Vaal University of Technology

Modelling of Pressurised Water Supply Networks that May Exhibit Transient Low Pressure - Open Channel Flow Conditions

by

Stephen Nyende Byakika BSc(Hons) Eng, MSc(Eng)

Thesis submitted in fulfilment of the requirements for the Degree of **Doctoris Technologiae** in the Department of Civil Engineering and Building, Faculty of Engineering and Technology, Vaal University of Technology.

Promoter: Prof. G Ngirane-Katashaya, BSc (Hons),DiplMet MSc,DIC,PhD

Co-Promoter: Prof. JM Ndambuki BSc (Hons) Eng, MSc, PhD

March 2011

Acknowledgements

My sincere appreciation goes to the Vaal University of Technology (VUT) for the bursary and opportunity to conduct part-time lectures while I was a Doctoral student. Let me also thank the Department of Civil Engineering for providing an enabling environment.

Sincere thanks go to my Promoters Prof. Gaddi Ngirane-Katashaya and Prof. Julius M. Ndambuki for having invited me to pursue this Doctorate at VUT, for supervising me adequately and supporting and encouraging me.

Heartfelt gratitude goes to my mother Mrs. Sarah Nabakaawa Kyobe Byakika who has been a powerful blessing to me for having supported, encouraged and shielded me not only through this course but all my life. I attribute all my success to her tireless and selfless efforts. Many thanks also go to Kisakye and Senkunja for loving and encouraging me. Special thanks go to Deborah and Richard Ssewakiryanga for their continuous encouragement.

I also extend thanks to my former bosses at the National Water and Sewerage Corporation of Uganda viz. Eng. Alex Gisagara, Eng. Peter Balimunsi and Charles Odonga for strongly inspiring me. Special thanks also go to my students who I have taught over the years, for helping me realise my true potential.

At the risk of offending some, I also thank my friends who have stood by me viz. Jackie Zirabamuzaale, Herman Kabanda, Andrew Okumu, Gilbert Akol, Kenneth Ssekanyo, Tonny Kaluuba (RIP), Joel Aita, Stephen Adeyemi, Kempi Inyang and Beltran Labana.

To God Almighty, the alpha and omega, may all thanks, glory and honour return.

Dedication

This Thesis is dedicated to the memory of my father G.S. Kasajja-Byakika

Abstract

Growing demand for water due to increasing populations, industrialisation and water consuming lifestyles puts stress on existing water supply systems. To cater for the rising demand, water distribution networks are expanded beyond their design capacities and this creates transient "low-pressure-open-channel flow" (LPOCF) conditions. Current water supply models use "demand driven approach" (DDA) methodology which is not able to simulate transient LPOCF conditions, that poses an impediment to management/analysis of pressure-deficient networks.

With a case study of the water supply network of Kampala City, LPOCF conditions were studied in this research. A "pressure/head driven approach" (PDA/HDA) was used in order to determine what demand is enabled by particular nodal pressures. Conversion of free surface to pressurised flow was analysed and modelled, with a view to clearly understanding occurrence of this phenomenon.

The research demonstrated that if adequate pressures and flows are to be maintained, effectiveness of the water distribution network should be given as much attention as water production capacity. The research also indicated that when network pressures are low, the head-driven approach to water distribution modelling gives more accurate results than the traditional demand-driven methodology. Coexistence of free-surface and pressurised flow in networks prone to LPOCF conditions was confirmed and modelled.

Results obtained highlighted the advantages of developing fully dynamic and transient models in the solution of transient LPOCF conditions in water distribution networks. Models developed allow application of PDA/HDA and DDA methodologies in systems that may exhibit LPOCF conditions thus enabling identification, understanding and analysis of the status of all sections of the network. These culminated in the development of a DSS to guide operational decisions that can be made to optimise network performance.

Contents

List of Figures

List of Tables

List of Acronyms

DDA	Demand Driven Analysis
DD-ADF	Demand-Driven Available-Demand-Fraction
DPA	Decoupled Pressure Approach
DSS	Decision Support System
EPS	Extended Period Simulation
FDE	Full Dynamic Equation
FDM	Full Dynamic Model
HDA	Head Driven Analysis
ID	Identification Number
IEC	Information, Education and Communication
ITM	Interface Tracking Method
KWSN	Kampala Water Supply Network
LPOCF	Low Pressure – Open Channel Flow
masl	metres above sea level
MoC	Method of Characteristics
NWSC	National Water and Sewerage Corporation
ODE	Ordinary Differential Equations
OMDST	Operations and Management Decision Support Tool
PDA	Pressure Driven Analysis
PDD	Pressure Driven Demand
PDE	Partial Differential Equations
WDS	Water Distribution System
WSS	Water Supply System

THE MAIN THESIS

1.0 INTRODUCTION

1.1 Background

Growing demand for water as a result of increasing urban populations, industrialisation and rising water consuming lifestyles, puts stress on existing water supply systems. In order to cater for additional demand, distribution networks are expanded, often beyond their design capacities, which creates bottlenecks such as development of transient flow conditions ranging from excessive pressures, fluctuating pressures to open-channel flow situations. This culminates into low pressures with low flows and sometimes no flow at all, thereby compromising service levels and giving planners and engineers a complicated task of supplying the additional resource in sufficient and reliable quantities in the most feasible way possible.

The effect of network extensions and increased demand on the hydraulic characteristics of water distribution networks is not clear and this lack of knowledge hinders the explanation of water shortages in the network, and consequently, formulation of appropriate remedial measures. Moreover, available water distribution models are unable to deal with cases of low pressures and the extreme case of flow in partially full pipes which also transition to full pipe flow.

In order to meet regulatory requirements and customer expectations, water utilities are feeling a growing need to explain better the movement and transformations undergone by water introduced into their distribution systems (Rossman 2000). If understanding of network behaviour under adverse conditions could be obtained and the impact of these conditions established, networks would be managed better, and more satisfactory customer service would be offered. The problem was studied and solutions obtained given the researched condition of its real recurrence.

A case in point is the water supply network of Kampala City where increased demand coupled with social and political pressures to provide sufficient water and the desire to maximise revenue has led to excessive network expansion. For example, the number of new water connections made increased by 58 percent from 14,045 in 2003/2004 to

22,218 in 2004/2005 (NWSC Annual Report 2006/2007), a rate of growth that is expected to increase further with consumer growth. However as the network is expanded, cases of intermittent supply and little or no water have been increasingly reported in several areas. Areas that previously used to receive enough water are presently inadequately served (Figures 1 and 2).

Figure 1: Water Scarcity
A vender transports water to residents of Kampala during a shortage recently. He said he sells a jerrycan at Shs1,000 (ZAR 3.00) up from Shs 200 during the crisis.

Kampala City is not the only area that faces water shortages but is among many urban areas that possess pressurised water supply systems with intermittent flows (Zyoud 2003; Lee & Schwab 2005; Vairavamoorthy 2008; Vairavamoorthy, Gorantiwar & Pathirana 2008; Basu & Main 2001; Biswas & Seetharam 2008; Rosenberg 2008; Khatri & Vairavamoorthy 2007) which are characterised by low pressures, low flows and open-channel flow conditions, whose analysis requires special treatment, different from that of fully pressurised systems. This persistent challenge is present in many developing countries in the world due to inadequate production capacities and distribution networks which require huge and sometimes, unavailable capital investments to expand. Consequently, demand is not satisfied yet despite system bottlenecks, consumers should get water. There is need to ensure continuity and sustainability of water supply during periods when pressures are low and even when pipes do not flow full.

The research was aimed at augmenting existing knowledge on supply of water during transient Low Pressure Open-Channel flow (LPOCF) conditions. To the best of my knowledge, no work has been done elsewhere in this regard; attempts have been made to study flow transitions in storm and wastewater drainage systems but none before has been made to study the same in potable water systems. Knowledge obtained will aid the provision of water supply services even under extreme situations of low flows and low pressures. This will not only improve the understanding of piped water supply systems but also ensure sustainable supply of the basic need for survival.

Uganda: Water Shortage Hits Kampala City

Conan Businge **13 August 2009**

Kampala — KAMPALA and its suburbs has suffered an intermittent water supply in the last two weeks. Ndejje, Ntinda, Mutungo, Kanyanya, Nabweru and Busabala have only been getting water at night.

Other suburbs with water shortages are Namuwongo, Namungoona and Rubaga. A technical supervisor from the National Water and Sewerage Corporation said the ongoing drought had affected the pumping of water from the Ggaba plant.

He explained that "as the water reduces, so does the pressure in the pumps, making them fail to push water to all the destinations." There is more dirt at the source of water. If the water is continuously pumped, there are more chances of supplying dirty water, the source added.

"There have to be breaks in pumping to allow the dirt to settle down," the source explained. The acting public relations officer, Ruth Ijang, said: "The drought is a very big problem, but we shall maintain the flow of water to our clients. The pressure is not enough to pump the water."

However, the water supply manager, Ssonko Kiwanuka, said: "We recently increased the production capacity. The drought has nothing to do with this."

He said the shortage was being caused by the increased number of consumers.

Figure 2: Press release about Water Shortages in Kampala

Most water distribution models consider closed systems i.e. pipes flow full and under pressure, with an assumption that analysis would be based on a demand driven rationale that further assumes demand as independent of pressure. In this approach, it is taken for granted that pressures are available to satisfy demands defined at nodes, thus allowing nodal hydraulic heads and pipe flows to be determined by solving a set of quasi-linear equations (Rossman 2000). This formulation is well developed and valid for scenarios in which pressures in a system are adequate for delivering required nodal demands.

The reality in the Kampala Water Supply Network (KWSN) is that there are sections which experience severe pressure shortfalls brought about by inadequate production, inadequate distribution and unplanned/design-exceeded network expansions, leading to several cases of very low flow or no flow at all and this directly limits supply (Nyende-Byakika, Ngirane-Katashaya & Ndambuki 2010). In such situations, traditional methods of analysis have limitations; demands in this case are not only a function of time but of pressure as well and consequently, demand driven analysis alone fails when abnormal conditions prevail (Zheng, Rong, Thomas, Shao, Bowdler & Baggett 2009; Hayuti, Burrows & Naga 2007).

Since Demand Driven Analysis (DDA) assumes that pressures are sufficient to fulfil any demands in both time and space, constitutive equations (conservation of mass and energy) are composed in order to solve both nodal head and link flows. Of course, since demand at nodes is predetermined and therefore fixed, this parameter in the continuity equation is also fixed. Thus nodal heads and link flows are calculated while the nodal demands are fixed.

Demand driven analysis, though commonly used, may produce unrealistic results under extreme operating conditions. Particularly, it can produce pressures that are negative or less than satisfactory when demand is excessive. This led to development of analysis that incorporates a relationship between demand and pressure, also called pressure/head driven analysis (PDA/HDA), which has resulted in desired solutions by showing compensation among nodal demands and available pressures (Cheung, van Zyl, & Reis 2005).

Hayuti et al. (2007) developed an approach to obtain a solution to pressure constrained outflows based on repetitive substitution into conventional computational algorithms where demands are generally predetermined. He was able to demonstrate that in terms of the nodes affected by shortfalls in pressure, the demands supplied by those nodes were actually less than what would have been the case with conventional investigations. Chandapillai (1991) having acknowledged that conventional methods just give the resulting pressures at various nodes in the prescribed demand condition, attempted to satisfy an additional constraint of head-flow relationship at each node in order to handle supply in adverse situations. He was able to demonstrate that under conditions of insufficient pressures, the amount of water that can be supplied at a node directly depends on the pressure available at the node. Martínez-Solano, Iglesias-Rey, Pérez-Garcí & López-Jiménez (2008) described an iterative procedure for demand load allocation in order to satisfy the continuity equation in the nodes, having considered that under normal conditions models are based on averaged demands based on customers' billings. He was able to determine the actual water that can be supplied at different nodes, depending on the pressures available at the nodes.

While head-driven analysis (HDA) is superior to demand-driven analysis in that a modeller is able to determine nodes with insufficient supply and the respective magnitudes of shortfalls, HDA is quite involved to apply due to the difficulty in calibrating a pressure-flow relationship at every node, which would require extensive field data collection and calibration. Moreover, models are schematized and some features are lost; secondary networks have variable headlosses and outlet elevations which would require all the more data to collect. Worse still, resulting equations have no computational methods to solve them. This explains why there are no commercial hydraulic solvers using HDA.

Modelling intermittent water supply systems of pipeline networks is a challenging task because these systems are not fully pressurised but networks with high water demands, very low pressures and sometimes restricted water supply hours per day. The alternate emptying and refilling followed by pressurisation and depressurisation of water pipelines make it problematic to apply standard hydraulic models because of low transient pressures and pipes flowing partially full (Ingeduld, Svitak, Pradhan & Tarai 2006). Transient pressures have traditionally been considered to bear destructive influence in systems leading to pipe or equipment failures and representing a threat to both water quality and smooth operation (Karney, Parente, Eerkes & White 2008). However, it has recently been realised that they can give very useful insight into system state and condition thus leading to inverse transient methods, where a transient signal is used to infer system characteristics and parameters. This is an area where more research needs to be carried out.

From the foregoing discussion, it can be inferred that:

i. Current water supply modelling philosophy assumes a demand driven approach which may not be applicable in networks experiencing low pressure situations.

ii. Many pressurised water supply systems exhibit transient low-pressure-open-channel flow (LPOCF) conditions. There are sections that operate as gravity mains under low reservoir conditions and as low-pressure force mains under high level conditions. This type of situation, where flows in pressurised networks shift from full bore flow to open channel flow is difficult to analyse and requires special treatment which is different from that of fully pressurised systems.

To this end, numerical simulation as a modern tool in the management of piped water supply plays a very important role as it would identify non-conformant LPOCF situations in a pressurised network.

1.2 Statement of the Problem

Transient low pressure and low flow situations in several water supply networks are a recurrent reality, which may not be eliminated in the near future. This is because water production and distribution capacities are not proportionately expanded in time to cater for demand growth, owing to the massive capital investments required. Yet, in spite of these bottlenecks, consumers should keep getting water; therefore, engineers cannot continue to design, build, rehabilitate or operate constrained systems as if the capacity to produce and deliver water is sufficient, but should optimise water supply within existing constraints. Thus, they have to analyse the systems and come up with models that allow prediction of system behaviour, thereby enabling timely corrective interventions.

Due to the fact that most water supply models operate on the assumption that pressure is sufficient to deliver adequate flows, in situations of low transient pressures the models do not give reliable results. Thus, in order to address this issue, a tool that treats transient pressures and flows which lie along the continuum between open and closed systems in both time and space needs to be developed. Currently there are no tools available to deal with Low Pressure - Open Channel (LPOCF) conditions in water supply networks. Hence this study is designed to develop a new approach to address this problem.

1. 3 Objectives of the Research

1. 3.1 Main Objective

The main objective of the research is to develop an Operations and Management Decision Support Tool (OMDST) of pressurised water supply networks prone to transient low pressure – open channel flow conditions. The tool would enable understanding of occurrence as well as identification of network sections with and without adequate pressures. This tool will guide informed actions that can be taken to improve the situation in the affected sections while not severely affecting supply elsewhere.

1.3.1.1Specific Objectives

Specific research objectives are:

i. To characterise the water supply situation in Kampala and the status of the network i.e. layout, pressures and flows (this was the study area for testing the performance of the developed management tool).

ii. To develop an operations and management decision support tool (OMDST) (multi-modular simulation model) of the water supply network under normal (adequate pressures and design demand) operating conditions.

iii. To study and analyse the model for extreme operating conditions with emphasis on transition to low network outflows.

iv. To analyse pressure transition from full bore flow to partial pipe flow tapering to no flow conditions.

v. To apply the developed operations and management decision support tool to a real world situation in order to test its performance.

1.4 Conceptualisation of the Study

1.4.1 Water Supply Network Modelling

Water supply models have in recent decades found increasing application in design and management of reticulation networks. This is due to the fact that they simulate pressures and flows thereby revealing system behaviour that makes it possible to judge the feasibility of a proposed network. Today, water distribution modelling is a critical part of designing and operating water distribution systems that are capable of serving communities reliably, efficiently and safely, both now and in future (Haestad Methods 2003). The availability of increasingly sophisticated and accessible models allows these goals to be realised more fully than ever before.

Network simulations, which replicate the dynamics of an existing or proposed system, are commonly performed when it is not practical for the real system to be directly subjected to experimentation or for the purpose of evaluating a system before it is actually built. Simulations can be used to predict system responses to events under a wide range of conditions without disrupting the actual system. Using simulations, problems can be anticipated in proposed or existing systems, and solutions can be evaluated before time, money, and materials are invested in a real-world project. These models are especially important for Water Distribution Systems (WDS) due to their complex topology, frequent growth and change, and sheer size. It is not uncommon for a system to supply hundreds of thousands of people (large networks supply millions); thus, the potential impact of a utility decision can be tremendous. Haestad Methods (2003) states that water distribution models can be used for:

i. Long-Range Master Planning

Planners carefully research all aspects of a water distribution system and try to determine which major capital improvement projects are necessary to ensure the quality of service for the future. This process may be used to project system growth and water usage for the next 5, 10, or 20 years. System growth may occur because of population growth, annexation, acquisition, or wholesale agreements between water supply utilities. The capability of the hydraulic network to adequately serve its customers must be evaluated whenever system growth is anticipated. Not only can a model be used to identify potential problem areas (such as future low pressure areas or areas with water quality problems), but it can also be used to size and locate new transmission mains, pumping stations, and storage facilities to ensure that the predicted problems never occur. Maintaining a system at an acceptable level of service is preferable to having to rehabilitate a system that has become problematic.

ii. Rehabilitation

As with all engineered systems, the wear and tear on a water distribution system may lead to the eventual need to rehabilitate portions of the system such as pipes, pumps, valves, and reservoirs. Pipes, especially older, unlined, metal pipes, may experience an internal build-up of deposits due to mineral debris and chemical reactions within water. This build-up can result in loss of carrying capacity, reduced pressures, and poorer water quality. To counter these effects of aging, a utility may choose to clean and reline a pipe, the pipe may be replaced with a new (possibly larger) pipe, or another pipe may be laid in parallel. Hydraulic simulations can be used to assess the impacts of such rehabilitation efforts, and to determine the most economical improvements.

iii. Fire Protection Studies

Water distribution systems are often required to provide water for fire fighting purposes. Designing the system to meet fire protection requirements is essential and normally has a large impact on the design of the entire network. The engineer determines the fire protection requirements and then uses a model to test whether the system can meet those requirements. If the system cannot provide certain flows and maintain adequate pressures, the model may also be used for sizing hydraulic elements, such as pipes and pumps, to correct the problem.

iv. Water Quality Investigations

Some models provide water quality modelling in addition to hydraulic simulation capabilities. Using a water quality model, the user can model water age, source tracing, and constituent concentration analyses throughout a network. For example, chlorine residual maintenance can be studied and planned more effectively, disinfection by-product formation (DBP) in a network can be analyzed, or the impact of storage tanks on water quality can be evaluated. Water quality models are also used to study the modification of hydraulic operations to improve water quality.

v. Daily Operations

Individuals who operate water distribution systems are generally responsible for making sure system-wide pressures, flows, and tank water levels remain within acceptable limits. The operator must monitor these indicators and take action when a value falls outside the acceptable range. By turning on a pump or adjusting a valve, for example, the operator can adjust the system so that it functions at an appropriate level of service. A hydraulic simulation can be used in daily operations to determine the impact of various possible actions, providing the operator with better information for decision-making.

vi. Operator Training

Most water distribution system operators do their jobs very well. As testimony to this fact, the majority of systems experience very few water outages, and those that do occur are rarely caused by operator error. Many operators, however, gain experience and confidence in their ability to operate the system only over a long period of time, and sometimes the most critical experience is gained under conditions of extreme duress. Hydraulic simulations offer an excellent opportunity to train system operators on how their system will behave under different loading conditions, with various control strategies, and in emergency situations.

vii. Energy Management

Figure 3: Pumps
Cost of pumping is one of water supplier's biggest expenses

Next to infrastructure maintenance and repair costs, energy usage for pumping (Figure 3) is the largest operating expense of many water utilities. Hydraulic simulations can be used to study the operating characteristics and energy usage of pumps, along with the behaviour of the system. By developing and testing different pumping strategies, the effects on energy consumption can be evaluated, and the utility can make an educated effort to save on energy costs.

viii. Emergency Response

Emergencies are a very real part of operating a water distribution system, and operators need to be prepared to handle everything from main breaks to power failures. Planning ahead for these emergencies by using a model may prevent service from being compromised, or may at least minimize the extent to which customers are affected. Modelling is an excellent tool for contingency and emergency response planning.

ix. System Troubleshooting

When hydraulic or water quality characteristics in an existing system are not up to standard, a model simulation can be used to identify probable causes. A series of simulations for a neighbourhood that suffers from chronic low pressure, for example, may point toward the likelihood of a closed valve in the area. Field crew can then be dispatched to this area to check nearby valves.

Networks are logically expected to operate with pressure for reasons of effective delivery and hygiene (Arsene, Bargiela & Al-Dabass 2002; Bahadur, Johnson, Janke & Samuels 2006) by flushing the system to ensure self cleansing and prevent deposition of sediments while also preventing infiltration. In this regard, models assume pressurised conditions everywhere in the network which, in practice, may not always be the case. Traditionally after assuming pressurised systems, water supply models analyse networks by using the demand driven approach (DDA), where it is taken for granted that all demands are satisfied. The visual responses become chaotic during pressure shortfalls when the required demands are shown to be satisfied in all circumstances including periods of inadequate pressure, which is unrealistic.

Failure of a water distribution network implies a deficiency in the level of service that is usually of limited areal extent (Tanyimboh 2000). Deficiency usually appears as either pressure or flow falling below some specified values at one or more nodes within the network. It therefore becomes important to isolate those sections (nodes) in order to study them closely with an aim of establishing the flows that can be enabled by the resulting pressures. DDA identifies pressure deficient nodes during abnormal conditions when demand cannot be met, however its weakness lies in the retention of actual demand values at those nodes even when it is clear that with shortfalls in pressure below certain threshold values, supply also falls short.

From a pressure-driven point of view the problem that needs to be addressed is to determine available flows at prevailing pressures, an objective which is of fundamental importance in this research. There then arises a need to employ an approach that recognises a relationship between pressure and discharge, also called a “pressure or head dependent demand approach (PDA/HDA)” in order to establish exactly the supply that is enabled by the available pressures at the nodes.

Furthermore, when pressures are zero water may not fill the pipes and no demand is satisfied. This concept led to the study of occurrence of partially-full flowing pipes using free surface flow equations, and simulation of flow regime changes between pressurised and free surface flow, a condition which current water distribution models do not tackle. During the transition from full-pressurised flow to partially-full flow, a moving water interface advances into the free-surface region. There is need to track the interface in order to explain the development of pressures in a pipeline. This will enable us to make a contribution in understanding the co-existence of pressurised and free-surface flows in a water supply network. The study of the flow regime transition also greatly enables understanding of the pressurisation process in pipelines.

In summary, while available water distribution models only consider the pressurised regime and the demand driven approach, this research aimed at understanding the occurrence of pressurised and free surface flow conditions. Consequently, a Decision Support System (DSS), that would predict conditions that occur in sections of the network that function well and those that do not, was developed. This DSS will assist in arriving at decisions which alleviate shortfalls ranging from insufficient nodal pressures to free surface flow and no flow at all in pipelines.

1.4.2 Decision Support Systems

Recent intensification of use of computers and information technology has enabled employment of modelling to support understanding of occurrence, behaviour and prediction of several phenomena in water distribution. Currently no model addresses all situations that occur in water supply networks including excessively low pressures and no pressure at all. This challenge is tackled by application of several simplifying assumptions underlying modelling techniques such as the Demand Driven Approach (DDA) in which fulltime pressurised networks are assumed. This means that these models, as stand-alone tools, fail to address all issues that networks face such as no water flows. A global perspective of the network should recognise the fact that what happens in one part of the network affects what happens elsewhere in the network (Nyende-Byakika 2006). It is therefore important to look at a ‘forest’ and not just ‘trees’, a concept also called systems analysis (Biswas 1976). If an entire network is to be analysed and solved, then required tools should be able to establish relationships between different problems that occur in a network; this research developed such tools.

In addition, it should be noted that inputs and outputs of models are often highly technical and can best be understood and made use of by technical staff yet water supply networks are also managed by non-technical staff. This factor implies that if the prevailing models are to be relevant to all managers and decision makers in water supply, they should yield results that can aid decision making for both technical and non technical members of the operations team. Models can be improved by coining their results into outputs that are easily understood by all decision makers and thereby assisting them to make appropriate and pragmatic decisions amidst the situations encountered. This calls for the development of decision support systems, also called decision aid techniques (Ndambuki 2001; Ngirane-Katashaya, Kizito & Thunvik 2006; Kizito 2009) which was the focus of this study.

It should be noted that this research was chiefly aimed at resolving problems faced in prevailing low pressure and intermittent flow situations and the research context is biased towards systems operating at inappropriate but inevitable conditions. In order to accomplish this research three areas were identified to guide the understanding and formulation of simulation tools: normal pressures, low pressures and nil pressure (atmospheric pressure). The latter two which comprise our major research focus are problems faced in water supply networks in many developing countries and fast growing urban areas and populations. The Decision Support System (DSS) developed (Figure 5) was conveniently divided and designed along three hierarchical levels as follows:

i. Normal pressures
ii. Low Pressures
iii. Nil Pressures (Atmospheric Pressure)

1.4.3 Explanation of the Developed Decision Support System

The decision support system developed (Figure 5) is capable of providing sufficient information to guide decisions that are made during the operation of water supply networks. A water distribution model was developed in the EPANET2 hydraulic network solver under the traditional demand driven approach as a starting point. This was mainly because available network models operate in this mode. The reasons for selecting EPANET2 are elaborated in Section 2.8. The developed water distribution model was loaded with various scenarios e.g. excessive demand or reduced supply, in order to examine what impact the scenarios would have on different parts of the network. While some sections of the network may experience normal pressures other sections bear low pressures and others still, bear no pressure at all.

1.4.3.1 Normal Pressures

In this research normal pressures were defined as the minimum pressures at which all demand is met, and also as design pressures. Nodes with normal pressures were identified. It is also worth noting that the focus of the study was on analysing and solving poor pressure systems and this being the major objective of this study, normal pressures were not accorded more attention than they already had.

1.4.3.2 Low Pressures

In this study low pressures were considered to lie between the threshold normal pressures (which are node specific) and zero. In this regime only a fraction of the demand can be fulfilled through a relationship between available pressure head and demand. This relationship should be exploited if actual discharges at nodes during the low pressure episodes are to be obtained. Heads of less than 2 metres at a main (as shown in Figure 4) cannot fully satisfy all consumers.

Figure 4: Water gushes out of a burst pipe.
The pressure in the main can be estimated by comparing the height of the car to the height to which water rose .

The demand driven approach (DDA) ceases to help in this case as it displays results indicating that demand as desired is fulfilled even though the pressure is low and at this juncture there is need to transform from DDA to PDA in order to determine available flows enabled by the prevailing low pressures.

Demand at every pressure-deficient junction was treated as an unknown value while a pressure threshold was imposed through the following modifications:

i. Elevation of the node was increased by the value of the threshold pressure head ultimately making the node pressure zero and disabling supply, necessitating the substitution of the original node demand value with zero.

ii. An outlet to the node was provided through a virtual reservoir installed at the same new elevation and connected through a pipe that was infinitesimally short in order to minimise head losses which would reduce discharge to the virtual reservoir when the model was run. It should be noted that the new node elevation does not enable any supply but the discharge into the virtual tank logically reflects the supply that can be made (i.e. the proportion of the original demand met) if the original nodal elevation were to hold.

If the discharge as calculated from the artificial node elevation exceeded the actual demand at that node, that would imply that the particular node was in fact not pressure deficient and the virtual reservoir was removed and the original elevation and demand restored. The process was carried out iteratively until equilibrium conditions were established in the network hence yielding network stability and reflecting the realistic and expected situation in the field.

1.4.3.3 Nil Pressure (Atmospheric Pressure)

Intermittent water supply systems encounter periods when pressures do not exist in sections of the network where pipes may be full, partially full or empty and consequently no supply is possible. Pressurised and free surface conditions (during adequate ventilation) can then be said to coexist. This is an interesting phenomenon in pressurised water supply networks that has not received any attention to date inasmuch as it is prevalent, thereby forming the subject of this study. Upon simulating the situation (Chapter 4), actions can be taken such as laying new pipelines (Figure 6) in order to minimise or eliminate the shortfalls using the support of the DSS developed.

It is worth noting that normal, low and nil pressure situations can occur at different times and locations in the same network and to study these phenomena, holistic network considerations should be taken of all variables, parameters and relationships that exist within the system. Figure 5 shows the conceptual framework for the DSS while Figure 6 shows process of laying new pipelines for the Kampala City Water Supply System.

1.5 Scope of the Research

The research addressed pressurised and free surface flows ranging from pressurised full bore to partially full flow as well as the transition from free surface to pressurised flow. The research was geared towards developing tools that would aid the understanding of the development of pressures in the systems with an aim of ensuring sustainable and successful management of networks during all flow conditions.

1.6 Real World Case Scenario

To validate the developed tools, field data was obtained from an existing network and compared with model outputs. The Kampala Water Supply System in Uganda (Figures 7 and 8) provided data used for this research as it is a typical system that exhibits low pressures and intermittent supply. The researcher was personally very familiar with it.

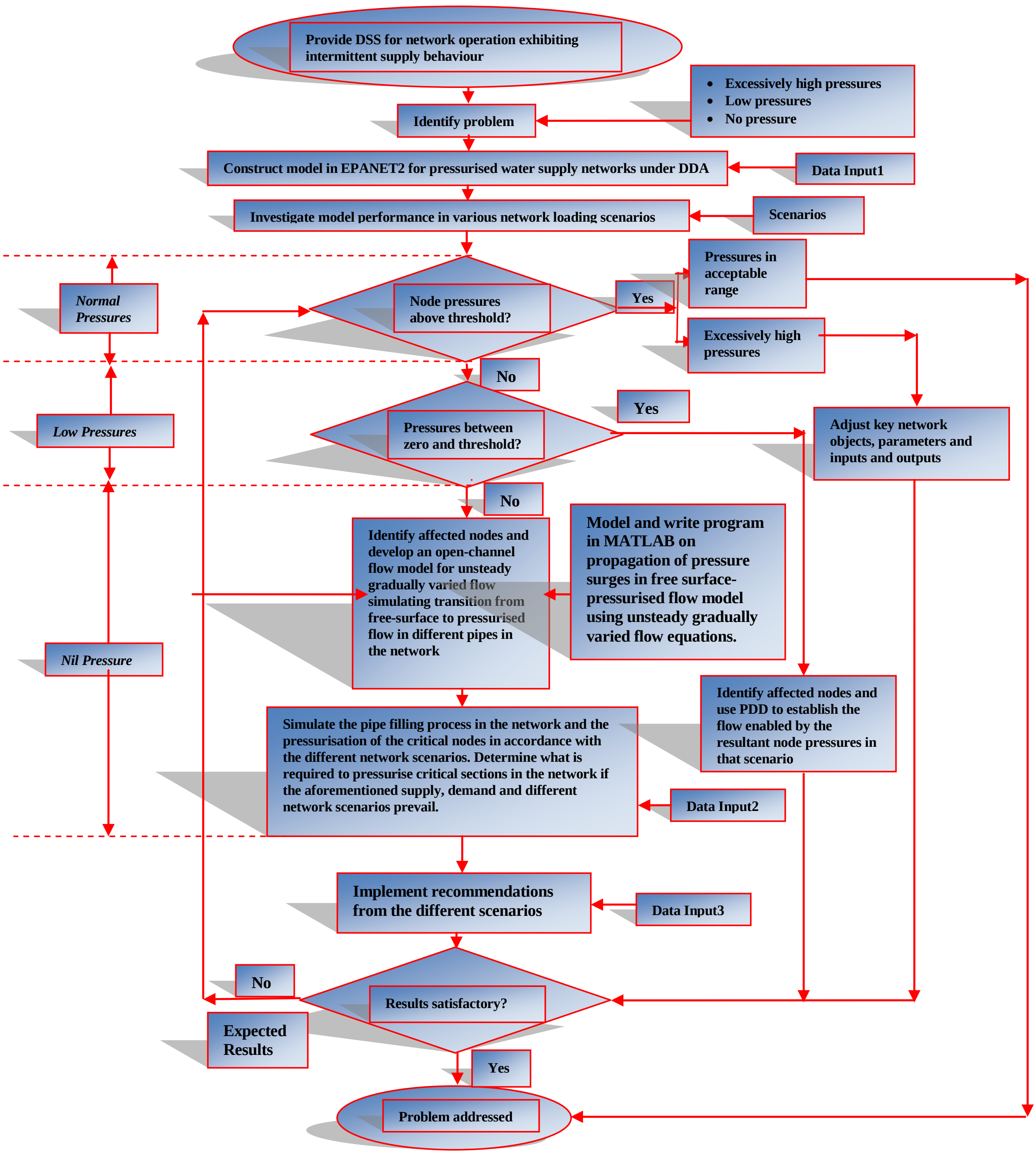

Figure 5: Conceptual Framework for Decision Support System

Legend to Figure 5

Data Input1

- Nodal demands
- Node elevations
- Pipe diameter
- Pipe lengths
- Reservoir shape
- Reservoir size
- Reservoir elevations
- Pump characteristics
- Head loss equation
- Friction coefficients

Scenarios

- Excessive Demand
- Reduced Supply

Data Input2

- Dirichlet conditions
- Boundary Conditions
- Friction factors
- Pressure wave

Data Input3

- Demand figures
- Supply figures
- Operation of network

Expected Results

- Equitable water supply
- Rationalised water supply

Figure 6: Pipe Laying DN 700 Gaba – Gun Hill Reservoir in Kampala

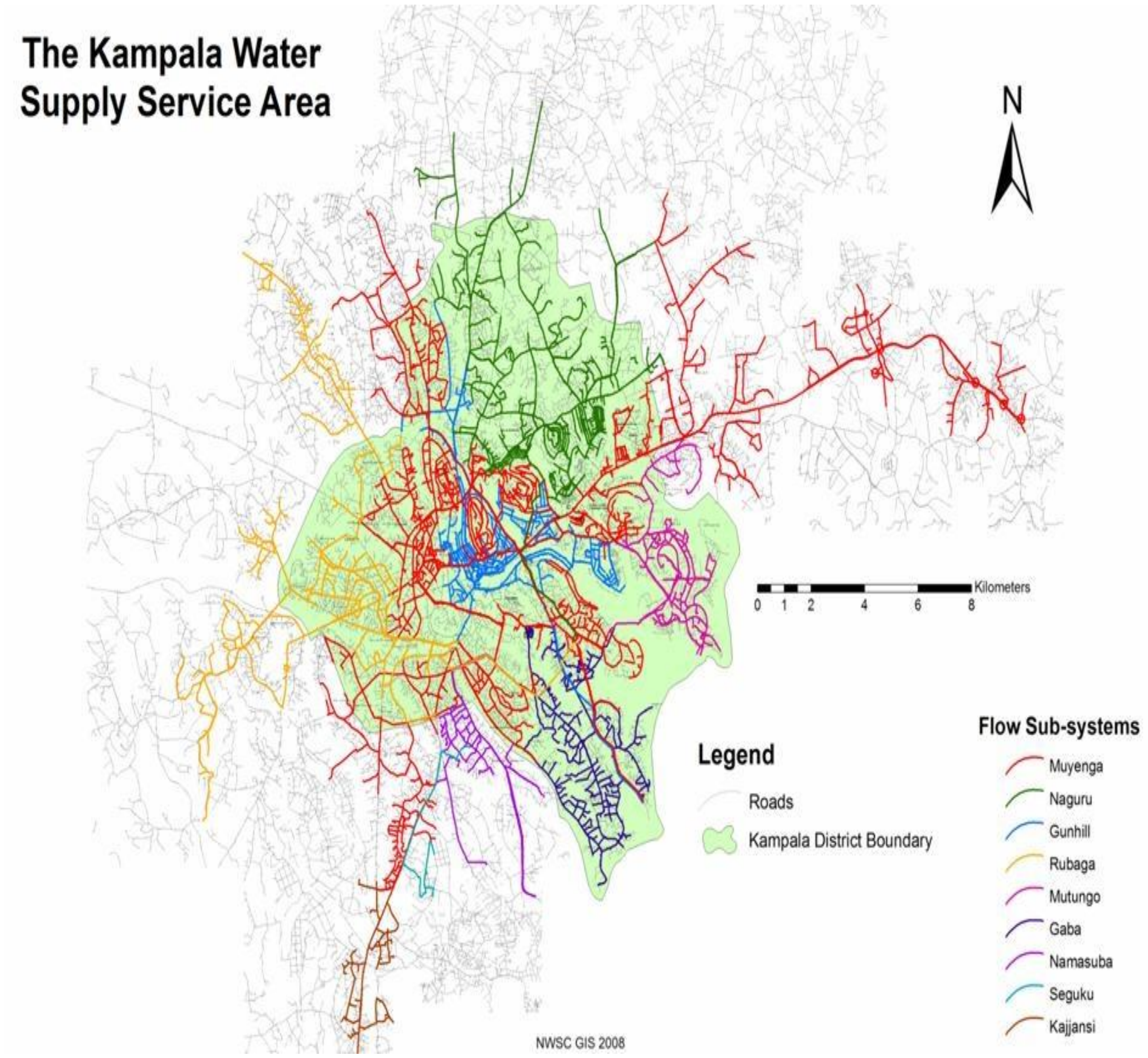

Figure 7: Kampala Water Supply Service Area

1.7 Study Area: Topography and Climate

Kampala ($0^{o}20'$ N, $32^{o}30'$ E) is the largest urban centre in Uganda (Figures 7 and 8), with a population of about 1.7 million people. This accounts for 47% of the urban population in Uganda. It is located on the northern shores of Lake Victoria at an altitude of 1,310 meters above sea level. The climate is typical of an inland tropical city modified by altitude and distance from the sea.

Figure 8: Map of Uganda showing Kampala City

The temperature of the City ranges from a mean minimum of 17°C to a mean maximum of 27°C with a mean of 22°C and a diurnal range of 10°C. However, temperature extremes of 35°C and 12°C have been recorded. The average humidity over the year is 75% typically high in the morning and low in the afternoon. Daily sunshine hours range from a low of 5.7 hours during the month of July to 7.7 hours during January (a mean of 6.5 hours per day). The average annual rainfall is 1,174mm, falling mostly during the two wet seasons of March to May and October to November. The dry seasons are December to February and June to July.

1.8 Structure of the Thesis

The Thesis has been structured into eight chapters; Chapter One, Introduction, contains the background to the study, the statement of the problem, the main and specific objectives, conceptual framework, scope of the research and the area of study. Chapter Two consists of the literature reviewed on water distribution modelling, open channel flow, unsteady flows and mixed flows.

Chapter Three contains mathematical formulations for the study; Chapter Four consists of the methodology of the study that includes characterisation and modelling of the case study network, development of pressure dependent demand approach, programming of pipe filling and the flow regime transition problem, problem analysis and development of a Decision Support System.

Chapter Five contains analysis and presentation of results of the developed models; Chapter Six contains discussion of results and Chapter Seven contains conclusions of the study and recommendations made. The rest of the report consists of references and appendices.

2.0 LITERATURE REVIEW

2.1 General

Most cities rely on networks of pressurised water supply systems for conveyance of potable water. Thus, networks are designed to operate under a pressurised flow regime. However, in practice, there can exist periods with shortfalls in pressure and this affects the quantity of water that can be supplied. The pressure dependency of withdrawals from water distribution systems under conditions of abnormal stress is well accepted (Hayuti et al. 2007).

In recent years, many researchers have attempted to predict behaviour of water distribution systems under pressure-deficient conditions (Ang & Jowitt 2006; Wu, Wang, Walski, Yang, Bowdler & Baggett 2006 & 2009) using models that assume full-flowing pressurised pipelines. Such models provide sensible results when pressures in the system are high enough to meet demand but do not reflect reality when pressures are low. The root of the problem lies in the traditional demand-driven approach (DDA) to water distribution modelling (further discussed in Section 2.3.1) which assumes ever high pressures and that water supply is demand driven, so that demand is a primary model input while pressure is a primary model output. When these parameters are used to complete the continuity and energy equations that underlie the operation of the models, demands are always fixed but pressures vary. Subsequently, when pressures are low, in order to satisfy fixed demand, the models can compute heads below the minimum required for outflow to occur physically at some or all of the nodes (Chandapillai 1991; Ang & Jowitt 2006; Wu et al. 2006 & 2009) and this represents a fundamental weakness of water distribution models.

In reality, when pressures are low, water is supplied in accordance with the available pressures, that is, the lower the pressure the less the water supplied (it is worth noting that while the reverse is true, vis the higher the pressure the more the water supplied, there is always a limit to consumption). If water distribution models are to play an important role in the simulation of low pressure networks and provide realistic results that can help in the solution of low network pressure problems, they should take cognisance of the pressure dependence of withdrawals in order to aid determination of how much water can be supplied during low pressure periods.

Furthermore, in extreme conditions when the pressure is grossly inadequate, the regime in the pipelines may shift from full-pressurised flow to partially-full flow with a free surface during adequate ventilation. This may be caused by a number of reasons such as very high demand, very low supply, inadequate distribution network or inappropriate operating conditions for example closed valves or pump shut down. During these extreme conditions, both the pressurised and free surface flow regimes may co-exist in the network, a situation sometimes referred to as mixed flow. This research focuses on low pressure systems and on systems that are supposed to operate under the pressure regime but in which water flows do not fill the pipe, thereby becoming free surface flow conditions.

During the transition from full-pressurised flow to partially-full flow, a moving water interface advances into the free-surface region. There is need to track the interface in order to explain the development of pressures in a pipeline so that we can make a contribution to the understanding of the co-existence of pressurised and free-surface flows in a water supply network. A major problem with mixed flow analysis is the difficulty involved in treating the moving interface similar to surges (Song, Cardle & Leung 1983) and requires a considerable amount of computational effort to detect its generation and trace its movement. Ingeduld et al. (2006) notes that hydraulic models of intermittent water supply need to simulate the “charging” process in pipes and this requires integration of the continuity motion equations to indicate the positions of the water front in the network at any time.

The major impact of the inadequacy and absence of pressures in a water distribution system is that demand cannot be met. However, there are other outcomes of pressure deficiency such as creation of opportunities for external contamination to enter the distribution system (Fleming, Gullick, Dugandzic & Lechevallier 2006). Leakage points in water mains, submerged air valves, cross-connections and faulty seals or joints can all serve as entry portals for external contaminants when the pressure of water surrounding a distribution system main exceeds the water pressure inside the main.

In addition, the impact of fluctuating pressures on the physical integrity of the distribution system is also a concern (Axworthy 1997). As with high pressure events, low pressure transients may also contribute to pipe fatigue and eventual failure if stress fluctuations of sufficient magnitude and frequency occur (Friedman, Radder, Harrison, Howie, Britton, Boyd, Wang, Gullick, Lechevallier, Wood & Funk 2004; Gullick, Lechevallier, Case, Wood, Funk & Friedman 2005; Fleming et al. 2006). Low pressure fluctuations greater than those occurring under normal operating pressures create stresses and strains that can progressively fatigue and weaken distribution system piping.

This Chapter reviews literature about previous/current research in the subject and applicable theory especially concerning the normal operational network parameters; intermittent water supply, open channel flow, mixed flow analysis, hydraulic transients, models of water supply and decision support systems. Conclusions are then made and gaps present in the literature are identified as a precursor to the formulation of appropriate Methodology.

2.2 Hydraulic Modelling of Water Distribution Networks

In order to meet regulatory requirements and customer expectations, water utilities are realising a growing need to understand better the movement and transformations undergone by treated water introduced into their distribution systems (Kritpiphat, Tontiwachwuthikul & Chan 1998; Rossman 2000; Huang & Fipps 2003). This is enabled by computer simulation of physical behaviour of water distribution systems in order to design, calibrate, rehabilitate, operate and control networks (Cheung et al. 2005) under various operating conditions.

Hydraulic models can be classified according to four broad theories (National Research Council, 2006).

i. Steady State Theory
ii. Extended Period Simulation

iii. Rigid Water Column Theory
iv. Surge Theory

2.2.1 Steady State Theory

The basic network hydraulic approach applicable to time-invariant conditions solves conservation of mass (at each node) and energy (around each loop) equilibrium expressions using an iterative scheme such as Newton-Raphson based on known (static) demand loading and operating conditions (Machell, Mounce & Boxall 2010). This theory is ideal as there are hardly any steady state situations in reality. Nevertheless, it forms part of traditional water distribution modelling when flow conditions between particular time intervals are regarded to be constant with time. This simplifying assumption is what facilitates the modelling process.

2.2.2 Extended Period Simulation

The second approach, applicable to very slow transients, is called Extended Period Simulation (EPS) or quasi-steady theory and involves solving a sequence of steady-state solutions linked by an integration scheme for the differential equation describing the storage tank dynamics. The theory is quasi-steady because within a particular time interval, flow conditions are assumed constant with time however they change in different time intervals. Both inertial and elastic effects are neglected. These models have become ubiquitous within the water industry and are an integral part of most water system design, master planning and fire-flow assessment studies.

2.2.3 Rigid Water Column Theory

Another category of unsteady flow that is suitable for faster (but still relatively slow) transients is called Rigid Water Column Theory (Lumped Parameter Approach). It considers gradually varied flow and slow moving transients under the assumption that water acts as a rigid column and elastic properties of the pipe walls are of no consequence. Thus the density of water is considered to be constant since no expansion is possible. In this approach, the inertia of the fluid in a particular pipe is treated as lumped instead of continuously distributed. This approach will later be used to simulate co-existence of free-surface and pressurised flows because the category of flow to be dealt with will be gradually varied flow and considering low and no pressures, the elastic properties of pipe walls will be irrelevant since no significant pressures will be exerted on the walls.

2.2.4 Water Hammer (Surge) Theory

The last category of unsteady flow applicable to rapid transients is called elastic or water hammer theory (Distributed Parameter Approach) and takes into account the elasticity of both the fluid and the pipe walls in calculations. It represents situations with more rapid and sudden changes in velocity (e.g. rapid valve closure, pump trip) that require consideration of liquid compressibility and pipe wall elasticity. The Water Hammer Theory is used to simulate rapid pressure transients in water pipelines with the aim of determining the maximum pressure that can be produced in a pipeline, so that the pipeline can be reinforced to withstand the pressure.

The last three hydraulic modelling categories are known as unsteady or dynamic flow analysis. These models, as just described, can effectively be used to identify susceptible regions in the distribution system that are of greatest concern for vulnerability to objectionable (low) surges and evaluate how they may be minimised (Boulos, Karney, Wood & Lingireddy 2005).

2.3 Approaches to Water Distribution Modelling

2.3.1 Demand Driven Approach

The principal objective of traditional water distribution modelling is to determine nodal pressures and link flows that can satisfy nodal demands i.e. Demand Driven Approach (DDA) to analysing Water Distribution Systems (WDS). In this method, constitutive equations for flow in water supply networks have to simultaneously satisfy nodal flow continuity and conservation of energy applied to each loop or path. Various methods have been used to solve conventional network analysis problems e.g. Hardy–Cross, Newton–Raphson / Gradient Algorithm. The Newton–Raphson method, illustrated as follows (Kalungi & Tanyimboh 2003), is widely used due to its convenient computational applicability.

The continuity equation for each node j (j=1,…,NJ) may be written as

$$\sum_{i:H_i<H_j} Q_{ij} - \sum_{i:H_i>H_j} Q_{ij} = Q_j^{req} \tag{2.1}$$

where Q_{ij} is the flow in the link ij and NJ is the number of nodes in the network. Q_j^{req} is the nodal outflow (demand). H_i and H_j are piezometric heads at nodes i and j respectively. The Hazen–Williams equation for headloss in a pipe link has the form:

$$Q_{ij} = K_{ij}^{\ 0.54} \left| H_i - H_j \right|^{-0.46} \left(H_i - H_j \right) \tag{2.2}$$

in which K_{ij} is a resistance coefficient for link ij and has the form

$$K_{ij} = \frac{\alpha L_{ij}}{CHW_{ij}^{1.852} D_{ij}^{4.87}} \tag{2.3}$$

where α =10.675 in SI units, L_{ij} = link length, CHW_{ij} = Hazen–Williams coefficient and D_{ij} = diameter.

Equation (2.1) can incorporate Equation (2.2) to become:

$$F_j = \sum_{i:H_i>H_j} \left(\frac{H_i - H_j}{K_{ij}} \right)^{0.54} - \sum_{i:H_i<H_j} \left(\frac{H_j - H_i}{K_{ij}} \right)^{0.54} - Q_j^{req} = 0 \tag{2.4}$$

in which F_j represents the continuity equation for node j. Other network components including pumps, non-return valves, flow-control valves and pressure-reducing valves can be included in Equation (2.4) in a similar way. Choosing nodal piezometric heads as basic unknown parameters, Equation (2.4) can be solved by the following iterative scheme (Kalungi & Tanyimboh 2003):

$$J_H^m \underline{\Delta H}^m = -\underline{F}\left(\underline{H}^m\right) \tag{2.5}$$

$$\underline{H}^{m+1} = \underline{H}^m + \underline{\Delta H}^m \tag{2.6}$$

in which J_H is a Jacobian matrix (matrix of all first-order partial derivatives of a vector-valued function with respect to another vector); $\underline{H}$ is a vector of unknown heads; $\underline{\Delta H}$ is a vector of the respective corrections to nodal heads and $\underline{F}$ is a vector of respective values of the nodal continuity expressions, i.e. F_j, for j=1,…,NJ. The iteration number is denoted by m. The elements of the Jacobian matrix, J_H, are given by

$$\frac{\partial F_j}{\partial H_i} = 0.54\left(\frac{\left|H_i - H_j\right|^{-0.46}}{K_{ij}^{0.54}}\right) = \frac{\partial F_i}{\partial H_j};\forall j.\forall i; i \neq j \tag{2.7}$$

$$\frac{\partial F_j}{\partial H_i} = -0.54\sum_{i \in N_j}\left(\frac{\left|H_i - H_j\right|^{-0.46}}{K_{ij}^{0.54}}\right);\forall j; i \neq j \tag{2.8}$$

where N_j represents nodes connected to node j. To solve the problem computationally, nodal demands are assigned values that are assumed to be fixed. These values are determined either through predetermined criteria or field data. By using a lumped parameter approach the demand values are concentrated at the nodes. The problem then consists of solving the system of equations to determine pipe flow rates and nodal pressures that are consistent with specified demands.

DDA clearly demonstrates that no consideration is given to the relationship between pressure and demand. It assumes that consumer demands are always satisfied regardless of the pressures throughout the system and formulates the constitutive equations accordingly to solve the unknown nodal heads. The pressures are computed when flow rates are determined. The approach, used by almost all traditional network hydraulic solvers works as long as the source pressure can supply minimum pressure head required at the demand nodes (Kalungi & Tanyimboh 2003; Cheung et al. 2005; Tanyimboh 2000; Tanyimboh & Templeman 2007).

The major drawback of DDA is when it purports to meet original (and fixed) water demand at very low or negative pressures. Clearly, when pressures are low, not all demands at nodes can be met. In such cases, if demand driven analysis is used, it may produce very unrealistic results. For example, it would not be surprising to see such warning messages as “negative pressures at 6:00 hrs” from an EPANET2 model (Rossman 2000) while nodal demands are presumed fully satisfied during the same time period. In reality, the volume of water delivered to a node begins to fall short of the required demand as the pressure at the node falls below some threshold value. Almost all demand-driven models possess a partial recognition of this weakness with similar warning messages when negative pressures are calculated from a network hydraulic analysis. Considering the weaknesses of the demand-driven simulation therefore, it is necessary to employ an approach that recognises the pressure dependency of demand and utilises the relationship between pressure and flow in order to establish exactly what flows can be enabled by particular pressures.

2.3.2 Pressure Driven Approach

As already observed, demand driven analysis may produce unrealistic results, that is, very low or negative pressures, especially under partially failed conditions of a system. Many critical operational scenarios involve subnormal operating conditions and to simulate these in a realistic way a new approach, different from demand-driven analysis is required. In such cases, a pressure driven approach also known as the pressure dependent or head-driven analysis (HDA) is superior to demand-driven analysis in that a modeller is able to determine nodes with insufficient supply and the respective magnitudes of the shortfalls. Primacy is given to pressures and a node is supplied its demand fully only if a minimum required supply pressure is satisfied at that node. If the minimum pressure requirement cannot be met, then the fraction of the nodal demand satisfied is determined by recognition of a relationship between nodal head and nodal outflow.

Attempts made to develop numerical methods of analysis which give primacy to pressure in determining flow rate by incorporating a relationship between demand and pressure utilise fixed demands above a critical pressure, zero demand below a given minimum pressure and some relationship between pressure and demand for intermediate pressures (Obradovi 2000; Tabesh & Karimzadeh 2000; Tabesh, Tanyimboh & Burrows 2001; Araujo, Coelho & Ramos 2003; Burrows, Mulreid, Hayuti, Zhang & Crowder 2003; Piller, Brémond & Poulton 2003; van Zyl, Borthwick & Hardy 2003; Todini 2003; Kalungi & Tanyimboh 2003; Cheung et al. 2005; Tanyimboh & Templeman 2007; Lamaddallena & Pereira 2007; Jayaram 2006). Gupta and Bhave (1996) provide a detailed comparison of various flow-head relationships proposed by researchers. Typically the relationship is expressed as follows:

$$h_f = KQ^n \tag{2.9}$$

where h_f represents a drop of pressure in the pipe and *K* denotes a resistance constant of the pipe. The pressure drop is usually related to flow in the pipe by a characteristic power *n*. Darcy-Weisbach and Hazen-Williams are two methods that are commonly used to define the values of *n* and *K.* For the Darcy-Weisbach method, *n* is set at 2 and

$$K = 8f\left(\frac{L}{g_c \pi^2 D^5}\right) \tag{2.10}$$

where *f*, *L*, *D* and g_c denote a Darcy-Weisbach friction factor, a length of pipe, a diameter of pipe and a force-mass conversion factor respectively. For the Hazen-Williams method, the value of *n* depends on the type of liquid in the system. For water it is set at 1.852 and

$$K = \frac{K_1 L}{c^{1.852} D^{4.8704}} \tag{2.11}$$

where K_1 is a constant (10.675 for SI units) and *C* is referred to as the Hazen-Williams coefficient.

The minimum pressure threshold below which no supply is feasible is junction-specific and also depends on demand, elevation as well as the type of service connection and type of development in the area served by that junction. This value is usually the minimum of outlet elevations in the locality served by junction. In the absence of such data, it can be taken as the elevation of the junction itself. The Office of Water Services in England specifies a minimum acceptable static pressure of 7 m (68.5 kPa) below which customers may be entitled to compensation for less than satisfactory service (Tanyimboh 2000). In general, nodal heads of 15 m to 25 m will guarantee satisfactory service at all related top taps in a distribution system (Tanyimboh, Burd, Burrows & Tabesh 1999). Tanyimboh (2000) states that pressures of 15 m to 25 m are the minimum acceptable standards with the use of network models.

2.3.3 Difficulties in Application of Head-Driven Analysis

Use of head-driven analysis is complicated by the following factors:

i. Water distribution networks are almost always skeletonised to some degree depending on the purpose of modelling, forcing demands from secondary networks to be lumped into nearby junctions. Moreover, at the core of pressure-driven analysis is establishment of a relationship between pressure and flow. Due to various headlosses associated with secondary networks and different elevations of outlets in these networks, it is very difficult if not impossible, to define a relationship between pressure at a junction and flow available from that junction into the secondary networks it serves. A tedious way of doing that requires extensive field data collection and calibration to determine *K* and *n* values possibly for each node (Mays 2004; Ozger 2003; Gupta & Bhave 1996).

ii. Lack of robust methods for the computational solution of constitutive equations in head-driven analysis makes the demand-driven network models the primary choice for hydraulic analysis. Due to this shortcoming, Kalungi & Tanyimboh (2003) states that although some researchers have considered pressure driven analysis in the past, computer programs for analysing systems with insufficient pressure in a routine manner are not commercially available.

The practical difficulties mentioned above have made it very desirable that techniques based on existing demand-driven algorithms be developed for predicting deficient network performances. Ozger (2003) and Mays (2004) developed a semi-pressure driven approach that was called the Demand-Driven Available-Demand-Fraction (DD ADF) method which starts with the usual demand-driven analysis. Then, nodes whose pressures are insufficient to fully satisfy their demands are identified and proceeds in an iterative manner using EPANET. Their findings indicated that unrealistic results from an initial demand-driven analysis in the form of pressure deficiencies could be transformed into partial fulfilment of nodal demands without losing computational efficiency and accuracy. In addition, Tanyimboh and Tabesh (1997) also found that when a network with locally insufficient heads is simulated using the demand-driven approach, the deficiency appears to be far more serious and widespread than it is in reality.

2.4 Open Channel Flow

Open channel flow is characterised by existence of a free water surface. In contrast to pipe flow, this constitutes a boundary at which pressure is atmospheric and across which shear forces are negligible. The longitudinal profile of the free surface defines a hydraulic gradient and determines the cross sectional area of flow. It also necessitates introduction of an extra variable, stage, to define the position of the free surface at any point in the channel (Chadwick, Morfett & Borthwick 2004). The stage enables us to establish the head at a point i.e. elevation and pressure head and also the hydraulic grade line which in this case is equivalent to the piezometric head in a full flowing pressurized pipe. This further enables us to work out the pressure difference that enables flow of water to a customer. Open channel flow can be divided into steady and unsteady flows depending on temporal variation of flow parameters i.e. depth and discharge. It can further be subdivided into uniform and non-uniform flow depending on the spatial rate of change of flow parameters.

2.4.1 Transient Low Pressure – Open Channel Flow Conditions

It is worth noting that in water supply situations where intermittent flow is manifested, transient Low Pressure - Open Channel flow (LPOCF) conditions can best be described as unsteady flow since flow depth and velocity vary with time and as gradually varied flow since the rate of change of flow depth and velocity along the channel is very low. Gradually Varied flow (GVF) is non-uniform flow whose spatial rate of flow is sufficiently low to imply translatory wave motion of long wave length and low amplitude (Chadwick et al. 2004) such that the assumption of parallel streamlines and hydrostatic pressure distributions is reasonable. In this research therefore, LPOCF conditions were modelled as unsteady gradually varied flow.

In unsteady non-uniform flow, the discharge Q, varies as a function of time and length along the pipe and all the hydraulic factors of a cross-section change as a function of time and length i.e. water depth, cross-sectional area and water surface width. The full dynamic wave equations that are used to solve unsteady gradually varied flow are the Saint Venant / Shallow Water Equations which were derived in 1871 by A.J.C Barre de Saint Venant (Chow 1959) based upon the following assumptions:

i. Flow is one-dimensional i.e. velocity is uniform over a cross section and the water level across the section is horizontal.

ii. The streamline curvature is small and vertical accelerations are negligible, hence the pressure is hydrostatic. Gradually Varied Unsteady flow implies translatory wave motion of long wave length and low amplitude in which case the assumption of parallel streamlines and hydrostatic pressure distributions is reasonable (Chadwick et al. 2004).

iii. Effects of boundary friction and turbulence can be accounted for through resistance laws analogous to those used for steady state flow.

iv. The average channel bed slope is small so that the cosine of the angle it makes with the horizontal may be replaced by unity.

Unsteady flows are a very complex flow type, requiring the solution of energy, momentum and friction equations with time (Chadwick et al. 2004). Three conservation principles i.e. conservation of water mass, conservation of mechanical energy content of water and conservation of momentum content of water are available for analysis of one dimensional unsteady flow (Franz & Melching 1997a).

Equations derived from application of the conservation of mass principle (which becomes the conservation of water volume if density is constant) are often referred to as continuity equations. The equation obtained with the conservation of momentum principle is simpler than the equation obtained with the conservation of energy principle. The simplicity of the equation obtained with the conservation of momentum principle is twofold; the equation includes fewer terms and less information is needed for each cross section. In addition, more accurate equations can be approximated with the momentum principle than with the energy principle (Franz & Melching 1997b; Bourdarias & Gerbi 2007; 2008; 2009; Liovic, Rudman & Liow 1999).

Any two of the three principles can be used to solve unsteady flow problems, depending on the available data. Unsteady flow equations are key to the understanding of the unsteady flows in pipelines.

2.4.2 Conservation of Mass

The continuity equation for one-dimensional unsteady open channel flow can be expressed as (Osman 2006; Chadwick et al. 2004; Franz & Melching 1997a&b)

$$\frac{\partial A}{\partial t}+\frac{\partial Q}{\partial x}=0 \tag{2.12}$$

where A is cross sectional area of flow and Q is flow rate. All quantities in the equation are algebraic expressions and can be positive or negative; therefore, a negative outflow is an inflow. The equation is a statement of the conservation of mass principle on a per-unit-length basis and can also be stated as

$$\frac{\partial y}{\partial t} + V\frac{\partial y}{\partial x} + y\frac{\partial V}{\partial x} = 0 \quad (2.13)$$

which is an alternative way of stating the continuity equation for unsteady flow.

2.4.3 Conservation of Momentum

The momentum conservation equation (Equation of Motion) can be written in the form (Osman 2006; Chadwick et al. 2004; Franz & Melching 1997a&b)

$$\frac{\partial Q}{\partial t} + \frac{\alpha\partial\left(Q^2/A\right)}{\partial x} + gA\frac{\partial h}{\partial x} - gA\left(S_0 - S_f\right) = 0 \quad (2.14)$$

where α is an energy coefficient normally equated to unity for SI units, *t* is the time interval being considered, x is the length of the reach being considered, *h* is depth of flow, *g* is gravitational acceleration, S_o and S_f are channel slope and friction slope, respectively.

Equations 2.13 and 2.14 are called St. Venant equations. Different types of formulations can be given for the Saint Venant equations depending on the problem.

2.4.4 Solution of Unsteady Flow Equations

An important family of equations that is often encountered in hydraulics is based on the following equation (Chadwick et al. 2004):

$$a\frac{\partial^2 f}{\partial x^2} + b\frac{\partial^2 f}{\partial y \partial x} + c\frac{\partial^2 f}{\partial y^2} = 0 \quad (2.15)$$

f is some variable/function such as velocity. If $b^2 - 4ac > 0$ then a typical form is

$$\frac{\partial^2 f}{\partial x^2} - c^2\frac{\partial^2 f}{\partial y^2} = 0 \quad (2.16)$$

This is the hyperbolic equation which can be applied to unsteady flows. The Saint Venant/Shallow wave equations are classified as Partial Differential Equations of the hyperbolic type. All flow variables are functions of both time and distance along the channel. In other words, at a given location, the flow depth, discharge and the other flow variables vary with time. Likewise, at a fixed time, the flow variables change along the channel. For a given channel of known properties (cross sectional geometry, roughness factor, longitudinal slope), the unknowns are the discharge *Q* and flow depth *y*. The other flow variables such as the area *A* and the friction slope S_f can be expressed in terms of *Q* and *y*. The independent variables are time *t* and distance along the channel x. There are several ways of attempting solutions to the Saint Venant equations as explained in section 2.4.5 and 2.4.6 below.

2.4.5 Simplification of St. Venant Equations

Unsteady gravity flows have been traditionally modelled by numerically solving the one-dimensional equations of continuity and momentum. Commonly used models range in sophistication from Kinematic Wave to Full Dynamic Wave solution of these equations (Yen 2001). Simplified solutions are used in special cases when the full Saint Venant equations are not needed and are discussed in the following subsections.

2.4.5.1Kinematic Approximation

In the Kinematic Formulation it is assumed that friction slope equals bottom slope (Osman 2006; Tucciarelli 2003), i.e. $S_f = S_o$. This implies that the momentum conservation equation collapses to a simple form and the equations can be written as:

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = 0 \qquad (2.17)$$

$$Q = \frac{1}{n} A R^{\frac{2}{3}} s_0^{\frac{1}{2}} \qquad (2.18)$$

i.e. discharge *Q* at any point can be calculated directly as a function of bottom slope S_o. This simplification leads to one single dependent variable (water depth *h*).

2.4.5.2Diffusion Analogy

The Kinematic approximation cannot take into account the influence of backwater in hydraulic computations (Helsinki University of Technology 2003) hence it is suited only for steep slopes where the water level in downstream sections does not significantly influence discharge in the channel (Sevuk 1973). For mild slopes, the method yields unsatisfactory results (Osman 2006). An approximation that eliminates the problem of the Kinematic Approximation is the Diffusion Analogy where it is assumed that acceleration terms in Equation 2.14 i.e. the first two terms, are neglected but the surface elevation term is included (Helsinki University of Technology 2003). The equations to be solved in the diffusion analogy are:

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = 0 \qquad (2.19)$$

$$\frac{\partial h}{\partial x} = S_0 - S_f \qquad (2.20)$$

Equation 2.20 can be solved in terms of S_f to get an equation for calculating discharge *Q*:

$$S_f = S_0 - \frac{\partial h}{\partial x} \qquad (2.21)$$

$$Q = \frac{1}{n} A R^{\frac{2}{3}} \sqrt{S_f} = \frac{1}{n} A R^{\frac{2}{3}} \sqrt{S_0 - \frac{\partial h}{\partial x}} \qquad (2.22)$$

2.4.6 Method of Characteristics

The differential equations of Saint Venant cannot be solved analytically unless certain simplifications are carried out such as the neglect of certain terms and simplification of boundary conditions as is the case in the Kinematic Approximation and Diffusion Analogy and this can lead to serious errors (Marriott 2009). With the advent of the digital computer numerical solutions can be obtained and thus no simplifying assumptions to the basic equations need be made.

A number of different numerical methods are available to solve hyperbolic differential equations. The best known is the Method of Characteristics (Wylie & Streeter 1978; Chaudary 1979; Chadwick et al. 2004). The method is widely used for transient flow in a closed conduit because it is simple and also provides good insight into behaviour of hyperbolic equations. In the method, the original set of partial differential equations in Equations 2.2 and 2.3 are transformed into two sets of simultaneous ordinary differential equations.

$$\frac{dx}{dt} = v \pm c \quad \text{and} \quad \frac{dy}{dt} \pm \frac{c}{g}\frac{dv}{dt} \pm c\left(S_f - S_0\right) = 0 \qquad (2.23)$$

The above equations are known as characteristic equations and are valid along two different characteristic lines, namely, the positive characteristic line (C^+) and the negative characteristic line (C^-) as shown in Figure 9 (Chadwick et al. 2004; Gomez & Achiaga 2008; Sturm 2001)**.**

By discretising the transformed equations we can obtain the velocity and depth of flow at point P as:

$$y_p = \frac{1}{C_R + C_S}\left[y_R c_s + y_s c_R \left(\frac{v_R - v_s}{g} - \Delta t.\left(S_{fR} - S_{fS}\right)\right)\right] \qquad (2.24)$$

$$v_p = v_R - \frac{g}{c_R}\left(y_p - y_R\right) + g.\Delta t.\left(S_o - S_{fR}\right) \qquad (2.25)$$

where the values of variables at the points R and S can be obtained by linear interpolation. If we are at a boundary then one of the characteristic equations is outside of the problem boundary, so we need a boundary condition that will be the pressure head at this boundary.

When converted into finite difference equations, the characteristic equations lead to a set of simultaneous algebraic equations for the unknowns. There is need to ensure that the size of the time step conforms to Courant's stability condition (Tullis 1989) that ensures convergence of the finite difference equations (Wylie & Streeter 1978; Chadwick et al. 2004). As we want information along the pipe to travel along the characteristic lines, we select the time interval Δt such that $\Delta t \leq \frac{\Delta x}{|v \pm c|}$ for free surface flow and the one for pressurised flow has c replaced with a. Thus, the size of Δx and the wave celerity c determine the size of the time interval. The parameter a is much greater than c so that $\Delta t < \frac{\Delta x}{v + a}$ is the stability condition that is applied to the whole grid.

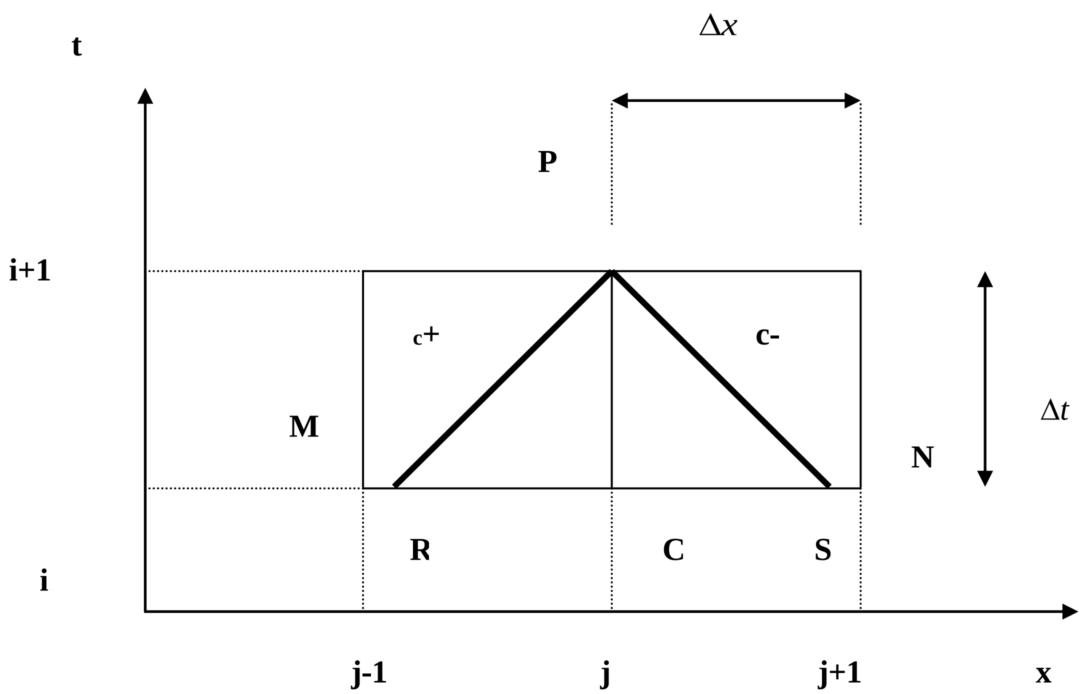

Figure 9: Fixed Grid System for Water Supply Pipeline

The characteristic equations when expressed in numerical (finite difference, finite element or finite volume form) can be programmed for automatic evaluation on a digital computer (Chadwick et al. 2004; Marriott 2009).

2.5 Free Surface Flows in Pressurised Water Supply Networks

Existing water distribution models can only be used both with demand driven and pressure driven approaches when pipes are flowing full and if not, these models do not give sensible results. Many a time the pipe may be near-full and can easily be pressurised so that water starts flowing out of the system. There is therefore, a need to resort to models that can simulate pressurised as well as free surface flows at the same time and in our case, with emphasis on the transition between pressurised and free surface flows. Such models are currently not available in water supply modelling and is what this research seeks to develop by analysing this scenario of mixed flow i.e. flow between full pressurised flow and partially-full (free surface) flow conditions by extensively studying the interface between full and partially-full flow conditions. This aids in understanding the development and propagation of pressure surges in pipelines so that during the transient state while supply is low, pressures can be determined simultaneously with discharges that can be availed to consumers.

2.5.1 Flow Regime Transition Problem

Equations of continuity and motion for a one dimensional unsteady flow in an open channel can be restated as (Trajkovic, Ivetic, Calomino & Dippolito 1999; Song et al. 1983; Leon 2007; Leon, Ghidaoui, Schmidt & Garcia 2010; Gomez & Achiaga 2008; Aldrighetti & Stelling 2006; Duchesne, Mailhot, Dequidt & Villeneuve 2001; Schmitt, Thomas & Ettrich 2004; Blais, Gatto, Bourdarious & Gerbi 2006)

$$\frac{\partial y}{\partial t} + v\frac{\partial y}{\partial x} + \frac{c^2}{g}\frac{\partial v}{\partial x} = 0 \qquad (2.26)$$

And $g\frac{\partial y}{\partial x} + \frac{\partial v}{\partial t} + v\frac{\partial v}{\partial x} + g\left(S_f - S_0\right) = 0$ (2.27)

in which c is the gravity wave celerity given by

$$c = \sqrt{\frac{gA}{T}} \tag{2.28}$$

and y = depth; v = mean velocity; g = gravitational acceleration; A = cross-sectional area of flow; T = top width of flow; S_f = friction slope, S_0 = bed slope, t = time and x = distance measured along the channel.

Corresponding equations for pressurised or closed conduit flow can be written as (Song et al. 1983)

$$\frac{\partial y}{\partial t} + v\frac{\partial y}{\partial x} + \frac{a^2}{g}\frac{\partial v}{\partial x} = 0 \tag{2.29}$$

$$g\frac{\partial y}{\partial x} + \frac{\partial v}{\partial t} + v\frac{\partial v}{\partial x} + g\left(S_f - S_0\right) = 0 \tag{2.30}$$

in which a = the speed of the water hammer wave and y should be regarded as the piezometric head measured from the pipe invert rather than the flow depth.

This research concentrated on analysis of situations where free surface and pressurised flows co-exist in water mains, a situation also called mixed flows. The two sets of equations that describe free surface and pressurised flows are deceptively similar (Song et al. 1983) and may invite difficulties if not treated carefully. A reasonable result cannot be expected if either the mixed flow regime or the pressurised regime is regarded as a simple extension of the other. In otherwords, the transition between the flow regimes in closed conduits poses some difficulties with regards to the numerical solution of the problem since the open-channel mass and momentum equations do not hold in pressurised flow regimes due to the absence of a free surface.

The subject requiring the greatest amount of research effort is the characteristic of the interface that joins the free surface flow regime and the pressurised flow regime (Song et al. 1983). During an initial stage of pressurisation the interface (or the pressurisation wave) behaves as a moving internal hydraulic jump of extremely large magnitude. During depressurisation, however, the interface may behave like a smooth negative wave. Obviously, a pressurisation interface must be treated as a shock wave. Less obvious is the fact that the depressurisation interface must also be treated as an area of abrupt flow change because $c \to \infty$ as $T \to 0$ as calculated by Equation 2.28. The third term in the left hand side of Equation 2.28 becomes undefined as the water level approaches the pipe crown and flow surface width approaches zero.

2.6 Models for Analysing Mixed Flow

The following Section discusses three methods or models for solving mixed flows i.e. Priessman Slot, Decoupled Pressure Approach and Interface Tracking Method.

2.6.1 Priessman Slot

In this method, the mixed flow problem is simplified to a free surface flow problem by adding a hypothetical/imaginary slot at the crown of the pipe which allows depth of flow to exceed pipe diameter and give the effect of pressurised flow (Yen 1986; Butler & Davies 2004; Cunge & Wegner 1964; Cunge, Holly & Verwey 1980; Song et al. 1983; Zhang & Lerner 2000). The width of the slot *b* is calculated precisely to suit the conditions and must not be so wide that it has a significant effect on continuity. Additionally, the slot is determined so that the gravity wave speed equals the pressure wave speed. The hypothetical slot allows the entire problem to be treated as an open channel flow problem. The Priessman-Cunge-Wegner model assumes that flow is ventilated to the atmosphere everywhere in the system which allows a phase change between open channel and closed conduit conditions to take place everywhere in the system.

2.6.2 Decoupled Pressure Approach

Recently, Vasconcelos, Wright and Roe (2006) introduced a Decoupled Pressure Approach (DPA), which is formulated by modifying the open-channel Saint-Venant equations to allow for overpressurisation, assuming that elastic behaviour of pipe walls accounts for the gain in pipe storage. The Preissmann slot and the DPA approach use free surface flow equations to simulate mixed flows. One of the limitations of this shock-capturing approach is presence of what the author calls “post-shock oscillations" near open channel-pressurised flow interfaces. To keep these oscillations small, lower values for the pressure wave celerity may be used, but this may compromise the accuracy of the simulation if pressurised transients are simulated. This approach was not used in this study since elastic pipe wall behaviour was not considered especially because of the low pressures which would hardly have any significant pressure force effect on the pipe walls.

2.6.3 Interface Tracking Method

This class of flow regime transition models is exemplified by the works of Wiggert (1983), Hamam and McCorquodale (1982) and Li and McCorquodale (1999). Wiggert (1983) modified the Priessman-Cunge-Wegner model by introducing a moving interface between the free surface and pressurised flow regimes. The method treats the two flow regimes separately but joined together by an interface which is regarded as a shock wave. Song et al. (1983) used the characteristic method for both open channel flow and closed conduit flow regimes. Identical equations and solution techniques were used throughout the system except for a special treatment at the interface. These models solve the ordinary differential equations (ODE) based on a momentum balance in a rigid column represented by the pressurised portion of the flow. In each time step, the ODE is solved and the velocity of the rigid column, speed, location and intensity of the shock wave is updated. The location of the pressurisation front is obtained using the continuity equation across the moving interface. The flow conditions near the interface are thus calculated using a mass and momentum balance in a control volume. The free surface portion of the flow is solved by the method of characteristics. This model, also called a shock-fitting model is appropriate when the energy contained in the flow is sufficient to pressurise the flow through a hydraulic jump. The water depths and velocities near the interface are obtained using two shock-boundary conditions plus three characteristic equations (Politano, Odgaard & Klecan 2005).

If velocity changes are more gradual, acceleration of flow between two adjacent sections can be neglected and the flow near the interface can be simulated using momentum and mass balance in a moving control volume. This method facilitates accurate tracking of the interface conserving mass and is the approach that was used in this study.

2.7 Modelling Flow Discontinuities

While the Method of Characteristics (MoC) is a valuable approach in the sense that it provides a deep understanding of the nature of shallow water fronts, the approach is limited by its inability to handle flow discontinuities. The hyperbolic nature of mass and momentum partial differential equations allows discontinuities in the solution in form of hydraulic bores (Vasconcelos 2005). Analysis of the moving interface or surge front i.e. solution for the surge location and velocity is carried out by solving the mass and momentum conservation equations across the interface, since in the transition region there are no valid characteristic equations that cross the interface trajectory (Gomez & Achiaga 2008).

Solution of the interface requires determination of the interface location x and velocity w. For flow upstream of the interface (pressurised flow), we need to determine the pressure head h_1 and velocity V_1 while for flow downstream of the interface (free-surface flow), we need to determine its flow depth y_2 and velocity V_2. The remaining unknowns require four equations to solve them:

i. two characteristic equations for free surface flow
ii. mass and momentum conservation equations

The Method of Characteristics (MoC) formulation is used for numerical integration of differential flow equations in the free surface region. The Conservation of Mass equation is (Politano et al. 2005):

$$\frac{d\left(A_1 - A_2\right)}{dt}\Delta x = A_1 V_1 - A_2 V_2 = 0 \tag{2.31}$$

or $$A_1\left(V_1 - w\right) = A_2\left(V_2 - w\right) \tag{2.32}$$

where

A_1, V_1 are respectively, the cross sectional area of flow and velocity at the upstream (pressurised) end of the interface

A_2, V_2 are respectively, the cross sectional area of flow and velocity at the downstream (free-surface) end of the interface

Δx is the length of the control volume that contains the interface

w is surge velocity

It can be seen that Equation 2.31 is more representative of the dynamic situation that is being considered than Equation 2.32, since the interface is moving. However, Equation 2.31 does not consider the effect of the interface velocity and hence can be modified to the following:

$$\frac{d(A_1 - A_2)}{dt}\Delta x = A_1(V_1 - w) - A_2(V_2 - w) \qquad (2.33)$$

The Conservation of Momentum shall be:

$$0 = A_2V_2^2 - A_1V_1^2 + g\left[\overline{y_2}A_2 - \left(h_1 - z_1 - \frac{D}{2}\right)A_1 + \Delta x\left(\frac{A_2 + A_1}{2}\right)S_o\right] \qquad (2.34)$$

or $$g\left(A_2\overline{y_2} - A_1h_1\right) = A_1(V_1 - w)(V_1 - V_2) \qquad (2.35)$$

where

$\overline{y_2}$ is the distance from the free surface to the centroid of the wetted cross section.

h_1 is the pressure head in the pressurized region

z_1 is elevation above datum

D is diameter of the pipe

Equations 2.31 and 2.32 are credited to Politano et al. (2005) while Equations 2.34 and 2.35 are credited to Fuamba (2003).

2.8 EPANET

EPANET (Rossman 2000) is a computer program that performs extended period simulation of hydraulic and water quality behaviour within pressurised pipe networks (a network consists of pipes, nodes or junctions, pumps, valves and storage tanks or reservoirs). EPANET tracks the flow of water in each pipe, the pressure at each node, the height of water in each tank and the concentration of chemical species throughout the network during a simulation period comprised of multiple time steps.

EPANET is designed to be a research tool for improving our understanding of the movement and fate of drinking water constituents within distribution systems. It can be used for many different kinds of applications in distribution systems analysis for example sampling program design, hydraulic model calibration, chlorine residual analysis and consumer exposure assessment. Running under Windows, EPANET provides an integrated environment for editing network input data, running hydraulic and water quality simulations, and viewing the results in a variety of formats. These include colour-coded network maps, data tables, time series graphs, and contour plots.

Full-featured and accurate hydraulic modelling is a prerequisite for doing effective water quality modelling. EPANET contains a state-of-the-art hydraulic analysis engine that includes the following capabilities: places no limit on the size of the network that can be analyzed; computes friction headloss using the Hazen-Williams, Darcy-Weisbach or Chezy-Manning formulas; includes minor head losses for appurtenances such as bends and fittings; models constant or variable speed pumps; computes pumping energy and cost; models various types of valves including shutoff, check, pressure regulating, and flow control valves; allows storage tanks to have any shape (i.e., diameter can vary with height); considers multiple demand categories at nodes, each with its own pattern of time variation; models pressure-dependent flow issuing from emitters (sprinkler heads); and can base system operation on both simple tank level or timer controls and on complex rule-based controls.

EPANET's hydraulic simulation model component computes junction heads and link flows for a fixed set of reservoir levels, tank levels and water demands over a succession of points in time. From one time step to the next reservoir levels and junction demands are updated according to their prescribed time patterns while tank levels are updated using the current flow solution. The solution for heads and flows at a particular point in time involves solving simultaneously the conservation of flow equation for each junction and the headloss relationship across each link in the network. The method used in EPANET to solve the flow continuity and headloss equations that characterise the hydraulic state of the pipe network at a given point in time can be termed as a hybrid node-loop approach also called the Gradient Algorithm (Todini & Pilati 1987). This process, known as "hydraulically balancing" the network solves the nonlinear equations iteratively.

Due to the aforementioned factors and because of its flexibility and adaptability to various research objectives, EPANET was selected from various water distribution softwares as one of the tools to be used in this research. Other hydraulic softwares reviewed were found inadequate since they deal with open channel flows, drainage and river hydraulics which could not find direct application in this study. Nevertheless they provided valuable insight and ideas for this research.

2.9 Conclusions from Literature Review

i. Demand driven analysis of water distribution networks produces inaccurate results when the network is constrained while pressure driven analysis yields more meaningful results. Although some researchers have considered pressure driven analysis in the past, computer programs for analysing systems with insufficient pressure in a routine manner are not commercially available.

ii. Work done so far has confirmed the existence of transient LPOCF conditions and mixed flow. Need has been identified to analyse the interface between full-flow and partially full-flow in pressurised systems. This calls for numerical methods and arrangements that can be subsequently modelled into a computer based management system for networks prone to these unusual mixed flow regimes.

Findings from the research provided valuable insight into the events surrounding development and propagation of pressures and intermittent water supply with the aim of designing more pragmatic systems.

iii. Water distribution systems are designed and modelled as full pressurised systems whereas sometimes pipelines flow partially full and under free surface flow conditions. A detailed hydraulic model of intermittent water supply system needs to simulate the 'charging' process in pipes. This requires integration of the momentum equation and the water-column velocity equation to generate positions of the water front in the network at any time. This may be a short or long period after supply is resumed when the pipes are filling from a near empty state to a fully charged pressurised state. A major problem with a mixed flow model is the difficulty involved in treating the moving interface in pressurised systems facing LPOCF conditions, a component that is lacking in water distribution modelling. No software is currently available to solve the transient low pressure – open channel flow problem and it was one of the objectives of this research that by solving, would help predict the exact time at which different consumers get water.

iv. Modelling the process of pipe filling and analysis of transient LPOCF conditions in pressurised water distribution networks can be best categorised as gradually varied unsteady flow as opposed to unsteady rapidly varied flow that yields hydraulic transients of water hammer (Chadwick et al. 2004). This means that the approach to the solution of gradually varied flow is very different from that of the water hammer transients.

v. Work on gradually varied unsteady flow has mostly been applied in the hydraulics of rivers and with regard to mixed flows (Song et al. 1983). The work closest to this is analysis of rapidly varied flows, thus modelling water supply in transient LPOCF conditions has not been studied before. Assuming the time rate of flow change in a water supply network is small, existing models often attempt to simulate the mixed flow and pressurised flow regimes in a pressurised network with only minor adjustments, or ignoring the mixed flow altogether. Considerable effort has got to be devoted to a proper representation of the moving interface.

vi. Analysis of unsteady flow equations was carried out using the two principles of Conservation of Mass and Conservation of Momentum. These two principles are adequate for the analysis; require fewer terms, less information and are more accurate than the Conservation of Energy principle.

vii. Existing water distribution models cannot be used to simulate mixed flow because they deal with either pressurised networks or free surface flow conditions alone. There is therefore a need to resort to models that can simulate pressurised as well as free surface flows at the same time. Such models are currently not available and it is that aspect that this research seeks to develop.

3.0 METHODOLOGY-1: DEVELOPMENT OF MATHEMATICAL FORMULATIONS AND MODELS USED

3.1 General

Analysis of unsteady flow in this research was carried out using Conservation of Mass and Conservation of Momentum principles. Two governing equations were explicitly solved because the flow and depth of the water surface were both unknown. One of the governing equations was the conservation of water mass and the other was the conservation of water momentum. Computational elements and approximations to differential or integral terms in the governing equations were used to develop two equations for each computational element written in terms of elevations and flows at the ends of the element. A computational element with respect to time was also considered; the time axis was divided into finite increments that, ideally, were short enough so that the approximations of the differential and integral terms were sufficiently accurate. Because of this dependence on time, the governing equations involve not only the unknown flow and depth at two points along the channel but also at two points in time.

This Chapter details development of physical and mathematical models that back up the study as well as a discussion of unsteady flow dynamic wave equations of St. Venant arising from conservation of mass and momentum. The Chapter also illustrates the solution of the equations using the Method of Characteristics and mathematical formulations to solve the moving interface.

3.2 Conservation of Mass

Considering a control volume with length Δx and cross sectional area A (shown in Figure 10), the water balance components of the section are as follows:

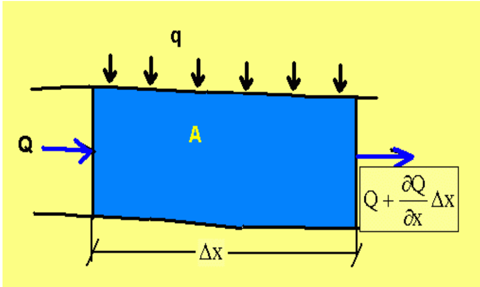

Figure 10: Water balance of a control volume

Incoming $= Q$; Outgoing $= Q + \frac{\partial Q}{\partial x}\Delta x$; Change in Storage = $\frac{\partial A}{\partial t}\Delta x$

where Q is the incoming discharge. The difference between incoming and outgoing mass equals the change of storage (volume) over time or the change in volume of the water in the control volume during any time interval. In other words,

$$\int_{X_L}^{X_R}\left[A(x,t_U) - A(x,t_D)\right]dx = \int_{t_D}^{t_U}\left[Q(x_L,t) - Q(x_R,t)\right]dt \qquad (3.1)$$

The time interval of integration is defined by two points in time, t_U and t_D, such that $t_U > t_D$.

The continuity equation for one-dimensional unsteady open channel flow can thus be expressed as (Osman 2006; Chadwick et al. 2004, Franz & Melching 1997a&b):

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial x} = 0 \qquad (3.2)$$

Density is constant and is not shown in the equation because each term would have a constant multiplier that cancels from the relation. Thus the conservation of mass is equivalent to conservation of water volume (assuming incompressible flow). The area in the above equation (Franz & Melching 1997a&b) should be considered the volume per unit length of channel. Thus the time derivative of area gives the rate of change of volume per unit length. The derivative of the flow rate in the channel with respect to distance should be considered the channel outflow per unit length of channel. All of the quantities in the equation are algebraic expressions and can be positive or negative; therefore, a negative outflow is an inflow. The equation is a statement of the conservation of mass principle on a per-unit-length basis.

By remembering the small change in the area of the cross section $dA = Bdh$ (defined as the area of a rectangle Bdh) as shown in Figure 11, the continuity equation can also be given in the form

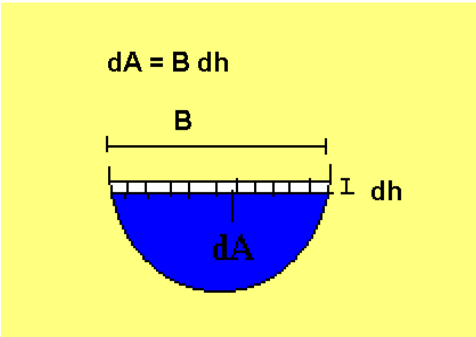

Figure 11: Incremental Area $dA = Bdh$

$$B\frac{\partial h}{\partial t} + \frac{\partial Q}{\partial x} = 0 \tag{3.3}$$

Taking $B = T$ and $y = h$ assuming a rectangular channel (for simplification purposes) with width T, depth of flow y and velocity V from Equation 3.3 we obtain

$$T\frac{\partial y}{\partial t} + \frac{\partial (Ty)V}{\partial x} = 0 \tag{3.4}$$

then

$$T\frac{\partial y}{\partial t} + V\frac{\partial (Ty)}{\partial x} + (Ty)\frac{\partial V}{\partial x} = 0 \tag{3.5}$$

and

$$T\frac{\partial y}{\partial t} + VT\frac{\partial y}{\partial x} + Ty\frac{\partial V}{\partial x} = 0 \tag{3.6}$$

ultimately giving

$$\frac{\partial y}{\partial t} + V\frac{\partial y}{\partial x} + y\frac{\partial V}{\partial x} = 0 \tag{3.7}$$

which is an alternative way of stating the continuity equation for unsteady flow.

3.3 Conservation of Momentum

Momentum is defined as mass multiplied by velocity thus momentum is zero if velocity is zero. The rate of change of momentum is equal to the forces acting on it. The rate of change of momentum of a body is given by (Franz & Melching 1997a&b):

Force = rate of change of momentum

$$F = \frac{(mv_2 - mv_1)}{\Delta t} = ma \tag{3.8}$$

where: *F* is the force, *m* is the mass and V_1 and V_2 are the initial and final velocities, respectively, Δt is the time over which the change of velocity occurs, and *a* is acceleration of the body. In hydraulics it is convenient to notice that density ρ (kg m^{-3}) multiplied by discharge *Q* (m^3 s^{-1}) gives the amount of mass flowing through a cross section over time i.e. $m/\Delta t$ in Equation 3.8 can be replaced by ρQ giving

$$F = \rho Q(V_2 - V_1) \tag{3.9}$$

The conservation of momentum in the *x*-direction requires that the change of momentum in the control volume during time step Δt must be equal to the sum of the net inflow of momentum into the control volume and the integral of the external forces acting on it over the same time interval. Momentum is the product of mass and velocity and momentum flux through the flow section is the product of the mass flow rate and velocity and therefore it can be written that

Momentum inflow = $\rho Q v = \rho Q^2 / A$ (3.10)

while

Momentum outflow = $\rho Q^2 / A + \frac{\partial(\rho Q^2 / A)}{\partial x}\Delta x$ (3.11)

thus

Change of momentum over time = $\frac{\partial(\rho Q)}{\partial t} = \rho \frac{\partial Q}{\partial t}$ (3.12)

The important external forces acting upon the control volume in the x-direction are pressure, gravity, and frictional resistance.

Gravity = $\rho g A \Delta x \sin\Theta = \rho g A \Delta x S_0$ (3.13)

Friction = $\rho g A \Delta x S_f$ (3.14)

Pressure = $\rho g h$; Pressure force = $\rho g A h$ but $h = \frac{\partial h}{\partial x}\Delta x$ (3.15)

and thus

Pressure Change = $\rho g A \Delta x \frac{\partial h}{\partial x}$ (3.16)

where the angle Θ is so small that sin Θ is equal to tan Θ = S_o i.e. bottom slope.
The total force F due to pressure over a certain cross section can be integrated from

$$F = \rho \int_o^h g(h-y)B(y)dy \quad (3.17)$$

where $B(y)$ is the width of the cross section. For a rectangular cross section with bottom width B_B and total height h, the force can be integrated:

$$F = \frac{1}{2}\rho g B_B h^2 \quad (3.18)$$

The change of pressure in the channel reach is

$$\frac{\partial F}{\partial x}\Delta x = \rho g B_B h \Delta x \frac{\partial h}{\partial x} = \rho g A \Delta x \frac{\partial h}{\partial x} \tag{3.19}$$

which is valid for cross sectional area, but it can be shown that Equation 3.19 holds for irregular cross sections as well.

The momentum conservation equation can now be written in the form

$$\frac{\partial Q}{\partial t} + \frac{\alpha \partial \left(Q^2 / A\right)}{\partial x} + gA\frac{\partial h}{\partial x} - gA\left(S_0 - S_f\right) = 0 \tag{3.20}$$

where α ,an energy coefficient, is normally equated to unity in SI units. Equation 3.20 is the governing equation for unsteady gradually varied flow in an open channel.

3.4 Method of Characteristics

Governing equations for gradually varied unsteady flow in open channel are given by

$$\frac{\partial y}{\partial t} + \left(\frac{A}{T}\right)\frac{\partial V}{\partial x} + V\frac{\partial y}{\partial x} = 0 \tag{3.21}$$

$$\frac{\partial V}{\partial t} + V\frac{\partial V}{\partial x} + g\frac{\partial y}{\partial x} = g\left(S_0 - S_f\right) \tag{3.22}$$

Equations (3.21) and (3.22) represent the continuity and dynamic equation, in non-conservation form, respectively. Here, A = Cross-sectional area, T = top width, y = flow depth, V = Velocity, g = acceleration due to gravity, S_0 = bed slope, S_f = friction slope, x = distance and t = time. The friction slope is given by $S_f = \frac{fV|V|}{2D}$ for pressurized flows and $S_f = \frac{n^2 V|V|}{R_h^{4/3}}$ for free-surface flows, with f the Darcy-Weisbach friction factor, n the Manning's Coefficient, D the pipe diameter and R_h the hydraulic radius.

Equations 3.21 and 3.22 are a set of coupled non-linear partial differential equations of hyperbolic type. There are no analytical solutions for these equations and they are usually solved by numerical methods. Here in we explain the "Method of Characteristics" for solving these governing equations (Katika & Pilon 2005; Chou 2009). This concept helps one to understand propagation of waves and formulation of boundary conditions. Formulation of boundary conditions is very crucial if one is to correctly solve the governing equations.

3.4.1 Characteristic Equations

Multiplying Equation 3.21 by an arbitrary parameter λ, adding it to Equation 3.22 and rearranging the terms gives

$$\left[\frac{\partial V}{\partial t}+\left(V+\frac{\lambda A}{T}\right)\frac{\partial V}{\partial x}\right]+\lambda\left[\frac{\partial y}{\partial t}+\left(\frac{g}{\lambda}+V\right)\frac{\partial y}{\partial x}\right]=g\left(S_0-S_f\right) \tag{3.23}$$

The total derivatives for V and y can be written as

$$\frac{dV}{dt}=\frac{\partial V}{\partial t}+\frac{\partial V}{\partial x}\frac{dx}{dt} \tag{3.24}$$

and

$$\frac{dy}{dt}=\frac{\partial y}{\partial t}+\frac{\partial y}{\partial x}\frac{dx}{dt} \tag{3.25}$$

If we choose λ such that

$$V+\frac{\lambda A}{T}=\frac{g}{\lambda}+V=\frac{dx}{dt} \tag{3.26}$$

then Equation (3.23) becomes

$$\frac{dV}{dt} + \lambda \frac{dy}{dt} = g\left(S_0 - S_f\right) \tag{3.27}$$

and

$$\lambda = \pm\sqrt{\frac{gT}{A}} \tag{3.28}$$

Therefore

$$\frac{g}{\lambda} = \pm\sqrt{\frac{gA}{T}} \tag{3.29}$$

The cross-sectional area *A* normal to the flow for circular ducts is (Politano 2005)

$$A = \frac{D^2}{8}\left(\alpha - \sin\alpha\right)\ ;\ \cos\left(\frac{\alpha}{2}\right) = 1 - \frac{2y}{D} \quad \text{for} \quad y \le \frac{D}{2} \tag{3.30}$$

or

$$A = \frac{D^2}{4}\left[\pi - \frac{\alpha - \sin\alpha}{2}\right]\ ;\ \cos\left(\frac{\alpha}{2}\right) = \frac{2y}{D} - 1 \quad \text{for} \quad y \ge \frac{D}{2} \tag{3.31}$$

It can be seen that $\frac{g}{\lambda}$ represents the celerity i.e. the propagation rate of a disturbance and $\frac{dx}{dt}$ represents the absolute wave velocity. Thus

$$\frac{dV}{dt} + \frac{g}{C}\frac{dy}{dt} = g\left(S_0 - S_f\right) \tag{3.32}$$

is valid along the curve $\frac{dx}{dt} = V + C$ (3.33)

while

$$\frac{dV}{dt} - \frac{g}{C}\frac{dy}{dt} = g\left(S_0 - S_f\right) \tag{3.34}$$

is valid along the curve $\frac{dx}{dt} = V - C$ (3.35)

Equation 3.32 is called the Positive Characteristic Equation and Equation 3.34 is called the Negative Characteristic Equation. Plots of Equations 3.33 and 3.35 in the *x-t* plane are shown in Figure 9 in which line LP represents Equation 3.33 and is known as the Positive Characteristic Line, C^+. Equation 3.32 is valid along this line. It may be noted that the discretised form of original partial differential equations can be written between any pair of points arbitrarily taken in the *x-t* plane. On the other hand, Equation 3.32 which is simpler than the original P.D.Es can be written between any two points in the *x-t* plane, L and P, only when they satisfy the condition given by Equation 3.32. Points L and P indicate that a wave traveling in the downstream direction takes Δt time to travel a distance Δx_L. In a similar manner, Equation 3.35 plots as line RP, C^- in the *x-t* plane. This line is known is the negative characteristic line. Equation 3.34 is valid along the line RP. Points R and P indicate that a wave travelling in the upstream direction takes Δt time to travel a distance of Δx_R.

Equation 3.34 is integrated along the characteristic line LP as shown below:

$$\int_L^P dV + \int_L^P \frac{g}{c} dy = g\int_L^P \left(S_0 - S_f\right) dt \tag{3.36}$$

Note that the celerity *C* depends upon the flow cross-sectional area *A* and top width *T* both of which are functions of time and distance therefore *C* is not a constant along the line LP in the *x-t* plane. In a similar manner, S_f is also not a constant along LP.

However, for the sake of simplification, assuming that C and S_f are constant along the line LP, and are equal to values of C and S_f at point L, Equation 3.36 can be shown to give

$$V_P - V_L + \left(\frac{g}{C}\right)_L [y_P - y_L] = g\left[S_0 - \left(S_f\right)_L\right][t_P - t_L] \tag{3.37}$$

Similarly, Equation 3.34 can be integrated along the line RP. The result is

$$V_P - V_R - \left(\frac{g}{C}\right)_R [y_P - y_R] = g\left[S_0 - \left(S_f\right)_R\right][t_P - t_R] \tag{3.38}$$

In Equations 3.37 and 3.38, subscripts P, R and L refer to the values at the points P, R and L respectively. If the values V and y are known at points L and R, values of V and y at point P can be obtained by simultaneously solving Equations 3.37 and 3.38. By taking points L and R at the same time level i.e. $t_L = t_R$ and

$$t_P - t_L = t_P - t_R = \Delta t \tag{3.39}$$

$$\left(\frac{g}{C}\right)_L = C_L \text{ and } \left(\frac{g}{C}\right)_R = C_R \tag{3.40}$$

$$V_P + C_L y_P = V_L + C_L y_L + g\left(S_0 - S_{f_L}\right)\Delta t \tag{3.41}$$

$$V_P - C_R y_P = V_R + C_R y_R + g\left(S_0 - S_{f_R}\right)\Delta t \tag{3.42}$$

Right hand sides of Equations (3.41) and (3.42) can be evaluated using values at points R and L. Therefore, V_p and y_p can be obtained by solving Equations 3.41 and 3.42. Variables at points R and L can be obtained by linear interpolation. If we are at a boundary then one of the characteristic equations is outside of our problem boundary, and therefore only one characteristic equation is relevant. We therefore need a boundary condition that can be either the flow depth or velocity, or a relationship between the flow depth and velocity.

In our case at R we need a boundary condition that will be the open channel depth in this boundary. When this open channel depth at the boundary is bigger than the height of the pipe a surge is created i.e. a transition from pressurised to free-surface flow. Because we want information along the pipe to travel along the characteristic lines, we select the time interval Δt such that $\Delta t \leq \frac{\Delta x}{|v \pm c|}$. This is the Courant Stability condition for free surface flow (Wylie & Streeter 1978; Chadwick et al. 2004; Tullis 1989). Thus, the size of Δx and the wave celerity C will determine the size of our time interval.

To solve the pressurised flow, we make a similar process that we did with the free-surface flow. The Courant condition for the pressurised flow is $\Delta t < \frac{\Delta x}{v + a}$. The parameter *a* is much greater than c so this is the stability condition that is applied for the whole grid. The characteristic equations expressed in finite difference form are then programmed for automatic evaluation on a digital computer (Chadwick et al. 2004; Marriott 2009).

3.5 Modelling the Transition Region

In the transition region of two different flows (Figure 12), there are no valid characteristic equations that cross the interface trajectory. In this region the equations that should be applied are the conservation of mass and momentum quantities and the characteristic equation for pressurised flow that does not cross the interface trajectory.

In the case of a surge that advances from upstream to downstream; the free-surface side of the interface can be solved separately to find the velocity and the pressure at P1, by using the linear equations C1- and C1+.

In order to solve the pressurised side of the interface the following equations can be applied:

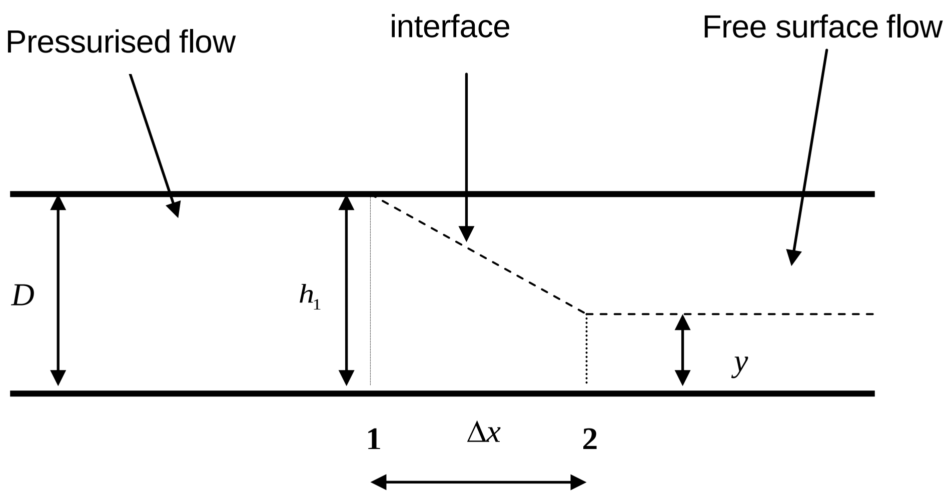

Figure 12: Control volume for the interface

a) Continuity Equation

$$A_2(v_2 - w) = A_1(v_1 - w) \quad (3.43)$$

where *w* is the velocity of the interface and subscripts 2 and 1 are locations at the pressurised and free-surface zones respectively.

b) Conservation Momentum Quantities

Momentum Equation $F = ma$

where

F is the net force causing acceleration, m is the mass of the fluid and a is the acceleration but $F = h\rho gA$ where h is the height/depth of fluid; ρ is the fluid density; g is gravitational acceleration and A is cross-sectional area of fluid. Therefore

$$F = \frac{mv_2 - mv_1}{t} = \frac{m}{t}(v_2 - v_1) = \rho\frac{Vol}{t}(v_2 - v_1) = \rho Q(v_2 - v_1) = \rho A_1 v_1 (v_2 - v_1) \quad (3.44)$$

where $Q = A_1 v_1$ i.e. mass is constant at $A_1 v_1$. The velocity at point 2, velocity at point 1 and the time between these velocities are represented as v_2, v_1 and t respectively. Vol is volume and Q is discharge. This implies

$$(\bar{y}_2 A_2 - \bar{y}_1 A_1)\rho g = \rho A_1 v_1 (v_2 - v_1) \quad (3.45)$$

in which ρ cancels out giving $\bar{y}_2 A_2 - \bar{y}_1 A_1 = (V_1 - w)\frac{A_1}{g}(v_1 - v_2)$ (3.46)

after incorporating the interface velocity. $\bar{y}$ is the depth from the water surface to the centre of gravity of the flow cross-sectional area.

Substitution into Equation 3.41 and 3.42 gives

$$V_2 + \left(\frac{g}{a}\right)_L y_2 = V_{L2} + \left(\frac{g}{a}\right)_L y_L + g\left(S_0 - S_{f_L}\right)\Delta t \quad (3.47)$$

or

$$V_2 - V_{L2} + \left(\frac{g}{a}\right)_L (y_2 - y_{L2}) = g\left(S_0 - S_{f_{L2}}\right)\Delta t \quad (3.48)$$

From Equation 3.43, $A_2 = \frac{A_1(V_1 - w)}{V_2 - w}$ is substituted into Equation 3.46 to give $\bar{y}_2 \frac{A_1(V_1 - w)}{V_2 - w} - \bar{y}_1 A_1 = (V_1 - w)\frac{A_1}{g}(V_1 - V_2)$ which is multiplied all through by $V_2 - w$ to give $\bar{y}_2 (V_1 - w) - \bar{y}_1 (V_2 - w) = (V_2 - w)(V_1 - w)\frac{(V_1 - V_2)}{g}$ or $\bar{y}_2 g(V_1 - w) - \bar{y}_1 g(V_2 - w) = (V_2 - w)(V_1 - w)(V_1 - V_2)$. This is expanded, rearranged and added to Equation 3.48 itself multiplied by $\frac{A_1^2}{A_2\left(A_2 - A_1\right)}$ to give a quadratic equation for the interface velocity as suggested Gomez and Achiaga (2008): $w^2 + Bw + C = 0$ where $B = -2v_2 + \frac{A_1}{A_2}a$ and

$$C = v_2^2 - \frac{A_1}{A_2 - A_1}\left(\bar{y}_2 g + v_2 a\right) + \frac{A_1^2}{A_2\left(A_2 - A_1\right)}\left[v_{l1} a + \left(y_1 - \bar{y}_1\right)g - a\Delta t.g\left(S_{fL1} - S_0\right)\right] \quad (3.49)$$

which gives $w = \frac{-B + \sqrt{B^2 - 4C}}{2}$ (3.50)

From the quadratic formula given in Equation 3.50, only the positive root makes physical sense, and thus, the model calculates the velocity at point 1 with the continuity equation as:

$$v_1 = w + \frac{A_2}{A_1}\left(v_2 - w\right) \quad (3.51)$$

and the pressure on the same point 1 from the pressurised positive characteristic equation as

$$y_1 = y_{L1} + a\left(\frac{v_1 - v_{L1}}{g} - \Delta t\left(S_0 - S_{fL1}\right)\right) \quad (3.52)$$

The new interface position is found from the Kinematic Condition according to:

$$\Delta x' = w.\Delta t \quad (3.53)$$

The following equations govern flow across the free surface:

$$y_p = \frac{1}{C_R + C_S}\left[y_R c_s + y_s c_R \left(\frac{v_R - v_s}{g} - \Delta t.\left(S_{fR} - S_{fS}\right)\right)\right] \quad (3.54)$$

$$v_p = v_R - \frac{g}{c_R}\left(y_p - y_R\right) + g.\Delta t.\left(S_o - S_{fR}\right) \quad (3.55)$$

The Conservation of Mass equation for the surge is:

$$\frac{d\left(A_1 - A_2\right)}{dt}\Delta x = A_1 V_1 - A_2 V_2 \quad (3.56)$$

The conservation of momentum equation is:

$$0 = A_2 V_2^2 - A_1 V_1^2 + g\left[\overline{y_2} A_2 - \left(h_1 - z_1 - \frac{D}{2}\right)A_1 + \Delta x\left(\frac{A_2 + A_1}{2}\right)S_o\right] \quad (3.57)$$

$$\text{or} \quad g\left(A_2 \overline{y_2} - A_1 h_1\right) = A_1\left(V_1 - w\right)\left(V_1 - V_2\right) \quad (3.58)$$

Referring to Equation 3.57 in the case of a pressurised pipeline, slope S_o is irrelevant and the term dies out. Also z_1 was assumed to be zero. Equation 3.57 then becomes

$$g\left(\overline{y_2} A_2 - A_1 h_1 + A_1 \frac{D}{2}\right) = A_1 V_1^2 - A_2 V_2^2 \quad (3.59)$$

Also, if the term on the left hand side of Equation 3.58 is modified to include $A_1 \frac{D}{2}$, then Equation. 3.58 can be re-written as

$$g\left(A_2 \overline{y_2} - A_1 h_1 + A_1 \frac{D}{2} \right) = A_1 (V_1 - w)(V_1 - V_2) \tag{3.60}$$

Equations 3.46 and 3.60 would then be used as the mass and momentum conservation equations during the solution of the interface characteristics.

It is also true that $\frac{dx}{dt} = w$ (3.61)

3.6 Programming

The aforementioned system of equations developed was programmed in MATLAB (Mathworks 2007; Hahn & Valentine 2006). The model equations were discretised using a fixed-grid method with a first-order finite difference approximation. The resulting nonlinear equations were solved using the Newton-Raphson Method. The time step was determined by the Courant's criteria.

4.0 METHODOLOGY-2: ANALYSIS AND ACTIVITIES LEADING TO SATISFYING THE SPECIFIC OBJECTIVES

This Chapter explains the methods and procedures that were undertaken in order to carry out this research and obtain results. These include characterisation of the Kampala Water Supply Network (KWSN), development of a model of the Kampala Water network, development of codes for pipeline filling and pressurisation and development of a decision support system using both hypothetical and actual field data.

4.1 Characterisation of the Kampala Water Supply Network

The first specific objective entailed collection of data from the National Water and Sewerage Corporation (NWSC) on the Kampala Water Supply Network (KWSN). The data included water produced and supplied, pipeline layout, pipe sizes and elevations, pipe lengths and materials, valves, reservoirs, pumps, consumption patterns and pressures, heads and flows at strategic sections, well supplied zones and poorly supplied zones (Figure 13).

Figure 13: Pipe Detection exercise using the Mala Easy Locater

4.2 Development of Network Decision Support System

A decision support system herein called the Operations and Management Decision Support Tool (OMDST) was developed from several models mentioned below, depending on the status of pressures in the network.

i. EPANET2 model of water distribution network under the traditional demand driven modelling approach
ii. Pressure driven demand model for low pressure and demand analysis
iii. Partially full pipes undergoing filling/unfilling and pressurisation/depressurisation.

The OMDST represented as a flow chart in Figure 5 and further described in Section 1.4.3, guided execution of different tasks in accordance with the EPANET2 model outcomes of different scenarios. The different models are further described in sections 4.3, 4.5 and 4.6.

4.3 Model of the Kampala Water Supply Network

The second specific objective entailed building a model of the Kampala Water Supply Network using the network data obtained under Section 4.1. The model was constructed using the EPANET2 hydraulic solver under the demand driven approach. The modelling process involved the following steps: input data collection, network schematisation, model building, testing and problem analysis.

The model was built in steps, gradually increasing the level of detail in order to ensure that all errors could be corrected before the model was too complicated and thereby avoid problems during the model testing procedure. The primary network through which the main reservoir supplies the subsystem reservoirs was modelled first followed by the secondary network supplied by the subsystem reservoirs.

4.3.1 Input Data Collection

4.3.1.1 General

The requirements needed to build the model included the following data:

i. Layout of the system i.e. pipeline routes and junctions and location of the main components.
ii. Topography i.e. ground elevations in the area of the system
iii. Type of system i.e. distribution scheme, gravity, pumping or combined.

4.3.1.2 Tanks

The primary input properties for tanks are bottom elevation; diameter; initial, minimum and maximum water levels. The principal output computed over time is the hydraulic head (water surface elevation).

4.3.1.3 Pipes

The principal hydraulic input parameters for pipes are start and end nodes, diameters, lengths and roughness coefficients for determining head loss. Computed outputs for pipes included flow rate, velocity and head loss. The hydraulic head lost by water flowing in a pipe due to friction with the pipe walls was computed using the Darcy-Weisbach formula (Equation 4.1), this being the most theoretically correct and considering that it applies over all flow regimes (Rossman 2000).

$$h_f = \lambda \frac{l}{d} \frac{v^2}{2g} \tag{4.1}$$

where $h_f =$ pressure loss (N/m^2); λ = Darcy-Weisbach friction coefficient; *l* = length of duct or pipe (m); *d* = hydraulic diameter *(m)*; and *v*=velocity (m/s).

4.3.1.4 System Operation and Monitoring

Figure 14: Pressure Measurement

Testing and calibration of the model of the existing system to field observed values was done for a range of operating conditions in order to evaluate the model's ability to represent actual situations. Important measurements for calibration and testing of the model included the pressure at a number of points in the network and in pumping stations (Figures 14 and 15) and the field data of residual pressures was compared with model outputs. More data collected included level variations in tanks and flows in a few main pipes in the network.

Figure 15: Flow Meters installed at a service point for monitoring purposes

However, in considering the network under this study, most of this information was not directly accessible from the on-line monitoring of the system. A large part of this information was either missing or incomplete and the only real sources were the operator in the field as well as actual measurements. In the absence of all the required measuring equipment some data was obtained in descriptive form.

4.3.2 Water Demand

Establishment of nodal demands is key to the model building process. The demand categories that were considered were domestic, industrial and institutional. A survey of numerous users spread all over the network was carried out and using an average household occupancy of five and a daily per capita water consumption of 40 litres, average demand values for different areas were computed. Assuming an even distribution of consumers so that the border between the supply areas of two nodes connected by a pipe is at half the interconnecting pipe length, base demand values were allocated to corresponding pipe nodes in order to make the network presentation suitable for a computer model. Because demand varies at different times of the day, multipliers were used in order to reflect the actual diurnal demand variation at the nodes.

4.3.3 Network Schematisation

Network schematisation was done up to a level where model accuracy would not be substantially affected. This was checked by comparing model results with field data (Section 5.1). The network was schematised by merging pipes of diameter less that 100 mm, concentrating demand at certain nodes, ignoring very small pipes or those very far from the main pipes as well as introduction of equivalent pipe diameters.

4.3.4 Model Testing

Trial solutions were run and their performance reviewed. With adjusted network inputs and parameters, the iterative process was repeated until a technically feasible solution was reached. While the model was built, local losses resulting from added turbulence that occurs at bends and fittings were neglected since these are very small as compared to the network and considering that no turbulence is expected due to low velocities. Only friction losses were considered when calculating head losses. In addition, no losses due to leakage were assumed to occur in the network except losses implicitly included in the fixed nodal consumptions.

4.3.5 Generation of Model Outputs

Principal model outputs that were desired for this research were flows, velocities and pressures under different scenarios. It should be noted that pressures at nodes were assumed to be the pressures at consumption points for simplification purposes. This was due to lack of relevant data arising from the impracticability of modelling the network as far as the consumer taps (the network was schematised in order to facilitate modelling). Besides, nodal pressures were the subject of study in the research.

4.4 Problem Analysis

Two broad scenarios were considered as follows:

i. Normal operating systems in which pressures are sufficient to meet the demand throughout the network. This scenario involved monitoring the response and performance of the model during ideal flow conditions i.e. periods and sections when pressures are sufficient, in order to find out system behaviour and pressures, heads and flows at various sections. This provided a control and benchmark to the subsequent scenarios.

ii. A constrained system which was created by imposing excessive demand loadings, insufficient supply and inadequate pipe sizes.

The theoretical case involved scenarios for demonstration and insightful purposes. The field cases involved comparison of real data on inputs and outputs from networks in the two scenarios with the model performance. Observation of the behaviour and performance of the system under low and no pressures as well as the perceived transition to partially full flow and the corresponding effects on water supplied was done. Examination of the model for critical sections in the system (low and no pressure) was done. In all cases the decision support system was followed as guided by the inputs from the model.

4.5 Pressure Dependent Demand

Having carried out demand driven analysis (DDA), nodes at which pressures were insufficient to fully supply their demands were identified. As already discussed, since demands are fixed under DDA while pressures vary, a nodal pressure value was considered insufficient if it was less than the pressure threshold, a situation that would result in less water supply than is required. The threshold value for each node can be approximated by the expected maximum outlet level in the locality served by that node represented by the height of the tallest building that can be agreeably supplied by the service provider without extra pumping, usually a value of 10 m. When lower nodal pressures are obtained then only a fraction of the original demand is met. There is then the need to determine the available flows at the identified pressure deficient nodes using the method summarised below (Ozger 2003; Mays 2004).

i. New node elevation = Original node elevation + Threshold pressure head
ii. Set demand to zero
iii. Connect an artificial reservoir to the node by an infinitesimally short pipe that allows flow only from the node to the reservoir
iv. Artificial tank elevation = New node elevation

With these modifications demand at each pressure-deficient function was treated as an unknown while a pressure threshold is imposed. If one or more artificial reservoirs received more water than their nodes demanded, those artificial reservoirs were removed from the network and the original elevations and demand at the corresponding nodes restored. This procedure was carried out in an iterative manner until acceptable demand values that corresponded to the available pressures were obtained.

4.6 Numerical Simulation of Free Surface – Pressurised Flow in Pipelines

The fourth objective was achieved by analysing and modelling transient pressures and partially-full pipes so as to locate the position of problem flow in the network at any time by simulating the “charging and discharging” process in pipes. This involved formulation of mathematical and numerical models as detailed in Chapter 3. Codes for pipe filling and pressurisation were programmed in MATLAB.

The simulation code for pipe filling involved the following inputs: pipe diameter, pipe length, pipe slope, Manning’s Coefficient and flow depths and discharges at two key points in the programme. Values of flow depths and discharges at two critical points along the pipeline were initially determined from boundary conditions and field values.

The simulation for pipeline pressurisation involved the following inputs: pipe length, pipe diameter, flow depth, pressure, velocity and friction slope. Flow depth and pressure values were obtained from initial boundary conditions and were measured from the pipe invert. The velocity was obtained from initial conditions and later confirmed from field values.

The MATLAB simulation was used locally i.e. at only affected sections of the network and not globally, so that inputs to the code were easily available. After pressure deficient pipes in the network as simulated in EPANET 2 were identified under different scenarios, they were isolated using valves inserted automatically onto the pipelines and this made the MATLAB codes functional.

5.0 ANALYSIS AND PRESENTATION OF RESULTS

This Chapter analyses and presents results obtained in this research. Valuable outcomes of the different models developed and used together with their responses to various practical scenarios are herein shown. The major models that are fundamental to this Chapter are the WATER DISTRIBUTION model, PIPE FILLING and PRESSURISATION models.

5.1 Analysis of Kampala Water Supply Network

Water supplied in Kampala City comes from the Gaba Water Works (Figure 16) on the shores of Lake Victoria which currently produces 170,000 m^3 of water per day and supplies it through two main reservoirs i.e 4000 m^3 capacity reservoir located at Gunhill, Nakasero and five 4000 m^3 capacity reservoirs located on

Figure 16: Gaba III water treatment plant commissioned on April 19, 2007

Muyenga hill from where the water is distributed to the rest of the City through an additional three 4000 m^3 capacity reservoirs (Figure 17). Every reservoir forms a subsystem in the network. The total pipe network length in Kampala is estimated to be 3,000 km and serves a population of one million people. A schematised network of the Kampala Water Supply system showing node elevations and pipe diameters is presented in Figure 19. In order to facilitate quick visualisation and comparison of elevations at various nodes in the network, a plot of selected nodal elevations has been made in Figure 18, observed from the left side to the right of Figure 19, through nodes with IDs 24,23,16,17,27,5 (Rubaga reservoir),1 (Muyenga reservoir), 2 (Mutungo reservoir) and 8, as identified in Figure 20. The highest node in the graph is the Muyenga reservoir.

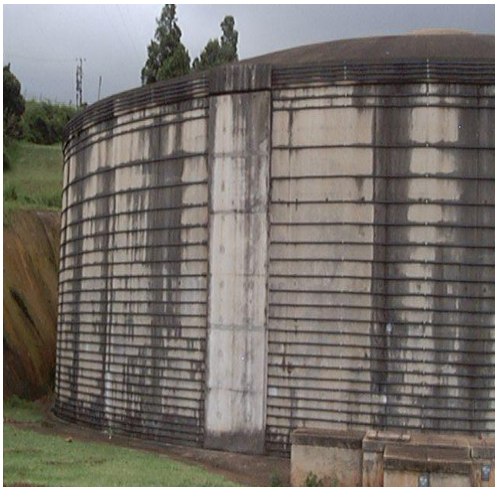

Figure 17: Some Reservoirs
Left: One of Muyenga 4000 m^3 Reservoirs
Right: Naguru 4000 m^3 Reservoir

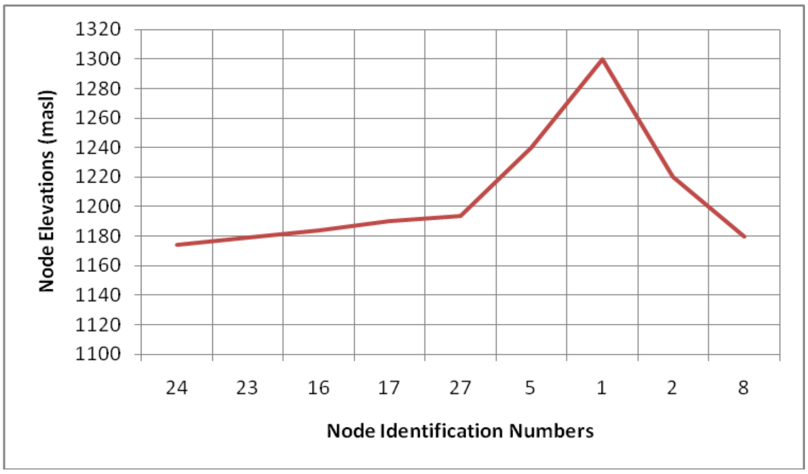

Figure 18: Variation of Selected Network Node Elevations

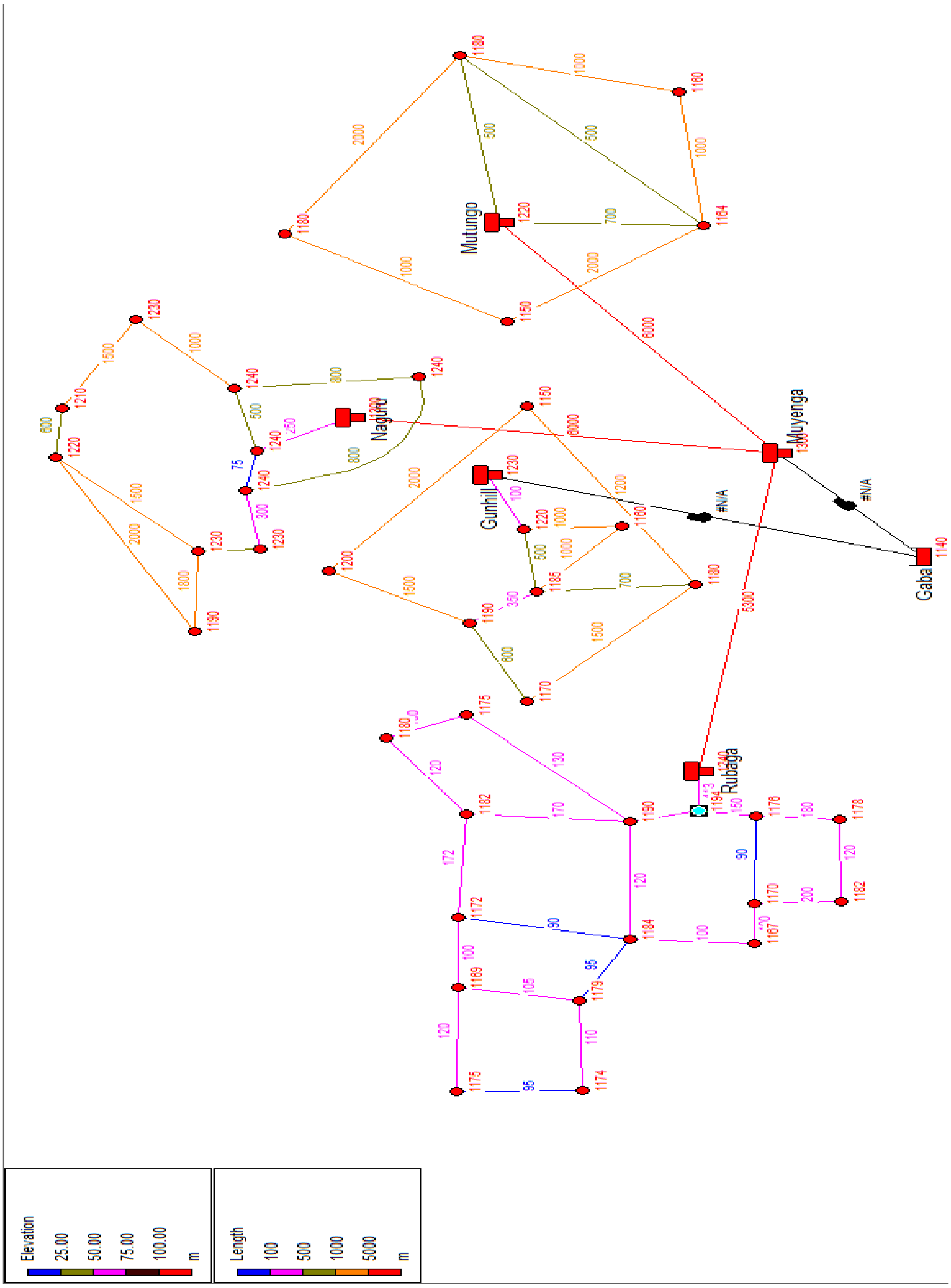

Figure 19: Schematised Pipe Network of Kampala Water Supply System

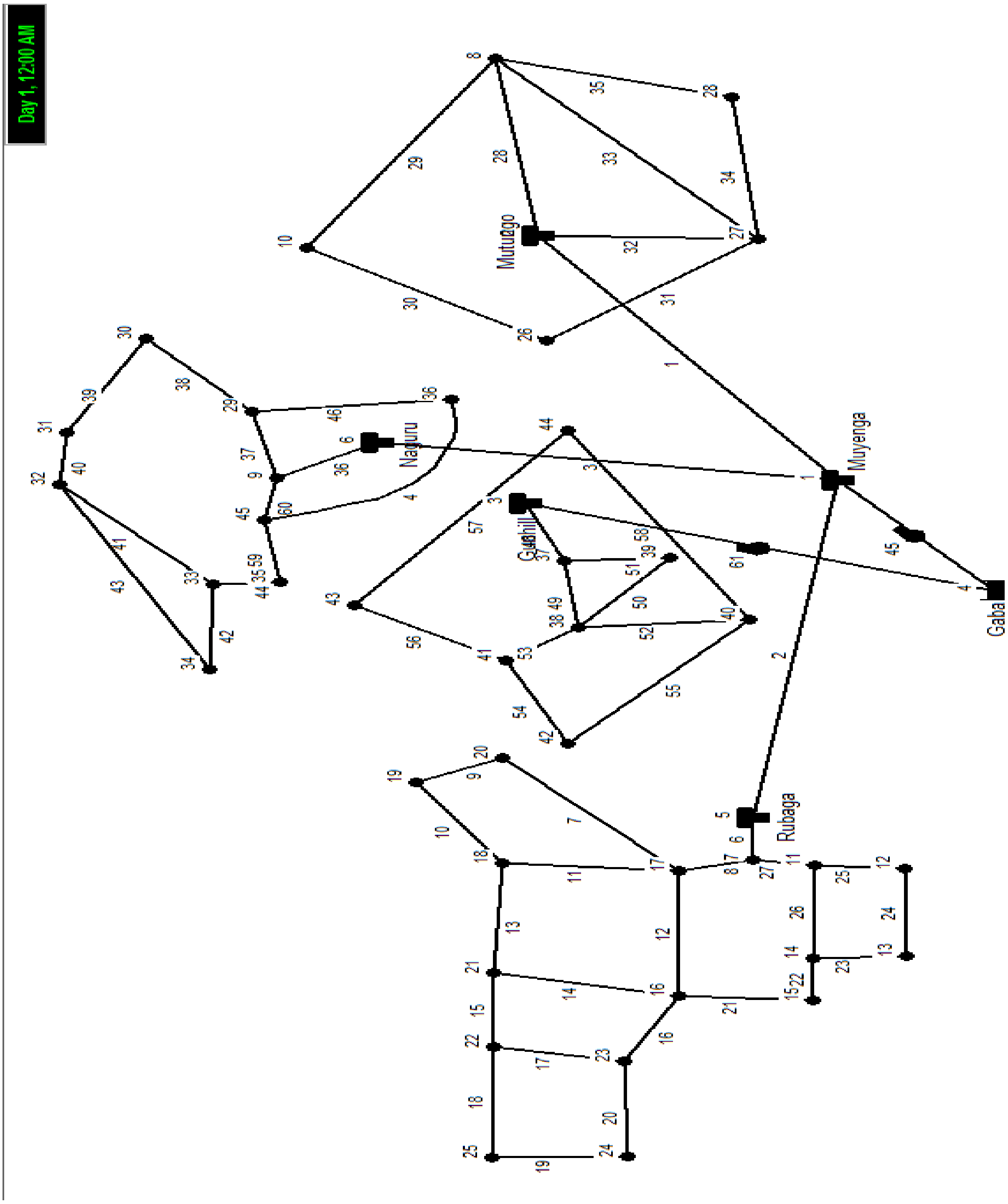

Figure 20: Network Map Showing Node and Link IDs

5.1.1 Calibration of Model

Table 1: Pressure Values

Time (hours)	Model Pressure Values (m)		Field Values (m)
	Node 25	Node 24	
00:00	18.82	20.82	19.04
01:00	18.82	20.82	18.95
02:00	18.82	20.82	18.74
03:00	18.82	20.82	18.87
04:00	18.58	20.58	18.90
05:00	17.65	19.65	16.93
06:00	17.96	19.96	17.62
07:00	18.36	20.36	17.96
08:00	18.58	20.58	18.85
09:00	18.88	20.88	19.23
10:00	18.88	20.88	19.37
11:00	18.88	20.88	19.14
12:00	18.88	20.88	19.08
13:00	18.36	20.36	18.66
14:00	17.65	19.65	17.55
15:00	16.69	18.69	16.45
16:00	15.51	17.51	15.56
17:00	16.69	18.69	16.54
18:00	18.58	20.58	18.45
19:00	18.58	20.58	18.83
20:00	18.58	20.58	18.94
21:00	18.58	20.58	19.24
22:00	18.58	20.58	19.64
23:00	18.58	20.58	20.42
24:00:00	18.82	20.82	21.74

Calibration of the model, which entails adjustment of model inputs and parameters in order to obtain realistic field outputs, was done by adjusting water demand values until pressure values obtained from the model were close to field values. A comparison of model and field values was done using field results from a pressure test carried out at node 25 (Figure 20) at Bulenga on a 150 mm diameter main at the boundary of the Rubaga subsystem (Table 1). It revealed pressures of approximately 20 metres at the mains for most of the time (Figure 21) which depicts a low delivery capacity of the network.

Figure 21 also shows that the lowest pressure values occur at 05 00 hours and 16 00 hours when water consumption goes up because people wake up to prepare for work at 05 00 hours and probably resume domestic water consuming chores at 16 00 hours.

Highest pressure values occur at 10 00 hours and at midnight when there is hardly any domestic activities being done and also start to rise at 18 00 hours when there is little activity in the homes until midnight when water consumption is least because people are asleep.

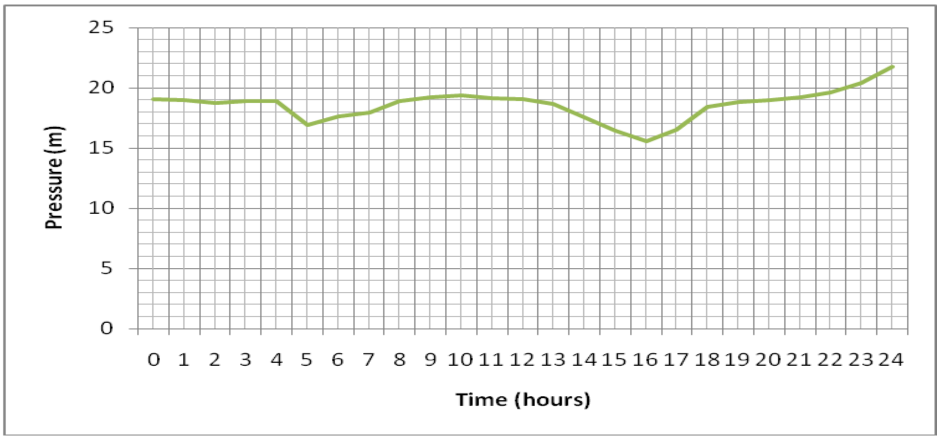

Figure 21: Pressure Test Done at Bulenga Trading Centre

After model calibration and adjustment of demand loadings in order to reflect the reality on the ground, a comparison between field pressure values and model pressure values for Node 25 is illustrated in Figure 22. It can be seen that there is good agreement between model values and field values.

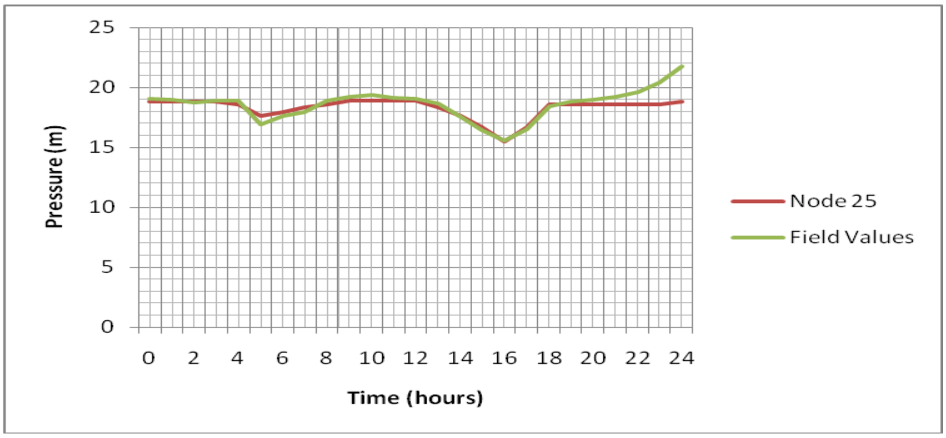

Figure 22: Comparison between field and model pressure outputs

5.1.2 Pressure and Demand Variation

Figure 23 shows the diurnal pressure variation at two selected nodes; 24 and 25 (Figure 20) in the Rubaga subsystem. The simulation revealed pressures of approximately 20 metres at the mains for most of the time (Figure 21) which depicts a low delivery capacity of the network. The nodes were selected because they were most critical since they formed take-off points for any major extensions.

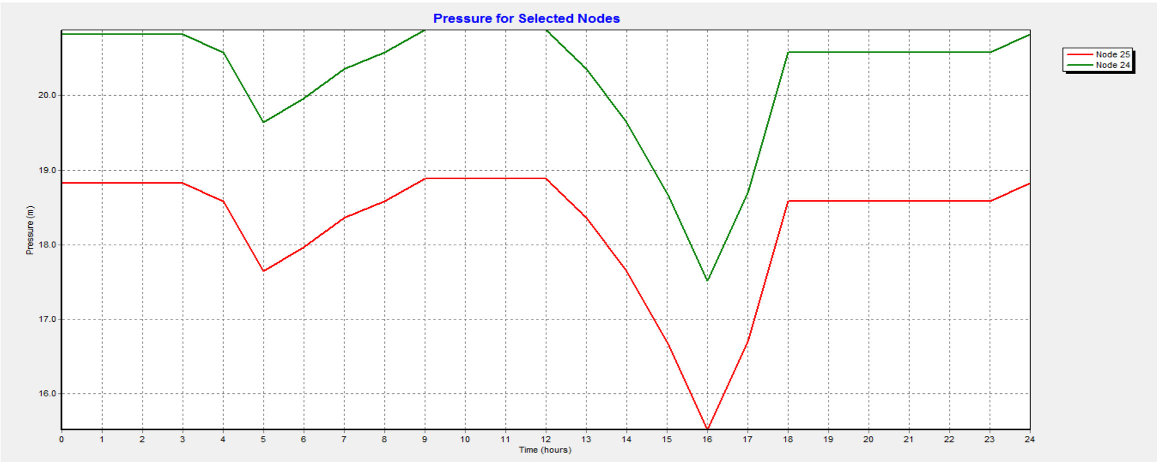

Figure 23: Diurnal Pressure Variations for Nodes

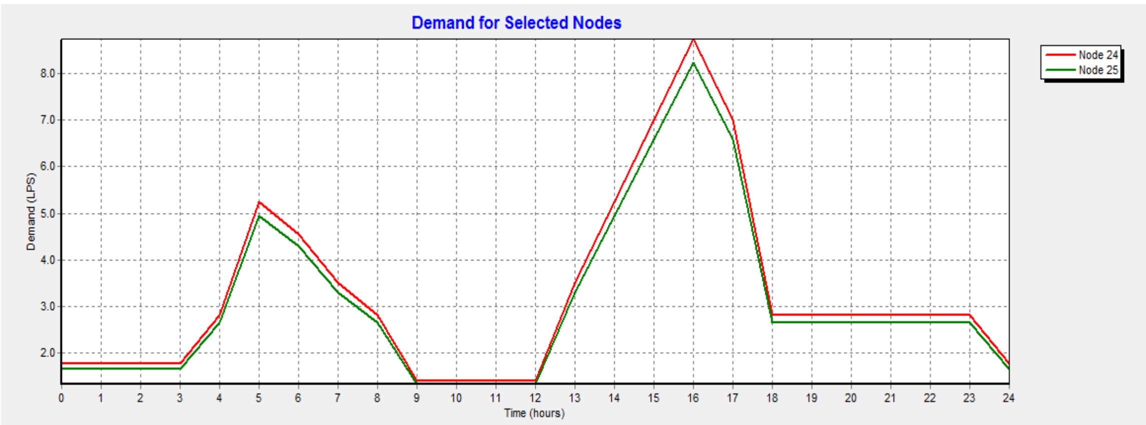

Figure 24: Diurnal Demand Variation for Selected Nodes

The plot in Figure 24 shows the diurnal demand variation at selected nodes also given in Table 2. These nodes are critical because they lie at boundaries and could form joints for further extensions. Their hydraulic characteristics are therefore worth studying. It can be observed that the lowest demand is at midnight whereas peak demand is at 16 00 hours.

Table 2: Demand Values

Time (hours)	**Demand (l/s)**	
	Node 24	**Node 25**
00:00	1.75	1.65
01:00	1.75	1.65
02:00	1.75	1.65
03:00	1.75	1.65
04:00	2.80	2.64
05:00	5.25	4.95
06:00	4.55	4.29
07:00	3.50	3.30
08:00	2.80	2.64
09:00	1.40	1.32
10:00	1.40	1.32
11:00	1.40	1.32
12:00	1.40	1.32
13:00	3.50	3.30
14:00	5.25	4.95
15:00	7.00	6.60
16:00	8.75	8.25
17:00	7.00	6.60
18:00	2.80	2.64
19:00	2.80	2.64
20:00	2.80	2.64
21:00	2.80	2.64
22:00	2.80	2.64
23:00	2.80	2.64
24:00:00	1.75	1.65

5.1.3 Velocity Variation

Table 3: Model Velocity Values

Time (Hours)	Velocity (m/s)	
	Link 20	Link 16
00:00	0.10	0.21
01:00	0.10	0.21
02:00	0.10	0.21
03:00	0.10	0.21
04:00	0.16	0.34
05:00	0.30	0.63
06:00	0.26	0.55
07:00	0.20	0.42
08:00	0.16	0.34
09:00	0.08	0.17
10:00	0.08	0.17
11:00	0.08	0.17
12:00	0.08	0.17
13:00	0.20	0.42
14:00	0.30	0.63
15:00	0.40	0.84
16:00	0.49	1.05
17:00	0.40	0.84
18:00	0.16	0.34
19:00	0.16	0.34
20:00	0.16	0.34
21:00	0.16	0.34
22:00	0.16	0.34
23:00	0.16	0.34
24:00:00	0.10	0.21

Figure 25 and Table 3 show velocity values for links 20 and 16 as indicated in Figure 20. These links are critical because they lie at boundaries and could form take off links for further extensions. Their hydraulic characteristics are therefore worth studying. The velocities are generally very low and just above the minimum requirement as illustrated.

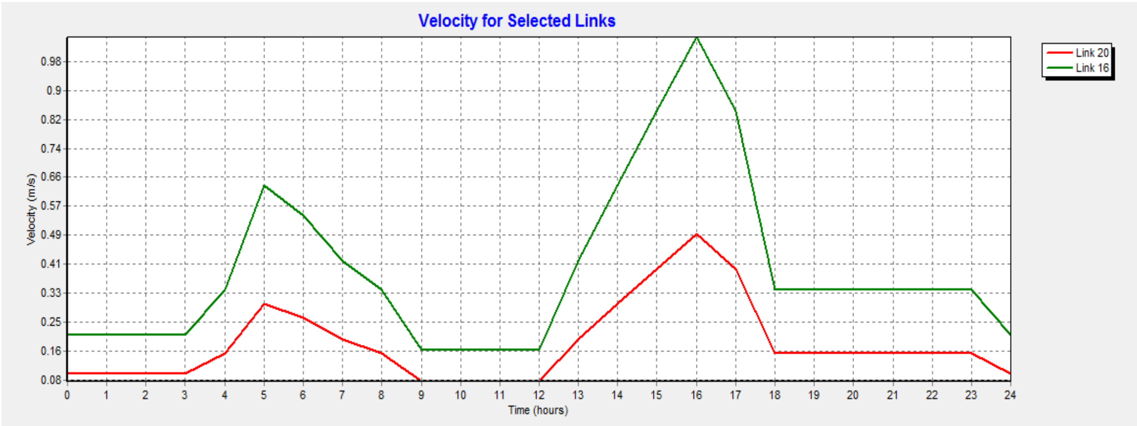

Figure 25: Velocity Variation for Selected Links

Velocities determine discharge and low velocities lead to low discharges. The velocities are low due to the fact that, generally, pressures in the network are low as can be evidenced in Figure 23. Velocities are highest at 05 00 hours and 16 00 hours which (Figure 25) are periods when lowest pressures occur, as observed in Figure 23 and explained in Section 5.1.1. Low velocities are also not desirable as they do not facilitate efficient flushing of the network making it unhygienic.

5.1.4 Effect of Pipe Size on Pressure, Head losses and Velocity

Table 4 shows the impact of pipe size on pressure, headloss and velocity. Figure 26 shows variation of pressure at node 7 (Figure 20) with size of a connecting pipe when satisfying a demand of 6.8 l/s at the node at a randomly selected time of 17 00 hours. The node connects the Rubaga tank to the Rubaga subsystem. It was interesting to observe that in this particular case a diameter of less or equal to 200 mm was unable to serve the subsystem because it yielded negative pressures. However use of diameters between 300 mm and 900 mm yielded higher pressures for a higher pipe size (Figure 26) in a relationship that can be estimated from Figure 26 in Equation 5.1 in which *y* is node pressure and *x* is connecting pipe diameter. This shows that the bigger the pipe size, the lower the headlosses which consequently yields higher pressures.

$$y = 8.133e^{-0.030x} \qquad (5.1)$$

However for diameters above 900 mm, the pressure remained the same due to the fact that headlosses reduce to a negligible value when the pipe diameter is sufficiently big. At this stage, friction between moving water and pipe walls is greatly reduced because water ceases to exert additional pressure forces against the pipe walls when the pipe cross sectional area is very big compared to the amount of water entering the pipe.

Table 4: Nodal Pressures, Link Diameters, Velocities and Headlosses

Pipe Size (mm)	Pressure (m)	Unit Head loss (m/km)	Velocity (m/s)
200	-25.1	87.42	3.45
300	5.99	12.13	1.53
350	8.64	5.72	1.13
400	9.77	2.99	0.86
450	10.3	1.68	0.68
500	10.58	1.01	0.55
550	10.74	0.63	0.46
600	10.83	0.41	0.38
700	10.92	0.2	0.28
800	10.96	0.1	0.22
900	10.98	0.06	0.17
1000	10.99	0.03	0.14
1100	10.99	0.02	0.11
1200	10.99	0.01	0.11

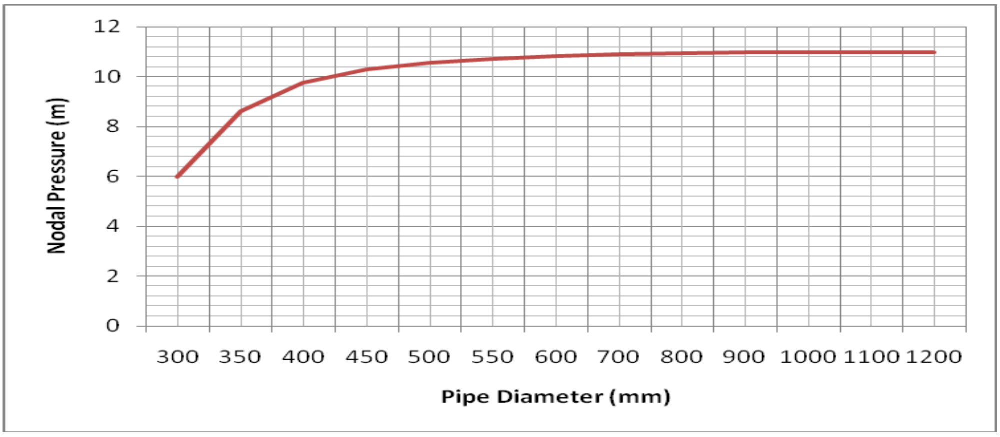

Figure 26: Variation of Nodal Pressure with Link Diameter

Figure 27 and Table 4 show the variation of headlosses with pipe diameter. A very small pipe size yields high headloss which reduce nodal pressures. The bigger the pipe size the smaller the headlosses. The relationship between pipe size and headlosses can be estimated from Figure 27 in Equation 5.2 where *y* is unit headloss and *x* is pipe diameter.

$$y = 19.02e^{-0.57x} \quad (5.2)$$

Figure 28 and Table 4 show the variation of pipe size with water velocity. The higher the pipe size the lower the flow velocity in line with the principle of mass conservation in a relationship that is estimated from Figure 28 to follow Equation 5.3 where *y* is water velocity and *x* is pipe size.

$$y = 1.756e^{-0.22x} \quad (5.3)$$

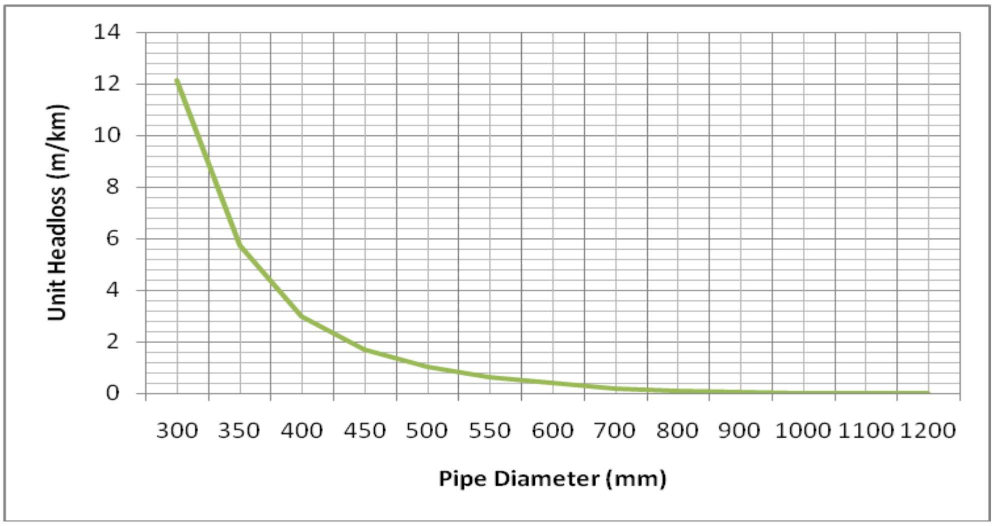

Figure 27: Variation of Pipe Size with Headlosses

Figure 29 reveals how high pressures result from low link head losses as is logically expected and can be predicted through an exponential relationship represented from Figure 29 by Equation 5.4 in which *y* is unit headloss and *x* is pressure.

$$y = 19.02e^{-0.57x} \quad (5.4)$$

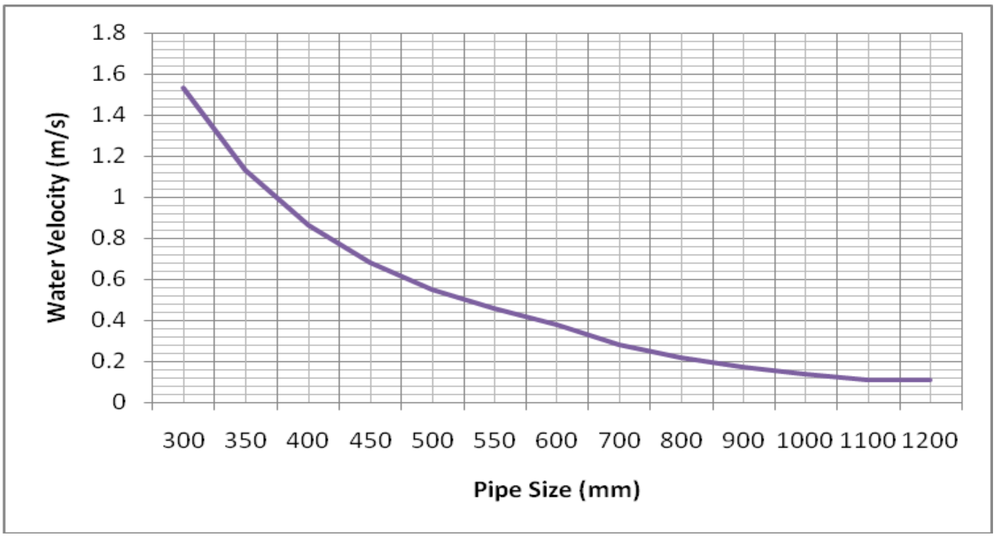

Figure 28: Variation of Pipe Size with Velocity

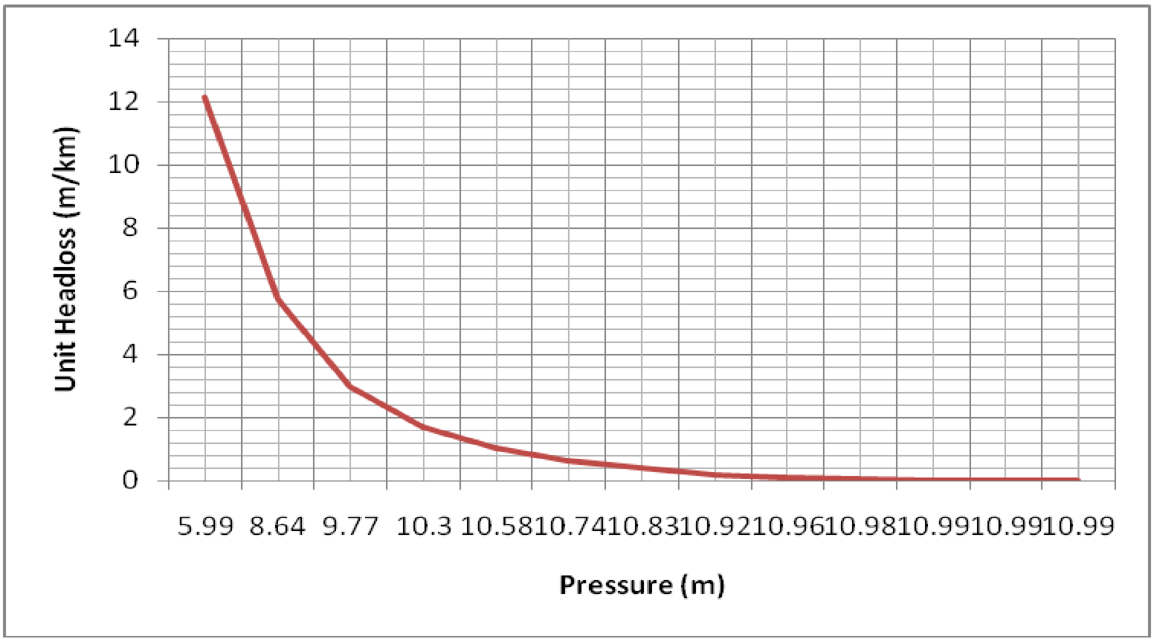

Figure 29: Variation of Nodal Pressure with Connecting Link Headloss

5.2 Analysis of Alternative Modelling Approaches

5.2.1 Demand Driven Analysis: Pressure Response to Demand

In this section we looked at the response of nodal pressures to changing demand. Plots of variations of demand with pressure at node 24 were done at midnight (Table 5 and Figure 30) when demand is lowest and at 16 00 hours (Table 6 and Figure 31) at peak demand. Demands were shown to be met at different pressures in a relationship of inverse proportionality whereby the higher the demands the lower the pressures at which the demands are fulfilled. When water abstracted from the system is a determinant of system performance, then more water withdrawn from the system leaves less water in the system and this directly lowers the pressure in the system.

It can be seen that for demand variation at midnight, pressures were much higher and varied gently because the demand was very low (Figure 30) and redundant demand in the system was very high so that changes in demand did not significantly affect the pressures at which the demand should be met. However at 16 00 hours, pressures were much lower and had a steeper slope. At 29 l/s, while the pressure value at node 24 was still high at midnight, negative pressures were exhibited at 16 00 hours. This showed that the system at this point was malfunctioning since not all nodes could supply water.

Table 5: Pressure outputs for given demand values at midnight

Demand (l/s)	Pressure (m)
1.75	20.82
2.00	20.81
2.25	20.80
2.50	20.79
2.75	20.78
3.00	20.77
3.25	20.76
3.50	20.75
3.75	20.74
4.00	20.73
4.25	20.71
4.50	20.70
4.75	20.69
5.00	20.67
5.25	20.66
5.50	20.65
5.75	20.63

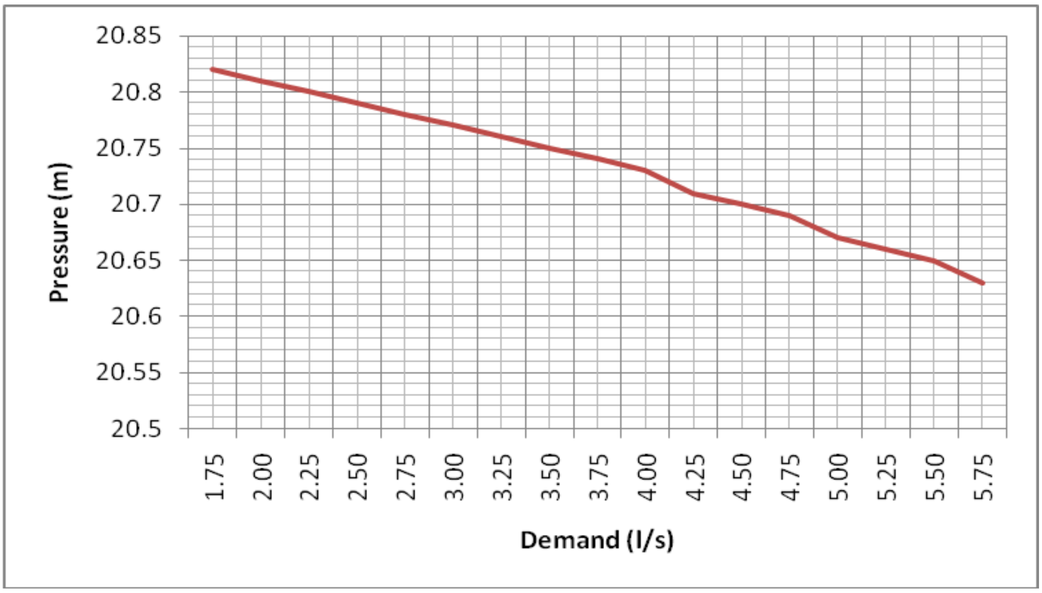

Figure 30: Plot of demand vs pressure at node 24 at 12 00 AM

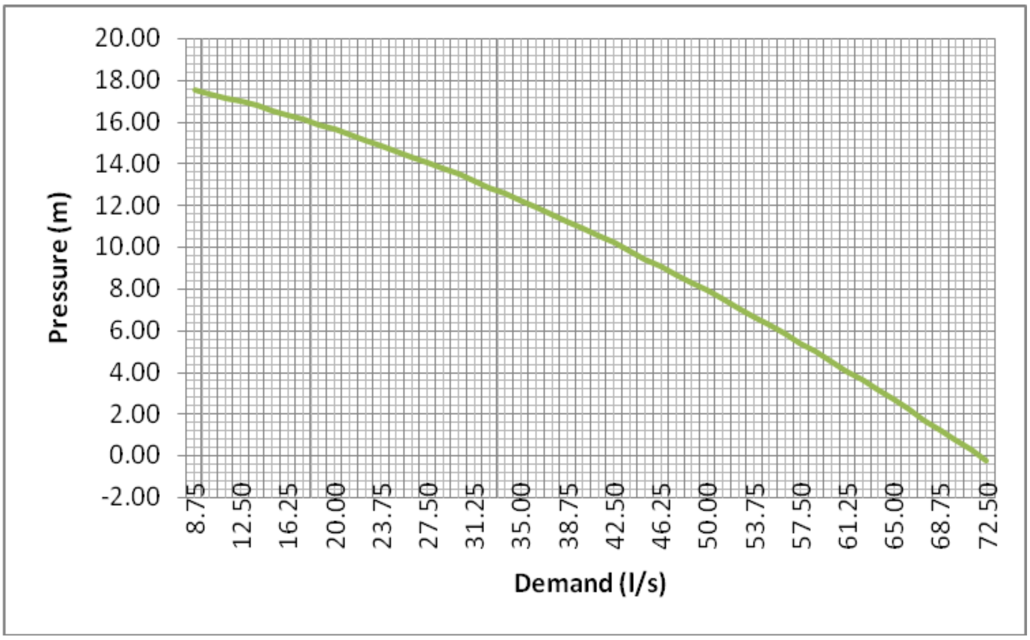

Figure 31: Plot of Demand vs Pressure at Node 24 at 16 00 hours

Table 6: Pressure outputs for given demand values at 16 00 hours

Demand (l/s)	Pressure (m)	Demand (l/s)	Pressure (m)
8.75	17.51	41.25	10.52
10.00	17.33	42.50	10.16
11.25	17.14	43.75	9.79
12.50	16.94	45.00	9.42
13.75	16.74	46.25	9.05
15.00	16.53	47.50	8.66
16.25	16.31	48.75	8.27
17.50	16.08	50.00	7.88
18.75	15.85	51.25	7.48
20.00	15.61	52.50	7.07
21.25	15.36	53.75	6.66
22.50	15.11	55.00	6.24
23.75	14.85	56.25	5.81
25.00	14.58	57.50	5.38
26.25	14.31	58.75	4.94
27.50	14.03	60.00	4.50
28.75	13.74	61.25	4.04
30.00	13.45	62.50	3.59
31.25	13.15	63.75	3.13
32.50	12.84	65.00	2.66
33.75	12.53	66.25	2.18
35.00	12.21	67.50	1.70
36.25	11.88	68.75	1.22
37.50	11.55	70.00	0.72
38.75	11.21	71.25	0.23
40.00	10.87	72.50	-0.28

5.2.2 Pressure Driven Analysis: Demand Response to Pressure

This section looked at the response of water supplied to pressure. In this section, pressure is the determining factor and independent variable and demand is the dependent variable. Table 7 shows the variation of pressure with nodal supply where by an increase in nodal elevation directly implies a reduction in nodal pressure of a similar magnitude due to the fact that total head is a summation of elevation, pressure head and velocity head as per Bernoulli’s equation stated in Equation 5.5.

$$\frac{P}{\rho g} + \frac{v^2}{2g} + Z = \text{Constant} \qquad (5.5)$$

where

$\frac{P}{\rho g}$ is the pressure head where ρ is water density and g is gravitational acceleration

$\frac{v^2}{2g}$ is the velocity head where v is water velocity, and

Z is the static head or elevation.

In order to maintain the same energy content of the water in accordance with the Principle of Conservation of Energy, when elevation increases by a certain magnitude, pressure has to decrease by the same magnitude if water velocity is kept constant.

Figure 32 illustrates the variation of available supply with pressure at Node 24 in the model at 16 00 hours. The initial pressure at the node (elevation 1184 masl) at 16 00 hours is 17.51 m. It can be seen that nodal supply increases with increasing pressure at the node and reduces with declining nodal pressures. When pressure is the determining factor of system performance, then the higher the pressure in the system the more the water supplied.

Table 7: Pressures, elevations and supply at node 24, 16 00 hours

Elevation (masl)	**Pressure Difference (m)**	**Node Pressure (m)**	**Node Supply (l/s)**
1184.0	0.0	17.51	65.51
1186.5	2.5	15.01	59.19
1189.0	5.0	12.51	52.45
1191.5	7.5	10.01	45.14
1194.0	10.0	7.51	37.12
1196.5	12.5	5.01	28.06
1199.0	15.0	2.51	17.34
1201.5	17.5	0.01	3.20

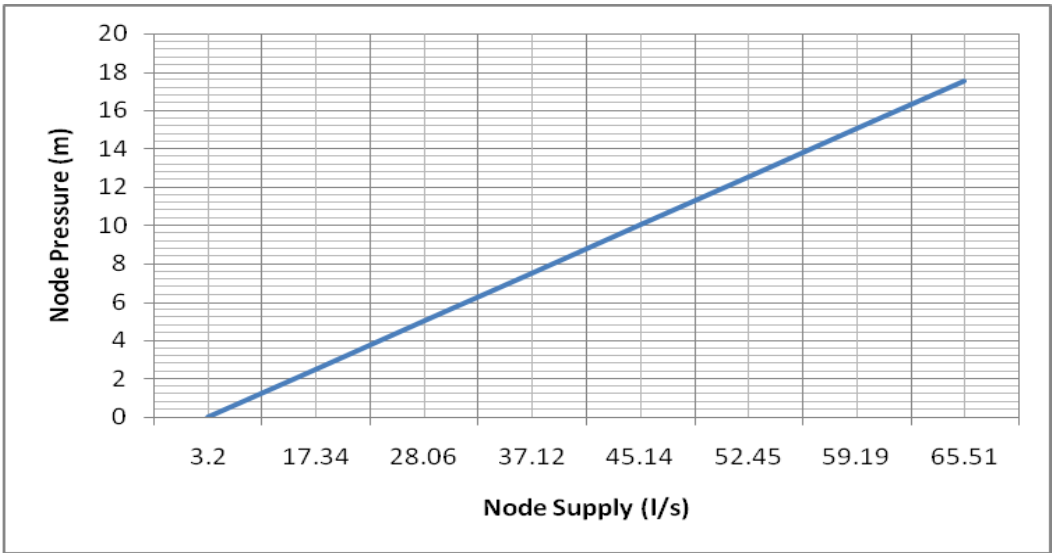

Figure 32: Response of Available Supply to Changing Pressures at 16 00 hours

5.3 Analysis of Pipe Filling

A source code that simulates the process of pipe filling and emptying was developed in this research and programmed in MATLAB programming language. A pseudocode of the program is presented here:

```
Solves the pressurised side of the interface. Uses three equations
Continuity Eq: A2(V2-w)=A1(V1-w). w is interface velocity
A2,V2 are on the pressurised side. A1,V1 are on the free surface side
Momentum Eq: y2A2-y1A1=(v1-w)(v1-V2)A1/g. V1=vp (already defined)
Pressurised Positive Characteristic Eq:
V2-VL2+(g/a)(y2-yL2)=g(So-SfL2)change_t
Solution of the equations above gives a quadratic equation of the
interface velocity (w^2)+(P*w)+ Q = 0
Set g = 9.81
Get pipe diameter D
Work out pipe cross sectional area
Get flow depth yp
Get flow velocity vp
Work out flow cross sectional area
Get pressure wave speed a
P = -2*vp+A2/A1*a;
Enter value of depth of flow yl2 at the start of the pressurised
characteristic equation
Enter pressure y2 at pressurised side of interface
y2_av = 2/3 times y2;
y1_av = 2/3 times yp
yl2_av = 2/3 times yl2
vl2 = input('Enter value of vl2: ')starting point of pressurised
positive characteristic line
Enter value of friction slope at pressurised side Sfl2
Get Pipe slope So
```

```
change_t = 0:60:300;
Q = vp^2-A2/(A1-A2)*(y1_av*g + vp*a)+ A2^2/A1/(A1-A2)*(vl2*a + ...
    (y2-y2_av)*g-a*change_t*g*(Sfl2-So));
w = (-P + sqrt(P^2-4*Q))/2;
Work out velocity v2 at pressurised side of interface v2 = w + A1/A2*(vp-w)
y2 = yl2 + a*((v2-vl2)/g - change_t*(So-Sfl2
change_x = w.*change_t
Calculates distance travelled by interface in change_t along the pipe the
new interface position is obtained by adding the former interface position
at pipe soffit along the pipe) to change_x.
plotchange_x vs change_t
```

Table 8: Model Inputs

Input Parameter	**Value**
Pipe diameter (m)	0.3
Pipe length (m)	150
Pipe slope	0.0001
Manning's Coefficient	0.015
Flow depth at point L (m)	0.002
Flow depth at point R (m)	0.001
Discharge at point L (m^3/s)	0.004
Discharge at point R (m^3/s)	0.006

A typical filling process of a pipe was studied with the chief aim of determining the rate at which water fills up the pipes until pressures start to build up. The simulation involved the following inputs: pipe diameter, pipe length, pipe slope, Manning's Coefficient and flow depths and discharges at two key points in the programme (Table 8). Values of flow depths and discharges at two critical points along the pipeline were initially determined from measurements carried out in the field.

Outputs of the program are presented in this Section. Figure 33 indicates it took 6.4 seconds to fill a 300 mm pipeline under the given initial conditions of inflow discharge and head. During the pipe filling process, no supply can be made under normal circumstances due to the fact that connections are usually made on top of the pipe and considering that the pipe is not yet full, neither water nor pressure is available to enable supply. After the pipe has filled, it starts to get pressurised. It can be reasonably argued that the rate at which the pipe fills is directly proportional to the initial boundary conditions of head and discharge. When the depth of flow exceeds the pipe diameter flow conditons change from free surface flow to pressurised flow as water starts to exert pressure on the pipe.

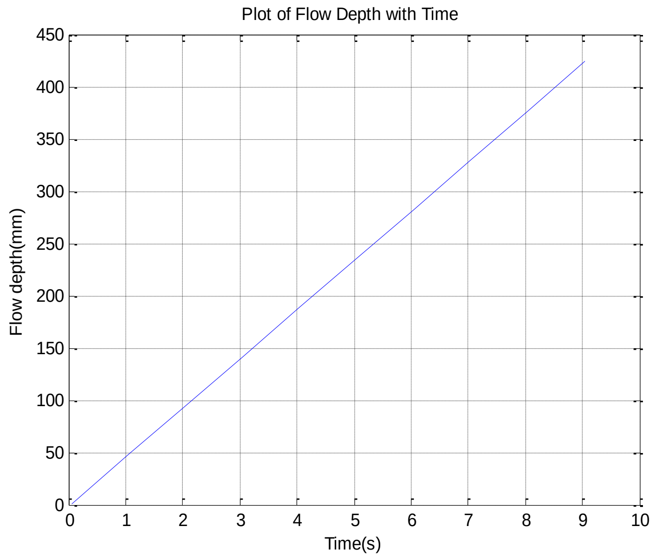

Figure 33: Process of Pipe filling

5.4 Analysis of Pipe Pressurisation Process

When the pipeline is full or near full and the input head is sufficient, it starts to get pressurised. A code that simulates the process of pipe pressurisation and depressurisation was developed in this research and programmed in MATLAB programming language. A pseudocode is presented below:

```
Set g to 9.81
Get pipe diameter D
Get pipe length Lx
Get flow depth at first point yl
work out cross sectional flow area Al
work out top flow width
Get flow depth at second point
Get top flow width at second point
Work out gravity wave speed a first point Cl
Work out gravity wave speed at second point Cr
CL = g/Cl
CR = g/Cr
Get pipe slope So
Get Manning's Coefficient n
Get discharge at first point
Work out velocity at first point vl = ql/Al
Bl = 8 times Al divided by D^2
pl = Bl times D divided by 2
Hydraulic radius R1 = Al divided by pl
Friction slope Sfl = n^2 times vl^2 divided by R1^(4/3)
Get discharge at second point qr
Work out velocity at second point vr = qr divided by Ar
Br = 8 times Ar divided by D^2
pr = Br times D divided by 2
Rr = Ar divided by pr
```

```
Friction slope Sfr = n^2 times vr^2 divided by Rr^(4/3)
change_x = 0.5:0.5:Lx
change_t = change_x/10:10
Value_1 = vl+CL*yl+g*(So-Sfl)*change_t
Value_2 = vr+CR*yr+g*(So-Sfr)*change_t
solution of simultaneous equations
vp+CLyp=value_1
vp-CRyp=value_2
a=[1,CL;1,-CR];
b=[Value_1;Value_2];
flow_values = a\b
vp = flow_values(1,:)
yp = flow_values(2,:)
Flow depth y = D + yp
Plot change_t vs
```

This section presents the results of the simulation of the pipe pressurisation process. The simulation involved the following inputs: pipe length, pipe diameter, flow depth, pressure, velocity and friction slope. Flow depth and pressure values were obtained from initial boundary conditions (typical field values) and were measured from the pipe invert. The velocity was obtained from initial conditions and later confirmed from field values. For the programme inputs in Table 9 the outputs are shown in Figures 34 and 35. Table 10 summarises key model outputs that are analysed subsequently.

Table 9: Model Input Parameters

Input Parameter	**Value**
Flow Depth at the start of the Pressurised Characteristic Equation (m)	0.08
Pressure at Pressurised Interface Side (m)	1.1
Velocity at the start of the Pressurised Characteristic Equation (m/s)	2
Friction slope	0.0008
Pipe length (m) Pipe diameter (m)	1000 0.1

Table 10: Sample Model Outputs

Pressure (m)	Water Velocity (m/s)	Interface Velocity (m/s)	Time (s)	Location (m)
0.4367	2.0035	2.0056	0	0
0.4358	2.1212	2.1935	60	131.6089
0.4332	2.2389	2.3814	120	285.7622
0.429	2.3566	2.5692	180	462.4551
0.423	2.4742	2.757	240	661.6828
0.4154	2.5919	2.9448	300	883.4408

5.4.1 Surge Front Characteristics

The plot in Figure 34 reveals the rate of movement of the hydraulic bore (pressure surge or interface) along the pipeline with time. After approximately 45 seconds the surge front had travelled 100 m giving a velocity of 2.2 m/s and after 250 s it had travelled 700 m giving a velocity of 2.8 m/s (Figure 35). This is a good indication of the rate of development of pressures along the pipeline and shows that the interface accelerates during pressurisation as a result of the net force from the water that overcomes its gravitational and frictional resistance.

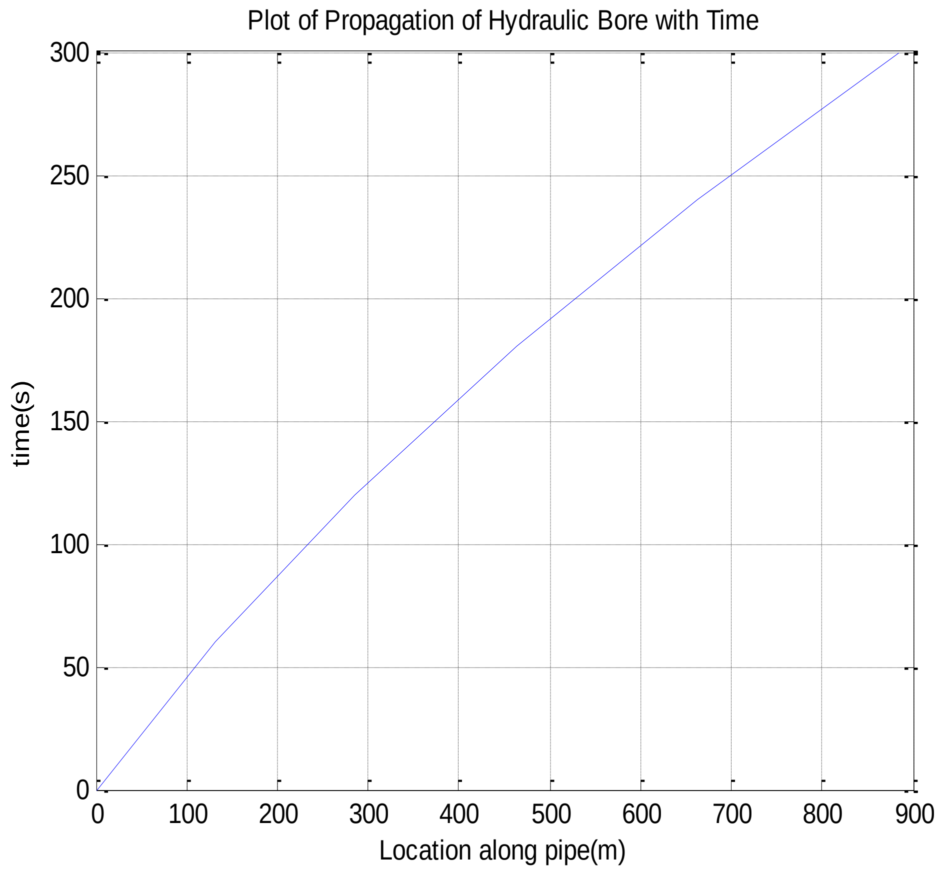

Figure 34: Behaviour of Pressure Surge

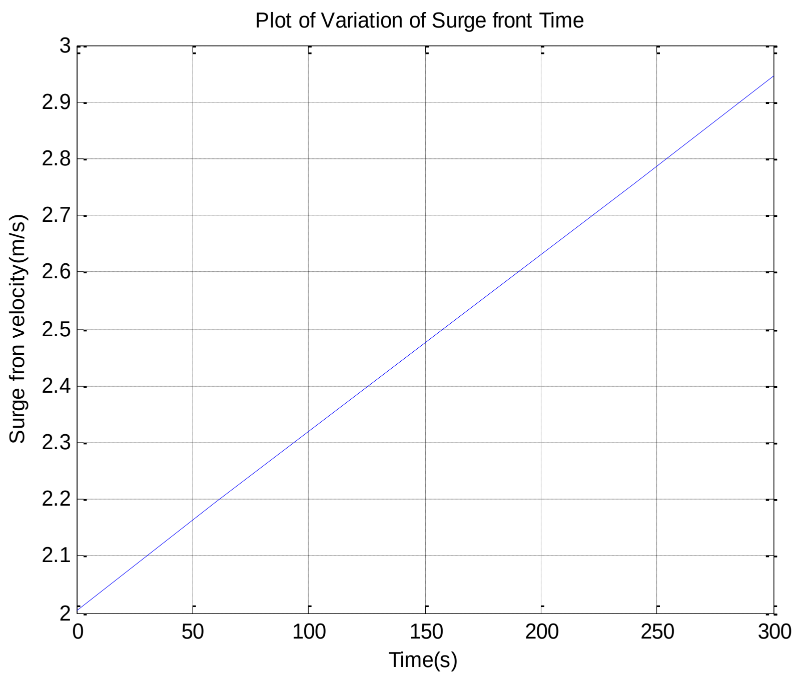

Figure 35: Behaviour of Pressure Surge Velocity

5.4.2 Pressure Characteristics

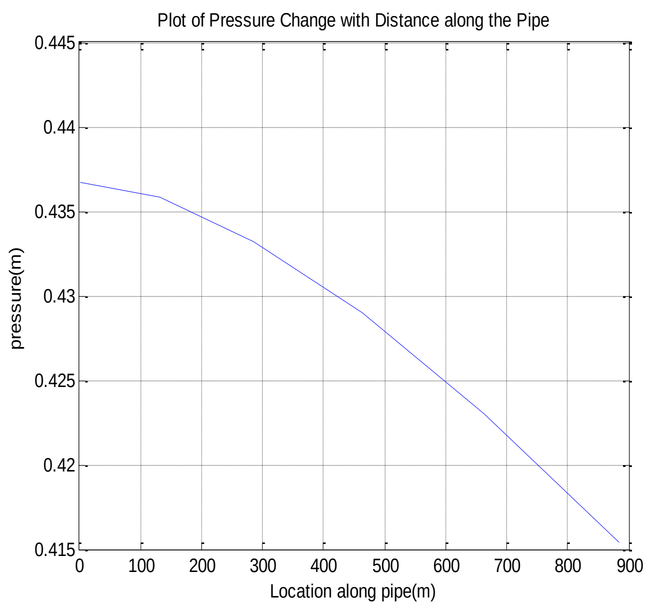

Figure 36: Pressure Variation along Pipeline

The plot in Figure 36 reveals development of pressure along a length of pipe at an instant. It can be observed that pressure builds up gradually along the entire pipeline in accordance with the propagation of the pressure surge or interface. Pressurisation occurs in the direction in which the interface moves. As the interface advances forwards along the pipeline, whichever point it touches starts to get pressurised and the pressure at this point continues to increase ahead of the pressure at the subsequent points along the pipeline. It can be interpreted that pressure will increase with time at this point on the pipeline at the same rate as that of the propagation of the interface along the pipeline.

It should be noted that the pressure values in Figure 36 and in Table 10 are measured from the pipe bottom.

5.4.3 Velocity Characteristics

The plot in Figure 37 reveals the variation of water velocity (not interface velocity) as the interface moves along the pipeline during initial pressurisation. This shows that water accelerates. It can be reasonably explained that the velocity is highest at the points that the interface just touches because it encounters no flow resistance at those points of free surface flow apart from friction.

From Figures 35 and 37, it can be observed as would be expected that the surge velocity is higher than the water velocity. A comparison of surge velocity with water velocity is shown in Table 11 for values picked at two locations; 100 m and 700 m along the pipeline.

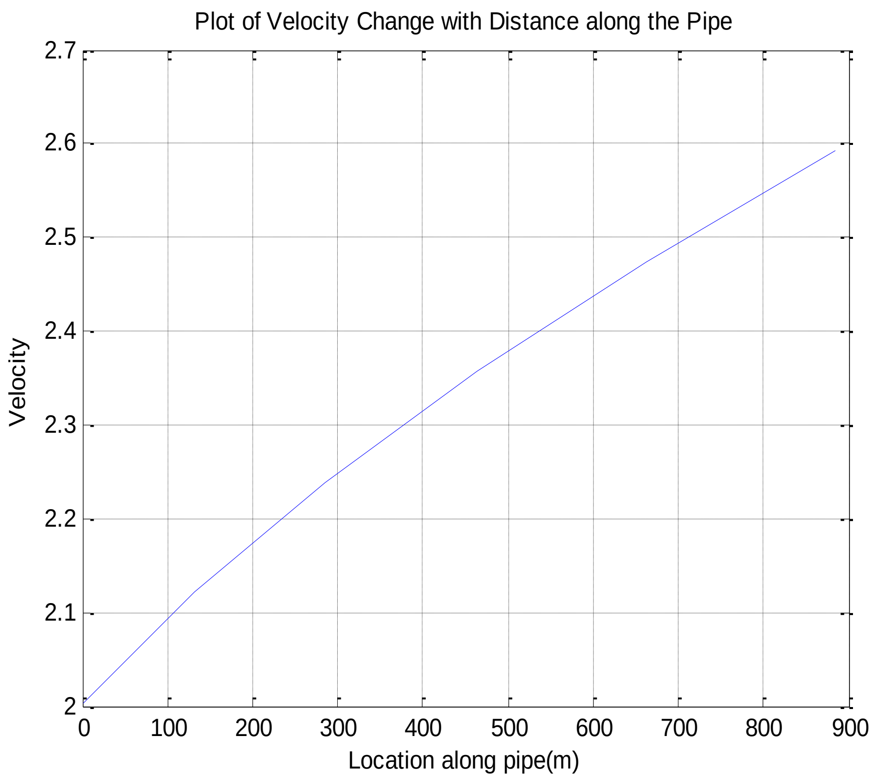

Figure 37: Velocity Variation along Pipeline

Table 11: Comparison of surge front and water velocities

Location (m)	Velocity (m/s)	
	Surge front	Water
100	2.2	2.09
700	2.8	2.5

The results obtained above highlight the advantages of developing a fully dynamic and transient model. Not only is the surge front location and propagation accurately predicted during the transient flow phase between free surface and pressurised flows but flow conditions as well.

5.4.4 Pipe Size and Pressurisation

It is shown that the pressure values obtained during the pressurisation of the pipeline increase with pipe size. For example, the pressures at the various points are higher with a 200 mm pipe (Figure 38) than with the 100 mm pipe used in Figure 36. Table 12 illustrates a comparison of pressures along the pipeline in Figures 36 and 38. Figure 39 shows a comparison of both pressure results in one graph. The bigger the pipe size, the lower the frictional head losses arising from interaction of water with pipe walls. This consequently leads to higher pressures and further justifies the use of bigger diameter pipes in water supply.

Table 12: Comparison of pressures along pipelines for different pipe diameters

Pipe Diameter (mm)	Location along pipeline (m)				
	0	200	400	600	800
100	437.0	434.5	430.5	425.0	418.5
200	442.8	442.3	441.0	439.2	437.0

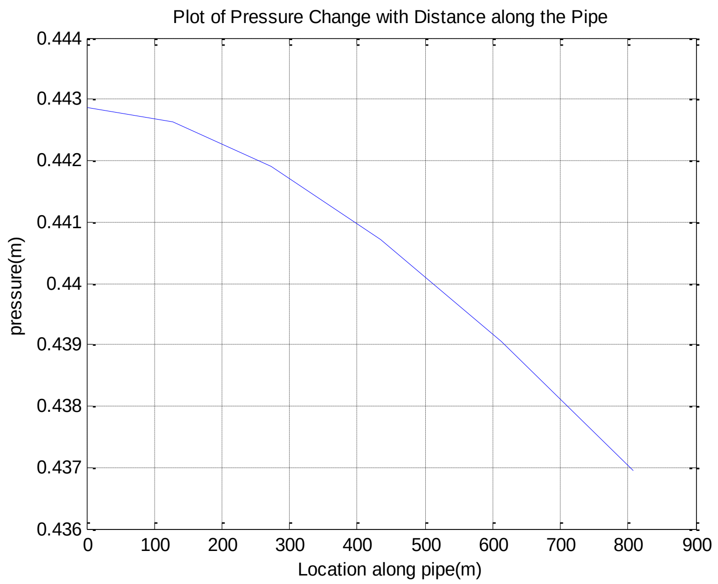

Figure 38: Pressure Variation along Pipeline

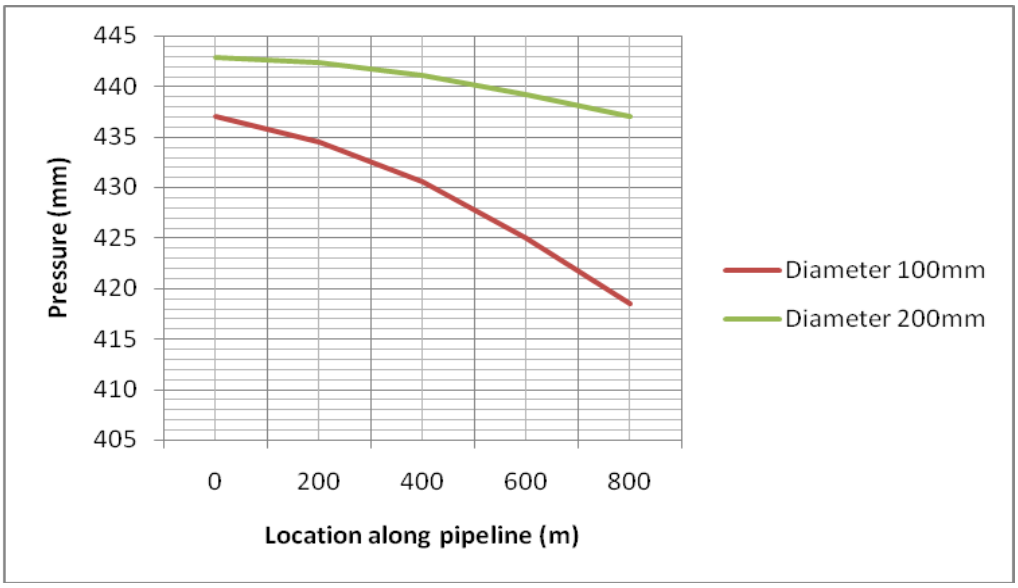

Figure 39: Pressure comparison for different pipe sizes

5.5 Testing of Operations and Management Decision Support Tool

The study developed an Operations and Management Decision Support Tool (OMDST) that would aid the operation and management of water supply systems across the continuum from normal pressures to low pressures to no pressure i.e. from full pressurised flow to partially full free-surface flow. The developed tool becomes a powerful asset for managing networks that exhibit continuous transient flow conditions of a low-pressure-open channel-flow (LPOCF) nature resulting from either inadequate production that cannot meet the demand or an inadequate distribution network. The decision support system developed was tested on the Rubaga subsystem. Figure 40 shows the subsystem with node and link identification numbers (ID) for quick reference; Figure 41 shows the subsystem with node elevations and link diameters while Figure 42 shows the same subsystem with pipe lengths. Scenarios covering normal, low and no pressure cases were investigated and actions taken in order to alleviate the problem and the outcomes are discussed in this section.

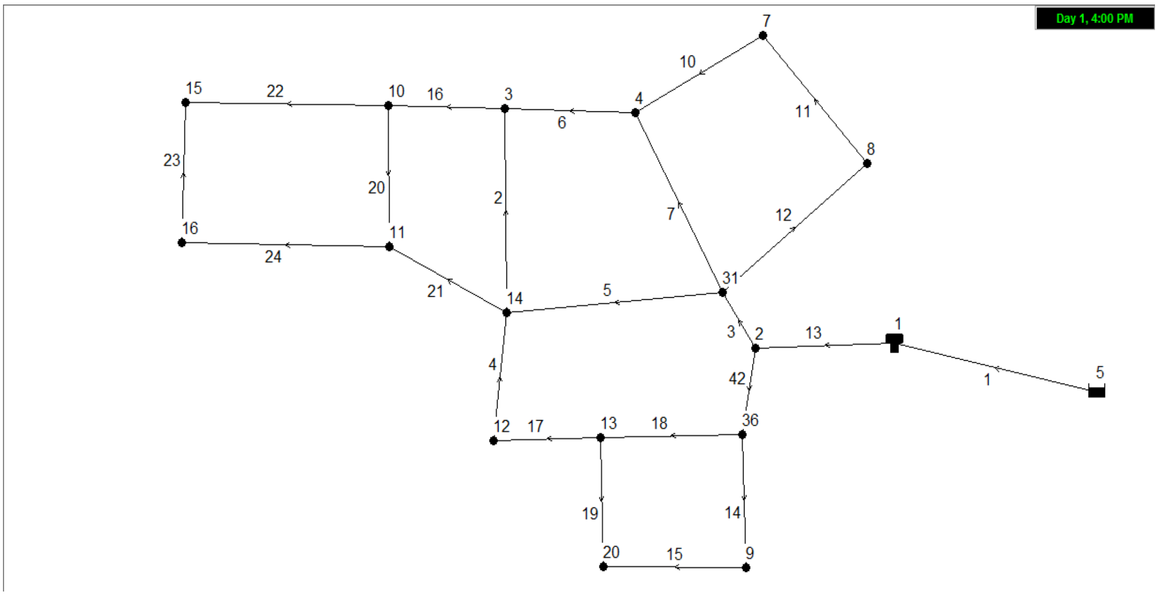

Figure 40: Rubaga Subsystem with node and link IDs

Table 13 shows the initial node demand and corresponding pressure values at 16 00 hours (peak period). The Table also shows a different scenario where higher demand loadings have been put on the network. Figure 43 shows initial pressures at 16 00 hours (peak demand hour) and Figure 44 shows pressures after higher demand loadings are made. It can be observed, as is expected of demand-driven analysis, that lower pressures arise from higher demand loadings. It is particularly observed that negative pressures developed at junction 16 highlighted in Figure 44, which implies an inability to meet the demand at that node.

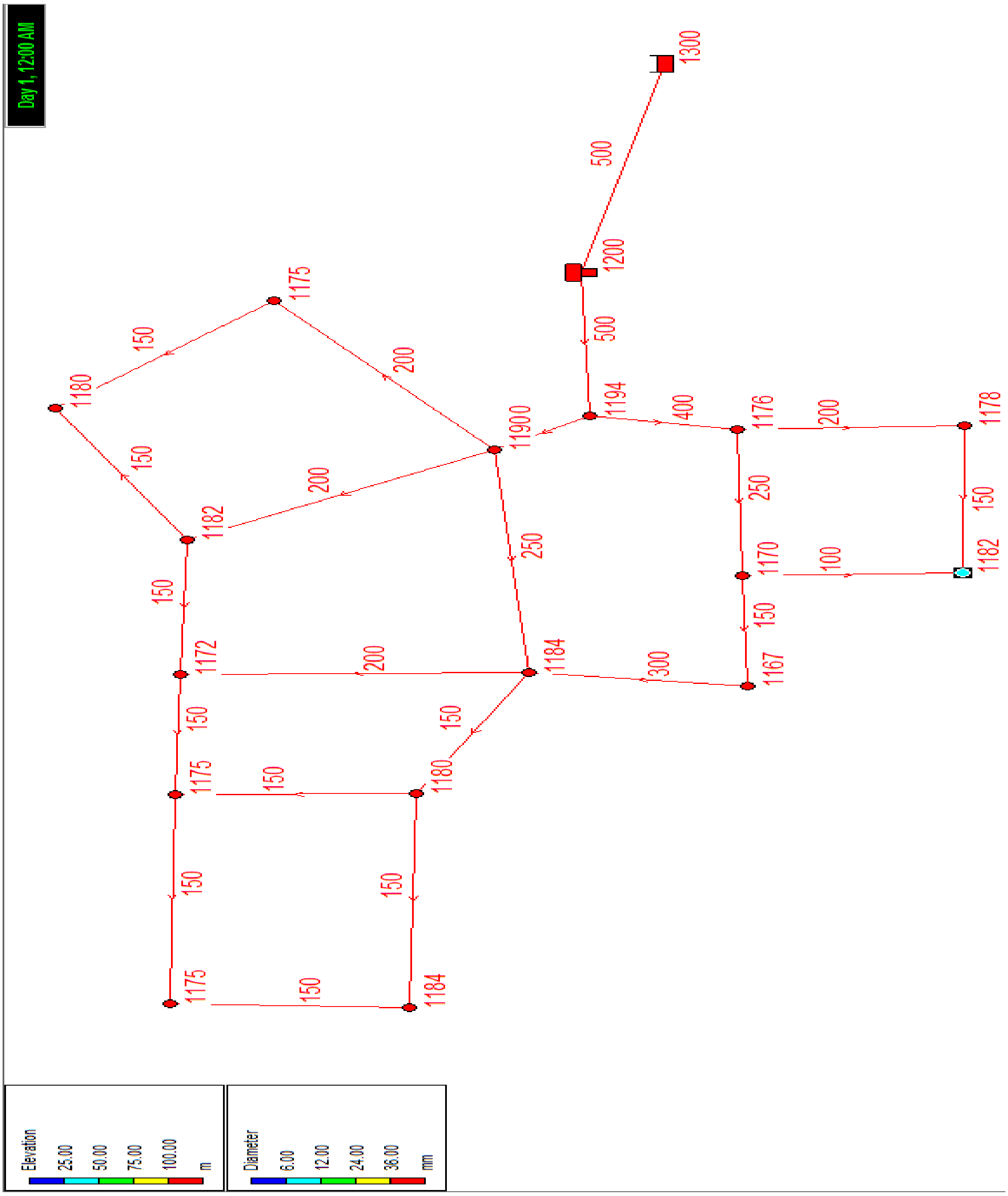

Figure 41: Rubaga Subsystem showing node elevations and pipe diameters

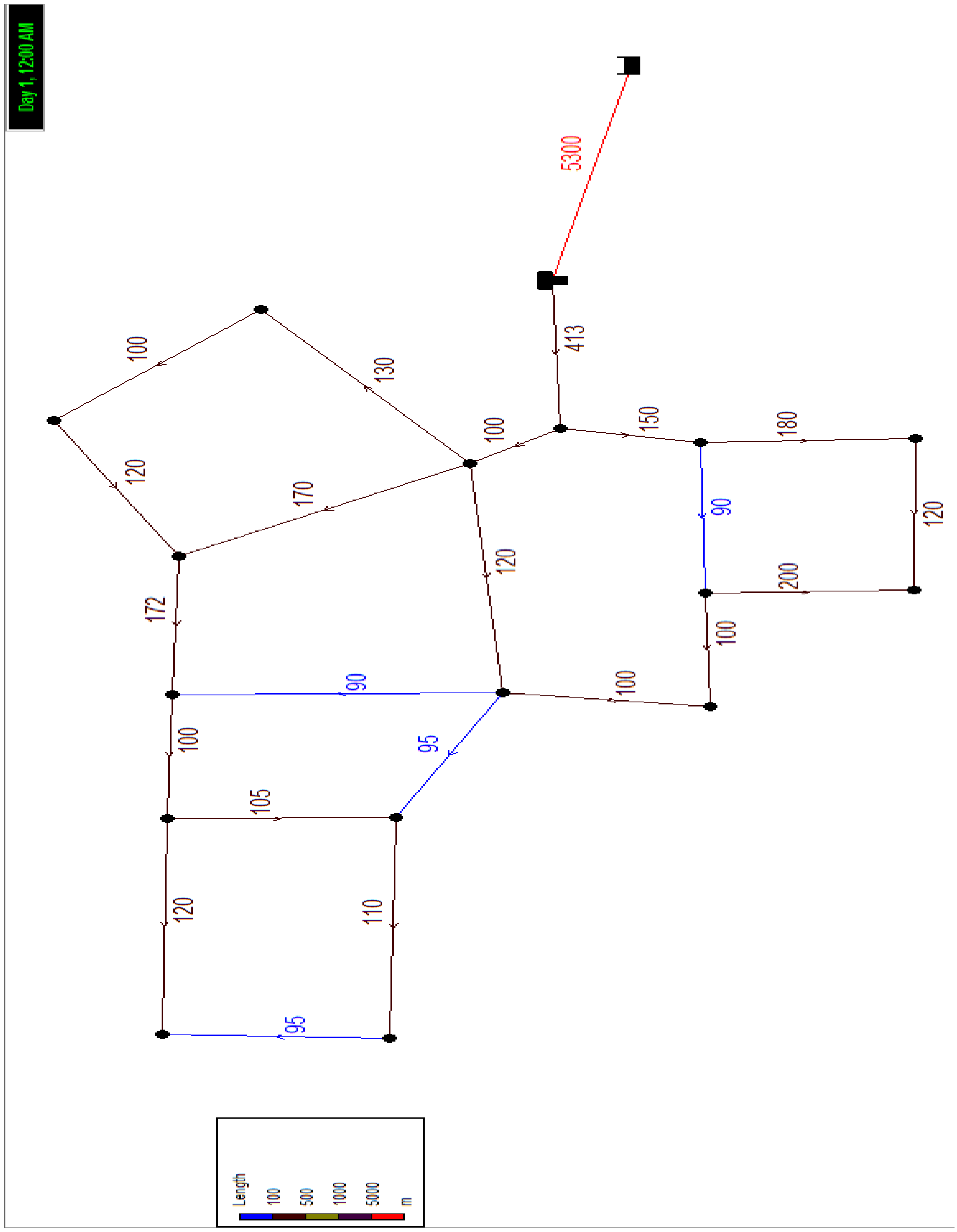

Figure 42: Rubaga Subsystem showing pipe lengths in metres

Table 13: Initial and subsequent pressure and demand values at 16 00 hours

Node ID	Initial Conditions		Subsequent Conditions	
	Demand (l/s)	**Pressure (m)**	**Demand (l/s)**	**Pressure (m)**
2	8.50	10.12	8.50	8.50
14	8.75	19.25	17.50	13.51
20	8.25	21.66	17.50	18.51
31	8.50	13.63	17.50	10.09
36	8.25	28.06	8.25	26.25
12	8.75	36.26	17.50	30.54
13	8.75	33.93	17.50	31.61
3	8.75	31.06	17.50	24.19
4	8.25	21.31	17.50	16.82
7	8.00	23.30	8.00	18.99
8	8.00	28.44	8.00	24.64
9	8.25	25.80	17.50	23.13
10	8.75	27.27	17.50	13.07
11	8.75	22.27	17.50	8.09
15	8.25	27.01	37.50	8.25
16	8.75	18.01	37.50	-0.75
Resvr 5	-584.97	0.00	-585.82	0.00
Tank 1	449.47	4.53	303.07	4.26

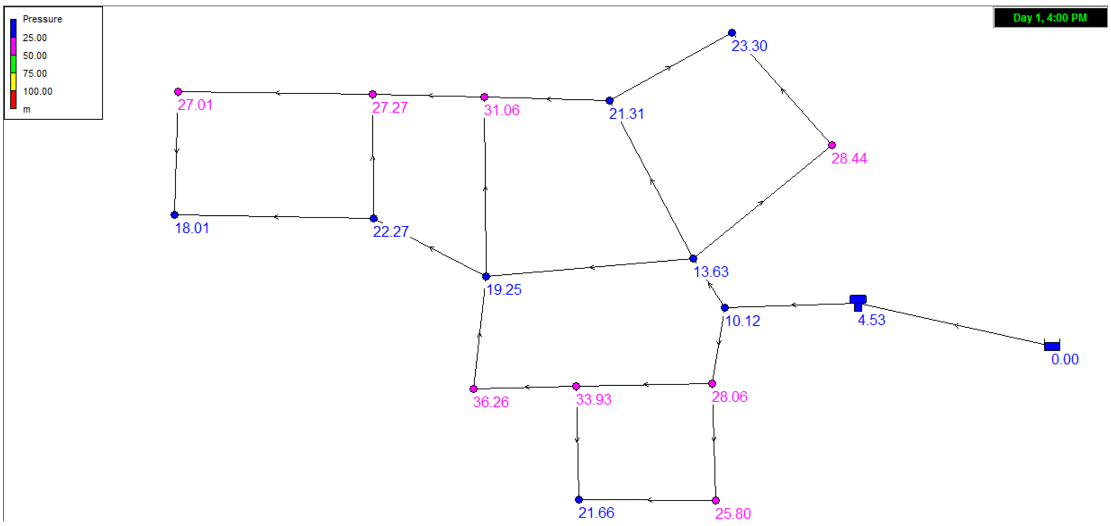

Figure 43: Initial pressures at 16 00 hours

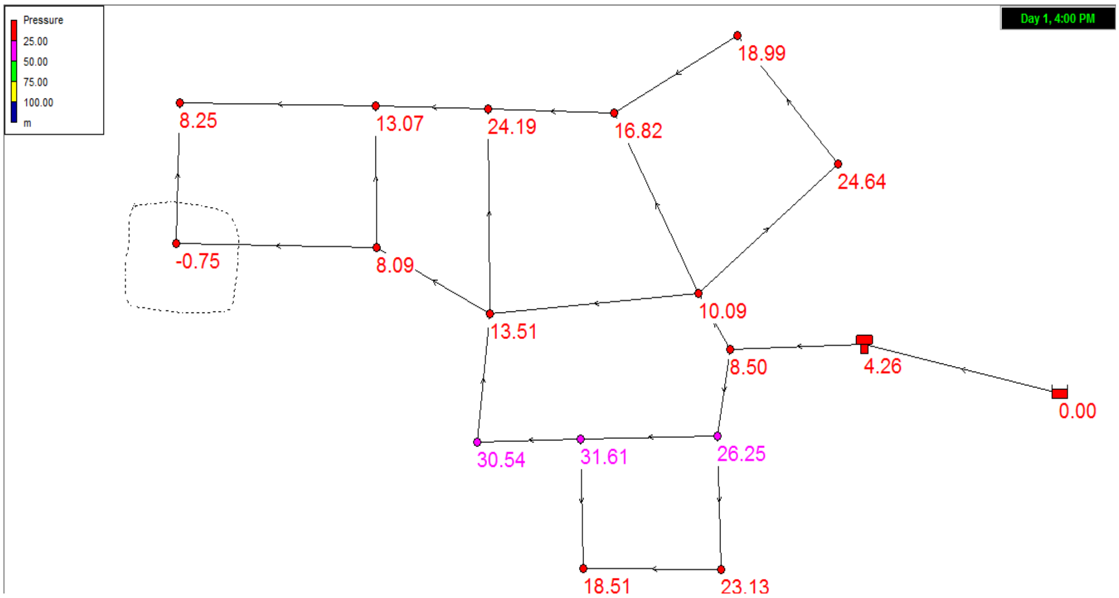

Figure 44: Pressures after higher demand loadings are made

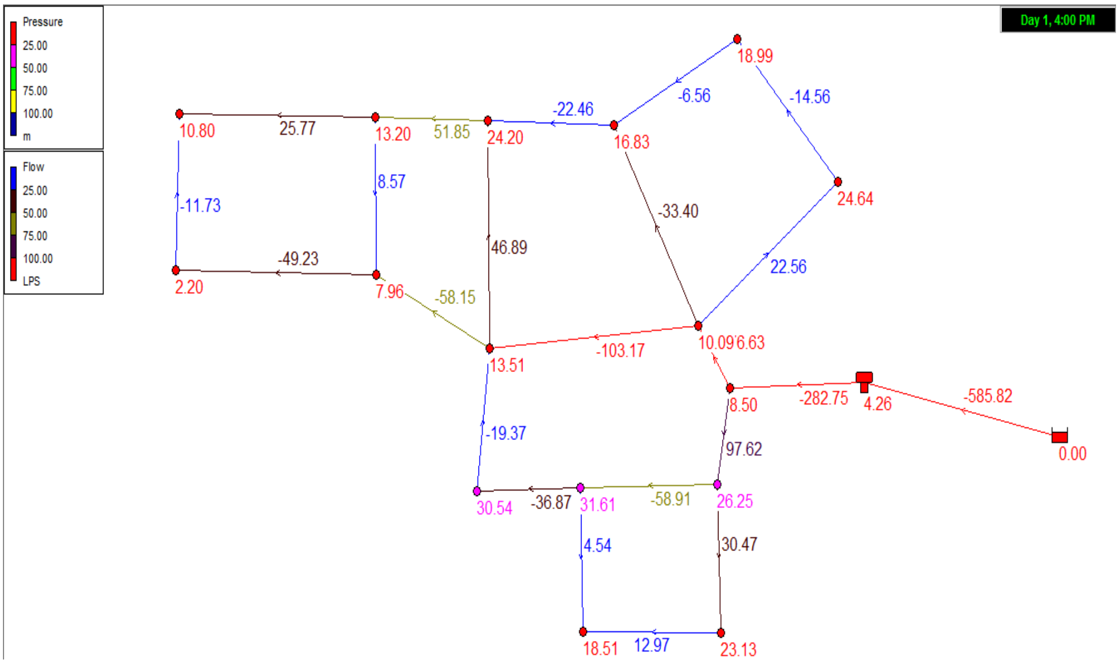

Figure 45: Node pressures and link flows

At junction 16 the pressure required to satisfy a demand of 37.5 l/s is negative which is logically interpreted to mean that at this demand value no supply is possible at this node. This is computational result however in reality some water will come out of this node at a discharge less than 37.5 l/s, in proportion to the prevailing pressure at the node and this further underlies the chief weakness of demand-driven analysis for water distribution networks. Using head-driven analysis however, it can be worked out that the demand that can be met at node16 is 9.73 l/s at midnight and 0.4 l/s at the peak hour.

Increasing pipe size, as discussed in Section 5, has the effect of increasing the rate of pressurisation in the pipe due to the reduction of head losses. For example, if link 24 connecting the pressure deficient node is increased by 50 mm from 150 mm, the node will start to have positive pressures (Figure 45).

6.0 DISCUSSION OF RESULTS

6.1 General

This Chapter discusses results analysed and presented in Chapter 5 from both scientific and operational perspectives with the aim of establishing a workable decision support system for the management of pressure deficient water supply networks. The Decision Support System that was developed in this research sought to answer four broad questions:

i. Can the network satisfy user demands at the nodes at the recommended pressures?
ii. If not, how much water can be supplied at the prevailing pressures at the nodes in question and what actions can be taken to make this possible?
iii. For nodes which cannot supply any water, when are they likely to start to get some water and what actions can be taken to make this possible?
iv. In overall system consideration, how do the various actions (e.g. Figure 46) taken affect the performance of the network at other nodes?

Figure 46: Pipe Laying DN 900 Gaba –Muyenga Reservoir

6.2 Decision Support System

The main purpose of water supply systems is to provide water to consumers at the required pressures and flows. The chief role of hydraulic models used and developed in this research is to simulate pressures and flows in order to predict systems that will fulfil consumer requirements of pressure and flows. According to the Conceptual Framework, the research was geared towards analysing the network in three categories when primarily modelled from a demand driven perspective:

i. Normal Pressures
ii. Low Pressures
iii. Nil Pressure (Atmospheric Pressure)

6.2.1 Normal Pressures

These are pressures above which all demand is satisfied. The pressures are above a limit herein called a pressure threshold which is node specific. Above this pressure threshold, no more water needs to be supplied as demand is already met. In Figure 31, a demand at a node of 45.14 l/s at 16 00 hours was satisfied by a pressure of 10 m which can be called the threshold or reference pressure. This means that above a pressure head of 10 m all of the demand is met while nodal pressure values of less than 10 m will only produce a fraction of the demand. This is very important information for purposes of water supply management due to the fact that it will be clear how much water can be supplied at a certain pressure and this will help invoke informed pressure adjustments of the network in order to supply water to various consumers.

6.2.2 Low Pressures

In this research, low pressures were defined as pressures between the threshold pressure value and zero. Between these values the water released at the nodes varies with the nodal pressure. In Figure 31, a demand at a node of 45.14 l/s at 16 00 hours was satisfied by a pressure of 10 m. Below 10 m head, not all the demand is satisfied.

It was in the interest of this research to predict what demands could be availed by the various pressures in this category, this being very important for real time water supply management and is discussed in Section 6.2.2.4.

6.2.2.1 Pressure and Demand

In the initial model of the demand driven network under analysis, before the network was subjected to additional loads, pressures were normal and customers had their demand fully satisfied. Additional demand loads on the network led to drops in nodal pressures especially at the peak hour (Figure 31) and this demonstrates the fact that there is a demand limit on the network beyond which all demand will not be satisfied, contrary to common belief.

6.2.2.2 Pressure and Head Loss

A small pipe size results in high headlosses which reduce nodal pressures due to increased friction between the water and the pipe walls, increased turbulence facilitated by a small flow cross sectional area and increased local losses. Headloss is inversely proportional to pipe size as was illustrated in Figure 27 and in the Darcy-Weisbach headloss Equation 6.1 where h_L is head loss, f is the Darcy-Weisbach friction factor, L is pipe length, D is pipe diameter, V is water velocity and g is gravitational acceleration.

$$h_L = f\frac{L}{D}\frac{V^2}{2g} \qquad (6.1)$$

Equation 6.1 also illustrates that headloss is directly proportional to pipe length and water velocity. Figure 28 shows that the higher the pipe size the lower the flow velocity in line with the principle of mass conservation and since water density is constant, what is kept constant in effect is volume due to the fact that mass is a product of volume and density. Therefore, in order to maintain the rate of volume delivery i.e discharge, the velocity will decline if the pipe diameter or cross-sectional area increases. It is important however to select a pipe diameter which will yield velocities which are high enough to flush the pipe and whose pressure is adequate to avoid infiltration in order to maintain high hygiene levels. It is also important to select pipe diameters which do not yield very high velocities in order to avoid scouring of the pipe walls.

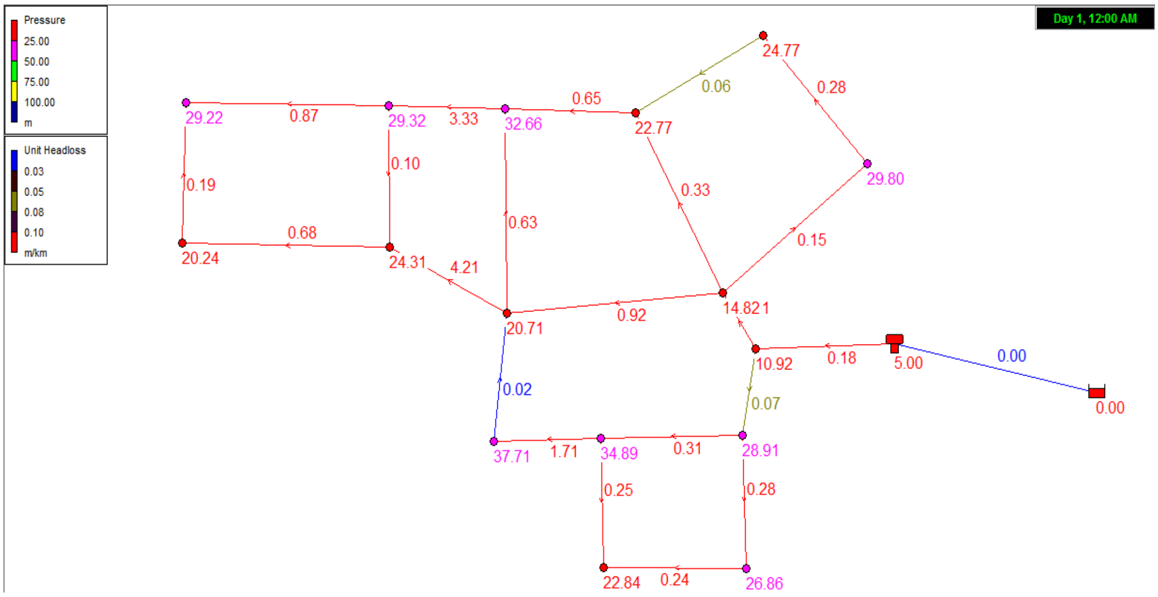

Figure 47: Rubaga subsystem showing head losses and node pressures

Figure 29 also revealed how high nodal pressures can result from low head losses in connecting pipes as is logically expected. However beyond a certain size headlosses cannot reduce any further and so nodal pressures remain constant (Figure 29). Figure 47 shows the Rubaga subsystem indicating head losses across pipes and pressures at nodes.

Pressure along a pipeline varies in accordance with head loss if other factors are kept constant e.g. supply, elevation and cross section. The higher the head loss in the pipeline the higher the pressure drop as demonstrated in Figure 47 showing head loss of 4.21 m/km across a 95 m pipe, ID 21 (see Figure 40 for link ID and Figure 42 for pipe length). The elevations at the two ends of the pipe are 1184 masl and 1180 masl (Figure 41) resulting into pressures of 20.71 m and 24.31 m. The following calculation illustrates the role played by headloss in reducing pressures. While the expected pressure should be 24.71m at node 11, the model shows a pressure of 24.31 m; the difference is brought about by the headloss across the pipeline.

Length of link connecting the two nodes		95 m
Elevation of higher node		1184 masl
Elevation of lower node		1180 masl
Pressure difference (equal to difference in elevations)		4 m
Pressure at higher node		20.71 m
Expected pressure at lower node	20.71 + 4 =	24.71 m
Actual pressure at lower node		24.31 m
Difference between expected and actual pressure at lowere node =		0.4 m
Total headloss across link (unit headloss * pipelength) = 4.21 * 0.095 =		0.4 m

Table 14: Variation of Pressure with Elevation and Head Loss

Location along pipe (m) from higher node	Pressure difference due to elevation (m)	Pressure (m)	Head Loss (m)	Final pressure (m)
0	0.00	20.71	0.00	20.71
20	0.84	21.55	0.08	21.47
40	1.68	22.39	0.17	22.23
60	2.53	23.24	0.25	22.98
80	3.37	24.08	0.34	23.74
95	4.00	24.71	0.40	24.31

The variation of pressure along the pipeline is demonstrated in Table 14 and Figure 48. Pressure increases along the pipe in the downstream direction because the subsequent elevation reduces along the pipeline. The lower the elevation the smaller the height (static head) that water has to rise to and this results in higher pressures.

From the foregoing discussion it is of crucial importance for engineers to select the most optimal pipe size in order to deliver the highest pressures and flows while avoiding both undersizing and oversizing which increase capital and operational costs and in this context the model plays a key role. Head losses in pipelines can also be controlled by using smooth and new pipes and by limiting pipe lengths. This can be done by connecting consumers to closer supply tanks in the network.

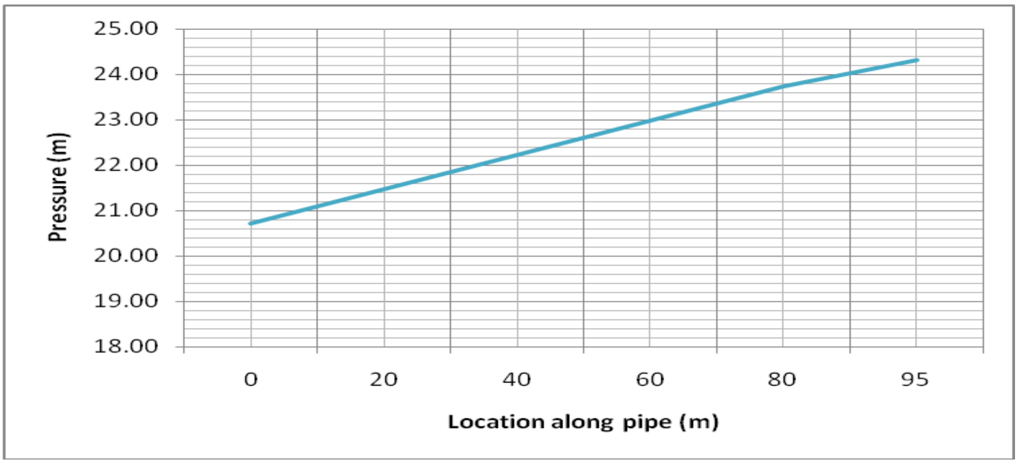

Figure 48: Variation of Pressure along pipe

6.2.2.3 Pressure and Elevation

Key determinants of pressures at nodes are elevations at the nodes and the supply tank. The higher the difference between the nodal and suppy tank elevations, the higher the nodal pressure. Table 15 shows the variation of the pressures at nodes 16 and 17 (located at 1184 masl and 1175 masl) with the elevation of the supply tank (located at 1200) at midnight. Figure 49 shows the variation of the pressure at node 16 with the elevation of the supply tank. Pressure at nodes increases with the elevation of the supply tank because the static head from which water is to be supplied increases as the supply tank is raised higher.

Table 15: Variation of supply tank elevation with pressure at nodes 16 and 15

Supply Tank Elevation (m)	Node 16 Pressure (m)	Node 15 Pressure (m)
1200	20.24	29.22
1210	30.24	39.22
1220	40.24	49.22
1230	50.24	59.22
1240	60.24	69.22
1250	70.24	79.22

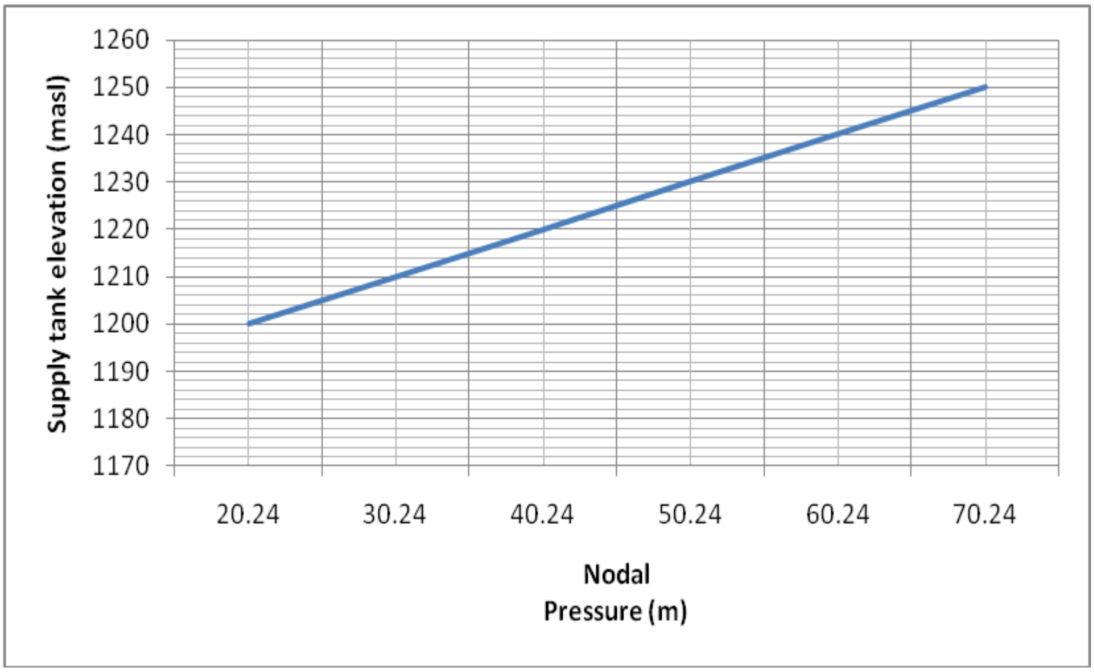

Figure 49: Variation of Nodal Pressure with Supply Tank Elevation

Table 16 and Figure 50 show the hydraulic grade line along several pipes connecting nodes from the supply tank up to the last node 16 along the route through nodes 2, 31, 14 and 11 (refer to Figure 40 and Figure 50). The hydraulic grade line is conviniently plotted along side the elevation for comparison purposes.

Table 16: Head variation in the network

Node	Connecting pipe length (m)	Location from Supply tank (m)	Elevations (masl)	Pressures (m)	Head (m)
1	0	0	1200	5	1205
2	413	413	1194	10.92	1204.92
31	100	513	1190	14.82	1204.82
14	120	633	1184	20.71	1204.71
11	95	728	1180	24.31	1204.31
16	110	838	1184	20.24	1204.24

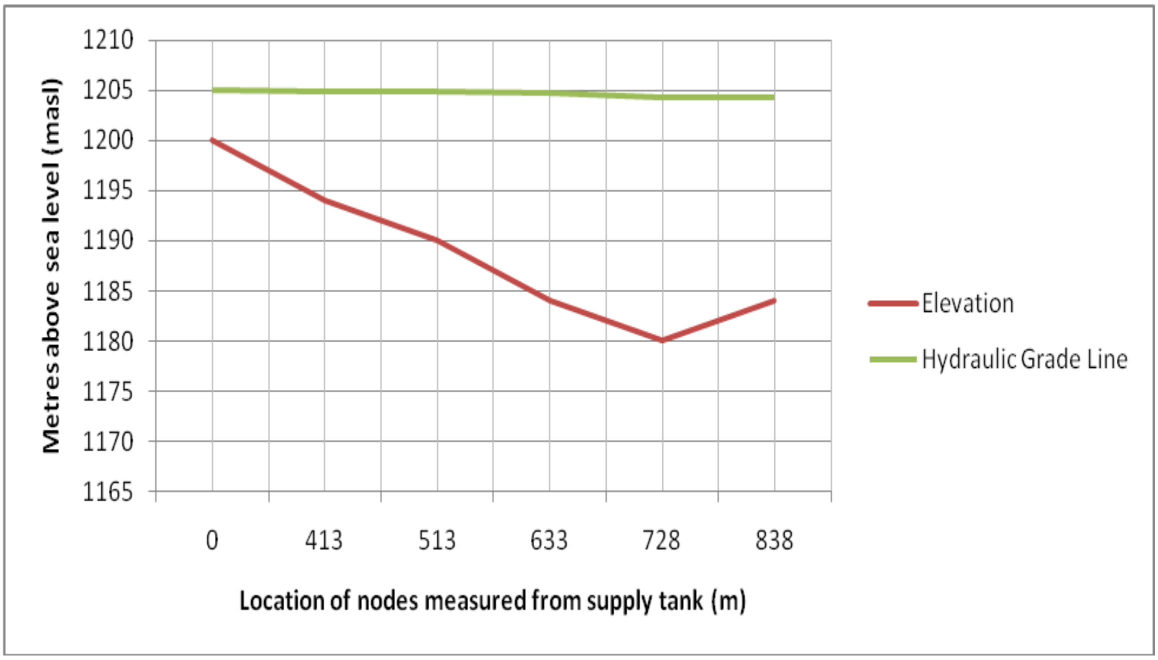

Figure 50: Head variation in the network

It can be reasonably expected that one of the ways to improve node pressures is by increasing the elevation of the supply tank or lowering the elevation of consumer nodes. Elevation of the supply tank is a more viable alternative than lowering of consumer nodes which would require consumers to draw water from low heights including below the ground surface.

6.2.2.4 Pressure and Velocity

Hydraulic logic has its limitations for example, a solution such as enlarging pipe diameters in an effort to increase discharge (Section 5.1.4) and which also reduces friction losses, consequently yields smaller velocities (Figure 28) hence it may appear difficult to optimise both pressures and velocities in the system. Figures 51 and 52 show pressure measurements that were carried out on two 150 mm and 80 mm diameter pipes supplied from the same main. The average pressure on the 150 mm pipe was 3.84 m while the average pressure measured on the 80 mm pipe was 2.38 m. This clearly shows that pressures tend to be higher on bigger pipes.

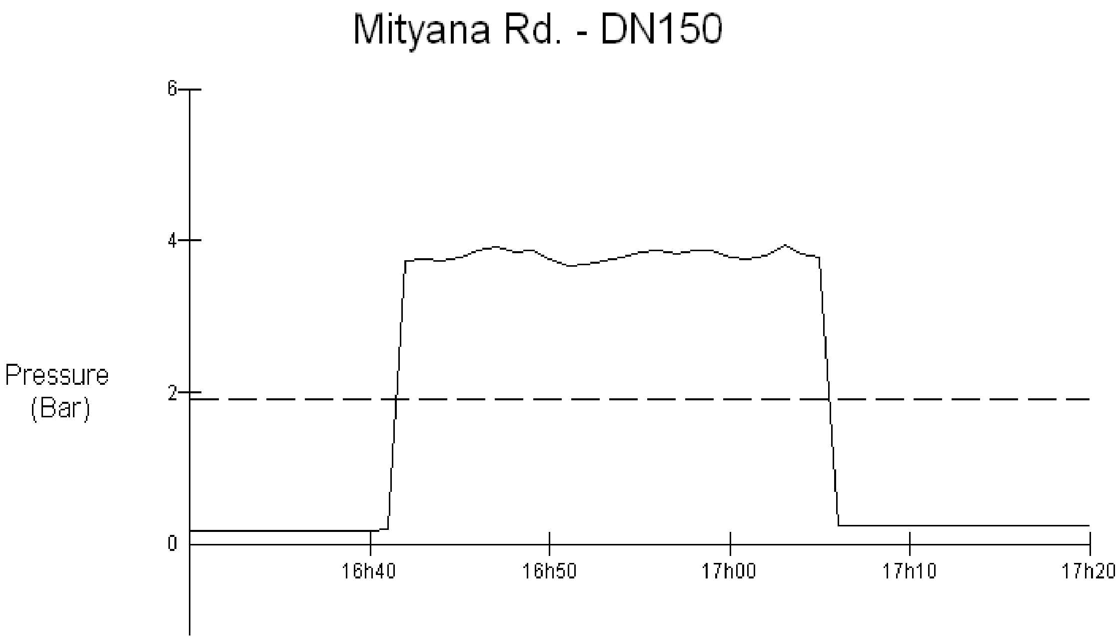

Figure 51: Pressure Measurement on DN150 main along Mityana Road

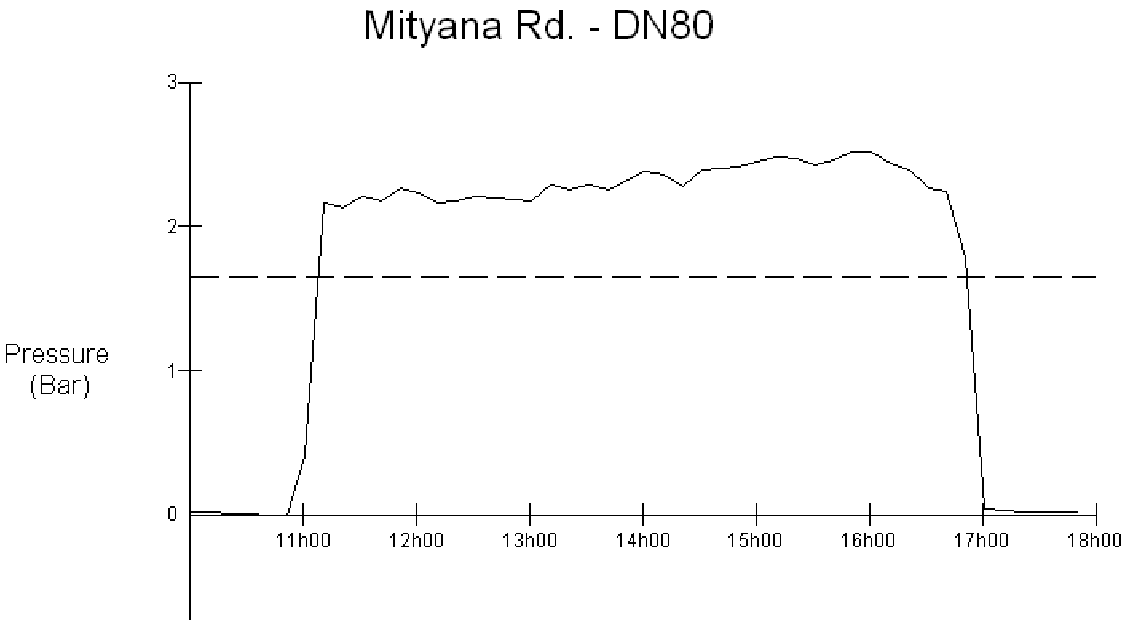

Figure 52: Pressure Measurement on DN80 main along Mityana Road

Furthermore, in systems where reliable and cheap energy is available, cost calculations may show that a lower investment in pipes and reservoirs justifies the increased operational costs of pumping. Hence, there are no rules of thumb regarding optimal pumping or ideal conveying capacity of the network. It is often true that more than one alternative can satisfy the main design parameters. Thorough analyses should therefore be conducted for a number of viable alternatives. It is important to adopt an appropriate distribution scheme and select independent and dependent variables for the situation. Pumping is an obvious choice in flat areas and in situations where the supply point has a lower elevation than the distribution area. In all other cases, the system may entirely or at least partly be supplied by gravity which translates into the flowing practical guidelines:

i. maximum utilisation of existing topography (gravity);
ii. use of pipe diameters that generate low friction losses;
iii. as little pumping as necessary to guarantee design pressures;
iv. valve operation reduced to a minimum.

Besides maintaining an optimum range, pressure fluctuations are also important. Frequent variations of pressure during day and night can create operational problems, resulting in increased leakage and malfunctioning of water appliances. Reducing the pressure fluctuations in the system is therefore desirable. The design criteria for hydraulic gradients depend on the adopted minimum and maximum pressures, the distance over which water needs to be transported, local topographic circumstances and the size of the network, including possible future extensions.

The following values can be accepted as a rule of thumb (Bhave 1991; American Water Works Association 2004):

i. 5-10 m/km for small diameter pipes;
ii. 2-5 m/km for mid-range diameter pipes;
iii. 1-2 m/km for large transportation pipes.

Velocity range can also be adopted as a design criterion. Low velocities are not preferred for hygienic reasons such as failure to flush the pipe and resultant low pressures which facilitate infiltration, while too high velocities cause excessive head-losses. Standard design velocities are:

i. ± 1 m/s in distribution systems;
ii. ± 1.5 m/s in transportation pipes;
iii. 1-2 m/s in pumping stations.

6.2.2.5 Comparison of Demand Driven and Pressure Driven Analysis

In this Section we looked at demand response to changing pressure. It is true that in reality, once the demand for a certain area has been fulfilled at a particular pressure, a further increase in pressure may supply more water but the demand will not change. This implies that if a certain amount of water is required for a certain purpose, no more water is necessary even if it is available. However below a certain threshold pressure, the amount of water that can be supplied begins to fall short of the required demand value. It is this relationship between pressure and demand that was studied in this section. Whereas demands in water distribution softwares are an input and nodal pressures are outputs (demand driven analysis), it is necessary to determine how much water different nodal pressures can supply i.e. pressure becomes an input and supply becomes an output (pressure driven analysis).

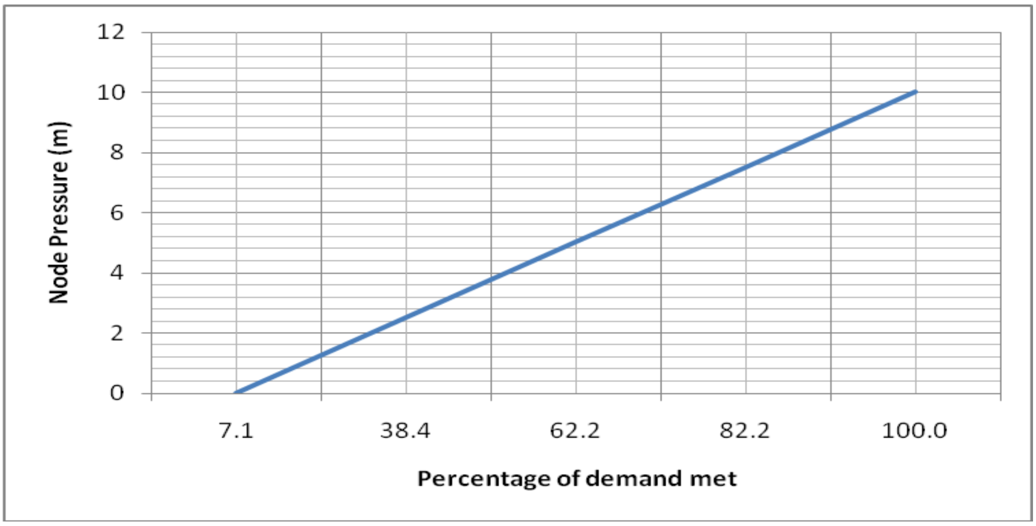

Figure 53: Proportion of original demand met at different pressures

This research enabled the determination of the quantity of water that can be supplied at various pressures, an ability that is absent in water distribution models in current use. Figure 53 shows what proportion of the original demand can be met at various pressures in accordance with Figure 32. The higher the pressure at any node, the more the discharge (supply). The more the supply, the higher the fraction of demand that can be met at the node.

In demand driven analysis, the higher the demand the lower the pressure (Figure 30) whereas in head driven analysis (also called pressure driven analysis), the higher the pressure, the higher the demand and the lower the pressure the lower the demand. Demand Driven Analysis (DDA) is valid when system pressures are adequate at all nodes so that demands are independent of pressure. If this is not the case, inaccurate outputs like demand fulfilment at negative pressures are obtained as is illustrated in Figure 44 (Section 5.5).

Determination of pressures and flows in water distribution models involves the solution of governing equations of energy and mass conservation. Since, under demand-driven analysis, demands are fixed model inputs while pressures are the demand-dependent outputs, in the event of excessive demand loading, the model returns very low and negative pressures. This is a purely mathematical result because practically and physically we cannot have negative pressures within the domain of what is being studied. It is important to note that the negative pressures returned in this model are different from the negative pressures that occur in pressurised flows causing cavitation. In this research, the system is stretched and there are no pressures at all. During cavitation sub-atmospheric pressures are indicated.

When this situation of no positive pressures occurs, it suggest two possibilities:

i. The program has sought to fulfil excessive nodal demands. In this case it is necessary to reduce the demands till the model produces positive pressure outputs under Demand Driven Analysis. Alternatively one can carry out an investigation into available pressures at the affected nodes and how much water can be supplied at those pressures i.e. what proportion of the demand can be met under Head Driven Analysis (HDA). HDA recognizes the dependency of demand on pressure and this is supported by the fact that practically during pressure deficient conditions the system supplies what is available in accordance with the prevailing pressures. This research addressed this situation during the low pressure regime.

ii. Alternatively, no pressures are physically available at the node. This is the situation tackled in Section 6.2.3.

6.2.3 Nil Pressure (Atmospheric Pressure)

During conditions of excessive withdrawals, inadequate supply or an inadequate distribution system, pressure is zero and consequently no water can be supplied. In this case the pipe may either be full, partially full or empty, exhibiting free surface flow conditions. In reality the pipes always have water which either fills or nearly fills their cross sectional area but the pressure is lacking to push it out. This is why in the majority of cases it is found necessary to make connections to the sides of the pipes in order for some water to be drawn. With conditions changing from pressurised to no pressure (free surface flow), existing water supply models are unable to describe neither the free surface flow nor the transition between the free surface to pressurised flow. The interest of this research lay in the pressurisation of these pipes and largely in the coexistence of the free surface and pressurised flow regimes as a moving pressurisation surge (interface) moves along the pipeline.

6.3 Modelling of Pipe Pressurisation

Modelling water distribution systems always assumes that pipes are flowing full and are under pressure, an assumption which yields undesirable results when system pressures are deficient. No regard is given to the process that leads to the pressurised full pipe flow state; that prior to this state the pipeline undergoes filling and inevitably goes through a free surface flow stage followed by a pressurisation stage. The pressurisation process can last a short or long time depending on discharge, pressure and demand. Situations have been observed whereby while water is being pumped or provided under pressure, there appears to be alternate pressurisation and depressurisation in the pipeline because downstream users continue to draw water from the pipeline, an event enabled by the pressure head or elevation difference between the water in the pipeline and the user. If the rate of draw-offs exceeds the rate at which water is being pumped, depressurisation in the pipeline will occur and if some valves are closed there will be pressurisation in the pipelines. In areas that suffer from water supply constraints, the network can dwell in the pressurisation stage for a very long time. Such a system cannot be effectively analysed using the traditional water distribution models.

This research studied the situation when pipes are not under pressure i.e. flow under free surface conditions and analysed the process of pipeline pressurisation. In order to enable the simulation of the pressurisation process and the determination of flow parameters along the pipe, additional nodes were inserted programmatically along pipelines at predetermined locations of interest (i.e. where flow parameters should be determined) and then isolated on both sides. The additional nodes provided particular locations at which flow parameters would be measured.

Pipes were simulated for pressurisation by a moving interface between pressurised and free surface flow. The pressurisation process of the pipes involved tracking the movement of the interface with the aim of determining where and when pressures would start to build up along the pipes. The process was analysed and modelled in this work with a view of clearly understanding what happens during this phenomenon and consequently aiding engineers in ultimately designing systems and operations that take full advantage of this phenomenon.

The motivation to study this flow phenomenon arose from the fact that it is the ‘pressurisation’ stage that leads to the ‘pressurised’ state that is of profound interest to water supply managers and engineers. As a management tool this would help inform when particular sections (nodes) of the pipes would start to pressurise upon filling of the entire cross sectional area thereby starting to release water and what actions should be taken for this to happen.

Since pressurisation represents a flow regime transition from free surface to pressurised flow and provides a link between the two regimes, any techniques used to study it should take cognisance of both regimes while at the same time providing a satisfactory link between them. The method adopted in the research involved the study of the propagation of the pressure surge represented by a moving interface between pressurised and free surface flow. The surge is a transition from pressurised flow to free-surface flow resulting into a moving interface with full, pressurised flow on one side and free surface flow on the other.

This particular stage in the research study involved coming up with a system of equations to back up the study. A model simulating the propagation of pressure surges was developed based on the Interface Tracking Method and the Method of Characteristics. The basic equations fundamental to the research were the unsteady gradually varied flow equations also known as St. Venant or shallow wave equations because of the very nature of the flow characteristics i.e. very low or no pressures, and low velocity flows whose non-uniform features vary gradually along the pipeline while at the same time varying with time. These equations were modified in order for them to be relevant to the study. Two codes were written in MATLAB to simulate pipe filling and track the movement of this interface and the build-up of pressures along the pipeline. Pressures at points which the interface touched continued to rise as more water entered the pipe until the entire pipeline was pressurized.

This research was aimed at enhancing our understanding of propagation of pressure surges through water supply pipelines by simulating the unsteady flows in closed conduits. The simulation covered all flow situations ranging from free surface flows to partly free surface-partly pressurized flows (mixed flows) to fully pressurised flows.

If we can track the movement of the hydraulic bore in the pipelines, then we can be able to tell when and where pressures are sufficient in the network, and the subsequent flows that can be enabled by these pressures. If downstream conditions are adjusted accordingly for example by varying the valve aperture, then the model can predict the various pressures and flows that are enabled by the implemented actions while the network undergoes pressurisation and depressurisation.

By obtaining the pressures along the pipeline in the network we are able to predict what pressures are available at particular locations in the network. Pragmatic management decisions can then be made to ensure that water is available at different sections of the network. Such decisions can include closure or opening of valves, installation of adequate pipe and reservoir sizes in the network or a logical rationing program for water supply.

The results obtained highlight the advantages of developing a fully dynamic and transient model in the solution of transient Low pressure – Open Channel Flow Conditions in water distribution networks. Not only is the surge front location and propagation accurately predicted during the transient flow phase between free surface and pressurised flows but flow conditions as well (St. Venant equations of gradually varied flow are full dynamic equations. Determination of precise locations of flow depths and velocities require use of momentum and continuity equations which requires full dynamic flow).

6.3.1 Practical Pressurisation

Herein are further discussed research findings about the nature of pipeline pressurisation, punctuated with theoretical back-up. A pipeline usually starts getting pressurised from the downstream end due to the slope and as it fills up, a pressurisation wave builds upstream-wards i.e. travels towards the upstream end of the system and behaves like a hydraulic jump. It should be noted that even for a rising main fed by pumping, the pressurisation wave starts from the point of low gradient although in this case the pressurisation wave is in the same direction as the water flow.

It is also worth noting that as conceptualised in this research, downstream does not always refer to a lower height, it only refers to the destination of water. In other words, for a rising main the upstream side of the water is actually at a lower height than the downstream side! If inflow rate is significantly larger than the outflow rate, the speed with which the pressurisation wave moves upstream can be very significant; infact in this case pressurisation can occur from upstream towards downstream.

It should be noted that pressurisation does not always occur when the pipeline is full; it can also occur when the pipeline is not completely full as long as the initial head and discharge can allow sufficiently rapid filling. It can be realized that if the initial discharge and head are insufficient, lower water depths cannot produce the transition to pressure flow, but only an increase in water depth. On the other hand, greater water depths may not show a pressure front but almost an instantaneous transition to the pressure flow for the whole pipeline, a case that is not envisaged under normal conditions because of the large pipeline lengths, diameters and slopes. In all cases the transition occurs (and was modelled) through a moving interface that advances into the free surface portions of the system.

During rapid filling, a surge moving against the flow may develop a steep front but in cases of gradual filling, gentle slopes or depressurization, the surge may have a very smooth interface. Even if the interface is smooth, the transition between free-surface flow and pressurised flow cannot be continuous because the gravity wave speed as computed in Equation 2.28 would be infinite at the point of transition where $T = 0$ i.e. $c \to \infty$ as $T \to 0$ and this represents an area of abrupt flow change creating a discontinuity which gives the St. Venant equations their hyperbolic character. For this reason, it is always necessary to assume a discontinuity at the interface.

6.3.2 Key Determinants of Pressurisation

Pressurisation occurs at a rate proportional to the net head (Section 5) and discharge i.e difference between input discharge and output discharge (supply). It can also be reasonably expected that pressurisation can occur even when the pipe is not entirely full as long as the initial head and discharge can allow sufficiently rapid filling. To improve the rate of pipeline pressurisation and enable customers to get some water therefore, it makes sense to improve pipe discharge by limiting output discharge or withdrawals from the network (nodal demand) as well as flow in other links connecting to the node.This is due to the Principle of Conservation of Mass which implies that flow into the node equals flow out of the node. In Figure 54 for example, the principle is obeyed since node inflows are equated to node outflows (flow out of the node also includes withdrawals from the network). At the highlighted node in which 3.50 l/s is withdrawn from the network,

Flows out of the node (l/s)	= 3.50 + 9.41 + 11.66	= 24.57
Flows into the node (l/s)	= 3.84 + 20.73	= 24.57

It can be inferred therefore that if flow out of the node is controlled, this can improve the discharge in the input links.

Figure 54 also reveals that the summation of all nodal demands in the subsystem (withdrawals from the network, arrows indicate direction of flow, if negative sign is ignored) equals the flow from the tank into the system (Table 17) i.e. 56.55 l/s thus if external withdrawals are limited, pressurisation is facilitated and better pressures are obtained in the system. This can be done by, for example, closing off certain valves at different times of the day either completely or partially. Pressurisation also improves with the pipe diameter as observed and explained in Section 5.4.4.

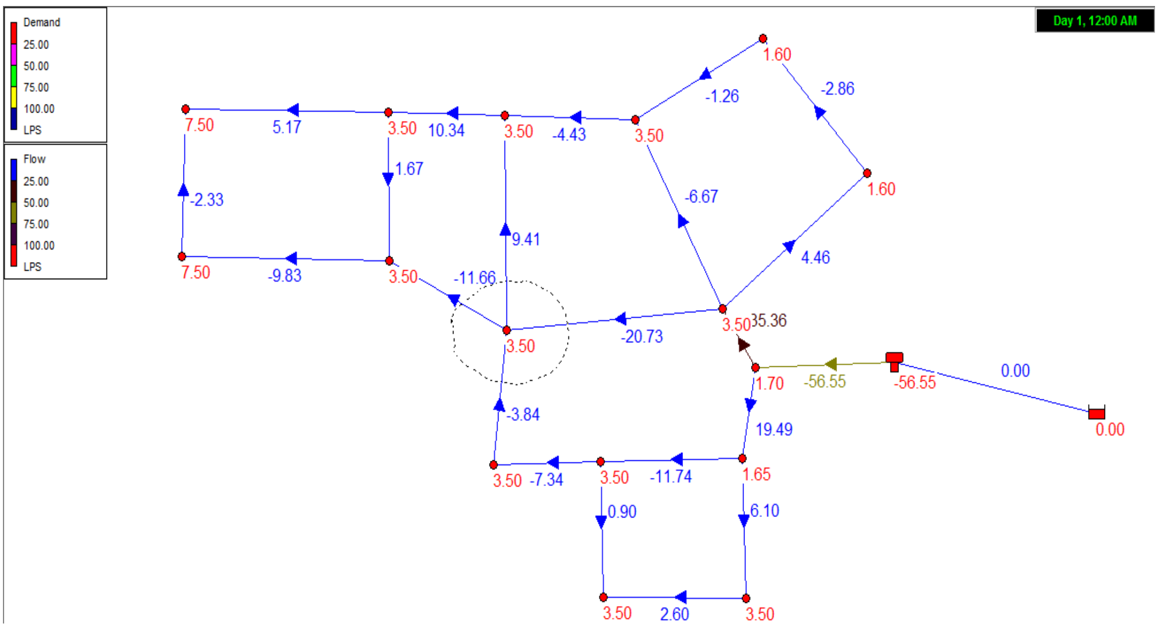

Figure 54: Link flows and Nodal demands in Rubaga subsystem

Table 17: Node Demand and system feeder tank withdrawals

Node ID	**Demand (l/s)**
2	1.7
14	3.5
20	3.5
31	3.5
36	1.65
12	3.5
13	3.5
3	3.5
4	3.5
7	1.6
8	1.6
9	3.5
10	3.5
11	3.5
15	7.5
16	7.5
Total	56.55
Tank Withdrawal	56.55

6.4 Operations and Management Decision Support Tool

The study developed an Operations and Management Decision Support Tool (OMDST) that would aid the operation and management of water supply systems across the continuum from normal pressures to low pressures to no pressure i.e. from full pressurized flow to partially full free-surface flow. The developed tool becomes a powerful asset for managing networks that often exhibit continuous transient flow conditions of a low-pressure-open-channel (LPOC) nature resulting from either inadequate supply that cannot meet the demand or an inadequate distribution network. The decision support system developed was tested on the Rubaga subsystem. It could be clearly observed that under demand-driven analysis, pressures dropped with increase in demand and even yielded negative pressures at the extreme.

Using head-driven analysis however it was demonstrated that some water could still be supplied at prevailing pressures.

At junction 16 the pressure required to satisfy a demand of 37.5 l/s is negative which is logically interpreted to mean that at this demand value no supply is possible at this node. This is computational result however in reality some water will come out of this node at a discharge less than 37.5 l/s, in proportion to the prevailing pressure at the node. Our interest therefore lies in determining how much water this node can supply and what other actions can be taken to alleviate the situation. If the threshold pressure head is predetermined at 10 m then it can be worked out that the demand that can be met at node16 is 9.73 l/s at midnight and 0.4 l/s at the peak hour. This provides much more accurate and meaningful information for real time management.

For a pipeline (node) which is unable to supply any water because it is under free surface flow, our interest lies in determining when the pipe will be pressurised so that pressures can start developing in order to push water out of the pipe. As was demonstrated in Section 5.4 of the previous chapter several actions can be taken such as:

i. Increase discharge and head in the pipes by limiting supply as discussed in Section 6.2.4 and Table 6.5. Water supply rationing which involves provision of water to different parts of the community at different times can be carried out. Water supply at certain nodes is totally cut off for some time in order to build pressures in other areas. This can be done in an orderly manner and accompanied by information, education and communication (IEC) campaigns against water wastage, reduced consumption can help improve service levels.

ii. Increase pipe size: This has the effect of increasing the rate of pressurisation in the pipe due to the reduction of head losses as discussed in Section 5 of Chapter 5.

It should be noted that ultimately, to improve water supply service provision, there should be an increase in discharge and head in the network by improving production and resizing mains i.e. replacing them with bigger mains as was demonstrated in Section 5.4. In the case of Kampala, inadequate pipe sizes are a problem as can be seen in the model and also noted by Mugisha (2010). In many places transmission mains are undersized and in other places the same mains laid 70 years ago are still serving without any additional mains. Added to this is the direct connection of consumers to transmission mains which drops the pressures in the mains (Figure 55). A good design is where dedicated transmission mains are used and any consumer connection is made from these. However, these solutions are not immediately possible in many poor countries due to the high capital investments required. In homes, the use of tanks that can fill during off peak hours for use during peak hours should also be encouraged. There is a lot of redundant pressure during off-peak hours that could supply water in the tanks for use during peak hours.

Figure 55: Inlet to the Rubaga Reservoir

The transmission main to the Rubaga Reservoir has a connection to the Cardinal's residence, Rubaga Hospital and Rubaga Cathedral! The inlet to the Rubaga Reservoir has a semi-permanent valve key that is used to close off water so that the connection upstream of the inlet can receive water to serve a Cathedral

6.5 Summary of Discussion

In the face of inadequate water supply and in order to provide equitable distribution of available water from constrained systems, intermittent water supplies with reduced system pressures are often introduced. In intermittent flow systems, consumers are forced to collect as much water as possible during the limited supply hours. The demand for water is not based on the notions of diurnal variations of usage but on the maximum quantity of water that can be collected during supply hours which in turn depends on the available pressure heads in the network. Modelling intermittent water supply systems is a challenging task because these systems are not fully pressurised networks but they are networks with very low pressures and restricted water supply hours per day. The alternate emptying and filling of water pipelines does not enable the application of standard hydraulic models because of low pressures and pipes which do not flow full. This ultimately means that a detailed hydraulic model of an intermittent water supply system should simulate the ‘charging’ process in pipes (Ingeduld 2006).

The research analysed water distribution systems facing normal pressures, low and nil pressures (atmospheric pressure). It is important to realise that in financially viable economies characterised by satisfactory water supply system performance, all customers are expected to receive the amount of water at the pressure that they applied for, thus pressure is a key performance indicator whose non fulfilment attracts penalties to the service provider. In less developed and water scarce regions pressure is a luxury with flow becoming the more critical performance indicator. In such regions it is good enough that some water can run through faucets. Focus then shifts from supplying water at pressures above a pre-established threshold value to supplying some water at all, at whatever pressure. In these areas the notion of “some for all rather than all for some” strongly holds. This means that while we cannot achieve the desired pressures, we need to maintain some flows to consumers, a challenge that this research has sought a overcome.

7.0 CONCLUSIONS AND RECOMMENDATIONS

The principal objective of the research was to understand the behaviour of water in distribution networks that face transient Low Pressure – Open Channel Flow Conditions (LPOCF) and to develop an operations and management decision support tool (OMDST) that could aid decision making in management of such conditions. The research demonstrated the usefulness of decision aid techniques in the understanding, quick identification and remediation of problems that arise in water distribution networks with special emphasis on the alleviation of no-water problems.

This Chapter presents conclusions arrived at during this research in satisfaction of set specific objectives and related findings/inferences. Recommendations for further study and restructuring/optimisation of the Kampala Water Supply System are also indicated.

7.1 Conclusions

i. It should be noted that ultimately, to improve water supply service provision; there should be appropriate management of discharge/production, head, and network configuration. In this consideration, network components such as pipelines, reservoirs and pumps should be well designed to optimise service provision.

ii. In water distribution networks, when pressure is high, water supplied does not need to depend on pressure but when pressure is low then the supply is dependent on the available pressure.

iii. Demand Driven Analysis (DDA) of water distribution networks produces inaccurate results when network pressures are low.

iv. The Pressure/Head Driven Approach (PDA/HDA) to water distribution modelling should be adopted when analysing networks subject to low pressure interludes and intermittent flows.

v. The network acts optimally as a system i.e. changes in parameters e.g. demand and pressure in a subsystem of the network will affect other parts of the network.

vi. Piped water supply is often analysed and modelled as pressurised full-bore flow but many a time water flows do not fill the entire pipeline due to low pressures (including free-surface open channel flow), inadequate delivery heads, and too many open takeoffs.

vii. In networks that exhibit intermittent flows, alternate emptying and filling of pipelines do not enable the application of standard hydraulic models because of low pressures and pipes which do not flow full.

viii. Pressurised and free-surface flow conditions can co-exist in the same network and pipelines temporally and spatially.

ix. Existing water distribution models cannot be used to simulate mixed flow because they deal exclusively with pressurised networks. Other hydraulic models consider either pressurised or free-surface/open channel flow conditions alone such as storm water and wastewater collection.

x. A network can dwell in a pressurisation state for a very long time, never getting fully pressurised, especially if the head is limited or the draw-off is large.

xi. Study of pipe-filling and pressurisation/depressurisation processes is key to understanding behaviour of water supply networks. Not only is the surge front location and propagation accurately predicted during the transient flow phase between free surface and pressurised flows but flow conditions as well.

xii. This research highlighted the importance of full dynamic models in the solution of transient Low Pressure – Open Channel Flow Conditions in water distribution networks, as the culprit in poor/no service delivery.

xiii. Unsteady flows in water supply occur not only as commonly known rapid transients (water hammer) but also as gradually varied flow (GVF).

xiv. Limiting supply through water supply rationing is one way of preserving pressures in networks.

With regard to the Kampala Water Supply Network, the following conclusions were made;

i. The subsystems are all interconnected and lack of delineation presents real difficulties in isolating systems for proper modelling and management control.

ii. The problem with Kampala is not production. The problem is an inadequate distribution network i.e. poorly sized reservoirs and pipelines.

iii. It is also important to note that proper explanation of system behaviour is currently hampered by the absence of a comprehensive model for the system which leads to guided network modifications.

iv. Data capture for important measurements to be used in modelling such as pressure in a number of points in the network, level variations in the tanks, pressures and flows in pumping stations and flows in a few main pipes in the network was mostly not available from on-line monitoring of the system. A large part of this information was either missing or incomplete and the most viable sources were the operator in the field and actual measurements. In the absence of all the required measuring equipment some knowledge was obtained in descriptive form.

7.2 Recommendations

The recommendations below have been categorised into two groups; recommendations from research and recommendations specific to the Kampala water supply system.

7.2.1 Global Research Recommendations

i. Use and development of pressure based water supply models should be encouraged.

ii. Inclusion of the study of partially - full flowing sections water supply network analysis greatly aids the improvement and management of pressure deficient water supply networks and should be incorporated in future water distribution models.

iii. In weak economies consideration should be given to designing low cost water supply networks herein termed as “low pressure systems” where priority is given to flows and not pressures, especially where the need for high pressures is not envisaged i.e. in areas where high pressures are not needed due to low levels of direct pressurised flow usage. These systems are cheaper to implement and operate. They offer pragmatic and most needed solutions to portable water supply shortfalls, yet their limitations do not adversely inconvenience the populations served. This means that current models in use which favour ‘high’ pressure systems will have to be replaced with mixed/full ‘low’ pressure models as have been formulated in this research.

iv. Further studies should target the formulation of models that tackle the dual character exhibited by several water supply systems i.e. co-existence of pressurised and free-surface flow conditions in the same network.

v. A detailed hydraulic model of the intermittent water supply system needs to simulate the ‘charging’ process in pipes. This would then help predict the exact time at which different users get water. This component is lacking in water distribution modelling and it is one of the objectives of this study.

vi. Full dynamic models should be used to analyse the free surface - pressurised flow co-existence in pipes in order to capture all data and get more accurate results.

vii. Proper modelling and management of a water supply system depends heavily on availability of data on network system parameters. Efforts should be made and monitoring points put in place as well data capture mechanisms instituted in order to collect and store valuable data for better network management and research purposes.

7.2.2 Performance Improvement of Kampala Network

i. The KWSN should be restructured i.e. increase reservoir capacities and pipeline sizes in order to carry the amount of water that can satisfy the demand of the City.

ii. Additional demand loadings on the network should be verified to ensure that they can be fulfilled by the network. Limiting supply through rationing can help preserve pressures in the network.

iii. More effort should be made to collect network hydraulic data in order to enhance management as mentioned above.

iv. Delineation of subsystems should be carried out in order to aid proper control of pressure and management. It is easier to deal with isolatable systems than a very big open system. For proper network management, systems analysis should be carried out prior to subsystems being created.

v. A good design should be where dedicated transmission mains are used for that purpose only and any consumer connections are made only from distribution mains from distribution reservoirs. In this way, reservoirs perform according to design.

8.0 REFERENCES

ALDRIGHETTI, E. & STELLING, G. 2006. A robust scheme for free surface and pressurised flows in channels with arbitrary cross sections. *Communications to SIMAI Congress,* 1:1-4.

AWWA. 2004. *Sizing water service lines and meters. Manual of water supply practices,* M22. Denver: AWWA.

ANG, W. K. & JOWITT P.W. 2006. Solution for water distribution systems under pressure-deficient conditions. *Journal of Water Resources Planning and Management*, 132(3): 175-182.

ARAUJO, L.S., COELHO, S.T., & RAMOS, H.M., 2003. Estimation of distributed pressure-dependent leakage and consumer demand in water supply networks. *Proceedings of the International Conference on Advances in Water Supply Management.* London.

ARSENE, C.T.C., BARGIELA, A. & AL-DABASS, D. 2002. Modelling and simulation of water systems based on loop equations. *International Journal of Simulation.* 5 (1): 61-72.

AXWORTHY, D.H. 1997. Water distribution network modelling: From steady state to water hammer. PhD Thesis. University of Toronto. Canada.

BAHADUR, R., JOHNSON, J., JANKE, R. & SAMUELS, W. B. 2006. Impact of model skeletonisation on water distribution model parameters as related to water quality and contaminant consequence assessment. *8th Annual Water Distribution Systems Analysis Symposium.* Ohio.

BASU, S. R. & MAIN, H. A. C. 2001. Calcutta's water supply: demand, governance and environmental change. *Applied Geography*, 21 (1).

BHAVE, P.R. 1991. *Analysis of flow in water distribution networks*. Lancaster : Technomic Publishing.

BISWAS, A. K. & SEETHARAM, K. E. 2008. Achieving water security for Asia. *International Journal of Water Resources Development,* 24(1).

BISWAS, A. K., 1976. *Systems Approach to Water Management.* New York: McGraw-Hill.

BLAIS, J.P., GATTO, L., BOURDARIOUS, C. & GERBI, S. 2006. Numerical modelling of mixed flows in hydroelectric schemes. *La Houille Blanche*, 1: 76-81.

BOULOS, P. F., KARNEY, B. W., WOOD, D. J. & LINGIREDDY, S. 2005. Hydraulic transient guidelines for protecting water distribution systems. *Journal of American Water Works Association,* 97(5): 111-124.

BOURDARIAS, C. & GERBI S. 2007. A finite volume scheme for a model coupling free surface and pressurised flows in pipes. *Journal of Computational and Applied Mathematics,* 209(1): 109-131.

BOURDARIAS, C. & GERBI, S. 2008. A kinetic formulation for a model coupling free surface and pressurised flows in closed pipes*. Journal of Computational and Applied Mathematics,* 218(2): 522-531.

BOURDARIUS, C. & GERBI, S. 2009. A kinetic scheme for unsteady pressurised flows in closed water pipes. *Universite de Savoie.* France.

BURROWS, R., MULREID, G., HAYUTI, M., ZHANG, J. & CROWDER, G. 2003. Introduction of a fully dynamic representation of leakage into network modelling studies using epanet. *In* Maksimovic, C., Butler, D. & Memon, F.A., eds. *Advances in Water Supply Management*: 109-118.

BUTLER, D. & DAVIES. J.W. 2004. *Urban Drainage.* UK: Taylor & Francis.

CHADWICK, A., MORFETT J. & BORTHWICK M. 2004. *Hydraulics in Civil and Environmental Engineering.* UK: Spon Press.

CHANDAPILLAI, J. 1991. Realistic Simulation of Water Distribution Systems. *Journal of Transportation Engineering,* 117(2): 258-263.

CHAUDRY, M.H. 1979. *Applied Hydraulic Transients.* New York: Van Nostrand Renhold Co.

CHEUNG, P.B., VAN ZYL, J.E. & REIS, L.F.R. 2005. Extension of Epanet for Pressure Driven Demand Modelling in Water Distribution System *CCWI2005 Water Management for the 21st Century, Exeter, UK.*

CHOU, T. 2009. The Method of Characteristics. University of California. Los Angeles.

CHOW, V.T. 1959. *Open-channel hydraulics.* New York: McGraw-Hill.

CUNGE, J.A., HOLLY, F.M. & VERWEY, A. 1980. *Practical aspects of computational river hydraulics.* London: Pitman Advanced Publishing Program. Cited by Osman (2006).

CUNGE, J.A. & WEGNER, M. 1964. Numerical Integration of Bane de Saint Venant's Flow Equations by Means of an Implicit Scheme of Finite Differences. Applications in the Case of Alternately Free and Pressurised Flow in a Tunnel. *La Houille Blanche* (1).

DUCHESNE, S., MAILHOT, A., DEQUIDT, E. & VILLENEUVE, J.P. 2001. Mathematical modelling of sewers under surcharge for real time control of combined sewer overflows*. Urban Water,* 3(4): 241-252.

FLEMING, K.K., GULLICK R.W., DUGANDZIC J.P. & LECHEVALLIER M.W. 2006. Susceptibility of Potable Water Distribution Systems to Negative Pressure Transients.

Department of Environmental Protection, Div. Science, Research and Technology. Trenton, NJ and American Water, Voorhees, NJ.

FRANZ, D.D. & MELCHING, C.S. 1997a. Full Equations (FEQ) model for the solution of the full, dynamic equations of motion for one-dimensional unsteady flow in open channels and through control structures. *U.S. Geological Survey Water-Resources Investigations Report.*

FRANZ, D.D. & MELCHING, C.S. 1997b. Full Equations Utilities (FEQUTL) model for the approximation of hydraulic characteristics of open channels and control structures during unsteady flow. *U.S. Geological Survey Water-Resources Investigations Report.*

FRIEDMAN, M., L. RADDER, S. HARRISON, D. HOWIE, M. BRITTON, G. BOYD, H. WANG, R. GULLICK, M. LECHEVALLIER, WOOD, D. & FUNK, J. 2004. Verification and Control of Low Pressure Transients in Distribution Systems. *American Water Works Association Research Foundation (AWWARF), Denver, CO.*

FUAMBA, M. 2003. Contribution on Transient Flow Modelling in Storm Sewers. *Journal of Hydraulic Research,* 40(6): 685-693.

GOMEZ, M. & ACHIAGA, V. 2008. Mixed Flow Modelling Produced by Pressure Fronts from Upstream and Downstream Extremes. *Proc. ASCE Conference, USA.*

GULLICK, R. W., LECHEVALLIER M. W., CASE J., WOOD D. J., FUNK J. E. & FRIEDMAN M. J. 2005. Application of pressure monitoring and modeling to detect and minimize low pressure events in distribution systems. *JWSRT – Aqua,* 54 (2): 65-81.

GUPTA. R. & BHAVE, P.R. 1996. Comparison of methods for predicting deficient network performance. *Journal of Water Resources Planning and Management. ASCE,* 123(6): 369-370.

HAESTAD METHODS. 2003. *Advanced Water Distribution Modelling and Management.* Waterbury: Haestad Press.

HAHN, B.D. & VALENTINE, D.T. 2006. *Essential MATLAB for Engineers and Scientists.* Oxford: Butterworth and Heinemann.

HAMAM, M. A. & MCCORQUODALE, J. A. 1982. Transient conditions in the transition from gravity to surcharged sewer flow. *Canadian Journal of Civil Engineering,* 9: 189-196.

HAYUTI M. H., BURROWS R. & NAGA D. 2007. Modelling water distribution systems with deficient pressure. *Water Management,* 160: 215-224.

HELSINKI UNIVERSITY OF TECHNOLOGY. 2003. Laboratory of Water Resources.

HUANG, Y. & FIPPS, G. 2003. Modelling flows in irrigation distribution networks-model description and prototype. *American Society of Agricultural Engineers Annual International Meeting*, Nevada, USA.

INGEDULD, P., SVITAK, Z., PRADHAN, A. & TARAI, A. 2006. Modelling intermittent water supply systems with EPANET. *8th Annual WD Symposium EPA Cincinnati.* August.

JAYARAM, N. 2006 .Reliability Based Optimisation of Water Distribution Networks. MTech Thesis. Indian Institute of Technology. Madras.

KALUNGI, P. & TANYIMBOH T.T. 2003. Redundancy model for water distribution systems. *Reliab Engng Syst Safety,* 82(3): 275-286.

KARNEY, B., PARENTE A., EERKES E. & WHITE C. 2008. Assessing performance of a water transmission system using an inverse transient method. *Proceedings of Pipelines Congress.* Atlanta GA.

KATIKA, K.M. & PILON, M. 2005. Modified method of characteristics in transient radiation transfer. University of California. Los Angeles.

KHATRI, K.B. & VAIRAVAMOORTHY, K. 2007. Challenges for urban water supply and sanitation in developing countries. Discussion draft paper for the Session on Urbanisation. UNESCO-IHE. The Netherlands

KIZITO, F. 2009. Water supply management in an urban utility: a prototype decision support framework. PhD Thesis. Royal Institute of Technology, Sweden. Makerere University, Uganda.

KRITPIPHAT, W., TONTIWACHWUTHIKUL P. & CHAN C. W. 1998. Pipeline network modelling and simulation for intelligent monitoring and control: a case study of a municipal water supply system. *Industrial & Engineering Chemistry Research,* 37(3): 1033-1044.

LAMADDALENA, N. & PEREIRA, L.S. 2007. Pressure-driven modelling for performance analysis of irrigation systems operating on demand, *Agric. Water Manage.* 90 (1): 36–44.

LEE, E.J. & SCHWAB, K.J. 2005. Deficiencies in drinking water distribution systems in developing countries. J*ournal of Water and Health,* 3(2): 109-127.

LEON, A.S. 2007. Improved modelling of unsteady free surface, pressurised and mixed flows in storm sewer systems. PhD Thesis. University of Illinois at Urbana-Champaign.

LEON, A.S., GHIDAOUI, M.S., SCHMIDT, A. R. & GARCIA, M.H. 2010. A robust two equation model for transient mixed flows. *Journal of Hydraulic Research,* 48(1): 44-56.

LI, J., & MCCORQUODALE, A. 1999. Modelling mixed flow in storm sewers. *J. Hydraul. Eng.,* 125(11): 1170-1180.

LIOVIC, P., RUDMAN, M. & LIOW, J.L. 1999. Numerical modelling of free surface flows in metallurgical vessels. *Second International Conference on CFD in the minerals and process industries.* Melbourne. Australia.

MACHELL, J., MOUNCE, S.R. & BOXALL, J.B. 2010. Online modelling of water distribution systems: a UK case study. *Drink. Water Eng. Sci.,* 3(1): 21-27.

MARRIOTT, M. 2009. *Civil Engineering Hydraulics.* 5th Ed.Oxford: Wiley-Blackwell.

MAYS, L.W. 2004. Water supply systems security. New York: McGraw-Hill.

MARTÍNEZ-SOLANO, J., IGLESIAS-REY, P.L., PÉREZ-GARCÍ &, LÓPEZ-JIMÉNEZ, P.A. 2008. Hydraulic analysis of peak demand in looped water distribution networks. *Journal of Water Resources Planning and Management,* 134(6): 504-510.

MATHWORKS. 2007. Matlab. Computer Program. Natick. Massachusetts.

MUGISHA, F. 2010. Water distribution modelling and decision support tools for Kampala Water network operation and management: Experiences, challenges and future perspectives. *15th International Africa Water Conference.* Kampala.

NATIONAL RESEARCH COUNCIL. WATER SCIENCE AND TECHNOLOGY BOARD. 2006. *Drinking water distribution systems: assessing and reducing risks.* Washington, DC : National Academies Press.

NDAMBUKI, J.M. 2001. Multi-objective groundwater quantity management: a stochastic approach. Delft University Press. The Netherlands.

NGIRANE-KATASHAYA, G., KIZITO, F. & THUNVIK, R. 2006. Decision support systems for water resources management in Uganda: the need. *Proceedings of the 32nd WEDC International Conference.* Colombo, Sri Lanka.

NYENDE-BYAKIKA, S. 2006. A study of the hydraulic impact of water supply network expansions. MSc Thesis. Makerere University. Kampala

NYENDE-BYAKIKA, S., NGIRANE-KATASHAYA G. & NDAMBUKI, J.M. 2010. Behaviour of stretched water supply networks. *Nile Water Science and Engineering Journal,* 3(1): 51-60.

NWSC. 2006/2007. *Annual Report.* Kampala.

OBRADOVI, D. 2000. Modelling of demand and losses in real life water distribution systems. *Urban Water.* 2(2): 131-139.

OSMAN, A.A. 2006. *Open Channel Hydraulics.* UK: Butterworth-Heinemann.

OZGER, S. 2003. A semi-pressure driven approach to reliability assessment of water distribution networks. PhD Thesis. Arizona State University. USA

PILLER, O., BRÉMOND, B. & POULTON, M. 2003. Least action principles appropriate to pressure driven models of pipe networks. *Proc. ASCE conference, Philadelphia, USA.*

POLITANO M., ODGAARD J. & KLECAN W. 2005. Numerical Simulation of Hydraulic Transients in Drainage Systems. *Mecanica Computacional,* XXIV . Argentina.

ROSENBERG, D.E. 2008. Integrated water management and modelling at multiple spatial scales. PhD Thesis. University of California, Davis. USA.

ROSSMAN A. L. 2000. EPANET Users' Manual. *National Risk Management Laboratory.* United States Environmental Protection Agency, Ohio.

SEVUK, A.S. 1973. Unsteady flow in sewer networks. PhD Thesis. Department of Civil Engineering, University of Illinois, Urbana, Il. Cited by Osman (2006).

SCHMITT, T.G., THOMAS, M. & ETTRICH, N. 2004. Analysis and modelling of flooding in urban drainage systems. *Journal of Hydrology,* 299(3): 300-311.

SONG, C.C.S., CARDLE. J.A. & LEUNG, K.S. 1983. Transient mixed flow models for storm sewers. *Journal of Hydraulic Engineering,* 109(11): 1487–1504.

STURM, T. 2001. *Open Channel Hydraulics.* 1st ed. New York: McGraw Hill.

TABESH, M. & KARIMZADEH, A. 2000. Optimum design of reliable distribution systems considering pressure dependency of outflows. *In* Savic, D. A. & Walters, G. A., eds. *Water network modelling for optimal design and management*: 211-220

TABESH, M., TANYIMBOH, T.T. & BURROWS, R. 2001. Head Driven Simulation Based Reliability Assessment of Water Supply Networks. *Proceedings of the World Water and Environmental Resources Congress,* ASCE, D. Phelps and G. Sehlke (eds.), Orlando, Florida.

TANYIMBOH, T.T. 2000. A quantified assessment of the relationship between the reliability and entropy of water distribution systems. *Engineering Optimisation,* 22(2): 179-199.

TANYIMBOH, T.T. & TABESH, M. 1997. Discussion of comparison of methods for predicting deficient network performance. *Journal of Water Resources Planning and Management. ASCE,* 123(6): 369-370.

TANYIMBOH, T.T. & TEMPLEMAN A.B. 2007. A new nodal outflow function for head-dependent modelling. *Adv Eng Softw.* www.sciencedirect.com.

TANYIMBOH, T.T., BURD R., BURROWS, R. & TABESH, M. 1999. Modelling and reliability analysis of water distribution systems. *Water Science Tech.,* 39(4): 249-255.

TODINI, E. 2003. A more realistic approach to the extended period simulation of water distribution networks. *In* Maksimovic, C., Butler, D. & Memon, F. A., eds. *Advances in Water Supply Management*: 173-183.

TODINI, E. & PILATI, S. 1987. A gradient method for the solution of looped pipe networks. *International Conference on Computer Applications in Water Supply and Distribution.* Leicester Polytechnic, UK.

TRAJKOVIC, B., IVETIC M., CALOMINO F. & DIPPOLITO A. 1999. Investigation of transition from free surface to pressurized flow in a circular pipe. *Water Science and Technology,* 39(9): 105-112.

TUCCIARELLI, T. 2003. A new algorithm for a robust solution of the fully dynamic St. Venant equations. *Journal of Hydraulic Research.* 41(3): 239-246.

TULLIS, J.P. 1989. *Hydraulics of pipelines, pumps, valves, cavitation, transients.* New York: John Wiley & Sons.

VAIRAVAMOORTHY, K. 2008. Cities of the future and urban water management. Paper presented during Thematic Week 2: Water and City. Water Tribune – Zaragoza International Exhibition.

VAIRAVAMOORTHY, K., GORANTIWAR, S.D. & PATHIRANA, A. 2008. Managing urban water supplies in developing countries – Climate change and water scarcity scenarios. *Physics and Chemistry of the Earth,*33(5): 330-339.

VAN ZYL, J. E., BORTHWICK, J. & HARDY, A. 2003. OOTEN: an object-oriented programmers toolkit for epanet. *Advances in water supply management,* supplementary paper.

VASCONCELOS, J. G. & WRIGHT, S. J. 2004. Numerical modelling of the transition between free surface and pressurised flow in storm sewers. *In* W. James, W., ed. *Innovative Modeling of Urban Water Systems.* Monograph 12, CHI Publications, Ontario, Canada.

VASCONCELOS, J.G., 2005. Dynamic approach to the description of flow regime transition in storm water systems. PhD Thesis. University of Michigan.

VASCONCELOS, J.G., WRIGHT, S. J. & ROE, P.L. 2006. Improved simulation of flow regime transition in sewers: two-component pressure approach. *J. Hydraul. Eng.*, 132(6): 553-562.

WIGGERT, D.C. 1983. Transient flow in free-surface, pressurised systems. *Journal of the Hydraulics Division, ASCE,* 98, No. HY1, 1972. Cited by Song.

WU, Z. Y., WANG, R.H., WALSKI, T.M., YANG S.Y., BOWDLER D. & BAGGETT C.C. 2006. Efficient pressure dependent demand model for large water distribution system analysis. *8th Annual Symposium on Water Distribution System Analysis.* Ohio.

WU, Z. Y., WANG R.H., WALSKI T.M., YANG S.Y., BOWDLER D. & BAGGETT C.C. 2009. Extended global-gradient algorithm for pressure-dependent water distribution analysis. *Journal of Water Resources Planning and Management,* 134(1):13-22.

WYLIE, E.B. & STREETER,V.L. 1978. *Fluid transients.* New York : McGraw Hill.

YEN, B.C. 1986. Hydraulics of sewers. *In* Yen, B.C., ed. *Advances in HydroScience.* New York: Academic Press, 14.

YEN, B. C. 2001. *Hydraulics of sewer systems in stormwater collection systems.* New York: McGraw-Hill.

ZHANG, B. & LERNER, D.N. 2000. An integrated groundwater, pipe and open channel flow model for simulation of aquifer-audit systems. In *Calibration and Reliability in ground water modelling.* (Proceedings of the modelCARE 99 Conference held at Zurich, Switzerland), IAHS 265.

ZHENG, Y. W., RONG, H. W., THOMAS, M. W., SHAO, Y. Y., BOWDLER, D. & BAGGETT C.C. 2009. Extended global-gradient algorithm for pressure-dependent water distribution analysis. *Journal of Water Resources Planning and Management,* 135(1): 13-22

ZYOUD, S.H.A.R. 2003. Hydraulic performance of Palestinian water distribution systems. MSc Thesis. An-Najah National University. Palestine.

Author's Biography

Stephen Nyende Byakika was born on 9 November 1980. He attended St. Mary's College Kisubi in Uganda from where he joined Makerere University Kampala, where he pursued a Bachelors Degree in Civil Engineering, graduating in 2003 with an Upper Second Class Honours Degree. He further graduated with a Master of Science Degree in Civil Engineering from the same university in 2007. Stephen has a Post Graduate Diploma in Project Planning and Management from the Uganda Management Institute (2008) and a Graduate Diploma of the Chartered Institute of Purchasing and Supply (CIPS) 2008.

Stephen initially worked as a trainee engineer in the Department of Water Resources Management in Uganda and in November 2004, joined the National Water and Sewerage Corporation of Uganda as a Zone Engineer before he was transferred to the Department of Planning and Capital Development, responsible for the planning and implementation of all capital development projects of the Corporation. As a zone engineer Stephen was responsible for the operation and maintenance of the water supply network in the zone. While in the Department of Planning and Capital Development, Stephen was Senior Engineer and Procurement Officer in charge of the initiation, study and designs, procurement and supervision of water supply and sewerage projects of the Corporation. Stephen later joined Lafarge East Africa (Hima, Bamburi and Mbeya Cement) where he was in charge of the supply of cement and all associated technical support to the World Bank funded Bujagali Hydro Power Project.

Stephen joined the Vaal University of Technology in South Africa at the start of 2009 to pursue doctoral studies. He has taught several courses in Civil Engineering at the university.

APPENDIX

Appendix A: Kampala Water Distribution Network Map

Appendix B: Sample Pressure Test Data

Site: Japanese Embassy in Kampala

This is a sample of Pressure Test results obtained.

Complete results may be obtained from www.nwsc.co.ug

Date	Time	Pressure (m)	Date	Time	Pressure (m)
4/17/2009	10:59:00	3.43	4/17/2009	11:18:00	3.36
4/17/2009	10:59:30	3.44	4/17/2009	11:18:30	3.36
4/17/2009	11:00:00	3.39	4/17/2009	11:19:00	3.36
4/17/2009	11:00:30	3.35	4/17/2009	11:19:30	3.34
4/17/2009	11:01:00	3.36	4/17/2009	11:20:00	3.42
4/17/2009	11:01:30	3.36	4/17/2009	11:20:30	3.42
4/17/2009	11:02:00	3.35	4/17/2009	11:21:00	3.43
4/17/2009	11:02:30	3.34	4/17/2009	11:21:30	3.43
4/17/2009	11:03:00	3.34	4/17/2009	11:22:00	3.4
4/17/2009	11:03:30	3.32	4/17/2009	11:22:30	3.28
4/17/2009	11:04:00	3.32	4/17/2009	11:23:00	3.27
4/17/2009	11:04:30	3.34	4/17/2009	11:23:30	2.85
4/17/2009	11:05:00	3.3	4/17/2009	11:24:00	3.14
4/17/2009	11:05:30	3.34	4/17/2009	11:26:30	3.07
4/17/2009	11:06:00	3.33	4/17/2009	11:27:00	3.08
4/17/2009	11:06:30	3.37	4/17/2009	11:27:30	3.1
4/17/2009	11:07:00	3.37	4/17/2009	11:28:00	3.09
4/17/2009	11:07:30	3.36	4/17/2009	11:28:30	3.08
4/17/2009	11:08:00	3.35	4/17/2009	11:29:00	3.05
4/17/2009	11:08:30	3.3	4/17/2009	11:29:30	3.12
4/17/2009	11:09:00	3.37	4/17/2009	11:30:00	3.11
4/17/2009	11:09:30	3.36	4/17/2009	11:30:30	3.09
4/17/2009	11:10:00	3.36	4/17/2009	11:31:00	3.09
4/17/2009	11:10:30	3.29	4/17/2009	11:31:30	3.01
4/17/2009	11:11:00	3.3	4/17/2009	11:32:00	2.99
4/17/2009	11:11:30	3.32	4/17/2009	11:32:30	2.95
4/17/2009	11:12:00	3.32	4/17/2009	11:33:00	2.91
4/17/2009	11:12:30	3.32	4/17/2009	11:33:30	2.88
4/17/2009	11:13:00	3.36	4/17/2009	11:34:00	2.87
4/17/2009	11:13:30	3.32	4/17/2009	11:34:30	2.86
4/17/2009	11:14:00	3.37	4/17/2009	11:35:00	2.84
4/17/2009	11:14:30	3.39	4/17/2009	11:35:30	2.94
4/17/2009	11:15:00	3.37	4/17/2009	11:36:00	2.9
4/17/2009	11:15:30	3.37	4/17/2009	11:36:30	2.88
4/17/2009	11:16:00	3.35	4/17/2009	11:37:00	2.82

Complete results may be obtained from www.nwsc.co.ug

Date	Time	Pressure (m)	Date	Time	Pressure (m)
4/17/2009	11:16:30	3.39	4/17/2009	11:37:30	2.84
4/17/2009	11:17:00	3.35	4/17/2009	11:38:00	2.78
4/17/2009	11:17:30	3.3	4/17/2009	11:38:30	2.78
4/17/2009	11:39:00	2.83	4/17/2009	12:02:00	2.65
4/17/2009	11:39:30	2.77	4/17/2009	12:02:30	2.6
4/17/2009	11:40:00	2.79	4/17/2009	12:03:00	2.77
4/17/2009	11:40:30	2.67	4/17/2009	12:03:30	2.75
4/17/2009	11:41:00	2.65	4/17/2009	12:04:00	2.75
4/17/2009	11:41:30	2.65	4/17/2009	12:04:30	2.73
4/17/2009	11:42:00	2.64	4/17/2009	12:05:00	2.74
4/17/2009	11:42:30	2.77	4/17/2009	12:05:30	2.74
4/17/2009	11:43:00	2.74	4/17/2009	12:06:00	2.75
4/17/2009	11:43:30	2.72	4/17/2009	12:06:30	2.75
4/17/2009	11:44:00	2.72	4/17/2009	12:07:00	2.73
4/17/2009	11:44:30	2.71	4/17/2009	12:07:30	2.75
4/17/2009	11:45:00	2.7	4/17/2009	12:08:00	2.75
4/17/2009	11:45:30	2.72	4/17/2009	12:08:30	2.75
4/17/2009	11:46:00	2.73	4/17/2009	12:09:00	2.74
4/17/2009	11:46:30	2.74	4/17/2009	12:09:30	2.74
4/17/2009	11:47:00	2.73	4/17/2009	12:10:00	2.72
4/17/2009	11:47:30	2.72	4/17/2009	12:10:30	2.74
4/17/2009	11:48:00	2.72	4/17/2009	12:11:00	2.72
4/17/2009	11:48:30	2.73	4/17/2009	12:11:30	2.72
4/17/2009	11:49:00	2.7	4/17/2009	12:12:00	2.75
4/17/2009	11:49:30	2.75	4/17/2009	12:12:30	2.75
4/17/2009	11:50:00	2.76	4/17/2009	12:13:00	2.75
4/17/2009	11:50:30	2.78	4/17/2009	12:13:30	2.76
4/17/2009	11:51:00	2.79	4/17/2009	12:14:00	2.76
4/17/2009	11:51:30	2.78	4/17/2009	12:14:30	2.75
4/17/2009	11:52:00	2.77	4/17/2009	12:15:00	2.77
4/17/2009	11:52:30	2.8	4/17/2009	12:15:30	2.77
4/17/2009	11:53:00	2.79	4/17/2009	12:16:00	2.77
4/17/2009	11:53:30	2.8	4/17/2009	12:16:30	2.62
4/17/2009	11:54:00	2.79	4/17/2009	12:17:00	2.62
4/17/2009	11:54:30	2.79	4/17/2009	12:17:30	2.61
4/17/2009	11:55:00	2.8	4/17/2009	12:18:00	2.56
4/17/2009	11:55:30	2.78	4/17/2009	12:18:30	2.54
4/17/2009	11:56:00	2.79	4/17/2009	12:19:00	2.56
4/17/2009	11:56:30	2.79	4/17/2009	12:19:30	2.55
4/17/2009	11:57:00	2.79	4/17/2009	12:20:00	2.37

Complete results may be obtained from www.nwsc.co.ug

Date	Time	Pressure (m)
4/17/2009	11:57:30	2.78
4/17/2009	11:58:00	2.76
4/17/2009	11:58:30	2.72
4/17/2009	11:59:00	2.62
4/17/2009	11:59:30	2.72
4/17/2009	12:00:00	2.64
4/17/2009	12:00:30	2.77
4/17/2009	12:01:00	2.76
4/17/2009	12:01:30	2.75
4/17/2009	12:25:00	1.61
4/17/2009	12:25:30	2.46
4/17/2009	12:26:00	2.54
4/17/2009	12:26:30	2.38
4/17/2009	12:27:00	2.48
4/17/2009	12:27:30	2.32
4/17/2009	12:28:00	2.53
4/17/2009	12:28:30	2.56
4/17/2009	12:29:00	2.74
4/17/2009	12:29:30	2.61
4/17/2009	12:30:00	2.76
4/17/2009	12:30:30	2.75
4/17/2009	12:31:00	2.78
4/17/2009	12:31:30	2.77
4/17/2009	12:32:00	2.78
4/17/2009	12:32:30	2.79
4/17/2009	12:33:00	2.8
4/17/2009	12:33:30	2.79
4/17/2009	12:34:00	2.79
4/17/2009	12:34:30	2.76
4/17/2009	12:35:00	2.8
4/17/2009	12:35:30	2.78
4/17/2009	12:36:00	2.75
4/17/2009	12:36:30	2.74
4/17/2009	12:37:00	2.75
4/17/2009	12:37:30	2.75
4/17/2009	12:38:00	2.75
4/17/2009	12:38:30	2.73
4/17/2009	12:39:00	2.75
4/17/2009	12:39:30	2.76
4/17/2009	12:40:00	2.76
4/17/2009	12:20:30	2.35
4/17/2009	12:21:00	2.33
4/17/2009	12:21:30	2.35
4/17/2009	12:22:00	2.68
4/17/2009	12:22:30	2.69
4/17/2009	12:23:00	2.65
4/17/2009	12:23:30	2.65
4/17/2009	12:24:00	2.39
4/17/2009	12:24:30	2.41
4/17/2009	12:48:00	2.61
4/17/2009	12:48:30	2.77
4/17/2009	12:49:00	2.73
4/17/2009	12:49:30	2.74
4/17/2009	12:50:00	2.6
4/17/2009	12:50:30	2.57
4/17/2009	12:51:00	2.71
4/17/2009	12:51:30	2.71
4/17/2009	12:52:00	2.71
4/17/2009	12:52:30	2.72
4/17/2009	12:53:00	2.72
4/17/2009	12:53:30	2.7
4/17/2009	12:54:00	2.72
4/17/2009	12:54:30	2.71
4/17/2009	12:55:00	2.71
4/17/2009	12:55:30	2.7
4/17/2009	12:56:00	2.7
4/17/2009	12:56:30	2.72
4/17/2009	12:57:00	2.72
4/17/2009	12:57:30	2.72
4/17/2009	12:58:00	2.72
4/17/2009	12:58:30	2.72
4/17/2009	12:59:00	2.73
4/17/2009	12:59:30	2.72
4/17/2009	13:00:00	2.72
4/17/2009	13:00:30	2.71
4/17/2009	13:01:00	2.76
4/17/2009	13:01:30	2.72
4/17/2009	13:02:00	2.72
4/17/2009	13:02:30	2.71
4/17/2009	13:03:00	2.69

Complete results may be obtained from www.nwsc.co.ug

Date	Time	Pressure (m)
4/17/2009	12:40:30	2.75
4/17/2009	12:41:00	2.76
4/17/2009	12:41:30	2.63
4/17/2009	12:42:00	2.64
4/17/2009	12:42:30	2.67
4/17/2009	12:43:00	2.38
4/17/2009	12:43:30	2.69
4/17/2009	12:44:00	2.46
4/17/2009	12:44:30	2.55
4/17/2009	12:45:00	2.78
4/17/2009	12:45:30	2.8
4/17/2009	12:46:00	2.7
4/17/2009	12:46:30	2.78
4/17/2009	12:47:00	2.77
4/17/2009	12:47:30	2.7
4/17/2009	13:11:00	2.66
4/17/2009	13:11:30	2.67
4/17/2009	13:12:00	2.67
4/17/2009	13:12:30	2.67
4/17/2009	13:13:00	2.68
4/17/2009	13:13:30	2.69
4/17/2009	13:14:00	2.7
4/17/2009	13:14:30	2.69
4/17/2009	13:15:00	2.69
4/17/2009	13:15:30	2.69
4/17/2009	13:16:00	2.68
4/17/2009	13:16:30	2.68
4/17/2009	13:17:00	2.68
4/17/2009	13:17:30	2.68
4/17/2009	13:18:00	2.68
4/17/2009	13:18:30	2.7
4/17/2009	13:19:00	2.7
4/17/2009	13:19:30	2.73
4/17/2009	13:20:00	2.71
4/17/2009	13:20:30	2.72
4/17/2009	13:21:00	2.7
4/17/2009	13:21:30	2.7
4/17/2009	13:22:00	2.71
4/17/2009	13:22:30	2.7
4/17/2009	13:23:00	2.7

Date	Time	Pressure (m)
4/17/2009	13:03:30	2.67
4/17/2009	13:04:00	2.69
4/17/2009	13:04:30	2.14
4/17/2009	13:05:00	2.7
4/17/2009	13:05:30	2.7
4/17/2009	13:06:00	2.71
4/17/2009	13:06:30	2.7
4/17/2009	13:07:00	2.7
4/17/2009	13:07:30	2.7
4/17/2009	13:08:00	2.7
4/17/2009	13:08:30	2.69
4/17/2009	13:09:00	2.67
4/17/2009	13:09:30	2.64
4/17/2009	13:10:00	2.64
4/17/2009	13:10:30	2.65
4/17/2009	13:34:00	2.64
4/17/2009	13:34:30	2.65
4/17/2009	13:35:00	2.61
4/17/2009	13:35:30	2.6
4/17/2009	13:36:00	2.59
4/17/2009	13:36:30	2.58
4/17/2009	13:37:00	2.55
4/17/2009	13:37:30	2.56
4/17/2009	13:38:00	2.57
4/17/2009	13:38:30	2.58
4/17/2009	13:39:00	2.6
4/17/2009	13:39:30	2.61
4/17/2009	13:40:00	2.63
4/17/2009	13:40:30	2.64
4/17/2009	13:41:00	2.65
4/17/2009	13:41:30	2.64
4/17/2009	13:42:00	2.65
4/17/2009	13:42:30	2.65
4/17/2009	13:43:00	2.64
4/17/2009	13:43:30	2.64
4/17/2009	13:44:00	2.64
4/17/2009	13:44:30	2.64
4/17/2009	13:45:00	2.65
4/17/2009	13:45:30	2.65
4/17/2009	13:46:00	2.65

Complete results may be obtained from www.nwsc.co.ug

Date	Time	Pressure (m)	Date	Time	Pressure (m)
4/17/2009	13:23:30	2.71	4/17/2009	13:46:30	2.64
4/17/2009	13:24:00	2.68	4/17/2009	13:47:00	2.66
4/17/2009	13:24:30	2.69	4/17/2009	13:47:30	2.67
4/17/2009	13:25:00	2.68	4/17/2009	13:48:00	2.66
4/17/2009	13:25:30	2.67	4/17/2009	13:48:30	2.67
4/17/2009	13:26:00	2.67	4/17/2009	13:49:00	2.67
4/17/2009	13:26:30	2.67	4/17/2009	13:49:30	2.65
4/17/2009	13:27:00	2.65	4/17/2009	13:50:00	2.65
4/17/2009	13:27:30	2.67	4/17/2009	13:50:30	2.65
4/17/2009	13:28:00	2.68	4/17/2009	13:51:00	2.63
4/17/2009	13:28:30	2.67	4/17/2009	13:51:30	2.64
4/17/2009	13:29:00	2.67	4/17/2009	13:52:00	2.56
4/17/2009	13:29:30	2.67	4/17/2009	13:52:30	2.55
4/17/2009	13:30:00	2.69	4/17/2009	13:53:00	2.56
4/17/2009	13:30:30	2.67	4/17/2009	13:53:30	2.57
4/17/2009	13:31:00	2.65	4/17/2009	13:54:00	2.56
4/17/2009	13:31:30	2.58	4/17/2009	13:54:30	2.62
4/17/2009	13:32:00	2.63	4/17/2009	13:55:00	2.66
4/17/2009	13:32:30	2.64	4/17/2009	13:55:30	2.65
4/17/2009	13:33:00	2.63	4/17/2009	13:56:00	2.65
4/17/2009	13:33:30	2.62	4/17/2009	13:56:30	2.66
4/17/2009	13:57:00	2.65	4/17/2009	14:20:00	3.12
4/17/2009	13:57:30	2.65	4/17/2009	14:20:30	3.11
4/17/2009	13:58:00	2.67	4/17/2009	14:21:00	3.11
4/17/2009	13:58:30	2.66	4/17/2009	14:21:30	3.09
4/17/2009	13:59:00	2.65	4/17/2009	14:22:00	3.09
4/17/2009	13:59:30	2.67	4/17/2009	14:22:30	3.11
4/17/2009	14:00:00	2.68	4/17/2009	14:23:00	3.08
4/17/2009	14:00:30	2.67	4/17/2009	14:23:30	3.09
4/17/2009	14:01:00	2.68	4/17/2009	14:24:00	2.99
4/17/2009	14:01:30	2.68	4/17/2009	14:24:30	3.01
4/17/2009	14:02:00	0.67	4/17/2009	14:25:00	3.01
4/17/2009	14:02:30	2.82	4/17/2009	14:25:30	2.91
4/17/2009	14:03:00	2.81	4/17/2009	14:26:00	3.02
4/17/2009	14:03:30	2.83	4/17/2009	14:26:30	3.07
4/17/2009	14:04:00	2.81	4/17/2009	14:27:00	3.12
4/17/2009	14:04:30	2.81	4/17/2009	14:27:30	3.13
4/17/2009	14:05:00	2.8	4/17/2009	14:28:00	3.13
4/17/2009	14:05:30	2.82	4/17/2009	14:28:30	3.13
4/17/2009	14:06:00	0.72	4/17/2009	14:29:00	3.12

Complete results may be obtained from www.nwsc.co.ug

Date	Time	Pressure (m)	Date	Time	Pressure (m)
4/17/2009	14:06:30	2.95	4/17/2009	14:29:30	3.11
4/17/2009	14:07:00	3	4/17/2009	14:30:00	3.13
4/17/2009	14:07:30	3.04	4/17/2009	14:30:30	3.13
4/17/2009	14:08:00	3.05	4/17/2009	14:31:00	3.13
4/17/2009	14:08:30	3.07	4/17/2009	14:31:30	3.12
4/17/2009	14:09:00	3.07	4/17/2009	14:32:00	3.11
4/17/2009	14:09:30	3.08	4/17/2009	14:32:30	3.16
4/17/2009	14:10:00	3.08	4/17/2009	14:33:00	3.17
4/17/2009	14:10:30	3.11	4/17/2009	14:33:30	3.25
4/17/2009	14:11:00	3.12	4/17/2009	14:34:00	3.25
4/17/2009	14:11:30	3.12	4/17/2009	14:34:30	3.25
4/17/2009	14:12:00	3.11	4/17/2009	14:35:00	3.25
4/17/2009	14:12:30	3.08	4/17/2009	14:35:30	3.26
4/17/2009	14:13:00	3.09	4/17/2009	14:36:00	3.24
4/17/2009	14:13:30	3.1	4/17/2009	14:36:30	3.29
4/17/2009	14:14:00	3.1	4/17/2009	14:37:00	3.26
4/17/2009	14:14:30	3.1	4/17/2009	14:37:30	3.23
4/17/2009	14:15:00	3.1	4/17/2009	14:38:00	3.19
4/17/2009	14:15:30	3.11	4/17/2009	14:38:30	3.15
4/17/2009	14:16:00	3.11	4/17/2009	14:39:00	3.11
4/17/2009	14:16:30	3.13	4/17/2009	14:39:30	3.1
4/17/2009	14:17:00	3.1	4/17/2009	14:40:00	3.09
4/17/2009	14:17:30	3.11	4/17/2009	14:40:30	3.12
4/17/2009	14:18:00	3.12	4/17/2009	14:41:00	3.16
4/17/2009	14:18:30	3.11	4/17/2009	14:41:30	3.2
4/17/2009	14:19:00	3.12	4/17/2009	14:42:00	3.24
4/17/2009	14:19:30	3.11	4/17/2009	14:42:30	3.26
4/17/2009	14:43:00	3.26	4/17/2009	15:06:00	0.72
4/17/2009	14:43:30	3.25	4/17/2009	15:06:30	0.69
4/17/2009	14:44:00	3.25	4/17/2009	15:07:00	3.29
4/17/2009	14:44:30	3.28	4/17/2009	15:07:30	3.25
4/17/2009	14:45:00	3.26	4/17/2009	15:08:00	3.26
4/17/2009	14:45:30	3.26	4/17/2009	15:08:30	3.26
4/17/2009	14:46:00	3.19	4/17/2009	15:09:00	3.28
4/17/2009	14:46:30	3.28	4/17/2009	15:09:30	3.29
4/17/2009	14:47:00	3.27	4/17/2009	15:10:00	3.3
4/17/2009	14:47:30	3.27	4/17/2009	15:10:30	3.29
4/17/2009	14:48:00	3.27	4/17/2009	15:11:00	3.27
4/17/2009	14:48:30	3.26	4/17/2009	15:11:30	3.3
4/17/2009	14:49:00	3.28	4/17/2009	15:12:00	3.29

Complete results may be obtained from www.nwsc.co.ug

Date	Time	Pressure (m)	Date	Time	Pressure (m)
4/17/2009	14:49:30	3.28	4/17/2009	15:12:30	3.29
4/17/2009	14:50:00	3.29	4/17/2009	15:13:00	3.29
4/17/2009	14:50:30	3.29	4/17/2009	15:13:30	3.26
4/17/2009	14:51:00	3.28	4/17/2009	15:14:00	3.26
4/17/2009	14:51:30	3.29	4/17/2009	15:14:30	3.2
4/17/2009	14:52:00	3.3	4/17/2009	15:15:00	3.15
4/17/2009	14:52:30	3.31	4/17/2009	15:15:30	3.14
4/17/2009	14:53:00	3.31	4/17/2009	15:16:00	2.35
4/17/2009	14:53:30	3.3	4/17/2009	15:16:30	1.75
4/17/2009	14:54:00	3.3	4/17/2009	15:17:00	1.64
4/17/2009	14:54:30	3.28	4/17/2009	15:17:30	1.64
4/17/2009	14:55:00	3.3	4/17/2009	15:18:00	1.61
4/17/2009	14:55:30	3.29	4/17/2009	15:18:30	1.57
4/17/2009	14:56:00	3.28	4/17/2009	15:19:00	1.53
4/17/2009	14:56:30	3.26	4/17/2009	15:19:30	1.51
4/17/2009	14:57:00	3.22	4/17/2009	15:20:00	1.48
4/17/2009	14:57:30	3.24	4/17/2009	15:20:30	1.48
4/17/2009	14:58:00	3.26	4/17/2009	15:21:00	1.47
4/17/2009	14:58:30	3.23	4/17/2009	15:21:30	1.47
4/17/2009	14:59:00	3.22	4/17/2009	15:22:00	1.48
4/17/2009	14:59:30	3.23	4/17/2009	15:22:30	1.45
4/17/2009	15:00:00	3.23	4/17/2009	15:23:00	1.45
4/17/2009	15:00:30	3.25	4/17/2009	15:23:30	1.45
4/17/2009	15:01:00	3.28	4/17/2009	15:24:00	1.49
4/17/2009	15:01:30	3.26	4/17/2009	15:24:30	1.49
4/17/2009	15:02:00	3.27	4/17/2009	15:25:00	1.54
4/17/2009	15:02:30	3.29	4/17/2009	15:25:30	1.55
4/17/2009	15:03:00	3.3	4/17/2009	15:26:00	1.56
4/17/2009	15:03:30	3.26	4/17/2009	15:26:30	1.55
4/17/2009	15:04:00	3.26	4/17/2009	15:27:00	1.59
4/17/2009	15:04:30	3.27	4/17/2009	15:27:30	1.6
4/17/2009	15:05:00	3.27	4/17/2009	15:28:00	1.59
4/17/2009	15:05:30	3.27	4/17/2009	15:28:30	1.87

Appendix C: Sample Model Inputs and Outputs

```
  Page 1                                        3/30/2011 12:55:34
PM

**********************************************************************
*
  *                           E P A N E T
*
  *                    Hydraulic and Water Quality
*
  *                    Analysis for Pipe Networks
*
  *                           Version 2.0
*

**********************************************************************
*

  Input File: Primary system with Rubaga&Mutungo&Naguru&Gunhill
subsystem4.net

  Schematized Kampala Water Supply Network
  This pipe network has been schematized to this simplified lay out
in order to faciltate analysis and research for my doctoral
studies.The model has been built in the EPANET software.

  Link - Node Table:
  --------------------------------------------------------------------
---
  Link            Start           End                 Length  Diameter
  ID              Node            Node                     m        mm
  --------------------------------------------------------------------
---
  1               2               1                     6000       500
  2               1               5                     5300       500
  3               6               1                     6000       500
  6               5               7                      413       500
  7               17              20                     130       200
  8               17              7                      100       300
  9               19              20                     100       150
  10              18              19                     120       150
  11              18              17                     170       200
  12              16              17                     120       250
  13              21              18                     172       150
  14              16              21                      90       200
  15              22              21                     100       150
  16              23              16                      95       150
  17              22              23                     105       150
  18              25              22                     120       150
  19              24              25                      95       150
  20              23              24                     110       150
  21              15              16                     100       300
  22              14              15                     100       150
  23              13              14                     200       100
  24              12              13                     120       150
  25              11              12                     180       200
  26              14              11                      90       250
  27              7               11                     150       400
```

28	2	8	500	150
29	10	8	2000	300
30	26	10	1000	100
31	26	27	2000	500
32	27	2	700	500
33	27	8	500	150
34	28	27	1000	250
35	28	8	1000	100
36	9	6	250	400
37	9	29	500	250
38	29	30	1000	250

Page 2 Schematized Kampala Water Supply Network

Link - Node Table: (continued)

Link ID	Start Node	End Node	Length m	Diameter mm	
39	30	31	1500	250	
40	32	31	600	250	
41	32	33	1500	250	
42	34	33	1800	150	
43	34	32	2000	150	
44	33	35	500	250	
46	36	29	800	200	
48	3	37	100	500	
49	38	37	500	400	
50	38	39	1000	200	
51	37	39	1000	300	
52	40	38	700	100	
53	38	41	350	350	
54	42	41	600	300	
55	42	40	1500	150	
56	43	41	1500	500	
57	43	44	2000	150	
58	40	44	1200	150	
59	35	45	300	300	
60	45	9	75	250	
4	45	36	800	150	
45	4	1	#N/A	#N/A	Pump
61	4	3	#N/A	#N/A	Pump

Energy Usage:

Pump	Usage Factor	Avg. Effic.	Kw-hr /m3	Avg. Kw	Peak Kw	Cost /day

```
  45                100.00  75.00      0.60   7272.90   7272.90
0.00
  61                100.00  75.00      0.31   4337.75   4337.75
0.00
  --------------------------------------------------------------------
---
                                                Demand Charge:
0.00
                                                Total Cost:
0.00

  Node Results at 0:00 Hrs:
  --------------------------------------------------------------------
---
  Node                    Demand      Head  Pressure   Quality
  ID                         LPS         m         m
  --------------------------------------------------------------------
---
  7                         1.70   1204.97     10.97      0.00
  11                        1.65   1204.96     28.96      0.00
  12                        1.65   1204.94     26.94      0.00
  13                        1.65   1204.93     22.93      0.00
  14                        1.75   1204.95     34.95      0.00
  15                        1.75   1204.91     37.91      0.00
  16                        1.75   1204.91     20.91      0.00


  Page 3                        Schematized Kampala Water Supply
Network
  Node Results at 0:00 Hrs: (continued)
  --------------------------------------------------------------------
---
  Node                    Demand      Head  Pressure   Quality
  ID                         LPS         m         m
  --------------------------------------------------------------------
---
  17                        1.70   1204.93     14.93      0.00
  18                        1.65   1204.91     22.91      0.00
  19                        1.60   1204.91     24.91      0.00
  20                        1.60   1204.92     29.92      0.00
  21                        1.75   1204.89     32.89      0.00
  22                        1.75   1204.84     29.84      0.00
  23                        1.75   1204.84     24.84      0.00
  24                        1.75   1204.82     20.82      0.00
  25                        1.65   1204.82     18.82      0.00
  8                         1.80   1204.94     24.94      0.00
  10                        1.60   1204.94     24.94      0.00
  26                        1.90   1204.99     54.99      0.00
  27                        1.55   1205.00     41.00      0.00
  28                        1.65   1204.98     44.98      0.00
  9                         1.75   1304.98     64.98      0.00
  29                        1.80   1304.92     64.92      0.00
  30                        1.55   1304.87     74.87      0.00
  31                        1.50   1304.85     94.85      0.00
  32                        1.75   1304.85     84.85      0.00
  33                        1.75   1304.88     74.88      0.00
  34                        1.60   1304.79    114.79      0.00
```

35	1.70	1304.93	74.93	0.00	
36	1.50	1304.91	64.91	0.00	
37	1.83	1225.00	5.00	0.00	
38	1.75	1224.98	39.98	0.00	
39	1.85	1224.99	64.99	0.00	
40	1.65	1224.79	44.79	0.00	
41	1.88	1224.96	34.96	0.00	
42	1.91	1224.95	54.95	0.00	
43	1.55	1224.96	24.96	0.00	
44	1.80	1224.77	24.77	0.00	
45	1.90	1304.95	64.95	0.00	
4	-7277.14	1140.00	0.00	0.00	Reservoir
1	2846.37	1305.00	5.00	0.00	Tank
2	-8.50	1205.00	5.00	0.00	Tank
5	500.30	1205.00	5.00	0.00	Tank
6	-18.03	1305.00	5.00	0.00	Tank
3	3890.40	1225.00	5.00	0.00	Tank

Page 4 Schematized Kampala Water Supply Network

Link Results at 0:00 Hrs:

Link ID	Flow LPS	VelocityUnit m/s	Headloss m/km	Status
1	0.00	0.00	0.00	Closed
2	527.40	2.69	18.87	Open
3	1.23	0.01	0.00	Open
6	27.10	0.14	0.08	Open
7	2.90	0.09	0.11	Open
8	-15.71	0.22	0.34	Open
9	-1.30	0.07	0.10	Open
10	0.30	0.02	0.01	Open
11	-3.30	0.10	0.14	Open
12	-7.80	0.16	0.23	Open
13	-1.35	0.08	0.11	Open
14	3.58	0.11	0.16	Open
15	-3.18	0.18	0.52	Open
16	-3.72	0.21	0.69	Open
17	-0.22	0.01	0.00	Open
18	-1.65	0.09	0.15	Open
19	0.00	0.00	0.00	Open
20	1.75	0.10	0.17	Open
21	1.24	0.02	0.00	Open
22	2.99	0.17	0.46	Open
23	-0.45	0.06	0.10	Open
24	1.20	0.07	0.08	Open
25	2.85	0.09	0.10	Open
26	-5.19	0.11	0.11	Open
27	9.69	0.08	0.03	Open
28	1.43	0.08	0.12	Open
29	-1.26	0.02	0.00	Open
30	0.34	0.04	0.06	Open

31	-2.24	0.01	0.00	Open
32	-7.07	0.04	0.01	Open
33	1.37	0.08	0.11	Open
34	-1.92	0.04	0.02	Open
35	0.27	0.03	0.04	Open
36	-16.80	0.13	0.09	Open
37	5.62	0.11	0.12	Open
38	3.24	0.07	0.04	Open
39	1.69	0.03	0.01	Open
40	-0.19	0.00	0.00	Open
41	-2.24	0.05	0.02	Open
42	-0.92	0.05	0.05	Open
43	-0.68	0.04	0.03	Open
44	-4.91	0.10	0.10	Open
46	-0.58	0.02	0.01	Open
48	14.21	0.07	0.02	Open
49	-9.89	0.08	0.04	Open
50	-0.64	0.02	0.01	Open
51	2.49	0.04	0.01	Open

Page 5 Schematized Kampala Water Supply Network

Link Results at 0:00 Hrs: (continued)

Link ID	Flow LPS	VelocityUnit m/s	Headloss m/km	Status
52	-0.78	0.10	0.27	Open
53	8.01	0.08	0.05	Open
54	-3.29	0.05	0.02	Open
55	1.38	0.08	0.11	Open
56	-2.84	0.01	0.00	Open
57	1.29	0.07	0.10	Open
58	0.51	0.03	0.02	Open
59	-6.61	0.09	0.07	Open
60	-9.43	0.19	0.32	Open
4	0.92	0.05	0.05	Open
45	3372.53	0.00	-165.00	Open Pump
61	3904.61	0.00	-85.00	Open Pump

Node Results at 1:00 Hrs:

Node ID	Demand LPS	Head m	Pressure m	Quality
7	1.70	1204.97	10.97	0.00
11	1.65	1204.96	28.96	0.00
12	1.65	1204.94	26.94	0.00
13	1.65	1204.93	22.93	0.00
14	1.75	1204.95	34.95	0.00
15	1.75	1204.91	37.91	0.00
16	1.75	1204.91	20.91	0.00

17	1.70	1204.93	14.93	0.00	
18	1.65	1204.91	22.91	0.00	
19	1.60	1204.91	24.91	0.00	
20	1.60	1204.92	29.92	0.00	
21	1.75	1204.89	32.89	0.00	
22	1.75	1204.84	29.84	0.00	
23	1.75	1204.84	24.84	0.00	
24	1.75	1204.82	20.82	0.00	
25	1.65	1204.82	18.82	0.00	
8	1.80	1204.90	24.90	0.00	
10	1.60	1204.89	24.89	0.00	
26	1.90	1204.95	54.95	0.00	
27	1.55	1204.95	40.95	0.00	
28	1.65	1204.94	44.94	0.00	
9	1.75	1304.88	64.88	0.00	
29	1.80	1304.82	64.82	0.00	
30	1.55	1304.78	74.78	0.00	
31	1.50	1304.76	94.76	0.00	
32	1.75	1304.76	84.76	0.00	
33	1.75	1304.79	74.79	0.00	
34	1.60	1304.70	114.70	0.00	
35	1.70	1304.84	74.84	0.00	

Page 6 Schematized Kampala Water Supply Network

Node Results at 1:00 Hrs: (continued)

Node ID	Demand LPS	Head m	Pressure m	Quality	
36	1.50	1304.82	64.82	0.00	
37	1.83	1225.00	5.00	0.00	
38	1.75	1224.98	39.98	0.00	
39	1.85	1224.99	64.99	0.00	
40	1.65	1224.79	44.79	0.00	
41	1.88	1224.96	34.96	0.00	
42	1.91	1224.95	54.95	0.00	
43	1.55	1224.96	24.96	0.00	
44	1.80	1224.77	24.77	0.00	
45	1.90	1304.86	64.86	0.00	
4	-7277.14	1140.00	0.00	0.00	Reservoir
1	2338.76	1305.00	5.00	0.00	Tank
2	484.83	1204.96	4.96	0.00	Tank
5	500.28	1205.00	5.00	0.00	Tank
6	-3.73	1304.91	4.91	0.00	Tank
3	3890.40	1225.00	5.00	0.00	Tank

Link Results at 1:00 Hrs:

Link ID	Flow LPS	VelocityUnit m/s	Headloss m/km	Status
1	-493.33	2.51	16.67	Open
2	527.38	2.69	18.87	Open

3	-13.07	0.07	0.02	Open
6	27.10	0.14	0.08	Open
7	2.90	0.09	0.11	Open
8	-15.71	0.22	0.34	Open
9	-1.30	0.07	0.10	Open
10	0.30	0.02	0.01	Open
11	-3.30	0.10	0.14	Open
12	-7.80	0.16	0.23	Open
13	-1.35	0.08	0.11	Open
14	3.58	0.11	0.16	Open
15	-3.18	0.18	0.51	Open
16	-3.72	0.21	0.69	Open
17	-0.22	0.01	0.00	Open
18	-1.65	0.09	0.15	Open
19	0.00	0.00	0.00	Open
20	1.75	0.10	0.17	Open
21	1.24	0.02	0.00	Open
22	2.99	0.17	0.46	Open
23	-0.44	0.06	0.10	Open
24	1.21	0.07	0.08	Open
25	2.86	0.09	0.10	Open
26	-5.19	0.11	0.11	Open
27	9.69	0.08	0.03	Open

Page 7 Schematized Kampala Water Supply Network

Link Results at 1:00 Hrs: (continued)

Link ID	Flow LPS	VelocityUnit m/s	Headloss m/km	Status
28	1.43	0.08	0.12	Open
29	-1.26	0.02	0.00	Open
30	0.34	0.04	0.06	Open
31	-2.24	0.01	0.00	Open
32	-7.07	0.04	0.01	Open
33	1.37	0.08	0.11	Open
34	-1.92	0.04	0.02	Open
35	0.27	0.03	0.04	Open
36	-16.80	0.13	0.09	Open
37	5.62	0.11	0.12	Open
38	3.24	0.07	0.04	Open
39	1.69	0.03	0.01	Open
40	-0.19	0.00	0.00	Open
41	-2.24	0.05	0.02	Open
42	-0.92	0.05	0.05	Open
43	-0.68	0.04	0.03	Open
44	-4.91	0.10	0.10	Open
46	-0.58	0.02	0.01	Open
48	14.21	0.07	0.02	Open
49	-9.89	0.08	0.04	Open
50	-0.64	0.02	0.01	Open
51	2.49	0.04	0.01	Open
52	-0.78	0.10	0.27	Open
53	8.01	0.08	0.05	Open

```
  54                    -3.29      0.05      0.02      Open
  55                     1.38      0.08      0.11      Open
  56                    -2.84      0.01      0.00      Open
  57                     1.29      0.07      0.10      Open
  58                     0.51      0.03      0.02      Open
  59                    -6.61      0.09      0.07      Open
  60                    -9.43      0.19      0.32      Open
  4                      0.92      0.05      0.05      Open
  45                  3372.53      0.00   -165.00      Open Pump
  61                  3904.61      0.00    -85.00      Open Pump

  Node Results at 2:00 Hrs:
  ----------------------------------------------------------------------
---
  Node                 Demand      Head  Pressure   Quality
  ID                      LPS         m         m
  ----------------------------------------------------------------------
---
  7                      1.70   1204.97     10.97      0.00
  11                     1.65   1204.96     28.96      0.00
  12                     1.65   1204.94     26.94      0.00
  13                     1.65   1204.93     22.93      0.00
  14                     1.75   1204.95     34.95      0.00
  15                     1.75   1204.91     37.91      0.00
  16                     1.75   1204.91     20.91      0.00

  Page 8                         Schematized Kampala Water Supply
Network
  Node Results at 2:00 Hrs: (continued)
  ----------------------------------------------------------------------
---
  Node                 Demand      Head  Pressure   Quality
  ID                      LPS         m         m
  ----------------------------------------------------------------------
---
  17                     1.70   1204.93     14.93      0.00
  18                     1.65   1204.91     22.91      0.00
  19                     1.60   1204.91     24.91      0.00
  20                     1.60   1204.92     29.92      0.00
  21                     1.75   1204.89     32.89      0.00
  22                     1.75   1204.84     29.84      0.00
  23                     1.75   1204.84     24.84      0.00
  24                     1.75   1204.82     20.82      0.00
  25                     1.65   1204.82     18.82      0.00
  8                      1.80   1204.90     24.90      0.00
  10                     1.60   1204.89     24.89      0.00
  26                     1.90   1204.95     54.95      0.00
  27                     1.55   1204.95     40.95      0.00
  28                     1.65   1204.94     44.94      0.00
  9                      1.75   1304.86     64.86      0.00
  29                     1.80   1304.80     64.80      0.00
  30                     1.55   1304.75     74.75      0.00
  31                     1.50   1304.73     94.73      0.00
  32                     1.75   1304.73     84.73      0.00
  33                     1.75   1304.76     74.76      0.00
  34                     1.60   1304.67    114.67      0.00
  35                     1.70   1304.81     74.81      0.00
```

36	1.50	1304.79	64.79	0.00	
37	1.83	1225.00	5.00	0.00	
38	1.75	1224.98	39.98	0.00	
39	1.85	1224.99	64.99	0.00	
40	1.65	1224.79	44.79	0.00	
41	1.88	1224.96	34.96	0.00	
42	1.91	1224.95	54.95	0.00	
43	1.55	1224.96	24.96	0.00	
44	1.80	1224.77	24.77	0.00	
45	1.90	1304.83	64.83	0.00	
4	-7277.14	1140.00	0.00	0.00	Reservoir
1	2338.79	1305.00	5.00	0.00	Tank
2	484.83	1204.96	4.96	0.00	Tank
5	500.28	1205.00	5.00	0.00	Tank
6	-3.77	1304.88	4.88	0.00	Tank
3	3890.40	1225.00	5.00	0.00	Tank

Page 9 Schematized Kampala Water Supply Network

Link Results at 2:00 Hrs:

Link ID	Flow LPS	VelocityUnit m/s	Headloss m/km	Status
1	-493.33	2.51	16.67	Open
2	527.38	2.69	18.87	Open
3	-13.03	0.07	0.02	Open
6	27.10	0.14	0.08	Open
7	2.90	0.09	0.11	Open
8	-15.71	0.22	0.34	Open
9	-1.30	0.07	0.10	Open
10	0.30	0.02	0.01	Open
11	-3.30	0.10	0.14	Open
12	-7.80	0.16	0.23	Open
13	-1.35	0.08	0.11	Open
14	3.58	0.11	0.16	Open
15	-3.18	0.18	0.51	Open
16	-3.72	0.21	0.69	Open
17	-0.22	0.01	0.00	Open
18	-1.65	0.09	0.15	Open
19	0.00	0.00	0.00	Open
20	1.75	0.10	0.17	Open
21	1.24	0.02	0.00	Open
22	2.99	0.17	0.46	Open
23	-0.44	0.06	0.10	Open
24	1.21	0.07	0.08	Open
25	2.86	0.09	0.10	Open
26	-5.19	0.11	0.11	Open
27	9.69	0.08	0.03	Open
28	1.43	0.08	0.12	Open
29	-1.26	0.02	0.00	Open
30	0.34	0.04	0.06	Open
31	-2.24	0.01	0.00	Open

32	-7.07	0.04	0.01	Open
33	1.37	0.08	0.11	Open
34	-1.92	0.04	0.02	Open
35	0.27	0.03	0.04	Open
36	-16.80	0.13	0.09	Open
37	5.62	0.11	0.12	Open
38	3.24	0.07	0.04	Open
39	1.69	0.03	0.01	Open
40	-0.19	0.00	0.00	Open
41	-2.24	0.05	0.02	Open
42	-0.92	0.05	0.05	Open
43	-0.68	0.04	0.03	Open
44	-4.91	0.10	0.10	Open
46	-0.58	0.02	0.01	Open
48	14.21	0.07	0.02	Open
49	-9.89	0.08	0.04	Open
50	-0.64	0.02	0.01	Open
51	2.49	0.04	0.01	Open