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DEPARTMENT OF CIVIL AND CONSTRUCTION ENGINEERING

Evaluation of Robustness of Water Distribution Networks by varying Peak Factors in Hydraulic Models: Case Study of Mombasa North Mainland

MSc. Thesis

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F56/7902/2017

Thesis submitted in partial fulfilment of the requirements for the award of the Degree of Master of Science in Civil Engineering (Water Resources Engineering) in the Department of Civil and Construction Engineering of the University of Nairobi

AUGUST 2022

DECLARATION

I, Mugaa Murithi Eric, hereby declare that this thesis is my original work. To the best of my knowledge, the work presented here has not been presented for a degree in any other Institution of Higher Learning.

	22nd August 2022
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Approval

This thesis has been submitted for examination with my approval as university supervisor.

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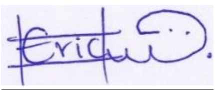
Signature: 

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DEDICATION

This thesis is dedicated to my family; my father Benson, my mother Tabitha, my siblings Robert, Moses, Jane, Reuben and Fridah. Thank you for your advice, encouragement, support, and prayers throughout my study. May you live long and fulfilled lives. God bless you all.

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LIST OF ABBREVIATIONS AND ACRONYMS

AC	Asbestos Cement
BWM	Bulk Water Meters
CI	Cast Iron
DI	Ductile Iron
DMA	District Metering Area
DMZ	District Metering Zone
EPA	Environmental Protection Agency
GIS	Geographical Information Systems
GMS	Galvanized Mild Steel
HL	Head Loss
IC	Individual Connections
IWA	International Water Association
LPS	Litres per Second
MNF	Minimum Night Flow
MOWASSCO	Mombasa Water Supply and Sanitation Company
MPa	Mega Pascals
MWI	Ministry of Water and Information
NC	Non-Individual Connections
NGO	Non-Governmental Organization
NRW	Non-Revenue Water
P. F	Peak Factor
SCADA	Peak Factor
WOP	Water Operator Partnership
WSP	Water Service Provider

ABSTRACT

Most old water distribution networks are extended overtime to accommodate new connections which make them heavily looped. These networks also lack well-documented mapping incorporating all alterations. Consequently, there are difficulties in the monitoring of water flow in pipelines and in the management of water losses. The result is poorly sized and managed water networks that do not meet flow velocities and pressure requirements leading to inadequate and erratic water supply. Upgrading of the networks involves testing the robustness of the network to handle varied flows and construction of capacity augmentation pipelines running parallel to the old pipelines. In this study, the existing alignments of the Mombasa North Mainland distribution network were mapped and modelled using KYPipe Software. Three models; namely, existing network model for year 2020, and future network models design years 2030 and 2040, were developed. Design flow is the projected peak flow in the distribution network and usually has a design peak factor of between 1.0 and 1.2. Flows greater than the design flow were simulated in the distribution networks by applying peak factors of 1.5 and 2.0 in the models. The adequacy of the network was determined based on the achievement of acceptable design pressures and pipeline flow velocities. Results showed that the existing and future Mombasa North Mainland hydraulic model analyzed under a peak factor of 1.2 met the water supply design criteria for pressures range 100 - 600 kPa and flow velocity range 0.01 - 2.0 m/s and pressures of 80 to 524 kPa. The 2030 model simulated using a peak factor of 1.5 was found robust to transmit the increased flow. However, under 1.5 peak factor, 2040 model was found insufficient. Under a peak factor of 2.0, the 2030 and 2040 models were found insufficient to transmit the increased flows with both models experiencing negative flows and zero flow pipelines in some parts. The study optimized the networks for possible increased flows by proposing augmented new pipelines for short distances along the main transmission mains to increase the robustness of the ultimate model for a 1.5 and 2.0 peak factor. The study recommended testing the robustness of water distribution networks to handle unexpected flows before implementation of the networks. Moreover, water utilities should develop, maintain, and update models for quick and informed decisions making on their network expansions.

CHAPTER ONE

1. INTRODUCTION

1.1 Background Information

Least cost water distribution networks that are needed in developing countries are designed to minimize network cost while meeting various design criteria such as nodal pressures and required flow. The networks are designed to meet projected water demands; however, water demand projections are affected by planning uncertainties that include population growths, spatial developments, and changes in public sentiment on water use (Kang & Lansey, 2012). Therefore, the networks need to be robust to handle increased and unexpected flow patterns. that result from design assumptions or unforeseen economic and population growths in the service area.

Robustness of a network ensures that a supply maintains the required water quality and quantity to customers at acceptable pressures under normal operation (Jung & Hoon, 2017; Jung, et al., 2014). One of the ways to improve robustness of a water distribution network is by installation of bigger pipes. However, bigger pipes impact the economic feasibility of the network adversely.

Water consumption varies hourly, daily, weekly, and seasonally therefore water systems are designed to convey the maximum water demand to users. To ensure that the maximum water demand is met, a peak factor is applied on the demand. Various factors such as climate and community characteristics influence peak factors. The commonly applied peak factors vary widely, for example, from 1.35 to 3.40 (Brandt, et al., 2017), and affect the pipeline sizes and therefore system costs. Consequently, adoption of high peak factors results in higher flows that require bigger pipelines that are costly. Pipelines costs account for more than 40% of the distribution network cost (Gurung, et al., 2016).

Hydraulic models use common hydraulic equations to analyze distribution networks considering parameters such as pipeline sizes, materials, and water sources to compute flow rates, velocities and nodal pressures. Hydraulic modelling assists water service providers build management scenarios for improving the efficiency and reliability of existing networks. They also help in design of new networks (Karaa, et al., 2016). Moreover, the models serve to store distribution network data in form of pipeline attributes.

The intended use of a hydraulic model determines the level of calibration and precision of validation. Hydraulic models can be used for activities such as extension of networks into new areas, division of the network into district metering areas, scheduling of network rehabilitations and determination of optimum pressures in the network (Tabesh, et al., 2009). In developed countries, most distribution networks are equipped and controlled with supervisory control and data acquisition (SCADA) systems to improve network efficiencies. These systems provide real time data on control points in the network for comparison with the model predictions while considering the temporal and spatial variations of water users in the network.

Many water distribution networks in Kenya such as the Mombasa North Water Distribution Network have been augmented and expanded over the years on need bases. These expansions have resulted in heavily looped networks that are difficult to monitor and may incur water losses. Establishment of geo-referenced hydraulic models can assist test the robustness of the networks for different flow capacities by varying the peak factor values. This study focused on modelling the Mombasa North Mainland distribution network in Kenya with different peak factors to test its robustness for different flow capacities.

1.2 Problem Statement

Distribution systems in Kenya are either community based or managed by water service providers (WSP). Most of these networks are more than 30 years old and have been extended overtime to accommodate new connections which has made them heavily looped. The networks lack well-documented mapping incorporating all alterations. Consequently, they are faced with difficulties in the monitoring of water flow in pipelines and water loss management. Moreover, water demand often exceeds design flows leading to inadequate and erratic water supply. The result is poorly sized and managed water networks that do not meet flow velocities and pressure requirements. Upgrading of the networks involves testing and updating them with new flow pipeline augmentations. Consequently, there is need to test the robustness of the network to handle various flows, while maintaining the required pressures. An innovative way to fluctuate flows in hydraulic models is to vary the peak factor, from design peak factor of 1.2 to the globally used factors of 1.5 to 2.0.

1.3 Research Objectives

The overall objective of this study was to evaluate the robustness of water distribution networks by varying flows in hydraulic models using different peak factors, with Mombasa North Mainland Water Distribution Network as a case study.

The specific objectives of the project are to:

a) Model the Mombasa North Mainland Water Distribution Network using KYPipe Software at the design peak factor of 1.2 and evaluate the adequacy of the pipeline velocities and nodal pressures.
b) Establish the robustness of pipeline network using peak factors of 1.5 and 2.0.
c) Evaluate augmentation of Mombasa North mainland water distribution network when simulated with flow in excess of the design flow.

1.4 Scope and Limitations of the Study

This study mapped and modeled the existing and proposed pipelines of Mombasa North Mainland network. The robustness of the network was gauged on the node pressures and water velocities in the transmission pipes by running simulations using KYPIPE modelling software. Global Mapper 18 and Arc GIS 10.5 Software were used for geo-referencing of the distribution network. The study tested the robustness of the proposed pipeline sizes to handle different flows by increasing the demand peak factor in the model from 1.2 to 1.5 and 2.0.

The limitations of the study included:

i. Mombasa North Mainland distribution network pipeline alignments mapped on Geographic Positioning Systems (GPS) Receiver limited to the knowledge of the operators.
ii. The accuracy of pipeline parameters such as materials and diameters depended on the accuracy of the information obtained from the water service provider.
iii. The ability to correctly identify the alignments of the water mains buried underground in instances where these alignments had not been earmarked.

CHAPTER TWO

2. LITERATURE REVIEW

2.1 Water Supply System

A typical water supply system for a supply area consists of infrastructure to abstract, treat, store, and convey water for use. The four major components of a water supply system are water source, treatment facilities, storage facilities and a distribution network (Hickey, 2008; Linsley & Franzini, 1979). A water supply system should guarantee the quality and quantity of water. The major components of a distribution network are illustrated in Figure 2.1.

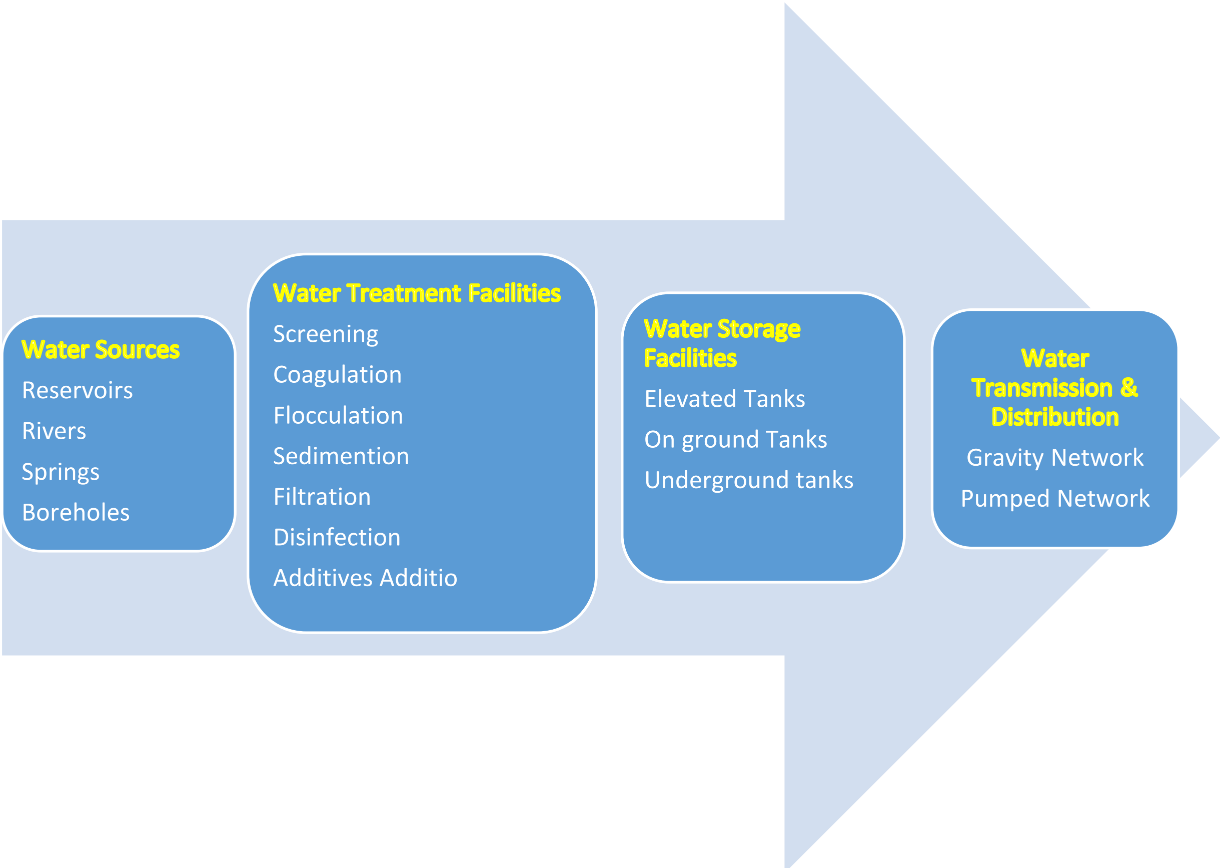


Figure 2.1: Components of a typical Piped Water Supply System

2.2 Water Distribution Network

An ideal water distribution network provides water to users in the required quality and quantity at a specified range of velocities and pressures. Physical utilities of a distribution network are the infrastructures that is built to facilitate the flow of water including terminal tanks, pipes, gate valves, bulk water meters, air valves, wash outs, fire hydrants and consumer water meters.

Intangible utilities of a water distribution network are the managerial aspects of the network to ensure efficient co-ordination and minimize water losses. Water utilities are tasked to formulate management policies and strategies to ensure resilience of distribution networks (Jung, et al., 2019).

2.3 Requirements of a Water Distribution Network

Drinking water should be colourless, palatable and odourless but in its natural state water is always marred with impurities during distribution considering that it is a natural solvent (Pohorille & Pratt, 2012). Therefore, a water distribution network should maintain the water quality in the distribution pipes without significant deterioration. The pipelines should be well jointed to minimize losses and be able to meet the pressure requirements for both domestic and firefighting purposes. In addition, the network configuration should ensure users access water even during repairs.

2.4 Pipe Layouts in a Water Distribution Network

Different pipe layout configurations in a water distribution network exist depending on geographic and physical characteristic of the area, degree and type of developments found in the locality. These layouts are broadly classified into gridiron and dead-end/tree/ dendritic layouts (Wagner & Lanoix, 1959). Over the years the gridiron network has broken down further into different layouts based on pipeline arrangement such as circular/ ringed layouts and looped layouts (King & Crocker, 1973). The following subsections briefly describe the two broad classifications of layouts.

2.4.1 Gridiron Water Distribution Network

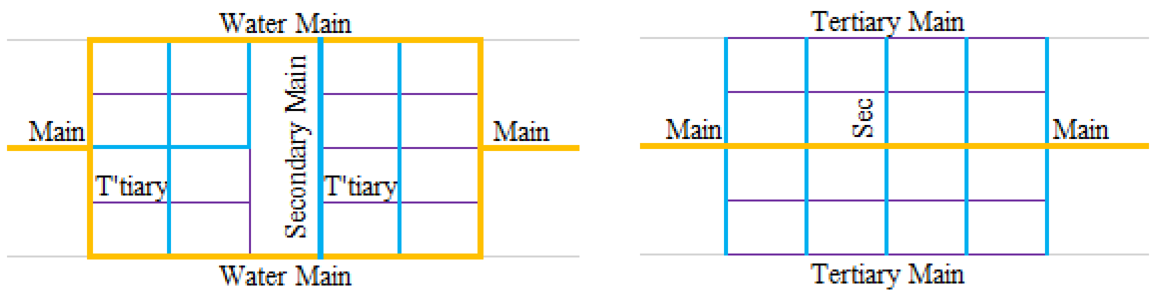
Various researchers including Hickey (2008), King and Crocker (1973) and Wagner and Lanoix 1959 recommended that the supply water main forms a ring/mesh around the supply area in gridiron networks. The secondary lines branch off from the main pipelines and from each other. The branches form loops (rectangles/ circular) and interconnect themselves ensuring that there are no dead ends amongst them (Hickey, 2008). These layouts are suitable for areas that are well planned as they follow road peripherals. Gridiron networks are illustrated in Figure 2.2 in form of ringed and looped distribution networks.

Advantages of gridiron water distribution networks include:

i. Any point in the system is at least supplied from two directions.
ii. Network operations and maintenance does not completely cut water supply.
iii. Minimum head loss during transmission due to the interconnections.

iv. Reduced contamination due to elimination of stagnant water (Wagner & Lanoix, 1959).

v. More supply to fire hydrants in the network.



Ringed Distribution Network **Looped Distribution Network**

Figure 2.2: Ringed and looped Distribution Network

Both Hickey (2008) and King and Crocker (1973) highlighted some disadvantages of gridiron water distribution network. They state that the network is:

i. Expensive due to cut-off valves required to control water in different branches.

ii. Difficult to design actual pipe sizes since problematic to calculate the discharge, pressures, and velocities in different pipes due to the interconnections.

iii. Results in larger pipe diameter than necessary (Wagner & Lanoix, 1959).

iv. Relatively expensive to lay and joint because of many pipe interconnections.

2.4.2 Tree / Dead End Water Distribution Network

Tree/ dead end water distribution network is a network with the main line running at the centre or along the main road of the supply area. Secondary and tertiary/ service water mains branch off from the main to transmit water to users (Hickey, 2008). Twort, et al, (2000) recommends dead end networks for trunk mains and local distributions in the rural areas. Ultimately, end users connect to the tertiary/ service lines as illustrated in Figure 2.3.

Dead end networks have the advantages of being simple and easy to design, relatively simple to lay pipelines in trenches and a reduced number of cut valves to control flow of water as compared to gridiron distribution networks. However dead-end networks risk water contamination in the pipeline dead ends and water shortages in some areas due to pipeline breakdowns. Supply shortage in the dead-end network during repairs is because of lack of alternative routes to supply water since only one pipe conveys water to some areas (GONU, 2009).

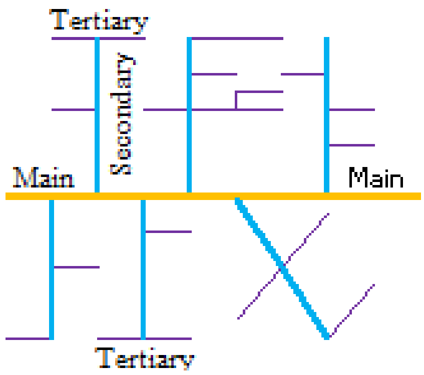


Figure 2.3: Tree/ Dead-End Distribution Network

In practice, Wagner and Lanoix recommends a compromise between gridiron and dead ends distribution networks where gridiron network is laid in residential areas and central business districts and dead ends network is used for the rest of the network.

2.5 Methods of Water Distribution in Networks

Depending on the topography of a supply area, the water source, the elevation of the terminal tank and other local factors, three main methods of water distribution that can be used to supply an area with water are the gravity method, the pumped and the combined pumping and gravity method.

2.5.1 Gravity Method of Water Distribution

Gravity method of water distribution is the most common, most reliable, and economical method of water distribution (Mohanty, 2012). Linsley and Franzini (1979) recommended having a water source/ tank at an elevated location to ensure water reaches all points of the network with adequate pressures. Water then flows to users by gravity via closed pipelines of the distribution network with major head loss occurring due to friction (Sodiki & Adigio, 2014).

2.5.2 Pumped Method of Water Distribution

Pumped method of water distribution involves pumping water directly into the distribution network when the elevation difference between water source and supply area is small. It is an expensive method of water distribution as high lift pumps are engaged considering power usage and their operations and maintenance (Twort, et al., 2000). Power Consumption of 3 to 6% is estimated to be used in water transmission (Bylka & Mroz, 2019) In addition, pumping water directly into distribution network pipes can cause periodic bursts of the pipes due to prevalent surges reducing pipes lifespan and increasing the cost of the project.

2.5.3 Combined Pumped and Gravity Method of Water Distribution

Researchers such as Mohanty (2012), and Twort, et al (2000) recommended that where elevation difference between water source and supply area is inadequate to allow gravity flow, treated water can be pumped to a high elevation terminal tank and gravitate to users. The tank serves as storage of excess water for supply during emergencies and supply of water during peak hours to offset the abnormally high peak hour demand.

2.6 Water Use in a Supply Area

The amount of water use in a supply area is location dependent and varies from one area to another. Globally water use has been on the rise growing at more than twice the rate of the population increase and already a number of regions are chronically short of water. (UN, 1996) estimated that by 2025, as much as two-thirds of the world population would be under stress conditions.

Factors affecting water use include population size and distribution, technology, economics, and environmental conditions (National Academy of Sciences, 1999). Linsley & Franzini (1979) and Twort et al (2000) cited climate and industrialization as part of factors that also affect water use in a supply area.

2.6.1 Categories of Water Use/ Demand

Water use is broadly split into two categories; domestic use and non-domestic use (Twort, et al., 2000; Linsley & Franzini, 1979). Domestic use includes water consumption in households, bathing, and food preparation (Howard & Bartram, 2003). Twort, et al (2000) includes the concepts of irrigating gardens, filling ponds and swimming pools, and washing cars into domestic use. Various factors that affect domestic water use include household characteristics (Syme, et al., 2004), attitudes on water consumption, supply restrictions (Andey & Kelkar, 2009), utility water tariff structures (Renwick & Green, 2000) and water variability in season and availability (Machingambi & Manzungu, 2003). According to various researchers, Syme et al (2004) and Howard & Bartram (2003) domestic water use is given in litres per capita per day (l/c/d). Domestic water use values range from 120 to 160 l/c/d in some European countries, 200 to 250 l/c/d in some Asian countries and 180 to 230 l/c/d in USA (Twort, et al., 2000). The values are lower in Africa and vary depending on the economic ability of involved households. In Kenya, the domestic water demand values range from 60 to 250 l/c/d for households with individual water connections as per the Ministry of Water.

Non- domestic water use comprises water used in commercial establishments, industrial use, institutional water uses such as schools, and agricultural water use. However, Linsley (1970) gives public use water as a separate category of water use. Public use water is water used in public parks, churches, and street washing. The average global freshwater withdrawal for agricultural use is approximately 90% of all water extraction for low -income countries; 79% for middle income countries and only 41% for high income countries. For industrial use, high income countries use 17% of all water extracted while 2% is used by low-income countries (Ritchie, 2017).

2.7 Peak Factors

Water distribution networks are designed based on maximum water demand therefore peak factors become a consideration (Gato & Gan, 2014). Peak factors consider seasonal, weekly, daily, and hourly variations in water use and are influenced by various factors such as climate and community (Haque, et al., 2015) and the service area (Brandt, et al., 2017).

Water use over a 24-hour period varies from the minimum night flow (MNF) to a peak hour demand, although the rate of draw may be reduced where the supply passes through in-house storage (Brandt, et al., 2017). Diurnal water use varies depending on the size of the population and the commercial activity that residents engage in. Daily water use also depends on the day of the week because demand peaks differently between a normal weekday and a weekend.

Peak factor coefficients vary worldwide with values ranging from 1.35 to 3.40 (Brandt, et al., 2017). Table 2.1 shows common peak factors used in the design of different water distribution networks all over the world.

Table 2.1: Range of Peak Factors used in various parts of the World (Brandt, et al., 2017)

	Location	Peak Factor
1.	Worldwide. Cities with hot dry summers	1.35 - 1.45
2.	Worldwide. Cities in equable climates	1.25 - 1.35
3.	Worldwide. Cities with substantial industrial demand	1.10 - 1.25
4.	UK. Seaside and holiday resorts	1.30 - 1.50
5.	UK. Residential towns, rural areas	1.20 - 1.30
6.	UK. Industrial towns	1.15 - 1.25
7.	UK. Peak due to garden watering in prolonged hot dry weather	1.50 - 1.70
8.	USA. Typical peak domestic demands – in-house only Western states Eastern states	 1.80 - 1.90 1.30 - 1.40

	Location	Peak Factor
9.	USA. Typical peak domestic demands due to lawn sprinkling Western states Eastern states	 2.20 - 3.40 2.00 - 3.00

Peak factors affect water flow in distribution networks therefore impact the pipelines sizes and the system costs (Gurung, et al., 2016). In Kenya, most principal towns and urban centres utilize water the entire day lowering the need for a high peak factor. Consequently, a range of peak factor coefficients between 1.0 and 1.2 are commonly used in design.

2.8 Water Loss and Non-Revenue Water (NRW)

Water loss is any water that gets into the distribution network upon treatment but does not get to users (IWA, 2012). According to International Water Association (2003) water losses are classified as: physical (real) and apparent losses. Real losses include leakages from pipes rough bursts and ruptures, leakage at joint fittings, overflows from reservoirs, apparent losses comprise of losses due to metering errors, unmetered water, data handling errors and illegal water tapping. Factors that affect water losses include the pressures in the pipelines; lengths of water mains; the water distribution network complexity, type of leakage controls. (Islam, et al., 2011). Additionally, the quantity of water loss varies from one utility to the next depending on factors such as topography, length of water mains, the number of connections and standard service and level of network operation and maintenance (Lambert & Hirner, 2000).

Real and apparent water losses in addition to unbilled water are often referred to as Non-Revenue Water (NRW) (Kamani, et al., 2012). Non-Revenue Water (NRW) is the difference between system input volume and billed authorized consumption (Alegre, et al., 2016) where system input volume is annual input to a defined part of the water supply system. Unbilled water is utility operation water including pipelines flushing water, firefighting water, and free water that certain users such as water supplied by water utilities to informal settlement dwellers.

The International Water Association (IWA) Task Force of 2000 stipulated an international ‘best practice’ standard approach for water balance calculations and the first step was the universal definition of terms to enable practical management of water losses (Lambert & Hirner, 2000; Alegre, et al., 2016). The IWA standard ‘best practice’ water balance is shown in Table 2.2.

Table 2.2: IWA Standard Water Balance (IWA Water Loss Task Force, 2000)

System Input Volume	Authorized Consumption	Billed Authorized consumption	Billed metered consumption (including water exported)	Revenue Water
			Billed unmetered consumption	
		Unbilled Authorized Consumption	Unbilled metered consumption	Non-Revenue Water (NRW)
			Unbilled unmetered consumption e.g. fire demand	
	Water losses	Apparent Losses	Unauthorized consumption e.g. illegal connections, meter tampering or bypassing	
			Customer metering inaccuracies	
		Real Losses	Leakage from transmission and distribution mains	
			Leakage and overflows at utility's storage facilities	
			Leakage from service connections up to the consumer's meter	

Real water losses cannot be completely eliminated from a network hence the concept of unavoidable annual real losses (UARL). Unavoidable annual real water loss is the lowest real water loss that can be achieved for a well-managed network (IWA, 2000). To reduce real losses IWA advocates for four strategies: water pressure management, active leakage control, water infrastructure and asset management and speed and quality of repairs (Lambert, 2003). The four strategies are shown in Figure 2.4.

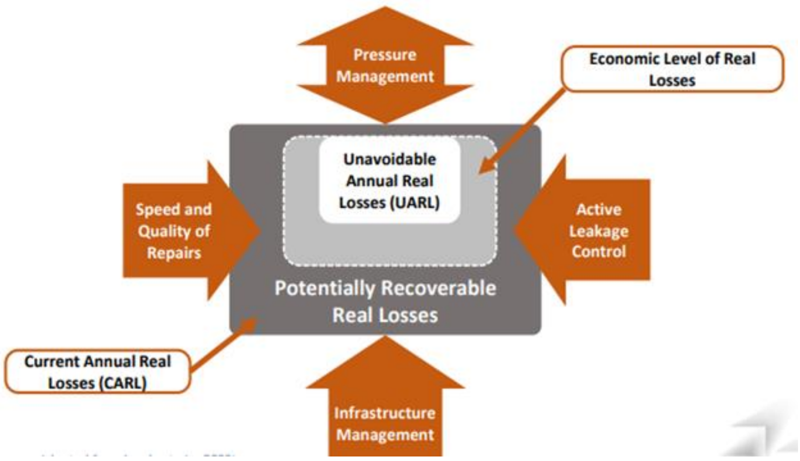


Figure 2.4: Methods of Managing Real Water Losses (Lambert, 2003)

The World Bank estimates that the actual figure for overall NRW levels in the developing world is probably in the range of 40 - 50% of the water produced (GIZ, 2011). Many urban and rural areas in Kenya with piped water have unaccounted for losses of up to 50% of the supply (United Nations, 2005).

2.8.1 District Metering as a Tool of Water Loss Management

District Metering Areas (DMAs) are zones within a distribution network that use bulk meters to monitor water flow and help to locate areas with losses and have been adopted in many distribution networks to manage NRW (Nardo, et al., 2013; Morrison, et al., 2017). A DMA is a leakage monitoring approach that requires installation of flow meters at strategic points throughout the distribution network. A DMA, therefore, is a hydraulically isolated area in a distribution network whose water flow and physical water losses can be monitored using bulk water meters (Farley, 2001). DMA meters can be equipped with telemetry data loggers, to allow continuous monitoring of zone consumption from which an estimate of leakage can be made. Isolation of a DMA area can be by closure of valves or by delinking (permanent pipes disconnection to neighboring areas) the distribution network entirely (Morrison, et al., 2007).

Morrison et al (2017 and 2007), Nardo et al (2013) and Farley (2001) highlight common advantages of establishing DMAs as an active leakage control technique. DMAs divide distribution network into manageable areas and facilitate quick leakage identification. The operations and maintenance teams can therefore prioritize repairs depending on the severity of leakages at different points of the network. Prompt repairs prevent water contamination through infiltrations and increase water supply to users due to minimized losses. In addition, pipeline asset life is increased strengthening the networks economic viability.

2.9 Robustness of a Distribution Network

A water distribution network needs to be resilient enough to recover from a catastrophic failure such as an earthquake or intentional contamination (Jung, et al., 2019). Jung, et al (2019) considered robustness of a water distribution network a key component in the network resilience. Robustness is the ability of a water distribution network to always supply quality and adequate water to users irrespective of variability in demand. Therefore, robustness of a distribution network is an important component of a distribution network.

Water demand varies with time. The demand can either increase or decrease. A decrease in demand has no major effect in the network supply while an increase in demand can strain the distribution network. Increase in demand could result from unforeseen future expansion of the supply area resulting in unexpected growth of industries and settlements. Other unforeseen higher demands could be because of higher population growth rates in the supply area as compared to those anticipated in the design of the network.

The robustness of greenfield distribution and pumped networks can be increased by installing bigger pipes. The robustness of an existing distribution network can be evaluated by varying flow in the pipelines. To vary flow, a hydraulic model of the existing distribution network is created and vary the model peak factor to increase the flow in the network. Model simulation analysis results are used to evaluate the robustness of the network.

2.10 Hydraulic Network Modelling

Network Modelling is an approach adopted by water network designers to optimize their designs, test rehabilitation and operation using network simulation software. Well calibrated models provide a platform to analyze a network under different conditions of operation. Modelling is routinely being adopted in daily operations of water utilities to facilitate proactive maintenance as compared to the old reactive maintenance. The results culminate in better efficiency in water head management, better prediction of bursts and identification of bursts in the network and ultimately improved services to the consumers.

These programs are built on basic water flow head loss equations of Hazen-Williams, Darcy - Weisbach and Chezy-Mannings. Some of the programs include optimization that represents a more detailed representation of operating policies. All contain menu-driven graphics-based interfaces that facilitate user interaction (Robert Ettema, 2000). These programs are appropriate for use in shared vision exercises involving stakeholder involvement in model building and simulations. The following subsections discuss two of the more common water distribution modelling software; EPANET and KYPipe Software.

2.10.1 EPANET

EPANET is a water modelling software developed in the United States by the Environmental Protection Agency (EPA) Water Supply and Water Resources Division in 1993. The software is used by designers to model water distribution networks, size new water infrastructure, analyze existing aging infrastructure, optimize operations of tanks and pumps, reduce energy usage, investigate water quality problems, and prepare for emergencies. The main advantage of EPANET is that it’s a public domain software that is compatible with all Windows based programs. Therefore, the software can be copied from one computer to another easily. In addition, EPANET is an open-source software, with continued development on the internet. Though a robust software, EPANET has faced criticism on its energy calculation. EPANET calculates incorrectly the efficiency of variable speed pumps and ignores natural energy in distribution of pressurized water (Gómez Sellés, et al., 2016).

EPANET models consists of pipes, nodes, and other network infrastructure such as tanks, reservoirs, valves. These are developed on the graphic user interface which has a visual network editor. EPANET also provides water quality modelling and water security and resilience modelling through the software extensions EPANET-MSX (Multi-Species eXtension) and EPANET-RTX (Real-Time eXtension).

2.10.2 KYPipe Software

KYPipe Network Modelling Software was developed by KYPIPE LL (USA) at the University of Kentucky. KYPipe Software is used for Hydraulic Network Analysis, Transient Analysis and Compressible Gas Analysis. The software can model networks of water, petroleum, and chemicals.

The over 40-year-old software has been continually updated and maintained and it is one of the most widely used and trusted software for hydraulic analysis in the world. KYPipe is easy to use and utilizes colour graphics extensively to show networks and simulation results. It is user friendly and has an attractive and an interrogative interface as shown in Figure 2.5.

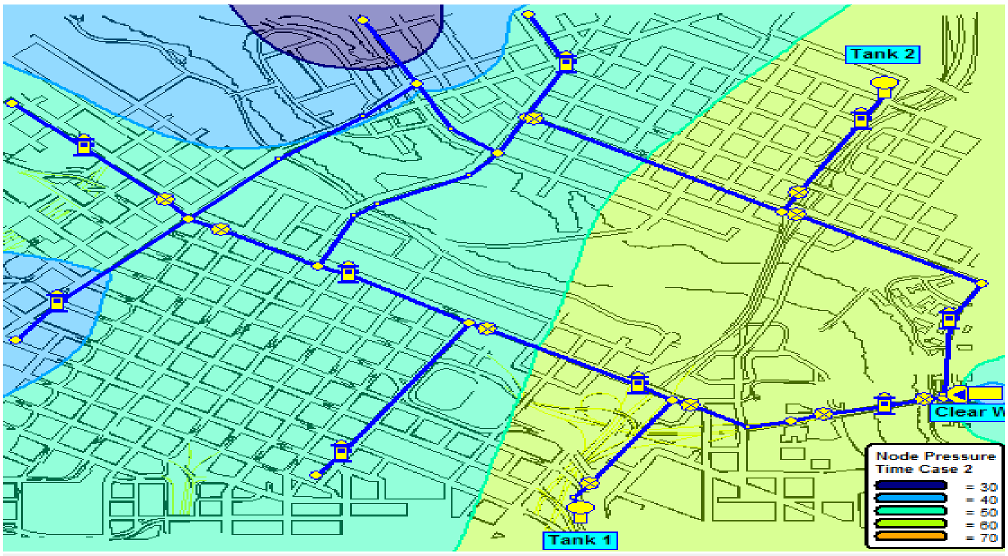


Figure 2.5: Nodal Pressure Contours (KYPIPE Manual-2016)

KYPipe is used for selecting and sizing pipes, pumps, valves, tanks. The calibration tools and pump operation optimization features help ensure sound modelling with high degree of accuracy.

Some other advantages of KYPipe are as follows:

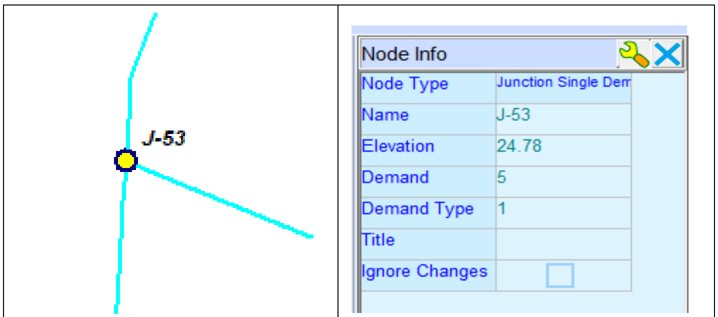
i. KYPipe is simple and easy to use.
ii. Fully representative in all design sectors.
iii. KYPipe is robust, precise and reliable.
iv. The software gives clear and complete results after network analysis.
v. There is free round the clock technical support for all KYPipe users.

2.10.2.1 KYPipe Software Inputs.

When KYPipe 2016 Software is used for Hydraulic Network Analysis, some key data input is required. Inputs into KYPipe 2016 to accomplish network modelling include:

i) A geo-refenced network layout in dxf / dwg format- This is imported in KYPipe as a background image to enable tracing/ digitization of the alignments that form the network. Where pipelines in the model meet, a junction/ node is formed.

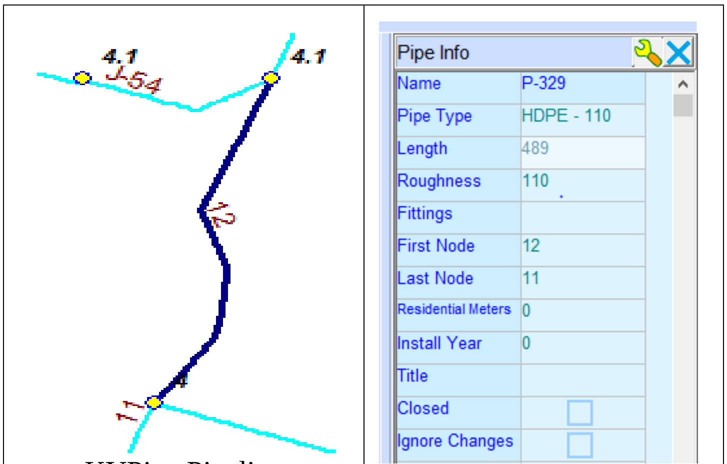
ii) Nodal/ junction ground elevations - These are obtained from a Digital Terrain Model (DTM) extracted from Global Mapper software and manually input into the node properties. A typical model node and its attributes are shown in Figure 2.6 below.



KYPipe Node/ Junction | KYPipe Node Properties

Figure 2.6: KYPipe 2016 node and its properties

iii) Pipeline properties/ attributes - These include pipeline diameters, materials and roughness coefficients. A typical modelled water pipe and its attributes are shown in Figure 2.7 below.



KYPipe Pipeline

Pipeline Properties

Figure 2.7: KYPipe 2016 Pipeline and Pipeline Attributes

iv) Nodal/ junction water use – Nodes serve as user connection points. Area water demand is distributed in the model nodes/ junctions accordingly. The demand allocated to each node is dependent on the area’s land use, residents’ consumer category, population densities and institutions in the area.

v) Reservoirs – These serve as water sources in the model. Reservoirs are modelled as Fixed Grade Node (FGN) assuming unlimited water supply into the distribution network.

2.10.3 Comparison of KYPipe and EPANET

EPANET and KYPipe software have many functionality similarities and differences as summarized in Table 2.3. KYPipe Software is more robust than EPANET and has easy compatibility with other common software such as Arc GIS and Global Mapper that require shapefiles.

Table 2.3: Functionality comparison between KYPipe and EPANET

FEATURES	**KYPIPE**	**EPANET**
Additional Modules	Yes	Yes
Devices (Pumps, Valves, Flow Control)	Yes	Yes
Calculate Darcy Weisbach/ Hazen-Williams/ Chezy-Mannings Methods	Yes	Yes
Minor Head Loss for Bends & Fittings	Yes	Yes
Compute Piping and Energy Cost	Yes	Yes
Consider multiple demand types at nodes	Yes	Yes
Tanks to take any shape	Yes	Yes
Technical Support	Yes (Free)	No
Meters	Yes	No
Branch Diameter Analysis	Yes	No
Import/Export AutoCAD and GIS Files	Yes	No

FEATURES	KYPIPE	EPANET
Import Internet Maps & Elevations	Yes	No
Locate Pressure Zones	Yes	No
Automatic Demand Distribution	Yes	No
Model Calibration	Yes	No
Multiple Pumps (series, parallel), Constant Head	Yes	No
Elevation Retrieval	Yes	No
Optimal Pump Scheduling	Yes	No
Import & Export Shape Files	Yes	No
Presentation Generator	Yes	No
CAD, GIS & Graphical Image Background Maps	Yes	No
Internet Background Maps	Yes	No
Certified Training Courses	Yes	No
Video Tutorials – online and on CD	Yes (Free)	No
Excel Import/Export	Yes	No
SCADA Connection	Yes	No
Pipe Schedules	Yes	No

2.10.4 Flow variation in KYPipe affected pipes

KYPipe software allows the user to vary the flows using a global factor provided that the distribution network remains constant. Therefore, upon model development and incorporation of demand, flows can be varied uniformly using a peak factor to simulate higher flows in the network. However, flow can also be varied on individual pipe nodes but this results to increased flow in affected pipe.

2.11 Modelling Criteria

2.11.1 Pressures and Velocities

The design criteria of the pipelines in the hydraulic network models were according to the Ministry of Water and Irrigation Practice Manual for water supply services in Kenya 2005. Table 2.4 below gives a summary of acceptable ranges of pressures and velocities in a distribution network.

Table 2.4: Pipeline Design Criteria

Parameter	Criteria Adopted	Justifications and Limitations
Minimum Flow Velocity	0.01 m/s	Reduce the possibility of sediments settling in the pipes
Maximum Flow Velocity	2.0 m/s	Minimize water hammer effects. Minimize head loss in the pipe

Parameter	Criteria Adopted	Justifications and Limitations
Minimum Pressure	0.15 MPa (15 m)	Pressure in pipes in areas with firefighting facilities
	0.1 MPa (10 m)	Pressure in pipes in areas without fire-fighting facilities **Caveat** Minimum pressure limit is terminal reservoir dependent. Pressures are lower in areas around the terminal reservoir due to a small elevation difference. In this case areas close to Nguu Tatu reservoir in Mombasa North Mainland distribution network are likely to have pressures lower than 0.1 MPa
Maximum Pressure	0.6 MPa (60 m)	This limit is to prevent damage on the fittings at the consumer connection point. **Caveat** This pressure limit is terrain dependent. The maximum pressure limit might be exceeded in occasions where the terrain makes higher pressures unavoidable

2.11.2 Friction Losses

Energy loss in flow is caused wall fictional losses and turbulence caused by change of direction or area of flow. Pipe friction losses are continuous over the length of the pipe and form the major head loss in a pipe (Brandt, et al., 2017).

To calculate friction losses, KYPipe has three friction formulas built in to choose from. These are Darcy-Weisbach (Colebrook-White) Equation, Hazen Williams Equation and Chezy-Manning's Equation.

a) Darcy Weisbach (Colebrook-White) Equation

The Darcy-Weisbach equation is a dimensionally correct formulae for friction loss. The Darcy – Weisbach Formula gives head loss in a pipe as follows:

$$H_L = \frac{\lambda L V^2}{2gd} \quad \text{.. Equation 2.1}$$

Where:

H_L	=	Head Loss in a Length of Pipe
λ	=	Frictional factor (non-dimensional coefficient)
L	=	Length of pipe in metres
V	=	Velocity of flow (m/s)
g	=	gravitational acceleration

d = Internal diameter of the pipe in metres

Colebrook and White showed that λ in the Darcy-Weisbach formula is a function of the relative roughness of the pipe surface, the viscosity of the flow and the Reynolds number, Re. Therefore Darcy-Weisbach equation is a theoretically based equation commonly used in the analysis of pressure pipe systems. It applies equally well to any flow rate and any incompressible fluid and is general enough to be applied to open channel flow systems.

Design charts / Diagrams for friction loss calculation in pipelines based on the Darcy-Weisbach (Colebrook-White) equation are available for use. The main disadvantage of Darcy-Weisbach (Colebrook-White) equation is the cumbersome iterative calculations involved. However, the advent of computers and published Design Charts have eliminated this disadvantage.

b) Hazen Williams Formula

Frictional losses in pipelines in the hydraulic model were calculated based on the

Hazen–William's equation is an empirical relationship relating the flow of water in a pipe with the physical properties of the pipe and the pressure drop caused by friction (Twort, et al., 2000). It is the most used formula for the calculation of pipe friction losses. Hazen Williams equation is most accurate for the pipe sizes and velocities typically found in water supply practice. The flow in the water distribution pipes is in the intermediate zone in the Moody diagram and the Hazen Williams formula can be applied with reasonable accuracy provided that velocity and pipe size do not vary greatly from these values (Brandt, et al., 2017). Therefore, Hazen Williams formula was adopted in the KYPipe modelling of Mombasa North distribution network.

Determination of Frictional Head Losses in pipelines using the Hazen Williams Formula is based on the following formula:

$$\mathrm{H} = \frac{6.78l}{d^{1.165}}\left(\frac{v}{C}\right)^{1.85} \text{ .... Equation 2.1}$$

Where:

H = Frictional head loss (m of water)

l = Length of pipe in metres

d = Internal diameter of the pipe in metres

v = Velocity of flow (m/s)

C = Roughness Coefficient

The Roughness Coefficient (C) varies depending on the Material and Condition/Age of the pipe as summarized in Table 2.5. These values of C include for minor losses due to fittings such as bends along the pipeline. The C value adopted in pipeline design for hydraulic modelling is 110 considering the age of the pipelines.

Table 2.5: Roughness Coefficients (C) for various Pipe Material and Condition/Age (Twort et al., 2000)

	Pipe Condition	**Good**	**Average**	**Poor**	**Very**
	Pipe Age	**< 10 Years**	**10 to 40 Years**	**>40 Years**	**Corroded**
	Pipe Type				
1.	uPVC, HDPE	150	140	140	-
2.	CI, DI, Steel with cement mortar lining, AC and Concrete	120	110	100	90
3.	Unlined Steel, GMS, CI	130	110	90	80

c) Manning's Equation

Manning's equation is used when flow is turbulent with either a high Reynolds numbers or when the conduit is particularly rough. Often, its used with open channel flow and not generally recommended for pipeline systems, except possibly in large, rough conduits such as unlined rock tunnels (Brandt, et al., 2017).

The Manning's equation is written in the form:

$$V = \frac{(R^{\frac{2}{3}} i^{\frac{1}{2}})}{n} \text{.. Equation 2.3}$$

Where:

R = Hydraulic mean depth/ hydraulic radius

i = Hydraulic gradient (H/L)

n = Roughness/ Manning coefficient

2.12 Implications of Literature Review

Mapping and then modelling of the networks serves to document the pipelines. Increasing the flow of the networks in the model by varying peak factors from 1.2 to 1.5 and 2.0 establishes the robustness of the networks in terms of pipe sizing. The results of hydraulic simulations indicate expected pipeline pressures and velocities at different sections of the network.

CHAPTER THREE

3. RESEARCH METHODOLOGY

3.1 Introduction

This chapter describes the methods used to meet the overall study objective of evaluating the robustness of water distribution networks by varying flows in hydraulic models using different peak factors, with Mombasa North Mainland Water Distribution Network as a case study. It describes the study area and its distribution network, the data used and their sources, software used to map and analyze the network and the modelling methodology and procedures used in variation of peak factors.

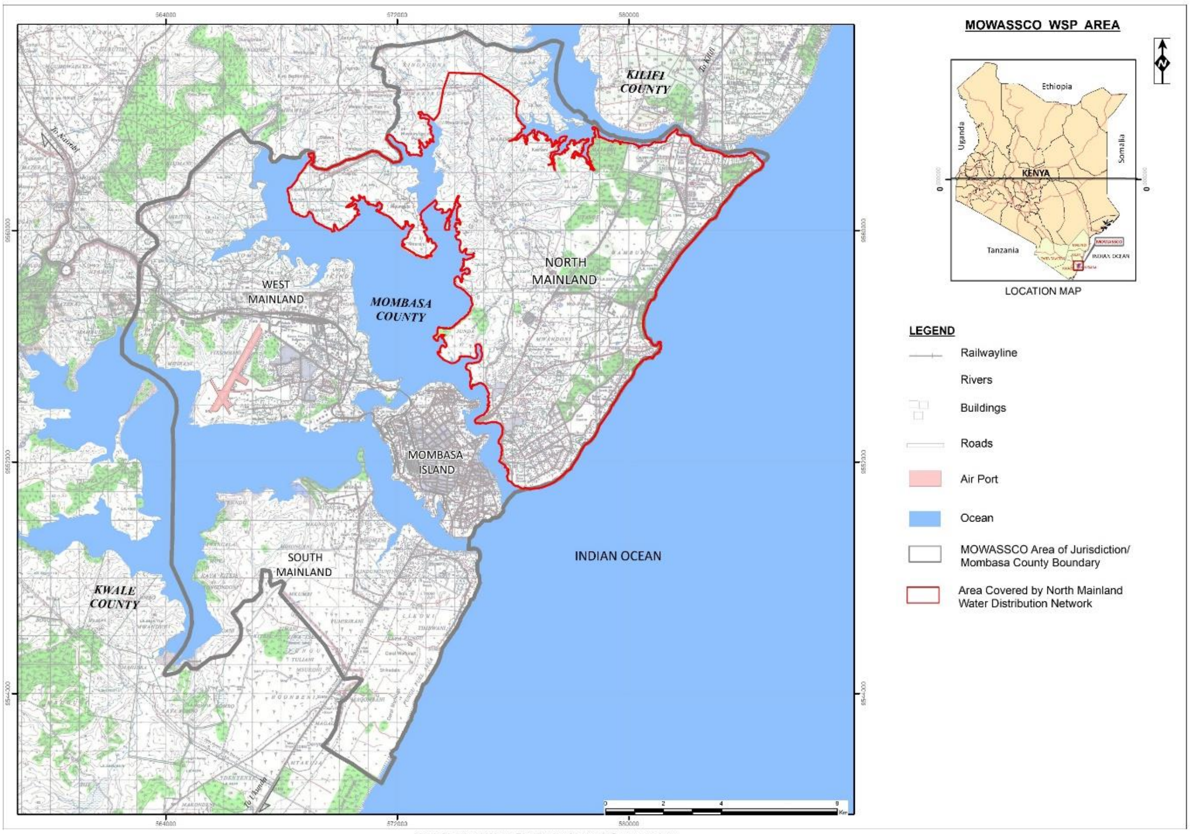
3.2 Description of the Study Area

3.2.1 Study Area – Mombasa North Mainland

Mombasa North Mainland is one of the four distinct geographical areas of Mombasa County in Kenya among Mombasa Island, Mombasa South Mainland, and Mombasa West Mainland. Mombasa County is located at the Kenya coastal strip bordering the Indian Ocean. This study concentrated on Mombasa North Mainland area within Mombasa County.

Mombasa North Mainland covers Kisauni and Nyali Sub Counties in Kisauni Division and has an approximate population of 508,507 (Kenya National Bureau of Statistics, 2019). The water distribution network in this area predominantly supplies residential houses and several other commercial centers. Nyali, is a high-income residential area originally occupied by single dwelling houses but now has multi-storey rental houses cropping up due to devolution. The Mombasa North Mainland coastal strip area is made up of sandy beaches occupied by tourist hotels and restaurants. All these developments put more pressure on the existing distribution network infrastructure to supply adequate water of the required quantity and quality.

Mombasa North Mainland area is under the jurisdiction of Mombasa Water Supply and Sanitation Company Limited (MOWASSCO) which is the water service provider tasked with the mandate to supply the entire Mombasa County with water. Figure 3.1 show Mombasa County Boundary (black demarcation) and Mombasa North Mainland Study Area (red demarcation).



North Mainland Water Distribution Network Coverage Area

Figure 3.1 Mombasa North Mainland Water Distribution Network Coverage Area

3.2.2 Mombasa North Mainland Water Supply

Water in Mombasa North Mainland is supplied through Sabaki Pipeline from Baricho Wellfields situated along Sabaki River. The Sabaki Pipeline terminates at Nguu Tatu Reservoirs that are located at Nguu Tatu Hill from where water gravitates into Mombasa North Mainland distribution network. The Nguu Tatu Reservoirs are at an elevation of 66 m above mean sea level (amsl). Three ductile iron water mains start from the tank to supply the network. Two of the main lines are of 700 mm nominal diameter (DN) each while the third water main has a nominal diameter of 1200 mm.

The three main lines from Nguu Tatu Reservoirs run parallel on Kengelani Road then diverge to follow different alignments until they join the two main roads traversing across Mombasa North Mainland at different locations. These roads are Mombasa - Malindi Road (Road B-8) and Coral Road. Coral Road branches from Mombasa-Malindi Road at approximately 300 metres from Nyali Bridge which connects Mombasa North Mainland to Mombasa Island. The primary water mains then split off into different diameter lines that follow roads alignments to supply the northern areas including Kiembeni, Shimo la Tewa, Shanzu, Ndengerekeni and the southern areas of Kiasuni, Nyali, Mwembe, Mwatamba, Bamburi. From the main lines, other secondary, tertiary and service water mains along smaller feeder roads and paths interconnect to cover the entire Mombasa North Mainland area.

Mombasa North Mainland network is a typical distribution network in Kenya. The network has been extended overtime to accommodate new connections and has become heavily looped. The water service provider proposes to rehabilitate and extend the network following a distribution network master plan formulated in the year 2017. The masterplan proposals are designed for implementation up to ultimate year 2040.

The 2017 Distribution Network Masterplan proposed three water distribution networks as follows:

i. Immediate design year 2020 water demand.
ii. Future design year 2030 water demand.
iii. Ultimate design year 2040 water demand.

3.2.2.1 Mapping of Mombasa North Mainland Network

This study mapped the three existing and proposed Mombasa North Mainland water distribution networks for the immediate design year 2020 water demand, future design year 2030 water demand and ultimate design year 2040 water demand. Figure 3.2 shows the layout of Mombasa North Mainland distribution network for the ultimate design year 2040. These distribution networks were used to develop three hydraulic models for year 2020, 2030 and 2040 using a peak factor of 1.2 that is typically used in design of the water supplies in Kenya and the network parameters observed. The models were then subjected to robustness test for pressure and velocities using 1.5 and 2.0 peak factors to vary the flow. Where simulation results showed a model was inadequate in terms of the design pressures and velocity, pipe capacity augmentations were proposed accordingly.

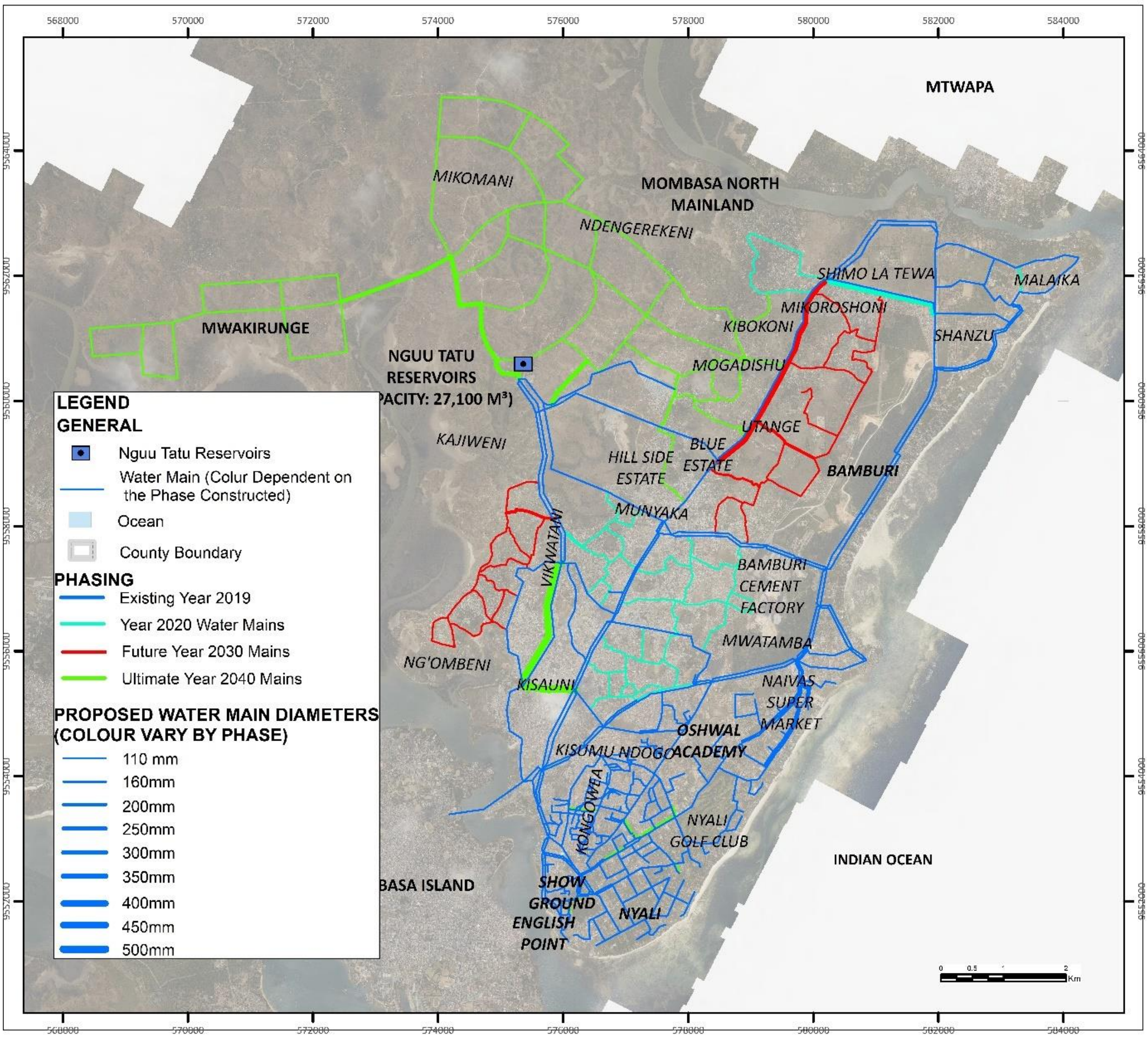


Figure 3.2 The Mombasa North Mainland Distribution Network for year 2040

3.3 Data Collection

3.3.1 Data Type and Sources

To achieve the objectives of this study, primary and secondary data was utilized. Primary data included identification and mapping of some of Mombasa North Mainland pipelines alignments using a Geographic Positioning System (GPS) receiver with the assistance of the water service provider. Pipelines were mapped as shapefiles while key locations were recorded as co-ordinates.

The secondary data was extracted from Google Earth and Global Mapper v18 software while other data was obtained from MOWASSCO GIS Department. The study area boundary was extracted from Google Earth while Mombasa North Mainland background image and contours were extracted from Global Mapper. MOWASSCO GIS Department provided alignments of pipelines proposed in the master plan as shapefiles. Attributes in the shapefiles were pipeline diameters, materials, and conditions.

The study evaluated pipelines of diameters equal or greater than 100 mm. Pipes with diameters less than 100 mm were considered as tertiary and service lines that connect to consumer connections and were not modeled.

3.3.2 Software and Hardware

Hardware resources used in this study were a GPS Receiver to map alignments and a laptop. The software used included ArcGIS 10.5 to load shapefiles data and generate maps, Global Mapper v18 to extract study area images and contours, Google Earth to extract study area boundaries, KYPipe 2016 hydraulic modelling software to model Mombasa North Mainland distribution networks and AutoCAD to polish the pressure maps.

3.4 KYPipe 2016 Modelling

Alignment shapefiles both mapped and those obtained from GIS department were first opened in Global Mapper. These were either existing or proposed pipelines depending on whether they have been constructed or not. Proposed pipelines were further sub divided into three categories namely immediate pipelines (under construction), future pipelines and ultimate pipelines depending on the timelines when the works are proposed to be done. All these categories of pipelines were colour coded for easy visual identification. Table 3.1 below illustrates different categories of pipelines and their adopted colour code.

Table 3.1: Adopted Pipe Colour Codes for different Construction Phases

Pipe State	Pipeline Category	Construction Timelines (Years)	Pipe Colour Code
Existing	Existing	Earlier than 2017	Blue
	Recently Constructed	2017 - 2019	Black
Proposed	Immediate – Under Construction	2020	Cyan
	Future	2030	Red
	Ultimate	2040	Green

3.4.1 Development of Hydraulic Models

The following steps were followed in development of Mombasa North Mainland distribution network hydraulic models in KYPipe:

i) Colour-coded alignments were first geo-referenced and exported form Global Mapper as a dxf file.

ii) The dxf layout was imported into KYPipe 2016 as a background image and traced/ digitized to create Mombasa North Mainland pipeline network. Intersection of pipes formed nodes/ junctions.

iii) Ground elevations for all nodes/ junctions were keyed in KYPipe 2016 network node attributes. The elevations were obtained from a Digital Terrain Model (DTM) extracted from Global Mapper.

iv) Pipe materials, diameters, and friction co-efficient extracted from alignment shapefile attributes were then defined for each pipeline in the network.

v) Model nodes were then allocated demand according to the enumeration areas with the quantity allocated on a node dependent on the area's land use, population density and institutions in that area. Mombasa North Mainland projected water demand incorporating a 20% allowance for water losses through leakages and wastage is summarized in Table A1 in Appendix 1.0. The 20% water leakage allowance is the industry water losses target as per the Water Design Manual from the Ministry of Water and Irrigation in the year 2005, used to project water consumption rates in the county.

vi) Nguu Tatu terminal reservoir was modelled as a Fixed Grade Node with unlimited water supply.

Figure 3.3 below shows a typical KYPipe 2016 model displaying pipes, junctions, and direction of water flow.

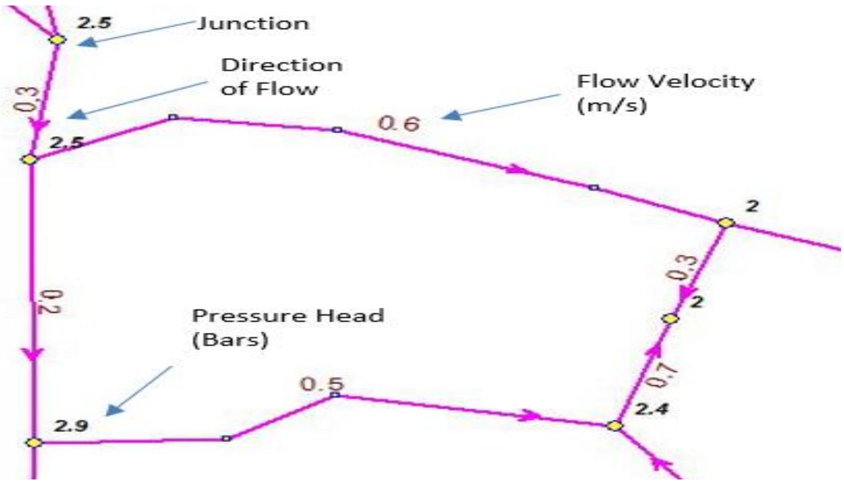


Figure 3.3: Typical KYPipe 2016 Model network and results of simulation analysis (KYPipe Manual, 2016)

3.4.2 Hydraulic Models Developed.

Following the modelling procedure and factoring total demand values given in Table A1 with a peak factor coefficient of 1.2, three models were developed.

1) Immediate Model – This model incorporated all existing pipes and proposed immediate works pipes. The demand distributed in the model nodes is the projected 2020 demand which was 98,607 m^3/day.
2) Future Model – This model was developed by incorporating proposed future works pipes into the Immediate Model. The demand distributed in the model nodes is the projected 2030 demand which is 142,893 m^3/day.
3) Ultimate Model – This model incorporated ultimate works pipes into the Future Model. The demand distributed in the model nodes is the projected 2040 demand which is 190,003 m^3/day.

Model simulations were run and analysis results including nodal pressures and pipe velocities tabulated. The analysis reports have been set to show the most 10 most critical junction pressures and pipes with critical velocities in both extremes (maximum and minimum).

3.4.3 Generation of Pressure Contour Maps

Pressure contour maps for three Master Plan models (1.2 peak factor) have first been generated from their respective KYPipe 2016 models; immediate model; future model and ultimate model. These contour maps have then been exported to AutoCAD for polishing and presentation. The same has been repeated for all peaked models.

3.5 Model calibration and Validation

Model calibration was key for results validation and consistency while the purpose of the model determined the extent of model calibration required.

Calibrating existing network model involved vetting of the water service provider database that contained parameters such as pipe sizes, diameters, materials, and year of installation. However, data for such an extensive network has been marred with errors and incompleteness, which posed a challenge to the study. This is due to lack of design documentation and other as-built drawings.

This study focus was to evaluate the robustness of water distribution network. Modelling of the Mombasa North Mainland distribution network was a means to achieve the robustness evaluation. Therefore, model calibration and validation were restrained to data vetting and error checks. However, in future with adoption of technology and proper record keeping, the Software KYPipe 2016 enables integration between Geographic Information Systems (GIS), Supervisory Control and Data Acquisition (SCADA) and Customer Information Systems (CIS) databases. These can provide immediate feedback that can continually be fed into the models to ensure accuracy and consistency of the model output.

3.6 Network Robustness Testing at 1.5 and 2.0 Peak Factors

Robustness tests on future year 2030 and ultimate year 2040 models were carried out by varying the nodal flows using a peak factor of 1.5 and 2.0. In the calculation of Mombasa North Mainland demand, water consumption figures included about 20% allowance for water losses through leakage and wastage. In varying flows using different peak factors, water loss was not varied.

Table 3.2 illustrates the concept and criteria adopted for calculating flows in Mombasa North Mainland, while Table 3.3 gives the computed values of varied flows using peak factors 1.5 and 2.0 respectively.

Table 3.2: Criteria for Calculation of Master Plan Flows and Modelling Flows

North Mainland Flows given in the Master Plan				
Total Master Plan Flow (1.0 Peak Factor)	=	Actual Flow*	+	NRW (20 % of Total Master Plan Flow)
North Mainland Flow using a Peak Factor of 1.2 as proposed in the Master Plan				
Master Plan Flow (1.2 Peak Factor)	=	1.2 x Actual Flow	+	NRW (20 % of Total Master Plan Flow)
Calculation of Flow using a Peak Factor of 1.5				
Robustness Test Flow (1.5 Peak Factor)	=	1.5 x Actual Flow	+	NRW (20 % of Total Master Plan Flow)
Calculation of Flow using a Peak Factor of 2.0				
Robustness Test Flow (2.0 Peak Factor)	=	2.0 x Actual Flow	+	NRW (20 % of Total Master Plan Flow)

**Actual Flow is the Demand calculated using rates that do not include water losses*

Table 3.3: North Mainland varied flows using 1.5 and 2.0 Peak Factors

Type of Flow	Design Flow (m^3/day)		
	Immediate Design year 2020	Future Design year 2030	Ultimate Design year 2040
Actual Flow	68,005	98,547	131,037
Master Plan Flow (1.2 Peak Factor)	98,607	142,893	190,003
Robustness Test Flow (1.5 Peak Factor)	119,009	172,463	229,317
Robustness Test Flow (2.0 Peak Factor)	153,011	221,731	294,832

3.6.1 System Robustness.

The following steps were followed to assess model robustness for future year 2030 and ultimate year 2040 models:

i) The Mombasa North Mainland future (2030) hydraulic model was analyzed using the varied flow obtained using a peak factor of 1.5. Results were tabulated and a pressure map generated. Depending on the model simulation results in terms of design criteria of pressures and velocities, pipeline capacity augmentations (if necessary) were proposed on the main lines emanating from Nguu Tatu Reservoir to make the model adequate. The resulting adequate future year 2030 hydraulic model analysis results and a pressure map were generated and a colour coded layout of the model made.

ii) Any pipeline capacity augmentations required in the varied future year 2030 model were propagated to ultimate year 2040 hydraulic model. Ultimate 2040 peaked flow obtained using a peak factor of 1.5 was applied on the model and simulation analysis run. Results of the varied ultimate year 2040 model were tabulated, and a pressure map generated. Depending on simulation results in relation to design criteria, more pipeline capacity augmentations were made (where necessary) for model adequacy. Afterwards, the resulting varied adequate ultimate year 2040 hydraulic model analysis results, pressure map and a colour coded layout of the model were generated.

iii) Steps (i) - (ii) were repeated by varying flow in the models using a peak factor of 2.0. This generated other flow varied models.

CHAPTER FOUR

4. RESULTS AND DISCUSSIONS

4.1 Introduction

This chapter presents the results of the evaluation of robustness of Mombasa North Mainland water distribution network by hydraulic modelling in KYPipe. The results are depicted using Global Mapper v18, AutoCAD and Arc GIS 10.5 software.

Three models prepared included a model of the existing network that comprised of pipelines proposed in the immediate 2020 flows and future and ultimate network models that included pipelines proposed for 2030 and 2040. The future year 2030 and ultimate year 2040 model flows were varied with peak factors of 1.5 and 2.0 to evaluate robustness of the networks for different flows. The following sections present the observed model parameters and associated discussions.

4.2 Mombasa North Mainland Distribution Network KYPipe Models

4.2.1 Model of Existing Mombasa North Mainland Network

The model of the existing North Mainland Network including pipelines that are under construction was developed in KYPipe software and incorporated the year 2020 daily flow of 98,607 m^3 (1,141.29 l/s) distributed accordingly to the pipeline nodes. Pipelines developed before the year 2017 are coloured blue, pipelines constructed between 2017 and 2019, are coloured black while immediate design pipelines (year 2020) that were under construction are coloured cyan as outlined in Table 3.1 of the Methodology. The resulting existing hydraulic model of Mombasa North Mainland network is shown on Figure 4.1.

4.2.2 The 2030 Mombasa North Mainland Network Model

The proposed future year 2030 hydraulic model of Mombasa North Mainland network was developed by incorporation of the 2030 design pipelines into the existing Network model which included pipelines proposed under the immediate year 2020 design. Therefore, 2030 model included all existing pipelines, immediate year 2020 pipelines and proposed year 2030 pipelines. Projected year 2030 flow of 142,893 m^3/day (1,653.85 l/s) for the area was distributed into model nodes and junctions as outlined in the methodology.

To differentiate construction phases of various network pipelines, pipelines proposed in the future year 2030 were coloured red as summarized in Table 3.1 under methodology. The resulting future year 2030 hydraulic model of Mombasa North Mainland network developed in KYPipe software is shown in Figure 4.2.

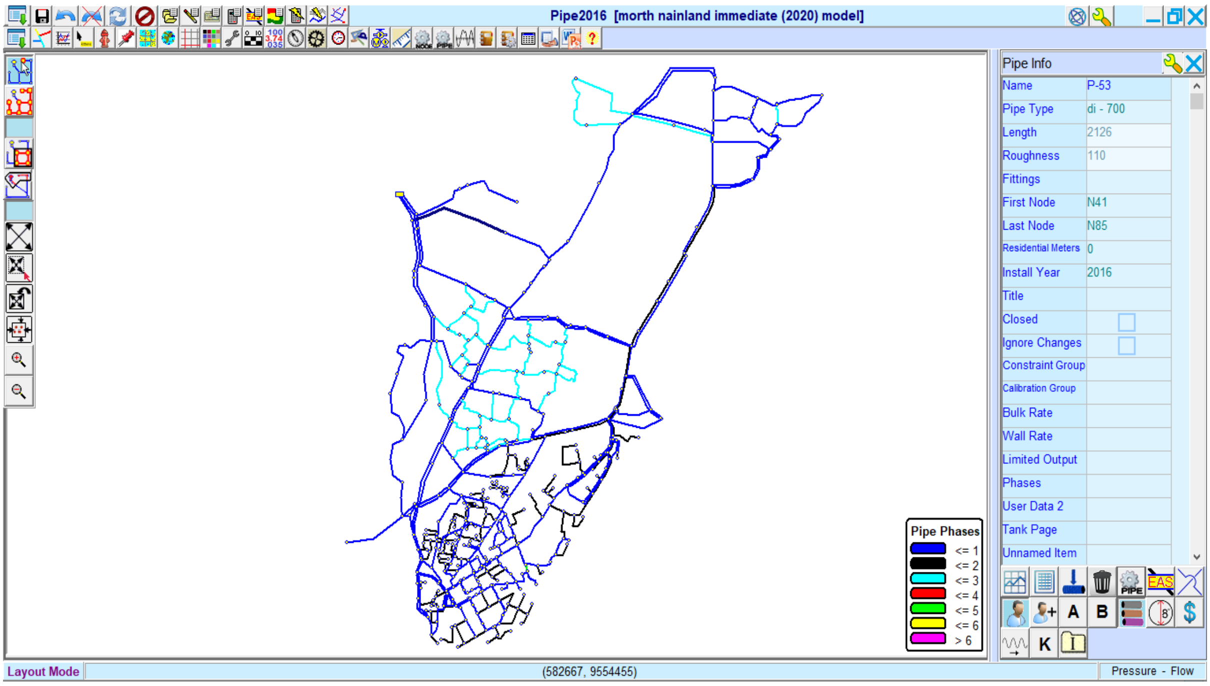


Figure 4.1: Existing Mombasa North Mainland Network KYPipe Model

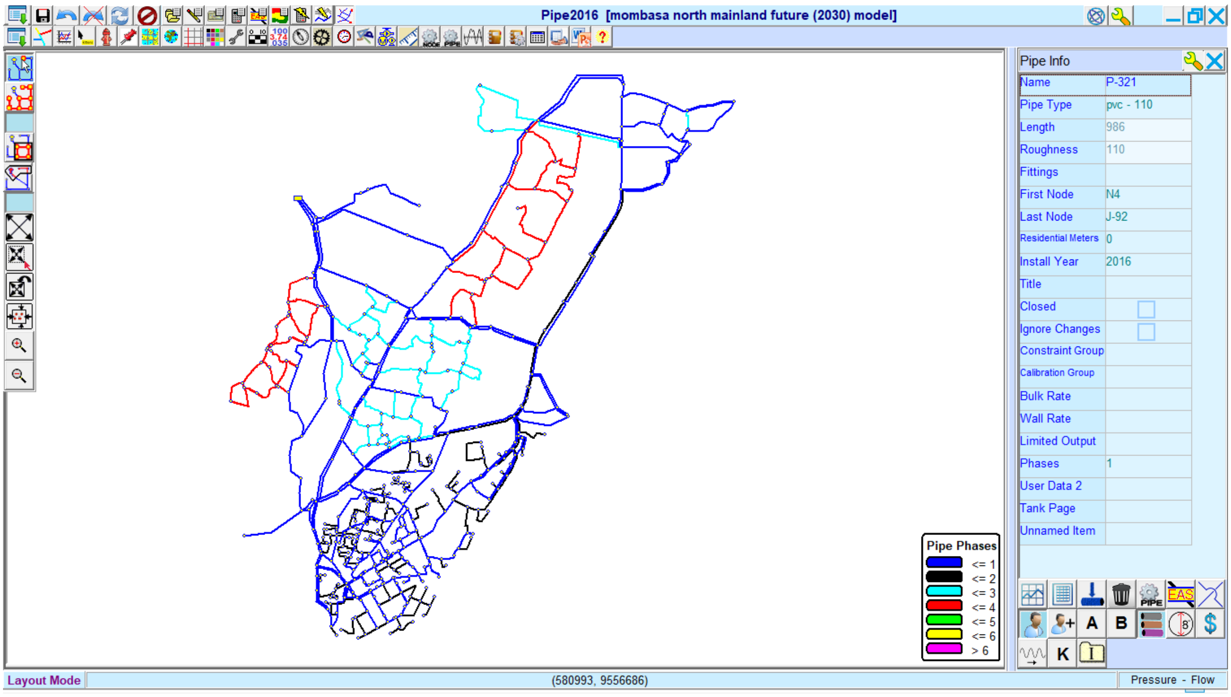


Figure 4.2: Year 2030 Mombasa North Mainland Network KYPipe Model.

4.2.3 The 2040 Mombasa North Mainland Network Model

Ultimate year 2040 North Mainland model was developed by adding the year 2040 design pipelines into the year 2030 model. The ultimate North Mainland model includes all the future year 2030 model water mains and all other pipelines proposed lines to be developed in the year

2040. Water mains proposed for implementation in 2040 were coloured green as summarized in Table 3.1 under methodology. Colour coding enabled differentiation of construction phases for various pipelines.

A daily flow of 190,003 m^3 (2,199.1l/s) projected for the area in the year 2040 was distributed into the ultimate year 2040 model as described in the methodology. This resulted to a complete study area model, encompassing all proposed pipelines up to design year 2040 as planned by the water service provider. Figure 4.3 shows the ultimate year 2040 North Mainland KYPipe Model.

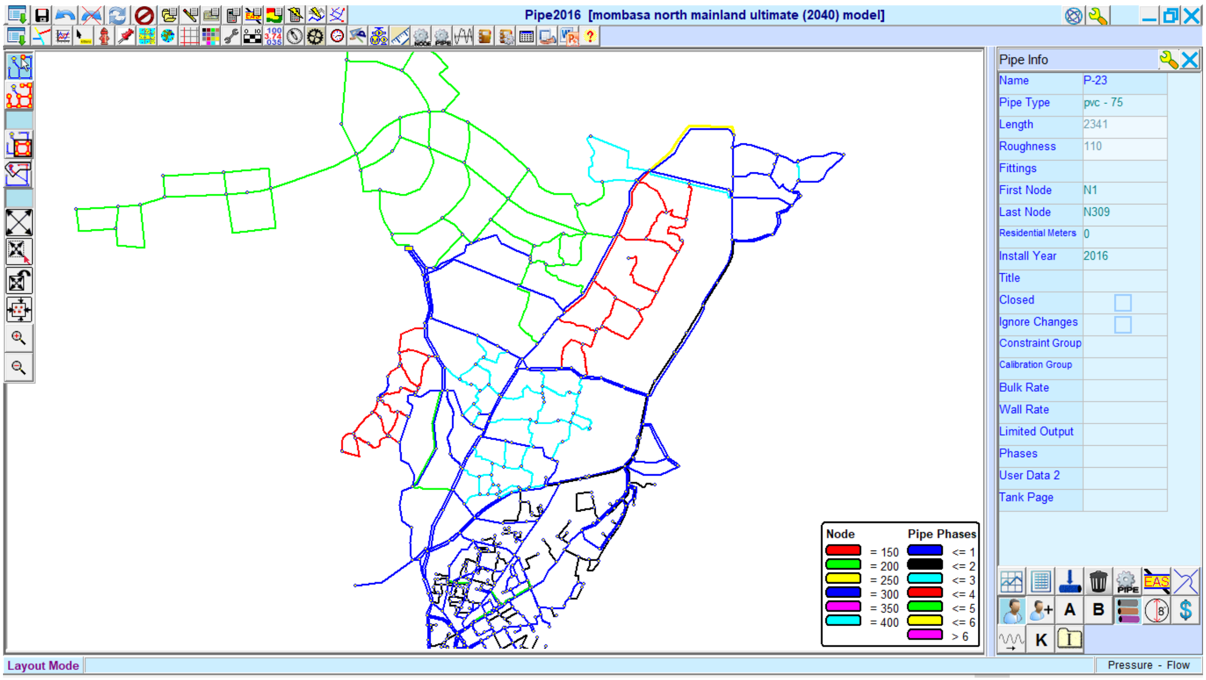


Figure 4.3: Year 2040 North Mainland Network Model in KYPipe

4.2.4 Model Layouts of Existing (Year 2020), Future (Year 2030) and Ultimate (Year 2040) Mombasa North Mainland Network

AutoCAD schematic layouts of existing hydraulic model including the immediate design (year 2020) pipelines, future year 2030 and ultimate year 2040 hydraulic model showing some model pipeline names, junctions and some node labels are shown in Figures 4.4, 4.5 and 4.6.

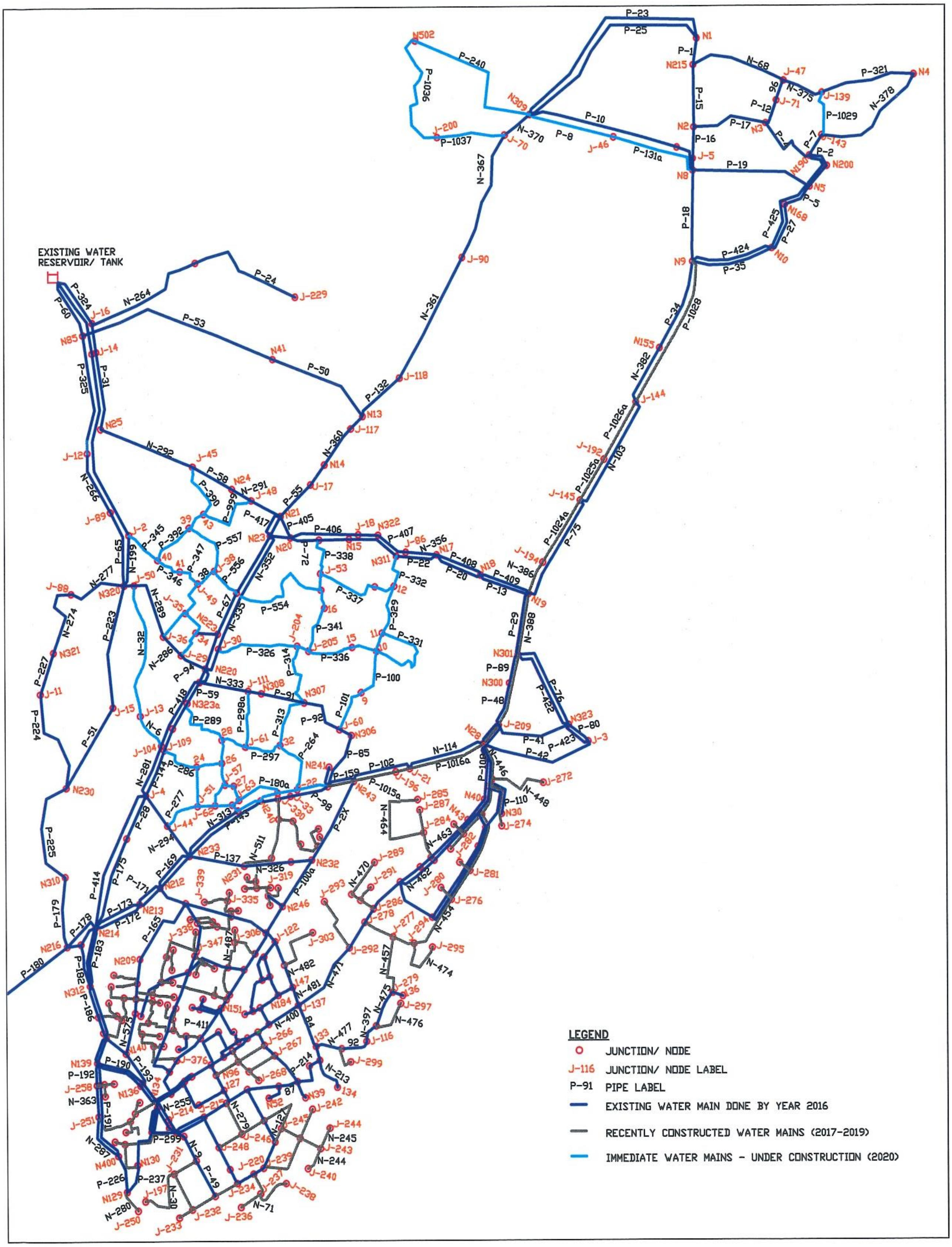


Figure 4.4: Schematic Layout Plan of the Existing Mombasa North Mainland Network showing Pipeline and Nodes Labels

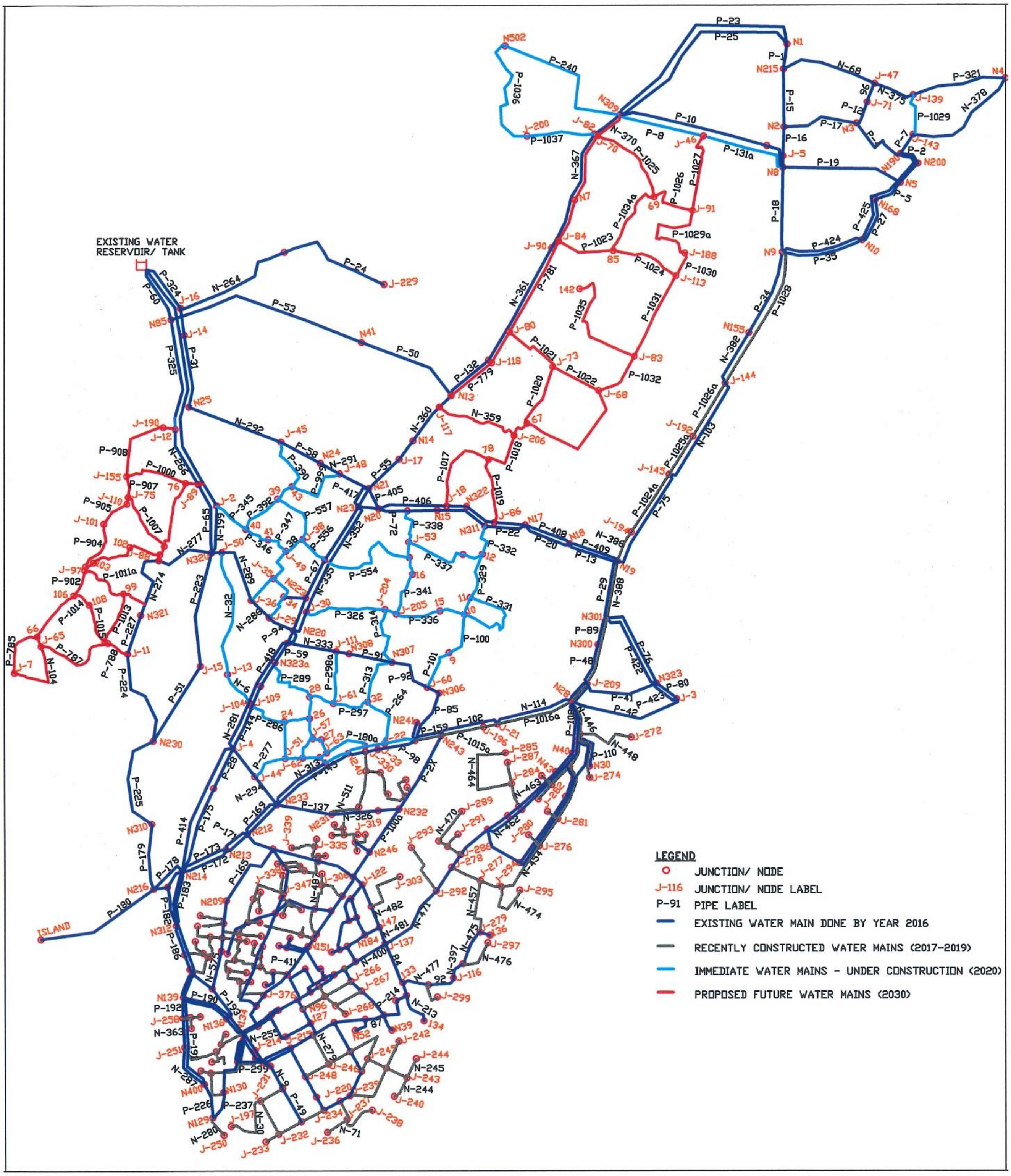


Figure 4.5: Schematic Layout Plan of Future Year 2030 Network Showing Pipe and Node Labels.

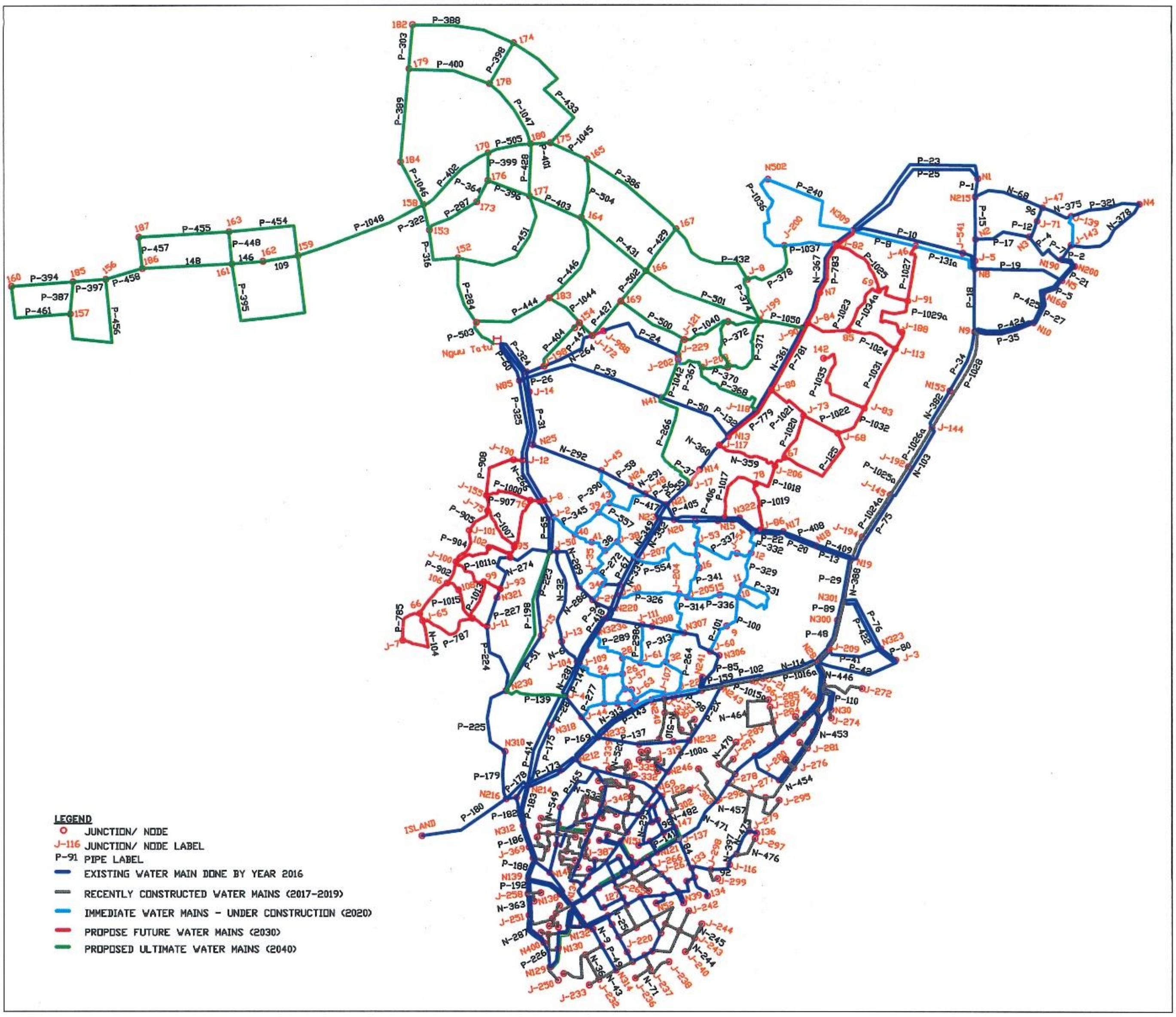


Figure 4.6: Schematic Layout Plan of Ultimate Year 2040 Network Showing Pipe and Node Labels.

4.3 Simulation Results of Existing, 2030 and 2040 Network Models under a Design Peak Factor of 1.2

Critical hydraulic simulation results of the existing, future year 2030 and ultimate year 2040 Mombasa North Mainland KYPipe models showing 10 highest and 10 lowest model node pressures as well as 10 highest and 10 lowest model pipe velocities are summarized in Table 4.1. The existing, year 2030 and 2040 model simulation results are presented graphically depicting pressure contours, with pipelines colour coded for velocity in Figures 4.7 and 4.8.

Table 4.1: Summary of Simulation of the Existing, Future and Ultimate Critical Maximum and Minimum Pressures and Velocities

Pressures											
Existing (Year 2020) Model Pressures				Future (Year 2030) Model Pressures				Ultimate (Year 2040) Model Pressures			
Node Label	Maximum (kPa)	Node Label	Minimum (kPa)	Node Label	Maximum (kPa)	Node Label	Minimum (kPa)	Node Label	Maximum (kPa)	Node Label	Minimum (kPa)
N220	612.47	Nguu Tatu	19.61	J-93	534.79	Nguu Tatu	19.61	175	524.41	183	11.4
J-11	549.22	J-16	92.5	J-11	518.78	J-16	87.94	162	500.41	Nguu Tatu	19.61
N321	542.67	N25	121.69	N321	514.93	J-110	106.04	J-93	500.39	J-49	80.25
J-192	529.38	J-89	128.74	J-98	473.16	J-49	107.79	J-42	500.08	J-16	81.44
J-7	529.03	41	134.77	J-96	463.74	N25	108.24	186	498.37	41	90.07
N323	521.85	J-49	137.31	J-192	463.36	41	111.15	J-230	490.99	J-71	91.55
J-3	521.19	J-2	137.46	J-72	461.99	76	111.84	165	487.7	N25	92.68
N26	520.82	J-14	148.6	N323	453.07	J-89	120.45	N321	480.31	N316	93.62
J-145	516.81	40	150.23	J-3	452.02	J-2	128.15	J-11	479.25	76	98.43
J-144	515.62	J-25	154.01	J-145	451.6	J-75	130.67	159	464.46	N183	105.71
Velocities											
Existing (Year 2020) Model Velocities				Future (Year 2030) Model Velocities				Ultimate (Year 2040) Model Velocities			
Pipe Label	Maximum (m/s)	Pipe Label	Minimum (m/s)	Pipe Label	Maximum (m/s)	Pipe Label	Minimum (m/s)	Pipe Label	Maximum (m/s)	Pipe Label	Minimum (m/s)
P-90	1.31	N-392	0	P-1005	2.1	N-252	0	P-196	2.2	N-384	0
P-415	1.11	N-33	0.01	P-1006	2.1	N-14	0.01	P-172	2.18	P-399	0
P-172	1.03	N-512	0.01	P-788	1.71	N-392	0.01	P-294	2.06	P-5	0.01
P-59	1.01	P-243	0.01	P-90	1.6	N-473	0.01	P-12	2.05	P-169	0.01
P-12	1	P-298	0.01	P-1012	1.56	N-263	0.02	P-90	1.92	P-387	0.01
P-66	0.98	N-279	0.01	P-909	1.48	N-33	0.02	P-312	1.92	N-491	0.02
P-289	0.97	N-30	0.01	P-172	1.48	N-71	0.02	P-57	1.79	N-371	0.02
P-286	0.96	N-40	0.01	P-415	1.43	P-201	0.02	N-333	1.78	N-89	0.02
P-284	0.95	N-71	0.01	P-196	1.36	P-298	0.02	P-311	1.77	P-1025a	0.02
P-196	0.95	N-74	0.01	P-1008	1.32	N-458	0.02	P-93	1.7	P-1026a	0.02

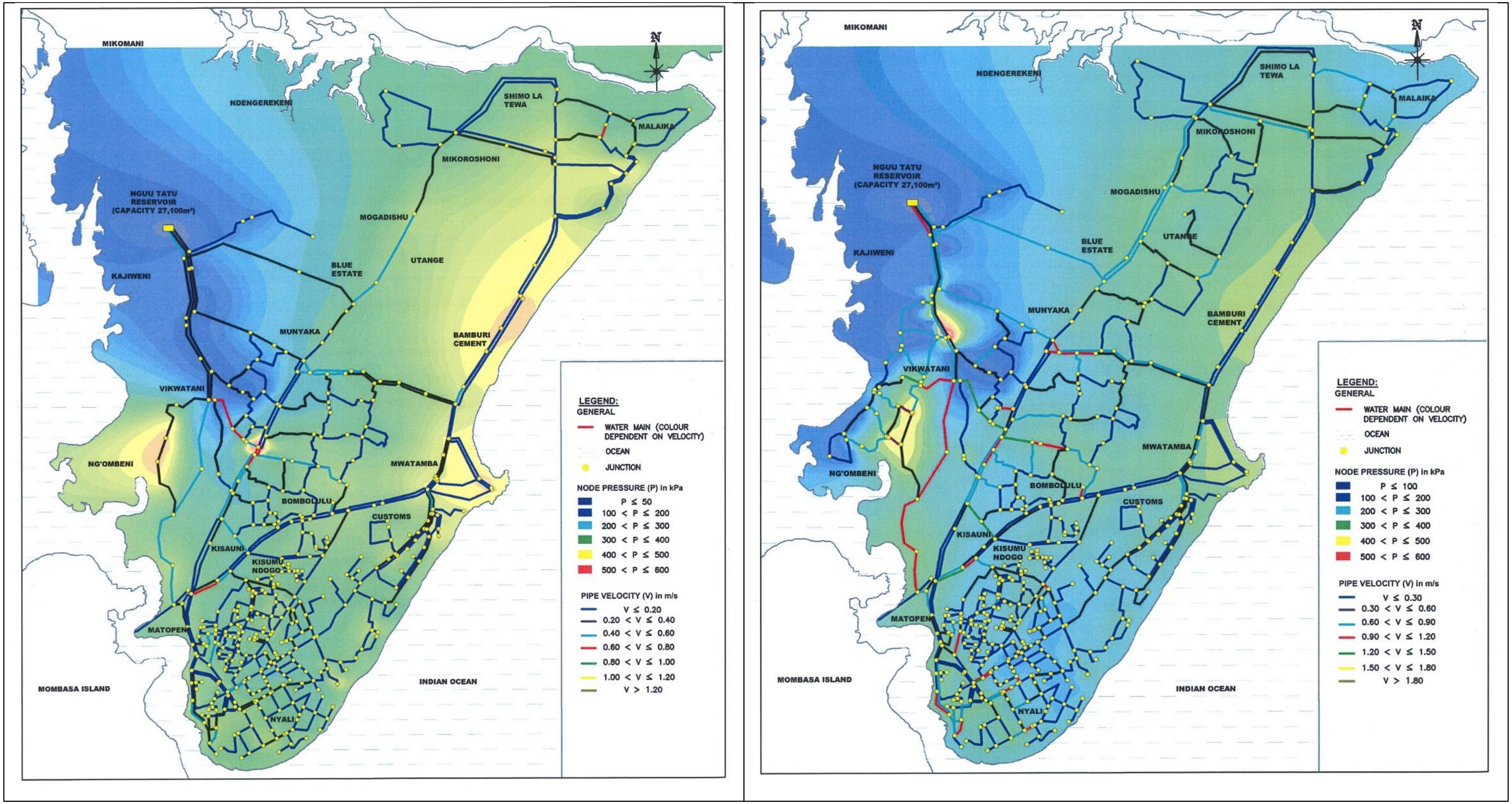


Figure 4.7: KYPIPE Pressure Diagrams for the Existing (left) and Future year 2030 (right) Mombasa North Mainland Network

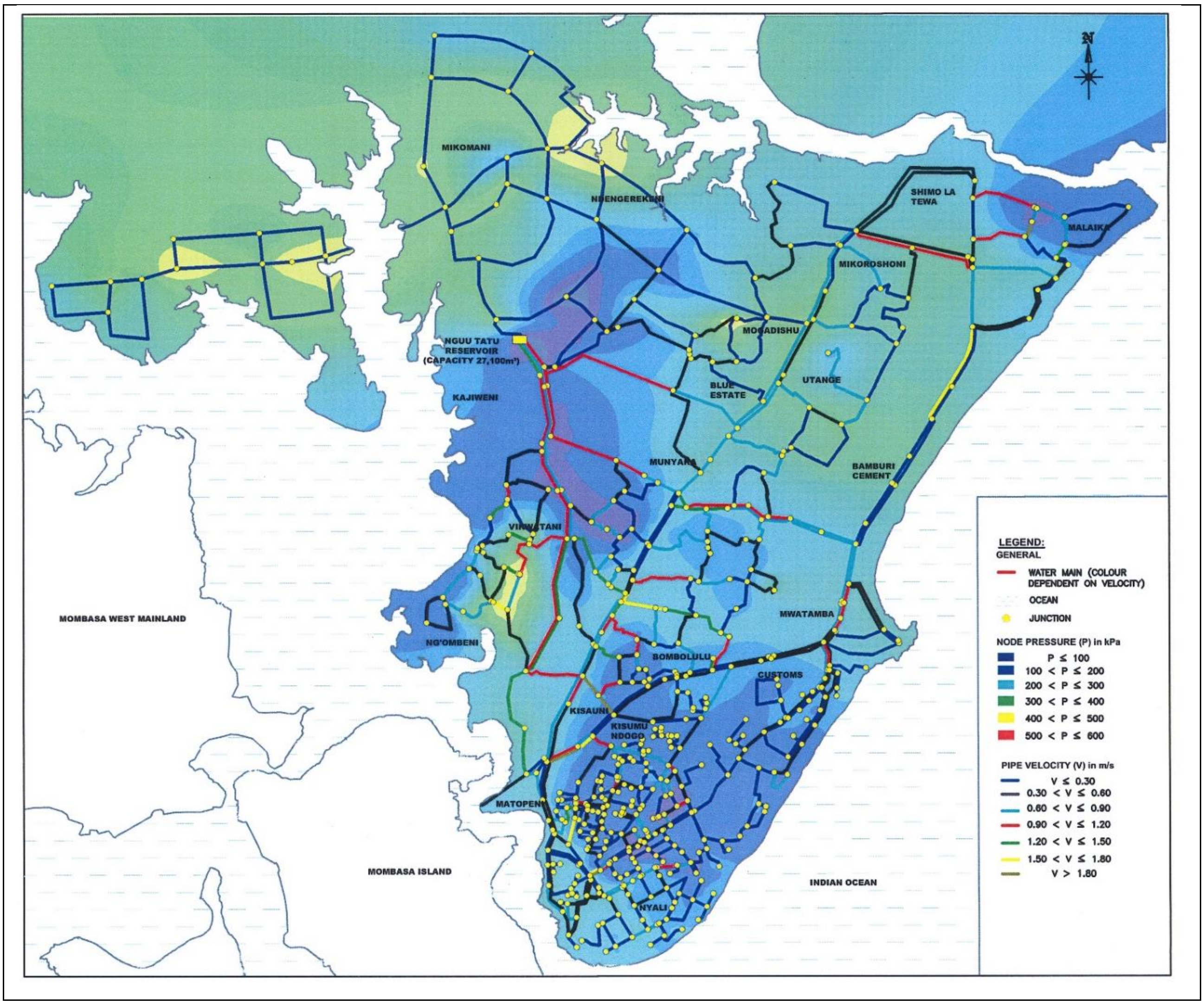


Figure 4.8: KYPIPE Pressure Diagram for Ultimate Year 2040 Distribution Network

4.3.1 Hydraulic Simulation Results Analysis for a design Peak Factor of 1.2.

Table 4.1 show that the highest pressures in the three models were within the design criteria of 600 kPa except for the isolated case of one node in the existing network model which showed a pressure of 612.47 kPa as shown in comparative line graph in Figure 4.9. Higher pressures are usually expected at isolated low elevation points as explained by King & Crocker (1973). The pressures generally decreased with each proposed construction phase with the least high nodal pressure of 524.41 kPa observed for ultimate model (year 2040).

The pressure contours shown in Figures 4.7 and 4.8 and the pressure summaries in Table 4.1 show that the areas around Nguu Tatu Reservoir would experience pressures below 100 kPa in the existing network and in the year 2030 and 2040 models. The low pressures were attributed to the close proximity of the nodal to the source leading to low residual heads as discussed by Twort, et al., (2000) and Linsley & Franzini (1979). If the water service provider wishes to increase the pressures of the critical nodes in the vicinity of the ground terminal tanks, a small, elevated water tank can be erected within the compound of the main tanks to only serve the demand of consumers in vicinity of the terminal tanks.

Pressures of between 100 and 200 kPa were observed in the southern parts of Mombasa North Mainland covering the areas of Kisumu Ndogo, Customs and Nyali, and the north-eastern area covering Malaika in both the future year 2030 and ultimate year 2040 models. Kisumu Ndogo, Customs and Bombolulu are the most populous areas in the study area. Therefore, irrespective of the high water demands in the highly populated areas, the pressure design criteria met up to year 2040 requirements.

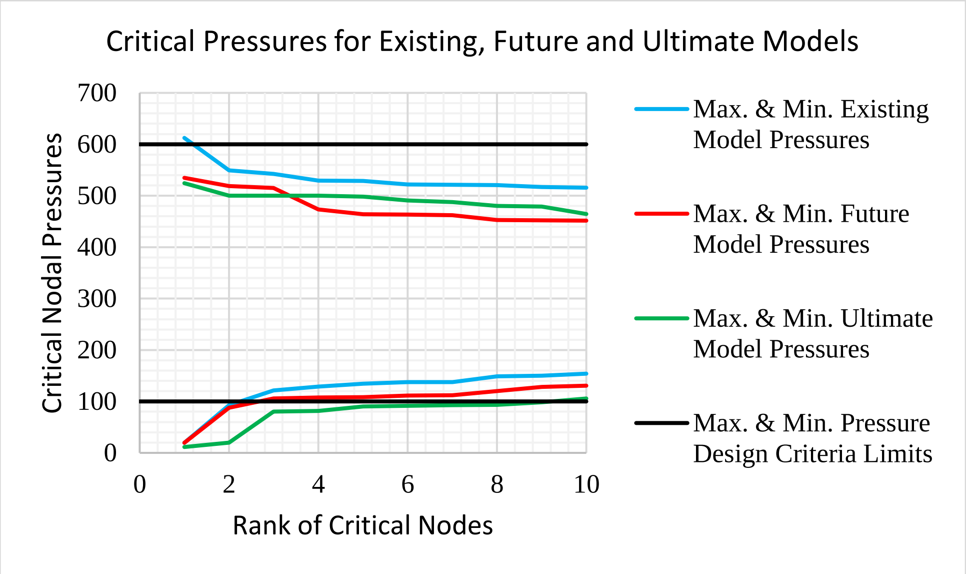


Figure 4.9: Critical Pressures for Existing, Future Year 2030 and Ultimate Year 2040 Network Models

Generally high pressures in the range of 300 to 500 kPa were observed in the western areas of Ng'ombeni, eastern areas of Utenge and Bamburi in all the models. However, these pressures decreased as flow increased in the models as seen in the decrease of pressure contour colour intensity (yellow and cyan) across the three models from existing network model, through year 230 to 2040 model in Figures 4.7 and 4.8.

The future year 2030 and ultimate year 2040 models have two and four pipelines respectively with flow velocities above the design criteria of 2 m/s. The highest flow velocity in the 2030 model is 2.1 m/s while the highest flow velocity in the 2040 model is 2.20 m/s. However, the highest pipeline flow velocity in the existing network model is 1.31 m/s. Generally, pipeline flow velocities increased with increase in model flow as shown across the three models. The flow velocities across the three network models; namely existing, future year 2030 and ultimate year 2040 is shown in a comparative pipeline flow velocity line graph in Figure 4.10.

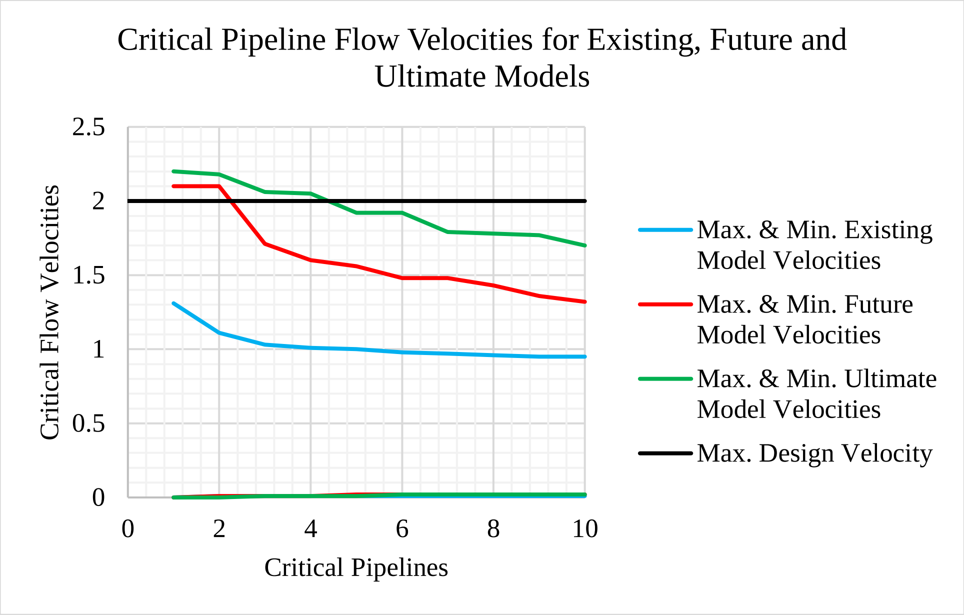


Figure 4.10: Critical Pipeline Flow Velocities for Existing, Year 2030 and 2040 Models

All the three hydraulic models had a pipeline with no water flow (0 m/s) irrespective of water demand (Figure 4.10). Both the existing network model and the 2030 model had one pipeline with no water flow while the 2040 model had two pipelines with no flow Pipelines with no water flow were found at localized high point locations in areas that do not receive water.

The critical low flow velocities observed in all the models were between 0.01 m/s and 0.02 m/s. The low velocities were observed on secondary water distribution pipelines that connect to water users. Due to the low velocity of flow especially in the existing network model, sedimentation of debris can occur at low points (Twort, et al., 2000). However, settling of sediments is expected to be minimal because the water is clear drinking water. In addition, washouts to flush the sediments are provided at low points.

From the model simulation results networks, the existing and proposed networks were found adequate to handle flows when simulated using a peak factor of 1.2. Apart from a single low point where the pressure was above the design criteria with 12 kPa, and the area around the supply reservoirs that had pressures below 100 kPa, the rest of the network had adequate pressures suitable for water distribution.

4.4 Model Robustness Flows at Peak Factors of 1.5 and 2.0

Model robustness of the future (year 2030) and ultimate (year 2040) Mombasa North Mainland models by varying flows using a peak factor of 1.5 and 2.0 were carried out. Model nodal flows were factored by 1.5 or 2.0 respectively after handling water losses. Varying nodal flows generated four models, two models under each peak factor. Resulting models were evaluated on design criteria and necessary steps taken to ensure their adequacy. Any pipeline augmentations required have been displayed in Arc GIS layout plans. Velocity and pressures result in terms of pressure contours for each peaked models are presented in the following sections.

4.4.1 Future year 2030 and Ultimate year 2040 Network Models Simulated with Flow Varied by a 1.5 Peak Factor

Future and ultimate Mombasa North Mainland network models were generated by peaking nodal flows with a peak factor of 1.5. The 2030 model nodal water flow increased from 142,893 m^3/day (1,653 l/s) to 172,463 m^3/day (1,996 l/s) while the 2040 model flow increased from 190,003 m^3/day (2,199 l/s) to 229,317 m^3/day (2,654 l/s).

Table 4.2. summarizes the 10 highest and 10 lowest nodal pressures as well as 10 highest and 10 lowest pipe flow velocities of 1.5 peak factor future year 2030, ultimate year 2040 and augmented ultimate year 2040 models. Simulation results of the 1.5 peak factor future year 2030, ultimate year 2040 and augmented ultimate year 2040 models are presented graphically in terms of pressure contours, with pipelines colour coded for velocity in Figures 4.11 and 4.12.

Table 4.2: Future and Ultimate Maximum and Minimum Pressures and Velocities.

Model Pressures when Flow is varied using a 1.5 Peak Factor											
Future (Year 2030) Model Pressures				Ultimate (Year 2040) Model Pressures				Augmented Ultimate (Year 2040) Model Pressures			
Node Label	Maximum (kPa)	Node Label	Minimum (kPa)	Node Label	Maximum (kPa)	Node Label	Minimum (kPa)	Node Label	Maximum (kPa)	Node Label	Minimum (kPa)
J-93	512.42	Nguu Tatu	19.61	175	508.83	J-71	-83.83	175	515.54	183	12.17
J-11	493.82	J-110	43.26	162	496.38	N316	-72.1	162	497.38	Nguu Tatu	19.61
N321	492.45	J-75	71.56	186	492.15	N183	-62.33	186	493.14	J-71	77.57
J-98	408.77	J-49	82.34	165	469.4	J-47	-61.46	J-93	490.55	66	77.76
J-192	403.5	J-16	83.73	J-93	464.25	N210	-61.44	J-42	487.02	J-49	83.75
J-90	399.45	J-155	84.59	159	460.85	J-193	-55.09	J-230	478.64	J-16	86.2
J-87	397.84	41	91.86	J-42	460.85	N180	-54.29	165	478.35	41	90.94
J-96	396.94	N25	96.17	J-230	456.7	N184	-52.81	N321	470.43	76	92.9
J-145	392.15	76	101.3	184	450.79	J-226	-51.94	J-11	469.27	N316	98.48
N323	388.62	J-7	109.82	176	444.83	J-123	-51.75	159	461.85	J-21	104.42
Model Pipeline Velocities when Flow is varied using a 1.5 Peak Factor											
Future (Year 2030) Model Velocities				**Ultimate (Year 2040) Model Velocities**				**Augmented Ultimate (Year 2040) Model Velocities**			
Pipe Label	**Maximum (m/s)**	**Pipe Label**	**Minimum (m/s)**	**Pipe Label**	**Maximum (m/s)**	**Pipe Label**	**Minimum (m/s)**	**Pipe Label**	**Maximum (m/s)**	**Pipe Label**	**Minimum (m/s)**
P-1005	2.09	N-252	0.01	P-196	2.66	N-384	0	P-317	2.14	N-384	0.01
P-1006	2.09	N-14	0.02	P-172	2.63	P-399	0	P-12	1.99	N-373	0.01
P-788	2.06	N-390	0.02	N-294	2.49	P-169	0.01	P-788	1.99	N-443	0.01
P-90	1.94	N-473	0.02	P-12	2.48	P-5	0.01	P-95	1.84	P-410	0.01
P-1012	1.88	N-263	0.02	P-90	2.35	P-387	0.02	N-133	1.83	P-399	0.01
P-909	1.79	N-33	0.02	P-312	2.31	N-371	0.02	N-114	1.82	P-5	0.01
P-172	1.79	N-71	0.02	P-311	2.25	N-491	0.02	P-1012	1.82	P-373	0.01
P-415	1.72	P-314	0.02	P-57	2.17	P-411	0.02	N-136	1.78	N-279	0.02
P-196	1.64	P-113	0.02	N-333	2.15	N-41	0.02	P-90	1.73	P-387	0.02
P-1008	1.6	P-201	0.03	P-93	2.05	N-89	0.03	N-89	1.7	N-491	0.02

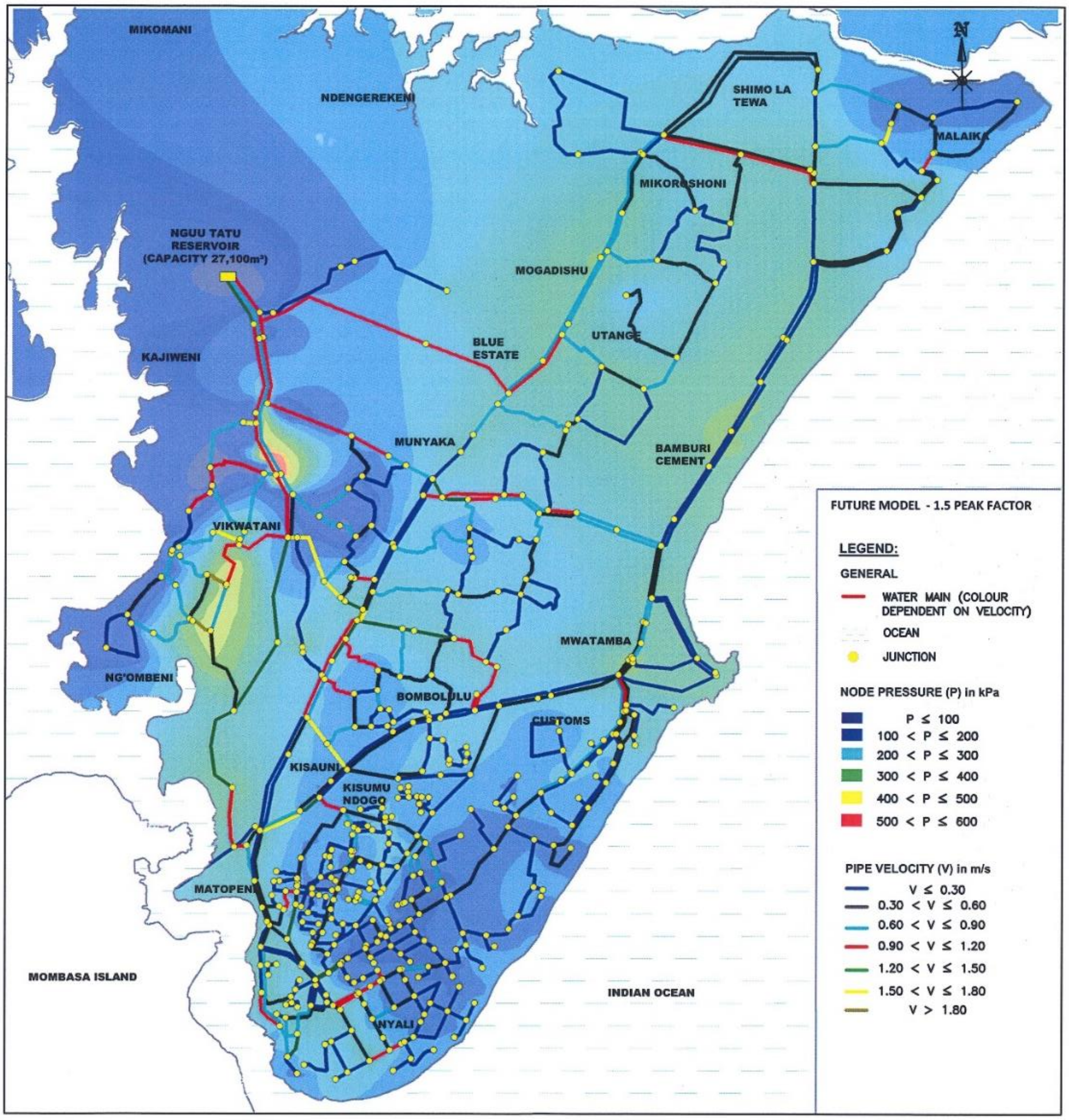


Figure 4.11: Pressure Diagram for the Future (Year 2030) Distribution Network Flow at a Peak Factor of 1.5

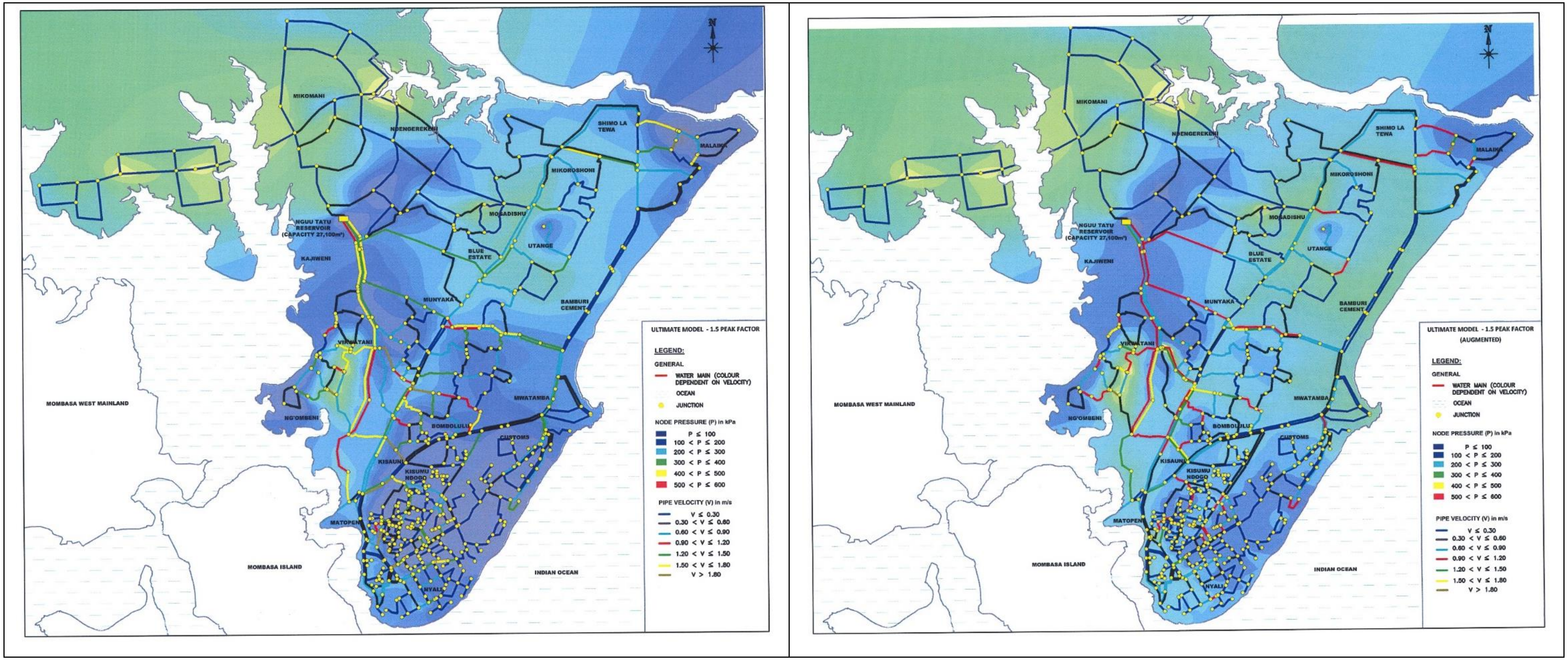


Figure 4.12: Pressure Diagrams for the Ultimate Year 2040 (left) and Augmented Ultimate Year 2040 (right) Network Flow at a Peak Factor of 1.5.

4.4.1.1 Hydraulic Simulation Results for year 2030 Model with a 1.5 Peak Factor

Simulation analysis for future model with flow varied with a peak factor of 1.5 indicated that the area around Nguu Tatu Reservoirs had pressures below 100 kPa. Specifically, these are the areas of Kajiweni and some parts of Vikwatani and Munyaka. Low pressures are expected in nodes close to the reservoir because of a relatively low elevation difference between the source and the nodes resulting to a low residual head as discussed by Linsley & Franzini (1979).

Comparison of proposed future model under 1.2 peak factor and under 1.5 peak factor, showed a general drop in pressures of between 3 m (30 kPa) to 5 m (50 kPa) in the model when analyzed using a 1.5 peak factor. The decrease in model nodal pressures when analyzed using a higher peak factor is because of the increased flow. However, all other nodes of the 1.5 peaked factor future model showed flow pressures that meet the design criteria.

The maximum flow velocity in the future year 2030 model analyzed using a 1.5 peak factor pipelines was 2.09 m/s. The highest velocity is slightly above the maximum design flow velocity of 2 m/s and only occurred in three pipes hence their long-term effect in terms of water hammer, pipe abrasion and internal corrosion are minimal.

Future (year 2030) model analyzed using a 1.5 peak factor was found robust to transmit an unexpected surge in flows to the order of 1.5 times the design flow. The study concluded that the year 2030 Mombasa North Mainland network meets the flow design criteria even when subjected to higher flows in the network. Therefore, there is no need to augment the future network pipelines at a peak factor of 1.5.

4.4.1.2 Hydraulic Simulation Results for year 2040 Model with a 1.5 Peak Factor

Simulation analysis for ultimate model with flow varied with a peak factor of 1.5 displayed that about 40% of the model experienced negative pressures. The least pressure is negative 80 kPa observed in areas around Nguu Tatu Reservoirs. Consequently, water cannot flow in these areas (zero velocity pipelines). In some other parts of the model, high flow velocities were observed in pipelines, with the highest being 2.6 m/s as shown in Table 4.3. The high velocities in these sections of the model induced high head losses in the distribution network.

Ultimate year 2040 Mombasa North Mainland distribution network model analyzed with flow varied by a 1.5 peak factor was found inadequate to transmit an unexpected surge in flow. If the water service provider considers increasing the robustness of the network in ultimate year 2040 design, to transmit almost one and half the average flow, the study identified and proposed augmentation pipes that were incorporated into the model. The augmentation water main proposed was a 10.3 km long, DN 500 mm pipeline starting from Nguu Tatu Reservoirs running parallel to other existing main lines towards Bombolulu and Kisumu Ndogo areas and terminating in Nyali area. This augmentation is shown in green and black dashes in Figure 4.13 layout. The resulting KYPipe model is termed as the augmented ultimate year 2040 North Mainland Network model.

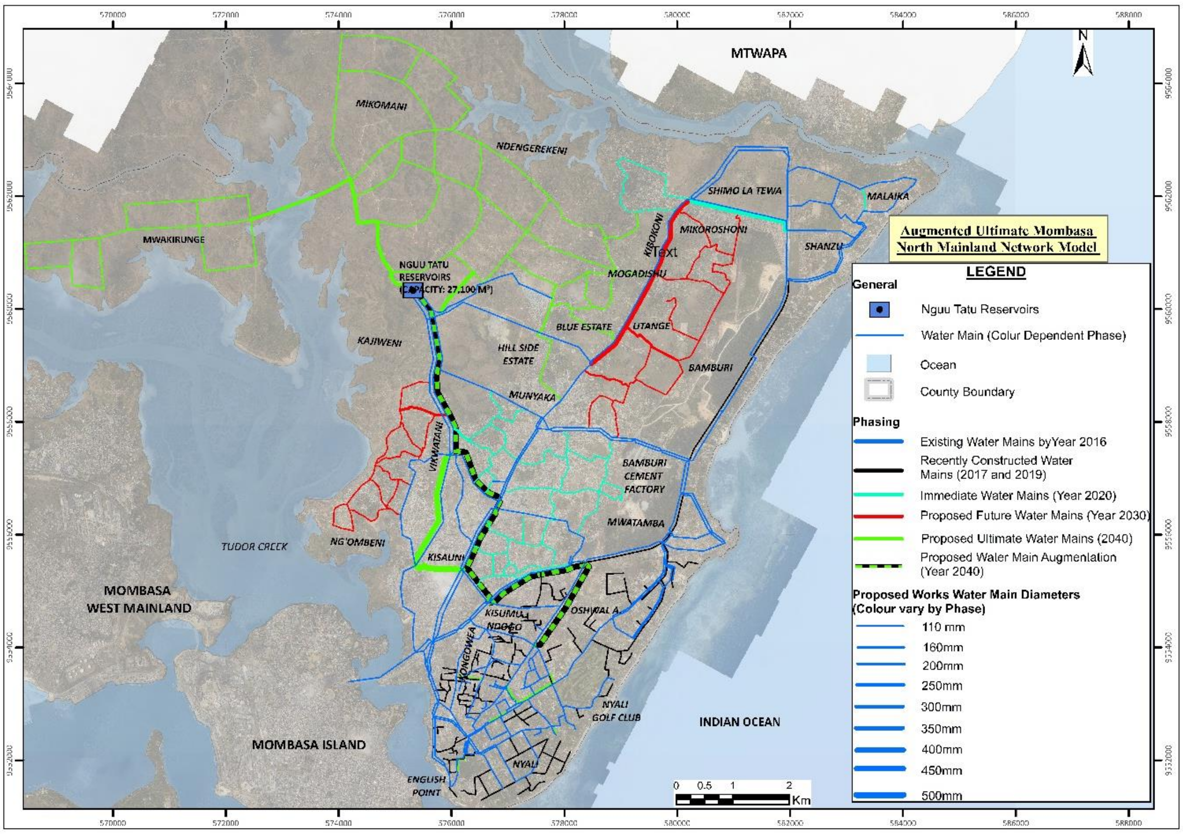


Figure 4.13: Proposed Pipeline Augmentation for Ultimate Year 2040 Design at 1.5 Peak Factor Flow

Pressures improved after the capacity augmentations with the design criteria met in 99% of the augmented ultimate model network. The highest pressure observed at a node was 516 kPa which was within the maximum design recommendation of 600 kPa. Pressures below 100 kPa were only observed in areas around the reservoir as expected, with complete elimination of negative pressures. The comparisons between ultimate model and augmented ultimate models for nodal pressures are shown in the line graph on Figure 4.14.

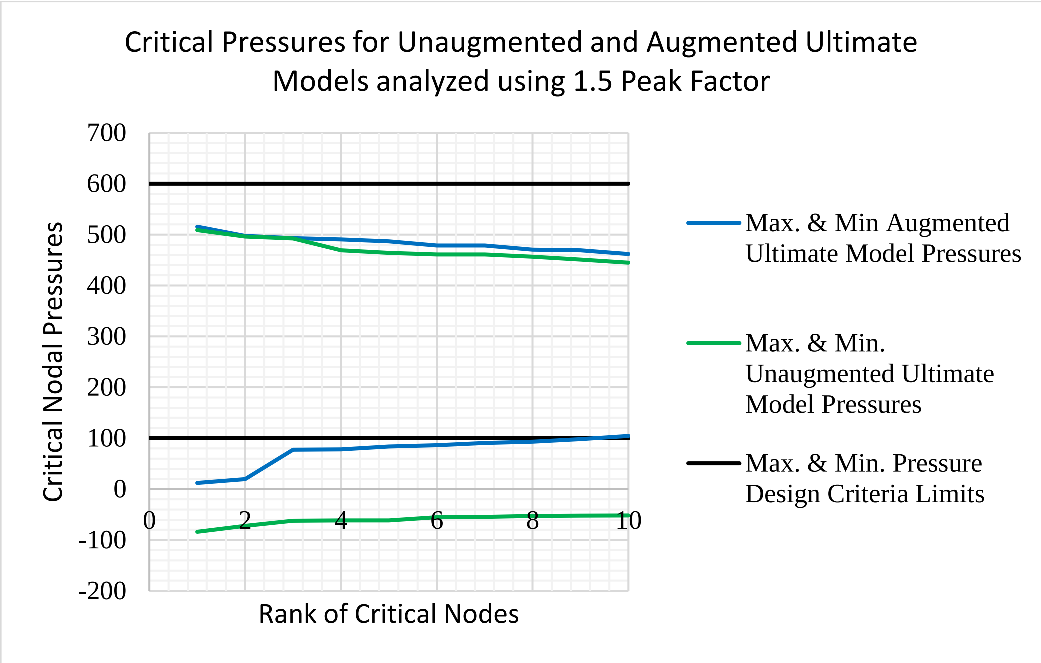


Figure 4.14: Critical Pressures for Year 2040 model flow for Peak Factor 1.5 with Augmentation.

The initially high velocities in the ultimate model pipes reduced after pipeline augmentation with the highest velocity in a single pipeline being 2.14 m/s, a drop of 0.46 m/s from the previous 2.6 m/s. Nonetheless, the high velocity was an isolated case as all the other pipe velocities are within the design criteria of less than 2 m/s. Areas that initially experienced zero velocities were eliminated by the pipeline augmentation. The comparisons between ultimate model and augmented ultimate models for flow velocities are shown in the line graph on Figure 4.15.

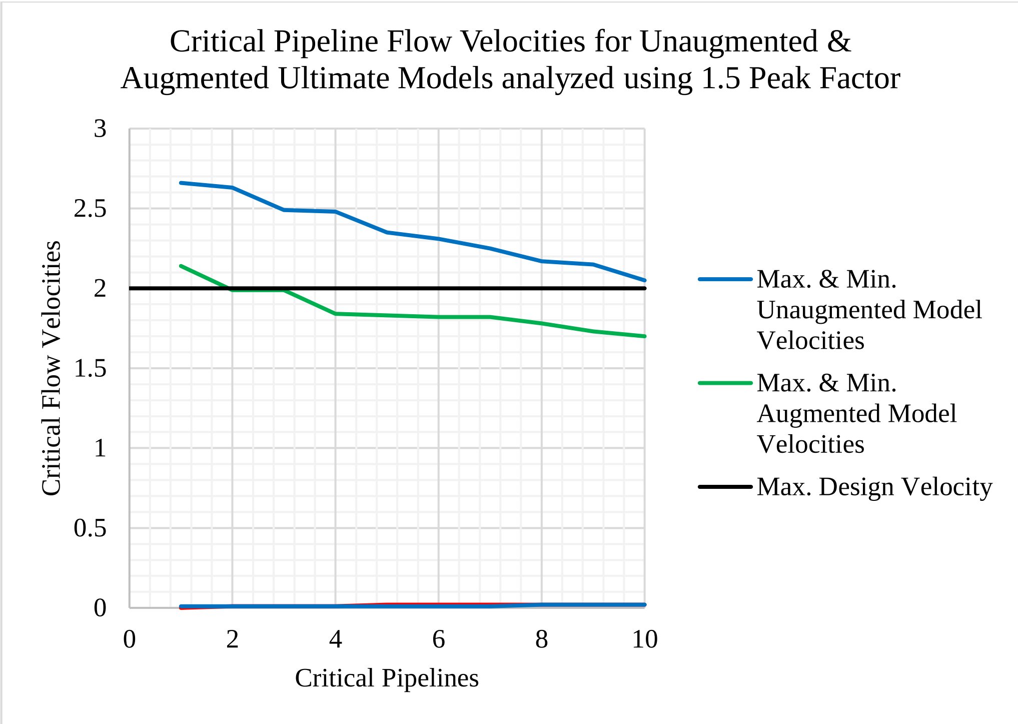


Figure 4.15: Critical Pipeline flow velocities for Year 2040 model flow for Peak Factor 1.5 with Augmentation.

Modelling depicted that a 10.3 km long diameter augmentation would be necessary to make the ultimate model (year 2040) robust to transmit flow varied using 1.5 peak factor.

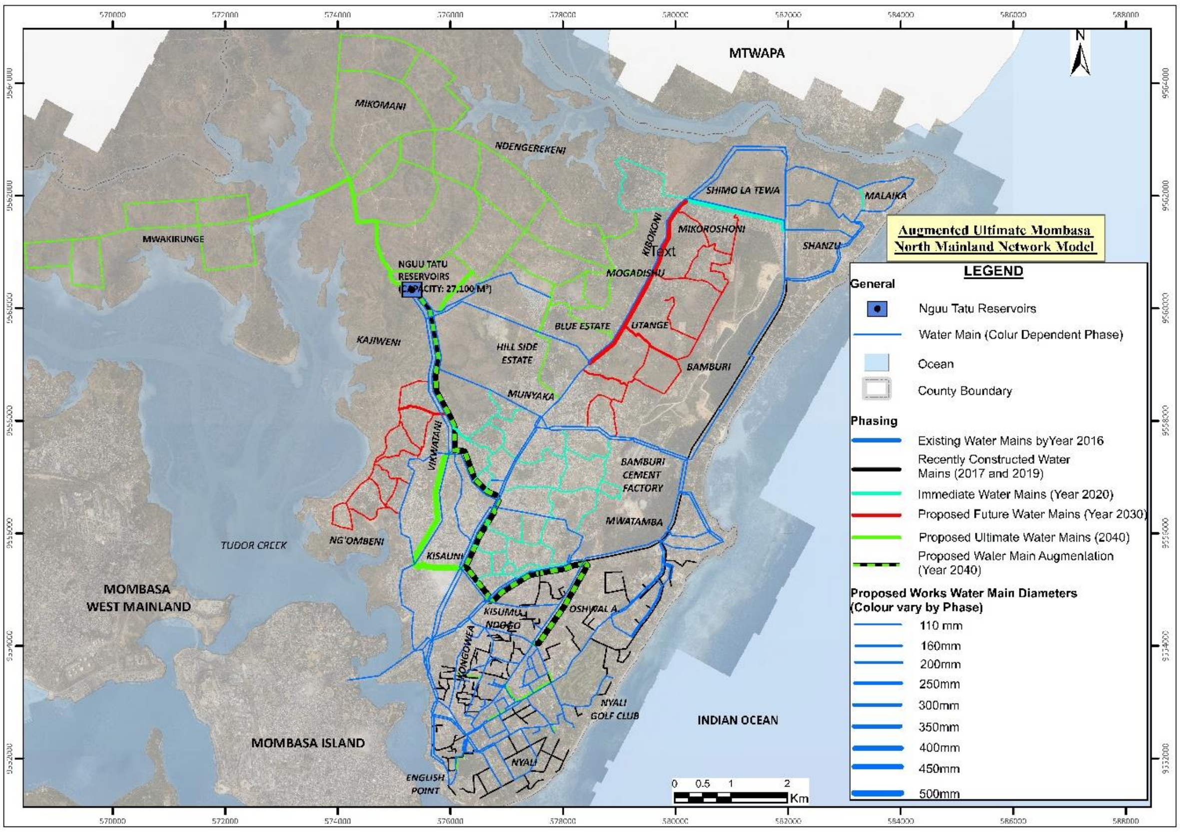


Figure 4.16: Proposed Pipeline Augmentation for Ultimate Year 2040 Design at 1.5 Peak Factor Flow

4.4.2 Future year 2030 and Ultimate year 2040 Network Models Simulated with Flow varied by a 2.0 Peak Factor

The future and ultimate models' flows were varied using a peak factor of 2.0. The future model nodal water flow increased from 142,893 m^3/day (1,653 l/s) to 221,731 m^3/day (2,566 l/s) while the ultimate model flow increased from 190,003 m^3/day (2,199 l/s) to 294,831 m^3/day (3,412 l/s).

Table 4.3. summarizes the 10 highest and 10 lowest nodal pressures as well as 10 highest and 10 lowest pipe flow velocities for 2.0 peak factor future year 2030, augmented future year 2030, ultimate year 2040 and augmented ultimate year 2040 models. Simulation results for the 2.0 peak factor future year 2030, augmented future year 2030, ultimate year 2040 and augmented ultimate year 2040 models are presented graphically in terms of pressure contours, with pipelines colour coded for velocity in Figures 4.16 and 4.17. During model simulations, the augmentation pipelines in both augmented future year 2030 and augmented ultimate year 2040 were also colour-coded as per the flow velocity conforming to other model pipes of flow velocities in the same range.

Table 4.3:Future and Ultimate Maximum and Minimum Pressures and Velocities.

Pressures when Flow is varied using a 2.0 Peak Factor															
Future Year 2030 Model Pressures				**Augmented Future Year 2030 Model Pressures**				**Ultimate Year 2040 Model Pressures**				**Augmented Ultimate Year 2040 Model Pressures**			
Node Label	**Max. (kPa)**	**Node Label**	**Min. (kPa)**	**Node Label**	**Max. (kPa)**	**Node Label**	**Min. (kPa)**	**Node Label**	**Max. (kPa)**	**Node Label**	**Min. (kPa)**	**Node Label**	**Max. (kPa)**	**Node Label**	**Min. (kPa)**
J-93	467.45	N91	-89.06	J-98	496.76	Nguu Tatu	19.61	162	451.2	Nguu Tatu	19.61	J-98	471.69	183	19.61
N321	447.27	N316	-88.31	J-96	491.52	J-49	54.95	175	448.15	183	19.61	J-42	463.03	Nguu Tatu	19.61
J-11	443.63	J-110	-82.06	J-93	487.21	J-250	78.83	186	444.88	J-242	43.58	J-96	461.13	J-49	69.54
37	356.75	J-7	-80.44	J-11	464.87	J-16	79.87	J-98	440.74	J-241	53.42	J-93	460.61	76	78.68
J-90	339.2	66	-71.12	J-87	427.36	N25	81.5	J-42	436.86	J-245	53.68	175	452.04	J-16	81.76
J-87	333.37	N183	-62	99	418.73	76	85.88	J-93	434.56	J-49	54.4	162	451.94	N25	92.13
J-118	328.44	J-65	-60.95	J-191	408.8	J-110	97.43	J-96	431.03	N154	65.28	186	445.62	J-21	92.03
1	328.42	J-193	-57.02	95	408.5	N129	98.63	159	427.71	76	71.44	N321	440.3	J-89	93.03
J-88	322.3	N184	-55.94	N323	369.84	J-89	102.32	184	418.29	N315	77.82	J-11	435.76	J-2	97.99
J-229	320.37	106	-55.35	J-3	368.78	40	102.65	N321	414.04	N52	78.98	159	428.45	40	100.7
Model Pipeline Velocities when Flow is varied using a 2.0 Peak Factor															
Future Model Velocities				**Augmented Future Model Velocities**				**Ultimate Model Velocities**				**Augmented Ultimate Model Velocities**			
Pipe Label	**Max. (m/s)**	**Pipe Label**	**Min. (m/s)**	**Pipe Label**	**Max. (m/s)**	**Pipe Label**	**Min. (m/s)**	**Pipe Label**	**Max. (m/s)**	**Pipe Label**	**Min. (m/s)**	**Pipe Label**	**Max. (m/s)**	**Pipe Label**	**Min. (m/s)**
P-1005	3.25	N-252	0.01	P-66	1.79	P-243	0.01	N-133	2.24	92	0	N-89	1.99	P-162	0.01
P-1006	3.25	N-473	0.02	P-90	1.79	N-295	0.01	N-136	2.21	N-486	0	P-405	1.9	N-486	0.01
P-788	2.65	N-390	0.02	N-289	1.77	N-315	0.01	P-90	2.16	P-54	0	P-60	1.8	N-212	0.01
P-90	2.49	N-14	0.02	N-286	1.77	N-397	0.01	N-89	2.16	P-64	0	P-72	1.78	P-54	0.01
P-1012	2.42	N-263	0.03	N-284	1.74	N-475	0.01	P-405	2.13	N-582	0.01	N-333	1.77	N-33	0.01
P-909	2.31	N-33	0.03	P-94	1.73	P-298	0.01	P-415	2.09	P-163	0.01	N-133	1.74	N-495	0.01
P-172	2.29	P-314	0.03	N-6	1.71	P-130	0.02	N-333	2.06	N-33	0.01	N-136	1.72	N-533	0.01
P-415	2.21	N-71	0.03	P-415	1.7	P-206	0.02	P-72	1.99	N-495	0.01	P-51	1.7	N-478	0.01
P-196	2.12	P-419	0.03	P-237	1.68	N-375	0.03	P-51	1.98	N-533	0.01	P-223	1.7	N-118	0.01
P-1008	2.05	P-201	0.04	P-60	1.62	N-33	0.03	P-223	1.98	N-478	0.01	P-164	1.68	N-102	0.01

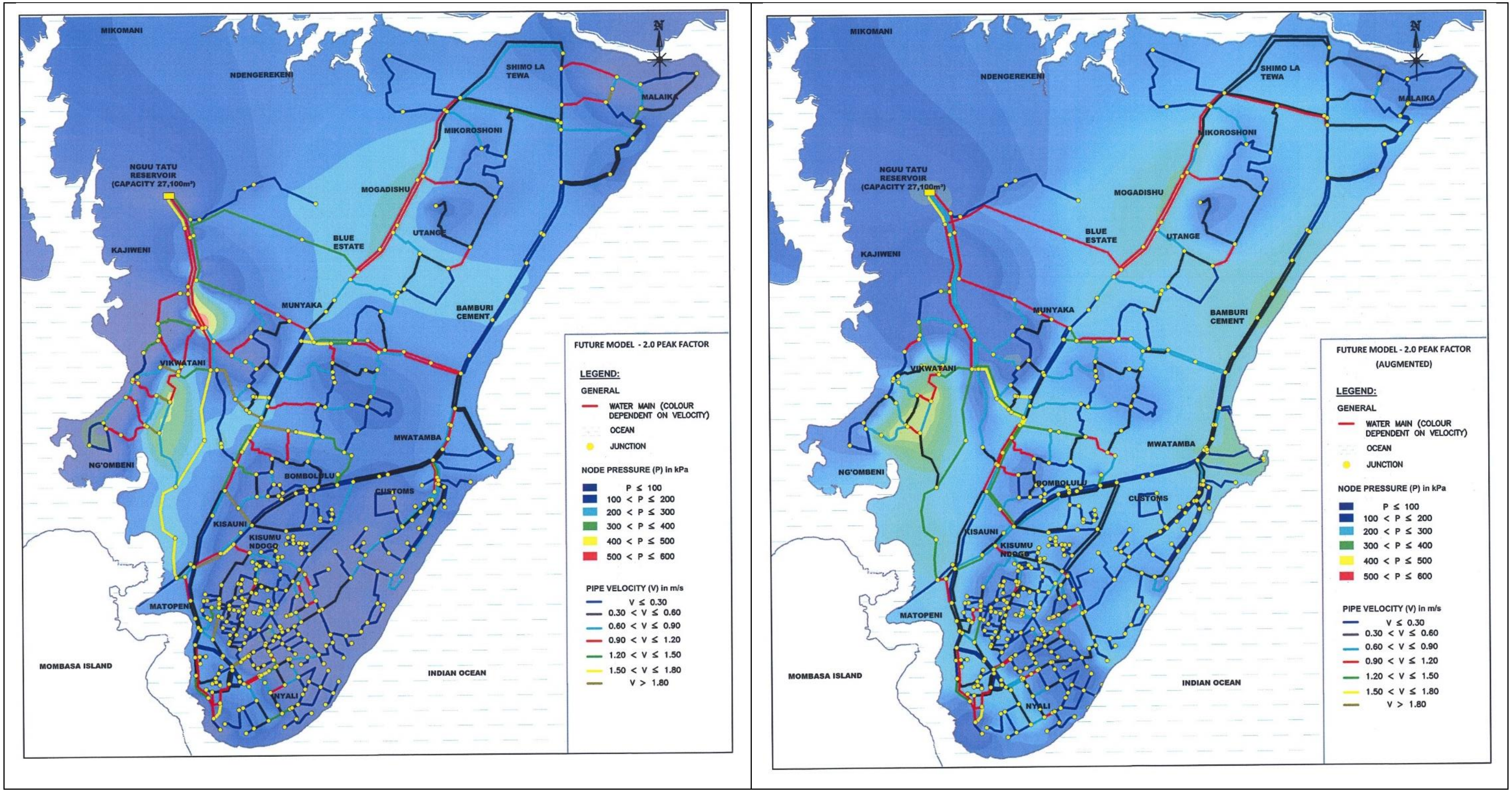


Figure 4.17: Pressure Diagrams for the Year 2030 (left) and Augmented Year 2030 (right) Network Flow for Peak Factor of 2.0.

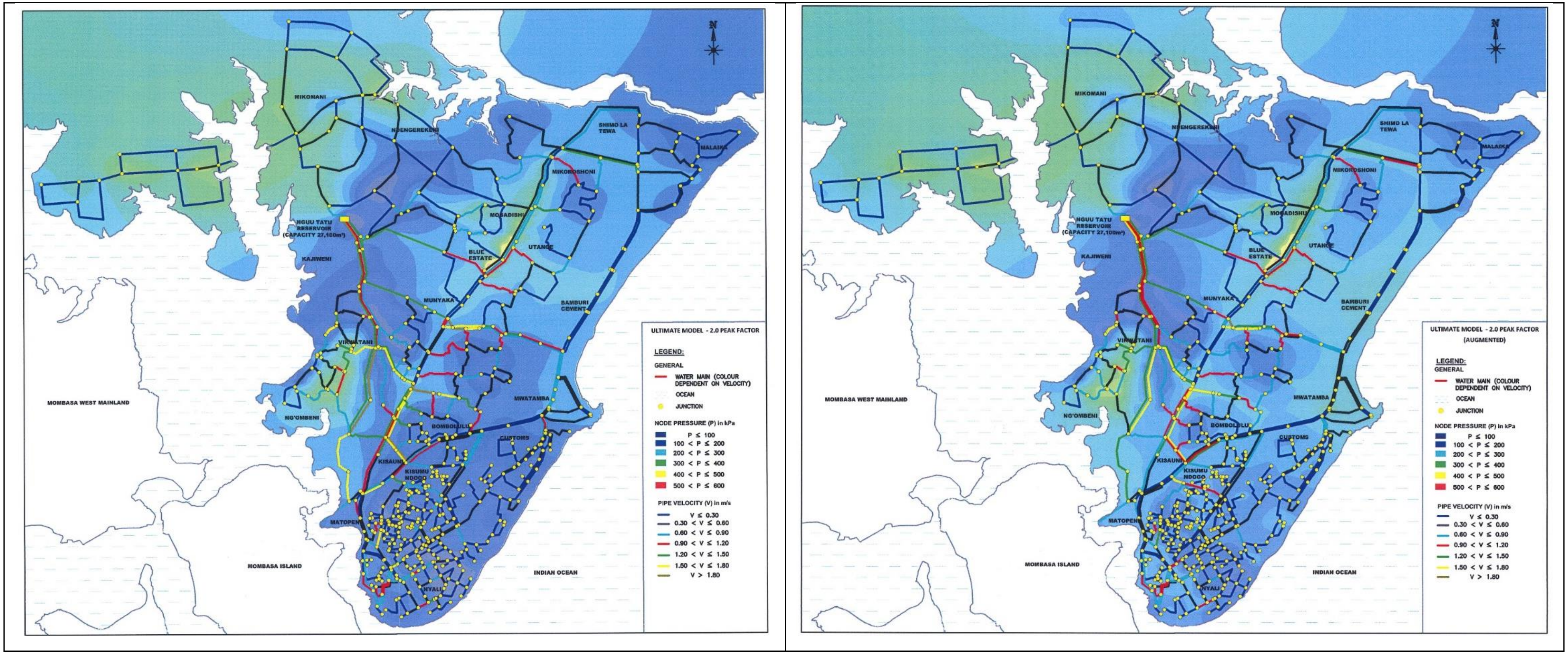


Figure 4.18: Pressure Diagrams for the Year 2040 (left) and Augmented Year 2040 (right) Network Flow for Peak Factor of 2.0.

4.4.2.1 Hydraulic Simulation Results for year 2030 Model with a 2.0 Peak Factor

The results of future model simulation after flow variation using a 2.0 peak factor depicted that about 50% of the model had negative pressures as shown in Figure 4.16. The least pressure is negative 89 kPa observed in areas around Nguu Tatu Reservoirs. Other parts of the model that had negative pressures include areas of Ng'ombeni, Kajiweni, Bombolulu, Kisumu Ndogo, Customs area and Nyali. These negative pressure nodes are shown in the comparative line graph in Figure 4.18. Consequently, water could not flow in the parts of the model that had negative pressures resulting to pipelines with zero flow velocity.

High flow velocities were observed in the future network model after flow variation using a 2.0 peak factor with all the 10 critical nodes having flow velocities above 2.0 m/s. The highest nodal velocity observed was 3.25 m/s as shown on Table 4.3. The highest nodal velocity was 62.5% above the velocity design criteria, and it induced high head losses in the model network. The future year 2030 model was therefore found not sufficiently robust to transmit an unexpected flow twice its average capacity hence the need for pipeline augmentations. If needed by the water service provider, augmentation proposed for the adequacy of the future model with flow varied using a 2.0 peak factor is an 11.1 km long, DN 500 mm pipeline. This pipeline started from Nguu Tatu Reservoirs running parallel to other existing main lines headed towards Bombolulu, Kisumu Ndogo, Customs Area and terminating in Nyali. The augmentation is shown Figure 4.18 layout.

Simulation analysis of the augmented future model with flow varied using a 2.0 peak factor showed that pressures improved, meeting the design criteria in 99% of all model parts as shown in Figure 4.16. The highest nodal pressures observed was 497 kPa while pressures below 100 kPa were observed in only eight nodes (Table 4.3) around the reservoir as expected. Additionally, the high flow velocities in the pipes of the future model reduced with the highest velocity observed in a single pipeline being 1.79 m/s, a 45% decrease from the previous 3.25 m/s. Subsequently, all the augmented future model with flow varied using 2.0 peak factor pipeline velocities are within the velocity design criteria of less than 2 m/s. All the pressures and velocity simulation results for the augmented future model with flow varied using a 2.0 peak factor are as summarized in Table 4.3. Visual comparisons between future model and augmented future model simulated using a 2.0 flow variation peak factor for nodal pressures and velocities are shown in the line graphs on Figure 4.18 and 4.19 respectively.

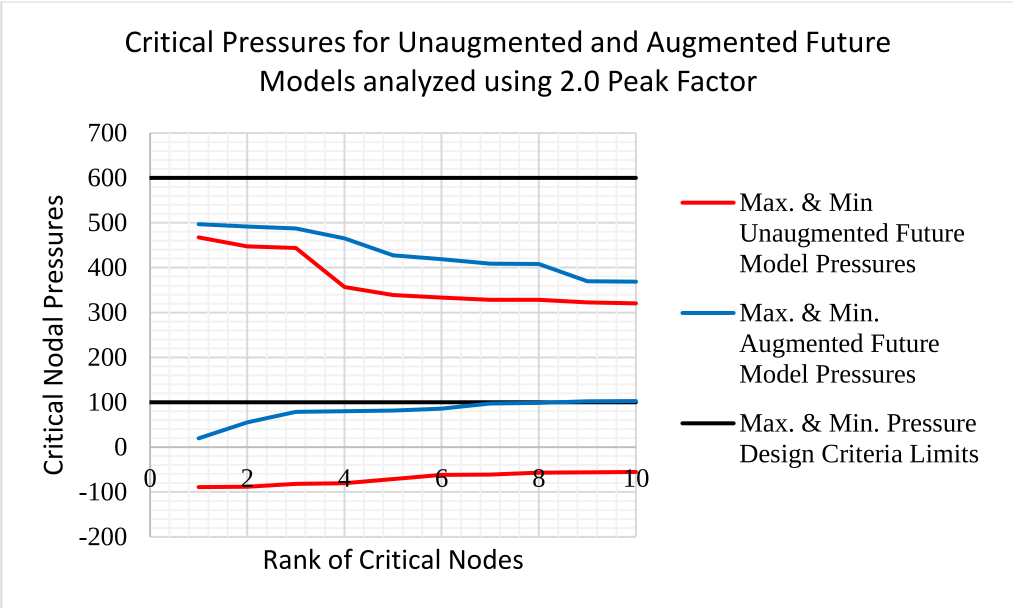


Figure 4.19: Critical Pressures for Year 2030 model flow for a 2.0 Peak Factor with Augmentation.

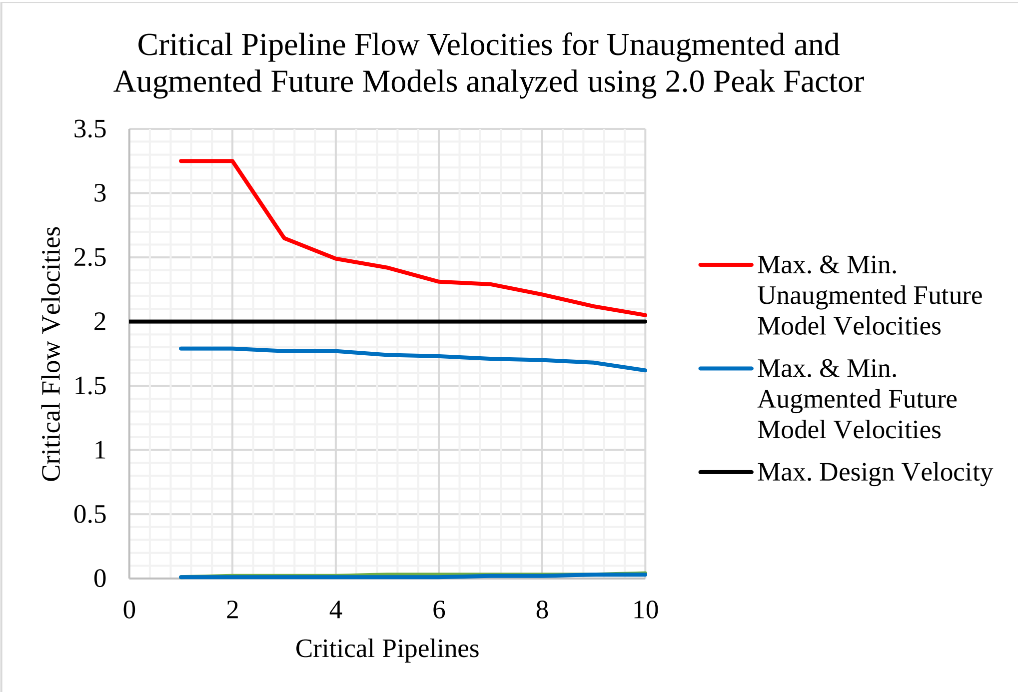


Figure 4.20: Critical Pipeline flow velocities for Year 2030 model flow for a 2.0 Peak Factor with Augmentation.

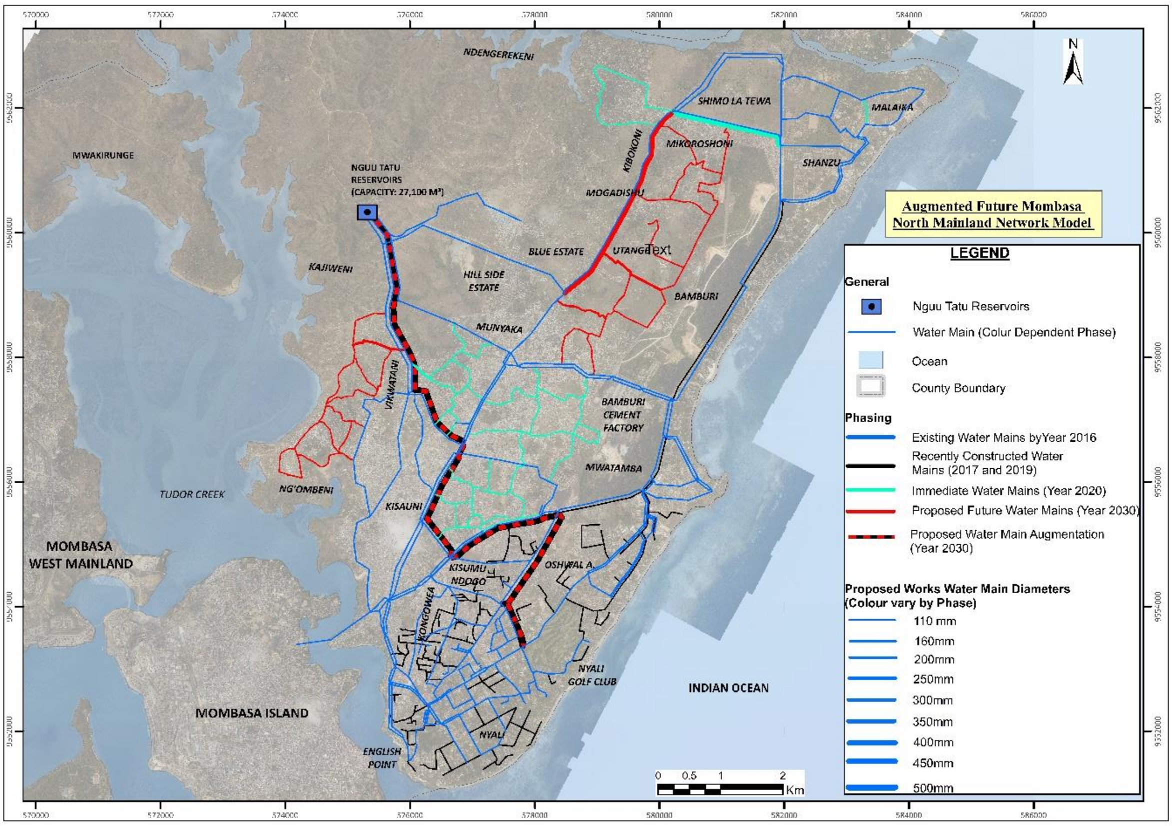


Figure 4.21: **Proposed Pipeline Augmentation Models for Future Year 2030 design at 2.0 Peak Factor Flow**

Visual inspection of line graphs in Figures 4.18 and 4.19 confirm future model adherence to the pressures design criteria and improvement of velocities after incorporation of the pipeline augmentation shown on Figure 4.20. The study concludes that the proposed 11.1 km capacity augmentation would make the future Mombasa North Mainland network model simulated after flow variation using a 2.0 peak factor sufficiently robust to handle unexpected flows in the network.

4.4.2.2 Hydraulic Simulation Results for Year 2040 Model with a 2.0 Peak Factor.

Simulation results of the ultimate model when simulated after flow variation using a 2.0 peak factor showed about 50% of the model had pressures between 50 kPa and 150 kPa. There were no negative pressures in this case because the augmentation proposed in the future network model simulated after flow variation using a 2.0 peak factor was carried to this model. However, low pressures less than 10 kPa were experienced in areas around Nguu Tatu Reservoirs, Kajiweni, Bombolulu, Kisumu Ndogo, Malaika, Customs area and Nyali as shown in pressure contours in Figure 4.17.

Four pipelines of the ultimate model simulated after flow variation using a 2.0 flow peak factor had no water flow as shown in the summary Table 4.3. On the contrary, seven pipes experienced flow velocities higher than the design criteria of 2.0 m/s with the highest being 2.24 m/s. The relatively fast movement of water induced relatively low pressures as velocity and pressure are inversely related (Alegre, et al., 2016).

Proposed ultimate year 2040 model incorporating the augmentation proposed in the future year 2030 model could not sufficiently transmit the increased flow with a 2.0 peak factor. If the water service provider considers increasing robustness of the ultimate year 2040 design network the study identified vulnerable pipes and proposed capacity augmentations that were incorporated into the model. The augmentation proposed is a 6.6 km long, DN 700 mm water main that tapers down to a 7 km long DN 500 mm and then branches to a DN 300mm, 0.86 km long. This augmented pipeline starts from Nguu Tatu Reservoirs and rans parallel to other existing main lines towards Bombolulu, Kisumu Ndogo, Customs Area and terminates in Nyali area as shown in green and black dash line in layout Figure 4.21.

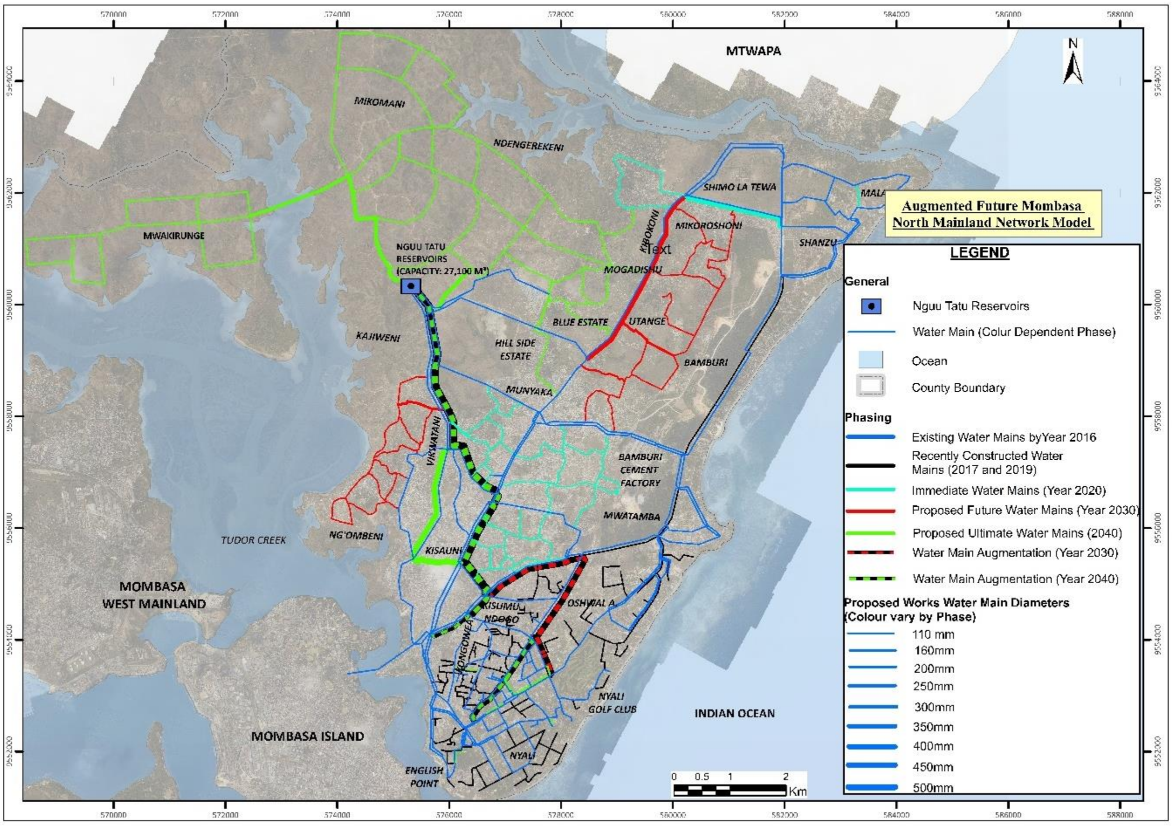


Figure 4.22: Proposed Pipeline Augmentation for Ultimate Year 2040 Network at 2.0 Peak Factor Flow

The pressures in augmented ultimate model with a 2.0 peak factor improved, meeting the design criteria in almost the entire model as shown in Figure 4.17. The highest nodal pressures observed was 472 kPa while pressures below 100 kPa were observed in 9 nodes (Table 4.4) around the reservoir. Additionally, the high flow velocities in the pipes of the ultimate model reduced with the highest velocity observed in a single pipeline being 1.99 m/s. Subsequently all the augmented ultimate Mombasa North Mainland network model simulated after flow variation using 2.0 peak factor pipeline velocities are within the velocity design criteria of less than 2 m/s. Comparisons of critical nodal pressures and velocities between ultimate model and augmented ultimate models simulated with a peak factor of 2.0 flow variation are shown in the line graphs on Figures 4.22 and 4.23, respectively.

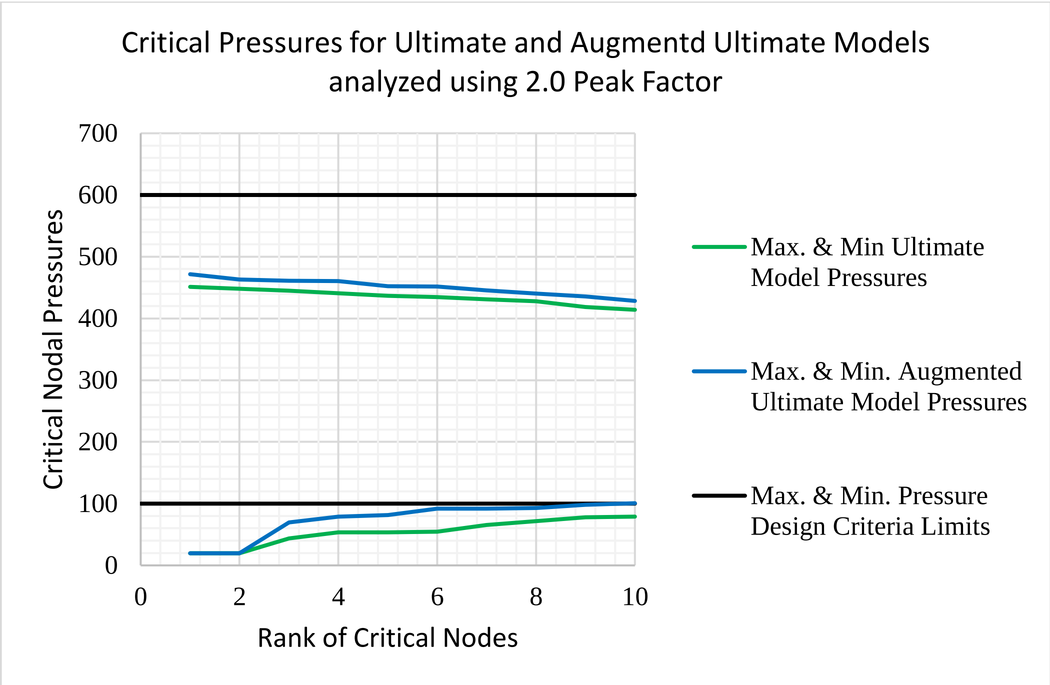


Figure 4.23: Critical Pressures for Ultimate year 2040 model flow for a 2.0 Peak Factor before and after Augmentation.

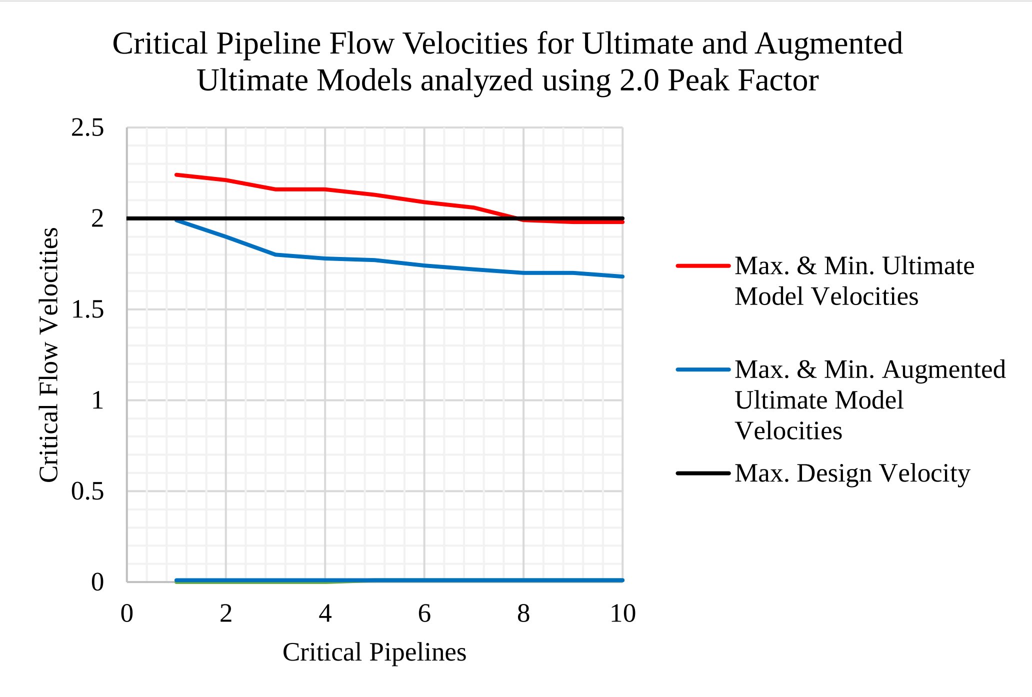


Figure 4.24: Critical Pipeline flow velocities for Ultimate year 2040 model flow for a 2.0 Peak Factor before and after Augmentation.

Visual inspection of Figures 4.22 and 4.23 confirm ultimate model adherence to the pressures design criteria and a slight improvement of velocities after incorporation of the pipeline augmentation shown on Figure 4.21. The study concludes that the proposed augmentation pipeline (6.6 km long, DN 700 mm water main that tapers down to a 7 km long DN 500 mm and branches to a DN 300mm, 0.86 km long) would make the ultimate Mombasa North Mainland network model simulated after flow variation using a 2.0 peak factor robust enough to handle unexpected flows.

4.5 General Results Discussions

The results of modelling Mombasa North water distribution networks showed how modelling can be used to analyze a network, run, and simulate future network scenarios under varying flow conditions and plan for expansions of the network.

In this study, the models have also been used to suggest pipelines augmentations that the water service provider can implement in case of unexpected water flows in the research area in the coming years. However, the augmentations have been based on global peak factors, but unexpected flows can occur in localized areas. In such cases, the model nodal flows in specific areas can be adjusted accordingly and the software used to identify other local lines to augment in the locality for adequacy.

CHAPTER FIVE

5. CONCLUSIONS AND RECOMMENDATIONS

5.1 Conclusion

The study evaluated the robustness of a water distribution network by varying flows of existing and proposed future networks model with different peak factors on KYPipe Modelling Software. The following conclusions were made:

1) The ultimate (year 2040) Mombasa North Mainland hydraulic model analyzed under a peak factor of 1.2 met the pressures requirements of greater than 100 kPa and not exceed 600 kPa with pressures of 80 to 524 kPa and flow velocity with the requirements of less than 2 m/s in 99% of nodes with the exceptional highest flow velocity of 2.2 m/s observed in one pipeline.

2) The future year 2030 network simulated with a peak factor of 1.5 was found robust to transmit the increased flow but insufficient at 2.0 peak factor, the ultimate year 2040 model was insufficient even at 1.5 peak factor.

3) Operating the Mombasa North Mainland distribution network with flow 1.67 times the design flow, would require augmentation of the existing DN 700 mm and 500 mm main pipes. with a 6.6 km long, DN 700 mm water main that tapers down to a 7 km long DN 500 mm and branches to a DN 300 mm, 0.86 km long pipeline.

5.2 Recommendations

Recommendations of this study are:

1) The robustness of water distribution network to handle unexpected flows should always be checked by varying flows before implementation of the networks.

2) Water utilities should develop, maintain, and update models for quick and informed decisions making on their expansion.

3) The models should use actual surveyed ground elevations instead of the ground elevations generated using Digital Terrain Models for more accurate results

4) Future studies on the robustness of other distribution networks in Mombasa South Mainland, Island and West Mainland under the MOWASSCO water service provider should be carried out to evaluate the adequacy of the existing and future distribution networks.

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Appendix 1

Table A1: Overall North Mainland Water Demand, m^3/day (MOWASSCO Master plan, 2017)

Area	Category	2016	2020	2025	2030	2035	2040
North Mainland	Domestic	62,312	72,896	89,729	104,020	121,822	137,831
	Educational	2,116	3,301	4,901	6,818	7,714	8,727
	Health Sector	551	645	766	888	1,004	1,136
	Administration	2,565	3,001	3,564	4,132	4,675	5,289
	Commercial	1,187	1,272	1,378	1,393	1,388	1,425
	Industrial	1,427	1,529	1,657	1,676	1,669	1,713
	Hotels	1,866	2,362	3,171	4,257	5,716	7,675
	Total	**72,024**	**85,006**	**105,166**	**123,184**	**143,988**	**163,796**