32, 40, 50, 65, 80, 100, 150, 200, 250, 300

Table 2-8: Pipes Pressure Level (NP) (ACF, 2008*)*

Pipe Type	**Nominal Pressure (NP)**	**Maximum Pressure (P_{Static})**
Plastic pipe (PVC or PE)	NP 6	60 meters
	NP 10	100 meters
	NP 16	160 meters
Galvanised Iron (GI)	NP 16	160 meters
	NP 25	250 meters

2.6.1.2.2 Dynamic pressure

The dynamic pressure is the pressure exerted by water against the pipe walls as it flows, (that is to say when the taps are open, and the pipes are full of water). The dynamic pressure is less than the static pressure (see figure 2-6) due to the established fact that when water runs in pipes, energy is lost. Dynamic pressure can be mathematically expressed as:

$P_{Dynamic} = P_{Static} - Head\ Loss$ ... (2.17)

But;

$P_{Static} = H_{Stat}\ (m)$ ... (2.18)

Therefore;

$$P_{Dynamic} = H_{Stat}\ (m) - \Delta H\ (m) \qquad \dots (2.19)$$

The pressure heads at the network nodes can also be computed on the premise that pressure head elevation at the inlet node is known as follows:

Pressure heads at node i = Known Head-Elevation at the network Inlet node –Elevation at node i– Head Loss in pipe i of loop j

$$P_i = HE_{inlet} - E_i - hL_{i,j} \qquad \dots (2.20)$$

2.6.1.2.3 Piezometric Lines

The piezometric line permits envisaging the advancement of the water pressure all along the conduit. In fact, it represents the height water would attain in a perpendicular pipe inserted into the pipeline. If we sketch the pressure line when water is flowing, we end up with the dynamic pressure profile as illustrated in figure 2-6 below.

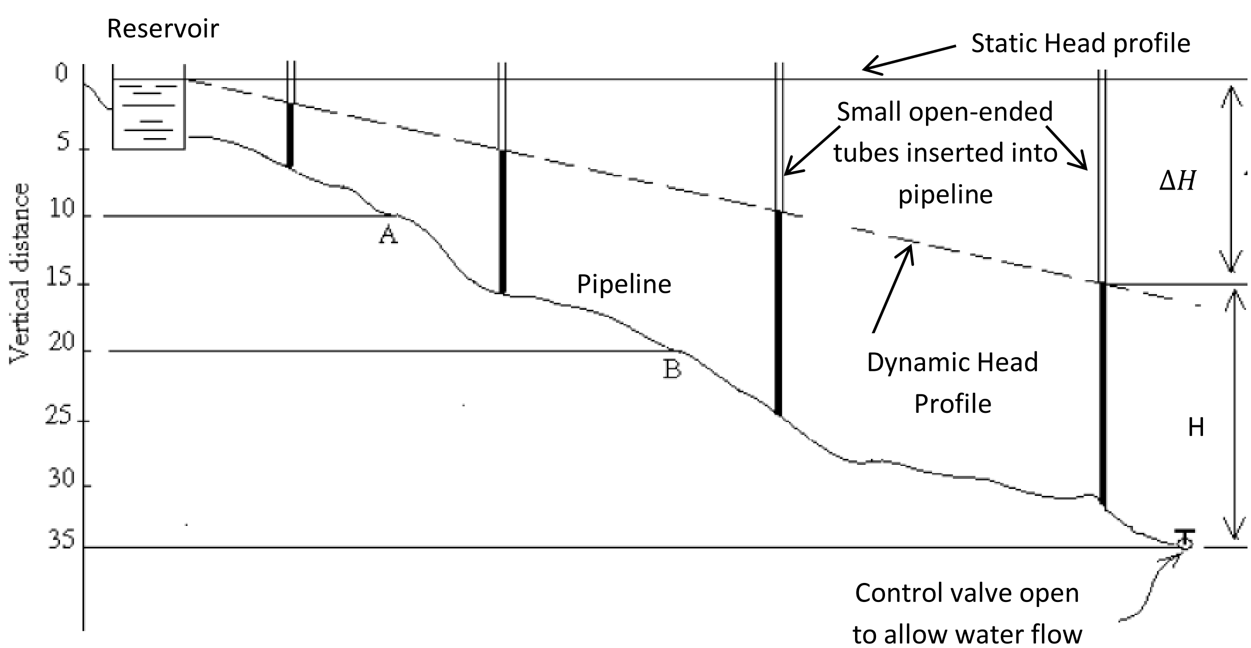


Figure 2-6: Static and Dynamic Piezometric Levels (ACF, 2008)

According to MWE(2013) water supply design guidelines, the recommended pressure ranges during peak hourly flow in the network are:

Table 2-9: Recommended Pressure in the Distribution Network (MWE,2013)

Main Parameter	Sub Parameter	Unit	Range	Recommended
Distribution Network	Minimum Pressure	bar	1.5-2.0	2.0
	Maximum Pressure (Static pressure)	bar	4.5-6.0	6.0

2.6.2 The Head Loss Equations

When the fluid flows through a pipe, it undergoes some opposition to its movement and as a result, its flow energy is reduced. This loss of head can be categorized as (Bansal, 2007) presented in figure 2-7 below:

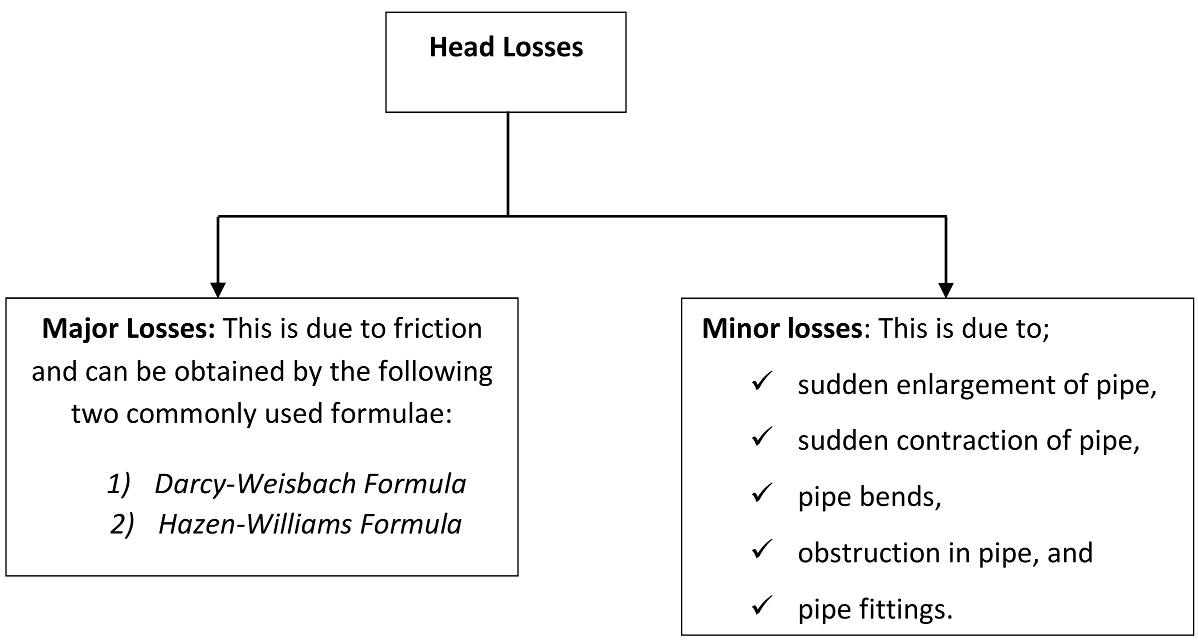


Figure 2-7: Classification of Energy Losses in a Pipeline

2.6.2.1 Major Energy Losses

The correlation between major energy losses and pipe flow along network pipelines is non-linear. The general relationship between flow and head losses due to friction in closed conduits is usually modeled in the following form:

$h_{fj} = K_j Q_j |Q_j|^{n-1}$ For all pipes j=1… N … (2.21)

Where;

$h_{fj} =$ Head loss due to friction in pipe j;

Q_j= Flow rate in pipe j;

K_j= Coefficient for pipe j; which is a function of pipes diameter, length, and material and

n is a constant in the range of 2 (Lansey & Mays, 1999).

For the Darcy-Weisbach Head Loss Equation, n = 2 (Lee, 1983)

Therefore, substituting for n=2 into equation (2.21) above, we shall end up with;

$$h_{fj} = K_j {Q_j}^2 \quad \dots (2.22)$$

Where;

$$K_j = \frac{f_j L_j}{D_j} \frac{1}{2gA_j^2} (For\ S.I\ Units) \quad \dots (2.23)$$

Where;

f_j = Darcy-Weisbach's friction factor for pipe j

L_j =the length of pipe j;

D_j =the diameter of pipe j.

The friction factor f_j has been appraised experimentally for several pipes and the results have demonstrated that f_j is dependent on pipe diameter, roughness, and Reynolds numberR_e. *Also* f_j has been proposed to be time dependent because with time, roughness may vary either due to deposition of solid particles or organic growths. Surface roughness and pipe diameter may also show a discrepancy due to permissible manufacturing allowance. The idea being communicated is that it is impossible to correctly determine the friction factor of any pipe. A designer is expected to use good engineering judgment in choosing a design value for f_j so that proper tolerance is made for these factors. The practical correlation of f_j with roughness, diameter ***d***, and R_e has quite been thoroughly studied (Jeppson, 1976). Nikuradse[7] (1933) and Colebrook (1939) were the first researchers to conduct experimental work, studying the correlation between f_j with roughness, diameter ***d***, and R_e (Larock, et al., 2000). In fact, the Moody Chart (1944) figure 2-8 below was founded on their work (Khamkham, 2000)

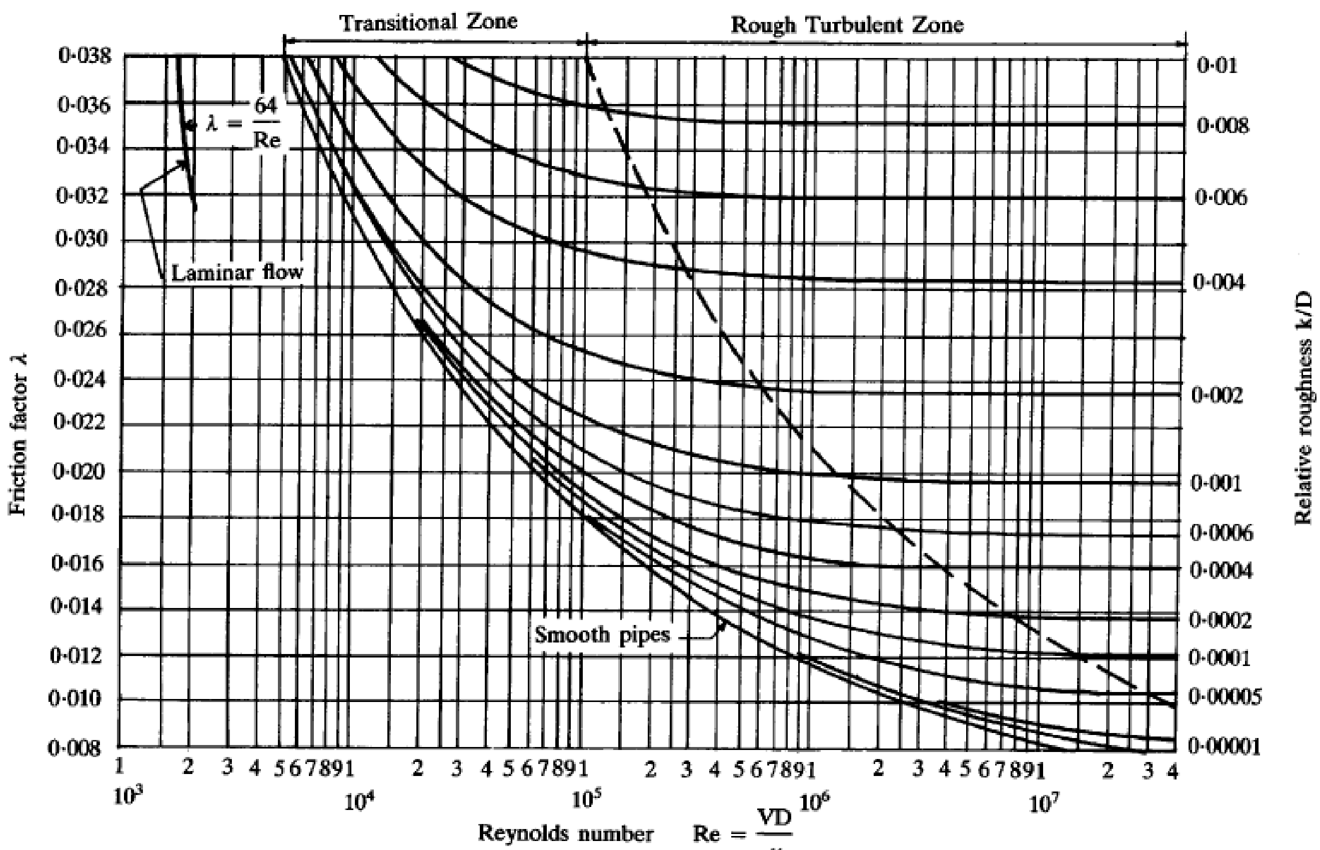


Figure 2-8: The Moody Chart (Featherstone & Nalluri, 1995)

Nikuradse (1933) used pipes that were roughened with uniform roughness and couldn't be practically realistic to use for commercial pipes undergoing turbulence (Khamkham, 2000)

[7] Johann Nikuradse (November 20, 1894- July 18, 1979) Georgia-born German Engineer and Physicist; PhD student (1920) of Ludwig Prandtl

However, observations by others, especially Colebrook (1937), revealed that at large R_e *and* large wall roughness*,* flows in commercial pipes can turn out to be independent of Reynolds number, ***R***e. Therefore, it is possible to calculate the relative roughness $^{\varepsilon}/_{D}$ for commercial pipes from the investigational formula Nikuradse (1933) verified for his entirely-rough pipes eq. (2.25) (Khamkham, 2000).

In case of Laminar flow ($R_e < 2000$), head loss (h_f) can be theoretically found using Hagen-Pouiseuille formula as (Featherstone & Nalluri, 1995):

$$h_f = \frac{32L\mu V}{2g\rho D^2} \qquad \text{... (2.24)}$$

The equation for entirely- rough ducts (White, 1998) is:

$$\frac{1}{\sqrt{f_j}} = -2log_{10}\left(^{\varepsilon_j}/_{D_j}\right) + 1.1364 \qquad \text{... (2.25)}$$

Ludwig Prandtl[8] and Von Kármán[9] suggested the friction factor equation for smooth-or rough-wall ducts, correspondingly as follows:

Smooth-wall ducts: $\frac{1}{\sqrt{f_j}} = 2.0log_{10}\left(\frac{R_{ej}\sqrt{f_j}}{2.51}\right)$ ... (2.26)

Rough-wall pipes: $\frac{1}{\sqrt{f_j}} = 2.0log_{10}\left(\frac{3.7D_j}{\varepsilon_j}\right)$ ... (2.27)

Colebrook and White (1939) combined the smooth wall Eq. (2.26) and rough-wall Eq. (2.27) into an iterative formula commonly known as Colebrook-White equation (2.28).

$$\frac{1}{\sqrt{f_j}} = -2.0log_{10}\left(\frac{\varepsilon_j}{3.7D_j} + \frac{2.51}{R_{ej}\sqrt{f_j}}\right) \qquad \text{... (2.28)}$$

The implicit nature of Colebrook-white formula makes solving for f_j quite problematic. After realizing glitches with the Colebrok-White transition formula, several researchers developed explicit formulae for approximating Colebrook-white equation:

Blasius[10] (1911) proposed the following explicit solution to imprecise f_j in a restricted range of $4x10^3 < R_{ej} < 1x10^5$ (White, 1998):

---

[8] Ludwig Prandtl (1875-1953), German engineer who introduced boundary-layer theory.

[9] Theodore von Kármán (1881-1963); Hungarian mathematician and aeronautical engineer; gave his name to the double row of vortices shed from a 2-d bluff body and now known as a Kármán vortex street.

[10] Heinrich Blasius, a student of Ludwig Prandtl

$$f_j = \frac{0.316}{R_{ej}^{0.25}} \qquad \ldots (2.29)$$

Moody (1944) proposed a simple formula to approximate the Colebrook-while transition formula to eliminate oppositions to its use. His formula has been reported (Featherstone & Nalluri, 1995) to give f_j values in the range $(4x10^3 < R_{ej} < 1x10^7)$ with the discrepancy of $\pm 5\%$:

$$f_j = 0.0055\left[1 + \left(\frac{20x10^3\varepsilon_j}{D_j} + \frac{10^6}{R_e}\right)^{1/3}\right] \qquad \ldots (2.30)$$

Swamee-Jain (1976) equation (implemented in EPANET) explicitly estimates friction factor (f_j) for a circular pipe under full flow conditions: The formula is more suitable for the Reynolds number in the range of $5x10^3$ and $1x10^5$

$$f_j = \frac{0.25}{\left[log\left(\frac{\varepsilon_j/D_j}{3.7} + \frac{5.74}{R_{ej}{}^{0.9}}\right)\right]^2} \qquad \ldots (2.31)$$

Barr (1975) suggested the following formula to explicitly approximate the Colebrook-White function for the turbulent flow (Featherstone & Nalluri, 1995):

$$f_j = \frac{1}{\left[-2log\left(\frac{\varepsilon_j/D_j}{3.7} + \frac{5.1286}{R_{ej}{}^{0.89}}\right)\right]^2} \qquad \ldots (2.32)$$

Barr further improved his formula in 1981 to give a more accurate approximation (Featherstone & Nalluri, 1995). The following modified Barr (1981) equation gives f_j values with the error of $\pm 0.04\%$:

$$\frac{1}{\sqrt{f_j}} = -2log_{10}\left(\frac{\varepsilon_j}{3.7D_j} + \frac{4.518\log\left(\frac{R_{ej}}{7}\right)}{R_{ej}\left(1 + \frac{R_{ej}^{0.52}}{29}\left(\frac{\varepsilon_j}{D_j}\right)^{0.7}\right)}\right) \qquad \ldots (2.33)$$

The Darcy-Weisbach friction factor (f_j) can also be explicitly approximated by Haaland Formula (1983). This works for Reynolds number greater than 4000:

$$\frac{1}{\sqrt{f_j}} = -1.8log_{10}\left(\left(\frac{\varepsilon_j}{3.7D_j}\right)^{1.11} + \frac{6.9}{R_{ej}}\right) \qquad \dots(2.34)$$

From the test results, the recommended effective roughness for commercial pipes is as shown in table 2-9 below.

Table 2-10: Values of Effective Roughness (ε) for Commercial Pipes *(White, 1998)*

Material	***Conditions***	***ε (ft)***	***ε (mm)***	***Tolerance, %***
Steel	Sheet metal, new	0.00016	0.05	± 60
	Stainless, new	0.000007	0.002	$\pm$ 50
	Commercial, new	0.00015	0.046	$\pm$ 30
	Riveted	0.01	3.0	$\pm$ 70
	Rusted	0.007	2.0	$\pm$ 50
Iron	Cast, new	0.00085	0.26	$\pm$50
	Wrought, new	0.00015	0.046	$\pm$20
	Galvanized, new	0.0005	0.15	$\pm$ 40
	Asphalted cast	0.0004	0.12	$\pm$ 50
Brass	Drawn, new	0.000007	0.002	$\pm$50
Plastic	Drawn tubing	0.000005	0.0015	$\pm$ 60
Glass	—	Smooth	Smooth	
Concrete	Smoothed	0.00013	0.04	$\pm$ 60
	Rough	0.007	2.0	$\pm$50
Rubber	Smoothed	0.000033	0.01	$\pm$60

For Hazen Williams; n = 1.852 (Lee, 1983).

Substituting for n= 1.85 into equation (2.21) yields;

$$h_{fj} = K_j Q_j^{\ 1.852} \qquad \dots(2.35)$$

Where;

$$K_j = L_j \frac{1}{\left(0.278 C_j D_j^{\,2.63}\right)^{1.852}} (for\ S.I\ Units) \qquad \dots (2.36)$$

Where;

C_j = Hazen-Williams discharge coefficient;

D_j =the diameter of pipe j;

L_j =the length of pipe j.

Table 2-11: H-W Friction Coefficient, C_{hw} for Common Pipes Material *(Jeppson, 1976)*

Pipe description	**H-W Friction coefficients (C_{hw})**
Polyvinyl Chloride (PVC) pipe	150
Very smooth pipe	140
New cast iron or welded steel	130
Wood, concrete	120
Clay, new riveted steel	110
Old cast iron, brick	100
Badly corroded cast iron or steel	80

2.6.2.2 Minor Energy Losses

The empirical proof reveals that the Head loss due to induced turbulence or secondary flow due to fittings, valves, meters and other elements in a network, will be roughly proportional to the velocity squared or the discharge squared. Minor losses are generally expressed in the form (Lee, 1983):

$$h_{LM} = K_m * Q^2 \qquad \dots (2.37)$$

In which;

$$K_m = {}^{M}\!/_{(2gA^2)} \qquad \dots (2.38)$$

The values of M for different common appurtenances are presented in Table 2-11 (Jeppson, 1976; Lee, 1983).

Table 2-12: Local Loss Coefficient Values, M, for various Common Appurtenances *(Lee, 1983)*

Appurtenances	**Loss Coefficient (M)**
Globe Valve (fully open)	10
Gate Valve (fully open)	0.19
Gate Valve (3/4 open)	1.0
Gate Valve (1/2 open)	5.6
Angle Valve (fully open)	5
Ball Check Valve (fully open)	70
Foot Valve (fully open)	15
Swing Check Valve (fully open)	2.3
45^0 Elbow	0.4

It should be definitely noted that for the analysis of reasonably long pipelines, minor losses can be ignored. Nevertheless, in short pipelines, they may profoundly offer a significant effect on the flow rate especially if a valve is partially closed.

2.6.3 The Continuity Equation

For an incompressible fluid flowing towards the network junction node, the mass conservation principle states that the algebraic sum of flow at each node is equivalent to zero. The general expression for flow continuity at junction node i is:

$$\bar{Q} = q_i - \sum_{j=1}^{NP} Q_j = 0 \quad (i = 1 \dots n) \qquad \dots (2.39)$$

Where;

Q_j = The flow in pipe j connected to node, i

q_i = The demand at the node;

NP = The number of pipes at the node;

NN = The total number of nodes in the network;

$\bar{Q}$ = The vector of unknown flows in all pipes in the system

NB: A sign convention is adopted where flow away from the node is positive

2.6.4 Systems of Equations for Analysing Pipe Networks

There are basically three dissimilar systems of equations applied in solving flow network problems under steady flow conditions. These include; *Q-equations, H-equations,* Δ*Q-equations.* Each of these three will be detailed individually in the proceeding subheadings.

2.6.4.1 The Q-Equations

The Q-Approach Involves solving for flow rates in pipes as the principal unknowns (Q_p) (Khamkham, 2000; Larock, et al., 2000). Two basic principles (continuity and work-energy) have been governing the analysis of discharge in pipe networks. For continuity to be satisfied, flow rate into a junction node must equal flow rate out of the junction node.

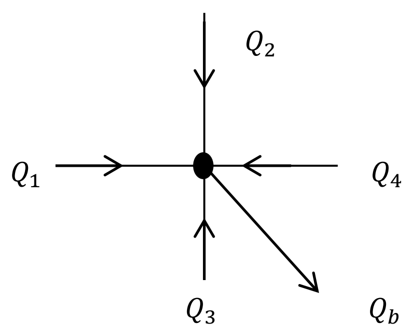


Figure 2-9: Flow of Water In and Out of the Junction Node

At each of the NN junction nodes, continuity expression is formulated as;

$$\sum_{n=1}^{Pj} Q_{nj} = q_{nj} \; (j = 1,2, \ldots NN) \qquad \ldots (2.40)$$

Where

Q_{nj} = Flow rate into node j from pipe n;

q_{nj} = Base demand at node j;

Additional equations can be formulated for work-energy principle that must also be satisfied. These equations are realized by totaling the energy losses along both real and pseudo loops to yield independent equations (Khamkham, 2000; Larock, et al., 2000).

$$\sum_{n=1}^{PL} h_{nL} - \sum_{n=1}^{Pm} h_{pkL} = dh_L \quad (\mathrm{L} = 1, 2 \; \ldots \; \mathrm{N}) \qquad \ldots (2.41)$$

Where;

h_{nL} = Head loss in pipe n in loop L;

h_{pkL} = Head supplied by pump k in loop L;

dh_L= Change in head between the nodes at the beginning and end of loop *L;*

Pm =Number of pumps in Loop L

PL =Number of Pipes in Loop L;

2.6.4.2 The H-Equations

The H-Method involves resolving for Heads at junctions as unknowns (H_j). If we initially consider the elevation of the energy line or hydraulic grade line all through a network as the fundamental set of unknown variables, then a set of *H*-equations can be derived and resolved (Khamkham, 2000; Larock, et al., 2000).

Deriving the set of *H*-equations, involves resolving the exponential equation for the flow rate in the arrangement (Khamkham, 2000; Larock, et al., 2000):

$$Q_{ij} = \left({hf_{ij}}/{K_{ij}}\right)^{1/n_{ij}} = \left[{(H_i - H_j)}/{K_{ij}}\right]^{1/n_{ij}} \quad \dots (2.42)$$

Where;

Q_{ij} *and* K_{ij}, Connote flow rate and loss coefficient for the pipe from node i to node, j.

Now, notice that the energy loss due to friction is substituted by the difference in Hydraulic Grade Line (*HGL)* values between the upstream and downstream nodes (Khamkham, 2000).

Substituting eq. (2.42) into continuity eq. (2.40) yields;

$$q_{nj} - \sum\left\{\left[{(H_i - H_j)}/{K_{ij}}\right]^{1/n_{ij}}\right\}_{in} + \sum\left\{\left[{(H_i - H_j)}/{K_{ij}}\right]^{1/n_{ij}}\right\}_{out} = 0 \quad \dots (2.43)$$

In which the summations are over all pipes that flow to and from junction *j*, respectively.

2.6.4.3 The ΔQ- Equation

These equations regard the loop corrective discharges or **ΔQ**'s as the prime unknowns. These equations can be expressed in the following form for each loop and path (Khamkham, 2000; Larock, et al., 2000).

$$\sum \mathrm{K_i} \left\{\mathrm{Q_{oi}} \pm \sum \Delta \mathrm{Q_k}\right\}^{\mathrm{n_i}} = 0 \qquad \dots (2.44)$$

2.5 Pipe Network Solution Approaches

There are different analysis techniques offered to compute discharges and pressures or head losses all through the pipe network. In this chapter, we shall review three widely applied solution techniques in water network and these include; Newton-Raphson, Linear Theory and Hardy Cross.

2.5.1 Newton-Raphson (NR) Method

NR scheme is a legendary method found in most of the mathematics textbooks of numerical analysis. The practical application of NR technique is seen in solving simple and intricate water supply systems. It is said to have a "quadratic convergence" compared to other iterative schemes which exhibit a linear convergence (Gerald & Wheatley, 2004; Lee, 1983). Just like Cross's (1936) method, NR method requires an initial assumption of unknown variables or a reference point. The choice of an initial guess is so relevant in determining the speed of convergence of NR scheme. The NR expression can be formulated either from the graph or Tailor series expansion theorem. For simplicity, we shall adapt the graphical method for its derivation.

2.5.1.1 Proof of NR Scheme

Let's give thought to figure 2-10 below. If x_i is the initial estimate that is near to the root of the function, $f(x) = 0$, drawing a tangent to the curve at, $f(x_i)$, then point x_{i+1} where the tangent intersects with the x-axis would become the next approximation.

The gradient of the curve will be given by the gradient of line tangent to the curve.

$$Gradient = tan\emptyset_i = f'(x_i) = \left[{}^{f(x_i) - 0}\big/_{x_i - x_{i+1}}\right] \qquad \dots (2.45)$$

Resolving eq. (2.42) generates

$$x_{i+1} = x_i - \frac{f(x_i)}{f'(x_i)} \qquad \textit{which is NR Formula} \qquad \dots(2.46)$$

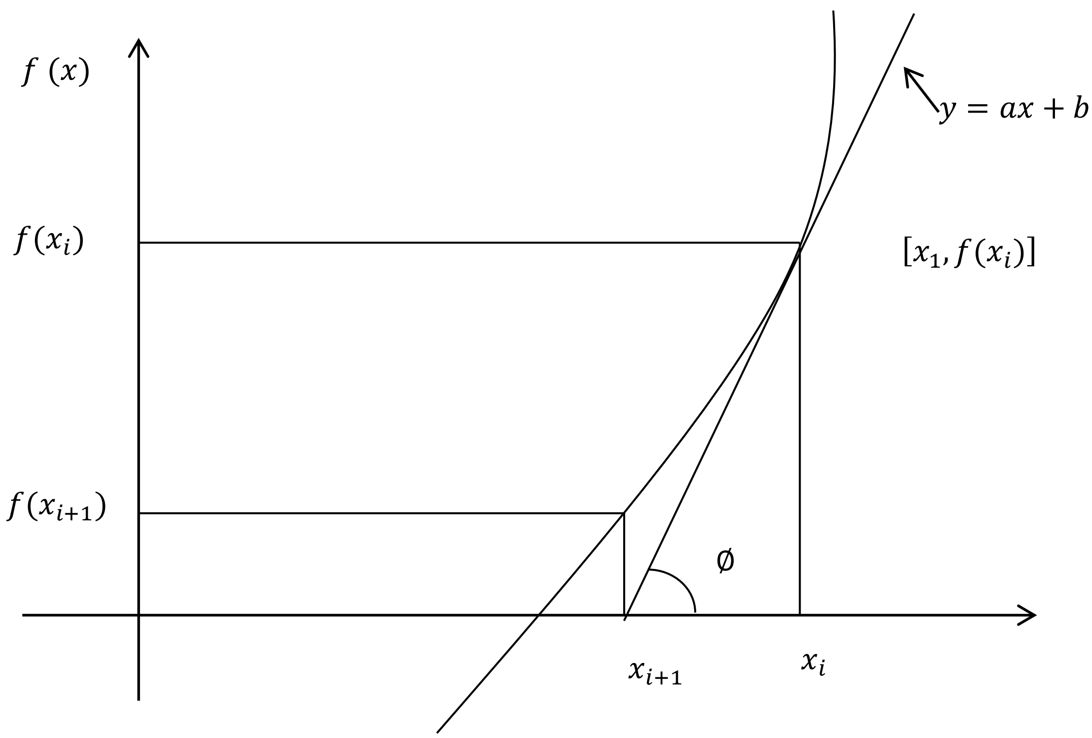


Figure 2-10: Geometric Interpolation of Newton's Method

From equation 2.43 above,

$$\frac{f(x_i)}{f'(x_i)} = \Delta x \qquad \dots(2.47)$$

x_i is the known initial x-value;

$f(x_i)$ denotes the value of the function at x_i;

$f'(x_i)$ is the slope or gradient of the graph above at x_i also written as ${f(x)}/_{dx}$ and

$x_{\ i+1}$ signifies the next x-value.

Therefore, for Δx to tend to zero (0) , the more the iterations you have to run.

It is inevitable in hydraulic analysis to be faced with a problem involving solving sets of non-linear equations defining flow in a network of pipes. In that scenario, the NR method comes into rescue. Indeed, the NR scheme can be used to solve any of the three (3) systems of equations (i.e. Q —equations, H —equations and ΔQ —equations) covered in the preceding sections.

2.5.1.2 The Newton-Raphson Formula for a System of Equations

The Newton-Raphson iterative scheme for solving a system of equations is expressed mathematically as:

$$x_{i+1} = x_i - D^{-1}F_i \qquad \dots (2.48)$$

Where;

x = the column vector of unknowns,

F_i = the demand column vector,

D^{-1} = the inverse of the Jacobian matrix $[\boldsymbol{D}]$.

Let it be known that for: H —equations, the column vector x becomes H, Q — equations, the column vector x becomes Q and ΔQ —equations, the column vector x becomes ΔQ.

The vector components are:

$$H = \begin{bmatrix} H_1 \\ H_2 \\ . \\ . \\ . \\ H_m \end{bmatrix}; Q = \begin{bmatrix} Q_1 \\ Q_2 \\ . \\ . \\ . \\ Q_m \end{bmatrix}; \ \Delta Q = \begin{bmatrix} \Delta Q_1 \\ \Delta Q_2 \\ . \\ . \\ . \\ \Delta Q_m \end{bmatrix} where; (m = 1,2,3, \dots\dots m\ ) \qquad \dots (2.49)$$

The Jacobian matrix *[D]* is a matrix[11] of derivatives. For example, the Jacobian matrix for ΔQ —equations can be expressed as:

[11] A matrix is “a rectangular array of numbers” (Gerald and Wheatley, 2006)

$$D = \begin{bmatrix} \frac{\partial F_1}{\partial \Delta Q_1} & \frac{\partial F_1}{\partial \Delta Q_2} & \cdots & \frac{\partial F_1}{\partial \Delta Q_m} \\ \frac{\partial F_2}{\partial \Delta Q_1} & \frac{\partial F_2}{\partial \Delta Q_2} & \cdots & \frac{\partial F_2}{\partial \Delta Q_m} \\ . & . & . & . \\ . & . & . & . \\ \frac{\partial F_m}{\partial \Delta Q_1} & \frac{\partial F_m}{\partial \Delta Q_2} & \cdots & \frac{\partial F_m}{\partial \Delta Q_m} \end{bmatrix} \quad \dots (2.50)$$

In applying the NR method, the inverse of the matrix of derivatives (Jacobian) is never found. Rather the term $D^{-1}F_i$ is substituted with the solution vector $[z]$.

We therefore end up with:

$$x_{i+1} = x_i - z_i \quad \dots (2.51)$$

However, From;

$$D^{-1}F = Z \quad \dots (2.52)$$

Which can be re-written in as:

$$Dz = F \quad \dots (2.53)$$

2.5.2 Linear Theory Method (LTM)

The Linear Theory Method (LTM) (Wood and Charles, 1972) solves a set of Q-equations at once after linearizing the system of non-linear equations. To avoid manual initialization, an initial estimate of $1.0\ cfs\ (0.0283m^3/s\ or\ 28.3l/s)$ for each pipe was an assumption suggested by Wood and Charles (1972) when applying LTM. Furthermore, for the first iteration, a constant velocity value of $1m/s$ may perhaps be assumed for all network pipes. Linear theory (LT) converts the $non-linear$ energy equations into linear by estimating the pressure drop in each pipe as:

$$h_{Lj} = \left[K_j Q_{jo}^{m-1}\right] Q_j = K_j' Q_j \quad \dots (2.54)$$

Where;

Q_{jo} = Estimated discharge in line j.

It has been reported (Khamkham, 2000; Larock, et al., 2000) that if you have a network made up of NP conduits, NJ Nodes and NL loops, then the following relationship is true for all network patterns:

$$NL = NP - NJ + 1 \qquad ..(2.55)$$

Where $NP =$ Number of Pipes; $NL =$Number of Loops and $NJ =$Number of Junctions

Continuity equations for junction nodes are written as:

$$\sum_{j=1}^{NPJ} Q_j = q_{ext} \text{ for all Junction nodes} \qquad ...(2.56)$$

Loop equations are written as;

$$\sum_{j=1}^{NPL} K_j |Q_{jo}|^{m-1} Q_j = 0 \text{ For all real loops} \qquad ...(2.57a)$$

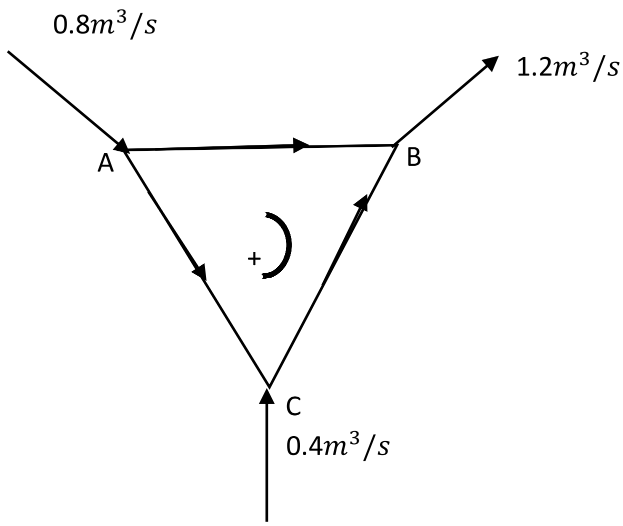
$$\sum_{j=1}^{NPL} K_j |Q_{jo}|^{m-1} Q_j = \Delta H \text{ For pseudo loops} \qquad ...(2.57b)$$

These equations formularize a set of linear equations that can be resolved using a number of schemes such as LU decomposition, Jacobi's method or Gaussian Elimination technique. The pendulous action of LTM around the final solution prompted Wood and Charles (1972) to suggest the use of the average values of flows from the previous two computations as the approximation for the next iterations $\left[Q_{jo} = \left(\frac{Q_{Old}+Q_{New}}{2}\right)\right]$. The drawback of LTM is that, it results into a non-symmetric matrix[12] . This thwarts the service of more proficient linear algebra solution schemes (Ellis, 2001).

[12] A symmetric matrix is one which is equivalent to its transpose (meaning it doesn't change when transposed). Whereas transposing a non- symmetric matrix results into a different matrix from the original matrix.

2.5.2.1 Worked Example Problem Illustrating Application of LTM

Consider a pipe Network below



Pipe Resistance

Pipe	AB	BC	AC
K	50	30	60
n	2	2	2

Figure 2-11: A 3 Pipe, 3 Node Network

Step 1: List down the continuity equations for junctions A and B. Junction C equation is ignored because it's the summation of the first two equations. Therefore, it would just stay redundant.

$$Q_{AB} + Q_{AC} = 0.8 \quad \dots (2.58a)$$

$$Q_{AB} + Q_{BC} = 1.2 \quad \dots (2.58b)$$

Step 2: Write the loop equation

$$K_{AB}(Q_{AB})^n - K_{BC}(Q_{BC})^n - K_{AC}(Q_{AC})^n = 0 \quad \dots (2.58c)$$

Substituting the values into equation (2.53c), we obtain;

$$50Q_{AB}^2 - 30Q_{BC}^2 - 60Q_{AC}^2 = 0 \quad \dots (2.58d)$$

Re-written as:

$$50|Q_{AB}|Q_{AB} - 30|Q_{BC}|Q_{BC} - 60|Q_{AC}|Q_{AC} = 0 \quad \dots (2.58e)$$

Dividing equation (2.58e) above by 10 we obtain:

$$5|Q_{AB}|Q_{AB} - 3|Q_{BC}|Q_{BC} - 6|Q_{AC}|Q_{AC} = 0 \quad \dots (2.58f)$$

Expressing equations $(2.58a, 2.58b\ and\ 2.58f)$ in a matrix form as;

$$[D]\{Q\} = \{F\} \quad \dots (2.58j)$$

Where;

$$D[Transformation\ Matrix] = \begin{bmatrix} 1 & 0 & 1 \\ 1 & 1 & 0 \\ 5\boldsymbol{Q_{AB}} & -3Q_{BC} & -6Q_{AC} \end{bmatrix}$$ and

$$F[Demand\ column\ Vector] = \begin{bmatrix} 0.8 \\ 1.2 \\ 0 \end{bmatrix}$$

$$Q[Unknown\ Discharges] = \begin{Bmatrix} Q_{AB} \\ Q_{BC} \\ Q_{AC} \end{Bmatrix}$$

To obtain unknown $\{Q_{New}\}$, perform the iteration as follows:

$\{Q_{New}\} = [D]^{-1}\{F\}\ and\ Q^0 = (Q^0_{old} + Q_{New})/2$ ... (2.58l)

Therefore, Equation 2.58j can be solved by one of the following methods: LU decomposition, Cramers Rule or Gaussian Elimination.

Let's Assume $Q^0_{AB} = Q^0_{BC} = Q^0_{AC} = 0.0283 m^3/s$ (Wood & Charles, 1972)

For the first iteration,

$$\begin{bmatrix} 1 & 0 & 1 \\ 1 & 1 & 0 \\ 5(0.0283) & -3(0.0283) & -6(0.0283) \end{bmatrix} x \begin{Bmatrix} Q_{AC} \\ Q_{BC} \\ Q_{AC} \end{Bmatrix} = \begin{bmatrix} 0.8 \\ 1.2 \\ 0 \end{bmatrix}$$

$Q_{AB} = 0.6 m^3/s;\ Q_{BC} = 0.6 m^3/s; Q_{AC} = 0.2 m^3/s$

For the second iteration

$$Q_{ABNew} = \left(\frac{0.6+0.0283}{2}\right) = 0.31415 m^3/s$$

$$Q_{BCNew} = \left(\frac{0.6+0.0283}{2}\right) = 0.31415 m^3/s$$

$$Q_{ACNew} = \left(\frac{0.2+0.0283}{2}\right) = 0.11415 m^3/s$$

$$\begin{bmatrix} 1 & 0 & 1 \\ 1 & 1 & 0 \\ 5(0.31415) & -3(0.31415) & -6(0.11415) \end{bmatrix} x \begin{Bmatrix} Q_{AB} \\ Q_{BC} \\ Q_{AC} \end{Bmatrix} = \begin{bmatrix} 0.8 \\ 1.2 \\ 0 \end{bmatrix}$$

$Q_{AB} = 0.525 m^3/s;\ Q_{BC} = 0.675 m^3/s; Q_{AC} = 0.275 m^3/s$

For the third iteration

$$Q_{ABNew} = \left(\frac{0.525+0.31415}{2}\right) = 0.4196m^3/s$$

$$Q_{BCNew} = \left(\frac{0.675+0.31415}{2}\right) = 0.4946m^3/s$$

$$Q_{ACNew} = \left(\frac{0.275+0.11415}{2}\right) = 0.1946m^3/s$$

$$\begin{bmatrix} 1 & 0 & 1 \\ 1 & 1 & 0 \\ 5(0.4196) & -3(0.4946) & -6(0.1946) \end{bmatrix} x \begin{Bmatrix} Q_{AB} \\ Q_{BC} \\ Q_{AC} \end{Bmatrix} = \begin{bmatrix} 0.8 \\ 1.2 \\ 0 \end{bmatrix}$$

$Q_{AB} = 0.572m^3/s$; $Q_{BC} = 0.628m^3/s$; $Q_{AC} = 0.228m^3/s$

Continue with the computation for the next iterations until convergence occurs to obtain the following final discharges:

$$Q_{AB} = 0.5605m^3/s; \quad Q_{BC} = 0.6395m^3/s; \quad Q_{Ac} = 0.2395m^3/s$$

2.5.3 Hardy Cross (1936) Method

One of the primary and oldest renowned, widely used analysis approaches is the Hardy Cross (1936) method. This method of course, was initially proposed for manual calculation especially for networks with few loops before the birth of digital computers. The distinct advantage this method has over the rest is the ability to perform simple arithmetic while self-adjusting the initially guessed flow values in each duct.

Cross' (1936) work has been the most cited pieces of work but with slight understanding by the writers. Hardy Cross (1936) invented two methods-that is the "method of balancing heads" and the "method of balancing flows". The "method of balancing heads" gained popularity and was accepted by industries until the late 1960's.

2.5.3.1 Method of Balancing Heads

This technique balances the initially guessed discharges in each network pipe founded on the loop-continuity equations. The point is that an initial guess of flows in the network which satisfy continuity must be provided. Then, proceed to calculate the counter balancing flow and use it to make corrections to the initial flows. The iterative process is carried on until the counter balancing flow decreases to within an acceptable range. Usually, continuity must be maintained at the nodes and the discharges are successively modified to satisfy the zero sum of head loss around the loops.

Table 2-13: Procedure for Solving Flow Network Problem by Method of Balancing Heads

Step	**Description of the process (Cross, 1936)**
1	Set up a grid pattern with closed loops to look like planned flow distribution arrangement
2.	Compute water demand on each street without excluding fire flow demand
3.	Sum up the flow used in the area without fire flow demand and allocate it to the nodes where known outflows are needed.
4.	Guess internally consistent distribution of flow, i.e. at any given junction node, flow continuity must be satisfied. ($\sum Q_{in} = \sum Q_{out}$)
5.	Decide the sign convention for each circuit. Normally Clockwise flows are positive and counter-clockwise flows are negative.
6.	Giving consideration to sign (+/-) convention, Calculate the head loss in each duct using the non-linear equation below. $h_L = KQ^m$
5.	Calculate the total head loss around each loop without forgetting the sign convention. $\sum h_L = \sum KQ^m$
6.	Without sign convention, calculate the sum of quantities, R = $\sum mKQ^{m-1}$ in each closed loop.
7.	Compute the counter-balancing flow for each loop as follows: $\Delta Q = \dfrac{-\sum KQ_o^m (Giving\ attention\ to\ sign\ convention)}{\sum mK\lvert Q_o^{m-1}\rvert\ (without\ refrence\ to\ sign\ convention)}$
8	Apply the counter-balancing flow to each pipe not common in both loops. $Q_{New1} = Q_o + \Delta Q_{loop1}$
9.	In case of pipes common in both loops, use the correction formula below. $Q_{New1} = Q_o + \Delta Q_{loop1} - \Delta Q_{loop2}$

10.	Repeat the process until the corrections are reasonably small.
11.	Calculate flow velocities in each duct and match to the standards to ensure adequate velocity and pressure is present in each element. If necessary, modify pipe sizes to increase or reduce flow velocity.
12.	Reiterate all the above process until an acceptable solution is attained.

2.5.3.1.1 Proof of HC Algorithm

Let's consider a single loop. The correcting flow term can be deduced from binomial expansion of KQ^m (Cross, 1936).

Where;

$Q = Q_o + \Delta Q$, ΔQ = rectifying term and Q_o =initial estimate of Q

$$KQ^m = K(Q_o + \Delta Q)^m = K(Q_o^m + mQ_o^{m-1}\Delta Q + \binom{m}{2} Q_o^{m-2}\Delta Q^2\ ) \qquad \ldots (2.59)$$

Since ΔQ is small, all terms beyond the first order may be neglected. Re-ordering eqn. (2.55) ends up generating a common correction term below;

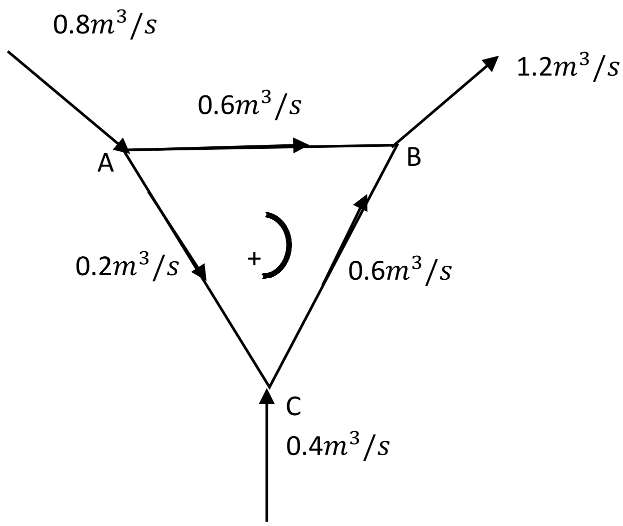
$$\Delta Q = \frac{-\sum KQ_o^m}{\sum mK|Q_o^{m-1}|} \qquad \ldots (2.60)$$

Of course, this may appear as only an approximation, but, several re-adjustments of the loops increases its efficacy to even handle multiple reservoirs and pumps (Ellis, 2001). It has been proven that errors are not cumulative but, the choice of initial flow estimate presents a great influence on the number of iterative steps before eventually arriving at the solution. Essentially, Cross (1936) noted that $\sum mK|Q_o^{m-1}|$ doesn't change any much during the iterative process and it is not necessary to usually be re-calculated for each change in flow (Ellis, 2001).

When implementing HC algorithm, for the starting successive iterations, conservation laws are likely not to be satisfied as the computed ΔQ will not make pressure drop around the closed loop zero although it will reduce it closer to zero than it was in the previous iteration.

2.5.3.1.2 Worked Example Problem Illustrating HC Method of Balancing Heads

Consider a pipe network in figure 2-12 below



Pipe Resistance

Pipe	AB	BC	AC
K	50	30	60
n	2	2	2

Figure 2-12: A 3 Pipe, 3 Node Network

Initial estimates of the discharges in each pipe are obtained, obeying the first law of Kirchhoff at the junction nodes.

Head losses are then calculated in the clockwise direction. Make sure the sign convention is obeyed. The results for the HC iterative process are presented systematically in Table 2-15.

Calculation for the first iteration;

Step1: Summation of head losses around the loop:

$$\sum h_L = K_{AB}(Q_{AB})^{n_{AB}} - K_{BC}(Q_{BC})^{n_{BC}} - K_{AC}(Q_{AC})^{n_{AC}} \qquad \dots (2.61)$$

In which; $n_{AB} = n_{BC} = n_{AC} = 2$;

$$\sum h_L = 50(0.6)^2 - 30(0.6)^2 - 60(0.2)^2 = 4.8$$

Step2: Summation of the first derivative of head losses

$$\sum h'_L = n_{AB}K_{AB}(Q_{AB})^{n_{AB}-1} + n_{BC}K_{BC}(Q_{BC})^{n_{BC}-1} + n_{AC}K_{AC}(Q_{AC})^{n_{AC}-1} \qquad \dots (2.62)$$

$$\sum h'_L = 2(50(0.6) + 30(0.6) + 60(0.2)) = 120$$

Step 3: The Loop Corrective flow:

$$\Delta Q_{cor} = {-\sum h_L}/{\sum h'_L} = -\left({4.8}/{120}\right) = -0.04\ you\ then\ proceed\ to\ correct\ the\ flows$$

$$Q_{new} = Q_o \pm \Delta Q_{cor}$$

$$Q_{ABNew} = 0.6 + (-0.04) = 0.56 m^3/s$$

$$Q_{BCNew} = 0.6 - (-0.04) = 0.64 m^3/s$$

$$Q_{ACNew} = 0.2 - (-0.04) = 0.24 m^3/s$$

Repeat the procedure above from step 1 to 3 until ΔQ_{cor} is so small (probably ΔQ_{cor}<Tolerance error (in this case let's use 0.0001).

Table 2-14: Computation for the 3 Pipe, 3 Node Network Problem

First Iteration					
Pipe	**K**	**N**	$\boldsymbol{Q_0}$	$\boldsymbol{h_L = K(Q)^n}$	$\boldsymbol{h_L/Q}$
AB	50	2	0.6	18	30
BC	30	2	-0.6	-10.8	18
AC	60	2	-0.2	-2.4	12
$\Delta Q_{cor} = -\left(\sum h_L / n\sum h_L/Q\right) = -\left(\frac{4.8}{2*60}\right) = -0.04$				$\sum = 4.8$	$\sum = 60$
Second Iteration					
Pipe	**K**	**N**	$\boldsymbol{Q_0}$	$\boldsymbol{h_L = K(Q)^n}$	$\boldsymbol{h_L/Q}$
AB	50	2	0.56	15.68	28
BC	30	2	-0.64	-12.288	19.2
AC	60	2	-0.24	-3.456	14.4
$\Delta Q_{cor} = -\left(\sum h_L / n\sum h_L/Q\right) = -\left(\frac{-0.064}{2*61.6}\right) = 0.000519$				$\sum = -0.064$	$\sum = 61.6$
Third Iteration					
Pipe	**K**	**N**	$\boldsymbol{Q_0}$	$\boldsymbol{h_L = K(Q)^n}$	$\boldsymbol{h_L/Q}$
AB	50	2	0.5605	15.708	28.025
BC	30	2	-0.6395	-12.269	19.185
AC	60	2	-0.2395	-3.442	14.372
$\Delta Q_{cor} = -\left(\sum h_L / n\sum h_L/Q\right) = -\left(\frac{-0.003}{2*61.582}\right) = 0.000024$				$\sum = -0.003$	$\sum = 61.582$

As it can be seen in the third iteration that $\Delta Q_{cor} = 0.000024 < Tol = 0.0001$, then $Q_{AB} = 0.5605 m^3/s;\ Q_{BC} = 0.6395 m^3/s;\ Q_{AC} = 0.2395 m^3/s$

2.5.3.2 Method of Balancing Flows

This method balances flows in each network pipe basing on nodal-continuity equations with nodal heads as unknowns. Therefore, an initial guess of the unknown heads in the network is required. The flows are then computed from these initial head estimates and re-adjusted sequentially. The iterative process is carried forward until the discrepancies are decreased to a specified tolerance. Zero (0) sum of head loss is conserved around the loops and flows are sequentially corrected to satisfy flow continuity at the nodes. Cross (1936) noted that the convergence was slow and that some loops could be out of balance with regard to head loss. Gessler (1981) maintains that probably this could have been the reason why it did not gain popularity related to the method of balancing heads.

2.6 Convergence Criteria

In pipe network analysis using numerical algorithms, iterations are continued until the defined convergence criterion is achieved. Generally, there are four criteria for convergence that can be useful to determine the acceptability of a solution.

1- Based on $\Delta \boldsymbol{Q}$ for each loop

This criterion is commonly implemented in HC and NR algorithms. The solution convergences once the absolute value of all corrective flows ΔQ is less than defined tolerance, i.e. $\Delta Q_i < TOL$

2- Based on $\Delta \boldsymbol{H_i}$

This is commonly used in HC and NR algorithms. The solution convergences once the absolute value of all corrective heads ΔH_i is less than the defined tolerance, i.e. $\Delta H_i < TOL$.

3- Based on $\sum \boldsymbol{h_i}$ for each loop

The convergence is achieved when the sum of head losses around a loop is zero

4- Based on % Change in flow rates

This criterion could be applied to HC, NR and LT solution approaches. The change in flow rates between the successive trials can be used to check convergence. This is termed as relative accuracy, i.e.

$$\left(\frac{Q_{new} - Q_{old}}{Q_{new}}\right) * 100\% < 0.5\%$$

2.7 Network Cost

Network cost is found by totaling the cost of each conduit. The total cost can be mathematically stated as (Sadafule, et al., 2013):

$$C = \sum_{j}^{N} C_j \left[L_j D_j\right] \qquad \text{... (2.63)}$$

Where;

C_j= Cost per unit length of pipe j with diameterD_j;

L_j = Length of the pipe j;

2.8 Program Validation Methods

Verifying the performance of the model requires conducting statistical analysis. The analysis schemes include; the coefficient of determination (R2), the root mean square error (RMSE), and the mean bias error (MBE). RMSE measures the variation of predicted figures around the observations. The smaller the RMSE, the more precise is the approximation. MBE is a representation of the mean deviation of the predicted values from the respective observations. The smaller the MBE, the more superior is the model performance (Maitha, et al., 2011).

The expressions for the aforementioned statistical parameters are:

$$R^2 = 1 - \frac{\sum(q_{obs} - q_{cal})^2}{\sum(q_{obs} - \bar{q}_{cal})^2} \qquad \text{... (2.64)}$$

$$RMSE = \sqrt{{}^{1}/_{N} \sum_{k=N}^{N} (q_{cal} - q_{obs})^2} \qquad \text{... (2.65)}$$

$$MBE = \frac{1}{N} \sum_{k=1}^{N} (q_{cal} - q_{obs}) \qquad \text{... (2.66)}$$

Where; q_{obs} =observed discharge, q_{cal} = Calculated discharge and $\bar{q}_{cal}$ =mean of Calculated discharge

It has been reported (Oke et al., 2015) that the credibility of any algorithm relies on its accurateness and rationality. The statistical methodology established to report about the reliability of any technique involves examining the assumption that no variance between the algorithm and the other algorithms exists. The consistency [C] between the algorithms have been statistically defined by Sartory (2005) and Oke (2007) as:

$$C = 100 - 100\left[\frac{\sum_{i=1}^{n}(q_{obs} - q_{cal})}{\sum_{i=1}^{n} Q_{obs}}\right] \qquad \dots (2.67)$$

Where;

q_{obs} =expected flows and q_{cal} =obtained flows

2.9 Existing Hydraulic Models for Water Supply Networks

There are some existing computer models that can be applied to solve a set of equations defining flow in pipe networks and to mimic a number of flow control devices. Some useful educational packages also do exist for analysing smaller network problems. Table 2-14 presents some existing hydraulic models used for solving flow network problems.

Table 2-15: Existing Hydraulic Models Applied to Solve Flow Network Problems

Software	Capability	Developer	Reference
KYPIPE2	This mimics pressure flow in pipe networks including water distribution, irrigation. It as well handles a tree-like pipe network with dead-ends. Output from the program includes; flow rates, velocities, head losses, junction pressures, energy grade line elevations and water surface elevations in the storage tanks. Additionally, the pump heads and the valve losses can be presented. It uses the LT Algorithm.	D. J. Wood, Department of Civil Engineering, University of Kentucky, Lexington, KY.	(Brater, et al., 1996)
EPANET	A computer model written in C programming language for performing extended-period simulation of hydraulic and water-quality behaviour within pressurized pipe networks. EPANET uses gradient algorithm proposed by Todini and Pilati (1988) for hydraulic analysis. A	L. A. Rossman , Drinking Water Research Division, Risk Reduction Engineering	(Brater, et al., 1996; Rossman, 2000; Sonaje & Joshi, 2015)

	network in EPANET is represented by; links (pipes), junction nodes, pumps, valves, and storage tanks or reservoirs (figure 2-1). Indeed, EPANET is able to perform water quality analysis as well as water age tracing.	Laboratory, U.S. Environmental Protection Agency.	
WaterCAD V8i (2014)	A hydraulic mode which implements Gradient Algorithm with an array of functionalities and advancements in GUI. It is capable of hydraulic and water quality analysis, steady state and extended period simulations, strong data management along with AutoCAD and GIS integrations.	Bentley's Haestad Methods (hydraulic and Hydrology) group	(Sonaje & Joshi, 2015)
FLOWMASTER	A general-purpose application for simulating fluid flow in complex pipe networks. The model mimics the real situation by offering mathematical representation of individual network components and joining them at nodes according to user need. The model can also analyse heat-transfer within the pipeline.	Amtech (UK) Limited.	(Brater, et al., 1996)

In most existing hydraulic network simulation models such as EPANET2, after manually estimating the base demand for a given the population at every network node, the demand can then be fed into the computer model to compute actual discharges in every pipe in the network. There is time wastage in the course of manual evaluation of base demand before the data could be keyed into the computer program. However, in this research thesis, the currently developed user friendly numerical hydraulic model has the capability to compute for the user the base demand at every network node given the available projected population figures at each junction node. This will save much time the designer would waste trying to determine the base demand with a hand calculator.

2.10 Computer Programming

Writing both simple and advanced commands that a computer implements is a practice termed as “computer programming”. The commands, commonly known as code or simply algorithm, are written in a programming language which a computer can interpret, comprehend and use to resolve a problem or execute a task.

2.10.1 Software Development Checks

Whatever the language of programming may be, the final package must meet these fundamental checklists.

a) **Program reliability:** How often precise is the output from the program. This always relies on the theoretical precision of the algorithms, and minimization of coding errors such as logic errors like division by zero [0].

b) **Usability of the application:** The success of any computer program entirely depends on the ease with which the user will use the software for its intended purpose.

c) **Program maintainability:** While developing a specific program, developers need to give considerations to the ease with which it can be improved upon by its current or future programmers in order to either customize or replicate it to new surroundings. This will depend entirely on the source code readability: Readability in computer language means the ease with which a different programmer can read and understand the lines of written codes.

For the past decades, numerous high-level languages have been advanced, however, only a hand full have become the de-facto industry standard, such as **(Visual Basic 6.0, FORTAN and Pascal)**.

2.11 Microsoft Visual Basic 6.0

Microsoft VB6.0 is an event- driven and graphical-based tool that enables development of user friendly, Microsoft windows based programs conversant to the users (Tylee, 1998). In an event-driven language, the command stays idle until called upon to react to some event procedure for example by pressing a button, or selecting from the menu. Graphical-based means the language can enable the user to work directly with graphical controls like text box, button among others.

2.11.1 Key Terms

Before any developer starts creating applications in Visual Studio 6.0 using VB language, they have to be well conversant with the succeeding key concepts:

1) **Distributable component:** The complete, compiled form of a project (application) that can be kept on any storage device for distribution to other computer users.
2) **Project:** A group of files compiled to build a distributable program.
3) **Solution:** A group of projects and files that make up a program.

Project (.VBP, .MAK)

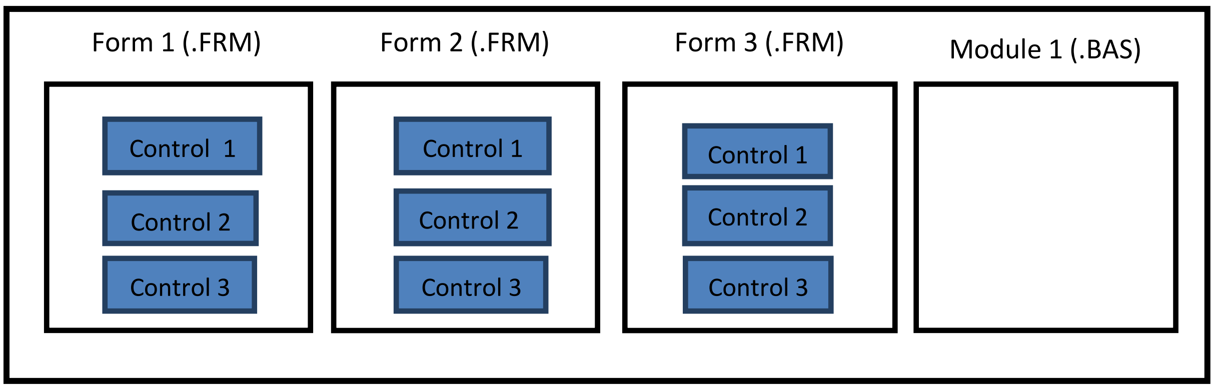


Figure 2-13: VB Application (Project) Structure *(Tylee, 1998)*

Table 2-16: Description of VB Project (Application) Elements

Element	Description
Forms	Windows generated for user interaction.
Controls	Graphical elements such as command buttons, picture box, list boxes drawn on forms.
Methods	In-built procedure executed to give some action to a certain object.
Event Procedures	Encryption linked to objects, invoked when a specific event results.
General Procedures	Encryption not connected to objects that must be run by the application.
Modules	Compilation of general procedures, variable declarations, and constant definitions used by the program.

2.11.2 Microsoft Visual Basic 6.0 Components

The components are what make up the program. The Ms. VB6.0 components are presented in Table 2-16 and figure 2-14 below.

Table 2-17: Components of Microsoft Visual Basic 6.0

Component	Description
Menu Bar	This is where you choose commands that are used to direct the functioning of Visual Basic Integrated development Environment (IDE).
Toolbox	This is a container from which controls such as image box, label are picked and drawn to the form.
Code Window	This is where the developer writes the code (command) that is executed when a specific event ensues.
Properties Window	This is the window which allows the developer to make modifications to the object property values.
Form Window	This window is significant when generating windows applications. It is where the application controls are drawn.

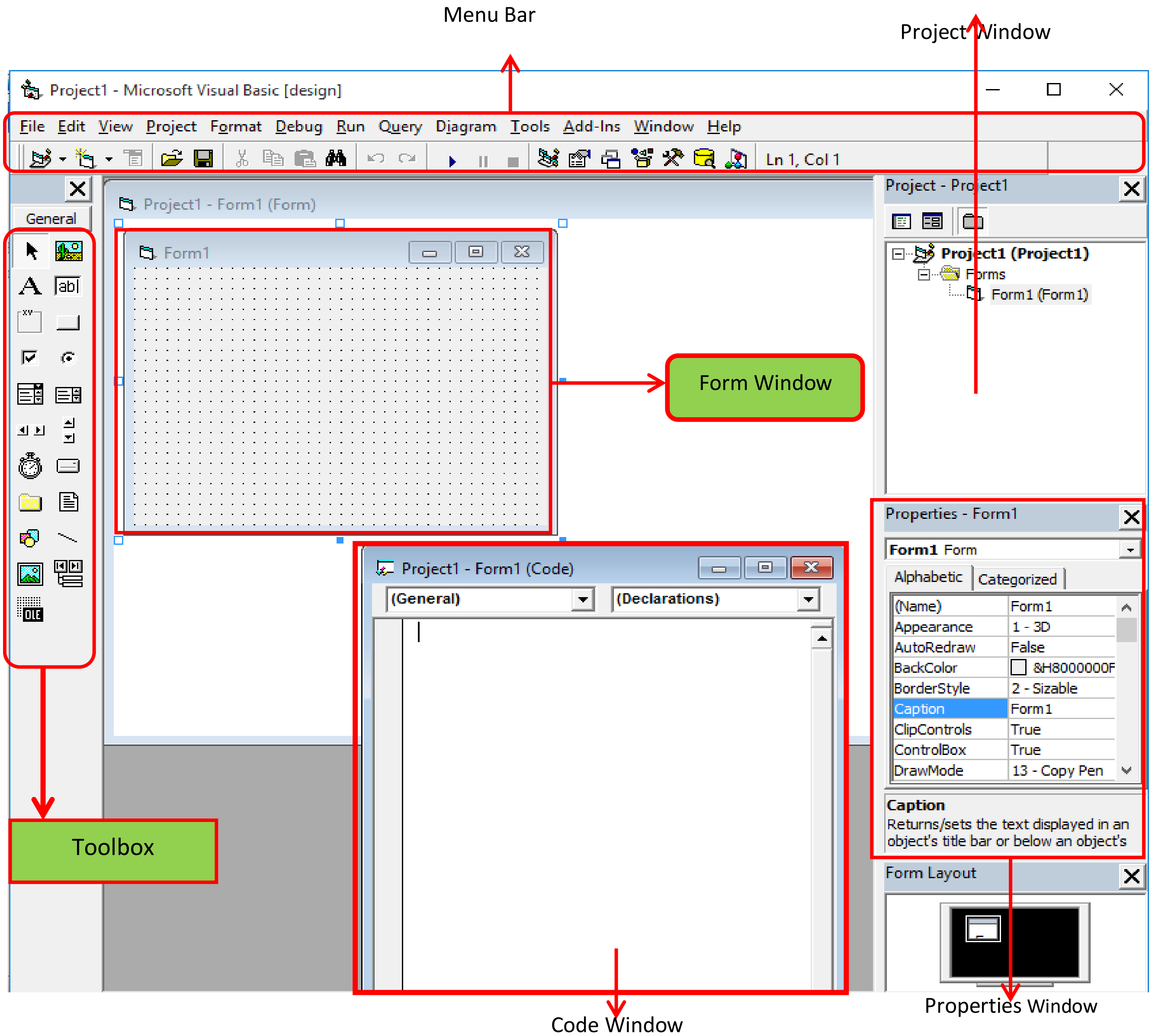


Figure 2-14: Microsoft VB 6.0 Integrated Development Environment (IDE)

2.12 Software Development Methodologies

For software development projects, there are basically two (2) fundamental questions that should be answered prior to choosing which methodology to adopt:

1) What is the project deadline?
2) What is the project scope?

Numerous approaches and modus operandi have been used to build a program[13]. However, there are three (3) widely used methodologies in Software Development (SD) discussed in the following subheadings and these include; iterative, water fall and V-Model techniques.

2.12.1 Iterative Methodology

Iterative method (figure 2-15) is a process which encompasses planning through design, implementation, testing and evaluation. In this approach, after accomplishing the initial planning, a couple of steps are reiterated over and over. In the Iterative model, the designer begins by building a small part of program requisites and continuously advancing the next versions until the built final product is ready for use.

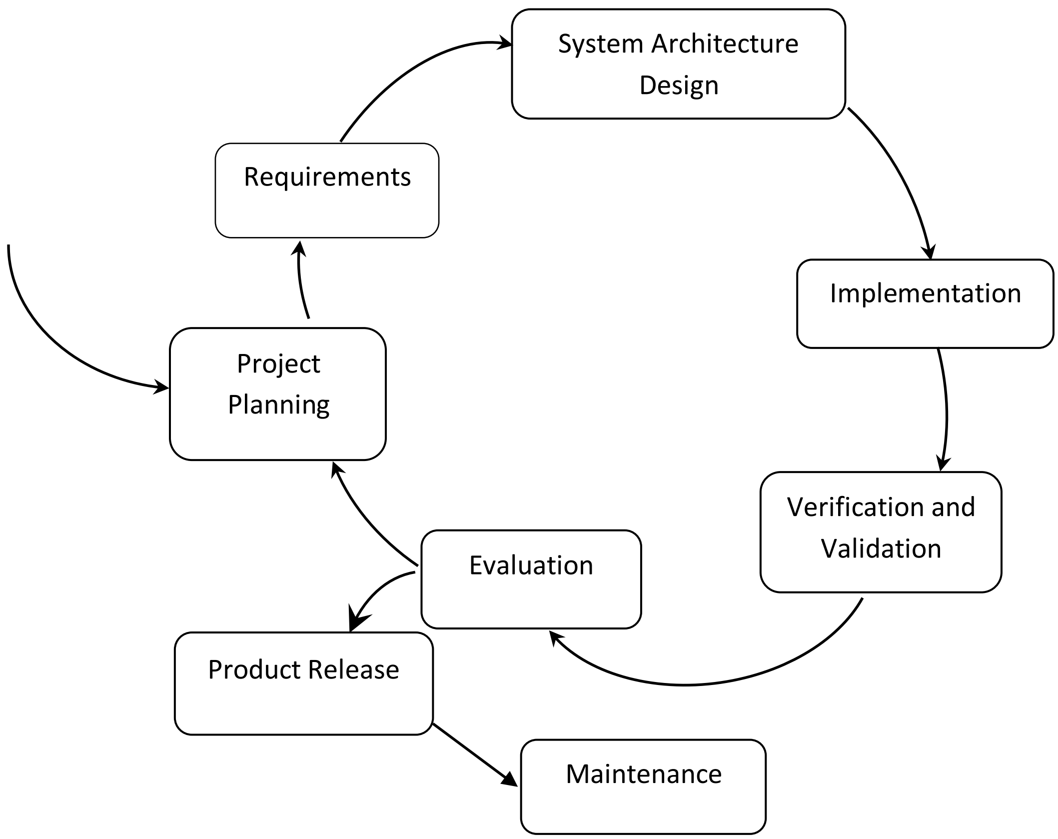


Figure 2-15: Iterative Model

Some of the key weaknesses of iterative approach include; lack of definite ending date of project, requires a lot of resources in reiterating the processes, and only appropriate for bigger projects.

[13] www.professionalqa.com/iterative.model

2.12.2 Waterfall Approach

The "waterfall" technique (figure 2-16) was the pioneering methodology (CTG, 1998) formally suggested by Royce (1970) for use in the framework of spacecraft mission software evolvement.[14] The method subdivides the entire course of program development into discrete steps thus, the result of one step is the input of the next step (Balajji & Murugaiyan, 2012) in succession. The following diagram explicitly shows the dissimilar steps of waterfall model.

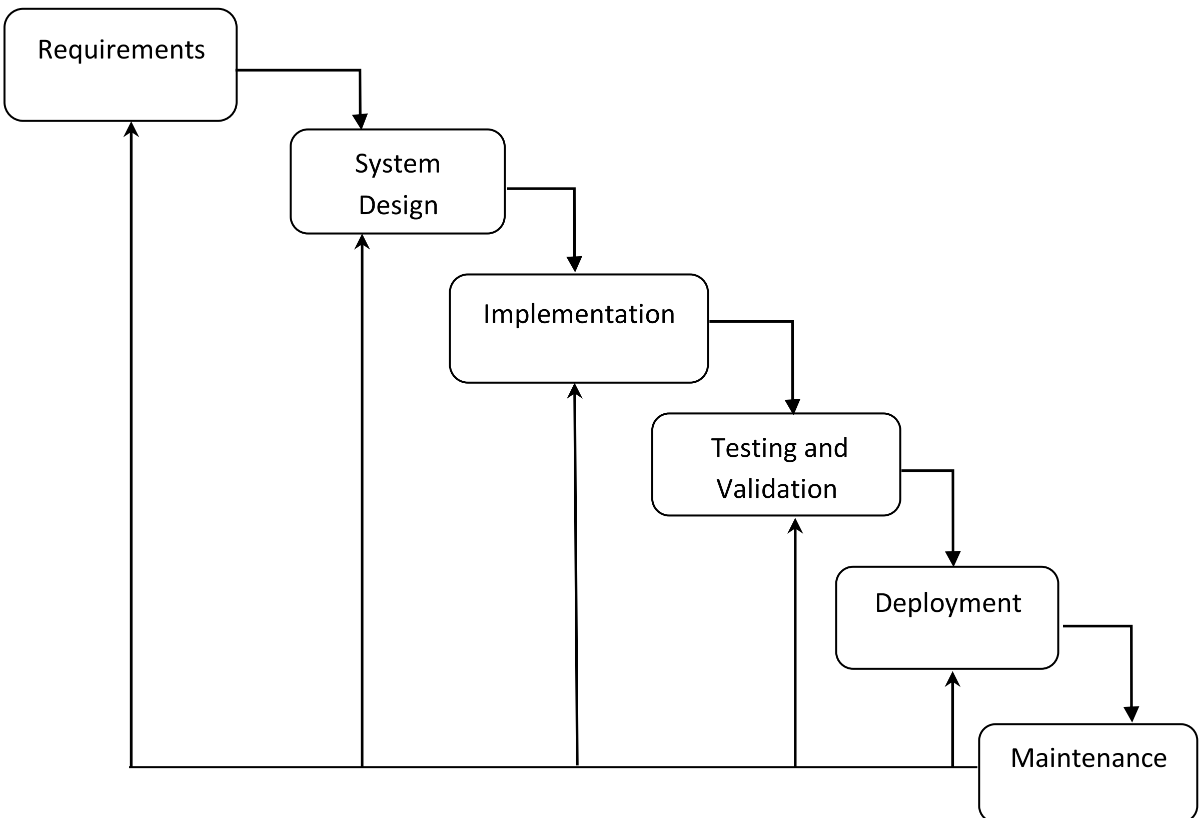


Figure 2-16: Waterfall Model (CTG, 1998)

Some conditions under which Waterfall methodology may be adopted are–

- ✓ Requirements are clear and static.
- ✓ Project definition is steady.
- ✓ The project is short.

[14] https://www.tutorialspoint.com/sdlc/sdlc_waterfall_model.htm

Some of the key merits of the Waterfall Model are as follows –

- ✓ The steps are completed one at a time.
- ✓ Applicable for minor projects.
- ✓ Well-defined stages.
- ✓ Clearly understood milestones.

2.12.3 V-Model

The V-Model (figure2-17) is an enhancement of the waterfall model (Mathur & Malik, 2010). In every step, there is verification before embarking on the next stage (Balajji & Murugaiyan, 2012). This is a well-ordered model where subsequent step begins only when the preceding stage is consummate.

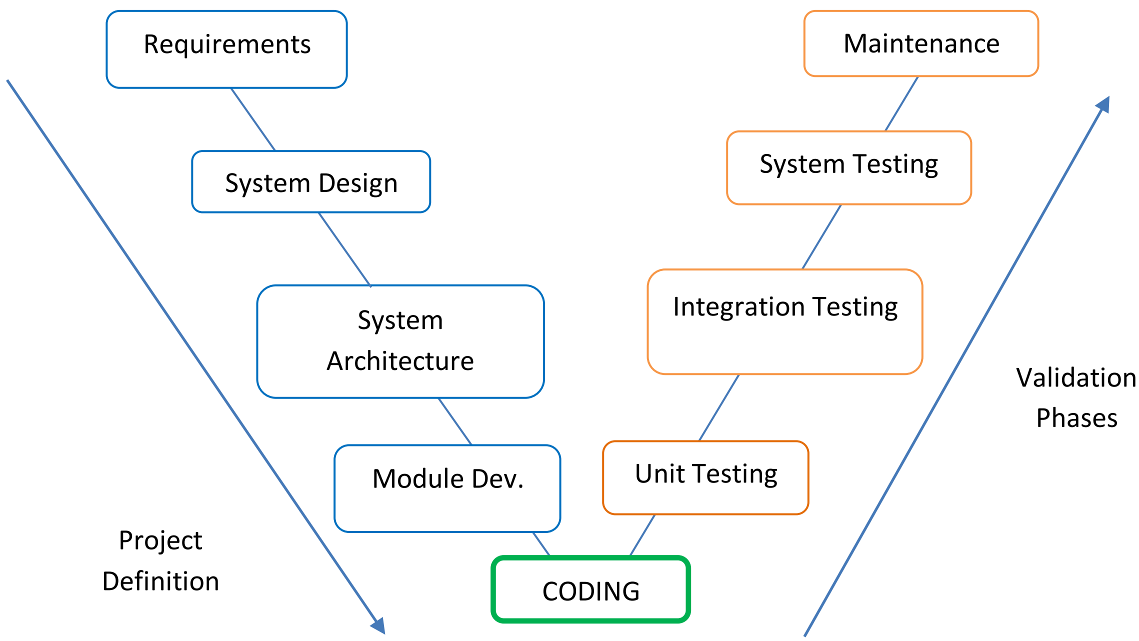


Figure 2-17: V-Model (Mathur & Malik, 2010)

The conditions under which V-Model can be used are similar to that of waterfall. However, some of the key benefits of the V-Model method are as follows (Balajji & Murugaiyan, 2012):

- Highly-disciplined model and stages are accomplished one at a time.
- There is proper time management.
- Very good for smaller projects where requirements are clearly-defined.
- Simple and easy to understand and use.

Chapter 3 : Research Methodology

3.0 Introduction

This chapter describes the methodology adopted to develop the user friendly hydraulic model for analysing and costing complex network pipelines. Due to time constraint, the overall methodology used was according to V- model[15](figure 3-1) structure of program design that encompasses; the project definition (Requirements specification, Architecture design, Module design), Implementation (coding) and the project test and integration (Integration, test, and verification, System verification and validation, operation and maintenance). The succeeding illustration describes the different phases in a V-model of the ADLC[16]

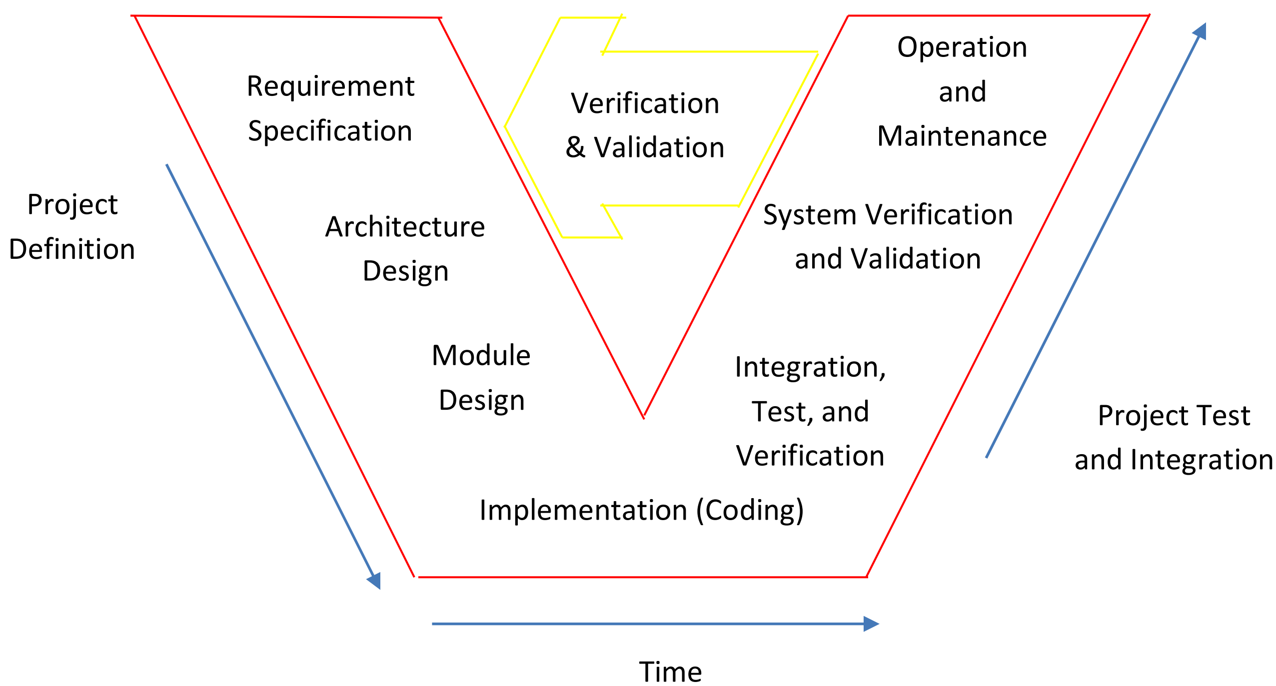


Figure 3-1: V-Model Methodology of Application Design

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[15] https://www.tutorialspoint.com/sdlc/sdlc_V_model.htm.

[16] https://en.wikipedia.org/wiki/V-Model_(software_development)

3.1 System Requirements

During this first phase, the system requirements were established to determine the feature set. Spending sufficient time and developing the model requirements was significant at this step. The approaches that were used to achieve this include;

- **Literature Review:** This involved reading written materials related to this study from different authors that include; textbooks, magazines, journals and documents from libraries and by utilizing internet through visiting several websites with the aim of acquiring more information based on the current systems.
- **Consultation:** This involved interacting with professionals in the fields of water resources, experts in the field of programming, and the public on the challenges they face while using the existing models.

3.1.1 Functional Requirements

The functional requirements describe the task of a system or its modules. This considers the system's ability to; compute the flow rates, head losses, flow velocities within the pipe network, pressure heads at the network nodes and cost network pipelines.

3.1.2 Non-Functional Requirements

This include; reliability (is the system able to give only correct output?), usability (is the system user friendly?), efficiency (is the system able to run without any intervention, thus maintaining the correct outputs ever?), performance requirements (is the system fast in executing its algorithm to provide results in case of any looping?)

3.2 Program Architecture

The program consisting of welcome form, user login form, main menu form and computation forms was written in a simple programming language called VB6.0.

The first form that loads after the program is started is called the welcome form. The next form that opens after clicking the start button on the welcome form is called the main menu form. This is where the user performs loops and head loss formula selection. After the user has selected a specific head loss formula and the loop to be analysed, the model entreats computation form for a particular loop and head loss formula. The computation form allows the user to enter pipe properties. The following figure 3-2 shows the system architecture hierarchy.

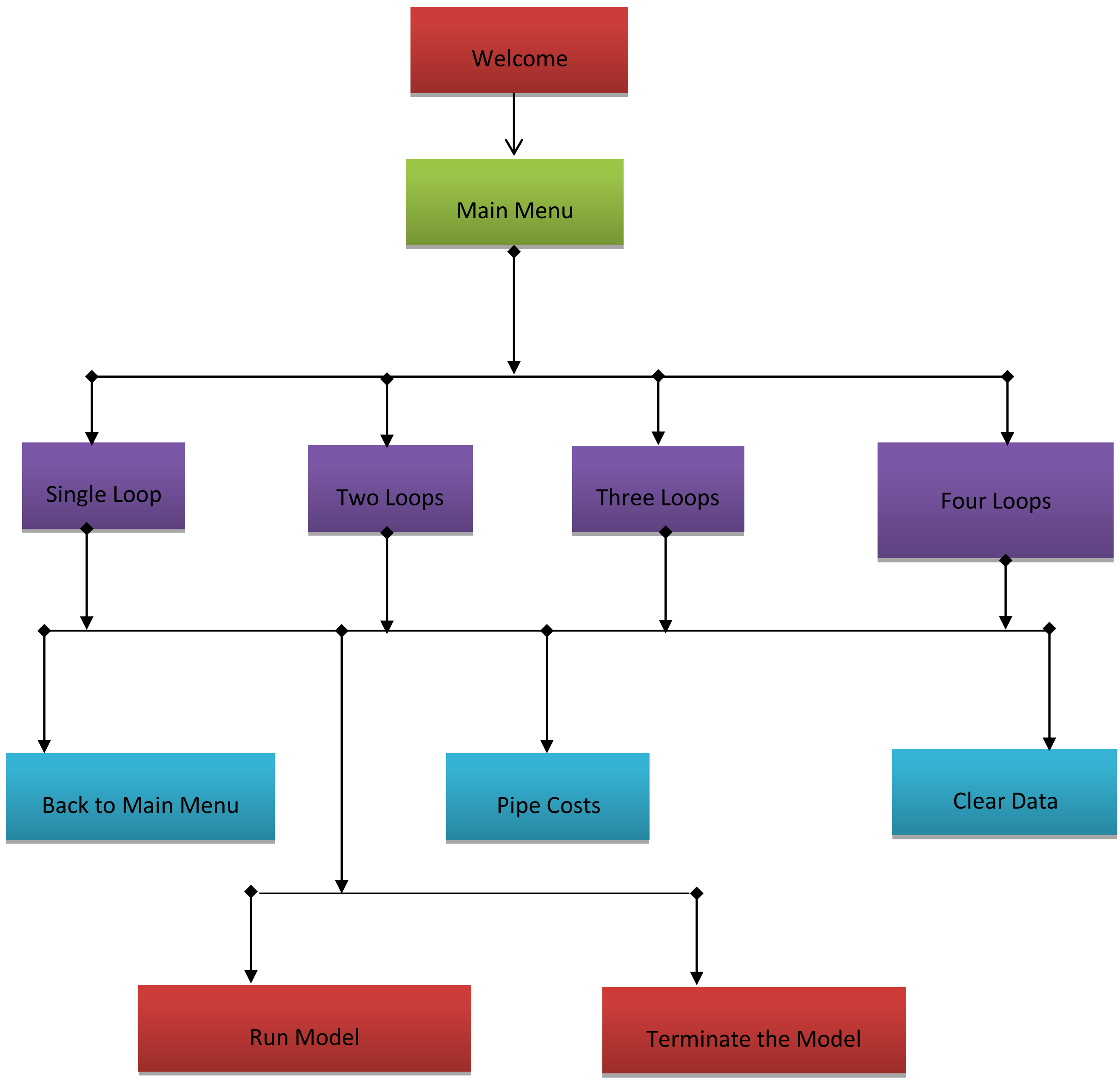


Figure 3-2: System Architecture Hierarchy

3.3 Implementation/Coding

At this phase, just half-way through the stages along the V-Model program development methodology, definite coding and implementation started. During this period, ample time was allotted to transform all the preceding steps into a coded, working model. In this study, an improved Hardy Cross Algorithm (Epp & Fowler, 1970) was modelled and implemented in Vb6.0.

3.3.1 Mathematical Formulation of Improved Hardy Cross Algorithm

As aforementioned in chapter 2, Hardy Cross (HC), a structural engineering professor at the University of Illinois (Cross, 1936) authored “the head balancing scheme” to examine the closed loop water pipelines. The original algorithm of HC has found much acceptability in industries. The HC method adheres to two (2) important laws: [1] the net flow at each junction must be equal to zero (0). [2] The head loss around a closed loop must be equal to zero. The algorithm operates on an initial guess of discharges in each conduit that must fulfill the first law of Kirchhoff. There is a number of iterative steps executed in the algorithm until both conservation principles are satisfied. One important feature to take note of is that, in the original HC method, the corrective flows $\{\Delta Q_{Cor}\}$ are determined separately and applied to obtain the next flow rates. The ease with which the corrective flows are independently obtained using the classic HC technique motivated most universities around the world to extensively teach this so called “single path adjustment algorithm” in their engineering faculties. However, the rationale of separately obtaining corrective flows somehow renders the algorithm slow. Therefore, it was until 1970 when Epp and Fowler suggested fundamental modifications to the original HC method to increase its convergence speed. This new modified approach simultaneously resolves the entire closed loop pipe network (Brkic, 2011). The proposed improved Hardy Cross scheme (Epp & Fowler, 1970) sometimes referred to as the “simultaneous path adjustment method” is a kind of Newton-Raphson approach that uses a matrix technique to simultaneously solve for unknown corrective flow rates.

The original HC formula determines the unknown flow modification factor as:

$$\Delta Q_k = \frac{-\sum K_j Q_{oj}^{n}}{\sum nK_j |Q_{oj}^{n-1}|} \qquad \ldots (3.1)$$

From Darcy-Weisbach formula;

$$K_j = {f_j L_j}/{2gD_j A_j^{\,2}} = {f_j L_j}/{12.106 D_j^{\,5}} \qquad \ldots (3.2)$$

Where; L_j=Length of pipe j, D_j=Diameter of pipe j, and f_j = Darcy-Weisbach friction factor of pipe j.

In which, f_j, can be explicitly computed by the modified Barr (1981) equation (Featherstone & Nalluri, 1995) expressed in the form:

$$\frac{1}{\sqrt{f_j}} = -2log_{10}\left(\frac{\varepsilon_j}{3.7D_j} + \frac{4.518\log\left(\frac{R_{ej}}{7}\right)}{R_{ej}\left(1+\frac{R_{ej}^{0.52}}{29}\left(\frac{\varepsilon_j}{D_j}\right)^{0.7}\right)}\right) \qquad \ldots (3.3)$$

Where;

${\varepsilon_j}/{D_j}$ = the relative roughness of pipe j

And, R_{ej} is the Reynolds number for pipe j solved by the subsequent equation (3.4)

$$R_e = {V_j D_j}/{\vartheta} \qquad \ldots (3.4)$$

Where;

V= The flow velocity in pipe j in (m/s),

D = Diameter of pipe j in (m) and,

ϑ = The kinematic viscosity in (m^2/s).

After obtaining the flow adjustment factor (ΔQ), it should be added to the initial flows to determine the new flows.

$$Q_{New1} = Q_o + \Delta Q_{loop1} \qquad \ldots (3.5)$$

In case of pipes common in both loops, the correction formula below will be applied

$$\mathrm{Q_{New1}} = \mathrm{Q_o} + \Delta\mathrm{Q_{loop1}} - \Delta\mathrm{Q_{loop2}} \qquad \ldots(3.6)$$

When implementing HC algorithm, for the starting successive iterations, conservation laws are likely not to be satisfied as the computed ΔQ will not make pressure drop around the closed loop zero albeit, it will reduce it closer to zero than it was in the previous iteration.

Expressing the original HC method (1936) (“single path correction method”) in a matrix arrangement (Brkic, 2011) would produce:

$$\begin{bmatrix} \sum n\frac{h_{11}}{Q_{11}} & 0 & 0 & 0 & \ldots & 0 \\ 0 & \sum n\frac{h_{22}}{Q_{22}} & 0 & 0 & \ldots & 0 \\ 0 & 0 & . & 0 & \ldots & 0 \\ . & . & . & . & . & . \\ . & . & . & . & . & . \\ 0 & 0 & 0 & 0 & \ldots & \sum n\frac{h_{mk}}{Q_{mk}} \end{bmatrix} x \begin{bmatrix} \Delta Q_1 \\ \Delta Q_2 \\ , \\ . \\ . \\ \Delta Q_k \end{bmatrix} = -\begin{bmatrix} \sum \mathrm{h}_1 \\ \sum h_2 \\ . \\ . \\ . \\ \sum h_k \end{bmatrix} \qquad \ldots(3.7)$$

(Epp & Fowler, 1970) revised the original HC method (Cross, 1936) by substituting the zeroes (0) in the off-diagonal of Eq. (3.7) with the first derivative of the head loss for pipes common in two loops Eq. (3.8). The work of Epp and Fowler (1970) led to what is today commonly known as the “modified HC Algorithm” which is capable of simultaneously solving for the flow rectification factor.

$$\begin{bmatrix} \sum n\frac{h_{11}}{Q_{11}} & -n\frac{h_{12}}{Q_{12}} & \ldots & -n\frac{h_{1k}}{Q_{1k}} \\ -n\frac{h_{21}}{Q_{21}} & \sum n\frac{h_{22}}{Q_{22}} & \ldots & -n\frac{h_{2k}}{Q_{2k}} \\ . & . & . & . \\ . & . & . & . \\ . & . & . & . \\ -n\frac{h_{m1}}{Q_{m1}} & -n\frac{h_{m2}}{Q_{m2}} & \ldots & \sum n\frac{h_{mk}}{Q_{mk}} \end{bmatrix} x \begin{bmatrix} \Delta Q_1 \\ \Delta Q_2 \\ , \\ . \\ . \\ \Delta Q_k \end{bmatrix} = -\begin{bmatrix} \sum \mathrm{h}_1 \\ \sum h_2 \\ . \\ . \\ . \\ \sum h_k \end{bmatrix} \qquad \ldots(3.8)$$

3.3.2 Developing a VB6.0 Program

There are a number of steps that the researcher undertook to create a VB6.0 project:

Step 1: The researcher used the forms onto which the object controls like button, picture box, among others were drawn.

Step 2: After which the form module was used to write the program or source code

Step 3: The program was then compiled using a VB compiler (a special program that transforms programs written in a high-level language such as VB into machine code (low-level language) that a computer comprehends). The compiler examines a program written in a language such as VB and transforms it into a form that is readable by a computer system (Khamkham, 2000).

Step 4: Eventually, the program was executed, tested on a case study pipe network and validated

3.3.3 Compiler hardware and software Requirements

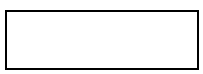
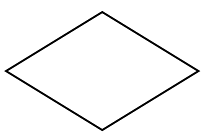
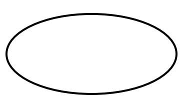
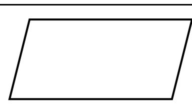
The compiler used in this project is the Microsoft visual basic compiler. The Microsoft VB compiler package provides a comprehensive IDE for creating all windows application. The compiler can be installed on Intel based Personal Computers (PCs) with Pentium processor or newer version, running Windows 95, 98 or Windows NT 4 or higher version (Khamkham, 2000).

3.3.4 Algorithm Flow Chart

As aforementioned, the algorithm implemented in this study is improved HC algorithm to solve single (1), two (2), three (3) and four (4) loops. Unlike in the previous studies such as (Yengale, et al., 2012; Demir, et al., 2008) where the user is required to first approximate the initial flow rates, in this model, the user just needs to enter the population or base demand data at the network nodes and per capital water demand. The program will then automatically guess the initial discharges for each pipe in the network while maintaining continuity. The model also requires the user to key in the nodes elevation data, pipe length, diameter, and loss coefficients. Following that, the program will proceed to compute the actual flow rates in pipe k, pressure head at node i, velocity in pipe k, head losses in pipe k and total head losses around loop j. The model will check whether or not the number of iterations is equal to the maximum iteration. The subsequent illustrations below represent the algorithms of the program for a single loop (figure 3-3) and multiple loops (figure 3-4) respectively.

3.3.4.1 Logic design

Table 3-1: Key Symbols used in HC Algorithm Flow Charts

Name	**Symbol**	**Description**
Data flow	→	This shows the movement of data through the system
Process		This transforms data from inputs to produce outputs
Decision box		This makes a decision depending on query. It is usually limited to only two options; YES and NO
Initiator or terminator		This shows the start and stop of the program
Input or Output box		This reads the input data into the program and prints the output.

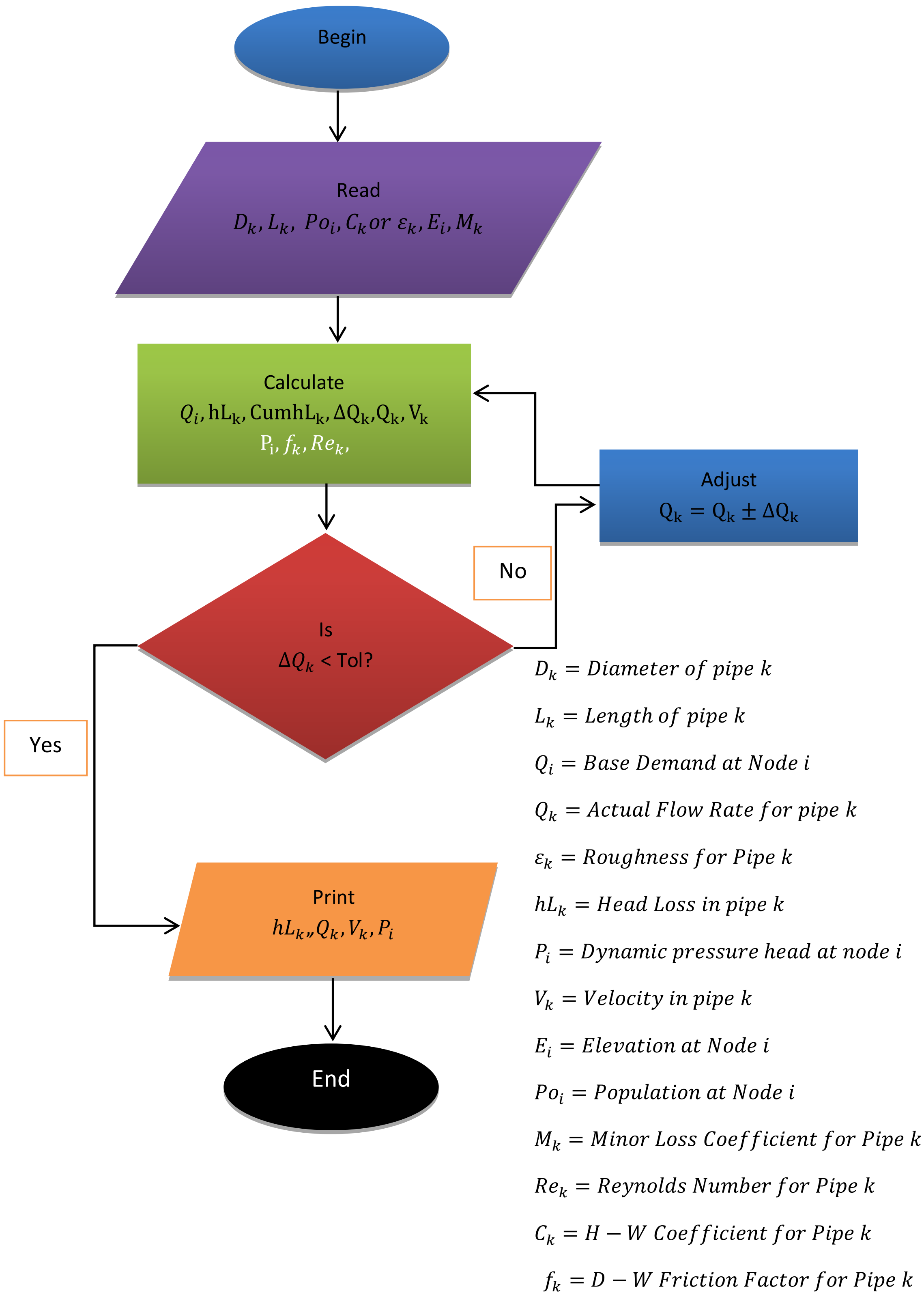
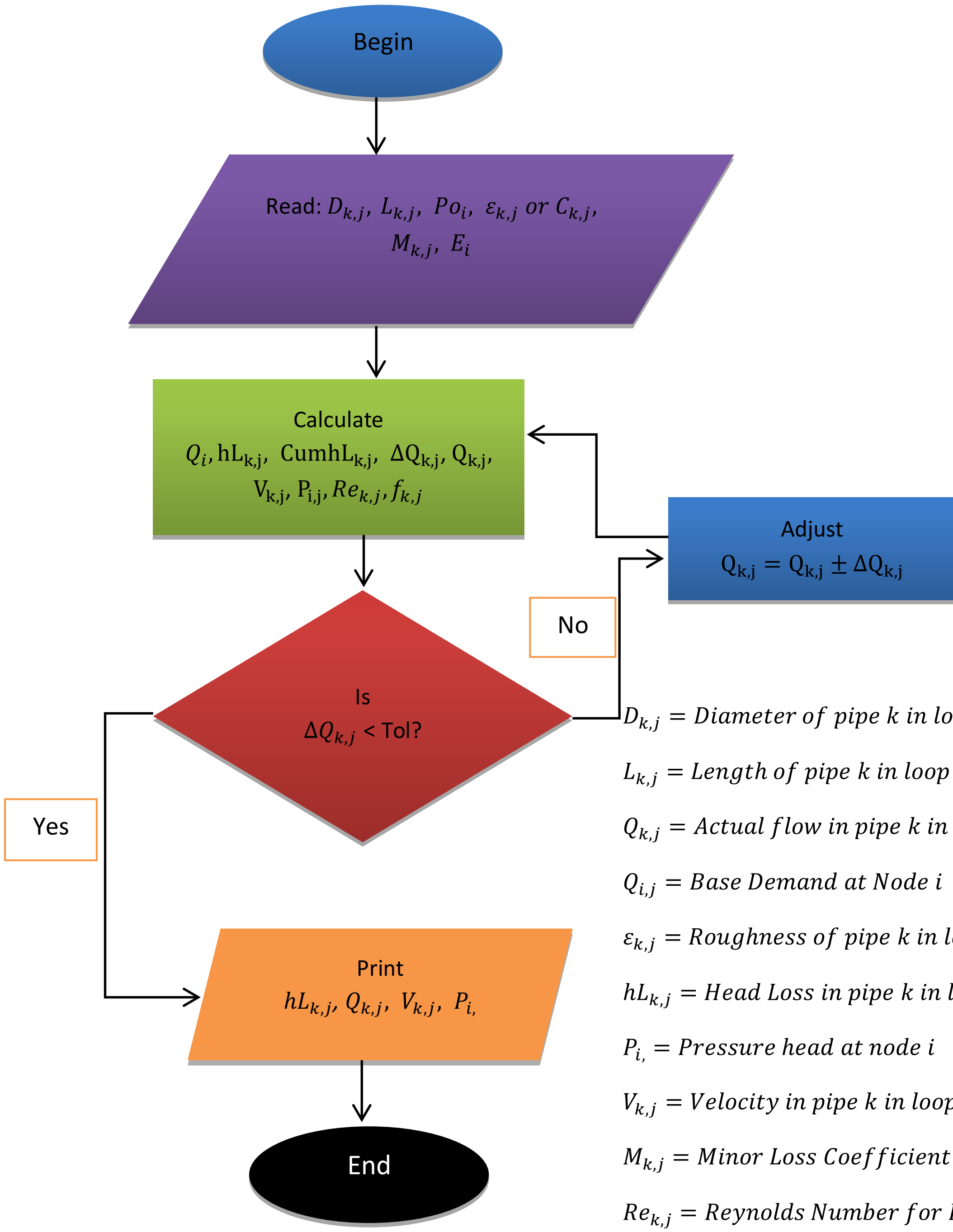


Figure 3-3: Flow Chart of HC Algorithm for a Single Loop



$D_{k,j} = Diameter\ of\ pipe\ k\ in\ loop\ j$

$L_{k,j} = Length\ of\ pipe\ k\ in\ loop\ j$

$Q_{k,j} = Actual\ flow\ in\ pipe\ k\ in\ loop\ j$

$Q_{i,j} = Base\ Demand\ at\ Node\ i$

$\varepsilon_{k,j} = Roughness\ of\ pipe\ k\ in\ loop\ j$

$hL_{k,j} = Head\ Loss\ in\ pipe\ k\ in\ loop\ j$

$P_{i,} = Pressure\ head\ at\ node\ i$

$V_{k,j} = Velocity\ in\ pipe\ k\ in\ loop\ j$

$M_{k,j} = Minor\ Loss\ Coefficient\ for\ Pipe\ k$

$Re_{k,j} = Reynolds\ Number\ for\ Pipe\ k\ in\ loop\ j$

$C_{k,j} = H - W\ Coefficient\ for\ Pipe\ k$

$f_{k,j} = D - W\ Friction\ Factor\ for\ Pipe\ k$

$E_i = Elevation\ at\ Node\ i$

$Po_i = Population\ at\ Node\ i$

Figure 3-4: Flow Chart of HC Algorithm for Multiple Loops

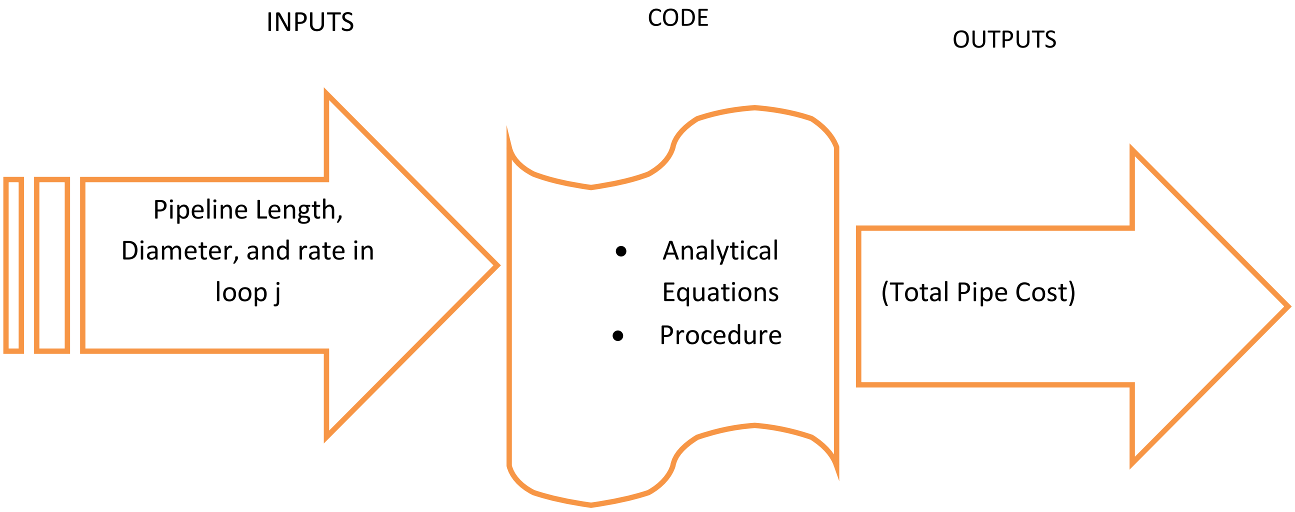


Figure 3-5: Pipe Cost Estimation Flow Chart

3.4 Testing, Verification and Validation

3.4.1 Testing

All the modules developed in the implementation phase were integrated into a single system. Several testing techniques were employed and these include testing of each unit, integration testing of combined units and post integration of the entire system where errors and failures were noticed and corrected. The techniques of testing commonly used in ADLC are detailed below;

- **Unit Testing:** This method of separately testing individual modules helped in correcting all errors (bugs) in the code to make certain each module fulfills its functional requirements.
- **System Integration Testing:** This process was used to ensure that the individual modules are properly integrated and working in harmony.

3.4.2 Verification

One of the central requirements for any software acceptability is reliability and accuracy. To ensure this, the model was tested on a case study water network. The output from the model was compared with EPANET software solution to determine any discrepancies.

3.4.3 Validation Method

In order to assess the reliability of the built numerical hydraulic model and prove whether there is any fundamental inclination in its performance, statistical study including the coefficient of determination (R^2), RMSE and MBE were adopted. The higher the R^2, the more accurate is the estimation. The expression for the aforementioned statistical parameter is in form:

$$R^2 = 1 - \frac{\sum\left(q_{E,k} - q_{p,k}\right)^2}{\sum\left(q_{E,k} - \frac{\sum q_{m,k}}{N}\right)^2} \qquad \text{...(3.9)}$$

Where $q_{p,k}$ is the program solution, $q_{E,k}$ is the EPANET solution and N is the number of observations.

$$RMSE = \sqrt{1/_N \sum_{k=N}^{N}(q_{cal} - q_{obs})^2} \qquad \text{...(3.10)}$$

$$MBE = \frac{1}{N}\sum_{k=1}^{N}(q_{cal} - q_{obs}) \qquad \text{...(3.11)}$$

Where; q_{obs} =observed discharge (for this research study obtained EPANET) , q_{cal} = Calculated discharge from program and $\overline{q}_{cal}$ =mean of Calculated discharge

3.5 Documentation and Reporting

For every creative and innovative research on system development, documentation report is a substantial prerequisite. Hence, this study was carefully documented from start to end for future reference and further research in the field of hydraulic network studies. The documentation covered problem formulation, setting of objectives and overall project plan, data collection, model conceptualization, model translation, verification, validation, findings, discussion and recommendation.

Chapter 4 : Computer Program Description and Testing

4.0 Introduction

Every computer software is always accompanied by the user manual which describes fully the software (that is, what the software is capable of doing and how to use it). This is done to ensure the user is well acquainted with the software package. Therefore, this chapter aims to describe the functionalities and the use of the developed user friendly hydraulic model for closed loop network.

The computer program herein developed has been written in VB6.0 language, to solve for flow rates, piezometric heads, head losses, and velocities using the improved HC algorithm.

Essentially the computer model reads input data defining the network links (pipes) and junction nodes.
A number of key things needs to be noted about this hydraulic model:

- ✓ Depending on the data at the user's disposal, the program can read in either node population data and compute the base demand plus assumed initial discharge in each pipe or it can read in base demand data at the nodes and then compute the initial discharge for each pipe in the closed loop.
- ✓ One type of liquid, essentially water is applied to this model.
- ✓ The head loss equations for simulation are: Darcy-Weisbach and Hazen-Williams formulae.
- ✓ Computation continues until a tolerance of 0.00001 is achieved.

4.1 Tasks Executed by the Model

1) Read the input data that define the network.
2) From the input data, assume initial flow rates for each pipe in the network while obeying principal of continuity at each node and the sign convention for the flows in the network.

3) Determine the sign convention without the user having to do it by themselves. There is a tendency of the users forgetting to assign the flow direction sign convention to the initially guessed pipe flows while working with the HC Method. Usually clockwise direction is assigned (+) and counter clockwise direction assigned (-). It should be noted that while determining the initial pipe discharges, the model keeps in memory the sign convention. The model will assign a negative (-) sign to the initial pipe flow in the counter clockwise direction and a positive (+) sign to the initial flow in the clockwise direction.
4) Make use of the improved HC Algorithm to analyse the closed loop network made up of a single loop, two loops, three loops and four loops.
5) Obtains the head loss at each pipe after the pipes flow rates have been found
6) Display the solution results in tables that can be readily understood.
7) And eventually evaluate the total cost of the network pipelines.

4.2 Model Input Variables

The data requirements for the program are as follows:

4.2.1 Pipe data

For each pipe in the closed loop network, diameter, length, and pipe roughness are required. If the pipe has any minor loss device specifically a valve, the number of valves and the value of minor loss coefficient needs to be fed in.

4.2.2 Junction data

For each junction in the closed loop network, the input data necessary to describe every junction is keyboarded in.

They are as follows:

1- First, the population or base demand (m^3/s) data is keyed in by the user. If for example the user enters population data at the nodes, the software will go ahead to compute the base demand plus initially assumed flow rates. The unit for base demand is in cubic meters per second (m^3/s). *In cases where you are faced with the external flow (Base Demand) into the junction node, then a negative (-) sign should be assigned to it by the user.*

2- Second, the elevation data at each node is fed in. This data is required to proceed with pressure head calculation. However, class room examples without elevation data can still be solved by the model but the user eventually receives "WARNING messages" which can be ignored. Elevation is in meters and the calculated pressure heads also in meters of water.

4.3 Starting/Running the Program

1- Go to windows search and type "PAU-NET".

2- Click on its Icon to execute the program. Figure 4-1 will appear on the screen. It is called the "welcome" window.

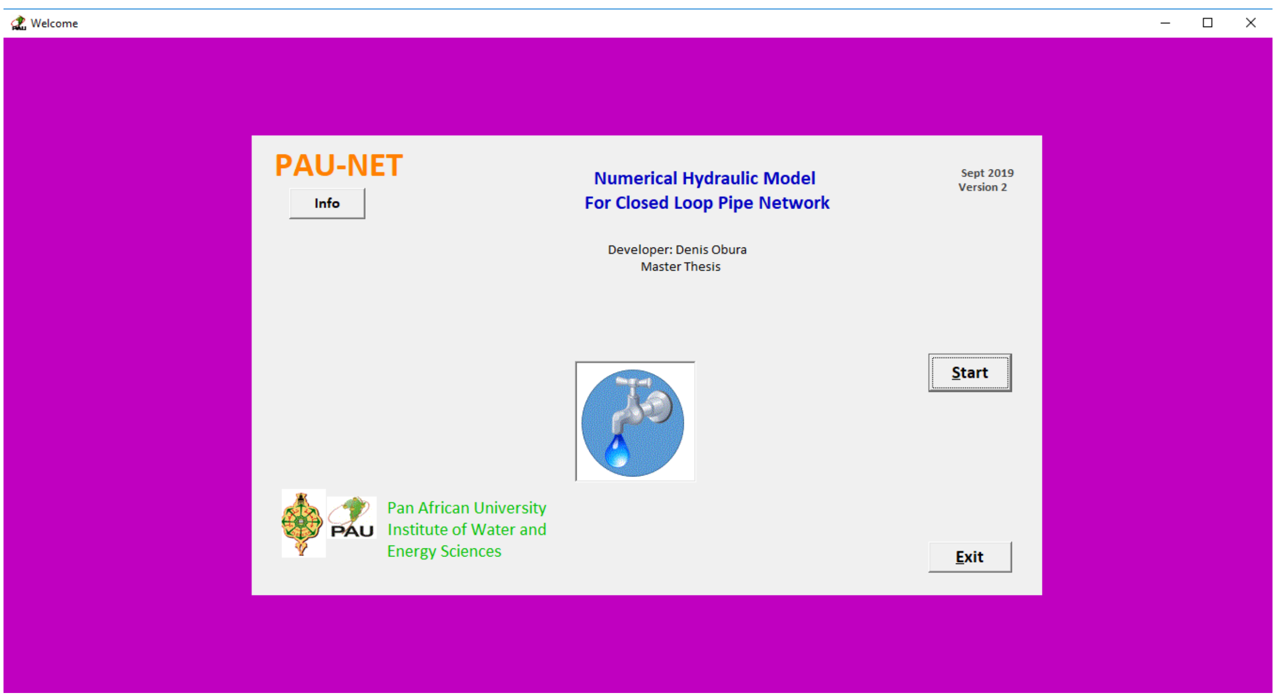


Figure 4-1: Welcome Window Interface

3- When you click on “Info” button, the form/window in figure 4-2 below is displayed. This window gives the user an introduction to the program, that is, what the program is all about, and the developer.

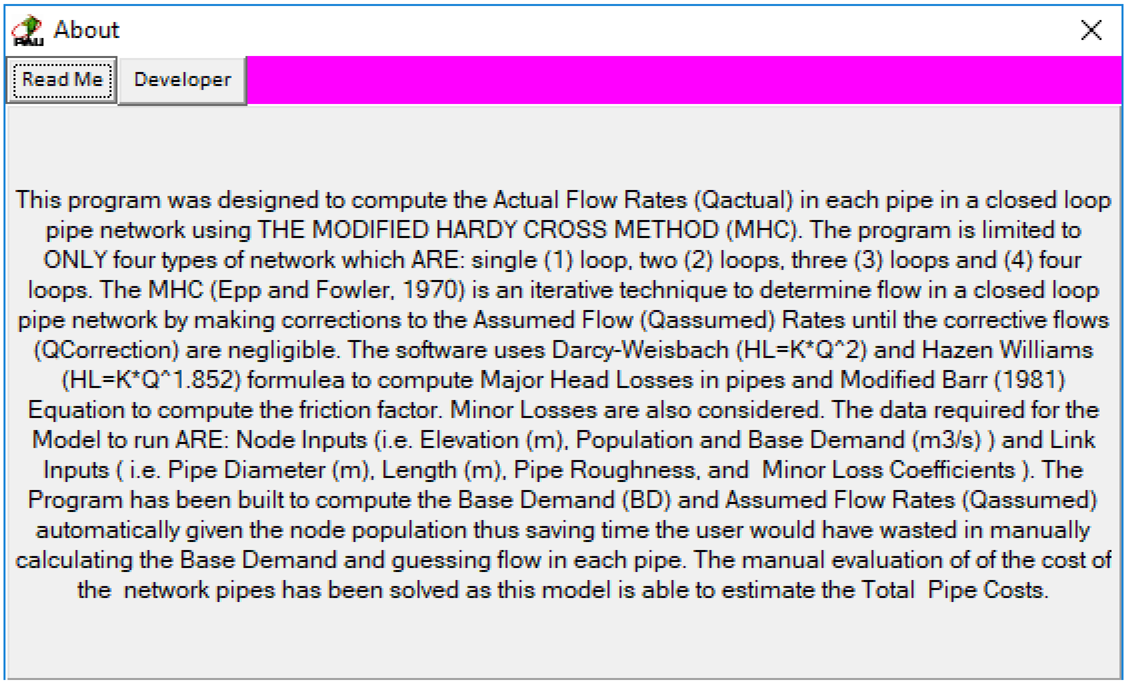


Figure 4-2: Introduction Form

4- Prior to performing analysis, click on the “Start” button and the "Main Menu” form (figure 4-3) below will appear. Selection of data type, head loss formula and loop to work with is done from this window.

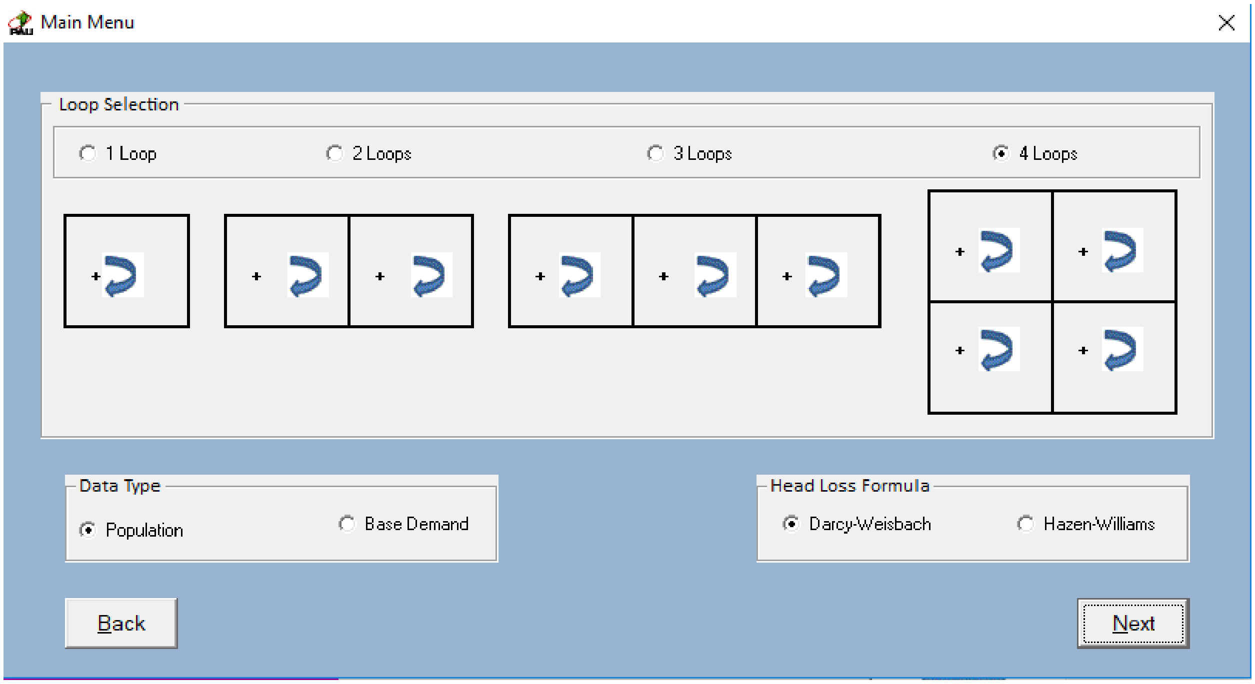


Figure 4-3: Main menu Interface

5- Click on the “Back” button on the “Main Menu” window to go back to the “Welcome” window (figure 4-1).

6- Click “Next” after selecting a particular loop, data type and head loss formula to execute network analysis. The computation form will then appear with a message dialog box informing the user that the current inputs are just default values. Therefore, the user needs to input new data to perform analysis (figures 4-4 to 4-7).

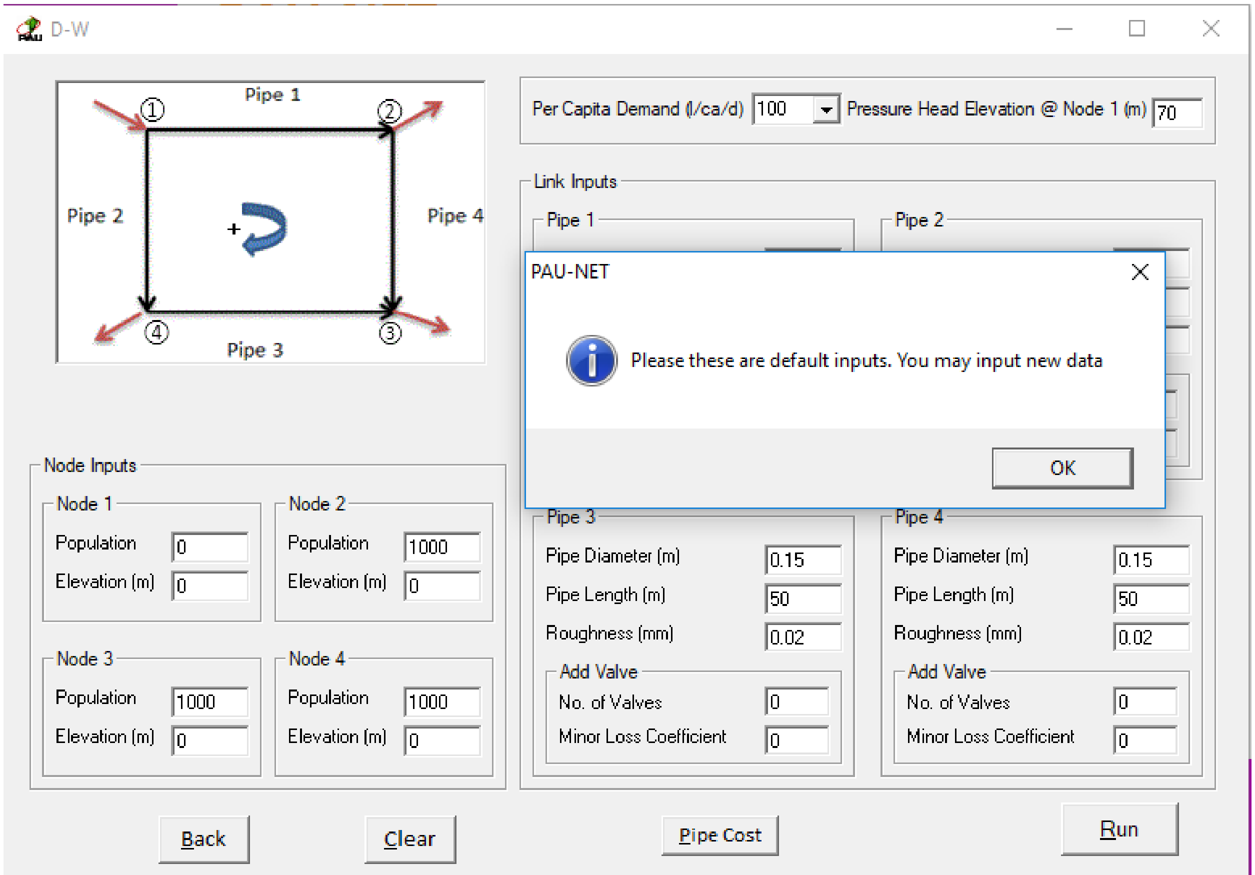


Figure 4-4: Computation Interface for Single (1) Loop

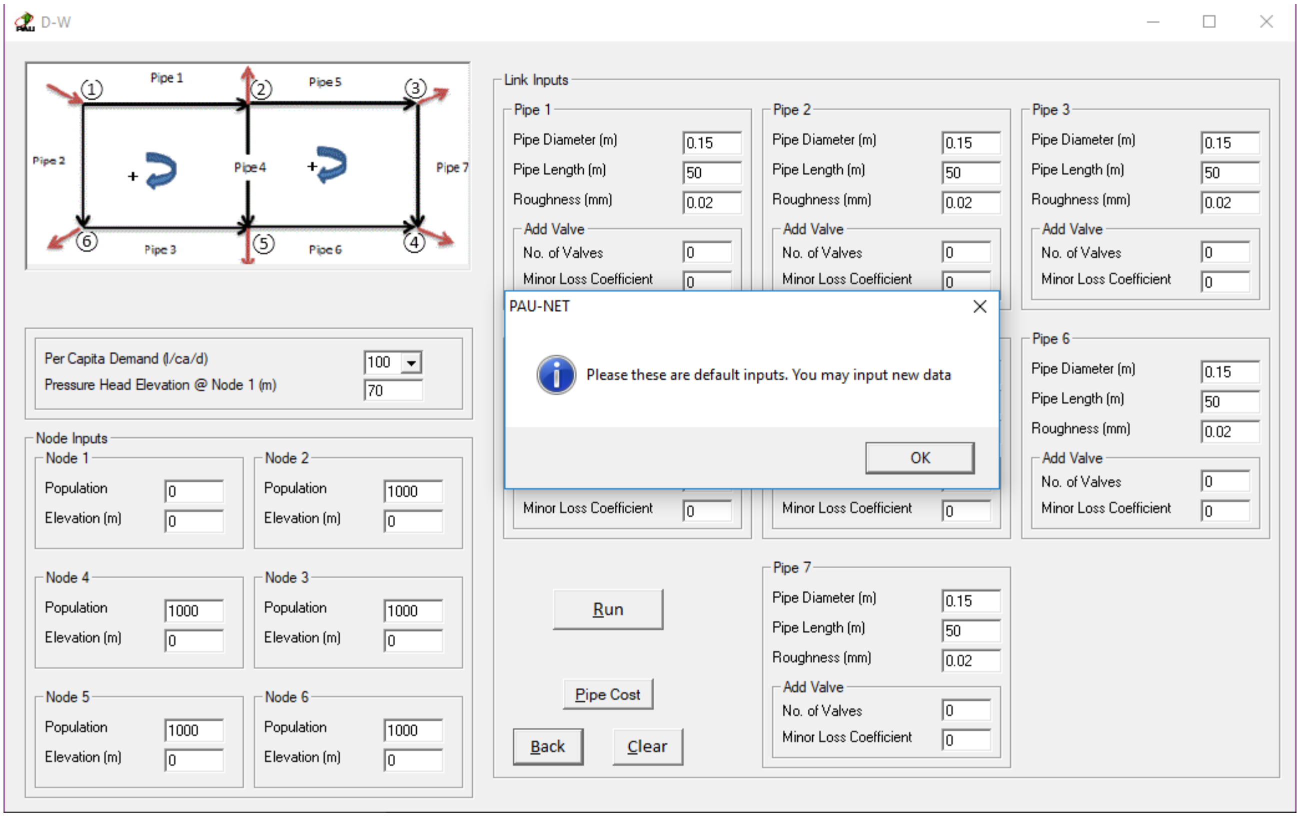


Figure 4-5: Computation interface for 2 loops

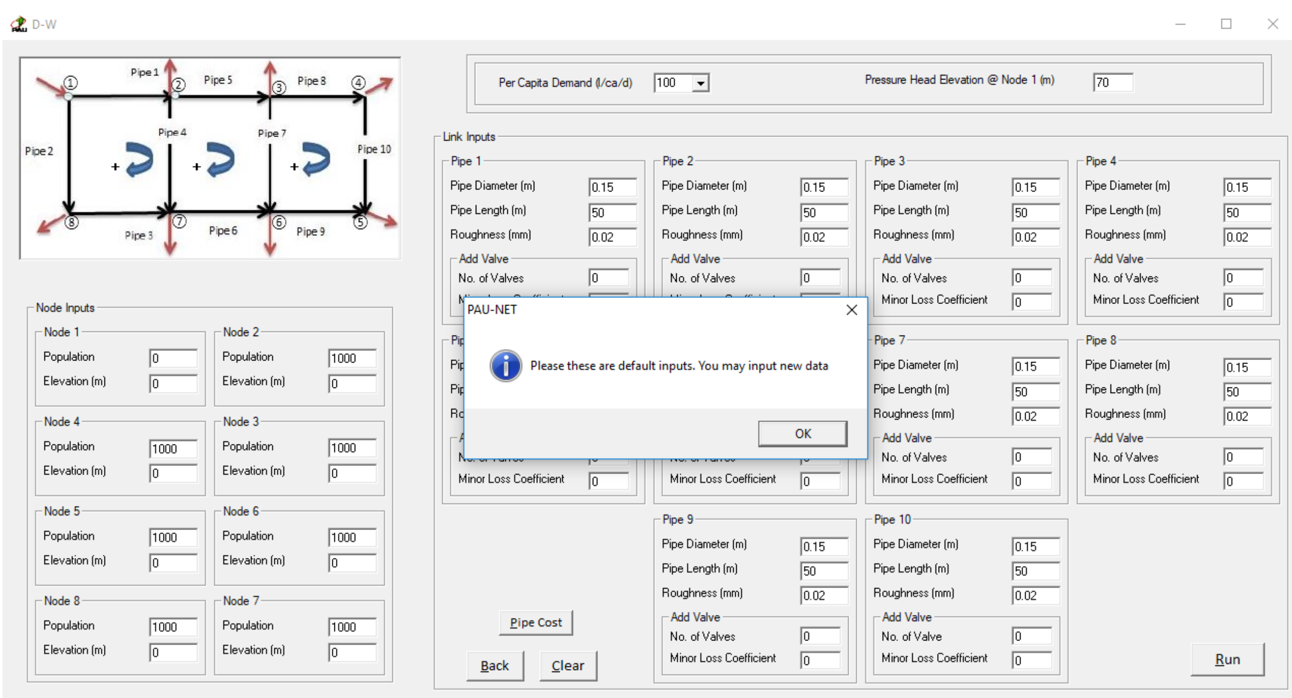


Figure 4-6: Computation Interface for 3 Loops

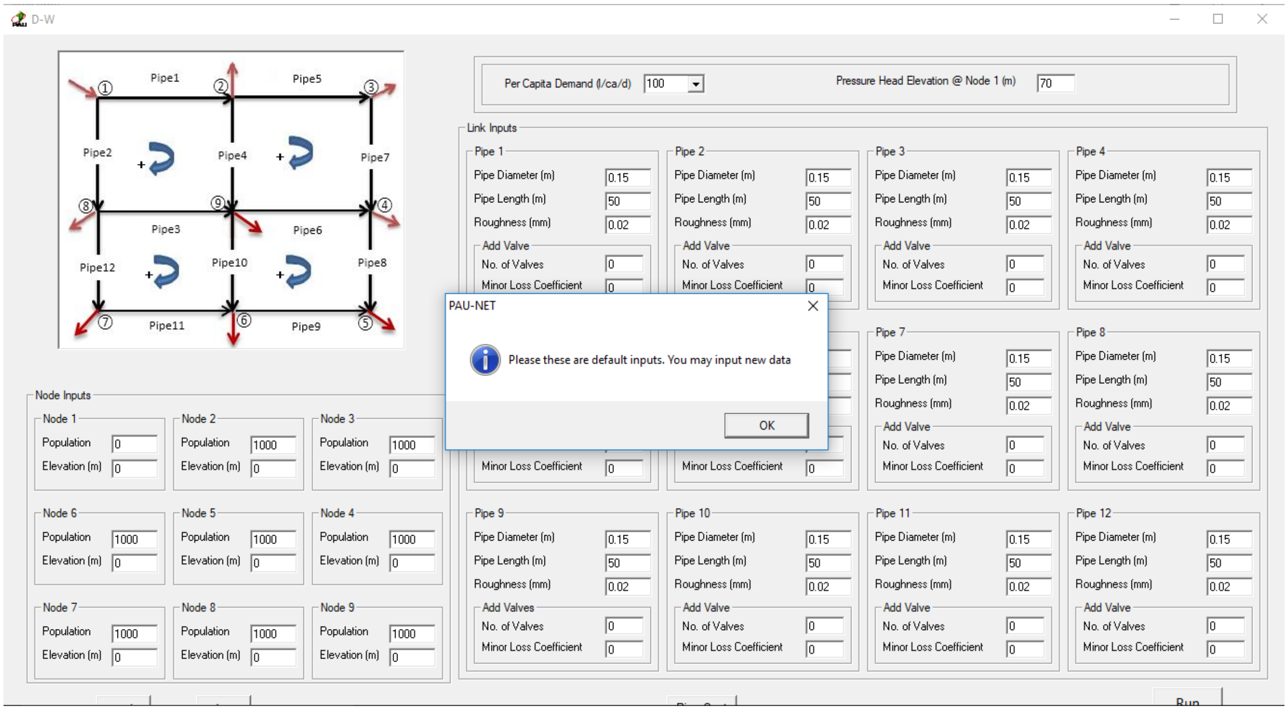
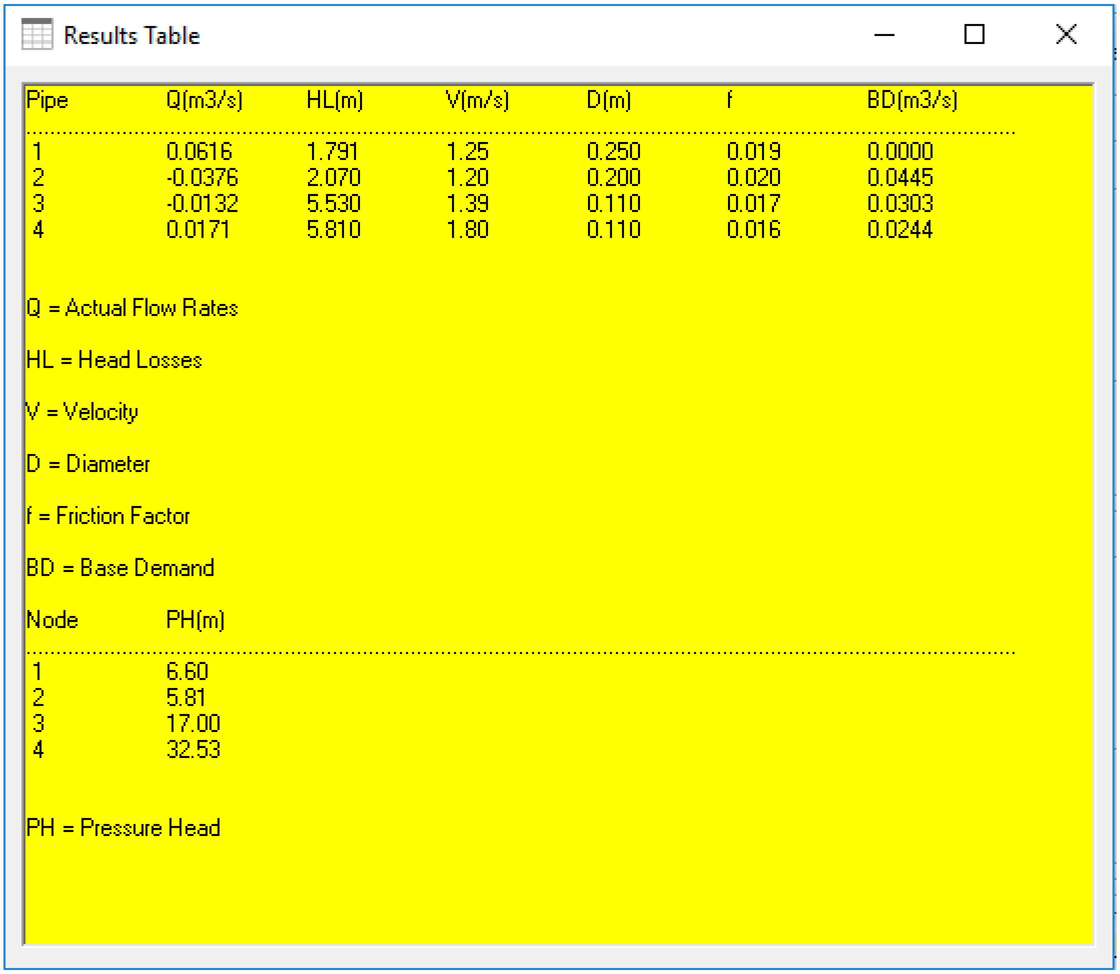


Figure 4-7: Computation Interface for 4 Loops

7- Enter both the node and the link data into the computation interface then click “Run” button to perform the loop analysis.

NB: *In cases where you are faced with the external flow (Base Demand) into the junction node, then a negative (-) sign should be assigned to it by the user when keying in the data.*

8- The analysis results are displayed in the table of results interface. After running the analysis, the “Results Table” interface will turn yellow in color (figure 4-8). The negative (-) flow output is okay. It describes a flow that is in the anti-clockwise direction. The flow in the clockwise direction maintains a positive (+) sign convention


Results Table

Pipe	Q(m3/s)	HL(m)	V(m/s)	D(m)	f	BD(m3/s)
1	0.0616	1.791	1.25	0.250	0.019	0.0000
2	-0.0376	2.070	1.20	0.200	0.020	0.0445
3	-0.0132	5.530	1.39	0.110	0.017	0.0303
4	0.0171	5.810	1.80	0.110	0.016	0.0244

Q = Actual Flow Rates

HL = Head Losses

V = Velocity

D = Diameter

f = Friction Factor

BD = Base Demand

Node	PH(m)
1	6.60
2	5.81
3	17.00
4	32.53

PH = Pressure Head

Figure 4-8: Results Table Interface

9- After running the analysis, you may want to know how much your network pipes will cost. By clicking "Pipe Cost" button, the "Pipe cost" interface will pop up. Enter the cost per unit length then click "cost" to perform costing.

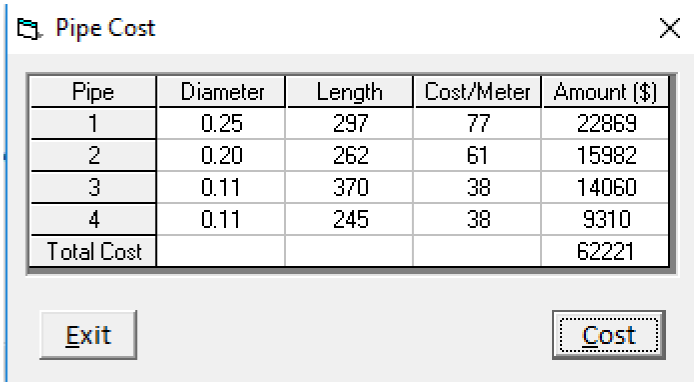


Figure 4-9: Pipe Cost Interface

4.4 Model Testing

The testing of the built program was conducted on a single (1), two (2), three (3) and Four (4) loop case study area water network. The four loop network represents the case study area. The different networks for each loop with the collected input data are presented in the below. The figure of per capita demand used in the analysis was 100 l/ca/d.

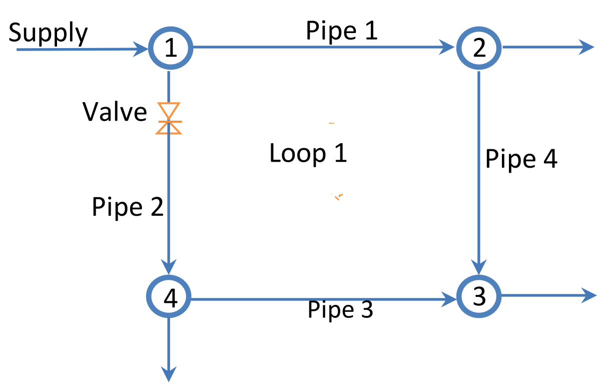


Figure 4-10: A single[1] loop, 4 Pipe Network Problem

Table 4-1:The Pipe Input Data for a Single[1] Loop Network Problem

Pipe	Pipe Type	Pipe Length[m]	Pipe Diameter[mm]	No. of Valves	Minor Loss Coefficient	Pipe Roughness [mm]	Pipe Unit Cost [$]
1	GI	297	200	-	-	0.15	61
2	UPVC	262	90	1	2.3	0.0015	25
3	UPVC	370	90	-	-	0.0015	25
4	UPC	245	90	-	-	0.0015	25

Table 4-2: The Node Input Data for a Single[1] Loop Network Problem

Node	Population	Elevation [m]
1	0	48
2	10000	47
3	6400	30
4	5000	20

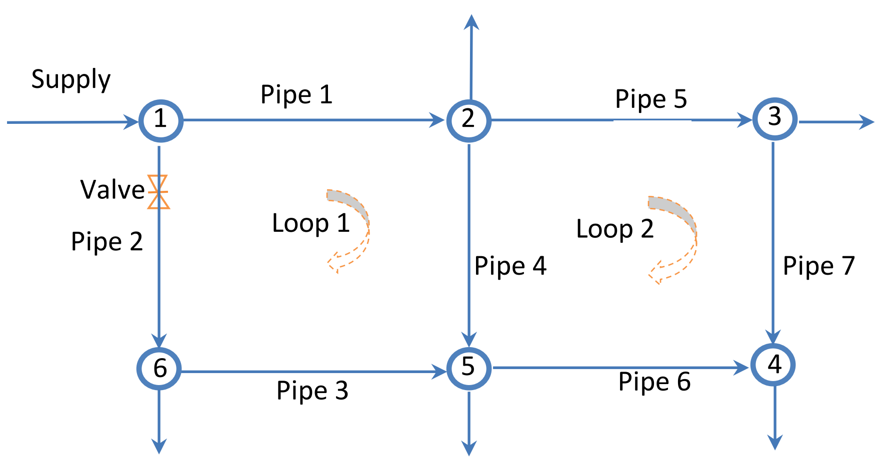


Figure 4-11: A 2 Loop, 6 Pipe Network Problem

Table 4-3: The Pipe Input Data for a 2 Loop Network Problem

Pipe	Pipe Type	Pipe Length[m]	Pipe Diameter[mm]	No. of Valves	Minor Loss Coefficient	Pipe Roughness[mm]	Pipe Unit Cost[USD]
1	GI	297	250	-	-	0.15	77
2	GI	262	200	1	2.3	0.15	61
3	UPV	370	110	-	-	0.0015	38
4	UPVC	245	110	-	-	0.0015	38
5	GI	128	150	-	-	0.15	46
6	UPVC	209	110	-	-	0.0015	38
7	UPVC	243	110	-	-	0.0015	38

Table 4-4: The Node Input Data for a 2 Loop Network Problem

Node	Population	Elevation [m]
1	0	48
2	4000	47
3	6110	44
4	2400	30
5	4000	30
6	5000	20

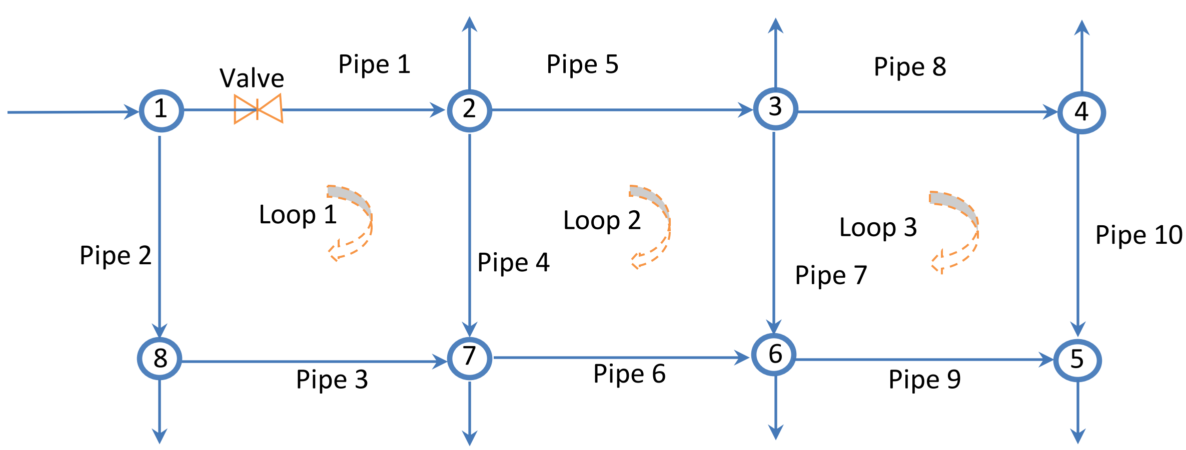


Figure 4-12: A 3 Loop, 10 Pipe Network Problem

Table 4-5: The Pipe Input Data for a 3 Loop Network Problem

Pipe	Pipe Type	Pipe Length[m]	Pipe Diameter[mm]	No. of Valves	Minor Loss Coefficient	Pipe Roughness[mm]	Pipe Unit Cost[USD]
1	GI	262	200	1	2.3	0.15	61
2	GI	297	250	-	-	0.15	77
3	UPVC	245	110	-	-	0.0015	38
4	UPVC	370	110	-	-	0.0015	38
5	UPVC	149	110	-	-	0.0015	38
6	UPVC	100	110	-	-	0.0015	38
7	UPVC	416	110	-	-	0.0015	38
8	UPVC	180	110	-	-	0.0015	38
9	UPVC	200	90	-	-	0.0015	25
10	UPVC	400	90	-	-	0.0015	25

Table 4-6: The Node Input Data for a 3 Loop Network Problem

Node	Population	Elevation [m]
1	0	48
2	1000	20
3	0	17
4	2100	14
5	2100	15
6	2400	9
7	4000	30
8	10000	47

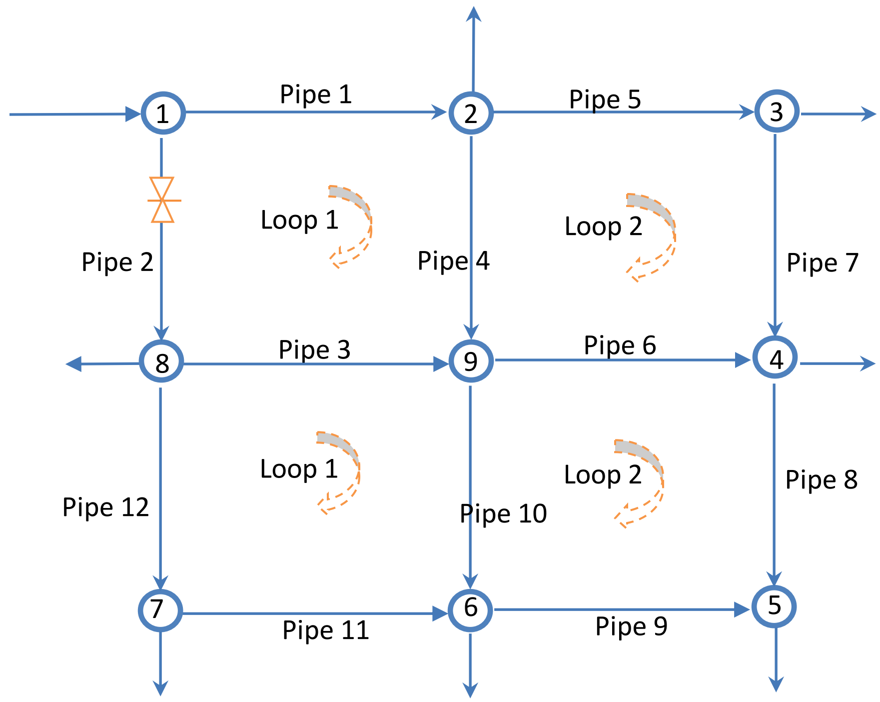


Figure 4-13: A 4 Loop, 12 Pipe Network Problem

Table 4-7: The Pipe Input Data for a 4 Loop Network Problem

Pipe	Pipe Type	Pipe Length[m]	Pipe Diameter[mm]	No. of Valves	Minor Loss Coefficient	Pipe Roughness[mm]	Pipe Unit Cost[USD]
1	GI	297	250	-	-	0.15	77
2	GI	262	200	1	2.3	0.15	61
3	UPVC	370	110	-	-	0.0015	38
4	UPVC	245	110	-	-	0.015	38
5	GI	128	150	-	-	0.15	46
6	UPVC	209	110	-	-	0.0015	38
7	UPVC	243	110	-	-	0.0015	38
8	UPVC	100	90	-	-	0.0015	25
9	UPVC	234	90	-	-	0.0015	25
10	UPVC	100	110	-	-	0.0015	38
11	UPVC	416	110	-	-	0.0015	38
12	UPVC	149	110	-	-	0.0015	38

Table 4-8: The Node Input Data for a 4 Loop Network Problem

Node	Population	Elevation [m]
1	0	48
2	4000	47
3	6110	44
4	0	30
5	2400	22
6	2100	9
7	2100	17
8	1000	20
9	4000	30

Chapter 5 : Results and Discussion

5.0 Introduction

This chapter aims at discussing and comparing the results of the developed user friendly hydraulic model with EPANET hydraulic tool. The first section presents and compares the results in a tabular format for one (1), two (2), three (3) and four (4). This chapter finally ends by presenting the summary of the chapter.

5.1 Results Comparison

The comparison of results produced from the program and EPANET software is done through tables 5-1 to 5-5 and the correlation through fig. 5-1, 5-4, 5-7, 5-10, as follows:

Table 5-1: Results of a single [1] Loop Network from the Program and EPANET

	Program Output			**EPANET Output**		
Pipe	Q_{New} (m^3/s)	HL (m)	V (m/s)	Q_{New} (m^3/s)	HL (m)	V (m/s)
1	0.0616	1.791	1.25	0.0616	1.791	1.25
2	0.0376	2.070	1.20	0.0376	1.907	1.20
3	0.0132	5.530	1.39	0.0132	5.491	1.39
4	0.0171	5.810	1.80	0.0171	5.775	1.80

Dynamic Pressure		
	Program Output	**EPANET Output**
Node	Pressure Head [m]	Pressure Head [m]
1	6.60	6.60
2	5.81	5.81
3	17.00	17.03
4	32.53	32.52

From Table 5-1 above, it can be clearly seen that the discharge results from both the program and EPANET software are consistent. The velocities are within the recommended range of (0.6-3.0) m/s. The pressures at the nodes are within the allowable range (2-60bars or 2m – 60m) (MWE, 2013). The head losses calculated in pipes 1 & 2 are 0.006 m/m and 0.008 m/m respectively. These values are less than the maximum allowable (0.01 m/m). Pipes 3 & 4 have 0.015 and 0.024 head loss per unit length values which are slighly higher than the recommended maximum allowable value.

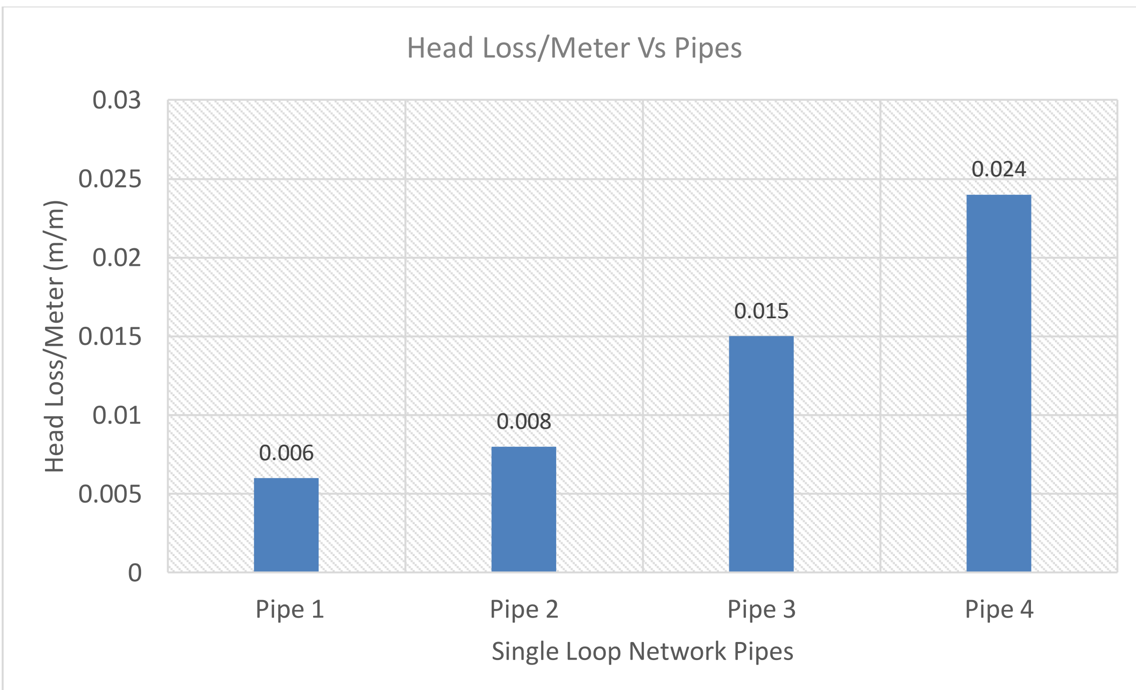


Figure 5-1: Head Loss Per Unit Length in a Single (1) Loop Network Pipes

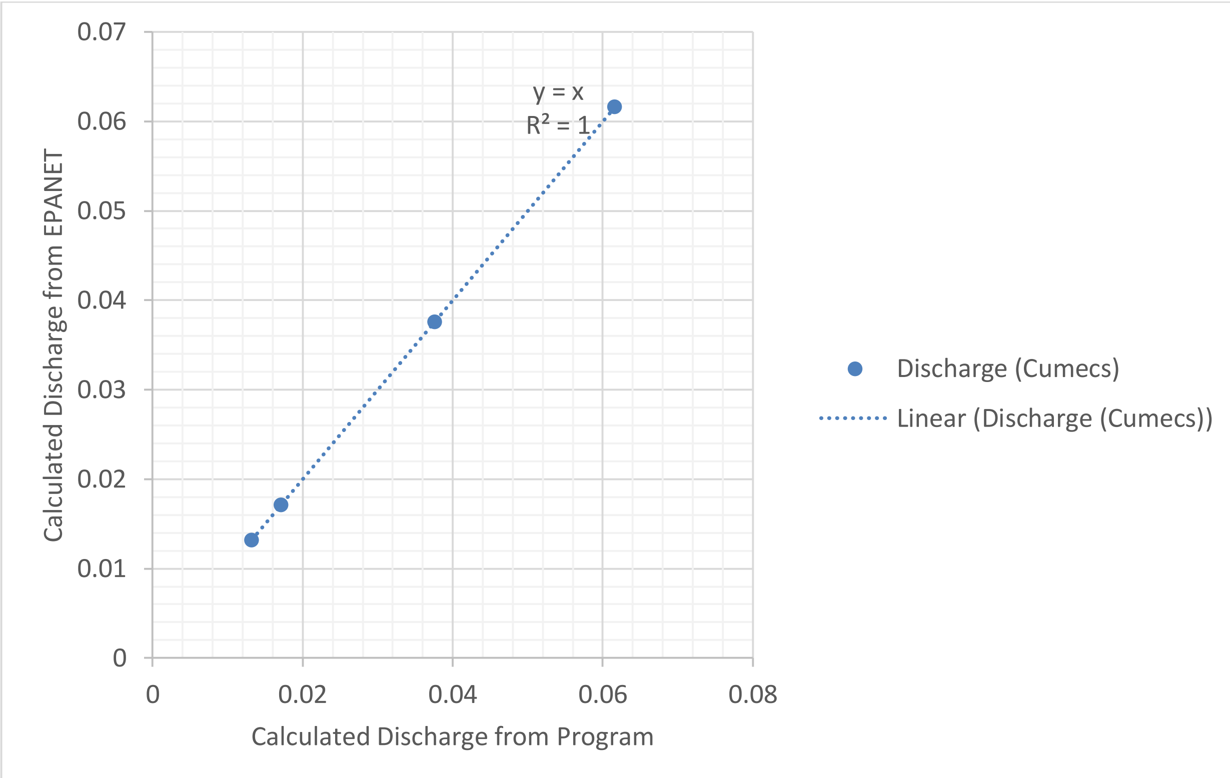


Figure 5-2: Comparison of a Single Loop Discharge Results from the Program and EPANET

Results Table

Pipe	Q(m3/s)	HL(m)	V(m/s)	D(m)	f	BD(m3/s)
1	0.0616	1.791	1.25	0.250	0.019	0.0000
2	-0.0376	2.070	1.20	0.200	0.020	0.0445
3	-0.0132	5.530	1.39	0.110	0.017	0.0303
4	0.0171	5.810	1.80	0.110	0.016	0.0244

Q = Actual Flow Rates

HL = Head Losses

V = Velocity

D = Diameter

f = Friction Factor

BD = Base Demand

Node	PH(m)
1	6.60
2	5.81
3	17.00
4	32.53

PH = Pressure Head

Figure 5-3: A single [1] Loop Network Results from the Program

Pipe Cost

Pipe	Diameter	Length	Cost/Meter	Amount ($)
1	0.25	297	77	22869
2	0.20	262	61	15982
3	0.11	370	38	14060
4	0.11	245	38	9310
Total Cost				62221

Exit   Cost

Figure 5-4: Cost of Pipes in a Single[1] Loop Network

Table 5-2: Results of a 2 Loop Network from the Program and EPANET

	Program Output			**EPANET Output**		
Pipe	Q_{New} (m^3/s)	HL (m)	V (m/s)	Q_{New} (m^3/s)	HL (m)	V (m/s)
1	0.0700	2.298	1.43	0.0700	2.305	1.43
2	0.0362	1.918	1.15	0.0362	1.769	1.15
3	0.0118	4.485	1.24	0.0118	4.466	1.24
4	0.0141	4.105	1.48	0.0141	4.087	1.48
5	0.0359	3.720	2.03	0.0359	3.724	2.03
6	0.0059	0.726	0.62	0.0059	0.729	0.62
7	0.0068	1.110	0.72	0.0068	1.091	0.72

Dynamic pressure		
	Program Output	**EPANET Output**
Node	Pressure [m]	Pressure [m]
1	6.60	6.60
2	5.30	5.30
3	4.58	4.58
4	17.47	17.48
5	18.20	18.21
6	32.68	32.68

From Table 5-2 above, the discharge results from both the program and EPANET software are the same which shows the reliability of model. The velocities in all pipes are within the recommended range (0.6-3.0) m/s. The pressure at all nodes for a two loop network are within the acceptable range (2bars – 60 bars) according to MWE, 2013 design guidelines for a water supply network. The head loss/meter in pipes 3, 4 and 5 are 0.012, 0.017 and 0.029 (figure 5-5) respectively which are slightly above the maximum allowable value.

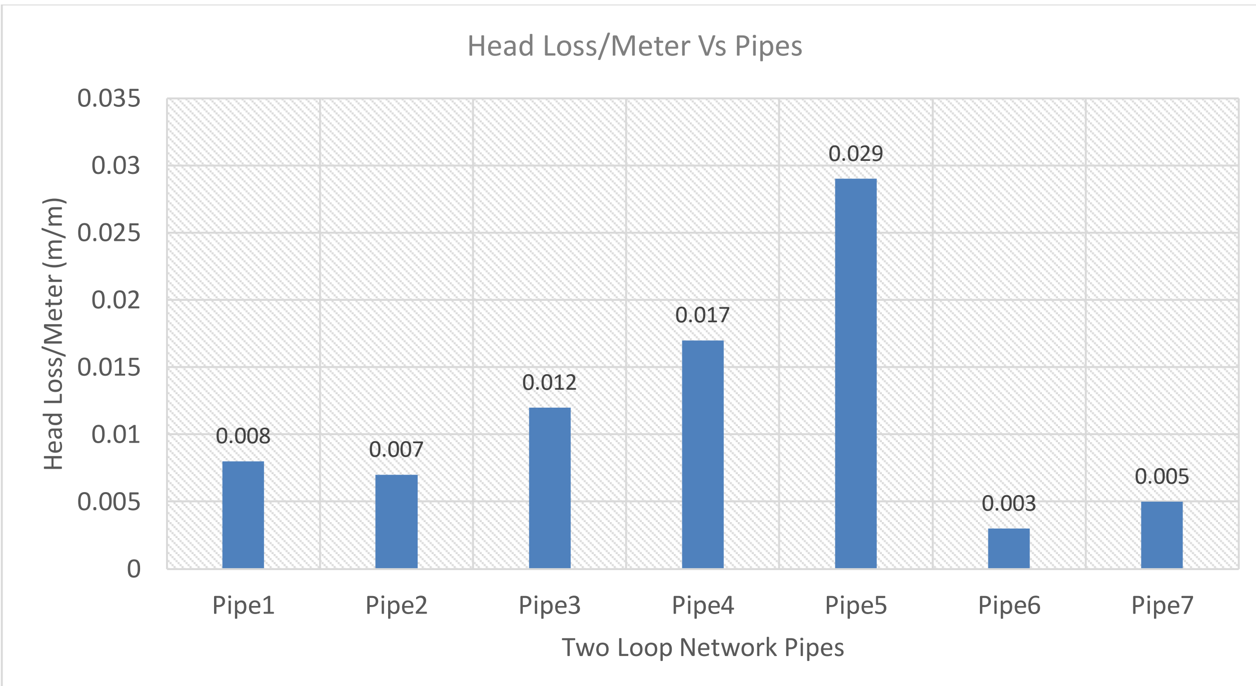


Figure 5-5: Head Loss Per Unit Length in a Two (2) Loop Network Pipes

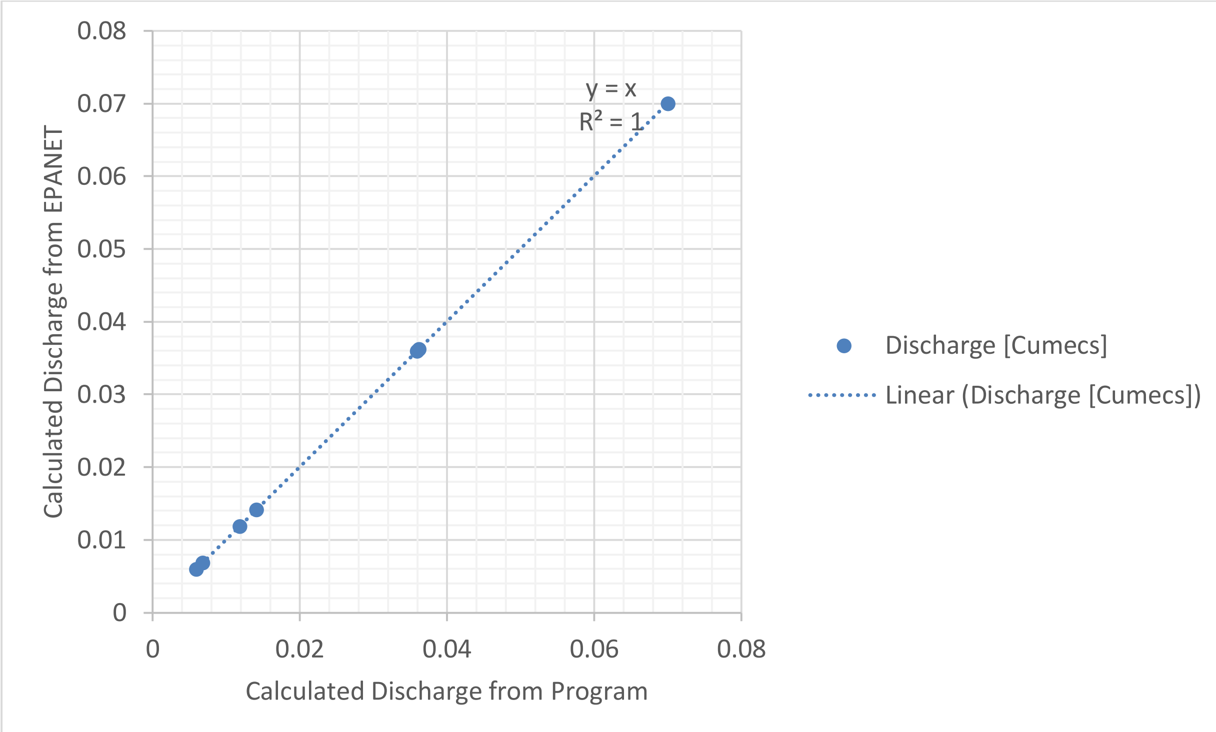


Figure 5-6: Comparison of a Two Loop Discharge Results from Program and EPANET

Results Table

Pipe	Q(m3/s)	HL(m)	V(m/s)	D(m)	f
1	0.0700	2.298	1.43	0.250	0.019
2	-0.0362	1.918	1.15	0.200	0.020
3	-0.0118	4.485	1.24	0.110	0.017
4	0.0141	4.105	1.48	0.110	0.016
5	0.0359	3.720	2.03	0.150	0.021
6	-0.0059	0.726	0.62	0.110	0.020
7	0.0068	1.110	0.72	0.110	0.019

Q = Actual Flow Rates

HL = Head Losses

V = Velocity

D = Diameter

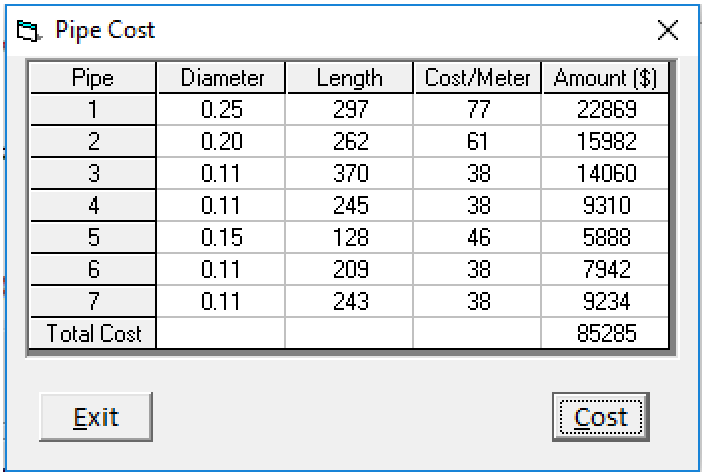
f = Friction Factor

Node	PH(m)	BD(m3/s)
1	6.60	0.0000
2	5.30	0.0200
3	4.58	0.0291
4	17.47	0.0127
5	18.20	0.0200
6	32.68	0.0244

PH = Pressure Head

BD=Base Demand

Figure 5-7: A 2 Loop Network Results from the Program



Pipe	Diameter	Length	Cost/Meter	Amount ($)
1	0.25	297	77	22869
2	0.20	262	61	15982
3	0.11	370	38	14060
4	0.11	245	38	9310
5	0.15	128	46	5888
6	0.11	209	38	7942
7	0.11	243	38	9234
Total Cost				85285

Figure 5-8: Pipe Cost Results for a 2 Loop Networks

Table 5-3: Results of a 3 Loop Network from the Program and EPANET

	Program Output			**EPANET Output**		
Pipe	Q_{New} (m^3/s)	HL (m)	V (m/s)	Q_{New} (m^3/s)	HL (m)	V (m/s)
1	0.0421	2.577	1.34	0.0421	2.371	1.34
2	0.0634	1.895	1.29	0.0634	1.901	1.29
3	0.0189	6.978	1.99	0.0189	6.936	1.99
4	0.0142	6.296	1.49	0.0142	6.253	1.50
5	0.0222	5.683	2.33	0.0222	5.647	2.34
6	0.0131	1.470	1.38	0.0131	1.461	1.38
7	0.0072	2.083	0.76	0.0072	2.068	0.76
8	0.0150	3.377	1.58	0.0150	3.353	1.58
9	0.0076	2.892	1.19	0.0076	2.874	1.19
10	0.0037	1.597	0.58	0.0037	1.588	0.58
Dynamic Pressure						

	Program Output	**EPANET Output**
Node	Pressure [m]	Pressure [m]
1	15.74	15.74
2	41.16	41.16
3	38.46	38.51
4	38.10	38.16
5	35.51	35.57
6	44.40	44.45
7	24.87	24.91
8	14.85	14.84

From Table 5-3 above, the discharge from the program and EPANET are the same, and the velocities except in pipe 10 are within the recommended range (0.6-3.0) m/s. The pressure at all nodes are acceptable according to MWE, 2013 design guidelines manual.

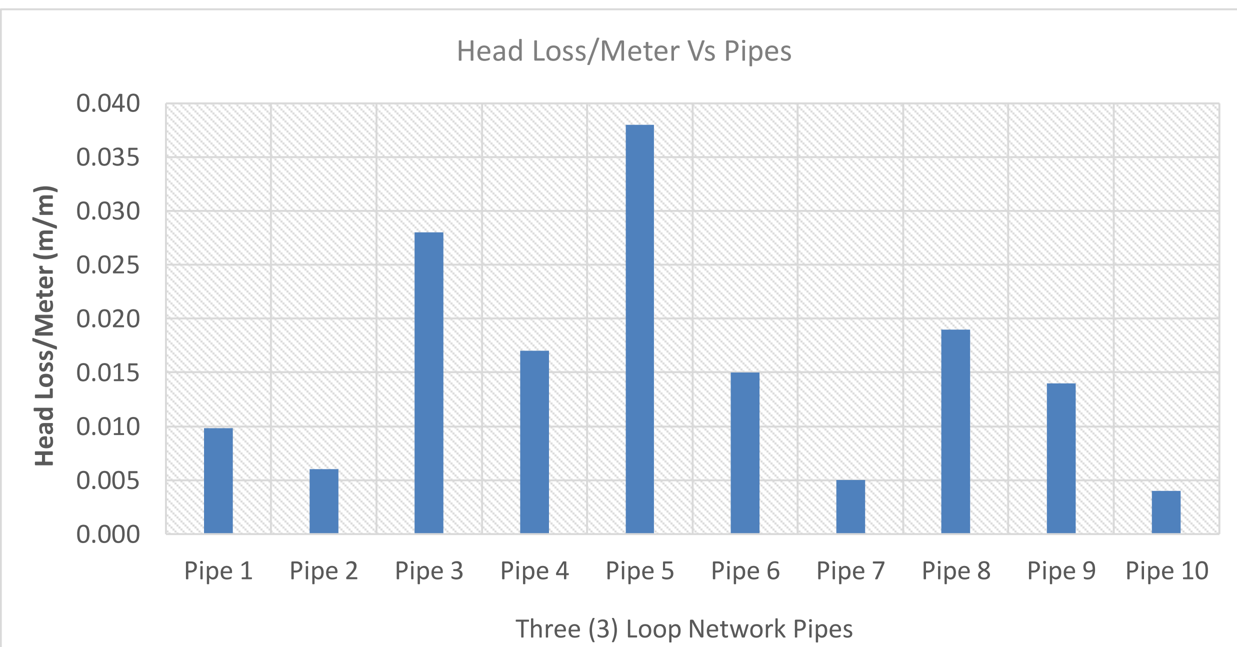


Figure 5-9: Head Loss Per Unit Length in a Three (3) Loop Network Pipes

From Figure 5-9 above, head loss per unit length in pipes 3, 4, 5, 6, 8, 9 are more than the maximum recommended value.

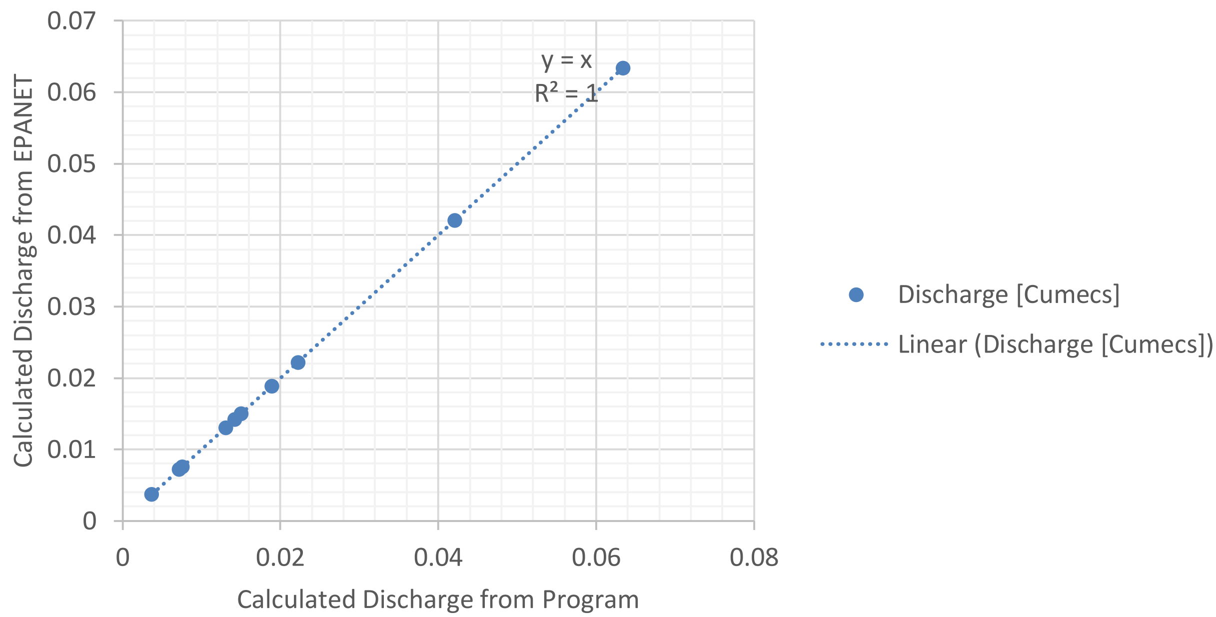


Figure 5-10: Comparison of a Three(3) Loop Discharge Results from Program and EPANET

Results Table

Pipe	Q(m3/s)	HL(m)	V(m/s)	D(m)	f
1	0.0421	2.577	1.34	0.200	0.020
2	-0.0634	1.895	1.29	0.250	0.019
3	-0.0189	6.978	1.99	0.110	0.015
4	0.0142	6.296	1.49	0.110	0.016
5	0.0222	5.683	2.33	0.110	0.015
6	-0.0131	1.470	1.38	0.110	0.017
7	0.0072	2.083	0.76	0.110	0.019
8	0.0150	3.377	1.58	0.110	0.016
9	-0.0076	2.892	1.19	0.090	0.018
10	0.0037	1.597	0.58	0.090	0.021

Q = Actual Flow Rates

HL = Head Losses

V = Velocity

D = Diameter

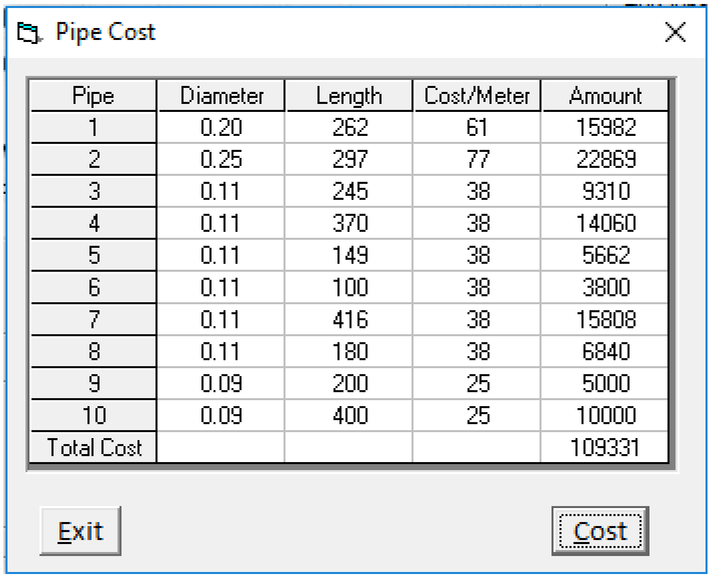
f = Friction Factor

Node	PH(m)	BD(m3/s)
1	15.74	0.0000
2	41.16	0.0057
3	38.48	0.0000
4	38.10	0.0113
5	35.51	0.0113
6	44.40	0.0127
7	24.87	0.0200
8	14.85	0.0445

PH = Pressure Head

BD=Base Demand

Figure 5-11: Program Results for a 3 Looped Network



Pipe	Diameter	Length	Cost/Meter	Amount
1	0.20	262	61	15982
2	0.25	297	77	22869
3	0.11	245	38	9310
4	0.11	370	38	14060
5	0.11	149	38	5662
6	0.11	100	38	3800
7	0.11	416	38	15808
8	0.11	180	38	6840
9	0.09	200	25	5000
10	0.09	400	25	10000
Total Cost				109331

Figure 5-12: Cost of Pipes in a 3 Loop Network

Table 5-4: Results of a 4 Loop Network from the Program and EPANET

	Program Output			**EPANET Output**		
Pipe	Q_{New} (m^3/s)	HL (m)	V (m/s)	Q_{New} (m^3/s)	HL (m)	V (m/s)
1	0.0726	2.466	1.48	0.0726	2.471	1.48
2	0.0375	2.057	1.19	0.0375	1.897	1.19
3	0.0133	5.560	1.40	0.0133	5.532	1.40
4	0.0160	5.150	1.68	0.0160	5.128	1.69
5	0.0366	3.862	2.07	0.0366	3.863	2.07
6	0.0010	0.031	0.10	0.0010	0.033	0.11
7	0.0075	1.319	0.79	0.0075	1.298	0.79
8	0.0085	1.773	1.34	0.0085	1.760	1.34
9	0.0042	1.164	0.66	0.0042	1.158	0.66
10	0.0083	0.640	0.87	0.0083	0.635	0.87
11	0.0072	2.102	0.76	0.0072	2.092	0.76
12	0.0185	4.098	1.95	0.0185	4.075	1.95

Dynamic Pressure		
	Program Output	**EPANET Output**
Node	Pressure [m]	Pressure [m]
1	2.87	2.87
2	1.51	1.51
3	0.65	0.65
4	13.33	13.35
5	19.56	19.59
6	33.72	33.75
7	27.83	27.84
8	28.92	28.92
9	13.36	13.38

From Table 5-4 above, the discharge from the program and EPANET are similar, and the velocities (except in pipe 6) are within the recommended range (0.6-3.0) m/s. The pressure at nodes 2 and 3 are below the minimum recommended value that is 2.0m or 2bars (MWE, 2013). The head loss per unit length of pipe is higher in pipes 3, 4, 5,8 and 12 (Figure

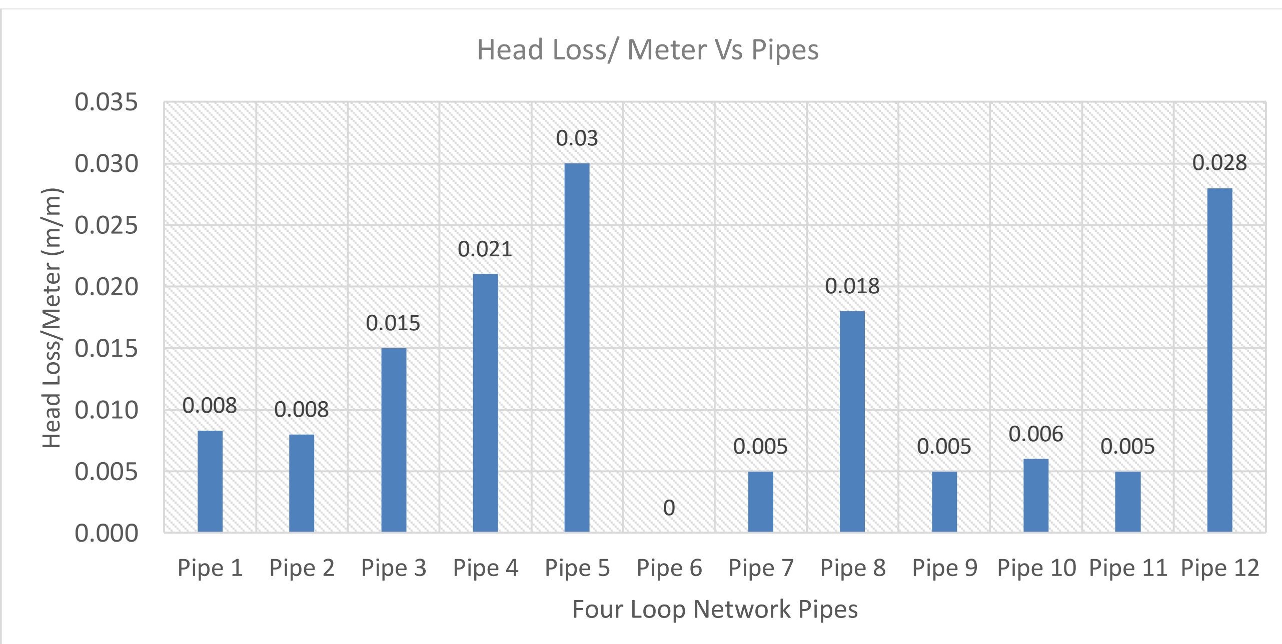


Figure 5-13: Head Loss Per Unit Length in a Four (4) Loop Network Pipes

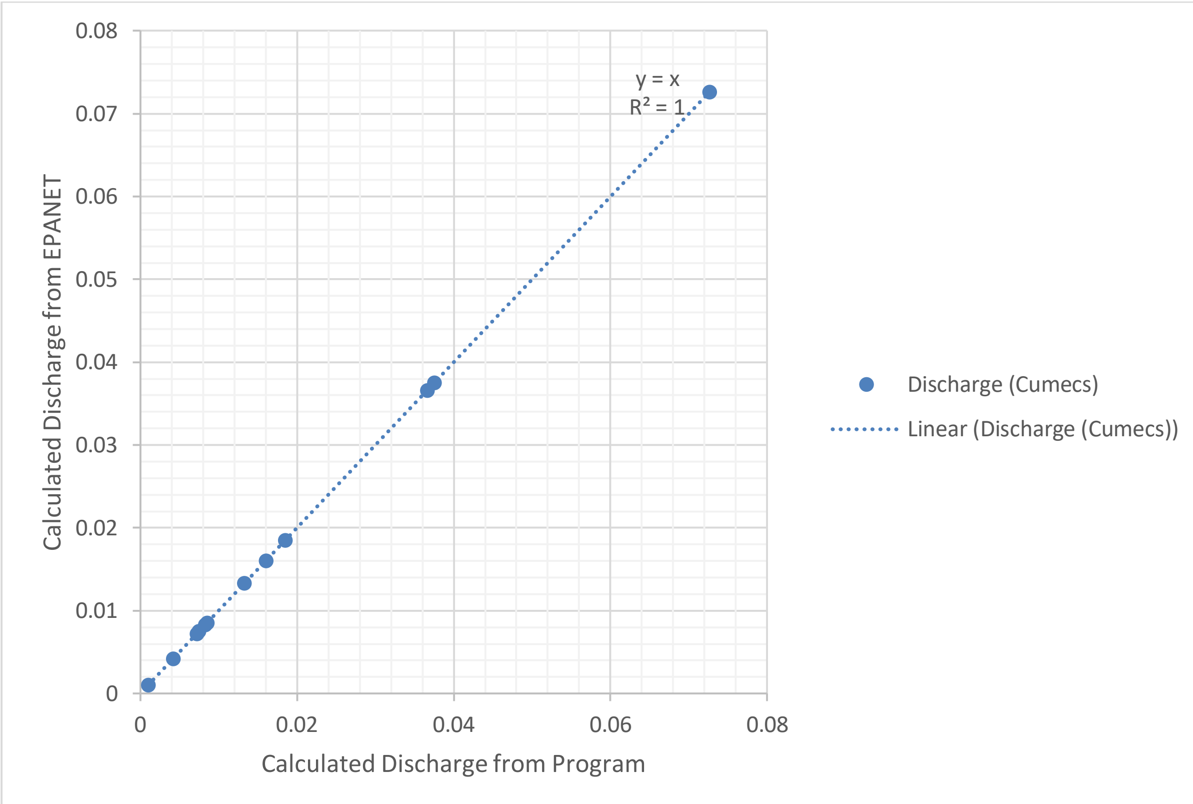


Figure 5-14: Comparison of a Four Loop Discharge Results from Program and EPANET

Results Table

Pipe	Q(m3/s)	HL(m)	V(m/s)	D(m)	f
1	0.0726	2.466	1.48	0.250	0.019
2	-0.0375	2.057	1.19	0.200	0.020
3	-0.0133	5.560	1.40	0.110	0.017
4	0.0160	5.150	1.68	0.110	0.016
5	0.0366	3.862	2.07	0.150	0.021
6	-0.0010	0.031	0.10	0.110	0.030
7	0.0075	1.319	0.79	0.110	0.019
8	0.0085	1.773	1.34	0.090	0.017
9	-0.0042	1.164	0.66	0.090	0.020
10	-0.0083	0.640	0.87	0.110	0.018
11	-0.0072	2.102	0.76	0.110	0.019
12	-0.0185	4.098	1.95	0.110	0.016

Q = Actual Flow Rates

HL = Head Losses

V = Velocity

D = Diameter

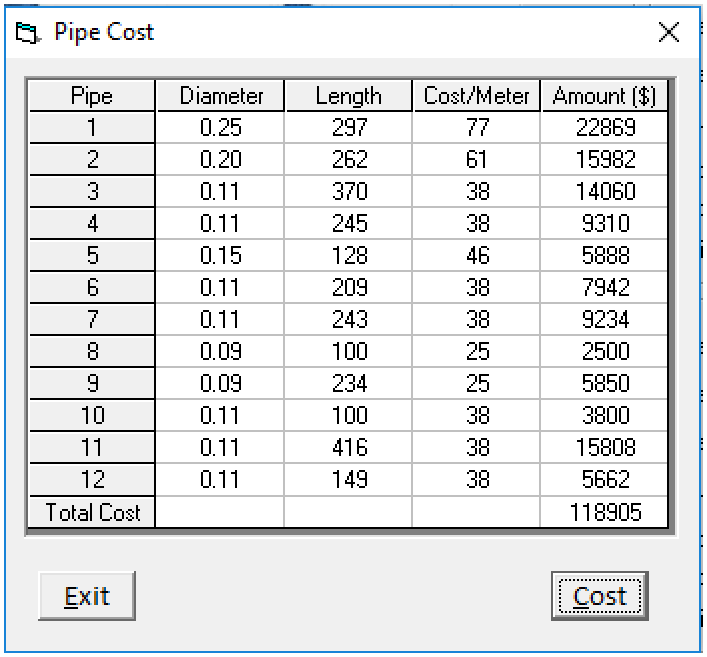
f = Friction Factor

Node	PH(m)	BD(m3/s)
1	2.98	0.0000
2	1.51	0.0200
3	0.65	0.0291
4	13.33	0.0000
5	19.56	0.0127
6	33.72	0.0113
7	27.83	0.0113
8	28.92	0.0057
9	13.36	0.0200

PH = Pressure Head

BD=Base Demand

Figure 5-15:Program Results for a 4 Looped Network



Pipe Cost

Pipe	Diameter	Length	Cost/Meter	Amount ($)
1	0.25	297	77	22869
2	0.20	262	61	15982
3	0.11	370	38	14060
4	0.11	245	38	9310
5	0.15	128	46	5888
6	0.11	209	38	7942
7	0.11	243	38	9234
8	0.09	100	25	2500
9	0.09	234	25	5850
10	0.11	100	38	3800
11	0.11	416	38	15808
12	0.11	149	38	5662
Total Cost				118905

Figure 5-16: Cost of pipes in a 4 Looped Network

5.2 Summary

Table 5-5: Statistical Analysis of Flow Results from the Program and EPANET

Statistical Method	**Discharge**			
	Loops			
	Single (1)	Two (II)	Three (III)	Four (IV)
Coefficient of Determination (R^2)	1.000	1.000	1.000	1.000
Root Mean Square Error (RMSE)	0.000	0.000	0.000	0.000
Mean Bias Error (MBE)	0.000	0.000	0.00	0.000

Table 5-6: Statistical Analysis of Velocity Results from the Program and EPANET

Statistical Method	**Velocity**			
	Loops			
	Single (1)	Two (II)	Three (III)	Four (IV)
Coefficient of Determination (R^2)	1.000	1.000	1.000	1.000
Root Mean Square Error (RMSE)	0.000	0.000	0.000	0.000
Mean Bias Error (MBE)	0.000	0.000	0.000	0.000

From the statistical analysis tables 5-5 and 5-6 above, the Coefficient of Determinant (R^2), Root Mean Square Error (RMSE), and Mean Bias Error (MBE) for both discharge and Velocity results from program and EPANET have been found to be: 1.000, 0.000, and 0.000 respectively for single (1), two (2), three (3) and four (4) loop networks. It should be noted that velocity is calculated from $\left(\frac{Discharge}{Area}\right)$. Therefore, if the discharge results from the program and EPANET agree, then velocities will automatically be the same for the same pipe diameter. This shows the reliability of the program in computing flows and velocities.

Table 5-7: Statistical Analysis of Head Loss Results from the Program and EPANET

Statistical Method	**Head Loss**			
	Loops			
	Single (1)	Two (II)	Three (III)	Four (IV)
Coefficient of Determination (R^2)	0.998	0.998	0.999	0.999
Root Mean Square Error (RMSE)	0.086	0.058	0.070	0.048
Mean Bias Error (MBE)	0.059	0.031	0.041	0.025

From table 5-7 above presenting statistical analysis of head loss results from the program and EPANET, it can be seen that the Coefficient of Determinant (R^2) is practically higher. Rounding off (R^2) to 2 decimal places makes R^2 approximately 1.000. The higher the (R^2), the better is the performance of the model. The Root Mean Square Error (RMSE), and Mean Bias Error (MBE) table 5-6 above can be clearly observed to be reasonably small values. It is stated that the lower the RMSE and MBE, the better is the performance of the model. The reasonably minimal variability in head loss results resulted into the above statistical analysis output. However, this does not affect the final actual flow results. There are about three (3) reasons that could be attributed to the very small variability in head losses which are: (1)- the convergence speed (dependent on initial assumed flows), (2)- the numerical algorithm implemented by both models to solve the system of non-linear equations, (3)- the implemented friction factor (f) formula. EPANET implements “Gradient Algorithm”(Todini and Pilati, 1987) and Swamee-Jain (1976) friction factor equation while the developed model implements improved hardy cross (Epp & Fowler, 1970) and modified Barr (1981) friction factor equation. Nonetheless, the statistical analysis outputs show that the model is reliable.

Table 5-8: Statistical Analysis of Pressure Results from the Program and EPANET

Statistical Method	**Pressure**			
	Loops			
	Single (1)	Two (II)	Three (III)	Four (IV)
Coefficient of Determination (R^2)	1.000	1.000	1.000	1.000
Root Mean Square Error (RMSE)	0.016	0.006	0.042	0.017
Mean Bias Error (MBE)	0.010	0.003	0.034	0.012

From table 5-8, the Coefficient of Determinant (R^2) is 1.000 for the considered loops. The values of the Root Mean Square Error (RMSE), Mean Bias Error (MBE) are rationally minimal. Small values of RMSE and MBE indicate good performance of the model. Therefore, it can be concluded from the statistical study conducted between EPANET and the program that the developed model is reliable and can be trusted in analysing flows in a closed loop water network.

Chapter 6 : Conclusion and Recommendation

6.0 Introduction

This chapter presents the conclusion and recommendation. Recommendations are very crucial for the further advancement of the study.

6.1 Conclusion

In the face of the challenges encountered during the hydraulic model development, the User Friendly Numerical Hydraulic model for analysing complex pipe networks using modified hardy cross algorithm has undergone complete and successful design, implementation and testing to meet the earlier stated objectives. The model has been limited to analyze one (1), two (2), three (3), and four (4) closed loops. For the purposes of model validation, a comparative study was conducted on the outputs from the program with EPANET. The statistical analysis revealed the model validity, since the discharge results from both EPANET software and the developed model showed no variations.

6.2 Recommendation

It should be noted that the developed hydraulic model may not possess adequate functions as there is in a commercially rated model. It may have a limited application for a compound pipe network. For the future advancement of the model, the following recommendations are presented:

1. Develop a model that implements modified HCM for both available population and the base demand data. That is to say, for the base demand entering and leaving the junction node.
2. Adopt other existing algorithms like; Newton-Raphson, and Linear theory methods to conduct a comparative study for the further advancements of this study.
3. Extending the present model to solve hydraulic grid of any number of loops greater than four.
4. Develop the graphical interface for the current model.

5. Extend the present model to handle water quality modelling plus extended period hydraulic analysis.
6. Extend the present program to model a network with a pump and pseudo-loops.

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