College of Engineering, Mathematics and Physical Sciences

Development of a methodology for sustainable conversion from an intermittent to a continuous water supply system

Submitted by

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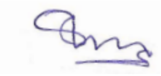
I certify that all material in this thesis which is not my own work has been identified and that no material has previously been submitted and approved for the award of a degree by this or any other University.

Signature: 

“To my late father who had a wish of at least one of his children studying in Great Britain and my late mother who passed on when I was pursuing my PhD studies at the University of Exeter in Great Britain”. Both believed in persistent hard work as the basis for successes.

ABSTRACT

Intermittent water supply systems (IWSS) supply water to about one-third of consumers in Africa, half in Asia and two-thirds in Latin America. Despite their wide application, IWSS present many problems that can be overcome by conversion to continuous water supply (CWS). However, conversion to CWS is difficult due to many interconnected IWSS causal factors. In this research, a methodology aimed at ensuring sustainable conversion from intermittent to CWS through increased understanding of the different causal factors is proposed. Among the eight identified causal factors are poor governance, demographic and economic dynamics. To incorporate uncertainties linked to the evolution of the causal factors, the methodology uses scenarios. Four scenarios (business as usual, consumption demand management, non-revenue water (NRW) management, and holistic) are developed for water supply systems using the drivers of water demand and the two axes scenario development method. For hydraulic analysis, a method for modelling IWSS is developed. Application of the "Global Scenario Group" scenarios to the Lusaka water supply network (LWSN) shows that Zambia is currently under the Market Forces scenario which has implications of poor revenue generation from water sales. Application of the developed scenarios shows that the NRW management scenario, which corresponds to the water supply investment master plan for Lusaka city, is not sustainable because the targeted reduction of NRW to 15% of the input volume by the year 2035 is not attainable. Moreover, if all the residents are considered, there will be a water deficit of 17,831 m^3/d even if the 15% NRW reduction target was met. The most sustainable scenario, which will result in a water surplus of 10,287 m^3/d, is the holistic scenario. However, its attainment requires concerted efforts to change the course of development from the unsustainable scenarios. The LWSN's Chelstone zone's 2035 topology is proposed following the optimisation of the zone's scenario-based rehabilitation problems.

LIST OF PUBLICATIONS

Simukonda, K., Farmani, R., Butler, D., 2017. Causes of intermittent water supply in Lusaka, in: Proceedings of the IWA Efficient 2017 Conference, Bath, United Kingdom.

Simukonda, K., Farmani, R., Butler, D., 2018. Causes of intermittent water supply in Lusaka City, Zambia. Water Practice and Technology13(2): 335–345. doi:10.2166/wpt.2018.046.

Simukonda, K., Farmani, R., Butler, D., 2018. Intermittent water supply systems: causal factors, problems and solution options. Urban Water Journal 15(5): 488–500. doi:10.1080/1573062X.2018.1483522.

Simukonda, K., Farmani, R., Butler, D., 2019. Modelling of intermittent water supply systems considering the distinctions between leakage and consumer water demand, in: Proceedings of the 1st Intermittent Water Supply Conference. Kampala, Uganda.

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NOTATION

A_{ind}	Total industrial area in a district metered area (DMA)
Bd_{Dj}	Consumption base demand (l/s)
Bd_{Lj}	Nodal average leakage (leakage base demand) (l/s)
Bd_{node}	Representative base demand for the node connecting (Chelstone) zone to other zones or water boreholes (l/s)
Bd_{pump}	Representative base demand for a borehole pump (l/s)
Bd_{Totalj}	Total base demand for node j (l/s)
C	Constant that depends on the network (-)
C_{Con}	Per capita water consumption in commercial facilities (l/c/d)
C_{Pop}	The population of people in commercial facilities (-)
$dmfactor$	Consumption demand multiplication factor (-)
E_{coef}	Artificial consumption demand emitter coefficient (l/sm$^{0.65}$)
El_{coef}	Artificial leakage emitter coefficient ((l/sm$^{\alpha}$). The value of α varies
$\bar{F}$	Mean pipe flow (l/s)
F_A	Flow values through GV-362 at time t after isolation (l/s)
$factor_{Bd}$	Factor by which 2010 base demands are increased to 2035 base demands (-)
F_B	Flow values through GV-362 at time t before isolation (l/s)
F_{F_i}	Field measured flows through pipe *i* (l/s)
f_{mf}	Flow pattern multiplication factor (-)
H_{aveDMA}	Average DMA pressure head (m)
HC_{sc}	High cost houses domestic per capita water consumption for a scenario (l/c/d)
H_i	Actual node pressure head for node i (m)
H_j	Actual node pressure head for node j (m)
${H_{ji}}^{av}$	Average pressure head of pressure head for node j and I (m)
H_m	Minimum node pressure head (m)
H_{req}	The required system pressure head (m)
Ind_{Consc}	Industrial water consumption per unit area for a scenario (l/ha)

$InfC_{sc}$	Informal housing domestic per capita water consumption for a scenario (l/c/d)
$LC_{S_{sc}}$	Low cost houses domestic per capita water consumption for a scenario (l/c/d)
$Leak_{DMA}$	Estimated DMA level of leakage for a scenario (m^3/d)
L_x	Length of pipe x
L_{xji}	Length of pipe *x* between nodes j and i (m)
$lmfactor$	Leakage multiplication factor (-)
MC_{sc}	Medium cost houses domestic per capita water consumption for a scenario (l/c/d)
mf	Multiplication factor at each time step t for the node connecting one zone to others (-)
n	Constant that varies from 0.5 to 2.5 (-)
$NDMAdn$	Number of demand nodes in a DMA (-)
N	Total number of pipes with field measured flows (-)
N_j	Number of nodes connected to node j (-)
N_n	Total number of new pipes to be installed (-)
N_v	Total number of new valves to be installed (-)
P_{Con}	Per capita water consumption in public facilities (l/c/d)
P_{DMA}	The total population of people in a DMA (-)
P_{HC}	The population of people in high cost houses (-)
P_{Inf}	The population of people in informal housing (-)
P_{kDMA}	The population of people living in housing type k (HC, MC, LC and $InfC$) in the DMA (-)
P_{LC}	The population of people in high cost houses (-)
P_{MC}	The population of people in medium cost houses (-)
P_{mf}	Pump supply pattern multiplication factor (-)
P_{Pop}	The population of people in public facilities (-)
Q_{ave}	Average flow through the flow control valve (l/s)
Q_{DMA}	District metered area consumption demand (l/d)
$Q_{DMA_{Billed}}$	Billed water consumption (l/s) or (m^3/d)
Q_{DMAd}	DMA domestic consumption water demand (l/d)
Q_{DMAi}	DMA industrial consumption water demand (l/d)

Q_{DMAp}	DMA public consumption water demand (l/d)
$Q_{DMAtotal}$	Total DMA water demand (l/s) or (m^3/d)
Q_j	The total water supplied from node j at a given time (l/s)
Q_j^d	Node consumption demand outflow (l/s)
Q_j^l	Node leakage outflow (l/s)
Q_j^{req}	Required consumption water demand (l/s)
Q_{Lx}	Pipe *x* leakage flow rate (l/s)
Q_{LWSN}	Total consumption water demand for LWSN (m^3/d)
Q_{pumpt}	Pump discharge at time t (l/s)
Q_{tGV}	Flow through GV-362 at time t (l/s)
Q_{totalj}	Total water outflow from node *j* (l/s)
Q_{Zone}	Zone consumption water demand (m^3/d)
$Q_{Zonetotal}$	Zone total water demand (m^3/d)
R	Correlation coefficient (-)
S_{Fi}	Simulated flows through pipe *i (l/s)*
$Supply_{DMA}$	Daily volume of water supplied to a DMA (m^3/d)
$Supply_{LWSN}$	Daily volume of water supplied to LWSN (m^3/d)
$Supply_{Zone}$	Daily volume of water supplied to a zone (m^3/d)
α	Leakage parameter representing the combination of bursts and background leakage (-)
γ	Consumption emitter exponent (-) found by $\gamma = \frac{1}{n}$

Other notations[1]

ABBREVIATIONS

ACDE	Artificial consumption demand emitter
ACV	Artificial check valve
AFCV	Artificial flow control valve
AGPV	Artificial general purpose valve
ALE	Artificial leakage emitter
AN	Artificial node
AR	Artificial reservoir
AT	Artificial tank
ATCV	Artificial throttle control valve
BD	Break down
CSH	Code for sustainable housing
CWS	Continuous water supply
CWSS	Continuous water supply system(s)
DDA	Demand driven analysis
DMA	District metered area
DN	Demand node
DTF	Devolution Trust Fund
EC	Eco-communalism
EPS	Extended period simulation
ESAWAS	Eastern and Southern Africa Water and Sanitation
FAVAD	Fixed and Variable Area Discharges
FW	Fortress World
GRZ	Government of the Republic of Zambia
GSG	Global Scenario Group
HC	High cost
HIPC	Heavily indebted poor countries
IBNET	International Benchmarking Network
IBT	Increasing blocks tariff
ILI	Infrastructure leakage index
$InfC$	Informal cost
IWSS	Intermittent water supply system(s)

[1] Other less frequently used notations are defined in the paragraphs where they are first mentioned.

IWS	Intermittent water supply
LA	Local authority
LC	Low cost
LPS or l/s	Litres per second
LWSC	Lusaka Water Supply and Sanitation Company
LWSN	Lusaka Water Supply network
MC	Medium cost
MDGs	Millennium Development Goals
MF	Market Force
MLGH	Ministry of Local Government and Housing
M-SIPDA	Modified Single-Iteration Pressure Driven Analysis
MSMSO	Multi-stage multi-scenario optimization
MWDSEP	Ministry of Water Development, Sanitation and Environmental Protection
NRW	Non-revenue water
NSGA II	Non-dominated Sorting Genetic Algorithm II
NSP	New Sustainability Paradigm
NWASCO	National Water Supply and Sanitation Council
OG	Offspring generation
PDA	Pressure driven analysis
PG	Parent generation
PR	Policy Reform
RMSE	Root mean square error
SDGs	Sustainable Development Goals
SIPDA	Single-Iteration Pressure Driven Analysis
STEEP	Social, Technological, Economic, Environmental and Political
UN	United Nations
WARMA	Water Resources Management Authority
WDS	Water distribution system
WRM	Water resources management
WSS	Water supply system
WTP	Water treatment plant
WWGs	Water Watch Groups
ZEMA	Zambia Environmental Management Agency

1 INTRODUCTION

1.1 Background and research justification

Intermittent water supply systems (IWSS) are piped water supply systems (WSS) that deliver water to users for less than 24 hours in one day (Charalambous and Laspidou, 2017). Since in some cases water is supplied only on some days of the week, IWSS can generally be defined as piped WSS in which water is supplied for a limited duration (Kumpel and Nelson, 2016). These systems supply water to more than one billion people in Low and Middle- Income countries worldwide (Charalambous and Laspidou, 2017). Despite being widespread, IWSS are inherently complex and are associated with many problems. Resolving these problems is difficult because of the complex interplay of factors that cause and sustain the intermittent water supply (IWS) mode coupled with many misconceptions about the causes and advantages of this water supply mode (Charalambous and Laspidou, 2017). Consequently, the subject of IWSS has become topical and highly discussed.

IWSS are complex because their development and sustenance encompass several interconnected factors with social, economic, political, natural and technical dimensions (Charalambous and Laspidou, 2017; Buurman and Santhanakrishnan, 2017; Simukonda et al., 2018b). Moreover, they manifest differently and as such, they have water supply schedules that vary greatly too (Charalambous and Laspidou, 2017). The implementation of the supply schedules adds to the operational complexities of IWSS. When there is no water supply, pipes are drained to varying extents. This results in very complex flow regimes (mixed-phase flows) in the early stages of water supply resumption and makes some parts of the water supply system (WSS) start receiving water long after supply is resumed (De Marchis et al., 2010). These complexities are a source of many uncertainties which contribute greatly to the difficulties in analysing, managing and planning for improving service delivery with these systems. Analysis and planning approaches that consider the uncertainties associated with IWSS are desirable.

One of the most popular ways of incorporating uncertainties in the planning process is scenarios (Lansey and Kang, 2012), but the development and application of scenarios to IWSS are uncommon.

Despite the complexities, IWS is seen as a coping strategy when there is water scarcity and a way of reducing background leakage (De Marchis et al., 2010). From this perspective, coupled with the lack of resources (skilled manpower, water and financial resources), IWSS will continue to be there into the foreseeable future and therefore tools and methods for analysing and designing them with intermittent supply rather than continuous supply concepts should be developed (Vairavamoorthy and Elango 2002; Vairavamoorthy et al., 2008). These methods and tools have not yet been developed. From the other viewpoint, IWS is seen as a major source of problems to the water supply utility, consumers and society such that IWSS are deemed to be failed systems and any recourse through intermittent mode should be viewed as intermediate and transitory while efforts are being made towards achieving continuous water supply (CWS) status (Myers 2003; McIntosh 2003; Klingel and Nestmann 2013; Simukonda et al., 2018b). From this perspective, conversion to CWS status is the goal. However, the conversion is difficult due to the various forms of lack of knowledge amongst the stakeholders. The development of a conversion approach that helps to reduce misconceptions and increase understanding amongst the stakeholders is critical.

Misconceptions are due to different viewpoints in many aspects. One of them is the advantages and disadvantages of IWSS as briefly discussed above. The other Misconception is about the scarcity or availability of water resources to ensure CWS. In the literature, attaining CWS is difficult for systems that have absolute water scarcity (Totsuka et al., 2004). This is true, but reality has shown that some water-scarce countries such as Bahrain, Qatar, Kuwait and Saudi Arabia have CWS, while some of the countries with abundant water resources such as Nigeria, Eastern part of India, Guinea and Democratic Republic of Congo have IWS and are among those with very poor water supply situations (International Monetary Fund, 2015; Charalambous and Laspidou, 2017). This shows that beyond water scarcity/abundance, there are other factors that either individually or collectively are responsible for IWS (Charalambous and Laspidou, 2017) and therefore determine the possibility of attaining CWS status. Lack of financial resources has been one of the major arguments against conversion to CWS in developing countries. However, where water resources are in abundance, such arguments are not supported by the literature as shown in Blair et al. (2005), Biswas and Tortajada (2010), International Monetary Fund (2015) and Simukonda et al. (2018b). Moreover, taking lack of financial resources as the reason for failure to convert to CWS implies that IWSS will continue deteriorating and water supply services will continue worsening thereby defeating the hope of improving water supply services for all as enunciated in the sustainable development goals (SDGs) (United Nations General Assembly, 2015). There are also different viewpoints between the need for equitable water supply to society and for revenue generation from water sales which is crucial to the financial sustainability of the water utility companies. These different viewpoints and others require that the subject of IWS is looked at from a broad perspective that seeks to understand the interplay between the social, economic, political, natural and technical factors key to the conversion to CWS (Buurman and Santhanakrishnan, 2017). This broad perspective can be attained through the development and application of scenarios.

Lack of knowledge has many angles. This can be from the consumers who are not aware or sure about the benefits of the CWS mode (Anand, 2017) and hence cannot demand better water supply services (Charalambous and Laspidou, 2017) or who do not see any possibility of converting to CWS because there are numerous limitations such as limited water resources and power outages and rapidly expanding cities (Water and Sanitation Program - WSP, 2010). In certain cases, consumers who feel water will be costlier under the CWS mode oppose efforts to convert to CWS (Buurman and Santhanakrishnan, 2017). Lack of knowledge can also be from WSS operators who, due to poor data management, have limited knowledge about the layout of the WSS infrastructure, its capacity and how much water is lost from it (Anand, 2017; Simukonda et al., 2018a). It may also be a lack of knowledge of what it takes to get the water supply services improved.

This is implied in the differences between the performance targets set by consultants versus the targets set by the local stakeholders and the failure to meet the consultants' targets as exemplified in the Lusaka Water Supply network (LWSN) case (Millennium Challenge Account - Zambia Limited, 2013; Lusaka Water Supply and Sanitation Company Limited, 2018a). Even though many technical approaches to the planning of water supply improvement interventions do not address these lack of knowledge aspects, they must be explored when planning for a sustainable conversion from IWS to CWS.

Since IWSS manifest differently and there exist numerous factors that lead to their development and sustenance, a systematic approach to converting these systems to CWS status requires the identification and analysis of the root causal factors rather than effects. The intertwined interplay between the root causal factors, various forms of lack of knowledge and the existence of controversies entails many uncertainties that need to be considered when developing a sustainable conversion approach. So far, the conversion processes reported in the literature have fallen short of identifying the root causes and inclusion of uncertainties in the planning process with the future outlook.

1.2 Research aim and objectives

The main aim of this research is to develop a methodology for the sustainable conversion of IWSS to CWSS. This aim is achieved through the attainment of the following objectives:

i. To review the literature on IWSS
ii. To identify and analyse the problems and the root causal factors of the IWSS.
iii. To develop scenarios for a sustainable conversion from IWS to CWS status.
iv. To develop an approach for modelling IWSS
v. To apply the developed scenarios and modelling approach to a real-world WSS.

1.3 The layout of the thesis

The thesis is divided into eight chapters which include this introductory chapter. The summary of the thesis layout is shown in Figure 1.1. The Figure also gives the link between the thesis chapters and the major aspects of the proposed process for the sustainable conversion to CWS status. A summary of the chapters other than the introduction is presented below.

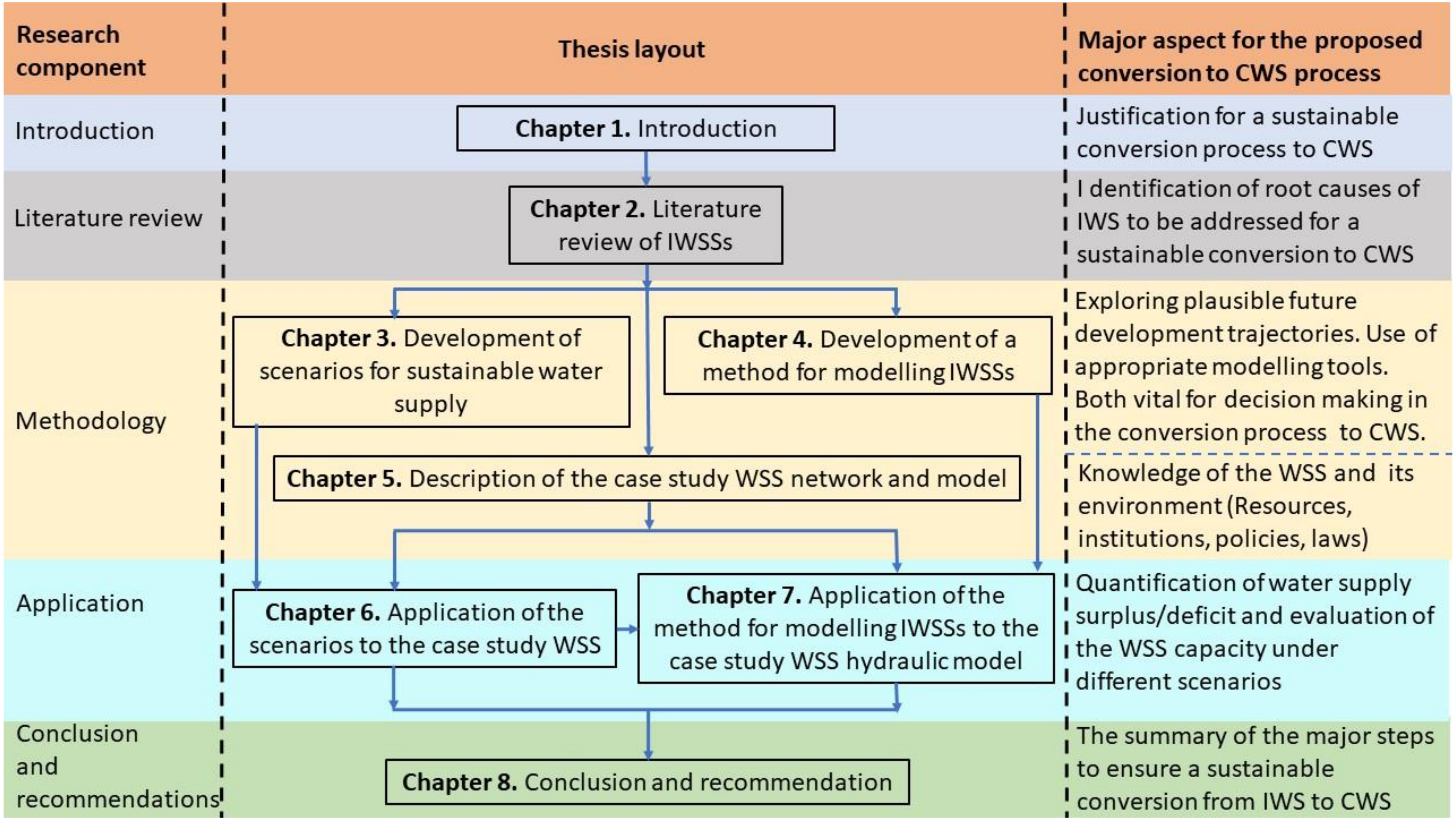


Figure 1.1: The thesis layout with research components and major aspects of the proposed conversion process to CWS status

Chapter 2 provides a critical review of IWSS problems, root causes and solution options for managing the problems. The chapter underscores the need to eliminate root causes rather than the effects of IWS to overcome problems associated with IWS. The intermittent water supply tree concept is proposed to liken the various aspects of IWS to the parts of a natural tree.

Chapter 3 highlights the significance of scenarios in incorporating uncertainties linked to future water supply improvement ventures. It also discusses the linkage between the root causes of IWSS (Chapter 2) and the drivers of change (for WSS, Water supply and demand management); and the linkage between the drivers of change and the Global Scenario Group (GSG) scenarios. Furthermore, the linkage between the GSG scenarios and the city expansion approaches are discussed. Then, the chapter presents how these linkages culminate in the development of the water supply sustainability scenarios which explore plausible future water supply situations under different water demand management approaches for a WSS.

Chapter 4 presents the development of the method for modelling IWSS. The developed method builds on the existing methods for modelling CWSS with deficient pressure. The mathematical model used in the method takes care of the multiple water supply schedules which are characteristic of IWSS and leakage modelling. Leakage modelling is important because in IWSS, the high leakage levels greatly affect the hydraulic behaviour of these systems.

Chapter 5 describes the Lusaka Water Supply Network (LWSN) as the case study network. The chapter highlights the challenges faced by the water supply sector in Zambia from the political, institutional, legal/policy and resource perspectives. This serves as an example of how the causes of IWSS (Chapter 2) affect the performance of a real-world WSS. Since the LWSN is large and complex, the chapter discusses how the network is decomposed into clusters using the modularity index. The chapter also highlights the complexities of the LWSN thereby justifying the analysis of a zone rather than the whole network in Chapters 6 and 7.

For isolating a zone, a new method for modelling the point where water supply to the zone is isolated from the rest of the network (the zone water offtake point) is proposed in this chapter. The method makes it possible to supply predetermined amounts of water to a zone. This is an important aspect for modelling of the zone's hydraulic capacity under different scenarios (Chapter 7).

Chapter 6 discusses the application of the water supply sustainability scenarios (developed in Chapter 3) to the LWSN. The chapter demonstrates the significance of considering different scenarios in the planning process of water supply improvement ventures.

Chapter 7 discusses the application of the developed method for modelling IWSS (Chapter 4) and the technique for isolating zones (Chapter 5) to the analysis of the Chelstone zone WSS capacity under different water supply sustainability scenarios developed in Chapter 3. This analysis helps to identify network elements that change according to scenarios and those that are the same across scenarios. This aspect helps to reduce the regret cost when a scenario other than the desired scenario occurs.

Chapter 8 presents the research conclusions and highlights the research contributions to the body of knowledge. The chapter also summarises the major steps for the sustainable conversion from IWS to CWS status as part of the key recommendations for practice and further research.

1.4 **Research major assumptions**

Filling of empty pipes: In IWSS, after water supply is resumed, the initial filling process of empty pipes and building up of pressure makes some parts of the system start receiving water far later than others (Sashikumar et al., 2003; Fontanazza et al., 2007; Cabrera-Bejar and Tzatchkov, 2009; De Marchis et al., 2010). Moreover, during the pipes filling process the Hazen Williams C-factor varies from low when the pipes have a lot of air pockets to high and normal values when the pipes are full and devoid of air pockets (Sashikumar et al., 2003). For simulations in this research, it is assumed that the pipes are full all the time and the C-factors are constant.

Leakage points: Leakage can take place along pipes, at joints, through valves, and as spillage from tanks and reservoirs. However, for modelling purposes, leakage is assumed to take place only at junctions. This is like the approach for modelling consumption demand in which several household connections are aggregated at a few selected junctions.

Minimum node pressure head: In this study, the minimum node pressure head is assumed to be equal to atmospheric pressure and as such, it is taken as zero.

Required pressure: This is the node pressure at which consumption demand at a node is fully met. It is assumed that pressure increase beyond the required pressure does not change the node outflow in respect of consumption demand. This assumption is based on the understanding that when using faucets, people regulate the water flows to suitable levels to enable them to get the required quantity. However, for IWSS in high-pressure areas, especially where water flows directly into storage facilities, consumption demand node outflow can exceed the actual consumption demand which leads to water wastage.

Non-revenue water: This comprises three components. These are real losses, apparent losses and unbilled authorised water consumption (Lambert and Hirner, 2000). However, for modelling purposes in this study, it is assumed that Non-revenue water **(**NRW) represents leakage (Taylor et al., 2019).

1.5 Originality and contribution to knowledge

This research has:

- Contributed to the understanding of IWSS root causal factors. Considering the correct root causal factors of IWS at each stage of the planning process of water supply improvement ventures is important. In the existing literature, the causes are often mixed with the effects of IWS leading to the proposal of water supply improvement interventions that seek to deal with effects while neglecting the causes completely. The identification and discussion of the root causes of IWSS in Chapter 2 provide the context of the major problems to contend with when planning and implementing any project aimed at handling problems associated with IWSS.

- Demonstrated the importance of the GSG scenarios (Market Force (MF), Policy Reform (PR), Fortress World (FW) and the New Sustainability Paradigm (NSP)) to the determination of the current policy, legal and institutional status of any country or state, and the exploration of plausible future development trajectories (Chapter 3 and Chapter 6). The institutional and policy/legal frameworks of a country or state determine the future development agenda and therefore affect the sustainability of all water supply improvement endeavours that have a future outlook. Conversion from IWS to CWS has a future outlook because it is supposed to be phased owing to financial resources limitations. The basic understanding of the relevant institution and policy/legal frameworks is important not only to the formulation of possible pictures of the future and the setting of benchmarks but also to the understanding of major stakeholders and limitations in the implementation of technical solutions to improving water supply from the current status through to the possible futures.
- Highlighted the process of developing scenarios for planning sustainable water supply improvement in IWSS which includes conversion from IWSS to CWSS (Chapter 3). The scenarios, which correspond to the GSG scenarios, are developed based on the water demand management concept. Water demand management is key to the environmental, social and economic sustainability of the water supply. The developed scenarios that explore the different water demand trajectories are important to the understanding of the sufficiency of water resources and the capacity of the WSS over the planning horizon. No studies have been found in the literature that have developed and applied scenarios to ascertain the sustainability of an IWSS improvement project with phased water resources development and different water demand management scenarios (Chapter 3 and Chapter 6).

- Developed a pressure driven analysis (PDA) hydraulic modelling method for IWSS that includes Leakage modelling (Chapter 4). The mathematical model for the method considers the similarities and differences between consumption water demand and leakage. It also considers multiple water supply schedules which are characteristic of IWSS. This contributes to the body of knowledge because currently, most of the existing PDA approaches that perform extended period simulations are developed for CWSS with deficient pressure, do not consider multiple water supply schedules and have no provision for modelling leakage which is significant in IWSS.
- Developed a technique for isolating a zone or district metered area (DMA) from the whole WSS. The technique enables the modelling of the water offtake point of an isolated part (zone or DMA) of a WSS such that predetermined amounts of water are supplied to the part. This is important in the application of scenarios to the isolated part of an IWSS because for these systems water supplied to a single part has to be proportional to its demand with that of other parts. No technique of isolating a part of an IWSS such that predetermined amounts of water can be supplied to the isolated part has been reported in the existing literature.
- Generated interventions required to attain CWS under different scenarios. These interventions are either from the water supply-demand balances (Chapter 6) or from the WSS hydraulic capacity (Chapter 7). This three-point application of scenarios to the analysis of an IWSS is novel.

2 LITERATURE REVIEW ON INTERMITTENT WATER SUPPLY SYSTEMS

This chapter sets the scene for this research by discussing IWS in terms of the extent to which it is practiced in the world, its causes, problems and solution options to the problems. The prevalence of IWSS is discussed in section 2.1. Section 2.2 discusses the root causes of IWSS. Section 2.3 discusses the problems of water supply intermittency to consumers, society and water utilities. The solution options for IWS related problems are discussed in section 2.4 which is followed by the chapter conclusion in section 2.5.

2.1 Introduction

WSS are complex infrastructures comprising raw water facilities, pump stations, treatment and finished water facilities which include the water distribution system (WDS). These systems are designed to process, store and deliver potable water to consumers reliably and adequately always (Walski et al., 2003; Farmani et al., 2005; United States Environmental Protection Agency, 2005). These objectives are generally met for WSS in developed countries. The situation is different in many developing countries where water supply is intermittent (Vairavamoorthy and Elango, 2002; McIntosh, 2003). Under IWS, the objectives of WSS are not met because IWSS have water supply schedules that vary greatly. Some of these systems supply water daily while others supply it less than 7 days a week with supply durations ranging from 1 to more than 19 hours on supply days in both cases (Sashikumar et al., 2003; Ingeduld et al., 2008; Danilenko et al., 2014; Simukonda et al., 2018b).

Water supply intermittency is practiced in many developing countries as a demand control technique and a way of reducing water losses through leakage (McIntosh, 2003; De Marchis et al., 2010; Charalambous and Laspidou, 2017). About one-third of consumers in Africa, half in Asia and two-thirds in Latin America are supplied water intermittently (Klingel and Nestmann, 2014). In South East Asia, 91% of WSS are intermittent (Vairavamoorthy and Elango, 2002).

Danilenko et al., (2014) provide data on the water supply and sanitation service indicators which include the average duration of water supply for countries reporting to the International Benchmarking Network (IBNET). Information on the average duration of water supply and hence the prevalence of IWSS in the world is depicted in Figure 2.1. While the Figure shows that the IBNET does not have data for the majority of countries, it still shows that IWS is a big issue. The magnitude of the problem should be understood from the fact that average hours of supply mask serious variations of water supply durations between utilities within a given country and between zones or district metered areas (DMAs) of the same WSS.

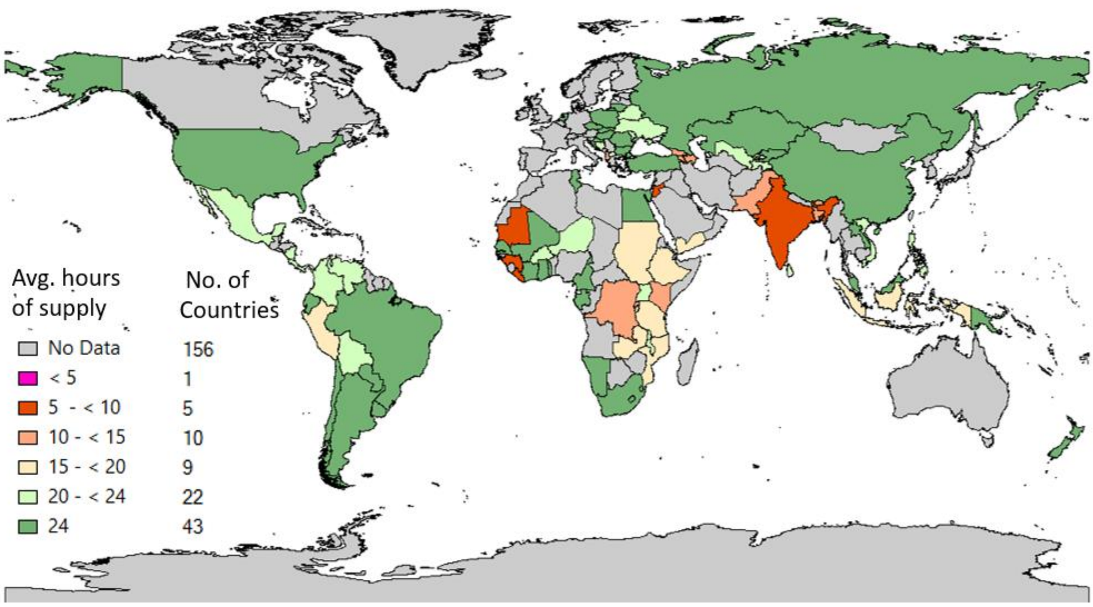


Figure 2.1: Average duration of water supply in the World (Adapted from: Sandvik, 2009; Danilenko et al., 2014)

2.2 Causes of IWSS

The first step towards solving a problem is the acknowledgment of its existence (Charalambous and Laspidou, 2017) which should be followed by the identification of its cause or causes. The causes of water supply intermittency in developed countries may be temporal events such as droughts, pollution accidents and earthquakes (Wagner et al., 1988; Solgi et al., 2013). This is not IWS per se because in such cases only part of the system is affected and water supply normalises when the causal factors are over. In many developing countries, however, the situation is more complex. In these, IWS (full-time intermittency) is practiced because water rationing and reduced duration of water supply are norms for the whole system and reverting to continuous supply mode is difficult. In these systems, temporal causes also exist, but they seem insignificant and are obscured by major causal factors.

IWS causes are so many and so interconnected that isolating more specific ones is difficult (van der Bruggen et al., 2010). From a water scarcity point of view, causes of IWS are grouped into three as those due poor management (technical scarcity), weak financial capability of a utility or government to expand existing infrastructure and poor planning (economic scarcity), and insufficient quantities of water at the source (absolute scarcity) (Totsuka, et al., 2004; Charalambous and Laspidou, 2017). From a technical perspective, the causes of IWS include deficient system planning, missing system concept, deficient infrastructure management and limited system knowledge. These causes result in consequences that aggravate them such as deterioration of the WSS infrastructure, increasing water losses and wastage (Klingel, 2012). Critical analysis of the causal – consequence pathways of water supply intermittency, results in the identification of internal factors that form the self-reinforcing mechanism of the water supply intermittency because they both cause and develop from it and external factors which include increased population, political misjudgments of progress indicators, development, hydrological regime changes and electricity blackouts (Galaitsi et al., 2016). The self-reinforcing mechanism is called the vicious downward spiral of intermittency (Ingeduld et al., 2008; Hunter et al., 2010; Charalambous and Laspidou, 2017) as exemplified in Figure 2.2. Internal factors to this mechanism include excessive NRW (real and apparent water losses) and poor compliance with the payment of water bills. The key external factors, identified through the literature review, are shown in Table 2.1 as root causal factors discussed below.

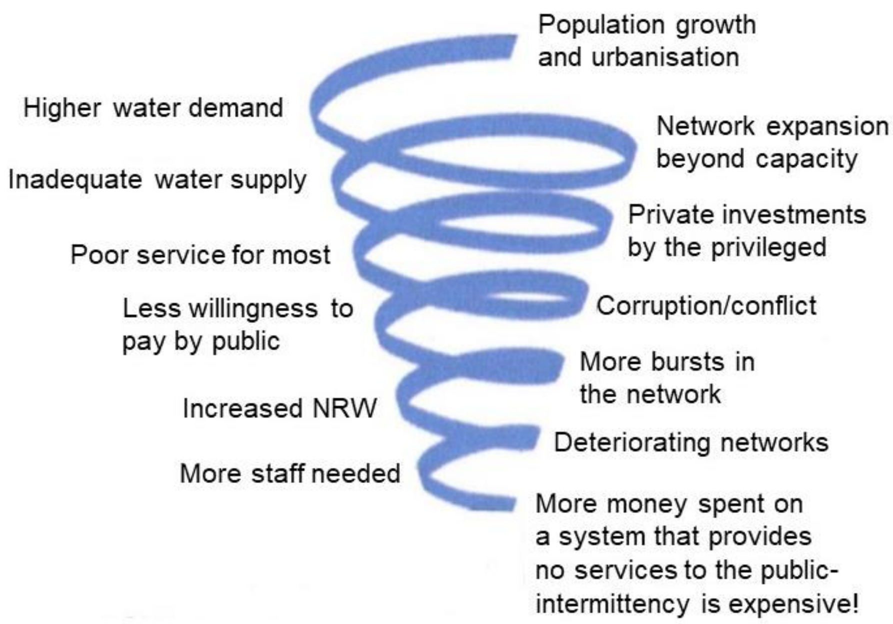


Figure 2.2: Downward spiral of water supply intermittency (Charalambous and Laspidou, 2017)

Table 2.1. Causal factors of intermittent water supply mode (Simukonda et al., 2018b)

Causal factors		**Key references**										
Root causal factors	**Elements of the causal factors**	Myers (2003)	McIntosh (2003)	Totsuka et al. (2004)	Vairavamoorthy et al (2001)	Bakker et al. (2008)	Government of India (2009)	van der Bruggen et al. (2010)	Lehmann (2010)	Klingel (2012)	Galaitsi et al. (2016)	Charalambous and Laspidou, (2017)
Poor governance	Poor infrastructure funding			√		√		√				
	Poor political judgment of progress		√								√	
	Interference by elected leaders		√						√			
	Poor policies/guidelines and corrupt practices		√					√	√		√	
Demographic and economic dynamics	Increasing water demand	√		√	√	√		√	√	√	√	√
Hydrological regime changes	Water shortage at source	√		√	√			√		√	√	√
Poor system management and operation	Poor database management						√		√	√		√
	Low tariffs	√	√	√		√	√		√			
	Poor system metering		√				√		√			
	Poor revenue collection	√	√	√			√					
	Poor maintenance	√	√	√		√	√	√		√		
	Demotivated staff			√		√	√	√		√		
Limited skilled manpower	Low usage of modern analytical tools	√				√	√			√		
Unplanned extension of systems	Poor pressure conditions in the system		√	√						√	√	√
Lack of customer awareness	Water use behaviour	√		√			√		√		√	√
Poor electricity supply	Affects water pumping schedules	√		√	√						√	

2.2.1 *Poor governance*

Governance is a highly context-based concept and it is very complex in terms of definitions and interpretations. Because of these, there is no agreed form of governance that fits all nations (Tortajada, 2010). However, principles of governance such as participatory decision-making processes, adherence to the rule of law, transparency in dealings, responsive institutions and processes, upholding equity, effectiveness and efficiency in processes and institutions, and accountable decision-makers are common (Rhodes, 1996; UNDP, 1997; Adeyemo, 2003; Rogers et al., 2002; Batchelor, 2009). Poor governance exists where adherence to these principles is lacking (Rogers and Hall, 2003) and it is linked to poor water supply in almost all developing countries (McIntosh, 2003) because it diminishes funding for infrastructure development and operation (Totsuka et al., 2004; Dahasahasra, 2007; Bakker et al., 2008; van der Bruggen et al., 2010). In this regard, government, being the key player in national or state political, economic and administrative governance (Rogers and Hall, 2003), has the responsibility to ensure resources are mobilised to develop the water infrastructure capable of adequately providing water to all its citizens (United Nations Organisation, 2010).

Understanding the extent to which governments acknowledge and undertake the responsibility to ensure that adequate potable water is provided for all nationals is important. This understanding can be better achieved from the human rights perspective. Initially, access to safe drinking water and sanitation was recognised as a basic human need but not a right (World Water Council, 2009; Moore, 2013). However, it became a non-legally binding right by the declaration of the United Nations (UN) in 2010 (United Nations Organisation, 2010; Brunner et al., 2015). Some countries have not explicitly endorsed access to water as a right while others have (Brunner et al., 2015). However, even those that have endorsed the right to water and included it in the bills of rights of their constitutions, the right to water is one of the economic, social and cultural rights, and as such it can only be realised progressively as resources permit (United Nations General Assembly, 1966; Government of South Africa, 1996; The Republic of Kenya, 2010; Brunner et al., 2015).

Thus, governments cannot be compelled through litigation to ensure adequate water supply for all citizens as they are only required to show that resources are not available (Government of the Republic Zambia, 1991). Accordingly, lack of resources is one of the most cited problems in developing countries and it is a major hindrance to improving water supply conditions (Gutierrez, 2007; Chitonge, 2011). This lack of resources is exacerbated by corruption (McIntosh, 2003; Lehmann, 2010; van der Bruggen et al., 2010) which is due to poor legislation enforcement (Batchelor, 2009). In this regard, the 2009 Istanbul Ministerial Declaration, pledged to improve water sector governance and prevent corruption (World Water Council, 2009).

Besides affecting investment to infrastructure development and operation, poor governance contributes to water scarcity (Ioris, 2012; Giglioli and Swyngedouw, 2008; Catalano et al., 2013) and affects WSS management and operations through political interference (McIntosh, 2003; Seetharam and Bridges, 2005; Mckenzie and Ray, 2009).

Since poor governance elements are rife in developing countries, all interventions aimed at improving water management and supply conditions depend on improved governance (McIntosh, 2003; Blair et al., 2005; Biswas and Tortajada, 2010; Jones et al., 2014; International Monetary Fund, 2015). However, the problem is that there are relatively few studies that attempt to make systematic links between governance and the effectiveness of service delivery. On one hand, studies that focus on governance factors do not explore in detail the causal link between governance dynamics and effective service delivery (which include water supply). On the other hand, studies that focus more on service delivery aspects do not cover governance issues in detail (Jones et al., 2014). It is noteworthy that issues affecting both service delivery and governance define the course of water sector reforms and the dynamics of water sector institutions with water law, policy and administration being the major components. Factors that lead to changes in water institutions can be considered as external from the water sector institutions and internal factors. However, the interrelationship of these factors makes it difficult to isolate their roles or to generalise the direction of their effects (Saleth et al., 2000).

Recognising the linkage between water supply service delivery, governance and water sector institution dynamics, improving water supply services inevitably requires improving governance which, in this case, involves strengthening water sector institutions that regulate the interactions between stakeholders – government (political) entities, cooperating partners including donors, civil society groups, water utilities and local communities. Through these interactions, water supply problems are identified and the need to mobilise resources to solve them is acknowledged (Dorado, 2005). The success or failure of technical solutions to be applied depends on the informed and agreed on position of the stakeholders within the existing policy/legal and institutional frameworks. To this effect, interdisciplinary approaches are indispensable. These do not only enable a proper understanding of the nature of the water supply problems, but also ensure effective information dissemination which helps in raising awareness of the problems, creating demand for improvement and reaching consensus on sustainable water supply improvement solutions in developing countries (Simukonda et al, 2018b).

2.2.2 *Demographic and economic dynamics*

As shown in Table 2.1 population growth, urbanisation and economic dynamics are among the key causes of water supply intermittency. Population growth and urbanisation are global problems. They raise concerns both in developed and developing countries (Buchberger et al., 2008). However, populations in developing countries are growing rapidly resulting in rapid increases in water demand (Myers, 2003; Totsuka et al., 2004; Dahasahasra, 2007; Bakker et al., 2008; Ingeduld et al., 2008; Lehmann, 2010; Klingel, 2012) and ultimately water scarcity. High population growths are worsened by urbanisation (Cohen, 2006) which is the movement of people, in search of improved livelihood, from agricultural dependent rural areas to manufacturing and service industry-dependent urban areas (Henderson, 2003; Cohen, 2006). This rural-urban drift contributes to the high concentration of people in cities (Henderson, 2003; Butler, 2004; Bao and Fang, 2012; Bao and Chen, 2015). This results in the rapid increase of demand for water and sanitation services beyond the capacity of existing infrastructure leading to numerous challenges with the effective provision of these services (Seetharam and Bridges, 2005; Le Blanc, 2008).

In many developing countries, problems related to population growth and urbanisation are linked to governance aspects of land allocation and land tenure. The failure of the formal land allocation system leads to unplanned settlements that are built without coordination (Lusaka City Council and Environmental Council of Zambia, 2008; Ioris, 2012; Criqui, 2015). Many migrants from rural areas settle in these settlements (van der Bruggen et al., 2010). The settlements lack water supply and sanitary facilities (de Waele and Follesa, 2003; Cohen, 2006; Lusaka City Council and Environmental Council of Zambia, 2008) because utilities or governments are unwilling to provide services in them (Bakker et al., 2008; Mason, 2009; van der Bruggen et al., 2010; Chitonge, 2011; Ioris, 2012). Reasons for the unwillingness include technical challenges in laying pipes (Mason, 2009; van der Bruggen et al., 2010; Criqui, 2015) as structures are built in a manner that does not provide space for laying pipes, the high cost of installing pipes over long distances with a few isolated individuals connecting hence low revenue, and some of these settlements are considered illegal (Lusaka City Council and Environmental Council of Zambia, 2008). Since unplanned settlements are poorly supplied with water, it is unclear to what extent they contribute to the development of IWSS.

Economic dynamics refer to economic development which directly contributes to changes in water availability and consumption. Water availability is reduced through quality deterioration caused by industrial effluent pollutants (Nachiyunde et al., 2013). Industrial water consumption also increases (Bao and Fang, 2012) though this depends on the area being considered, type of industry and temporal scales that affect technology development and adoption (Bao and Chen, 2015). Economic development also makes people wealthier and studies show that per capita water consumption increases as people become more affluent (Totsuka et al., 2004; Butler, 2004; Vásquez and Espaillat, 2014). However, this is not conclusive as the correlation between affluence and water consumption is low (Nauges and Whittington, 2016).

2.2.3 *Hydrological regime changes*

Hydrological regime changes can be climate or land-use related. Climate change is caused by both natural climate variability and anthropogenic factors (IPCC, 2014), but the separation of the effects of the two causal factors is difficult. Moreover, climate change effects are difficult to generalise because they vary from region to region and from one scale to another (van Lanen et al., 2007). The uncertainties are even more at river basin scales and when it comes to future projections (IPCC, 2007). Despite these uncertainties, current evidence shows that it will increase occurrences of extreme droughts and floods in various regions of the world (IPCC, 2007; Commission of the European Communities, 2007; Vairavamoorthy et al., 2008).

Concerning water supply, droughts are critical because they are more complex and have greater consequences than floods. A look at droughts requires that they are distinguished from water scarcity and aridity. A drought is a temporal decrease in average water availability owing to lower than normal precipitation worsened by high air temperatures (or low temperatures for winter drought) and evapotranspiration (Wilhite, 1996). Water scarcity is a long-term inadequacy of water when demands exceed available water resources (Commission of the European Communities, 2007). Thus, water scarcity is linked to factors that increase water demand such as demographic and economic dynamic factors. On the other hand, aridity is the permanent climatic condition of regions with low rainfall (Wilhite, 1996). Aridity is not the subject of this research. Droughts are complex to handle because they develop slowly, affect many people and are long-lasting with effects on water supply going beyond the drought duration (Wilhite, 2000; Commission of the European Communities, 2007). Droughts affect water supply through water quantity reduction in surface water bodies and aquifers (Dracup et al., 1980; Wilhite, 2000; Charalambous, 2012; Miyan, 2015).

Land use practices contribute a lot to hydrological regime changes. Land covered with forests has high infiltration rates and hence low runoff (Mishra et al., 2007). High infiltration rates sustain dry season stream base flows. Several land use factors contribute to deforestation or loss of natural land cover. These include agriculture, settlements, timber harvesting and energy provision (Allen and Barnes, 1985). Increasing cultivated land, increases non-point source water pollution, surface runoff, sediment loads (Bullard, 1966; Mango et al., 2011) and irrigation water demand in dry seasons (Wagner et al., 2013). In urban areas, settlements increase water resources pollution and rainfall runoff because of paved surfaces and soil compaction. These factors reduce usable surface and ground water in terms of quantity, quality or both (van der Bruggen et al., 2010).

Increases in drought frequency and duration have been observed in many parts of the World (Commission of the European Communities, 2007; European Commission, 2012; Miyan, 2015). Droughts have had effects in the least developed Asian countries (Miyan, 2015). In Europe, the drought period 2007–2009 subjected the town of Lemesos now known as Limassol in Cyprus to IWS (Charalambous, 2012). In Africa, a combined effect of climate change and anthropogenic activities in Zambia is attributed to the drying up of boreholes and Chongwe River which results in critical water shortages in Chongwe district (NWASCO, 2014a, Beekman, 2016).

The IPCC (2014) states how climate change will affect WSS and advises on how to attain resilience. In developed countries, there are some guidelines on how climate-related extreme events should be handled (Commission of the European Communities, 2007), but in developing countries, there are no clear measures in place to manage such events (O'Hara and Georgakakos, 2008). In these, climate change effects are not a priority (Danilenko et al., 2010; Oates et al., 2014), land use planning is poor (Ioris, 2012) and little is being done to solve current hydrological problems and to develop water utilities adaptive capacities against future extreme events (Simukonda et al., 2018b).

2.2.4 *Poor system management and operation*

Poor system management and operation have many aspects almost all of which relate to database management. Poor data management makes revenue collection inefficient (Chan, 2009; Biswas and Tortajada, 2010), tariffs setting difficult (Lehmann, 2010) and proper analysis of the state and performance of the WSS infrastructure unattainable (Klingel, 2012).

Suitable tariffs are useful water resources management tools (Boland and Whittington, 2000) because they ensure efficient water resources allocation, water affordability and utility sustainability through full cost recovery (Rogers et al., 2002; Lehmann, 2010; NWASCO, 2014b; Nauges and Whittington, 2016). Very few water utilities worldwide charge tariffs that cover the full cost of supplying water (Nauges and Whittington, 2016). However, in developing countries, tariffs are too low even to meet operation and maintenance costs (McIntosh, 2003; Government of India, 2009; Danilenko et al., 2014). Part of these costs and basically all capital costs are paid by governments, donors and cooperating partners (Lehmann, 2010; Republic of Zambia, 2011b; Nauges and Whittington, 2016). Since government funding is generally erratic, donors and cooperating partners remain the main funders (Blair et al., 2005; Biswas and Tortajada, 2010; International Monetary Fund, 2015).

Many tariff structures exist, but the increasing blocks tariff (IBT) structure is the most common in developing countries (Boland and Whittington, 2000; McIntosh, 2003; Komives et al., 2005; Nauges and Whittington, 2016). The IBT is said to make the rich subsidise water consumed by the poor and industries subsidise domestic water consumption (Boland and Whittington, 2000; Rogers et al., 2002; NWASCO, 2014b). However, the poor do not benefit from current subsidy arrangements because they are poorly supplied with water by utilities (Boland and Whittington, 2000; Rogers et al., 2002; Mason, 2009; Government of India, 2009; Lehmann, 2010; Nauges and Whittington, 2016). Consequently, the issue of tariff structures remains unresolved. This is because some studies show the need for subsidies (McIntosh, 2003; Komives et al., 2005; Mckenzie and Ray, 2009; Mason, 2009; Biswas and Tortajada, 2010) while others argue against them (Whittington et al.,1991; Rogers et al., 2002). The latter group is to some extent supported by those that show that the poor are willing to pay more for improved services (Whittington et al., 1991; McIntosh, 2003; Franceys and Jalakam, 2010; Vásquez and Espaillat, 2014; Rananga and Gumbo, 2015). The contrasting views on the poor is an indication that the current subsidised (IBT) tariffs are based on insufficient information on 1) the role of tariffs in the water supply sector, 2) the performance of IBT tariffs as compared to other tariff structures (Boland and Whittington, 2000) and 3) the poor/rich divide in terms of levels of service, willingness and ability to pay for water services. In this respect, comprehensive information is vital not only to facilitate the setting of cost-reflective tariffs for those that are willing and able to pay full water supply costs for improved services, but also for identifying the befitting subsidies beneficiaries who will also benefit from raised tariffs through improved supply conditions (Whittington et al., 1991; Rogers et al., 2002; McIntosh, 2003).

The state and performance of WSS infrastructure depend on the infrastructure maintenance regime. Poor infrastructure maintenance is a common but acute problem in developing countries (International Monetary Fund, 2015). Poor maintenance schedules and repair works contribute to increasing WSS operation and maintenance costs (Buchberger et al., 2008) and lead to increasing leakage (Giglioli and Swyngedouw, 2008; Catalano et al., 2013).

Four factors contribute to the poor maintenance problem. The first factor is the poor water sector institutions. This defeats any investment in the water sector. It makes even donor support ineffective (International Monetary Fund, 2015). The second factor is the lack of utility financial resources due to poor revenue collection (Water and Sanitation Program, 2011). The third factor is poor data management which leads to poor WSS knowledge (Seetharam and Bridges, 2005; Klingel et al., 2012) and poor knowledge of water consumers (Biswas and Tortajada, 2010). The last factor pertains to the lack of technical and managerial skills which result in poor system analysis, complicated procurement procedures and poor workmanship (McIntosh, 2003) worsened by the usage of substandard materials (Republic of Zambia, 2011b).

2.2.5 *Limited skilled manpower*

Limited skilled manpower affects WSS through different aspects including planning, management and operation (McIntosh, 2003; Government of India, 2009). Managing limited water resources efficiently by reducing water losses is data and technology intensive. However, the management of databases is poor plus technical skills and use of technology are low in developing countries (Farley and Liemberger, 2005) especially, where local municipalities provide water services (Vásquez and Espaillat, 2014). This is attributed to a lack of know-how, demotivated staff and the failure by decision-makers to recognise the importance of functional databases (Klingel and Deuerlein, 2009; Klingel, 2012). Lack of the necessary technical and managerial skills to improve water services delivery to match the rapid population growth and urbanisation in developing countries is well known (Myers, 2003; Blair et al., 2005; Cohen, 2006; Government of India, 2009; Franceys and Jalakam, 2010). But solutions to this problem are rare because capacity building plans, though common, lack details on *what* skills or capacities need to be developed.

2.2.6 *Unplanned extension of systems*

Extension of WDSs is important in developing countries because many residential areas do not have access to safe drinking water. The need for these extensions is reflected both in the Millennium Development Goals (MDGs) (Danilenko et al., 2014) and SDGs (United Nations General Assembly, 2015). The MDGs were- and SDGs are- high-level political commitments by member states. Thus, from the political perspective, the extension of WDSs is very desirable in developing countries (McIntosh, 2003; Le Blanc, 2008) and this contributes to making WDS extension one of the utilities' performance indicators (Chitonge, 2011; Danilenko et al., 2014). The problem is that these extensions are implemented without informed planning or any planning at all as utility databases are poorly managed (Klingel, 2012) and networks are extended to new areas after they are developed (Rakodi, 1987; Lusaka City Council and Environmental Council of Zambia, 2008). Because knowledge of such systems is often poor (Seetharam and Bridges, 2005; Klingel, 2012) and water quantities are limited, WDSs are consequently, extended beyond their hydraulic capacities (McIntosh, 2003).

2.2.7 *Lack of customer awareness*

Lack of awareness affects water consumers as well as utilities. Consumers are affected by the poor water supply services which have become normal (McIntosh, 2003; Biswas and Tortajada, 2010). They are also affected by water tariffs because they feel water should be supplied for free (Hunter et al., 2010) or because tariffs are unjustifiable due to poor services. Utilities are affected through i) challenges of implementing water pricing as a cost recovering and water conservation measure (Lehmann, 2010), ii) consumers' unwillingness to pay water bills (Chan, 2009; Biswas and Tortajada, 2010), iii) consumers' poor cooperation with improvement measures such as conversion to continuous supply status (Franceys and Jalakam, 2010), and iv) vandalism and water theft.

Since many people in developing countries have lived with poor water supply services all along, imagining and demanding for better supply conditions is difficult without extensive awareness campaigns to influence the perception of levels of service, water consumption trends and cooperation with improvement measures by utilities.

2.2.8 *Poor electricity supply*

Electricity supply is critical to water utilities for pumping. Because of this, power outages (blackouts) are among the root causes of water supply intermittency (Myers, 2003; Vairavamoorthy et al., 2001; Totsuka et al., 2004; Franceys and Jalakam, 2010; International Monetary Fund, 2015). This complicates matters because poor electricity and water supply have many causal factors in common (Meier, 1990).

Electricity generation in developing countries is either fossil fuel or hydropower dependent. In countries that depend on hydropower, electricity generation is highly linked to the effects of climate change (Vergara et al., 2007; IPCC, 2007; NWASCO, 2015; Beekman, 2016).

In those that depend on fossil fuels, electricity generation is affected by their local availability and quality, imported fuel prices, the mix of fuels and efficiency of technologies used (Eberhard et al., 2011; International Energy Agency, 2015). Whether from hydropower or a fuel-powered plant perspective, environmental requirements compound challenges of increasing electricity generation capacities by developing countries. With numerous challenges faced by these countries, electricity shortages are likely to continue affecting WSS negatively. Establishing the extent to which WSS are affected by poor electricity supply and developing measures to mitigate the effects are important (Simukonda et al., 2018b).

2.3 Problems associated with intermittent water supply

Problems associated with water supply intermittency can be viewed from two perspectives as shown by the IWS tree (Figure 2.3). From the first perspective, the mere intermittent operation of a system designed to supply water continuously has inherent problems. These include inequitable water distribution, inconvenience to consumers and meter malfunctioning. Possibly, these problems can be mitigated by appropriate designs of IWSS, and the use of the right technologies and equipment. Mitigation of some IWS problems is important because of the understanding that due to challenges such as absolute water scarcity, IWSS will continue existing into the foreseeable future. From the second and broader perspective, problems will always exist provided WSS are intermittent (Totsuka et al., 2004). From this view, problems can be categorised as those faced by utility companies, consumers and society as discussed in detail below.

2.3.1 *Problems faced by utility companies*

Pressure fluctuations, from on and off pumping routines and continuous opening and closing of valves, increase pipe failure rate due to pressure surges and fatigue (Batish, 2003; Myers, 2003; Totsuka et al., 2004; Klingel and Nestmann, 2014). Joints are also weakened (Myers, 2003). These increase bursts and background leakages.

Alternating between dryness and full flow increases meter deterioration and inaccuracies thereby increasing apparent water losses (Dahasahasra, 2007). Valve repairs and replacement frequencies are raised leading to high maintenance costs (Totsuka et al., 2004). There are also increased costs due to overtime payment of staff operating valves and pumps to implement zonal water allocation (Dahasahasra, 2007; Charalambous, 2012).

2.3.2 *Problems faced by consumers*

Water consumers are negatively affected by IWS through coping costs in terms of storage facilities, pumps and pumping costs. Questionable water quality adds to coping costs through household water treatment methods and facilities. Under IWS, water quality degradation takes place due to two mechanisms. The first one is intrusion and backflow and the second one is biofilms, loose deposits, and microbial growth (Kumpel and Nelson, 2016). Moreover, when there is no water supply from the WDS, alternative water sources are either more expensive (Totsuka et al., 2004; Rosenberg et al., 2008) or are far away (Klingel, 2012). Furthermore, a lot of time is wasted on water collection (Totsuka et al., 2004; Klingel, 2012), and in general, poor people are the most affected by water supply intermittency as they tend to pay more (Whittington et al., 1991; Bakker et al., 2008) even through bribes to induce those operating the valves or pumps to supply water to a given location (McIntosh, 2003).

2.3.3 *Problems faced by society*

Society is affected by water supply intermittency in different ways. This supply mode results in inequitable water distribution. This is because when supply is restored, consumers who are far from water sources or in high elevation locations are subjected to low pressure and as a result, they get less water than those near sources or in low lying areas (Batish, 2003; Vairavamoorthy and Elango, 2002; Totsuka et al., 2004; Ingeduld et al., 2008; De Marchis et al., 2010). Inequity also occurs when water is wasted by consumers in high-pressure areas who replace old stocks with fresh stocks of water each time supply is restored (McIntosh, 2003). Water is also wasted through taps that are left open when there is no water (CPHEEO, 1999; Batish, 2003; Mohapatra et al., 2014) so that when water comes people can notice or it goes straight into storage facilities some of which tend to overflow (Government of India, 2009). These wastage acts take place while those in low-pressure areas have little or no water at all.

Intermittent systems expose consumers to health hazards, which can have dire socio-economic consequences, firstly through water contamination (Totsuka et al., 2004) and secondly through poor sanitary conditions and practices (CPHEEO, 1999; Dahasahasra, 2007; Rosenberg et al., 2008; Rananga and Gumbo, 2015). From the second perspective, IWS may impair the functionality of school toilets thereby exposing children to health hazards (Lundblad and Hellström, 2005). Some female children's education is affected during menstruation as they feel uncomfortable when there is little water to wash themselves at home or school and so they do not go to school (Maimaiti and

Siebert, 2009). Moreover, children's education (especially females) is affected through their involvement in fetching household water instead of going to school (Blair et al., 2005; Brunner et al., 2015; International Monetary Fund, 2015).

Society is also exposed to fire hazards because when there is no water supply, fire hydrants are affected (CPHEEO, 1999; Batish, 2003). In some cases, this is mitigated by installing firefighting water reserves on sensitive premises (Batish, 2003). Otherwise, fire trucks go long distances to fetch water before attending to a fire scene (Simukonda et al., 2018b).

2.4 Options for addressing problems associated with IWS

In the literature, there are two options for minimising the negative effects of water supply intermittency. The first is to develop approaches and tools for designing and analysing intermittent supply systems (Vairavamoorthy and Elango, 2002; Vairavamoorthy et al., 2008). This option is aimed at overcoming problems associated with the intermittent operation of systems designed to be operated on a CWS basis. The second is that of converting to CWS. This aims at overcoming all IWS problems. From the second option's perspective, IWSS are failed systems and any recourse through intermittent mode should be viewed as intermediate and transitory while efforts are being made towards achieving continuous supply status (Myers, 2003; McIntosh, 2003; Klingel and Nestmann, 2014).

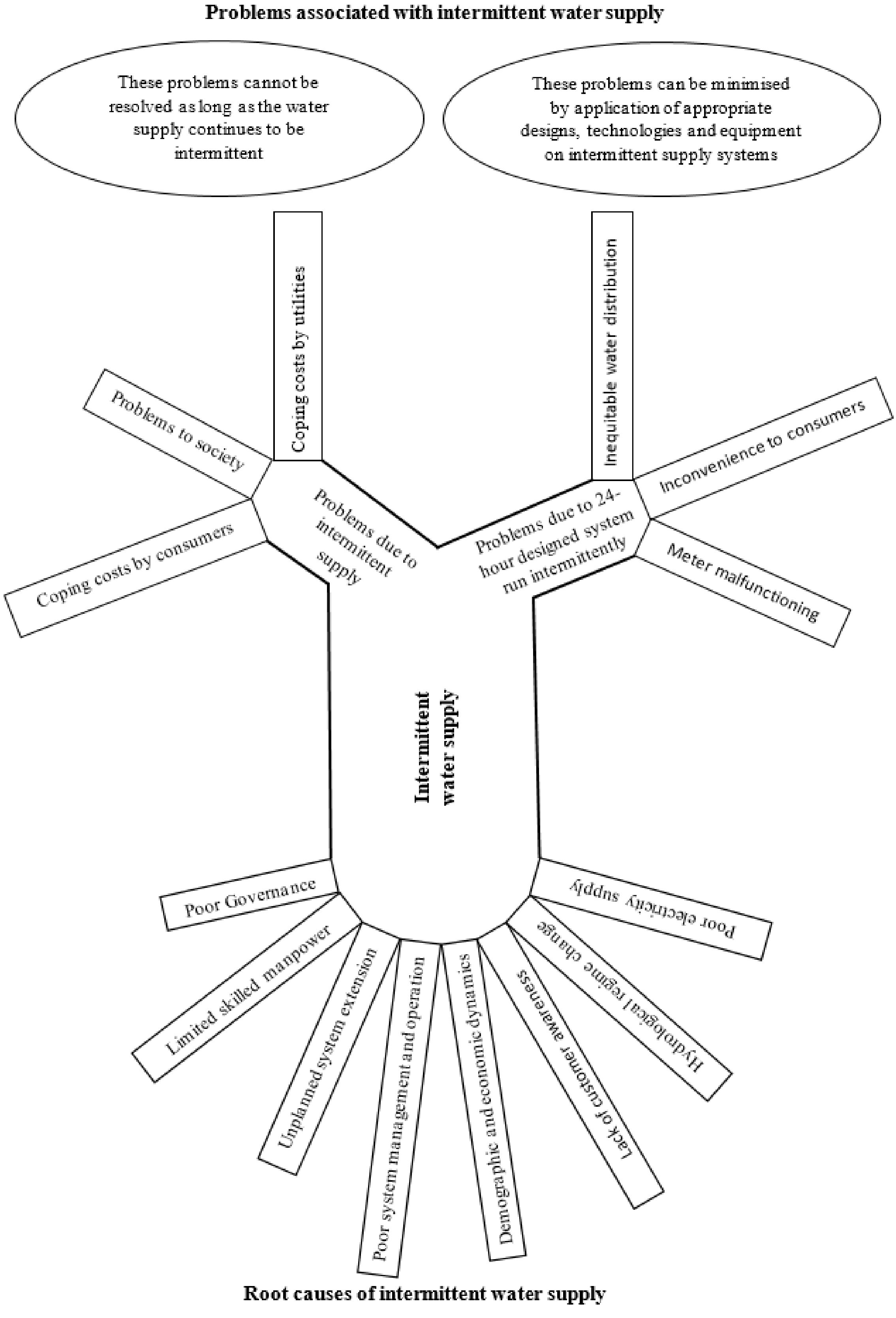


Figure 2.3: The intermittent water supply tree (Simukonda et al., 2018b)

2.4.1 *Analysis of intermittent water supply systems*

Analysis of IWSS is a challenge because many metrics do not apply to these systems. For instance, water demand is difficult to ascertain due to storage and wastage (De Marchis et al., 2010; Mohapatra et al., 2014). Leakage assessment is also difficult because night flows can be as high as day flows (Ndirangu et al., 2013) in certain cases even higher (Brian Colquhoun Hugh O'Donnell and Partners, 2010). Aside from metrics, modelling of WSS provides vital information for decision making. Simulation of the behaviour of WSS facilitates quicker analysis of different scenarios and provides valuable information for their planning, designing, operation and management that would be impossible if studies were conducted directly on the physical system elements. For modelling purposes, WSS can be categorised as (Figure 2.4):

i. CWSS with adequate pressure
ii. CWSS with deficient pressure
iii. IWSS with deficient pressure

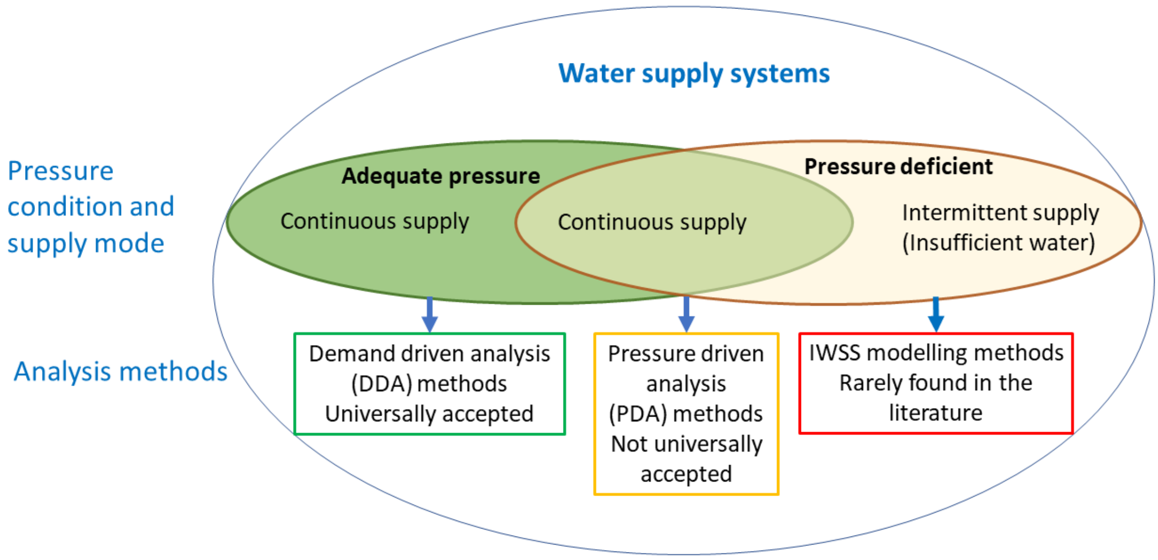


Figure 2.4: Alternative water supply systems and the corresponding modelling approaches (Adapted from: Simukonda et al., 2019)

CWSS with adequate pressure are those that normally have pressure at all demand nodes that enable the supply of adequate amounts of water always. For these, many universally accepted demand driven analysis (DDA) methods and software tools exist. The most common example of DDA-based software tools is

EPANET 2 (Rossman, 2000). The DDA methods' assumption that known and fixed nodal demands are fully supplied regardless of the nodal pressure (Giustolisi and Walski, 2012; Paez et al., 2018) does not manifest problems in CWSS with adequate pressure because these systems have high hydraulic capacities.

For pressure deficient systems (which include IWSS), DDA methods provide unreliable results as they show full water supply even when demand node pressure is below the threshold that guarantees full water supply (Wagner et al. 1988; Giustolisi and Walski, 2012). Moreover, in DDA models, leakage is assumed to be independent of pressure (Germanopoulos, 1985) which is against the fact that leakage depends on pressure (Germanopoulos, 1985; Sebbagh et al., 2018). Leakage is more realistically modelled as a pressure dependent demand (Sebbagh et al., 2018) for which pressure driven analysis (PDA) methods are recommended. Consequently, existing software tools that are based on DDA methods such as EPANET 2.0 cannot directly be used to analyse IWSS which are pressure-dependent and multi-phase flow systems. The recently realised EPANET 2.2 (Rossman et al., 2020) can perform PDA, but it was not yet released during the period of this research. Because IWSS are pressure-dependent, PDA models are appropriate (Germanopoulos, 1985; Chandapillai, 1991; Giustolisi et al. 2008; Lee et al., 2016). In these models, both nodal pressure and water supply are variables dependent on known pressures and flows at system water entry points and the defined relationships between nodal pressure and outflows (Lee et al., 2016). Although no universally accepted PDA method exists, there are many PDA methods that are used for modelling pressure deficient systems other than IWSS (Discussed in Chapter 4).

2.4.2 *Pressure driven analysis methods*

Since IWSS are also pressure deficient systems, the existing PDA methods that are developed for CWSS with deficient pressure form the basis for developing the method for modelling them. The existing PDA methods can be categorised into three groups as: 1) models that do not involve interfacing with an external hydraulic solver, 2) EPANET extension methods, and 3) methods that add artificial elements to the EPANET input file.

Models that do not involve interfacing with an external hydraulic solver

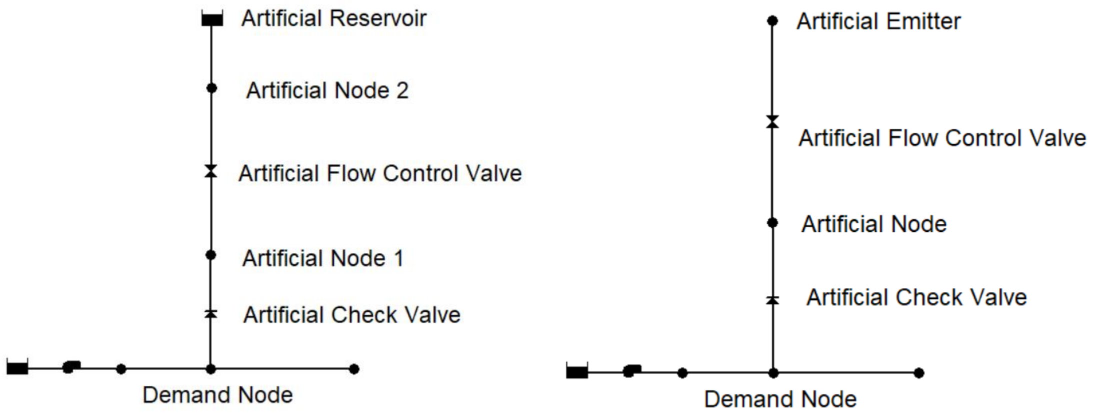
This category of PDA methods comprises approaches that have mathematical formulations that include PDA aspects. These mathematical formulations are used to determine pipe discharges and node pressure heads using procedures or algorithms such as the Newton-Raphson procedure, Newton method, and the global gradient algorithm (GGA). Because they use their own procedures to solve the WSS conservation of energy and mass equations, they can be seen as new software tools which do not use an external hydraulic solver. These include the methods by Germanopoulos (1985), Giustolisi et al. (2008), Elhay et al. (2016) and Ciaponi and Creaco (2018). Some of these methods cannot be used for extended period simulations (EPSs). Also, the lack of information on how to apply them is a common limitation (Mahmoud et al., 2017).

EPANET extension methods

These methods add the PDA aspects to the EPANET original hydraulic solver by modifying its mathematical models and the source code (Mahmoud et al., 2017; Paez et al., 2018). The methods include the EPANET-PDX (Siew and Tanyimboh 2012), the EPANET-MNO (Jun and Guoping, 2013), the WaterNetGen (Muranho et al. 2014), the EPANETPDD (Morley and Tricarico 2014) and the EPANET-IMNO (He et al. 2016). These methods can do both steady state and EPSs, and some include leakage modelling. However, they have limited information on how best to use them (Mahmoud et al., 2017). Bentley Systems Inc (2014) has developed WaterGEMS which is a commercial software that can perform both steady state and EPSs and it has provision for modelling pressure deficient systems. However, there are no records of its performance on IWSS.

Methods that add artificial elements

These PDA methods add artificial elements to demand nodes in EPANET. The artificial elements are the ones that implement the pressure dependent flows. There are two approaches in this category: the reservoir and emitter methods where the last in the series of the added artificial elements is the reservoir and emitter respectively (Figure 2.5). Lack of provisions for leakage modelling is the common missing aspect in these methods.



(a) (b)

Figure 2.5: Demand node with a series of artificial elements for (a) Reservoir method and (b) Emitter method

There are many reservoir methods in the literature. These include the pressure-deficient network algorithm (PDNA) (Ang and Jowitt, 2006) in which artificial reservoirs are progressively introduced into the network to initiate nodal flows, but are replaced by nodes with full demand loads once the nodal outflow satisfies the full demand. To determine the extent of water supply shortfalls, Suribabu and Neelakantan (2011) developed the steady state simulation complementary reservoir solution (CRS) in which complementing reservoirs are progressively added to pressure deficient nodes. The amount of water supplied by the complementing reservoirs is the shortfall in meeting the full nodal demand. The iterative nature of the PDNA and the CRS makes them computationally not suitable for EPSs. Babu and Mohan (2012) presented the modified pressure deficient network algorithm (MPDNA) in which water supply suddenly changes from zero to actual consumption water demand when the actual node pressure equals the minimum pressure head. This is not realistic hydraulically. Gorev and Kodzhespirova (2013) developed a non-iterative EPS method which proposes the use of check valve minor loss coefficients, but the procedure for determining the diameter of the check valves on which the coefficients depend is unclear. Pacchin et al. (2017) also proposed a non-iterative approach, but it was used for steady state simulations only. Paez et al. (2018) developed a method in which an artificial flow control valve (AFCV) was added to each demand node to ensure that the delivered flow does not exceed the required consumption water demand and an artificial throttle control valve (ATCV) was added to implement the pressure-flow relationship when the actual nodal pressure is less than the required pressure. However, the performance of the method depends on the correct setting of the diameter of the ATCV and the procedure for specifying this diameter is unclear.

Examples of emitter methods are the non-iterative method developed by Abdy Sayyed et al. (2015) and the one by Mahmoud et al. (2017) which modifies the former into the Single-Iteration Pressure Driven Analysis (SIPDA). The major limitation for these methods is the dynamic setting of AFCVs, emitter coefficients and exponents for EPS which makes the models computationally slow.

Studies that directly refer to modelling of IWSS

Limited studies directly refer to modelling of IWSS. These include Sashikumar et al. (2003), Dahasahasra (2007), Fontanazza et al. (2007), Cabrera-Bejar and Tzatchkov (2009), De Marchis et al. (2010), Ameyaw (2013), Lieb et al. (2016), Ilaya-ayza et al. (2016) and Ilaya-Ayza et al. (2018). However, the methods used in these studies are either not for IWSS because they do not provide means of handling multiple supply schedules or inadequate information on how the methods can be applied to these systems has been publicised. Battermann and Macke (2001) developed a PDA method for modelling IWSS that considers consumption demand and leakage explicitly. The method does not involve nodal demands to induce flow in the system, but pressure head forces water flows into tanks with defined volume capacity that represent aggregated consumer tanks and those that have very large capacity for leakage flows. The major problems with the method, which affect its accuracy greatly are the difficulty in determining the correct sizes of the domestic water consumption tanks and the distribution of the leakage tanks in the model network (Battermann and Macke, 2001). Moreover, the method presents numerical convergence problems when applied to large networks (Taylor et al., 2019).

2.4.3 *Design based on the intermittent water supply systems concept*

Intermittent WSS will continue to exist in many developing countries. This justifies the need to find suitable ways of designing or redesigning them (Batish, 2003). Fundamentally, designing of IWSS aims at achieving equitable water supply (Vairavamoorthy and Elango, 2002; Totsuka et al., 2004). This design requirement has been elusive due to limited studies aimed at developing design guidelines (Ameyaw et al., 2013). IWSS are generally designed using the continuous supply peak factors which are in the range of 2 to 3 (CPHEEO, 1999; Vairavamoorthy and Elango, 2002; Andey and Kelkar, 2009). Since demand patterns do not exist in these systems (Batish, 2003; Vairavamoorthy et al., 2001; Vairavamoorthy et al., 2008), peak factors are simply multiplication factors calculated from the division of 24 hours by the equivalent duration of supply in hours per day (Batish, 2003; Abu-Madi and Trifunovic, 2013). These factors can vary from as low as less than 2 to as high as greater than 12 (Sashikumar et al., 2003; Dahasahasra, 2007; Bose et al., 2012; Abu-Madi and Trifunovic, 2013). The high limits of peak factors confirm the fact that systems designed using continuous supply peak factors are undersized in most cases. The situation calls for trade-offs because larger pipes are costly (Bose et al., 2012) and have water quality problems.

Chandapillai (1991) developed a technique for both designing and improving intermittent systems. However, the technique implies that each time the network is extended, the source head should be adjusted to maintain the same node pressure conditions. This may not be practical. Vairavamoorthy and Elango (2002) developed the first known guidelines which were expected to result in optimal designs of intermittent systems (Vairavamoorthy et al., 2001; Vairavamoorthy et al., 2008). However, the guidelines are not in the public domain and their application has not been publicised (Batish, 2003). Batish (2003) suggested a designing procedure using artificial reservoirs in place of consumer roof tanks. To achieve equitable water distribution, the method involves changing of pipe diameters and installation of suitable throttle valves. The method or procedure for changing the pipe diameters and selecting suitable throttle valves is not provided.

The small number of trials to develop methodologies for designing IWSS shows that little effort is being made in this direction. Maybe because intermittent systems are discouraged and conversion to continuous supply status is encouraged (Vairavamoorthy and Elango, 2002; Myers, 2003; McIntosh, 2003; Dahasahasra, 2007; Biswas and Tortajada, 2010; Klingel and Nestmann, 2014).

2.4.4 *Conversion from intermittent to continuous water supply*

Conversion to CWS is necessitated by the understanding that provided intermittent supply exists, problems will be there. Like a natural tree cannot be killed by cutting its branches, but by uprooting, removing the intermittent supply tree, through conversion to continuous supply, also calls for removing the root causes.

Studies that considered the root causes of water supply intermittency

A few studies (Table 2.2) considered some of the root causes explicitly or implicitly and recommended approaches on how the conversion should be done. The studies had notable differences in terms of identifying factors that are critical to the success of the conversion process. For instance, McIntosh (2003), and Biswas and Tortajada (2010) identified governance and tariffs as critical, Dahasahasra (2007) stressed on hydraulic modelling and Klingel and Nestmann (2014) emphasised on database management. Although all the studies recommended phased conversion processes, the selection of parts of the WSS to be converted was different in some cases and the criteria for selecting these parts were unclear. The studies did not consider some factors such as poor electricity supply challenges and hydrological regime changes, and they did not suggest possible ways of making converted WSS resilient to uncertain future changes in demographic, economic and climatic conditions. The similarities, differences and omissions in these studies highlight the fact that many water supply intermittency root causes are common, their relative importance, however, is case-specific and there is a need for more interdisciplinary approaches to contextualising them and developing integrated conversion solutions.

The possibility of converting from IWS to CWS is a subject of debate sometimes based on long-standing generalisations. For instance, the general argument against conversion has been a lack of financial resources in developing countries, and the citizenry believes in this position. However, where water resources are in abundance, such arguments are not supported by the literature which shows that financial resources to achieving such conversions are not mainly met by the concerned governments or utilities, but by donors and cooperating partners (Blair et al., 2005; Biswas and Tortajada, 2010; International Monetary Fund, 2015). To this effect, poor governance, though typically not discussed in the water resources management literature from a technical perspective, is one of the water supply intermittency root causes that influences all others (McIntosh, 2003; Blair et al., 2005; Biswas and Tortajada, 2010; Simukonda et al., 2018b). Thus, in many developing countries, whether conversion is possible or not greatly depends on governance which includes political, administrative and economic governance. This means, the development of solutions (including technical solutions) to improve any water supply situation should be cognisant of the dynamics of relevant governance factors that are likely to affect the WSS at present and in the future. The GSG scenarios, (discussed in Chapter 3) provide comprehensive narratives that enable the understanding of the existing and future governance directions.

Table 2.2: Studies that show processes of converting from intermittent to continuous supply mode (Simukonda et al., 2018b)

Conversion process	Targeted root causal factor	Elements of root causal factor targeted	Action taken	Reference
Phased conversion	• Poor governance • Poor system management and operation • Unplanned extension of systems • Limited skilled manpower • Customer awareness	• All elements of governance shown in Table 2.1 • All elements of system management and operation shown in Table 2.1 • Poor pressure conditions in the system • Limited technical and managerial skills • Water use behaviour and bills payment	Transformed the Phnom Penh (Cambodia) water distribution system to continuous supply	Chan (2009) Biswas and Tortajada (2010)
Phased conversion	• Poor governance • Poor system management and operation	• All elements of governance shown in Table 2.1 • All elements of system management and operation shown in Table 2.1	Recommendation	McIntosh (2003)
Phased-random zone conversion	• Poor system management and operation • Limited skilled manpower • Customer awareness	• Poor database management • Poor system metering • Poor maintenance • Low tariffs • Non- use of modern analytical tools • Water use behaviour and bills payment	Feasibility demonstration based on Delhi, Indore and Guwahati. But proposed pilot implementation was on Delhi and Karnataka (India)	Myers (2003)
First analysed as “whole to part” but followed phased conversion.	• Population and economic dynamics • Poor system management and operation • Unplanned extension of systems	• Increasing water demand • Poor database management • Poor system metering • Poor maintenance • Poor pressure conditions in the system	Process demonstrated by converting 10 out of 34 wards in Badlapur (India)	Dahasahasra (2007)
Phased – successive zone conversion	• Poor governance • Unplanned extension of systems • Poor system management and operation • Limited skilled manpower • Customer awareness	• Poor policies/guidelines • Poor pressure conditions in the system • Poor database management • Poor maintenance • Poor system metering • Low tariffs • Non- use of modern analytical tools • Water use behaviour and bills payment	Process demonstrated on one zone in Béni Abbès (Algeria)	Klingel et al. (2012) Klingel and Nestmann (2014)

Considering the future in current water supply improvement interventions

Naturally, WSS infrastructure is designed to operate for decades and as such any plans to improve the system should consider the future. This is even more important with plans to convert from IWS to CWS because other than the expected long-life span, the conversion process itself may go on at least for a decade. This is so because of the limited resources which are the key reason for the phased conversion as the most recommended approach in the literature (Table 2.2). In this approach, one or more parts (DMA or zone) of a system are initially converted to CWS status. Over time the number increases until the whole system is converted. This process provides chances to mobilise financial resources, to learn from mistakes and build relevant skills, and it reduces the cost of mistakes associated with investing huge amounts of resources at once (Myers, 2003; Ilaya-Ayza et al., 2018). Since the conversion process is anticipated to complete long into the future, and the system is expected to be reliable long after conversion, numerous uncertainties are encountered in the planning process because of the complex evolution of the IWS root causal factors. A good option that incorporates the uncertainties in the interrogation of the plausible futures is the development and analysis of scenarios (Butler, 2004; Kang and Lansey, 2011).

2.5 Conclusion

This chapter sets the scene by discussing the prevalence, causes, problems associated with IWSS and the existing options for managing these problems. It also discusses existing PDA methods which form the basis for modelling IWSS.

IWS is dominantly practiced in developing countries, with few developed countries having resorted to it due to temporal causes. For full-time IWS status, the causes are so many and so interconnected that it is difficult to isolate root causes. Some of the causes are internal factors because they are worsened by the water supply intermittency itself while they at the same time have effects that worsen the intermittency thereby forming the self-reinforcing mechanism called the vicious downward spiral of intermittency. These include high NRW and poor compliance with the payment of water bills.

Other factors are external or root causes because they contribute to the development of the IWS mode, but the water supply intermittency does not influence their development and sustenance. These include poor governance, demographic and economic dynamics, hydrological regime changes, poor system management and operation, limited skilled manpower, unplanned extension of systems, lack of customer awareness and poor electricity supply.

To help in the visualisation of IWS root causes, problems of IWSS and management options, the chapter represents the intermittent supply tree concept. Problems associated with water supply intermittency are branches of the tree and attempts to mitigate the effects of the problems are likened to the cutting of tree branches while leaving the trunk and roots intact. Such attempts do not solve problems related to water supply intermittency, but temporarily reduce their effects while in the long run worsen the problems. The target of any sustainable intervention to managing problems of water supply intermittency, therefore, should be the root causes (roots of the intermittent supply tree). Since conversion to the CWS status is the ultimate solution in cases where water supply is in abundance, the chapter recommends interdisciplinary approaches to dealing with the root causes because they originate from political, social, economic, natural and technical factors.

The chapter has presented the information that highlights the fact that where water resources are in abundance, generalised arguments against conversion to CWS due to lack of financial resources in developing countries are not correct. This is because the literature shows that the financial resources to achieve such conversions are not mainly met by the concerned governments, but by donors and cooperating partners. The case of Phnom Penh in Cambodia attests to this assertion. Moreover, the greater part of the funding, even for intermittent supply systems, is still from donors and cooperating partners. In this regard, governance is found to be one causal factor of the development of the IWS status that influences all other causes.

Finally, the critical review presented in the chapter shows that even if financial support was to be available, there are many knowledge gaps that prevent the sustainable conversion from IWS to CWS. The following are among the major gaps:

i) Lack of solution approaches that focus on causes of IWS

In the literature on IWS, the classification of causes of IWS has been either on the basis of water scarcity as technical scarcity, economic scarcity and absolute scarcity or from the technical perspective as deficient system planning, missing system concept, deficient infrastructure management and limited system knowledge (Klingel, 2012). Causes of IWS are also categorised as external factors which include increased population, political misjudgments of progress indicators, development, hydrological regime changes and electricity blackouts (Galaitsi et al., 2016). The literature lacks information on the identification of the root causes and how they cause IWS. This has resulted in the development of solutions approaches that are not sustained because they focus on effects of IWS rather than the root causes.

ii) Limited or no conversion methods involve scenarios

In the literature, conversion from IWS to CWS is acknowledged as the ultimate solution to the problems associated with IWS and phased conversion approaches are recommended. However,, there are no conversion methods that involve scenario development and analysis in the planning process despite the knowledge that causes of IWS are complex and intertwined. Because the causes of IWS are complex and interconnected, their evolution into the future is also complex and presents numerous uncertainties in the planning process. Scenarios provide a means of incorporating uncertaininties in planning processes with the future outlook. Thus, scenario development and analysis are critical to the planning approach for systematic and sustainable conversion from IWS to CWS because they enable informed adaptation of WDSs to future and uncertain changes of governance, climatic, demographic and economic factors. Scenarios are discussed in Chapter 3.

iii) No literature on IWSS that has characterised the prevailing trend of government policies using the GSG scenarios narratives

For sustainability, a complete understanding of the operating environment of a WSS is required. This understanding does not only involve the analysis of environmental (natural) and technical aspects but also regional or national social, political and policy/legal frameworks that affect the WSS. For planning water supply improvement internvetion for IWSS which include their conversion to CWS, the “Global Scenario Group” (GSG) scenarios provide objective and comprehensive narratives to describe the sustainability implications of the planned water supply improvement interventions under the prevailing trend of regional or national government policy frameworks. There is no literature on IWSS that has characterised the prevailing trend of government policies using the GSG scenarios naratives and thereafter link the possible success or failure of water supply improvement interventions to the prevailing trends of government policies. The GSG scenarios narratives and their linkage to various aspects of water supply systems are discussed in Chapter 3.

iv) Modelling approaches for IWSS which incorporate leakage are uncommon.

While there are many PDA methods for CWSS with pressure deficiency in the literature, modelling approaches for IWSS which incorporate leakage are uncommon. These modelling approaches would aid in the understanding of the hydraulic behaviour of these systems during their planning and operation.

3 DEVELOPMENT OF SCENARIOS FOR SUSTAINABLE WATER SUPPLY

In the literature, conversion from IWS to CWS status is recommended as the most effective approach to solving all the problems associated with IWSS. However, issues of sustainability in all the attempted conversions have not been exhausted. The methodology developed in this study covers three major analysis stages for generating information during the planning phase that would improve the sustainability of any intermittent water supply improvement intervention including conversion to continuous water supply mode. The first and second analysis stages involve the use of scenarios. The development of scenarios and their analysis approaches is the subject of the current chapter (Figure 3.1). The third stage involves the use of hydraulic models to investigate the hydraulic capacity of the WSS under different future scenarios and expand the hydraulic capacity where needed. Development of the modelling approach for IWSS is discussed in Chapter 4 (Figure 3.1) and the optimisation of the rehabilitation problem to increase the hydraulic capacity of a WSS is discussed in Chapter 7 (Figure 3.1).

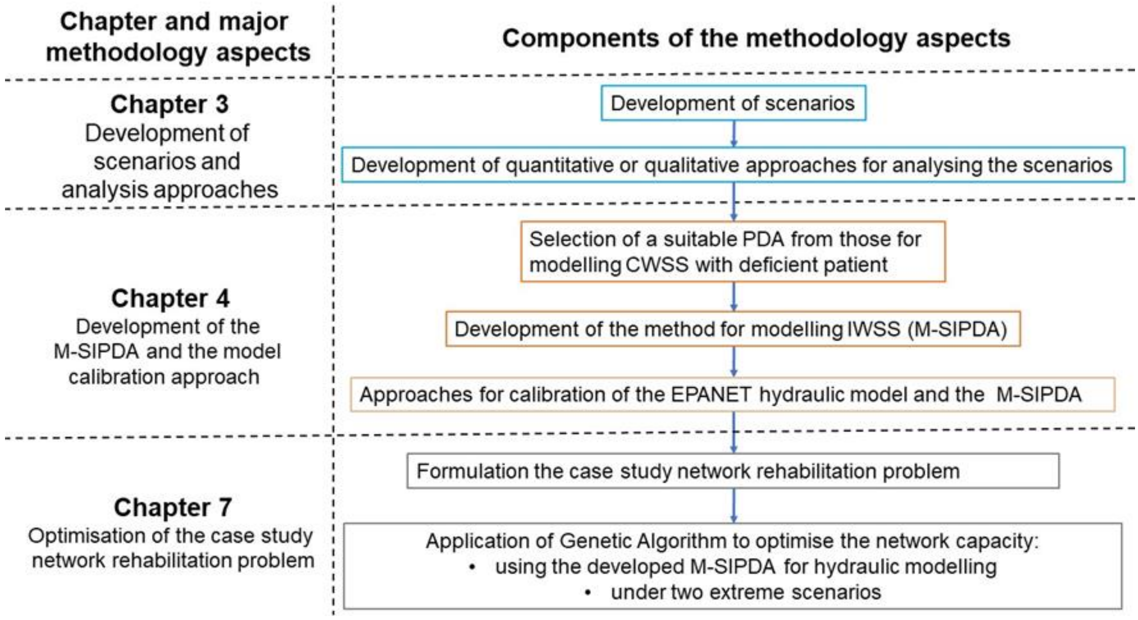


Figure 3.1: Major study methodology aspects

Owing to the numerous uncertainties with IWSS and the future planning horizons involved, scenario analysis is suited for the assessment of the sustainability of the proposed conversion approaches. This chapter provides a generic approach to the understanding of the national (state) policy direction that influences the sustainability of WSS, the development of scenarios for sustainable conversion from IWS to CWS status and to the development of qualitative and quantitative approaches for analysing the developed scenarios.

Section 3.1 is the introduction to scenarios which provides the justification for their use in the planning process of the conversion to CWS status. In section 3.2, information is provided on the scenario development processes and major aspects of the GSG scenarios. Section 3.3 discusses the two axes method used to develop the sustainability scenarios in which the drivers of total water demand (consumption demand and leakage) form the axes. Section 3.4 presents aspects that are critical to the management of consumption water demand and NRW. The chapter conclusion is presented in section 3.5.

3.1 **Introduction to scenarios**

Planning of any water supply improvement undertaking that must be implemented well into the future should consider several sources of uncertainties. For the conversion from IWS to CWS, the major sources of uncertainties are population growth, people's lifestyles which affect per capita water demand, economic and industrial development patterns, technology development and utilisation, climate change, urbanisation trends and governance systems which influence the effectiveness of policies and regulatory mechanisms. One of the major approaches to incorporating these uncertainties in the planning process is the development and analysis of scenarios (Lansey and Kang, 2012).

Scenarios provide a description of the plausible future system states including paths of development that may lead to them (Kosow and Gaßner, 2008). Scenarios are not meant to represent or predict the future, but to present the possible state of the system under investigation. They provide insight into what the future *might* rather than what the future *will be* (Butler, 2004). Scenario development and analysis are based on three recognitions – that there is some knowledge of the future, that this knowledge is limited, and that the future can be influenced by human choices and actions at present and those which are yet to be made (Gallopín et al., 1997; Butler, 2004; Kosow and Gaßner, 2008). From the limited knowledge of the future, scenarios are developed based on assumptions of what direction certain trends might take, which developments might remain constant and which ones might change in the future and how the change might affect the state of the system. Because scenarios are based on these assumptions, there are many possible future system states which increase with the increasing planning horizon resulting in a spreading out structure called the scenario funnel (Kosow and Gaßner, 2008) as shown in Figure 3.2. However, for the human mind to keep track of the implications of each scenario, the large number of scenarios is compressed into a small number comprising only of scenarios that cover the key drivers and their possible developments. Godet (2000) limits the tractable range of scenarios to between four and six. Schwartz (1991) and Hunt et al. (2012) recommend a minimum of two and a maximum of four scenarios. For easy explanation in this section, three scenarios S1, S2 and S3 represent the possible futures which are arrived at by possible paths of development a1, b1, c1 for scenario S1, a2, b2, c2 for scenario S2 and a3, b3, c3 for scenario S3 respectively (Figure 3.2).

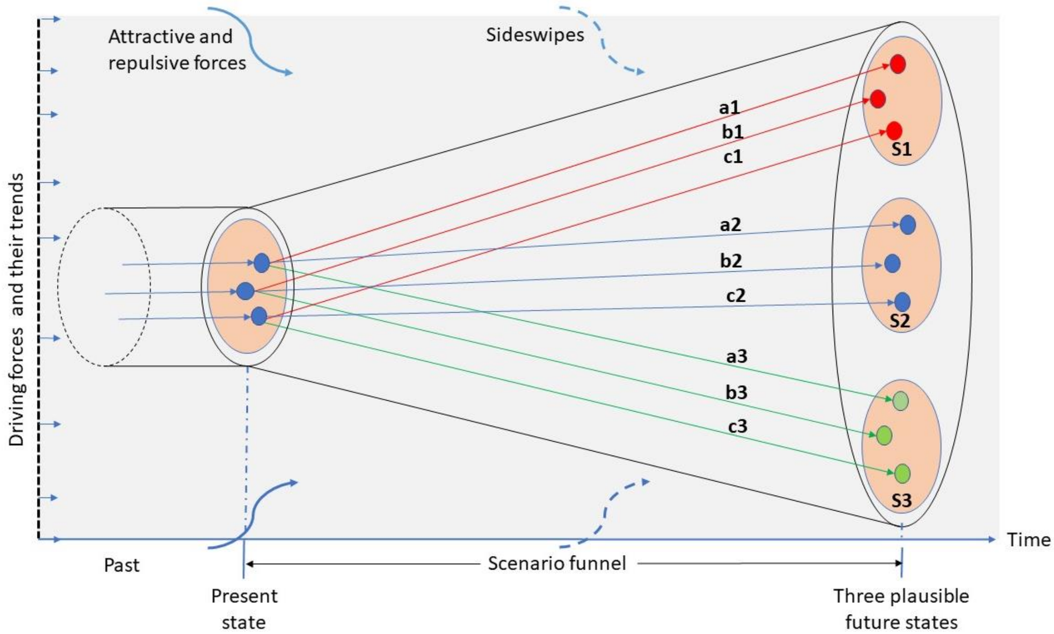


Figure 3.2: The interaction of driving forces, attractors and sideswipes in the development of the scenario funnel (Adapted from: Raskin et al., 1996; Gallopín et al., 1997; Kosow and Gaßner, 2008)

When planning and implementing projects with a long-term planning horizon, there are surprises (sideswipes) which may result in sudden changes to the trends because they affect the drivers. These include a major natural disaster, a global epidemic, a breakdown of the climate to mention but a few (Raskin et al., 1996). As shown in Figure *3.2*, there are also attractive or repulsive forces (attractors), such as consistency with sustainability principles, which can substantially redirect beliefs, behaviours, policies and institutions towards some futures and away from others (Raskin et al., 1996). The making of demonstrable commitments to poverty reduction by the heavily indebted poor countries (HIPC) to qualify for debt relief (United Nations General Assembly, 2000) was also an attractor. This shows the possibilities of donors to develop attractors that can direct efforts towards attaining the CWS mode in developing countries where water is in abundance. If the attractors are weak, such that they have no effects on the current drivers (such as political driver), the present scenario continues into the future (Raskin et al., 1996; Gallopín et al., 1997) as indicated by scenario S2 in Figure *3.2*.

The general aim of scenarios and their analysis is that they provide the required flexibility for project implementors to take actions in response to unexpected developments at reasonable regret cost (Kang and Lansey, 2011). Specifically, scenarios can be seen to have four functions. These are the explorative or knowledge, communication, goal setting, and decision-making and strategy formation function (Kosow and Gaßner, 2008).

The explorative or knowledge function of scenarios stems from the fact that they provide a systematic approach that deepens the existing knowledge of present development conditions and factors that influence the performance of systems such as WSS. Through the explorative or knowledge function, scenarios provide a means of focusing on possible interactions of the key drivers at present and in the future, possible development paths of these drivers, their salient characteristics and the range of possible eventualities in the process (Kosow and Gaßner, 2008).

In line with the communication function, scenarios are used as a communication tool which promotes debate that leads to a common understanding of a problem.

They also promote the exchange of ideas and the integration of different viewpoints on a topic. Furthermore, scenarios can be used to generate information that leads to a better understanding of problem situations and better planed actions (Kosow and Gaßner, 2008).

Goal setting involves the understanding of the current position and having a vision of the desired future system state. In this regard, scenarios provide a means of answering questions such as: "*Where do we want to go from here?"* and *"What do we hope to achieve?"* In so doing, scenarios become helpful in developing images of the desired future (Kosow and Gaßner, 2008).

Scenarios are used for decisions and strategic planning because they make clear the available options and enable the development of indicators as benchmarks which when met call for action. Moreover, scenarios can be used to test the reliability, robustness, and effectiveness of decisions, strategies and policies by comparing the consequences of developments based on different scenarios (Kosow and Gaßner, 2008).

As a result of their functions, scenarios have been used extensively in developed countries to probe into the future in many fields including the assessment of alternative solutions for WSS development or assessment of the sustainability and resilience of various solutions relating to these systems at present and in the future. Kang and Lansey (2011) used scenario-based optimization approaches for the planning of integrated water/wastewater systems for sustainable water supply in which they noted that the effective application of these approaches calls for care during the formulation of the objective function and the generation of scenarios that sufficiently cover the range of uncertainties. Lansey and Kang (2012) developed a multi-stage multi-scenario optimization (MSMSO) approach to find a compromise solution that minimizes the regret costs across several scenarios because there are common elements that can be identified between them. This approach requires that scenario planning processes are regularly revised during the planning horizon to adjust the scenarios and to revise the decisions if planning conditions and assumptions change, or certain benchmarks are met.

Scenarios have been applied to regeneration sites like the Luneside East site (UK) for which sustainable development depended more on the policy directives towards reducing potable water consumption using technology alone. The implications of various technological water management strategies (water-efficient appliances, greywater recycling and rainwater harvesting) in line with the UK government policy can be analysed based on the (GSG) scenarios' narratives (Raskin et al., 2002) to better understand the problems associated with this government policy as done in Hunt et al. (2012). The GSG scenarios are the Market Forces (MF), Policy Reform (PR), New Sustainability Paradigm (NSP) and the Fortress World (FW) (discussed in detail in subsection 3.2.2). Using these scenarios, Hunt et al. (2012) show that technology change is more effective if coupled with changes in human behaviour which also potentially depends on policy directives. Boyko et al. (2012) acknowledge the usefulness of scenarios as aids to thinking about and visualising the future, but notes that scenarios have not been used to explore, in the urban context, the actual sustainability of the currently highly-acclaimed solutions and how resilient these solutions are in the face of future uncertainties. Farmani et al. (2012) applied the GSG scenarios to describe the Luneside East site water management plausible futures and to assess how solutions that are sustainable at present might cope with any future development. In this case, the use of a diverse range of water management options is seen to improve and increase the overall system resilience in the future.

The above studies are not directly related to IWSS, but they highlight three aspects that are important to the discussion of scenarios for IWSS. The first is that the policies and planning strategies in the involved countries such as UK prompted the studies. The second aspect is that the studies discuss the urban futures with respect to the sustainability of the current solutions and their resilience in view of future uncertainties. The third aspect is that the studies demonstrate the acknowledged usefulness of scenarios as tools for investigating the sustainability of the currently proposed solutions (designs and plans) for urban systems and their resilience in the future. These aspects are not considered as important in many developing countries. In these, there are no policies that are informed by research and research rarely assesses the future implications of policies. This could be due to the lack of policies that stipulate how the city should be developed which should be expected as there are no comprehensive planning strategies for urban development in the future. Where these exist, the plans are developed by foreign entities (mainly attached to aid) without any real input from the local experts and as a result, their enforcement is grossly lacking. Moreover, there are no serious codes of practice in line with sustainable development such as the code for sustainable housing (CSH) in the UK (Communities and Local Government, 2010). In most cases, the reference to sustainability and resilience lacks the basis and meaning. It is difficult to discuss resilience from a water supply perspective because going by its definition (Butler et al., 2014),

> "*the degree to which the system minimises level of service failure magnitude and duration over its design life when subject to exceptional conditions.*"

the level of service failure duration in IWSS is continuous and the magnitude increases with time. These systems, therefore, are not resilient, but the people or societies subjected to them are. Moreover, these systems are not sustainable because they do not meet even the needs of the current generations.

The use of scenarios to have insight into what might happen in the future if certain developments took place is very uncommon in developing countries. Concerning IWSS, it is known that there are several sources of uncertainties related to them, but to the knowledge of the author of this study, no proposed water supply improvement solution in the literature of these systems has been found that uses scenarios in the planning process to gain insight into the sustainability and future resilience of the proposed interventions. Studies have proposed conversion from intermittent to CWS mode and some pilot conversions have been done (discussed in Chapter 2), but none of them suggests or uses scenarios to consider future uncertainties and to assess the sustainability and resilience of the proposed conversion approaches.

In the Water Supply Investment Master Plan for Lusaka City (Republic of Zambia, 2011b) three scenarios called "operation scenarios" are developed which are named according to the rate at which network expansion is to be done as; Aggressive option (Scenario A), Moderate option (Scenario B) and Passive option (Scenario C). However, in terms of drivers that affect the sustainability of the interventions such as water demand and NRW, these scenarios are not different as they all depend on the same scenario in which there is no change in demand, but the focus is on NRW reduction (The NRW management scenarios discussed in section 3.3).

Unlike the earlier studies that have suggested water supply improvements in IWSS including conversion to CWS mode, but have not included the uncertain developments in the major drivers, this chapter develops scenarios that are used to assess the sustainability of converting from intermittent to CWS using the water supply-demand balance.

3.2 **Theoretical background**

To apply scenarios to the planning process of improvement measures, the broader understanding of the scenario development process and the expanded interpretation of the GSG scenarios with respect to urban forms and water demand is critical.

3.2.1 ***Scenario development process***

The methodology for developing and applying scenarios is summarised in Figure 3.3. The initial analysis of the past and present WSS operating environment provides general information on aspects that may affect the sustainability of the WSS operations. The analysis uses the narratives for the GSG scenarios with their linkage to urban forms and water demand (subsections 3.2.2 to 3.2.4). The drivers of change constitute both indirect and direct drivers. The drivers of change, their predetermined elements and critical uncertainties are discussed in subsection 3.2.5. For the sustainability of WSS, the water supply-demand balance provides information as to whether water resources are adequate or not. Universally, water demand management is critical for the sustainability of WSS. The direct drivers for water demand, their predetermined elements and critical uncertainties are discussed in subsections 3.3.1 and 3.3.2. The axes (two axes) scenario development method is discussed in subsection 3.3.3. This is followed by the narratives for the developed scenarios and the qualitative analysis framework (subsection 3.3.4). The quantitative approach for determining the quantities for the direct drivers for each scenario is discussed in subsection 3.3.5.

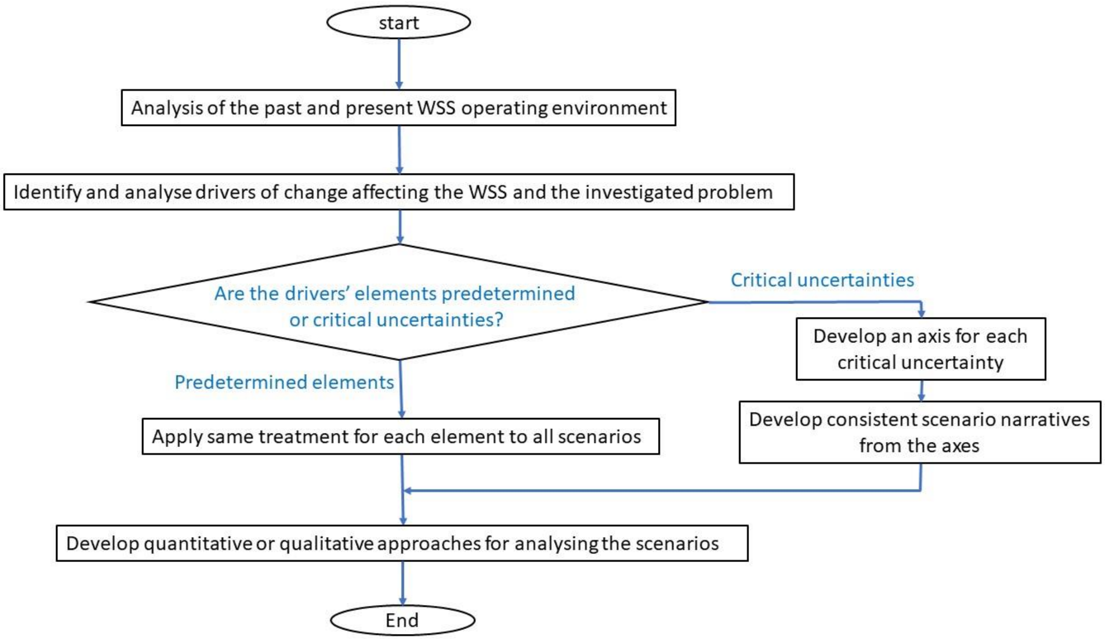


Figure 3.3: Major steps for WSS improvement interventions scenarios development

3.2.2 *The GSG scenarios*

The development of scenarios starts with the knowledge of the past and the current state in terms of the system operating environment and its state. For sustainability, the GSG scenarios are important. These are archetype scenarios which apply to the evaluation of project strategies for sustainable development at global, regional, national levels and even lower scales. The scenarios are based on a two-tier hierarchy where the first tier comprises the three different social visions called classes and the second tier shows two possible scenarios from each class (Electris et al., 2009). The three classes and scenarios proposed by Raskin et al. (2002) are the Conventional Worlds with MF and PR as the scenarios, Barbarization with FW and *Break down* (BD) as the scenarios, and the Great Transition which has NSP and *Eco-communalism* (EC) as scenarios. The BD is the extreme form which the FW would try to prevent, and the prevailing globalisation trends make the EC unlikely. Some of the characteristics of the remaining four GSG scenarios are shown in Table 3.1 which is a summary of the comprehensive narratives for each scenario. The narratives reveal four critical issues on the drivers and the scenarios themselves.

Table 3.1: Selected global scenarios characteristics (Adapted from: Raskin et al., 2002, Rogers et al., 2012)

Global scenarios class	Scenario	Scenario characteristics
Conventional Worlds	MF	• Market-enabling government and a consumerist public • Increasing transnational corporation power • Sustainability issues addressed more through rhetoric than action • High income disparities • Unplanned and fragmented urban development • Built environment expanding onto other land use classes • Dwelling densities drop slightly due to urban sprawl and little land recycling • Social and environmental concerns are secondary
	PR	• Government policies directed to align markets for poverty reduction and sustainability • Reduced income disparity and improved social equity and welfare • No net increase in the land devoted to the built environment • Dwelling densities increase as urbanisation continues and more compact settlements develop, supported by policy. • More economic centres are created, and sprawl is contained by strong policy and high land recycling.
Barbarization	FW	• Unresolved deepening social and environmental tensions • Social and environmental problems overwhelm market and policy responses • Authoritarian response by powerful actors with vested interests to prevent the breakdown scenario • Built environment sprawls because of increasing population and urbanisation • The poor masses live in poor while the privileged elites live in favourable environments
Great transitions	NSP	• Human happiness and fulfillment are central themes for human development • No net increase in the land devoted to the built environment • Civil society and engaged citizens are critical sources of change for the new values • The shift to values emphasising quality of life, human solidarity and environmental sustainability supports much greater civic participation

The first issue is that governance (government as the major entity) is the critical driver for all the scenarios as noted in Burdett et al. (2006). Governance (poor governance) is also one root causal factor for IWSS that affects all others (Simukonda et al., 2018b). This shows how serious governance issues should be considered when planning the conversion from IWS to CWS status including any other water supply improvement in IWSS.

The second issue is that the NSP scenario though desirable, requires a lot of effort by the civil society groups to promote awareness of the problems of IWSS and solution options, to ensure good policies are formulated and implemented, and to promote water consumer behaviour change. These are the aspects that make the NSP the most sustainable scenario because the effects of technology advancement are actualised by the consumers' improved behaviour (Figure 3.4) and progress is driven by the participation of the informed populace. However, currently, the kind of efforts by the civil society groups to bring about the NSP scenarios is missing because many of these groups focus on civil and political rights which are justiciable unlike the economic and social rights such as the right to water which are not justiciable (directive principles of state policy) as they are to be met progressively as resources permit (Republic of Zambia, 1996; The Republic of South Africa, 1996; The Republic of Kenya, 2010; Government of India, 2019). Consequently, all that governments have to say is that there is a lack of resources and this is a common problem in developing countries (Simukonda et al., 2018b). Unless surprises occur, the limited civil society involvement in water issues means the NSP scenario is less likely to take place in many developing countries possibly until at the end of 2030 when their development policies will be guided by a different set of goals and targets other than the SDGs where national policies and priorities are respected in the pursuit of sustainable development and system resilience. This is because, with respect to developing countries, the SDGs have serious inconsistencies in as much as on paper they proclaim a balance between economic, social and environmental dimensions of sustainable development, in reality, the economic dimension (free market economy) has an upper hand over the other two dimensions making many developing countries, in which corruption is also rife, powerless against multinational corporations. This dominance is reflected in the characteristic of the MF scenario in Table 3.1.

The third issue is that because of the targets set by the international community such as the SDGs in the face of liberalised economic activities, the PR scenario seems to be one of the most common scenarios at present at the first sight. In this scenario, some policies are developed that promote the liberalisation of the economy while others are developed to counteract the effects of this liberalisation in line with environmental sustainability and poverty reduction. To achieve this sustainability, policies should not only promote both technology development and human behaviour improvement in this case with respect to water consumption (Farmani et al., 2012), but also constrain the environmental and social exploitations by corporations. In line with these arguments, Raskin et al. (2002) presents two difficulties with the PR scenario. The first is the huge technical challenges of countering conventional development under this scenario in view of the scenarios' assumption that the underlying values, lifestyles and economic structures of the Conventional Worlds remain unchanged. The second difficulty is that the scenarios' plausibility lies on the existence of enough political will that gives rise to an unprecedented and unyielding governmental commitment to achieving sustainability goals. This level of political will is non-existent in many developing countries. Consequently, many well formulated policies are poorly implemented, and sustainability issues are addressed more through rhetoric than action (Rogers et al., 2012).

The fourth issue which arises from the above discussion is that at present the MF and the FW are the most common scenarios in many developing countries. These scenarios will most likely dominate up to 2030 and possibly beyond unless surprises occur which result in the increase in political will by governments and the role of civil society groups shifts from a focus on civil and political rights to awareness creation among the populace so that they can demand the fulfillment of their social and economic rights. In terms of user behaviour and technological efficiency (Figure 3.4), in the MF scenario the situation worsens which corresponds to high water demand while for the FW, the use of technology is poor and user behaviour is poor too among the affluent. Amongst the poor (have nots), the use of technology is nonexistence, but their water consumption behaviour is improved because of the restricted water availability (Hunt et al., 2012).

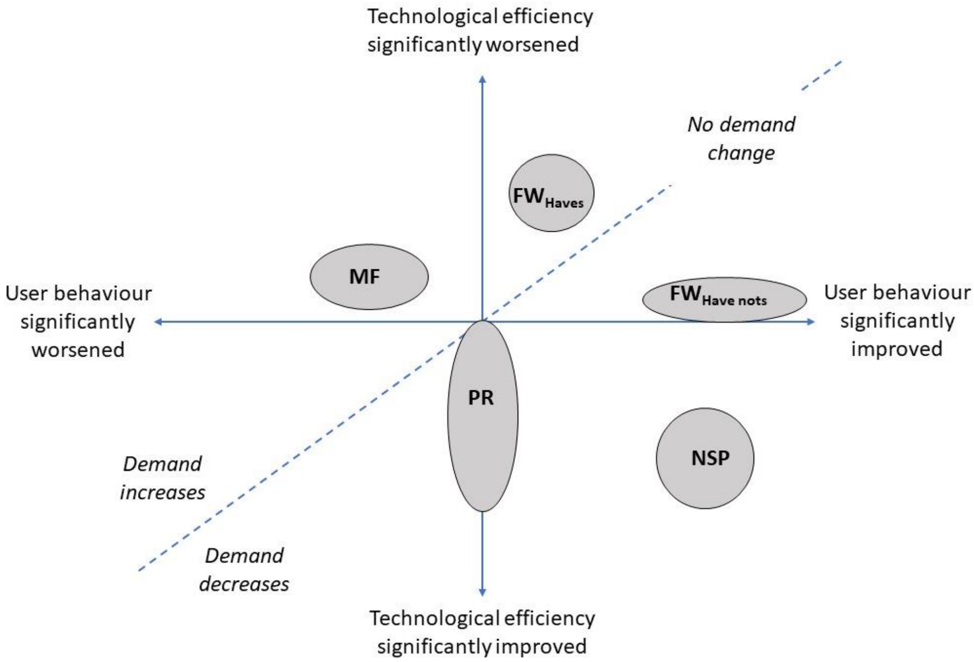


Figure 3.4: Four GSG scenarios mapped onto technological efficiency and user behaviour axes (Adapted from: Hunt et al*.* 2012))

3.2.3 *The link between the GSG scenarios and the city expansion*

In Table 3.1 it is shown that the four scenarios have different effects on land use such that to different extents, some scenarios have the effect of city sprawls while others do not. Farmani and Butler (2014) link the urban forms widely applied in urban planning with these scenarios and then investigate the relationship between urban forms and the performance of WSS. The urban forms are (Farmani and Butler, 2014; Bucur et al*.*, 2015) :

i. Compact: Uniform development within the existing urban where most commercial activities take place
ii. Planned development - monocentric: Growth is concentrated in a single region of the existing urban area.
iii. Planned development - polycentric: Growth is concentrated in several regions of the existing urban area.

iv. Edge expansion. Development of new settlements as urban extensions with new nodes of activities emerging as part of a bigger network of the diffused urban metropolis.

The urban forms linkages with the GSG scenarios are compact/uniform, monocentric, polycentric and edge developments with PR, FW, NSP and MF respectively. Population growth rate and the type of urban development have significant effects on the total cost of redesigning and operating existing WSS infrastructure (Farmani and Butler, 2014). Uniform expansion (PR) is the most cost-effective system and polycentric expansion (NSP) is the costliest development if it relies on centralised water sources to supply water to consumers. The cost effectiveness of edge expansion (MF) depends on the location of the expansion and the monocentric (FW) is not the most cost-effective development contrary to reports in the literature (Farmani and Butler, 2014).

3.2.4 *The GSG scenarios and their implications on the total water demand*

The analysis of the GSG scenarios can be linked to the total water demand which is one of the major factors of WSS management. This factor has consumption demand (domestic demand, commercial and industrial demand) and leakage (NRW) as its components. Butler (2004) mentions some of the drivers of domestic water demand as the cost of water, personal washing, garden watering, technology efficiency and regulation while for leakage; metering, regulation and resource availability are cited as drivers. The implications of the occurrence of any of the four GSG scenarios are here highlighted with respect to the total water demand.

MF scenario

The occurrence of the MF scenario would entail high leakage and domestic water demand due to limited regulation coupled with poor usage of efficient technologies and deteriorating consumer behaviour (Figure 3.4) because of people’s consumerist mindset (Table 3.1). Moreover, the unplanned and fragmented urban development (urban sprawl or edge development) implies that the cost of delivering water to all parts of the concerned city is likely to be high thereby making water unaffordable to the poor. This is a vital aspect in developing countries where WSS are intermittent and their extension is necessary.

PR scenario

The PR scenario aims at countering the negative effects of the MF scenario (Table 3.1) and achieving this relies on the existence of strong government policies and regulations aimed at controlling the urban sprawl to compact/uniform development. This is important in developing countries because being the most cost-effective development (Farmani and Butler, 2014), it leads to affordable means of connecting to new consumers which would increase water demand. With effective government action, in developing countries, consumption water demand is also likely to increase because of the possibility of raising incomes for the poor to reduce income disparities, improving or upgrading of peri-urban areas or slums (Government of India, 2019) and due to population increase plus urbanisation. The PR scenario also depends on the existence of efficient technology and its use to reduce water demand (Figure 3.4). This is so in developed countries, but the lack of technology in developing countries may result in insignificant demand and leakage reductions especially when coupled with week regulatory frameworks. It is important to note that technology or policy alone without change in people’s behaviour may not yield great success in reducing total water demand (Farmani et al., 2012; Anand, 2017). Consumer behaviour under the PR scenario is a challenge because the Conventional Worlds characteristics are unaltered.

The FW scenario

Under the FW scenario, it is not clear how water demand will be affected. It would be expected that demand will increase because of urban sprawl which results from population growth and urbanisation, but since the poor masses live in poor environments (Table 3.1) where water supply is restricted, the net increase in demand depends on the extent of the restriction. The net increase in demand also depends on the population of the rich because for these, water demand is high. The discriminatory nature of this scenario shows that improvement of water supply in the poor residential sites is difficult and any interventions aimed at improving water supply in IWSS under this scenario, would meet a lot of challenges because the rich who have the power have adequate storage facilities and treatment equipment such that they can mimic the CWS mode. This makes the rich not interested in improving the water supply situation especially for the poor, but instead, capitalise on their poverty and vulnerability.

NSP scenario

The NSP scenario is the scenario envisaged in the SDGs in which world leaders show the desire to pursue development that leaves no one behind (United Nations General Assembly, 2015). Under this scenario, the masses are well informed because of the active role of the civil society. Government policies put the welfare of the people and environmental sustainability as a priority. Water consumer behaviour is very good and the use of technology is high (Figure 3.4). Even though population growth and urbanisation take place, leakage and per capita water consumption are likely to be reduced by the use of technology, the availability of resources which the government provides to improve the quality of life for all and the willingness of informed people to follow policies and regulations. Although this scenario is very desirable, for developing countries and for improving water supply in IWSS, the scenario faces the challenge of balancing sustainable development and economic development under free-market economies in which powerful multinational corporations are dominant over governments and their regulatory policies for sustainable development as observed by Burdett et al. (2006). Moreover, in developing countries where urbanisation is high, the costly polycentric expansion which corresponds to this scenario (Farmani and Butler, 2014) may be a drain to the limited resources of these countries when extending WDS networks.

3.2.5 *Drivers of change for WSS*

The operating environment is interconnected with the drivers of change and their trends which develop and change the system's state (Schwartz, 1991; Gallopín et al., 1997; Raskin et al., 2002).

For developmental projects which include conversion from intermittent to CWS, major drivers are demographics, economics, social issues, culture, technology, environment, and governance (Raskin et al., 2002; Hunt et al., 2012). If culture and demographics are combined into social issues, the major drivers become

Social, Technological, Economic, Environmental and Political (STEEP) (Hunt et al., 2012). With respect to IWSS, these drivers match well with the root causes of the IWS mode discussed in Chapter 2. In Chapter 2, poor governance (matches with political, economic and social issues as drivers), demographic and economic dynamics (matches with a combination of social and economic drivers), hydrological regime changes (corresponds to environmental drivers), poor system management and operation (corresponds to a combination of technological and social drivers) and Limited skilled manpower (matches with social and technological drivers), unplanned extension of systems (has elements of political, technological and economic drivers), lack of customer awareness (combines social drivers) and poor electricity supply (combines environmental, technological, economic, and political drivers). The identification of drivers (root causes of IWSS) helps to build coherent scenarios if these drivers are considered with their predetermined elements and critical uncertainties.

Predetermined elements are aspects of a driver that are certainly going to take place no matter which scenario occurs (Schwartz, 1991). This means for these aspects; all scenarios are treated the same. Consequently, differences in scenarios lie in critical uncertainties. These are aspects whose evolution is not known with certainty (Schwartz, 1991). Since predetermined elements are the same for all scenarios, the backbone of scenarios constitutes the current state, drivers (root causes) and critical uncertainties (Gallopín et al., 1997). In the paragraphs that follow below, elements of the drivers that are not captured in the discussion of root causes of IWSS in Chapter 2 are discussed.

The discussion highlights aspects that are important in understanding the prevailing and future social, economic, technological, environmental and political aspects that may affect the success of any planned conversion of IWSS to CWSS. For this understanding, the GSG scenarios provide the most comprehensive and all-encompassing narratives.

Governance

Governance is a driver that includes political, economic and legal institutions and interactions of stakeholders within these institutional frameworks at international, regional and national levels. At the international level, the largest institution is the UN under which the Millennium Development Goals (MDGs) declarations were made. These are among the major world declarations which provided the goals to be met for the sustainable development agenda. The major thrust for the MDGs was to mitigate the negative effects of globalisation on developing countries. These were followed by the SDGs which continued and amplified the MDGs not only for furthering the sustainable development agenda, but also for promoting development that strengthens the resilience of communities and systems. These international declarations form part of the basis of the Policy Reforms (critical to the PR scenario in subsection 3.2.2). Many development plans and policies are aligned with these UN set of goals. However, in the world where free-market economy policies dominate (MF scenario), the implementation of many sustainable development policies lacks the will power of national governments (Raskin et al., 2002) especially in developing countries due to poor governance problems. As a result, in these countries, sustainability and resilience are mere words for political rhetoric with very little or no meaning at all (Rogers et al. 2012). In developing countries, governance issues are critical uncertainties and there is the need to comprehensively consider the position of the responsible government entities or simply the government and the wider range of governance issues when planning to convert from intermittent to CWS mode.

Demographics (social)

Population growth and urbanisation are the major drivers of consumption water demand in cities especially those for developing countries.

These two demographic factors are the forces behind the expansion of cities which is the genesis of urban forms discourse. They increase pressure on many infrastructures including those for WSS and they contribute to environmental degradation which leads to the decline of water resources (Simukonda et al., 2018b). Consequently, failing to consider demographic changes in the planning of water supply improvement in IWSS simply implies planning for the failure of the planned intervention.

Concerning demographics, the predetermined element for a given WSS in a developing country is that population growth and urbanisation will surely take place regardless of the scenario. However, the per capita water consumption depends on the type of future scenario, namely: the MF, PR, FW and NSP. This is because the per capita water consumption is linked to the development and use of technology, the water consumer behaviour and the proportion of the poor and the rich people. The evolution of all of these into the future cannot be predicted with certainty.

Economics

The three dimensions of sustainable development (also called the 3Ps) are economic (Prosperity or Profit) social (People) and environment (Planet) (Butler et al., 2014; United Nations General Assembly, 2015). Economic activities and growth, are necessary in all countries in the world, but they are more so in developing countries (Asafu-Adjaye, 2005; Blair et al., 2005). There is a strong differentiation of economic activities between rural and urban areas. Rural economies are mainly dependent on agriculture and their growth is very slow while urban areas' economies are industry and service-based and they grow relatively fast. This makes urban areas attractive because they provide better livelihood opportunities and as a result, there is a pull effect on rural-urban drift which leads to urbanisation and the sudden rise of pressure on urban water supply and other services infrastructures. It should be noted that with globalisation and liberalisation of the economy, the operations of multinational corporations in developing countries have increased, but being profit-oriented in pursuit of the *prosperity or profit* dimension of sustainable development and in line with the consequences of the free market policies (MF scenario), these corporations have not helped to improve the quality of life of many poor people in developing countries thereby violating the *social* dimension of sustainable development. The multinational corporations have continued to contribute to environmental degradation and the pollution of water resources and hence the reduction in water availability in countries with very poor regulatory measures and high levels of corruption. These are a violation of the *environment* dimension of sustainable development and directly conflicts SDG 6 which aims at ensuring the availability and sustainable management of water and sanitation for people.

Economic dynamics present a complex mix of critical uncertainties with respect to improvement of water supply in developing countries where the real government will power to reduce inequality within (and among) countries (SDG 10) and to genuinely champion the course of sustainable development is lacking (Raskin et al., 2002).

Social issues

The major social issues in many developing countries are high levels of poverty which is the basis for making the first block of the IBT structure to be the social tariff (NWASCO, 2014b) and the inequality in terms of service provisions between the poor and the rich (Raskin *et al.*, 2002b) which is contrary to the provision of SDG 10. In many developing countries water supply services are better in areas where the affluent live than where the poor live. How this inequality unfolds in the future depends on the developed and implemented national policies which affect the gap in terms of incomes and proportions between the poor and the rich thereby affecting the level of water demand. Differences in the treatment of the poor and the rich in terms of water availability and supply durations under the IWS mode are discussed in Simukonda et al. (2018b) (also Chapter 2) and reported in the literature on developing countries such as India (Dahasahasra, 2007) and Zambia (Republic of Zambia, 2011b). From this perspective, improvement of water supply in IWSS faces contradictory interests between the social (people) and economic (prosperity) dimensions. Therefore, such improvements should be planned bearing in mind the needs of the people and the sustainability (financial sustainability) of utility companies. For instance, supplying water continuously requires that people should pay the cost-reflective tariffs for the utility to maintain the service. In line with this, inequitable water supply in IWSS should be the problem when people are supplied water differently due to system pressure deficiency. However, whenever people of lower status are disadvantaged for whatever reason, inequality of water supply conditions should be the issue and it should be looked at not only in terms of system pressure adequacy but also in terms of water supply durations. Ensuring equality of water supply duration would ensure equitability because it would enable every person to draw the water according to the quantity needed and the ability to pay according to the tariff structure dictates. This should be understood from the fact that inequitable water supply in terms of quantities based on socio/economic class differences of people is institutionalised in many developing countries such as Zambia where per capita water consumption in the design stands for WSS depends on the housing types (Government of the Republic of Zambia, 2010; Republic of Zambia, 2011b).

Culture

Culture has been affected by the increased mixing of people from different regions. Depending on the extent of mixing, people from certain cultures consume more water than others. When culture is used in terms of work culture or institution culture, it affects the willingness to change and as a result, it affects the acceptance and use of some technologies. Moreover, it has a bearing on the level of determination to finding solutions by society.

Technology

Technology development is very important in many aspects. Through advances in computer technology, models for simulating the behaviour of WSS have become commonplace in developed countries with adequate pressure CWSS. However, for developing countries, where IWSS are dominant, the use of models is low. The use of technology is vital for leakage and consumption water demand reduction in developed countries with adequate pressure CWSS, but it is low in many developing countries (Kingdom et al., 2006). Moreover, many technologies do not apply to IWSS. The rate of technology development and application to the management of leakage and consumption demand in IWSS is a critical uncertainty and it is one of the key determinants of the sustainability of these systems.

Environmental

Environmental degradation involves the human-induced deterioration of the atmosphere, land and water resources. This is a problem without boundaries, and it was a significant driver for the development of the MDGs and SDGs. The predetermined aspect of this driver is that many global countries will develop their policies in line with these goals and others related to them because it is realised that no country can insulate itself from the effects of environmental degradation

(Raskin et al., 2002). However, critical uncertainties come from the implications of the first part of Article 21 of the SDGs declarations which states,

> *"All of us will work to implement the Agenda within our own countries and at the regional and global levels, taking into account different national realities, capacities and levels of development and respecting national policies and priorities."*

The fact that consideration should be given to national realities and capacities, and respect to be given to national policies and priorities casts doubts on the extent to which SDGs are being implemented. It is most likely that the major causes of failure for implementing the MDGs will prevail even stronger this time. The major causes of failure include lack of institutional capacity, inadequate financial resources, poor policy and legal frameworks, and lack of skilled manpower to monitor and enforce the developed policies and laws (Asafu-Adyaye, 2005). There is also a problem of considering the minimum conditions as '*adequate*' in the SDGs. For instance, the SDG 6 provides the minimum condition of *equitable access to safe and affordable drinking water for all,* but due to lack of knowledge about the effects of CWS (Anand, 2017) and lack of awareness that continuous rather than IWS is the norm, IWS is deemed to be an attainment of SDG 6 in many developing countries where governments cannot be pressured for better services by the uninformed nationals (Charalambous and Laspidou, 2017). Accordingly, it is important to note that improving water supply conditions in IWSS which include conversion to CWS requires going beyond the minimum conditions by striving to attain what is acceptable for a decent life and this is not the requirement of the SDG 6 in many developing countries as reflected in SDG 6.1 read together with Article 21 of the SDGs declaration (United Nations General Assembly, 2015). Consequently, whether awareness among the masses to demand better than the minimum level of accessing water or whether SDGs 6 and 10 are simultaneously considered as priority or not presents critical uncertainties which should be considered when planning conversion from IWS to CWS and any other IWSS improvement interventions during the SDGs period and beyond.

3.3 Development of scenarios for converting to continuous water supply status

In the literature, the recommended solution to improving water supply conditions and solving all IWSS problems is their conversion to CWS status (Simukonda et al,. 2018b). Conversion to CWS status calls for new relationships and mindsets among the water sector stakeholders (Anand, 2017). The critical aspect for the conversion is sustainability in terms of the utility's financial position (prosperity), environmental impact of the utility operations (planet) and in terms of consumer demand satisfaction (people) which can be assessed using the water supply-demand balance and the WSS pressure conditions. In the development of scenarios, in this case, the water supply-demand balance is considered first, but it is later linked in the explanations to utility financial position and the protection of the environment.

3.3.1 *Predetermined elements*

Variation over the years in the water supply from the source is the same for all scenarios because it must follow planned projects usually supported by government entities or donors. In this case, the total volume of water supplied by the water utility, rather than the different sources for the water (surface or ground water) is important. The increase in population and urbanisation are also taken as predetermined. This aspect is reflected in the GSG scenarios where population growth and urbanisation are a common factor, but the difference is in the resulting urban forms.

3.3.2 *Critical uncertainties*

Regarding water demand, the two direct drivers that have major critical uncertainties are consumption water demand and NRW (leakage) (Butler 2004).The major uncertainties are with the variation of per capita water consumption and leakage (NRW). For sustainable water supply, reduction of these or maintaining them to the lowest possible levels is the concern of every water utility. However, the effectiveness of the efforts aimed at reducing per capita water consumption and NRW depends on the preparedness of the concerned utility with respect to the handling of uncertain combined effects of the drivers presented by the occurrence of any of the GSG scenarios in the country where the utility is. Consequently, understating the future ranges of consumption demand and leakage through scenarios is critical to the utility's preparedness.

3.3.3 *The two axes scenario development method*

The two-axes method is used in which one axis represents the first total water demand driver with critical uncertainties and the other axis represents the second driver (Lindgren and Bandhold, 2003; Burdett et al., 2006). Since domestic demand is the largest component of consumption demand, domestic per capita water consumption is used for scenario development without considering commercial, public and industrial components which also vary. NRW is also used in the development of scenarios because it is a major component of the total water demand which also has many uncertainties. Thus, as shown in Figure 3.5, domestic per capita water consumption and NRW form the horizontal and vertical axes in line with the two-axes method. Since per capita water consumption and NRW are the two major drivers that determine the sustainability of water supply, the two axes result in a quadrilateral dubbed the water supply sustainability quadrilateral.

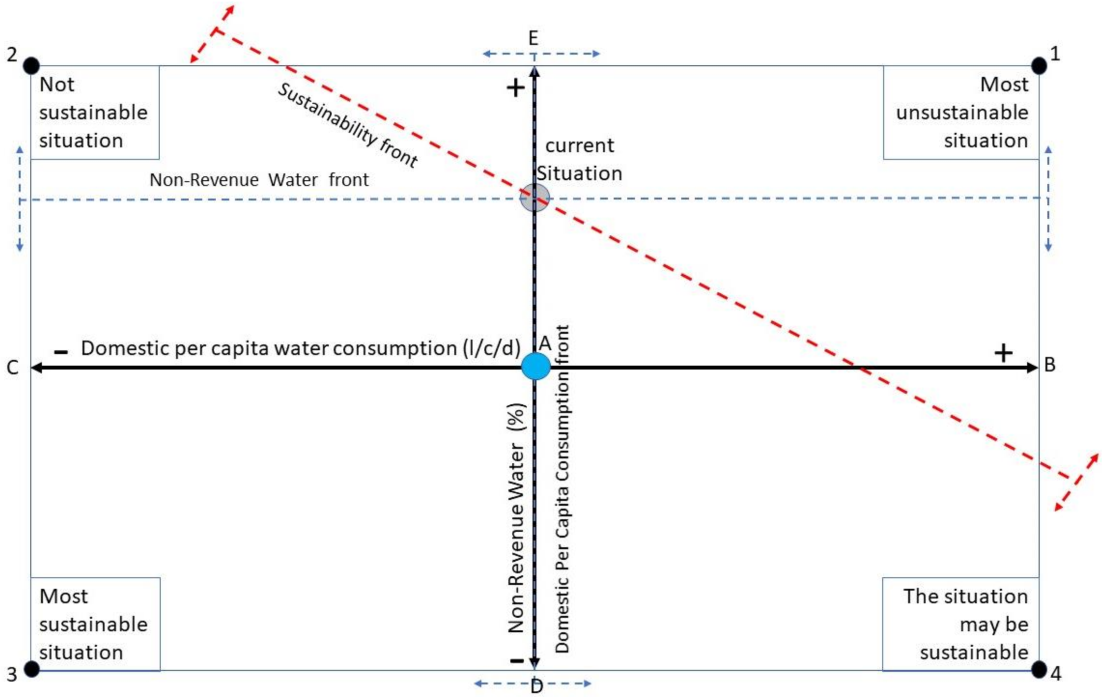


Figure 3.5: The water supply sustainability quadrilateral

3.3.4 *The water supply sustainability quadrilateral*

The two axes of the water supply quadrilateral intersect at point A (Figure 3.5) which is the reference point and represents the domestic per capita water consumption and NRW values either set by the government through the regulator or directly through policy directives. The values for point A can also be set through benchmarking. The present situation (state) of the WSS is at the intersection of the NRW front and the domestic per capita water consumption front. At that point, the sustainability of the WSS can be described and the line passing through the point (parallel to a virtual line joining 2 and 4) is the sustainability front.

Vertex 1 represents the most unsustainable situation because both per capita consumption demand and NRW are at their highest values. Very high domestic per capita water consumption when there are inadequate water resources may result in the water needs of some consumers being unmet. Moreover, very high domestic per capita water consumption may also be a sign of very low water charges which translates to inadequate revenues from the water sales. Very high NRW has negative environmental consequences, deprives consumers of the needed water and the water utility loses the water sales revenue. Vertex 2 represents a situation that is not sustainable because of the negative environmental, social and financial implications of very high NRW. Vertex 3 represents the most sustainable situation because the domestic per capita water consumption is the lowest which means efficiency in water usage due to either high water charges or improved consumer behaviour. The result is increased availability of water to meet the needs of additional consumers or improving the quality of water supply services in terms of pressure and duration of supply for IWSS. The lowest NRW means minimised negative environmental effects, improved financial resources for the utility and more water for improving the water supply services or for supplying additional consumers. For vertex 4, the situation may be sustainable because of the lowest NRW level provided there is adequate water to supply all the consumers and the water charges cover all the cost of supplying water.

In the water supply sustainability quadrilateral, the reduction of the domestic per capita water consumption or NRW is represented by (-) and an increase is represented by (+). Reduction in the domestic per capita water consumption shifts the domestic per capita water consumption front to the left towards C while an increase shifts the front towards B. Where either the domestic per capita water consumption or NRW is reduced or increased, the sustainability front moderately shifts towards 3 or 1 respectively. However, a reduction or increase of both the domestic per capita water consumption and NRW results is a large shift of the sustainability front towards 3 or 1 respectively. There is no maximum limit for the domestic per capita water consumption, but there are minimum acceptable limits for basic needs or social wellbeing. The acceptable social wellbeing values or the minimum possible domestic per capita water consumption determines point C. Reduction of the NRW only shifts the non-revenue water front downwards towards D while the increase shifts the front towards E. Again, there is no set maximum level of NRW, but, the minimum level (point D) is when the Unavoidable Annual Real Losses and Unavoidable Annual Apparent Losses are reached (Mutikanga, 2012).

The water supply sustainability quadrilateral helps to qualitatively visualise the implications of water demand management (discussed in section 3.4) targets set in a bid to improve the sustainable management and operation of a WSS. It is used qualitatively to understand the sustainability of the occurrence of any of the four scenarios for water demand management which are the business as usual, NRW management, demand reduction and the holistic scenario as discussed in detail below.

The business as usual scenario

This scenario is represented by quadrant BEA1 of the water supply sustainability quadrilateral (Figure 3.5) and corresponds to the MF scenario of the GSG scenarios. Under this scenario, water supply is left to commercial utilities, which are poorly regulated as there are no strict government policies and directives to set the levels of leakage and water consumption. Utility companies try to reduce NRW for the sake of improving their revenues. However, water demand is not controlled directly, unless through tariffs which are generally low. Thus, there are possibilities of demand increasing. However, connection of new consumers can be a challenge because of the urban sprawl. There is also a possibility of the leakage situation worsening to some levels determined by the state of the WSS. Leakage also may increase if there is an increase in the system input volume while the WSS remains poorly maintained.

The consumption demand management scenario

This scenario is represented by quadrant CAE2 and matches the FW scenario in which policies and government actions are designed not to promote sustainability because they do not support intrageneration equity in the use of resources. In this scenario, the affluent (the haves) who leave in high class residential areas consume more water than the poor masses (the have nots) who leave in poor residential areas (peri-urban areas or slums). However, the affluent may exert low water demand pressure on the WSS because they have private pumping, treatment and adequate water storage facilities such that they can mimic CWS conditions and as result, they do not care about the poor state of the WSS. Water demand for the poor masses is reduced due to the reduced duration of water supply. NRW management is not a priority in this case and it may be high in the high-class residential areas. However, in the poor residential areas, NRW in terms of leakage may be low because of the highly constrained hours of water supply, but in general, leakage is high due to the poor state of the WSS elements and due to lack of technological development.

The NRW management scenario

This scenario denotes the effect of reducing NRW only to various set levels. It is represented by quadrant BAD4 and corresponds to the PR scenario in developing countries. Government policies and actions are directed towards achieving equitable water distribution. In this regard, the government encourages the extension of WSS, but water consumption behaviour remains poor and regulation of water demand is not emphasised such that the affluent use potable water without much care including for watering gardens simply because it is cheap and they can pay for it easily. Management of NRW is critical for all water utilities because NRW makes utilities lose revenue after incurring treatment and energy costs, results in a lot of water abstracted from the source which is lost while depleting water for aquatic life and reduces the available water to the consumers. Thus, NRW reduction is important because it delays the investment in the costly new water sources, it directly improves the finances of the utility companies and it makes water available for the new consumers.

Holistic scenario

This scenario is represented by quadrant DAC3 and corresponds to the NSP scenario. The scenario shows the effects of effective water demand management (management of both NRW and per capita water consumption). In this scenario, government policies are complemented by consumers who willingly adopt water-saving technologies and attitudes. The water utility staff are also motivated and improve their work culture. For the sake of equality and equity, the possible management combinations can be reducing NRW coupled with increasing water supply in IWSS where water supplied currently is less than demand or reducing both NRW and per capita water consumption where the water supply is more than demand. The occurrence of this scenario depends on the extent to which awareness is raised among the masses and the participation of all stakeholders is encouraged. These lead to consensus in the identification of societal problems or needs and the development of solutions to the problems which includes the identification of suitable technologies (Simukonda et al., 2018b).

3.3.5 *Quantification of consumption demand and leakage for the scenarios*

For each scenario *sc*, the generic equation for calculating DMA consumption demand for a given year within the planning horizon is:

$$Q_{DMA} = P_{HC} * HC_{sc} + P_{MC} * MC_{sc} + P_{LC} * LC_{S_{sc}} + P_{Inf} * InfC_{sc} + P_{Con} * P_{Pop} + C_{con} * C_{Pop} + Ind_{Consc} * A_{ind} \quad (3.1)$$

Where P_{HC}, P_{MC}, P_{LC} and P_{Inf} are populations in high cost, medium cost, low cost and informal housing respectively, HC_{sc}, MC_{sc}, $LC_{S_{sc}}$ and $InfC_{sc}$ are high cost, medium cost, low cost and informal housing domestic per capita water consumption per day respectively; P_{Con} is the per capita public water consumption, P_{Pop} is the population in the public facilities, C_{Con} is the per capita commercial water consumption, C_{Pop} the population in the commercial facilities, Ind_{Consc} per unit area industrial water consumption for scenario sc, A_{ind} the total industrial area in a DMA.

The population in the various housing categories P_{kDMA} in a DMA is found by:

$$P_{kDMA} = P_{DMA} * \%P_{kDMA} \tag{3.2}$$

Where P_{DMA} is the total DMA population; P_{kDMA} the population of people living in housing type k in the DMA; k represents HC_{sc}, MC_{sc}, $LC_{S_{sc}}$ and $InfC_{sc}$ housing types.

Equation 3.1 is based on the Zambian case where domestic per capita water consumption is assumed to be dependent on the status of the water consumers which is reflected by their occupied housing types as high cost, medium cost, low cost and informal (Government of the Republic of Zambia, 2010; Republic of Zambia, 2011b). Where such segregation is not practiced, the appropriate domestic per capita water consumption can be used for the entire DMA population. The total consumption demand for a zone or the whole WSS is calculated by summing up demand values for the DMAs calculated using Equation 3.1.

Projections of DMA level of NRW for each scenario are calculated as percentages of the DMA input volume. The zone or system NRW values are calculated as percentages of the zone or system input volumes. To visualise the water supply-demand balance, graphs such as Figure 3.6 are plotted for each scenario. In the Figure, water sources are groundwater (boreholes) and surface water from the water treatment plant (WTP). The process of developing the graphs of the type shown in Figure 3.6 for each of the four water supply sustainability scenarios applied to a real-world WSS is demonstrated in Chapter 6.

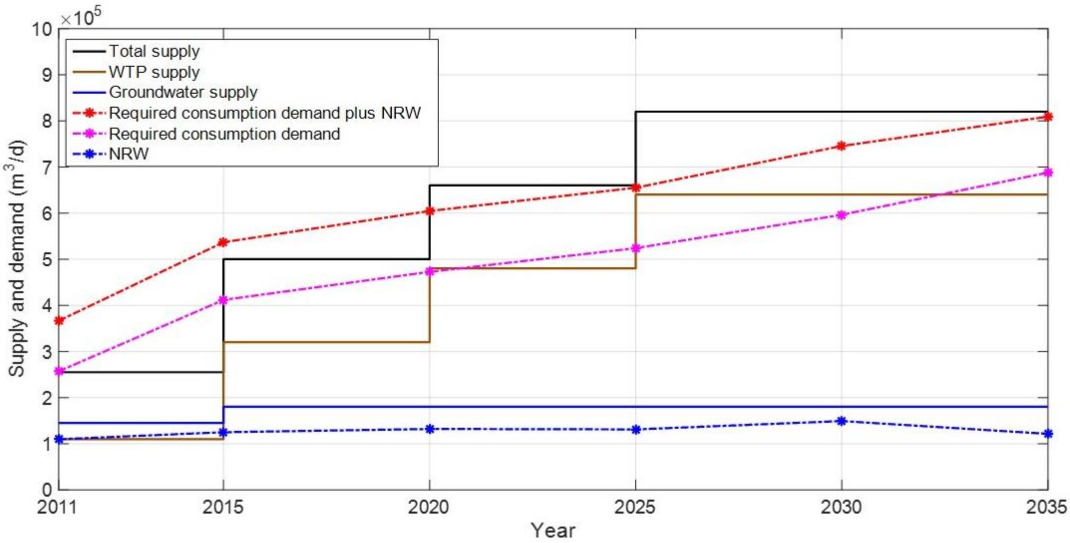


Figure 3.6: Example of the graph for the projected water supply-demand balance

3.4 Water demand management

Water demand management involves undertaking a combination of technical, economic, administrative, financial and social strategies that are aimed at reducing consumption water demand and NRW (Deverill et al., 2001; Brooks, 2007). The water supply sustainability scenarios show the importance of water demand management, especially that there is an ever-increasing consumption water demand due to population growth and urbanisation. In certain instances, such as the case is in many developing countries where populations have outgrown the capacity of WSS by many folds, water saved from reducing demand and NRW alone is not enough to improve the quality of water supply services and supply new consumers. In such situations, the twin-track approach should be followed in which water demand management is implemented while new water supply sources are being developed after considering all the costs and benefits (Defra, 2008). The twin-track approach prevents the development of new water supply sources alone to respond to increasing demand because the development of new water sources has the potential of causing environmental degradation (Kayaya and Smout, 2011).

3.4.1 *Consumption demand reduction*

Reduction of consumption water demand involves undertaking one or a combination of two or more of the measures which include, metering, tariffs, public awareness campaigns, introduction of water-saving devices and utilisation of alternative water sources (Kayaya and Smout, 2011).

Metering

Metering of all consumers is important because it enables consumers to pay for the volume of water consumed thereby providing fairness or equity. Metering reduces household water consumption for newly metered households which were initially unmetered. This helps utilities distinguish leakages for which they are responsible and which are the responsibility of the customer because they occur in the service lines after the customer meter. Metering also enables innovation in the development of tariff structures (Defra, 2008). High metering ratios eases the control of domestic water consumption and ascertaining of NRW. Even though meters provide such advantages, many utilities do not have 100% metering. For instance, of the 9 utilities that were analysed by the Eastern and Southern Africa Water and Sanitation (ESAWAS) regulators association in the period 2016/2017, only 3 utilities reported a 100% metering ratio (Table 3.2). In Zambia, in 2016 two water and sewerage companies (North-western and Eastern) had a 100% metering ratio while Lusaka Water Supply and Sanitation Company (LWSC) was at 64 % (NWASCO, 2017).

Table 3.2: Metering ratio for some utilities from 9 Eastern and Southern African Countries (ESAWAS Regulators Association, 2018)

Country	Water utility	Metering ratio (%)
Kenya	NCWSC	94.2
Zambia	LWSC	63.7
Tanzania	DAWASCO	94.0
Mozambique	AdeM	81.3
Lesotho	WASCO	100
Rwanda	WASC	100
Burundi	REGIDESO	100
Zanzibar	ZAWA	11.1
Uganda	NWSC	99.9

Public awareness campaigns

Public awareness campaigns are very important for the success of water demand management, improved revenue correction by the utility and effective institutional reform measures (Kayaya and Smout, 2011). Awareness campaigns promote behaviour change (Defra, 2008), increase acceptance of water supply improvement measures and help to remove misconceptions and myths. However, for the awareness campaigns to be successful, they should be continuous and not one-off activities, their implementation strategies should be developed by all stakeholders and there should be support from the leaders at all levels especially political leaders (Deverill et al., 2001; Water and Sanitation Program - WSP, 2010; Kayaya and Smout, 2011). The importance of political leadership support was demonstrated in the improvement of the WSS for Phnom Penh in Cambodia (Biswas and Tortajada, 2010).

Introduction of water-saving devices

Water-saving devices can be introduced through retrofitting toilets and showers in old houses, use of water efficient equipment (dishwashers and washing machines) and enforcing design codes of new housing units that ensure meeting prescribed levels of portable water consumption levels like the case is in the UK (Communities and Local Government, 2010). Retrofitting for water conservation is not common in developing countries. McIntosh (2003) reports that in Asia, with the exception of Singapore and China, there is little attempt to save water through toilet or shower retrofitting. In Zambia, retrofitting of toilets and showers is not yet a common practice in households and government institutions presumably because there are no incentives attached to the replacement of old with new toilets or showers. The use of washing machines for laundry and dishwashers is also uncommon in Zambia.

Utilisation of alternative water sources

Alternative water sources include harvested rainwater, harvested stormwater, reclaimed wastewater and greywater. Of these, harvested rainwater is more popular followed by greywater. Both of these types of water can be used for flushing urinals and toilets and for watering gardens and lawns thereby reducing the use of potable water for these purposes (Defra, 2008).

3.4.2 *NRW management*

NRW comprises three components. These are real losses, apparent losses and unbilled authorized consumption (Lambert and Hirner, 2000). Real losses are caused by poor operations and maintenance, the lack of active leakage control and poor quality of underground assets. Apparent losses are caused by customer meter under registration, data-handling errors and theft of water in various forms. Unbilled authorized consumption comprises unbilled metered consumption and unbilled unmetered consumption (Lambert and Hirner, 2000). These would include water used by the utility for operational purposes, water used for firefighting and water provided for free to certain consumer groups (McIntosh, 2003; Kingdom et al., 2006; Mutikanga et al., 2011). Of the three components, real and apparent losses are critical in the NRW reduction.

The reduction of real losses in developing countries would make water available for more consumers thereby postponing investment in the development of new water source infrastructure (Kingdom et al., 2006). Effective leakage reduction requires that all the four pillars of real losses management which are pressure management, active leakage control, asset management, and speed and quality of repairs (Figure 3.7) are followed (Mutikanga, 2012).

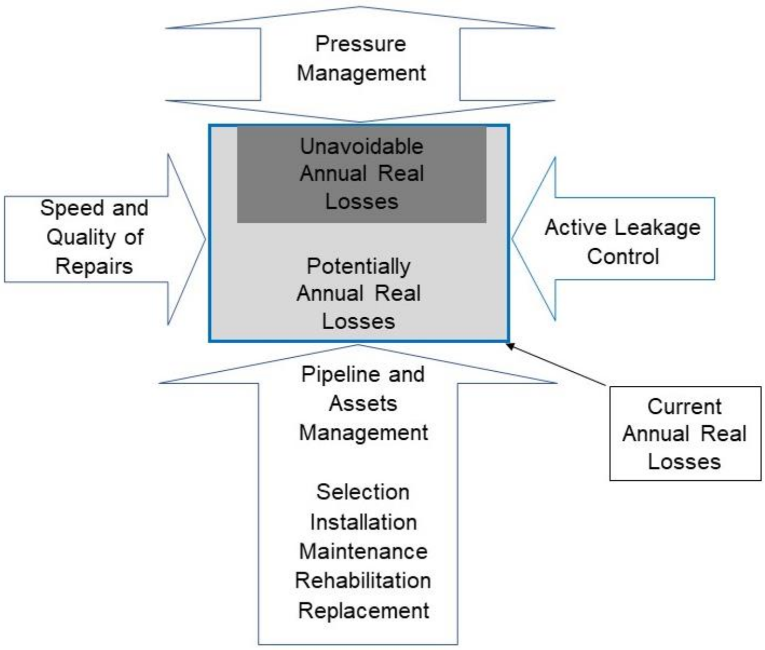


Figure 3.7: The four pillars of a successful leakage management policy (Liemberger and Farley, 2004)

Pressure management

Pressure management is one of the key elements in a well-developed leakage management strategy (Farley et al., 2008). This is because leakage (background leaks and pipe burst) is dependent on pressure such that high pressure results in high leakage flow rates and vice versa (Skipworth et al., 1999; Puust et al., 2010). This dependency of leakage on pressure is demonstrated by the application of the *Fixed and Variable Area Discharges* (FAVAD) principle (Liemberger and Farley, 2004; Lambert et al., 2017).

To effectively develop and implement a pressure management system, knowledge of the WSS based on comprehensive flow and pressure data is important, but for IWSS, data is generally not comprehensive due to poor database management (Klingel and Nestmann, 2014; Simukonda et al., 2018b). The availability of comprehensive data for a WSS also enables effective development of DMAs and hydraulic models both of which are not only important for pressure management but also for leakage assessment and awareness (Puust et al., 2010).

Active leakage control

The active leakage control policy involves four aspects: active leakage assessment, awareness, localisation and pinpointing (Puust et al., 2010).

i. Leakage assessment (water audit)

This is meant to quantify the amount of water lost, but without concern as to where the leaks are located (Puust et al., 2010). Two methods are used for leakage assessment. These are the top-down and the bottom-up method. The top-down method is used to estimate the amount of water loss by subtracting the amount of water consumed from the system input volume. Estimating water losses using the top-down method is called a crude estimate (Puust et al., 2010). This is the approach that is used in many IWSS. On the other hand, the bottom-up method involves all operational areas of the utility company including billing records, distribution system pressure and flow monitoring and accounting principles to determine the WSS efficiency (Puust et al., 2010). For real losses, the bottom-up method can be done using either the 24 Hour Zone Measurement (HZM) or

Minimum Night Flow (MNF) analysis (Puust et al., 2010; Mutikanga, 2012). For a network with defined DMAs or zones, the MNF analysis is used. This method is not suitable for IWSS because night flows can be as high as day flow (Ndirangu et al., 2013) or even higher (Brian Colquhoun Hugh O'Donnell and Partners, 2010).

ii. Leak detection (awareness)

The term leak detection is in certain cases used to mean leak localisation as defined in Mutikanga (2012) or leak localisation and location (pinpointing) as in Li et al. (2015). However, as shown in Figure 3.8, Leak detection or leak awareness is the realisation that a leak has occurred in the WDS, zone or DMA without necessarily knowing where it has occurred (Puust et al., 2010; Romano et al., 2011; 2014). The major leak detection approach is the DMA flow monitoring which enables the quantification of leakage and prioritisation of leak localisation and location activities (Liemberger and Farley, 2004).

iii. Leak localisation

Leak localisation makes leak location or pinpointing easier because it is used to isolate possible leakage areas or pipes. The most common leak localisation technique is step-testing (Liemberger and Farley, 2004; Puust et al., 2010). Unless the part of the system being investigated is temporarily converted to CWS and water overflow from storage facilities is controlled, step-testing is difficult to apply to IWSS because continuous water overflow can be seen as leakage (Harmilton, 2017) and any temporal increase in water supply without proper communication to the consumers is met with high water drawing for storage. Two methods in the literature that are specifically developed for IWSS and used to quantify leakage, localise and locate leaks (with the aid of other techniques) are the stop tap method and the mobile tanker method (Farley, 2001).

iv. Leak location (pinpointing)

Under IWS, leak location or pinpointing (the determination of the exact position of leaks), can be done by the walking field clue using visual observation of signs that signify the presence of water (CPHEEO, 2005) such as soil moisture and vegetation growth. For leaks that do not appear on the surface, the basic acoustic instrument for Leak location is the listening rod also called the sounding stick (Liemberger and Farley, 2004; Li et al. 2015). The listening rod is frequently used because it can be used to listen to leaks along pipes and on fittings such as valves and hydrants to confirm leaks found by other methods (Farley, 2001; Hamilton and Charalambous, 2013; Hamilton, 2017). Other techniques for leak location are the gas tracer and Leak Noise Correlator (CPHEEO, 2005).

Speed and quality of repairs

The aspects of speed and quality of repairs are discussed with reference to the leak lifetime (Figure 3.8). Other than bursts which are reported quickly due to the large flows involved, background leaks take long to be noticed. This results in a long duration between the occurrence of a leak and the time of its detection (Mounce et al., 2010). For IWSS, this duration may be extremely long because rather than the active leakage control, these systems follow the passive leakage control policy which tends to increase leakage levels as it does not involve any systematic approach (Puust et al., 2010; Mutikanga, 2012). Under the passive control policy, some leaks or bursts are not reported on time or are not even reported at all, and those that discharge in holes are not even detected.

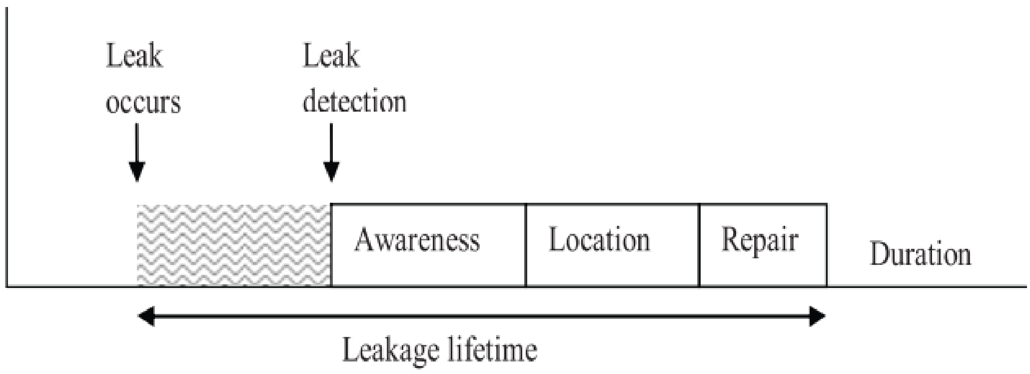


Figure 3.8: The life cycle of a leak (Mutikanga, 2012)

For reported leaks, their location or pinpointing is not a problem, but the problem is the long time it takes before they are repaired. After a leak is detected or reported, it should be repaired as soon as possible to reduce the volume of water lost (Farley et al., 2008). Liemberger and Farley (2004) indicate that reported visible leaks should be repaired within 24 hours for leaks on mains and within 7 days for small leaks on service connections. However, due to the poor financial positions by many IWSS which lead to a lack of repair materials, it takes long before reported leaks are repaired (Simukonda et al., 2018a). The lack of repair materials results in the use of improvised materials thereby resulting in poor quality of repairs (Republic of Zambia, 2011b). Consequently, to reduce leakage, there should be efficiency in the utility organisation and procedures to ensure there is timely detection of all leaks and the availability of resources to facilitate speedy and high quality leak repairs (Farley et al., 2008).

Asset management

Asset management is the application of all aspects of management and operation of the physical assets of a WSS that ensures that it continues to provide the desired level of service using the most cost-effective way of rehabilitating, repairing, or replacing of the assets (Environmental Protection Agency, 2008). According to the asset management framework developed by the Environmental Protection Agency (2008), there are five core elements of an asset management plan (Figure 3.9). In the figure, the bullet points are some of the best practices or strategies that should be followed.

Asset management is very important for long term leakage control (Farley et al., 2008), and the result of poor asset management is indicated by high leakage levels (Mutikanga, 2012). For many, if not all, IWSS, asset management is poor partly as a result of the poor financial positions of the utilities due to poor revenue collection and partly as a result of poor database management which affects all the five core elements of the asset management plan and virtually all WSS management and operation activities (Simukonda et al. 2018a; 2018b; Klingel, 2012). In instances where asset management is extremely poor, the reduction of real losses to improve water supply services has called for a complete system overhaul (Water and Sanitation Program - WSP, 2010). To avoid such huge investments, utilities running IWSS should ensure that asset management is not only well planned, but also well implemented, followed by progress evaluation and improvement actions based on the evaluation information (Environmental Protection Agency , 2008).

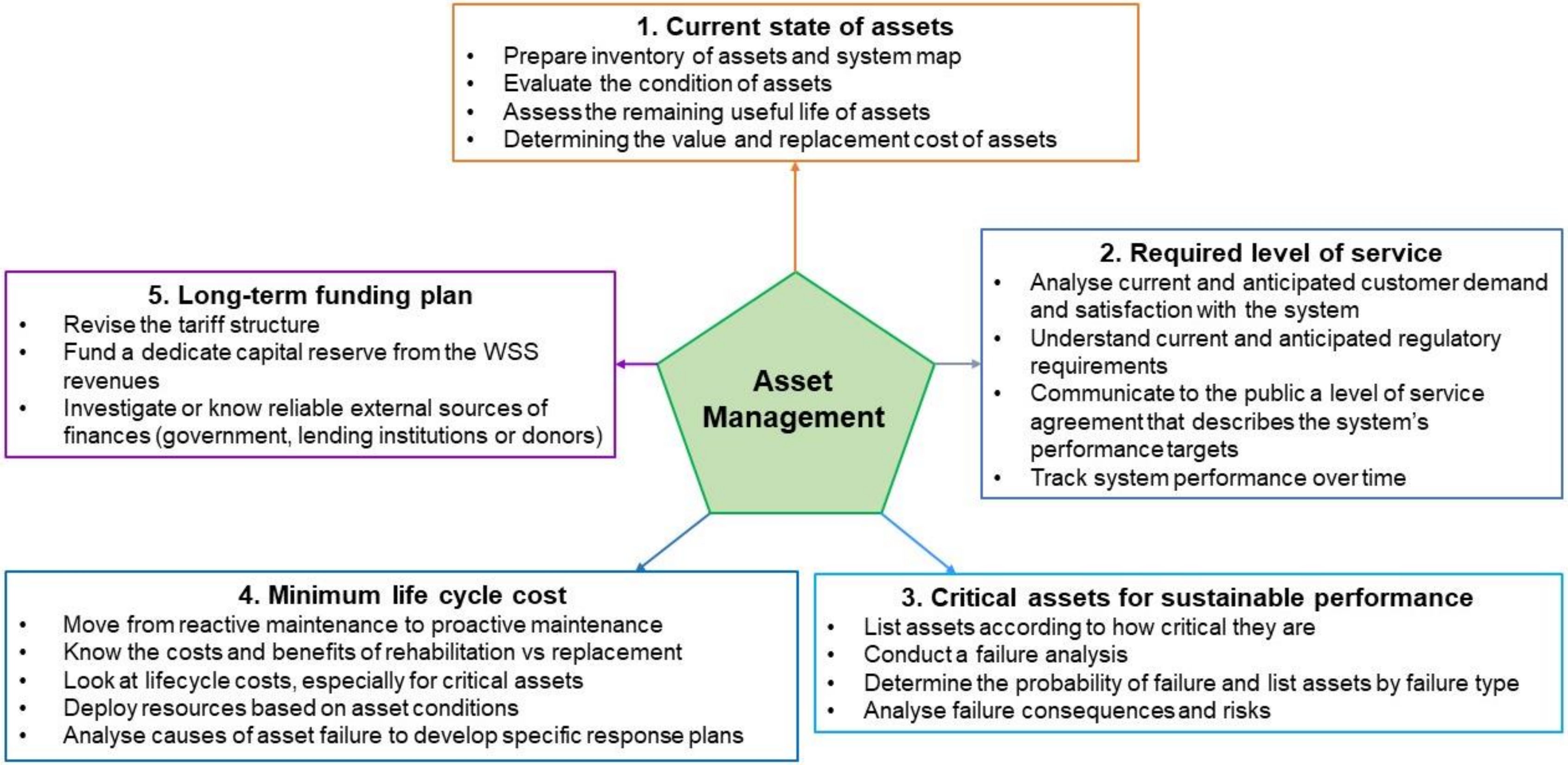


Figure 3.9: The five core elements of asset management planning (Adapted from: Environmental Protection Agency, 2008)

In comparison with real losses, the reduction of apparent water losses is said to be more attractive because it leads to increased revenues which the utility needs to reduce real losses and it improves the utility financial viability (Mutikanga, 2012). This is because, water charges are expected to be far much higher than production costs which means saving even a small volume of apparent losses results in a huge financial benefit (Farley et al., 2008). For many IWSS with less than 100% metering ratio, determination of the actual apparent water losses is a challenge because of the possibility that those on fixed charge, but supplied with excess water, consume more water than they are billed and those supplied with less water consume less water than billed (Mastaller and Klingel, 2018). This problem can effectively be controlled by 100% metering. The reduction of apparent water losses involves the control of aspects that fall under the four pillars of apparent losses management (Figure 3.10).

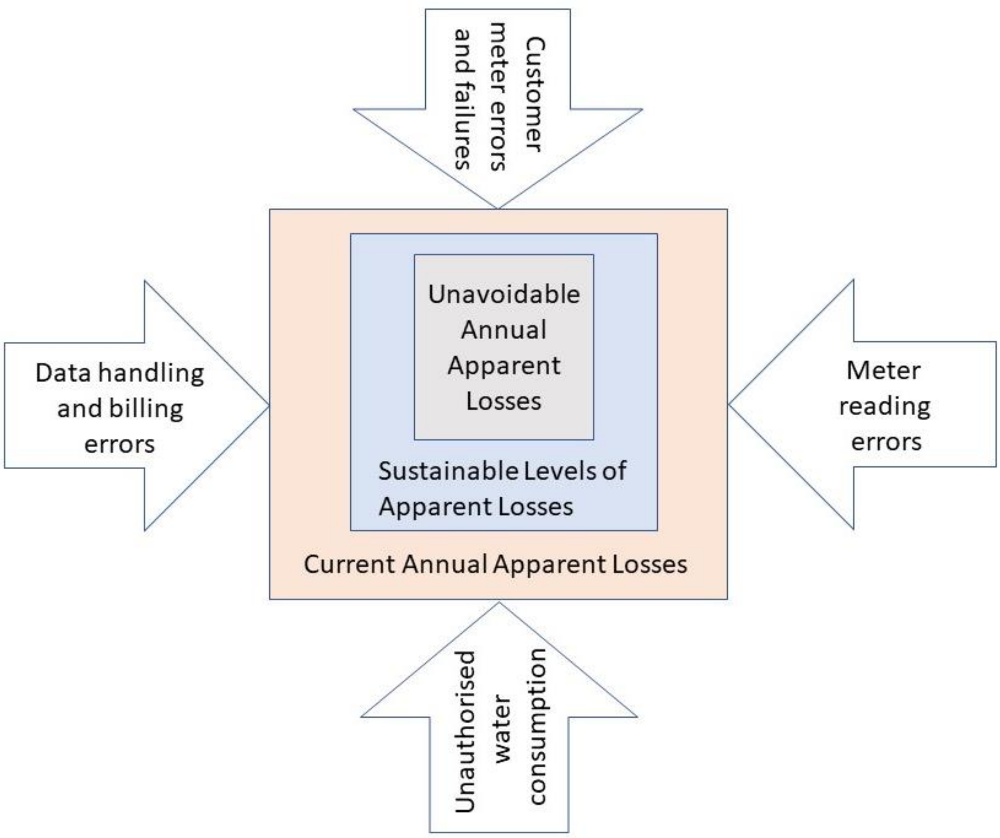


Figure 3.10: The four pillars for the management of apparent water losses (Adapted from: Mutikanga, 2012)

Meter inaccuracy and failures

Meter inaccuracy by the under-registration of consumed water can be due to wear and tear over time (Van Zyl, 2011; Mutikanga et al., 2011). However, it can be exacerbated by poor water quality resulting in sediments settling in pipes and meter internal parts. The accumulation of sediments in meters increases friction which results in the slow movement of meter parts thereby under-registering water consumption (Farley et al., 2008). For IWSS, meter inaccuracy increases due to the alternating drying and wetting of the meters between no water supply and supply durations. This routine leads to meter inaccuracy in three ways. The first is that the routine promotes leaks and pipe bursts due to pipe joint weakening and fatigue and increases valve deterioration all of which increase the frequency of repair works (Simukonda et al., 2018b). These in turn increase the number of suspended particles in the water. Moreover, when pipes are not pressurised, there is the ingress of particles through leak openings (Van Zyl, 2011). The second way is that the routine leads to over-registration of water consumption due to airflow through the meters before pipes are fully pressurised (Van Zyl, 2011; Foufeas and Petroulias, 2017; Mastaller and Klingel, 2018). Meter inaccuracy is also linked to household overhead storage tanks. However, the effects of these tanks (as a result of the installed ball valves) depend on their diameter which has bigger effects if large and the duration of water supply which affects the accuracy of the meter more if it is long (Mutikanga, 2012).

Meter failure is caused by similar factors as those for meter inaccuracy except for the effect of the household overhead storage tanks and the two-phase flow (Farley et al., 2008; Van Zyl, 2011). Another cause of meter failure is vandalism by people who would like to use water free of charge (Van Zyl, 2011, Mutikanga, 2012). Consequently, meter inaccuracy and failures can be reduced by converting from IWS to CWS status and by awareness campaigns to minimise vandalism.

Meter reading errors

Meter reading is commonly done manually in many developing countries. This method of meter reading is prone to human errors (Mutikanga et al., 2011) which may be due to negligence or even corruption by the meter reader, incompetence or inexperience meter readers who may read meters wrongly or place decimal figures in wrong places, and unclear meter screens (Farley et al., 2008). Meter reading errors can be reduced through the establishment of clear systems and procedures for meter reading, greater supervision, training and motivating of meter readers and rotating their meter reading routes to prevent their being corrupt (Farley et al., 2008). The use of smart meters would help not only to reduce the meter reading errors, but also data handling errors (Farley et al., 2008; The Republic of Zambia, 2014).

Data handling and billing errors

These errors can result in disputed bills which may distort the utility revenue correction greatly (Chanda, 2009). The errors can take place at any stage during the process of writing the figures read from the meter on the meter reading sheets, transmitting or capturing data from the sheets into the customer billing database and giving or sending the bill to the customer (Farley et al., 2008; Mutikanga et al., 2011). Moreover, sometimes water consumption figures are just estimated using past trends because meter readers cannot gain access to some customer meters located inside customer premises as they find no one at home, but in certain cases only find vicious dogs. Many cycles of estimating these water consumption figures increase the chances of grossly inaccurate estimations (Mutikanga et al., 2011). Another possible source of error is the incorrect tariff allocation. The Republic of Zambia (2014) highlights two instances of this. The first is where an unmetered customer is recorded as metered. The water consumption value for such a customer would be zero despite having consumption because there is no meter to record the water use. The second is where a commercial customer is indicated as domestic which corresponds to a lower tariff. Consequently, data handling and billing errors can be reduced by the same measures for reducing meter reading errors besides regular updating of customer databases (The Republic of Zambia, 2014).

Unauthorised water consumption

Unauthorised water consumption includes illegal connections, meter bypassing, illegal use of hydrants and missing data in the billing system (Farley et al., 2008; Banda, 2009). It may also be excessive water that has been poorly recorded by meters that have been tampered with (Van Zyl, 2011). Illegal connections include consumers who are disconnected but illegally reconnect themselves. This leads to situations where the utility's system shows that the consumer is disconnected and gets no water bill while on the ground, the consumer is drawing water (Farley et al., 2008; The Republic of Zambia, 2014). Another aspect of unauthorised consumption is that some consumers who are disconnected resort to getting water from neighbours who are on fixed charge which results in one bill for the household with the fixed charge connection while the water consumed could be for two households (Banda, 2009). Consumers with missing data in the billing system, maybe legally connected but because there are no records of their property in the utility billing system, the consumer uses water for free (Farley et al., 2008; Chanda, 2009; Republic of Zambia, 2011b).

Unauthorised water consumption can be reduced through public awareness programmes that would increase the alertness and willingness of members of the public to report any form of water theft and development of regulations and effective enforcement of stiffer punishments for illegal water users (Farley et al., 2008; Chanda, 2009). Unauthorised water consumption can also be reduced by regular database updates and increased metering.

3.5 Conclusion

The information presented in this chapter is critical to the sustainable conversion from IWS to CWS status and the success of any water supply improvement in IWSS. This is because the chapter discusses the functions of scenarios, the process of scenario development, the linkage between the GSG scenarios to urban forms and the water supply sustainability quadrilateral which is vital in the qualitative representation and implications of the water demand management targets.

The chapter focuses on scenarios and scenario analysis because they are an important way of incorporating future uncertainties in the planning process especially for improvement interventions for IWSS which have numerous unknowns because of poor database management. Under great uncertainties, scenarios are useful in deepening the understanding of the problem or the system being investigated, to setting informed targets, in making decisions and in the development of strategies. They are also useful as sources of information and means of communication. They enable the creation of the picture of the desired future and the interrogation of what the future *may* be rather than *will* be if certain development paths occur thereby providing the range of possible futures and enabling the setting of benchmarks which when met trigger actions intended to steer development towards the desired future. These are premised on the understanding that human beings (planners) have some knowledge of the future, that the knowledge is limited and that the future can be influenced by human choices and actions.

Because of their usefulness, scenarios have been used extensively in developed countries to probe into the future in many fields including the water sector. Concerning the water sector, scenarios are used to analyse alternative solutions for WSS development, assessment of future resilience of solutions that are presently seen to be sustainable and to analyse the effectiveness of policy directives that focus on the use of technology only on urban regeneration sites. For these, scenarios have helped to see that the application of technology in the reduction of water demand is more effective if complemented with change in human behaviour. The usefulness of scenarios however has not been exploited in developing countries in relation to the planning process for improving water supply in IWSS and for converting these systems to CWS status. This lack of use of scenarios in IWSS which have several sources of uncertainties prompted the development of scenarios for these systems in this study. The GSG scenarios, which apply to the analysis of the sustainability of strategies at any scale of development because their narratives are comprehensive and cover all drivers, provide a fairly objective basis for analysing the governance systems of nations and the linkage of the governance systems to the developed scenarios such as the water supply sustainability scenarios. The GSG scenarios' narratives reveal four critical issues as follows:

- Governance is the major driver for all the scenarios. This echoes the assertion that governance (poor governance) is one root causal factor for IWSS that affects all others. This underscores the need to take governance issues seriously when planning the conversion from IWS to CWS including any other water supply improvements for IWSS.

- The NSP is a very desirable scenario, but its occurrence in the future depends on the occurrence of surprises or a strong attractor that may change the status quo. For this to happen civil society organisations should create a lot of awareness about the need to improve the quality of lives for all human beings and to overcome the dominance of multinational corporations in developing countries. Concerning IWSS, this level of civil society is presently missing. Donors can be a good attractor to change several aspects of governance and IWSS management and operation.
- The PR scenario seems to be the most common scenarios from the political perspectives because of the international set goals such as the SDGs, but in practical terms the developed policies and laws fall short of enforcement. The PR scenario faces huge technical challenges to reduce the effect of the MF scenario’s maintained underlying values, lifestyles and economic structures. Another challenge is that its plausibility lies in the currently nonexistence “political will” which should give rise to an unprecedented and unyielding governmental commitment to achieving sustainability.
- At present, the MF and the FW are practically the dominant scenarios in many developing countries and they will most likely be the ones that will dominate up to 2030 and possibly beyond unless surprises occur which result in the increase in political will by governments to effectively develop and enforce policies that counter the effects of these scenarios and shifts the role of civil society groups from a focus on civil and political rights to awareness creation among the populace so that they can demand for the fulfilment of their social and economic right by governments.

The four critical issues reflect the fact that the current world political dispensation makes the occurrence of MF and the FW scenarios more likely in developing countries than the PR and the NSP scenarios for which the necessary means of attaining them do not yet exist.

The linkage of GSG scenarios with the urban forms is important because in developing countries WSS are extending to new places. For this, different urban forms exhibit different levels of cost-effectiveness in terms of WSS extension and operation. For instance, while the uniform expansion which corresponds to the

PR scenario is the most cost-effective, the polycentric expansion which corresponds to the NSP scenario is the least cost-effective development if it relies on centralised water sources.

Concerning the total water demand, the occurrence of the MF scenario entails high leakage and domestic water demand, and high cost of delivering water to new areas which makes water unaffordable to the poor. The PR scenario relies on policies and regulations aimed at improving equity and controlling the urban sprawl to compact/uniform urban development. The implication of these is the increase in consumption water demand especially with poor technology use in IWSS (developing countries) which also contributes to high levels of leakage. For the FW scenario, it is not clear how water demand will be affected because water demand depends on the level of water supply restrictions to the poor masses, the population of the affluent and their use of the limited available technology. The NSP is the scenario envisaged in the SDGs. Under this scenario, equity is high and (water) consumer behaviour is very good and the use of technology is high. These result in the reduction of leakage while consumption demand may increase or reduce depending on which direction promotes equality and equity in terms of water supply services.

Basing on the discussion of the scenario development process and the narratives of GSG scenarios, four scenarios for assessing the sustainability of planned water demand management targets to improve water supply services for IWSS are developed using the two-axes scenario development method. The water supply-demand balance is used to assess the sustainability of the system in terms of consumer satisfaction. Since water supply is taken as a predetermined element which makes its variations the same for all scenarios, only total demand drivers are used to develop the water supply sustainability scenarios. Thus, for the two axes method, the horizontal and vertical axes are represented by the domestic per capita water consumption and the NRW (leakage) which are the two major drivers of the total water demand. Since these drivers are key to the sustainability of water supply, the axes form a quadrilateral which has been named the water supply sustainability quadrilateral. This quadrilateral helps to visualise the implications of any of the four developed scenarios: the business as usual, NRW management, consumption demand management and the holistic scenario. These scenarios correspond to the MF, FW, PR and the NSP scenario. However, the perspective of the PR scenario under IWSS is different. Policies are not directed to the management of consumption demand, which is already restricted, but NRW which is critical as it affects the water utility financial base and the extension of the WDS to new areas which is politically very desirable. For the NSP scenario for IWSS, the requirements of equality and equity lead to two possible management combinations. The first is increasing water supply while reducing NRW in IWSS where the water supplied currently is less than demand and the second one is reducing both NRW and per capita water consumption (if it is very high) where water supply is more than demand.

To assess the four scenarios quantitatively, a generic equation for calculating consumption demand has been developed in which it is assumed that water consumption depends on the status of consumers as reflected by the type of their houses. NRW values are calculated as percentages of the system input volumes. The analyses in this chapter and the use of the developed scenarios for quantitative analysis of the water supply-demand balance should be followed judiciously when planning the conversion from intermittent to continuous water supply or any water supply improvements related to IWSS. Chapter 6 demonstrates the analyses on a real-world WSS.

4 MODELLING OF INTERMITTENT WATER SUPPLY SYSTEMS

To plan any water supply improvement measures, the approximation of the WSS behaviour through modelling provides a good basis for decision making. However, modelling of IWSS is a challenge because there is a lack of models for these systems. This chapter describes the method developed for modelling IWSS which considers the similarities and differences between consumption water demand and leakage.

The chapter is arranged into 6 sections. Section 4.1 is the introduction which briefly differentiates the three categories of WSS from the modelling perspective thereby providing the basis for the development of a new modelling method for IWSS. Section 4.2 provides the fundamental equations that describe nodal water outflows in WSS. It also gives the theoretical background on the existing PDA approaches and leakage modelling. Section 4.3 describes the methodology for developing the modelling approach for IWSS. In section 4.4 the literature on the model calibration process is presented. Section 4.5 explains the graph theory-based technique for developing pressure distribution maps of the WSS and finally, section 4.6 provides the chapter conclusion.

4.1 Introduction

Modelling of a WSS requires the understanding of its characteristics and the isolation of its critical distinguishing aspects. As stated in Chapter 2, for modelling purposes, there are three possible categories in which a WSS can fall. These are CWSS with adequate pressure, CWSS with deficient pressure, and IWSS with deficient pressure.

The major characteristic of CWSS with adequate pressure is that water outflows from demand nodes always meet the demand (Germanopoulos, 1985; Giustolisi and Walski, 2012; Paez et al., 2018). Because of this, CWSS with adequate pressure have traditionally been modelled using DDA-based models, but they can also be modelled using PDA-based approaches. The DDA modelling approach for CWSS with adequate pressure is discussed in subsection 4.2.1.

The aspect that differentiates CWSS with deficient pressure from CWSS with adequate pressure is the pressure deficiency that occurs at some demand nodes during certain periods of the day. This makes the node outflow during pressure deficient periods to be less than the demand. The node water outflow rate is not known as it depends on the nodal pressure. This pressure dependent node water outflow during pressure deficient periods cannot be realistically modelled using DDA, but it is possible using PDA approaches (Germanopoulos, 1985; Chandapillai, 1991; Lee et al., 2016). The PDA modelling approach for CWSS with deficient pressure is discussed in subsection 4.2.2.

The major characteristic that distinguishes IWSS from CWSS is that water is supplied to different parts of the WSS at different times sometimes even for different durations. This results in the continuous process of turning on and off of water supply to different parts of the system either using pumps or valves resulting in different water supply schedules to the different parts. The pressure deficiencies and the multiple water supply schedules result in inequitable water supply problems. The multiple water supply schedules characteristic differentiates the PDA modelling approaches for IWSS from those for CWSS with deficient pressure. Since PDA modelling approaches for IWSS are uncommon in the literature, in this chapter one approach has been developed. The developed PDA modelling approach for IWSS is discussed in section 4.3.

4.2 Background

In hydraulic models, there are differences in the nodal outflow relationships between DDA and PDA approaches. The differences are in terms of the way base demands are implemented in the models and the way these demands are influenced by the demand (supply schedule) multiplication factors.

4.2.1 *Nodal outflow relationships for demand driven analysis (DDA) approaches*

In DDA approaches, several demands can be allocated to a node and the term base demand mainly applies to average domestic and commercial consumption demand. Other base demands are allocated separately and identified by their specific names such as fire, industrial and leakage. For all the base demands, corresponding demand multiplication factors are directly applied to them to vary the nodal water outflow according to time. The fundamental DDA nodal outflow relationship for demand node *j* at time *t* is given by Equation 4.1 (Sebbagh et al., 2018):

$$Q_j = Bd_{Dj} * dmfactor + Bd_{Lj} * lmfactor \tag{4.1}$$

Where Q_j is the total water supplied (which is equal to the total water demand)*;* Bd_{Dj} the consumption base demand; $dmfactor$ the consumption demand multiplication factor; Bd_{Lj} the nodal average leakage (leakage base demand) and $lmfactor$ is a leakage multiplication factor.

In Equation 4.1, the consumption demand outflow ($Bd_{Dj} * dmfactor$) does not depend on pressure and as such, consumption demand will always be met regardless of the demand node pressure. For leakage outflow ($Bd_{Lj} * lmfactor$), two cases are possible. The first is where leakage is assumed constant over the simulation duration represented by $lmfactor = 1$ (Sebbagh et al., 2018). This does not reflect the fact that leakage is pressure dependent. The second case is where leakage is implicitly part of the nodal demand (Brian Colquhoun Hugh O'Donnell and Partners, 2010; Khedr et al., 2015). In this case, leakage outflow varies by the same factor as consumption demand ($lmfactor = dmfactor$). This is against the fact that when consumption demand is high, pressure and consequently leakage becomes low and vice versa (Liemberger and Farley, 2004; Puust et al., 2010). This relationship between consumption demand, pressure and leakage is shown in Figure 4.1 which is developed from the 24 hours application of the FAVAD principle to a DMA (Liemberger and Farley, 2004). Thus, both approaches of modelling leakage ($lmfactor = 1$ and $lmfactor = dmfactor$), do not reflect the fact that leakage is pressure dependent. Consequently, Equation 4.1 is not realistic especially for pressure deficient systems.

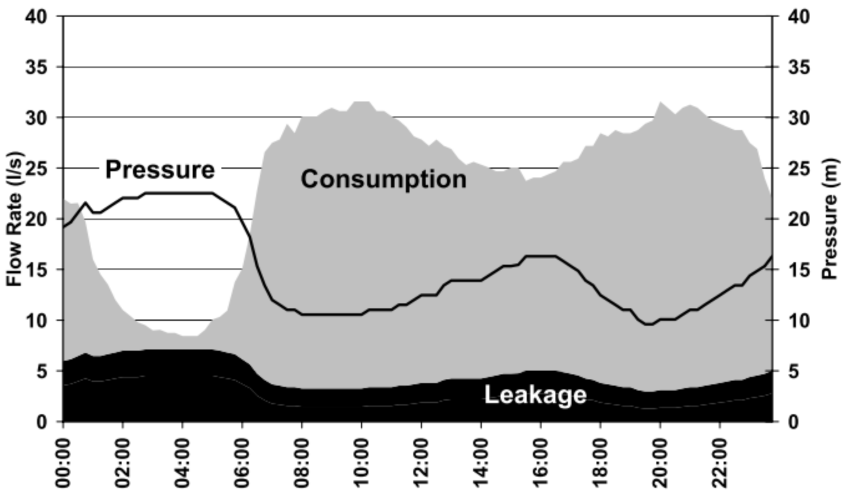


Figure 4.1: Pressure and leakage variations in relation to consumption demand (Liemberger and Farley, 2004)

4.2.2 *Nodal outflow relationships for pressure driven analysis (PDA) approaches*

For PDA methods, even though base demands are allocated to demand nodes, they are not directly influenced by demand pattern multiplication factors or supply schedule multiplication factors in the case of IWSS. These factors are used to calculate coefficients in the mathematical models that express the relationships between nodal outflows and pressure heads. The fundamental PDA methods' relationship for the nodal pressure head and total water outflow from node *j* (Q_{totalj}) at time *t* is:

$$Q_{totalj} = Q_j^d + Q_j^l \tag{4.2}$$

Where Q_j^d is the node consumption demand outflow and Q_j^l the node leakage outflow.

In this work Q_j^d is calculated, using the Wagner et al. (1988) model (Equation 4.3) because it is the most commonly used PDA model (Giustolisi and Walski, 2012; Paez et al., 2018) and it gives good flow predictions (Gupta and Bhave, 1996; Shirzad et al., 2013).

$$Q_j^d = \begin{cases} Q_j^{req} & for\ H_j \geq H_{req} \\ Q_j^{req}\left(\dfrac{H_j - H_m}{H_{req} - H_m}\right)^{\gamma} & for\ H_m < H_j < H_{req} \\ 0 & for\ H_j \leq H_m \end{cases} \tag{4.3}$$

Where Q_j^{req} is the required consumption water demand calculated as $Bd_{Dj} * dmfactor$; H_j the actual node pressure head; H_m the minimum node pressure head; H_{req} the required system pressure head at which water demand is fully supplied and $\gamma = \frac{1}{n}$. γ is an exponent that can be determined through calibration (Cheung et al., 2005) and n varies between 0.5 to 2.5 (Mahmoud et al., 2017).

In Equation 4.3 and many existing PDA methods, the base demand allocated to a demand node represents consumption demand only with no explicit consideration for leakage. Although leakage takes place at the junctions (nodes) as well as along the pipe, for modelling purposes, it is assumed that it takes place at the nodes. Thus, in Equation 4.2, the Q_j^l is the contribution of all the pipes connecting node *j* with other nodes. First, leakage through a single pipe joining two nodes is considered. For a pipe *x* joining nodes *j* and *i,* its leakage outflow at time *t* is (Germanopoulos, 1985):

$$Q_{Lx} = C\left(L_{xji}{H_{ji}}^{av}\right)^{\alpha} \tag{4.4}$$

Where Q_{Lx} is the pipe *x* leakage flow rate*;* C the constant that depends on the network; L_{xji} the length of pipe *x* joining node *j* and *i*; ${H_{ji}}^{av}$ the average pressure of H_j and H_i which are pressure heads for nodes *j* and *i* respectively and α the leakage parameter representing the combination of bursts and background leakage, and the types and areas of the leaking holes (Giustolisi et al., 2008). α is determined through calibration, but it varies between 0.5 and 2.5.
Since the leakage value described by Equation 4.4 is for a single pipe joining two nodes, for it to be analysed as a nodal demand, half of it is allocated to each node. Thus, for a node *j* that is connected to N_j nodes by N_j pipes, Q_j^l is determined by:

$$Q_j^l = \frac{1}{2}\sum_{i=1}^{N_j} C\left(L_{xji}H_{ji}{}^{av}\right)^{\alpha} \tag{4.5}$$

Equation 4.5 is a realistic way of modelling leakage from nodes, but it cannot be used for EPS because at the beginning of each simulation run, the pressure values are unknown for all the N_j nodes (Gupta et al., 2016) while during the simulation run, they are evaluated at different times. Gupta et al. (2016) approximated Q_j^l as a function of pressure head only by:

$$Q_j^l = \left(\frac{Bd_{Lj}}{H_{req}{}^{\alpha}}\right)H_j{}^{\alpha} \tag{4.6}$$

For a system where nodal pressure is higher than H_{req}, consumption water supplied by the node is assumed to be equal to Q_j^{req} and there is no increase beyond that level of supply while leakage is dependent on the actual pressure rather than H_{req}. For a system with many DMAs, leakage is influenced by different pressure values which necessitates the use of average pressure for each DMA rather than H_{req} (Lambert, 2001; Lambert et al. 2017; Sebbagh et al. 2018). In that case, Equation 4.6 becomes:

$$Q_j^l = \left(\frac{Bd_{Lj}}{H_{aveDMA}{}^{\alpha}}\right)H_j{}^{\alpha} \tag{4.7}$$

Where H_{aveDMA} is the average DMA pressure head which should be known by the system operators through measurements.

4.3 **Methodology**

The proposed methodology for developing the modelling approach for IWSS is summarised in (Figure 4.2)The methodology involves three general activities. The first is the selection of the suitable PDA methods (discussed in detail in from sub-section 4.3.1 to 4.3.3). This general activity has three specific activities. The first is the selection of the methods based on the literature following what the authors of the methods say. This activity is followed by the selection of better performing methods after evaluation (by the author of this work) of the methods by comparing their EPS performance using a known benchmark network in the EPANET- MATLAB interface and the final specific activity is the selection of the best performing PDA method after the comparison of the performance of the selected PDA methods on the real-world network (presented in detail in Chapter 5). This results in the selection of the most suitable existing PDA method which is used for the development of the IWSS modelling approach by incorporating the leakage modelling component and the use of multiple water supply schedules.

The second general activity is discussed in detail under sub-section 3.3.4. Under this general activity, the Single-Iteration Pressure Driven Analysis (SIPDA) (Mahmoud et al. 2017) is modified to develop the tool for modelling IWSS dubbed the Modified SIPDA (M-SIPDA). The specific activities involved are the development of the mathematical model for the M-SIPDA, the development of the demand node topology for the M-SIPDA and the setting of the artificial elements for M-SIPDA demand node topology so that the mathematical model can be implemented during EPS.

The third general activity is the calibration of both the EPANET hydraulic model and the M-SIPDA. Calibration methods for the hydraulic model and the M-SIPDA are discussed in section 4.4.

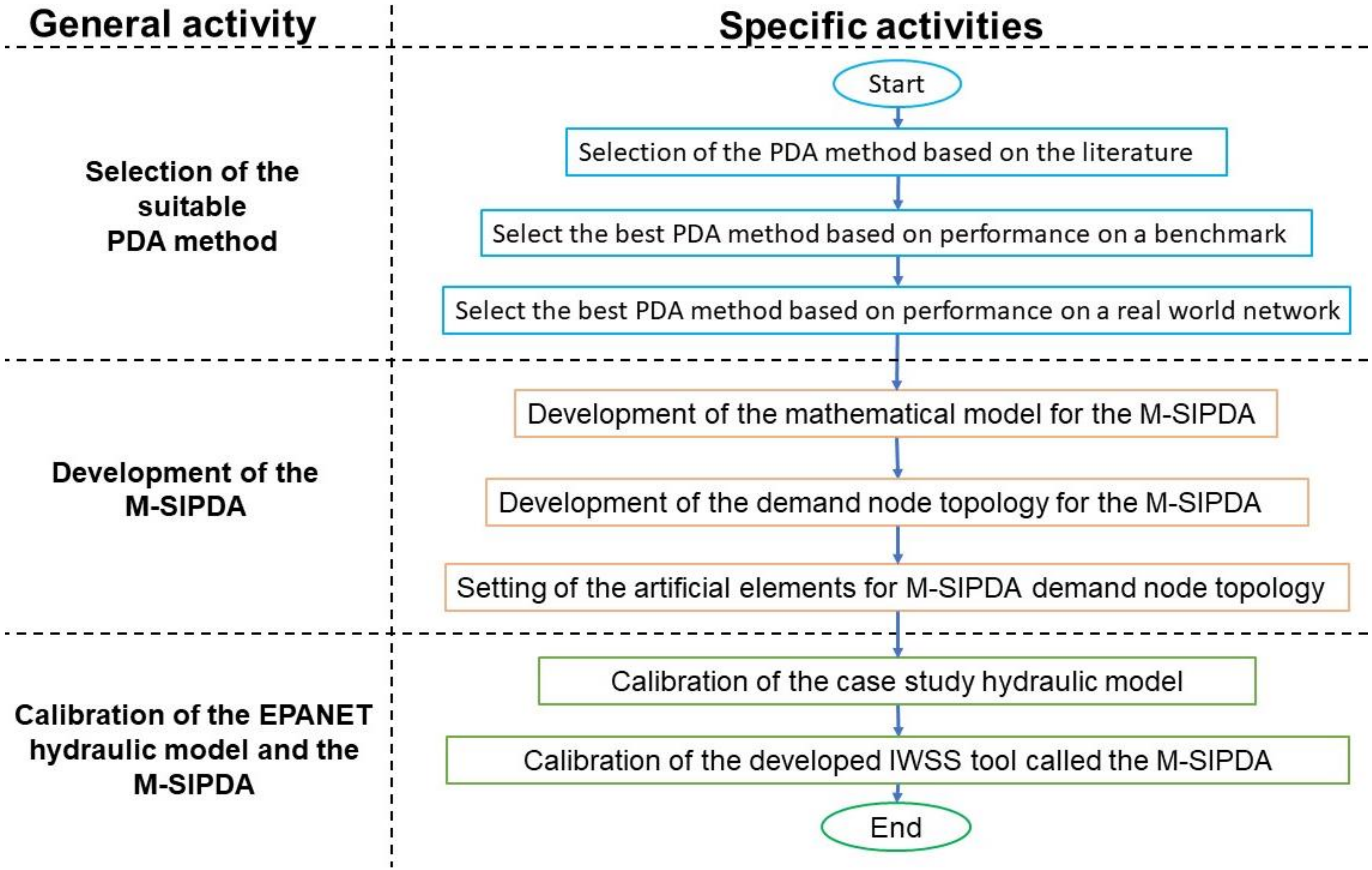


Figure 4.2: Methodology for developing the approach for modelling IWSS

4.3.1 *The selection of the PDA method*

The criteria for selecting the PDA method for this study are that it should be able to perform EPS, provides hydraulically realistic results, there should be adequate information to give clear guidelines on its application and there should be practical means of incorporating pressure dependent consumption demand and leakage.

From the three categories of PDA methods (discussed in Chapter 2), no method is selected from methods that are not linked to a hydraulic solver (EPANET) because most of them are for steady state simulations. For the few that can perform EPS, little information is available to enable their correct use. Limited information is also a limitation for the PDA approaches in the category of EPANET extension methods. Despite this limitation, the EPANETPDD (Morley and Tricarico, 2014) is selected due to the additional information provided by one of the authors. The information includes the EPANETPDD dynamic link library and the header files. PDA methods that modify the input files are subdivided into reservoir and emitter methods. No reservoir method is selected because they have one or more of the following limitations: provision of results that are not hydraulically realistic, used for steady state simulation only and the difficulty of incorporating consumption demand and leakage in the modelling process. From the emitter subdivision, the SIPDA (Mahmoud et al., 2017) is selected because it meets all the selection criteria. With the SIPDA, EPS can be performed and much of the needed information hinges on coding in MATLAB. Through the application of leakage mathematical models, a second emitter can be added to implement pressure dependent leakage outflows.

4.3.2 *Comparison of the selected PDA methods on a benchmark network*

In subsection 4.3.1, the methods are selected based on the literature narratives. In this subsection, the methods are evaluated based on their actual performance using the EPANET- MATLAB interface developed in this study. The benchmark network used is the C-town network (Figure 4.3) (Ostfeld et al., 2011).

For the SIPDA, both the normal and pressure deficient conditions are simulated as described in Mahmoud et al. (2017). The H_{req} is 15 m. The pressure deficient conditions are created by imposing a fire flow demand of 15 l/s on node N1

(Figure 4.3) as done in Mahmoud et al. (2017). The fire flow demand is from 12:00 to 21:00. The reporting timestep is set to 15 seconds. This is necessary to capture detailed variations in demand and pressure, especially under pressure deficient conditions. Node J278 is used for analysis because it has the lowest pressure head under pressure deficient conditions.

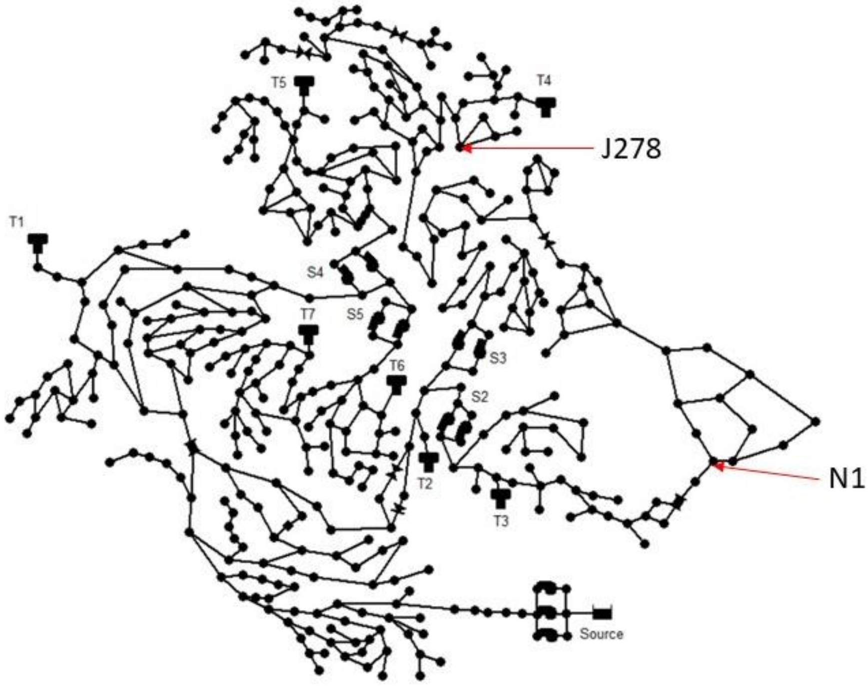


Figure 4.3: The C_Town Network (Ostfeld et al., 2011)

For the EPANETPDD method or tool, the EPANET input file is first modified in Microsoft Excel according to the guidelines given in Morley and Tricarico (2014). Two key modifications are done. The first is to insert the new [PDD] section on top of the input file before the [JUNCTIONS] section (Figure 4.4a). Under the added [PDD] section, the type of the mathematical model used is specified. Three general options are given as WAGNER, TUCCIARELLI and FUJIWARA. The option that includes leakage is the BBLAWN which stands for the Battle of Background Leakage Assessment for Water Networks. The second modification is the insertion of the [PDD_JUNCTION] section at the end of the entries for the [JUNCTIONS] section (Figure 4.4b). This section provides two selection options of demand nodes for PDA implementation. The first is the selection of pressure deficient demand nodes only and the second option is where all demand nodes are selected. For each demand node that is selected for PDA implementation, the value of the network required pressure (H_{req}) is put uder the heading $P_{critical}$

and the minimum pressure (H_m) under the heading P$_{minimum}$. In Figure 4.4b (as is the case in this study), all the demand nodes are selected for PDA implementation.

```
[TITLE]

[PDD]
;Options     Value
TYPE         WAGNER

[JUNCTIONS]
;ID                  Elev            Demand          Pattern
 J511                105.08          1.76            DMA2_pat            ;
 J411                8.95            1.33            DMA1_pat            ;
 J414                34.36           1.45            DMA1_pat            ;
 J415                65              0                                   ;
 J416                42.6            0                                   ;
 J417                37.38           1.94            DMA1_pat            ;
```

(a)

```
[PDD_JUNCTIONS]
;ID                 Pcritical    Pminimum
 J511               15           0                  ;
 J411               15           0                  ;
 J414               15           0                  ;
 J417               15           0                  ;
```

(b)

Figure 4.4: Modifications of the input file when using the EPANETPDD

Coding for the single-iteration pressure driven analysis

The SIPDA coding is done as described in Mahmoud et al. (2017). However, rather than adding artificial elements to pressure deficient nodes only, they are added to all demand nodes. The sequence of artificial elements connected to a demand node (DN) are the artificial check valve (ACV), artificial node (AN), artificial flow control valve (AFCV) and artificial consumption demand emitter (ACDE) (Figure 4.5). The elevation of the AN and the ACDE are equal to the elevation of the DN plus H_m. The base demand on the DN is set to zero. The base demand is used in the calculation of the emitter coefficient for ACDE which ensures that the amount of water supplied is dependent on pressure. The ACV is used to prevent reverse flow from the ACDE to the DN when $(H_j < H_m)$. The AFCV ensures that the maximum flow to the ACDE does not exceed the demand and its flow setting is equal to the actual nodal consumption demand. For EPS, maximum allowable flow through the AFCV for each time step is equal to the product of the consumption base demand for the corresponding DN and the demand pattern multiplication factor. The artificial valves are made to have negligible head losses across them by setting them with very large diameters (1000 mm as used in Mahmoud et al., 2017). The ACDE implements the pressure-depended node outflow using the Wagner et al. (1988) model as described in Equation 4.3.

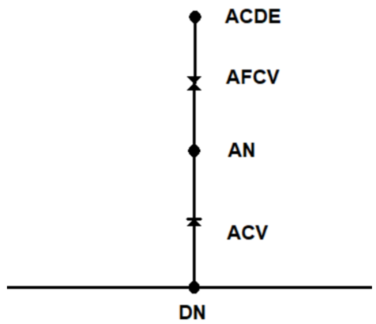


Figure 4.5: Connection of artificial elements to each demand node DN (Adapted from: Mahmoud et al., 2017)

For the normal operation condition, the 24 hours demand and pressure variations for node J278 are shown in Figure 4.6. The figure shows the simulation results for a DDA software (EPANET) and the two selected PDA approaches are basically identical. This is also shown in Table 4.1 which shows practically no difference between the two PDA methods. The similar performance of all the methods (DDA and the two PDAs) is because under normal operation, the C-Town network is a CWS system with adequate pressure. For such systems, PDA methods that are correctly developed and DDA approaches are expected to produce identical results. This is reflected in Table 4.1 which shows that the differences are practically negligible. In Table 4.1, the demand difference at time t ($Demand\ diff_t$) is calculated using:

$$Demand\ diff_t = PDA\_sim_{Demandt} - EPANET\_sim_{Demandt} \tag{4.8}$$

Where $PDA\_sim_{Demandt}$ is the node outflow simulated by PDAs (EPANET PDD or SIPDA) and $EPANET\_sim_{Demandt}$ the node outflow simulated by EPANET.

The demand difference mean at time t ($Demand\ diff\ mean_t$) is determined by:

$$Demand\ diff\ mean_t = \frac{1}{T}\sum_{t=1}^{T} Demand\ diff_t \qquad (4.9)$$

Where T is the total EPS duration.

The standard deviation at time t (D$emand\ diff_t\ std$) is calculated using:

$$Demand\ diff_t\ std = \left(\frac{\sum(Demand\ diff_t - Demand\ diff\ mean_t)^2}{N-1}\right)^{\frac{1}{2}} \qquad (4.10)$$

Where N is the total number of node outflow values for node J278.

Other than the hydraulic performance, the two PDA methods were compared with respect to simulation running time. For the complete 24 hours EPS, the EPANETPDD and SIPDA took 5.23 and 4.38 seconds respectively.

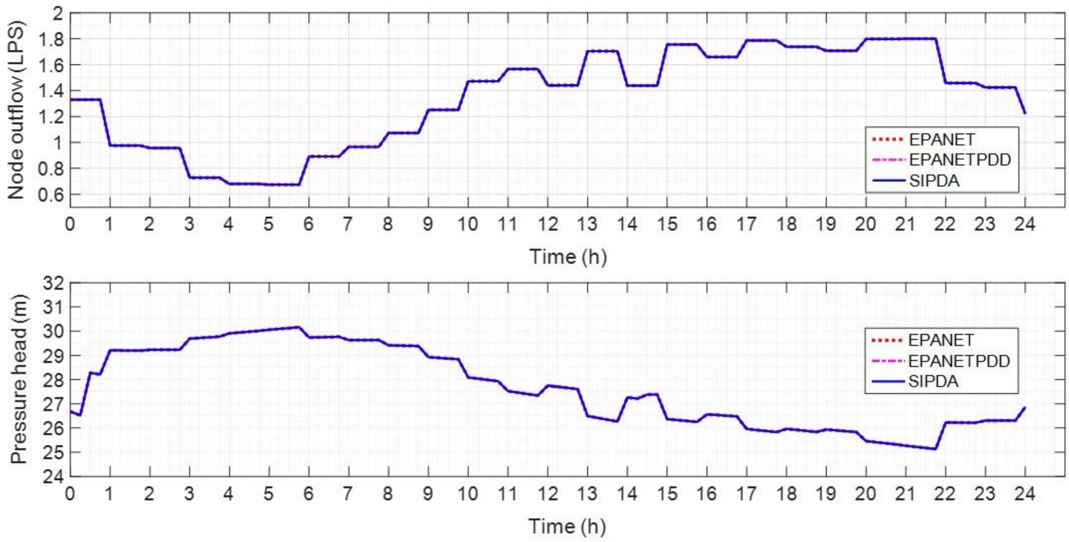


Figure 4.6: Demand (node outflow) and pressure variations for C_Town node J278 under normal operation

Table 4.1: Comparison of PDA methods with respect to EPANET simulation results

Statistics	Method	
	EPANET PDD	SIPDA
Mean of water demand difference (LPS)	$2.29*10^{-8}$	$1.64*10^{-5}$
Standard deviation of demand difference (LPS)	$1.76*10^{-8}$	$1.47*10^{-6}$
Mean of pressure difference (m)	$1.96*10^{-4}$	$4.66*10^{-4}$
Standard deviation of pressure difference (m)	$1.39*10^{-4}$	$1.85*10^{-4}$

When the network is subjected to excessive fire flow demand, its hydraulic capacity becomes insufficient and some nodes experience pressure heads that are lower than the required 15 m (Figure 4.7). Identical nodes are identified by all the methods as having pressure deficiency problems at 20:30 (Figure 4.7) although usually EPANET, as a DDA based software, tends to show more deficient nodes than PDA methods. This is because DDA methods assume that demand is fully met even where pressure is deficient which results in higher pipe flows and higher friction head losses than those for PDA methods (Hayuti et al., 2007; Mahmoud et al., 2017).

The fact that EPANET shows full demand supply even when the pressure cannot guarantee it is shown in Figure 4.8. Since when there is pressure deficiency, the node outflow (node supplied demand) is less than the amount of water demanded, the term node outflow, rather than demand, is used in Figure 4.8. The figure shows that the pressure deficiency conditions for node J278 simulated by the SIPDA start earlier (18:30) than for EPANET and EPANETPDD (18:45), but last up to 22:00 for all the three methods. For all the methods, the minimum pressure for node J278 is between 20:00 and 21:00, but the EPANET minimum pressure values are lower than those for the PDA methods because, as earlier stated, during pressure deficient conditions, EPANET simulations have higher flows and friction head losses than PDA methods.

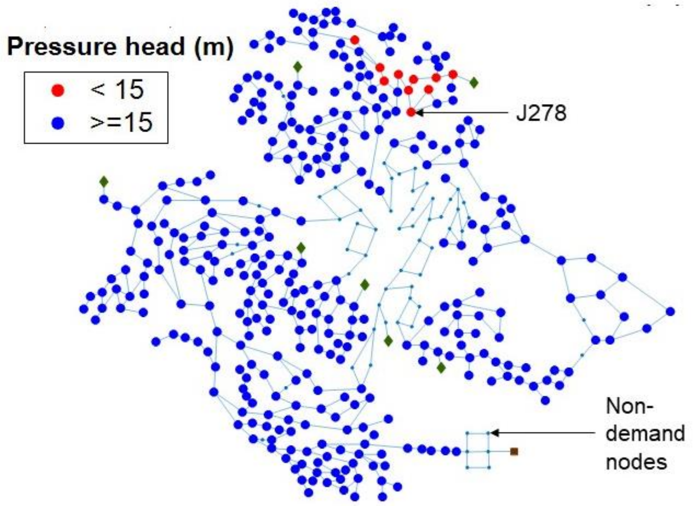


Figure 4.7: Nodes with and without pressure deficiency at 20:30 for EPANET, EPANETPDD and SIPDA

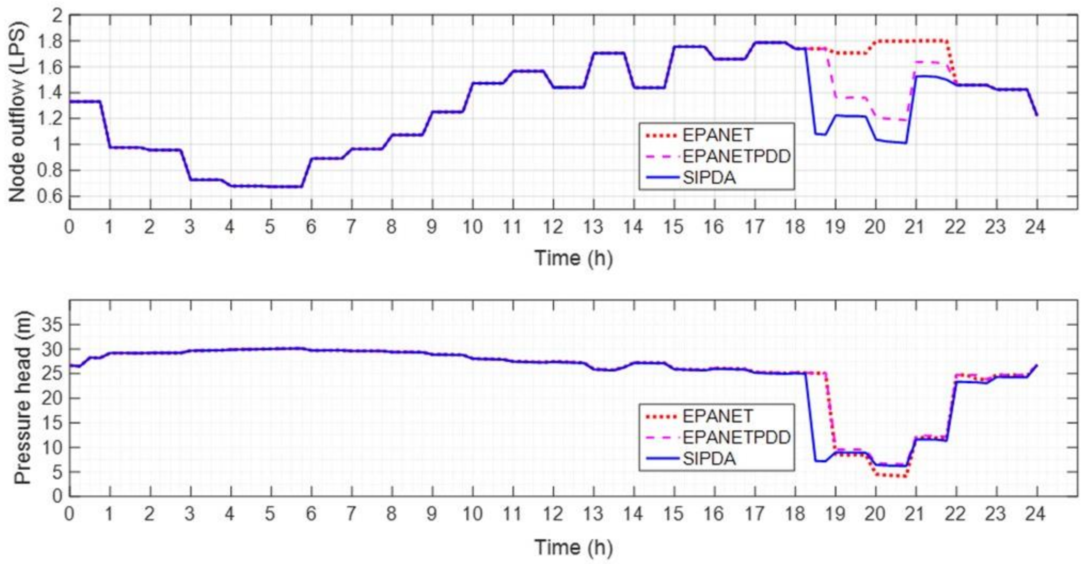


Figure 4.8: Node outflow and pressure for C_Town node J278 under pressure deficient conditions

A comparison of some statistics for the simulation results by the SIPDA and the EPANETPDD is shown in Table 4.2. The slight differences in the average water supply pressure are mainly because of the differences in the low pressure duration, ranging from about 18:45 to 22:00. However, from the statistics in Table 4.1 and Table 4.2, it is not possible to tell which of the two methods is performing better than the other. Even the difference in simulation durations is very small. To be conclusive, the methods are subjected to a large real-world network.

Table 4.2: Comparison of the simulation results by EPANETPDD and SIPDA

Statistics	Method	
	EPANET PDD	**SIPDA**
Supply range (l/s)	0.67 - 1.79	0.67 - 1.79
Supply mean (l/s)	1.30	1.28
Supply standard deviation (l/s)	0.34	0.34
Pressure range (m)	6.56 - 30.16	6.68 - 30.15
Pressure mean (m)	25.41	25.15
Pressure standard deviation (m)	6.28	6.54

4.3.3 *Comparison of the PDA methods on a large real network*

The large network used is the 2010 LWSC network reported in Brian Colquhoun Hugh O'Donnell and Partners (2010) (described in detail in Chapter 5). The setting procedures for the EPANETPDD and the SIPDA are as described in subsection 4.3.2 except for the H_{req} which is set to 7 m; the value expected to guarantee full demand supply in Zambia (NWASCO, 2014a). The reporting time step is reduced to 10 minutes to increase the accuracy of approximating the simulated values at times that are not multiples of 10 minutes, but which are the times when field data measurements were taken and used to report the WaterGEMS simulated results (Brian Colquhoun Hugh O'Donnell and Partners, 2010). Table 4.3 shows the first few entries for the field measured pipe flow data, reported WaterGEMS simulation results, simulated results for EPANET, SIPDA and EPNETPDD. The complete Table is presented in the appendix (Table A.1).

The WaterGEMS results are for comparing the accuracy of the pipe flows simulated by the EPANETPDD and the SIPDA. This is because the EPANET model used with the EPANETPDD and SIPDA is converted from the WaterGEMS model, without calibration at this stage. The EPANET simulation results have also been included (Figure 4.9) to help explain if any discrepancies could be due to the conversion process. Calibration of the converted EPANET model is not necessary at this stage because if it is to be calibrated, it has to be with respect to both the EPANETPDD and the SIPDA before either is selected.

Other than pipe 18 where the EPANET simulated flow is in the opposite direction to these simulated by the WaterGEMS and SIPDA, in many other cases, the EPANET simulated results are close to those by these two modelling approaches. The EPANETPDD, has many discrepancies. Its simulated flows have notable opposite flows to the results of the other modelling approaches in pipes 17, 18, 20, 28 and 72. Where flows are in the same direction, the Figure shows several pipes for which the simulated results by the EPANETPDD significantly deviate from the other 3 approaches. The biggest discrepancies are in pipes 53 and 60 where the EPANETPDD results show no flows while the other approaches show flows of more than 100 m^3/h and 900 m^3/h respectively.

Table 4.3: Field flow measurements data, simulated results for WaterGEMS, EPANET, EPANETPDD and SIPDA

Pipe ID	**Time of measurement (h)**	**Field data (m^3/h)**	**WaterGEMS (m^3/h)**	**EPANET (m^3/h)**	**EPANETPDD (m^3/h)**	**SIPDA (m^3/h)**
1	14.12	116.65	116.72	116.72	116.72	116.72
2	12.58	19.65	19.62	19.62	19.62	19.62
3	12.88	100.15	100.2	100.20	100.21	100.21
4	12.48	49.02	48.96	48.96	8.16	48.96
5	12.50	42.02	42.14	42.14	42.92	42.15
6	12.33	53.03	53.11	53.60	25.49	53.61
7	15.75	156.87	152.82	151.07	164.32	151.03
8	12.37	80.5	80.58	80.58	64.33	79.85
53	12.72	121.3	121.43	121.43	0.00	121.43
60	13.00	974	971.81	959.00	0.00	958.97

Another aspect used to compare the performance of the two modelling approaches is the simulation run time. The 24 hours EPS for the SIPDA takes about 12 minutes while that for the EPANETPDD takes about 58 minutes. These are significant duration differences.

Since the SIPDA is more accurate on the large network and it takes shorter duration to complete the 24 hours EPS than the EPANETPDD, it is selected as the PDA method used to develop the method for modelling IWSS.

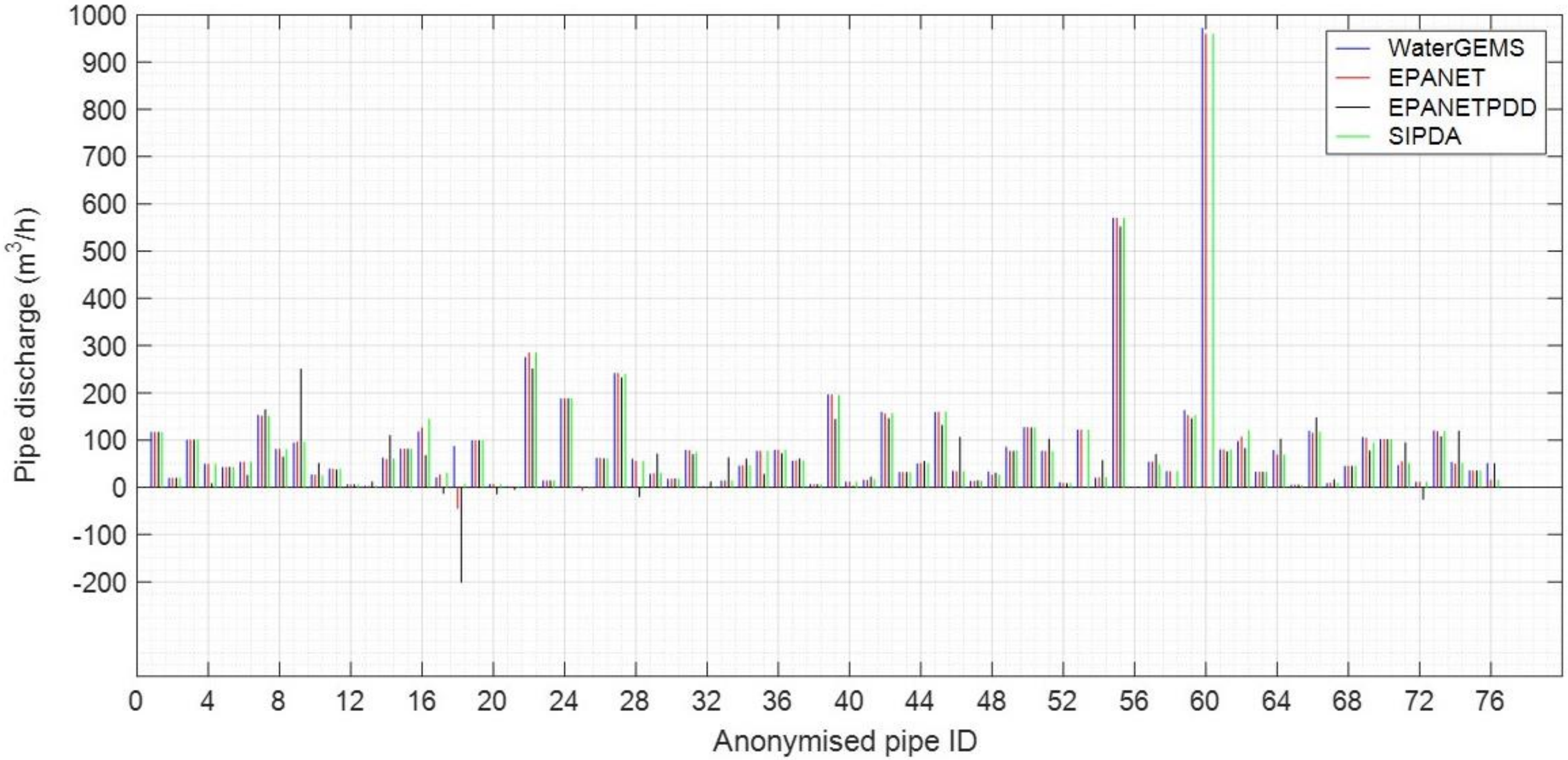


Figure 4.9: Pipe flows simulated by four modelling approaches

4.3.4 *The development of the Modified Single-Iteration Pressure Driven Analysis (M-SIPDA)*

As discussed in subsection 4.3.1, the SIPDA was developed for modelling CWSS with deficient pressure rather than IWSS. The few methods that have reference to modelling IWSS have major limitations (Chapter 2). Because of the above reasons, this work proposes a new PDA approach for modelling IWSS. The method, which is based on the emitter methods, considers the similarities and differences between consumption water demand and leakage, takes into account the multiple water supply schedules, and provides for modelling of pressure dependent demand and leakage which is normally very high in these systems and cannot be ignored. The method also incorporates a technique for developing pressure distribution maps which makes visualisation of the pressure in the water supply system and the effects of any modification to the system easier.

The proposed new PDA approach for modelling IWSS builds on the SIPDA (Mahmoud et al. 2017) and it is called the Modified SIPDA (M-SIPDA). The three major steps in developing the M-SIPDA are the development of the mathematical model that considers consumption demand as well as leakage, development of the M-SIPDA topology including the setting of its main elements and the calibration of the M-SIPDA.

Development of the mathematical model of M-SIPDA

The mathematical model considers the similarities and differences between water supplied for consumption demand and leakage. The main similarities are that both quantities are pressure dependent regardless of the WSS and that when pressure at a node is zero, both quantities are zero. There are two major differences between them. The first is that leakage increases with pressure without bound while it is assumed that consumption demand water outflow does not increase beyond Q_j^{req} which is met when $H_j = H_{req}$ (Giustolisi and Walski 2012) as depicted in Figure 4.10. The second is based on the consumption demand – pressure – leakage relationship in which leakage is directly influenced by how much water is withdrawn from the nodes to meet consumption demand while consumption demand is not directly influenced by leakage. These differences require that nodal consumption demand and leakage outflows are modelled explicitly differently. This requires that the consumption base demand (Bd_{Dj}) and the leakage base demand (Bd_{Lj}) are determined separately.

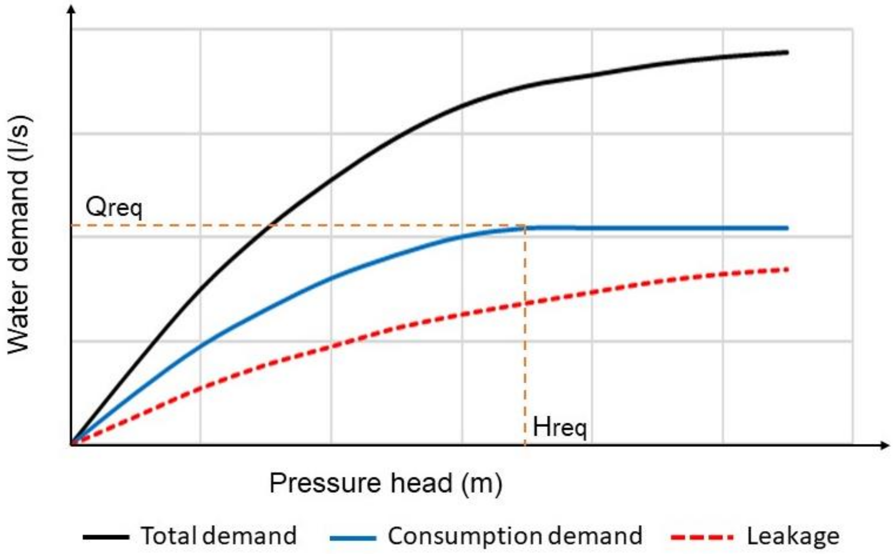


Figure 4.10: The behaviour of consumption demand and leakage in relation to pressure head (Simukonda et al., 2019)

In the M-SIPDA, the Bd_{Dj} and Bd_{Lj} are calculated respectively by:

$$Bd_{Dj} = \left(\frac{Q_{DMA_{Billed}}}{NDMAdn}\right) \tag{4.11}$$

Where $Q_{DMA_{Billed}}$ is the billed water consumption approximated from both metered and fixed charge billing records and $NDMAdn$ is the number of demand nodes in a DMA.

$$Bd_{Lj} = \left(\frac{Q_{DMAtotal} - Q_{DMA_{Billed}}}{NDMAdn}\right) f \tag{4.12}$$

Where $Q_{DMAtotal}$ is the total DMA water demand

In Equation 4.12, it is assumed that NRW is dominantly leakage such that from the law of conservation of mass, the DMA system input volume is equal to the $Q_{DMAtotal}$ which is the sum of leakage and consumption demand as implied in Skipworth et al. (2002), Butler (2004) and Farmani et al. (2012). Bd_{Lj} is only computed when there is water supply to the DMA containing node *j* since that is the only time leakage can occur. At that time, $f = 1$; otherwise $f = 0$.

The proposed model describes the total water outflow at node *j* (Q_{totalj}) for the three pressure conditions at time *t* by combining Equation 4.3 and Equation 4.7 as:

$$Q_{totalj} = \begin{cases} Q_j^{req} + \left(\dfrac{Bd_{Lj}}{H_{aveDMA}{}^{\alpha}}\right) H_j{}^{\alpha} & for\ H_j \geq H_{req} \\ Q_j^{req} \left(\dfrac{H_j - H_m}{H_{req} - H_m}\right)^{\gamma} + \left(\dfrac{Bd_{Lj}}{H_{aveDMA}{}^{\alpha}}\right) H_j{}^{\alpha} & for\ H_m < H_j < H_{req} \\ 0 & for\ H_j \leq H_m \end{cases} \tag{4.13}$$

In this formulation, the value of γ is kept constant because it is calculated using the constant *n* = 1.54 as used in Mahmoud et al. (2017). The value for α is determined through calibration which is discussed later in the chapter.

Development of the M-SIPDA topology and the setting of its main elements

The M-SIPDA is developed by the addition of the artificial elements to the demand node (DN) to transform the SIPDA (Figure 4.11a) to the M-SIPDA topology (Figure 4.11b) using the developed EPANET- MATLAB interface code. The elements that make the M-SIPDA different from the SIPDA are the second artificial check valve (ACV2) and the artificial leakage emitter (ALE). The ACV2 is meant to prevent backflows from ALE to the artificial node (AN) in case of higher hydraulic heads at the former. The ALE is used for implementing the pressure dependent nodal leakage outflow which is not considered in the SIPDA. Besides the AN, ACV2 and the ALE, other artificial elements include the artificial check valve (ACV), the artificial flow control valve (AFCV) and the artificial consumption demand emitter (ACDE) which are common artificial elements for both methods (Figure 4.11). The ACV is meant to prevent backflows from the ACDE to the DN in cases when $H_j < H_m$. The AN is used to connect the DN, ACDE and ALE through the ACV, AFCV and the ACV2 respectively (Figure

4.11b). The diameter for the ACV, ACV2 and the AFCV are very large (1000 mm as used in Mahmoud et al. 2017) so that the head loss across them is negligible.

In this study, the minimum pressure head (H_m) is assumed to be zero as such Equation 4.13 is written in terms of the coefficients for the artificial consumption demand emitter (ACDE) and artificial leakage emitter (ALE) as:

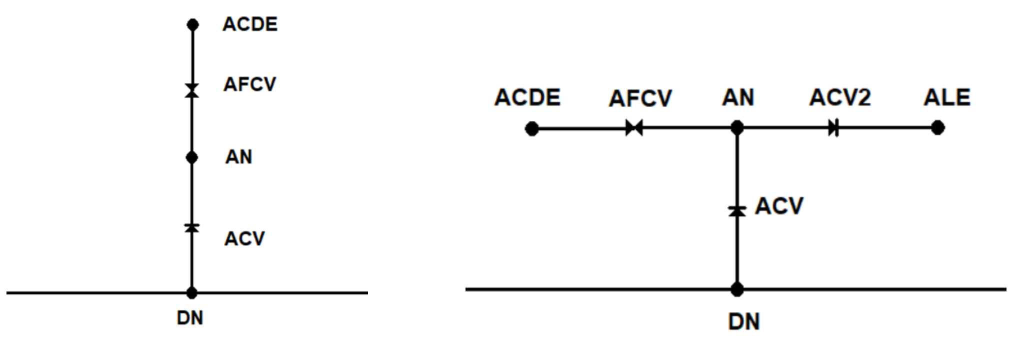
$$Q_{totalj} = \begin{cases} Q_j^{req} + El_{coef} * H_j{}^{\alpha} & for\ H_j \geq H_{req} \\ E_{coef} * H_j{}^{\gamma} + El_{coef} * H_j{}^{\alpha} & for\ 0 < H_j < H_{req} \\ 0 & for\ H_j \leq 0 \end{cases} \quad (4.14)$$

Where E_{coef} is the ACDE coefficient defined by Equation 4.15 and El_{coef} is the ALE coefficient defined by Equation 4.16.

$$E_{coef} = \frac{Q_j^{req}}{H_{req}{}^{\gamma}} \quad (4.15)$$

$$El_{coef} = \frac{Bd_{Lj}}{H_{aveDMA}{}^{\alpha}} \quad (4.16)$$

The maximum flow through the AFCV is set to be equal to Q_j^{req} to ensure that the flow (Q_j^d) does not exceed the required flow. The parameter settings for the ACDE are the elevation which is equal to the DN elevation (EL), the value of *n* and the value of E_{coef}. For the ALE, the parameter settings are the EL, the El_{coef} and the α.



(a) (b)

Figure 4.11: The DN and the artificial elements for (a) SIPDA and (b) M-SIPDA

Both the initial and the EPS dynamic settings for the M-SIPDA's ACDE and ALE are based on Equation 4.14. Since DMAs have different water supply schedules, the setting of the AFCV and the application of Equation 4.14 (with Bd_{Lj} determined by Equation 4.12) takes care of the multiple supply schedules of IWSS. This is because when a DMA is supplied water, Q_j^{req} > 0 and E_{coef} > 0 since $dmfactor$ > 0. When the DMA is not supplied water $dmfactor$ = 0 and Q_j^{req} = E_{coef} = 0. For leakage, the changes of Bd_{Lj} when there is water supply or not (Eq. 9) means $El_{coef} > 0$ when there is water supply and $El_{coef} = 0$ when there is no supply. The summary of the steps for these dynamic settings of the AFCV, ACDE and ALE is shown in Figure 4.12. For these settings, the input information comprises the $Q_{DMAtotal}$, $Q_{DMA_{Billed}}$, the determined DMA leakage level ($Q_{DMAtotal}$ - $Q_{DMA_{Billed}}$) and average pressure head. Figure 4.12 also shows that the simulation run at each time step t produces results that form vectors, which when combined for all the time steps up to t =T form matrices of results. The elements in the matrices are used in the automated post-processing of the simulated DMA supplied consumption demand and leakage to determine which DMAs have water supply or leakage problems.

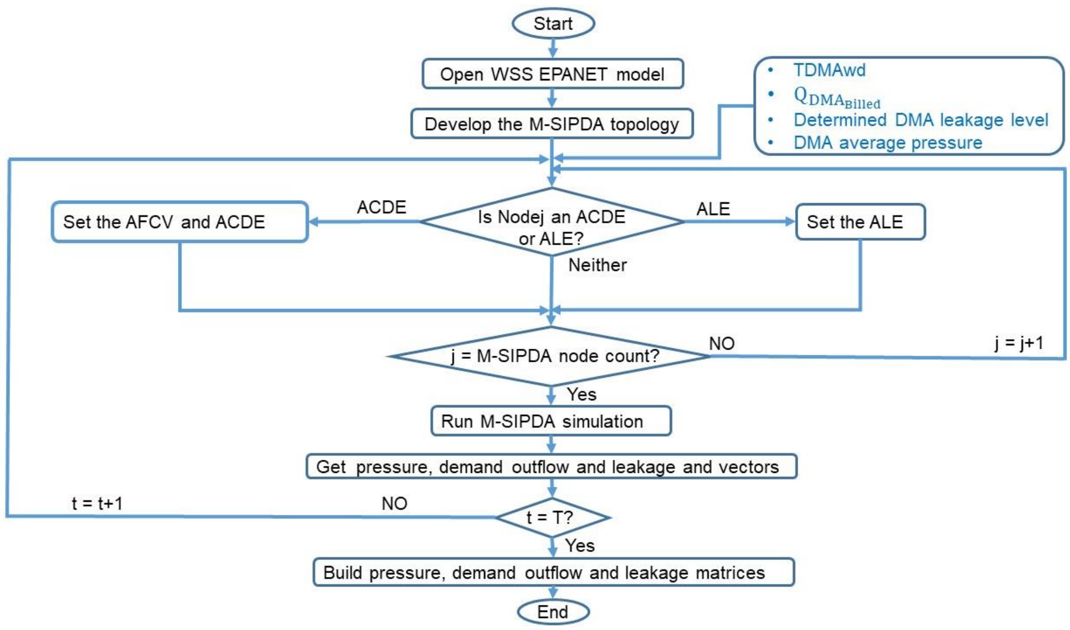


Figure 4.12: Dynamic setting of the AFCV, the ACDE and the ALE during EPS

4.4 Calibration of a hydraulic model

This section discusses the significance of calibration in the modelling process and highlights how most of the PDA models have been calibrated. It provides information and steps that guide the calibration of the EPANET hydraulic model for the case study network and the M-SIPDA (Chapter 7). Through the information provided in this section, the quality of the available calibration data is evaluated, the suitable calibration method is selected and the quality of the final level of calibration for both the EPANET hydraulic model and the M-SIPDA is evaluated.

Calibration of a model using field data is important because it improves the accuracy of the model's representation of the real system (Walski, 1986). However, most of the PDA models' simulation results are not compared with real field data for calibration (Hayuti et la., 2007). Their performance and calibration are evaluated using benchmark networks by comparing their simulation results with those of other earlier models as exemplified by Morley and Tricarico (2014). In these cases, significant factors that are adjusted during calibration are unique to the suggested PDA model factors, but the hydraulic solver parameters such as demand, pipe roughness, valve settings are not adjusted when trying to match the model predictions against those of the earlier models. In this study calibration of both the hydraulic model parameters and the proposed PDA model are vital, hence the discussion of the calibration process in detail.

Calibration, as part of the development process for hydraulic models (Figure 4.13), is a continuous but complex process because while it is applied to WDS model which are built on a scientific foundation, it is an art which heavily depends on experience (Ostfeld et al., 2011). It is a process that requires the use of *reliable* field measurement data taken over a range of operation conditions (Walski, 1983; Ormsbee and Wood, 1986; Savic and Walters, 1995; Ormsbee and Lingireddy, 2004; Ostfeld et al., 2011). Besides improving the accuracy of the model, calibration enables the identification of the model's strengths and weaknesses and improves the understanding of the WDS (AWWA Water Distribution Model Calibration Subcommittee, 2013).

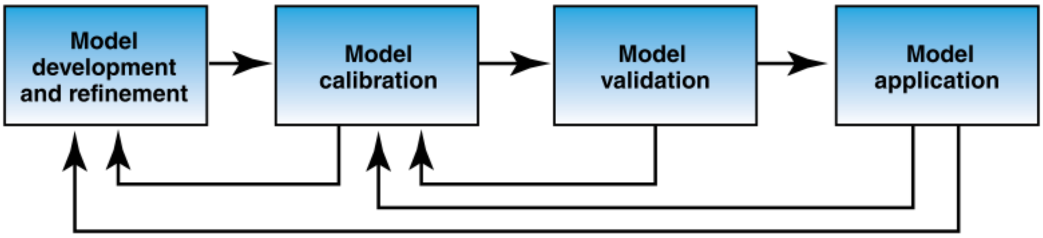


Figure 4.13: Basic steps in model development (AWWA Water Distribution Model Calibration Subcommittee, 2013)

4.4.1 *Aspects that complicate hydraulic model calibration*

The art of model calibration is complex for three reasons. The first is that there are many possible sources of errors that make it difficult to identify the exact causes of the hydraulic model – field data discrepancies and to select the correct parameters for adjustments in any particular instance (United States Environmental Protection Agency, 2005; Walski, 2017). Possible sources of errors include erroneous model parameters data, incorrect operational data such as nodal demand distribution and wrong pump controls, incorrect network topology such as the connection of pipes to wrong nodes, incorrect pressure zone boundary definitions, errors in boundary conditions (incorrect PRV value settings, tank water levels, pump curves), measurement equipment errors (pressure gages not properly calibrated, etc.), measurement errors such as reading the wrong values from measurement instruments and wrong measurement instrument elevations (Ormsbee and Lingireddy, 2004; Edwards et al., 2006; Walski, 2017). The large number of possible sources of errors presents the possibility for the occurrence of compensating errors in which one parameter is adjusted to calibrate the model and yet the poor simulation results are caused by a different parameter (Walski 1983, 2017).

The second reason is that there are no explicit guidelines on how much data is required for a successful calibration and what affects the quality of the data. A set of guidelines for the quantity of data for hydraulic model calibration according to the model intended use was drafted and published by the AWWA Engineering Computer Applications Committee in 1999 (United States Environmental Protection Agency, 2005) (Table 4.4), but there has been no final confirmation of these criteria. Moreover, the cost of measuring such amount of data is very high.

In terms of quality, data collected for calibration can be categorised as good, bad and useless data (Walski, 2000).

Good data

Good data is the data collected when the head loss between the source and the measurement location or between two measurement points is about five times the head loss measurement error (Walski, 2017). For transmission mains, this level of head loss can be achieved by taking measurements between two points that are far apart when there is peak demand or when tanks are low such that many pumps are running. For smaller pipes, fire hydrants are the usual way of increasing flow velocities and head loss (Walski, 2000; 2017).

Bad data

Bad data is associated with errors arising from factors such as incorrect reading of a pressure gauge values, incorrect elevation of the pressure gauge, or poor knowledge of pumps that are operating during the time when calibration data is being collected. When bad data is identified, it should be discarded (Walski, 2000).

Useless data

Useless data is collected when the velocities in the system are very low such that head losses are also low and are of the same order of magnitude as their measurement errors (Walski, 2000). It is difficult to distinguish between good and useless data and most of the data used for many system model calibrations are suspected to be useless (Walski, 2000). While useless data can be used for macro-calibration, it cannot be used for micro-calibration (Walski, 2000). Models calibrated using useless data may perform well under low demand (static) conditions but perform badly under stressed (high demand) conditions (Walski, 1983; 2000). For design purposes, a model should perform well under low and stressed conditions (Walski, 2000). This can be achieved by calibrating the model for several operating conditions which also eliminates compensating errors (Walski, 1983). The summary of the various aspects that need to be considered to ensure that good model calibration data is collected is presented in the appendix (Table A.2).

Table 4.4: The 1999 AWWA Engineering Computer Applications Committee draft criteria for hydraulic model calibration data collection and accuracy (United States Environmental Protection Agency, 2005)

Intended use	Level of detail	Type of simulation	Number of pressure readings	Accuracy of pressure readings	Number of flow readings	Accuracy of flow readings
Long-Range Planning	Low	Steady state or EPS	10% of Nodes	±3.5 m for 100% Readings	1% of Pipes	± 0%
Design	Moderate to High	Steady state or EPS	5% - 2% of Nodes	±1.4 m for 90% Readings	3% of Pipes	±5%
Operations	Low to High	Steady state or EPS	10% - 2% of Nodes	±1.4 m for 90% Readings	2% of Pipes	±5%
Water Quality	High	EPS	2% of Nodes	±2.1 m for 70% Readings	5% of Pipes	±2%

The third reason is that there is no standard level of calibration to be attained because the level of calibration or level of model accuracy to be attained depends on the expected use of the model and the type of simulation (EPS or steady state), the cost the model user is ready to sustain for calibration which is a continuous process because WDSs are very dynamic in terms of operations and size (AWWA Water Distribution Model Calibration Subcommittee, 2013), and the performance of the model (Edwards et al., 2006). A well calibrated model may still have areas where it does not do well, but the utility may not have the resources to continue with further calibration such that the model is still useful provided its limitations are known. In this regard, even uncalibrated models can provide some useful insight for decision making (AWWA Water Distribution Model Calibration Subcommittee, 2013).

Concerning the simulation type, there exists the level of calibration guidelines for steady state simulations provided by the Sewers and Water Mains Committee of the Water Authorities in the United Kingdom in 1999 as follows (United States Environmental Protection Agency, 2005):

1. Flows should agree to:
    a. 5% of measured flow when flows are more than 10% of total demand (Transmission lines).
    b. 10% of measured flow when flows are less than 10% of total demand
       (Distribution lines).
2. Pressure should agree to:
    a. 0.5 m or 5% of head loss for 85% of test measurements.
    b. 0.75 m or 7.5% of head loss for 95% of test measurements.
    c. 2 m or 15% of head loss for 100% of test measurements

For EPS calibration, the level of the model accuracy for different intended uses is set out in Table 4.4. It is noteworthy that in certain instances, due to the type and quality of data available or due to the calibration method used, calibration may not meet the above accuracies.

4.4.2 *The seven basic steps for the calibration process of a hydraulic model*

To make the calibration process systematic, seven basic steps are followed (Ormsbee and Lingireddy, 2004):

i. Identification of the intended use of the model
ii. Estimation of initial parameter values
iii. Collection of calibration data
iv. Evaluation of the model results
v. Performing macro-level calibration
vi. Performing sensitive analysis
vii. Performing micro-level calibration

Intended use of the model

The first step before starting to develop a model is the identification of the need for a model and its intended short and long term application (Walski et al., 2003). The intended uses of the hydraulic model which have influence on the data collected for calibration and accuracy of the calibrated model are discussed in Walski et al. (2003), Ormsbee and Lingireddy (2004) and are shown in Table 4.4.

Estimation of initial model parameter values

In the development of a WSS hydraulic model, data on pipes, valves, pumps, junctions, tanks and reservoirs is collected for parameters as shown in Table 4.5.

Table 4.5: WSS elements and parameters

WSS elements	WSS hydraulic model elements	Parameters
Pipes	Links	Pipe identification, length, start and end nodes, diameter, roughness.
Pumps	Links (or node depending on software used)	Pump identification, start and end nodes, pump capacity (characteristic curves or power)
Valves	Links	Valve identification, diameter, start and end node, valve type, valve status, valve setting
Junctions	Nodes	Junction identification, location, elevation
Tanks	Nodes	Tank identification, location, elevation, tank type (underground, ground, elevated), dimensions, inlet and outlet elevation, minimum and maximum water levels
Reservoirs (sources)	Nodes	Reservoir identification, location, elevation

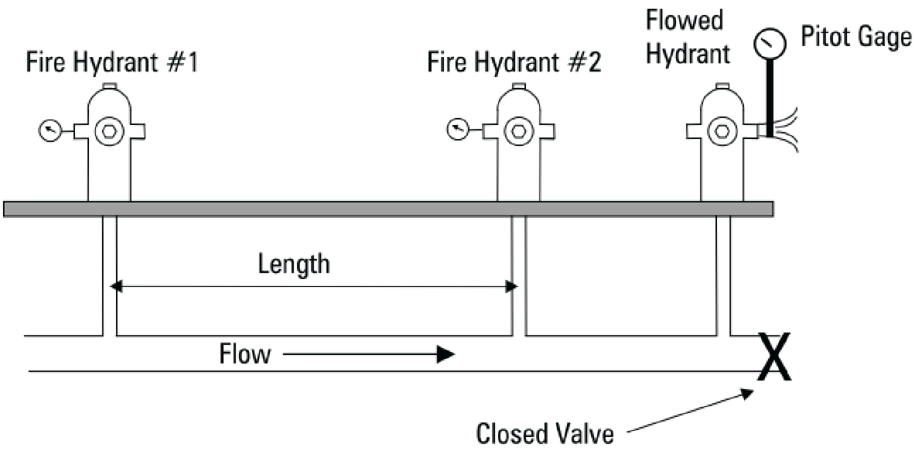
Besides the data on the physical WSS elements, demand and operation data is also acquired and applied to the model at demand nodes and through controls (simple or rule-based) respectively. In most cases, the parameters' data, demand and operation data are estimations characterised by numerous uncertainties. Besides, the mathematical equations used in the models are approximations rather than exact representations of reality. Moreover, because data collection is costly, not all parameters are measured directly (Lansey and Basnet, 1991; Savic and Walters, 1995). Data for state variables (nodal pressure head, pipe flows, nodal demands and tank water levels) is the source of information that is used to determine better values of WSS parameters (Lansey and Basnet, 1991). Because of these factors, model simulation results usually differ considerably with the real system output (United States Environmental Protection Agency, 2005).

Collection of calibration data

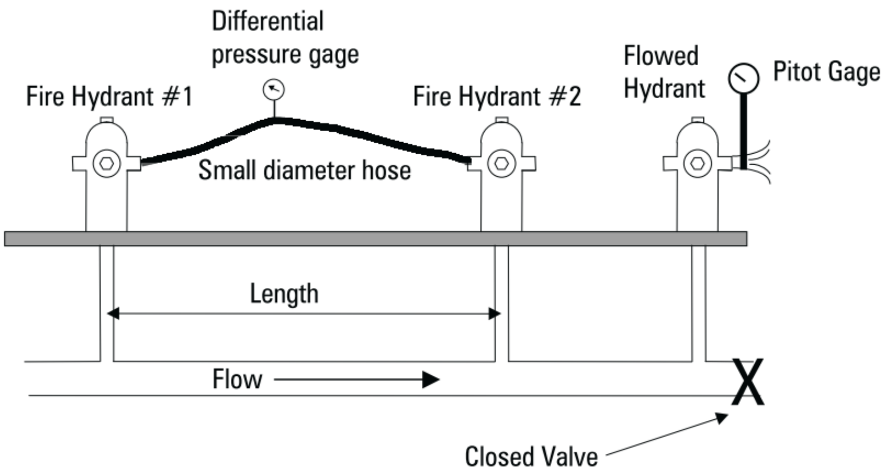
Data for model calibration is collected according to the types of simulation for the model which include steady state (static), extended period (dynamic) and water quality simulations. Ideally, calibration data is collected within a short enough period to ensure that boundary conditions are practically constant (Walski, 1986). In general, the four types of data that are important for calibration which can be obtained from field measurements or network telemetry systems are nodal pressure heads, nodal demands, pipe flows and tank water levels. Model calibration can be done according to the application of the model such as steady state simulation and EPS.

For steady state simulation calibration, the most common approach for data collection is the hydrant fire-flow test using the double–gauge method (Figure 4.14a) or the parallel pipes method (Figure 4.14b). Data collected for steady state simulations can be for low flow or high flow conditions.

When low flow conditions are considered, the flow in the WSS is determined by normal water demands (no hydrant flow) and as a result, the velocities are low and friction head losses are also low. The pressure head difference measured by gauges at hydrant 1 and 2 is small. Under these conditions, the hydraulic grade lines (HGL) for the system are expected to be flat and the model results are expected to match with field data otherwise some of the possible sources of the disagreement include wrong tank water level data, differences between model and actual pressure reducing valve (PRV) settings or reference elevations, Interconnections between pressure zones that are shown to be closed in the model and different demands between those at the time of measurements and the ones entered in the model (Walski, 2017).



(a)



(b)

Figure 4.14: Hydrant fire flow test configuration for (a) the double gauge and (b) parallel pipes methods (United States Environmental Protection Agency, 2005)

When the WSS is stressed due to high flows induced by opening the flowed hydrant, the head loss between the pressure measurement gauges 1 and 2 increases (Walski, 2017). The high head loss is important to reveal discrepancies that cannot be revealed under low flow conditions. If there are discrepancies between the model and field data under high flow conditions, the most probable sources include (Walski, 2017):

- Closed valves

- Pipe roughness
- Demand locations
- Demand magnitude
- Network connectivity
- Pipe diameter

For instance, closed valves usually lead to differences between measured and modelled hydraulic head values which only occur in a few locations while incorrect pipe roughness factors or demands manifest in uniformly distributed errors across a pressure zone (Walski, 2017). In the literature, roughness factors and demand are known to be the two factors that are with the highest degree of uncertainty in the development of WSS hydraulic models (Bhave 1988; Ormsbee, 1989; Lansey et al., 2001; Ormsbee and Lingireddy, 2004). Thus, micro-level calibration normally involves adjusting either the pipe roughness factors or nodal demand, or both till the model simulated pressure and flows match, within set accuracy, with the corresponding field measured values. Demands can be verified by conducting water balance analysis for a concerned zone. The number of roughness factors can be reduced by performing the C-factor test on some pipes or a group of pipes. According to Walski (2017), systems with mostly plastic or lined pipes have hydraulic heads that have very low sensitivity to roughness adjustments under normal circumstances.

EPS follows steady state calibration. EPS involves temporal variations in demand and pump settings that lead to variations in pipe flows and pressure. There are two types of EPS calibration. The first considers specific application conditions. In this case, the major system state variable considered is water level in tanks. The fluctuations of tank water levels measured in the field are compared with the model predictions. Walski (2017) gives two kinds of discrepancies that may occur which should be treated separately. The first one is when the pattern of tank water level as measured in the field is changing differently from the pattern predicted by the model. This is due to pump control problems and can be removed by controlling pumps using time-based controls. The second is when tank water levels predicted by the model are moving in the same direction as those measured in the field but have different slopes. This problem is due to wrong demand patterns and can be resolved by considering flow (not pressure) data especially from flow meters located at the entrance to the DMA.

The limitation of the time-based controlled model is that it can only be reliably used on the day for which it is calibrated. For the model to be used on days other than when it is calibrated, rule-based controls are used (Walski, 2017). Even in this case, variations in tank water levels, pipe flows, or nodal pressures over a long period for field measurements and model predictions are compared. Walski (2017) gives two types of discrepancies that arise with this calibration process. The first is the time shift discrepancies which are mostly due to small errors in demand patterns and control settings. Even though this type of errors is difficult to solve, their effect on the usefulness of the model is negligible. The second is the magnitude errors which have serious implications on the usefulness of the model and efforts should be made to identify the source(s) of these errors (Walski, 2017). If the model is to be used for water quality analysis, water quality calibration follows hydraulic EPS calibration.

Macro-level calibration

When the simulated flow or pressure results differ from the measured data by a margin that is greater than 30 %, there is the likelihood that the cause of the difference may not be due to poor estimation of either the pipe roughness values or the nodal demands. The possible causes for the difference include closed or partially closed valves, inaccurate pump curves or tank levels monitoring data, incorrect pipe sizes, incorrect pipe lengths, incorrect network geometry and incorrect pressure zone boundaries (Ormsbee and Lingireddy, 2004). Thus, before any model adjustment are made, it is vital to verify field data because in certain instances, it is the field data that may be erroneous rather than the model (Walski, 2000; 2017). Field data verification helps to reduce the number of sources of errors which is very vital in reducing the number of parameters to adjust during micro-level calibration (Ormsbee and Lingireddy, 2004). Consequently, the extent of success in this respect affects the effectiveness of micro-level calibration techniques (Ormsbee and Lingireddy, 2004; Ostfeld et al., 2011).

Sensitivity analysis

Even though the model simulated results and field measured values are in reasonable agreement after macro-level calibration, the differences are usually still significant and there are many possible causes of these differences. To identify the most likely sources of model errors, model sensitivity analysis is performed (Ormsbee and Lingireddy, 2004; Ostfeld et al., 2011). This is the varying of the model parameter values by different amounts and then measuring the associated effects. Parameters that have the largest effects have the most influence on the model performance and are taken as candidates for micro-level calibrations (Ormsbee and Lingireddy, 2004; Walski, 2017). The correct identification of these parameters is also very critical to the success of the micro-level calibration process (Ostfeld et al., 2011).

Micro-level calibration

Three micro-level calibration methods have been discussed in the literature. These are the explicit, manual and implicit methods as discussed below.

i. Explicit methods

Explicit methods (simulation-based models) are sometimes called analytical or direct methods because they are based on mathematical models which are solved analytically to determine the unknown values (Lansey et al., 2001). The major limitation of explicit methods is that they can only be conveniently applied to small networks while for large networks, their application requires a lot of network simplifications or skeletonization (Savic and Walters, 1995).

ii. Manual calibration

Manual calibration is a trial-and-error technique which involves several iterative model simulations until the results reach an acceptable level of calibration (Bhave, 1988). The method relies on manually changing model parameters and running simulations after changes to observe the level of agreement of the system state variables with field data. With this technique, it is difficult to effectively calibrate a large network and to achieve high quality calibration levels. However, models that are manually calibrated to levels that are far below the recommended accuracy levels can still perform well (Edwards, et al., 2006).

Manual calibration is used in this study owing to the limitations of the available data quality (Chapter 7)

iii. Implicit calibration

Implicit calibration (automated) methods use optimisation techniques that randomly search for a suitable combination of parameters that give the best simulation results. To use optimisation techniques, an objective function must be developed which should be used to test the fitness of the solution. In calibration problems, the objective function is developed to minimize the square of the differences between measured and simulated values of pressures and flows as (Savic and Walters, 1995; Lingireddy et al., 2005):

$$G = \sum_{i=1}^{N} W_Q \left(F_{F_i} - S_{F_i}\right)^2 + \sum_{j=1}^{J} W_H \left(F_{Hj} - S_{Hj}\right)^2 \qquad (4.17)$$

Where N is the total number of pipes with field measured flows used for calibration; W_Q the weighting factor for flows; F_{Fi} the field measured flows through pipe *i;* S_{Fi} the simulated flows through pipe *i*; W_H the weighting factor for node pressure heads; F_{Hj} the field measured pressure head for node *j*; S_{Hj} the simulated pressure head for node *j* and J the total number of nodes with field measured pressure heads used for calibration.

Since pressure head and flow values used in the calibration process define the state of the system in response to changes in parameters, it is handy to interface a hydraulic solver such as EPANET and a programming software in which the optimisation algorithm is implemented. The hydraulic solver handles the numerous linear node conservation of mass and non-linear link or loop conservation of energy equations to determine the water distribution network link flows, nodal water withdrawals and pressure. The numerous equations coupled with varying operation conditions, especially during EPS make WSS very complex which makes genetic algorithms rather than classical optimisation techniques suitable for finding near-optimum solutions (Savic and Walters, 1995; Farmani et al., 2007).

Evaluation of the quality of calibration

Evaluation of the quality of model calibration is a process that shows how well the model parameters have been adjusted thereby indicating the model reliability. It is done through the calculation of various statistics, visualisation through graphs and using model validation.

The statics used for the evaluation of the model calibration include the mean ($\bar{F}$), relative percentage error ($Error$), standard deviation (std), root mean square error (RMSE) and the correlation coefficient (R). The $Error$, $\bar{F}$, std, RMSE and R are calculated respectively by:

$$Error = \left(\frac{S_{F_i} - F_{F_i}}{F_{F_i}}\right) * 100 \tag{4.18}$$

$$\bar{F} = \frac{1}{N_m}\sum_{i=1}^{N_m} F_i \tag{4.19}$$

Where N_m is the number of pipes with field measured flows and F_i is the pipe flow at time t (F_{F_i} or S_{F_i}).

$$std = \left(\frac{\sum(F_i - \bar{F})^2}{N_m - 1}\right)^{\frac{1}{2}} \tag{4.20}$$

$$RMSE = \left(\frac{\sum_{i=1}^{N_m}\left(S_{F_i} - F_{F_i}\right)^2}{N_m}\right)^{\frac{1}{2}} \tag{4.21}$$

$$R = \frac{\left(N_m \sum_{i=1}^{N_m} S_{F_i} * F_{F_i}\right) - \left(\sum_{i=1}^{N_m} S_{F_i}\right)\left(\sum_{i=1}^{N_m} F_{F_i}\right)}{\sqrt{\left[N_m \sum_{i=1}^{N_m} {S_{F_i}}^2 - \left(\sum_{i=1}^{N_m} S_{F_i}\right)^2\right]\left[N_m \sum_{i=1}^{N_m} {F_{F_i}}^2 - \left(\sum_{i=1}^{N_m} F_{F_i}\right)^2\right]}} \tag{4.22}$$

The percentage errors calculated by Equation 4.18 are plotted against pipe ID to aid with the visualisation of the accuracy of the simulated results in respect of each pipe. The graphs help the easy identification of pipes for which the error is larger than stipulated in the guidelines.

Model validation involves subjecting a model to data that has not been used on it (unseen data). The unseen data in this case is field data from a period not used during the calibration of the model (Walski, 2017). While model validation is not done in this study due to data limitations, Equations 4.18 - 4.22 are used (Chapter 7).

4.4.3 *Calibration of the M-SIPDA*

The calibration process discussed here is for the M-SIPDA alone because the calibration of the EPANET hydraulic model is done separately as in subsection 4.4.2. Genetic algorithms (Non dominated sorting genetic algorithm II) (Deb et al., 2002) can be used in which the leakage emitter exponent and the adjustment factor for total base demand can be treated as two separate decision variables besides others such as roughness factors. However, in this study, only leakage exponent and the adjustment factor for total base demand are considered for calibration because roughness factors (C-factors) were fully adjusted separately during the original hydraulic model development as discussed in Chapter 7 under sensitivity analysis.

Since the M-SIPDA is developed for complex IWSS, explicit calibration techniques are not practical. Depending on the quality of the available calibration data, either manual or implicit calibration can be used. The calibration parameters are the leakage emitter exponent (α) and the total base demand (Bd_{Totalj}). The α value is initially set based on the combination of the WSS pipe materials that have fixed and variable leak openings as pressure varies (Lambert 2001; Lambert et al. 2017). For the Bd_{Totalj} adjustment, the initial factor of 1.0 is used which means the initial total base demand for the M-SIPDA is equal to that for the original DDA EPANET model. For the manual calibration method used in this study, first α is adjusted until the DMA simulated level of leakage is equal to or close to the known level. After adjusting α, the Bd_{Totalj} values are adjusted by multiplying them by small increments above 1.0 until the pipe flows are reasonably close to the measured values. In cases where the allocation of the Bd_{Totalj} is certain, its values need not be adjusted during calibration. Then, only the α can be adjust during calibration. The maximum pipe flow error limit is 10% based on the guidelines presented by the Sewers and Water Mains Committee of the Water Authorities in the United Kingdom in 1999 (United States Environmental Protection Agency, 2005). The assessment of the quality of the calibration process for the M-SIPDA is done using Equations 4.18 to 4.22.

4.5 Development of the visualisation technique for node parameters

Visualisation of the node parameters such as elevation, demand patterns and node pressure heads is very important for decision making. For instance, lower pressure heads than H_{req} indicate nodes with water supply problems and nodes with pressure head above 60 m (Republic of Zambia, 2011b) are indicative of areas where leakage problems are likely to be high. This is because high pressure is linked to high leakage rates (Republic of Zambia, 2011b; Ghorbanian et al., 2016) which is the basis for the pressure management pillar of a successful leakage management policy (Lambert, 2001; Liemberger and Farley, 2004). In EPANET and commercial software tools such as WaterGEMS, the generation of distribution maps for elevation, pressure and other parameters is built in. However, many PDA approaches, especially the reservoir and emitter-based methods, have no provision for generating pressure distribution maps. In this section, the technique for visualising node parameters is developed by writing a MATLAB code that uses graph theory to draw the WSS graph and develop the selected parameter's (pressure) distribution map at any time.

4.5.1 *Mapping the WSS into an undirected graph*

The first part is the mapping of the undirected WSS graph into an undirected graph in MATLAB. The nodes (vertices) of the graph are defined by their coordinates and the links (edges) are defined by their 'start and end nodes. Both, the nodes' coordinates and the links' start, and end nodes are read from the EPANET input file through the EPANET-MATLAB interface.

4.5.2 *Colour coding of the nodes to develop parameters distribution maps*

After drawing the graph, the second part involves the reading of the pressure heads (node parameter) for all the nodes at a given time. Then, the pressure head ranges are created into which each node pressure head fits. The creation of the pressure head ranges is guided by the specific country's expected minimum and required pressure heads. Each pressure head range is allocated a colour using the Red, Green and Blue (RGB) colour codes and these pressure rage colours are reflected in the WSS graph and the legend.

4.6 Conclusion

In this chapter, a new PDA method named the M-SIPDA is proposed for modelling IWSS. These systems are characterised by multiple water supply schedules which are considered in the mathematical model for the M-SIPDA using the fact that when a DMA is being supplied water, the supply schedule multiplication factors are greater than zero and when there is no supply, the multiplication factors are zero.

Moreover, the M-SIPDA considers leakage explicitly as well as a demand. While for the modelling of CWSS, omission of leakage or its inclusion as part of the consumption demand may not have serious implications, for IWSS, modelling of leakage is important because leakage constitutes a large portion of water flows. In the M-SIPDA, leakage is modelled by including an artificial leakage emitter which implements the pressure dependent leakage outflows. Since the leakage base demand is the 24 hours leakage average, when there is water supply to a DMA, leakage base demand is multiplied by 1, when there is no supply, it is multiplied by 0. This takes care of the multiple water supply schedules. Having a separate artificial leakage emitter from the artificial consumption demand emitter is important because while leakage has no upper limit when the nodal pressure head is equal to or above the required pressure, consumption demand is assumed to have.

To guide the calibration of the case study hydraulic model and the method for modelling IWSS (Chapter 7), the chapter presents a review of the literature on the calibration process. The fact that the type and source of the data collected for calibration depends on the application of the model and the type of simulation is highlighted. The distinction of good, bad and useless calibration data is noteworthy, especially when dealing with IWSS where data is in most cases of poor quality due to poor database management which leads to poor system knowledge. Moreover, the chapter shows that model calibration is a continuous process and that even models that seem poorly calibrated can be useful provided their limitations are known. The chapter presents metrics for evaluating the quality of the calibration process.

Finally, the chapter presents the technique for visualising node parameters such as pressure. The technique for visualising is achieved by writing a MATLAB code that uses graph theory to draw the WSS graph and develop the distribution map for the selected parameter (pressure) at any time. This visualisation is important as an aid to the quick identification of the water supply problem areas and the quick understanding of the effects of implemented interventions.

5 THE LUSAKA WATER SUPPLY NETWORK AND ITS MODELLING ASPECTS

This chapter provides information on the LWSN and aspects critical to its modelling process. The chapter is divided into five sections. Section 5.1 introduces the LWSN in terms of its zones and water supply mode. Section 5.2 discusses the two subsystems of the WSS (bulk and satellite system) and the causes of low system capacity. In section 5.3, the LWSN hydraulic model and aspects critical to the modelling process of the LWSN are discussed. The aspects include sources of data and analysis of the network complexities which provide reasons for working with a zone rather than the whole WSS to achieve a more comprehensive analysis using scenarios developed in Chapter 3. The isolation of the selected zone from the rest of the network is discussed in section 5.4, and section 5.5 provides the chapter conclusion.

5.1 Introduction

The LWSN is divided into five zones and three peri-urban area agglomerations (Figure 5.1). The zones are Lusaka Central, Chelstone, Kabulonga, Kabwata, and Lumumba including Matero (Simukonda et al., 2018a), and the three peri-urban areas agglomerations which are peri-urban Eastern, Southern and Western (Ministry of Local Government and Housing et al., 2009).

Water is supplied to the zones and peri-urban area agglomerations from the distribution reservoirs and more than 116 boreholes which are controlled by the production division. Since the production division supplies water intermittently, it is the first to implement the IWS mode. Then, the zones and peri-urban area agglomerations also implement the water supply intermittency by rationing the water between DMAs connected to different sources to try and balance supply durations (Simukonda et al., 2018a). This rationing is implemented through valve and pump operations.

The IWS operation of the LWSN is contrary to the Guidelines on Required Minimum Service Level according to which all parts of the WSS should be on CWS status (NWASCO, 2000). From the consultants' perspective, CWS is now taken as the long term target (Ministry of Local Government and Housing et al., 2009; Millennium Challenge Account - Zambia Limited, 2013), but meeting this target is proving difficult because of the many challenges that the water supply sector is facing (Simukonda et al., 2018a) and the fact that the vision for a CWSS is not part of the LWSC and the Zambian authorities yet. Information on the water sector which includes challenges of improving water supply in Zambia and Lusaka City is contained in Simukonda et al. (2018a) and Appendix B.

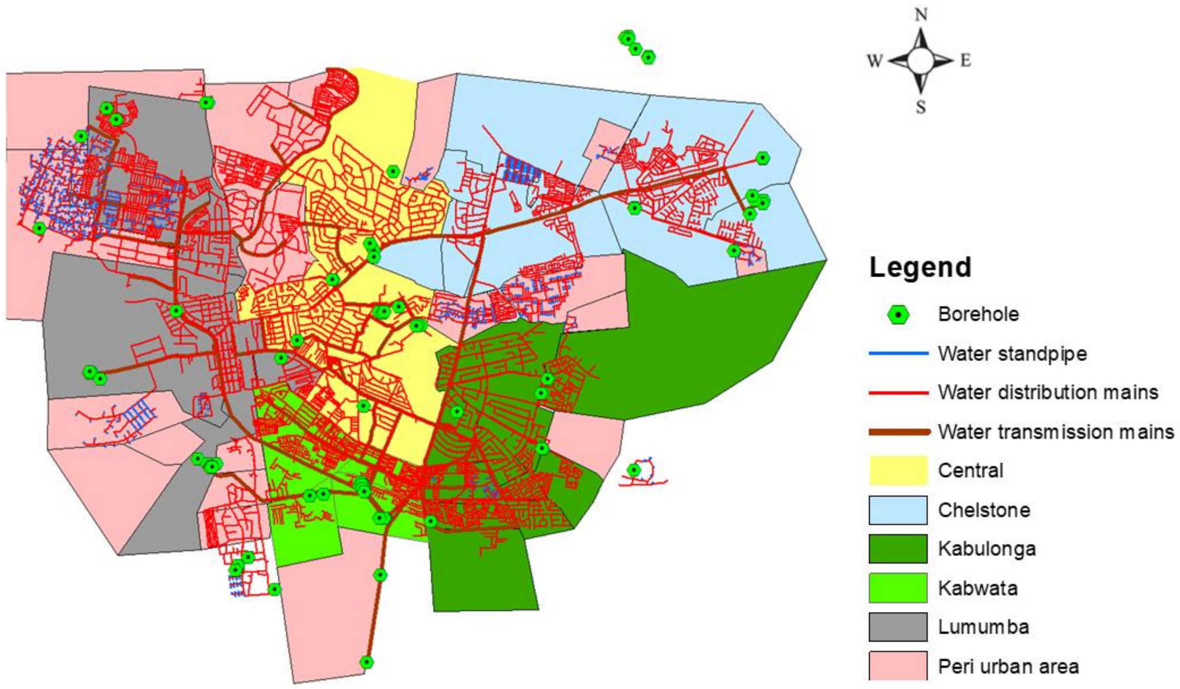


Figure 5.1: Lusaka Water Supply Network

5.2 The two water supply subsystems

There are two water supply subsystems for Lusaka City. These are the bulk and the satellite WSS (Lusaka City Council and Environmental Council of Zambia, 2008; Ministry of Local Government and Housing et al., 2009; Republic of Zambia, 2011b). The bulk WSS is the main system in which water from the sources (surface and ground water sources) must enter the main WSS network. The satellite WSS constitutes groundwater sources (boreholes) that are not connected to the main WSS network because they supply water only to nearby peri-urban areas (Ministry of Local Government and Housing et al., 2009; Republic of Zambia, 2011b). While some peri-urban areas are supplied water by the satellite systems only, others are supplied, in varying proportions, by both the satellite and bulk supply systems through the zones to which they are connected (Ministry of Local Government and Housing et al., 2009; Republic of Zambia, 2011b). This study is concerned with the bulk WSS, but since it is difficult to distinguish peri-urban areas that are supplied by both water supply subsystems from non-peri-urban areas, no distinction is made in this study between peri-urban area agglomeration and zone DMAs. The LWSN, which is the concern for this study, is part of the bulk WSS.

5.2.1 *Water supply schedules for the bulk supply system*

There are 69 existing DMAs for the bulk WSS. Some of the DMAs are supplied water intermittently while others are supplied continuously. Most of the DMAs with intermittent supply schedules have their supply starting at about 05:00 am and ending at about 22:00. Even for those with CWS schedules, most of them have peak demands around 05:00 am. The supply schedules for various regions of the WSS according to the hydraulic model (discussed in section 5.6) are shown in Figure 5.2. However, according to the data in the report on NRW (Brian Colquhoun Hugh O'Donnell and Partners, 2010), some regions that are shown to be continuously supplied with water are in fact not. Basing on this data, the proportions of the different water supply durations are shown in Figure 5.3. A comparison of Figures 5.2 and 5.3 shows a clear contrasting representation of facts. While Figure 5.2 shows that more than 50 % of the WSS is supplied water continuously, Figure 5.3 shows that only 16 % of the DMAs are supplied water continuously. Moreover, Figure 5.3 shows that there are DMAs where water is supplied for a total duration of 10 hours. This total water supply duration is not reflected by any of the water supply time intervals in Figure 5.2. One explanation for these differences could be the setting of some supply schedules with low multiplication factors other than zero while in actual sense the supply is zero when there is no water supply. This is the case for the DMAs with the water supply schedules in the hydraulic model as shown in Figure 5.4a which are actually supplied by the water supply schedule shown in Figure 5.4b. Also, DMAs that are shown in the hydraulic model to be supplied using supply schedules, as shown in Figure 5.4c, are actually supplied following schedule Figure 5.4d. Even though these differences in the information cannot be resolved in this study, they show the need to treat the model with caution which is important to varying degrees even for any other models.

Figure 5.4e and Figure 5.4f show the 24/7 block demand patterns. The name is used to distinguish them from those with peak demand factors at certain times of the day. However, since the exact characteristics of diurnal water demand patterns for Lusaka city are not known for DMAs that have a CWS, the patterns are in the stepwise form (Brian Colquhoun Hugh O'Donnell and Partners, 2010). Furthermore, Figure 5.4f shows a pattern for DMAs that have higher night than day demands.

With reference to the housing types (High cost, medium cost, low cost and informal), continuous supply is found in DMAs that have high or medium housing types. Due to the high hydraulic connectivity, some areas that have IWS schedules, have continuous supply. These are mostly areas with low elevations.

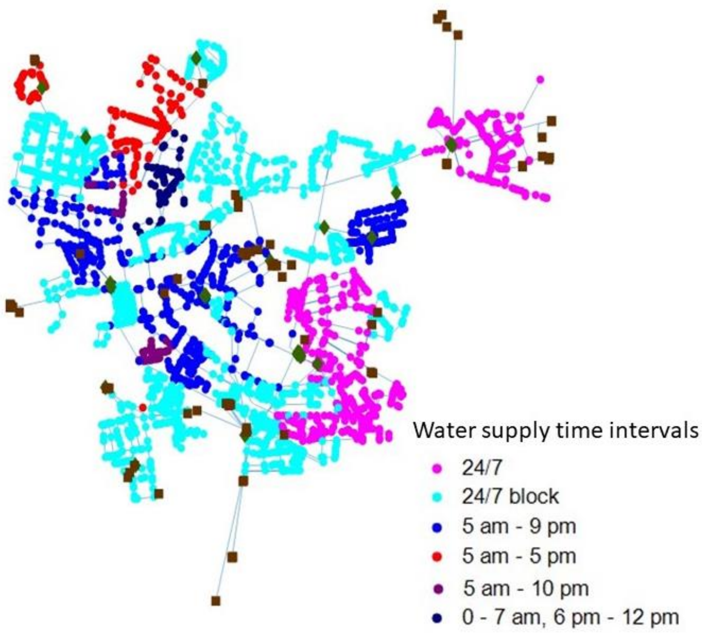


Figure 5.2: Water supply time intervals for various regions

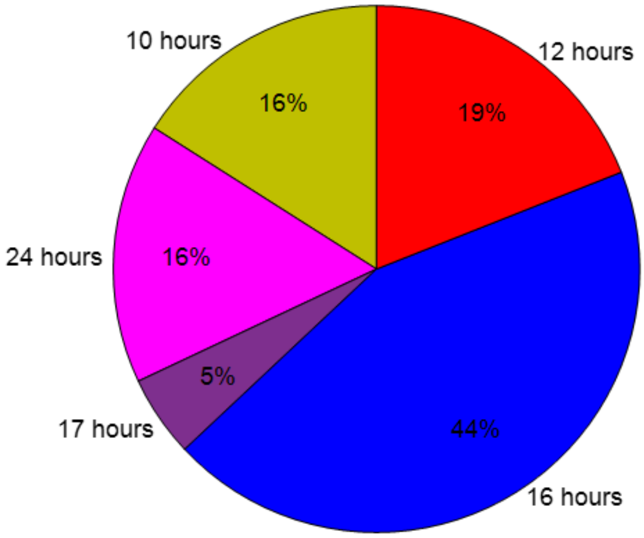


Figure 5.3: Supply duration and the proportion of the regions supplied during the duration (Adapted from: Brian Colquhoun Hugh O’Donnell and Partners, 2010).

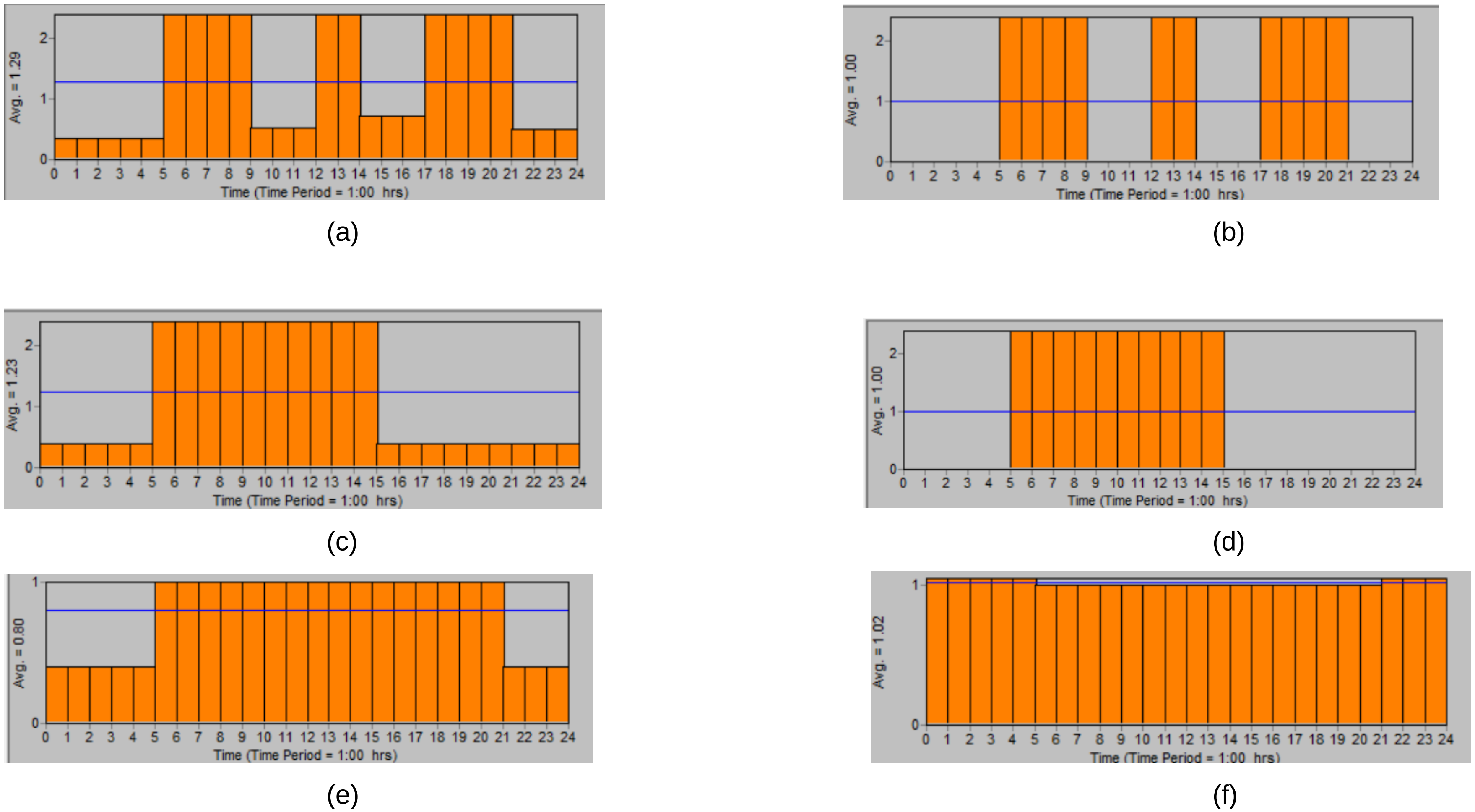


Figure 5.4: Water supply schedules and demand patterns for LWSN DMAs

5.2.2 *Low WSS capacity*

The LWSN has low capacity due to three factors: increasing consumption water demand, high non-revenue water (NRW) and the low capacity of the WTP.

Increasing consumption water demand

Consumption water demand is increasing mainly due to population growth, urbanisation and industrial development. Consumption demand comprises domestic and non-domestic (public or institutional, commercial and industrial) demands.

The rapidly increasing domestic demand has not been matched with the expansion of the water supply and distribution infrastructure. New residential areas (which are predominantly unplanned) are developed without the water distribution network (Rakodi, 1987). The unplanned or reactive extension of the WDS to these areas reduces the hydraulic capacity of the system thereby reducing the duration and the amount of water supplied to those that are already supplied (Simukonda et al., 2018a). Since the extension of the WSS to unserved areas is one of the indicators for the minimum service level for water utilities in Zambia (NWASCO, 2000), it influences all future water supply improvement plans as it implies ever increasing domestic water demand.

To determine the domestic water demand for the entire WSS, per capita water demands are multiplied by the population served or expected to be served. For the LWSN, the domestic per capita demands are differentiated according to the housing types as shown in Table 5.1. Hence, the total domestic water demand for a DMA in the LWSN is determined by (Republic of Zambia, 2011b):

$$Q_{DMAd} = P_{HC} * HC_{sc} + P_{MC} * MC_{sc} + P_{LC} * LC_{S_{sc}} + P_{Inf} * InfC_{sc} \qquad (5.1)$$

Where Q_{DMAd} is the DMA domestic consumption water demand and the other components are as defined under Equation 3.1 and the domestic per capita water consumptions as shown in Table 5.1. The definitions of the various housing types are presented in Table 5.2.

Table 5.1: Domestic per capita water demands (Republic of Zambia, 2011b)

Housing type	Actual per capita demand in 2010 (l/c/d)	Zambian design standard per capita demand (l/c/d)
High Cost Housing	143	280
Medium Cost Housing	110	150
Low Cost Housing	60	100
Informal Housing	20	40

Table 5.2: Definition of the housing types (Republic of Zambia, 2011b)

Housing type	Water Affairs definition	Zambian design standards definition
High Cost Housing (HC)	Individual house connection with: • Internal plumbing • Kitchen • Toilet and bathroom with shower • Separate servant quarter	House with: • Floor area > 120 m^2 • Good building finish • Multiple taps • More than one WC • Water born sanitation
Medium Cost Housing (MC)	Individual house connection with: • Internal plumbing • Kitchen • Toilet with shower • Plots are smaller than HC housing • No separate servant quarter	House with: • Floor area of 90 - 100 m^2 • Average building finish • Multiple taps • One or more WC • Water born sanitation
Low Cost Housing (LC)	House with: • Yard connection • No internal plumbing • Shower is outside connected to the yard connection	House with: • Floor area of < 60 m^2 • Basic building finish • Reduced number of taps • One WC • Water born sanitation
Informal Cost Housing ($InfC$)	Dwelling: • Of temporal construction • Use communal standpipe, kiosks, hand pumps or shallow wells • Use pit latrines	Dwelling: • Use communal standpipe or one tap in the plot • Use pit latrines

Even though the definitions of the housing types are not consistent between those by the Water Affairs department and the Zambian design standards, Table 5.1 and Table 5.2 show that water supply to the informal housing type is poor. The differentiation of the expected domestic per capita water consumption depending on the housing types is a clear indication and acknowledgment that water consumption in Zambia increases with affluence. For planning purposes, the values in the Zambian design standard for WDS are used which are higher than the per capita demand values in 2010 (Table 5.1).

The public or institutional demands are categorised into educational facilities, medical facilities and administrative offices. The average per capita water demand of 14 l/c/d is used for public or institution demand and for commercial users, the per capita demand is 3.05 l/c/d (Ministry of Local Government and Housing et al., 2009). The DMA public consumption water demand (Q_{DMAp}) is the product of the per capita demand and the number of people in the public facilities (Republic of Zambia, 2011b) and is determined using a component of Equation 3.1 represented as Equation 5.2.

$$Q_{DMAp} = P_{Con} * P_{Pop} \tag{5.2}$$

The DMA commercial water consumption (Q_{DMAc}) is calculated using Equations 5.2 by replacing the public with commercial per capita water demand and the population in public facilities with the population in commercial facilities.

Industrial demand is based on per unit area. The estimated actual per unit area industrial water demand for 2010 was 80,000 l/ha/day. The value for the planning horizon up to 2035 is 60,000 l/ha/day (Republic of Zambia, 2011b) and the industrial demand (Q_{DMAi}) is determined by Equation 5.3 which is a component of Equation 3.1:

$$Q_{DMAi} = Ind_{Con} * A_{ind} \tag{5.3}$$

Where Q_{DMAi} is the DMA industrial consumption water demand, Ind_{Con} is the industrial per unit area water consumption, A_{ind} is the area occupied by industries in a DMA.

Consequently, the total DMA consumption water demand (Q_{DMA}) is given by Equation 5.4 which is a summary of Equation 3.1. The total WSS consumption water demand (Q_{LWSN}) is given by Equation 5.5.

$$Q_{DMA} = Q_{DMAd} + Q_{DMAp} + Q_{DMAc} + Q_{DMAi} \tag{5.4}$$

$$Q_{LWSN} = \sum Q_{DMA} \tag{5.5}$$

High NRW

High NRW levels are an indication of a poorly managed water utility with poor governance, lacking autonomy, accountability, and the requisite technical and managerial skills for reliable service delivery (Kingdom et al., 2006). High levels of NRW lead to significant financial losses by the utility resulting in its failure to invest in infrastructure maintenance programmes which leads to the deterioration of the physical condition of the WSS (Farrow et al., 2016).

All WSS in Zambia have high NRW levels. Efforts to reduce the NRW levels have proved futile and workable solutions to reduce these levels are sought (NWSCO, 2017; 2018). For the LWSN, the levels of NRW are consistently above 40% of the system input volume (LWSC, 2017b) (Figure 5.5). These NRW levels are not only far above the upper acceptable limit of 25% set by the regulator (Table 5.3), but are also completely off the target for achieving the intended results of implementing the Water Supply Investment Master Plan for Lusaka city outlined in Republic of Zambia (2011b). The success of the Master Plan interventions, such as the effectiveness of developing new WTPs, increasing water supply hours and extension of the network coverage depends on reducing NRW to 15% of the input volume by 2035 (Ministry of Local Government and Housing et al., 2009; Republic of Zambia, 2011b; Millennium Challenge Account - Zambia Limited, 2013). However, since no significant progress is being made towards reducing NRW, the set target of reducing NRW to 15% by 2035 is apparently unattainable. With the high NRW levels, the utility financial position is negatively affected because NRW contributes significantly to the poor revenue collection by the company (Ministry of Local Government and Housing et al., 2009).

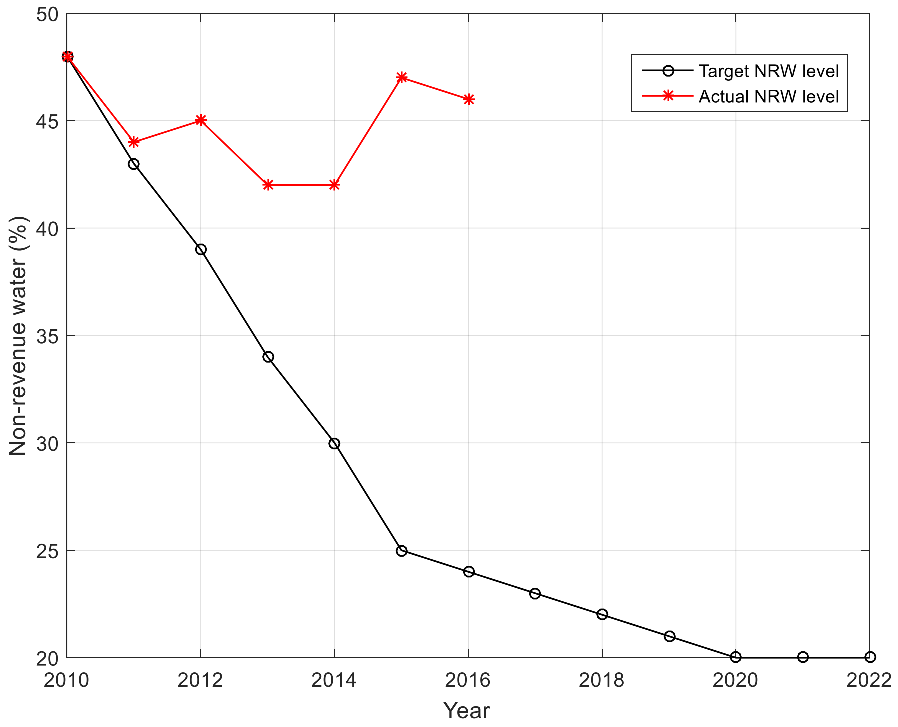


Figure 5.5: High actual NRW as compared to the targeted levels (Simukonda et al., 2018a)

Table 5.3: NWASCO benchmarks for NRW (NWASCO, 2018)

Benchmark for NRW	Good	< 20%
	Acceptable	20 – 25%
	Unacceptable	>25%

NRW consists of apparent and real losses. However, the quantification of these components is difficult especially in IWSS which are very poorly metered (Anand, 2015). No clear proportions of apparent and real losses are known, but assumptions of 50/50 have been reported (Millennium Challenge Account - Zambia Limited, 2013). For one DMA, the NRW value was reported to constitute

5.4% and 94.6% of apparent and real water losses respectively (The Republic of Zambia, 2014). Two apparent losses that directly contribute to the loss of system capacity are water theft and consumers on fixed charge tariffs since they tend to consume more water than they are billed (Millennium Challenge Account - Zambia Limited, 2013) and these can be significant for LWSN with a metering ratio of 66 % in 2018 (NWASCO, 2018).

Though in most cases real losses are looked at in terms of pipe leaks, they also occur from tanks and reservoirs in form of overflows and leaks. Leaking tanks and reservoirs for the LWSN are either filled to less than their capacity or abandoned completely due to their state of disrepair (Republic of Zambia, 2011b). The storage facility plus pipe leakage (background leaks and bursts) put extra pressure on the water sources because they increase the volume of water required to supply a given consumption demand (Skipworth et al, 2002) thereby directly affecting the system capacity. The developing country technical performance category for LWSN for 2017 and 2018 was D with the infrastructure leakage index (ILI) of 42.2 and 39 respectively (NWASCO, 2018). Performance category D (Table 5.4) indicates that the utility is highly inefficient in the use of resources and that leakage reduction programs are imperative and high priority (Kingdom et al., 2006; Farley et al., 2008; NWASCO, 2018). The ILI is the ratio of the current annual volume of real losses to the minimum achievable annual volume of real losses at the current average pressure (Kingdom et al., 2006; Farley et al., 2008). The ILI indicates how well a WSS is managed in terms of maintenance, repairing and rehabilitation (Farley et al., 2008). The various performance categories for developing countries and their corresponding ILIs are shown in Table 5.5. The ILI performance category for LWSN shows that leakage reduction programs should be a high priority, but so far leakage reduction efforts have proved futile.

Table 5.4: International Water Association (IWA) Infrastructure Leakage Index categories (NWSACO,2018)

Performance category	Comments/ Recommendations
A	Further loss reduction may be uneconomic unless there are shortages; careful analysis needed to identify cost effective improvement
B	Potential for marked improvements; consider pressure management; better active leakage control practices, and better network maintenance
C	Poor leakage record: tolerable only if water is plentiful and cheap; even then, analyse level and nature of leakage and intensify leakage reduction efforts
D	Horrifically inefficient use of resources; leakage reduction programs imperative and high priority

Table 5.5: Infrastructure leakage Index guidelines (NWASCO, 2018)

Technical performance category for developing countries	ILI	Physical losses (Litres/connection/day) (when the system is pressurized) at an average pressure of:				
		10 m	20 m	30 m	40 m	50 m
A	1 - 4	< 50	< 100	< 150	< 200	< 250
B	4 - 8	50 - 100	100 - 200	150 - 300	200 - 400	250 - 500
C	8 - 16	100 - 200	200 - 400	300 - 600	400 - 800	500 - 1000
D	> 16	> 200	> 400	> 600	> 800	> 1000

Low WTP capacity

There are two factors that contribute to the low WTP capacity. The first is that it has been operated below its design capacity for a long time because of the poor performance of various components which need repairs (Republic of Zambia, 2011b). This aspect is being resolved through the Millennium Challenge

Corporation project (Millennium Challenge Account - Zambia Limited, 2013). The second aspect is that even if the existing WTP is rehabilitated to its design capacity, the water produced will still be less than the increased demand because of population growth, urbanisation and industrial development. This will be resolved if plans to build two new WTPs near the existing one before 2035 materialise. However, realisation of these plans depends on the investment by the donors or cooperating partners as has been the case with all major water infrastructure development projects in Zambia (Simukonda et al., 2018a; Government of the Republic of Zambia, 2010). The master plan's current and projected demand and water production from the sources are shown in Figure 5.6. The figure shows that failure to reduce the NRW as planned in the master plan will result in water deficit in 2035 even if all the planned new plants were built. However, if the master plan leakage reduction target is met, the master plan scenario shows that there will be a total production (supply) above total demand (water demand plus NRW) surplus of about 10,688 m^3/day.

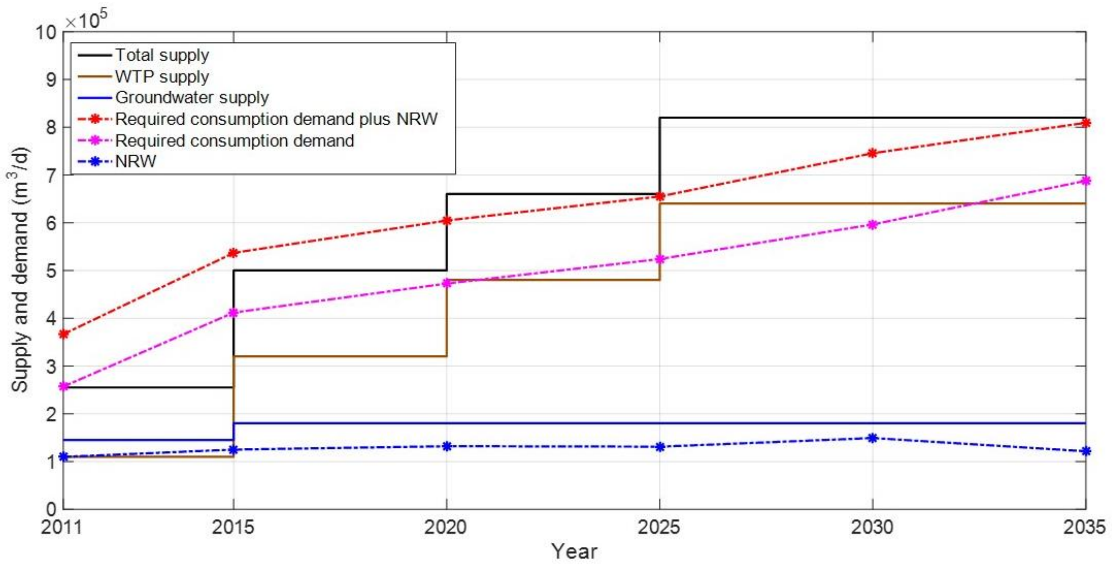


Figure 5.6: Water supply and demand (Adapted from: Republic of Zambia, 2011b)

5.3 The LWSN hydraulic model

The most effective way of analysing the hydraulic capacity of the LWSN is through modelling. Hydraulic models are dynamic because they are in a continuous process of updating and calibration each time there are changes to the WSS in terms of topology and operation (Walski et al., 2003; AWWA Water Distribution Model Calibration Subcommittee, 2013). For the LWSN, the existing hydraulic model is not regularly updated, and calibration for the existing model is poor due to limited quantity and quality of field measurement data. To make use of such a model, many secondary data sources and adaptive approaches to the modelling process are required.

5.3.1 *Data sources for the existing LWSN hydraulic model*

The existing LWSN hydraulic model was developed in 2010. The major sources of data for this model are the report on NRW (Brian Colquhoun Hugh O'Donnell and Partners, 2010), the Water Supply Investment Master Plan for Lusaka City (Republic of Zambia, 2011b) and the study on comprehensive urban development plan for the City of Lusaka in the Republic of Zambia report (Ministry of Local Government and Housing et al., 2009).

The report on NRW (Brian Colquhoun Hugh O'Donnell and Partners, 2010) contains the hydraulic model which was developed in WaterGEMS v3.0. The model calibration data constitutes measured flows for 76 pipes. The data includes the pipe ID and time when measurements were done. The flow measurements are one-off and were done during the day between 10:00 and 16:00 for a period of 4 months. The details of the data are in the appendix (Table A.1). Moreover, the report contains the simulation flows corresponding to the times when the measurements were done and pressure distribution maps at specified times. Since there were no pressure head measurements and tank level fluctuations were not observed, the WaterGEMS pressure distribution maps are also used as data sets. Lastly, the report also contains information on the DMA consumption demand, level of NRW and total demand which is the sum of consumption demand and NRW.

The Water Supply Investment Master Plan for Lusaka City (Republic of Zambia, 2011b) provides information on projections of water demands and supplies up to the year 2035. In the projections of demands, the domestic per capita water demands for each housing type (Table 5.1) are used taking into account the variation in numbers of the housing types during the planning horizon. The per capita water consumption for commercial and public entities plus industrial per unit area water consumption are also provided. The master plan also contains maps that show planned network extensions.

On the water supply part, the study on comprehensive urban development plan for the City of Lusaka in the Republic of Zambia (Ministry of Local Government and Housing et al., 2009) contains detailed information on water demand and leakage aspects which are in the master plan, but beyond that it contains approaches and maps that show how Lusaka City is expected to expand from the planning perspective.

5.3.2 *Conversion from WaterGEMS to the EPANET hydraulic model*

The original WaterGEMS v3.0 hydraulic model is first converted to a WaterGEMS v8i model following the steps provided by Dringoli (2018). This is because WaterGEMS v3.0 is not directly compatible with WaterGEMS v8i which is used by the LWSC. It is after conversion that, the WaterGEMS v8i model is converted to the EPANET 2.0 hydraulic model (Figure 5.7). The network model has many valves (Table 5.6) that regulate flows either through throttling or through minor head loss settings. The valves affect flows in complex ways because multiple of them can influence water flow through a single pipe. Table 5.7 shows some of the valves that have major influence on pipe flows. The complete list is in the appendix (Table A. 3).

Figure 5.7: The converted EPANET hydraulic model

Table 5.6: Summary of network elements

Network Elements	**Quantity**
Junctions	4217
Reservoirs	72
Tanks	25
Pipes	4033
Pumps	114
Valves	564

Table 5.7: Valves with flow control and minor loss settings

		Valve settings		
Pipe ID	**Affecting valves**	**Flow control (l/s)**	**Loss Coefficient**	**Fixed Status**
P-2055	GV-1045	43.575	100	Open
	GV-326	6.111	220	Open
P-2213	GV-814	27.8	383	Open
P-2715	GV-368	16.7	75	Open
	FCV-511	0	50	Open
	GV-369	14.5	420	Open

5.3.3 *District Metered Areas (DMAs) for the LWSN*

The 69 DMAs for the LWSN correspond to the development of urban settlements (townships) for which they are named after. While the expansion of the WSS in line with the expansion of the urban settlements is normal (Diao et al., 2013), the fact that the expansion of the settlements lacks planning is a source of problems. As a result of this situation, some DMAs are well defined in terms of water sources and distribution while others are too large or have no proper boundaries because they have many hydraulic connections with other DMAs. Having DMAs that are properly defined is vital for WSS operation and management and for converting from intermittent to continuous supply mode (Ilaya-Ayza et al., 2018). This is because by measuring the DMA pressure together with inflows and outflows, its performance can be fully analysed independent of the whole WSS (Diao et al., 2013). There has been no study that has ascertained the delineation of the LWSN DMAs.

LWSN is a Large WSS with multiple water sources and as such it is very complex in terms of topology and behaviour. For such systems, it is difficult to identify their systems' network structure, operational scheme, and interactions between their components (Diao et al., 2014; Diao et al., 2015). Since the expansion of the LWSN mainly follows the settlement areas, its topological characteristics fall in the category of nearly decomposable systems. These systems have community structures comprising smaller subsystems that have stronger or more frequent interactions within than between them (Courtois, 1985; Newman, 2006) (Figure 5.8). Decomposition of such systems makes their analysis easier (Courtois, 1985) because it results in smaller subsystems and layouts which may explicitly reveal system properties such as connectivity (Diao et al., 2015). Consequently, to understand the LWSN well, the delimitation of its DMAs should be ascertained. The clustering technique based on graph theory and knowledge-based judgment is used.

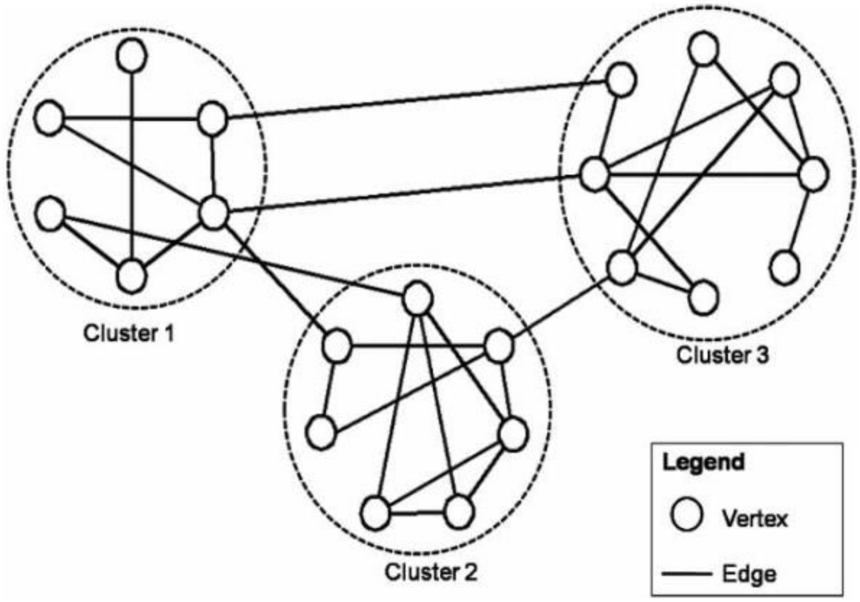


Figure 5.8: Network clustering using modularity index-based algorithm (Diao et al., 2014)

Network clustering

Graph theory methods form the basis for WSS decomposition because they map the system into graphs which are then divided into subgraphs (Diao et al., 2015). In a graph (G), vertices (V) such as junctions, tanks and reservoirs are joined by edges (E) such as pipes, valves and pumps (Perelman and Ostfeld, 2011; Diao et al., 2015). The developed graph is used to divide the WSS into clusters that have more connected internal nodes than there are with external nodes. The quality of the clusters is measured using the modularity index. The higher the modularity index (Modularity index > 0.3) the well-defined the clusters are (Diao et al., 2014).

Clustering (the division of the WSS graph) is achieved using the modularity index-based algorithm implemented in GEPHI which is a public domain software that is used for graph and network analysis (Bastian et al., 2009). GEPHI uses the data relating to the interplay between nodes (junctions, tanks and reservoirs) and edges (pipes, valves and pumps) to divide a complex system into subsystems (clusters). With respect to DMAs, network clusters detected by GEPHI may not all fit the specification for a DMA with respect to size (500-5000 consumer connections) and connection to water sources. As such those that are not well-defined through clustering are adjusted using knowledge-based judgment according to size, topography and the location of the identified transmission and distribution mains. The aim is to improve on the number of DMAs and their connection to water sources so that the understanding of the WSS is enhanced and its management efficiency improved.

Clustering of the LWSN network using GEPHI

The input file for the LWSN EPANET model is used to develop two files one for nodes (vertices) and the other for links (edges) which have the format as exemplified in Table 5.8 and Table 5.9 respectively. Since this is topological-based clustering, no hydraulic simulations are required and information about nodes (elevation and demand) and links (diameter, length and roughness) is not needed. For the nodes file, only the ID and Label information is needed. The ID is the node identification number while the label is the node identification name. For the edges file, the required information is the source, target and ID. The source and target refer to the edge’s start and end node respectively while the ID is the edge identification number. The files are saved with the extension .csv.

Table 5.8: LWSN nodes file

ID	Label
1	J-1361
2	J-733
3	J-2360
4	J-2429
5	J-757

Table 5.9: LWSN edges (links) file

Source	Target	ID
317	1666	1
453	730	2
1745	419	3
689	669	4
519	787	5

The saved nodes and edges files are exported into GEPHI starting with the nodes file. Since clustering is intended, in GEPHI, modularity is used and the randomise option is selected. First, the default resolution of 1.0 is used. Basing on the knowledge of the network, after trying a few resolutions, the resolution of 1.3 is found to be suitable. This results in a modularity index of 0.945 and 44 communities (clusters) which are identified by the community numbers (Modularity classes) that are given to all the nodes according to their communities as illustrated in Table 5.10. The Table containing the community numbers is exported to Excel and saved with the extension .xlsx.

Table 5.10: Example of nodes with their community numbers (Modularity class)

ID	312	1666	453	730	1745	419	689	669	519
Modularity class	22	22	33	33	35	35	33	33	30

Visualisation of DMAs

The modularity class is considered as a nodal parameter. This makes it possible to use the node parameter distribution map development technique developed in Chapter 4. To develop the modularity class distribution map, the saved Excel file is used and nodes that belong to the same class (community) are coloured the same colour. The coloured communities represent the DMAs. The resulting colour coded Modularity class (community) based DMA distribution map is shown in Figure 5.9. Since the communities have more internal than external connections, the existence of nodes of one colour among those of a different colour shows that there are parallel pipes that connect nodes belonging to different communities (Figure 5.10). These parallel pipes make the DMA boundaries defined from Modularity classes (based on connectivity) (Figure 5.9) differ from those based on township names which are also water supply schedule names (Figure 5.11).

It is noteworthy that the DMAs based on Modularity classes are closely linked to the hydraulic connectivity because nodes are connected by links. Thus, the difference between DMA connectivity and township boundaries is one of the complicating factors when identifying the DMA water sources for the LWSN. This is so because some nodes within a township-based DMA can have water sources different from the rest simply because they are supplied water by a different pipe that is parallel to the one supplying the rest (Figure 5.10). There are also township-based DMAs that are supplied water from two different sources and the network is disjointed. This is exemplified by Kany which is shown as one DMA in the original township-based DMAs while GEPHI has divided it into two parts belonging to different Modularity class based clusters as shown in Figure 5.9 and Figure 5.11 respectively. Figure 5.11 shows that nodes belonging to one DMA are found amongst those of the others as shown in Figure 5.12. However, in this case it is not because of connectivity, but because of multiple water sources for some DMAs and inconsistencies or errors in allocating supply schedules to demand nodes during the development of the hydraulic model.

A comparison of Figure 5.9 and Figure 5.11 shows that there are more DMAs based on township than there are based on system connectivity using the modularity class (communities). This is because some township DMAs are represented by less than 5 nodes and some institutions are considered as DMAs represented by one node in the model. Since these DMAs with very few nodes cannot be identified as independent communities using the modularity index, they are combined as part of the nearest connected communities. However, there are also cases, like the far east part, where the modularity index approach has identified more DMAs (Figure 5.9) than those based on township (Figure 5.11). For the eastern part, it is observed that the same supply schedule is used for three townships thereby making them appear as one DMA. On the northern, central and southern parts, the modularity index approach identifies fewer DMAs than the township approach. This is due to skeletonisation which results in some DMAs being represented by very few supply nodes while others have nodes that are found in the same township, but have different water sources and are represented in the hydraulic model without any connection. In these situations, the modularity index-based approach will not show the disconnected nodes as part of the same DMA while the township-based approach will.

For the western part, the modularity index-based approach identifies more DMAs than the township-based approach because the latter is based on the name of the township than the difference in the way the parts of the township are connected. Other than these differences, there are also DMAs which are identified in the same way by both approaches. These similarities and differences in the DMA identifications suggest the fact that neither method is perfect on its own, but that the two methods can complement each other when applied with a good knowledge of the WSS and engineering judgment. Even though it is shown that the township-based approach identifies DMAs that have some hydraulic connectivity and supply schedules allocation shortcomings, this study uses the DMAs based on the approach because the available information is based on such DMAs. The importance of the Modularity class based DMAs is that it has revealed the limitations of the township based DMAs.

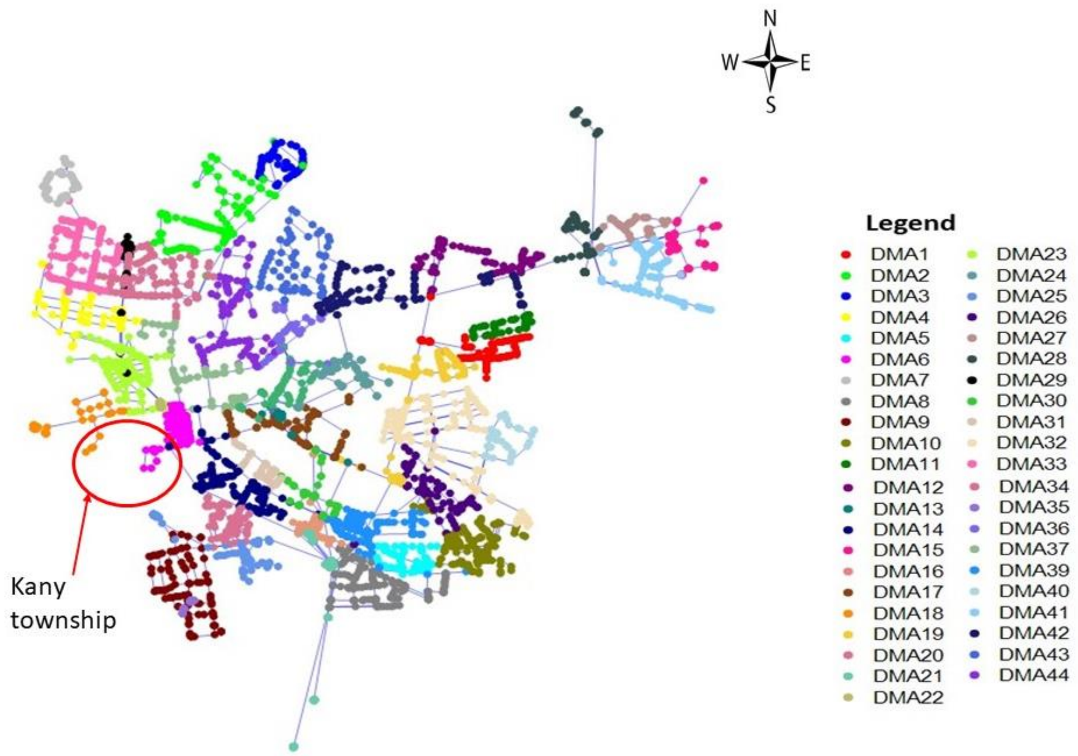


Figure 5.9: DMA identified as clusters using the modularity index

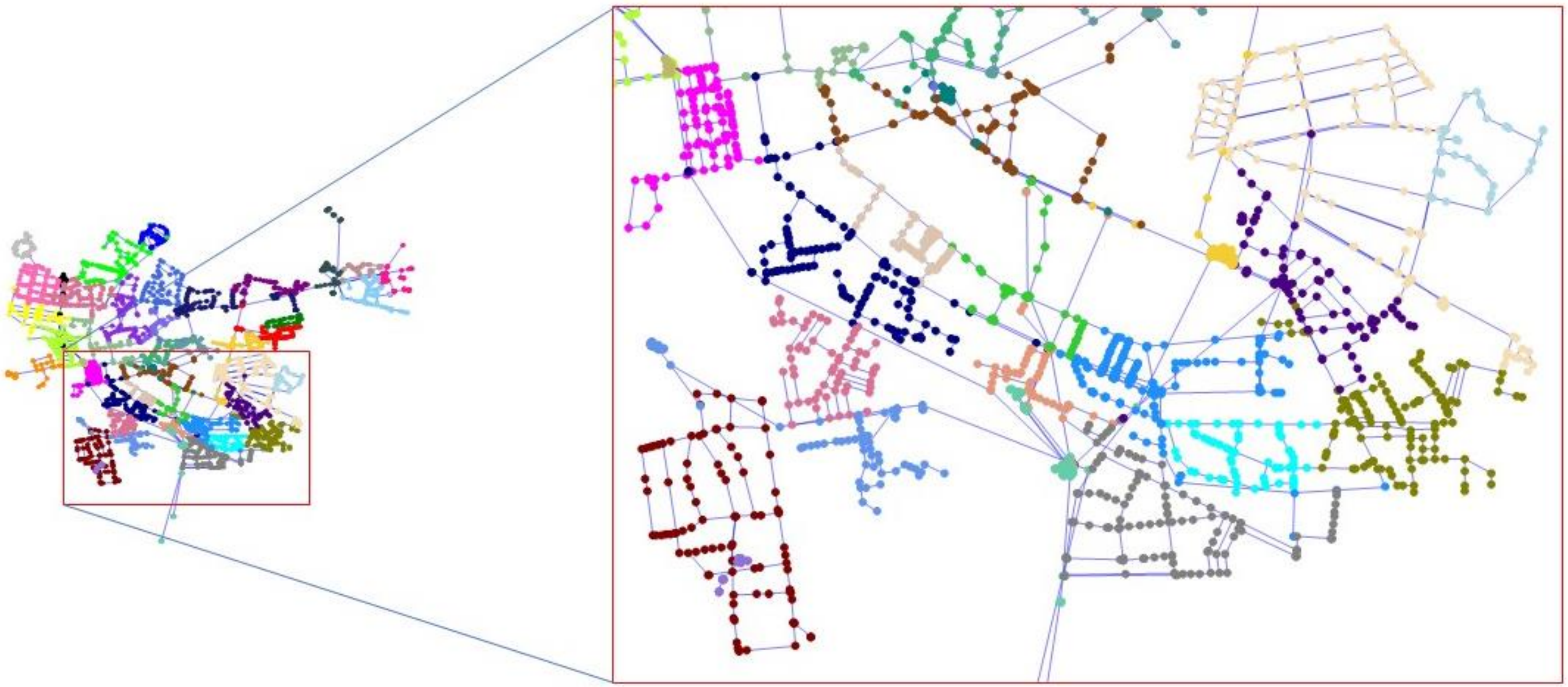


Figure 5.10: Parallel pipes and different water sources reflected by nodes of one DMA found in other DMAs

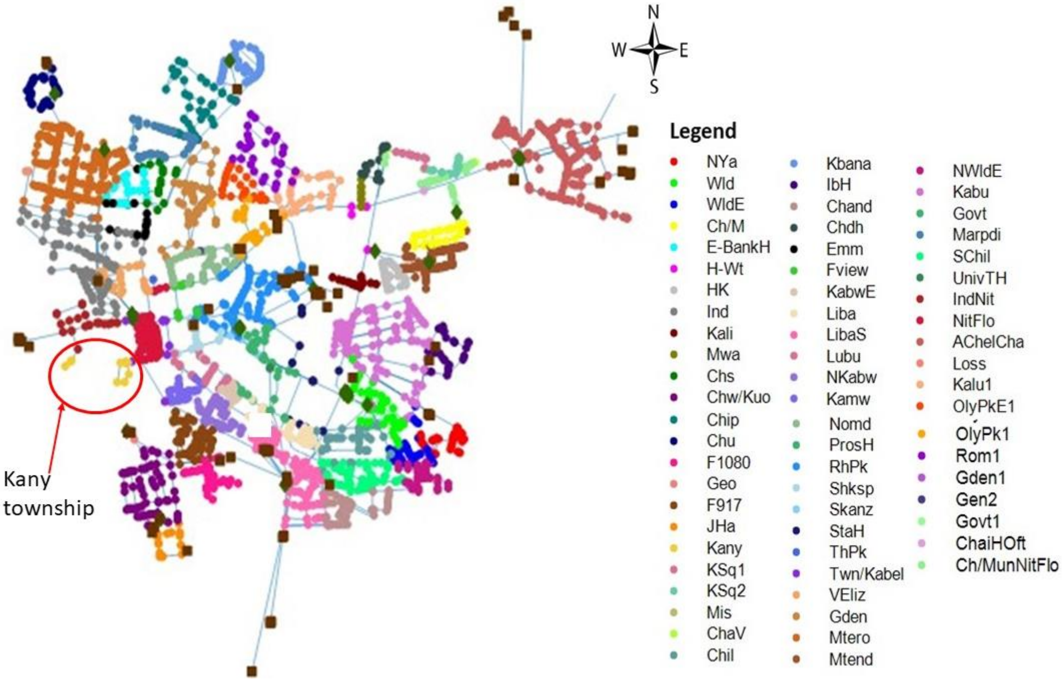


Figure 5.11: DMAs defined according to townships water supply schedules (Adapted from: Brian Colquhoun Hugh O'Donnell and Partners, 2010)

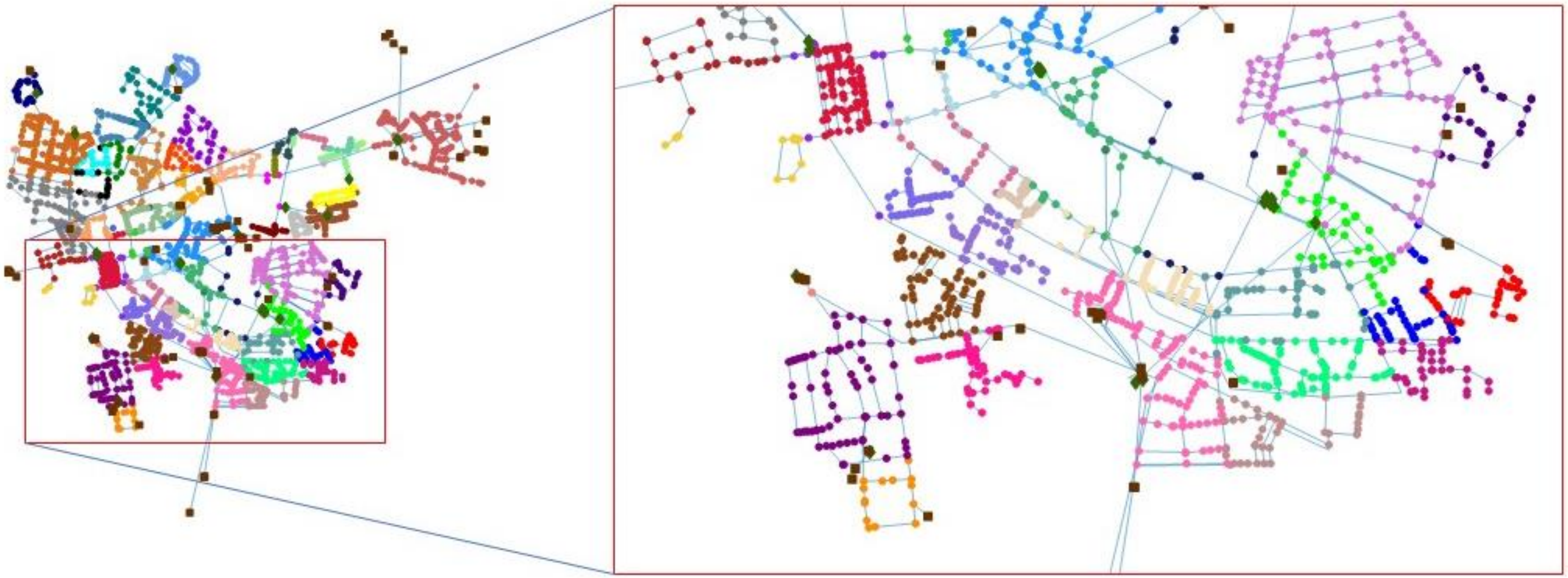


Figure 5.12: Multiple DMA water sources and Supply schedules allocation inconsistencies reflected by nodes of one DMA found in other DMAs

5.3.4 *The Interconnectivity of the LWSN zones and DMAs*

Besides the aspect of parallel pipes, the clustered LWSN model reveals that various parts of the system are hydraulically connected as shown in the schematic representation (Figure 5.13). The figure explicitly reflects the fact that of the many boreholes, some supply water directly into the distribution system, others supply indirectly via the distribution reservoirs and yet others supply both directly and indirectly. The distribution reservoirs also supply water to zones and DMAs that are hydraulically highly connected. This aspect of interconnectivity is against the DMA concept and makes it very difficult to isolate and independently analyse a zone or a DMA through modelling.

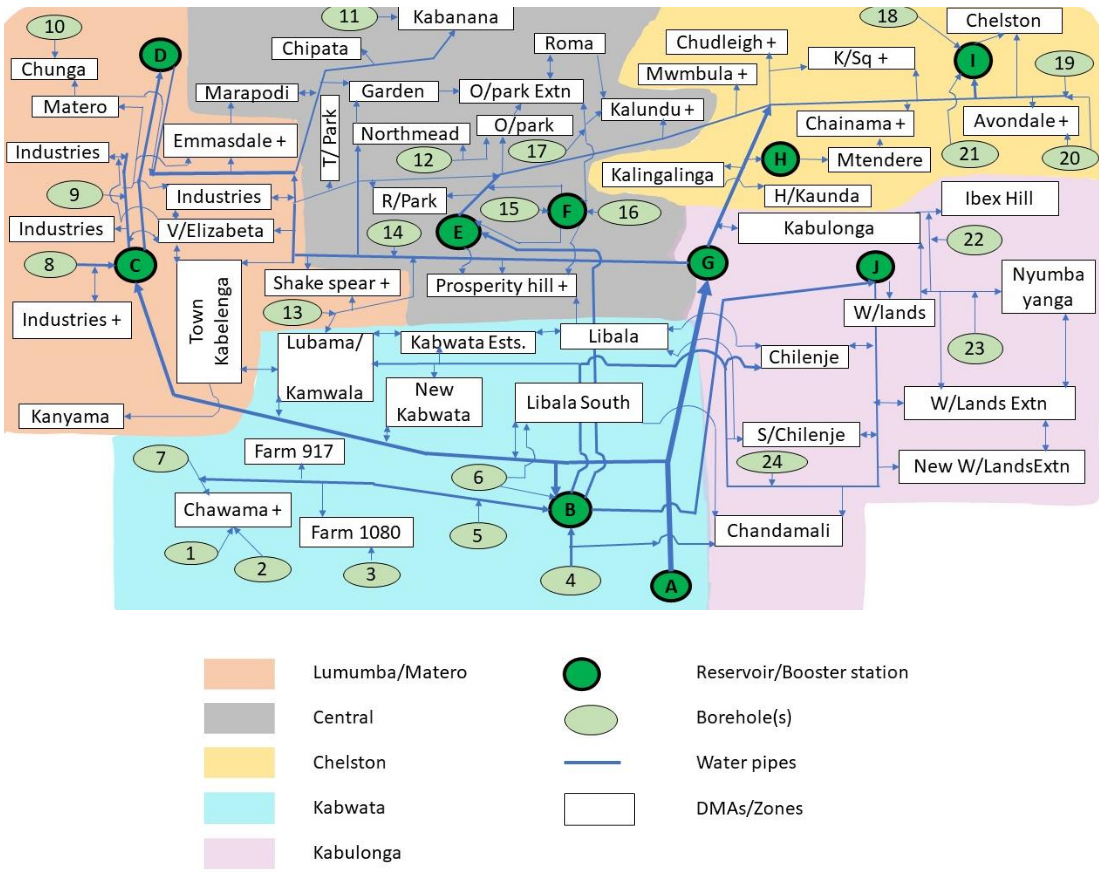


Figure 5.13: The LWSC network interconnected zones and DMAs

5.3.5 *LWSN complexities*

Complexities of the LWSN can be classified based on either operational or modelling aspects.

Operational complexity

Operationally, the LWSN is highly complex because of its large size with multiple water sources, large number of pumps and valves which are involved in the implementation of the IWS mode. The large number of pumps is a concern with respect to energy costs. Moreover, with the problem of electricity outages, maintaining the quality of water supply services is difficult because the LWSN is predominantly a pumped system with few elevated storage tanks to reduce direct pumping which is more negatively affected by electricity outages.

The pumps and valves are operated based on both the time of the day and the water levels in the tanks or reservoirs (LWSC, 2017c). These duo control aspects (using pumps and valves) are applied both by the production division which supplies water to the zones and by the zones which try to balance supply durations between DMAs with different water sources (Simukonda et al., 2018a). While some pumps are switched on and off automatically, all valves are operated manually. This means, a lot of man hours are used in the operation of the valves.

Modelling complexity

Modelling of the LWSN has several challenges because of several factors. One of them is the lack of updated information on the topology and water consumers. This results in the difference between the elements of the WSS as found in the field and the hydraulic model. This is because some of the activities by the field teams such as short pipe connections and permanent closure or opening of valves are not reported to the GIS team. Moreover, because of the limited number of modellers, some extensions that may be known by the GIS team are not reflected in the hydraulic model. Furthermore, outdated consumers database results in wrong demand allocation on the demand nodes. These factors are difficult to resolve in the highly connected LWSN and ultimately affect the accuracy of modelled results against the measured values.

The other complicating factor is the lack of measured data. Since DMA inflows and outflows are not comprehensively measured and only about 66 % of the consumers are metered, water consumption demands and NRW values used in modelling are merely estimates (NWASCO, 2018). To improve the model accuracy, good calibration data is required (Walski, 2017). However, lack of measured data precludes the existence of good data thereby making calibration difficult and the use of hydraulic models only becomes possible with good knowledge about them and the WSS (AWWA Water Distribution Model Calibration Subcommittee, 2013).

The absence of modelling tools for IWS is also a factor. This is the problem that Chapter 4 attempts to solve. In the meantime, without an appropriate PDA modelling tool, even the results of the WaterGEMS model which were obtained from DDA simulation runs are not completely accurate. This implies the calibration of the WaterGEMS model using DDA simulations (as was done in Brian Colquhoun Hugh O'Donnell and Partners, 2010) is inevitably not completely accurate, but the hydraulic model is still useful (AWWA Water Distribution Model Calibration Subcommittee, 2013).

The high hydraulic connectivity between zones or DMAs adds another dimension to the complexities, especially when coupled with the implementation of the intermittent supply mode. There are times when certain DMAs are scheduled to have no water supply, but due to the connectivity, these DMAs have water. This leads to differences between the schedules and the actual supply duration. There is also the pipe filling process which makes consumers far away from the source to start receiving water long after supply is resumed (De Marchis et al., 2010). The effect of time lag means the actual water supplied to some nodes is not as simulated under the assumption that all nodes start receiving water as soon as supply resumes.

Due to the complexities of the LWSN, comprehensive analyses for the whole network are not tractable. As a result, the model cannot reliably be used as an analysis tool for the effects of operating the system under different scenarios. To reduce the complexities, smaller and semi-autonomous subsystems can be used. In this case, a zone is used because for operation and management, the LWSN is divided into zones, but the zones are hydraulically interconnected (subsection 5.6.4). For one of the zones to be analysed individually, it has to be hydraulically isolated. However, isolation of a zone of an IWSS with numerous water sources and inadequate measurements is a challenge. In cases where the flow and pressure of the water entering or leaving a zone are consistently measured, the boundary conditions are set using the measured values. Where such measurements do not exist, setting the boundary conditions must rely on simulation values obtained before the zone isolated. A methodology is developed in this section which is used to isolate the Chelstone zone from the whole LWSN.

5.4 **Chelstone zone**

Chelstone zone has 15 DAMs (Figure 5.14). The zone is selected for the application of the simulation method for IWSS with scenarios because it has the highest number of pipes with field measurement data (20 as compared to the second highest zone with 15 pipes). Moreover, it only has 2 interconnections with other zones unlike the other four zones which have more than 2 interconnections. The small number of interconnections with other zones entails reduced errors during its isolation from the rest of the LWSN. Furthermore, the flow at the connection points for the Chelstone zone is in one direction unlike the other zones where some connection points have flows that reverse directions at different time steps which increases the chances of more and larger isolation errors. The zone is also representative of the characteristic of the whole LWSN because besides multiple water supply schedules, it has multiple water sources. There is one surface water source through the main LWSN distribution reservoir and 15 boreholes. Some of these boreholes supply water to the ground reservoir before it is pumped to the tower for distribution, some supply directly to the distribution system while others supply both to the ground reservoir and directly to the distribution system.

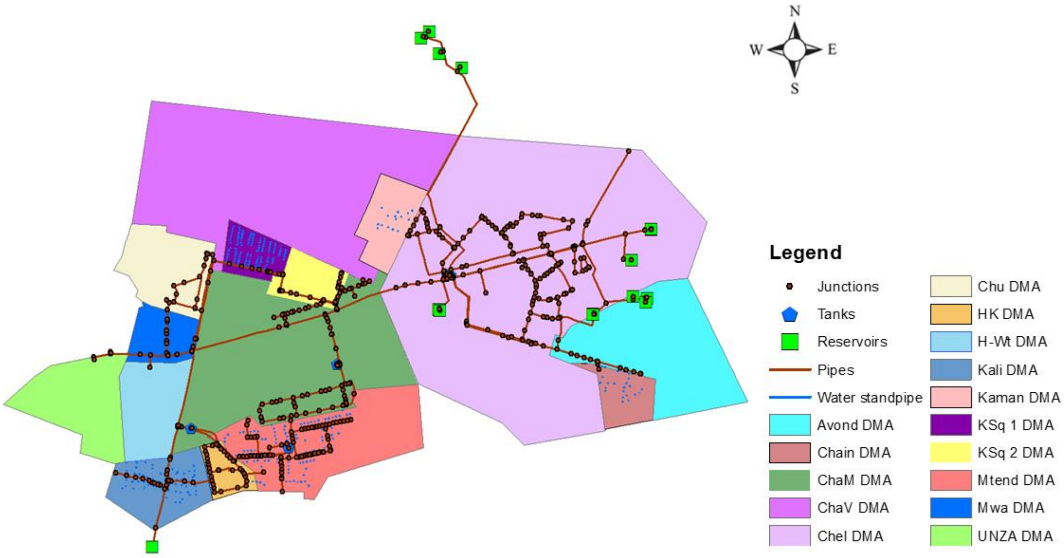


Figure 5.14: The Chelstone zone network and DMAs

The isolation of the subsystems such as zones and DMAs is important for consumption demand and leakage management. For CWSS with adequate pressure, the isolation of subsystems poses no challenges as is the case with IWSS. For IWSS in which subsystems share water sources and are connected via distribution pipes, the isolated subsystem should maintain the amount of water supplied to it before its isolation. If this is achieved, then increases in the water supplied to the isolated subsystem would not be arbitrarily, but match either the planned overall water supply increase for the entire system or plausible increases postulated in scenarios. However, in the literature, typical isolated IWS subsystems are those with their own water sources independent of others (Klingel and Nestmann, 2014) or temporary have increased water supply to reach continuous supply status during the study period (Andey and Kelkar, 2009). For these, efforts needed to maintain hydraulic conditions that existed before the isolation do not arise. However, for planning analyses, such as scenario analyses, that require proportionate allocation of the limited amounts of water to all the zones (Chapter 7), modelling techniques that allow the predetermined amount of water to flow to the isolated zone are required. Such modelling techniques are uncommon in the literature. The zone isolation process discussed in this section includes a new technique for isolating a zone in such a way that the pressure and flow conditions before isolation are closely maintained in the isolated zone. The technique provides a way of limiting how much water should be supplied to the zone thereby making it possible to proportionately supply water to the isolated zone when there is an increase in the total supply quantity for the entire WSS. The isolation process involves the identification of the zone hydraulic connection points to other parts of the WSS, disconnection of the zone from the other zones at these hydraulic connection points, representation of the zone borehole pumps as negative demand nodes and modelling of the water offtake point(s) from the distribution mains.

5.4.1 *Identification of the zone hydraulic connections to other parts of the WSS*

The first step in the zone isolation process is the identification of the points where the zone is hydraulically connected to other parts of the whole network and to characterise the zone water sources. For the Chelstone zone, two FCVs are identified as the hydraulic connection points to other parts of the whole LWSN. These are the FCV with the identification name of GV-362 which connects the DMA in the Lusaka Central zone to the water supply for Chelstone zone through node J-280 and the one identified by FCV-65 which is on the distribution main that connects the Chelstone zone to the main water distribution reservoir through node 4739-BBB (Figure 5.15a). Valve GV-1202 also connects Chelstone and the Lusaka Central zone, but it is closed. Concerning internal water sources, Chelstone zone has 15 boreholes as enumerated in Table 5.11.

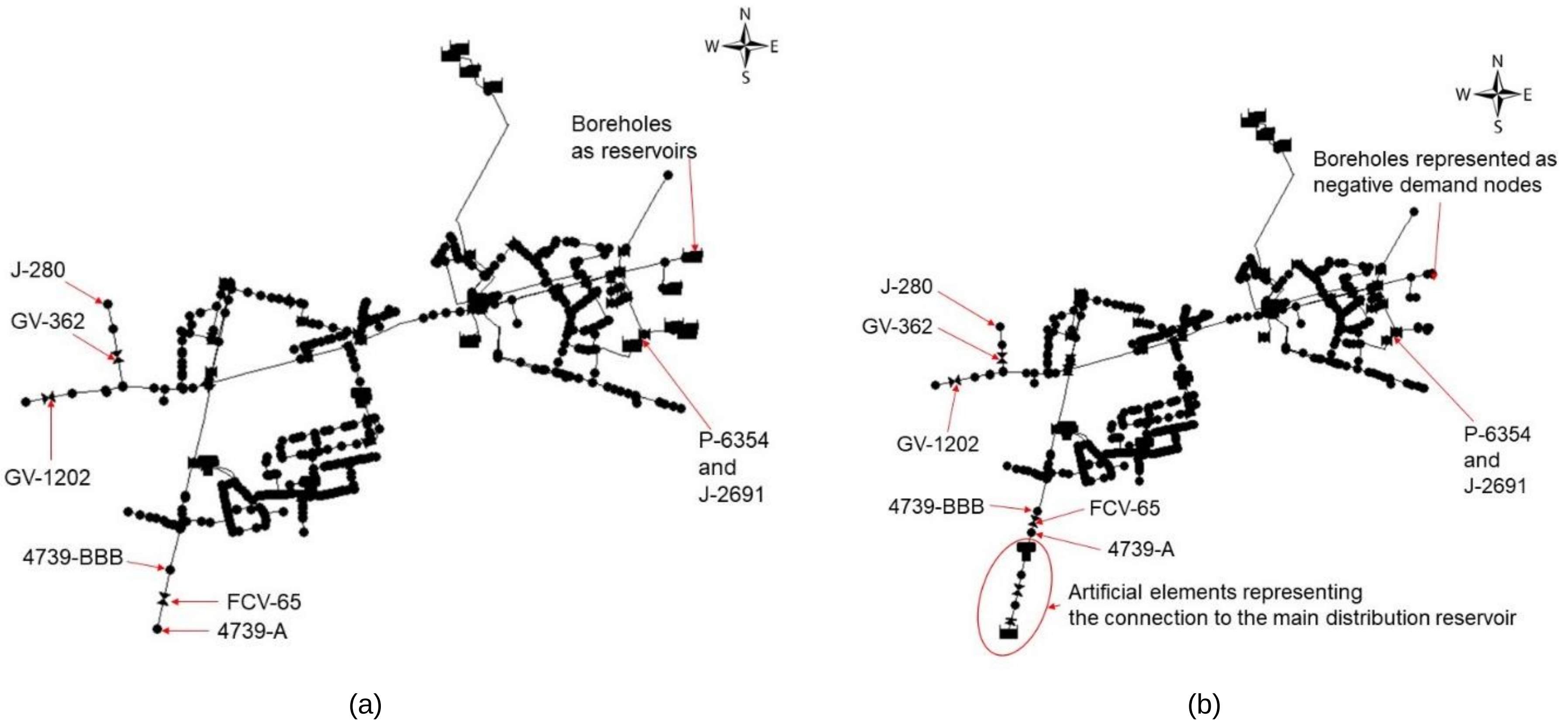


Figure 5.15: Isolated Chelstone zone (a) before and (b) after the modification of hydraulic connection points and replacement of borehole pumps with negative demand nodes

Table 5.11: Chelstone zone boreholes

No	Borehole
1	Avond1
2	Avond2
3	Avond3
4	ChelBH1
5	ChelBH2
6	ChelBH3
7	NewAvond1
8	NewAvond2
9	MaloF1
10	MaloF2
11	MaSh (Not functional)
12	NRDC4(B6-3/55)
13	NRDC2(B10-30N)
14	NRDC1(B6-3/55)
15	NRDC3

5.4.2 *Disconnection of the zone from the downstream zones*

The second step involves disconnecting the zone from all those parts of the network that are supplied water through the zone. In this case, only one DMA in the Lusaka Central zone is supplied water via Chelstone zone node J-280 (Figure 5.15). The hydraulic conditions of this node after isolation should not deviate from the conditions before the isolation. To determine the DMA demand that is imposed on node J-280, the 24-hour flow values through GV-362 are obtained from DDA simulation results of the whole LWSN prior to the isolation. DDA is used at this stage because the connection to the main distribution reservoir is represented by an artificial reservoir (AR) while the necessary regulatory artificial elements are not yet implemented (Figure 5.15b). With only the AR without the regulatory artificial elements, using PDA would result in excess water flowing into the network through node 4739-BBB. Using the DDA simulation values, the base demand for node J-280 is the average flow value determined by:

$$Bd_{node} = \left(\frac{\sum_{t=1}^{T} Q_{tGV}}{T}\right) \tag{5.6}$$

Where Bd_{node} is the representative base demand for the node connecting the zone to other zones or water boreholes. T is the total simulation duration (24 hours), Q_{tGV} the flow through GV-362 at time t.

The time variation of demand is implemented through the supply schedule (demand pattern) for which the multiplication factor at each time step t (mf) is calculated from:

$$mf = \frac{Q_t}{Bd_{node}} \tag{5.7}$$

The modified connection point between the Lusaka Central zone DMA and the Chelstone zone does not significantly affect the discharge through GV-362 and pressure heads at J-280 as shown in Figure 5.16 and Figure 5.17 respectively. In Figure 5.16, the flows before and after isolation modifications are practically identical with the maximum error of 0.017%. The error for each time step t ($Error$) is calculated by:

$$Error = \left|\frac{F_A - F_B}{F_B}\right| * 100 \tag{5.8}$$

Where F_B and F_A are flow values through GV-362 at time t before and after isolation modifications respectively.

There are some differences in pressure values especially between 05:00 am and 08:00 where the pressure values after the zone isolation are noticeably higher than those before (Figure 5.17). The maximum error is 5.38%.

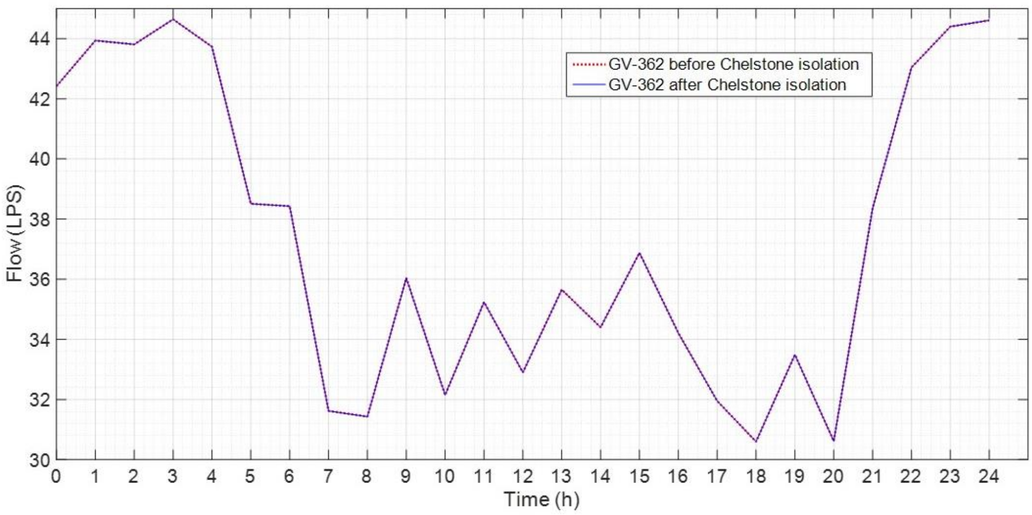


Figure 5.16: The 24 hours variations of flow through GV-362 before and after isolation

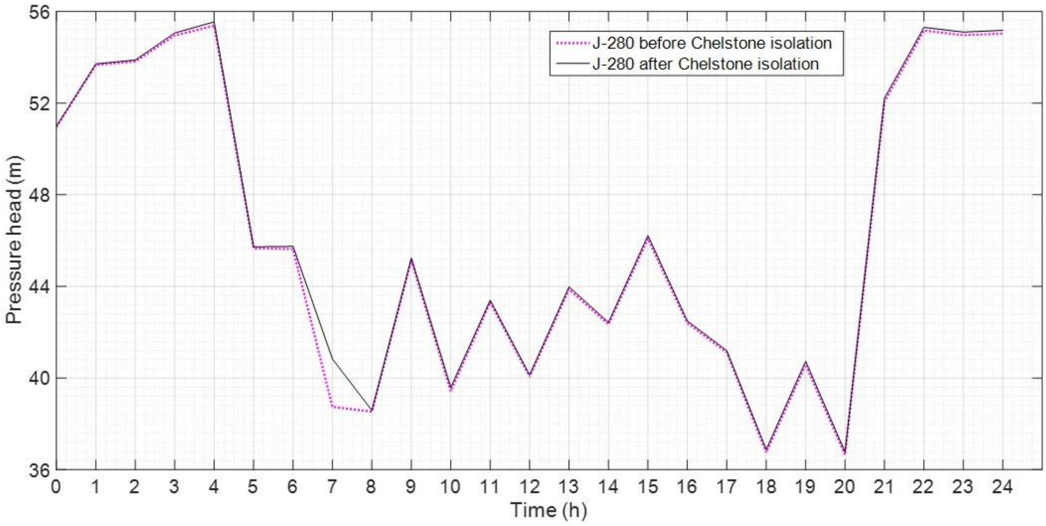


Figure 5.17: The 24 hours pressure variations at J-280 before and after the isolation

5.4.3 *Representation of the zone boreholes as negative demand nodes*

For boreholes from which water is pumped directly into the distribution system, there is a possibility of pumping rates being more than the borehole yield capacities. This is when the pressure at some nodes in the distribution systems reduces due to increases in water demand. This possibility would be there for pumps 1-11 (Table 5.11) that pump directly into the distribution system as well as the reservoir.

To prevent pumps in the model from pumping beyond the boreholes' yield capacities, pumps for boreholes (1-11) are replaced by negative base demand (Bd_{pump}) (Edwards, et al., 2006). For this, Equation 5.6 is used by replacing the flow (Q_{tGV}) by the negative pump discharge value ($-Q_{pumpt}$). The pump supply pattern multiplication factors (P_{mf}) are developed for each replaced pump using Equation 5.7 by substituting Q_{tGV} and $Bd_{node}$with Q_{pumpt} and Bd_{pump}) respectively. At each time step t, the actual supply from each borehole is the product of the base demand for the pump (Bd_{pump}) and the P_{mf}.

Since there are many pumps, the flow from pipe P-6354 and one of its end nodes (J-2691) (Figure 5.15) are used for comparing the *before* and *after* situation for boreholes. The 24 hours variation of the flows and pressure are shown in Figure 5.18 and Figure 5.19 respectively. In both Figures, there is a deviation in the last simulation hour reading to the maximum error for flows and pressure of 3.4% and 43.0% respectively.

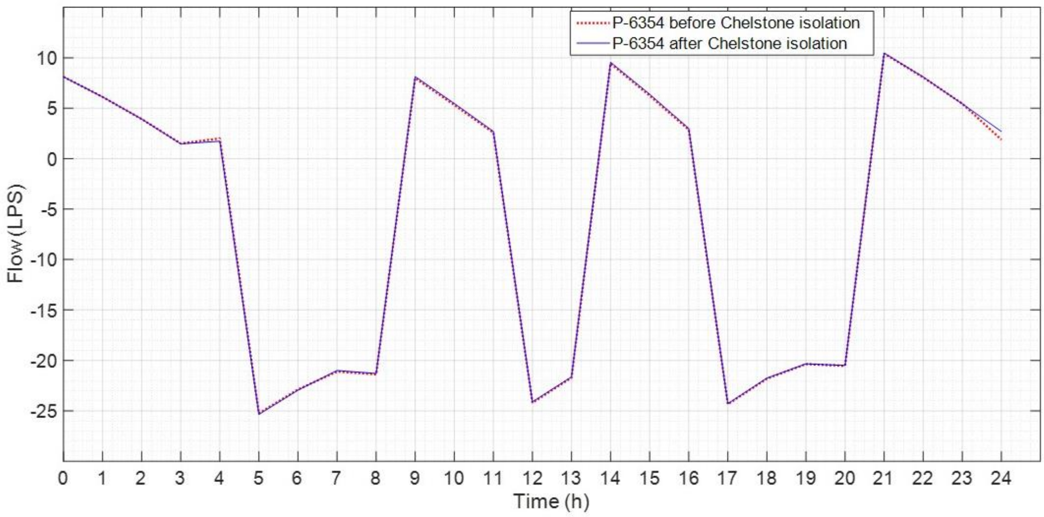


Figure 5.18: The 24 hours variations of flow through P-6354 before and after isolation

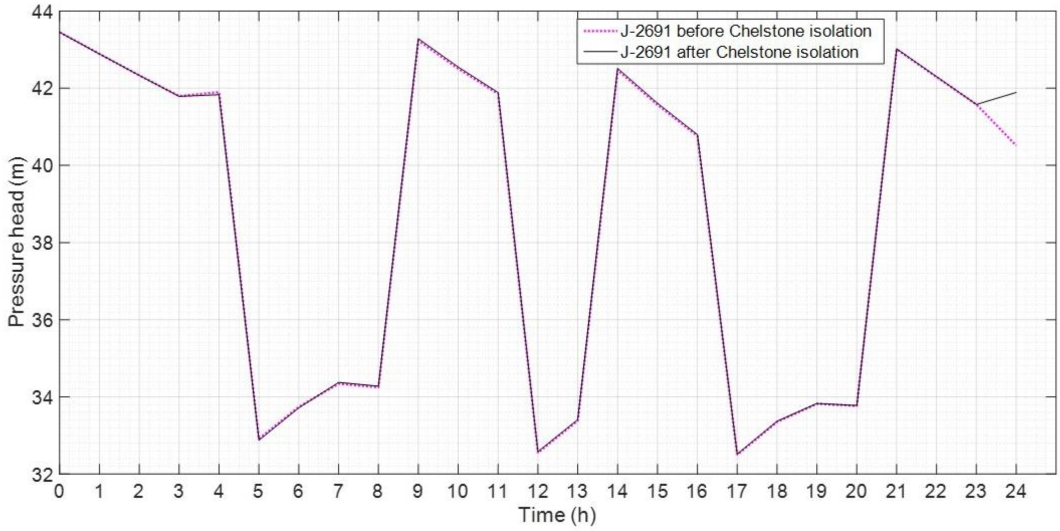


Figure 5.19: The 24 hours pressure variations at J-2691 before and after the isolation

5.4.4 *Chelstone zone water offtake point from the transmission main connected to the main distribution reservoir*

The pipe that supplies water from the main distribution reservoir to Chelstone zone has two hydraulic connections (through pipes P-312 and P-318 both not shown on the maps) with parts of Kabulonga zone before node 4739-A (Figure 5.15). Since the flows across these connections reverse directions as the day progresses (Figure 5.20), it shows that the hydraulic head at the water offtake point is influenced by both the main distribution reservoir and Kabulonga zone. In both pipes that join the Kabulonga zone to the supply pipe for Chesltone zone, flows reverse direction at about the same time. Negative flow values show that the total head is higher in the Kabulonga zone resulting in the flow being towards the lower head in the pipe from the main distribution reservoir to the Chelstone zone. The positive flow values show that the flow is from the main pipe to Chelstone which is at a higher head to the Kabulonga zone which is at the lower head. This influence of another zone on the hydraulic condition of the water flowing to the Chelstone zone is taken care of by modelling the water offtake point (main water source for the Chelstone zone) downstream of the joining points for the pipes linked to Kabulonga zone. Two aspects are considered when modelling the water offtake point to implement EPS. These are the water source including its diurnal head variations and the water demand of the isolated zone plus the diurnal flow variations. The details of the modelling technique for the water offtake point are discussed below.

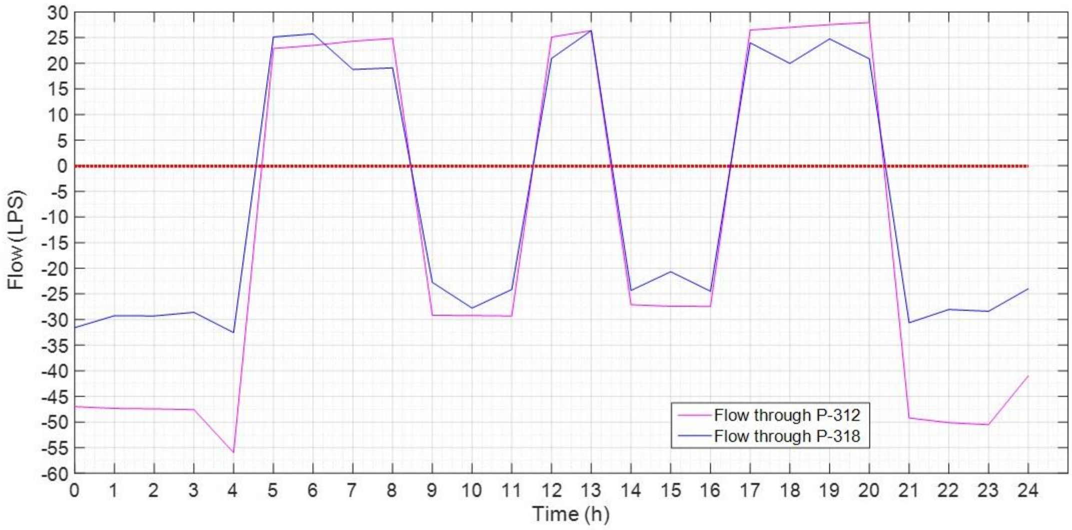
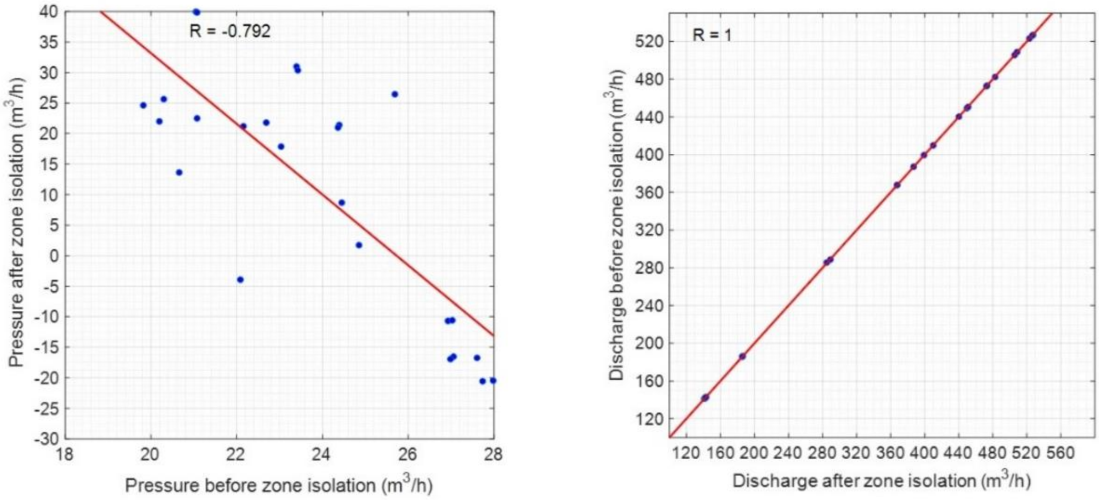


Figure 5.20: Interaction between Kabulonga zone and the pipe supply water to Chelstone zone before node 3749-A

5.4.5 *The modelling technique for the water offtake point*

The water offtake point for Chelstone zone is modelled in such a way that it performs two functions. The first is to ensure that the EPS flow and pressure variations before and after the zone isolation are as close as possible. The second function is to ensure that the total flow to the zone is according to a predetermined value. This aspect is important when dealing with scenarios where water supply and demand values are varied (Chapters 6 and 7). To achieve these, the first attempt is to model the water offtake point using the negative demand technique. The technique fails because it results in node 4739-BBB pressure values after isolation that differ greatly with those before isolation as shown by the negative correlation coefficient (Figure 5.21a). This is despite the exact match of flows through pipe P-1758 (Figure 5.21b).



(a) (b)

Figure 5.21: Negative demand technique correlation coefficient for (a) node 4739-BBB pressure before and after zone isolation (b) pipe P-1758 flow before and after zone isolation

Following the failure of the negative demand technique, trials involving six different combinations of artificial nodes and valves (Figure 5.22) are implemented first and the combination that performs the best is used to model the water offtake point in this chapter and Chapter 7. The artificial elements involved are the artificial reservoir (AR), the first artificial node (AN1), the second artificial node (AN2), artificial check valve (ACV), artificial tank (AT), artificial flow control valve (AFCV), Artificial general purpose valve (AGPV) and the artificial throttle control valve (ATCV). Node 4739-A, FCV-65 and node 4739-BBB which are common in all the arrangements are not necessarily part of the artificial elements as they are originally part of the WSS elements, but they are used as part of the water offtake point modelling arrangement.

The AR and the ACV are also common artificial elements for all the arrangements. The AR represents the water source which is the distribution main point where supply to Chelstone zone starts. Where water offtake points have continuously measured pressure variations, the hydraulic head variations can be determined by the sum of the fixed elevation and the measured varying pressure heads at each time step. However, since in this case there is no pressure data, the hydraulic head variations are obtained from the PDA simulations of the whole network prior to the zone isolation. The 24 hours average of the simulated hydraulic heads (calculated using Equation 5.6) is used in Equation 5.7 to determine the head multiplication factor (h_{mf}) at each time step t. The h_{mf} is used to vary both the reservoir head and the AT hydraulic head. The ACV is there to prevent backflow in situations where the hydraulic head downstream the AR is higher than that of the AR. The diameter of the ACV is equal to that for the distribution main from the main distribution reservoir. The elevation of the AN1, which is used as one of the end nodes for the ACV, is equal to that for node 4739-A.

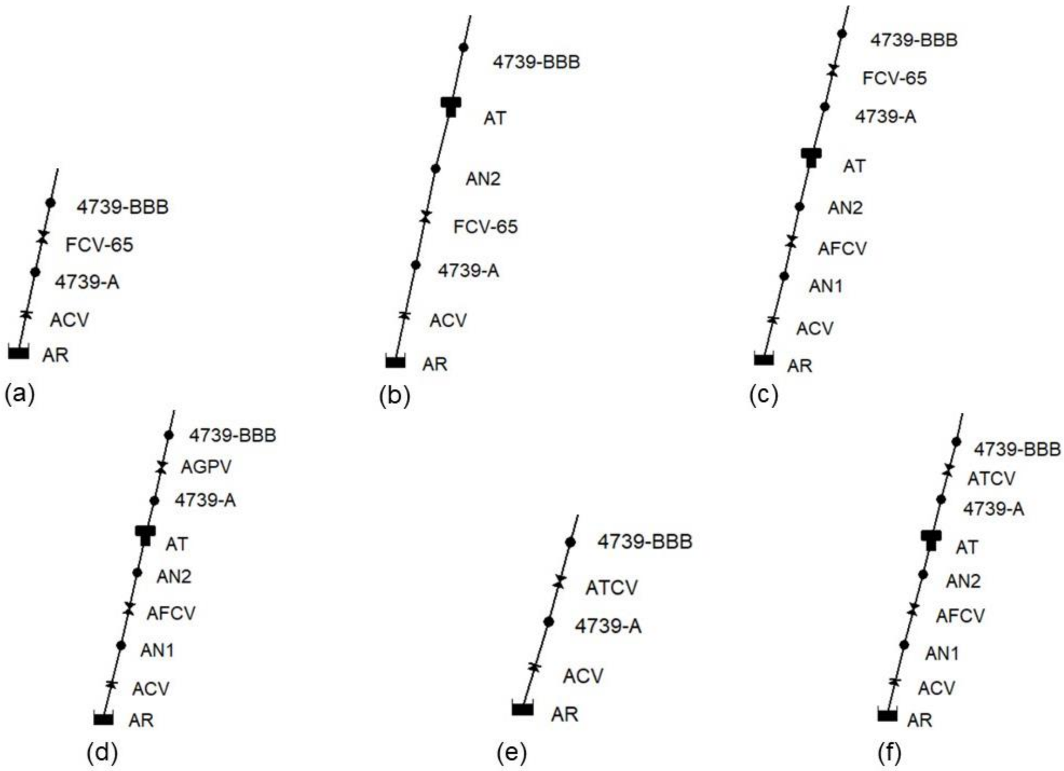


Figure 5.22: Six trials for modelling the water offtake point

From all these trials, the sixth arrangement (Figure 5.22f) gave the best performance and is used in Chapter 7. Details of the analyses and results are provided in Appendix A from Figure A.1 to Figure A-27.

5.5 Conclusion

The information presented in this chapter shows that the LWSN is supplied water from multiple sources. The network has limited capacity because of three major contributing factors. These are the high NRW, demographic dynamics (increasing population growth and urbanisation) and the low capacity of the WTP.

The chapter highlights the hydraulic interconnectivity of the zones and DMAs. The interconnectivity is either because different zones are supplied water from the same distribution reservoir(s) or because the zones and DMAs’ pipe networks are connected at many boundary points. This interconnectivity makes it difficult to attribute reservoir or tank water level variations to the water consumption of a single zone or DMA.

For the modelling process, the chapter discusses how the WaterGEMS 3.0 version is converted to an EPANET 2.0 hydraulic model. To ascertain the goodness of the existing residential area-based DMAs, clustering of the converted model is done using graph theory implemented in GEPHI. To visualise the clusters clearly, a technique for developing distribution maps for node parameters such as DMAs based demand patterns or supply schedules and pressure (developed in Chapter 4) is used. Differences and similarities are noticed between the DMAs based township names and those identified using clustering (modularity index). These differences are due to the combination of some DMAs that use the same supply schedule resulting in the clustering based delimitation to show one DMA and skeletonization of the WSS network in the model which makes the clustering technique fail to detect DMAs that are represented by very few nodes. Using the DMA based on the clustering technique, the existence of parallel pipes is observed. This cannot be noticed with the township area-based DMAs. These merits and demerits of each DMA delimitation approach show that to have proper demarcation of DMAs, both methods should be used coupled with knowledge-based judgment.

The chapter discusses factors that make the operation of the LWSN and its modelling a challenge. Modelling challenges include lack of updated information on the topology and consumers, non-existent of the modelling tool, the high hydraulic interconnectivity of the zones or peri-urban area agglomerations or DMAs and lack of measured data. The lack of updated information and data is overcome using secondary data. However, the complexities of the network prevent the thorough analysis of the system, especially using scenarios. Hence, rather than simulating the behaviour of the whole LWSN, a zone by zone approach to the analysis of the WSS is seen to be useful.

In the chapter, a technique for isolating zones is developed. The technique that employs a series of artificial valves and nodes to model the water offtake point, ensures that the total flow to the zone after isolation closely matches the total flow before isolation. It also ensures that the 24 hours flow and pressure variations at the offtake point after zone isolation do not differ completely to those before. With these controls at the water offtake point, the behaviour of the isolated zone is maintained and the delivery of pre-set quantities of water is possible. This aspect is important in the modelling of the isolated zone using scenarios under conditions that dictate that the limited and insufficient water resources should be shared amongst the various zones in proportional to the demands as discussed in Chapters 6 and 7.

6 APPLICATION OF SCENARIOS TO THE LUSAKA WATER SUPPLY NETWORK

In this chapter, the GSG scenarios and the water supply sustainability scenarios developed in Chapters 3 are applied to the real world WSS discussed Chapter 5. The various chapter sections highlight aspects that are critical in understanding the environment in which a WSS is operating and factors that would determine the success or failure to meet planned WSS improvement targets.

Section 6.1 discusses the current and future development targets for Zambia. These lead to the identification of Zambia's current and future situation from the GSG scenarios perspective. Section 6.2 discusses the current and future state of the Zambian water supply sector. The current and desired future state of the Lusaka city is presented in section 6.3. Section 6.4 discusses the current and desired future state of the LWSN which leads to the application of the water supply sustainability scenarios in section 6.5. The quantitative application of the developed water supply sustainability scenarios to the LWSN is discussed in section 6.6. The chapter conclusion is presented in chapter 6.7.

6.1 Zambia's current and desired future development state

6.1.1 *Current development state*

To understand the operational environment for water utilities in Zambia, the major development aspects and the direction of the economic policies of the country must be understood. Zambia's seventh national development plan is anchored on the Vision 2030 and regional, international, bilateral and multilateral development strategies (Government of the Republic of Zambia, 2017). The most prominent of the international strategies are the declarations of the MDGs and SDGs in which sustainable development and resilience are major concepts. For sustainable development, both the MDGs and SDGs emphasise the balance of the three dimensions of sustainable development: social, environmental and economic or people, planet and prosperity (3Ps) (United Nations General

Assembly, 2015). In Zambia, sustainable development is one of the national values and principles enshrined in the national constitution (Government of Zambia, 2016). However, Zambia's development agenda is biased more towards economic growth which is necessary for poverty reduction. Zambia's emphasis on economic growth can be understood because currently, the country's economic growth rate is less than 3%, yet for the purpose of poverty reduction it should at least be 7% (Blair et al., 2005). To improve economic growth, the major directions of Zambia's economic policies are (Government of Zambia, 2016):

i. Creation of an economic environment which encourages individual initiative and self-reliance among the people, promoting investment, employment and wealth creation.
ii. Promotion of the economic empowerment of citizens so that they contribute to sustainable economic growth and social development.
iii. Promotion of local and foreign investment and protecting and guaranteeing such investment through agreements with investors and other countries.

In Zambia therefore, environmental sustainability is not a priority and sustainable development is dominantly discussed in socio-economic terms rather than social, economic and environmental terms. The equitable access to - and protection of land, the environment and natural resources are provided for in the Zambian constitution, but the provisions are not justiciable such that the claim of lack of resources by the government is enough to justify the failure to meet the requirements of these constitutional provisions (Republic of Zambia, 1996). This dependency on the availability of resources to meet social, economic and cultural human rights obligations is an international practice (United Nations General Assembly, 1966) though applied to varying extents depending on the country's economic development classification. Thus, there is a general failure to enforce policies and laws when it comes to social, land, environmental and natural resources issues such that the poor are always disadvantaged (Wragg and Lim, 2015; Cerutti et al., 2018). Social inequalities and inequities between the affluent and the poor are high because such disparities are promoted by policy directions. Under this development approach, improving the conditions of the people living in peri-urban areas and preventing environmental degradation is difficult because of high poverty levels. Poverty reduction is one of the major targets of the MDGs and SDGs concerning developing countries, and in Zambia, poverty reduction seems to be the focus of every development programme. However, that is mere rhetoric. As it is, reducing poverty and economic inequality amongst the population continues to be one of the major challenges in Zambia (Lusaka City Council and Environmental Council of Zambia, 2008, Central Statistical Office, 2016). Poverty in all its forms has increased in the country since 2006 (Government of the Republic of Zambia, 2017). Four aspects are noteworthy.

1) The perpetual claim of the *lack of resources* by the Zambian government as a reason for the failure to meet the social and environmental requirements means the current development activities are not sustainable as they are based on resources that cannot meet even the current generation’s needs.
2) The lack of resources coupled with the ever-acknowledged lack of skilled manpower and institution capacities, implies in Zambia systems which include WSS are not resilient.
3) The formulation of policies that are targeted to reducing poverty which is supported by public political pronouncements seems to show that the current national state is represented by the PR scenario of the GSG scenarios. However, there is no political will to effectively enforce the policies and legal provisions. Thus, the free market economy policies, lack of transparency, poor policy and law enforcement and limited emphasis on environmental sustainability show that the current state of Zambia fits the MF scenario of the GSG scenarios (discussed in Chapter 3).
4) With high poverty levels, the financial sustainability of improved water supply services in Zambia requires strategic planning of both the infrastructure and tariff structures. This aspect is important as it is at the centre of a sustained water supply improvement programme and is the genesis of an implicit controversy between the need to supply water services equally and equitably to all consumers regardless of their status and the sustainability of the provision of these services.

6.1.2 *Desired future development state*

Zambia's vision as contained in the Vision 2030 document is to become a Prosperous Middle-Income Nation by 2030. In this document part of the national vision is (Republic of Zambia, 2006):

> "*to have an economy which is competitive, self-sustaining, dynamic and resilient to any external shocks, supports stability and protection of biological and physical systems and is free from donor dependence.*"

This vision is to be attained through the implementation of the 5-year National Development plans. The desired socio-economic development in this vision is to attain and sustain annual real growths of 6 percent (2006-2010), 8 percent (2011-2015), 9 percent (2016-2020), and 10 percent between 2021 and 2030 (Republic of Zambia, 2006).

During the 5th National development (2006 – 2010) and the 6th National development plan (2011-2015), economic growth averaged at 6.9%. The implementation of these plans was negatively affected by (Government of the Republic of Zambia, 2017):

i. Poor funding and unpredictable budgetary releases of funds
ii. Insufficient resources for planned human development programmes
iii. Failure to implement the decentralisation policy (mainly fiscal decentralisation)
iv. Poor coordination among various implementing entities and stakeholders mainly due to limitations in the institutional arrangements and technical capacities
v. Poor application of the monitoring and evaluation information and the general poor data availability

Unlike the earlier national development plans where sectoral-based planning and development was used, the 7th National development plan (2017-2021) has an integrated multi-sectoral approach under the theme, "*accelerating development efforts towards the Vision 2030 without leaving anyone behind*."

Notably however, while the aspirations in the Vision 2030 and the plans provide a lot of hope for the future of the country on paper, what is going on shows a different future. The economy and basically all systems in the country are crumbling and are not resilient except the people themselves. The projected growth rate of 9% (2016-2020) has not been met instead the growth rate has been less than 3% since 2015 (Government of the Republic of Zambia, 2017). Improving the economy soon such that the growth rate would reach 10% between 2021 and 2030 is not practically feasible because of the debt burden the country is facing (National Assembly of Zambia, 2017). The period of the 7th National development plan is coming to an end, rather than accelerated development, it is decelerated and rather than leaving no one behind, many Zambians are left behind. Poverty levels are increasing and the gap between the *haves* and *have nots* is widening. Concerning WSS, the future perspective is uncertain given the already existing controversy in the country that water supply tariffs are too high because of high poverty levels and yet the tariffs are apparently too low as they do not cover the full cost of water supply (Simukonda et al., 2018a).

6.2 The state of Zambia’s water supply sector

6.2.1 *The current state of the water supply sector in Zambia*

The performance of the water supply sector in Zambia is greatly affected by the poor economic performance of the country which greatly depends on governance. In the analysis of root causes of IWSS (Chapter 2, Simukonda et al., 2018b), governance is found to be the major root cause that affects all others. In the analysis of drivers of change (Chapter 3) governance is found to be the major driver. Moreover, poor leadership and governance have been cited as the real and fundamental reasons for poor water supply in developing countries (Biswas and Tortajada, 2010). Aspects of poor governance in Zambia concerning the water sector include corruption that acts as a drain to the national resources (Government of the Republic of Zambia, 2017), failure to implement relevant policies, unclear roles of institutions (ministries) and political interference (Chapter 5). Consequently, the water supply sector currently faces many challenges which include poor investment in infrastructure, electricity outages, high water losses and many others as discussed in Chapter 5, Simukonda et al.

(2018a) and in urban and peri-urban water supply and sanitation sector reports (NWASCO, 2019b).

6.2.2 *The desired future state of the Zambian water supply sector*

According to the Vision 2030 document, all Zambians should have access to clean water supply by 2030 (Republic of Zambia, 2006). However, docility and complacency of Zambians are a problem as noted in the Seventh Development Plan (Government of the Republic of Zambia, 2017):

> *"**Docility and complacency** – Zambians have of late developed a trait of being docile and complacent in the business of their daily lives. This has made citizens accept sub-standard products and services and violations of their human rights. Therefore, there is need to inculcate a spirit of alertness, activism and active engagement based on appropriate and relevant knowledge."*

Consequently, to improve water supply services in Zambia, especially for the poor, at least the PR scenario or better still the NSP scenario (discussed in Chapter 3) should occur. For that, civil society organisations should do a lot to create awareness among the citizens so that they can know that water supply services can be improved and that CWS is better than IWS for individual water consumers, the society and utility companies. This awareness coupled with an understanding that there is more to poor water supply services than mere lack of resources can engender pressure on leaders which can result in strong political will to change the policy direction and to ensure that clear and firm decisions are made, and future-oriented actions are taken for the good of all Zambians than a few corrupt elites. Then resources would be available to improve water supply services and donor aid will be effective.

6.3 The current and desired future state of the Lusaka city

6.3.1 *Lusaka city's current state*

Due to population growth and urbanisation coupled with the complex land delivery process which involves the formal and informal methods, settlement patterns in

Lusaka are difficult to monitor and control. The formal land delivery system enables development which involves relevant planning authorities such as Local Authorities as provided for in many Acts of parliament including the Urban and Regional Planning Act (Republic of Zambia, 2015). However, this method of land delivery is prone to corruption as it has many, long and costly procedures that mainly favour the affluent (Lusaka City Council and Environmental Council of Zambia, 2008). Plots illegally created by subdividing land for public use such as play-parks, markets, schools, etc. have been corruptly legalised (Lusaka City Council and Environmental Council of Zambia, 2008). Thus, owing to the government's economic policies that are promoting private initiatives, private sector driven development of Lusaka city which only benefits the affluent minority continues unabated in ways that appear neither transparent nor planned (Wragg and Lim, 2015). The other formal method is through institutions that build and sell completed houses. These too cater for the affluent minority as the houses are too expensive for an average Zambian (Wragg and Lim, 2015).

The informal land delivery system is chiefly exercised by traditional leaders and it is the system that gives most of the land. For instance, in Lusaka, 60% of the land is delivered through this method (Lusaka City Council and Environmental Council of Zambia, 2008). For over two decades, political party cadres have also been involved in the illegal land allocation. Thus, the informal land delivery system has led to the illegal demarcation of plots and building of structures haphazardly which has promoted the mushrooming of unplanned settlements called peri-urban areas (Lusaka City Council and Environmental Council of Zambia, 2008, Simukonda et al., 2018b). In Lusaka city, about 70% of the population is found in these areas (Lusaka City Council and Environmental Council of Zambia, 2008). As a reflection of the national trend, the current state of Lusaka city, therefore, reflects the MF scenario where the affluent have more opportunities to resources than the poor. Even consumerism is high in the city (Wragg and Lim, 2015) and the income disparities between the affluent and the poor are high.

6.3.2 *The desired future state for Lusaka city*

The desired future state of the city is that by 2030 it should be Economically strong, **E**nvironmentally Friendly and provide the **C**ommunity with **H**ope and **O**pportunity (ECHO) Garden City (Figure 6.1). To achieve this, the planners propose a sustainable environmental development strategy in which there is supposed to be a balance amongst environmental conservation, environmental use and urban development (Ministry of Local Government and Housing et al., 2009). The sustainable environmental development strategy is proposed following a study undertaken by the Ministry of Local Government and Housing et al. (2009) that looked at the development approach for Lusaka city which would effectively and efficiently accommodate the rapid population growth, urbanisation and economic development (Ministry of Local Government and Housing et al., 2009). Having a planned and predictable city development pattern is vital for the effective provision of services such as water supply. This is because city development patterns do not only affect the water demand on different sections of the WSS infrastructure, but also its operation and maintenance (Farmani and Butler, 2014).

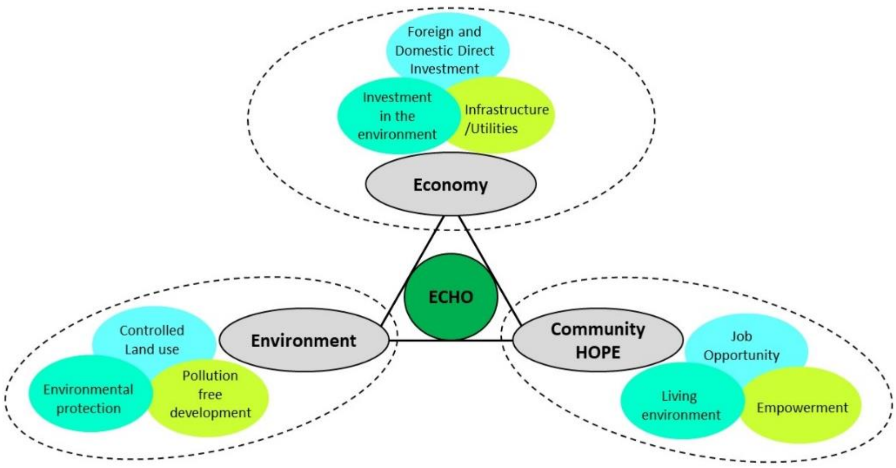


Figure 6.1: Development vision of Lusaka city by 2030 (Ministry of Local Government and Housing et al., 2009)

The common development approaches widely applied in urban planning are compact/uniform, monocentric, polycentric and edge developments (Farmani and Butler, 2014). Some similarities can be drawn from these and the development plan for Lusaka which has two components. These are the Lusaka city (district) as defined by its boundary and the greater Lusaka which includes satellite cities (towns) adjoining territories for Chibombo, Chilanga and Chongwe districts. With these components, three approaches were considered in the study reported in the Ministry of Local Government and Housing et al. (2009). These are:

i. ***New Urban Expansion*** *which considers satellite town formulation in urban fringe areas in combination with development that links these areas with existing urban areas.*
ii. ***Upgrading Existing City*** *which is based on a strong urban growth management for compact city formulation with intensive development in the interior area.*
iii. ***New Capital Development*** *by decentralizing the capital such as the development of twin city or new capital city in a remote area.*

Of the three approaches, the favoured concept is the *New Urban Expansion* (edge development). The argument for its selection is that it supports economic growth, is effective and efficient in absorbing the fast-growing population and urbanisation and provides benefits to adjoining districts (Ministry of Local Government and Housing et al., 2009). This future city expansion plan corresponds to the MF scenario (Farmani and Butler, 2014). In the existing inner city however, compact city development is envisaged (Ministry of Local Government and Housing et al., 2009) and this corresponds to the PR scenario (Farmani and Butler, 2014). Because of this setup, the development plan for the Lusaka city territory (inner city) is supposed to involve controlled dense settlements with efficient land use coupled with population density distribution and limiting urban expansion to within outer ring (Figure 6.2). The adjoining three Satellite cities (Chibombo, Chongwe and Chilanga-Kafue) are supposed to be self-sustaining cities with dense planned settlements.

The vision of the ECHO Garden city and the proposed sustainable environmental development strategy seem to be brilliant development approaches for Lusaka city or the greater Lusaka. However, three issues must be noted. The first issue is that the Economy aspects (Figure 6.1) take precedence over the Environment and Community Hope aspects. The second issue is that the lack of clear vision by the Zambian entities (Ministry of Local Government and Housing et al., 2009) including the water utility company can negatively affect the implementation of the planned development. The third issue is that all these planned developments squarely depend on donor support and the role of the private sector. This is a consequence of the poor state of the economy and the fact that government is overburdened with debts. Both donors and the private sector are strongly influenced by national governance which in this case is very poor. These coupled with poor skills in technology and management (Ministry of Local Government and Housing et al., 2009) make it unclear as to whether these developments will be accomplished within the planning horizon, and if they were accomplished whether they will last or not.

Figure 6.2: The development plan for the future Lusaka city showing the outer ring road and the satellite towns (Ministry of Local Government and Housing et al., 2009)

6.4 The current and desired future state of the LWSN

6.4.1 *The current state of the LWSN*

The current state of the LWSN is discussed in Chapter 5 and Simukonda et al. (2018a).

6.4.2 *The desired future state of the LWSN*

The desired future for the LWSN is not clear apart from the broad mission statement for the utility company which operates the network as (Lusaka Water Supply and Sanitation Company Limited, 2017):

> *"To provide quality water and sanitation services at commercially and environmentally sustainable levels to the delight of our customers and other stakeholders"*

The government acknowledged docility and complacency of Zambians implies what may delight them may not necessarily be of good quality. This means the desired future state for the LWSN is not clear. With this lack of clarity, the desired future for the LWSN can be extrapolated from five perspectives. These are the duration of water supply, the extension of the water supply network, consumption demand management, the reduction of NRW and the increase in the volume of water supply from the sources.

Duration of water supply

The future with respect to this perspective is blurred. While LWSN should have started supplying water continuously to all consumers six months after 1990 when it started operating as a private water utility (NWASCO, 2000; Republic of Zambia, 2011b), it still has most of the parts supplying water intermittently and there seem to be no measures by the utility company and the regulator towards attaining CWS status. The utility company is struggling to ensure that water supply duration by the LWSN meets the Service Level Guarantees agreed with the regulator in which the average duration of water supply is 22 hours (Lusaka Water Supply and Sanitation Company Limited, 2018a). The norm should be 24/7 water availability. However, concerning conversion from IWS to CWS, there seems to be no explicit indication from the utility company in that direction and nor is there any from the regulator other than the apparently disregarded guidelines of 2000 which required that networks such as the LWSN should have been converted to CWS six months after the incorporation of the utility company operating them (NWASCO, 2000). This lack of emphasis on the conversion to CWS is at variance with the planning documents upon which water supply improvement projects are based. In these documents, attainment of the CWS status by 2035 is the target (Ministry of Local Government and Housing et al., 2009; Republic of Zambia, 2011b; Millennium Challenge Account - Zambia Limited, 2013). This contrast in targets shows that awareness programmes on the planned conversion of the LWSN from IWS to CWS (by consultants) are not only important to consumers but also to the utility company (the network operators) as well as the regulator.

Extension (expansion) of the water supply network

Extension of the water supply network is supported in Lusaka because the number of residents connected to the network is still very low. The extension of the WSS is in line with SDG 6.1 that aims at ensuring access to safe and affordable drinking water for all by 2030 (United Nations General Assembly, 2015). It is also in line with the Zambian government’s vision of ensuring that there is 100% access to clean water by 2030 (Republic of Zambia, 2006). However, this kind of access includes water from hand-operated boreholes and networks operated under the satellite water supply sub-system (discussed in Chapter 5). For the main water supply system, the planned network expansions up to the year 2035 are shown in Figure 6.3. The problem with these network extensions has been poor planning or no planning at all because the network is extended to already unplanned and built up areas (Simukonda et al., 2018a; 2018b; Chapter 5).

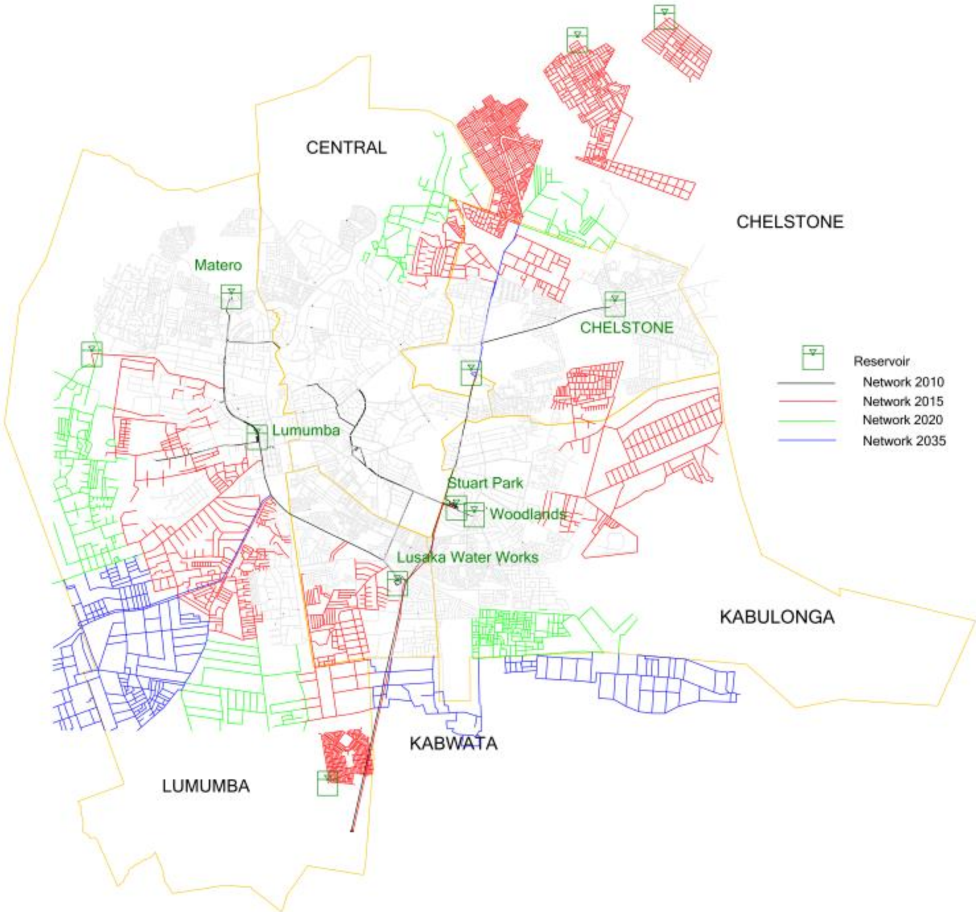


Figure 6.3: Planned LWSN expansion (Republic of Zambia, 2011b)

The planned city development pattern (Figure 6.1) can be compared with the planned LWSN extension for the period up to 2035 (Figure 6.3). A few aspects can be noted from the two planned developments.

i. The planned LWSN expansion is not covering the Chilanga and Chibombo satellite towns. This seems to be in line with the fact that the satellite towns will be self-sustaining. However, it is not clear as to how their WSS will be configured because on one hand, the two satellite towns are not part of the expanded LWSN, while Chongwe satellite town is part, but it is shown to be independent of the main LWSN. On the other hand, the calculation of the water to be abstracted from the Kafue river includes the demand for all the three satellite towns (Ministry of Local Government and Housing et al., 2009).
ii. Part of the city within the outer ring is not considered in the LWSN expansion plan. This means the planned LWSN expansions are not adequate to meet the expected consumption water demand from the expanding city.

iii. Many parts of the planned LWSN expansion are connected to the old system in several places, but without new distribution reservoirs or tanks. This absence of additional reservoirs means the expansion has not considered the problem of power outages which can be mitigated by increasing elevated storage capacity. Moreover, the multiple connections are potential problems for setting DMA boundaries. Since these planned network expansions were done without any functional hydraulic model, they are a potential cause of the WSS capacity reduction if implemented without proper analysis.

Consumption water demand management

Consumption demand management receives minimal attention in Zambia. Although there have been talks of reducing water consumption by washing cars using buckets and not hose pipes, brushing teeth using water in cups rather than keeping the taps running, nothing much has been done. People still use potable water to water gardens and lawns in winter. Unlike NRW, there are no set target levels of water consumption or clearly defined policy for efficient water use and consumption water demand management in Zambia (Republic of Zambia, 2008). WSS design standards values exist, but they are too high (Republic of Zambia, 2011b) and the block tariff structure (Lusaka Water Supply and Sanitation Company, 2018b) is not effective in deterring excessive water consumption by the affluent.

In Lusaka city, the use of alternative water sources such as greywater recycling can be beneficial in the dry season when there is high domestic water demand due to increased garden and lawn watering activities. Since potable water is used, switching to the use of recycled greywater would free the potable water thereby leading to improved water supply services. However, it should be noted that recycling of greywater is not a serious topic in Zambia and as such there have been no investigation on the merits and demerits of recycling greywater in the context of water demand management in Lusaka city. From the drainage perspective, handling of greywater is seen as a problem because of lack of drainage facilities and is not considered as an alternative water source in the National Urban Water Supply and Sanitation Programme 2011-2030 (Government of the Republic of Zambia, 2010). Concerning rainwater harvesting, Handia, et al. (2003) reported that it is a technology which exists traditionally in some parts of Zambia, but it is not widespread. Rainwater harvesting tanks are installed at very few schools in Lusaka city, but the tanks are normally dry in the dry season. With this lack of popularity, limited studies have investigated the impact of rainwater harvesting as an alternative source of water in the context of water demand management in Lusaka city.

NRW reduction

Reduction of NRW receives a lot of attention. The regulator has set the acceptable level of NRW as 25 % (NWASCO, 2017), and the Millennium Challenge Corporation targeted 29% by 2017 (Millennium Challenge Account - Zambia Limited, 2013). The success of the interventions in the investment master plan for Lusaka depends on the reduction of NRW to 15% by 2035 (Ministry of Local Government and Housing et al., 2009; Republic of Zambia, 2011b). However, attaining any of these set levels of NRW has eluded the LWSN because the level of NRW has constantly been above 40 % (Chapter 5; Simukonda et al., 2018a). This failure to reducing NRW in the presence of these targets shows that stating what should be achieved (what to do) without knowing how to achieve it (how to do it) does not yield much and the problem of high NRW may continue into the future unless drastic measures on the "*how to do it*" are implemented.

One challenge with the reduction of NRW stems from metering related problems. Metering is highly advocated and it is one of the performance indicators for utilities in Zambia, however since LWSN is operated intermittently metering has not provided the intended results because meters tend to have more inaccuracies and failures in IWSS and it has been advised in the literature that customer meters should be installed in systems (DMAs) that are on CWS status (McIntosh, 2003; Heymans et al., 2014). Moreover, since in Lusaka some customers are charged based on fixed charges, the determination of NRW is merely an approximation with many uncertainties.

Amount of water supply from the source

If water supply from the source is low, NRW reduction alone is not adequate to improve water supply to the population that has outgrown the WSS by many folds. In the case of Lusaka, there are projections of increasing water supply by increasing the WTP capacity in phases depending on financial and management capability of the LWSC (Ministry of Local Government and Housing et al., 2009; Republic of Zambia, 2011b).

Two critical aspects are noteworthy about the perspectives for understanding the desired future state for the LWSN.

i. Conversion to CWS in Lusaka is an issue that even operators of the utility company do not believe is feasible on account of insufficient water from the limited capacity WTP rather than raw water sources because the Kafue river has adequate water to meet the demands of Lusaka city (Chapter 5). Thus, while the international consultants set the target of 24 hours water supply, the local stakeholders do not have that target in mind.
ii. The setting of performance indicators such as reduction of NRW to a certain level are mere statements of the desired level of service. They do not provide guidance to answer the question *"How?"* and because of that NRW targets are missed.

6.5 Projections of consumption water demand and supply for Lusaka city up to 2035

Consumption water demand comprises domestic, public, commercial, and industrial demand. The equations for calculating these consumption demand components are presented in Chapter 5 (Equations 5.1 to 5.5). Also presented in Chapter 5 are the per capita domestic, public and commercial water consumption and the industrial water consumption per unit area. To use the equations, the required population components and the industrial areas are needed.

6.5.1 *Population components*

During the planning horizon, consumption water demand is expected to increase due to the increasing city population, urbanisation, economic and industrial growth. The summary of the 2010 and the projected 2035 population used in the water supply investment master plan for Lusaka (Republic of Zambia, 2011b) is shown in Table 6.1. The Table shows that the number of people that will still be in the informal housing type in Lusaka city in 2035 will be almost the same as they were in 2010. The importance of this large number of people in informal housing type should be seen from considering the target in Zambia's vision 2030 document where 100% access to clean and drinking water is envisaged. Population details in all the DMAs, the satellite towns and the housing types for years 2010 and 2035 are in the appendix (Table A.4). Table A.4 shows population components for each DMA for the two years only. The total populations for each DMA and satellite town for the 6 years of interest within the planning horizon (2010, 2015, 2020, 2025, 2030 and 2035) are given in Table A.5.

6.5.2 *The areas occupied by industries*

The areas occupied by industries are determined from the industrial demands provided in the master plan (Republic of Zambia, 2011b).

6.6 Quantitative application of the developed scenarios to the LWSN

The development and application of scenarios involve a rigorous analytical process of a wide spectrum of possibilities. Rigorous analysis is very important for IWSS which normally have limited data resulting in a lot of uncertainties in the planning of any improvement interventions. The application of the water supply sustainability scenarios (developed in Chapter 3) to the LWSN provides insights on possible occurrences that are difficult to imagine without the scenario analysis approach.

For all the four scenarios, (business as usual, demand management, NRW management and the holistic scenario) increase in water supply (Table 6.2) is a predetermined element. Population increase from 2010 to 2035 and the proportions of people in HC, MC, LC and InfC housing types are also assumed as predetermined elements. For these, the master plan assumptions are followed for all the scenarios. Moreover, the public and commercial per capita water consumption are predetermined and for each scenario, the values in the master plan are used. The critical uncertainties (aspects whose evolution is not known with certainty) are per capita domestic water consumption, NRW levels and the industrial water consumption per unit area (ha). In what follows, the four scenarios are applied to the LWSN.

Table 6.1: Lusaka City and the Greater Lusaka 2010 and projected 2035 population (Republic of Zambia, 2011b)

	2010 POPULATION					**2035 POPULATION**				
DESCRIPTION	**Total**	**HC**	**MC**	**LC**	**Inf**	**Total**	**HC**	**MC**	**LC**	**Inf**
LUSAKA CITY	**1,509,169**	**290,542**	**423,184**	**286,309**	**509,134**	**2,817,767**	**687,484**	**790,449**	**864,522**	**475,313**
SATELLITE TOWNS										
Kafue	42,071	14022	14,022	14,022	0	261,897	8,7290	8,7290	87,290	0
Chongwe	26,740	8912	8,912	8,912	0	202,386	67,455	67,455	67,455	0
Chibombo	17,788	5929	5,929	5,929	0	106,927	35,639	35,639	35,639	0
Total satellite towns	**86,599**	**28863**	**28,863**	**28,863**	**0**	**571,210**	**190,384**	**190,384**	**190,384**	**0**
TOTAL GREATER LUSAKA	**1,595,768**	**319405**	**452048**	**315172**	**509,134**	**3,388,977**	**877,868**	**980,833**	**1,054,906**	**475,313**

Table 6.2: Total water produced and supplied from the water sources

Year	Groundwater (10^5 m^3/d)	Water Treatment Plant (10^5 m^3/d)	Total Production 10^5 m^3/d)
2010	1.3	0.95	2.2
2011	1.5	1.1	2.6
2015	1.5	1.1	2.6
2015	1.8	1.1	2.9
2015	1.8	3.2	5.0
2020	1.8	3.2	5.0
2020	1.8	4.8	6.6
2025	1.8	4.8	6.6
2025	1.8	6.4	8.2
2030	1.8	6.4	8.2
2035	1.8	6.4	8.2

6.6.1 *Business as usual scenario*

Under this scenario, it is assumed that little or no effort is made to control the water consumption habits of consumers as such domestic per capita water consumption increases from the 2010 values to the ones for 2015 – 2035 (referred to as Zambian design standard per capita consumption in Chapter 5, Table 5.4). Concerning industrial water consumption, the high per unit area consumption value for 2010 (80,000l/ha/d) is assumed to continue and because there is no effective mechanism to reduce NRW, it is assumed that it will maintain the 2010 level of 48% of the system input volume (Republic of Zambia, 2011b). For this scenario, the level of leakage can even increase beyond 48%.

The total water demand-supply balance for the planning horizon is shown in Figure 6.4. As reflected in the Figure, under this scenario, the total water demand exceeds water supply for all the years during the planning horizon. By 2035, the total water supply deficit is 299,080 m^3/d. If the population of 475,313 people in the informal housing type is converted to low-cost housing an additional water volume of 28,519 m^3/d would be needed thereby increasing the deficit to 327,599

m^3/d. This is greater than the projected total capacity of the WTP at the beginning of 2020 (Table 6.2).

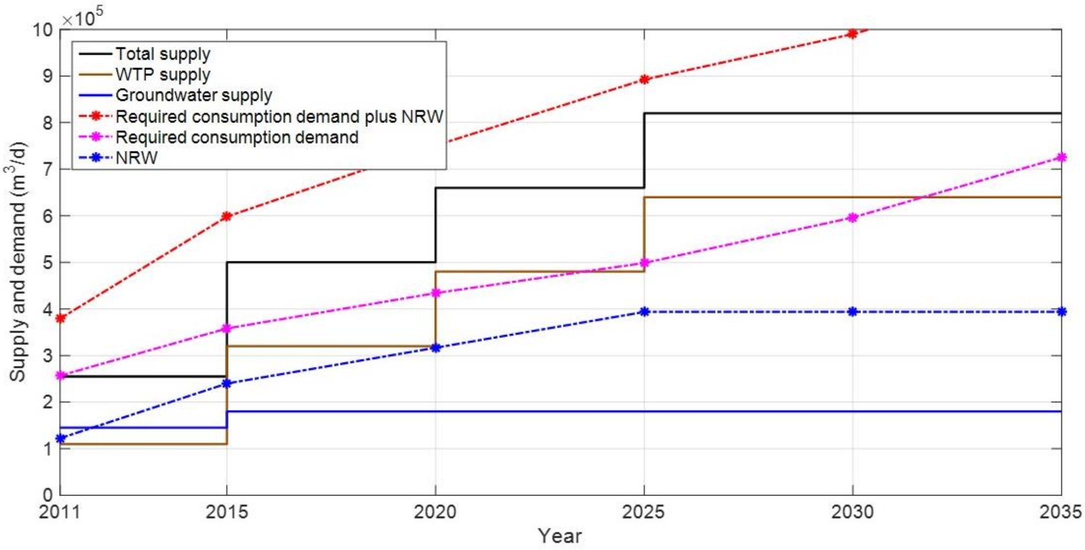


Figure 6.4: The business as usual scenario water supply and demand

6.6.2 *The consumption demand management scenario*

For this scenario, per capita domestic water consumption is maintained at the level obtained in 2010 (Chapter 5, Table 5.4). The industrial per unit area consumption is also maintained as the one obtained in 2010 (80,000l/ha/d). There is no effective NWR reduction strategy and as a result, the NRW value of 48% for 2010 continues.

The total water demand – water supply balance for this scenario is shown in Figure 6.5. The Figure shows that for this scenario, water supply will exceed the total water demand between 2025 and 2028. In 2035, there will be a water supply deficit of 129,190 m^3/d. In this case, water demand management is outweighed by the high NRW levels. If the population of 475, 313 people in the informal housing type is converted to low-cost housing, under this scenario, the additional water required would be 19,013 m^3/d thereby increasing the deficit to 148,203 m^3/d.

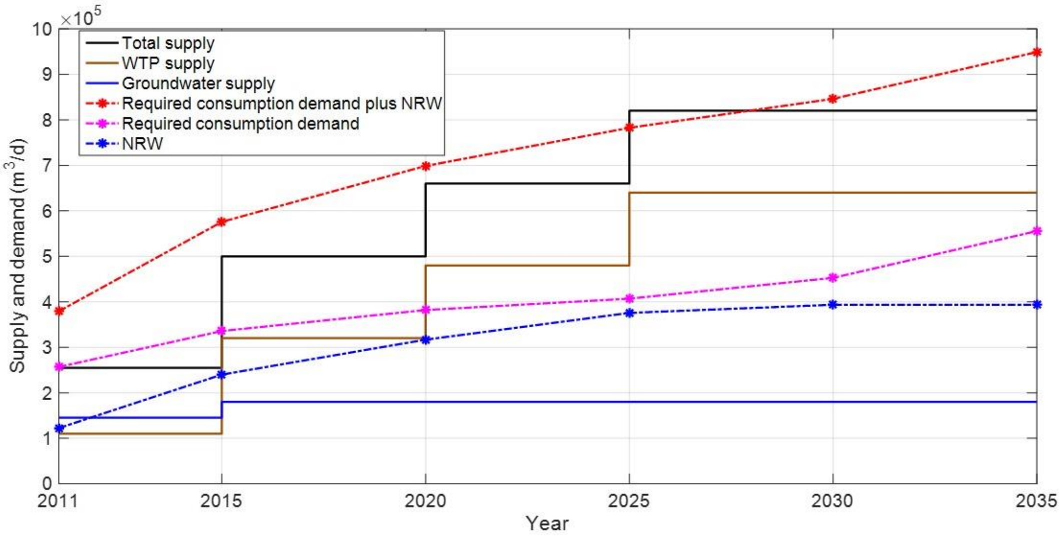


Figure 6.5: The consumption demand management scenario water supply - demand balance

6.6.3 *The NRW management scenario*

This scenario corresponds to the approach in the water supply investment master plan for Lusaka city. For this scenario, the domestic per capita water consumptions for 2015 – 35 are the same as the Zambian design standard per capita consumption (Chapter 5, Table 5.4). The industrial per unit area water consumption for 2015 – 2035 (60,000 l/ha/d) is used. The assumed reduction of NRW levels are as shown in Table 6.3.

Table 6.3: NRW reduction from 2010 to 2035 (Adapted: Republic of Zambia, 2011b)

Year	2010	2011	2015	2020	2025	2030	2035
NRW (%)	48	43	25	20	20	20	15

The consequence of this scenario can be explained both qualitatively and quantitatively. Qualitatively, the sustainability quadrilateral can be used (Figure 6.6). The horizontal axis (the domestic per capita water consumption) meets the vertical axis (the NRW) at point A (the blue circular spot). For the LWSN, point A can be defined by the average 2010 level of per capita domestic water consumption (83 l/c/d) and the regulator acceptable level of NRW (25%). The

2010 situation (grey circular spot) is defined by the average of 83 l/c/d and the level of leakage for 2010 which is 48%. Since the domestic per capita water consumption is the same as the reference value, the domestic per capita consumption front coincides with the vertical axis and the sustainability front passing through the spot representing the 2010 situation is close to the most unsustainable situation vertex 1. Under this scenario in 2035, the per capita domestic water consumption will be expected to increase from an average of 83 to 143 l/c/d, the domestic per capita front is moved towards B to the vertical dotted line passing through the grey spot representing the 2035 situation. Consequently, qualitatively, the expected 2035 per capita domestic water consumption is less sustainable than the 2010 because the per capita domestic consumption front is moved towards the most unsustainable vertex. This is not a good planning target in a world of increasing water scarcity and increasing requirement for more sustainable solutions.

With respect to NRW, the NRW front moves from the 2010 situation to the horizontal line passing through the 2035 situation spot. This represents a huge movement towards the most sustainable situation vertex 3. This is a desirable move because low levels of NRW mean more social, financial and environment sustainability. Thus, because of the huge, expected reduction of NRW, the overall movement of the sustainability front is towards the most sustainable solution (the middle slant red dotted line compared to the original line). However, if the average domestic per capita water consumption was kept at 83 l/c/d, with the huge NRW reduction, the movement of the sustainability front was going to be huge also resulting in the green slant dotted line that is passing through the green circular spot. This represents the 2035 situation described by the holistic scenario discussed in subsection 6.7.4 except that the level of NRW is 25% rather than 15%. From the qualitative perspective, the water supply sustainability quadrilateral can be used to visualise the setting of realistic and sustainable targets for water demand management.

Quantitatively, the total water demand – water supply balance for the scenario is shown in Figure 6.7 which shows that there will be a water surplus between 2020 and 2035 if this scenario occurred. The surplus in 2035 will be 10,688 m^3/d. However, if the population of 475,313 people in the informal housing type is converted to low-cost housing, under this scenario, the additional water required would be 28,519 m^3/d. This would result in a deficit of 17,831 m^3/d. This shift from water surplus to deficit reflects the fact that overall, the scenario is not sustainable because water is not enough to meet the needs of all the consumers when there are some who consume too much which violates the need for intragenerational equity (Asafu-Adjaye, 2005) of the sustainable development concept especially that in this case, it is the poor who are left out in informal housing conditions with poor or no water supply. It is important to note that the planning for water supply in Lusaka which explicitly leaves out a class of people (poor people) in water supply improvement plans (although planned before the inception of the SDGs but being implemented in their era) is against the spirit of the SDGs which promotes development that leaves no one behind and endeavours to reach the furthest behind first (United Nations General Assembly, 2015).

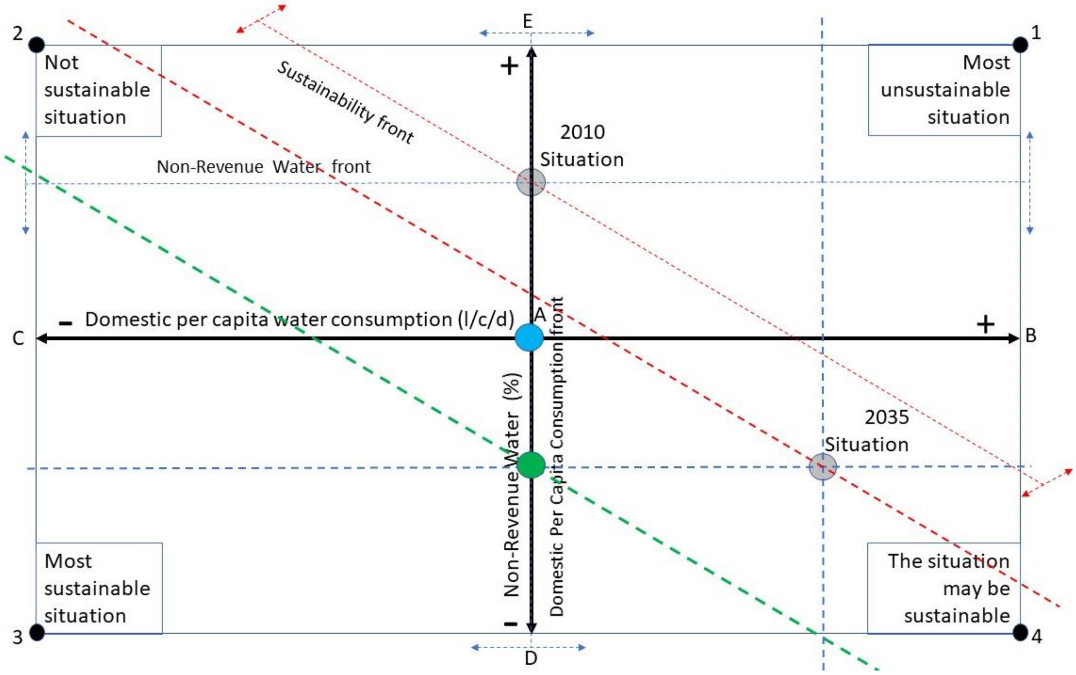


Figure 6.6: Application of the water supply sustainability quadrilateral to LWSN's master plan approach

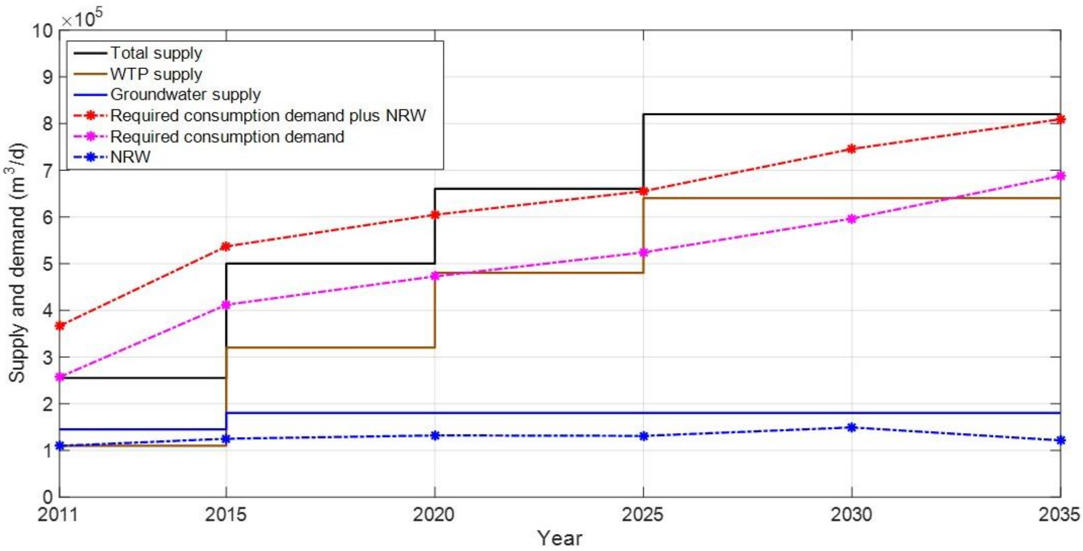


Figure 6.7: The NRW management scenario projected water supply and demand (Adapted from: Republic of Zambia, 2011b)

6.6.4 *Holistic scenario*

Under this scenario, all efforts are made to ensure that both water consumptions and leakage are controlled. Per capita domestic water consumptions are kept at the 2010 values. Industrial per unit area consumption is reduced from 80,000 to 60,000 l/ha/d which is the 2015 – 2035 assumed industrial water consumption (Chapter 5). NRW is progressively reduced from 48% in 2010 to the acceptable level of 25% in 2035.

The total water demand - water supply balance shows that there will be a water supply surplus of 129,300 m^3/d in 2035 (Figure 6.8) if this scenario occurred. However, if the population of 475,313 people in the informal housing type is converted to low-cost housing, under this scenario, the additional water required would be 19,013 m^3/d. This would reduce the water supply surplus to 110,287 m^3/d. Consequently, if this scenario occurred, there would be adequate water to supply the entire population in the city of Lusaka without leaving the poor out.

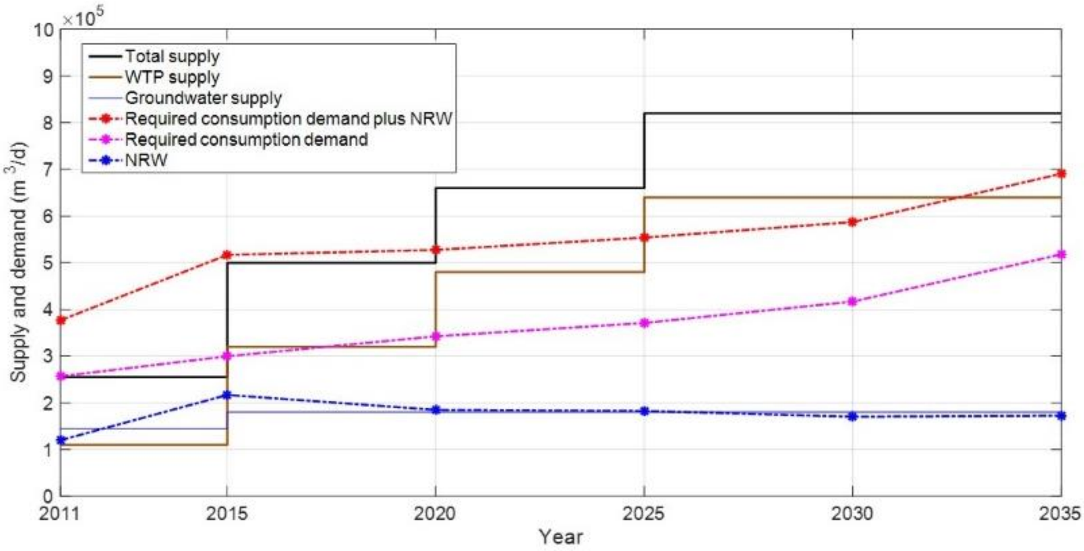


Figure 6.8: The holistic scenario projected water supply and demand

Five aspects are noteworthy from the quantitative application of the four water supply sustainability scenarios to the LWSN:

i) Even though the population values are considered as predetermined elements, there are uncertainties on how the people considered under the low cost and informal housing types will behave. The people under the low-cost housing type may choose to consume more water than what is prescribed in the design standards. With respect to the informal housing type, planning improvement measures that condemn some people to kiosks, communal standpipes and hand pumps is against the nation's vision that all should have safe drinking water because moving distances to fetch water cannot be considered safe at all. From this perspective, the master plan approach is an underestimation of the demand for planning purposes especially that population growth and urbanisation are still there and that the plan does not consider all the people under the informal housing type. Once these people are considered, the master plan total water demand – supply results in a water supply deficit.

ii) The NRW management (Master plan scenario) calls for serious decisions and actions to reduce NRW towards the desired course, otherwise, the master plan interventions will not result in the intended results. This is because NRW is not being reduced and the current development trajectory follows that of the business as usual scenario which results in a huge water supply deficit.

iii) The reduction of demand alone when NRW is high will not improve the sustainability of the system much. In fact, this may even increase the leakage component of NRW.

iv) For the NRW management and the holistic scenarios, the NRW in cubic metres in 2035 is more than the values in 2010 despite the very much reduced percentages. This is because the system input volumes increase over the years and the reduced percentages are applied to the large system input volumes.

v) There is a chance to aspire for a better desired future for the LWSN because as shown by the scenarios, the master plan desired future is not the best and it is unrealistic. The holistic scenario shows some hope, but for it to occur, not only should there be clearly defined water demand management policies or guidelines, but there should also be a commitment to their enforcement. This should include a commitment to the conversion of the LWSN to the CWS status. This commitment is currently missing.

6.7 Conclusion

In this chapter, the application of the GSG and water supply sustainability scenarios to the LWSN in Zambia has been presented. For the LWSN, it is important to take a broader perspective that includes analysing the current and desired future development states of the country - Zambia, the national water sector, city of Lusaka and the LWSN. These provide an understanding of trends in the development of the drivers, sources of challenges in changing the course of these trends, and identification of the possible strategies to be applied and key stakeholders to be involved to increase the chances of succeeding with the planned water supply improvement interventions. This is premised on the fact that the future can be influenced by the decision and actions being made at present and in the future. From the analyses the following specifics for Zambia, Lusaka city and LWSN are drawn:

Zambia’s current state, policy trends and desired future state dominantly conform to the MF scenario of the GSG scenarios. This has serious implications for water supply improvements because of the increasing number of people living in poverty. Some of these people leave in informal housing types with poor or no water supply services. Others leave in low-cost housing types and have difficulties in paying for water services at social tariffs and yet cost-reflective tariffs should be implemented to ensure social, environmental and utility financial sustainability. In a nutshell, the following can be said:

- For the whole water sector in Zambia, governance is the major issue. Policies and laws are poorly enforced. There is a lot of political influence (implicit and explicit) in the operation of water utilities. Much of governance issues can be resolved by an enlightened citizenry who know what is right and wrong and can air their grievances. For many Zambians, the issue of docility and complacency coupled with the reverence of those in authority makes them prone to sub-standard services and are not capable of making those in authority accountable. In such a situation, changing the attitude of the citizenry is vital for successful water supply improvement interventions. For this, involving appropriate "*non-partisan*" civil society organisations in the planning and implementing of water supply improvement interventions is critical.
- Currently, the development of Lusaka city lacks coordination. Most of the new settlements are being developed without water supply and sewerage systems. If this trend continues unabated, the current water supply problems in the city are likely to continue. There exists a plan on how Lusaka city should be developed in a coordinated manner by the year 2030. Under this plan, Lusaka city is considered into the inner city (areas that are already developed) and outer city (new areas) in which compact development and edge expansion are planned respectively. However, attaining the planned city developments is highly doubtful given the many challenges the city is facing. Whether the planned development takes place or not, what is certain is that the city will continue expanding due to population growth and urbanisation and any improvement in the water supply situation that does not consider this aspect is likely to be ill-planned.
- The current situation for the LWSN is that water supply is intermittent in most DMAs and efforts to reduce NRW have proved futile. Currently, LWSN faces many challenges (discussed in Chapter 5). There is no clear desired future from the water utility and the regulator perspectives but planning documents (developed by consultants dominantly from developed countries) have

continuous water supply by 2035 as a desired future state for the LWSN. The importance of the conversion to CWS status as a targeted desired future state must be understood by the utility company and the regulator first if this aspect must be supported by the policymakers and be a success. Because of the lack of desired future state from the local actors, the current chapter has discussed the desired future in terms of the duration of water supply, extension (expansion) of the water supply network, domestic water consumption management, NRW reduction and the amount of water supply from the source.

- Network extension is desirable in Lusaka city because many people are not yet connected to the LWSN. However, the future planned network extensions are less than the planned city expansion which means the planned capacity of the WSS is less than the expected city water demand. The network extensions also have multiple connections to the existing network. These multiple connections present DMA demarcation challenges and will result in poor system analyses. Moreover, the extensions do not have new elevated distribution reservoirs or tanks. This means the extensions are oblivious to the problems of electricity outages which could be mitigated by the elevated storage facilities.
- For domestic water consumption, the use of alternative water sources is not common in Zambia. There are no set target levels of water consumption or clearly defined policies for efficient water use and consumption water demand management in the country. However, in Zambia (Lusaka) it is accepted that people are segregated according to their status whereby the affluent who live in high cost housing types are expected to consume more water than the poor. This institutionalised segregation of water consumers means the concept of equity in water supply should be treated with caution. Equality may be more applicable because it balances the need for the financial sustainability of the utility company and that for equitable water supply. This is important considering that in the water supply investment master plan for Lusaka city, some people are considered to continue being under the informal housing types which have very poor or no

water supply at all. The omission of some category of people is not only against the nation's vision 2030 aspirations, but also the SDGs spirit that development should not leave anyone behind. Considering these aspects leads to the understanding that domestic consumption demand is underestimated for planning purposes.

- The application of the water supply sustainability scenarios to the LWSN shows that the best scenario would be the holistic scenario which corresponds to the NSP scenario of the GSG scenarios. Under the holistic scenario, both demand and NRW are reduced to minimum possible values. The NRW management scenario, which corresponds to the master plan projections, is problematic because should more people in the informal housing type convert to the low-cost housing type than projected, there would be a water supply deficit of 17,831 m^3/d in 2035. Moreover, the failure to reduce the levels of NRW to those projected in the master plan means the failure of all intended benefits. Hence, the application of scenarios has shown that improvement interventions in the water supply investment master plan for Lusaka city are most likely not going to yield the intended outcomes. The worst case is the business as usual scenario which corresponds to the MF scenario of the GSG scenarios. The business as usual scenario would result in the water supply deficit of 129,190 m^3/d if some people under the informal housing type are left out as assumed in the master plan. However, if those in the informal housing type converted to the low-cost housing type, the deficit would be 148,203 m^3/d.

7 APPLICATION OF THE METHOD FOR MODELLING IWSS TO THE CASE STUDY WATER SUPPLY SYSTEM HYDRAULIC MODEL

The chapter presents the application of the method for modelling IWSS in section 7.1. Modelling of the whole LWSN is discussed first. Then, the calibration of the M-SIPDA and its application to the whole LWSN is discussed. Section 7.2 deals with the application of the M-SIPDA to the Chelstone zone network. The section presents a comparison of the M-SIPDA simulated results and those reported in the reports on the LWSN. Section 7.3 focuses on the modelling of the Chelstone zone using two extreme scenarios (the holistic and the business as usual). Since by 2035 water supply and demand are expected to increase, the rehabilitation problem is posed as an optimisation problem. Detailed discussions are presented on how the demands and the proportionate quantity of water to be supplied to the Chelstone zone were determined. The optimisation results are presented in section 7.4, Section 7.5 discusses the occurrence of the business as usual scenario and section 7.6 presents the chapter conclusion.

7.1 Modelling of the whole LWSN

The complete WSS modelling process which involves model development and refinement, calibration, validation and application (AWWA Water Distribution Model Calibration Subcommittee, 2013) is discussed in Walski et al. (2003). For LWSN, an already developed WaterGEMs hydraulic model which is converted to an EPANET model (discussed in Chapter 5) is used.

7.1.1 *Calibration of the LWSN EPANET hydraulic model*

The calibration process for the LWSN EPANET hydraulic model follows the three major steps discussed in Chapter 4. These are macro calibration, sensitivity analysis and micro calibration.

The pipe flow calibration data for LWSN is for steady state calibration because field measurements were one-off type. There is no node pressure or tank water level measurement data. These greatly compromise the quality of the data for

EPS model calibration leading to increased chances of making compensating errors (Walski, 1983). Moreover, because demand varies greatly in WDSs, the 4 months duration of calibration data collection means the measurements were most likely taken when system hydraulic conditions were very different. Additionally, the measurements at individual points were not done for varying conditions. Consequently, the calibration process for the LWSN model for EPS as required in this study in which the model is subjected to a wide range of operating conditions, cannot be expected to meet the standard level of calibration, but it is still valuable because it enables understanding the strengths and limitations of the model (AWWA Water Distribution Model Calibration Subcommittee, 2013).

To do calibration with the limited data available, the following assumptions and calibration method were used:

i) The level of calibration of the original model is assumed to be the best the original WaterGEMS model developers (Brian Colquhoun Hugh O'Donnell and Partners, 2010) could achieve with the calibration data used in the current study. Basing on this, it is reasonable to evaluate the quality of calibration of the EPANET hydraulic model and the M-SIPDA not only with respect to the original data but also with respect to the WaterGEMS model simulation results so that major deviations in the WaterGEMS results should not be expected to be better in the EPANET model results.

ii) In the absence of field measured pressure or tank water levels data, the WaterGEMS simulation results which were obtained by the developers of the hydraulic model used in this study are used as a reference for pressure simulation results where needed (based on the assumption in (i)).

iii) Manual calibration is used in this study because LWSN and its Chelston Zone are too large networks for explicit calibration methods. Implicit or optimisation techniques cannot be used because the field measurements for the calibration data are one-off and were taken during a four months period at random times of the day such as 11.82, 12.58, 12.88, 14.12, 14.37 (hours), and so on. A comprehensive Table for the field measurement data and the WaterGEMS calibration results is contained in the appendix (Table A.1).

iv) For the EPANET hydraulic model, the calibration parameters are valve settings. This is because most of the model is not sensitive to pipe roughness factors presumably because most of the pipes have the Hazen-Williams C-factors in the range of 130 and 140 which represent relatively new pipes.

v) In models where both consumption demand and leakage are subjected to the same peak factors, pipe flows during high demand times (daytime) are overestimated because both consumption demand and leakage have high flows at the same time. During calibration of such models, base demand is reduced to match model simulated flows with field data. This was the case during the calibration of the original WaterGEMS hydraulic model (Brian Colquhoun Hugh O'Donnell and Partners, 2010). For the M-SIPDA which considers the differences between consumption demand and leakage, reduced base demand must be increased. Therefore, the calibration parameters for the M-SIPDA are the base demand adjustment (increment) factor and the leakage emitter factor (exponent).

vi) Due to the limitation of data, validation of the performance of the EPANET hydraulic model and the M-SIPDA was not done.

Macro calibration

For the LWSN, macro calibration involves checking if the converted EPANET hydraulic model has serious distortions from the original WaterGEMS model due to the conversion process. No distortions are found. The next aspect is the comparison between simulated pressure and flow results with the existing data. The flow errors between simulated and field-measured flows are shown in Figure 7.1. The errors are calculated using Equation (4.18) presented in Chapter 4. In Figure 7.1, six pipes have errors that are great than 30% with respect to the measured data. However, of the six pipes, only pipes number 18 and 76 have flow differences that are different from the simulation results from WaterGEMS which was calibrated by the hydraulic model developers (Brian Colquhoun Hugh O'Donnell and Partners, 2010). These pipes (pipe 18 and 76) are targets for the macro-level calibration.

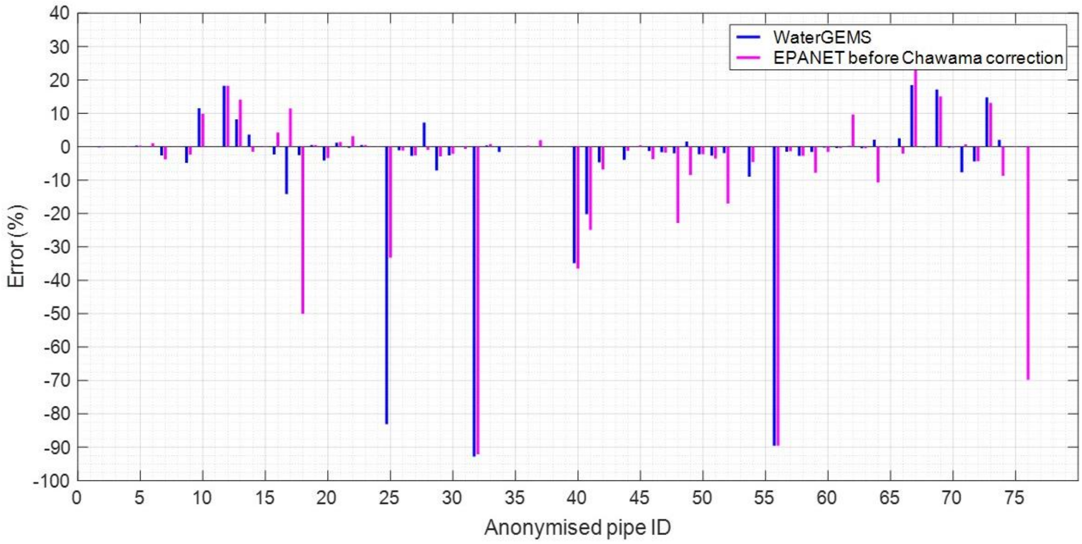


Figure 7.1: Flow errors of WaterGEMS and EPANET simulated results with respect to field measured data

Since there is no field measured pressure data, the WaterGEMS distribution map for simulated pressure results at 12:30 (Figure 7.2) is used to compare with the pressure distribution map developed from the EPANET simulated pressure results at the same time (Figure 7.3). For the legend on the EPANET pressure distribution map, the pressure ranges are set equal to those used on the WaterGEMS pressure distribution map. Significant differences between the two maps are noticed in regions A, B and C (Figure 7.3). Of the three regions, region A (Chw DMA) is connected to other parts of the network through pipe 18 while pipe 76 has no direct connection to regions A, B and C.

Figure 7.2: WaterGEMS network pressure distribution at 12:30
(Brian Colquhoun Hugh O'Donnell and Partners, 2010)

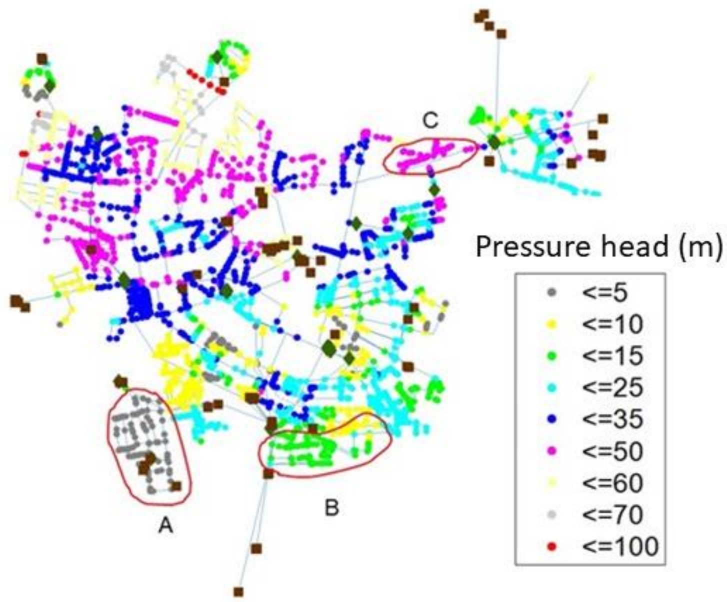


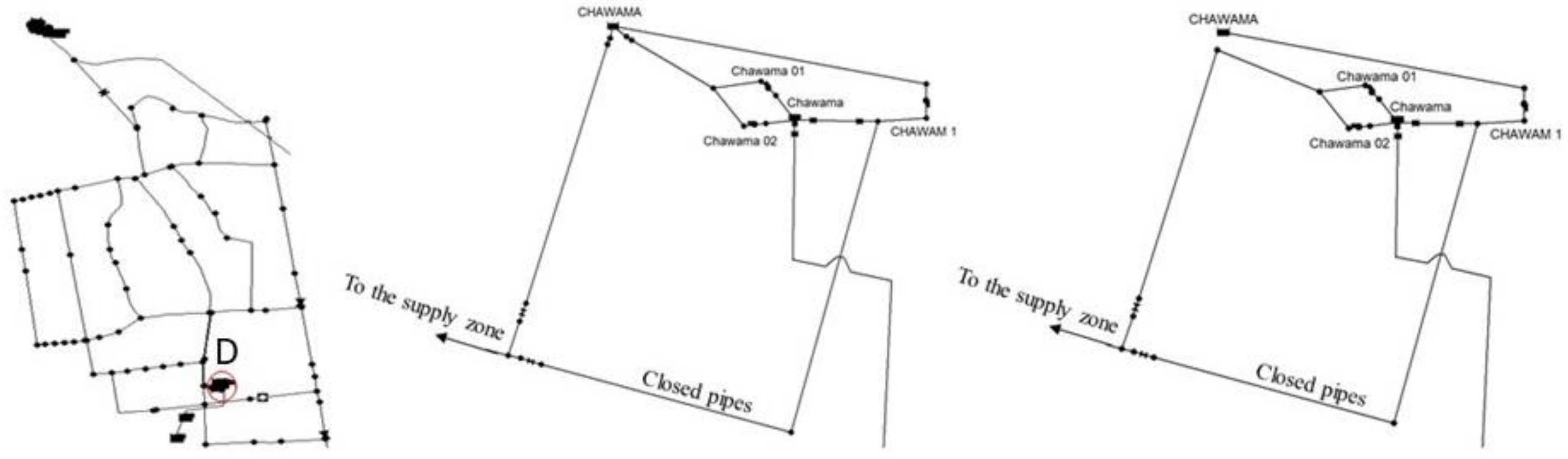
Figure 7.3: EPANET simulated network pressure distribution at 12:30

Examination of the water sources within region A, shows that there is a topological problem which is made apparent by magnifying the region as shown in Figure 7.4a to show the location of region D which has inconsistent connections of pumps, a reservoir and a tank as shown in Figure 7.4b. In Figure 7.4b, the

CHAWAMA reservoir has an elevation of 1273.68 m and tank Chawama's elevation is 1281 m. Chw/Kuo DMAs are flat areas and the fact that the nodes around tank Chawama have elevations of 1281 m, implies the tank is a ground tank and the CHAWAMA reservoir is most likely a borehole and pump CHAWAMA 1 is a borehole pump located at 1272.61 m which is about 1.07 m below the borehole water surface. Thus, it is correct that pump CHAWAMA 1 pumps water from the borehole to a ground tank. However, Figure 7.4b shows that pumps Chawama 01 and 02 pump water from the ground tank back to the borehole and that water from the borehole flows by gravity to the rest of the demand nodes which are at a higher elevation. This arrangement is wrong and is corrected by the connections shown in Figure 7.4c where pumps Chawama 01 and 02 pump water from the ground tank to the demand nodes. However, only pump Chawama 01 is working as pump Chawama 02 is closed (Brian Colquhoun Hugh O'Donnell and Partners, 2010).

The flow differences after correcting the Chw DMA connections are shown in Figure 7.5. The corrections make the flow through pipe 18 to be more than the field measured value. For pipe 76, the corrections in Chw DMA have no effects as the flow differences before and after corrections are identical. The pressure distribution map (Figure 7.7) shows that pressure heads in region A have increased from the highest of 5 m to the highest of 15 m. For region B, no changes are observed after the Chw DMA corrections.

After the Chawama correction, the flow error for pipe 18 is still higher than 30%. The other likely cause of this high error could be valve settings (Ormsbee and Lingireddy, 2004). The valve affecting flows in pipe 18 is FCV-24. In EPANET, there are three fixed status options. These are 'none', 'open' and 'closed' status. When the valve is on the 'none' status, it performs the regulatory functions according to the set value. This is when the valve is active. When it is on the 'open' and 'closed' status, the valve allows free water flow like a pipe and does not allow any flow respectively. Although FCV-24 has a maximum flow limit of 55.56 l/s, this is not implemented as the fixed status for valve is 'open'. When valve is turn to the active status ('none' status), the flow error is reduced to 0.11% which is almost zero as shown in Figure 7.6.



(a) (b) (c)

Figure 7.4: The Chawama pump, tank and reservoir location, original configuration (a, b) and modified configuration (c)

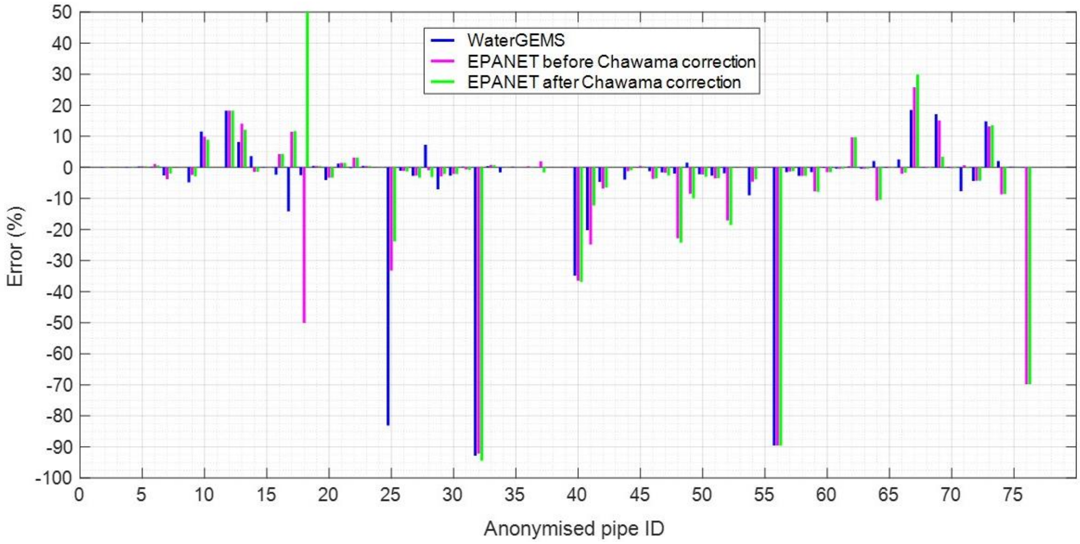


Figure 7.5: Comparison of pipe flow errors before and after Chawama corrections

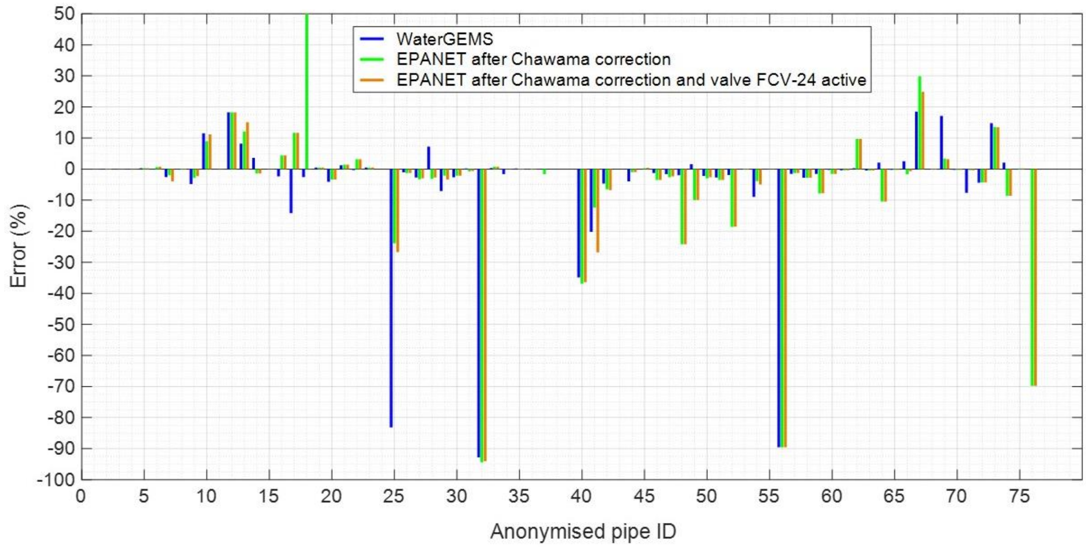


Figure 7.6: Comparison of pipe flow errors before and after Chawama corrections and activation of valve FCV-24

Other than pipe 18, Figure 7.6 shows that the activation of valve FCV-24 affected flow in other pipes located far from pipe 18 as shown in Table 7.1. The greatest percentage change from the flow after Chawama correction before activation of FCV-24 to the flow after FCV- 24 activation was 16.51% corresponding to pipe 41 (Table 7.1). The error of the flow after activation of FCV-24 with respect to the field measured flow for pipe 41 was 20%. This value is within the accepted values for the termination of the macro level calibration. The second greatest change was 7.48% corresponding to pipe 32. For this pipe, the error of the flow after activation of FCV-24 with respect to the field measured flow was 94 %. Since this error magnitude was close to the one for the WaterGEMS simulation results (Figure 7.6), pipe 32 was not the target pipe for calibration. The effect on many pipes by the change of flow in one pipe could be due to the high hydraulic connectivity of the network as discussed in Chapter 5.

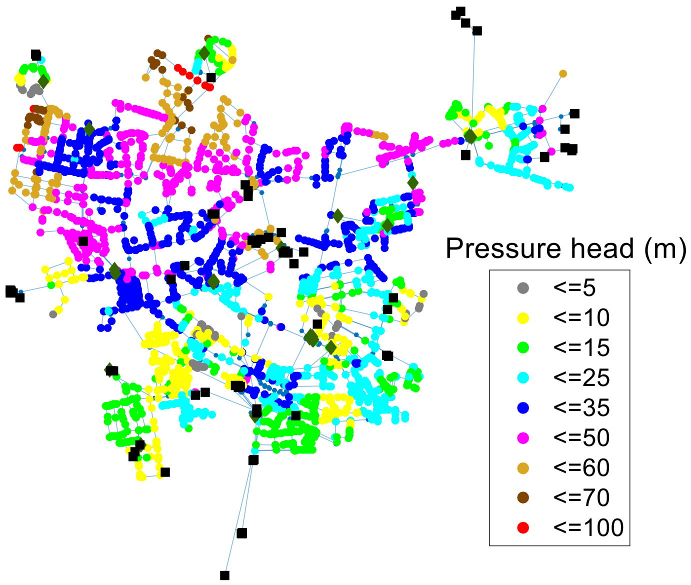


Figure 7.7: Pressure distribution after adjusting the Chw DMA connections

Table 7.1: The effects of activating the settings of FCV-24 on pipes far from its location

Pipe ID	Field measured flow (m^3/h)	Flow before FCV-24 activation (m^3/h)	Flow after FCV-24 activation (m^3/h)	Absolute flow difference (m^3/h)	Percentage flow difference (%)
7	156.87	153.77	150.70	3.06	1.99
10	23.67	25.79	26.32	0.53	2.06
13	2.2	2.47	2.53	0.07	2.66
25	11.74	8.94	8.60	0.33	3.74
32	22.63	1.26	1.35	0.09	7.48
41	17.7	16.96	14.16	2.83	16.51
67	7.14	9.28	8.92	0.36	3.89

Sensitivity analysis

Sensitivity analysis for the LWSN model at this stage (using the DDA approach) does not involve node consumer demand changes because they were adjusted during the calibration of the WaterGEMS model which also used the DDA approach (Brian Colquhoun Hugh O'Donnell and Partners, 2010) and there is no dataset to base the adjustments on. The Hazen Williams C-factors are varied, but they do not affect flows or pressure significantly because most of the pipes in the model have C-factors for new pipes. It is found that changes to valve settings (flow and minor head loss) result in significant flow changes. Consequently, valve settings are the only parameter adjusted during micro calibration.

Micro calibration

The micro calibration method used is manual calibration because of the size of the networks (excluding the use of explicit methods) and the random field data measurements times (excluding the application of optimisation techniques).

The first step in the manual calibration method used for the LWSN hydraulic model is the identification of all the valves that regulate flow through pipes with higher than the standard 10% value (Figure 7.5). After that, adjustments of the valve settings for pipes with the highest difference are done, and each change to the valve setting is followed by a simulation run and a check of the closeness of fit between the measured and simulated results. The complete set of changes to all the valves is shown in Table 7.2. For pipe 76, no valve is found that directly affects it. For all the adjustments, the 24 hours flow variations for pipe 76 are the same as shown in Figure 7.8. In the figure, the simulated flow at 14:33 is 4.26 l/s. The measured flow is 14.09 l/s which is the same as the simulated value at 13:33. Since some measurement times were recorded manually, it is taken that there was a time recording error for pipe 76 and an assumed correct time of 13:33 is used. This assumed time is used in all the other analyses involving pipe 76.

Table 7.2: Modifications to the settings of FCVs that affect the various pipes

Anonymised ID	Time of field data measurement (hrs)	Affecting valves	Original settings			Changes made (bold text)		
			Setting	Loss Coefficient	Fixed Status	Setting	Loss Coefficient	Fixed Status
7	15:45	GV-1045	43.575	100	Open	43.575	100	**Active**
18	12:47	FCV-24	55.56	0	Open	55.56	0	**Active**
22	14:39	GV-561	76.644	5	Open	76.644	5	**Active**
42	12:20	GV-1317	22.05	105	Open	22.05	**10**	Open
		GV-625	0	100	Open	0	**10**	Open
54	12:10	GV-941	15.517	0	Open	15.517	0	**Active**
59	15:15	PSV-40	4.01	0	Active	**0**	0	Active
60	13:00	PSV-40	4.007	0	Active	**0**	0	Active
62	14:00	GV-362	33.417	0	Open	33.417	0	**Active**
		GV-367	6.583	150	Open	6.583	150	**Active**
76	14:33							

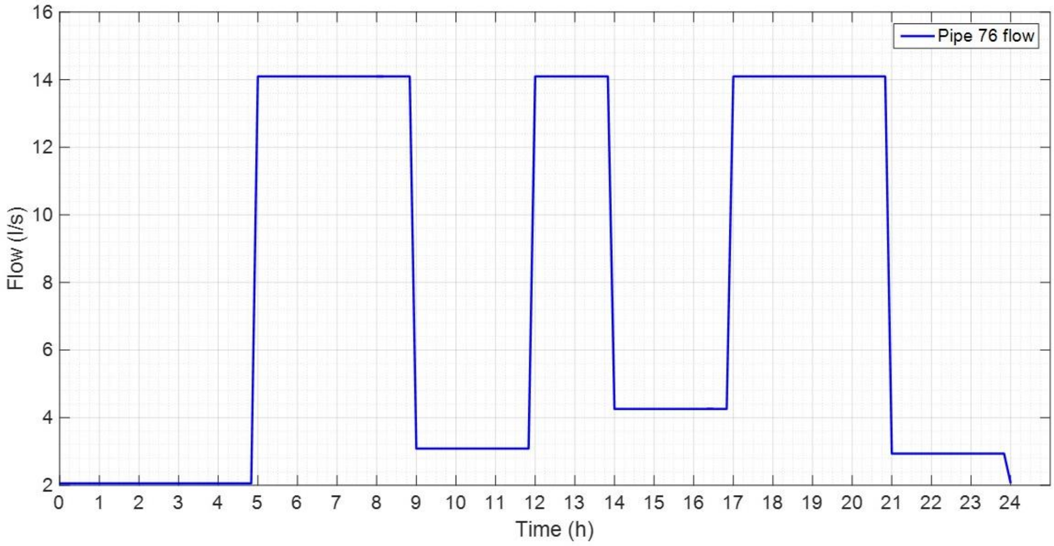


Figure 7.8: Diurnal flow variations for pipe 76

To assess how well the calibration has been done, evaluation of the quality of the calibration is done using statistics which included the mean flow ($\bar{F}$), relative percentage error ($Error$), standard deviation (std), root mean square error (RMSE) and the correlation coefficient (R).

Error

The comparison of errors of the WaterGEMS and the calibrated EPANET model simulation results is shown in Figure 7.9. To analyse the implication of the error values better, a further comparison of field measured flow data with the simulated results is shown in Figure 7.10. In Figure 7.9, 8 pipes have their errors greater than 10%. Part of the failure for these pipes is that LWSN is a large and complex system which makes it difficult to attain the accuracy of 10% or less for all the pipes using manual calibration. Moreover, since the calibration data was taken over a long period, hydraulic conditions were most likely different and cannot therefore easily converge to the same values. The error for each pipe i ($Error_i$) is calculated using S_{F_i} and F_{F_i}, if the values for these parameters are small, their difference is also small and the division of a small value by another small value (Chapter 4, Equation 4.18) results in large error values. Because of this, the pipes with high errors (Figure 7.9) are predominantly those with small flows (Figure 7.10).

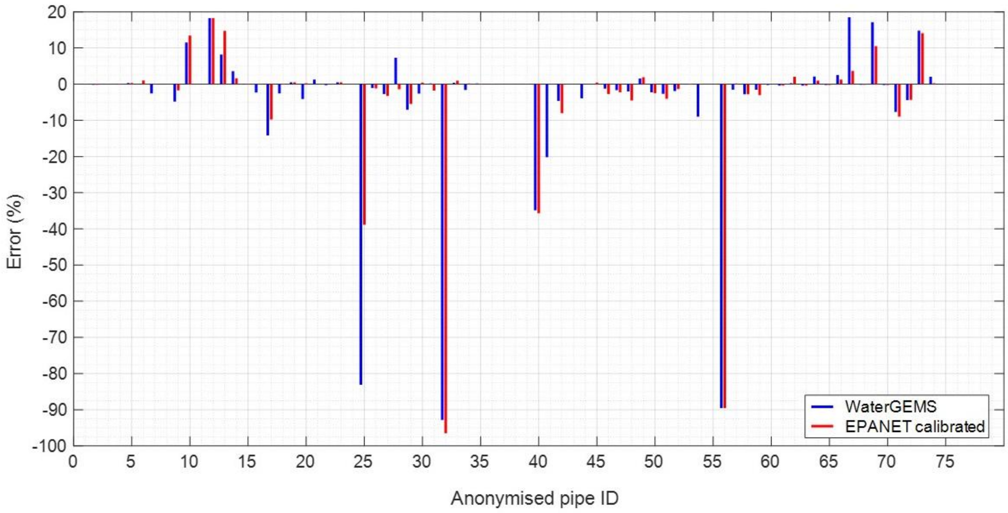


Figure 7.9: Pipe flow errors for WaterGEMS and Calibrated EPANET simulation results

Mean flow, standard deviation and the root mean square error

The comparisons between the measured field data and the simulated results for the EPANET model and the WaterGEMS simulation results using $\bar{F}$, std and the RMSE is presented in Table 7.3. The $\bar{F}$, for the LWSN model's simulated results is the lowest and the RMSE is lower than that for WaterGEMS results. Since RMSE gives the average flow difference between measured and simulated values for each pipe, The RMSE for the EPANET model is better than that for the WaterGEMS. The statistics show that the model can be used to predict the system performance characteristics, but with caution. This is because both the LWSN model and the WaterGEMS are used as DDA based models meant for WSS that have no pressure deficiencies as the case is with the LWSN which is intermittently supplied. Pressure deficient systems which include IWSS are supposed to be analysed using pressure driven analysis (PDA) approaches as discussed in Chapter 4. Moreover, the statistics information should be viewed within the context of uncertainties in data and the hydraulic model.

Table 7.3: Comparison of the mean, standard deviation and the RMSE between the field measured and simulated data

	Statistics		
Data source	**Mean (m^3/h)**	**std (m^3/h)**	**RMSE (m^3/h)**
Field measurement	84.8	132.3	--
EPANET model	83.9	132.3	3.9
WaterGEMS	84.0	129.6	4.3

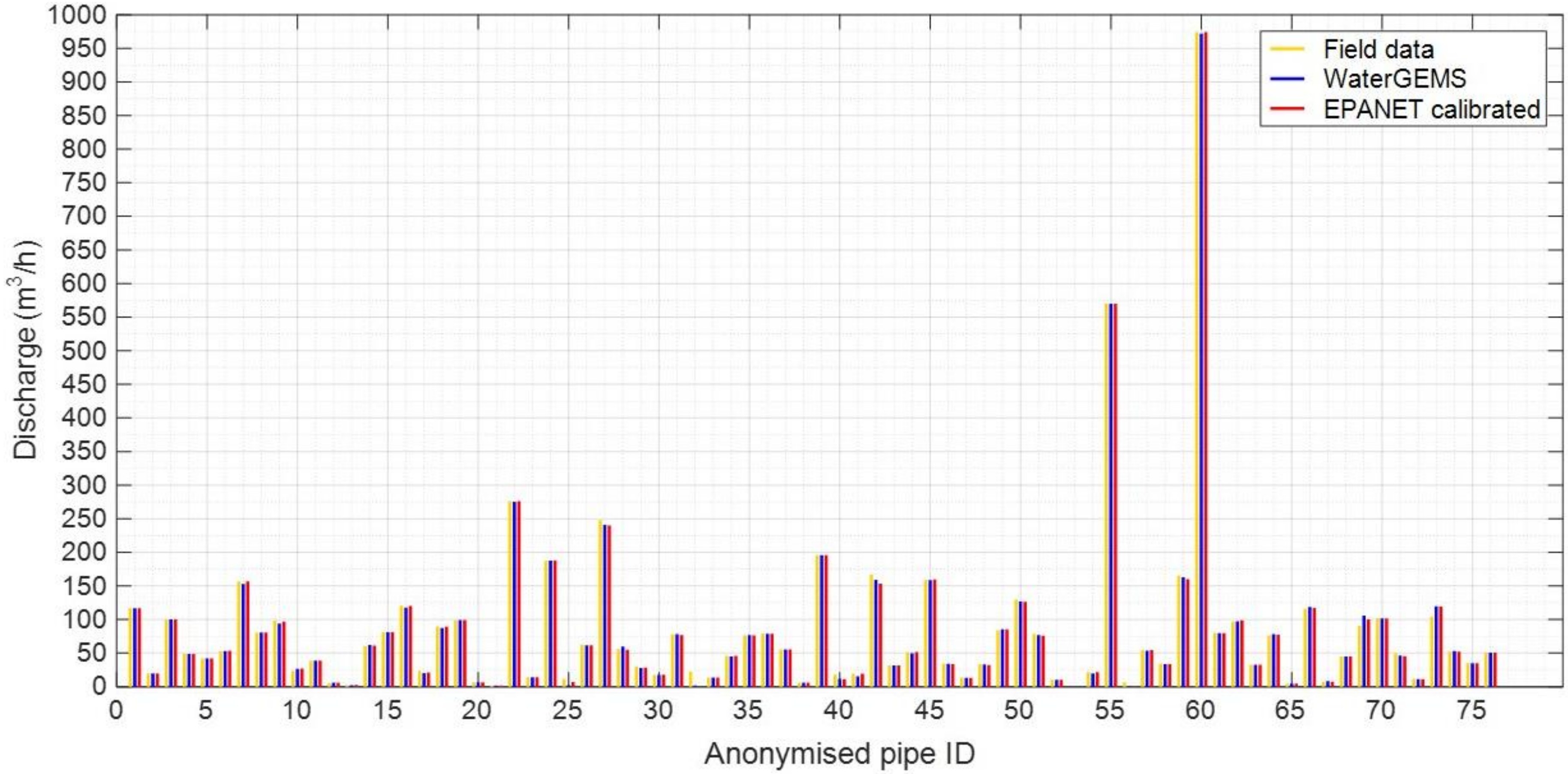
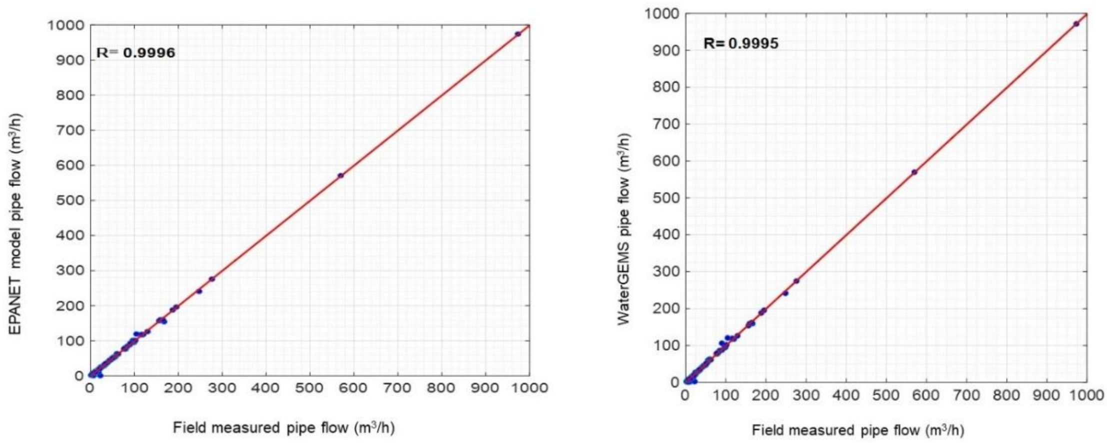


Figure 7.10: Comparison of field measured pipe flows, WaterGEMS and calibrated EPANET simulation results

Correlation coefficient

In the determination of the correlation coefficients, the number of pipes with field measured flows, N is 76. The correlation coefficient (R) for both the WaterGEMS and EPANET model with the field measured data is about 1 (Figure 7.11). The high R values do not reflect the flow errors observed in Figure 7.9 because the flows with large errors are very small and their effects on the calculation of the correlation coefficients are negligible.



(a) (b)

Figure 7.11: Correction coefficient for LWSC WDS pipe simulated flow (a) and Field measured pipe flow (b)

7.1.2 *Calibration and application of the M-SIPDA to the LWSN*

In this subsection, the application of the M-SIPDA calibration process to LWSN is discussed. The information used for calibration of the M-SIPDA includes the DMA water supply schedules, consumption demands, leakage, total demands and field measurements of flow for the selected 76 pipes all contained in Brian Colquhoun Hugh O'Donnell and Partners (2010). The method used for calibration is manual. This is because explicit calibration methods are not applicable on account of the network complexity and implicit calibration (optimisation) techniques are not possible because of the poor-quality data. The report in Brian Colquhoun Hugh O'Donnell and Partners (2010) also contains WaterGEMS simulation results at these times which are also used to compare the final accuracy of the M-SIPDA as contained in the appendix (Table A.1).

There are two calibration parameters for the M-SIPDA. These are the leakage emitter exponent (α) and the total base demand (Bd_{Totalj}). The initial value for the leakage emitter exponent (α) is 1.15 based on the recommended values from the literature for WSS with a combination of pipe materials that have fixed and variable leak hole opening areas as pressure varies (Lambert, 2001; Lambert et al., 2017). For the Bd_{Totalj}, the initial factor of 1.0 is used which means the Bd_{Totalj} for the M-SIPDA is equal to that for the original DDA model. After the calibration process (discussed in Chapter 4, subsection 4.4.3), The α value for the whole LWSN is found to be 0.59 and the Bd_{Totalj} increment factor is found to be 1.24 implying that the Bd_{Totalj} values are increased by a factor of 0.24 from the ones initially allocated to demand nodes under DDA when leakage was implicitly considered.

Like in the process of calibrating the EPANET hydraulic model, the quality of the calibration process is evaluated using statistics which include the $\bar{F}$, $Error$, std, RMSE and the R. The comparison of the $Error$ for the M-SIPDA and WaterGEMS simulation results for the whole LWSN is shown in Figure 7.12 which shows that the simulated flows through some pipes for both the WaterGEMS and M-SIPDA have larger errors than the criteria of 10%. However, the poor quality of data makes the 10% criteria for all the points unattainable.

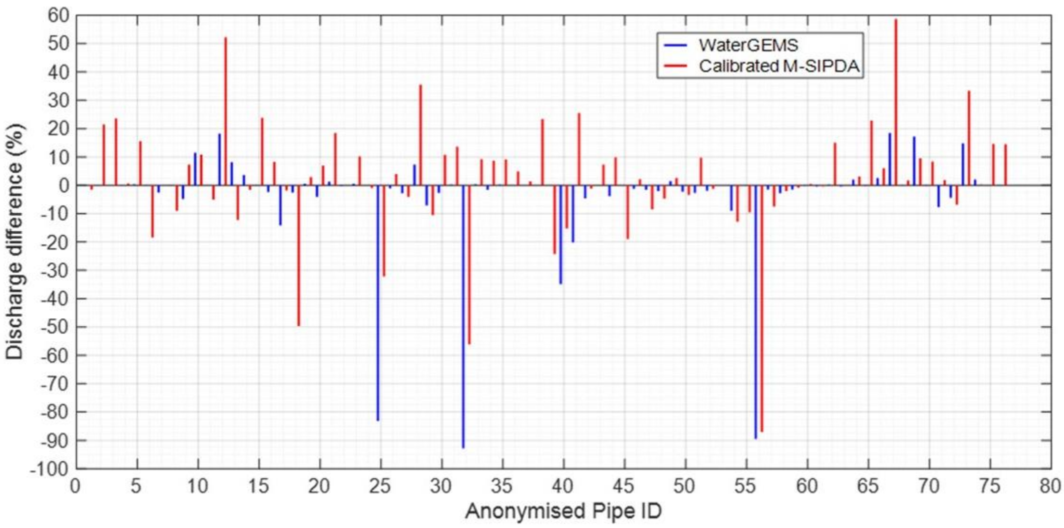
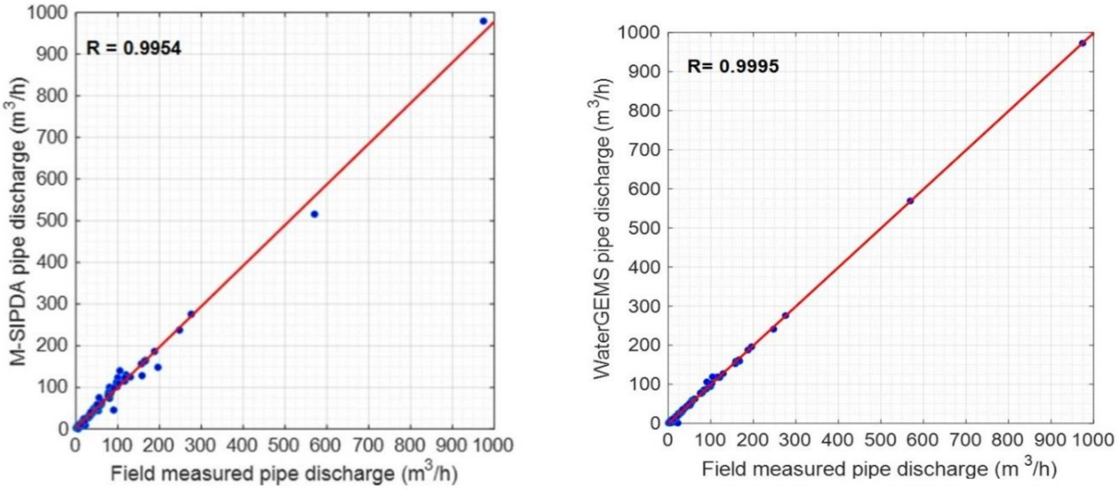


Figure 7.12: The comparison of the WaterGEMS and the calibrated M-SIPDA errors with respect to the field measured pipe flows

Table 7.4 shows the comparison of the $\bar{F}$, std, and RMSE between the field measured and simulated pipe flow data. The $\bar{F}$ for the M-SIPDA is larger than the one for field data and WaterGEMS. The RMSE for the M-SIPDA is also higher than that for the WaterGEMS data. However, the R value of about 1 (Figure 7.13) shows that the calibrated M-SIPDA can be used to analyse and predict pipe flows for the LWSN. This assertion is important for two reasons. The first reason is that in Brian Colquhoun Hugh O'Donnell and Partners (2010), the WaterGEMS was used as a DDA software and the problem of DDA for pressure deficient systems is that they simulate higher flows even when pressure is too low to guarantee the flows. This, coupled with the fact that leakage was implicitly treated as part of consumption demand, made the WaterGEMS produce higher flows for lower total based demands. The M-SIPDA as a PDA, is aimed at solving these limitations of modelling IWSS with DDA approaches. It also includes explicit modelling of leakage which is very significant in these systems. The second reason is that the one-off calibration data collected over a four months period means calibration was prone to compensating errors (Walski, 1983) such that the DDA model could appear to be calibrated for specific operating conditions and settings which makes any change to the operating conditions such as from DDA to the PDA result in poorer results as exhibited by the M-SIPDA. Thus, considering the advantages of the M-SIPDA over the DDA models and acknowledging the limitations of the calibration data which makes high correlation of measured and simulated data more important than the exact matching (Battermann and Macke, 2001), the findings suggest that the M-SIPDA can be used to model IWSS. However, to be conclusive, the M-SIPDA should be applied to more IWSS with good calibration data.

Table 7.4: Comparison of the mean, standard deviation and the RMSE between the field measured and simulated data

	Statistics		
Data source	**Mean ($\bar{F}$) (m^3/h)**	**Standard Deviation (std) (m^3/h)**	**RMSE (m^3/h)**
Field measurement	84.8	132.3	--
WaterGEMS	84.0	132.3	4.3
M-SIPDA	85.7	129.6	12.8



(a) (b)

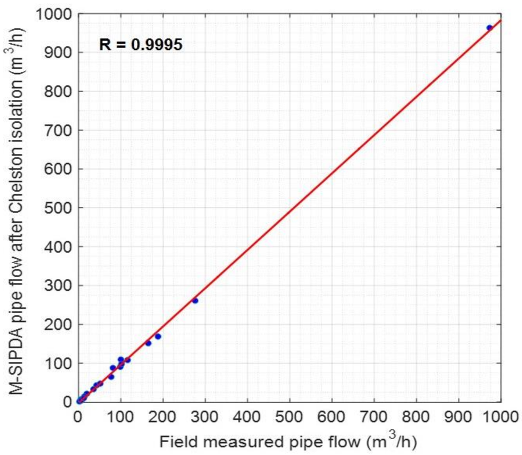
Figure 7.13: The correlation coefficients for (a) M-SIPDA and (b) WaterGEMS for the whole LWSC simulation

7.2 Application of the M-SIPDA to Chelstone zone

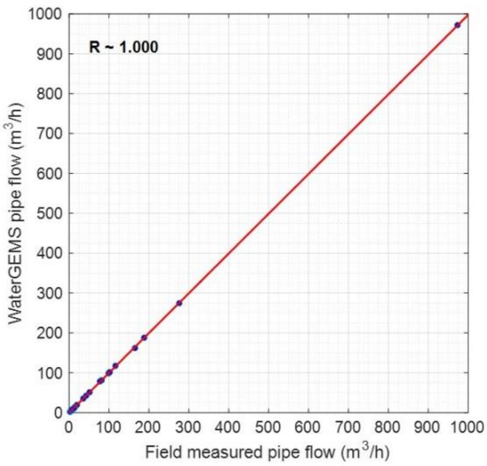
The M-SIPDA is applied to the Chelstone zone of the LWSN (discussed in Chapter 5). The zone is also subjected to the same calibration process as the whole LWSN as discussed in subsections 4.4.3 and 7.1.2. The M-SIPDA simulation results for the zone are discussed in comparison with the results for the whole LWSN. The Bd_{Totalj} increment factor and the leakage emitter exponent (α) for the zone are 1.09 and 0.61 respectively. The values for the whole LWSN are 1.24 and 0.59. The smaller increment factor suggests that the demand values for the zone are more accurate than those for the whole LWSN. In other words, leakage is lower as explained by the larger α. In Equation 4.16, $H_{aveDMA}{}^{\alpha}$ is the denominator as such, the larger α makes the quotient smaller. Thus, the simulation results for the zone show that leakage is slightly lower than the whole LWSN. Reported leakage values are 42% and 36% (Brian Colquhoun Hugh O'Donnell and Partners, 2010), and simulated values are 42% and 38% for LWSN and Chelstone zone respectively. Table 7.5 shows the improvements in the RMSE values for the zone which means with large quantity and high-quality data, the M-SIPDA can provide improved results. The higher mean values for the zone show that the pipes on which measurements were done have high flows. The smaller zone M-SIPDA mean suggests that either the Bd_{Totalj} increment factor was slightly underestimated, or the leakage emitter exponent value was slightly overestimated. From this analysis, the important point is that the consumption base demand used for the DDA in Brian Colquhoun Hugh O'Donnell and Partners (2010) was an underestimation. Finally, the improved simulation results are also reflected by the higher correlation coefficients for Chelstone zone results (Figure 7.14) than those for the whole LWSN (Figure 7.13).

Table 7.5: Statistics comparison between the whole LWSC and Chelstone zone network simulations

Data source	Whole LWSC simulation			Chelstone zone simulation		
	Mean (m^3/h)	**Standard deviation (m^3/h)**	**RMSE (m^3/h)**	**Mean (m^3/h)**	**Standard deviation (m^3/h)**	**RMSE (m^3/h)**
Field measurement	84.8	132.3	--	118.6	213.8	--
WaterGEMS	84.0	132.3	4.3	118.4	213.3	0.9
M-SIPDA	85.7	129.6	12.8	114.3	210.9	8.3



(a)



(b)

Figure 7.14: The correlation coefficients for (a) M-SIPDA and (b) WaterGEMS for Chelstone zone simulation results

7.2.1 *Visualisation of the network pressure situation*

Visualisation using system pressure distribution maps gives a quick and convenient way of identifying problem areas in the WSS because pressure is a good indicator of the water supply performance. For instance, the pressure distribution map (Figure 7.15) shows that at 05:00 am the eastern part of the network has water supply problems because most of the nodes have pressure heads that are lower than the required head (H_{req}) of 7 m. Moreover, the map shows a sudden change from pressure heads greater than 35 m to those less than or equal to zero which suggests that there is a bottleneck at the transition point near CH Reservoir. The bottleneck is found to be a closed valve (valve FCV-13) which is meant to prevent direct water flow from the main pipe to the affected areas. Water should first enter the reservoir then pumped to the tower from which it is distributed to the distribution system by gravity. If FCV-13 is opened, the pressure deficiency condition in the eastern part of the zone is eliminated as shown in Figure 7.16. However, in this Chapter, the valve remains closed because the analyses have to follow the conditions under which the EPANET hydraulic model is calibrated, but in subsection 7.3.2, the valve is opened as the aim in that subsection is to improve the WSS capacity.

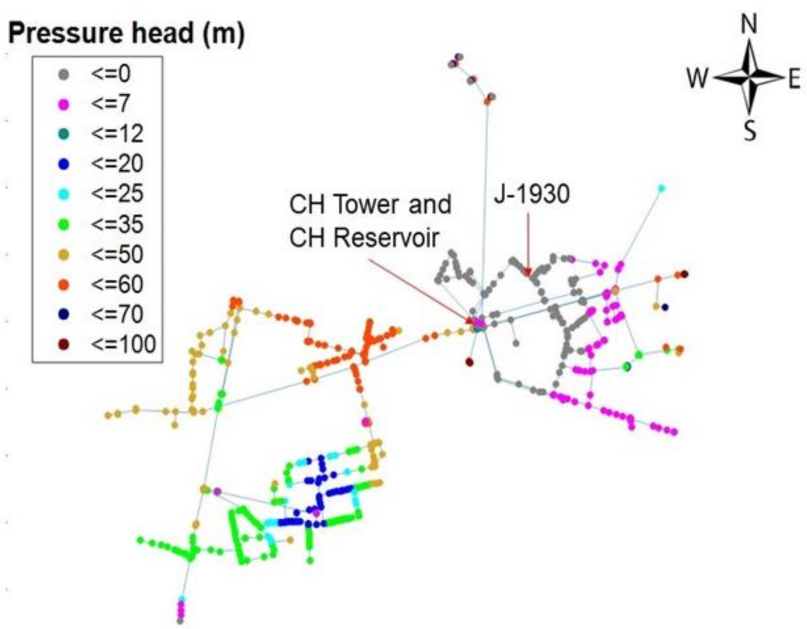


Figure 7.15: Closed FCV-13 Chelstone zone 05:00 am pressure distribution map

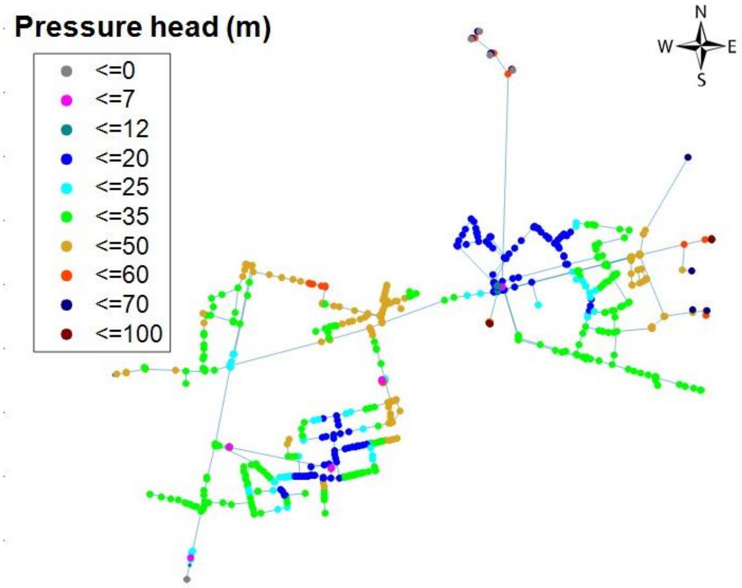


Figure 7.16: Open FCV-13 Chelstone zone 05:00 am pressure distribution map

7.2.2 *Comparison of nodal outflows for SIPDA and M-SIPDA simulated results*

In this comparison, the SIPDA represents PDA methods that either consider leakage implicitly as part of the consumption demand or consider consumption demand only. For the discussion, demand node J-1930 is considered because it has the lowest pressure for the M-SIPDA results. The water supply schedule for node J-1930 is presented in Figure 7.17. The schedule provides the values for the demand multiplication factors ($dmfactor$) at each time step as used in Equations 4.3, 4.14 and 4.15.

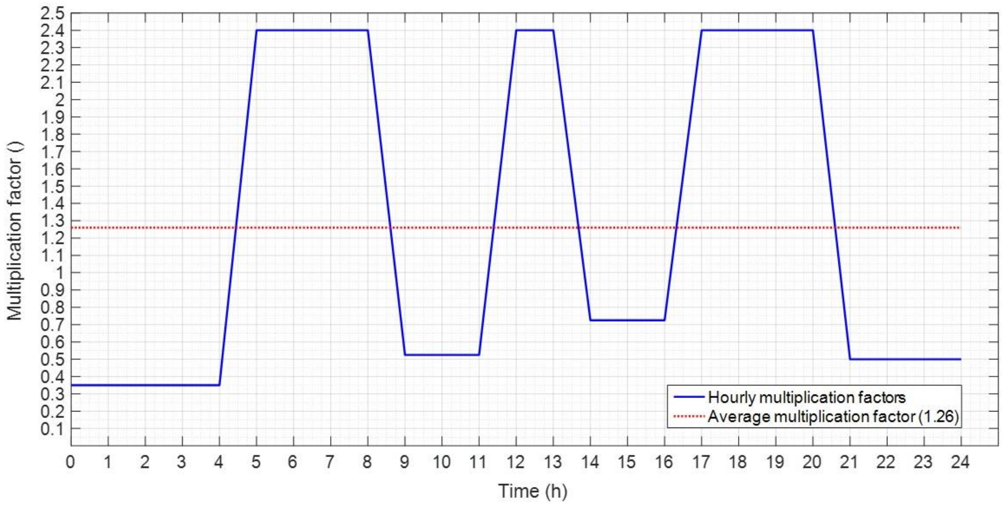


Figure 7.17: Water supply schedule for node J-1930

The 24-hour pressure variation (Figure 7.18) shows that the major pressure deficiency occurs only at 05:00 am. This is also reflected in Figure 7.19 where both leakage and consumption demand outflows are zero at 05:00 am for the M-SIPDA simulated results. However, for the SIPDA, the outflows do not drop to zero. The difference is because of the higher M-SIPDA simulated outflows before 05:00 am which drain water to the minimum level in the CH tower (Figure 7.20) while the lower SIPDA outflows do not drain water in the tower to the minimum. Figure 7.20 also shows that during peak demand hours, the tower water levels for the M-SIPDA are higher than those for the SIPDA results. These differences correspond to the lower peak nodal outflows for the M-SIPDA than the SIPDA (Figure 7.19). However, knowing the method that gives correct outflows requires water level data for the water tower which is not available in this case. Despite this aspect, Figure 7.18 and Figure 7.19 demonstrate the effects of different leakage modelling approaches on pressure and outflow variations respectively. In Figure 7.19, the SIPDA total demand represents simulations where leakage is implicitly modelled such that Q_j^l is zero. Since the Bd_{Totalj} is applied to the Artificial consumption demand emitter only, it implies leakage base demand (contained in the Bd_{Totalj}) is subjected to the same multiplication factor as consumption demand leading to higher flows during peak demand times and much lower flows during low demand times than those predicted by the M-SIPDA consumption demand plus leakage outflows. This shows that failure to model leakage explicitly when analysing IWSS results in excessive peak flows for a given consumption demand leading to excessive head losses thereby underestimating the system capacity as depicted by the dominantly lower pressure heads during peak demand hours (Figure 7.18). The reduced peak hours flows for the M-SIPDA which models consumption demand and leakage explicitly are because when consumption demand is high, pressure and leakage are low (the demand – pressure - leakage relationship) as reflected in Figure 7.19. The Figure also shows that modelling consumption demand only without leakage (demonstrated by SIPDA demand only) underestimates peak flows leading to overestimation of system capacity represented by higher pressure heads (Figure 7.18).

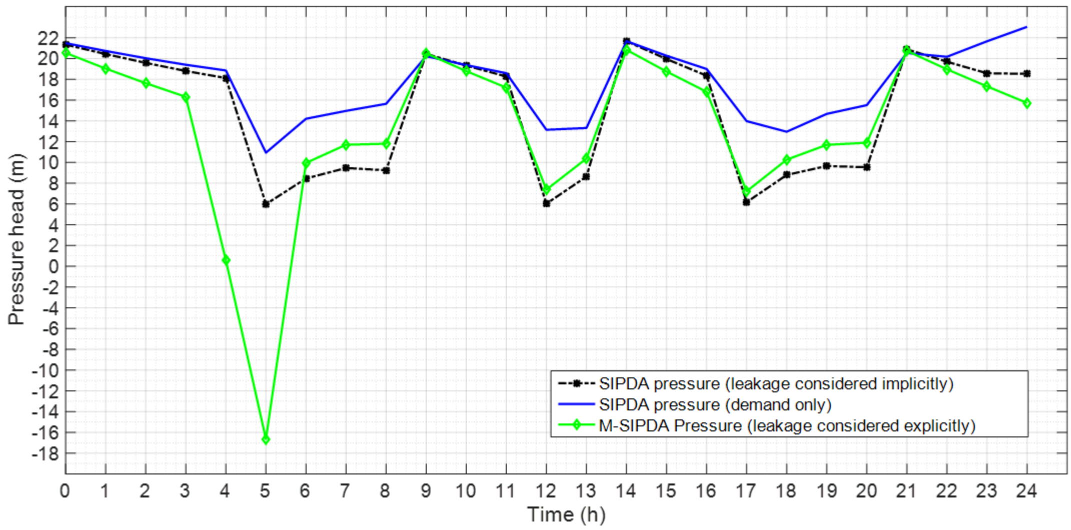


Figure 7.18: Node J-1930 pressure variations simulated by the SIPDA and M-SIPDA

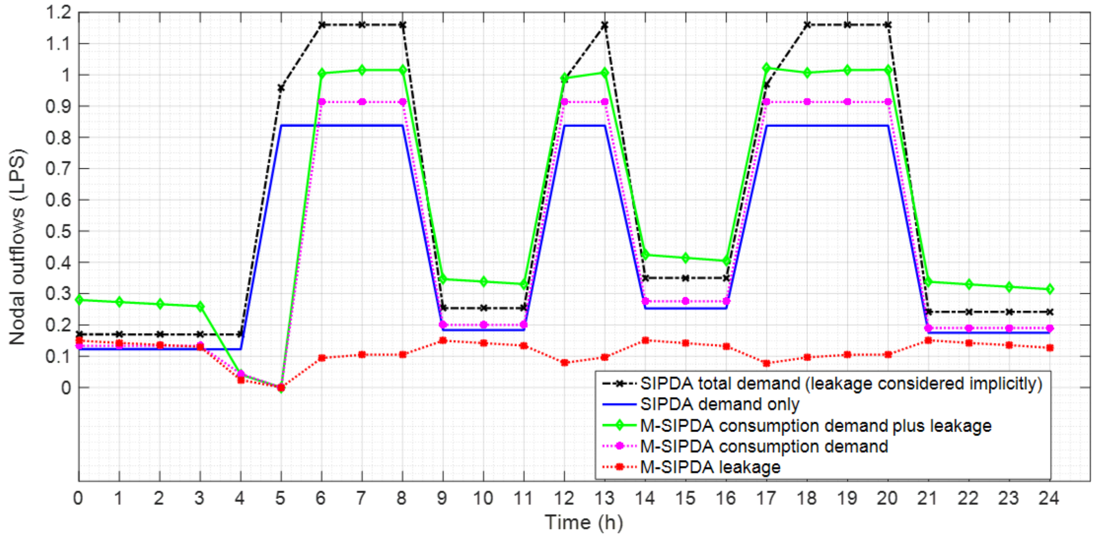


Figure 7.19: Effects of different approaches to modelling consumption demand and leakage

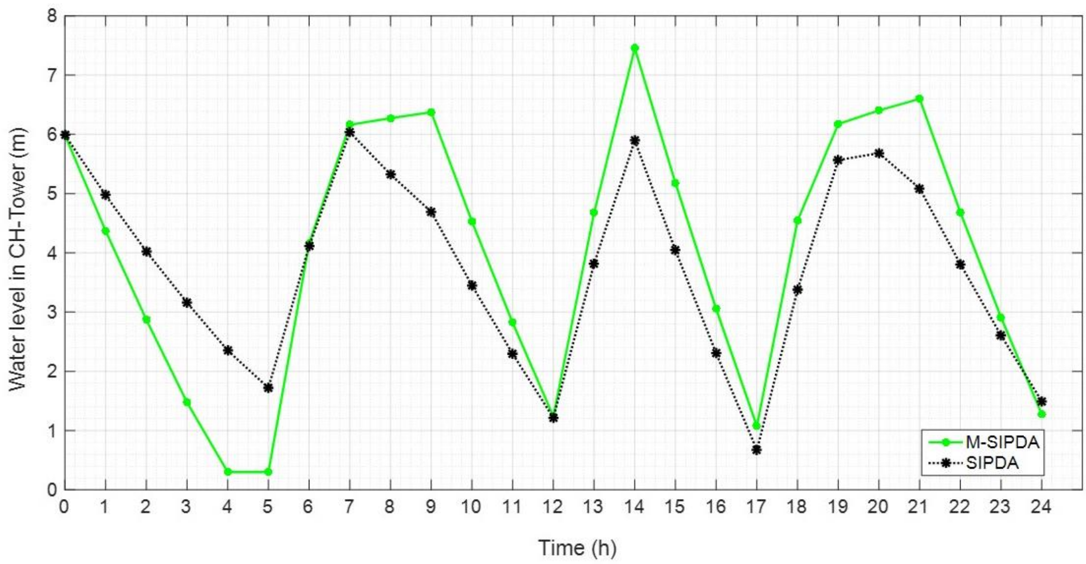


Figure 7.20: CH-Tower water level variations for the M-SIPDA and SIPDA simulation results

7.2.3 *Comparison of the M-SIPDA simulated demands with the reported data for Chelstone zone*

Table 7.6 shows that the consumption and the total demand determined by the M-SIPDA are higher than reported values. However, the maximum total demand relative error of 9% and 0.9% with respect to the two reported values show that the M-SIPDA simulation results are acceptable especially that the hydraulic model used was based on highly estimated data. For instance, in Brian Colquhoun Hugh O'Donnell and Partners (2010) the reported billed consumption demand included bills based on fixed charge tariffs (NWASCO, 2014) which makes the consumption water demand prone to underestimation. In the Brian Colquhoun Hugh O'Donnell and Partners (2010) report, the total demand was determined through the model calibration process by adjusting the nodal Bd_{Totalj} till simulated pipe flows were close enough to the measured values and the DMA leakage values were calculated by Equation 7.1. Since in WaterGEMS, leakage and consumption demand were subjected to the same peak factors, the Bd_{Totalj} had to be lowered for the pipe flows to closely match the measured values. This had the effect of reducing the WaterGEMS simulated DMA total demand ($Q_{DMA_{WGSim}}$) thereby resulting in the low leakage ($Q_{LknownDMA}$) value since the billed consumption ($Q_{DMA_{Billed}}$) was fixed. The Master plan values (Republic of Zambia 2011b) were also affected by the unmetered consumptions. The leakage values for Chelstone zone in the Republic of Zambia (2011b) were estimated using the overall utility leakage level. This most likely overestimated the Chelstone zone leakage level because as shown (subsection 7.1.2) and reported in Colquhoun Hugh O'Donnell and Partners (2010), the average Chelstone zone leakage level is less than that for the whole LWSN.

$$Q_{LknownDMA} = Q_{DMA_{WGSim}} - Q_{DMA_{Billed}} \tag{7.1}$$

Table 7.6: Chelstone zone M-SIPDA simulated and reported water demands

Information Source	Parameter Considered		
	Consumption Water demand (m^3/day)	**Leakage (m^3/day)**	**Total demand (m^3/day)**
Brian Colquhoun Hugh O'Donnell and Partners 2010	23,043	12,956	35,999
Republic of Zambia 2011b	22,188	16,738	38,926
M-SIPDA Simulation	25,116	14,155	39,271

The M-SIPDA simulation results are also estimations because they are based on the uncertain initial base demand values used in Brian Colquhoun Hugh O'Donnell and Partners (2010). However, since the Bd_{Totalj} values are increased by 0.09 and leakage varied differently from consumption demand but dependent on pressure for the respective DMAs in the zone, the simulated total demand of the zone is larger than the reported values, but the simulated leakage is in between. The difference between the reported total demand simulated by the WaterGEMS (Brian Colquhoun Hugh O'Donnell and Partners 2010) and the M-SIPDA simulated total demand should be understood in the context of the system flows. The total volume of 35,999 m^3/day was the total demand in the WaterGEMS simulation results which resulted in the flows that are compared with the field data as shown in Figure 7.12. For the M-SIPDA, the same WaterGEMS total demand is increased by a factor of 0.09 (M-SIPDA total demand = 39,271 m^3/day ≈ 35,999 *1.09 = 39,239 m^3/day) and results in flows that also compare with the field data as shown in Figure 7.12. These comparisons show that the WaterGEMS simulation approach which considered leakage implicitly underestimated the total demand the system can handle.

7.3 Modelling the Chelstone zone under different scenarios

When considering a conversion from intermittent to CWS, simulating the behaviour of the WSS under different scenarios is important. This is because it provides insights into management and operational challenges that are difficult to foresee if the system improvements were made following the performance characteristics predicted using one planning scenario. Due to the complexities of the complete LWSN, the application of scenarios with the method for modelling IWSS is demonstrated using the Chelstone Zone. For this, two extreme sustainability scenarios (developed in chapter 3) are used. These are the holistic and the business as usual scenarios.

Since water supply to Chelstone zone is supplied partly from the groundwater sources (boreholes) within the zone and partly from the main water distribution reservoir from which other zones are supplied, the modelling approach for isolating the zone has tried to consider these aspects as much as possible (discussed in Chapter 5). Since in the isolation of the zone, the flows from the groundwater sources are fixed to avoid boreholes discharging beyond their yield capacities when demand increases, the major increase in the water supplied, when demand increases, is expected to come from the main distribution reservoir. However, increasing the water supply from the main distribution reservoir to meet the increased demand in Chelstone zone would require an increase in the hydraulic head of the water offtake point for the zone. This is because hydraulic heads that would deliver different amounts of water towards meeting different levels of demand are different (Ang and Jowitt, 2006). Another way to increase water supply to the zone would be to increase the pipe diameters. For Chelstone zone, the uncertainties in the demands and the expansion of the pine network capacity, make it difficult to tell what the adequate hydraulic head at the zone offtake point should be to supply the zone demand. This makes the existing gravity supply system from the main distribution reservoir prone to failure when higher volumes of water need to be delivered to the zone. Even if the supply was pumped, the sizing of the pump station depends on the required quantity of water and the pressure head in the zone. The extent of the hydraulic heads can be identified by simulating the zone water flows under different water demand and supply scenarios which should be able to show the different heads that would be needed to supply the specified quantity of water to the zone. For each scenario, the required head is not known with certainty and should be determined. The determined hydraulic head should enable the supply of the limited water quantities as planned because water from the main distribution reservoir is shared between the five zones of the LWSN.

Due to financial resources constraints, water supply improvements are expected to be phased as indicated in the water supply investment master plan for Lusaka (Republic of Zambia, 2011b). While the water demand-supply balance analyses are done for all the four sustainability scenarios and years of interest (2011,2015,2020,2025, 2030 and 2035), the hydraulic simulation analyses are done only for the two extreme scenarios (the holistic and business as usual) and they are based only on the final year of the planning horizon (2035). This is because the water demands for the other two scenarios (The NRW management and the demand management) fall within the extreme scenarios demands and the WSS set up in the last year of the planning horizon is the goal. The analyses are in three parts. The first part is to determine whether or not the zone water offtake point and WSS, as they are, have the capacity to supply water under the 2035 water supply-demand conditions. This is to determine the extent of the system pressure deficiencies which have to be resolved through optimisation. The second and third parts follow if the system is found to have deficiencies after the first part simulations. The second part is to use optimisation to determine the optimum zone water offtake point's hydraulic head which should allow the supply of the allocated quantity of water to the zone and to increase the capacity of the WSS. The third part is to determine which pipes would be changed in the same way should either the holistic or the business scenario occur and which network configuration would result in the lowest regret costs if the scenario other than the selected one occurs.

7.3.1 *Determination of the zones WSS capacity before rehabilitation*

The Chelstone zone's WSS capacity is first assessed under the holistic scenario. To perform the hydraulic analysis of the zone network under the holistic scenario, the 2035 projected total water demand for the zone is determined first. This is used to determine the proportion of the 2035 projected total water produced from the sources that is supplied to the zone. By considering the fixed volume of water supply from the boreholes, the volume of water supplied from the main water distribution reservoir through the Chelstone zone water offtake point is determined as the difference between the total water supplied to the zone and the water supplied by the boreholes within the zone. The steps for determining consumption demand, leakage and water to be supplied to the zone for the year 2035 are as follows:

i. Determination of DMA consumption water demand (Q_{DMA})

This considers the housing types, commercial, public service facilities and industries in each DMA for both the 2010 and 2035 demands. For the holistic scenario (*hol*), Equation 5.1 is written as:

$$Q_{DMAd} = P_{HC} * HC_{hol} + P_{MC} * MC_{hol} + P_{LC} * LC_{hol} + P_{Inf} * InfC_{hol} \qquad (7.2)$$

Where Q_{DMAd} is the domestic DMA water demand, P_{HC}, P_{MC}, P_{LC} and P_{Inf} are defined under Equation 5.1, $HC_{hol} = 143\ l/c/d$ $MC_{hol} = 110\ l/c/d,$ $LC_{hol} = 60\ l/c/d$ and $InfC_{hol} = 20\ l/c/d$ are holistic scenario per capita domestic water consumption for high, medium, low and informal housing types respectively (Table 5.1 in Chapter 5) as discussed in subsection 6.6.4.

To exemplify the calculation process, Mtend DMA is used (Any DMA can be used as the process is the same). For Mtend DMA, $P_{HC} = 0$ and $P_{MC} = 0$ because the DMA has no high and medium cost houses. $P_{LC} =$ 94,271 and P_{Inf} = 31,423 people. Inserting these population figures and the domestic per capita water consumption values for the holistic scenario in Equation 7.3 gives:

$$Q_{DMAd} = \mathbf{6,285\ m^3/d}$$

For water consumption in public and commercial facilities, projections in the master plan are used for all the scenarios. For Mtend DMA, the sum of public and commercial water demand in 2035 is **2,143 m^3/d.** There are no industries in Mtend DMA as a result, industrial consumption demand (Q_{DMAi}) is zero. From these, the Mtend total DMA consumption water demand is (Equation 5.4 Chapter 5):

$$Q_{DMA} = 6{,}285 + 2{,}143 = \mathbf{8{,}428\ m^3/d}$$

This process of considering domestic, public, commercial and industrial water demand is followed to determine Q_{DMA} for each DMA.

ii. Determination of Chelstone zone consumption water demand (Q_{Zone})

In this case, Equation 5.5 is applied to Chelstone zone rather than to the entire WSS. The total Chelstone zone consumption water demand (Q_{Zone}) is the sum of all the zone's DMA consumption demands.

$$Q_{Zone} = \sum Q_{DMA} = \mathbf{44{,}662\ m^3/d}$$

iii. Determination of the water supplied to Chelstone zone ($Supply_{Zone}$)

The total LWSN consumption water demand (Q_{LWSN}) for the year 2035 under the holistic scenario is determined first using Equation 5.5 as:

$$Q_{LWSN} = \sum Q_{DMA} = \mathbf{518{,}025\ m^3/d}$$

The projected total water supply to the whole LWSN ($Supply_{LWSN}$) for 2035 is **820, 000 m³/d** (Republic of Zambia, 2011b)**.** The $Supply_{LWSN}$ is the same for all the scenarios as water supply is a predetermined element (discussed in subsections 3.2.1 and 3.3.1 and section 6.6). To determine the projected amount of water to be supplied to Chelstone zone, its proportion of the total LWSN consumption demand is used.

$$Supply_{Zone} = \left(\frac{Q_{Zone}}{Q_{LWSN}}\right) * \ Supply_{LWSN} = \mathbf{70{,}696\ m^3/d}$$

iv. Determination of the water supply to the Mtend DMA ($Supply_{DMA}$)

$$Supply_{DMA} = \left(\frac{Q_{DMA}}{Q_{Zone}}\right) * \ Supply_{Zone} = \mathbf{13{,}341\ m^3/d}$$

v. DMA leakage ($Leak_{DMA}$)

For the holistic scenario, the leakage in 2035 is assumed to be equal to 25% of the system (DMA) input volume. 25% is the acceptable level of NRW set by the regulator in Zambia (NWASCO, 2018).

$$Leak_{DMA} = Supply_{DMA} * 0.25 = \mathbf{3,335\ m^3/d}$$

vi. DMA Consumption demand plus leakage (DMA total demand ($Q_{DMAtotal}$))

$$Q_{DMAtotal} = Q_{DMA} + Leak_{DMA} = \mathbf{11,763\ m^3/d}$$

The total demand for Chelstone zone ($Q_{Zonetotal}$) is:

$$Q_{Zonetotal} = \sum Q_{DMAtotal} = \mathbf{62,336\ m^3/d}$$

The comparison of the $Q_{Zonetotal}$ and the $Supply_{Zone}$ shows that there is a water surplus of **8,360 m^3/d.** This shows that under the holistic scenario, supplying water continuously is possible. Therefore, for this scenario, the 2035 Chelstone zone system is analysed as a CWSS. However, in this case, DMA leakage values are computed from the Q_{DMA} values using the assumed 2035 leakage percentage of 25% and the assumption that for this case, the $Q_{DMAtotal}$ is equal to the $Supply_{DMA}$ (Conservation of mass).

vii. Determination of the actual total demands and water supply to fulfil the conservation of mass law

The total water supply to a DMA or zone X is given by:

$$Supply_X = \frac{Q_X}{0.75} \quad (7.3)$$

Where $Supply_X$ is the water supplied to a DMA or zone X, Q_X is the zone or DMA projected consumption water demand.

For the Mtend DMA, $Supply_{DMA} = Q_{DMAtotal} = \frac{8428}{0.75}$ = **11,237 m^3/d**

The $Leak_{DMA}$ = 11,232*0.25 **= 2,809 m³/d**

These calculations are done for all the DMAs and from the values for the DMAs, the zone total demand ($Q_{Zonetotal}$) is determined as:

$$Q_{Zonetotal} = \sum Q_{DMAtotal} = \mathbf{59{,}549\ m^3/d}$$

Determination of the 2035 nodal base demands for the holistic scenario

The 2010 total demand for each DMA ($Q_{DMAtotal\,2010}$) is known and the 2035 projected total demand for each DMA ($Q_{DMAtotal}$) and leakage are determined using the process presented in (vii). The ratio of these two total demand values gives the factor by which the 2010 total base demands (Bd_{Totalj}) in each DMA is increased. For Mtend DMA, to increase from the $Q_{DMAtotal\,2010}$ (5,455 m^3/d) to the 2035 $Q_{DMAtotal}$ (11,237 m^3/d), the 2010 total base demands are multiplied by a factor ($factor_{Bd}$) calculated as:

$$factor_{Bd} = \frac{Q_{DMAtotal}}{Q_{DMAtotal\,2010}} = \mathbf{2.06}$$

Since if the holistic scenario occurred, the zone would be supplied water continuously, all the zone's DMAs should be subjected to the same CWS demand pattern (Figure 7.21).

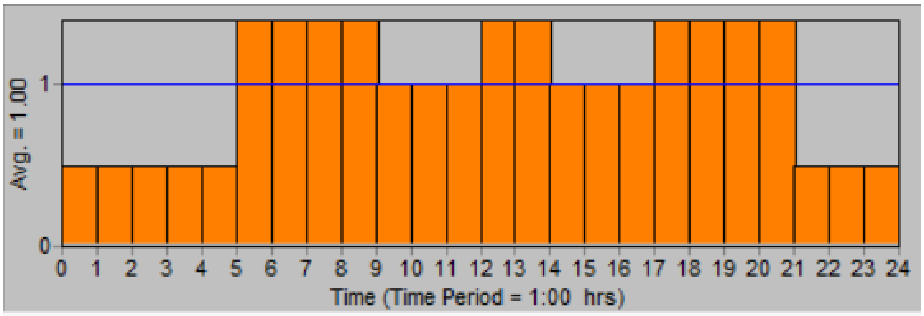


Figure 7.21: Demand pattern for Chelstone zone

Water flows from the offtake point for the holistic scenario

The water offtake point of pipe P-1758 (Figure 7.22) that connects the zone to the main distribution reservoir is represented by a series of artificial elements (Chapter 5). The flow through this pipe includes the flow to the DMA for the Central zone (Kalu1 DMA) but excluding the supply from boreholes within Chelstone zone. The total amount of water supplied to Kalu1 DMA through P-1758 is approximated to be **7,062 m^3/d** and the amount supplied by the 15 boreholes within the zone is approximated to be **3,725 m^3/d**. Consequently, the total water flow through pipe P-1758 ($F_{\mathrm{P-1758}}$) from the water offtake point is:

$$F_{\mathrm{P-1758}} = 59{,}549 + 7{,}062 - 3{,}725 = \mathbf{61,772\ m^3/d} = \mathbf{714.95\ l/s}$$

Determination of the 2035 nodal base demands for the business as usual scenario

The process for determining demands and leakage discussed for the holistic scenario is followed except the use of Equation 7.3. This is because in this case $Supply_{Zone}$ and $Supply_{DMA}$ as determined in parts iii and iv respectively, are used directly as system input volumes from which leakage various are determined. For the business as usual scenario, there will be a water supply deficit of **299,080 m^3/d** and as such CWS for the whole network will not be possible. For this scenario, the average discharges through pipe P-1758 from the offtake point is:

$$F_{\mathrm{P-1758}} = 75{,}465 + 7{,}538 - 3{,}725 = \mathbf{79{,}278\ m^3/d = 917.97\ l/s}$$

Simulation of the Chelstone zone WSS under the holistic scenario

For the hydraulic simulation of the Chelstone zone WSS under the holistic scenario, the average elevation of the artificial tank is 1302.57 (discussed in subsection 5.7.5). The simulation results at 05:00 am show that 67 nodes have pressure heads that are less than 7 m (Figure 7.22). These results entail the need to increase the capacity of the Chelstone zone WSS.

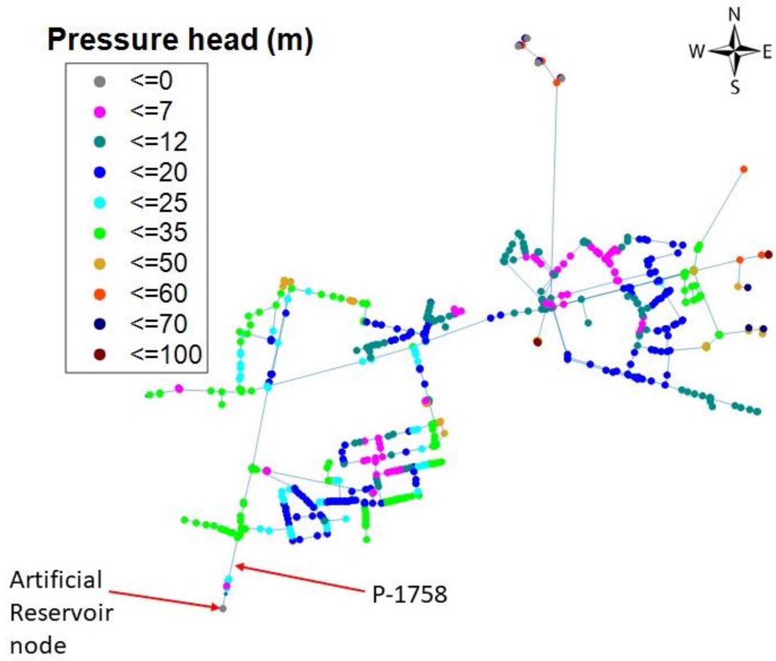


Figure 7.22: Holistic scenario 05:00 am simulation results pressure distribution map for Chelstone Zone WSS before rehabilitation

7.3.2 *Optimisation problem for the Chelstone zone water supply system rehabilitation*

Rehabilitation of the Chesltone zone is important because the capacity of the existing WSS will be low in 2035. Rehabilitation involves either the addition of parallel pipes or replacement of smaller pipes with larger ones or both (Savić et al., 2018). However, rehabilitation by way of pipe replacement is used in this study. This is because known works done on the Chelstone Zone network during the Millennium Challenge Account project (Millennium Challenge Account - Zambia Limited, 2013) involved the replacement of old pipes by new ones where old pipes existed. The rehabilitation problem is posed as a minimisation of the cost of installing larger pipes and their corresponding flow control valves in place of small ones (Equation 7.2).

$$Min\ C_T = \sum_{j}^{N_n} C_x L_x + \sum_{v}^{N_v} C_v + \sum P_{constv} \tag{7.2}$$

Where $C_T$is the total cost of installing new pipes and valves (US \$), N_n is the total number of new pipes to be installed, C_x is the cost per meter for pipe x, L_x is the length of pipe x, C_v is the unit cost for valve v, N_v is the total number of new valves to be installed, P_{constv} is the penalty cost for constraint violation

The constraints for the optimisation problem are:

i. Nodal pressure heads should not be less than the required pressure (7 m)
ii. The difference between the simulated and set total flows through the pipe connecting Chelstone zone to the main distribution reservoir (pipe P-1758) should be zero
iii. Head loss in each pipe should not exceed 10 m/km as recommended in Brian Colquhoun Hugh O'Donnell and Partners (2010).

If no constraint is violated, the penalty cost is zero $\left(\sum P_{constv} = 0\right)$, but if there some constraints that are violated, the optimised objective function value has a penalty cost component (Coello, 2000) which indicates how much the generated solution has violated the constraints. When determining the cost of rehabilitation, the penalty cost is deducted from the optimised objective function value.

The design variables for the optimisation of the zone network rehabilitation are the AT elevation (there are no costs attached because of the tank's artificial nature), pipe and valve diameters. The AT is one of the elements in the modelling of the water offtake point. It ensures that the AFCV discharges the set amount of water from the AR and it also influences the flow through the ATCV (Chapter 5, subsection 5.7.5). The required hydraulic head under different scenarios to supply different amounts of water is not known (Indicated in section 7.3). Hence for each scenario, determination of the required hydraulic head for the AT is an optimisation problem. The range for the AT hydraulic heads is from 1295.82 m to 1362.57 m. This range encompasses the minimum and maximum hydraulic heads obtained from the simulations of the whole LWSN (Discussed in subsection 7.1.2).

Pipes and valves are classified in terms of their diameters as small, intermediate and large pipes and valves (Table 7.7). The division of the pipes and valves diameters is aimed at speeding up the optimisation process. In comparison with the original system pipe categorisation (Table 7.8), the lower and upper limit for the diameter ranges for small pipes including the lower limit for the intermediate pipes are raised in Table 7.7 to facilitate the increase of the system hydraulic capacity in the optimisation process. This is because the initial limits show system hydraulic capacity deficiency (Figure 7.22). The splitting of the design variables reduces the number of pipe diameters in each range thereby increasing the possibility of finding optimum solutions. The splitting of the diameters is motivated by the usual classification of pipes into transmission mains, distribution mains and distribution pipes (Simukonda et al., 2010) which in the case of LWSN, are named as shown in Table 7.8 (Brian Colquhoun Hugh O'Donnell and Partners, 2010). In total, there are 694 design variables split into 1 artificial tank elevation setting, 555 small pipes, 58 intermediate pipes, 11 large pipes, 53 small valves, 13 intermediate valves and 3 large valves.

Table 7.7: Pipe size categorisation in this study

Pipe and valve category	Diameter range (mm)
Small	225, 280, 300, 350
Intermediate	400, 450, 500
Large	600, 700, 900

Table 7.8: The pipe categories in the LWSN

Pipe category	Diameter range (mm)
Primary pipes	125 ≤ diameter < 300
Secondary pipes	300 ≤ diameter < 600
Tertiary pipes	600 ≤ diameter ≤ 900

The Chelstone zone hydraulic model has both time-based and tank water level-based controls. Models with time-based controls can only be used on the specific day or conditions under which they were calibrated (Walski, 2017). To remove this limitation on the operation conditions for the zone during and after optimisation pump controls that are time-based rather than tank water level-based are removed from the zone's EPANET hydraulic model which is transformed into the M-SIPDA format before rehabilitation optimisation. This is because, after the rehabilitation optimisation, hydraulic conditions in the system are expected to change greatly and hence make the time-based controls unreliable as explained in Walski (2017). Moreover, valve FCV-13 is opened so that water can flow directly into the system and CH-Reservoir thereby increasing the capacity of the system. The system capacity increase effect of opening FCV-13 is demonstrated in subsection7.2.1.

7.3.3 *The Genetic Algorithm (GA)*

The optimisation process involved: 1) the MATLAB-EPANET interfacing code for inserting the design variables and extracting the hydraulic simulation results, 2) an appropriate MATLAB GA syntax (MathWorks, Inc. 2021) for optimising the developed single objective function (Equation 7.2).

Genetic algorithm (GA) is used because as a metaheuristic it is designed to be adapted to solve a wide range of hard optimisation problems, rather than specific problems (Savić et al., 2018). Since a single objective function was developed, the MATLAB-based single objective optimisation GA was used.

GAs mimic the natural process of evolution of species in which reproduction, crossover and mutation are the major operators. The reproduction operator enables the production of two offspring (new solutions) from two parents (initial solutions). The crossover operator makes the new offspring have a combination of genetic information from both parents and the mutation operator introduces a slight perturbation to the offspring's genes to introduce some extra variability within the population (Michalewicz, 1996, Farmani et al., 2007). Through these operators, the GA performs a random (multi-directional) search for better organisms (solutions) by maintaining a population of potential solutions and encouraging information exchanges in these directions. At each generation, the relatively good solutions are made parents for the next generation offspring while the relatively bad solutions die. To evaluate the fitness of different solutions, the objective (evaluation) function is used in the role of the environment (Michalewicz, 1996).

For the optimisation of the Chelstone zone network under the holistic and business as usual scenarios, the number of generations is kept at 35 and the population size is 50. This is because of the computer space limitations of the laptop for optimisation using M-SIPDA for hydraulic simulations. These numbers of generations and the population size are too low for the normal GA optimisation. However, they are applicable here because of the splitting of the design variables into three categories and the selection of best solutions from each generation based on elitism. Since GA gives priority to elite members of a generation to become parents of the next generation, it is assumed that solutions improve with increasing generations with the best solution (member) being expected to be in the last generation. It is the best member of the last generation that is saved by the MATLAB GA used in the optimisation process. Consequently, a code is written to insert the artificial tank elevation and diameters for pipes and valves corresponding to the best solution in the EPANET hydraulic model and saves it with a new name. This way, the pipes and valves that have been replaced are easily identified by using the EPANET input files before and after optimisation.

7.4 **Optimisation results**

The optimisation results for the holistic scenario and the business as usual scenario shows that the cost of rehabilitation (cost of replacing smaller old distribution pipes (small pipes), distribution mains (intermediate pipes) and transmission mains (large pipes) and flow control valves with bigger new pipes and valves) for the Chelstone zone is US $\$10.1*10^6$ and US $\$10.5*10^6$ respectively.

For the holistic scenario, the optimisation duration is 4.7 days. The variations (improvements) of the fitness values with the increasing number of generations are shown in Figure 7.23. It is noteworthy that the last generation's best objective function value in Figure 7.23 is greater than the value ($\$10.1*10^6$) indicated in the first paragraph of this section. This is because the penalty cost component of the objective function (Equation 7.2) is not zero. The total volume of water expected to flow through pipe P-1758 to Chelstone zone per day under the holistic scenario is 61,772 m^3/d. The simulated volume is 62,622 m^3/d. Thus, there is an error of 1.37 % which contributes to the penalty cost. There are no constraint violations regarding pressure heads because all pressure heads at demand nodes are above the required pressure (Figure 7.24). Regarding the head loss constraint violation, an hourly maximum value of 9.14 m/km occurs in pipe P-385 (Figure 7.24).

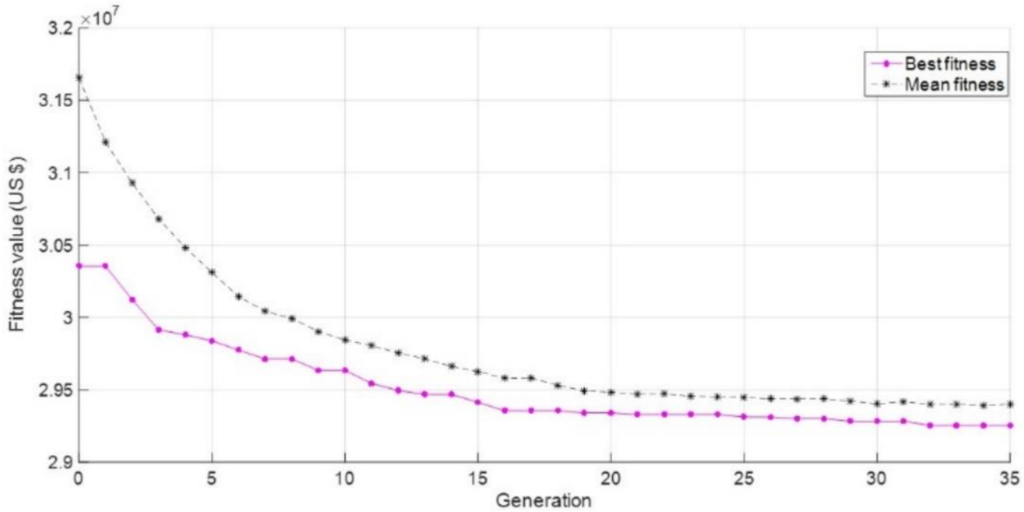


Figure 7.23: Best and mean fitness values of individuals in each generation (Holistic scenario)

For the artificial tank, the average elevation is 1301 m. This is lower than the average hydraulic head before rehabilitation which is at 1302.57 m. The lower hydraulic head can supply adequate pressure in the WSS because of reduced head losses in larger pipes. The pressure distribution map at 05:00 am is shown in Figure 7.24. In the figure, the nodes that appear to have pressure heads equal to or less than the required 7 m are tanks.

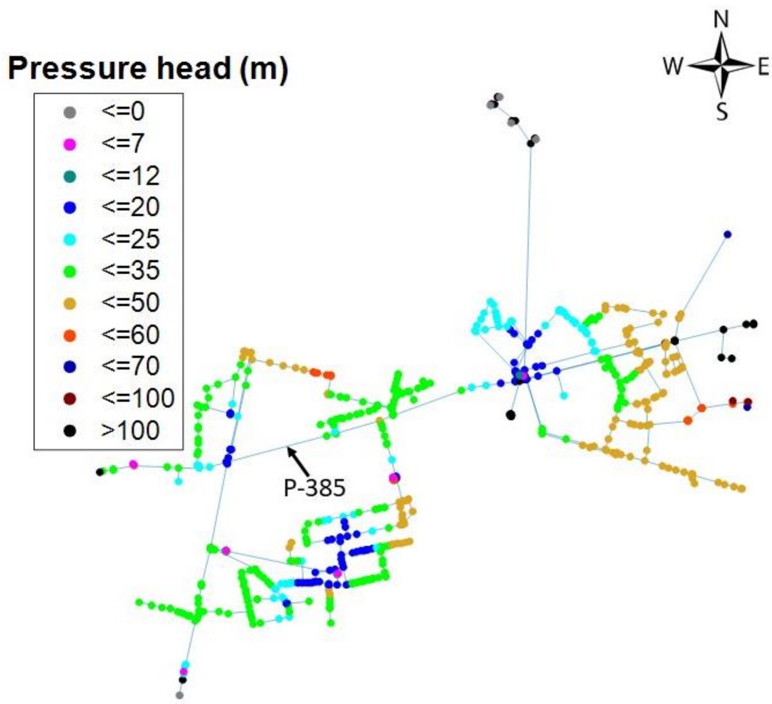


Figure 7.24: Optimised holistic scenario pressure distribution map

For the business as usual scenario, the optimisation duration is 4.8 days. The variations (improvements) of the fitness values with the increasing number of generations are shown in Figure 7.25. The last generation's best objective function value in Figure 7.25 is greater than the value indicated in the first paragraph of this section ($10.5*$10^6$). This is because of the penalty cost component of the objective function (Equation 7.2) which is not zero. The total volume expected to flow through pipe P-1758 to Chelstone zone per day under the business as usual scenario is 79,278 m^3/d. The simulated volume is 83,228 m^3/day. This gives an error of 4.98% which, though acceptable (less than 5%), contributes to the penalty cost. Pressure constraint violation is the major contributing factor to the penalty cost. The worst network node pressure deficiency situation for the business as usual scenario occurs at 08:00 rather than at 05:00. The pressure distribution map for the situation is shown in Figure 7.26. As shown in the figure, there are many (about 138) nodes that have pressure heads that are less than 7 m. The large number of nodes with pressure deficiencies is attributed to the water supply deficit for the business as usual scenario (discussed in Chapter 6, subsection 6.6.1). For the head loss constraint, pipes P-385 and P-2805 (Figure 7.26) have maximum hourly unit head losses of 31 and 11 m/km respectively which are higher than the maximum allowable unit head loss of 10 m/km.

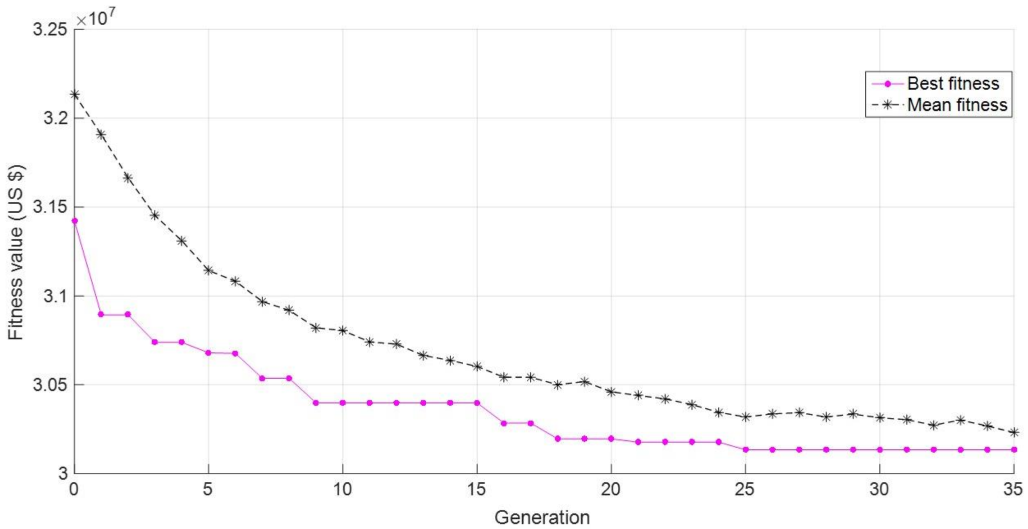


Figure 7.25: Fitness values variations with increasing number of generations (Business as usual scenario)

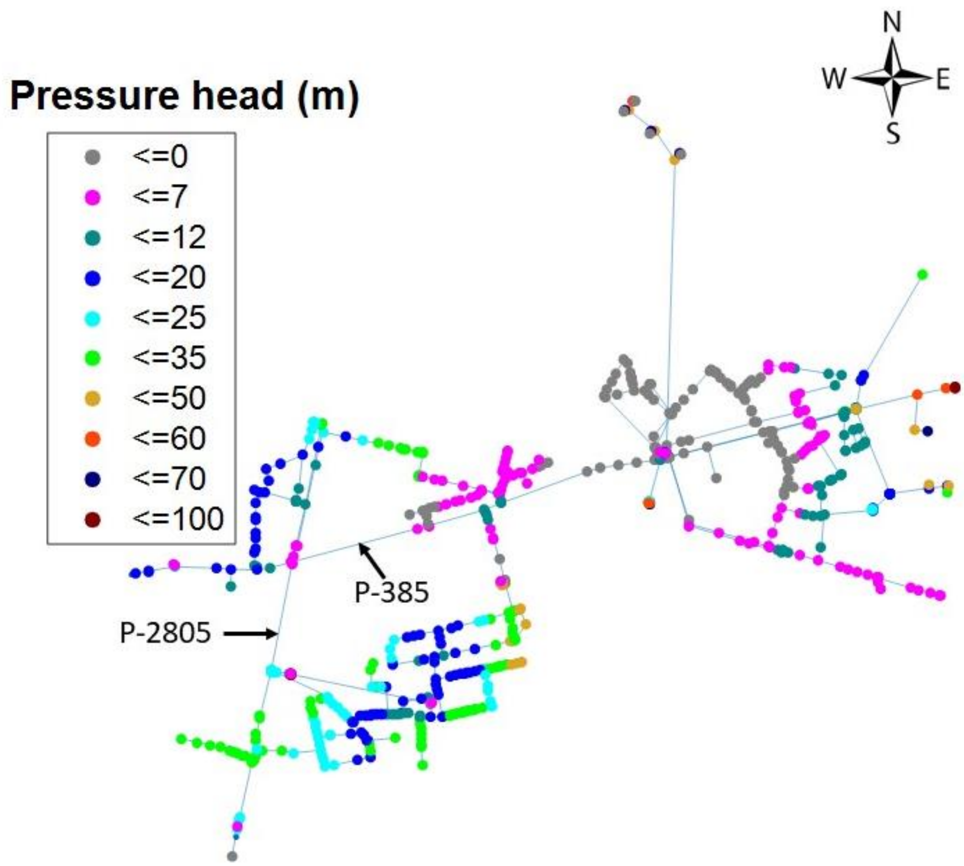


Figure 7.26: Optimised business as usual scenario pressure distribution map

The summary of the various pipe diameters and valves for the original network, the holistic and business as usual scenario are presented visually in Figure 7.27, 7.28 and Figure 7.29 respectively. Figure 7.27 shows that the major pipes for the original Chelstone zone WSS are the 375, 400 and 600 mm diameter pipes. The 450 mm diameter pipe is very short and of no significant effect on the system hydraulic capacity. After the rehabilitation optimisations for the two scenarios, the longest single stretch of the 600 mm diameter (the stretch just before joining with the 375 mm pipe) pipe is unchanged for both scenarios (Figure 7.28 and Figure 7.29). This is because increasing the diameter of this pipe would raise costs for reaching the same performance levels that can be achieved through the combination of other smaller pipes. For both scenarios, the dominant set of pipes suggested for rehabilitation for the small pipes are of diameters 225, 280 and 300 mm. For the intermediate sizes, 400 and 450 mm pipes are dominantly suggested (leaving out 500 mm) as shown in Figure 7.28. Noting that for pipe P-385 the head loss constraint was violated under both scenarios, with a diameter of 450 mm suggested under the holistic scenario, the 500 mm diameter would be ideal.

This could not be done during the optimisation process presumably because the head loss constraint violation was more acceptable than the higher cost or the violation of the constraint for limiting the water supply to Chelstone zone. For the large pipes, Figures 7.28 and 7.29 show that the dominant diameters are 600 and 700 mm (leaving out 900 mm). Figure 7.30 shows the 179 pipes that are suggested for rehabilitation in the same way under both the holistic and the business as usual scenario. Of the 69 valves, 28 was common to both scenarios. Since the holistic scenario represents the ultimate desire for solving the IWS problems, the rehabilitation following this scenario (Figure 7.28) would reduce the regret costs if the business as usual scenario occurred.

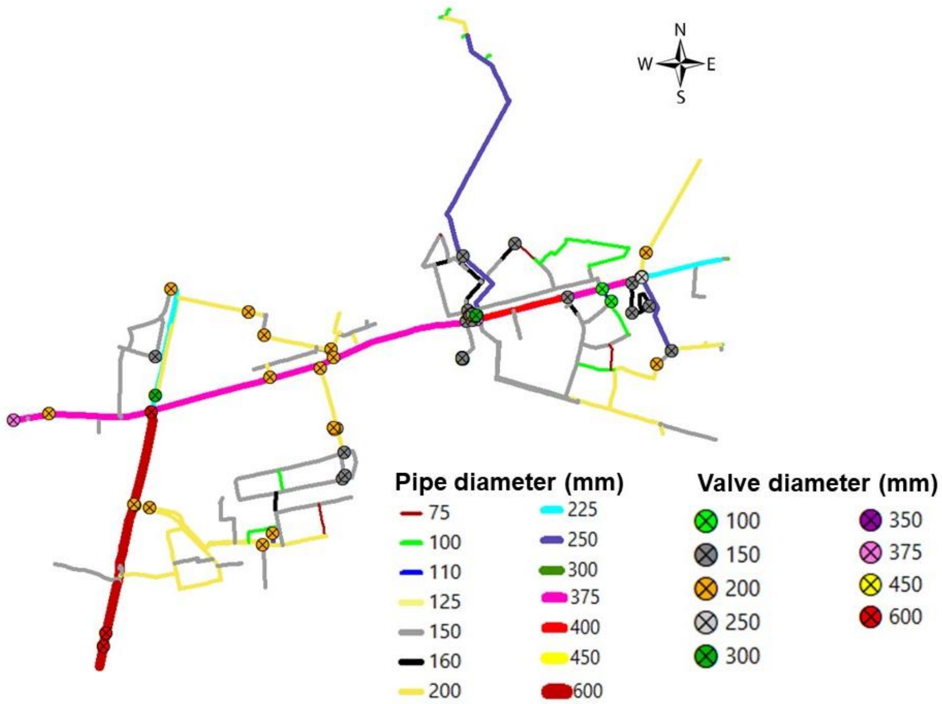


Figure 7.27: Pipes and valves for the original Chelstone zone network

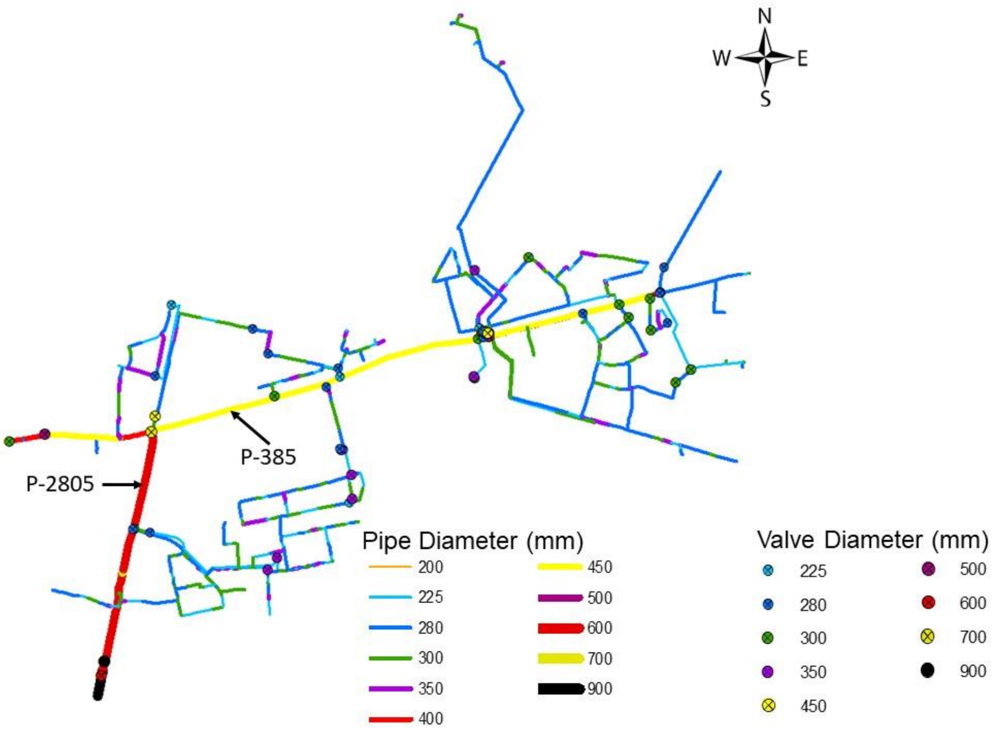


Figure 7.28: Pipes and valves for Chelstone zone optimised rehabilitation network under the holistic scenario conditions

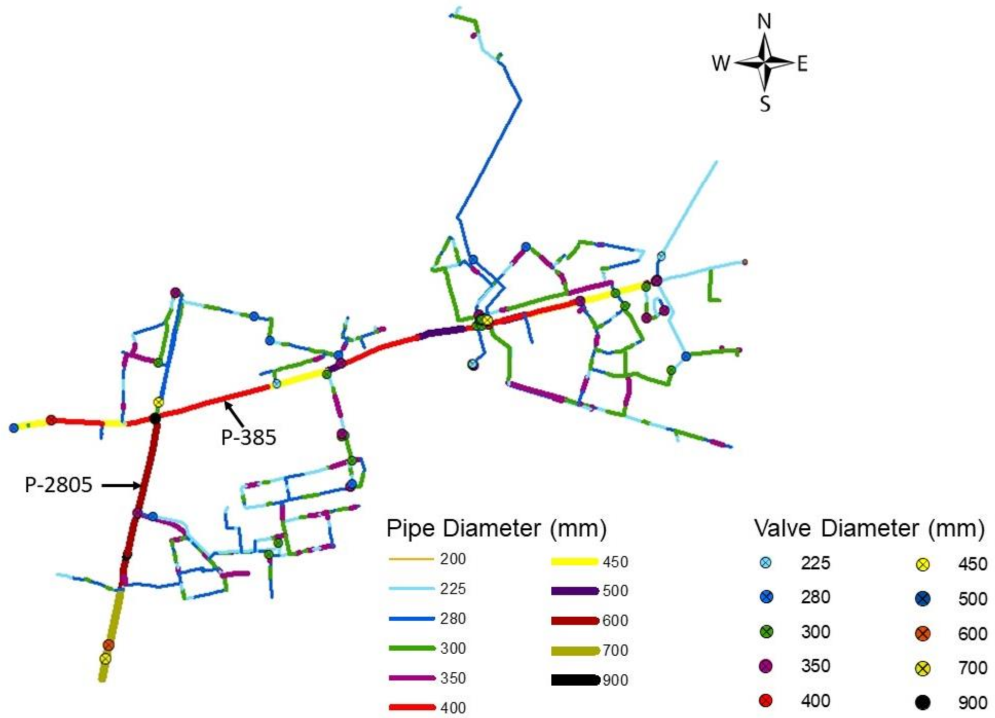


Figure 7.29: Pipes and valves for Chelstone zone optimised rehabilitation network under the business as usual scenario conditions

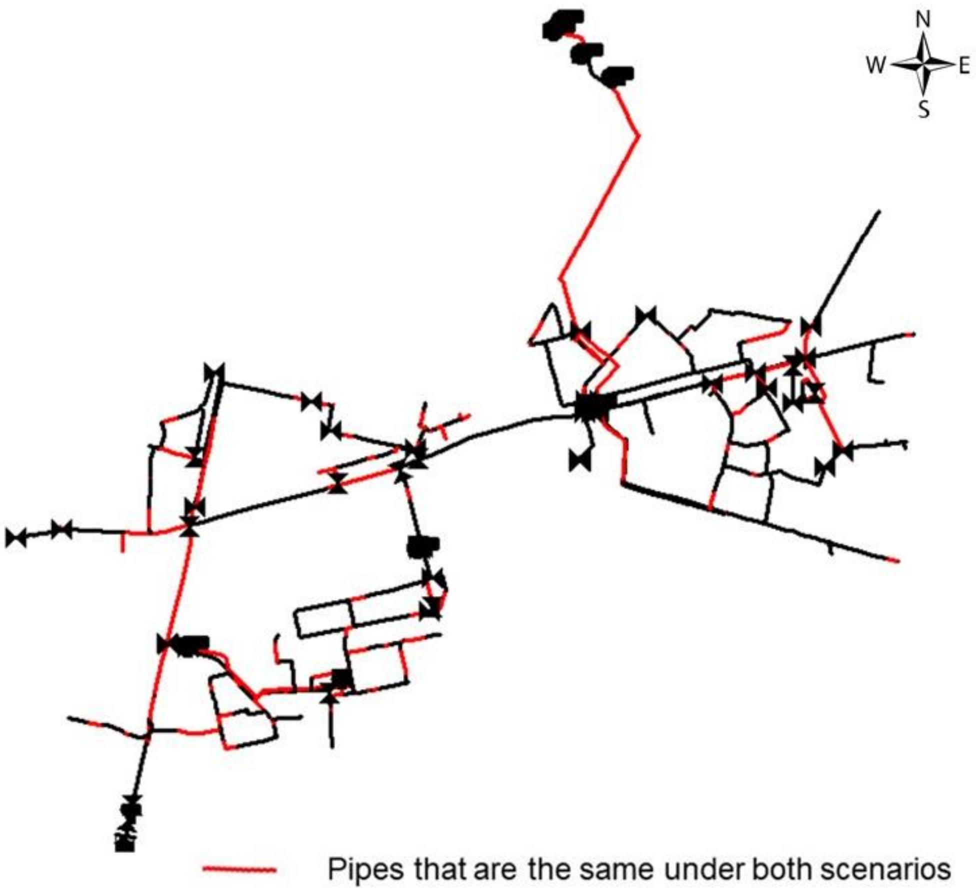


Figure 7.30: Pipes that are found to be the same for all cases of the holistic scenario and the business as usual scenario

7.5 Occurrence of the business as usual scenario

Although the water supply investment master plan for Lusaka is based on the NRW management scenario projections, the scenario is difficult to attain because of the failure to reduce NRW to 15% of the system input volume by 2035. NRW for the LWSN has persistently been above 40%. Since there are no policies or directives in Zambia to reduce consumption water demand, water demand is at least going to be as high as it is or may even become higher. This situation shows that the business as usual scenario is likely to be the scenario for the LWSN. Since for the business as usual scenario, there is a water deficit, conversion to CWS for the whole LWSN or the whole Chesltone zone by 2035 is not possible. Some DMAs will be on CWS status while others will be on IWS. For DMAs that are already on CWS status, leakage management is important so that water is made available to improve water supply services in other DMAs. For DMAs to be converted to CWS their selection is a challenge in Zambia because:

i. There is the institutionalised segregation of water consumers according to their status which is measured in terms of their housing types. The people who are expected to have the poorest water supply are those in the informal houses followed by those in the low cost. Both housing types are in peri-urban areas. Dominantly, the people who are subjected to the social tariffs (the lowest block of the IBT structure) are found in these housing types. In view of the need to recover full water supply costs, providing water continuously to these consumers before those who can pay the full cost of water supply is not a sustainable approach.
ii. The division of the WSS into the main and satellite systems with some peri-urban areas supplied water by both systems. For peri-urban areas where water supply is from the satellite system, improving water to CWS requires increasing water supply from the groundwater sources. Since no increase in groundwater supply is expected currently, increasing water resources to these peri-urban areas entails connecting them to the main system. For peri-urban areas that are supplied water from the main system, they are supplied water through their adjoining urban DMAs. This means improving water supply to these peri-urban areas depends on improving water supply to their adjoining DMAs.
iii. The decision of water supply improvements is highly influenced by the directives of the politicians who would require improvement of water supply conditions to certain areas or DMAs because of their potential to provide supporting voters.

7.6 Conclusion

The quantitative application of the scenarios gives information on the water supply-demand balance, but it does not provide information on the WSS hydraulic capacity under the different scenarios. This information is provided by modelling of the WSS. The method for modelling IWSS developed in Chapter 4 is applied first to the whole LWSN as it was in 2010 and then to the Chelstone zone of the network. The modelling results show that treating leakage as implicitly contained in the demand allocated to the nodes results in excessive flows during peak hours which leads to the underestimation of the system capacity because of excessive head losses that lead to low system pressure heads. This is because in modelling leakage as part of consumption demand, leakage is subjected to the same peak factors as consumption demand during peak times. This is contrary to the consumption demand-pressure-leakage relationship in which high consumption demands lead to lower pressure heads and reduced leakage flows and lower consumption demands lead to high-pressure heads and high leakage flows. Modelling consumption demand alone while omitting leakage completely as the case is in many PDA methods, underestimates system flows at peak hours thereby leading to overestimation of the system hydraulic capacity because of high pressures which show that the system can handle additional demand loads.

In comparison with the reported Chelstone zone total demand values, the M-SIPDA simulated values are slightly higher, but acceptable because its values reflect an increase in the demand by a factor of 0.09 and there are many uncertainties in the hydraulic model, the quality of data used for calibration and the application of the manual calibration method to a large network. These uncertainties require that the M-SIPDA should be applied to more systems with well updated hydraulic models and high-quality calibration data which can be used with the implicit calibration techniques. For the visualisation of the simulation results, the use of pressure distribution maps makes it easy to see areas of the supply zone that have water supply problems due to low or excessively high pressure. Using the pressure distribution maps, the closed valve (FCV-13) was identified which caused pressure deficiencies at nodes in the eastern part of the Chelstone zone WSS. When FCV-13 is opened, the pressure deficiency conditions for all the nodes in the eastern part of the WSS are eliminated.

Due to the anticipated increased water supply from the sources and total water demand by the year 2035, hydraulic capacity of the Chelstone zone WSS is expected to be insufficient thereby resulting in pressure deficiencies even after opening FCV-13. There is a need to increase the network hydraulic capacity to match the anticipated high flows and minimise the pressure deficiency conditions. The rehabilitation problem for the zone to increase the hydraulic capacity to respond to the increased pipe flows is posed as an optimisation problem with a total of 694 design variables. To determine the pipe flows and node heads necessary for evaluating the extent of constraint violations, hydraulic analyses are done using the M-SIPDA. The cost of rehabilitating the zone network under the holistic scenario and the business as usual scenario are US $\$10.1*10^6$ and US $\$10.5*10^6$ respectively. Of the 624 Chelstone zone WSS pipes, 179 are found to be rehabilitated by the same sizes of pipes under both the holistic and the business as usual scenarios. For pipes that are different, for both scenarios there are sets of pipes that are dominant under the category of small, intermediate and large. For the small pipes category, 225, 280 and 300 mm pipes are dominant, 400 and 450 mm are dominant for the intermediate, and for the large pipes, 600 and 700 mm are dominant diameters. Since the ultimate solution for the IWS problems is conversion to CWS, implementing the holistic scenario topology would result in low regret costs if the business as usual scenario occurs in 2035. On account of limited resources, the rehabilitation process following the proposed topology should be phased according to the changes in water demand and supply over the planning horizon.

Even though conversion to CWS is desirable, the business as usual scenario is likely to be the scenario into the foreseeable future unless efforts to avert it are doubled. Since the business as usual scenario leads to water supply deficits, the need for conversion to CWS requires that some DMAs be added to those that are already on CWS status to the extent that the water supply condition for those on IWS do not deteriorate. However, the Zambian situation presents a challenge on how to select the DMAs to be converted to CWS because of the segregation of water consumers based on their status, the interconnectedness of the LWSN, the need for water supply sustainability in view of increasing poverty levels and the role of politicians.

8 CONCLUSIONS AND RECOMMENDATIONS

This chapter presents the research conclusions and recommendations besides highlighting the research contributions to the body of knowledge. Section 8.1 presents a concise outline of the thesis. Section 8.2 presents the main conclusions of the research. The research contributions to the body of knowledge are presented in section 8.3, while the recommendations for practice and future research are presented in section 8.4.

8.1 Thesis summary

8.1.1 *Research aim and objectives*

Research aim

To develop a methodology for the sustainable conversion from intermittent to continuous water supply systems.

Specific objectives

i. To review the literature on IWSS
ii. To identify and analyse the problems and the root causal factors of the IWSS.
iii. To develop scenarios for a sustainable conversion from IWS to CWS status.
iv. To develop an approach for modelling IWSS.
v. To apply the developed scenarios and modelling approach to a real-world WSS.

The objectives are exhaustively discussed under seven chapters which are linked to the major research components and steps for the sustainable conversion from IWS to CWS status.

- The first objective is critical to ensuring that this research remained relevant given that the field of IWSS is topical and new contributions to the body of knowledge are coming which should be known if the current research is to add new knowledge.

- Specifically, the second objective is covered in Chapter 2 which reviews the literature from different disciplines to discuss the causes of water supply intermittency, problems, and solution options for managing the problems.
- Chapter 3 is concerned with the third objective. The chapter presents the significance of scenarios in incorporating future water supply improvement ventures' uncertainties. The chapter also discusses the GSG scenarios and their linkage to city expansion forms and WSS sustainability.
- Chapter 4 corresponds to the fourth objective. The chapter presents the development of the method for modelling IWSS which takes care of the multiple water supply schedules characteristic of IWSS and leakage modelling.
- Chapter 5 describes the Lusaka Water Supply Network (LWSN) as the case study network. The chapter highlights the challenges faced by the water supply sector in Zambia from the political, institutional, legal/policy and resource perspectives. The chapter also discusses a new method for modelling a water offtake point for isolating a zone or DMA such that the isolated zone or DMA can be supplied predetermined quantities of water. This is important when modelling the isolated zone or DMA under different scenarios in which the limited water resources must be shared among many zones or DMA proportionate to their water demands.
- Chapters 6 and 7 cover the fifth objective. Chapter 6 discuss the application of scenarios to the case study network to assess the adequacy of the water resources if any of the four scenarios occurred. The chapter demonstrates the significance of considering different scenarios in the planning process of water supply improvement ventures. Using the water supply-balance for different scenarios, scenarios under which CWS is feasible and those under which it is infeasible can be identified. Chapter 7 discusses the application of the developed method for modelling intermittent water supply systems and the technique for isolating zones to the analysis of the Chelstone zone WSS capacity in 2035 under the holistic and the busines as usual scenarios. Due to the reduced capacity of the

current Chelstone zone WSS, there is a need to increase the capacity through rehabilitation. The approach involves optimisation with the view to identifying the future network pipe combination that would reduce regret costs when a scenario other than the desired scenario occurs.

8.1.2 *Major research assumptions*

- For modelling purposes in this research, it is assumed that the pipes are always full and the Hazen Williams C-factors are constant.
- For modelling purposes, leakage is assumed to take place only at junctions. This is like the approach for modelling consumption demand in which several household connections are aggregated at a few selected junctions.
- In this study, the minimum node pressure head is assumed to be equal to atmospheric pressure and as such, it is taken as zero.
- It is assumed that pressure increase beyond the required pressure does not change the node outflow in respect of consumption demand.
- Nonrevenue comprises real losses, apparent losses and unbilled authorised water consumption. However, for modelling purposes in this study, it is assumed that Non-revenue water **(**NRW) represents leakage.

8.2 Conclusions

8.2.1 *Main conclusions*

- ❖ Intermittent water supply systems have so many interconnected causes that it is difficult to isolate the root causes. Some of the causes are internal factors while others are external to the IWSS. External factors which have been identified in this research as root causes of the IWS mode are poor governance, demographic and economic dynamics, hydrological regime changes, poor system management and operation, limited skilled manpower, unplanned extension of systems, lack of customer awareness and poor electricity supply.
- ❖ The intermittent water supply tree is presented to illustrate the fact that the target of any sustainable intervention to managing problems of water supply intermittency should be the root causes (roots of the intermittent supply tree). A reflection on the root causes shows the importance of

interdisciplinary approaches to dealing with them because they have political, social, economic, natural and technical origins.

- Where water resources are in abundance, generalised arguments against conversion to CWS status due to lack of financial resources in developing countries are not correct because the literature shows that the financial resources to achieve such conversions are not mainly met by the concerned governments but by donors and cooperating partners. Moreover, the greater part of the funding, even for intermittent supply systems, is still from donors and cooperating partners.
- Even if financial support was to be available, the conversion process is not yet clear because there is no modelling approach for intermittent water supply systems which would aid in the understanding of the hydraulic behaviour of these systems during planning and operation. Moreover, there is limited use of scenario analysis in the planning process for IWS improvement even though scenarios are a critical component to the systematic conversion approach that will enable informed adaptation of water supply systems to future and uncertain changes of governance, climatic, demographic and economic factors.
- Understanding the functions of scenarios, the process of scenario development, the linkage between the GSG scenarios to urban forms and the approaches to water demand management is important because scenarios provide a means of incorporating future uncertainties in the planning process for IWSS improvement interventions for which there are numerous unknowns because of poor database management. The importance of scenarios lies on the understanding that human beings (planners) have some knowledge of the future, that the knowledge is limited and that the future can be influenced by human choices and actions.
- The narratives of the GSG scenarios are comprehensive and cover all drivers of change. Consequently, these scenarios provide an objective basis for analysing the governance systems of nations and the linkage of the governance systems to the developed scenarios such as the water supply sustainability scenarios developed in this study.
- Basing on the narratives for the GSG scenarios, it can be seen that the current world political dispensation makes the occurrence of Market Force

and the Fortress World scenarios more likely in developing countries than the Policy Reform and the New Sustainability Paradigm scenarios for which the necessary means of attaining them do not yet exist.

- Concerning the total water demand, the occurrence of the Market Force scenario entails high leakage and consumption water demand, and high cost of delivering water to new areas which makes water unaffordable to the poor. The PR scenario relies on policies and regulations aimed at improving equity and controlling the urban sprawl to compact/uniform urban development. The implication of these is the increase in consumption water demand especially with poor technology use in IWSS (developing countries) which also contributes to high levels of leakage. For the FW scenario, it is not clear how water demand will be affected because water demand depends on the level of water supply restrictions to the poor masses, the population of the affluent and their use of the limited available technology. The NSP is the scenario envisaged in the SDGs. Under this scenario, equity is high and (water) consumer behaviour is very good and the use of technology is high. These result in the reduction of leakage while consumption demand may increase or reduce depending on which direction promotes equality and equity in terms of water supply services.
- Basing on the discussion of the scenario development process and the narratives of GSG scenarios, four scenarios for assessing the sustainability of planned water demand management targets to improve water supply services for IWSS are developed using the two-axes scenario development method. The horizontal and vertical axes are represented by the domestic per capita water consumption and the nonrevenue water (leakage) which are the two major drivers of the total water demand. Since these drivers are key to the sustainability of water supply, the axes form a quadrilateral which has been named the water supply sustainability quadrilateral. The extent to which the total demand is met determines whether the WSS has sufficient water resources to become a CWSS or not. After assessing the adequacy of the water resources, the WSS hydraulic capacity is assessed through modelling.
- For modelling purposes, water supply systems should be categories into three categories according to the water supply mode and pressure status. The three categories are continuous water supply systems with adequate

pressure, continuous water supply systems with deficient pressure and intermittent water supply systems with deficient pressure. This classification clearly shows the need for more studies aimed at developing modelling techniques for intermittent water supply systems.

- The Modified Single-Iteration Pressure Driven Analysis (M-SIPDA) developed in this study is proposed for modelling IWSS. This method considers the multiple water supply schedules characteristic of IWSS, explicitly modelling of leakage and consumption demand taking care of their similarities and differences.
- Modelling leakage as implicitly contained in the demand allocated to the nodes results in excessive flows during peak hours which leads to the underestimation of the system capacity because of excessive head losses that lead to low system pressure heads. This is because in modelling leakage as part of consumption demand, leakage is subjected to the same peak factors as consumption demand during peak times. Modelling consumption demand alone while omitting leakage completely as the case is in many PDA methods, underestimates system flows at peak hours thereby leading to overestimation of the system hydraulic capacity because of high pressures which show that the system can handle additional demand loads.
- Since the addition of artificial elements complicates the topology of the water supply system, a technique for visualising node parameters such as pressure is developed in this study using the graph theory implemented in MATLAB to draw the WSS graph and develop the distribution map for the selected parameter (pressure) at any time.

8.2.2 *Conclusions specific to the Lusaka water supply system*

- Before the planning of sustainable water supply improvement approaches which include conversion to CWS, the operating environment and the state of the WSS must be described. The operating environment for water supply systems constitutes the physical, ecological, social, economic, cultural, legal and technical entities.
- For the LWSN, problems that led to the deterioration of the water supply infrastructure and services still exist to date as evidenced by: (i) the poor

implementation of the seven water sector principles, (ii) poor policy and legal framework that have no effective enforcement mechanisms, (iii) political interference in water sector institutions that lack capacity, autonomy and have conflicting responsibilities. While these factors cannot be handled using technical solutions, they affect the choice, funding, and implementation of technical solutions and ultimately influence the effectiveness of NRW management. Currently, NRW is persistently high and efforts to reduce it have proved futile.

- The IWS for the LWSN is implemented at two levels. First by the production division that supplies water to some zones and peri-urban area agglomerations intermittently. Then the zones or peri-urban area agglomerations implement it by rationing between the DMAs that are connected to different water sources.
- The zones or peri-urban area agglomerations are not independent of each other because of their hydraulic interconnectivity which is either because different zones and peri-urban area agglomerations are supplied water from the same distribution reservoir(s) or because the zones and the peri-urban area agglomerations' pipe networks are connected at many boundary points. This interconnectivity makes it difficult to attribute reservoir or tank water level variations to the water consumption of a single zone, peri-urban area agglomeration or DMA.
- The complexity of the LWSN due to the interconnectivity of zones and peri-urban area agglomerations is worsened by the fact that while some of the boreholes pump water directly into the WDS, others pump it both directly into the WDS and indirectly into the distribution reservoirs first before it is delivered into the WDS. This aspect of boreholes makes the distinction of the two water supply subsystems (the satellite and the bulk subsystem) very unclear for areas that are supplied water by both systems.
- The IWS status is due to lack of capacity by the WSS which is due to the combination of the increasing consumption water demand, high NRW and low WTP capacity.
- The modelling of the LWSN is a big challenge because of the lack of updated information on the topology and consumers, non-existent of the modelling tool, the high hydraulic interconnectivity of the zones or peri-urban area agglomerations or DMAs and lack of measured data.

- Due to the complexities of the whole LWSN, comprehensive analysis is done through an isolated zone. A technique for isolating zones is developed. The technique employs a series of artificial valves and nodes to model the water offtake point ensuring that the total flow to the zone after isolation closely matches the total flow before isolation and enables the supplying of predetermined amounts of water to the isolated zone. This is a very important aspect of modelling the isolated zone under different scenarios.
- Applying the GSG scenarios to the Zambian policy/legal framework shows that the policy trends, current state and the desired future state dominantly conform to the MF scenario. This has serious implications for water supply improvements because of the increasing number of people living in poverty.
- There exists a plan on how Lusaka city should be developed in a coordinated manner by the year 2030. Under this plan, Lusaka city is considered into the inner city (areas that are already developed) and outer city (new areas) in which compact development and edge expansion are planned respectively. Whether the planned development takes place or not, what is certain is that the city will continue expanding due to population growth and urbanisation and any improvement in the water supply situation must consider this certainty.
- Although conversion to CWS status is advocated in planning documents for the LWSN (by consultants from developed countries), the importance and the feasibility of the conversion to CWS status as a targeted desired future state for the LWSN must be understood by the utility company and the regulator first if it is to be supported by the policymakers and be a success. Currently, the importance of converting to CWS and the conviction that conversion to CWS is feasible are lacking.
- For domestic water consumption, the use of alternative water sources is not common in Zambia. There are no set target levels of water consumption or clearly defined policies for efficient water use and consumption water demand management in the country. However, in Zambia (Lusaka) it is accepted that people are segregated according to their status whereby the affluent who live in high cost housing types are expected to consume more water than the poor. This institutionalised

segregation of water consumers means the concept of equity in water supply should be treated with caution. Equality may be more applicable because it balances the need for the financial sustainability of the utility company and that for equitable water supply.

- The application of the water supply sustainability scenarios to the LWSN shows that the best scenario would be the holistic scenario which corresponds to the NSP scenario of the GSG scenarios. Under the holistic scenario, both demand and NRW are reduced to minimum possible values. The worst is the business as usual scenario which corresponds to the MF scenario of the GSG scenarios. Its occurrence would result in large water supply deficits.
- Due to the anticipated increase of water supply from the source and total water demand by the year 2035, there is a need to increase the WSS hydraulic capacity to match the anticipated high flows. From an optimisation problem applied to both the holistic scenario and the business as usual scenario, the pipe combinations for the holistic scenario under the assumed CWS condition would be better for implementation. This is because conversion to CWS is the ultimate goal for solving problems of IWSS. The regret cost will be reduced if the business as usual scenario occurred because of the 179 common pipes for the holistic and the business as usual scenario and the closely related sets of small, intermediate and large pipes between both scenarios.
- Since there are no policies of reducing consumption water demand in Zambia, leakage reduction is a challenge and conversion to CWS status is not yet a major target for local stakeholders, it is most likely that the business as usual scenario will persist into the near future even up to 2035 unless advocacy efforts are doubled. Moreover, there are some DMAs which are on CWS status already. These coupled with the institutionalised segregation of water consumers according to their status and the high/low tariffs debate linked to poverty, the role of politicians and the interconnectedness of the LWSN present a unique challenge on how to select DMAs to be added to those that are on CWS status if the business as usual scenario continued.

8.3 Research contributions

Challenges posed by IWSS require extensive research to broaden the body of knowledge and develop techniques that will help effectively understand these systems. The contributions of the current study to the existing body of knowledge include:

- The understanding of IWSS root causal factors. This is important because, in the existing literature, the causes are often mixed with the effects of IWS leading to the proposal of water supply improvement interventions that seek to deal with effects while neglecting the causes completely. The identification and discussion of the root causes of IWSS provide the context of the major problems to contend with when planning the conversion to CWS status.
- Demonstration of the importance of the GSG scenarios to the determination, with respect to sustainable development, of the current policy, legal and institutional status of any country or state, and the exploration of their plausible future development trajectories. The institutional and policy/legal frameworks of a country or state determines the future development agenda and as part of the operating environment of a WSS, affect the sustainability of all water supply improvement endeavours that have a future outlook such as the conversion from IWS to CWS.
- Highlighting of the process of developing scenarios for planning sustainable conversion from IWS to CWS. The scenarios, which correspond to the GSG scenarios, are developed based on the water demand management concept because water demand management is key to the environmental, social and economic sustainability of water supply. The developed scenarios are important to the understanding of the sufficiency of water resources and the capacity of the WSS over the planning horizon. No studies have been found in the literature that have developed and applied scenarios to ascertain the sustainability of an IWSS improvement project with phased water resources development and different water demand management scenarios.

- Development of a pressure driven analysis (PDA) hydraulic modelling method for IWSS. The mathematical model for the method considers the similarities and differences between consumption water demand and leakage. It also considers multiple water supply schedules which are characteristic of IWSS. This is a contribution because currently, most of the existing PDA approaches that perform extended period simulations are developed for CWSS with deficient pressure, do not consider multiple water supply schedules and have no provision for modelling leakage which is significant in IWSS.
- Development of a technique for isolating a zone or DMA from the whole WSS. The technique enables the modelling of the water offtake point of an isolated part (zone or DMA) of a WSS such that predetermined amounts of water are supplied to the isolated part. This is important in the application of scenarios to the isolated part of an IWSS because for these systems, water supplied to a single part has to be proportional to its demand with respect to that of other parts. No technique of isolating a part of an IWSS such that predetermined amounts of water can be supplied to the isolated part has been reported in the existing literature.
- Generated interventions required to attain CWS under different scenarios. These interventions are from three ways in which scenarios have been applied in this study. The first part of the interventions comes from the application of the GSG scenarios to Zambia, Lusaka city and the LWSN. The second part of interventions comes from the water supply-demand balances for the four sustainability scenarios (Business as usual, consumption demand management, the NRW management, and the holistic) and the third part comes from the WSS hydraulic modelling which incorporates optimisation using the holistic and the business as usual scenario. This three-point application of scenarios to the analysis of an IWSS with a view to improving its water supply services is uncommon in the literature.

8.4 Recommendations

Intermittent water supply systems are complex in terms of analysis, operation and management. Solving of problems inherent with these systems requires understanding many intertwined factors. For any improvement approach such as conversion from IWS to CWS status, a step by step process is vital for effective analysis and development of sustainable solutions. Moreover, due to the complex nature of these systems and the limited research output in the literature, so much still calls for research. The limited durations for research projects, inherently entail that this study could not exhaust all the aspects that need to be thoroughly understood. Consequently, the following recommendations are made:

8.4.1 *Recommendations for practice*

Based on the finding and experience during this study, for a successful conversion process, 6 major steps should be followed boldly and objectively. The steps as part of the research process are outlined in Figure 1.1 and are as flows:

i. *Justification for the conversion from intermittent to continuous water supply*

   To win the support of all the stakeholders (including policy makers), advantages and disadvantages of both IWS and CWS should be well explained to the various stakeholders so that they can see the need for the conversion process. The problems of IWS to consumers, society and utility companies should be extensively discussed and all sorts of myths should be dispelled and possible challenges highlighted.

ii. *Identification of root causes of the IWS status*

   The identification of the root causes of water supply intermittency, using multidisciplinary approaches, is very important as it forms the basis for developing appropriate approaches to the sustainable conversion to CWS status. Approaches that treat effects or problems of water supply intermittency without addressing the root causes may be good in the short run but may not be sustainable in the long run. The root causes of water supply intermittency are closely related to the drivers of change, and of all the root causes (drivers of change), governance is a root cause (driver of change) that influences all others. Consequently, the relevant policy/legal and institutional frameworks in the concerned country or state should be well understood.

iii. *Deepened knowledge of the water supply system and its operating environment*

A detailed description of the WSS in the context of the current state of the root causes of the water supply intermittency is critical. It adds to the understanding of which root causes have greater effects on the performance of the WSS than others. It also sets the baseline on which the progress of future development is referenced.

iv. *Understanding the current state and exploring the plausible future trajectories of the root causes*

The root causes (drivers of change) at any instance define the operating environment of a WSS. Thus, while it is important to know the current state of the root causes and their influence on the WSS, it is equally important to know their development trends from the past and their plausible future trajectories. To influence the future trajectory of the root causes, a picture of the desired future is cardinal because without the picture of the future, there is no justification for any efforts. Since developments into the future can take several pathways and have many uncertainties, to incorporate the many possible pathways and uncertainties in the evolution of the root causes, the development and analysis of scenarios provide a means of including the many possible pathways and uncertainties of the root causes when planning IWS improvement interventions.

v. *Quantification of the water surplus/deficit and evaluation of the WSS capacity under different scenarios*

Quantification of the water surplus/deficit under different scenarios refers to the water supply-demand balance. For each scenario, the total water supply for the year defining the planning horizon should be estimated based on the existing plans and/or shown commitment by the authorities (stakeholders) responsible for water supply resources development. The total water demand for the planning horizon should be determined based on the projected evolution of consumption water demand and NRW which are the two major drivers of total water demand. For consumption demand, population growth, urbanisation and industrial development should be estimated. The per capita water consumption or per unit area industrial water consumption changes should be assessed based on the possible policy regulatory measures and technology development and adoption. Also, for NRW the current state, trends and future policy or regulatory measures should set the basis for the various future projections.

vi. *Summary of major steps to ensure a sustainable conversion from IWS to CWS*

The summary of the steps that should be taken to ensure a sustainable conversion from IWS to CWS should be based on the stakeholder's consensus within the accepted existing or established policy/legal and institutional frameworks. Stakeholders unity of purpose after understanding the problems, the resolve to solving IWS problems by ensuring transparent information flow, objective and informed decisions making without undue coercion on these steps is vital not only to the identification of acceptable technical solutions but also to the mobilisation of resources and ensuring that the implementation of the solutions is sustained.

8.4.2 *Recommendations for future research*

Concerning IWSS, areas that need more research include political, social, economic, natural and technical aspects. Thus, recommendations for future research are as follows:

- This study has found that there are different perspectives between stakeholders from the developed countries and those in developing countries on the need and possibility of converting from IWS to CWS. There is a need to identify (in different utilities where this problem has been experienced) reasons for these differences and what leads to communication breakdown in instances where projects are being implemented with different targets by the two groups of stakeholders.
- In the development of scenarios in this study, it was assumed that population growth and urbanisation were predetermined elements and as

such, the population projections in the water supply investment plan for Lusaka applied to all the scenarios. There is a need to develop scenarios that consider population growth as a critical uncertainty which can follow different pathways within the planning horizon. Moreover, the scenarios in this study were based on the segregation of water consumers which was used in the water supply investment plan for Lusaka. Where water consumers are segregated according to status, scenarios development approaches are needed that would consider a wider spectrum of water demand possibilities within the extremes of all consumers being in the low cost houses to that of all consumers being in the high cost houses.

- Leakage is a very important aspect of the hydraulic characteristic of IWSS because leakage levels are high in these systems. Limited pressure driven analysis studies have been reported in the literature that include leakage modelling. The method developed in this study has incorporated leakage modelling. However, the method must be applied to more IWSS with comprehensive data including that for calibration.
- Since good data from real-world IWSS may be difficult, it is recommended that benchmark networks should be developed for these systems as is the case with CWSS. The data for the benchmark networks should be obtained from deliberate agreements between consultants and the client water utilities because during a consultancy, utilities cooperate more and are willing to divulge a lot of data. The other way to get data is through research projects funded to develop such networks using case study utilities.
- For large and complex IWSS, simulation of their hydraulic performance under different scenarios is practicable when isolated parts are used one at a time rather than the whole system at once. For the application of scenarios, the isolated part should be able to be supplied the proportionate quantity of water with respect to its demand and that of the other parts. A water offtake point that would ensure that predetermined amounts of water are supplied to the isolated part was developed in this study, but it is not yet perfect and as such, it is recommended that more research should be done in the development of the water offtake point for isolating parts of IWSS taking into account the proportionate share of the limited water resources.

- Concerning the selection of parts (DMAs) of the LWSS that should be converted first, sustainability requires that the selection criteria go beyond technical solution recommendations. Issues of payment for the water supplied and the influence of consumer status on the quality of water supply services to various consumers require that case-specific approaches are developed which should be in collaboration with concerned water utilities.
- In this study, the topology for the 2035 network was selected based on the optimisation problem applied to the holistic and business as usual scenario for 2035 only rather than for all the years within the planning horizon when there are increases in the water supply from the sources are projected. This approach does not consider the different possible topologies which would be effective during the intermediate phases before the year 2035. It is recommended that analyses are done of the water distribution system topologies and capacities under the two extreme scenarios for each phase of water supply increment.

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Table A.1: Field data measurements, simulated results for WaterGEMS, EPANET, EPANETPDD and SIPDA

Pipe ID	Time of measurement (h)	Field data (m^3/h)	WaterGEMS (m^3/h)	EPANET (m^3/h)	EPANETPDD ((m^3/h)	SIPDA (m^3/h)
1	14.12	116.65	116.72	116.72	116.72	116.72
2	12.58	19.65	19.62	19.62	19.62	19.62
3	12.88	100.15	100.2	100.20	100.21	100.21
4	12.48	49.02	48.96	48.96	8.16	48.96
5	12.50	42.02	42.14	42.14	42.92	42.15
6	12.33	53.03	53.11	53.60	25.49	53.61
7	15.75	156.87	152.82	151.07	164.32	151.03
8	12.37	80.5	80.58	80.58	64.33	79.85
9	12.70	98.56	93.82	96.28	250.32	95.65
10	12.95	23.67	26.4	25.98	50.88	25.29
11	12.63	38.92	38.93	38.93	37.23	38.93
12	11.82	4.88	5.77	5.77	5.77	5.77
13	10.33	2.2	2.38	2.54	11.64	2.56
14	13.70	60.12	62.29	59.16	109.94	60.93
15	12.00	81.08	81	81.00	81.00	81.00
16	12.00	120.37	117.55	125.79	67.76	144.58
17	11.35	23.7	20.34	26.54	-12.98	30.55
18	12.78	89.49	87.22	-44.67	-200.82	6.95
19	14.47	98.5	99	99.00	99.00	99.01
20	14.23	6.57	6.3	6.35	-13.91	6.34
21	13.92	1.62	1.64	1.64	-4.25	1.64
22	14.65	275.92	275.07	284.66	250.88	284.69
23	14.83	14.03	14.1	14.10	14.10	14.10
24	13.00	187.59	187.52	187.52	187.52	187.53
25	13.97	11.74	1.98	-6.60	0.95	-2.13
26	11.90	62.41	61.74	61.70	60.45	60.35
27	12.77	247.9	241.09	241.38	231.96	239.54

28	14.17	55.86	59.92	55.38	-19.97	54.78
29	12.62	30.05	27.92	29.16	70.67	29.64
30	12.65	17.86	17.39	17.47	18.41	17.20
31	12.15	78.25	78.38	77.75	69.41	73.82
32	13.42	22.63	1.63	-1.77	11.62	1.23
33	12.25	13.65	13.7	13.76	62.81	13.76
34	12.25	45.95	45.22	46.01	60.11	45.89
35	12.63	76.5	76.65	76.60	27.54	76.61
36	12.28	78.58	78.62	78.84	71.44	78.78
37	11.83	55.56	55.58	56.65	60.57	56.41
38	13.03	5.85	5.85	5.85	5.85	5.85
39	13.87	195.95	195.9	195.90	144.04	194.63
40	12.07	17.7	11.53	11.25	2.27	11.24
41	14.58	19.35	15.44	14.63	22.17	16.63
42	12.33	167.1	159.31	155.69	145.59	156.53
43	13.33	31.65	31.65	31.65	31.65	31.65
44	10.83	51.85	49.82	51.21	55.15	50.38
45	13.53	158.75	158.73	159.48	131.18	159.40
46	13.75	34.7	34.26	33.57	106.07	33.56
47	13.72	13.43	13.21	13.19	14.37	13.03
48	11.67	33.64	32.97	25.95	30.26	25.88
49	11.00	84	85.29	76.88	76.69	76.94
50	13.03	129.66	126.77	126.68	126.00	125.87
51	13.13	78.96	76.83	76.27	102.48	76.27
52	11.50	10.37	10.17	8.60	8.12	8.57
53	12.72	121.3	121.43	121.43	0.00	121.43
54	12.17	21.5	19.57	20.50	56.90	20.81
55	13.23	569.8	569.93	569.93	551.52	569.20
56	14.10	6.4	0.67	-0.67	0.43	-0.67
57	10.83	54.34	53.51	53.61	69.76	48.01
58	10.07	34.72	33.76	33.76	-0.69	34.03
59	15.25	165.32	162.78	152.48	145.50	152.48
60	13.00	974	971.81	959.00	0.00	958.97
61	11.32	79.98	79.67	79.67	75.24	79.67

62	14.00	96.97	97.21	106.57	82.64	120.71
63	11.83	32.6	32.46	32.46	32.46	32.46
64	14.80	76.55	78.14	68.54	102.32	68.53
65	11.28	4.81	4.8	4.80	4.80	4.80
66	13.85	115.9	118.83	114.80	147.21	117.49
67	11.97	7.14	8.46	8.90	16.38	8.90
68	11.50	45	44.93	44.93	44.93	44.93
69	11.33	90.43	105.93	104.06	77.21	93.85
70	11.08	101.93	101.69	101.69	101.14	101.70
71	11.28	50.12	46.27	54.18	94.51	50.76
72	12.33	11.65	11.14	11.14	-25.29	11.14
73	14.87	104.32	119.77	118.20	107.61	118.51
74	13.48	52.18	53.25	49.21	119.15	50.82
75	13.20	34.99	35.04	35.04	34.99	35.05
76	14.55	50.78	50.74	15.33	50.48	15.33

Table A.1: Guidelines to promote good data collection and handling (Adapted from: Walski, 2000).

Activity	Details
Ensure data for model calibration is collected properly	• Use good pressure gauges • Understand the accuracy of the gauges and the accuracy used in the model
Maximise head loss	• Perform dynamic hydrant measurements to increase velocities for smaller pipes with low velocities • For large pipes correct data during peak flows or when tank levels are low so that many pumps are running
Know boundary conditions when pressure readings are taken	• Static hydrant test can be used to identify boundary conditions • Know exactly the exact water level in the tanks that supply water to the pressure gauge • Know the pumps that are running during the measurement period and measure the pump station pressure • Know PRV that have influence on the pressure gauge and know their status
Measure pressure far from boundary head	• Pressure should not be measured near the water tank, the pump or PRV discharge side • Pressure near the boundary elements is useful for checking boundary HGL not for actual micro model calibration
Water demand patterns should be known	• Know the expected demand at the time of field measurements • Take account of any unusual events, such as fire, that may have taken place and as a result distorted the flows
Good data are more important than abundant data	• Care should be taken to ensure that useless data is not used for calibration • Data can be collected under different operating conditions to complete calibration of the model. This helps to remove compensating errors • Relatively small good data is better that a large data set which contains bad data as well
Use HGL units to compare field and model results	• HGL rather than pressure units should be used as they give clear information which makes it easier to identify bad data

Table A.3: Valves with flow control and minor loss settings

Valve	Original settings		
	Setting	**Loss Coefficient**	**Fixed Status**
GV-1045	43.575	100	Open
GV-326	6.111	220	Open
GV-814	27.8	383	Open
GV-368	16.7	75	Open
FCV-511	0	50	Open
GV-369	14.5	420	Open
FCV-24	55.56	0	Open
GV-362	33.42	0	Open
GV-367	6.58	150	Open
GV-561	76.64	5	Open
GV-1009	3.26	1600	Open
FCV-428	0	600	Open
FCV-429	0	600	Open
GV-630	8.35	1650	Open
FCV-278	0	1000	Open
GV-1000	0	2000	Open
GV-624	5.375	800	Open
GV-1317	22.05	105	Open
GV-1114	14.73	0	Open
GV-362	33.42	0	Open
GV-367	6.58	150	Open
GV-1088	21.26	325	Open
FCV-510	0	100	Open
GV-369	14.5	420	Open
GV-1377	13.92	0	Open
GV-625	0	100	Open
GV-624	5.375	800	Open
FCV-74	0	0	Open
PSV-31	0	43	None
GV-220	4.96	5200	Open
GV-814	27.38	383	Open
GV-513	0	125	Open
FCV-146	33.73	0	None
PSV-40	4.00	0	None

Table A.4: Lusaka City and the Greater Lusaka's 2010 and projected 2035 population components (Adapted: Republic of Zambia, 2011b)

DMAs	**2010 population**					**2035 population**				
DISCRIPTION	**Total**	**HC**	**MC**	**LC**	**InfC**	**Total**	**HC**	**MC**	**LC**	**InfC**
Existing DMAs										
Avon	3612	0	3612	0	0	6744	0	6744	0	0
Bau	5873	0	0	0	5873	10966	0	0	5489	5477
Ch//M	14936	14936	0	0	0	27891	27891	0	0	0
Chai	1376	0	0	0	1376	2569	0	0	1287	1282
Chs	22262	0	0	0	22262	41570	0	0	20787	20783
ChaV	16275	0	16275	0	0	30393	0	30393	0	0
Chand	10293	0	10293	0	0	19221	0	19221	0	0
Chw/Kou	86760	0	0	43380	43380	162012	0	0	121509	40503
Chel	33560	0	33560	0	0	62669	0	62669	0	0
Chibo	5970	0	0	0	5970	11148	0	0	5568	5580
Chil	8766	0	8766	0	0	16370	0	16370	0	0
SChil	8274	0	8274	0	0	15451	0	15451	0	0
Chip	13050	0	0	3915	9135	24370	0	0	15837	8533
Chdh	5161	5161	0	0	0	9638	9638	0	0	0
Chu	2026	0	0	2026	0	3784	0	0	3784	0
Emm	11523	0	11523	0	0	21519	0	21519	0	0
E-BankH	10709	0	10709	0	0	19999	0	19999	0	0
Fview	1820	0	1820	0	0	3399	0	3399	0	0
F1080	7108	0	7108	0	0	13274	0	13274	0	0
F917	6442	0	6442	0	0	12029	0	12029	0	0
Gden	64207	0	0	12841	51366	119899	0	0	71934	47965
Geo	110718	0	0	0	110718	206753	0	0	103381	103372

Govt	3992	3992	0	0	0	7455	7455	0	0	0
H-Wt	2909	2909	0	0	0	5432	5432	0	0	0
HK	6740	0	6740	0	0	12587	0	12587	0	0
Ind	68995	68995	0	0	0	128839	128839	0	0	0
JComp	13379	0	0	0	13379	24977	0	0	12482	12495
JHa	16139	0	0	0	16139	30138	0	0	15075	15063
JLai	18211	0	0	0	18211	34005	0	0	17003	17003
Kbana	7149	0	0	7149	0	13349	0	0	13349	0
Kabu	10916	10916	0	0	0	20383	20383	0	0	0
KabwE	3923	0	3923	0	0	7326	0	7326	0	0
Kali	8516	0	0	8516	0	15904	0	0	15904	0
Kalu1	4444	4444	0	0	0	8298	8298	0	0	0
Kama	4367	0	0	0	4367	8155	0	0	4080	4075
Kany	21195	0	0	0	21195	39579	0	0	19797	19782
KSq1	2098	0	0	2098	0	3917	0	0	3917	0
KSq2	2098	0	0	2098	0	3917	0	0	3917	0
Liba	4663	0	4663	0	0	8708	0	8708	0	0
LibaS	29815	0	29815	0	0	55696	0	55696	0	0
Lila	4288	0	4288	0	0	8007	0	8007	0	0
Lubu	1964	1964	0	0	0	3668	3668	0	0	0
Marpdi	41649	0	0	0	41649	77773	0	0	38885	38888
Mtero	48470	0	0	48470	0	90512	0	0	90512	0
Mis	19583	0	0	0	19583	36569	0	0	18287	18282
Mtend	67311	0	0	33656	33656	125694	0	0	94271	31423
Mwa	3289	3289	0	0	0	6143	6143	0	0	0
IbH	2388	2388	0	0	0	4459	4459	0	0	0
NKabw	8140	0	8140	0	0	15200	0	15200	0	0

Kamw	1148	0	1148	0	0	2143	0	2143	0	0
NWldE	2514	2514	0	0	0	4694	4694	0	0	0
Nomd	8501	8501	0	0	0	15873	15873	0	0	0
NYa	4536	0	4536	0	0	8071	0	8071	0	0
OlyPk1	3321	3321	0	0	0	6200	6200	0	0	0
OlyPkE1	2367	2367	0	0	0	4421	4421	0	0	0
ProsH	4408	4408	0	0	0	8231	8231	0	0	0
RhPk	25170	25170	0	0	0	47001	47001	0	0	0
Rom1	13143	13143	0	0	0	24542	24542	0	0	0
Shksp	5249	0	5249	0	0	9802	0	9802	0	0
Skanz	2559	0	2559	0	0	4780	0	4780	0	0
StaH	11258	11258	0	0	0	21023	21023	0	0	0
ThPk	1535	0	1535	0	0	2866	0	2866	0	0
Twn/Kabel	41659	41659	0	0	0	77793	77793	0	0	0
Unza	10010	10010	0	0	0	18692	18692	0	0	0
VEliz	7942	7942	0	0	0	14830	14830	0	0	0
Wld	6655	6655	0	0	0	12429	12429	0	0	0
WldE	2120	2120	0	0	0	3959	3959	0	0	0
N'gom	13311	0	0	0	13311	24856	0	0	12429	12427
Kalik	17680	0	0	0	17680	33015	0	0	16507	16508
New DMAs										
DMA Rest Ward 3	18718	0	18718	0	0	34955	0	34955	0	0
DMA Rest Ward 4	47215	23608	23608	0	0	88166	44083	44083	0	0
DMA Rest Ward 7	4594	1378	3216	0	0	8577	0	8577	0	0
DMA Rest Ward 8	21915	0	21915	0	0	40922	0	40922	0	0
DMA Rest Ward 9	13119	0	13119	0	0	24497	0	24497	0	0

DMA Rest Ward 10	64990	0	16248	16248	32495	121360	0	0	91027	30333
DMA Rest Ward 11	3807	0	3807	0	0	7110	7110	0	0	0
DMA Rest Ward 12	29731	0	29731	0	0	55520	0	55520	0	0
DMA Rest Ward 15	7494	7494	0	0	0	13995	13995	0	0	0
DMA Rest Ward 16	49241	0	49241	0	0	91952	91952	0	0	0
DMA Rest Ward 17	25945	0	12973	12973	0	48450	48450	0	0	0
DMA Rest Ward 22	12070	0	3621	8449	0	22539	0	22539	0	0
DMA Rest Ward 23	75237	0	22571	52666	0	140496	0	140496	0	0
DMA Rest Ward 27	39129	0	0	11739	27390	73043	0	0	47505	25538
DMA Rest Ward 29	14482	0	7241	7241	0	27043	0	27043	0	0
DMA Rest Ward 32	2428	0	1214	1214	0	4534	0	4534	0	0
DMA Rest Ward 33	16616	0	4985	11631	0	31029	0	31029	0	0
LUSAKA	**1509169**	**290542**	**423184**	**286309**	**509134**	**2817767**	**687484**	**790449**	**864522**	**475313**
Satellite towns										
Kafue	42071	14022	14022	14022	0	261897	87290	87290	87290	0
Chongwe	26740	8912	8912	8912	0	202386	67455	67455	67455	0
Chibombo	17788	5929	5929	5929	0	106927	35639	35639	35639	0
SATELLITE TOWNS	**86599**	**28863**	**28863**	**28863**	**0**	**571210**	**190384**	**190384**	**190384**	**0**
GREATER LUSAKA	**1595768**	**319405**	**452048**	**315172**	**509134**	**3388977**	**877868**	**980833**	**1054906**	**475313**

Table A.5: Total population for each DMA and Satellite town (Adapted: Republic of Zambia, 2011b)

DMAs	YEARS AND TOTAL POPULATION					
DESCRIPTION	**2010**	**2015**	**2020**	**2025**	**2030**	**2035**
Existing DMAs						
Avon	3612	4165	4596	5223	5935	6744
Bau	5873	6772	7473	8492	9650	10966
Ch//M	14936	17223	19006	21598	24544	27891
Chai	1376	1587	1751	1990	2261	2569
Chs	22262	25670	28328	32191	36581	41570
ChaV	16275	18767	20710	23535	26745	30393
Chand	10293	11869	13098	14884	16914	19221
Chw/Kou	86760	100043	110401	125458	142568	162012
Chel	33560	38698	42705	48529	55148	62669
Chibo	5970	6884	7597	8633	9810	11148
Chil	8766	10108	11155	12676	14405	16370
SChil	8274	9541	10529	11965	13597	15451
Chip	13050	15048	16606	18871	21445	24370
Chdh	5161	5951	6567	7463	8481	9638
Chu	2026	2336	2578	2930	3330	3784
Emm	11523	13287	14663	16663	18936	21519
E-BankH	10709	12349	13628	15487	17599	19999
Fview	1820	2099	2316	2632	2991	3399
F1080	7108	8196	9045	10279	11681	13274
F917	6442	7428	8197	9315	10585	12029
Gden	64207	74037	81703	92846	105509	119899
Geo	110718	127669	140888	160103	181939	206753
Govt	3992	4603	5080	5773	6560	7455
H-Wt	2909	3354	3701	4206	4780	5432
HK	6740	7772	8577	9747	11076	12587
Ind	68995	79558	87795	99769	113376	128839
JComp	13379	15423	17020	19341	21979	24977
JHa	16139	18610	20537	23338	26521	30138
JLai	18211	20999	23173	26333	29924	34005
Kbana	7149	8243	9096	10337	11747	13349
Kabu	10916	12587	13890	15784	17937	20383
KabwE	3923	4524	4992	5673	6447	7326
Kali	8516	9820	10837	12315	13995	15904
Kalu1	4444	5124	5655	6426	7302	8298
Kama	4367	5036	5557	6315	7176	8155
Kany	21195	24440	26971	30649	34829	39579
KSq1	2098	2419	2669	3033	3447	3917
KSq2	2098	2419	2669	3033	3447	3917
Liba	4663	5377	5934	6743	7663	8708
LibaS	29815	34480	37940	43114	48994	55696
Lila	4288	4944	5456	6200	7046	8007
Lubu	1964	2265	2500	2841	3228	3668
Marpdi	41649	48025	52997	60225	68439	77773
Mtero	48470	55981	61678	70090	79649	90512

Mis	19583	22581	24919	28318	32180	36569
Mtend	67311	77616	85652	97334	110609	125694
Mwa	3289	3793	4186	4757	5406	6143
IbH	2388	2754	3039	3453	3924	4459
NKabw	8140	9386	10358	11771	13376	15200
Kamw	1148	1324	1461	1660	1886	2143
NWldE	2514	2899	3199	3635	4131	4694
Nomd	8501	9802	10817	12292	13968	15873
NYa	4536	5230	5772	6559	7454	8071
OlyPk1	3321	3829	4225	4801	5456	6200
OlyPkE1	2367	2729	3012	3423	3890	4421
ProsH	4408	5083	5609	6374	7243	8231
RhPk	25170	29023	32028	36396	41360	47001
Rom1	13143	15155	16724	19055	21597	24542
Shksp	5249	6053	6680	7591	8626	9802
Skanz	2559	2951	3257	3701	4206	4780
StaH	11258	12982	14326	16280	18500	21023
ThPk	1535	1770	1953	2219	2522	2866
Twn/Kabel	41659	48037	53011	60241	68457	77793
Unza	10010	11543	12738	14475	16449	18692
VEliz	7942	9158	10106	11484	13050	14830
Wld	6655	7674	8469	9624	10937	12429
WldE	2120	2445	2698	3066	3484	3959
N'gom	13311	15349	16938	19248	21873	24856
Kalik	17680	20387	22498	25566	29053	33015
New DMAs						
DMA Rest Ward 3	18718	21584	23819	27068	30760	34955
DMA Rest Ward 4	47215	54443	60080	68274	77585	88166
DMA Rest Ward 7	4594	5297	5845	6642	7548	8577
DMA Rest Ward 8	21915	25270	27886	31689	36011	40922
DMA Rest Ward 9	13119	15127	16693	18970	21557	24497
DMA Rest Ward 10	64990	74940	82699	93978	106795	121360
DMA Rest Ward 11	3807	4390	4845	5506	6257	7110
DMA Rest Ward 12	29731	34283	37833	42993	48857	55520
DMA Rest Ward 15	7494	8641	9536	10837	12315	13995
DMA Rest Ward 16	49241	56780	62659	71205	80916	91952
DMA Rest Ward 17	25945	29917	33015	37518	42635	48450
DMA Rest Ward 22	12070	13918	15359	17454	19834	22539
DMA Rest Ward 23	75237	86756	95739	108796	123634	140496
DMA Rest Ward 27	39129	45120	49792	56583	64300	73043
DMA Rest Ward 29	14482	16699	18428	20941	23797	27043
DMA Rest Ward 32	2428	2800	3090	3511	3990	4534
DMA Rest Ward 33	16616	19160	21144	24028	27305	31029
LUSAKA CITY	**1509169**	**1740408**	**1920401**	**2182364**	**2479949**	**2817767**
Satellite Towns						
Kafue	42071	50000	100000	137840	190000	261897
Chongwe	26740	50000	100000	126491	160000	202386

Chibombo	17788	30000	30000	45826	70000	106927
SATELLITE TOWNS	**86599**	**130000**	**230000**	**310157**	**420000**	**571210**
GREATER LUSAKA	**1595768**	**1870408**	**2150401**	**2492521**	**2899949**	**3388977**

Table A.6: Comparison of old and changed pipe diameters for the holistic and business as usual scenarios

		Original network	Holistic scenario	Business as usual scenario
Pipe ID	**Length (m)**	**Origin Dia. (mm)**	**Dia. (mm)**	**Dia. (mm)**
P-903	212.97	150	280	225
P-1083	188.93	150	280	350
P-1182	103.65	150	350	280
P-879	58.29	150	225	300
P-949	8.93	200	350	300
P-1124	124.17	150	280	300
P-738	48.74	150	300	280
P-963	115.90	200	350	225
P-1217	83.36	150	225	280
P-782	59.86	150	300	300
P-1170	76.02	150	300	280
P-812	44.09	200	350	300
P-1456	144.75	150	300	350
P-816	23.22	200	300	300
P-752	68.26	200	300	280
P-827	44.79	200	280	225
P-934	12.44	150	280	225
P-1190	5.50	200	280	280
P-989	240.48	200	280	225
P-751	112.88	150	225	300
P-863	59.08	150	225	280
P-820	24.31	200	280	300
P-1210	50.79	200	280	300
P-917	106.07	150	300	300
P-865	35.94	150	280	225
P-1134	51.61	150	280	280
P-1486	84.38	150	280	225
P-1533	101.04	150	280	280
P-1522	12.60	150	225	280
P-1329	46.55	150	280	300
P-1144	71.93	150	280	300
P-1081	30.13	150	280	280
P-1285	88.68	150	280	280
P-740	224.31	200	225	280
P-890	232.60	150	300	280
P-908	231.63	150	280	225
P-734	42.74	150	280	280
P-843	106.88	200	225	350

P-1506	98.56	150	300	300
P-825	55.44	200	350	280
P-1322	46.21	150	280	225
P-972	157.02	375	400	400
P-1135	188.49	150	300	280
P-851	61.46	150	280	300
P-1262	8.40	150	300	300
P-888	110.00	150	280	300
P-883	292.33	150	350	225
P-1306	179.66	150	280	225
P-1419	17.12	150	280	350
P-1691	171.00	375	400	400
P-1014	132.78	200	350	225
P-1497	62.85	150	280	300
P-822	24.31	200	225	225
P-918	63.69	150	280	300
P-817	32.01	200	350	280
P-771	65.09	200	300	350
P-1140	28.81	150	300	280
P-1292	122.23	200	300	225
P-1344	76.34	375	400	400
P-936	114.91	150	280	280
P-1247	68.63	200	300	300
P-873	52.98	150	280	300
P-1346	17.53	375	500	450
P-826	23.59	200	280	280
P-895	159.15	150	280	350
P-805	28.40	200	350	225
P-867	60.06	150	350	350
P-1312	173.84	200	300	280
P-1178	11.31	150	280	300
P-874	60.14	150	280	280
P-864	24.37	150	300	350
P-1076	148.52	150	300	225
P-1283	145.43	200	300	300
P-923	220.04	150	280	350
P-913	70.83	150	300	225
P-1107	57.64	150	300	280
P-838	62.93	200	280	350
P-790	45.87	200	300	280
P-1272	8.85	150	350	280
P-833	34.42	200	350	350
P-821	36.15	200	300	280
P-763	78.52	200	280	280
P-1077	109.01	200	280	225
P-766	73.14	150	280	300
P-872	19.25	150	280	280
P-753	73.50	200	225	300
P-1156	224.17	200	280	300
P-1115	211.09	375	450	500

P-905	84.69	150	300	280
P-1010	154.08	150	280	280
P-1286	43.45	150	300	280
P-1264	57.69	150	300	280
P-974	481.79	375	450	400
P-1278	62.47	150	280	225
P-897	73.21	150	300	225
P-1159	65.64	200	280	280
P-1228	85.88	150	350	350
P-1467	28.10	150	300	300
P-728	37.87	150	350	225
P-1263	61.06	150	280	300
P-889	222.36	150	225	300
P-811	28.56	200	225	350
P-876	7.78	150	300	225
P-1207	119.61	150	300	280
P-796	68.41	200	225	300
P-742	11.28	200	350	300
P-1383	21.97	150	300	225
P-1062	109.19	200	300	300
P-868	50.71	200	300	300
P-784	48.93	200	280	225
P-768	58.20	150	280	300
P-885	31.90	150	225	280
P-985	251.59	200	280	225
P-1265	64.13	200	350	225
P-810	13.04	200	300	280
P-815	39.57	200	300	280
P-769	47.56	200	280	350
P-1152	64.37	150	280	280
P-1457	239.33	150	225	225
P-882	38.79	150	300	225
P-1759	500.08	600	600	600
P-828	38.37	200	280	300
P-839	80.98	200	225	280
P-1046	116.95	150	300	280
P-791	58.23	150	225	280
P-809	44.68	200	225	225
P-1043	7.09	200	280	280
P-1211	42.15	150	350	225
P-808	15.83	200	280	350
P-1270	4.55	150	280	300
P-881	55.67	150	300	280
P-1189	77.28	150	280	225
P-832	22.60	200	300	300
P-858	63.34	150	225	300
P-804	46.37	200	300	280
P-861	59.08	150	225	225
P-1169	28.79	150	300	225
P-730	63.25	150	280	300

P-774	280.58	150	300	280
P-928	220.96	150	225	280
P-1078	43.31	150	350	225
P-837	20.77	200	350	225
P-961	119.56	200	225	280
P-1136	5.79	150	225	300
P-1545	181.35	150	280	300
P-755	152.82	150	280	280
P-1488	101.17	150	225	225
P-1479	71.65	150	280	280
P-760	66.99	150	280	300
P-880	60.42	150	280	350
P-1314	82.75	150	300	350
P-875	52.39	150	300	300
P-1473	73.73	150	280	225
P-1023	49.08	150	350	350
P-855	50.95	150	300	280
P-1110	36.72	150	280	280
P-970	285.57	375	450	450
P-802	49.23	200	280	280
P-741	71.60	200	300	225
P-807	45.76	200	350	300
P-856	60.28	150	225	280
P-870	56.35	150	300	225
P-836	32.98	200	350	225
P-1112	97.20	150	280	300
P-1318	87.51	150	300	300
P-871	44.79	150	225	350
P-1196	15.62	200	350	280
P-922	73.02	150	225	225
P-1405	91.88	150	350	300
P-1019	55.01	200	350	280
P-757	45.13	200	300	300
P-1315	200.93	150	280	300
P-857	52.14	150	300	225
P-893	145.71	150	280	225
P-834	29.52	200	300	350
P-1018	64.04	200	350	280
P-1313	87.64	150	300	280
P-1519	5.90	150	280	280
P-1227	55.81	200	280	280
P-1114	159.40	150	350	280
P-1118	106.23	150	350	300
P-1402	322.19	150	300	280
P-1760	33.93	600	700	900
P-1413	128.85	150	280	300
P-1104	652.95	375	450	450
P-799	37.50	200	300	300
P-884	66.81	150	225	300
P-1138	124.54	150	225	280

P-835	39.06	200	280	225
P-814	20.36	200	280	225
P-1171	67.23	200	225	350
P-803	18.85	200	350	300
P-1025	107.91	200	280	300
P-806	11.42	200	225	300
P-901	68.45	150	225	300
P-877	61.02	150	300	300
P-733	311.17	200	280	280
P-1453	128.27	150	350	280
P-1254	87.01	150	280	280
P-1332	655.59	375	400	450
P-845	125.56	200	300	350
P-1460	28.25	150	300	300
P-862	55.91	150	280	300
P-1185	63.96	200	300	280
P-818	36.73	200	280	300
P-1181	25.37	150	300	300
P-1039	239.85	200	300	350
P-1517	68.49	150	280	300
P-1277	276.78	375	400	500
P-1151	107.41	150	280	300
P-869	14.34	600	600	700
P-1130	26.69	200	300	225
P-2730	7.16	200	300	300
P-3995	201.57	375	450	400
P-3998	9.32	375	500	500
P-4035	29.66	150	280	280
P-4036	600.31	150	225	280
P-4070	62.76	75	225	280
P-4063	196.61	160	280	225
P-100	100.62	150	280	225
P-99	59.91	150	225	300
P-97	133.38	160	300	225
P-63	139.50	200	280	280
P-31	166.25	250	225	225
P-1	61.72	150	280	280
P-2	68.07	100	280	300
P-3	109.31	100	300	300
P-4	2.00	500	450	400
P-32	73.94	250	225	280
P-4105	60.67	200	300	350
P-4171	0.01	100	350	280
P-4172	0.01	100	225	350
P-4173	0.01	100	350	225
P-4174	0.01	100	280	280
P-4175	121.78	100	300	225
P-4509	3.00	200	350	280
P-720	49.98	200	300	300
P-4553	60.28	160	280	280

P-951	389.04	150	280	300
P-6128	199.52	200	350	300
P-6130	185.85	200	280	300
P-6131	145.05	200	300	280
P-6132	282.72	200	225	280
P-6220	24.80	150	225	350
P-6222	25.41	100	300	300
P-6224	80.65	150	280	225
P-6225	154.01	200	280	300
P-6226	374.44	200	280	350
P-6175	59.00	150	300	225
P-6178	71.49	150	300	280
P-6287	47.66	100	300	350
P-6290	2.50	225	225	350
P-6291	2.33	225	300	350
P-6218	134.29	400	450	400
P-6294	2.14	250	225	300
P-6296	2.61	250	280	225
P-6295	2.07	250	300	280
P-6299	2.14	225	280	300
P-6300	126.44	375	400	450
P-6301	2.10	200	300	280
P-6302	11.63	200	225	280
P-6303	197.18	200	225	300
P-6293	1074.88	250	225	225
P-6306	153.42	150	280	300
P-6307	404.75	150	280	300
P-6308	326.69	225	280	225
P-6309	713.13	225	280	225
P-6310	2.39	375	450	500
P-6311	2.29	375	450	450
P-6312	19.13	375	450	450
P-6316	7.57	150	225	280
P-6317	9.96	350	400	450
P-6320	94.65	350	400	500
P-6321	4.94	350	450	450
P-6322	43.75	350	500	450
P-6325	32.00	200	280	300
P-6326	11.90	200	300	300
P-6327	3965.44	250	280	280
P-6328	4.74	250	350	280
P-6330	2.20	250	350	280
P-6333	29.40	375	450	500
P-6304	453.21	200	225	300
P-6349	2.09	150	225	280
P-6351	2.00	150	350	300
P-6350	2.05	150	300	300
P-6352	2.03	150	300	300
P-6353	2.21	150	350	280
P-6354	248.96	150	280	225

P-6364	11.64	200	280	300
P-6365	21.51	125	280	225
P-6367	20.90	125	225	225
P-6369	4.21	200	280	280
P-6370	3.57	200	225	280
P-6374	4.97	200	225	300
P-6375	1.95	200	300	280
P-6376	667.89	200	280	350
P-6377	401.95	200	280	300
P-6378	71.75	200	280	300
P-6387	277.21	150	225	280
P-6388	169.74	150	280	280
P-6389	144.95	150	300	225
P-6390	138.96	150	280	350
P-6396	17.32	150	300	280
P-6397	124.00	150	300	300
P-6427	163.66	200	280	225
P-6428	101.25	200	300	300
P-6432	111.84	200	225	280
P-6433	61.80	200	300	225
P-6434	74.04	200	280	280
P-6435	62.62	200	280	300
P-6436	59.60	200	280	280
P-6437	82.58	200	225	350
P-6438	168.54	200	300	300
P-6439	55.42	200	300	225
P-6440	6.86	200	300	300
P-6441	53.07	200	280	300
P-6442	64.87	200	300	225
P-6443	8.92	150	300	280
P-6323	14.41	350	400	450
P-6449	82.49	150	225	300
P-6450	114.06	150	280	225
P-6459	8.74	100	280	225
P-6461	34.78	200	300	300
P-6467	0.01	200	350	225
P-6468	0.01	200	280	350
P-6470	9.37	200	300	350
P-6471	24.34	200	300	300
P-6472	194.93	160	280	350
P-6473	127.09	150	300	280
P-6474	92.19	150	280	280
P-6475	64.97	160	280	280
P-6476	36.94	200	300	280
P-6477	0.01	200	300	350
P-6502	0.01	300	450	500
P-6504	0.01	450	500	450
P-6514	0.01	200	225	300
P-6540	0.01	350	400	400
P-6541	12.00	150	300	225

P-6542	0.01	150	300	225
P-6289	1877.86	200	225	225
P-6292	0.01	200	300	280
P-6318	8.64	250	225	280
P-6324	0.01	250	280	280
P-6329	490.83	160	280	280
P-6332	465.45	160	350	225
P-8	8.14	250	300	300
P-9	64.52	150	280	225
P-10	18.31	150	300	300
P-11	160.26	375	400	450
P-61	139.28	150	280	300
P-62	1.28	150	280	280
P-256	4.36	150	300	300
P-368	2.67	350	400	450
P-370	22.70	350	500	450
P-388	128.40	375	450	450
P-389	150.34	375	450	500
P-399	210.70	375	450	500
P-2793	87.42	225	225	300
P-2794	10.09	225	350	225
P-1367	927.78	225	225	280
P-2797	13.51	300	500	450
P-2798	5.70	300	450	450
P-2805	1243.44	600	600	600
P-2806	13.88	600	700	700
P-2837	191.40	200	225	280
P-2838	3.57	200	280	300
P-2855	214.16	150	280	280
P-2856	12.14	150	280	280
P-2858	15.15	150	280	280
P-906	247.95	150	300	300
P-2861	12.00	150	300	280
P-2865	10.55	150	350	350
P-927	10.60	150	280	280
P-2870	14.43	200	300	280
P-2871	110.25	200	350	225
P-2872	89.82	200	350	225
P-2873	8.57	200	280	280
P-2878	122.89	150	300	225
P-2879	6.27	150	225	280
P-2880	3.10	150	300	350
P-2890	4.48	160	350	350
P-2897	7.12	160	280	300
P-2901	294.36	160	280	350
P-2902	8.23	160	280	350
P-2904	7.55	160	350	350
P-2905	256.24	160	225	280
P-2970	5.07	200	280	300
P-2971	39.88	200	280	280

P-1122	65.21	150	300	300
P-2989	5.19	200	300	225
P-1155	7.59	160	280	300
P-1157	246.33	160	300	225
P-1160	4.25	200	300	350
P-1161	2.22	200	225	280
P-1163	6.56	200	280	280
P-1164	12.56	200	225	300
P-1167	310.29	200	280	280
P-1168	72.24	200	300	300
P-973	100.88	200	280	300
P-969	114.54	200	300	225
P-1186	3.56	150	350	350
P-1188	7.39	150	225	225
P-1197	13.73	200	225	300
P-1212	7.32	200	280	280
P-1214	185.91	200	280	280
P-4240	3.56	150	300	350
P-1158	52.98	160	280	280
P-4256	128.74	200	300	225
P-4266	140.14	375	400	450
P-4271	421.97	200	300	300
P-4273	4.49	600	600	700
P-4279	427.76	250	280	225
P-4466	2.00	500	450	450
P-4467	2.00	500	500	450
P-4472	6.24	350	500	400
P-4471	6.19	350	350	300
P-4506	3.00	200	280	225
P-4507	3.00	200	300	225
P-4508	3.00	200	225	300
P-1213	82.49	100	280	300
P-1225	119.66	75	280	280
P-1226	59.66	75	280	280
P-1230	118.94	75	225	225
P-1296	115.74	100	280	350
P-1299	53.16	75	225	280
P-1301	14.31	75	300	225
P-1302	72.94	75	300	300
P-1310	86.01	75	350	300
P-1317	12.35	75	280	280
P-1320	59.83	75	300	350
P-1323	49.80	100	280	225
P-1328	52.14	100	300	280
P-1337	128.04	100	225	225
P-1343	49.86	100	300	350
P-1353	93.94	100	300	280
P-1356	132.60	100	280	300
P-1369	408.97	100	280	280
P-1375	48.99	100	350	280

P-1379	17.96	100	280	280
P-1385	61.51	100	280	300
P-1387	71.62	100	300	225
P-1398	82.85	100	300	300
P-1401	91.41	100	280	300
P-37	6.47	200	225	280
P-353	212.18	200	300	280
P-538	1.00	350	500	450
P-546	2.00	350	450	450
P-610	1.00	350	500	400
P-703	2.00	350	500	400
P-705	1.00	350	450	450
P-719	2.00	350	500	450
P-722	117.08	150	225	280
P-723	0.50	200	350	280
P-724	0.50	200	300	350
P-1490	31.07	150	280	350
P-1968	26.73	350	450	450
P-1990	382.12	150	300	280
P-1991	2.62	110	350	225
P-1992	19.85	110	225	225
P-1995	189.93	160	280	300
P-1996	132.98	160	280	300
P-1997	13.83	160	280	280
P-2045	183.55	110	280	280
P-2067	12.47	110	280	280
P-2410	12.12	150	280	280
P-2412	11.49	150	300	280
P-2414	30.18	150	225	225
P-2420	28.65	150	350	225
P-2421	2.16	150	300	300
P-2454	7.51	200	350	300
P-2488	100.56	150	300	280
P-2490	131.48	150	280	350
P-2528	210.58	160	350	225
P-2527	172.12	160	225	280
P-2529	54.01	160	225	225
P-2530	716.81	200	280	300
P-2531	2097.95	400	450	400
P-2533	7.34	150	300	280
P-4758	74.67	100	280	225
P-4789	63.11	100	350	225
P-4790	3.76	100	300	300
P-4892	47.93	300	400	500
P-4893	3.10	300	450	450
P-2957	81.59	200	280	225
P-953	46.27	200	225	300
3	15.93	150	225	300
4	226.61	200	280	300
5	24.46	150	350	350

6	383.59	150	300	225
7	106.44	150	300	350
P-2935	365.81	150	280	350
P-2941	405.82	150	280	350
P-2943	6.29	150	350	350
P-2945	170.73	150	225	300
P-2948	159.26	150	280	280
P-2932	901.21	150	300	225
P-1116	826.71	150	300	300
P-2927	369.07	150	300	280
P-2197	115.82	150	280	225
P-2145	223.21	100	300	280
P-1988	64.21	100	225	300
P-1986	39.44	100	350	300
P-1985	236.48	75	280	300
P-1982	100.43	75	280	300
P-3779	12.57	150	300	225
P-2079	467.79	100	280	300
P-1172	1314.98	200	280	225
P-2382	139.11	150	280	300
P-1979	163.60	150	300	280
P-1977	77.81	150	225	300
P-2401	95.18	150	300	350
P-3909	97.53	150	300	300
P-2398	68.18	150	300	280
P-3910	127.45	150	225	300
P-3919	69.34	150	300	350
P-2907	88.15	150	280	280
P-1993	828.90	375	450	450
P-1416	149.25	150	225	280
P-2913	78.00	150	300	350
P-2917	6.78	150	350	300
P-2919	127.78	150	300	225
P-4787	70.55	100	225	300
P-4792	143.80	100	280	225
P-4794	385.99	100	300	280
P-4783	173.65	100	350	300
P-4782	77.32	100	225	300
P-4780	11.70	100	225	280
P-2900	162.41	150	225	225
P-2892	16.63	150	280	280
P-2894	78.38	150	280	350
P-2896	153.20	150	300	280
P-1399	485.21	150	225	350
P-1143	773.30	150	280	300
P-2884	111.32	150	300	280
P-2889	142.27	150	280	280
P-2887	1112.24	150	280	300
P-2882	54.75	200	280	350
P-6313	44.27	400	400	400

P-6503	7.83	450	450	400
P-4253	620.99	150	225	280
P-398	217.08	375	450	500
P-2875	1034.47	375	450	400
P-1191	25.10	150	225	300
P-2960	139.11	200	280	280
P-2961	42.35	150	300	300
P-2965	38.77	150	280	300
P-2967	58.79	200	280	280
P-2969	14.57	200	225	280
P-2974	16.93	200	280	300
P-2972	193.93	200	225	300
P-2977	13.04	150	225	300
8	81.56	150	280	280
P-2980	72.29	150	225	350
P-1193	229.58	200	300	280
P-1203	264.42	150	350	300
P-1215	202.13	200	225	300
P-1221	261.47	200	300	225
P-2791	395.84	150	350	300
P-2789	2.43	150	350	280
P-2787	363.81	150	350	350
P-2784	125.65	150	280	350
P-66	51.70	150	300	280
P-2782	37.23	150	350	280
P-2780	28.97	150	350	225
P-2807	248.63	375	400	400
P-2804	80.30	225	300	300
P-2802	131.93	225	300	300
P-2800	477.34	225	225	280
P-6122	942.06	200	280	280
P-385	1480.72	375	450	400
P-2983	65.63	150	225	300
P-6429	27.25	150	225	280
P-6430	147.61	150	280	280
P-6431	66.04	200	300	300
P-2863	319.85	200	225	300
P-966	202.96	150	280	280
9	166.44	375	450	400
P-2728	289.34	375	400	450
P-2727	10.14	150	280	300
P-6451	312.31	150	300	350
P-2822	83.58	200	225	300
P-4775	102.81	200	225	225
P-4778	75.00	200	300	300
P-2827	130.53	150	225	280
P-2821	138.22	150	280	280
P-2817	69.75	200	280	300
P-4774	33.80	200	300	280
P-4772	111.66	200	280	280

P-756	188.08	150	280	225
P-2999	115.49	150	300	280
P-725	110.22	150	225	280
P-714	43.37	200	300	300
P-2992	49.67	200	350	280
P-2990	135.44	150	300	350
P-6822	512.50	600	600	600
P-2859	53.87	150	350	225
P-2864	56.17	150	280	350
P-2867	154.02	150	280	280
P-4759	78.12	100	350	280
P-6480	132.16	100	225	280
P-2853	149.61	150	300	280
P-4760	60.23	75	225	350
P-4763	67.42	75	225	280
P-2847	11.34	200	300	280
P-2850	171.36	150	225	225
P-2851	51.71	150	280	300
P-4766	112.12	100	350	350
P-2846	28.65	200	280	300
P-2843	247.82	150	280	300
P-4255	124.08	200	280	280
P-2835	59.42	150	280	350
P-4765	207.34	100	225	225
P-4769	139.02	100	300	280
P-4038	206.71	150	225	280
P-2841	28.32	150	350	280
P-2829	310.87	150	300	280
P-2832	203.98	150	280	280
P-2833	79.30	150	280	225
P-4239	3.65	150	300	225
A_cnct_Resev	0.10	600	900	700
P-1758	653.56	600	600	700
P_4739-BB	1.00	600	900	700
P-K1	25.84	200	200	200
P-1758_1	1.00	600	600	700

Table A.7: Comparison of old and changed valve diameters for the holistic and business as usual scenarios

Valve ID	Valve Type	Original network Dia. (mm)	Holistic scenario Dia. (mm)	Business as usual scenario Dia. (mm)
GV-232	FCV	200	280	280
GV-82	FCV	200	280	280
GV-1359	FCV	150	225	300
GV-1360	FCV	150	350	300
GV-574	FCV	150	280	300
GV-499	FCV	200	300	280
GV-221	FCV	150	300	280
GV-584	FCV	200	225	280
GV-1388	FCV	150	225	300
GV-1392	FCV	150	350	300
GV-234	FCV	200	280	300
GV-416	FCV	150	300	300
GV-421	FCV	200	280	280
GV-481	FCV	150	300	280
GV-413	FCV	150	300	280
GV-429	FCV	200	225	300
GV-1389	FCV	150	350	300
GV-428	FCV	200	280	280
GV-1160	FCV	200	350	300
GV-1350	FCV	300	450	450
GV-559	FCV	200	280	280
GV-731	FCV	600	700	700
GV-1088	FCV	200	300	225
GV-223	FCV	150	300	350
GV-561	FCV	200	225	300
GV-52	FCV	150	280	225
GV-53	FCV	350	450	500
GV-54	FCV	350	500	450
FCV-7	FCV	250	300	280
FCV-8	FCV	250	280	300

FCV-9	FCV	250	350	300
FCV-10	FCV	200	280	300
FCV-11	FCV	200	225	300
FCV-12	FCV	250	280	300
FCV-13	FCV	375	500	450
FCV-14	FCV	150	280	280
FCV-15	FCV	200	300	280
FCV-17	FCV	150	280	280
FCV-18	FCV	150	225	280
FCV-19	FCV	150	300	280
FCV-20	FCV	200	300	300
FCV-26	FCV	200	280	280
FCV-81	FCV	150	280	300
FCV-74	FCV	350	500	450
FCV-99	FCV	200	280	300
FCV-100	FCV	350	450	500
FCV-101	FCV	350	450	450
FCV-102	FCV	350	450	450
FCV-103	FCV	200	350	300
FCV-134	FCV	100	300	280
FCV-136	FCV	150	300	300
FCV-137	FCV	150	280	300
FCV-138	FCV	100	300	225
FCV-145	FCV	150	280	300
FCV-146	FCV	150	300	280
FCV-531	FCV	300	500	450
PSV-1	PSV	200	280	350
PSV-18	PSV	200	280	280
PSV-31	PSV	350	450	450
PSV-32	PSV	150	280	300
PSV-37	PSV	200	350	300
PSV-39	PSV	250	280	225
PSV-40	PSV	450	500	450
PSV-41	PSV	300	450	450
GV-2	FCV	150	350	300
AFCV	FCV	600	600	700

GV-1202	FCV	375	300	280
GV-362	FCV	200	500	450
ATCV	TCV	600	900	700

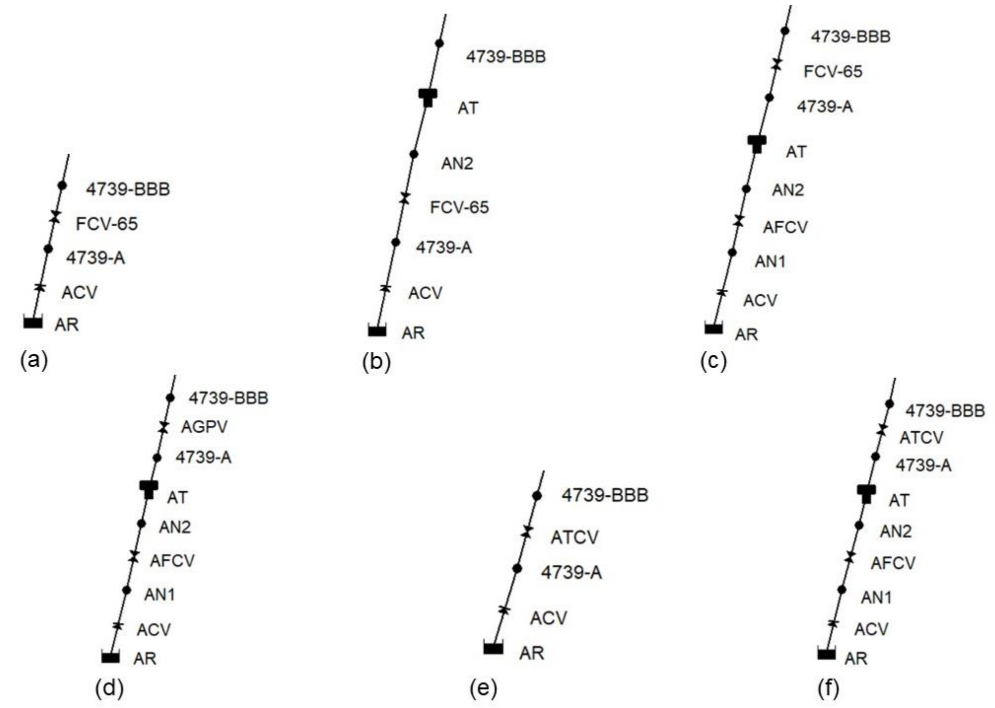


Figure A.1: Six trials for modelling the water offtake point

The first arrangement (AR, ACV, 4739-A, FCV-65, 4739-BBB)

This is represented by Figure A.1a. The FCV-65 is used to regulate the flow downstream by limiting the maximum flow values through it. At each time step t, the maximum flow value for the setting of the valve is determined as the product of the average flow (Q_{ave}) and the flow pattern multiplication factor (f_{mf}). The Q_{ave} through the FCV-65 and the f_{mf} are determined using Equations 5.6 and 5.7 respectively.

As shown in Figure A.2, the 24 hours variation of discharge for the AFCV and that for pipe P-1758 before and after isolation are almost identical. The absolute error between the total flow to Chelstone zone before and after isolation is 0.19%. The correlation coefficient (R) for the discharge for pipe P-1758 before and after the zone isolation is about 1 (0.9997) as shown in Figure A.3. This shows that the first combination of artificial elements (Figure A.1a) is very accurate in simulating the offtake point flows. However, for pressure the arrangement performs badly (Figure A.4). As compared to the pressure distribution map for the whole LWSN (Figure A.5), the pressure distribution map for the isolated Chelstone zone (Figure A.6) is significantly different especially in the western side. Therefore, the arrangement cannot be used to model the water offtake point.

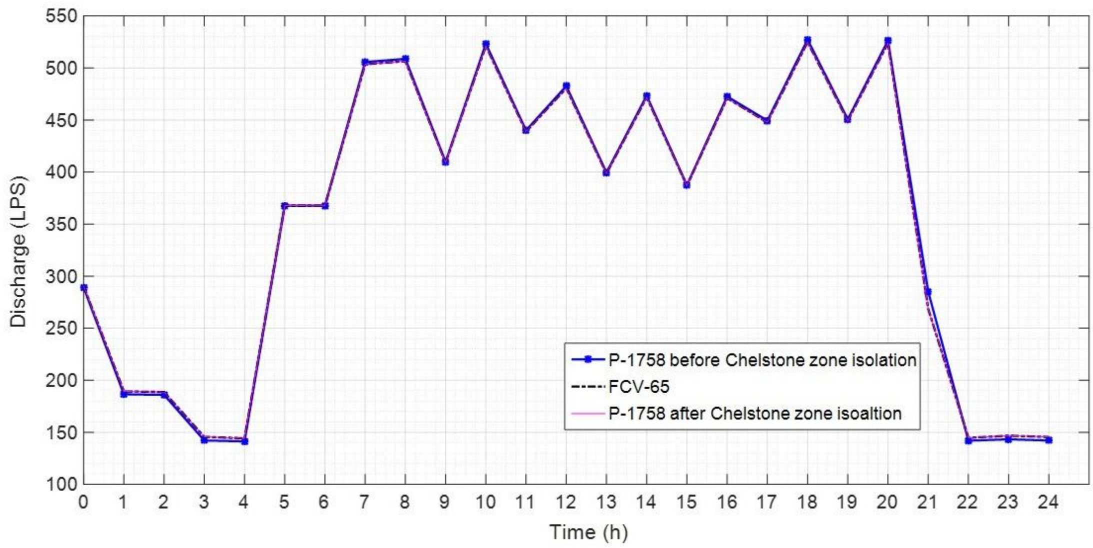


Figure A.2: First arrangement 24 hours discharge variations

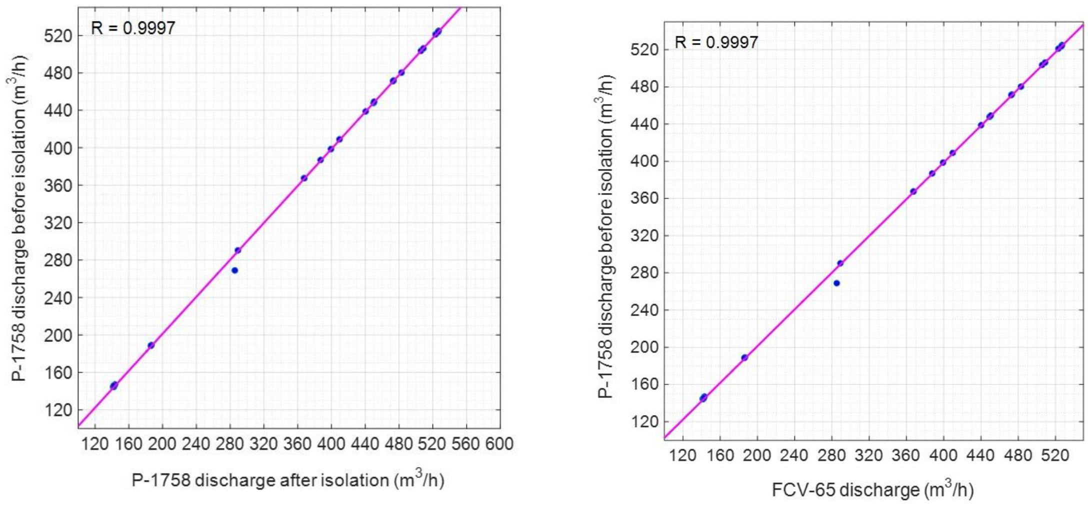


Figure A.3: First arrangement correlation coefficient for P-1758 discharge (a) before and after zone isolation (b) before and FCV-65 after zone isolation

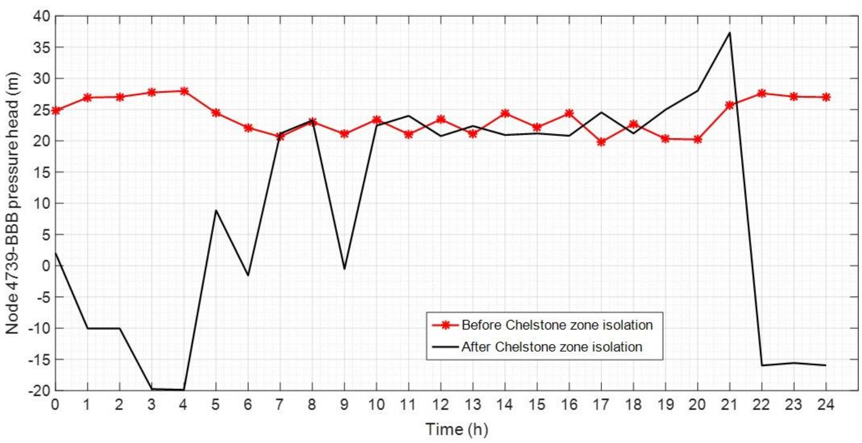


Figure A.4: First arrangement 24 hours pressure variations

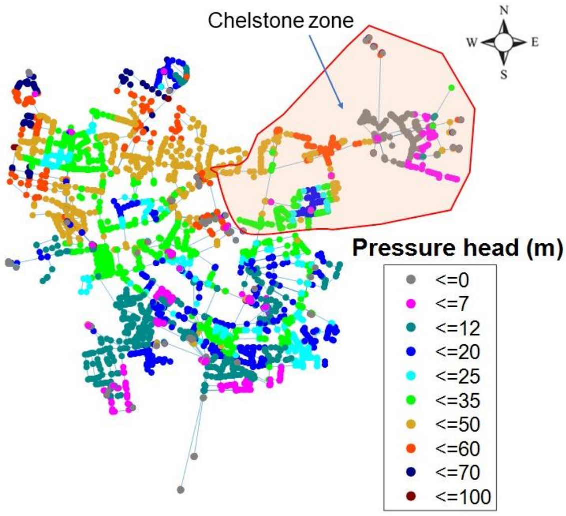


Figure A.5: LWSN pressure distribution map at 05:00 am

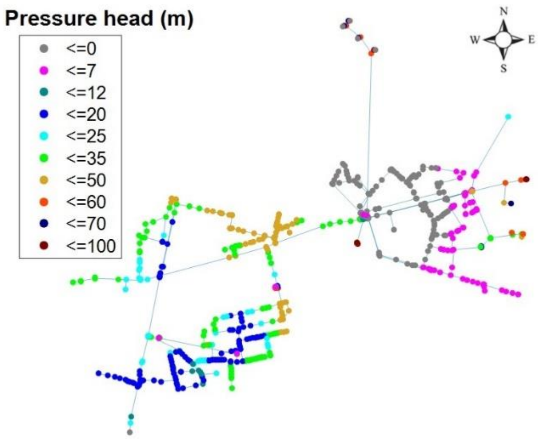


Figure A.6: First arrangement Chelstone zone pressure distribution map at 05:00 am

The second arrangement (AR, ACV, 4739-A, FCV-65, AN2, AT, 4739-BBB)

For this arrangement (Figure A.1b), the AT is used to ensure that the FCV-65 discharges accurately the predetermined amounts. Simulated flow values to Chelstone zone are dependent on the elevation of the AT which must vary in the same way as the total head for AR. To achieve this, the elevation for the AT is dynamically set at each time step using the AR head pattern. How large the difference between the two should be, is determined based on the knowledge of the system. The average elevation of the AT is 1302.57 m. To ensure that there is no significant AT head variation due to the water column, the water heights in the tank are kept very small. The initial, minimum and maximum values are 0.101 m, 0.1 m and 0.102 m respectively. To compensate for the small tank heights, a very large tank diameter of 600,000 m is used. These values are found by trial and error.

Discharge variations for the FCV-65 and P-1758 before isolation are almost identical, but that of P-1758 after isolation shows clear deviations (Figure A.7). Consequently, the discharge correlation coefficient for the FCV-65 and P-1758 before isolation is 1 (Figure A.8a) and that for after and before isolation is 0.8395 (Figure A.8b). Compared to the first arrangement, with a correlation of 0.9997, the correlation coefficient of 0.8395 shows that AT distorts the downstream flows, but the correlation is still high. The absolute error for the total flows before and after isolation is 6.41%.

The comparison of the pressure before and after Chelstone zone isolation shows that there are differences between the two 24 hours pressure variations graphs (Figure A.9). However, there is a similarity in the general trend of the graphs as reflected by the high correlation coefficient of (0.8286) as shown in Figure A.10. The pressure distribution map at 05:00 am (Figure A.11) compares well with that of the Chelstone part of the LWSN before isolation (Figure A.5). The AT seems to be a good final regulator for the downstream flow to Chelstone zone, however, under conditions that demand is higher that supply (discussed in Chapters 6 and 7), the AT is not effective.

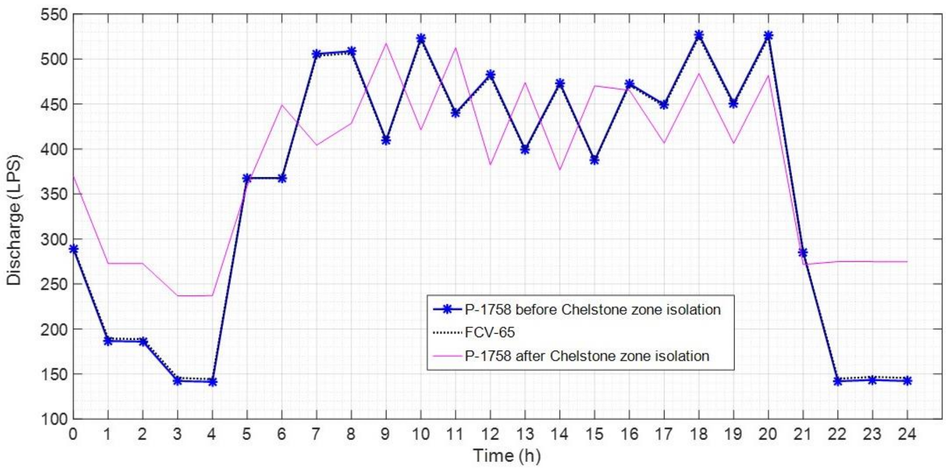
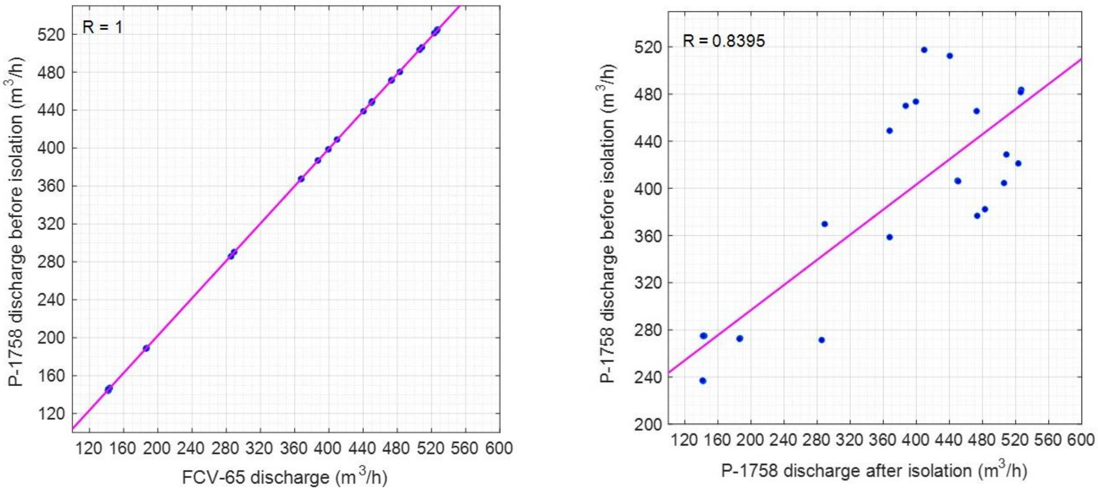


Figure A.7: Second arrangement 24 hours discharge variations



(a) (b)

Figure A.8: Second arrangement correlation coefficient for P-1758 discharge (a) before zone isolation and FCV-65 (b) before and after zone isolation

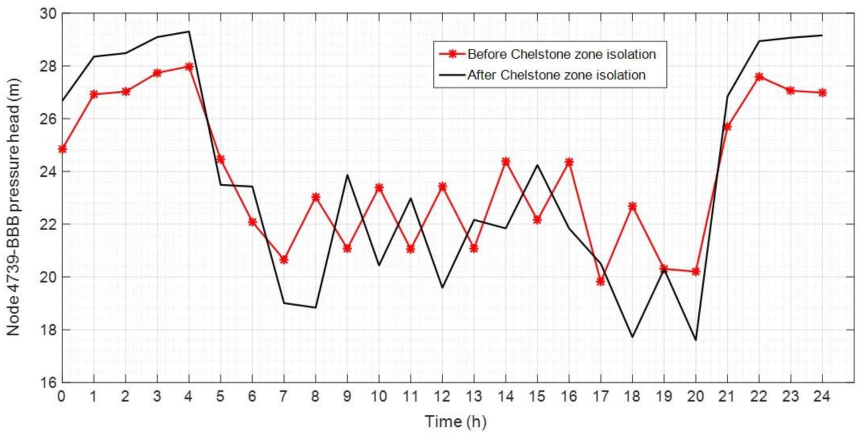


Figure A.9: Second arrangement 24 hours pressure variations

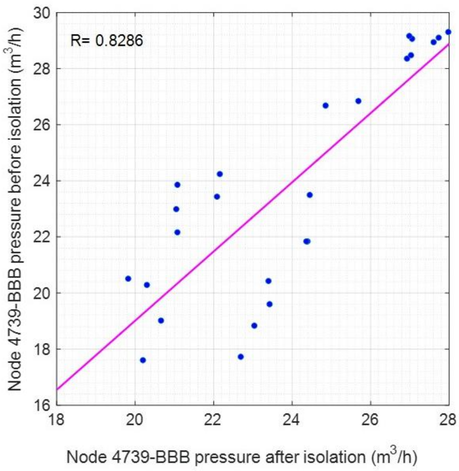


Figure A.10: Second arrangement correlation coefficient for Node 4739-BBB pressure before and after zone isolation

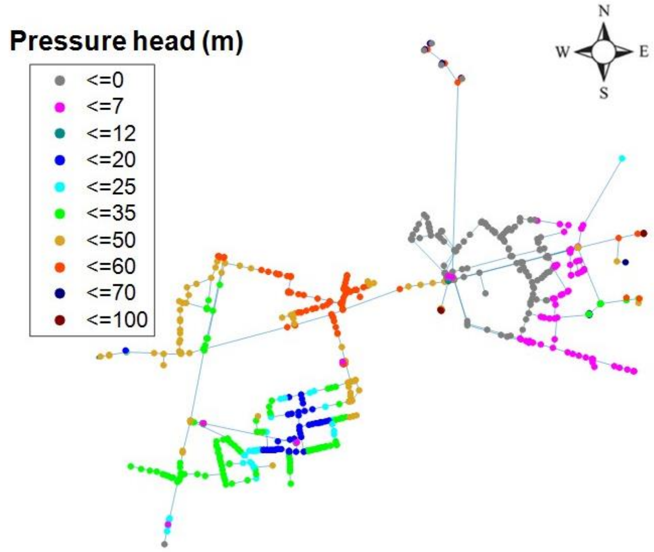


Figure A.11: Second arrangement Chelstone zone pressure distribution map at 05:00 am

The third arrangement (AR, ACV, AN1, AFCV, AN2, AT, 4739-A, FCV-65, 4739-BBB)

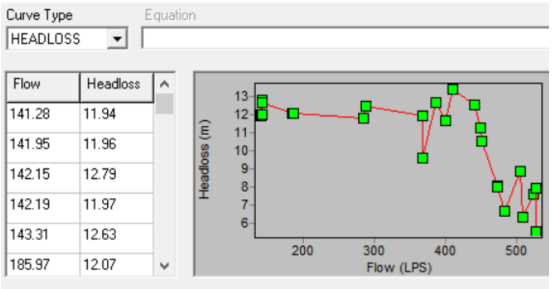
This arrangement is shown in Figure A.1c. The AFCV and the FCV-65 are used to set the maximum flow values at each time step t which are the product of the average flow (Q_{ave}) and the flow pattern multiplication factor (f_{mf}). The Q_{ave} through the AFCV and the f_{mf} are determined using Equations 5.6 and 5.7 respectively. The pattern that is used with the AFCV and FCV-65 is loaded on the AN2 which has the same elevation as AN1. For both nodes, the base demands are set to zero. The average flow (Q_{ave}) is loaded as a fixed value in the parts of the MATLAB code that dynamically set the two valves.

To ensure that the flow through FCV-65 matches that through AFCV, the AT head is adjusted from the minimum $\mathrm{H_{ave}}$ = 1302.57 m to any value less than or equal to the AR head.

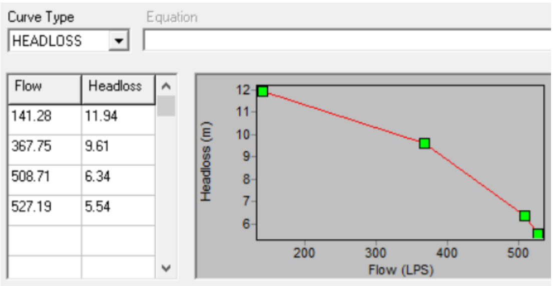
The simulation results for the third arrangement are very similar to the first arrangement. The discharge for the AFCV and P-1758 before and after the zone isolation are almost identical. The correlation coefficient for P-1758 discharge before and after isolation is 0.9993. The Correlation coefficient for the discharge for the AFCV and P-1758 before isolation is 1. However, for pressure, similar 24 hours pressure variations to that of the first arrangement (Figure A.4) and a similar 05:00 am pressure distribution map (Figure A.6) are produced. This arrangement is therefore not good for modelling the water offtake point.

The fourth arrangement (AR, ACV, AN1, AFCV, AN2, AT, 4739-A, AGPV, 4739-BBB)

This arrangement is like the third arrangement except that the FCV-65 is replaced with the AGPV (Figure A.1d). AGPV is used to vary the water supply to the Chelstone zone using the predetermined flow-head loss curve. To develop the flow-head loss curve, first PDA simulation is done for the whole LWSN. The 24-hour hydraulic heads for node 4739-BBB are obtained. The hydraulic heads for the source (AT) are also obtained. The head loss between the AT and node 4739-BBB is the difference between the corresponding heads. For each time step, the head loss is paired with the discharge in pipe P-1758 which is the immediate downstream pipe after node 4739-BBB. The curve developed from this process is shown in Figure A.12a. The curve is manipulated by strategically deleting out some points to develop the final curve shown in Figure A.12b.



(a)



(b)

Figure A.12: The AGPV (a) original head loss curve and (b) modified curve

The discharge for the AFCV and pipe P-1758 before and after the zone isolation are depicted in Figure A.13. The discharge for the AFCV matches well with that for P-1758 before isolation as reflected by the correlation coefficient of 1 (Figure A.14a). The discharge for P-1758 after isolation does not exactly match with the one before the isolation, but there are similarities in the general trend as reflected by the correlation coefficient of 0.8319 (Figure A.14b). The error between the total flow to Chelstone zone before and after isolation is 6.51%.

There are some major differences between the variation of pressure before and after the isolation (Figure A.15) which is reflected by the relatively low, but still significant correlation coefficient of 0.6517 (Figure A.16). However, the pressure distribution map (Figure A.17) compares well with that of the Chelstone part of the LWSN before isolation (Figure A.5). The fourth arrangement, therefore, is a candidate for use as a modelling technique for the water offtake point.

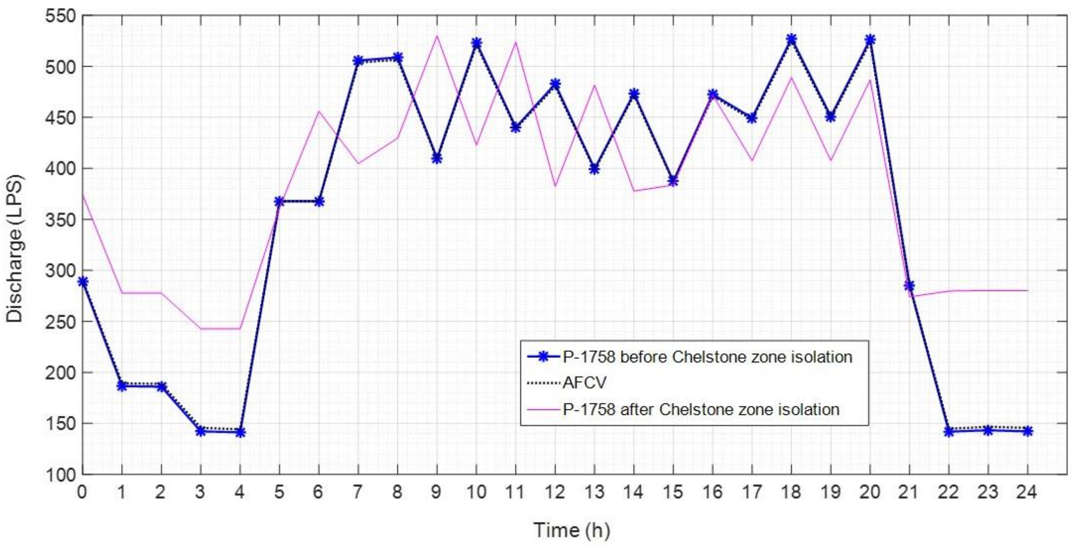
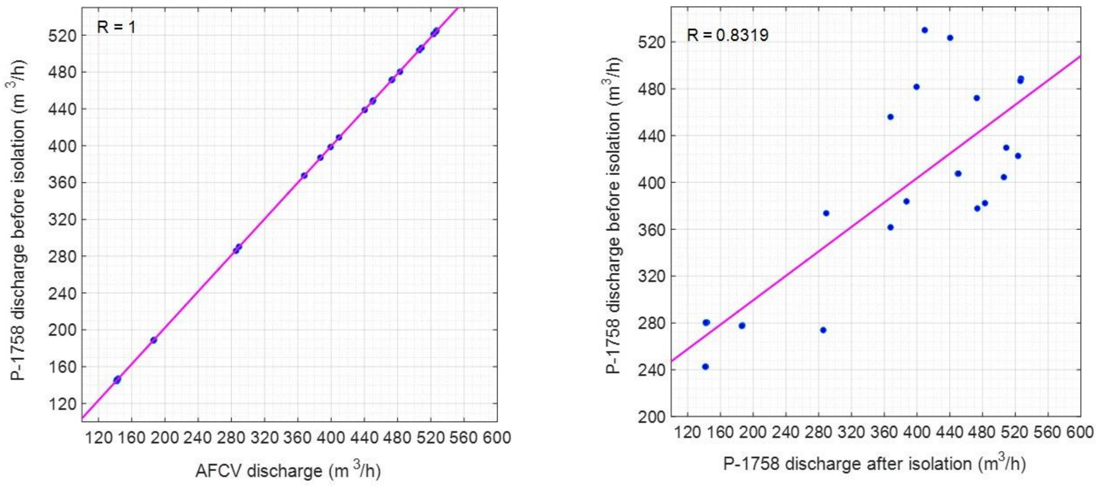


Figure A.13: Fourth arrangement 24 hours discharge variations



(a) (b)

Figure A.14: Fourth arrangement correlation coefficient for P-1758 discharge (a) before and AFCV after zone isolation (b) before and after zone isolation

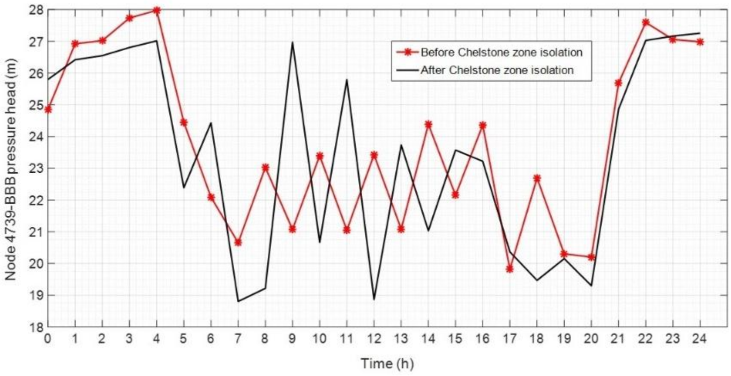


Figure A.15: Fourth arrangement 24 hours pressure variations

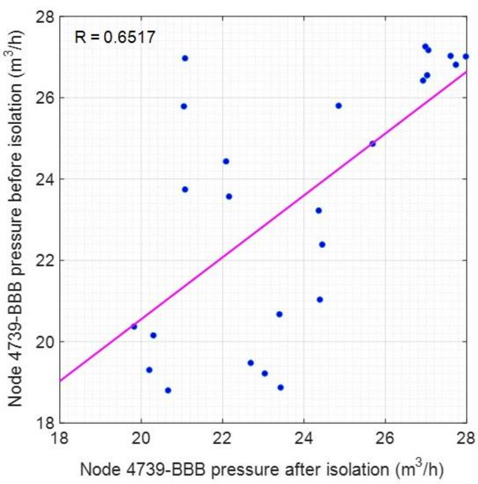


Figure A.16: Fourth arrangement correlation coefficient for Node 4739-BBB pressure before and after zone isolation

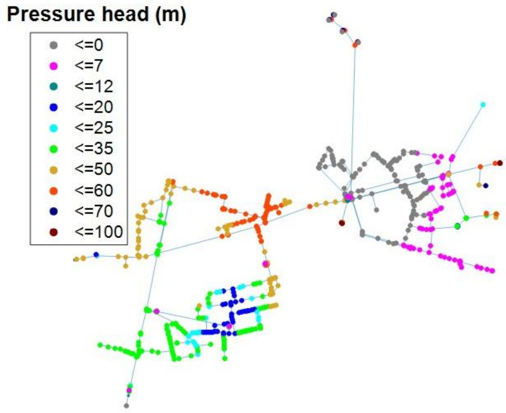


Figure A.17: Fourth arrangement Chelstone zone pressure distribution map at 05:00 am

The fifth arrangement (AR, ACV, 4739-A, ATCV, 4739-BBB)

The arrangement is shown in Figure A.1e. The ATCV is used in place of the FCV-65. For a throttle control valve, the major setting is the minor loss coefficient. The minor head loss coefficient at time step t is determined by Equation A6.1 after rearranging the Wagner et al. (1988) Equation (Paez et al., 2018).

$$K_{TCVt} = \frac{g\pi^{\gamma} D^{2\gamma} H_t}{{Q_t}^{\gamma} 2^{(2\gamma-1)}} \qquad (A5.1)$$

Where K_{TCVt} is the minor head loss coefficient at time t, g is the acceleration due to gravity, D the valve diameter, H_t and Q_t are pressure head for node 4739-BBB and discharge through pipe P-1758 respectively at time t predetermined from the PDA simulation of the whole LWSN.

When the ATCV is used alone to regulate flow, the discharge through the ATCV and P-1758 after the zone isolation are identical, but they are different from the discharge for P-1758 before the zone isolation (Figure A.18). Because of this, the correction coefficients of the ATCV and P-1758 discharge after isolation versus P-1758 discharge before isolation are both 0.8437 (Figure A.19).

In terms of pressure, the pressure variations before and after the isolation show differences. However, the graphs have a similar trend (Figure A.20) which results in a high correlation coefficient (Figure A.21). The pressure distribution map

(Figure A.22) compares well with that of the Chelstone part of the LWSN before isolation (Figure A.4).

There is a need to have the discharge through one of the values closely matching with the discharge of P-1758 before the zone isolation so that it acts as a reference point in situations where there is no simulation data before the isolation. Since ATCV discharge does not match the flow before the isolation, the fifth arrangement cannot be used as a modelling approach for the water offtake point.

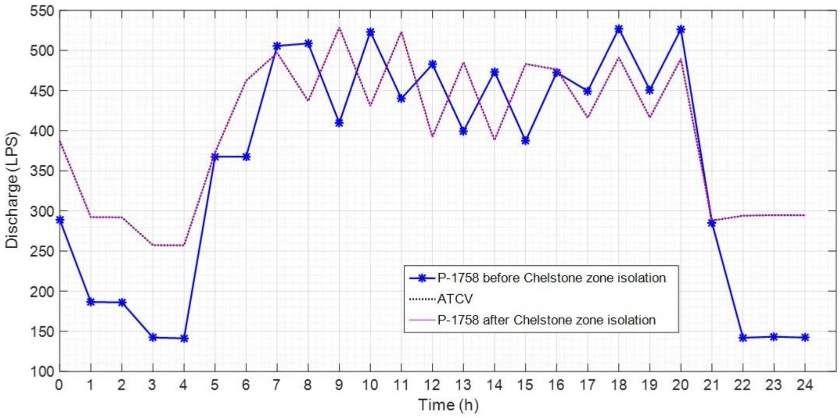
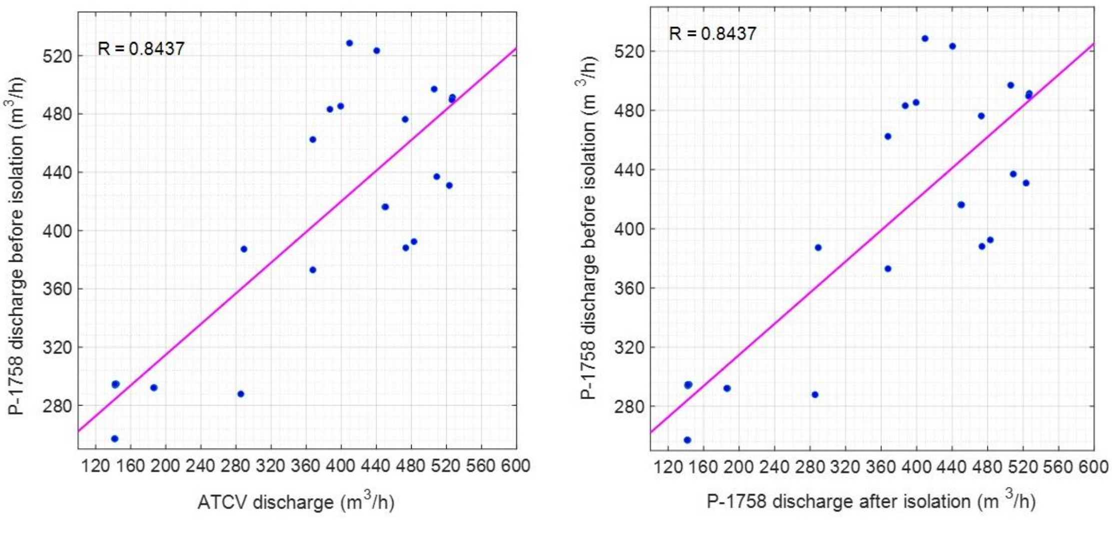


Figure A.18: Fifth arrangement 24 hours discharge variations



(a) (b)

Figure A.19: Fifth arrangement correlation coefficient for P-1758 discharge (a) before zone isolation and the ATCV (b) before and after zone isolation

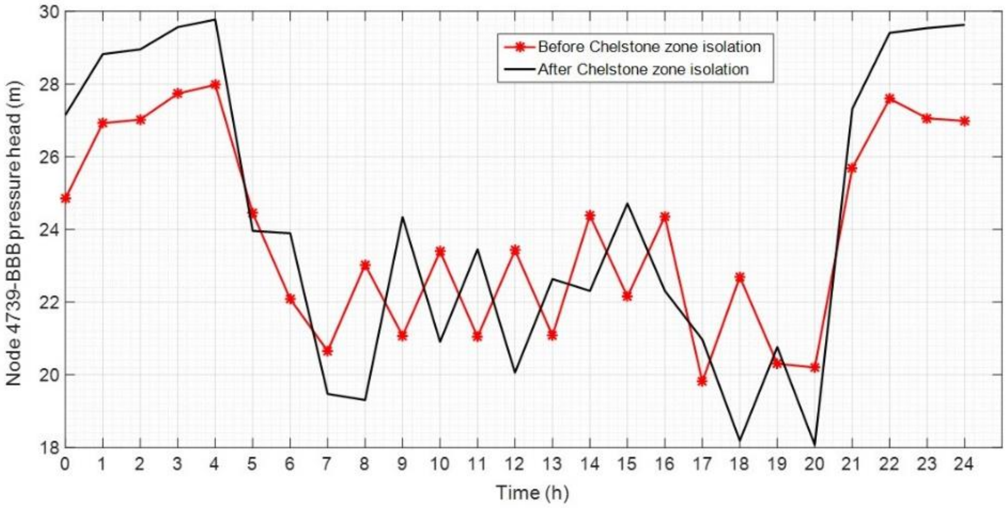


Figure A.20: Fifth arrangement 24 hours pressure variations

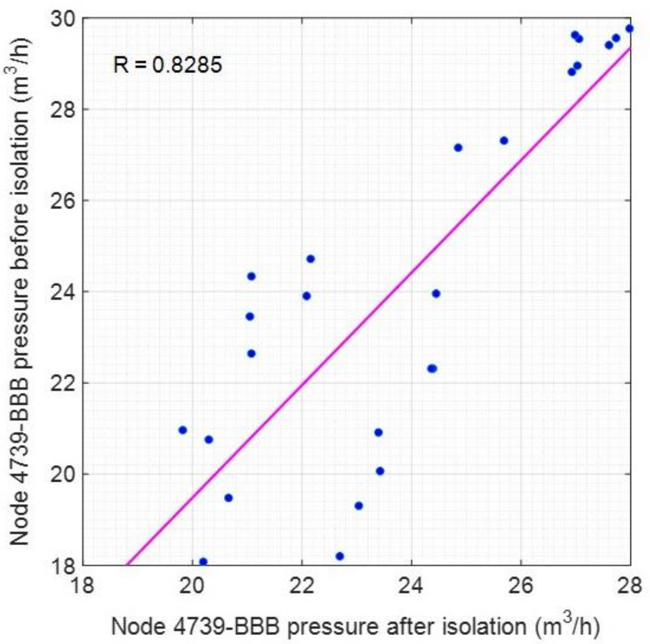


Figure A.21: Fifth arrangement correlation coefficient for Node 4739-BBB pressure before and after zone isolation

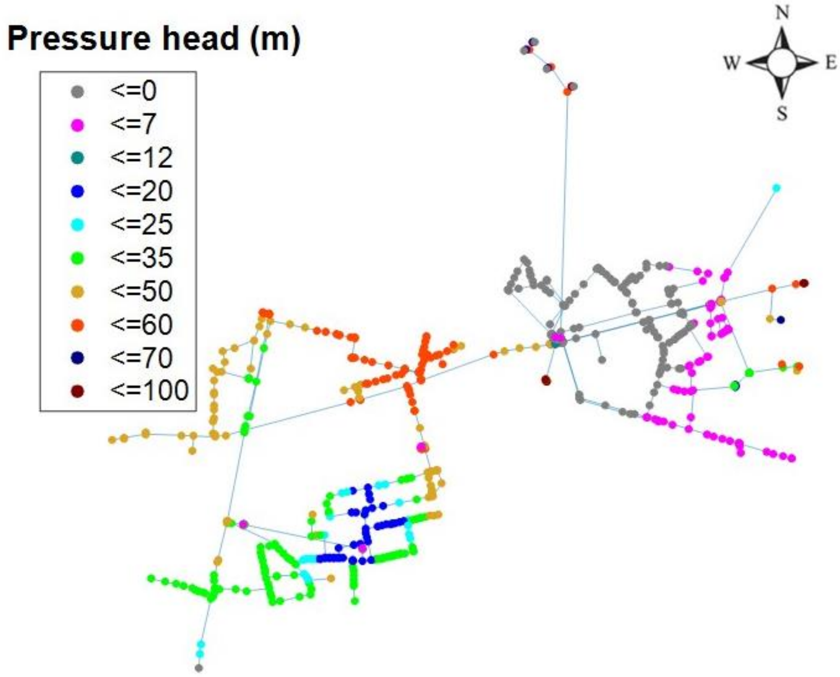


Figure A.22: Fifth arrangement Chelstone zone pressure distribution map at 05:00 am

The Sixth arrangement (AR, ACV, AN1, AFCV, AN2, AT, 4739-A, ATCV, 4739-BBB)

In the sixth arrangement, the AFCV is used to ensure that the predetermined amounts of water flow to the AT. At each time step t, the maximum flow value for the setting of the valve is determined as the product of the average flow (Q_{ave}) and the flow pattern multiplication factor (f_{mf}). The Q_{ave} through the AFCV and the f_{mf} are determined using Equations 5.6 and 5.7 respectively.

The AT is used as a tool to ensure that AFCV discharges the predetermined amounts of water accurately. The hydraulic head of the AT influences the flow through the ATCV. Thus, simulated flow values to Chelstone zone are dependent on the elevation of the AT which must vary in the same way as the total head for AR. To achieve this, the elevation for the AT is dynamically set at each time step using the AR head pattern. How large the difference between the AR and AT head should be, is determined based on the knowledge of the system. The average elevation of the AT is 1302.57 m. This is determined from the variation of hydraulic heads of node 4739-A during the simulation of the whole LWSN. However, for analyses using scenarios (Chapter 7), the determination of the AT elevation is one of the optimisation problems. To ensure that there is no significant AT head variation due to the water column, the water heights in the tank are kept very small. The initial, minimum and maximum values are 0.101 m, 0.1 m and 0.102 m respectively. To compensate for the small tank water heights, a very large tank diameter of 600,000 m is used. These values are found by trial and error.

The accuracy of the AFCV (which depends on the presence of the AT) is important because the flows through it are used as reference predetermined values against which the simulated flows through ATCV and ultimately through pipe P-1758 are compared. This is particularly important in the analyses using scenarios (Chapter 7).

The ATCV is used to regulate the flows to the isolated zone depending on the predetermined pressure head at the immediate downstream node. In this case, the predetermined pressure head at each time step t for the downstream node (node 4739-BBB) is obtained from the PDA EPS of the LWSN before isolation. The setting for the ATCV minor loss coefficient (K_{TCVt}) at each time step t is determined by Equation A6.2 after rearranging the Wagner et al. (1988) Equation (Paez et al., 2018).

$$K_{TCVt} = \frac{g\pi^{\gamma} D^{2\gamma} H_t}{{Q_t}^{\gamma} 2^{(2\gamma-1)}} \qquad (A5.2)$$

Where K_{TCVt} is the minor head loss coefficient at time t, g is the acceleration due to gravity, D the valve diameter, H_t and Q_t are pressure head for node 4739-BBB and discharge through pipe P-1758 respectively at time t predetermined from the PDA EPS simulation of the whole LWSN.

The discharge through the AFCV and pipe P-1758 before and after the zone isolation are depicted in Figure A.23. The discharge for the AFCV matches well with that for P-1758 before isolation with the correlation coefficient of 1 (Figure A.24a). The discharge for P-1758 after isolation does not exactly match with the one before the isolation, but there are similarities in the general trend as reflected by the correlation coefficient of 0.8303 (Figure A.24b). The discharge for P-1758 after zone isolation is achieved through the operation of the ATCV. The error between the total flow to Chelstone zone before and after isolation is 6.74%.

There are some differences between the variation of pressure before and after the isolation, but there is a similarity in the general trend in the graphs (Figure A.25) as reflected by the correlation coefficient of 0.8285 (Figure A.26). The pressure distribution map (Figure A.27) compares well with that of the Chelstone part of the LWSN before Chelstone zone isolation (Figure A.5).

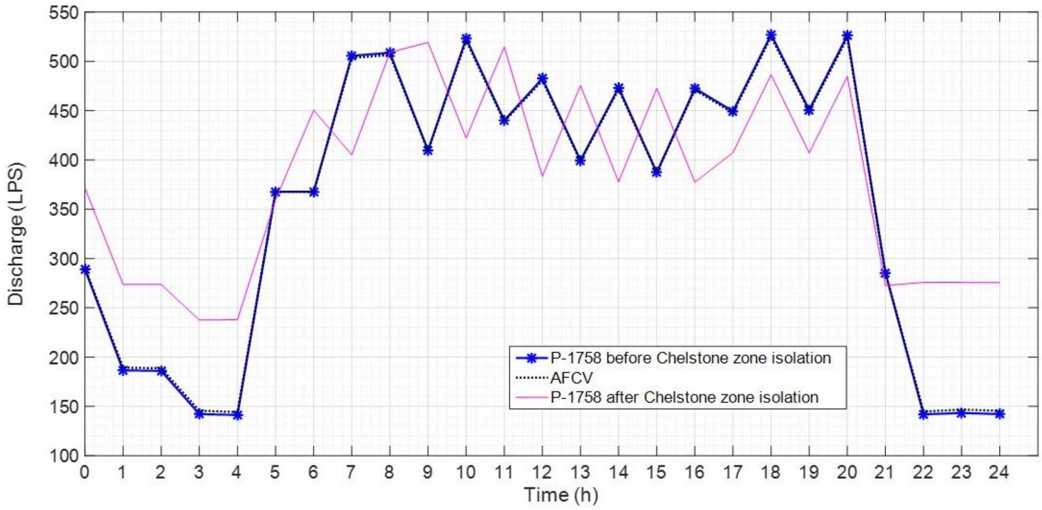
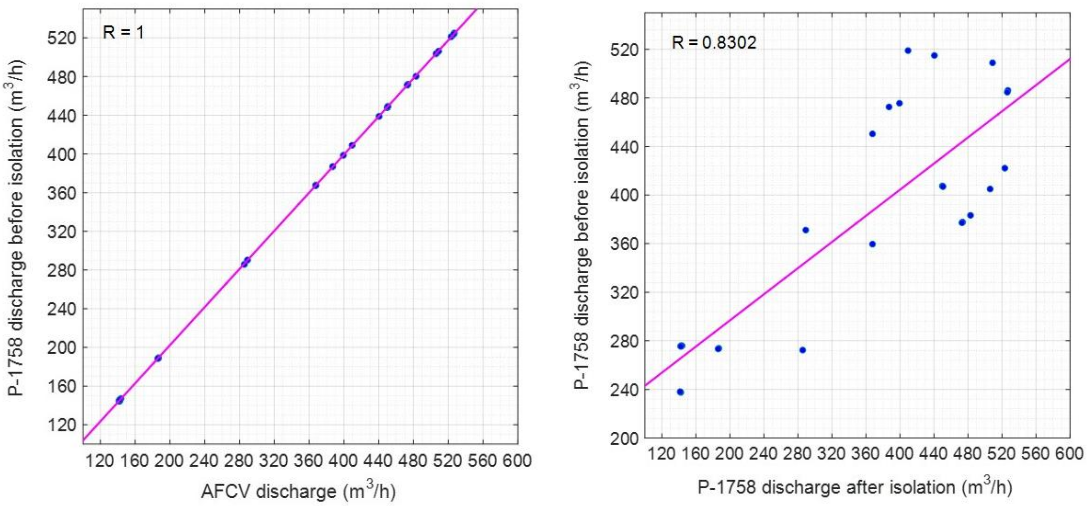


Figure A.23. Sixth arrangement 24 hours discharge variations



(a) (b)

Figure A.24. Sixth arrangement correlation coefficient for P-1758 discharge (a) before zone isolation and AFCV (b) before and after zone isolation

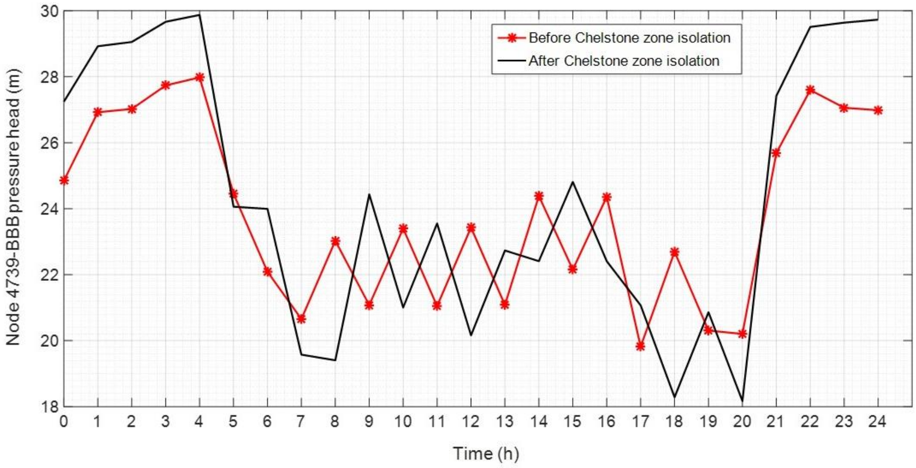


Figure A.25. Sixth arrangement 24 hours pressure variations

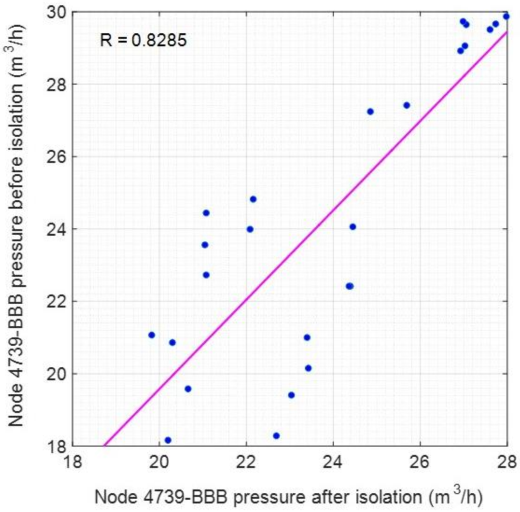


Figure A.26. Sixth arrangement correlation coefficient for Node 4739-BBB pressure before and after zone isolation

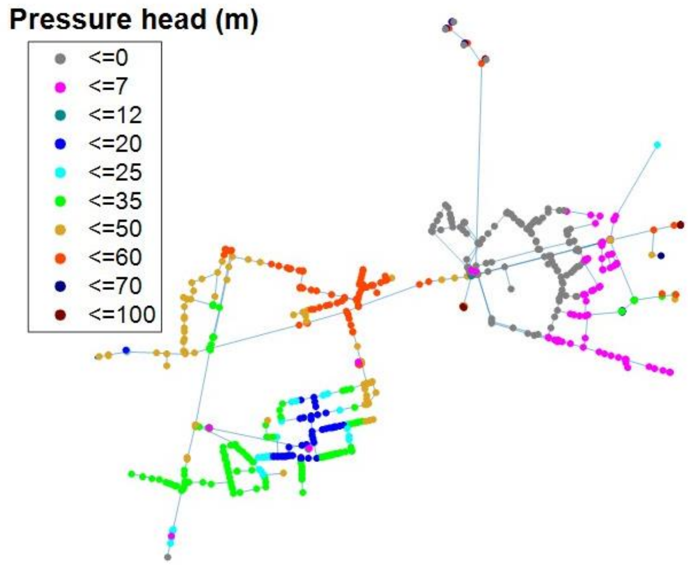


Figure A.27. Sixth arrangement Chelstone zone pressure distribution map at 05:00 am

Appendix B

Background of the water sector in Zambia

Before the late 1990s, the provision of water supply and sanitation services both in urban and rural areas was a direct responsibility of local authorities (LAs). However, due to their poor financial position and limited skilled manpower, most of the LAs could not meet the rapidly increasing demand for these services in urban areas because of the population growth and urbanisation (Chitonge, 2011). The Water supply and sanitation sector was facing many problems including poor legal and institutional frameworks, deteriorating water supply and sanitation services, inadequate human resource capacity, low stakeholder and community participation, limited and ever-decreasing capital investments, and the need to adapt to emerging international trends in water management (Republic of Zambia, 1994; 2010; Chitonge, 2011). To address these challenges, the water sector reforms of the 1990s were undertaken which led to the development of new policies, enactment of new laws and development of new institutions. One of the key outcomes of these reforms was the National Water Policy of 1994 which was to be implemented following the guidance of the seven sector principles which are as follows (Republic of Zambia, 1994; Government of the Republic of Zambia, 2010):

i) Separation of water resource functions from water supply and sanitation
ii) Separation of regulatory and executive functions within the water supply and sanitation sector
iii) Devolution of authority to LAs and private enterprise
iv) Achievement of full cost recovery, in the long run, for water supply and sanitation services through user charges
v) Human resource development leading to more effective institutions
vi) Technology appropriate to local conditions
vii) Increased Government of the Republic of Zambia (GRZ) spending priority and budget spending in the sector

The water sector institutional framework

The major institutions involved in the Zambian water supply and sanitation sector are outlined in Figure B.1. These are the Ministry of Local Government and Housing (MLGH) which is responsible for policy formulation and owns the water supply and sanitation infrastructure, and the Ministry of Water Development, Sanitation and Environmental Protection (MWDSEP) which is responsible for mobilizing financial resources for water supply and sanitation infrastructure development and service delivery. MWDSEP is the lead water development ministry. However, it is not clear how the MWDSEP is financing the development and operation of the water supply and sanitation infrastructure owned by the MLGH. The regulator for water supply and sanitation services is the National Water Supply and Sanitation Council (NWASCO) under which are the Devolution Trust Fund (DTF) and the Water Watch Groups (WWGs). Besides the regulation of water utilities to ensure efficient and sustainable provision of water supply and sanitation services, NWASCO advises the two ministries and LAs on water supply and sanitation issues (Republic of Zambia, 1997). For water resources management (WRM) and environmental management, the Water Resources Management Authority (WARMA) and the Zambia Environmental Management Agency (ZEMA) are the regulatory bodies respectively.

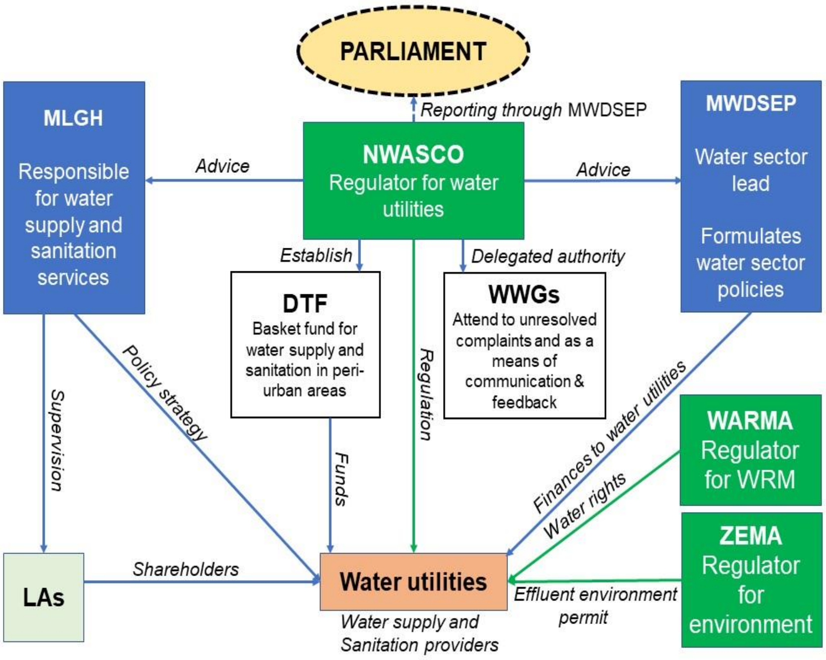


Figure B.1: Institutional framework (NWASCO, 2017)

Water sector policy and Legal framework

The policy and legal framework provide for the setting up of various regulatory bodies and how water resources should be developed, consumed and protected. Critical policies and Acts of parliament for the water supply and sanitation sector are summarised in **Table B.1.**

Table B.1: The water sector policy and legal framework (Simukonda et al., 2018a; NWASCO, 2019)

Policy/legislation	Some key provisions
The Water Act 1948	• Use of surface water resources within Zambia but not applicable to shared watercourses • The distinction between primary and secondary water uses
National Water Policy 1994	• The seven sector principles • Full cost recovery tariffs
Water Supply and Sanitation Act 1997	• Establishment and functions of the National Water Supply and Sanitation Council (NWASCO)
National Water Policy 2010	• Review of the 1994 National Water Policy • Realignment of the water sector's legal and institutional framework with modern international trends
Water Resources Management Act 2011	• Establishment of the WARMA • Provision for water resources management principles
Environmental Management Act 2011	• Establishment and functions of The Zambia Environmental Management Authority • Prohibition of water pollution

Political interference

In Zambia, the operation of any entity that is not absolutely a private concern is subjected to political interference in one way or another. Water utilities which are registered as private companies, but wholly owned by LAs, are directly affected by political interference in many ways. The first two are through the formulated policies and strategies and through financing (Figure B.1). The third way is through governance. This is either through failure to appoint board members for utility companies on time leading to the lack of oversight and accountability (NWASCO, 2017, 2018), or through politically oriented directives. For instance, the directive by the government that the water utility board of directors should comprise the Mayor, Provincial Local Government Officer and the Town clerk among others does not provide a suitable mix of skills, technical or analytical experience beneficial to the boards and the water utilities (NWASCO, 2017).

The fourth one is the appointments of board members of all the water sector-related regulators (Figure B.1) by the line ministries' cabinet ministers. For the water supply and sanitation sector, the irony is that board members for both the regulator (NWASCO) and the regulated (water utilities) are appointed by the same authority (Government) which greatly weakens the boards' decisions in case of conflicting viewpoints because of the allegiance to the same authority (NWASCO, 2017) – the government (cabinet) and its political party.

Failure by the government to show leadership is the fifth way of political interference. The overarching aspect is poor governance (DFID, 2012; Simukonda et al., 2018a). This is linked to the failure by the government to pay water bills on time (NWASCO, 2017; 2018). This makes revenue collection efficiency from the government to be one of the utility performance indicators in Zambia (Millennium Challenge Account – Zambia Limited, 2013; Simukonda et al., 2018a). The other way is the failure of the formal land allocation system that has promoted the illegal plot allocation leading to the mushrooming of unplanned settlement (Lusaka City Council and Environmental Council of Zambia, 2008) which are even in groundwater recharge areas and have encroached the water utilities facilities such as boreholes (Simukonda et al., 2018a). Moreover, the government apparently disregards the protection of the groundwater resource, by setting the Lusaka South Multi-Facility Economic Zone in an area that is known to be a recharge zone for the Lusaka aquifer (Bäumle and Nkhoma, 2008)

Continuation of the water supply problems

The setting up of the legal/policy and institutional frameworks has not solved the problems as expected because these frameworks still have inadequacies (Beekman, 2016; NWASCO, 2017; 2018) and the implementation of the seven sector principles has been poor. By 2010 only separation of regulatory and executive functions within the water supply and sanitation sector, and devolution of authority to LAs and private enterprises were achieved (Government of the Republic of Zambia, 2010; Simukonda et al., 2018a). Consequently, the problems identified before the reforms still exist and new ones such as climate change effects and increasing elements of poor governance are adding to the challenges faced by the water supply and sanitation sector in Zambia in general (Table B.2) and in Lusaka city in particular (Beekman, 2016).

Table B.2: Key challenges faced by the Zambian water supply sector (Simukonda et al., 2018a)

	Year of report									
Challenges	**2007**	**2008**	**2009**	**2010**	**2011**	**2012**	**2013**	**2014**	**2015**	**2016**
Inadequate investment in infrastructure	√	√	√	√	√	√	√	√	√	√
Frequent power outages		√	√	√	√	√	√	√	√	√
Poor payment of bills by government entities					√	√		√	√	√
Poor coordination between town planning, and water supply and sanitation service delivery	√	√				√			√	
Inadequate business orientation by commercial utilities	√	√	√		√	√		√	√	
Drying of reservoirs and low yields from boreholes							√	√	√	√
High water losses, mainly attributed to dilapidated infrastructure, low metering and poor water network management			√	√	√		√	√	√	√
Poor cost coverage due to higher increases in costs than revenue				√	√			√	√	√
Delayed/failed completion of projects coupled with poor workmanship							√	√	√	
Poor water quality due to pollution of sources and inadequate treatment infrastructure			√	√				√	√	√
Poor record-keeping and customer database management										√
Lack of regulation on groundwater resource use			√							
Inadequate storage capacity to counter power outage effects									√	√
Low metering ratio making water demand management very difficult									√	√
Lack of an enforcement mechanism in the Water Supply and Sanitation Act 1997						√	√			

Introduction to the Lusaka water supply network (LWSN)

Involvement of the private sector (sector principle iii) started in the late1980s (before the reforms) through commercialisation (rather than privatisation) of water supply and sanitation services to bring efficiency to the sector and to reduce the financial burden on the state (Chitonge, 2011; Simukonda et al., 2018a). The LWSC, a utility that owns the LWSN, was commercialised in 1988 and started operating as a limited company in 1990. The utility company provides water supply and sanitation services to the whole Lusaka province and the shareholders in the utility company are the LAs for all the districts of the province where water supply and sanitation services are provided by the company. For Lusaka City, the shareholder (owner of the LWSC and its infrastructure) is the Lusaka City Council which falls under the MLGH. This ownership of the utility company's infrastructure puts the responsibility of investing into infrastructure development back to the government, though using the MWDSED rather than the MLGH (Figure B.1). The government was failing to invest into infrastructure development prior to the reforms and is still failing on its own (Table B.2) unless donors come in. With many problems unresolved, water supply continues to be poor and the LWSN (Like all others in the country) is an IWSS though with some parts operating on a CWS basis.

Management and operation

WSS management and operation have many aspects almost all of which relate to database management. The aspects include system zoning, revenue collection efficiency (Chan, 2009; Biswas and Tortajada, 2010), informed tariff setting (Lehmann, 2010) and proper analysis of the state and performance of WSS infrastructure (Klingel, 2012). This section discusses poor data management, LWSN zones and tariff setting (including some aspects of revenue collection). The other aspects are reflected in Table B.2.

Poor data management

The data on the LWSN is inadequate. LWSC took over the pipe network from the Lusaka City Council without proper records. This made it difficult to locate some network pipes and junctions. This failure to locate some network elements has continued. For instance, some network connections are not clear and the sources of water in some parts of the network cannot be identified (Simukonda et al., 2018a). The need to clean up customer data (Republic of Zambia, 2011b) is an indication of lapses in customer database management. Concerning hydraulic modelling, there is a WaterGEMS v3.0 hydraulic model which was developed in 2010 (Brian Colquhoun Hugh O'Donnell and Partners, 2010). At the time of this research, there was no updated hydraulic model for the WSS yet. The available infrastructure data is not adequate to develop a very reliable hydraulic model and the operational data is not useful for a comprehensive analysis of the hydraulic performance of the WSS.

LWSN zoning

For operation and management, the LWSN is divided into five zones and three peri-urban area agglomerations (Figure B.2). The zones are Lusaka Central, Chelstone, Kabulonga, Kabwata, and Lumumba including Matero (Simukonda et al., 2018a), and the three peri-urban area agglomerations are peri-urban Eastern, Southern and Western (Ministry of Local Government and Housing et al., 2009). Water is supplied to the zones and peri-urban area agglomerations from the distribution reservoirs and more than 116 boreholes which are controlled by the production division. Since the production division supplies water intermittently, it is the first to implement the IWS mode. Then, the zones and peri-urban area agglomerations also implement the water supply intermittency by rationing the water between DMAs connected to different sources to try and balance supply durations (Simukonda et al., 2018a). This rationing is implemented through valve and pump operations. While the separation of supply and distribution between the water supply division and the zones is administratively clear, operationally it is not. This is because in many cases water reaching some DMAs in zones or peri-urban area agglomerations is directly from the supply division and the personnel at the zone or peri-urban area agglomeration level have minimal or no control at all.

The IWS operation of the LWSN is contrary to the Guidelines on Required Minimum Service Level according to which all parts of the WSS should be on CWS status (NWASCO, 2000). From the consultants' perspective, CWS is now taken as the long term target (Ministry of Local Government and Housing et al., 2009; Millennium Challenge Account - Zambia Limited, 2013), but meeting this target is proving difficult because of the many challenges that the water supply sector is facing (Table B.2) and the fact that the vision for a CWSS is not part of the LWSC and the Zambian authorities yet.

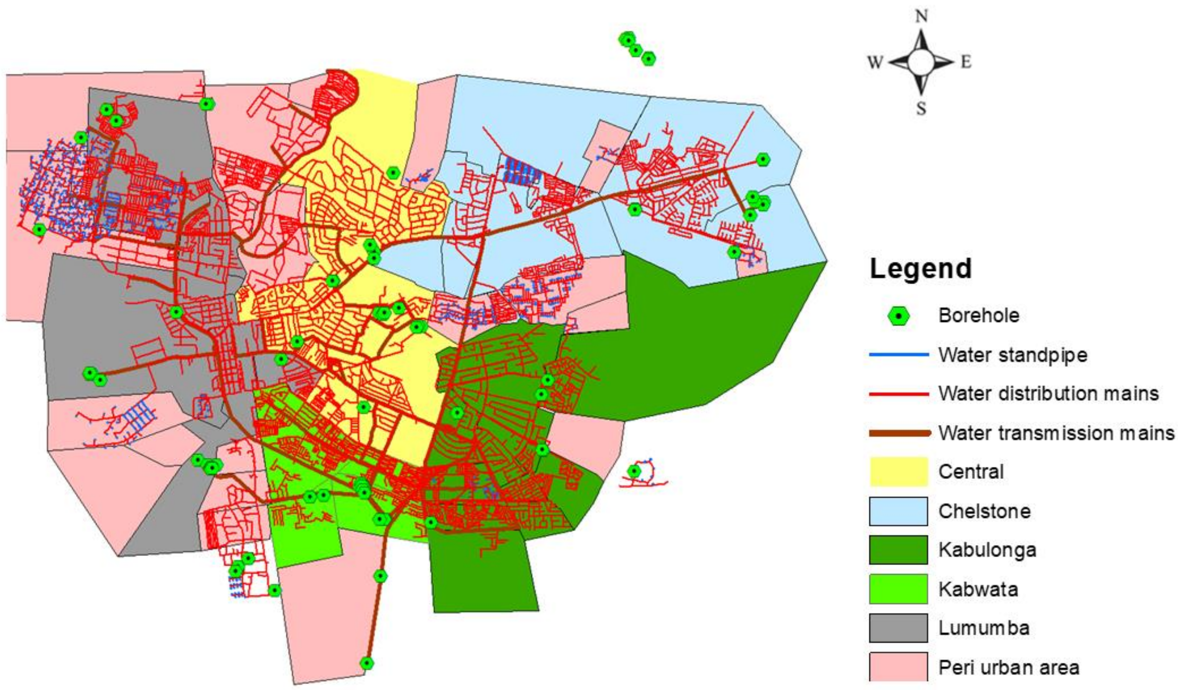


Figure B.2: Lusaka Water Supply Network

Tariffs

Tariffs for water supply services by the LWSC (and all the utilities in Zambia) are approved by NWASCO. The objectives of tariff setting (in Zambia) include the provision of; financial sustainability of utilities, distributive justice and affordability of water supply and sanitation services, protection of water consumers and transparency of the process (NWASCO, 2014). For metered domestic water consumers, a rising block structure (with at least three blocks) is applied. The first block is the social tariff which is for water consumption up to 6 m^3/month for a family of 6. Water within this block is supplied at a cost equal to or less than the cost of service provision while for the other blocks water is charged at the economic price and they are aimed at subsidising water consumption under the lower block (NWASCO, 2014; NWASCO, 2019). For unmetered domestic consumers, a fixed charge tariff is used. This is estimated by metering a representative sample of consumers in the area in question and computing an average consumption for 12 months. Where there is a similar category of customers already metered, the metered customers' average consumption is applied (NWASCO, 2014; Simukonda et al. 2018a). For non-domestic consumers, cost reflective tariffs are applied, but even for them, to instill a sense of water conservation, consumers that consume large water volumes are charged more (NWASCO, 2014).

Some studies show that the LWSC's tariffs are too high considering that many consumers are poor especially in peri-urban areas (Ministry of Local Government and Housing et al., 2009), but since the tariffs are targeted at covering only the maintenance and operation costs, they are low. There is a need to progressively move to full or total cost recovery which is one of the seven sector principles (principle iv) (Republic of Zambia, 2010; NWASCO, 2014; Simukonda et al. 2018a), but progress to achieving this (and the other principles) is poor (Republic of Zambia, 2010; Simukonda et al. 2018a).

The low/high tariffs paradox with respect to LWSC calls for alternative solutions that will enable segregation of consumers beyond what the rising block structure is doing. This is because the cost of supplying water to Lusaka City is likely to increase for at least two reasons. The first is that there is an increasing dependence on surface water from Kafue river through the WTP because groundwater is limited by the aquifer capacity and the water from the WTP is about 5 times more costly than the groundwater (Republic of Zambia, 2011b). The second reason is that the energy tariffs are rising which translates into rising pumping costs. Thus, supplying of the more costly water inevitably translates into increased tariffs which may be dire in view of the high poverty levels amongst the majority of the Lusaka residents and raises concerns of the revenue collection efficiency especially that water supply extension is a requirement.

Water sources

There are two water sources for the LWSN, namely ground and surface water sources. Groundwater has been the main water source for Lusaka City for a long time. More than 116 boreholes within and around the boundary of the district (Figure B.3) provide about 57% of the water supplied to the city while the surface water source supplies the remaining 43% (Simukonda et al. 2018a).

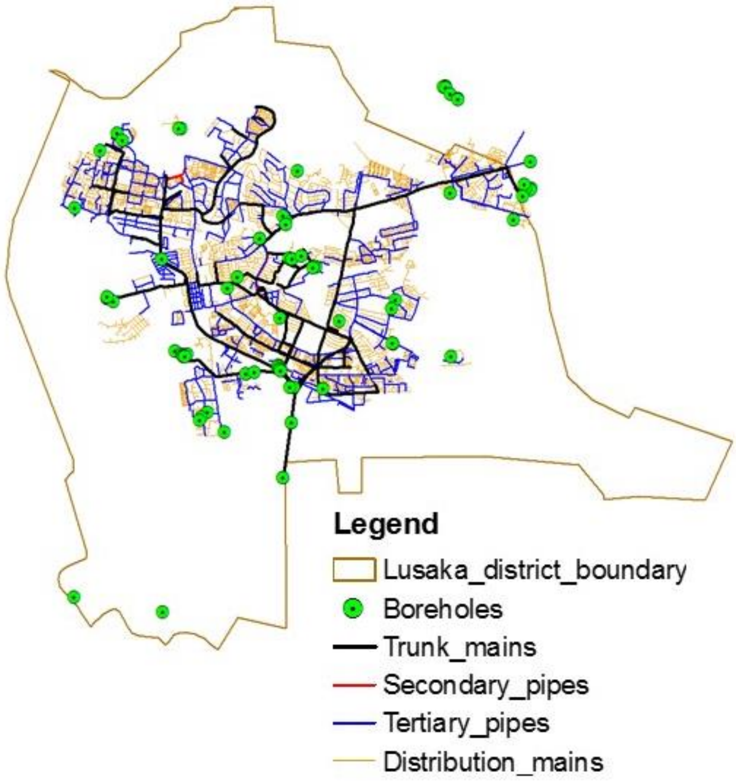


Figure B.3: The LWSN's boreholes as the groundwater sources (Simukonda et al., 2018a)

Groundwater sources

The groundwater sources have both advantages and disadvantages. The advantages include proximity of the sources (boreholes) to where the water is consumed thereby reducing the cost of water reticulation systems, the relatively good quality water which does not require sophisticated treatment processes and also as stated in subsection 5.3.3, from the LWSC's perspective, the energy consumed (a measure of cost) for supplying water from groundwater sources is about 5 times less than the energy consumed for supplying water from the surface water source (Republic of Zambia, 2011b). The disadvantages are discussed below:

Groundwater contamination

The major sources of groundwater contamination in Lusaka City are the use of septic tanks and pit latrines in most parts of the city (Figure B.4), poor solid waste management, industrial effluent and leaking underground petroleum storage tanks (de Waele and Follesa, 2003; Ministry of Local Government and Housing et al., 2009; Kang'omba and Bäumle, 2013; Beekman,2016). The use of septic tanks is becoming even a bigger threat to the groundwater sources for LWSC because residential areas dependent on these onsite wastewater disposal systems have encroached not only the company borehole areas but also the major Lusaka aquifer recharge zone. The effects of the sources of groundwater contamination are magnified by two factors. The first is the lack of coordination and cooperation in the planning and development of the city between public authorities either managing water, or impacting water through their instructions and decision-making (International Water Stewardship Programme, 2016). The second is the karstic nature of the Lusaka aquifer which has characteristics that promote rapid groundwater contamination from anthropogenic related contaminants (Ministry of Local Government and Housing et al., 2009; Kang'omba and Bäumle, 2013).

Figure B.4: Types and distribution of Sanitation systems in Lusaka city (Ministry of Local Government and Housing et al., 2009)

The limited capacity of the aquifer

Another problem with the groundwater resource is the limited amount of water that can be abstracted from the aquifer to meet the projected water demand up to the year 2035 (Kang'omba and Bäumle, 2013). This has raised the need to develop more capacity for the surface WTP.

Electricity outages and equipment breakdowns

Since most of the boreholes are located near or in residential areas, their electricity supply lines are the same as the residential areas where they are located. Electricity outages in residential areas are rampant especially in drought years, and they adversely affect boreholes' water production (LWSC, 2017a). Another problem with the boreholes is the breakdown of the equipment (pumps and other related facilities). Every day about 20% of the borehole supply is lost either due to breakdowns, scheduled maintenance activities or electricity outages (Republic of Zambia, 2011b). The water production capacity lost due to electricity outages, equipment breakdown and repairs is large. For instance, in the period from 10th April 2017 to 9th May 2017, the actual monthly water production was 3.86 $*10^6$ m^3. The total lost production was $1.61*10^6$ m^3 (41.9 % of the actual monthly production) split into total loss due to electricity outages, equipment or plant breakdown and scheduled works as 5.58 $*10^5$ m^3 (14.4 %), $1.02*10^6$ m^3 (26.4 %) and $3.37*10^3$ m^3 (0.9 %) respectively (LWSC, 2017a).

Surface water source

The surface water source for the LWSN is the Kafue River (Figure B.5). The WTP is located downstream of the part of the river called the Kafue flats sub basin. The flow of the river before the WTP intake is regulated by the Itezhi-Tezhi dam and the Kafue flats just downstream the dam. Downstream the WTP intake, the flow is regulated by the Kafue gorge dam. Although the river flow varies according to seasons, its average discharge at the WTP intake of 234 m^3/s (dry season month and normal rainfall year) is enough for Lusaka City water demand for the planning horizon up to 2035 (Ministry of Local Government and Housing et al., 2009).

Figure B.5: Kafue River downstream the WTP intake in June 2017

Water abstraction from the Kafue River

The amount of water that can be abstracted from the river by LWSC is limited by rights, but for water supply by a utility, additional rights can easily be obtained because water supply is a government responsibility (Republic of Zambia, 2011a). Consequently, the abstraction of water from the Kafue River for supplying the Lusaka City is practically unlimited. However, the limiting factor for this source is the capacity of the WTP.

Water transfer from Kafue river to Lusaka City

The distance between the WTP (which is along the Kafue River) at an elevation of about 1010 m and the main water distribution reservoir at the elevation of 1304 m is about 47 km. Water is pumped over a total height of 294 m (~300 m) in two phases (Figure B.6). The first phase is for a height of about 140 m from the WTP to Chilanga booster station which is 24 km from the plant and at an elevation of 1050 m. From Chilanga, water is pumped to the main water distribution reservoir. The treatment processes plus the pumping to the distribution reservoir lead to the higher energy costs than for the water supplied from boreholes.

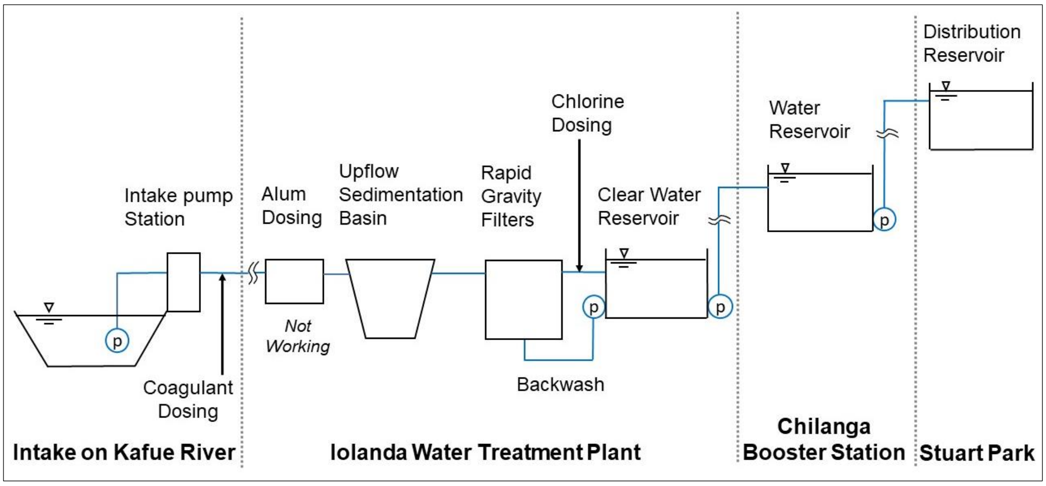


Figure B.6: Water transfer stages from the WTP intake to the main distribution reservoir at Stuart Park (Republic of Zambia, 2011b)