# Strategy to Transition from an Intermittent to a Continuous Water Supply Using a District Metering Area Approach

Case Study for Greater Accra Metropolitan Area, Ghana

**P. den Dekker**

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**By**

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*By: Charlie Mackesy*

## Abstract

In this thesis, I develop and evaluate a demand-side approach to transition from an intermittent to a continuous water supply using a district metered areas approach in Accra, Ghana, an urban metroplex of 4.0 million persons in western Africa.

Intermittent water supply (IWS) is inherently inefficient. These systems foment health hazards and are expensive with an inherent low return on investment. Various causes and effects with positive feedback loops exacerbate intermittency. And building more robust IWS systems does not help because of rapid system degradation and high socio-economic costs render them unsustainable.

Continuous water supply (CWS) systems are superior in every respect. Therefore, I view transitioning from IWS to CWS as the optimal choice for urban water districts. The general strategy to make this transition is to increase production and sharply improve the transmission capacity and efficiency of the drinking water system, the so-called supply-side approach.

But because the supply side approach does not factor the underlying causes of IWS-- leakage and variable pressure levels, I used a demand-side approach for this study and designed a novel method, based on leakage theory with the application of district metered areas (DMAs). We tested this method on a case study performed in Accra, Ghana.

The method is essentially a set of requirements and boundary conditions used to facilitate the transition from IWS to CWS from a demand-side perspective. A decision tree gives insight into the causes of IWS per DMA and the interventions required to reduce intermittency. Based on the foregoing, water engineers can build strategies to roll out a demand-side CWS systems in DMAs for an entire region.

In our Accra study, we tested three DMAs using this approach with a focus on supply security. Data from those DMAs was collected and a top down NRW assessment was performed. We selected one DMA to collect and analyse flow and pressure data (minimum night flow, non-revenue water, billing, intermittency level, average zonal pressure). Furthermore, the supply conditions and proposed intervention (pressure adjustments) were hydraulically modelled for the district. Based on the DMA data, a transition strategy was developed for the Greater Accra Metropolitan Area. We also assessed the applicability of the demand-side approach to other areas through a survey among water supply specialists.

The study shows that gains in water savings at the DMA level to more than justify the costs of transitioning to CWS. In cases of high pressure and high real loss volumes, pressure management improves supply conditions for customers. And, after attaining CWS, water recovered from loss reductions can be redistributed into neighboring districts, increasing their water availability. Because hydraulic pressure dependent demand (PDD) modelling cannot model leakage accurately, it was not possible to evaluate leakage reduction interventions. And it was not possible to create a working hydraulic model to assess the effect of this DMA approach for the entire Accra metroplex because of limited data availability at a district level, leakage parameters that were difficult to verify with field measurements, and the lack of a proven hydraulic PDD software able to distinguish between domestic demand and real losses.

In conclusion, the developed method looks promising. But two factors limited fully exploring its real-world application: (1) the absence of mature modelling software; (2) detailed water district data. Developing a working hydraulic modelling software was beyond the scope of an MSc. thesis. But such hydraulic modelling software should be developed to help water utilities with limited technical and human resources improve services. Further research is required into leakage component modelling and validation from field measurements, after which interventions can be more properly and easily applied.

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## List of Acronyms

| | |
|---|---|
| AZP | Average Zonal Pressure [m] |
| C | Leakage component EPANET emitter [-] |
| CARL | Current Annual Real Losses [$m^3$/day] |
| CWS | Continuous Water Supply |
| DDA | Demand Driven Analysis (Hydraulic) |
| DEM | Digital Elevation Model |
| DMA | District Metered Area |
| DMM | Domestic Demand Management |
| DN | Diameter [mm] |
| GAMA | Greater Accra Metropolitan Area |
| GPS | Geographic Positioning System |
| GSS | Ghana Statistical Service |
| GWCL | Ghana Water Company Limited |
| HDPE | High Density Poly Etylene |
| HPZ | High Pressure Zone |
| ILI | Infrastructure Leakage Index [-] |
| IWS | Intermittent Water Supply |
| KPI | Key Performance Indicator |
| MDA | Ministries, Departments and Agencies |
| MNF | Minimum Night Flow [$m^3$/hour] |
| $N_1$ | leakage exponent [-] |
| NDF | Night Day Factor [-] |
| NPV | Net Present Value [EURO] |
| NRW | Non-Revenue Water [%, $m^3$/month] |
| PDA | Pressure Dependent Analysis |
| PDD | Pressure Dependent Demand |
| PM | Pressure Management |
| PN10 | PVC pipe, quality level 10 |
| PRV | Pressure Reducing Valve |
| $Q_A$ | Authorized Consumption [$m^3$/day] |
| $Q_{AL}$ | Apparent Losses [$m^3$/day] |
| $Q_{BA}$ | Billed Authorized Consumption [$m^3$/day] |
| $Q_{CRL}$ | Current Level of Real Losses = CARL [$m^3$/day] |
| $Q_{DR}$ | Domestic Demand Required [$m^3$/day] |
| $Q_{DR, hourly}$ | Domestic Demand Required [$m^3$/hour] |
| $Q_{ERL}$ | Economic Level of Real Losses [$m^3$/day] |
| QGIS | Quantum Geographic Information System |
| $Q_I$ | Inflow [$m^3$/day] |
| $Q_L$ | Water Losses [$m^3$/day] |
| $Q_P$ | Production Capacity [$m^3$/day] |
| $Q_{RL}$ | Real Losses [$m^3$/day] |
| $Q_{UA}$ | Unbilled Authorized Consumption [$m^3$/day] |
| $Q_{URL}$ | Unavoidable Level of Real Losses = UARL [$m^3$/day] |
| RLR | Real Loss Reduction |
| SIV | System Input Volume (Qi) [$m^3$/day] |
| Stdev | Standard Deviation |
| USD | US Dollar ($) |
| VEI | Vitens Evides International |
| WDS | Water Distribution System |
| WHO | World Health Organization |
| WOP | Water Operators Partnership |

# 0. Context of this MSc. Thesis – 'Setting the Scene'

In February 2019, Vitens Evides International (VEI), started their drinking water operating partnership (WOP) with Ghana Water Company Limited (GWCL) under the WaterWorX program which was initiated by the Dutch Ministry of Foreign Affairs.

One of the objectives of this partnership for VEI was to help GWCL improve their supply conditions in the Greater Accra Metropolitan Area (GAMA), since intermittent supply conditions were rampant in this urbanized area.

In their quest to improve these supply conditions in GAMA, VEI sought for a deeper scientific understanding to achieve this goal. Therefore, this masters' thesis was commenced.

VEI, together with Dutch engineering firm Royal Haskoning-DHV, wants to develop investment proposals in collaboration with GWCL on how to effectively and efficiently improve these supply conditions in GAMA. This thesis will be part of the scientific background to make such proposals. Ultimately, VEI will assist GWCL in the implementation of the proposed investments that enhance the supply conditions of this metropolitan.

# 1. Backgrounds of Intermittent and Continuous Water Supply

## 1.1. Introduction

Over 663 million people worldwide lack access to enhanced drinking water services while around 2.1 billion people lack access to safely managed drinking water (United Nations, 2018; WHO, 2017). Losses in piped water supplies are a major obstacle to making better drinking water more widely available. Water losses were of lesser concern in times of plenty. But the adverse effects of rapid population growth, urbanization and climate change has reduced water availability. Annual water losses across the globe are 126 billion $m^3$, 36% of total global municipal water, and valued an astounding USD 39 billion annually (Cosgrove & Rijsberman, 2014; Liemberger & Wyatt, 2018). Remarkably, a one-third reduction in these losses would supply 800 million people with basic drinking water service (Liemberger & Wyatt, 2018).

Due to high levels of water loss, rapid population growth, and urbanization, water demand has outstripped supply in many areas. Intermittent water supply (IWS) occurs when the supply of water is less than 24 hours per day or seven days a week. Currently, over 1.3 billion people suffer the adverse effects of IWS, 45% of the world's population with access to piped water (Charalambous & Laspidou, 2016).

This chapter provides the background and current literature study regarding salient topics to identify the critical factors in designing a transition strategy from IWS to CWS. First, IWS is explained in detail. Challenges, causation, and solutions IWS are identified. Transition approaches are explained, after which more attention is given to a bottom-up approach. The requirements for this type of approach are described, followed by a real-world application to a case study of Greater Accra Metropolitan Area (GAMA), Ghana which is described in the next chapter.

## 1.2. Terminology

Terminology for water intermittency is not standardized. Terms referring to IWS include: irregular, unreliable, outages and scarcity, inadequate supply and poor supply. However, these terms are not used consistently in the industry, causing confusion in some studies. The lack of standard terminology may reflect the amorphous nature of the problem itself (Galaitsi et al., 2016). Intermittency takes many forms across time and space. There are annual patterns of availability with water supply reliable in one season and less predictable in others. Over time, a community's water access may shift as populations grow, or as infrastructure decays or is replaced. And water availability can fluctuate because of natural inter-annual variability.

Galaitsi, et al., proposed three definitions, listed from least disruptive in consumers' lives to most disruptive (Galaitsi et al., 2016):

1) Predictable Intermittency – known timeframe of supply & sufficient quantity
2) Irregular Intermittency – inconsistent timeframe of supply & sufficient quantity
3) Unreliable Intermittency – inconsistent timeframe of supply & unknown quantity

Predictable intermittency does not mean insufficient household supply. Though water rationing in situations of scarcity is often used as a justification for intermittent supply, in many situations it does not reduce water consumption, which is a function of user habits and socio-economic levels (Marchis et al., 2011). It provides a temporal change in water access because water must be stored. However, if enough water is delivered and consumers have adequate storage, consumption be unaffected can resemble continuous supply. The definitions used by Galaitsi do not give a full synopsis of the different supply states. To get a clear picture of the differences in intermittency, an overview of the different supply definitions is given in Figure 1, based on Galaitsi definitions and extended, by necessity, by the author. (Galaitsi et al., 2016).

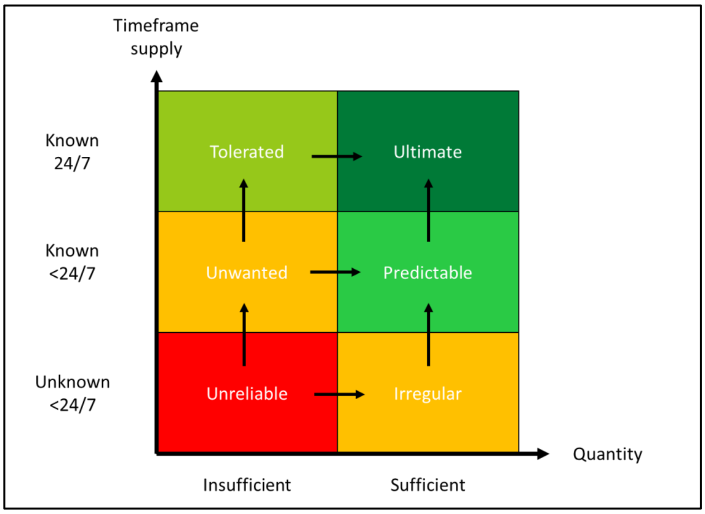



*Figure 1 Water Supply States Matrix (Design by author, 2019)*

**Supply States**

The colour codes in the matrix represent the desirability of the supply state. The arrows indicate what steps can be made in order to improve the current supply state, where going from insufficient to a sufficient supply (from left to right) is preferred over improving the timeframe of supply (from bottom to top). Predictable supply is preferred over tolerated supply, since there is sufficient water for the first. Tolerated supply is often caused by scarcity, either physical (water) or economic. It is important to distinguish partial and temporal intermittency from full-time intermittency. Temporal events can be droughts, pollution accidents, earthquakes and maintenance (Solgi, Haddad, Seifollahi-aghmiuni, & Loáiciga, 2015). When water rationing and reduced duration of supply are norms for the system, the operation is called full-time intermittency and cannot be easily be reverted to continuous supply (Simukonda, Farmani, & Butler, 2018). Full-time intermittency has many interconnected causes, that are difficult to isolate (Bruggen & Borghgraef, 2010).

### 1.3. Challenges Due to IWS

Water distribution systems (WDSs), in the vast majority of cases, were designed for CWS (K. Vairavamoorthy, Gorantiwar, & Pathirana, 2008). In a minority of cases, IWS was implemented intentionally. The distribution network emptied due to the high demand and insufficient supply. IWS has negative consequences on the network infrastructure, water quality and water losses. Furthermore, it creates high coping costs for utilities and customers and increases inequality between the poor and middle class as well as men and women/girls. Some of these effects have positive impact on intermittency, they increase the intermittent supply. Figure 2 shows the effects of IWS in a fish-bone diagram, with positive feedback loops for effects that increase intermittency. The effects are organized in their respective themes: technical, water quality, financial, socio-economical and governance. An example of an effect that has a positive influence on intermittency is the increase in undesired coping mechanisms, illegal connections, for example. This causes the cost-recovery of the utility to decrease, which makes it harder for the utility to invest in maintenance further exacerbating IWS. In the figure a plus sign indicates the positive influence of that specific effect on IWS, increasing it further. In Appendix M, a detailed overview is given of the different effects that are caused by IWS systems.

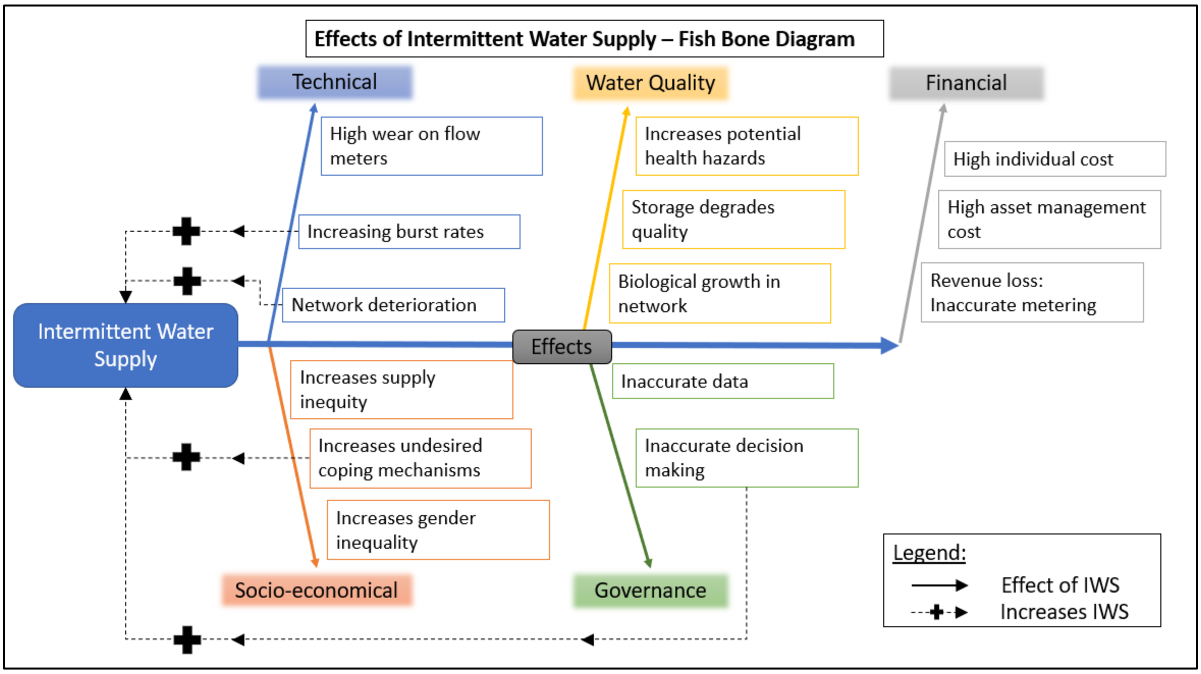



*Figure 2 Fishbone diagram - Effects of Intermittent Water Supply and positive feedback loops (Design by author, 2019)*

### 1.4. Causes of IWS

To find adequate solutions to reduce or solve the challenges regarding IWS, the causes need to be understood. A distinction must be made between external, outside control of utilities and internal causes, which are inside the control of utilities. Figure 3 sets forth these causes and effects of IWS in a fish-bone diagram. External causes are climate change, changing demographics, poor governance and economic development. Finally, the power supply, or lack thereof, is a more isolated causal factor. Internal causes are limited human capacity and lack of customer awareness as well as poor operations and management. Intermittency causes physical losses, substandard operations and maintenance and increasing poor governance hampering economic development.

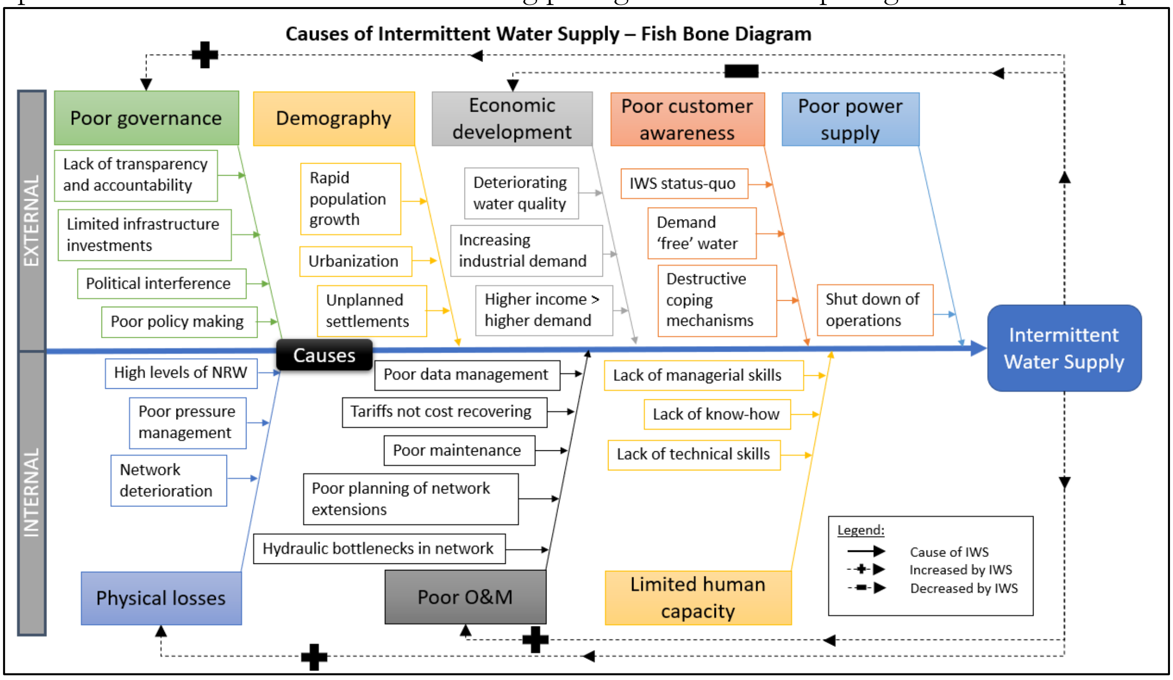



*Figure 3 Fishbone diagram- Causes of Intermittent Water Supply with feedback loops (Design by author, 2019)*

The upper section of the diagram represents external causes, the lower internal causes. Plus, and minus signs are assigned to different causal factors based on the relative effect on IWS, these are referred to as positive or negative feedback loops. In Appendix M, a detailed overview is given of the different causes of IWS systems.

### 1.5. Reasons to Avoid IWS

Charalambous, et al., articulates reasons to avoid IWS systems (Charalambous, 2019). They include network deterioration, increased leakage levels, pipe and service connections breaks, water quality problems, adverse financial effects and customer dissatisfaction. IWS especially has bad effects on the poor, since they do not have other suitable options to get water. Women and girls are also among the disadvantaged by IWS, since they normally are the ones that need to fetch water from other (unreliable) sources (Charalambous, et al., 2016). IWS should also be avoided since it promotes unwanted customer behavior such as creating private storage capacity, creating illegal connections, meter tampering, and creating a system where all valves are open and thus limited pressure can be developed inside the network.

### 1.6. Ways to Solve IWS

To avoid the adverse effects of IWS, two solutions are proposed: Increasing the IWS system robustness and changing to a CWS system.

Vairavamoorthy, et al. described the design requirements for the network conditions for IWS under improved conditions (Vairavamoorthy, Akinpelu, Lin, & Ali, 2001; K. Vairavamoorthy et al., 2008). For the design of IWS systems, peak factors are used using the common CWS peak factor of 2-3 and dividing by the amount of supply hours per day. This results in peak factors ranging between 2 and 12 (Abu-madi & Trifunovic, 2013). This enhanced IWS system still faces network deterioration and undesired customer behavior. Making it more robust is expensive, since the infrastructure must deal with much higher peak factors.

In other studies, the enhancement of IWS systems is discarded, and the transition to CWS systems is promoted since it diminishes the detrimental effects on the network and supply, increases water quality and therefore reduces health hazards and increases revenues (Klingel & Nestmann, 2014; Mcintosh, 2003). Different approaches exist to transition from IWS to CWS. External approaches, like governance improvement and social behavior are outside the control of the utility. Technical, management, operations and maintenance and the use of tools are inside the control of the utility as shown in table 1. Governance and social behaviors can be influenced by the government.

| Approaches to transition to CWS | | | |
|---|---|---|---|
| Internal | Technical | Top down network improvements | • Increase production capacity<br>• Increase hydraulic capacity network |
| | | Bottom up network improvements | • Leakage management<br>• Pressure management<br>• Successive zone conversion |
| | Operations & Maintenance | Metering | • Improve billing by accurate metering |
| | Management | Decision making | • Database management<br>• Analyzing data<br>• Long term perspective |
| | | Financial | • Financial planning |
| | | Human Capacity | • Capacity development<br>• Network and distribution know-how |
| | Tools | | • Hydraulic modelling<br>• GIS improvement |
| External | Governance | | • Institutional improvement<br>• Cost recovering tariff setting<br>• Reduce political influence<br>• Improving enabling environment for infrastructure investments |
| | Social | | • Improving water use behavior |

*Table 1 Overview approaches to transition from IWS to CWS.*

### 1.7. Supply-side vs. Demand-side Approach

Distinctions must be made among the different approaches available to transition to CWS. Water supply systems consist of different elements. Starting at a source, where water is derived, it is treated and distributed to storage reservoirs through the primary network. From the storage reservoir, water flows under gravity or is pumped into the secondary and tertiary networks where it reaches the customer at the tap. According to Vitens Evides International[1] (VEI), many engineering firms, contractors and development banks focus on developing a top-down approach when asked to solve IWS issues, specifically designing and installing capital intensive hardware. This approach focusses on the supply-side of the system and does not address reducing physical losses in the system as well as pressure optimization. The problem is that additional capacity generated at the top is lost passing through the network, with only a fraction reaching the customer. This approach increases pressures in the network, resulting in higher leakage rates.

In the case of the bottom-up approach or the demand-side focusses approach, step one is to optimize pressures and leakage in the tertiary and secondary network. When the performance is improved, i.e. leakages reduced and pressures managed, it then moves up to the next level and determines the bulk water distribution and storage requirements that are necessary.

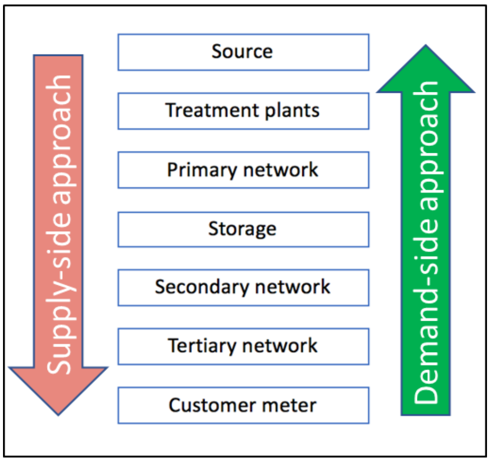



*Figure 4 Supply side and demand-side (bottom-up) approach for transitioning to CWS*

An overview of the supply-side and demand-side approach is shown in figure 4. The probable reason the demand-side approach is not used by development banks, engineering firms and contractors is because data unavailability and uncertainty increase at each stage after the treatment plant, with the highest data unavailability

[1]Nijsse, Martin (regional program manager in Uganda and team leader for Ghana, Rwanda and Tanzania at VEI) and Veenstra, Siemen (Director Africa at VEI) during meeting with author, March 2019.

and complexity at the customer level. Therefore, measuring the expansion at the supply side of the system is often easier to do, than measuring the level of satisfied customer demand.

### 1.8. Non-Revenue Water

Non-revenue water (NRW) is the volume of water not converted into revenues, i.e. water lost in the distribution and billing process, between the production plant and customer meter. International experience shows that water losses are the primary cause of IWS (Charalambous et al., 2016). There are several significant factors causing water losses. In the early 2000s, the International Water Association (IWA) developed a universal benchmark to standardize the categories of water losses, shown in figure 5. This includes water losses ($Q_L$) and Unbilled Authorized Consumption ($Q_{UA}$). Water losses can be apparent ($Q_{AL}$) —meter inaccuracies, illegal connections, and inaccurate billing; and real ($Q_{RL}$) —leaking pipes, overflow of reservoirs, and poor fitting fixtures (A. O. Lambert, 2002).

<table>
<tr><td rowspan="10">System input volume $Q_I$</td><td rowspan="5">Authorised consumption $Q_A$</td><td rowspan="3">Billed authorised consumption $Q_{BA}$</td><td>Billed water exported</td><td rowspan="3">Revenue water</td></tr>
<tr><td>Billed metered consumption</td></tr>
<tr><td>Billed unmetered consumption</td></tr>
<tr><td rowspan="2">Unbilled authorised consumption $Q_{UA}$</td><td>Unbilled metered consumption</td><td rowspan="7">Non-revenue water</td></tr>
<tr><td>Unbilled unmetered consumption</td></tr>
<tr><td rowspan="5">Water losses $Q_L$</td><td rowspan="2">Apparent losses $Q_{AL}$</td><td>Unauthorised consumption</td></tr>
<tr><td>Customer meter inaccuracies and data handling errors</td></tr>
<tr><td rowspan="3">Real losses $Q_{RL}$</td><td>Leakage on transmission and distribution mains</td></tr>
<tr><td>Leakage and overflows at storage tanks</td></tr>
<tr><td>Leakage on service connections up to point of customer meter</td></tr>
</table>

*Figure 5 IWA Water Balance* (Ziegler, Klingel, Happich, & Mutz, 2011)

#### 1.8.1. Real Losses

Real losses are a big contributor to IWS, since water is physically lost before it can reach the customer. The level of real losses can be analysed through minimum night flow (MNF) measurements and step testing. Real losses can be classified by their (a) location and (b) size and runtime (Morrison et al., 2007).

*Location*

1) **Leakage from transmission and distribution lines** may occur at pipes, joints and valves and usually has high to medium flow velocities and short to medium runtimes.
2) **Leakage from service connections** up to the point of the customer meter are often difficult to detect and therefore have long runtimes.
3) **Leakage and spills from storage reservoirs** are caused by deficient or damaged level controls.

*Size and runtime*

1) **Reported or visual leaks** primarily come from sudden bursts in transmission mains

2) **Unreported or hidden leaks** have flow rates over 250 m/h at over 50 m pressure, due to unfavorable conditions that do not appear at the surface.
3) **Background leakage** has flow rates below 250 m/h at 50 m pressure that do not appear at the surface. Since they cannot be detected acoustically, background leaks are never detected and repaired and leak until the defective part is replaced.

In order to assess and control the level of real losses and manage pressures at the tertiary and secondary network levels, discrete zones are required. These zones are called district metered areas (DMAs). A DMA is typically created by closure of valves or disconnection of certain pipe works in a way that quantities of water entering and leaving the area are metered (Morrison, Tooms, & Rogers, 2007). Appendix P provides a detailed description on NRW and DMAs. The appendix explains design requirements for DMAs, the effect of pressure on NRW levels that are assessed and the different components of NRW and how to solve them. Studies with regards to leakage and pressure management in DMAs often assume these districts to be supplied continuously. Under IWS conditions DMA performance and assessment is different.

#### 1.8.2. Apparent Losses

Apparent losses contribute to IWS for their share of illegal connections. It can contribute or cause IWS since it reduces the cost recovery ratio for the utility, i.e. how much revenues are generated based on the cost price of one cubic meter of water. This prevents the utility to make bigger long-term investments and additional (predictive) maintenance. Furthermore, under IWS conditions customer meter degradation is worse compared to CWS. This results in the meter under registering actual volumes of water that are consumed, which leads to incorrect billing and a loss of revenues.

#### 1.8.3. DMAs under IWS

In literature, studies focusing on DMAs assume CWS as the de facto mode of operation. However, in DMAs with IWS, many customers have private storage tanks. The demand pattern changes at the inlet of the DMA under IWS conditions. For IWS systems, the conditions for an MNF analysis, therefore, do not apply since many customers fill their private reservoirs that at night, causing elevated $Q_{LNC}$. A proposed solution is to supply the DMA with CWS for a number of days until private tanks and reservoirs are filled. This can be observed in flow measurements over a period of days, when the MNF lowers at night and eventually stabilizes (Al-washali, Sharma, Al-nozaily, & Haidera, 2019; AL-Washali, Sharma, AL-Nozaily, Haidera, & Kennedy, 2018). Then, when a steady state is reached and $Q_{NNF}$ can be calculated. This solution, however, is prone to errors, because some customers lack functional floating valves leading to higher $Q_{MNF}$.

### 1.9. Pressure

Intermittent water supply is a problem of pressure since, water cannot be supplied in a distribution network without pressure. Sufficient pressure is required to supply water to customers (service pressure) and to overcome friction losses from the flow of water through the network. High pressures and pressure waves are undesired in water distribution since they lead to higher leakage levels and higher burst rates. Pressure management can be considered to optimize pressures. Thornton et al. defined pressure management as, "*the practice of managing system pressures to the optimum levels of service ensuring sufficient and efficient supply to legitimate uses and consumers, while reducing unnecessary excess pressures, eliminating transients and faulty level controls all of which cause the distribution system to leak unnecessarily*" (Thornton & Lambert, 2005). Simply put, pressure management not only reduces pressure variations but eliminates superfluous pressure from the network and thus lessening leak flow rates and real water losses.

When reducing pressure, the minimum required supply pressure must be maintained at the critical point in the network. This critical point might vary depending on fluctuations in demand or infrastructure changes. Furthermore, negative pressures must be avoided that can occur at times of peak demand of fire flow conditions. Hydraulic modeling is necessary to accurately operate and control pressure strategies in the distribution network. It gives predictive insight about if and where hydraulic bottlenecks can occur. Hydraulic bottlenecks are locations within the network, with very high energy losses in relatively short distances, or locations where there is no or negative pressure. Often at these locations, velocities are either too low or too high. In addition, hydraulic modelling advances and facilitates the decision-making process regarding network extension, operations and maintenance (Ziegler et al., 2011).

### 1.9.1. Pressure Dependent Demand

The most straight-forward way to model distribution networks is demand driven analysis (DDA), meaning customer demands are appended to nodes. Based on these demands and the demand pattern, flows and pressures are calculated. EPANET is widely used WDS simulation software using the DDA mode. DDA is, however, incapable of simulating pressure-deficient conditions properly because it factors fixed demands regardless of pressure variations. The inadequacy of DDA can be remedied by pressure dependent analysis (PDA), where pressure dependent demand (PDD) is considered and nodal outflows and pressures vary with changes in pressure (Germanopoulos, 1985; Giustolisi, Savic, & Kapelan, 2008; Gupta & Bhave, 1996; Walski, Blakley, Evans, & Whitman, 2017; Zheng Yi Wu, Walski, & Bowdler, 2002).

Several methods have been proposed over the years to model PDD. Some authors suggest modelling approaches based on DDA combined with artificial reservoirs ((Ang & Jowitt, 2006; Ozger & Mays, 2003; Todini, 2003). Others proposed approaches based on a PDD relationship (Zheng Yi Wu et al., 2002). The artificial reservoir approach has the disadvantage that it changes the network topology and increases computation times. EPANET can assign emitters to nodes. Emitters are used to model flow through a nozzle or orifice, like sprinklers or fire hydrants but they can also be used to model PDD. However, when pressure turns negative in nodes, it generates flawed data because there is no upper limit for the emitter flow (Todini, 2008; Zheng Y Wu et al., 2009) Others have proposed extensions for EPANET for PDA (Cheung, Van Zyl, & Reis, 2005; Giustolisi et al., 2008; Hayuti, Burrows, & Naga, 2007; J. Muranho, Ferreira, Sousa, Gomes, & Sá Marques, 2014a; Todini, 2008). Muranho et al, have developed a PDD modelling extension for EPANET that also includes physical leakage modelling (J. Muranho et al., 2014a). In this model, leakage and domestic demand are separated and can be modelled independently, i.e. leakage with PDD and domestic demand with DDA. This mode of modelling is required to assess the effects of leakage interventions on a network.

# 2. Problem Definition

## 2.1. Knowledge Gap

In order to avoid the destructive effects of IWS and the high (coping) costs of maintaining an IWS system, a transition should be made to CWS. Different strategies exist to convert to CWS, as shown in figure 6, namely: (1) additional production capacity, (2) additional transmission capacity, (3) real loss reduction & management, (4) apparent loss reduction and (5) water demand management. Currently the conversion is often performed through a top-down i.e. supply-side approach where extra production and transport capacity (1 & 2) is made available. However, this approach does not target the underlying major causes of IWS, namely leakage and pressure inefficiencies. These lead to environmental and energy inefficient use of water resources and production facilities. Strategies 3, 4 and 5 focus on these underlaying causes at distribution and customer level. At this level, studies have focused on water quality aspects under IWS conditions and coping mechanisms of individual customers and how to increase supply equity. However, a bottom up i.e. demand-side approach to convert to CWS through a balanced and sequential restructuring of the distribution network into DMAs, in which NRW and water demand are managed, has not fully been studied.

**Strategies to transition to CWS**

| | Strategy | Explanation | Goal | Effect | Benefit | Negative aspect | Governed by |
|---|---|---|---|---|---|---|---|
| 1 | Increase production capacity | Increases the amount of water going into the system. | Get more water to customer | Increase of Qin | Increases flow through pipes, can avoid stagnant water in 'dead' zones. | Expensive and laborous strategy | Supply side |
| 2 | Increase transmission capacity | Optimizes the distribution of water going into the system | Get more water to customer | Reduction of hydraulic bottlenecks | Reduction in energy costs and reduction in burst frequency on transmission mains. | Expensive and laborous strategy | Supply side |
| 3 | Water demand management | Supply equity optimization. Focus: increase demand satisfaction by increasing duty cycle | Reduction of hydraulic load | Reduction of Qreceived | Also reduces or controls illigal custumer demand from Qapparent lossses. | Potential negative impact on water sales, revenue collection and water quality. | Demand side |
| 4 | Real loss reduction and management | Focusses on reduction of physical losses. Background, burst and transmission leakage | Reduction of hydraulic load | Reduction of Qreal losses | Save water that is lost. Possible additional sales. Defered investments in production and transport capacity | Lack of data and decision support (hydraulic, geographic, sales, etc.) Time consuming strategy | Demand side |
| 5 | Apparent Loss reduction for demand | Focusses on the illigal customer demand that is part of apparent losses. | Reduction of hydraulic load | Decrease Qapparent losses | Additional sales, since customers are added | Very personal engagement with 'new' customers. Time and personel intensive. | Demand side |

*Figure 6 Strategies to Transition to CWS with their respective goals, effects, benefits, negative aspects and from what perspective they are governed.*

## 2.2. Objective

The main objective of this thesis is to develop criteria and describe important conditions for implementing a demand-side approach, a balanced and sequential roll-out of DMAs, to transition to CWS and to test the feasibility of this demand-side approach in a specific case study.

## 2.3. Approach

The approach of this study is twofold:

1) The development of a general methodology to transition from IWS to CWS with a demand-side focus. This methodology is developed based on important criteria and requirements found in literature and by consulting water supply experts and engineering practices.

2) The application of this methodology for the case study in Accra, Ghana. In this case study, three DMAs in GAMA region will be developed, of which one will be monitored and measured. Based on the foregoing, NRW reduction and pressure optimization interventions will be modeled, followed by a description of a comprehensive implementation in the GAMA region. Finally, the applicability of this DMA approach is evaluated for other utilities and countries.

## 2.4. Research Questions

To meet the objectives, the primary question to be answered is:

"*To what extent could a demand-side approach on a DMA level aid in transitioning from an intermittent to a continuous water supply*?"

The following sub-questions will support answering this research question:

1) What is the current situation of the Amasaman, Santo and Amasaman districts in GAMA?
*2)* How can a demand-side method to roll-out DMAs be appropriated, to effectively re-distribute the recovered water in GAMA and progressively increase continuously supplied areas?
3) What investments from a supply side and management decisions are required by GWCL to use the demand-side DMA approach?
4) What is the applicability of this demand-side method in other cities/countries?

## 2.5. Scope

This research focusses primarily on the technical aspects of IWS. Management, governance and social aspects required to implement these technical measures are described and will embedded and implemented through VEI. The study also addresses key concerns for GWCL that need to be addressed when investment proposals are being made regarding the future of water supply in GAMA.

In this research the words methodology and approach are used interchangeably. Both reference to the procedure developed in this research with which the transition from IWS to CWS could be made by using a demand-side focused DMA perspective.

# 3. Description of the Case: Accra, Ghana

A major national utility confronting IWS stress is Ghana Water Company Limited (GWCL). It is a 100% state owned limited liability company, established by Act 461 of 1993 as amended by LI 1648, on July 1st, 1999. GWCL is responsible for the planning and development of water supply systems in urban communities in the country and the design, construction, rehabilitation and expansion of new and existing works.

A broader analysis of GWLC's objects reveals the company is mandated to:

- Provide, distribute and conserve the supply of water in urban Ghana for public, domestic and industrial purposes.
- Prepare long-term plans for the supply of water in consultation with the Water Resources Commissions.
- Promote research relative to water supply and connected subjects for any of the purposes mentioned above.

The company supplies water to over 330,000 connections fed from two sources: the Weija and Kpong treatment plants.

In 2018, GWCL agreed to a Water Operators Partnership (WOP) with VEI under the WaterWorX program of the Dutch Ministry of Foreign Affairs and all ten Dutch drinking water utilities. VEI is tasked with assisting GWCL with their bulk water supply, NRW reduction, transition to CWS and expansion of services in Accra West, East and Tema regions. For this purpose, more than 20 different flow and pressure meters were installed on distribution lines within the greater Accra metropolitan area (GAMA). A control center was created to gather hydraulic data.

GWCL needs the data and analysis from these measurements to improve and expand service levels for their customers by transitioning to CWS. In order to do so, it granted permission to VEI to improve three districts. Adenta (Accra East), Santo (Tema) and Amasaman (Accra West), shown in figure 7.

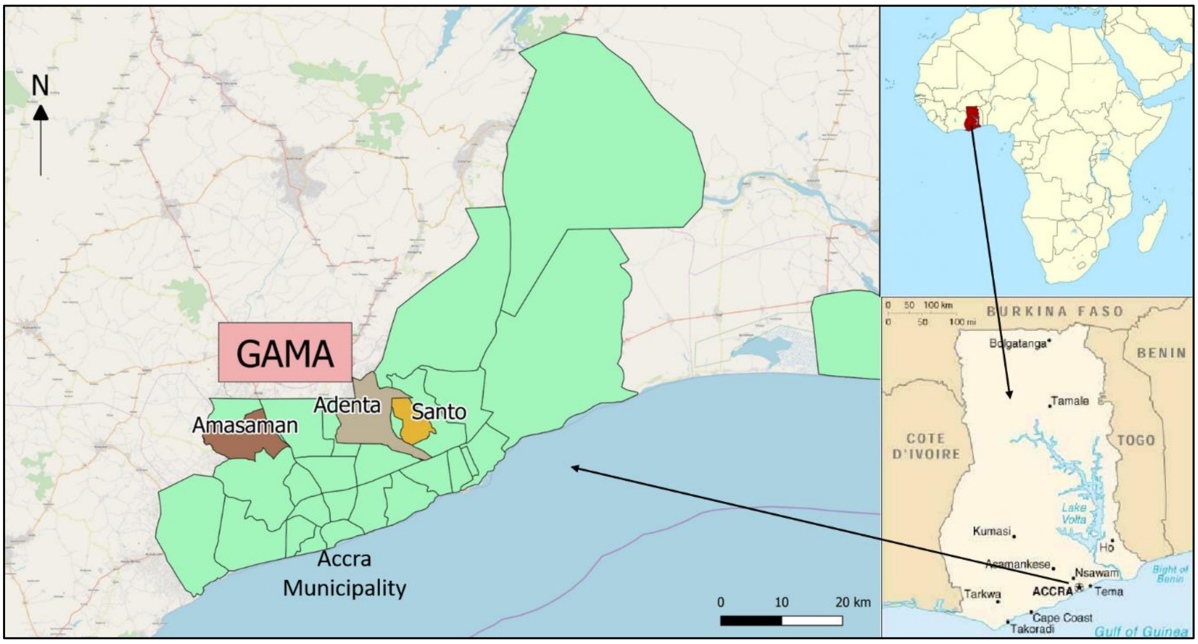



*Figure 7 Location Adenta, Amasaman and Santo districts within GAMA, Ghana. (Open Street Maps and own work, 2019)*

Each district has a district office with a general manager and a distribution officer, as well as a commercial manager and operational staff. Districts are under the responsibility and supervision of the regional offices. Based on what is learned and the improvements made in these three test districts, interventions and best practices can be implemented more broadly in other districts in the nation. An overview of the three districts and GAMA is given in table 2. Santo is the smallest district in terms of size and population but is growing rapidly due to urbanization. Amasaman is bigger in size and population but has a similar number of connections compare to Santo. Adenta is the largest district and is well established but is expanding still.

| | Adenta | Amasaman | Santo | GAMA | |
|---|---|---|---|---|---|
| **No. connections** | 13,367 | 437 | 400 | 337,228 | [-] |
| **Population** | 113,000 | 120,000 | 20,000 | 5,378,000 | [-] |
| **Pop. Growth rate** | 4.2 | 2.3 | 4.6 | 3.6 | [%] |
| **Length network** | 178.0 | 63.7 | 29.2 | 3820 | [km] |
| **Size of district** | 96.5 | 67.4 | 31.6 | 3241 | [$km^2$] |
| **Age of district** | 4.5 | 0.5 | 0.5 | 20-80 | [years] |
| **Water sold** | 224,920 | 10,200 | 11,200 | 7,845,000 | [m3/month] |

*Table 2 Overview of Adenta, Amasaman, Santo and GAMA.*

# 4. Development of the General Methodology

This chapter describes the development of the methodology to transition from IWS to CWS by using a demand-side approach. Since some of the main causes of IWS are real losses and pressure inefficiencies, I developed a framework to describe the causes of IWS from the perspectives of leakage and pressure. This framework then translates into the requirements to determine causes of IWS and interventions to transition to CWS that was developed by myself and VEI. The general methodology is derived from combining the requirements and the framework of IWS causes.

## 4.1. Framework: Types of Intermittent Water Supply

Different types of intermittency can be observed in a water distribution network. These types of IWS are generated by applying mass balances, starting at the DMA level. This mass-balance consists of ingoing and outgoing fluxes. Ingoing fluxes are either the system input volume (SIV) to a district or the production capacity. Outgoing fluxes are the required domestic demand and real losses. The required domestic demand is set by WHO to be 50 L/person/day (WHO, 2017).

The real losses appear in different levels: unavoidable, economical and current. Unavoidable losses can be calculated based on more detailed network information (S Hamilton, Mckenzie, & Seago, 2006; A. O. Lambert, Brown, Takizawa, & Weimer, 1999). Economic leakage levels can be calculated by using the cost of water, the level of real losses and the cost of active leakage control, as initially described by Mckenzie, et al. (A. O. Lambert & Fantozzi, 2005; R. Mckenzie & Lambert, 2002; Pearson & Trow, 2005). The current level of real losses is assessed through minimum night flow measurements. The concepts of unavoidable, economic and current levels of real losses are described in more detail in Appendix P – Non-Revenue Water.

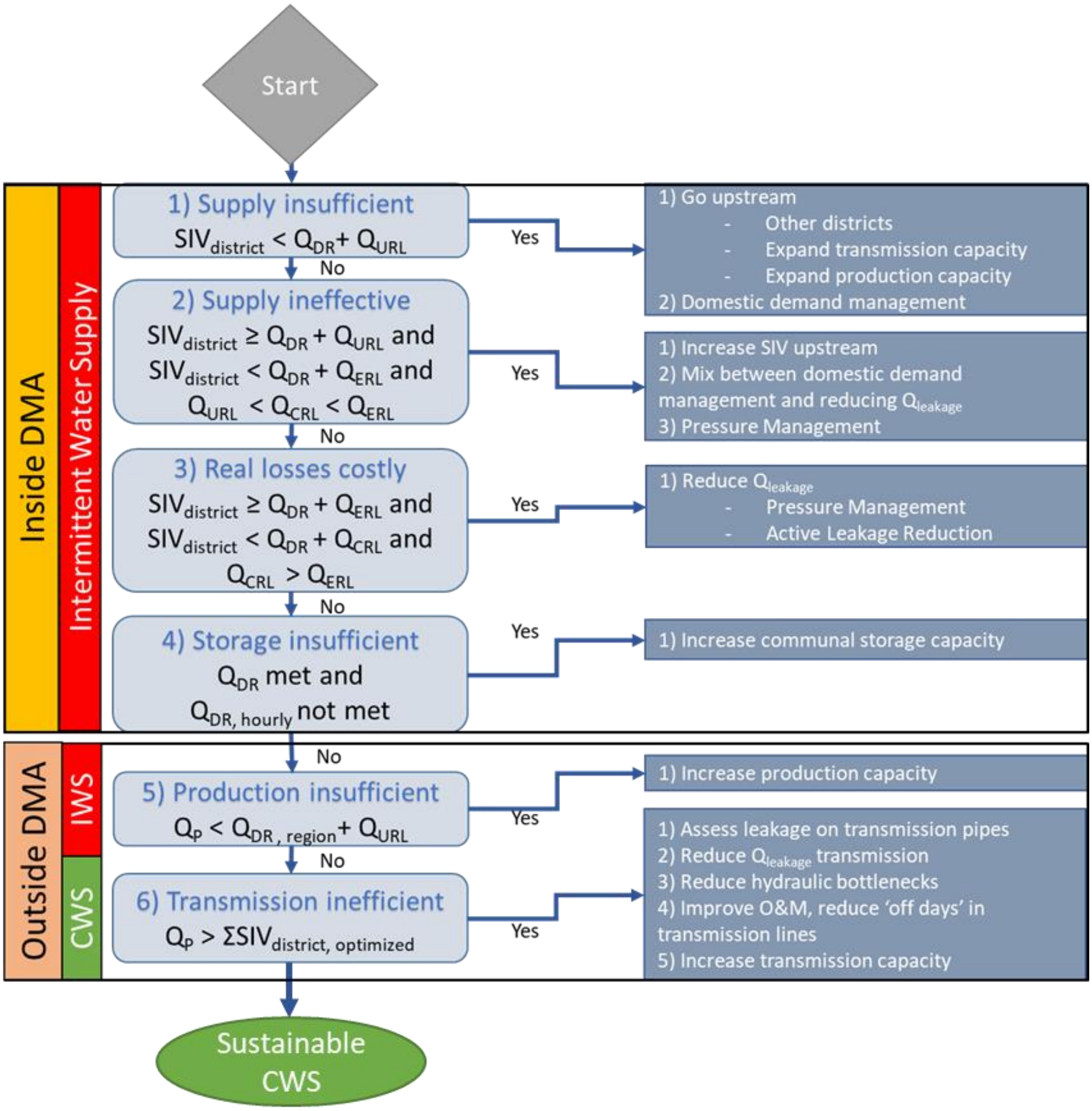
Furthermore, a distinction can be made between daily and hourly satisfied domestic demand. Once the SIV is optimized for each district by applying interventions such as demand management, pressure management, increasing storage capacity or active leakage control, the transmission efficiency can be evaluated. This way six different types of IWS can be introduced for the intermittency assessment from an operational leakage and supply management perspective. Four of these originate inside the DMA, the other two have effect outside a specific DMA. These types of IWS are shown in figure 8. This figure shows a decision tree for what type of IWS one must deal with and the possible interventions to reduce intermittency for this specific type.

The types of IWS are:

1) Supply Insufficient
   There is currently not enough water to satisfy domestic demand and the unavoidable real losses. The district experiences IWS that is caused by upstream conditions. SIV needs to be increased to reduce intermittency. In the meantime, demand management can be used to supply existing water equitable.
2) Supply Ineffective
   There is enough water to satisfy domestic demand and unavoidable real losses. However, current real losses are below economic level of real losses. This indicates that the level of real losses is below the economic beneficial level and therefore it will be more expensive to reduce those compared to when real losses are above economic leakage level. A mix between demand management and real loss reduction interventions needs to be considered. Pressure management is such an intervention.
3) Costly Real Losses
   There is enough water to satisfy domestic demand and the economic level of real losses. However, since the current real losses are higher than the economic level, domestic demand cannot be satisfied. The water lost is therefore ‘expensive’, it could generate immediate revenues when saved. Pressure management and active leakage control are effective interventions to reduce these real losses.

4) Storage Insufficient
   There is sufficient water to satisfy required domestic demand on a daily basis. However, at some points during the day not all required domestic demand can be satisfied. Therefore, additional buffer capacity is required in the form of storage.
5) Production Insufficient
   When considering a whole city or region, the production capacity is insufficient to satisfy the required domestic demand and unavoidable real losses. In this case additional production capacity is required and demand management proposed to supply existing capacity equitable.
6) Transmission Inefficient
   This is the case when the production capacity is lower than the sum of SIV of all the districts after these are optimized with their respective interventions. Transmission capacity is lacking, or bottlenecks have appeared in the network or transmission pipes loose high volumes of water. This can be reduced by reducing leakage on these transmission pipes, resolving hydraulic bottlenecks and increasing transmission capacity.

CWS is warranted once these types of IWS do not occur in the distribution network anymore. Ultimately the current level of real losses is equal to the economic level of real losses throughout the distribution network and customers are served with a service pressure of 15 meters. In that case, the distribution network is highly efficient and effective.




| Legend | | | | | |
|---|---|---|---|---|---|
| $SIV_{district}$ | = system input volume district | [m$^3$/day] | $Q_{URL}$ | = unavoidable level of real losses | [m$^3$/day] |
| $SIV_{optimized}$ | = SIV after interventions in district | [m$^3$/day] | $Q_{ERL}$ | = economic level of real losses | [m$^3$/day] |
| $Q_{DR}$ | = domestic demand required | [m$^3$/day] | $Q_{CRL}$ | = current level of real losses | [m$^3$/day] |
| $Q_{DR, hourly}$ | = domestic demand required | [m$^3$/hour] | $Q_p$ | = production capacity | [m$^3$/day] |

*Figure 8 Decision tree for types of IWS in distribution systems (blue box) and their respective interventions to reduce intermittency and secure CWS.*

The decision tree in figure 8 only focusses on improving supply conditions (CWS) in a DMA and does not show how additional water savings at a DMA level can lead to improved supply conditions in surrounding districts. When these effects are considered, it can lead to more effective, cost efficient and robust optimizing of the water distribution network. Therefore, these effects of interventions should be evaluated per DMA and for multiple DMAs at once from an economic and supply based perspective. To evaluate and appreciate these effects hydraulic modelling is required.

## 4.2. Process to Determine Causes and Interventions

To assess the different types of intermittency in a distribution network and to model interventions correctly, a set of steps can be drawn that are needed to develop the general methodology to transition to CWS.

Firstly, different data is required in terms of flow, pressure, leakage, demand and production capacity. This data can be gathered at different levels (nation, region, city, neighborhood). The accuracy of the decision which intervention is suitable will increase when the assessment is done at the lowest level, i.e. neighborhood, but will also be the most resource expensive and vice-versa. Interventions focused on active leakage control and pressure management require smaller hydraulically isolated areas that are metered. Therefore, the DMA methodology is used, described by the IWA Water Loss Taskforce as well as the methods described by Al-washali, et al. (Al-washali, Sharma, & Kennedy, 2016; Morrison et al., 2007). To obtain the data that is required DMAs can be utilized. Therefore, the step by step requirements are as follows:

1) Input data on:
    a. Network - Customers
    b. Geography
    c. Population
    d. Production capacity
2) Divide water distribution network into districts
3) Isolate districts hydraulically
4) Place flow meters and pressure loggers inside districts
5) Gather data on districts
    a. Flows
    b. Pressures
    c. Demand from billing
    d. Demand pattern
6) Analyze status quo of the districts
    a. SIV
    b. Top-down NRW assessment
    c. MNF
7) Determine bottlenecks for improving supply time, based on figure 8
8) Determine adequate interventions to reduce bottlenecks
9) Determine impact, cost and requirements per intervention
10) Assess impact of intervention by hydraulic and economic modelling
    a. Model domestic demand and real losses with PDD independently
11) Plan and execute interventions
12) Monitor improved state
13) Maintain network, district, metering and update databases regularly

The process of requirements that need to be met to transition to CWS with a bottom up approach are further described in Appendix A.

## 4.3. Approach to Transition to CWS by using DMAs

This paragraph describes what the methodology to transition to CWS from a demand-side approach looks like. The approach is created by combining the types of IWS framework with the requirements described in the previous paragraphs. To make the right assessment on how to transition to CWS for a region, DMAs are obligatory. Their individual performance influences the supply conditions of the whole region. To make the assessment, data is needed from these DMAs to assess their current status. Based on this status, interventions can be proposed which can be hydraulically modelled for individual DMAs as well as for a multiple of DMAs representing a whole region. To develop DMAs, input is required on the current situation of the distribution network, geography of the area and customers and population data.

Figure 9 gives an overview of this approach to transition to CWS by using DMAs. The methodology consists of five sub-sections: input, DMA development, data collection, data analysis and interpretation, and modelling.

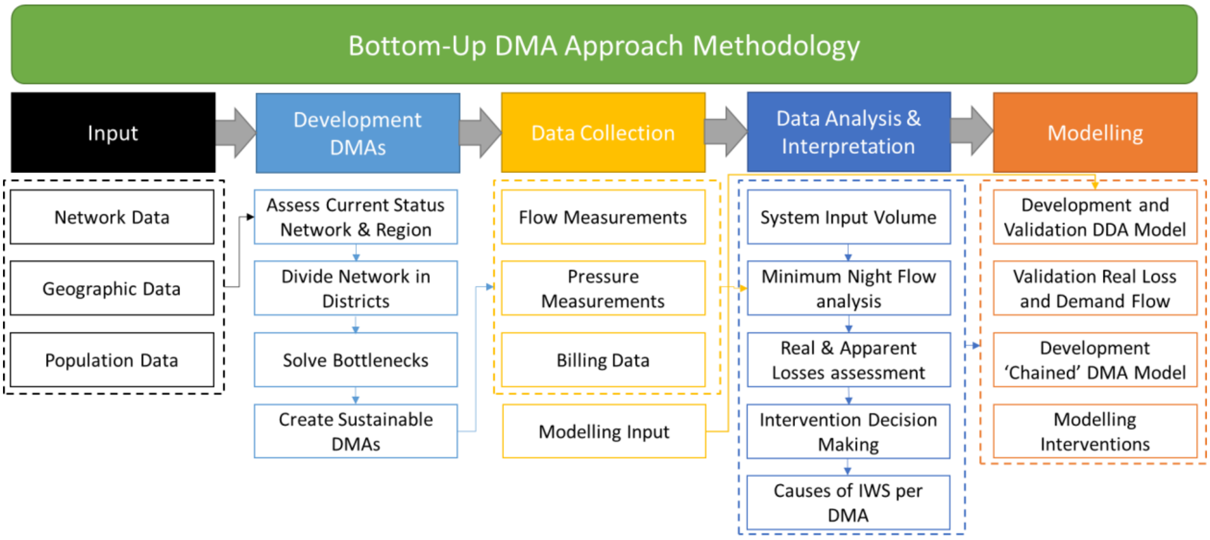



*Figure 9 Overview Bottom Up DMA Approach Methodology*

### Input

For the development of DMAs, input is required. Network data consists of a spatial overview of the distribution network, pipe diameters, valves, flow meters and customer locations. Geographic data is needed in terms of elevation, landscape, physical boundaries such as rivers and other boundaries such as roads, railroads, town centers and buildings. Population data is needed to include the current inhabitants in a certain area, as well as the future growth in terms of population. Developing DMAs based on this information makes them more robust and durable.

### Development of DMAs

To create DMAs according to the methods of the IWS Water Loss Taskforce (Galdiero, De Paola, Fontana, Giugni, & Savic, 2015; A. O. Lambert, 2002; Morrison et al., 2007), the current status of the network and region needs to be assessed. Based on this assessment, the water distribution network area can be divided into districts that are hydraulically isolated. When these districts are developed, some bottlenecks will occur such as boundary disputes, closing valves and end-capping pipes. This needs to be documented well and all stakeholders should be included in this process. Finally, sustainable DMAs can then be established by employing meters and pressure sensors throughout the district which is now hydraulically isolated. Ultimately when a whole region is divided into DMAs that each have their own flows and pressures recorded, the performance of each DMA can be benchmarked in terms of revenues, NRW, pressure, bursts and supply conditions (Fantozzi, Calza, & Lambert, 2009).

### Data Collection

Data from the (newly) developed DMAs is gathered. Flow measurements from the flow meters and pressure measurements from pressure readings throughout the district. Furthermore, billing data is required to eventually calculate NRW levels, described in the next step. Furthermore, modelling input is required to develop an initial hydraulic model. This input consists of assumptions as well as the demand assessment.

**Data Analysis and Interpretation**
Firstly the volume of water used by the district needs to be calculated, the so called SIV. Based on this information and billing data NRW levels can be determined. From pressure and flow readings from a longer period, an MNF can be performed. This indicates the level of real losses in the district. Based on this information the types of IWS can be evaluated based on the decision tree in figure 8. Finally, based on the occurrence of IWS suitable interventions can then be decided upon for the district.

**Modelling**
To assess the effect of the interventions, a hydraulic model is developed of the district. Firstly, this model is developed in DDA where flows and pressures can be validated with the data that was previously collected. The real losses can then be modelled in PDD fashion and can be validated by the results of the MNF analysis. Next, the intervention can be applied and its effect on pressure, flow, supply to customers and leakage reduction be modelled with PDD as well as its economic effect. Finally, the multiple DMAs can be connected, and the impact of their interventions assessed in a chained DMA model for the whole area.

# 5. Materials and Methods used for Application of Methodology

## 5.1. General

The materials and methods that were used to apply the general methodology from the previous chapter to the case in Accra, Ghana is described in this chapter. The methodology for this case study differs in two aspects from the bottom-up DMA approach, shown in figure 9.

Firstly, due to a lack of resources (time, finances, flow and pressure meters and staff), it was not possible to study all the districts of GAMA. Based on the current situations of the districts, the design and engineering of DMAs, and the top down NRW assessments for each district, one district was selected to be developed into a DMA.

Secondly, the overall feasibility of the general methodology was examined. For this purpose, a set of critical requirements was generated, and interviews held with water operators outside of Ghana as well as a sensitivity analysis on the modelling that was performed. These steps are further explained in paragraph 5.5.

Figure 10 shows the methodology applied to this case study including the selection of one district and the feasibility of the general methodology. This figure is used as a support structure for the rest of this chapter. The color-coded themes are described under each header and their respective content in sub-headers when required by the quantity of information in that section.

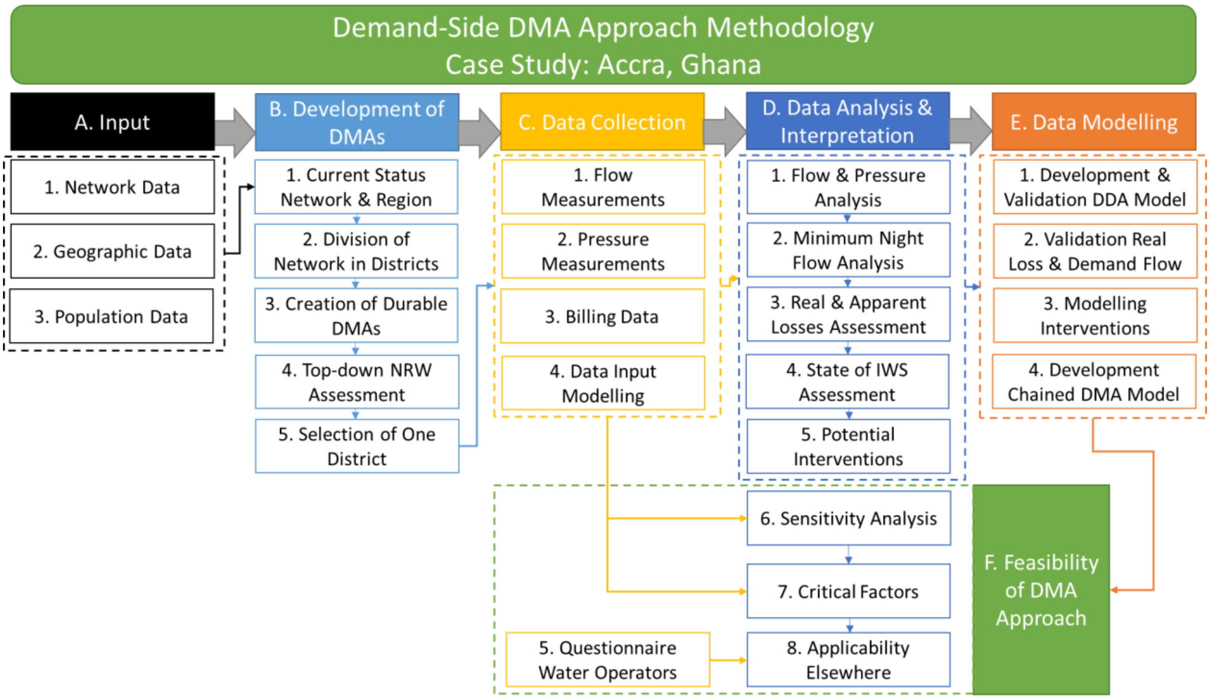



*Figure 10 Bottom Up DMA Approach Methodology for the Case Study of Accra, Ghana.*

## 5.2. Input - A

For the development of DMAs (step B), network, geographic and population data was needed. GWCL provided the most recent network and customer data from the three regions as well as the production capacity from the different production facilities. Network data consisted of pipe locations, diameters, materials as well as valves, reservoirs and flow meters. Furthermore, customer locations were received from GWCL. The network data was provided from a GIS database. The network data did not include the respective elevations of each object. Therefore, geographic data was used. This data included a digital elevation model (DEM) as well as an overview of physical objects such as rivers, roads and railroads. Population data was provided by the Ghana Statistical

Service (GSS) as well as local municipalities. For each district the current population as well as the population growth predictions were received.

## 5.3. Development of DMAs - B

Due to limited resources in terms of available trained staff from GWCL, financial constraints by VEI, limited flow and pressure meters and ultimately time, only one DMA was selected for further research, from the three DMAs that have been developed and engineered by me in collaboration with GWCL field staff.

### 5.3.1. Current Status Network & Region (B.1.)

Firstly, the current situation of the three districts was assessed, with a focus on supply time, network overview, physical boundaries and current demand. This was done through spatial analysis based on GIS and customer data, as well as field visits. During field visits, spatial data was checked and adjusted for when erroneous. This was necessary in areas with varying pipes and the district boundaries. Furthermore, district managers and officers were asked about flow directions, pressures and future extensions, and supply and intermittency levels. After initial boundaries were created for the DMAs observations were made about potential bottlenecks in the network and supply into and within the districts. These observations indicated potential or existing threats to establishing a hydraulic isolated district.

### 5.3.2. Division of Network in Districts and Creation of Durable DMAs (B.2-3.)

Based on the assessment of the current status of the districts DMAs were developed. Since GWCL explicitly stretched that the DMAs were to be sized alongside the current districts as much as possible, this request was honored. The three districts were designed and engineered to become DMAs. GWCLs policy is that the district has a maximum capacity of 10,000 service connections. This is larger than the advised IWA size of 3,000 for DMAs. In order to accurately assess and control pressures and NRW, proposals were made to divide the larger DMA into smaller sized sub-DMAs.

For the engineering of DMAs, QGIS 3.6 was used as explained in the next paragraph. The current network was analyzed as well as (hydraulic) bottlenecks and physical boundaries. Based on the analysis, more optimal boundaries were proposed, and pipes were marked that needed to be removed or end-capped in order to reduce inflow and outflow points in the district. And plans were proposed to ensure the durability of the DMAs, making them future proof.

### 5.3.3. Top-Down NRW Assessment (B.4.)

In order to appreciate and ultimately select one DMA which could be tested, the current situation of the three different districts assigned to VEI by GWCL was assessed. For each district a top-down NRW assessment was performed, as well as field visits to find bottlenecks and discern which district could be used to be studied further. EasyCalc, developed by Roland Liemberger in collaboration with the World Bank, was used to make this assessment.

An annual water balance audit is required to determine the losses per segment of the IWA water balance. This tool is used since it is used by practitioners in the field and does not require much human capacity and is usable in situations with scarce data availability. The standard procedure for this water audit is (Busschel, 2017):

1) Calculate annual system input volume $Q_I$
2) Determine the annual billed authorized consumption $Q_{BA}$ from meter readings.
3) Estimate the unbilled authorized consumption $Q_{UA}$.
4) Calculate authorized consumption, $Q_A$, by adding $Q_{BA}$ and $Q_{UA}$. Total water losses $Q_L$ follow from $Q_I - Q_A$.

5) Estimate apparent losses $Q_{AL}$. Initially, 5% of $Q_{BA}$ is used. This is adjusted after a good analysis is performed.
6) Calculate real losses $Q_{RL}$, by subtracting $Q_{AL}$ from $Q_L$.

The results of the water balance depend highly on accurate measurements and estimates. Therefore, confidence limits below 15% real losses are difficult to achieve (A. O. Lambert, 2003). In order to develop an appropriate water loss reduction strategy with the top-down water balance audit, a bottom-up assessment is required (Charalambous & Hamilton, 2011).

For each of the three DMAs, a top-down NRW assessment was performed by using the World Bank EasyCalc model, further explained in the next paragraph. By doing so, the current level of apparent losses was estimated, and a level of real losses determined. Baghirathan, et al., developed an overview for the procedure of this top down assessment, that also gives an outline of the data required, which can be found in Appendix D (Baghirathan & Parker, 2017).

#### 5.3.4. Selection of One District (B.5.)

Based on the EasyCalc assessment and in discussion with VEI and GWCL, one DMA was chosen to model the transition to CWS by proposing and modelling interventions. Based on the results of the previous steps, Amasaman district was chosen to be modelled further, for the following reasons:

- It had intermittent supply conditions
- It was the district with the highest pressures
- It was available immediately to turn into a DMA
- It had over 450 customers

#### 5.3.5. Tools for Development of DMAs

*QGIS*

QGIS 3.6 Noosa is used for spatial data analysis and design of DMAs, creation of network extensions and demand calculations. Quantum Geographic Information System (QGIS) is an open source and cross-platform software, rapidly developed and steadily adopted over the last 12 years by an international community of developers (Baghdadi, Mallet, & Zribi, 2018). Different plug-ins were used that are supported by QGIS 3.6. Point Sampling Tool version 0.5.2 developed by Borys Jurgiel, QEPANET version 2.04 developed by UNIBZ-UNITN and GRASS 7 version 2.0 developed by QGIS.

*World Bank EasyCalc*

WB-EasyCalc 5.16 is used to assess the leakage components and determine the level of apparent losses as well as real losses. The tool was developed by Liemberger and is regarded best practice when it comes to a top down assessment of the NRW fluxes.

### 5.4. Data Collection - C

#### 5.4.1. Flow Measurements, Pressure Measurements and Billing Data (C.1-3.)

In order to accurately assess and model the current status of Amasaman as well as future interventions, different data was required: Flows and pressure recordings throughout the district, and last month's billing reports. These were indicated in figure 10 with three boxes, here described in one paragraph.

In order to set up the measurements, the district was isolated hydraulically. Flow measurements were performed on the north-west and south-east boundaries of the district to determine the SIV. Two pressure sensors were placed at the ultra-sonic flow meters and four sensors were employed throughout the network. Figure 11 indicates the locations of the ultra-sonic flow meters and six pressure loggers.

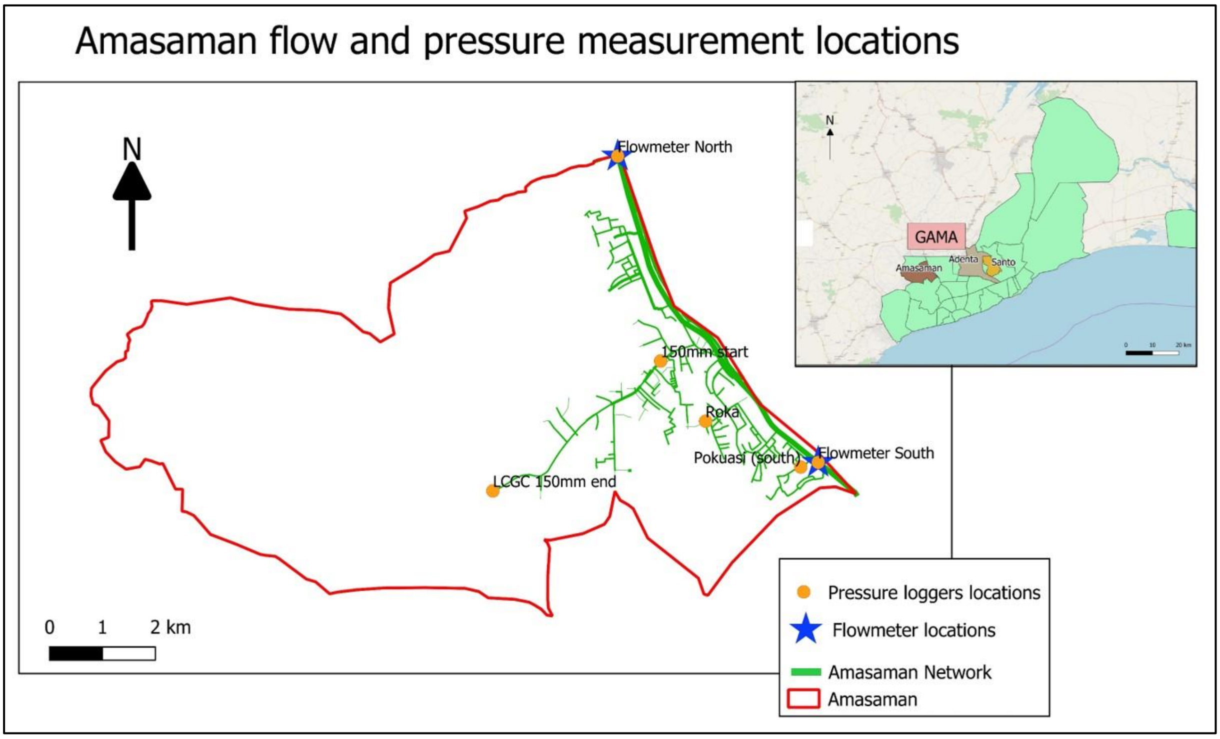



*Figure 11 Amasaman flow and pressure measurements locations (Design by author, 2019)*

For the purpose of accurately carrying out the MNF analysis, the DMA was supplied continuously for 8 days starting from 16th of August at 8:00 a.m. until 23rd August 2019 08:00 a.m. This way the studied area was fully 'saturated', which means all private containers and tanks are assumed to be filled and the readings will show accurate night flows. Pressures were recorded at an interval of two minutes, flow readings at ten-minute intervals. And billing reports from the month of August 2019 were reviewed.

### 5.4.2. Data Input Modelling (C.4.)

To create an initial hydraulic model, different data was required. Customer and network data were received from GWCL, flow and pressure data from field measurements and elevation data from Earth Explorer. QEPANET was used to create the network (pipes and nodes) within QGIS. The demand from the billing was distributed spatially over the customers by using client ID's. Around the network nodes, Voronoi polygons were created to assign demands per node which was done with the Point Sampling Tool. In addition, elevations were assigned to the network nodes with the same tool. The model was then converted to an EPANET input (.inp) file.

**Assumptions**

In order to model Amasaman district, different assumptions had to be made. An overview of these assumptions is given in the table 3. The GAMA Masterplan developed in 2016 was used for some of these assumptions, for other assumptions best practices in engineering were used as well as financial reports from GWCL.

| Domestic use | Value | | Source |
|---|---|---|---|
| Household size | 4.9 | Inhabitants | GAMA Masterplan, 2016 |
| Connections/household | 1.0 | - | |
| Ann. Pop. Growth | 2.1 | % | GAMA Masterplan, 2016 |
| Domestic demand | 132 | LCD | GAMA Masterplan, 2016 |
| (from water vendors | 32 | LCD | GAMA Masterplan, 2016 |
| **Pressure** | | | |
| Min. service pressure | 1.0 | bar | |
| Max. pressure | 8.0 | Bar | |
| **Velocities** | | | |
| Minimum (self-cleaning) | 0.6 | m/s | |
| Maximum | 3.0 | m/s | |
| **If unknown** | | | |
| Non-domestic demand | 10 | % domestic demand | GAMA Masterplan, 2016 |
| NRW | 55 | % SIV | GAMA Masterplan, 2016 |
| Peak factor seasonal | 1.2 | - | |
| Peak factor hourly | 1.67 | - | |
| Total peak factor | 2.0 | - | |
| **Interventions** | | | |
| Min. storage | 8 | Hours of $Q_{domestic\ demand}$ | |
| Max. storage | 12 | Hours of $Q_{domestic\ demand}$ | |
| Reduction repair time | 30% | With 6 extra staff | |
| Reduction reported bursts | 20% | With 5 extra staff | |
| Reduction burst freq. | X % | With X% pressure reduction | |
| **Economics** | | | |
| Marginal water cost | 0.42 | €/m$^3$ | GWCL 2016 report |
| Discount rate | 8.0 | % | |

*Table 3 Overview assumptions used to model district.*

### Domestic Demand

Several demand calculation and estimation methods were used.

- Billing based demand was obtained from the billing data from Amasaman. It turned out that a few (34) connections within the district were billed by another district (Nsawam). Their demand was added to the customers of Amasaman. Due to the relative newness of the district, no clear billing procedures were set in place in the early months, and therefore billing was off for these months. Since July and August, the customer demand started to go to regular monthly volumes. The billing demand was connected to the geographic location of the different customers.
- Scenario based demand was considered an alternative. With the size of households and per capita demand, demands for connections could be calculated. Assumptions from table 9 were used to calculate scenario demands.
- Survey based demand was initiated, but due to the high cost and time consumption, this method was discarded.
- MNF demand based on field measurements uses inflow and real loss measurements and analysis, to determine the domestic demand.

Ultimately MNF based demand was used to model Amasaman district, since it is accurate and includes apparent losses. Apparent losses are not considered in the other demand methods.

#### 5.4.3. Questionnaire Water Operators (F.5.)

To assess the applicability of the bottom up DMA approach a questionnaire was developed and answered by VEI water operators, representing different utilities from four different countries: Zambia (Meijer, Leo at Southern WSC & Nkana WSC), Rwanda (dr. Kabaasha, Asaph at WASAC), Zimbabwe (Ramaker, Toine at Harare Water) and Indonesia (Lagendijk, Vera at PDAM

Tirtawening Kota Bandung). In the questionnaire the methodology to transition to CWS by using a demand-side approach was described as well as the requirements for implementing this approach.

Surveyors were asked whether their utility faced IWS and if they met the requirements for the transition to CWS posed in this methodology. Furthermore, they were asked if they deemed this DMA approach to transition to CWS feasible for their utility based on their experience and expertise. Finally, they were asked for critical boundary conditions for this approach as well as their recommendations or critique on the approach. The full questionnaire can be found in Appendix Q.

#### 5.4.4. Data Collection Tools

*Flow meters*

For the flow measurements two portable Flexim ultra-sonic clamp-on flow meters are used, one F601 and one G601. The flow is logged every 10 minutes for seven days. The flowmeters have an uncertainty of ±1.0% ±0.005m/s. Both flowmeters can operate with an internal battery as well as directly connected to the electricity grid. The battery allows for a maximum of 14 hours of autonomous measurements. Both meters were company calibrated.

*Pressure loggers*

Pressure loggers that were used are the Supco LPT LOGiT. The loggers have a reach of 0 to 500 psi (0 to34.47 bar) and an accuracy of +/- 3 psi (0.207 bar) and a resolution of 0.15 psi (0.01 bar). Pressure is factory calibrated. The loggers operate with a 9V battery and can store 21,500 data points. Pressures were logged at a two minutes interval for seven days.

*Questionnaire*

Google Forms was used to develop and send a questionnaire to different VEI water operators in different countries. This tool collects the questionnaires individually and provides an overview of the different responses.

### 5.5. Data Analysis and Interpretation - D

#### 5.5.1. Flow and Pressure Analysis (D.1.)

Based on the flow and pressure measurements, the SIV was determined according to the following water balance (Eq. 4).

$$SIV = Q_{North} - Q_{South} \tag{4}$$

With:
SIV System Input Volume [$m^3$/day]
$Q_{North}$ Inflow [$m^3$/day]
$Q_{South}$ Outflow [$m^3$/day]

Furthermore, the relationship between flow and pressure throughout the measuring period was analyzed. Finally, the average zonal pressure (AZP) was calculated based on the elevations of the inflow and outflow points of the district, as well as the critical (highest) elevation. Then the weighted average ground level was calculated based on the elevation of service connections, which was retrieved from GIS. The AZP is further used in pressure management interventions and calculation of the infrastructure leakage index (ILI).

#### 5.5.2. Minimum Night Flow Analysis (D.2.)

MNF analysis was performed after the district was fully saturated on 9-18-2019 at 03:00 at the lowest night flow into the district (32,1 $m^3$/hr). The average inflow was calculated, based on which a monthly inflow volume was calculated. Subtracting the billed volume for August from this volume gave the NRW volume for the month of August. A residential night consumption of three liters per connection per hour was used, similar to Fantozzi, et al. (Fantozzi & Lambert, 2012). An N1 factor of 1.2 was used initially, since pipe materials were only PVC and HDPE(Cassa & Van Zyl, 2014; van Zyl, Lambert, & Collins, 2017). After further investigation, N1 was calculated for the emitter coefficient as 0.983 based on Equation 6 described in the next paragraph and this value was used for the final MNF analysis. From the moment of MNF, leakage flow can be calculated for the day, considering pressures. An introduction into leakage modelling is required to understand the formulae used to calculate this leakage flow.

### 5.5.3. Real and Apparent Losses Assessment (D.3.)

A leak in a pipe can be considered an orifice. The hydraulics of orifices are well understood and are described by Torricelli's Law in Equation 5:

$$Q = C_d A\sqrt{2gh} \tag{5}$$

With:

| | |
|---|---|
| Q | flow rate through orifice [$m^3/s$] |
| $C_d$ | discharge coefficient [-] |
| A | orifice area [$m^2$] |
| g | acceleration due to gravity [$m/s^2$] |
| h | pressure head [m] |

Where the discharge coefficient accounts for energy losses and jet contraction.

Experimental research has shown that the discharge in orifices varies greatly with pressures depending on the pipe material, called the Fixed and Variable Area Discharges (FAVAD) principle, described in the following relationship (Schwaller & Van Zyl, 2014) (Eq. 6):

$$\frac{Q_i}{Q_{MNF}} = \left(\frac{P_i}{P_{MNF}}\right)^{N_1} \tag{6}$$

With:

| | |
|---|---|
| $Q_i$ | leakage rate in DMA at time *i* [$m^3/h$] |
| $Q_{MNF}$ | leakage rate in DMA at time *MNF* [$m^3/h$] |
| $P_i$ | average zonal pressure (AZP) in DMA at time *i* [m] |
| $P_{MNF}$ | average zonal pressure (AZP) in DMA at time *MNF* [m] |
| $N_1$ | leakage exponent [-] |

Where the leakage exponent is dependent on the flexibility of the pipe material.

While orifice equation Eq. (4) predicts the leakage component to be 0.5, values as high as 2.9 have been reported in field studies, although the vast majority of leakage components are between 0.5 and 1.5 (Farley & Trow, 2003). A significant proportion of background leakage can consist of transitional flow, and thus have a leakage coefficient above 0.5 (Van Zyl, 2014). Furthermore, the type of material influences the leakage component significantly and therefore coefficient is often assumed as 1.0 (Morrison et al., 2007). The zonal night test is used to determine leakage exponent $N_1$, which is affected by changing pressures inside the DMA. Therefore, it is only possible to determine the exponent while $Q_{LNC}$ is minimal and $Q_{MNF}$ in the DMA almost equals the leakage rate.

In order to calculate a daily leakage rate, a night-day factor (NDF) is introduced (Eq. 7 & 8).

$$Q_{RL} = Q_{NNF} * NDF \tag{7}$$

$$NDF = \sum_{i=0}^{24} \left( \frac{P_i}{P_{MNF}} \right)^{N_1} \quad (8)$$

With:

$Q_{RL}$ Real losses inside DMA [$m^3$/day]

$Q_{NNF}$ Net night flow in DMA [m3/h]

NDF Night Day Factor [h/day]

Apparent losses are then determined as follows (Eq. 9):

$$Q_{AL} = NRW - Q_{RL} \quad (9)$$

With:

$Q_{AL}$ Apparent losses inside DMA [$m^3$/day]

NRW Non-revenue water inside DMA [$m^3$/day]

$Q_{RL}$ Real losses inside DMA [$m^3$/day]

### 5.5.4. State of IWS Assessment (D.4.)

Ultimately, for each district the state of IWS can be assessed. The assessment can be performed based on the decision tree shown in figure 8 and explained in the previous chapter.

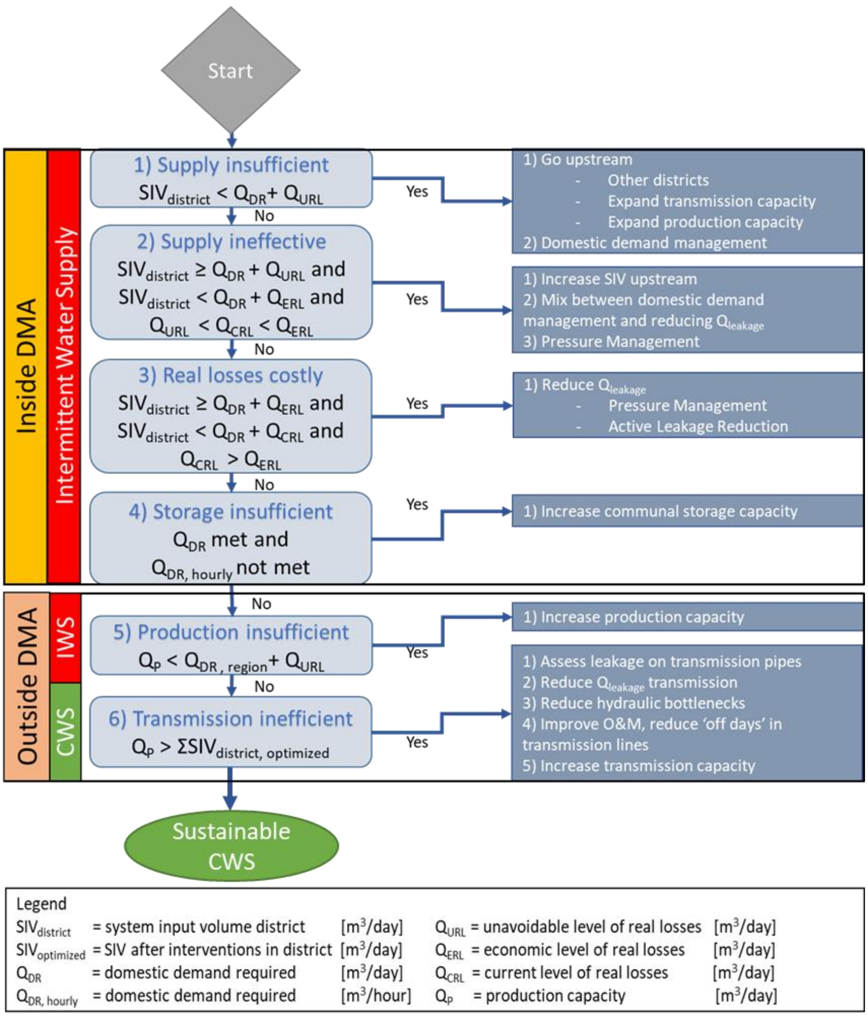



*Figure 8 Overview different Causes of IWS per DMA and their Solutions.*

### 5.5.5. Potential Interventions (D.5.)

In order to reduce intermittency and transition to CWS, several interventions are proposed when following the decision tree in figure 8. The interventions focus on pressure management (PM), creating additional storage reducing real losses (RLR) or management of domestic demand (DMM). Reducing the level of apparent losses greatly influences the cost recovery of the water utility but will not save water and is therefore not considered an intervention. Furthermore, creating additional production capacity is an intervention to increase supply conditions in the network.

**Pressure Management**

When pressure management is applied, pressure reduction *and* pressure increase are considered. In the case of a lack of pressure on the distribution network, booster stations are used to increase pressures. This is done with variable speed pumps (VSP), because it enables the required daily pressure patterns to be set and optimal pressure to be delivered.

In the case of too much pressure, pressures can be reduced with pressure reducing valves (PRVs). This way the energy that the water contains is throttled. PRVs are to be placed at the entry points of the district, at offtakes from the transport main.

It is important to note that PRVs are not the optimal solution long term, due to the wear and tear on the valve as well as the high cost to supply this energy to the system in the first place and then throttle it. A more long-term solution would be the construction of reservoirs, in the case when there is a direct offtake from the transmission line. With different reservoirs the pressures can be better managed.

**Additional storage**

Water storage has many benefits. It can compensate for fluctuations in water consumption during the day, when water is delivered at a constant rate. It acts as a provision of emergency reserve against interruptions due to mechanical and electrical failures or shut down of supply mains for repair or maintenance. It equalizes pressures within the distribution system as well as achieving stabilization of pumping heads. In IWS systems where customers have private tanks, creating additional storage can enhance the control of water demand by the water utility. Creating water storage for a district is not beneficial in case the district is supplied from a reservoir outside of the district by gravity.

The reservoir should be placed in such a way that it can preferably supply the district under gravity. If this is not possible, a booster pump can be used to increase the head. When there are big fluctuations in elevation, each elevation zone should have its own offtake from the reservoir with each having a PRV to operate under optimal pressures.

**Real loss reduction**

Real loss reduction measures include Active Leakage Control (ALC), Speed and Quality of Repairs and Spare Parts (SQR) and Asset Management (AM). ALC focusses mainly on unreported bursts, SS emphasizes reported bursts as well as background bursts and AM focusses on background leakage. To apportion the right measure(s), the aforementioned leakage components need to be appraised. Each measure has its unique costs, KPI, parameters and has different impact. Generally, the cost of the total measures is considered in terms of extra staff that needs to be employed, material costs and equipment costs to carry out the measure. For ALC the economic intervention frequency (EIF) can be determined, once the intervention costs are known, as well as the rate of rise.

**Domestic Demand Management**

Domestic demands can be managed in case there is too little water to supply to all the customers after or while real losses are addressed. Although different solutions exist to manage domestic demands, pressure management is very effective, as shown in the case study of Cape Town (Loubser, 2019).

**Additional Production Capacity**

Extra production capacity can be generated by extending or building additional treatment plants. Based on the assessment from the GAMA Masterplan, both Weija and Kpong treatment plant capacity can be extended (GAMA, 2016). Generating additional production capacity should be the last solution to resolve to, since the whole reason for this study is to have a demand-side approach where real losses and pressure inefficiencies are confronted first as opposed to the supply-driven approach.

#### 5.5.6. Sensitivity Analysis (F.6.)

A sensitivity analysis was conducted for the main parameters that were used to calculate the real loss flow for Amasaman District on which the EPANET leakage model was calibrated. These parameters were: average inflow, N1 factor, night consumption and residential leakage. Since the probability distribution of each of the parameters is unknown, the minimum and maximum values were used. The minimum and maximum values for each parameter were obtained differently. The average, minimum and maximum inflow was calculated based on the measurements from a full week. The leakage exponent N1 and night consumption values were obtained from the studies done by Lambert and Fantozzi (Fantozzi & Lambert, 2012; A. Lambert, 2001). Three liters /connection /hour was used as the night consumption and one and five liters/connection/hour as the minimum and maximum values respectively. The residential leakage values were obtained by assuming either 25% (minimum value), 50% (used value) or 75% (maximum value) of the connections have leakages of 20 L/hour. Standard deviations were calculated based upon the variance between the used real loss flow and the real loss flow influenced by the minimum and maximum value of each parameter.

#### 5.5.7. Critical Factors (F.7.)

Based on the available literature on transitioning to CWS, the results from the questionnaire with water distribution and leakage experts from VEI, field visits to districts in Accra and the sensitivity analysis a set of critical factors to make the transition to CWS with a demand-side DMA approach was obtained. These factors were divided in different themes; availability, know-how and training, and management aspects: willingness and mandate. Furthermore, critical factors that are required for implementing this approach at a DMA level are also described, which focus mainly on the availability of specific data.

#### 5.5.8. Applicability Elsewhere (F.8.)

The applicability of the DMA approach was assessed for other countries and utilities based on a list of criteria and requirements that was developed. WaterWorX Water Operating Partners (WOPs) and NRW experts were asked their opinion about the approach and if it would be applicable to their situation and why or why not. Furthermore, the results form the sensitivity analysis were included to assess the applicability in a different place. The result of this item is a conclusion, drawn from the previously mentioned input.

### 5.6. Data Modelling - E

For the purpose of modelling the water supply of Amasaman district accurately different software was used. The next chapter describes the reasons this software was used, its boundary conditions and requirements and advantages and disadvantages. The sub-headers of this paragraph do not follow the boxes shown in figure 10, since different software did not work properly. Therefore, an additional experiment had to be set up and a decision had to be made which software was going to be used, based on the results of this experiment.

#### 5.6.1. Development and Validation DDA Model (E.1.)

First a hydraulic model for the distribution network of Amasaman district was generated using GIS data on pipelines. Pipe diameters were assigned including roughness factors and pipe material. Since all pipes were either PVC or HDPE, a roughness factor of 0.0048mm was used. Elevations were assigned to the nodes in QGIS by using the digital elevation model (DEM) and Point Sampling Tool and translated into an EPANET .INP file. Initially, billing demand was assigned to the nodes as explained in 'Data Input', but due to the unknown level of apparent losses a water balance was used to account for the district demand, based on Equation 10. The demand was distributed equally over all nodes, since no reliable data was available to assign demands otherwise (Eq. 10 & 11).

$$\sum Qnode = Q_{in} - Q_{out} \tag{10}$$

$$Qnode = \frac{\sum Qnode}{137} \tag{11}$$

Due to changing flow directions at the southern flowmeter, a reservoir was used to simulate pressures in the South, this way the reservoir could fill or empty depending on its state. The head and head pattern of the reservoir were assigned based on the pressure reading in the south and elevation of the pressure logger. The northern flowmeter was modelled as a node with negative demand, with a demand pattern based on the flowmeter measurements. The elevation assigned was the static head. Within the network, the nodal demand was assigned to each node according Equation 11 and a demand pattern for each hour $t_i$ was assigned according to Equation 12.

$$Demand\ pattern\ t_i = \frac{(Q_{in} - Q_{out})_i}{(Q_{in} - Q_{out})_{average}} \tag{12}$$

The model was validated by inserting calibration data. The data used were head and pressures at the measuring points as well as flows at the flowmeter locations. The validated model for DDA was then used as input for the PDD model.

#### 5.6.2. Validation Real Loss and Demand Flow (E.2.)

For the PDD model, two types of PDD were considered: nodal consumptions and leakage. For nodal consumption the following relationship holds (Eq. 13) (J. Muranho et al., 2014a):

$$q_z^{avl}(P_z) = q_z^{req}\ x \begin{cases} 1 & P_z \geq P_z^{ref} \\ \left(\dfrac{P_z - P_z^{min}}{P_z^{ref} - P_z^{min}}\right)^a & P_z^{min} < P_z < P_z^{ref} \\ 0 & P_z \leq P_z^{min} \end{cases} \tag{13}$$

With:

| | |
|---|---|
| $q_z^{avl}$ | available demand at node *z* [m³/h] |
| $q_z^{req}$ | requested demand at node *z* [m³/h] |
| $P_z$ | pressure at node *z* [m] |
| $P_z^{ref}$ | reference pressure to satisfy requested demand at node *z* [m] |
| $P_z^{min}$ | minimum required pressure at node *z* [m] |
| a | PDD exponent (typically a = 0.5) [-] |

The pressure leakage relationships is as follows (Eq. 14)(J. Muranho et al., 2014a):

$$q_k{}^{leak}(P_k) = \begin{cases} \beta_k l_k (P_k)^{\alpha_k} + C_k (P_k)^{\delta_k} & P_k > 0 \\ 0 & P_k \leq 0 \end{cases} \tag{14}$$

With

| | |
|---|---|
| $q_k^{leak}$ | total leakage along line *k* [m³/h] |
| $P_k$ | pressure in line *k* [m] |
| $\beta_k$ | pipe deterioration parameter [m/h] (initial value = $10^{-7}$) |
| $l_k$ | length line *k* [m] |
| $\alpha_k$ | leakage exponent ($\alpha_k$) |
| $C_k$ | burst leakage parameter [m²/h] (Toricelli's Law) [m²/h |
| $\delta_k$ | burst leakage exponent [-] (Toricelli's Law) |

An initial pipe deterioration parameter of $10^{-7}$ m/h is used but should be set by calibration according to the pipe condition within the network. The leakage component $N_1$ is used for $\alpha_k$ and should be between 0.5 and 2.5 according to Lambert (A. Lambert, 2001). And the burst parameter should be between 0 and 1.0 and a burst exponent between 0.5 and 1.0 (Franchini & Lanza, 2014). For an in depth description of the composition of pipes, nodes, heads and flows, see Todini and Giustolisi (Giustolisi et al., 2008; Todini, 2003).

The leakage volume is calculated over the pipe length and is distributed to its end nodes according to Equation 15.

$$q_i{}^{leak} = \frac{1}{2}\sum_k q_k{}^{leak} \tag{15}$$

With $q_i^{leak}$ representing total leakage at node *i* [m³/h], where *k* iterates over all pipes connected to node *i*.

#### 5.6.2.1. WaterNetGen

WaterNetGen, developed by Muranho, et al. uses the above-mentioned parameters to model leakage pressure dependently (J. Muranho, Ferreira, Sousa, Gomes, & Sá Marques, 2014b). A pressure lower bound of 98.064 kPa (10m) was set, as well as a reference pressure threshold of 100%. This indicates the level of demand that is met at the lower bound pressure.

In order to accurately model leakage, leakage flow calculated with MNF analysis was used. The nodal demands were adjusted, only representing domestic demand and apparent losses. For the purpose of retrieving the leakage flow, a water balance was set up according to Equation 16 and 17.

$$Q_{leakage}{}^{modelled} = Q_{North} - Q_{South} - Q_{Domestic\ Demand\ \&\ Apparent\ Losses} \tag{16}$$

$$Q_{leakage}{}^{modelled} = Q_{leakage}{}^{calculated} \tag{17}$$

Where the modelled leakage flow [m³/h] should fit the calculated leakage flow from the MNF analysis. This was done by adjusting the background and burst leakage parameters. However, the model did not respond correctly to the change in parameters and the modelled leakage flow could not be fitted to the calculated leakage flow. Therefore, an investigation and experiment were performed with different hydraulic modelling software to see whether one was capable of modelling leakage flow pressure dependent.

While (whilst) modelling the domestic demand and real losses pressure dependently, it turned out that WaterNetGen did not function properly. Coefficients and exponents needed to be set for both burst leakage and background leakage. While trying to fit the modelled real losses to the calculated real losses by changing these parameters, the pressure dependency did not change. It was observed that the exponents for both burst and background leakage were not functioning properly. Indeed, while trying to model either burst leakage or background leakage by turning of the other parameters, the outcome did not change accordingly. For an overview of these results, see Appendix H. Therefore, the software was tested for the most basic pressure dependent model, based on (Walski et al., 2017). The detailed results of the test can be found in Appendix G.

#### 5.6.2.2. Pressure Dependent Demand Experiment

A basic pressure dependent demand model was set up, in accordance with Walski, et al. (Walski et al., 2017). A reservoir is constructed with an elevation of 100m and a DN100mm, 1000 km pipe with a roughness of 0.0048mm (Darcy Weisbach) is connected to a node with a demand of 10 L/s. See figure 12 for an overview of the set up. The elevation of the node is the only variable in this experiment, starting with an elevation of 60m and increasing with intervals of 10m, until elevation exceeds reservoir elevation. This experiment was run in EPANET 2.0, the recently released EPANET 2.2, EPANET's extension WaterNetGen and Bentley's WaterGEMS. First the DDA was performed, after which PDA was executed. Flows could be calculated for the PDA experiment according Equation 13 and therefore the PDA model could be validated.

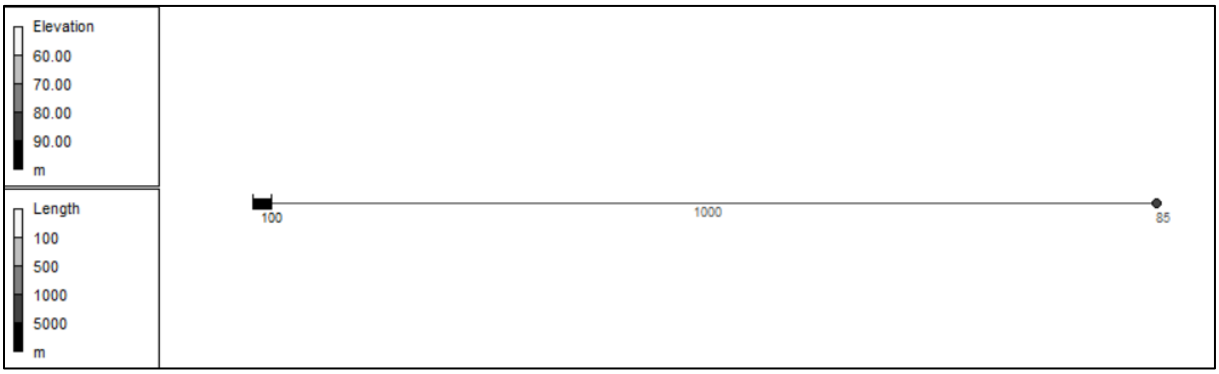



*Figure 12 PDD test experiment set up. Reservoir elevation = 100m, Pipe length= 1000m, Diameter=100mm, Roughness=0.0048mm, Base demand=10L/s, Node elevation=85m.*

Then, assessment was made for the above-mentioned software on leakage modelling. The requirement was to have the ability to split total demand in domestic demand (incl. apparent losses) and real losses. And to see the effect of interventions independently for both domestic demand and real losses.

Testing the software gave the following results, see table 4. EPANET 2.0 does not have a PDD module. The other software packages do have PDD functionality. When applying the PDD test, WaterNetGen did not function according to PDD theory, but was the only software that could model leakage pressure dependently and separate leakage flow from domestic demand. Ultimately, no software passed the leakage modelling test.

| Software | DDA | PDD | Modelled with? | Passed PDD test? | Models leakage pressure dependent? | Real losses separated from domestic demand? | Passed leakage modelling test? |
|---|---|---|---|---|---|---|---|
| Epanet 2.0 | yes | no | yes | n.a. | no | no | n.a. |
| Epanet 2.2 | yes | yes | yes | yes | no | no | n.a. |
| WaterNetGen | yes | yes | yes | no | yes | yes | no |
| WaterGEMS | yes | yes | yes | yes | no | no | n.a. |
| Synergi | yes | yes | no | n.a. | no | no | n.a. |

*Table 4 Overview of hydraulic software investigated. (n.a. is not available)*

Since none of the investigated software was able to model real losses pressure dependently, while modelling the domestic demand separately, the best possible alternative was to use EPANET's emitter function to model real losses.

#### 5.6.2.3. EPANET Emitters

Emitters are devices associated with junctions that model the flow through a nozzle or orifice that discharges to the atmosphere. The flow rate through the emitter varies as a function of the pressure available at the node, see Equation 18 (Rossman, 2000):

$$q = C \cdot P^{\gamma} \tag{18}$$

With

q flow rate [m³/h]
P pressure [m]
C discharge coefficient [m²/h]
γ pressure exponent [-]

For nozzles and sprinkler heads γ equals 0.5. The equation is consistent with the FAVAD theory, where the pressure exponent mainly depends on the predominant pipe material (Cobacho, Arregui, Soriano, & Cabrera, 2015).

Emitters are used to model flow through sprinkler systems and irrigation networks. They can also be used to simulate leakage in a pipe connected to the junction (if a discharge coefficient and pressure exponent for the leaking crack or joint can be estimated) or compute a fire flow at the junction (the flow available at some minimum residual pressure). In the latter case one would use a very high value of the discharge coefficient (e.g., 100 times the maximum flow expected) and modify the junction's elevation to include the equivalent head of the pressure target. EPANET treats emitters as a property of a junction and not as a separate network component (Rossman, 2000).

Emitter values were calculated at the time of MNF and an hour before MNF, since values are required to solve for the emitter value, on a log scale plot, following Equation 19.

$$\log\left(q_{MNF} - q_{night\ consumption}\right)_t = \log C + N_1 \cdot \log(P_t) \tag{19}$$

The emitter coefficient *C* was then distributed over the nodes according to its connected pipe length. The pipe length connected to each node was calculated through QGIS using Voronoi polygons for each node and calculating the total pipe length in each polygon. A 24h simulation was performed after which the total modelled leakage is compared to the total calculated leakage. If the difference was bigger than 0.5% of the calculated leakage flow a new iteration was performed. The coefficient was then modified accordingly ($C_{node\ leakage}^{h+1}$) until the convergence criteria was satisfied. Figure 13 shows this iterative process.

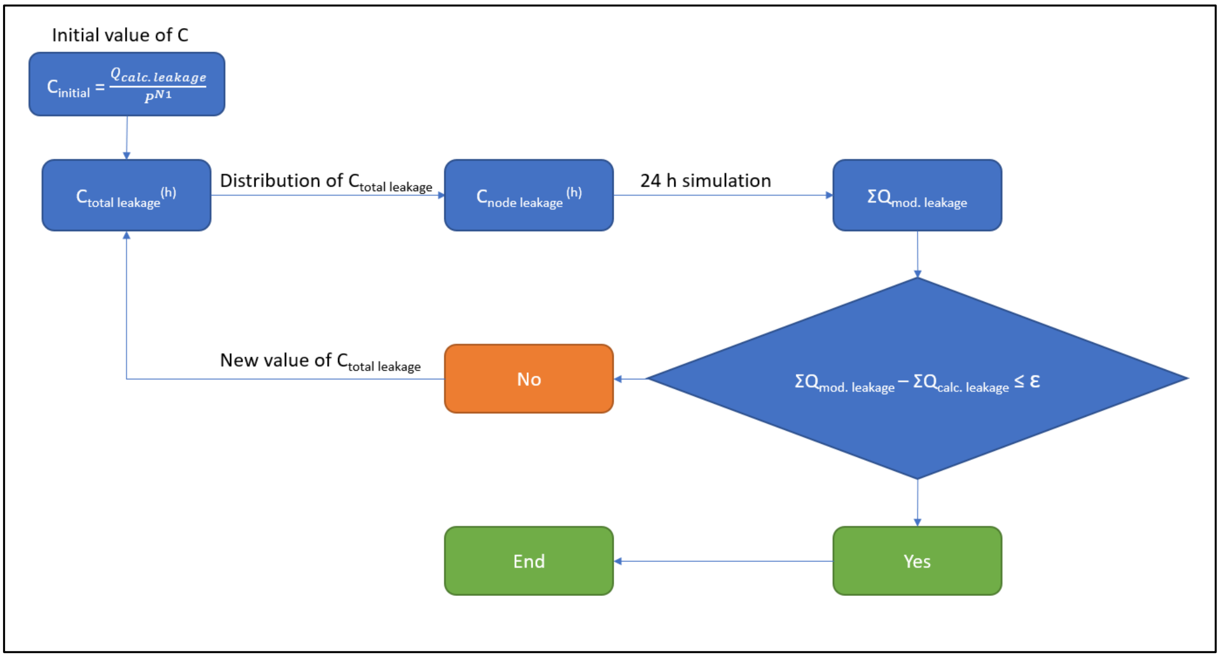



*Figure 13 Iteration procedure regarding distribution of emitter coefficient over nodes.*

#### 5.6.2.4. Validation Real Loss and Demand Flow – Part 2

Initially, the model was run without base demand, this way it could be observed if leakage was modelled. After leakage was modelled, the base demand was adjusted to the domestic demand and apparent losses. A new demand pattern was assigned, that corresponded with this nodal demand.

For the purpose of accurately modelling the impact of pressure management, the hydraulic model was divided into a transport part and a distribution part, as shown in figure 14. The demand and leakage on the transport main were set to zero and the demand and leakage within the district were adjusted accordingly. The total pipe length was now distributed only over the nodes within the district. The model is validated when the difference between the model and measured or calculated values is <5% of the average measured/calculated value. After this last validation step, interventions could be modelled, and their impact calculated.

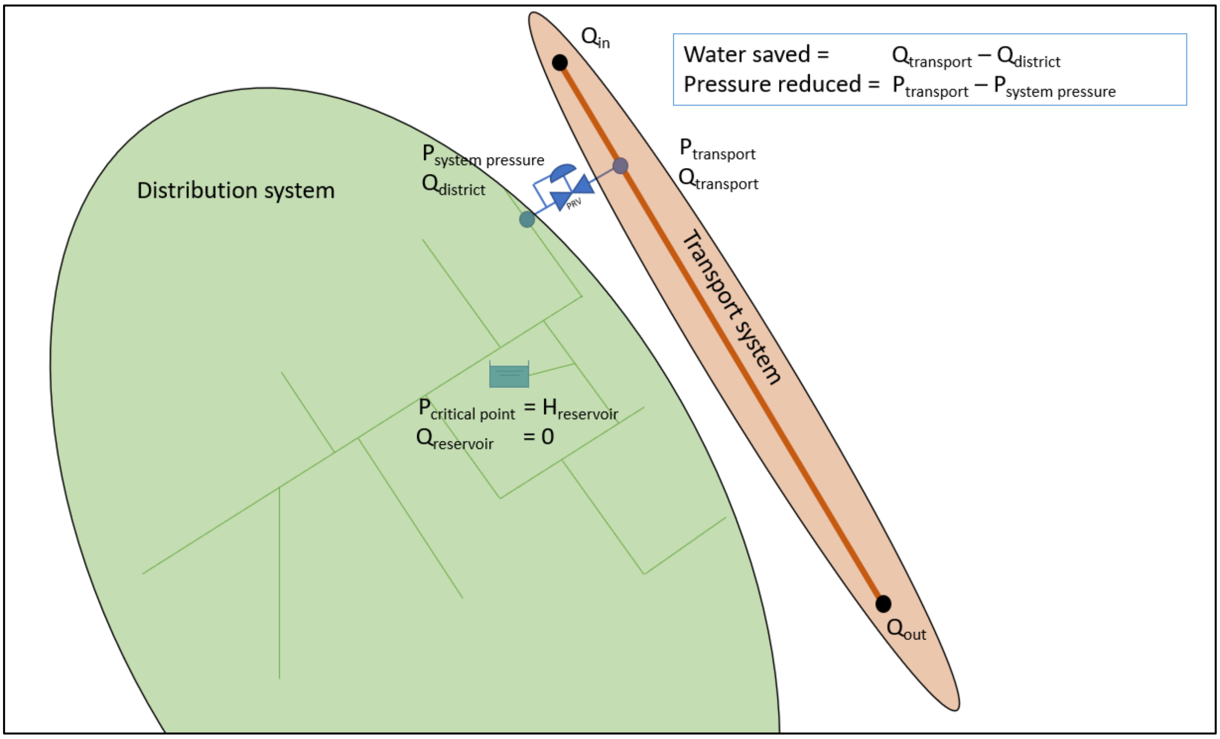



*Figure 14 Modelling transport & distribution system with reservoir. Connected with one pipe (PRV) and two nodes 1) PRV node within district 2) PRV node transport system.*

### 5.6.3. Modelling Intervention: Pressure Management (E.3.)

To model the effect of pressure management by placing a PRV, use was made of the previous validated model. The system service pressure, the pressure required to serve the district with at least 10m. pressure at each point within the District was determined. First, the critical pressure point in the district was assessed and an artificial reservoir was constructed at this node. The elevation of the reservoir was set to the elevation of the node plus the minimum required service pressure of 10m. The transport and distribution systems were isolated by closing pipes Pi75 and Pi130. At the PRV location, a node was constructed with negative demand (Node 72). The demand was set to equal the sum of the domestic demand in the district, the apparent losses and real losses. The demand pattern of domestic demand and apparent losses was used. Iteratively, the $Q_{PRV\ in}$ was adjusted until $Q_{reservoir}$ was zero, representing no in- / outflow from the reservoir and therefore a stable head equal to the minimum service level in the critical node. The adjustment in demand represents the reduction in real losses, with a pressure dependent demand.

The pressure pattern of the PRV node represents the system service pressure. The difference in pressure pattern between the transport system node and PRV node is the pressure reduction that can be applied with a PRV. Based on this pressure reduction pattern, an optimal PRV was selected and leakage volume reduction was calculated after which a net present value calculation was performed to indicate the cost effectiveness of the intervention.

### 5.6.4. Development of Chained DMA Model (E.4.)

Based on the DMA approach, a transition strategy to CWS could be developed for the whole of GAMA. A model was developed in QGIS, representing all the districts within GAMA and includes the transport mains (> DN 400mm pipes). This model was then converted and transferred to EPANET. For an initial assessment, the EasyCalc tool was used to determine different key parameters for each district. District managers and operators were asked to give an initial estimate on the performance and status of their district. The districts were then selected based upon pressure, supply time, leakage volume and the ratio of apparent losses vs. real losses. For each district a multi-criteria analysis can support decision making for suitable interventions, which is further explained in Appendix R.

#### 5.6.5. Data Modelling Tools

*EPANET 2.0*

EPANET 2.0 is used to model the WDS hydraulically. It is an open source software, developed by the U.S. Environmental Protection Agency. It can perform extended period simulation of hydraulic behavior within pressurized networks (Rossman, 2000). EPANET is widely applied in practice, as well as scientific research.

*WaterNetGen*

WaterNetGen version 1.0.0.942 (05-27-2015) is used for to perform PDD calculations in the EPANET environment. It is an EPANET extension for automatic WDS models generation and PDA. And was developed by Muranho, et al. (João Muranho, Ferreira, Sousa, Gomes, & Marques, 2012).

*EPANET 2.2 Beta*

EPANET 2.2 Beta is an updated version of EPANET 2.0 and includes a pressure dependent demand model as well as the normal DDA and it has click-wheel support for map zoom. The beta version was launched as of August 2019.

*WaterGEMS*

Bentley developed an advanced water distribution analysis and design software called WaterGEMS. Version 10.0.2 (2018) was used. It works for both DDA as PDD. It was developed by Wu, et al. (Zheng Y Wu et al., 2009).

# 6. Results

This chapter describes the results from applying the methods, shown in figure 9 for the case of Accra, Ghana. The chapter is divided into the same color-coded themes from this figure: development of DMAs, data analysis and interpretation, data modelling and feasibility of the DMA approach. The code in brackets indicates the specific subtheme from this figure.

## 6.1. Development of DMAs - B

### 6.1.1. Current Status of Network and Region (B.1.)

**Amasaman**

Amasaman District was added to Ghana Water Company Limited on 27 September 2018, after being operated by a private owner. The network mainly consisted of PVC (PN10) pipelines. Water is supplied from Nsawam treatment plant through a 315mm HDPE main coming in from the north, running parallel to the N6 highway. Several offtakes have been made on this line into the Amasaman district (8) and to the Eastern Accra Region (3), as shown in figure 15.

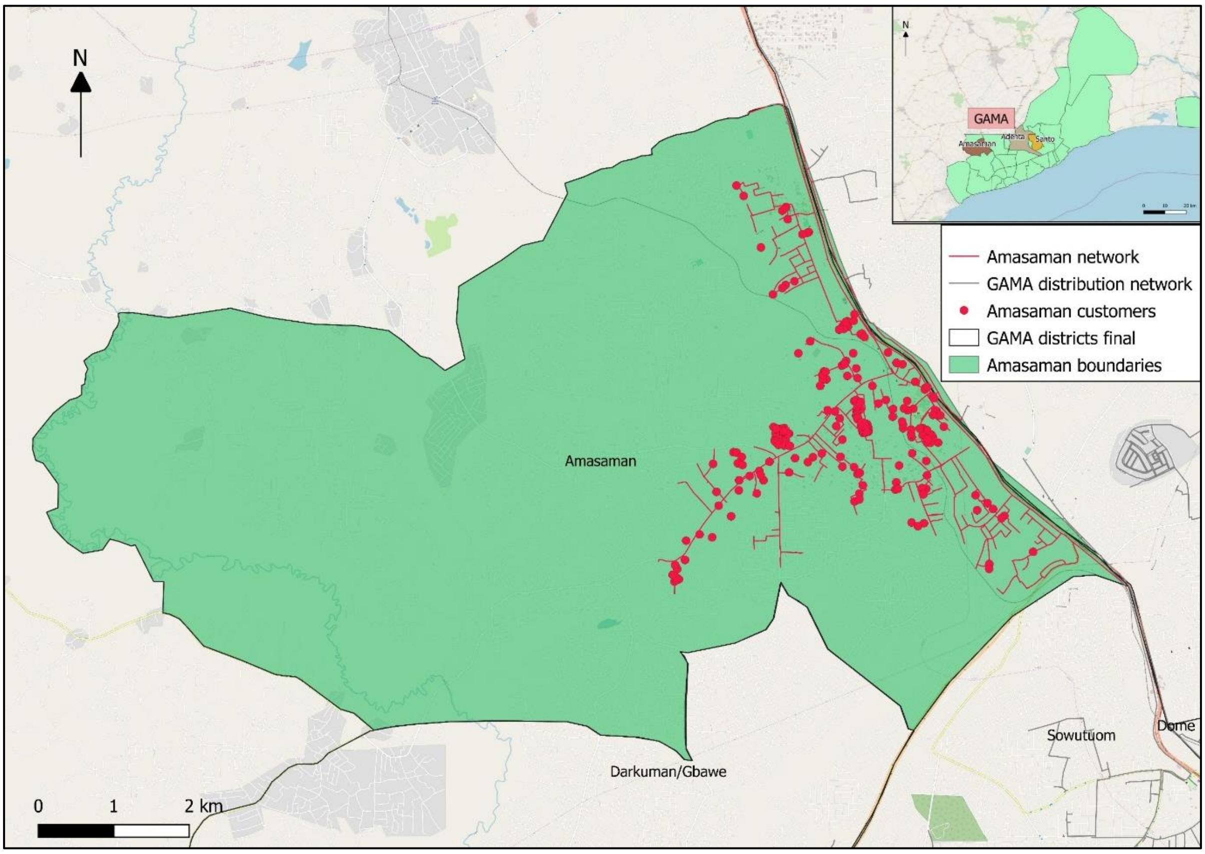



*Figure 15 Amasaman Current situation*

The offtakes (100mm HDPE) to East Accra are metered mechanically and did not serve customers until recently. During meetings with the regional and districts office it was observed that Amasaman faces intermittency issues mainly caused by maintenance and power outages at the Nsawam treatment plant as well as burst repairs within the district. Furthermore, relatively high pressures (7.0 bars at night, 5.5 during the day) were observed in the 150mm offtake from the 315mm line. Which cause higher leakage flows and their pressure transients higher burst rates.

In 2018, 115,000 people lived in Amasaman District, based on the municipal consensus. It is important to stress that some communities within the District are expanding usage. Currently

Amasaman District has 486 billed and 73 unbilled customers. Since opening in Amasaman District, GWCL receives about 15 connection applications per month from potential customers and making about 50 connections per month. It has been said that over a thousand applications are received up until the end of April 2019. Customers must pay a connection fee, but many potential customers lack the funds to pay this fee up front.

In order to increase supply to other regions around Amasaman District, a 400mm HDPE line is being constructed from BOI reservoir, feeding into the eastern corner of the district. From this corner a 400mm HDPE line will feed into a 300mm HDPE line coming from Weija. However, still 1.7km of mains has to be placed to connect these lines. Additionally, within the next month, construction will be started to supply a community within the Sowutuom District. This will be done by connecting a 150mm HDPE pipeline to the 100mm line over there. In case of emergencies when Nsawam does not supply water, this line is considered to supply (part of) Amasaman District.

*Bottlenecks*

Interviews at the district and regional offices revealed that some of the bottlenecks for the development of a DMA for the Amasaman district are caused by:

- High burst rates
- Weak pipes and fittings (>8 years old network)
- High pressure in network (>7.0 bars at night at 150 mm HPDE line)
- High pressure variations within network
- Intermittent supply due to maintenance Nsawam treatment works and power outages (up to 3 days a week)
- Old treatment works is not rehabilitated and therefore not in use
- Current network is hydraulically limited, extensions cannot be supplied in future, due to too small pipes

**Santo**

Santo is the newest district in the Tema region (Sept 2018). The District is supposed to serve as a model and example for the older districts and good practices will be adapted from here. The district is fed from an offtake in the south from Tema reservoir with pressures being regulated by a PRV set at 7.0 bar pressure. The 280mm HDPE line runs parallel to the main road and western boundary, up to the construction company. Currently Santo district has over 400 connections and has 20,000 inhabitants per estimate of the district office.

High pressures occur in the network and Santo is reported to now receive water continuously. However, in the future, with the network extending and demand increasing, this might not be the case, as reported by the Regional Manager[2]. Figure 16 shows an overview of the current situation for Santo.

*Bottlenecks*

Some of the bottlenecks for the development of a DMA for Santo that need to be addressed are:

- Boundaries are not set
    - Western boundary between Adenta and Santo

[2] Akoto, Evans Walter (District Manager Santo, GWCL) during meeting with author, May 2019

- Northern boundary between Adenta and Santo

- 180mm line feeding into Ashaiman West district
- 150mm line feeding into Adenta district
- High burst frequencies at 280mm line

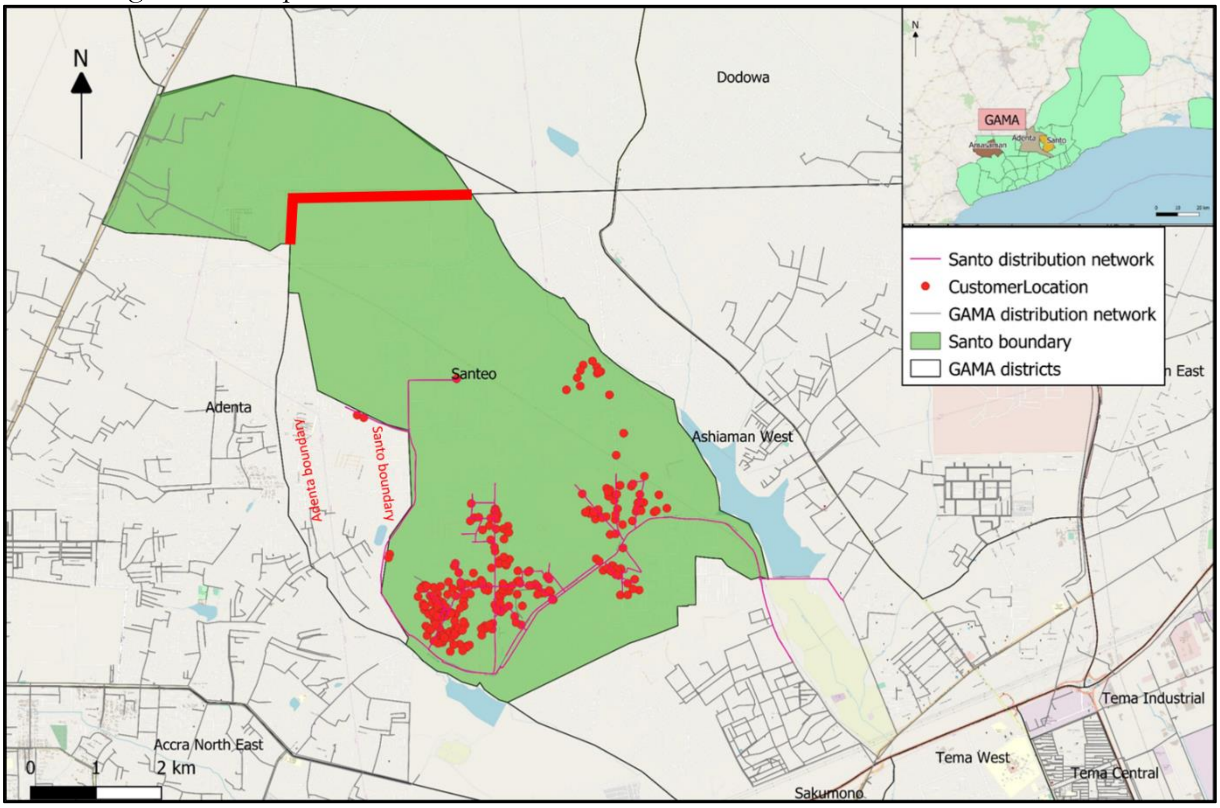



*Figure 16 Santo current situation. Red line indicates the boundary dispute between Adenta and Santo*

**Adenta**

*Background – Continuous Supply in GAMA*

Currently, Accra reservoir (45,000m3) faces challenges receiving sufficient water. Some of the feeding lines for the reservoir include the 1200mm from Dodowa running through Adenta District and the 800mm line from Tema with an offtake to Santo district. It is important to address this issue, since the decisions that must be made both for Santo and Adenta are depending on the vision GWCL has with the supply of this reservoir and its downstream supply zones. Currently, there are several offtakes made from the 1200mm line within Adenta, in order to feed the district and BOI reservoir. This was not the initial design during the China Ghazuba Project (2015). Within one year after construction, additional offtakes were made from this 'dedicated' line that is supposed to serve Accra reservoir. Supply to districts is cut on Wednesdays and Sundays in order to feed Accra Reservoir. At the reservoir, bypasses are made, so that the 1200mm line is feeding directly into a High-Pressure Zone (HPZ), not through the reservoir, as well as for the 800mm from Tema feeding into a Medium Pressure Zone. Whenever the goal of continuous supply will be achieved in Adenta, it will either come at the expense of the supply conditions downstream (Accra reservoir and its supply zones) or additional production capacity is required, or leakages must be reduced.

*Adenta Current Situation*

Adenta has over 13,000 connections, with 113,000 people living within the District boundary, as per national census (Nyarko, 2012). However, the national census might consider other boundaries than GWCL does for Adenta. Intermittent supply is an issue, there is no water going into Adenta on Wednesdays and Sundays, in order to feed Accra Reservoir. There are two reservoirs within the district that are not used/ by passed. Both are in the Adenta Municipality. Figure 17 shows an overview of the current situation for Adenta.

*Bottlenecks*

Bottlenecks for the development of a DMA for the Adenta district are:

- Multiple offtakes from supply line Dodowa – Accra Reservoir (1200mm) within the district.
- Ritz – junction. This is the place were the biggest offtake from the 1200 is made, feeding different districts. It is on the south border of the district. Challenges are:
    - High pressures
    - Low discharge into the feeder lines.
    - Currently squeezed to reduce pressures on feeder lines
- The 250mm line from Ritz-junction to University Farm alongside Ecowash road has at least seven offtakes going to Madina district.
- The district has more than 10,000 connections, which is the upper limit for a GWCL district.
- The district has multiple connections and pipelines outside of its boundaries.
- Reservoirs are bypassed and water is directly fed into the district from a transport line, resulting in reduced service levels within the district and high wear and tear on the distribution network.

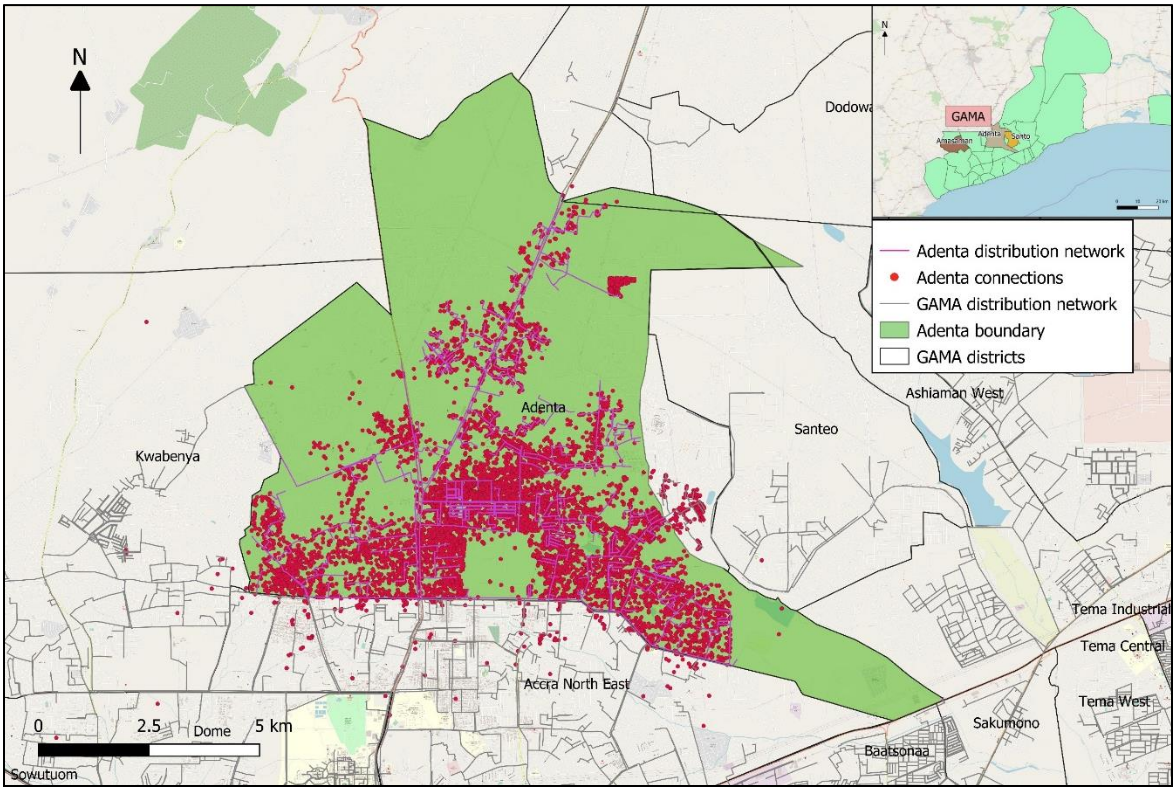



*Figure 17 Adenta Current situation.*

### 6.1.2. Division of Network in Districts and Creation of Durable DMAs (B.2-3.)

Based on the assessment of the current states of the district and their bottlenecks, the DMA requirements for each district were investigated which resulted in the DMA design for each district. An overview of these DMA designs is given for each district and highlights are explained. A detailed description of the DMA requirements for each district and the design and engineering of the DMAs can be found in Appendix B and C.

**Amasaman**

For Amasaman district to be converted into a DMA four pipes have to be cut, and end capped as indicated in figure 18. Two flowmeters have to be placed, one north and one south-east of the district.

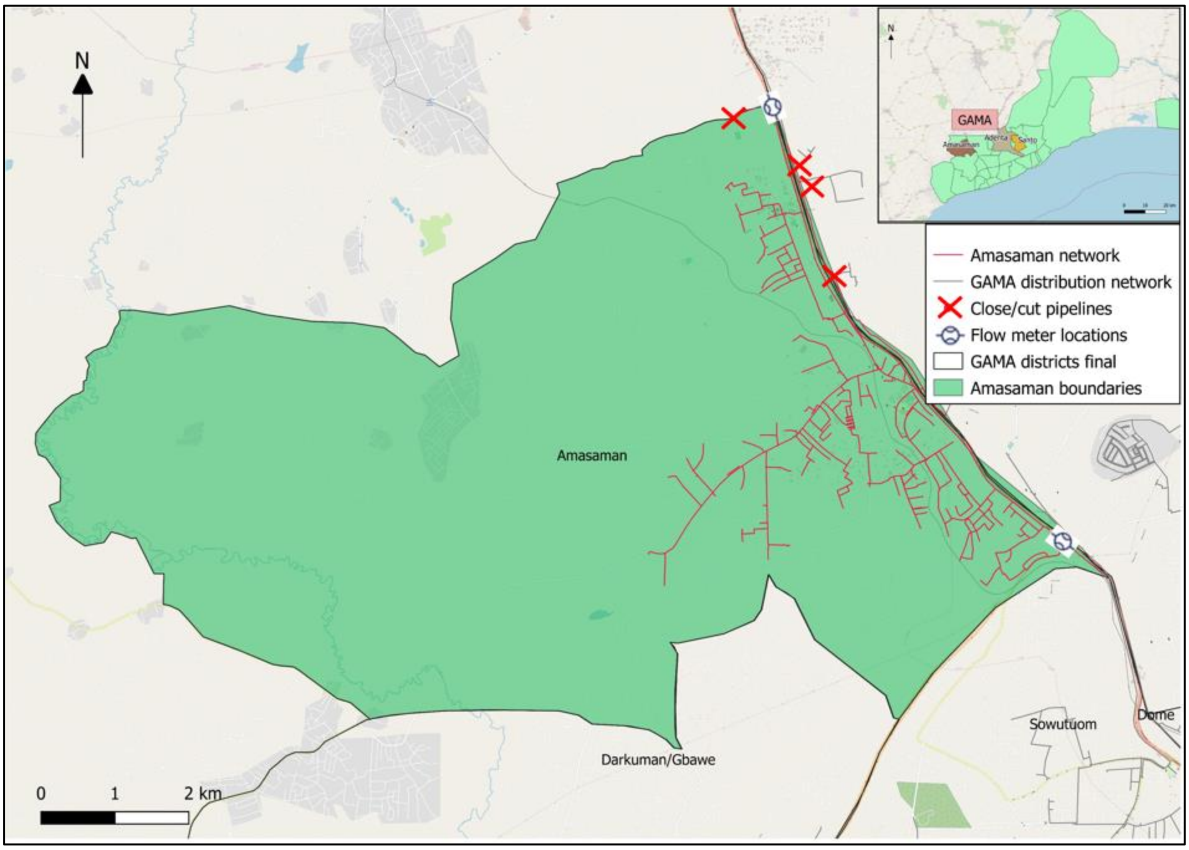



*Figure 18 Amasaman DMA requirements. Flowmeters in north and south east corners. Pipelines closed or cut in the north-eastern part of the district.*

**Santo**

For Santo district to be converted into a DMA two pipes have to be cut, and end capped as shown in figure 19. One flow meter must be placed in south-east corner of the district. Furthermore, the boundary dispute between Adenta and Santo should be solved.

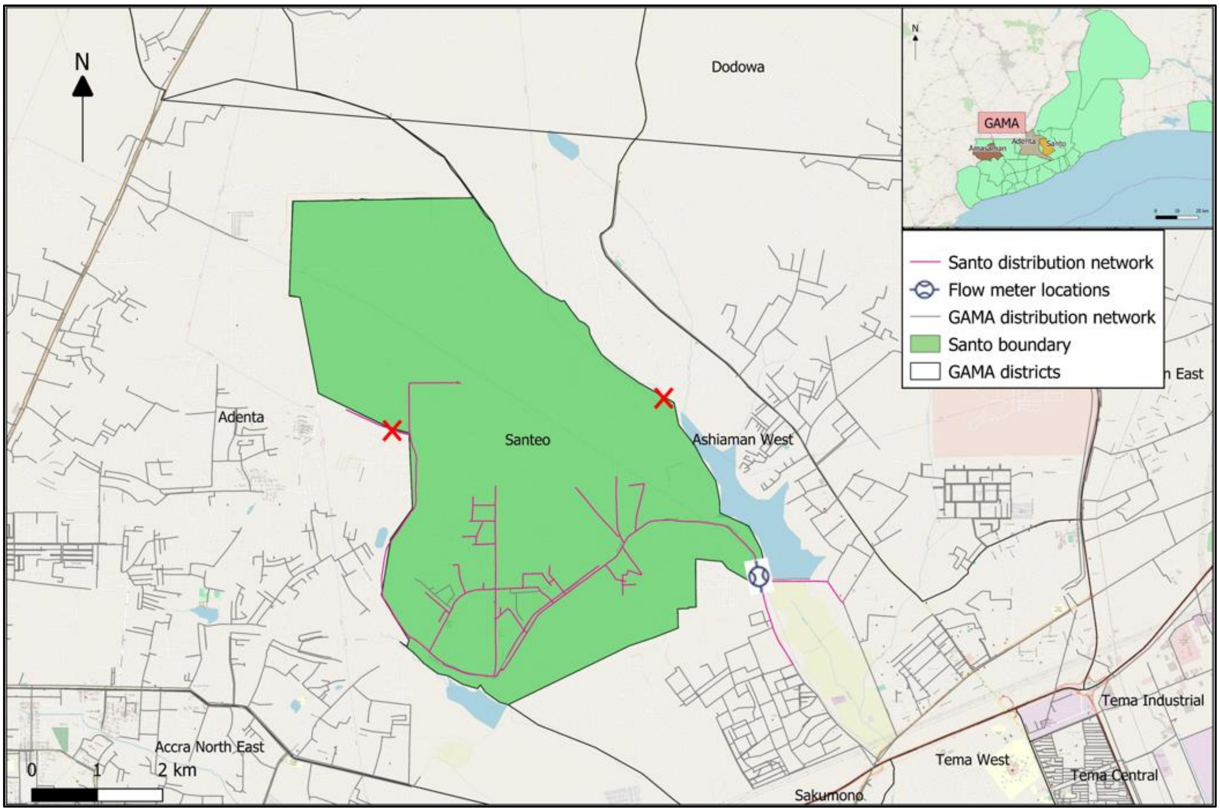



*Figure 19 Santo DMA overview. Red crosses indicate location where pipes need to be cut off. Flowmeter placed in the south-east corner.*

During the design and engineering of the DMAs it became evident that Adenta district is not ready yet to become a DMA due to the complexity of supply issues and the costly investments it requires to be solved. However, it can be divided into sub-districts who can become DMAs.

**Adenta**

Adenta is not ready yet to be converted into a DMA due to the complexities of supply issues and the costly investments it requires to be solved. Figure 20 shows how the district can be divided into two or more sub-districts that can become DMAs.

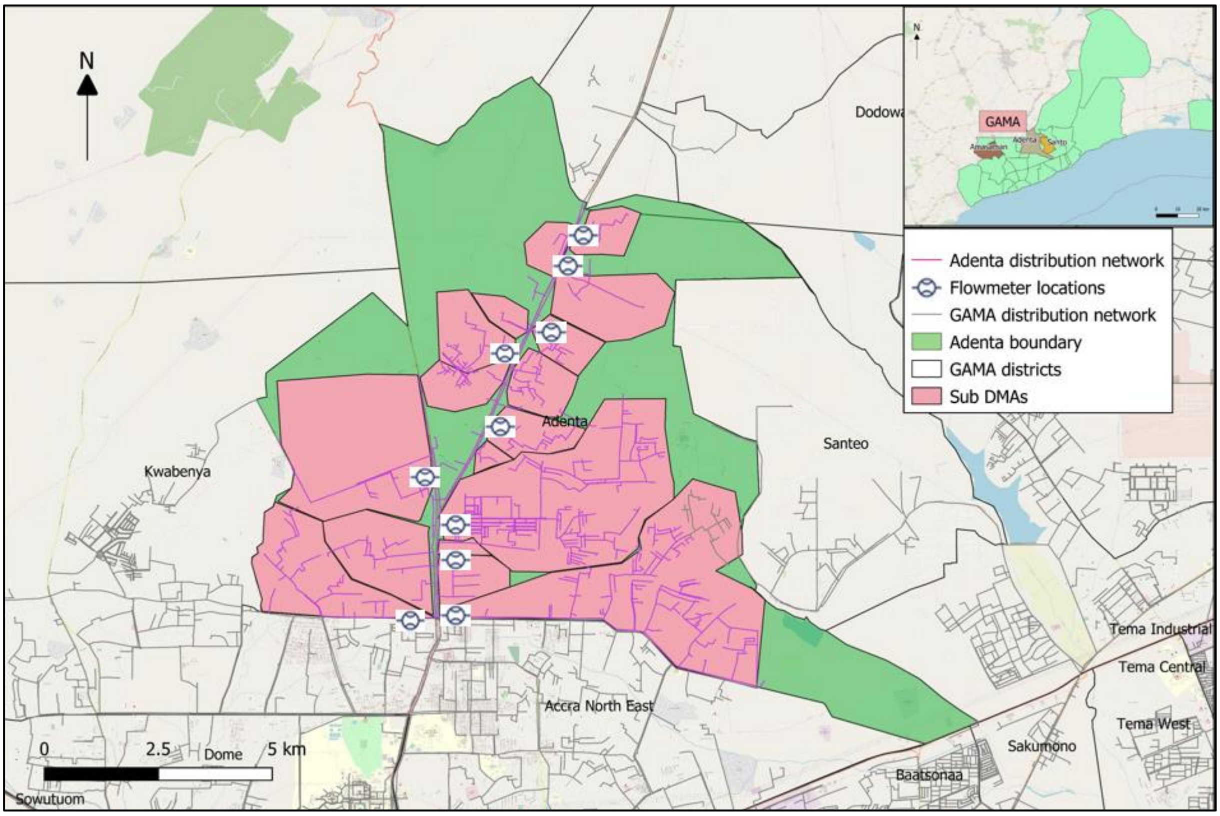



*Figure 20 Adenta with DMA requirements. Sub DMAs are an easy way to create a DMA out of the district.*

### 6.1.3. Top-Down NRW Assessment (B.4.)

The top down water balance assessment was carried out with EasyCalc for each district. A summary of the results is shown in table 3. The NRW levels are quite high for the districts of Adenta and Amasaman. Santo has low values in terms of apparent losses, and NRW. However, it has high real losses. Adenta has a high infrastructure leakage index (ILI), which indicates high opportunities for real loss reduction. Finally, Amasaman has a high share of apparent losses compared to the other districts. The detailed results of the EasyCalc top down assessment are shown in Appendix E.

| District | Adenta | Amasaman | Santo |
|---|---|---|---|
| **Average Supply Time [h/day]** | 17 | 18 | 24 |
| **Average Pressure [m]** | 75 | 65 | 55 |
| **CARL [$m^3$/day]** | 11,974 | 728 | 200 |
| **UARL [$m^3$/day]** | 749 | 136 | 25 |
| **ILI [-]** | 16 | 5 | 8 |
| **Real losses [L/con./day/P/h]** | 18 | 13 | 20 |
| **Apparent Losses [L/con./day]** | 34 | 185 | 7 |
| **NRW [% of SIV]** | 63 | 72 | 36 |
| **NRW [L/con./day/h]** | 1244 | 1060 | 508 |

*Table 3 Overview top down NRW EasyCalc assessment for the districts of Adenta, Amasaman and Santo.*

## 6.2. Data Analysis and Interpretation - D

### 6.2.1. Flow and Pressure Analysis (D.1-2.)

Appendix F shows the full and corrected flow-pressure graphs for Amasaman. The pressures that were measured at different sites can be found in Appendix F as well. Since the northern flowmeter registered flows continuously throughout the whole period, continuous water supply was ensured for the district during the period. MNF was observed at 8/18 at 03:00 after five days of CWS.

It was communicated that at 08-22 at 08:00, the 315mm HDPE pipeline experienced a burst explaining the pressure drop within the system, since it was closed at several points. This could be the same case for 08-20 at 06:00, but cannot be concluded with certainty, since one of the flow meters was not measuring during this period. At several times, high pressures were observed, when flow variation was only minimal. This happened at 8-19 at night, 8-20 in the evening and at night and 08-21 during the night. The increase in pressures is most likely due to the decrease in flow and increase in pressures in districts and offtakes upstream of Amasaman District.

### 6.2.2. Minimum Night Flow Analysis (D.3.)

An MNF analysis was performed for August 18. Figure 22 shows the real loss flow and unavoidable real losses, at current pressures.

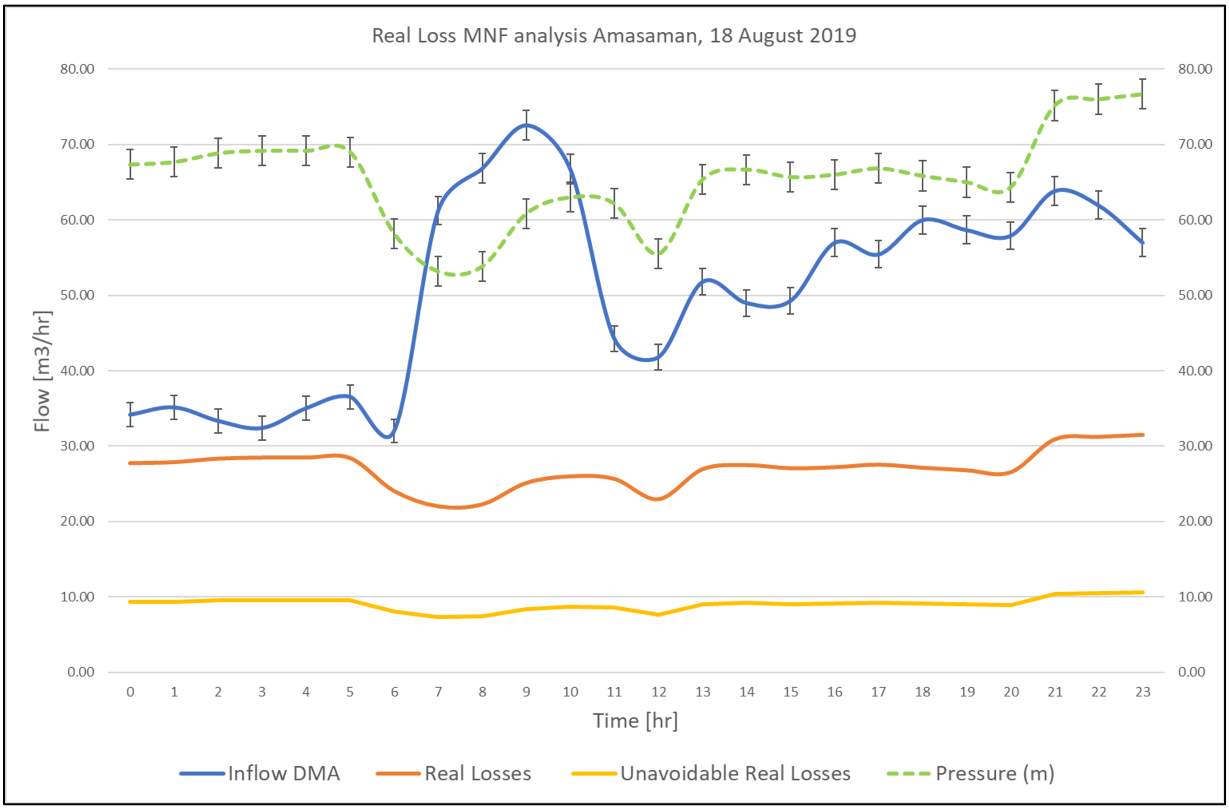



*Figure 21 Real loss MNF analysis, 18 August 2019. Measurement uncertainty indicated with error bars. Blue and green lines are measured, red and yellow are calculated.*

As can be seen in figure 22, flow and pressure are not fully dependent, most likely caused by upstream changes in flow and pressure. Based on the analysis, the real loss flow has a maximum of 31.5 $m^3$/hr, a minimum of 22.0 $m^3$/hr and an average of 27.0 $m^3$/hr. With an average unavoidable real loss flow of 9.05 $m^3$/hr. The uncertainty of the flow readings became larger at peak flows (1.95 $m^3$/hr at 72,00 $m^3$/hr), for the pressure readings the uncertainty was constant.

### 6.2.3. Real and Apparent Losses Assessment (D.3.)

The volumes for August were calculated by extrapolating average flows based on a 24-hour cycle. This resulted in a total inflow and NRW volume based on the billing for August, shown in table 4. The NRW volume was split in both real losses (19,429$m^3$, 53% of SIV) and apparent losses (7,157$m^3$, 19% of SIV).

| | Flow [m³/month] | Percentage [% of SIV] | Percentage [% of NRW] |
|---|---|---|---|
| **SIV** | 36,916 | 100% | |
| **Billing** | 10,329 | 28% | |
| **NRW** | 26,587 | 72% | 100% |
| **Real losses** | 19,429 | 53% | 73% |
| **Apparent losses** | 7,157 | 19% | 27% |

*Table 4 Overview Non-Revenue Water for Amasaman in August 2019.*

#### 6.2.4. State of IWS Assessment (D.5.)

The billing reports for the months of August and September were compared, a summary of the reports is shown in table 5.

| | August | September | |
|---|---|---|---|
| **Sales Private** | 8,384 | 6,909 | m3 |
| **Sales MDA** | 1,230 | 1,225 | m3 |
| **Customers private** | 485 | 537 | no. |
| **Customers MDA** | 7 | 7 | no. |
| **Private demand** | 17.29 | 12.87 | m3/customer |
| **Intermittency** | 0% | 26% | Private demand |

*Table 5 Overview intermittency based on billing reports for August and September 2019.*

As shown in table 5, private sales in September are lower than August, while the customer base had an 11% growth during the period and sales to Ministries, Departments and Agencies stayed constant. If the private sales for August per customer are assumed to be 100% the customer demand, then for September the customer demand is suppressed with an intermittency of 26%, indicating the district is not (well) supplied for over one quarter of the time. This is a best-case scenario, since periods without supply might have occurred in August as well. The above-mentioned was verified by the districts' commercial manager[3], who told the district experienced IWS at multiple moments in September.

IWS in Amasaman district is due to the high levels of real losses. SIV is large enough to supply the domestic demand as well as the unavoidable leakage level and the economic leakage level. The total real loss level makes the district experience IWS, besides power-outages and regular maintenance on the treatment plant. The type of IWS Amasaman has to deal with is therefore type III in the decision tree in figure 8. For this type of IWS, active leakage control and pressure management are possible solutions to reduce intermittency.

### 6.3. Data Modelling - E

#### 6.3.1. Development and Validation DDA Model (E.1.)

The hydraulic model of Amasaman developed in demand driven analysis (DDA) was validated for its measured flows and pressures. To check the validity of the model, the head (static and dynamic pressure) at the flowmeter North and the demand at the flowmeter South of the model need to correspond with the field data. The dynamic head pattern proved correct but underestimated by a few meters at each pressure logger location. This proved that the static head needed adjustment and the dynamic head was modelling correctly. The main reasons the static head was off was due to inaccuracies in the DEM, caused by high rise housing close to the measuring point as well as

[3] MacCarthy Danqua, Dennis (Commercial Manager Amasaman District, GWCL) in text message to author, October 2019.

elevations of each pressure logger under surface level. After looking into the elevations of each pressure logger and using more accurate GPS elevations for the coordinates, the model proved valid. An overview of the distribution network is given in figure 23. Figures 24 and 25 show the modelled head and demand at the locations of the flowmeters.

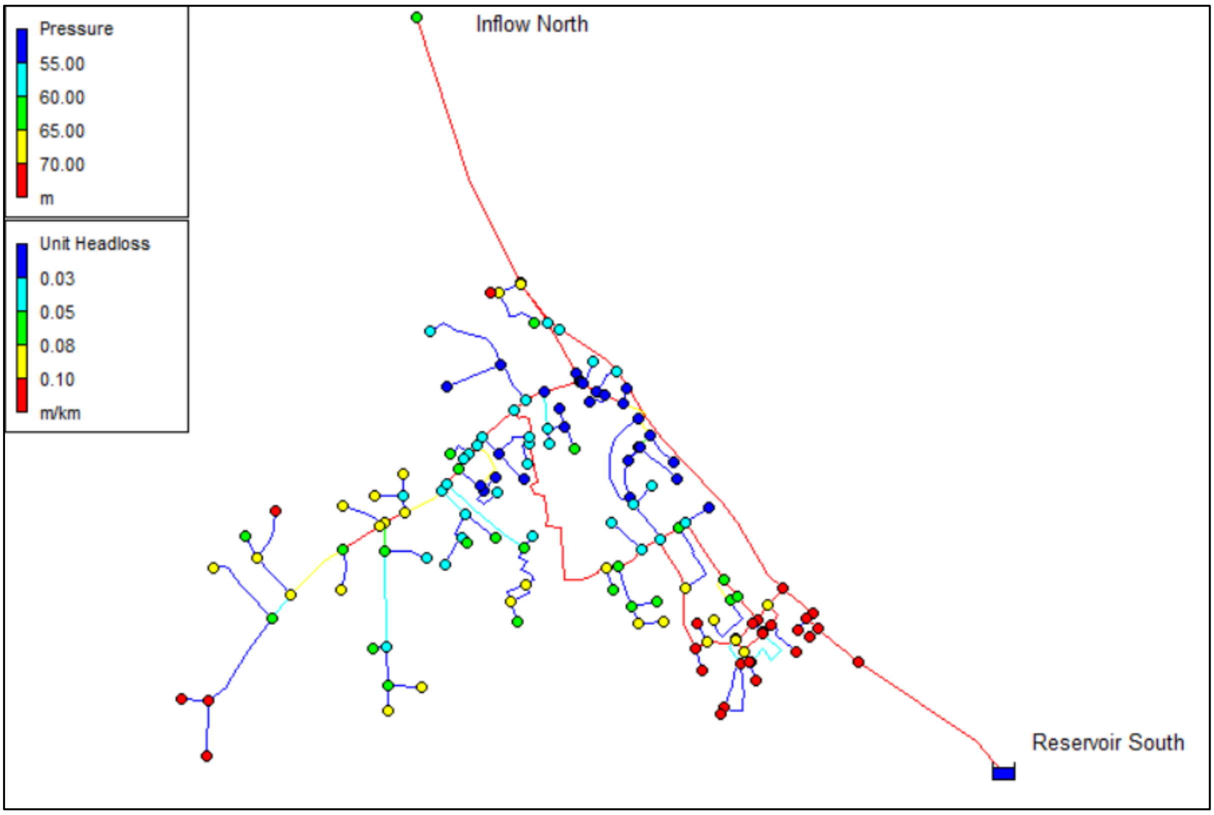



*Figure 22 Overview of the hydraulic model developed in EPANET 2.0 The flowmeter in the south was modelled through a reservoir, since positive and negative flows were observed at the location. Legend 'Pressure' indicates the pressure in meters at each node. Legend 'Unit Headloss' indicates the headloss in meter per kilometre in the pipes.*

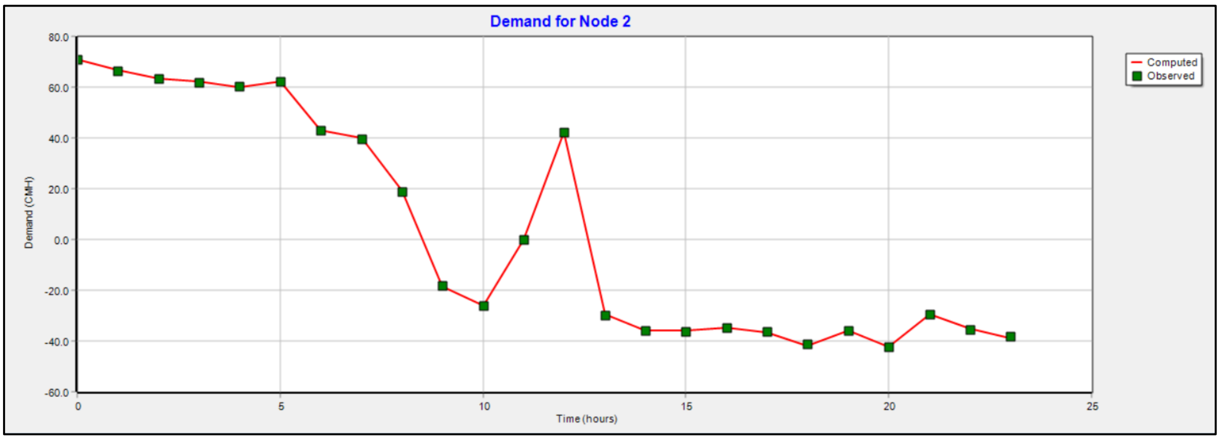



*Figure 23 Demand at flowmeter South (Node 2), modelled as a reservoir. Green dots represent field measurements. When red line follows green dots, it means that the volume flow within the district is accurately modelled.*

The difference between the modelled demand at node 2 and measured flow in the field, is termed $\Delta_{Demand}$. The average, maximum and standard deviation for $\Delta_{Demand}$ are shown in table 6.

| $\Delta_{Demand}$ | | |
|---|---|---|
| **Absolute Average** | 0.02 | m3/hour |
| **Absolute Max.** | 0.13 | m3/hour |
| **Max.** | 3.5 | % of measured flow |
| **Standard deviation** | 0.07 | m3/hour |

*Table 6 Difference between measured flow (demand) and modelled demand.*

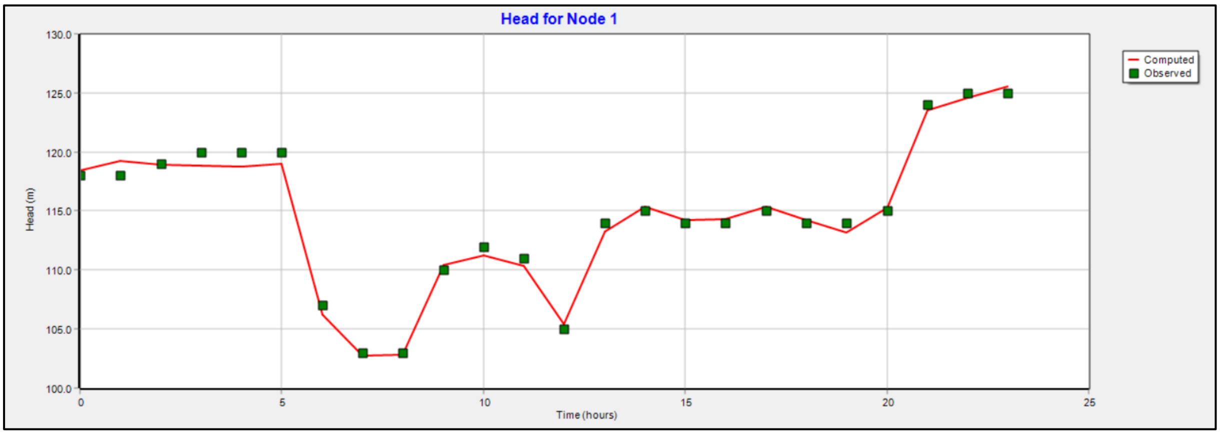



*Figure 24 Head in meters at flowmeter North (Node 1), green dots represent static and hydraulic pressure measurements in the field.*

The difference between the modelled head at node 1 and measured head in the field, is termed $\Delta_{Head}$. The average, maximum and standard deviation for $\Delta_{Head}$ are shown in table 7.

| $\Delta_{Head}$ | | |
|---|---|---|
| **Absolute Average** | 0.14 | m |
| **Absolute Max.** | 1.28 | m |
| **Max.** | 1.1 | % of measured head |
| **Standard deviation** | 0.42 | m |

*Table 7 Difference between measured head and modelled head.*

Based on the maximum difference and standard deviations from table 6 and 7, it can be concluded that the model represents field measurements accurately. Based on this validation in DDA, domestic demand and real losses were then modelled in PDD.

### 6.3.2. Validation Real Loss and Demand Flow (E.2.)

Real losses were modelled through EPANETs emitter function. The emitter coefficient and exponent were calculated, see table 8.

| Leakage Emitter | Value |
|---|---|
| **C (coefficient)** | 0.454 |
| **N1 (exponent)** | 0.983 |

*Table 8 Leakage emitter coefficient and exponent.*

The coefficient values were adjusted for each node, corresponding to the total length of pipe connected to each node and further adjusted until the modelled real losses corresponded with the calculated (MNF) real losses. The network input file with the corresponding emitter coefficients can be found in Appendix H. Figure 26 shows the final result for this iteration.

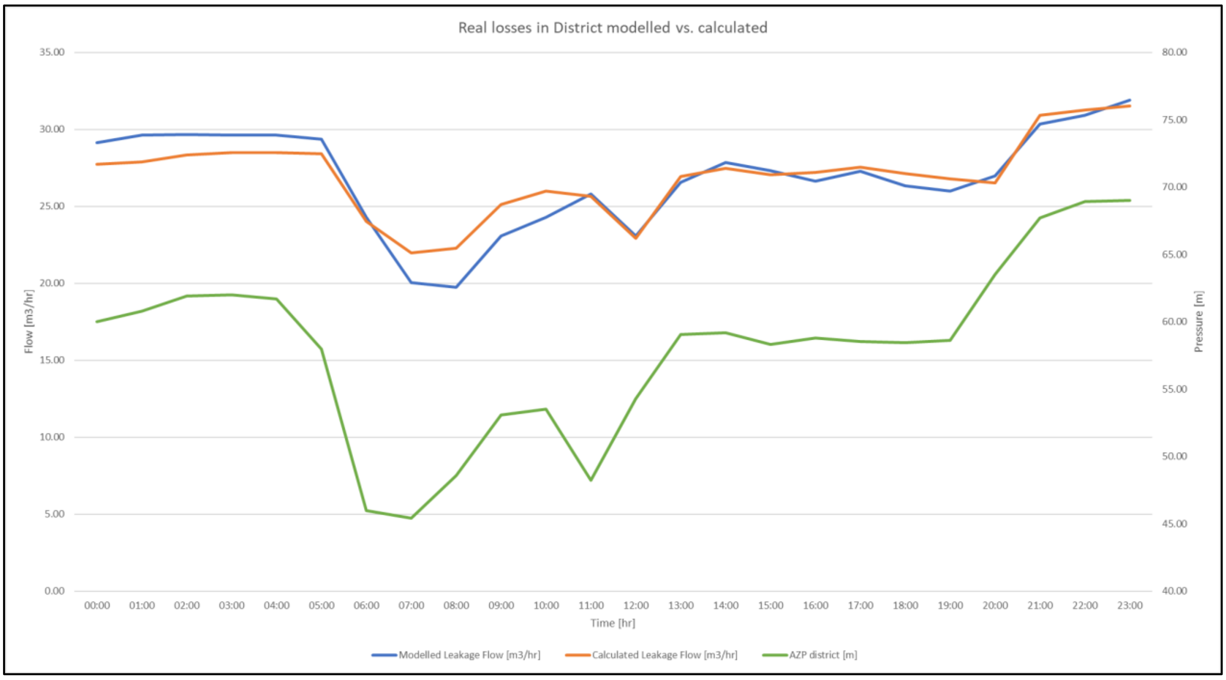



*Figure 25 Real losses modelled (blue) vs. calculated (orange). Pressure indicated in green.*

The modelled leakage flow is overestimated during the night and underestimated in the morning, however only slightly. The difference between calculated leakage flow and modelled leakage flow is termed ΔLeakage. The average, maximum and standard deviation for ΔLeakage are shown in table 9. The average difference between modelled and calculated leakage is 0.11 $m^3$/hour, with a maximum of 2.53 $m^3$/hour which corresponds to 9.4% of the average calculated leakage flow.

| ΔLeakage | | |
|---|---|---|
| **Absolute Average** | 0.11 | m3/hour |
| **Absolute Max.** | 2.53 | m3/hour |
| **Max.** | 9.4 | % average calculated flow |
| **Standard deviation** | 1.24 | m3/hour |

*Table 9 Difference between calculated leakage flow and modelled leakage flow.*

### 6.3.3. Modelling Intervention (E.3.)

In paragraph 6.2.4 the type of IWS that Amasaman is dealing with was deducted. This type is 'Costly Real Losses', where there is sufficient water to supply the district with its domestic demand and economic real loss level, but the current real loss level is much higher than the economic real loss level. This type of IWS requires pressure management and active leakage control as its solutions to improve supply conditions and transition to CWS, according to the decision tree in figure 8. Since there were not multiple MNF recordings over a longer period and due to the incapability of the PDD modelling software to model leakage and the different leakage components adequately, active leakage control could not be modelled as an intervention. Therefore, only pressure management could be modelled with the EPANET emitters function, since this intervention only relies on pressure adjustments as input.

**Pressure Management**

A pressure reducing valve (PRV) was modelled. The critical point in the network was determined to be at Node 102, with an elevation of 66 meters. Therefore, the artificial reservoir was connected to this point and the reservoir head was set to 76 meters (elevation + minimum service pressure). Figures 27 and 28 show the graphs for the pressure reduction in the system and the impact of a

PRV installed on the flows within the district, where the system pressure could be reduced by almost 50% and the volume reduction of real losses over 50%. Table 10 provides an overview in the costs, benefits and impact of this intervention based on the volume of water saved and the production costs of water.

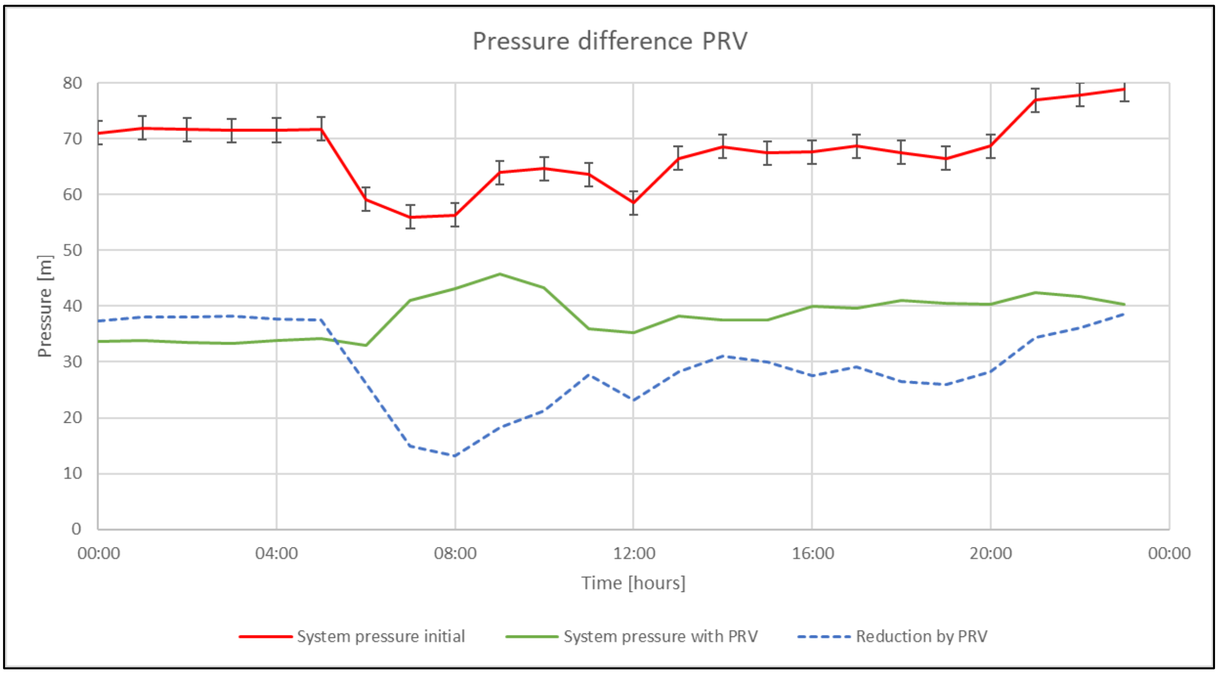



*Figure 26 Impact PRV on pressures within the district. Error bars indicate measurement uncertainty.*

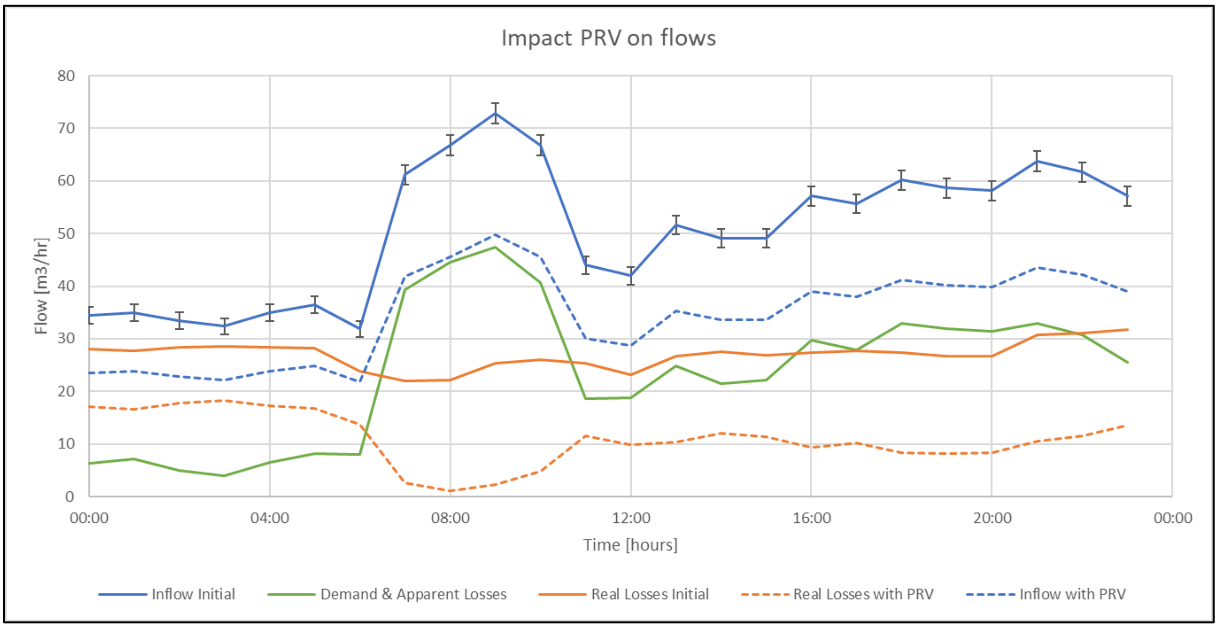



*Figure 27 Impact of PRV on flows within the district. Solid lines indicate status quo of district. Dashed lines indicate flows with PRV installed. Error bars indicate measurement uncertainty*

| Impact PRV for Amasaman | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| **Cost** | | | **Benefits** | | | **Impact** | | |
| Material cost | € 10,000 | | Water saved | 16 | m3/hr | Required return | 8% | |
| Installation cost | € 2,000 | | | 140.16 | ML/year | Lifetime PRV | 10 | years |
| Operational cost | € 1,000 | /year | Cost price water | € 0.42 | /m3 | NPV | € 400,294 | |
| CAPEX | € 12,000 | | Value saved | € 58,867 | /year | | | |
| OPEX | € 1,000 | /year | | | | | | |

*Table 10 Overview of impact PRV for Amasaman.*

Based on a ten-year life, the PRV will have a net present value (NPV) of € 400,294 and is therefore financially viable as an intervention strategy. Even within one year, the PRV will have a positive NPV. It is important to note that only the direct impact of a PRV is assessed, the indirect impact (reduction in burst frequency and the costs involved with their repairs) could not be assessed but will increase the NPV for pressure management in Amasaman only further.

#### 6.3.5. Development Chained DMA Model (E.4.)

An overview the existing transport network (>DN400mm) in GAMA and the districts and their feeding points is shown in figure 29. The flow and pressure data, as well as EasyCalc and MNF assessments for each district are conveyed to the feeding point (inlet) of each district. The types of IWS can be assessed per district once the SIV, domestic demand and leakage levels are known. Based on the type of IWS in a DMA an intervention is then proposed. The effects of the interventions (i.e. pressure management, active leakage control etc) are determined with the EasyCalc tool. The EasyCalc assessment is done every month to calculate KPIs and determine further adjustments for the district. The performance according to the EasyCalc assessments can be benchmarked for all the districts. Herewith the influence of one district on other districts upstream and downstream in terms of pressure, water availability and supply times is predicted.

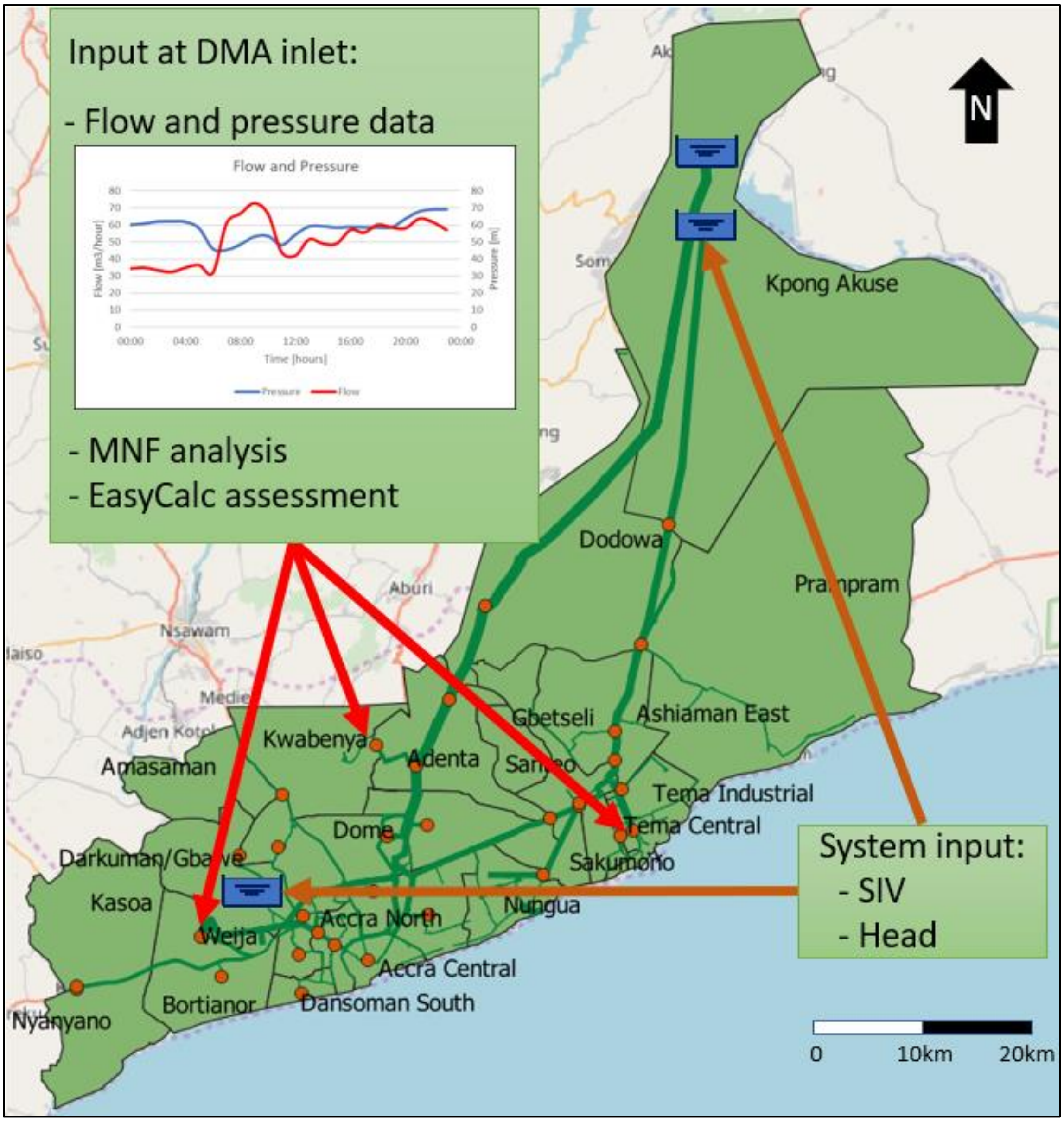



*Figure 28 Overview of GAMA districts and their feeding point (orange dots) with required input for each district inlet and system input. (Design by author, 2019)*

Since data from only 10 districts was received (all in the Tema region), a partial overview of supply time per district and pressures per district could be generated. Figure 30 shows the hours of supply per week for each of these 10 districts. Tema West and Tema Industrial experience the highest level of intermittency, over 40 hours per week. Figure 31 shows the average pressures for each of the 10 Tema districts. Six of them experience pressures between 20-25 meters. Gbetselli district faces low supply time and higher pressures.

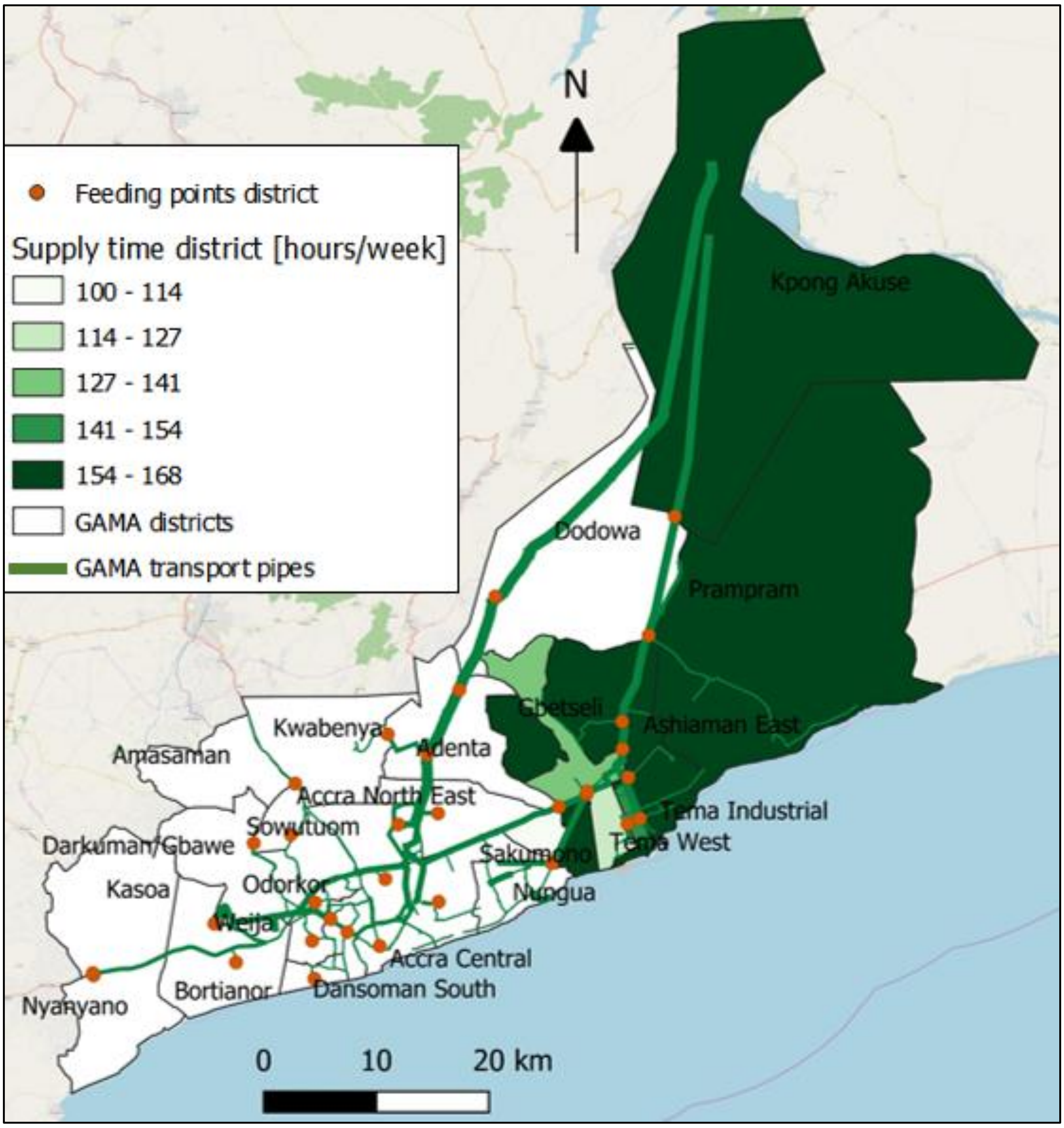



*Figure 29 Weekly supply time [hours/week] for GAMA district. Data from only 10 of 33 districts was received. Therefore, Accra East and Accra West regions show white.*

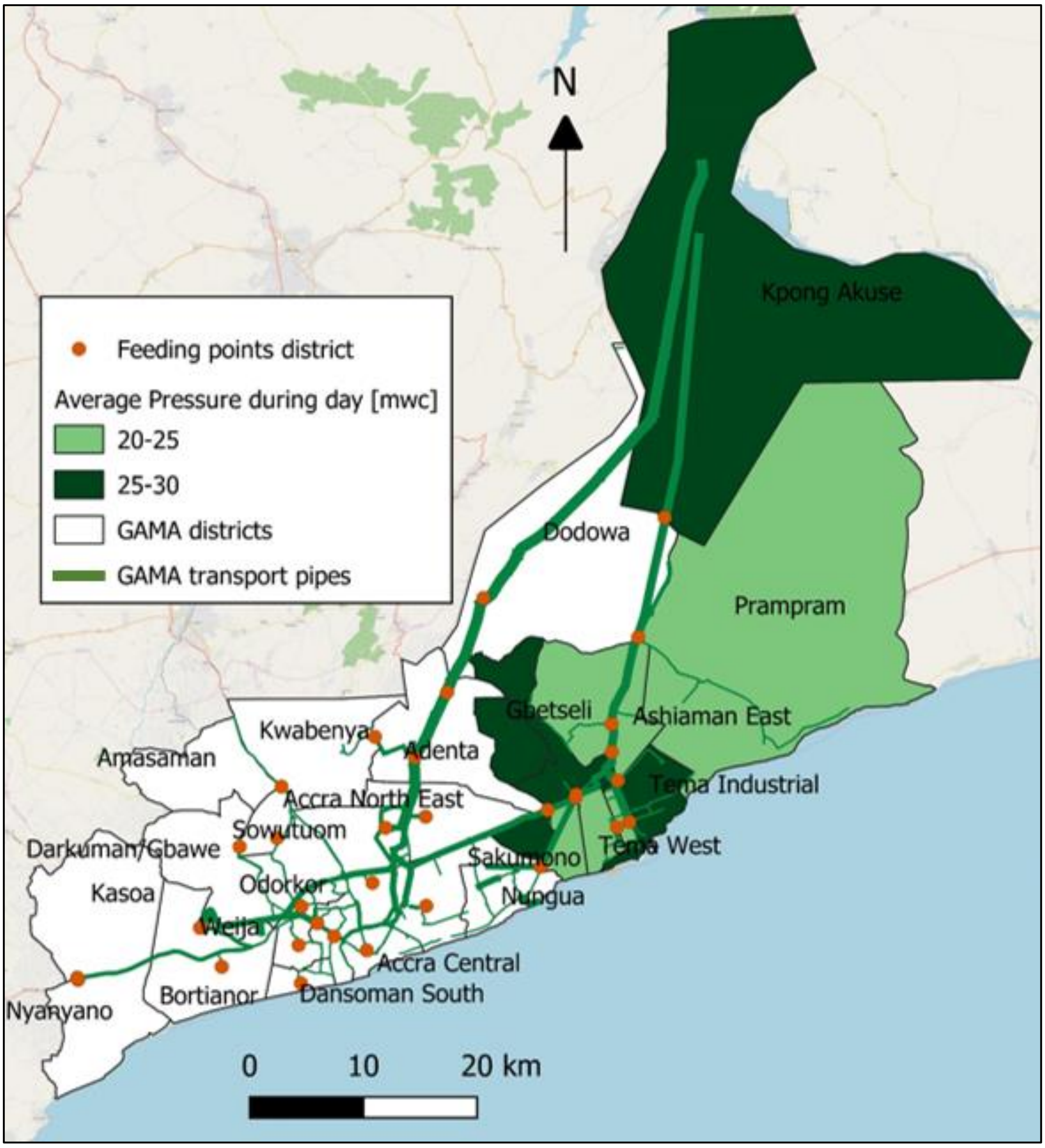



*Figure 30 Average Pressures [m] in GAMA districts. Data from only 10 of 33 districts was received. Therefore, Accra East and Accra West regions show white.*

## 6.5. Feasibility DMA Approach - F

### 6.5.1. Questionnaire Water Operators (F.5.)

The four utilities that were represented, all experienced IWS in their distribution zone. Three of them do not meet the requirements for the bottom up DMA approach. All three need operational DMAs, one utility needs an improved customer database. All VEI water operators agreed that based on their experience and expertise the DMA approach can be applied to transition to CWS for their utility. They unanimously responded that NRW reduction measures lie at the heart of reducing intermittency.

According to their expertise a trusted GIS database and working boundary valves are essential boundary conditions as well as strong leadership within the utility that makes decisions according NRW data.

When asked about critique and feedback, one operator responded that the establishment of DMAs in an existing network can be very complicated. And that a DMA can also be a larger area initially where water supply is stabilized. One operator said that the causes of IWS are complex and may not be resolved by DMA control only.

### 6.5.2. Sensitivity Analysis (F.6.)

The sensitivity of different variables is shown in table 11. Here an overview is provided of the parameters, their input values, standard deviations and the effect of the parameter on the output in terms of real losses.

| | Input | | | Stdev m3/hour | Stdev m3/hour | Output Real Losses [m3/hour] | |
|---|---|---|---|---|---|---|---|
| **Parameter** | Min. | Used | Max. | Min. | Max. | Min. | Max. |
| **Average inflow district [m3/hour] based on 24hour cycle** | 50.6 | 50.6 | 58.5 | 0.00 | 0.00 | 21.99 | 31.52 |
| **N1 factor [-]** | 0.5 | 0.983 | 2.5 | 1.41 | 3.85 | 24.97 | 36.84 |
| **Night consumption [L/connection/hour]** | 1 | 3 | 5 | 0.97 | 0.97 | 22.74 | 30.44 |
| **Residential leakage [L/connection/hour]** | 1.0 | 2.4 | 7.3 | 1.45 | 4.83 | 23.12 | 26.14 |
| **AZP [m]** | 63.4 | 65.5 | 67.6 | 0.00 | 0.00 | 21.99 | 31.52 |

*Table 11 Local sensitivity analysis real loss flow parameters. Percentages indicate the relative value of the deviation compared to the real loss flow for that moment.*

Based on table 11 and figure 32, the sensitivity analysis shows that the calculated real loss flow depends highly on the residential leakage as well as the N1 leakage exponent. Residential leakage is highly unknown, as it also includes spillage from private tanks. The average inflow into the district on a 24-hour cycle as well as the average zonal pressure (AZP) does not influence the real loss flow.

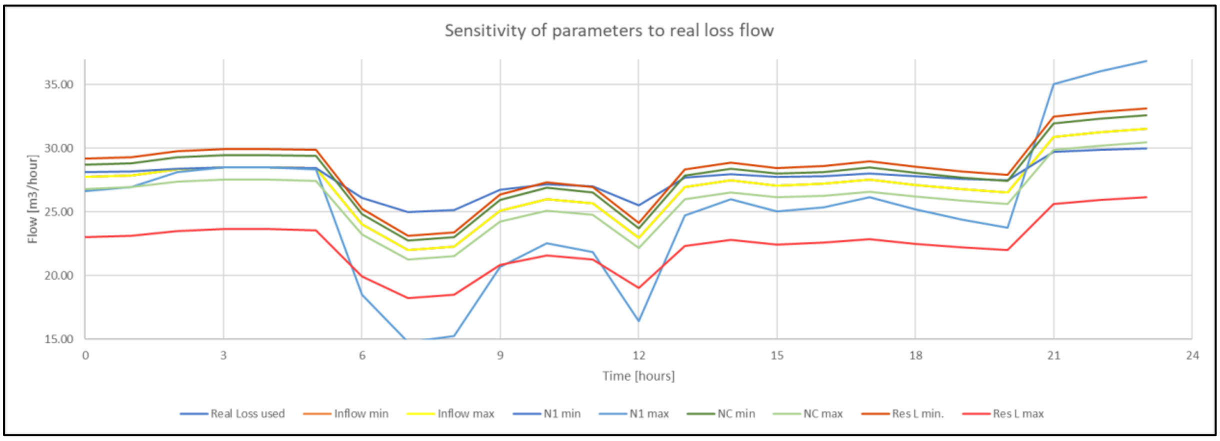



*Figure 31 Sensitivity of parameters to calculated real loss flow. Leakage exponent N1 and Residential Leakage have biggest influence on real loss flow.*

### 6.5.3. Critical Factors (F.7.)

Based on the DMA approach, a transition strategy is developed to move to CWS within the whole of GAMA. Based on the fieldwork, sensitivity analysis and results from the questionnaire, critical factors for applying this approach are observed. An overview is given of these factors that are critical to make the transition to CWS in a DMA in table 12. Table 13 shows the critical factors for expanding the DMA approach to a region (GAMA) or other countries.

| Critical factors | |
|---|---|
| Customers in GIS | DMA hydraulically isolated |
| Elevations in district | Network in GIS |
| SIV into district | Hydraulic model (based on the above) |
| Pressures within district | Clearly defined interventions |
| MNF measurements (multiple per year) | MCA of interventions. |
| Specific burst reports (pipeline, material, diameter, age, time, pressure, type of burst) | |

*Table 12 Critical factors to make transition to CWS inside a DMA.*

| Criteria crucial to expansion of DMA approach: | |
|---|---|
| **Availability of:** | Network data (spatially) |
| | Flow data |
| | Pressure data |
| | Trained staff |
| **Know-how and training on:** | Hydraulics (incl. modelling) |
| | Leakage reduction |
| | Maintenance of network and interventions |
| | Decision making interventions |
| **Willingness to:** | Invest (time, resources & know-how) |
| | Review and abandon old practices (if necessary) |
| | Benchmark performance of the DMAs |
| **Mandate to:** | Make CWS a standard in all districts by management |

*Table 13 Critical factors to make transition to CWS when expanding DMA approach to a region or other countries.*

To determine the feasibility of the DMA approach, the results of the sensitivity analysis, questionnaire held among water operators and the critical factors are synthesized into the last part; Applicability Elsewhere (F.8.). This synthesis is described in the conclusion.

### 6.6. Overall Result

In summary, the overarching result of this study is that CWS can be implemented in a DMA. Furthermore, the additional volume of water that can be saved in this district by reducing real losses with pressure management allows this water to be transported to neighbouring districts and increase their supply conditions. The effect of interventions on improved supply conditions in other districts could not be hydraulically modelled. This resulted that the economic impact of the intervention on a whole region could not be assessed as well.

# 7. Discussion

This chapter critically reflects on the results from this study and how they were reached. Firstly, the general methodology is discussed. Secondly, the application of the methodology to the case in Accra, Ghana is discussed and structured in themes; development of method, data collection, data analysis and interpretation and modelling. At the end a summary is given on the points of discussion.

## 7.1. Discussion on the Development of the General Methodology

The general method that was developed did not previously exist. Since one of the main causes of IWS is leakage, the best practices from the field of leakage were applied in the development of the methodology. DMAs are the standard for measuring and controlling real losses. Therefore, the methodology was developed in such a way that it incorporates DMAs and best practices from the field of NRW. Much has been written about NRW and leakage, which is well summarized and explained in the manual of Ziegler, et al. (Ziegler et al., 2011). This manual, which is a collection best practices of relevant literature, was used to develop the methodology. However, an import factor is that almost all leakage theory developed for a CWS system or assumes CWS.

The practical contribution of this work lies in the combination of leakage theory, reduction fo pressure inefficiencies and IWS by the development of a general methodology. In the methodology, the 'Types of IWS Framework' was developed by me and was combined with the requirements to determine these types of IWS inside and outside DMAs results in an approach to transition to CWS from a demand-side perspective. The framework was developed by observing leakage theory from an intermittent supply perspective, which resulted in a decision tree. This decision tree helps to select the type of IWS, and its suitable interventions based on leakage and distribution theory.

Only recently a paper was published on the implications of IWS on the assessment of leakage in distribution networks by Al-Washali, et al. (AL-Washali et al., 2020). Their work focussed on giving insight in the different methodologies to assess leakage in water distribution systems and the effect of IWS on these different leakage assessments. Previously it was known that IWS influenced these assessments, the extend was still unknown. Recommendations from the work of Al-Washali could not be adopted, since we did the leakage assessments months prior to their publication.

Furthermore, in a yet to be published article, Achi, et al. described the simulation to achieve gradual continuous supplies in different districts (El Achi & Rouse, 2020). In their approach they only used pressure management to transition to CWS. This resulted in improved supply conditions in some of their districts. The limitation of their model was that it did not take into account the water losses. It is interesting that their criteria for successful transition to CWS almost overlap the set of criteria gathered in this research. The major difference was their focus on social aspects as well in these criteria. One of their conclusions is that 'transition (to CWS) necessitates the presence of DMAs' (El Achi & Rouse, 2020).

## 7.2. Discussion on the Application of the Methodology to the Case Accra, Ghana

### 7.2.1 General

The method that was developed in this study relied heavily on availability of data (flow, pressure and billing) as well as PDD modelling software. Unfortunately, both data and modelling software proved to be insufficient to develop a detailed strategy for the roll out of the bottom up (demand-side) DMA approach.

Only in one DMA (Amasaman) flows and pressures could be recorded due to the time constraint and availability of other resources (finances, staff, ultrasonic flowmeters). If more DMAs could have been measured, it is expected that a more accurate picture of the GAMA situation would have been obtained.

Furthermore, GWCL districts have a maximum of 10,000 connections which is larger than the IWA advised maximum of 3,000 connections for a DMA. For this approach this will not be harmful, since in the design of the DMAs measures were taken to create multiple sub-DMAs inside the districts. These sub-DMAs will be required when active leakage control measures will be implemented.

### 7.2.1. Data Collection- C

Flow measurements were initially planned for seven days. However, one extra pipeline was feeding Nsawam district from Amasaman (outgoing). The valve on this line was closed Friday 08/16 at 08:00 a.m. Therefore, this moment was used as a starting point for the measurements. Power-outages and a weak internal battery from the (older) Flexim F601 ultrasonic flowmeter (south-side) caused no flows to be registered several times throughout the measuring interval. Therefore, in case no flow was registered at one of the flowmeters, for over 30 min., flow was not considered during these times.

A clear saturation point for the district could not be observed. This could be due to the relatively small number of domestic customers. Since Amasaman is a relatively small district, it was assumed that the saturation point was reached within the five days before the MNF sampling, but this was not proven.

During the fieldwork, pipes were discovered that were not registered in GIS and sometimes unknown to GWCL staff. For Amasaman the district boundaries were physically checked twice for this occurrence. When such pipes are still present in Amasaman, this can possibly bloat the volume of real losses that were measured in the district.

The billing data that was received from GWCL for several districts was not up to data. Since Amasaman and Santo are new and smaller districts there is only one person assigned to take meter readings. This means that sometimes not all connections have had a meter reading each month, which influences monthly billing values.

The EasyCalc top-down NRW assessment per district is based on estimates from district managers and distribution officers and not on physical measurements. This causes the output to have a high degree of uncertainty.

### 7.2.2. Data Analysis and Interpretation - D

The MNF shows high SIV and low billing for Amasaman district. Although unlikely, since it was checked twice, this could indicate another pipe going outside of the district unnoticed.

Although Amasaman district was assumed to be saturated at time of MNF, residential night consumption might be higher, due to filling of private reservoirs and spillage that can occur in case this tank does not have a working ball valve. This will decrease the real loss flow, previously calculated.

The rate of rise parameters could not be determined due to a lack of measurements. Assuming a rate of rise was not considered a good solution, due to the large spectrum of values for rate of rise.

When measured, multiple measurements over a longer time span (months, preferable years) are required to calculate the rate of rise of real losses for the district. Therefore, EIF could not be calculated and no good estimates for the cost of the reduction of physical losses could be made.

When IWS in district based on billing data might not give the full picture. It indicates the minimum level of IWS, the actual level might be higher.

#### 7.2.3. Data Modelling - E

The DEM data was not accurate at locations with high rise buildings. Elevation of model needed adjustment by using GPS sensors.

Hydraulic modelling software WaterNetGen did not function properly. Coefficients and exponents needed to be set for both burst leakage and background leakage. While trying to fit the modelled real losses to the calculated real losses by changing these parameters, the pressure dependency did not change. It was observed that the exponents for both burst and background leakage were not functioning properly. While trying to model either burst leakage or background leakage by turning of the other parameters, the outcome did not change accordingly.

Real losses were modelled as total leakage with EPANET's emitter coefficients, which could not split total leakage into background losses, unreported and reported bursts which is essential when the effects of leakage reduction measures need to be appreciated. The difference between modelled and calculated leakage in the night and morning is pressure dependent, modelled flow tends to react stronger to strong pressure fluctuations. The only way to modify this would be to change the exponent N1, but since N1 is calculated and represents Amasaman district specifically, it is not to be changed. Therefore, the model is validated for the modelled leakage flow. And it was difficult to attain a high level of accuracy for total leakage flow, when only the leakage coefficient C can be factorized. Finally, EPANET emitter coefficients do not work correctly when negative pressures occur in the network.

The network data from April 2019 was used to construct a hydraulic model from. However, network expansions within the district were made from that moment until November 2019, these expansions were not covered in the hydraulic model set up. The model set up and validation is assumed to be still valid since these were minor adjustments.

For Amasaman District only pressure management was modelled and no other real loss reduction measures since the latter depends on year-long measurements that were not available, as well as hard to verify burst and background leakage parameters. The district is supplied by gravity from a reservoir at the production plant at Nsawam. So, there was no need for an extra reservoir within the district to improve pressure levels and have emergency reserves. It would only be beneficial to have more control over the domestic demand, when water is stored by the utility and not by all the customers, but this does not need to be within the district itself.

Due to the unavailability of data for each district in GAMA and a proper working hydraulic PDD software, the chained DMA model could not be developed for GAMA. Although efforts have been made to build this model under the circumstances, the model cannot predict interventions that are required for each district, the effects of those interventions on other districts and ultimately the optimal investment strategy to introduce CWS in GAMA.

#### 7.2.4. Feasibility of DMA Approach - F

The questionnaire that was held was only responded by four different water utilities from four different nations. The goal of the questionnaire was to receive feedback from a very operational and engineering perspective. Therefore, this does not allow for easy replicability for a thorough scientific application. The main reason the DMA approach is not ready to be implemented for each of them is since none of them have operational DMAs. This might be a disadvantage of this approach, since developing and maintaining DMAs is resource intensive.

Leakage parameter N1 is very sensitive. Based on calculations N1 for the district should be 0.98, however most of the pipe material is plastic and therefore N1 should be around 1.5 according to Van Zyl (Van Zyl, 2014). It is unlikely that the maximum value for N1 will become 2.5 in the district, therefore the sensitivity of N1 is likely to be overestimated.

# 8. Conclusions

In this study a method was developed to transition water supply systems from IWS to CWS using a demand-side DMA based approach. We tested the method in the case study in Accra, Ghana. From the development and testing of this method, several conclusions can be drawn. Section 8.1 describes the conclusions on the general methodology, section 8.2 the conclusions on the application of the methodology to the case of Accra, Ghana and section 8.3 concludes on the feasibility of the methodology.

## 8.1 Conclusions on the General Methodology

The study shows that gains in water savings at the DMA level to more than justify the costs of transitioning to CWS. In cases of high pressure and high real loss volumes, pressure management improves supply conditions for customers. And, after attaining CWS, water recovered from loss reductions can be redistributed into neighboring districts, increasing their water availability.

Hydraulic Pressure Dependent Demand modelling does not accurately model leakage. Although the PDD theory on demand and lekage is clear and precise, PDD models like WaterGems, WaterNetGEN, EPANET 2.0 and the recently released EPANET 2.2 are not able to model domestic demand and leakage flows separately. Separate modelling is required for assessing the impact of interventions on leakage flow. The examined models are not mature and do not allow for easy adaptation by water utilities in developing countries, where human capacity is often lacking to develop such models themselves.

Due to the incapability to model and calibrate leakage pressure dependently, interventions that focussed on leakage reduction could not be modelled. And the unavailability of fundamental flow and pressure data on a district level also made it impossible to develop hydraulic models to analyse the interrelationships between different districts based on their interventions and the effects of the interventions on surrounding districts.

Leakage parameters that are required to model the volume of water that can be saved in each DMA are hard to verify with field measurements. Currently, methods exist on how to obtain leakage parameters easily, such as, burst component and exponent as well as background component and exponent. However, we were not able to apply these measuring campaigns with many sensors in this study. This lack of valid leakage parameters impeded the development a model for a specific (set of) district(s).

## 8.2 Conclusions on the Application of the Methodology for the Case of Accra

Currently, two districts experience varying degrees of intermittent water supply. IWS in Adenta is ultimately a management decision, forced by the mismanagement of the dedicated transmission lines, which inhibits Accra Reservoir to be filled when Adenta receives water. Amasaman experiences unreliable IWS, caused by shut-offs of the treatment plant due to maintenance, operational and power-outages and high leakage rates.

Amasaman district has high level of NRW in terms of volume (26,587$m^3$/month) and ratio (72%) and experienced at least 26% IWS in terms of volume based on billing accounts. An additional 2,300 $m^3$/month is required to introduce CWS based on domestic demand. Real losses accounted for 19,429 $m^3$/month, which was 188% of the billed volume. The type of IWS in Amasaman was 'Type 4: Costly Real Losses', for which IWS can be avoided when real losses are reduced. Therefore, domestic demand management does not have to be applied in Amasaman district.

GWCL has not prioritized CWS in its districts in GAMA. The focus is on extending services and increasing its coverage to a larger number of people. This comes at the expense of ensuring CWS, since the same volume of water available needs to be shared with more customers. Furthermore, the knowledge of the hydraulic capacity of their assets is inadequate, causing hydraulic bottlenecks to appear at different locations within the transport and distribution networks and incorrect solutions to be posed.

Although multiple interventions, increasing storage capacity, pressure and demand management and reduction of real losses, were considered, it was not possible to conduct a meaningful decision analysis when one type of IWS had different solutions, because 1) increasing storage capacity was not considered when the district was supplied by gravity and 2) accurate hydraulic leakage modelling based on the leakage reduction intervention was not possible and could therefore not be appraised.

The water supply of Amasaman district was modelled and validated. Only pressure management was modelled as an intervention. Pressure management can save 140 ML/year in the district and the placement of a PRV will have a positive NPV of over $400,000 in ten years. With pressure management, intermittency in the district can be reduced and might give the district CWS, although this could not be tested due to time restrictions.

GAMA districts currently do not exist as DMAs. Therefore, for each district a top down NRW assessment was used to determine the initial KPIs for each district. Since data from 23 of the 33 districts was not (fully) made available by GWCL, the initial full-scale strategy for GAMA could not be developed. A selection was made based on high pressures and high supply times for the district with available data from the Tema region. It could be concluded that Santo, Kpong Akuse and Tema Industrial are key districts that could yield quick gains in terms of water saved. These districts require flow and pressure measurements in order to cross-check the top-down NRW assessment and to determine suitable interventions to save water, which can then be distributed to other districts with lower supply times (higher intermittency). The impact of the interventions can then be monitored and benchmarked with the EasyCalc assessments for each district, as well as hydraulic flow and pressure readings throughout GAMA

It is uncertain how the planning of interventions for the whole of GAMA can be done over time in an efficient and (cost) effective way and its impact measured, since this depends on many parameters that would require more cutting-edge (hydraulic) modelling software that was used in this research and then currently available.

### 8.3 Conclusions on the Feasibility of the Methodology - F

The methodology that was developed and applied in Accra, has certain conditions in order to be successful. Firstly, data on flow and pressure from various districts is a prerequisite before applying this method. Furthermore, there needs to be a management environment where old practices are reviewed and abandoned if necessary and know-how and training is provided and integrated within the water utility.

To improve modelling of the districts, the sensitivity should be reduced by getting more accurate values for leakage exponent N1 and more accurate data on residential leakage. This can be done by field measurements and a house to house survey.

### 8.3.1. Applicability Elsewhere (F.8.)

Based on our interviews with different water operators, relevant literature, field experience in Accra, Ghana and a list of critical factors that are required to use the general methodology, we conclude that the methodology developed in this research can be applied in different water supply circumstances. A zonation of the distribution network and gathering of flow and pressure data are prerequisites to apply this approach elsewhere. Furthermore, a mandate from management has to be given to introduce interventions and review and abandon old practices in case they are hurting supply conditions.

## 8.4. Conclusions on the Final Result

The developed bottom-up DMA approach proves to be a good concept to transition to CWS. However, for this case study, it was not possible to apply the full demand-side DMA approach method. In the future, when PDD modelling has matured and GWCL has more data on each individual district, this method is expected to give good insight in the relevant investments GWCL has to plan in order to enjoy a continuous supply in GAMA.

# 9. Recommendations

Recommendations are made regarding the outcome of this study. Advice is given on how to improve the general methodology and specific recommendations are given regarding the case of Accra, Ghana. Finally, the overall work is personally reflected upon and recommendations are made regarding future research for others wanting to pick up a topic within the IWS theme.

## 9.1. Recommendations Regarding the General Methodology

New research should focus on developing an open source hydraulic pressure dependent model where different interventions can be modelled per district. Python WNTR package could be used as a base to develop a working PDD modelling software, that includes more detailed, easy to use leakage parameters. This model could be developed and tested in a region which is divided in DMA who have sufficient flow and pressure data.

More research is required to develop appropriate methods to assess background and burst leakage parameters. These should be developed in an easy and straightforward way to make them easy to adopt for utilities with limited human capacity.

## 9.2. Recommendations Regarding the Case of Accra, Ghana

GWCL should start measuring leakage levels in all districts at several times throughout the year and to develop a detailed database on burst events. This way back and burst leakage levels can be assessed for each district as well as the supply conditions. And sampling of residential night leakage assessments is recommended to improve modelling of real losses.

Amasaman District is expanding its water coverage rapidly. This puts more pressure on the existing network and water availability and will increase the occurrence and length of IWS. This increases the importance of reducing leakage levels so water which is saved can be directly be used by customers. Domestic demand management might be required to secure CWS for the whole district and GAMA in the future.

It is recommended focusing on the implementation of this DMA approach to transition to continuous water supply, before adding additional production capacity at the top. Eventually, due to the growth of GAMA additional capacity needs to be considered. Smart networks can be developed that can enhance the insight and understanding of supply conditions throughout the network. And the impact of interventions can be better monitored this way both upstream and downstream of the intervention.

To further increase the performance of the network and to ensure CWS, it is recommended to create a 'Distribution Decision Department' at the head office level, with a mandate to nullify plans that are not hydraulically sound. The department ensures economic diameters of pipes to be constructed, optimal pressures and heads of hydraulic objects. This would save energy costs and improve supply conditions. With the GAMA Masterplans recommendations in mind, the department can take the lead in the planning and design of extra extensions to customers, production capacity to be created and improved bulk water supply.

A house to house (HtH) survey was developed for Amasaman district. However, the field survey was not conducted, since GWCL and VEI thought it best to wait and include an illegal connection survey as well. It is recommended to start implementing HtH surveys at districts with high apparent losses vs. real losses ratio, since it will greatly improve the cost recovery for GWCL. The HtH survey can be found in Appendix L.

Santo District will experience IWS due population growth and urbanization, therefore it is recommended to change the supply directions and construct a reservoir on the north side of the district, further explained in Appendix J. Other specific recommendations are made to GWCL and VEI and can be found in Appendix K.

### 9.3. Reflection and Recommendations on Overall Work

While reflecting the overall work that was performed in this thesis, I would have done many things differently. With the knowledge and experience I have at this moment; I would have first looked better into pressure dependent modelling and the separation between domestic demand and pressure dependent real losses. I would have spent more time initially with experts in the field of modelling and (software) programming to develop an initial working PDD model that can distinguish between the two. Furthermore, I would have focussed more on developing specific easy to adopt methodologies for measuring real loss components such as background and burst flows as well as rate of rise parameters.

Once this model was developed and the leakage components can be measured easily, I would apply it to a case study where data on DMAs such as SIV, real loss levels and domestic demand is more readily available then in this case Accra, Ghana. The model can then be updated and upgraded by including interventions and a financial assessment of the costs and benefits of these interventions. With this mature model, the situation of GAMA can then be more thoroughly studied.

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# Appendix

## Appendix A: CWS requirements

A detailed description is given for the requirements to reach CWS by the DMA approach.

1) Divide WDN into districts.
   a. Requirements for districts: between 500 and 3,000 connections, minimum elevation difference, due to pressure variations (<50m), establish physical boundaries and one feeding source.
   b. Keep in mind the redundancy and vulnerability of the network.
   c. Use graph method and integrate with pressure management areas

*Approach:*
Start with district with highest pressures and highest supply times (preferably 24/7) and work in downstream direction (hydraulically) to other districts, with high supply time and lower pressures, after which districts are targeted with lower supply times and higher pressures. The assumption is that too much water enters districts with 24/7 supply and high pressures and interventions have a high impact there.

2) Isolate district hydraulically
   a. Close valves, end cap pipes
3) Place flow meters at inlet (and outlet), as well as pressure loggers at the flow meter locations and at points within district to assess AZP (preferably on main feeding lines into/within the district)
4) Assess status-quo of district
   a. Supply time and pressures
   b. Flow into district
   c. MNF assessment (Be aware: higher error due to intermittency)
   d. NRW calculation: Extrapolate average flow to monthly values and subtract billing for the month
   e. Cross checking top down NRW assessment with MNF assessment gives insight in:
      i. Real losses
      ii. Apparent losses
   f. Demand pattern
   g. Customer demand → from billing data
      i. Domestic
      ii. Commercial
      iii. Industrial
      iv. Government
      v. Water bottling/sachet
5) Determine physical bottlenecks for improvement of supply time.
   a. Pressure too high
   b. Pressure too low
   c. Real losses too high

    d. SIV too high
    e. SIV too low
    f. Storage within district too low
6) Determine adequate interventions to reduce / discard bottlenecks
    a. Pressure Management
        i. Modulation → PRV
        ii. Storage → Reservoir
    b. Buffer capacity → Reservoir
        i. Shaving off peak factors
        ii. Stable supply
    c. Active Leakage Control, combined with high quality and speed of repairs
    d. Replacement of (old/worn) meters
    e. House to House survey to assess illegal connections and missing connections in billing system. Update billing system.
7) Determine impact, cost and requirements (time, staff, material dependencies, etc.) per intervention
8) Pre-assess impact of interventions by hydraulic and economic modelling
    a. Supply time
    b. Pressures
    c. SIV
    d. Volume Billed
    e. Real losses
    f. Apparent losses
9) Plan interventions
10) Monitor new/updated state
11) Maintain network, district and metering. Update databases (GIS, billing, flow, pressures) regularly

*Approach:*

After step 7, the input is generated for the neighbouring districts. Based on this input, these districts can then be developed and modelled in the same fashion. This way, all districts undergo the transition iteratively. The districts cannot transition all at the same time since the districts are interconnected by flow, pressure, geographically and logistically. Water that is saved in one district will increase pressures and availability of water going into the other districts. This will result in higher SIV (possibly), higher supply times, higher real losses and higher apparent losses (pressure dependent part and increase in metering errors) within the other districts.

## Appendix B: Overview districts

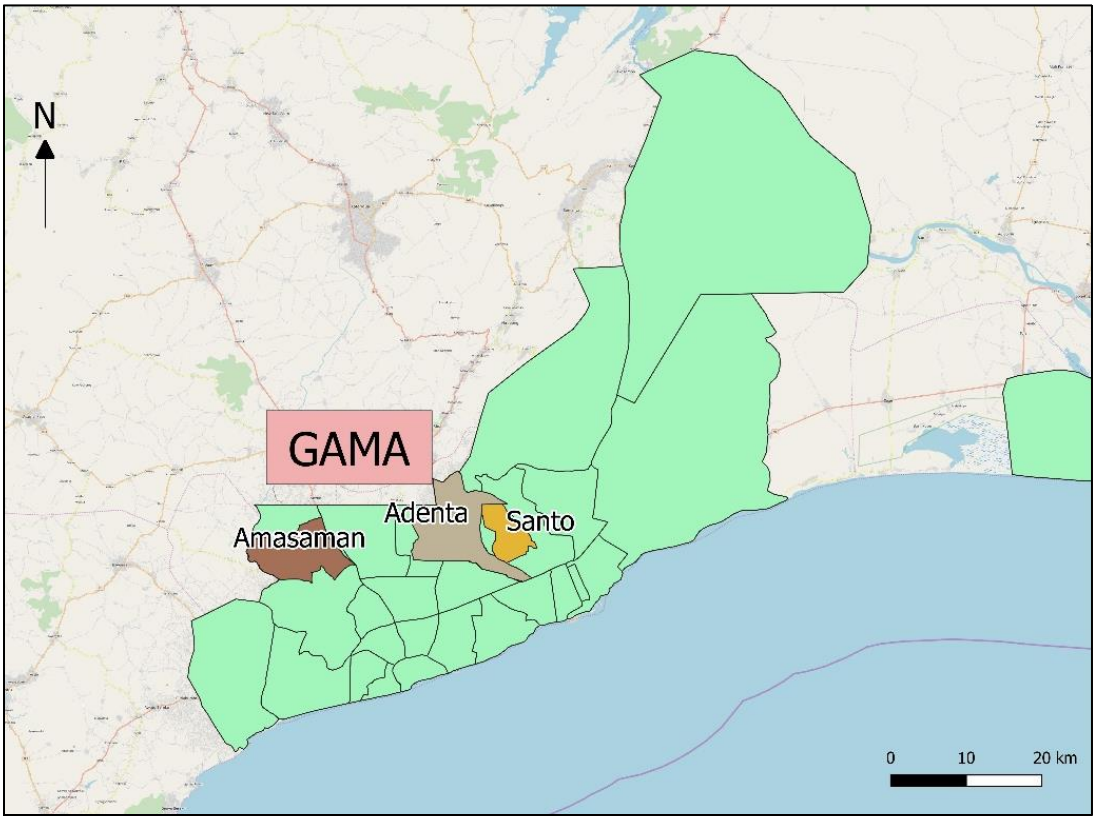



*Figure 32 Location of districts inside GAMA*

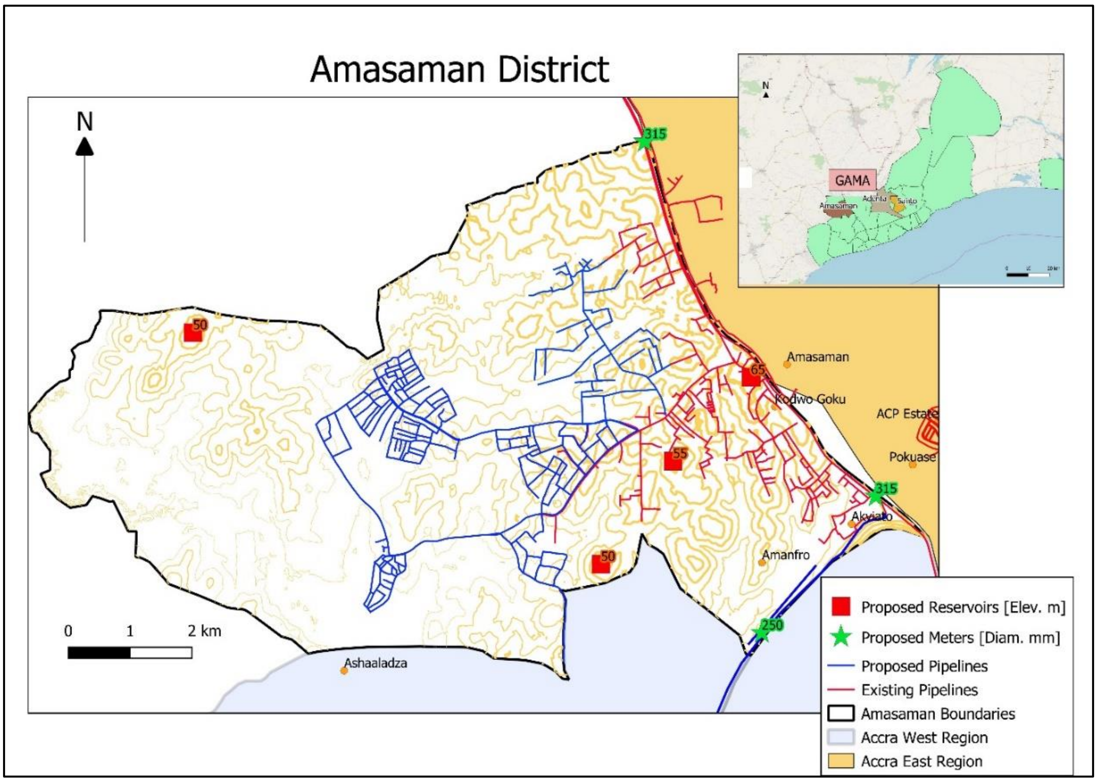



*Figure 33 Proposed pipelines by the district/region, including contour lines*

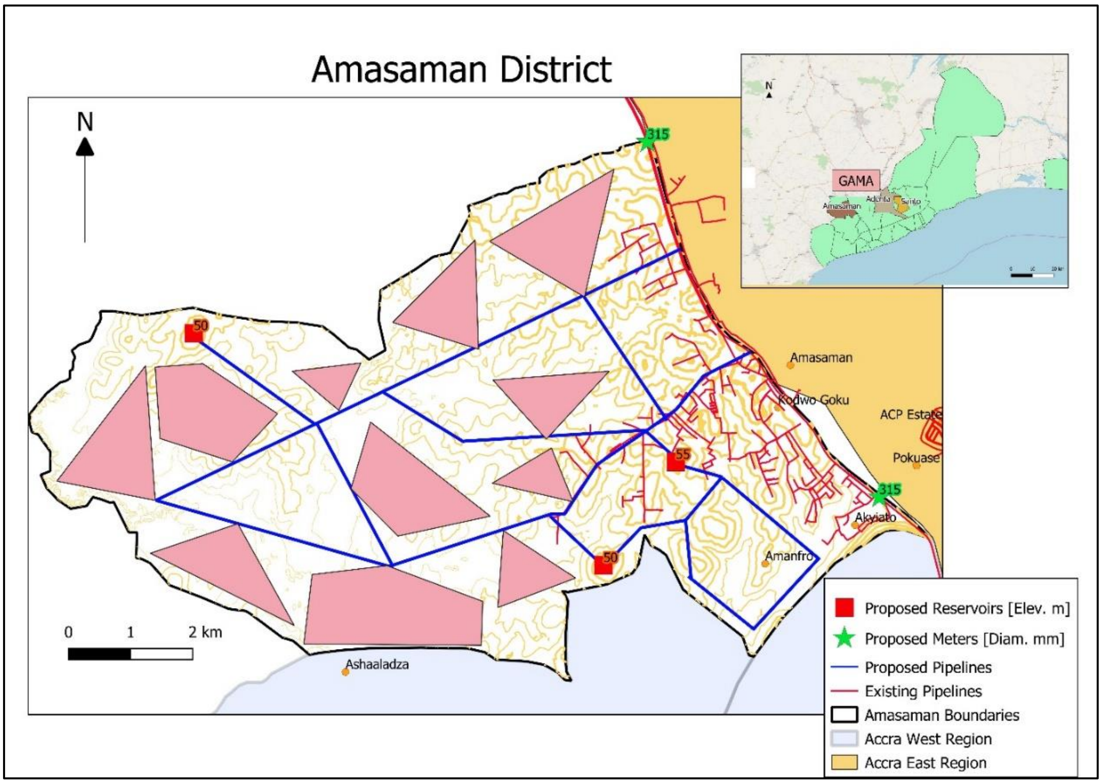



*Figure 34 Backbone structure and reservoirs*

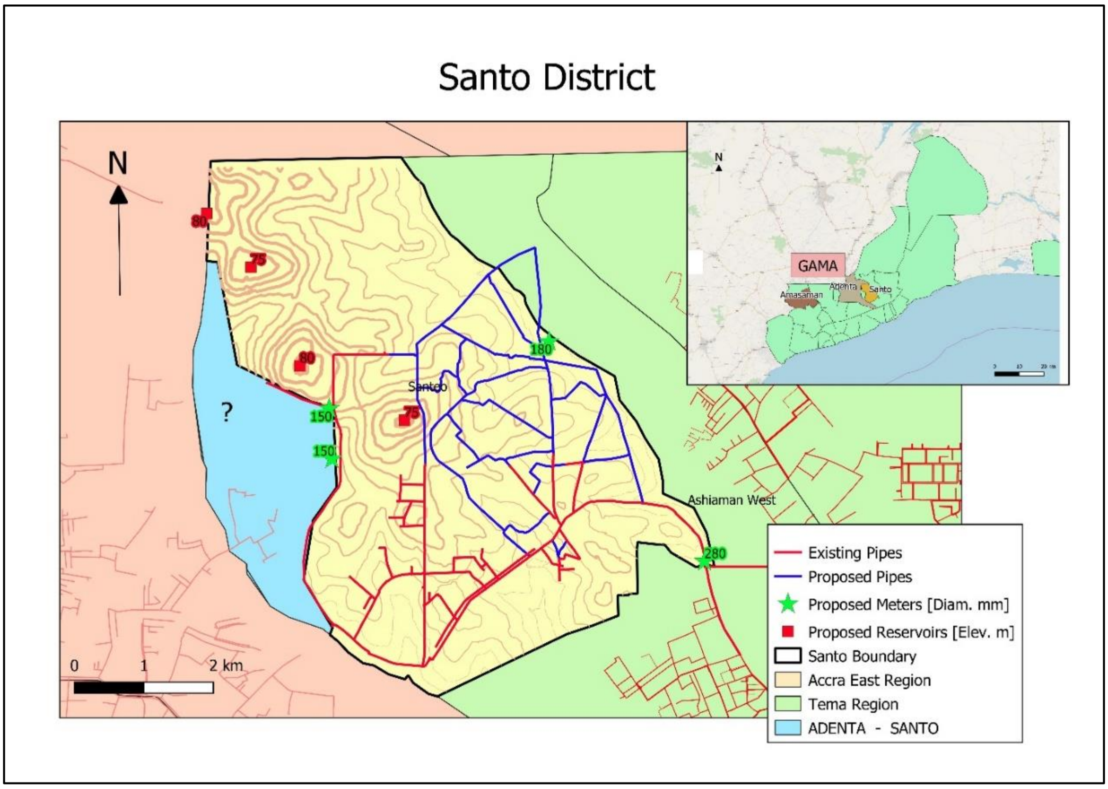



*Figure 35 Current and proposed lines in Santo*

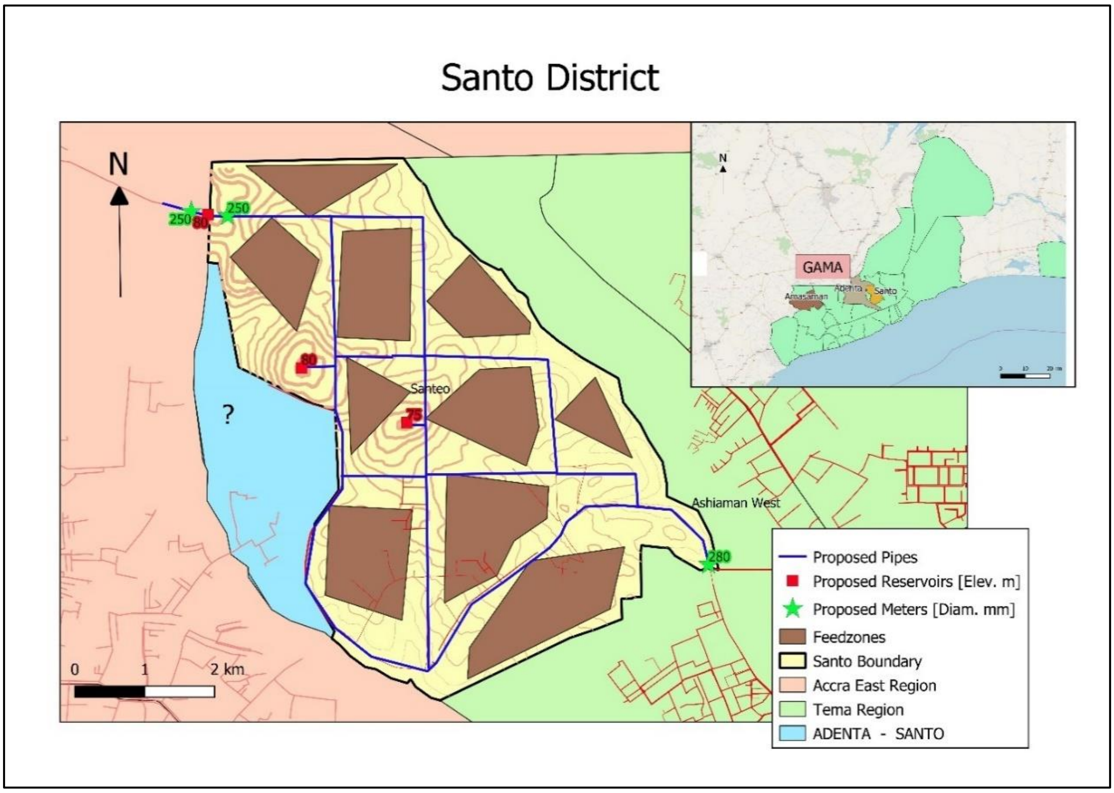



*Figure 36 Proposed backbone structure and feeding point from Dodowa*

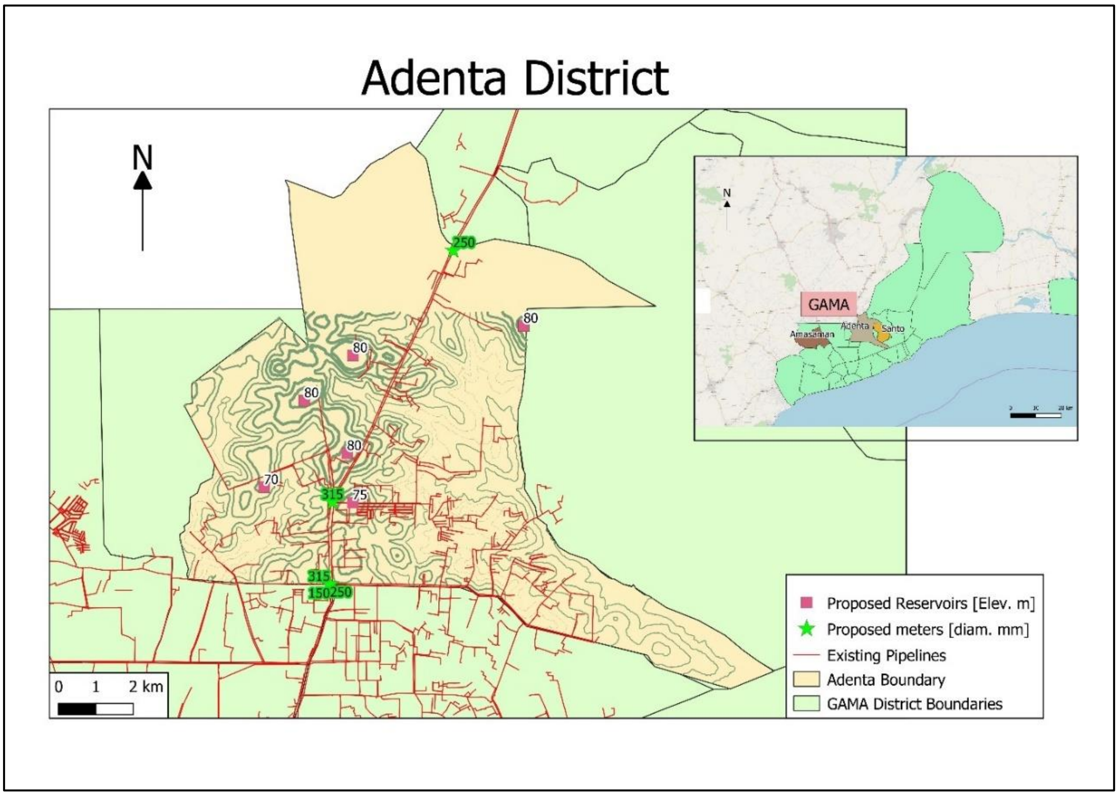



*Figure 37 Adenta overview*


*Figure 38 Possibilities for DMA's within Adenta. [connections per polygon]*

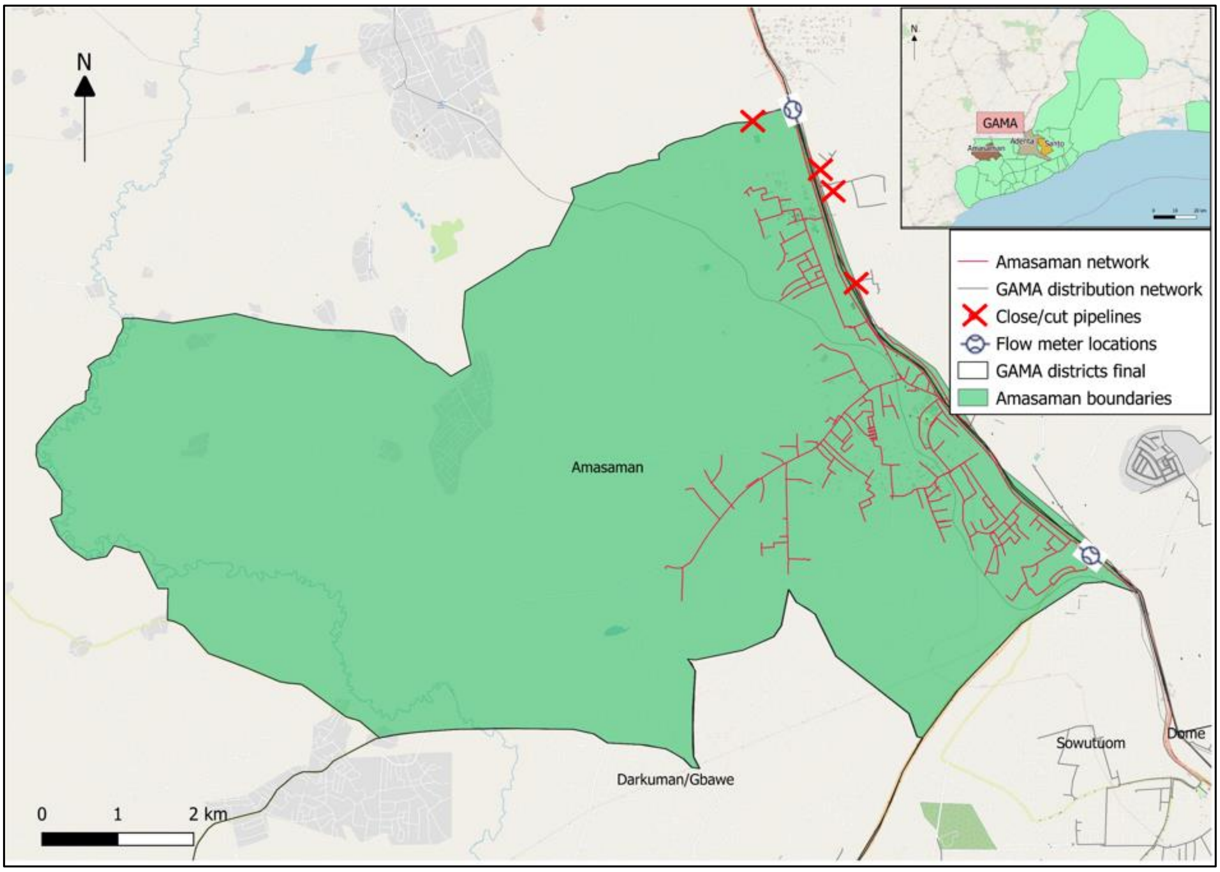
## Appendix C: DMA Overview of Santo and Adenta

*DMA Requirements Amasaman*

To construct a DMA for the Amasaman District, the District needs hydraulic isolation. The current situation is straightforward. When 5 people per connection are assumed, the District has the potential to have 25,000 connections currently, without regard for population growth and urbanization.

Requirements:

- Offtakes on the 315mm need to be cut-off (currently three). These lines feed 500 meters into East Accra region and are connected to the 150mm line parallel to N6 highway. For this region it is recommended to eventually replace this parallel line for a bigger diameter.
- EM flowmeter (on 315mm HDPE) at the northern boundary
- EM flowmeter (on 315mm PVC) in the eastern boundary, before BOI connection (400mm HDPE) and Sowutuom connection (400mm HDPE).
- Valves throughout the district for step-testing
- District will split in the future, therefore plan and design accordingly.




**Santo**

*DMA requirements*

Measures required to establish a DMA of Santo district are:

- Cut off 180mm line feeding into Ashiaman West district or place meter on the line
- Cut off 150mm line feeding into Adenta district
- Decide on the boundaries on the West and North.
- Place EM meter on the 280mm feeding the district.
- Place valves throughout district for step-testing

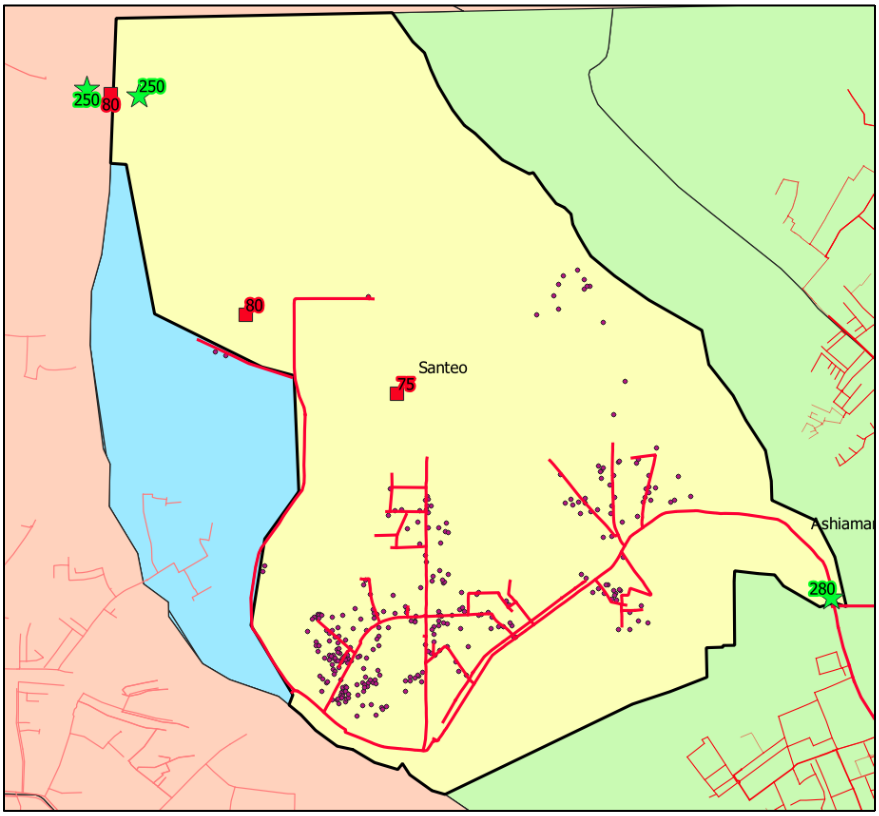



*Figure 39 Santo - Customer Connections. One flow meter needed for the southern 280mm line. In case a reservoir is constructed in the north, two additional flow meters are required there as well.*

*DMA requirements Adenta*

- Disconnect connections and offtakes on the 150mm pipeline coming in at Ecowash road west. Construct a new line within the district, where the connections can be made.
- 250mm pipeline at Ecowash road east needs to be isolated.
- Ritz-junction challenges need to be solved
    - Develop pressure reduction strategy
    - Analysis of hydraulic performance of district and subzones
- One offtake (150mm) on the 600mm to BOI reservoir needs to be cut and connected to the 200mm north of it.
- Boundary between Adenta and Santo on the East and North side needs to be clear. It is decided to use current Santo boundary, as a boundary for both districts.
- Depending on the above, multiple (2 to >10) flowmeters are required to measure the flow into the district.
- Split district in at least two others, since it has over 10,000 connections.

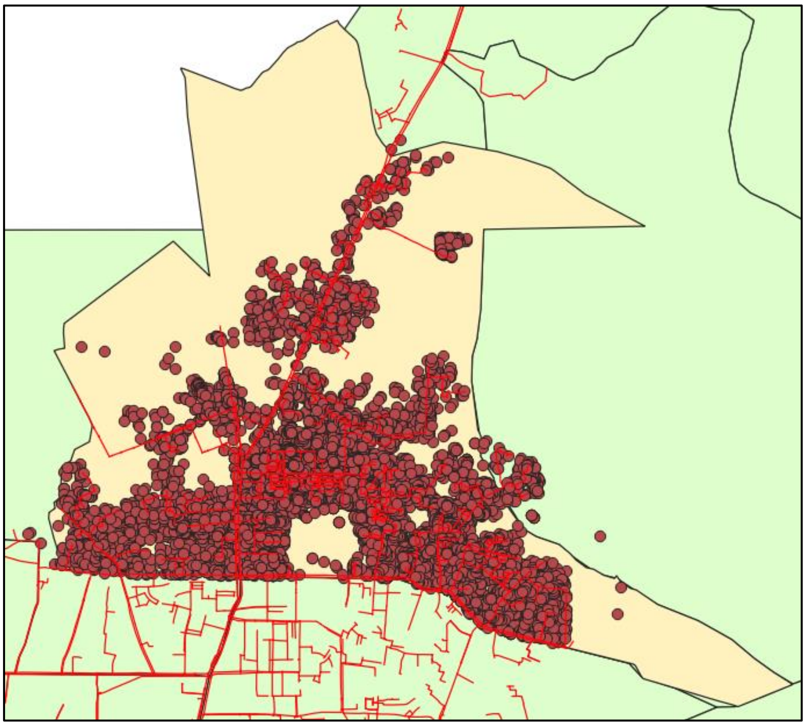


*Figure 40 Adenta - Customer Connections*

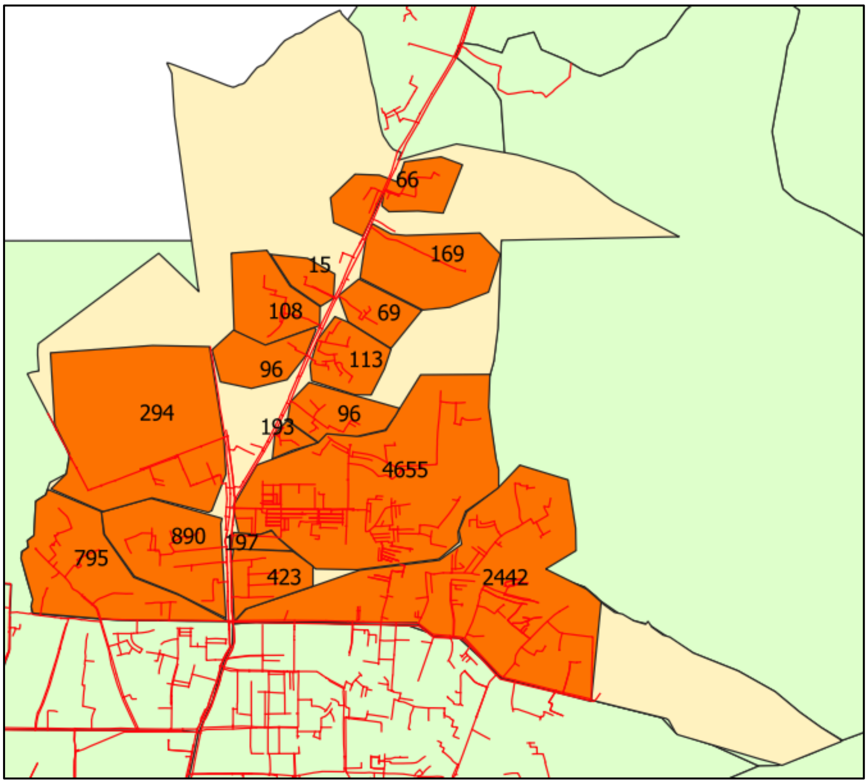
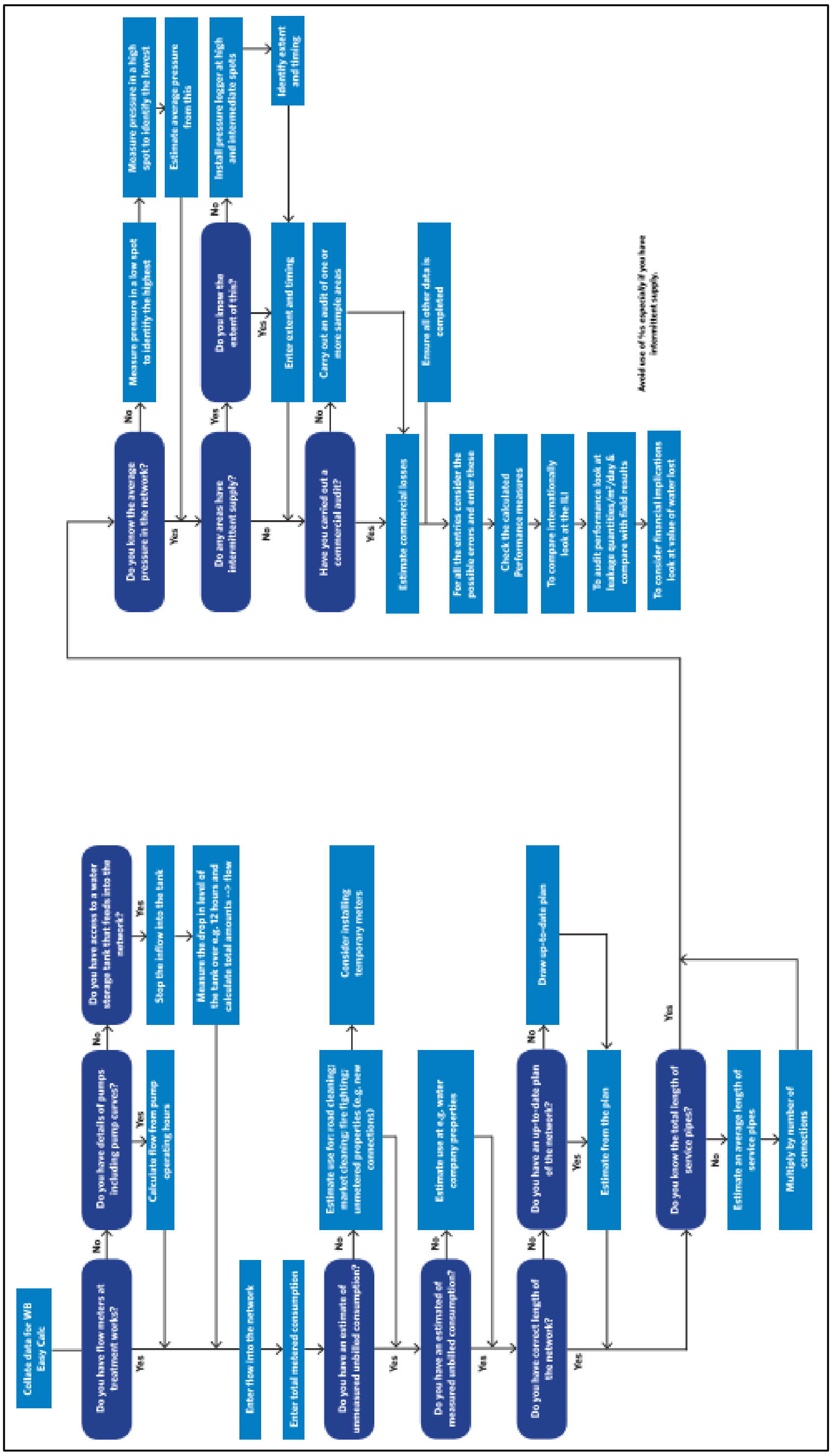



*Figure 41 Adenta - Connections per feeding point/line*

## Appendix D: Top-Down NRW Assessment Procedure Overview. (Baghirathan & Parker, 2017)



# Appendix E: EasyCalc Top Down Assessment Districts

## Performance Indicators

### Level of Service

| | Best Estimate | Error Margin [+/- %] | Lower Bound | Upper Bound |
|---|---|---|---|---|
| Average Supply Time [h/day] | 24.0 | 5% | 22.8 | 24.0 |
| Average Pressure [m] | 25.0 | 5% | 23.8 | 26.3 |

Home

### Volume of Physical Losses

| | Best Estimate | Error Margin [+/- %] | Lower Bound | Upper Bound |
|---|---|---|---|---|
| CAPL - Current Annual Volume of Physical Losses [m3/day] | 200 | 87% | 26 | 374 |
| MAPL - Minimum Achievable Volume of Physical Losses [m3/day] | 25 | 2% | 24 | 25 |

### Physical Loss Performance Indicators

| | Best Estimate | Error Margin [+/- %] | Lower Bound | Upper Bound |
|---|---|---|---|---|
| Infrastructure Leakage Index (ILI) | 8 | 87% | 1 | 15 |
| Liters per Connection per Day (w.s.p.)<br>w.s.p.: when the system is pressurized - this means the value is already corrected in the case of intermittent supply | 492 | 87% | 63 | 922 |
| Liters per Connection per Day per meter Pressure (w.s.p.) | 20 | 87% | 2 | 37 |
| m3/km mains per hour (w.s.p.) | 0.27 | 87% | 0.03 | 0.50 |

**Performance Group**

| Standard | Low and Middle Income Countries |
|---|---|
| D | C |
| Explanations | Explanations |

### Commercial Loss Performance Indicators

| | Best Estimate | Error Margin [+/- %] | Lower Bound | Upper Bound |
|---|---|---|---|---|
| Commercial Losses expressed in % of Authorized Consumption | 1% | 71% | 0% | 1% |
| liters/connection/day | 7 | 42% | 4 | 11 |
| liters/customer/day | 8 | 42% | 4 | 11 |

### NRW Performance Indicators

| | Best Estimate | Error Margin [+/- %] | Lower Bound | Upper Bound |
|---|---|---|---|---|
| Volume of Non-Revenue Water expressed in % of System Input Volume | 36% | 84% | 6% | 66% |
| Value of Non-Revenue Water expressed in % of Annual Operating Cost | 141% | 84% | 22% | 260% |
| Liters per Connection per Day (w.s.p.)<br>w.s.p.: when the system is pressurized - this means the value is already corrected in the case of intermittent supply | 508 | 84% | 79 | 937 |

**Performance Group**

| Standard | Low and Middle Income Countries |
|---|---|
| D | C |
| Explanations | Explanations |

*Figure 42 EasyCalc supply and leakage KPIs Santo*

## Performance Indicators

Home

### Level of Service

| | Best Estimate | Error Margin [+/- %] | Lower Bound | Upper Bound |
|---|---|---|---|---|
| Average Supply Time [h/day] | 24.0 | 5% | 22.8 | 24.0 |
| Average Pressure [m] | 65.5 | 5% | 62.2 | 68.8 |

### Volume of Physical Losses

| | Best Estimate | Error Margin [+/- %] | Lower Bound | Upper Bound |
|---|---|---|---|---|
| CAPL - Current Annual Volume of Physical Losses [m3/day] | 728 | 19% | 593 | 863 |
| MAPL - Minimum Achievable Volume of Physical Losses [m3/day] | 136 | 6% | 129 | 144 |

### Physical Loss Performance Indicators

| | Best Estimate | Error Margin [+/- %] | Lower Bound | Upper Bound |
|---|---|---|---|---|
| Infrastructure Leakage Index (ILI) | 5 | 19% | 4 | 6 |
| Liters per Connection per Day (w.s.p.)<br>w.s.p.: when the system is pressurized - this means the value is already corrected in the case of intermittent supply | 871 | 25% | 656 | 1,085 |
| Liters per Connection per Day per meter Pressure (w.s.p.) | 13 | 25% | 10 | 17 |
| m3/km mains per hour (w.s.p.) | 0.45 | 19% | 0.37 | 0.54 |

Performance Group

| Standard | Low and Middle Income Countries |
|---|---|
| C | B |
| Explanations | Explanations |

### Commercial Loss Performance Indicators

| | Best Estimate | Error Margin [+/- %] | Lower Bound | Upper Bound |
|---|---|---|---|---|
| Commercial Losses expressed in % of Authorized Consumption | 45% | 36% | 28% | 61% |
| liters/connection/day | 185 | 39% | 113 | 258 |
| liters/customer/day | 319 | 36% | 204 | 434 |

### NRW Performance Indicators

| | Best Estimate | Error Margin [+/- %] | Lower Bound | Upper Bound |
|---|---|---|---|---|
| Volume of Non-Revenue Water expressed in % of System Input Volume | 72% | 14% | 62% | 82% |
| Value of Non-Revenue Water expressed in % of Annual Operating Cost | 97% | 14% | 84% | 111% |
| Liters per Connection per Day (w.s.p.)<br>w.s.p.: when the system is pressurized - this means the value is already corrected in the case of intermittent supply | 1,060 | 15% | 904 | 1,217 |

Performance Group

| Standard | Low and Middle Income Countries |
|---|---|
| D | C |
| Explanations | Explanations |

*Figure 43 Easycalc supply and leakage KPIs Amasaman*

## Performance Indicators

Home

### Level of Service

| | Best Estimate | Error Margin [+/- %] | Lower Bound | Upper Bound |
|---|---|---|---|---|
| Average Supply Time [h/day] | 17.1 | 5% | 16.3 | 18.0 |
| Average Pressure [m] | 65.5 | 5% | 62.2 | 68.8 |

### Volume of Physical Losses

| | Best Estimate | Error Margin [+/- %] | Lower Bound | Upper Bound |
|---|---|---|---|---|
| CAPL - Current Annual Volume of Physical Losses [m3/day] | 11,974 | 50% | 5,971 | 17,976 |
| MAPL - Minimum Achievable Volume of Physical Losses [m3/day] | 749 | 6% | 704 | 794 |

LM

NC

LP

### Physical Loss Performance Indicators

| | Best Estimate | Error Margin [+/- %] | Lower Bound | Upper Bound |
|---|---|---|---|---|
| Infrastructure Leakage Index (ILI) | 16 | 50% | 8 | 24 |
| Liters per Connection per Day (w.s.p.)<br>w.s.p.: when the system is pressurized - this means the value is already corrected in the case of intermittent supply | 1,189 | 51% | 588 | 1,790 |
| Liters per Connection per Day per meter Pressure (w.s.p.) | 18 | 51% | 9 | 27 |
| m3/km mains per hour (w.s.p.) | 10.42 | 50% | 5.17 | 15.68 |

**Performance Group**

| Standard | Low and Middle Income Countries |
|---|---|
| D | C |
| Explanations | Explanations |

### Commercial Loss Performance Indicators

| | Best Estimate | Error Margin [+/- %] | Lower Bound | Upper Bound |
|---|---|---|---|---|
| Commercial Losses expressed in % of Authorized Consumption | 6% | 34% | 4% | 9% |
| liters/connection/day | 34 | 34% | 23 | 46 |
| liters/customer/day | 36 | 33% | 24 | 48 |

### NRW Performance Indicators

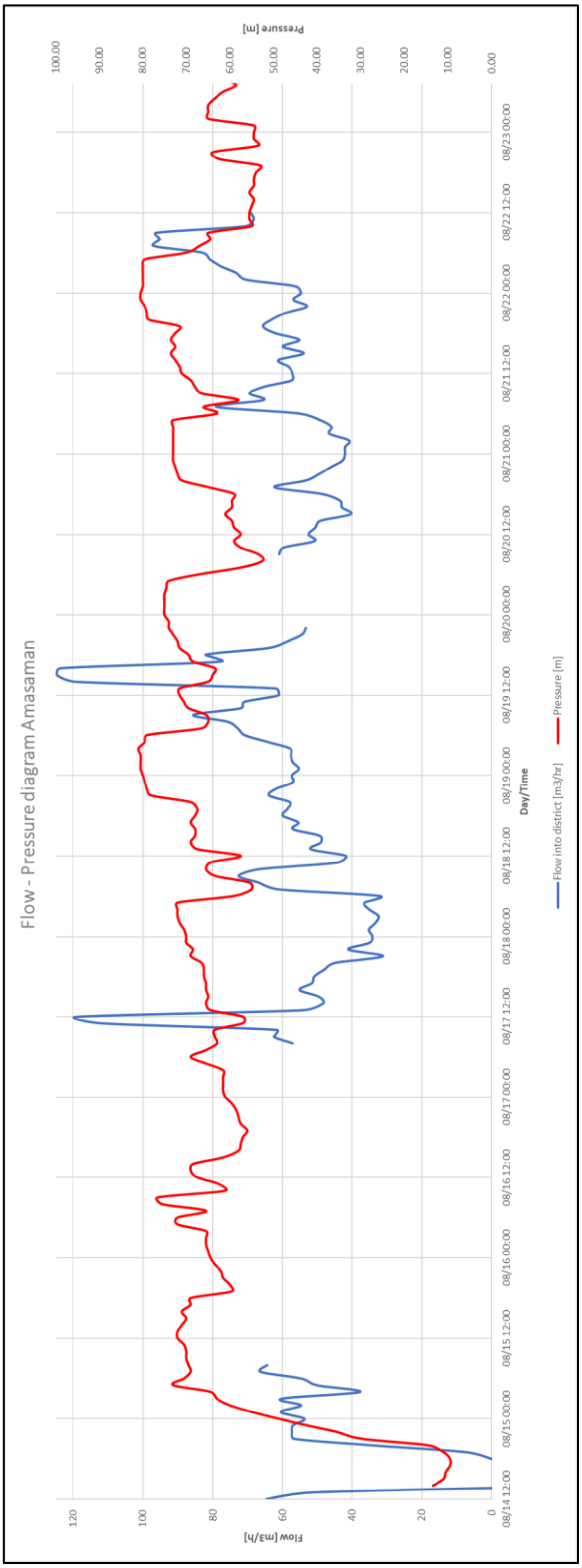
| | Best Estimate | Error Margin [+/- %] | Lower Bound | Upper Bound |
|---|---|---|---|---|
| Volume of Non-Revenue Water expressed in % of System Input Volume | 63% | 48% | 33% | 93% |
| Value of Non-Revenue Water expressed in % of Annual Operating Cost | 633% | 48% | 330% | 936% |
| Liters per Connection per Day (w.s.p.)<br>w.s.p.: when the system is pressurized - this means the value is already corrected in the case of intermittent supply | 1,244 | 48% | 645 | 1,843 |

**Performance Group**

| Standard | Low and Middle Income Countries |
|---|---|
| D | D |
| Explanations | Explanations |

*Figure 44 Easycalc supply and leakage KPIs Adenta*

# Appendix F: Flow and Pressure Measurements



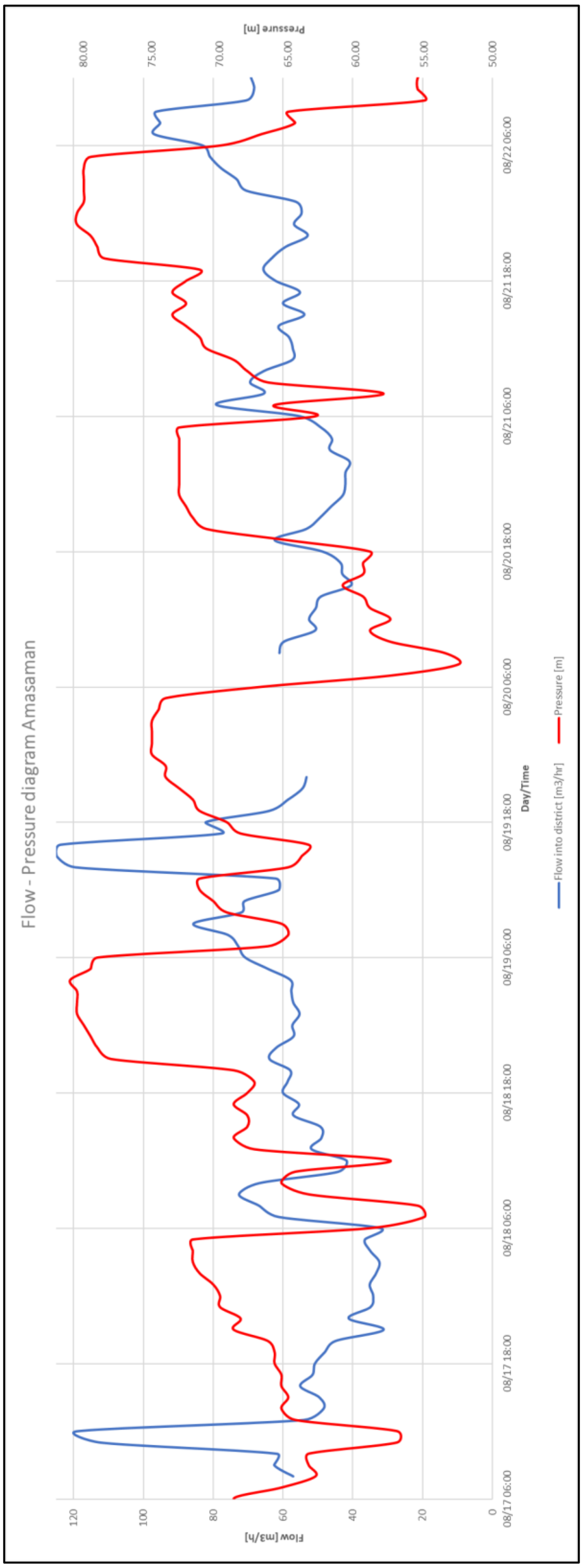




*Figure 45 Flow and Pressure diagrams. Left shows whole measurement period. Right shows only period when outflowing pipe to Nsawam District was shut at 17th 08:00.*

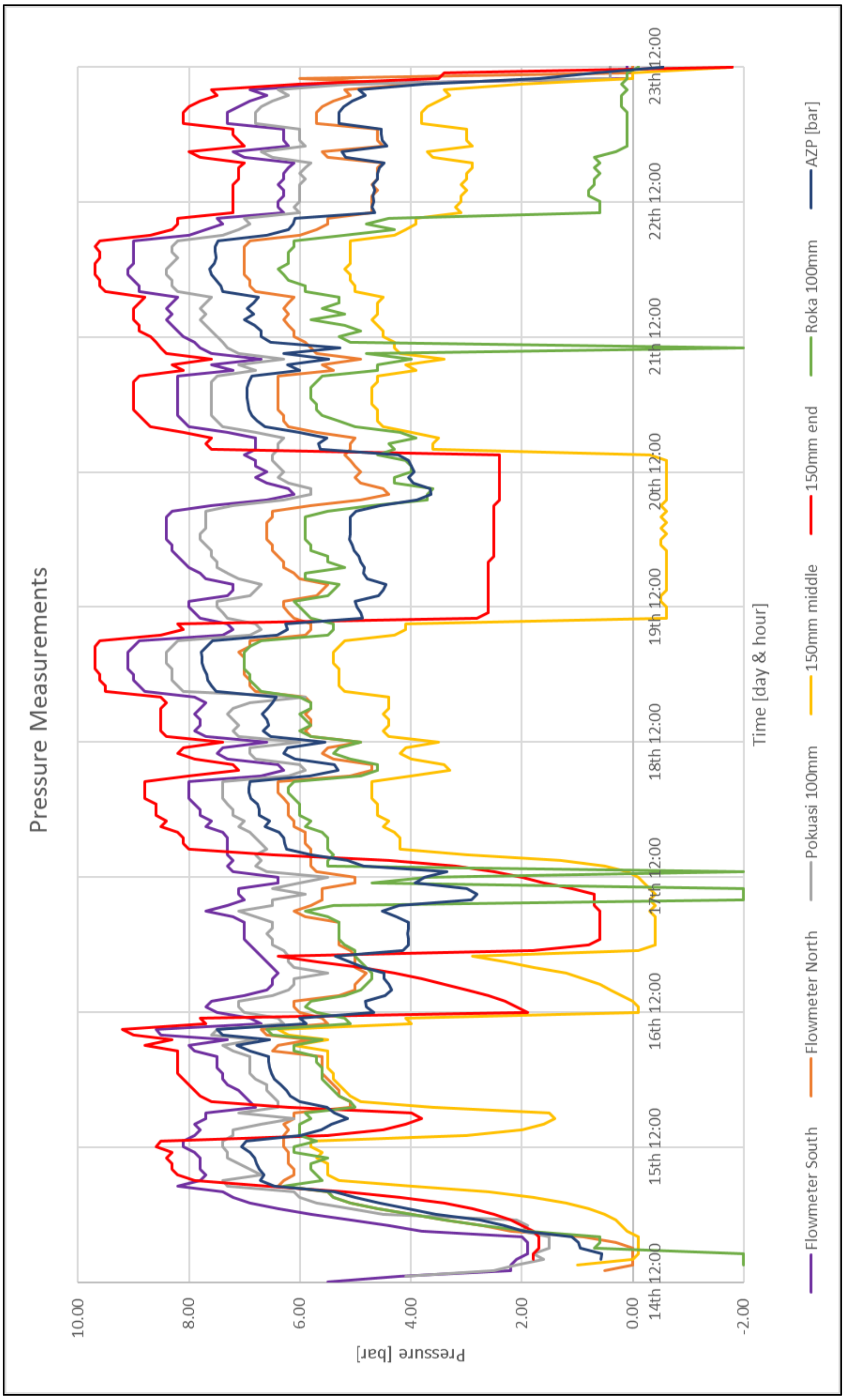



*Figure 46 Pressure measurements. Loggers were not able to measure below -2.0 bar. Big pressure variations due to bursts and closure of pipes at several days.*

## Appendix G: PDD Test Model

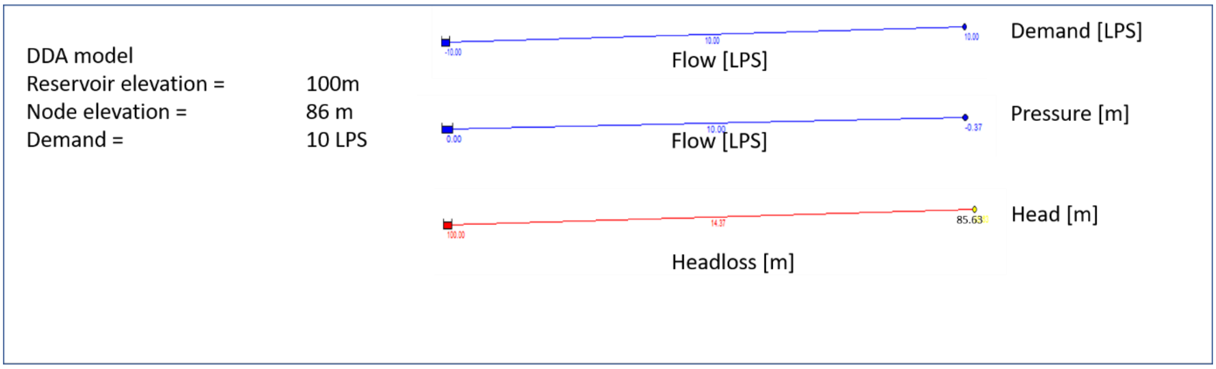



*Figure 47 Demand Driven Analysis test model.*

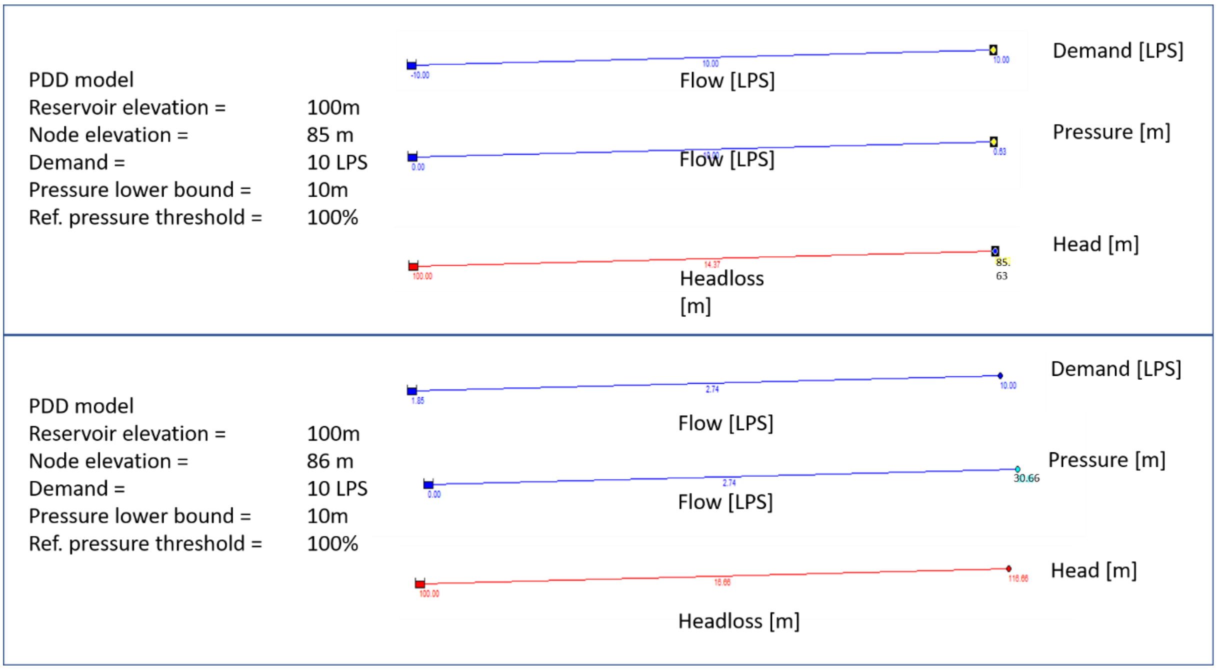



*Figure 48 Pressure Dependent Demand test model for WaterNetGen. At 85 meters elevation, model works as DDA. At 86 meters elevation, the head is much larger than physically possible.*

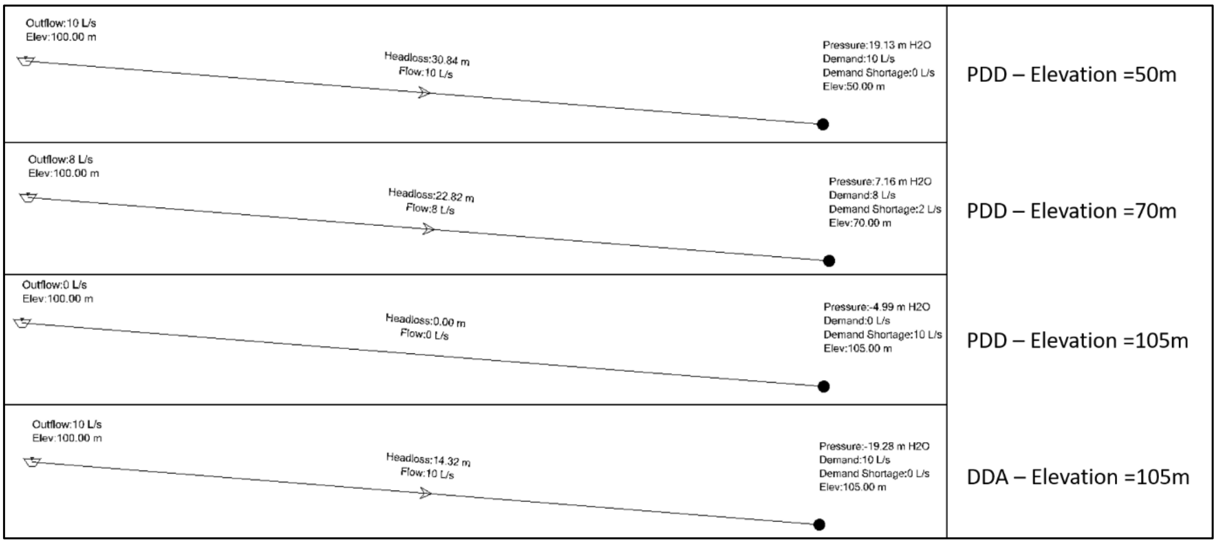
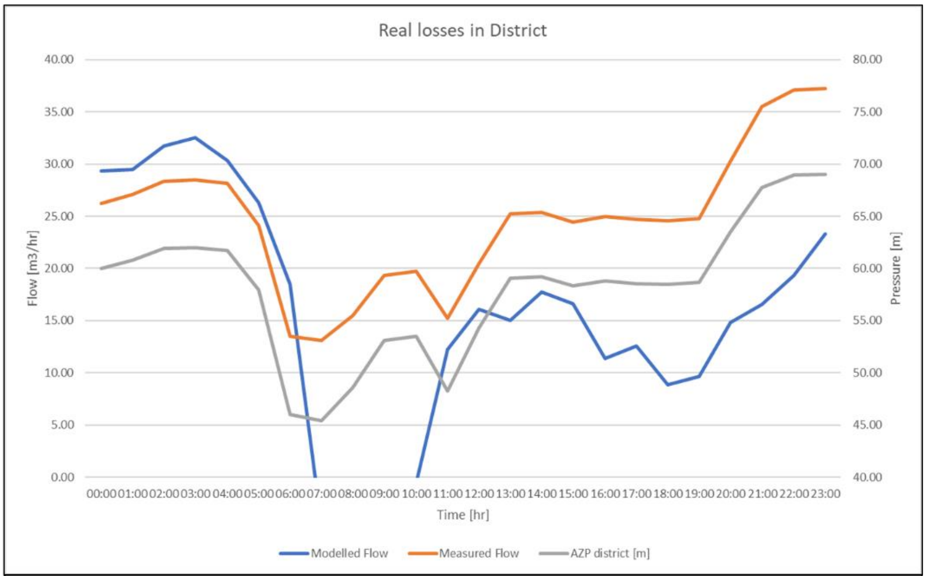



*Figure 49 Demand Driven and Presure Dependent Demand test model for WaterGEMS. Model works fine for PDD.*

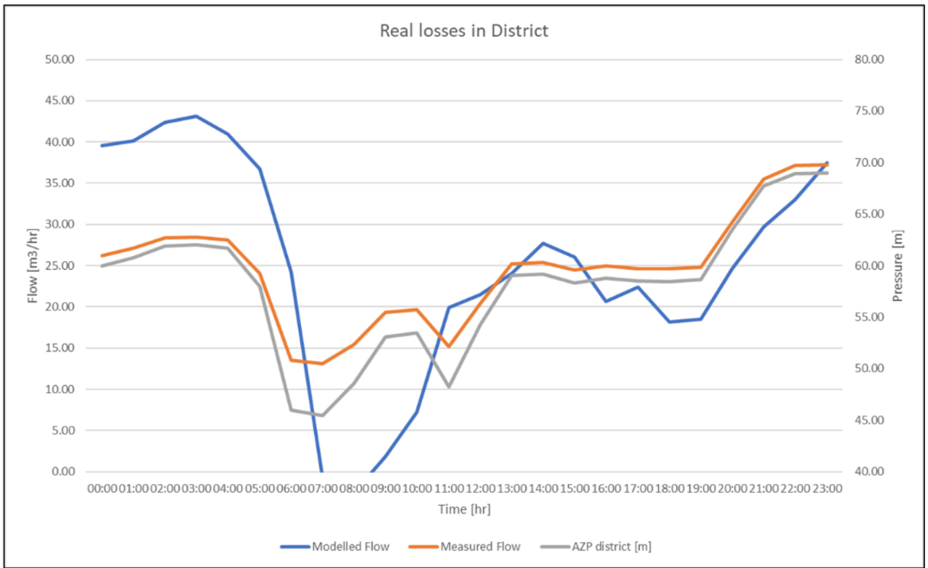
## Appendix H: Leakage Modelling WaterNetGen

Based on figure 56, 57 and 58 it can be concluded that WaterNetGen does not properly function under PDD modelling conditions. When background and burst coefficients and exponents are used, between the proper boundaries, the leakage cannot be fitted to the calculated leakage. Only when all the leakage parameters are set to zero, the model is able to model leakage correctly (Figure 58).



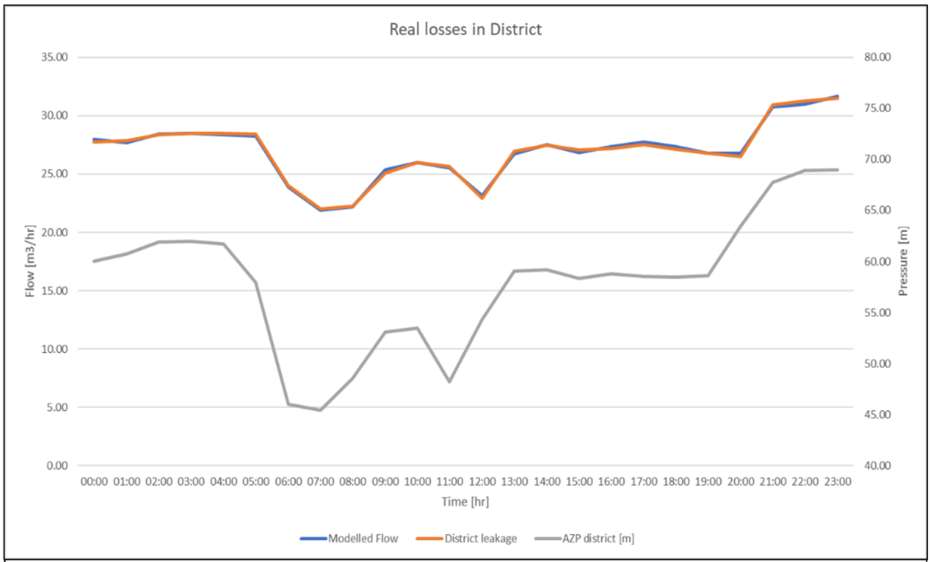

| | | |
|---|---|---|
| Background coef | 0.00000001 | Volume real losses modelled = 593.32 m3/day |
| Background exp | 1.2 | Volume real losses measured = 596.62 m3/day |
| Burst coef | 0.3 | |
| Burst exp | 0.000001 | |

*Figure 50 Modelled leakage volume with WaterNetGen.*




| | | |
|---|---|---|
| Background coef | 0.00000001 | Volume real losses modelled = 593.32 m3/day |
| Background exp | 1.0 | Volume real losses measured = 596.62 m3/day |
| Burst coef | 0.005 | |
| Burst exp | 1.0 | |

*Figure 51 Modelled leakage volume with WaterNetGen.*




| | | |
|---|---|---|
| Background coef | 0.0 | Volume real losses modelled = 593.32 m3/day |
| Background exp | 0.0 | Volume real losses measured = 596.62 m3/day |
| Burst coef | 0.0 | |
| Burst exp | 0.0 | |

*Figure 52 Modelled leakage volume with WaterNetGen.*

# Appendix I: EPANET Input Data

[JUNCTIONS]

| ;ID | Elev | Demand | Pattern | |
|---|---|---|---|---|
| No1 | 36 | 0.17218351 | 2 | ; |
| No2 | 32 | 0.17218351 | 2 | ; |
| No3 | 30 | 0.17218351 | 2 | ; |
| No4 | 49 | 0.17218351 | 2 | ; |
| No5 | 47 | 0.17218351 | 2 | ; |
| No6 | 47 | 0.17218351 | 2 | ; |
| No7 | 41 | 0.17218351 | 2 | ; |
| No8 | 49 | 0.17218351 | 2 | ; |
| No9 | 52 | 0.17218351 | 2 | ; |
| No10 | 47 | 0.17218351 | 2 | ; |
| No11 | 50 | 0.17218351 | 2 | ; |
| No12 | 47 | 0.17218351 | 2 | ; |
| No13 | 47 | 0.17218351 | 2 | ; |
| No14 | 53 | 0.17218351 | 2 | ; |
| No15 | 50 | 0.17218351 | 2 | ; |
| No16 | 50 | 0.17218351 | 2 | ; |
| No17 | 47 | 0.17218351 | 2 | ; |
| No18 | 55 | 0.17218351 | 2 | ; |
| No19 | 43 | 0.17218351 | 2 | ; |
| No20 | 47 | 0.17218351 | 2 | ; |
| No21 | 47 | 0.17218351 | 2 | ; |
| No22 | 53 | 0.17218351 | 2 | ; |
| No23 | 45 | 0.17218351 | 2 | ; |
| No24 | 44 | 0.17218351 | 2 | ; |
| No25 | 56 | 0.17218351 | 2 | ; |
| No26 | 45 | 0.17218351 | 2 | ; |
| No27 | 56 | 0.17218351 | 2 | ; |
| No28 | 50 | 0.17218351 | 2 | ; |
| No29 | 53 | 0.17218351 | 2 | ; |
| No30 | 51 | 0.17218351 | 2 | ; |
| No31 | 56 | 0.17218351 | 2 | ; |
| No32 | 59 | 0.17218351 | 2 | ; |
| No33 | 54 | 0.17218351 | 2 | ; |
| No34 | 58 | 0.17218351 | 2 | ; |
| No36 | 53 | 0.17218351 | 2 | ; |
| No37 | 51 | 0.17218351 | 2 | ; |

| | | | | |
|---|---|---|---|---|
| No38 | 52 | 0.17218351 | 2 | ; |
| No39 | 53 | 0.17218351 | 2 | ; |
| No40 | 53 | 0.17218351 | 2 | ; |
| No41 | 54 | 0.17218351 | 2 | ; |
| No42 | 53 | 0.17218351 | 2 | ; |
| No43 | 51 | 0.17218351 | 2 | ; |
| No44 | 63 | 0.17218351 | 2 | ; |
| No45 | 60 | 0.17218351 | 2 | ; |
| No46 | 57 | 0.17218351 | 2 | ; |
| No47 | 60 | 0.17218351 | 2 | ; |
| No48 | 58 | 0.17218351 | 2 | ; |
| No49 | 54 | 0.17218351 | 2 | ; |
| No50 | 47 | 0.17218351 | 2 | ; |
| No51 | 45 | 0.17218351 | 2 | ; |
| No52 | 47 | 0.17218351 | 2 | ; |
| No54 | 44 | 0.17218351 | 2 | ; |
| No55 | 51 | 0.17218351 | 2 | ; |
| No56 | 55 | 0.17218351 | 2 | ; |
| No57 | 47 | 0.17218351 | 2 | ; |
| No58 | 43 | 0.17218351 | 2 | ; |
| No59 | 55 | 0.17218351 | 2 | ; |
| No60 | 56 | 0.17218351 | 2 | ; |
| No61 | 59 | 0.17218351 | 2 | ; |
| No62 | 56 | 0.17218351 | 2 | ; |
| No63 | 58 | 0.17218351 | 2 | ; |
| No64 | 59 | 0.17218351 | 2 | ; |
| No65 | 62 | 0.17218351 | 2 | ; |
| No66 | 56 | 0.17218351 | 2 | ; |
| No67 | 54 | 0.17218351 | 2 | ; |
| No68 | 61 | 0.17218351 | 2 | ; |
| No69 | 52 | 0.17218351 | 2 | ; |
| No70 | 59 | 0.17218351 | 2 | ; |
| No71 | 57 | 0.17218351 | 2 | ; |
| No72 | 47 | 0.17218351 | 2 | ; |
| No73 | 57 | 0.17218351 | 2 | ; |
| No74 | 56 | 0.17218351 | 2 | ; |
| No75 | 43 | 0.17218351 | 2 | ; |
| No76 | 51 | 0.17218351 | 2 | ; |
| No77 | 52 | 0.17218351 | 2 | ; |
| No78 | 50 | 0.17218351 | 2 | ;Roka |
| No81 | 47 | 0.17218351 | 2 | ; |
| No82 | 48 | 0.17218351 | 2 | ; |

| | | | | |
|---|---|---|---|---|
| No83 | 52 | 0.17218351 | 2 | ; |
| No84 | 54 | 0.17218351 | 2 | ; |
| No85 | 54 | 0.17218351 | 2 | ; |
| No86 | 54 | 0.17218351 | 2 | ; |
| No87 | 61 | 0.17218351 | 2 | ; |
| No88 | 62 | 0.17218351 | 2 | ; |
| No89 | 60 | 0.17218351 | 2 | ; |
| No90 | 59 | 0.17218351 | 2 | ; |
| No91 | 59 | 0.17218351 | 2 | ; |
| No92 | 62 | 0.17218351 | 2 | ; |
| No93 | 58 | 0.17218351 | 2 | ; |
| No94 | 54 | 0.17218351 | 2 | ; |
| No96 | 61 | 0.17218351 | 2 | ; |
| No95 | 63 | 0.17218351 | 2 | ; |
| No97 | 62 | 0.17218351 | 2 | ; |
| No98 | 59 | 0.17218351 | 2 | ; |
| No99 | 65 | 0.17218351 | 2 | ; |
| No100 | 57 | 0.17218351 | 2 | ; |
| No101 | 63 | 0.17218351 | 2 | ; |
| No102 | 66 | 0.17218351 | 2 | ; |
| No103 | 66 | 0.17218351 | 2 | ; |
| No104 | 61 | 0.17218351 | 2 | ; |
| No105 | 62 | 0.17218351 | 2 | ; |
| No106 | 59 | 0.17218351 | 2 | ; |
| No107 | 44 | 0.17218351 | 2 | ; |
| No108 | 40 | 0.17218351 | 2 | ; |
| No109 | 40 | 0.17218351 | 2 | ; |
| No110 | 44 | 0.17218351 | 2 | ; |
| No111 | 38 | 0.17218351 | 2 | ; |
| No113 | 38 | 0.17218351 | 2 | ; |
| No112 | 46 | 0.17218351 | 2 | ; |
| No114 | 42 | 0.17218351 | 2 | ; |
| No115 | 49 | 0.17218351 | 2 | ; |
| No116 | 50 | 0.17218351 | 2 | ; |
| No117 | 46 | 0.17218351 | 2 | ; |
| No118 | 49 | 0.17218351 | 2 | ; |
| No119 | 55 | 0.17218351 | 2 | ; |
| No120 | 54 | 0.17218351 | 2 | ; |
| No121 | 51 | 0.17218351 | 2 | ; |
| No122 | 63 | 0.17218351 | 2 | ; |
| No123 | 49 | 0.17218351 | 2 | ; |
| No124 | 41 | 0.17218351 | 2 | ; |

| | | | | | |
|---|---|---|---|---|---|
| No125 | | 41 | 0.17218351 | 2 | ; |
| No126 | | 38 | 0.17218351 | 2 | ; |
| No127 | | 41 | 0.17218351 | 2 | ; |
| No128 | | 47 | 0.17218351 | 2 | ; |
| No129 | | 42 | 0.17218351 | 2 | ; |
| No130 | | 43 | 0.17218351 | 2 | ; |
| No131 | | 47 | 0.17218351 | 2 | ; |
| No132 | | 43 | 0.17218351 | 2 | ; |
| No133 | | 40 | 0.17218351 | 2 | ; |
| No134 | | 40 | 0.17218351 | 2 | ; |
| No135 | | 40 | 0.17218351 | 2 | ; |
| No136 | | 41 | 0.17218351 | 2 | ; |
| No137 | | 39 | 0.17218351 | 2 | ; |
| No138 | | 37 | 0.17218351 | 2 | ; |
| No140 | | 40 | 0.17218351 | 2 | ; |
| No141 | | 40 | 0.17218351 | 2 | ; |
| No142 | | 61 | 0.17218351 | 2 | ; |
| 1 | 57 | -54.32 | 7 | ;Flow North | |

[RESERVOIRS]

| ;ID | Head | Pattern | |
|---|---|---|---|
| 2 | 113.25 | 5 | ; |

[TANKS]

| ;ID | Elevation | InitLevel | MinLevel | MaxLevel | Diameter | MinVol | VolCurve | | Overflow |
|---|---|---|---|---|---|---|---|---|---|

[PIPES]

| ;ID | Node1 | | Node2 | | Length | Diameter | Roughness | MinorLoss | Status | |
|---|---|---|---|---|---|---|---|---|---|---|
| Pi1 | No1 | | No2 | 214.306564 | 105.100006 | 0.0048 | 0 | OPEN | ; | |
| Pi2 | No1 | | No3 | 435.610596 | 105.100006 | 0.0048 | 0 | OPEN | ; | |
| Pi3 | No4 | | No1 | 812.754089 | 105.100006 | 0.0048 | 0 | OPEN | ; | |
| Pi4 | No4 | | No5 | 636.666992 | 105.100006 | 0.0048 | 0 | OPEN | ; | |
| Pi5 | No6 | | No7 | 405.24704 | 105.100006 | 0.0048 | 0 | OPEN | ; | |
| Pi6 | No6 | | No8 | 189.744446 | 105.100006 | 0.0048 | 0 | OPEN | ; | |
| Pi7 | No9 | | No10 | 539.692383 | 105.100006 | 0.0048 | 0 | OPEN | ; | |
| Pi9 | No10 | | No6 | 395.531952 | 105.100006 | 0.0048 | 0 | OPEN | ; | |
| Pi8 | No10 | | No4 | 241.539078 | 105.100006 | 0.0048 | 0 | OPEN | ; | |
| Pi10 | No11 | | No12 | 194.891174 | 105.100006 | 0.0048 | 0 | OPEN | ; | |
| Pi11 | No11 | | No13 | 258.538635 | 105.100006 | 0.0048 | 0 | OPEN | ; | |
| Pi12 | No14 | | No11 | | 302.30481 | 105.100006 | 0.0048 | 0 | OPEN | ; |
| Pi13 | No14 | | No15 | | 99.42691 | 105.100006 | 0.0048 | 0 | OPEN | ; |
| Pi14 | No16 | | No14 | | 752.451782 | 105.100006 | 0.0048 | 0 | OPEN | ; |

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Pi15 | No17 | No16 | 219.6026 | 105.100006 | 0.0048 | 0 | OPEN | ; |
| Pi16 | No16 | No18 | 345.946503 | 105.100006 | 0.0048 | 0 | OPEN | ; |
| Pi17 | No9 | No19 | 323.853271 | 105.100006 | 0.0048 | 0 | OPEN | ; |
| Pi18 | No20 | No9 | 344.930664 | 105.100006 | 0.0048 | 0 | OPEN | ; |
| Pi19 | No21 | No17 | 172.501068 | 105.100006 | 0.0048 | 0 | OPEN | ; |
| Pi20 | No22 | No23 | 163.278305 | 105.100006 | 0.0048 | 0 | OPEN | ; |
| Pi21 | No22 | No24 | 222.70369 | 105.100006 | 0.0048 | 0 | OPEN | ; |
| Pi22 | No25 | No21 | 340.104126 | 154.799988 | 0.0048 | 0 | OPEN | ; |
| Pi23 | No17 | No20 | 42.396305 | 105.100006 | 0.0048 | 0 | OPEN | ; |
| Pi24 | No20 | No26 | 332.429077 | 105.100006 | 0.0048 | 0 | OPEN | ; |
| Pi25 | No21 | No22 | 134.893723 | 105.100006 | 0.0048 | 0 | OPEN | ; |
| Pi26 | No27 | No28 | 293.780426 | 105.100006 | 0.0048 | 0 | OPEN | ; |
| Pi27 | No29 | No30 | 55.582504 | 105.100006 | 0.0048 | 0 | OPEN | ; |
| Pi28 | No27 | No29 | 236.444885 | 105.100006 | 0.0048 | 0 | OPEN | ; |
| Pi29 | No29 | No31 | 244.79541 | 105.100006 | 0.0048 | 0 | OPEN | ; |
| Pi30 | No32 | No33 | 293.402954 | 105.100006 | 0.0048 | 0 | OPEN | ; |
| Pi31 | No32 | No34 | 142.972794 | 105.100006 | 0.0048 | 0 | OPEN | ; |
| Pi32 | No142 | No32 | 40.344173 | 105.100006 | 0.0048 | 0 | OPEN | ; |
| Pi33 | No36 | No142 | 417.24176 | 105.100006 | 0.0048 | 0 | OPEN | ; |
| Pi34 | No37 | No39 | 151.306931 | 105.100006 | 0.0048 | 0 | OPEN | ; |
| Pi36 | No39 | No38 | 783.044434 | 105.100006 | 0.0048 | 0 | OPEN | ; |
| Pi35 | No39 | No25 | 67.357857 | 154.799988 | 0.0048 | 0 | OPEN | ; |
| Pi37 | No25 | No27 | 261.995605 | 105.100006 | 0.0048 | 0 | OPEN | ; |
| Pi38 | No40 | No37 | 223.415894 | 154.799988 | 0.0048 | 0 | OPEN | ; |
| Pi39 | No41 | No42 | 52.758396 | 105.100006 | 0.0048 | 0 | OPEN | ; |
| Pi40 | No40 | No41 | 121.810951 | 105.100006 | 0.0048 | 0 | OPEN | ; |
| Pi41 | No41 | No43 | 176.513977 | 105.100006 | 0.0048 | 0 | OPEN | ; |
| Pi42 | No44 | No45 | 458.061584 | 105.100006 | 0.0048 | 0 | OPEN | ; |
| Pi43 | No46 | No47 | 175.952545 | 105.100006 | 0.0048 | 0 | OPEN | ; |
| Pi44 | No48 | No46 | 324.101166 | 154.799988 | 0.0048 | 0 | OPEN | ; |
| Pi45 | No46 | No36 | 69.245537 | 105.100006 | 0.0048 | 0 | OPEN | ; |
| Pi46 | No36 | No40 | 7.768437 | 105.100006 | 0.0048 | 0 | OPEN | ; |
| Pi47 | No44 | No49 | 716.749817 | 105.100006 | 0.0048 | 0 | OPEN | ; |
| Pi48 | No50 | No51 | 66.038017 | 105.100006 | 0.0048 | 0 | OPEN | ; |
| Pi49 | No52 | 1 | 2223.726318 | 277.600006 | 0.0762 | 0 | OPEN | ; |
| Pi50 | No54 | No55 | 171.46582 | 105.100006 | 0.0048 | 0 | OPEN | ; |
| Pi51 | No56 | No57 | 1642.445068 | 105.100006 | 0.0048 | 0 | OPEN | ; |
| Pi52 | No58 | No54 | 220.980606 | 105.100006 | 0.0048 | 0 | OPEN | ; |
| Pi53 | No58 | No54 | 217.42244 | 105.100006 | 0.0048 | 0 | OPEN | ; |
| Pi54 | No38 | No58 | 459.453888 | 105.100006 | 0.0048 | 0 | OPEN | ; |
| Pi55 | No38 | No59 | 111.13343 | 105.100006 | 0.0048 | 0 | OPEN | ; |
| Pi56 | No56 | No60 | 196.954025 | 105.100006 | 0.0048 | 0 | OPEN | ; |

| | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|
| Pi57 | No47 | | No61 | | 284.424988 | 105.100006 | 0.0048 | 0 | OPEN | ; | |
| Pi58 | No47 | | No62 | | 294.553741 | 105.100006 | 0.0048 | 0 | OPEN | ; | |
| Pi59 | No63 | | No44 | | 344.204834 | 105.100006 | 0.0048 | 0 | OPEN | ; | |
| Pi60 | No64 | | No63 | | 154.346283 | 105.100006 | 0.0048 | 0 | OPEN | ; | |
| Pi61 | No65 | | No96 | | 65.552147 | 105.100006 | 0.0048 | 0 | OPEN | ; | |
| Pi62 | No66 | | No67 | | 121.938622 | 105.100006 | 0.0048 | 0 | OPEN | ; | |
| Pi63 | No48 | | No56 | | 369.302551 | 105.100006 | 0.0048 | 0 | OPEN | ; | |
| Pi64 | No64 | | No66 | | 283.809998 | 105.100006 | 0.0048 | 0 | OPEN | ; | |
| Pi65 | No68 | | No69 | | 195.115891 | 105.100006 | 0.0048 | 0 | OPEN | ; | |
| Pi66 | No66 | | No68 | | 131.378998 | 105.100006 | 0.0048 | 0 | OPEN | ; | |
| Pi67 | No68 | | No70 | | 146.79921 | 105.100006 | 0.0048 | 0 | OPEN | ; | |
| Pi68 | No63 | | No48 | | 119.725281 | 105.100006 | 0.0048 | 0 | OPEN | ; | |
| Pi69 | No71 | | No52 | | 485.728027 | 277.600006 | 0.0048 | 0 | OPEN | ; | |
| Pi70 | No72 | | No73 | | 374.698456 | 105.100006 | 0.0048 | 0 | OPEN | ; | |
| Pi71 | No50 | | No74 | | 416.009796 | 105.100006 | 0.0048 | 0 | OPEN | ; | |
| Pi72 | No72 | | No50 | | 186.066086 | 105.100006 | 0.0048 | 0 | OPEN | ; | |
| Pi73 | No75 | | No71 | | 2694.240234 | 277.600006 | 0.0048 | 0 | OPEN | ; | |
| Pi74 | No73 | | No65 | | 445.636536 | 105.100006 | 0.0048 | 0 | OPEN | ; | |
| Pi75 | No52 | | No72 | | 11.700282 | 154.799988 | 0.0048 | 0 | OPEN | ; | |
| Pi76 | No76 | | No77 | | 199.460892 | 96.800003 | 0.0048 | 0 | OPEN | ; | |
| Pi77 | No78 | | No76 | | 325.652588 | 105.100006 | 0.0048 | 0 | OPEN | ; | |
| Pi79 | No76 | | No81 | | 151.457794 | 96.800003 | 0.0048 | 0 | OPEN | ; | |
| Pi80 | No81 | | No82 | | 199.530533 | 96.800003 | 0.0048 | 0 | OPEN | ; | |
| Pi81 | No57 | | No83 | | 187.411697 | 105.100006 | 0.0048 | 0 | OPEN | ; | |
| Pi82 | No57 | | No78 | | 95.460014 | 105.100006 | 0.0048 | 0 | OPEN | ; | |
| Pi83 | No78 | | No84 | | 235.25975 | 96.800003 | 0.0048 | 0 | OPEN | ; | |
| Pi84 | No85 | | No84 | | 170.297821 | 96.800003 | 0.0048 | 0 | OPEN | ; | |
| Pi85 | No84 | | No86 | | 314.612 | 105.100006 | 0.0048 | 0 | OPEN | ; | |
| Pi86 | No87 | | No88 | | 136.122223 | 105.100006 | 0.0048 | 0 | OPEN | ; | |
| Pi87 | No89 | | No90 | | 374.658905 | 105.100006 | 0.0048 | 0 | OPEN | ; | |
| Pi88 | No89 | | No91 | | 419.689819 | 105.100006 | 0.0048 | 0 | OPEN | ; | |
| Pi89 | No92 | | No91 | | 808.966858 | 105.100006 | 0.0048 | 0 | OPEN | ; | |
| Pi90 | No93 | | No85 | | 335.324066 | 105.100006 | 0.0048 | 0 | OPEN | ; | |
| Pi91 | No91 | | No93 | | 71.981003 | 105.100006 | 0.0048 | 0 | OPEN | ; | |
| Pi92 | No93 | | No94 | | 201.001251 | 105.100006 | 0.0048 | 0 | OPEN | ; | |
| Pi94 | No96 | | No64 | | 287.051239 | 105.100006 | 0.0048 | 0 | OPEN | ; | |
| Pi93 | No95 | | No96 | | 18.638063 | 105.100006 | 0.0048 | 0 | OPEN | ; | |
| Pi95 | | No65 | | No95 | | 81.524956 | 105.100006 | 0.0048 | 0 | OPEN | ; |
| Pi96 | | No95 | | No97 | | 17.811394 | 105.100006 | 0.0048 | 0 | OPEN | ; |
| Pi97 | | No97 | | No98 | | 186.218109 | 105.100006 | 0.0048 | 0 | OPEN | ; |
| Pi98 | | No97 | | No99 | | 130.52594 | 105.100006 | 0.0048 | 0 | OPEN | ; |
| Pi99 | | No99 | | No100 | | 229.470581 | 105.100006 | 0.0048 | 0 | OPEN | ; |

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Pi100 | No99 | No101 | 68.085518 | 105.100006 | 0.0048 | 0 | OPEN | ; |
| Pi101 | No101 | No102 | 158.445358 | 105.100006 | 0.0048 | 0 | OPEN | ; |
| Pi102 | No103 | No104 | 114.962227 | 105.100006 | 0.0048 | 0 | OPEN | ; |
| Pi103 | No101 | No103 | 157.930054 | 105.100006 | 0.0048 | 0 | OPEN | ; |
| Pi104 | No105 | No106 | 274.855164 | 105.100006 | 0.0048 | 0 | OPEN | ; |
| Pi105 | No103 | No105 | 415.990784 | 105.100006 | 0.0048 | 0 | OPEN | ; |
| Pi106 | No105 | No87 | 122.498268 | 105.100006 | 0.0048 | 0 | OPEN | ; |
| Pi107 | No87 | No89 | 11.165176 | 105.100006 | 0.0048 | 0 | OPEN | ; |
| Pi108 | No107 | No108 | 516.177307 | 96.800003 | 0.0048 | 0 | OPEN | ; |
| Pi109 | No108 | No109 | 51.968353 | 105.100006 | 0.0048 | 0 | OPEN | ; |
| Pi110 | No107 | No108 | 368.9935 | 96.800003 | 0.0048 | 0 | OPEN | ; |
| Pi111 | No110 | No113 | 512.338989 | 96.800003 | 0.0048 | 0 | OPEN | ; |
| Pi113 | No113 | No111 | 176.379959 | 96.800003 | 0.0048 | 0 | OPEN | ; |
| Pi112 | No112 | No113 | 112.336029 | 96.800003 | 0.0048 | 0 | OPEN | ; |
| Pi114 | No112 | No114 | 158.991959 | 96.800003 | 0.0048 | 0 | OPEN | ; |
| Pi115 | No115 | No112 | 232.818054 | 96.800003 | 0.0048 | 0 | OPEN | ; |
| Pi116 | No116 | No117 | 538.861572 | 96.800003 | 0.0048 | 0 | OPEN | ; |
| Pi117 | No118 | No119 | 66.286942 | 105.100006 | 0.0048 | 0 | OPEN | ; |
| Pi118 | No85 | No118 | 168.064911 | 96.800003 | 0.0048 | 0 | OPEN | ; |
| Pi119 | No118 | No110 | 625.08905 | 96.800003 | 0.0048 | 0 | OPEN | ; |
| Pi120 | No85 | No110 | 433.438721 | 96.800003 | 0.0048 | 0 | OPEN | ; |
| Pi121 | No119 | No120 | 529.440857 | 96.800003 | 0.0048 | 0 | OPEN | ; |
| Pi122 | No120 | No121 | 166.2939 | 96.800003 | 0.0048 | 0 | OPEN | ; |
| Pi123 | No120 | No116 | 240.100983 | 96.800003 | 0.0048 | 0 | OPEN | ; |
| Pi124 | No119 | No122 | 217.968887 | 105.100006 | 0.0048 | 0 | OPEN | ; |
| Pi125 | No115 | No123 | 15.282726 | 96.800003 | 0.0048 | 0 | OPEN | ; |
| Pi126 | No107 | No124 | 76.409081 | 96.800003 | 0.0048 | 0 | OPEN | ; |
| Pi127 | No123 | No107 | 235.143204 | 96.800003 | 0.0048 | 0 | OPEN | ; |
| Pi128 | No125 | No126 | 147.971756 | 96.800003 | 0.0048 | 0 | OPEN | ; |
| Pi129 | No127 | No128 | 140.333664 | 96.800003 | 0.0048 | 0 | OPEN | ; |
| Pi130 | No128 | No75 | 173.34552 | 96.800003 | 0.0048 | 0 | OPEN | ; |
| Pi131 | No129 | No115 | 188.259628 | 96.800003 | 0.0048 | 0 | OPEN | ; |
| Pi132 | No127 | No129 | 42.87582 | 96.800003 | 0.0048 | 0 | OPEN | ; |
| Pi133 | No129 | No130 | 106.470566 | 96.800003 | 0.0048 | 0 | OPEN | ; |
| Pi134 | No130 | No132 | 7.192253 | 96.800003 | 0.0048 | 0 | OPEN | ; |
| Pi135 | No123 | No131 | 102.007034 | 96.800003 | 0.0048 | 0 | OPEN | ; |
| Pi136 | No131 | No125 | 92.250565 | 96.800003 | 0.0048 | 0 | OPEN | ; |
| Pi138 | No132 | No131 | 207.018616 | 96.800003 | 0.0048 | 0 | OPEN | ; |
| Pi137 | No125 | No132 | 560.997009 | 96.800003 | 0.0048 | 0 | OPEN | ; |
| Pi139 | No133 | No128 | 198.71019 | 96.800003 | 0.0048 | 0 | OPEN | ; |
| Pi140 | No130 | No133 | 86.281502 | 96.800003 | 0.0048 | 0 | OPEN | ; |
| Pi141 | No133 | No134 | 305.724701 | 96.800003 | 0.0048 | 0 | OPEN | ; |

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Pi142 | No121 | No127 | 239.912292 | 96.800003 | 0.0048 | 0 | OPEN | ; |
| Pi143 | No75 | No135 | 295.45285 | 285 | 0.0048 | 0 | OPEN | ; |
| Pi144 | No136 | No137 | 111.897972 | 285 | 0.0048 | 0 | OPEN | ; |
| Pi145 | No135 | No136 | 66.795471 | 285 | 0.0048 | 0 | OPEN | ; |
| Pi146 | No136 | No138 | 114.081032 | 285 | 0.0048 | 0 | OPEN | ; |
| Pi147 | No121 | No116 | 59.45755 | 96.800003 | 0.0048 | 0 | OPEN | ; |
| Pi149 | No137 | No140 | 422.275299 | 285 | 0.0048 | 0 | OPEN | ; |
| Pi150 | No137 | No141 | 97.162575 | 285 | 0.0048 | 0 | OPEN | ; |
| Pi151 | No142 | No37 | 218.097672 | 105.100006 | 0.0048 | 0 | OPEN | ; |
| Pi180 | No140 | 2 | 1000 | 285 | 0.0084 | 0 | Open | ; |

[PUMPS]

;ID Node1 Node2 Parameters

[VALVES]

;ID Node1 Node2 Diameter Type Setting MinorLoss

[TAGS]

[DEMANDS]

;Junction Demand Pattern Category

[STATUS]

;ID Status/Setting

[PATTERNS]

;ID Multipliers

;

~FLAT 1

;

| | | | | | | |
|---|---|---|---|---|---|---|
| 1 | 0.72 | 0.71 | 0.7 | 0.7 | 0.7 | 0.72 |
| 1 | 1.05 | 1.5 | 1.3 | 1.2 | 1.2 | 1.2 |
| 1 | 1.15 | 1.15 | 1.1 | 1.1 | 1.1 | 1.1 |
| 1 | 1.2 | 1 | 1 | 0.8 | 0.8 | 0.8 |

;Flow pattern North-South

| | | | | | | |
|---|---|---|---|---|---|---|
| 4 | 0.68 | 0.69 | 0.66 | 0.64 | 0.69 | 0.72 |
| 4 | 0.63 | 1.21 | 1.32 | 1.44 | 1.32 | 0.87 |
| 4 | 0.83 | 1.02 | 0.97 | 0.97 | 1.13 | 1.1 |
| 4 | 1.19 | 1.16 | 1.15 | 1.26 | 1.22 | 1.13 |

;Pressure South Flow

| | | | | | | |
|---|---|---|---|---|---|---|
| 5 | 1.01 | 1.02 | 1.02 | 1.02 | 1.02 | 1.02 |
| 5 | 0.92 | 0.88 | 0.89 | 0.97 | 0.98 | 0.97 |

| | | | | | | |
|---|---|---|---|---|---|---|
| 5 | 0.91 | 1 | 1.02 | 1.01 | 1.01 | 1.02 |
| 5 | 1.01 | 1 | 1.02 | 1.09 | 1.1 | 1.11 |
| ;Flow Pattern North | | | | | | |
| 7 | 1.94 | 1.87 | 1.78 | 1.74 | 1.75 | 1.82 |
| 7 | 1.38 | 1.86 | 1.58 | 1 | 0.75 | 0.81 |
| 7 | 1.55 | 0.41 | 0.24 | 0.24 | 0.41 | 0.35 |
| 7 | 0.34 | 0.42 | 0.29 | 0.63 | 0.49 | 0.34 |
| ;Domestic+ApparentLoss | | | | | | |
| 2 | 0.27 | 0.31 | 0.21 | 0.16 | 0.28 | 0.34 |
| 2 | 0.34 | 1.67 | 1.89 | 2.01 | 1.73 | 0.79 |
| 2 | 0.8 | 1.05 | 0.91 | 0.94 | 1.26 | 1.18 |
| 2 | 1.39 | 1.35 | 1.33 | 1.4 | 1.3 | 1.08 |

[CURVES]

;ID X-Value Y-Value

[CONTROLS]

[RULES]

;WARNING:SynergidoesNOTexportanyrulestoEpanet

[ENERGY]

| | |
|---|---|
| Global Efficiency | 75 |
| Global Price | 0 |
| Demand Charge | 0 |

[EMITTERS]

| ;Junction | Coefficient |
|---|---|
| No1 | 0.008660075 |
| No2 | 0.00126885 |
| No3 | 0.002579131 |
| No4 | 0.010011711 |
| No5 | 0.003769531 |
| No6 | 0.005864617 |
| No7 | 0.002399357 |
| No8 | 0.001123425 |

| | |
|---|---|
| No9 | 0.007155057 |
| No10 | 0.006967293 |
| No11 | 0.004474497 |
| No12 | 0.001153897 |
| No13 | 0.001530736 |
| No14 | 0.006833604 |
| No15 | 0.000588679 |
| No16 | 0.007803522 |
| No17 | 0.002572555 |
| No18 | 0.002048254 |
| No19 | 0.001917446 |
| No20 | 0.004261478 |
| No21 | 0.003833663 |
| No22 | 0.003083962 |
| No23 | 0.000966726 |
| No24 | 0.001318567 |
| No25 | 0.003963675 |
| No26 | 0.001968221 |
| No27 | 0.004690523 |
| No28 | 0.001739393 |
| No29 | 0.00317838 |
| No30 | 0.000329089 |
| No31 | 0.001449366 |
| No32 | 0.002822528 |
| No33 | 0.001737158 |
| No34 | 0.000846503 |
| No36 | 0.002926353 |
| No37 | 0.003509927 |
| No38 | 0.008014482 |
| No39 | 0.005930845 |
| No40 | 0.002089988 |
| No41 | 0.002078668 |
| No42 | 0.000312368 |
| No43 | 0.001045091 |
| No44 | 0.008993678 |
| No45 | 0.002712057 |
| No46 | 0.003370665 |
| No47 | 0.004469741 |
| No48 | 0.004814314 |
| No49 | 0.004243679 |
| No50 | 0.003955719 |
| No51 | 0.000390993 |

| | |
|---|---|
| No52 | 0.01611121 |
| No54 | 0.003610866 |
| No55 | 0.001015202 |
| No56 | 0.013077116 |
| No57 | 0.011399272 |
| No58 | 0.005315965 |
| No59 | 0.000657991 |
| No60 | 0.001166111 |
| No61 | 0.001684002 |
| No62 | 0.001743972 |
| No63 | 0.003660645 |
| No64 | 0.004293755 |
| No65 | 0.003509295 |
| No66 | 0.003180185 |
| No67 | 0.000721965 |
| No68 | 0.002802244 |
| No69 | 0.001155228 |
| No70 | 0.000869158 |
| No71 | 0.018827719 |
| No72 | 0.003389407 |
| No73 | 0.004856978 |
| No74 | 0.00246308 |
| No75 | 0.018727484 |
| No76 | 0.004005793 |
| No77 | 0.001180953 |
| No78 | 0.003886201 |
| No81 | 0.002078106 |
| No82 | 0.001181366 |
| No83 | 0.001109613 |
| No84 | 0.004263926 |
| No85 | 0.006554986 |
| No86 | 0.001862731 |
| No87 | 0.001597327 |
| No88 | 0.000805942 |
| No89 | 0.004769227 |
| No90 | 0.002218253 |
| No91 | 0.007700719 |
| No92 | 0.004789671 |
| No93 | 0.003601615 |
| No94 | 0.001190073 |
| No96 | 0.002198019 |
| No95 | 0.000698494 |

| | |
|---|---|
| No97 | 0.001980811 |
| No98 | 0.001102546 |
| No99 | 0.002534556 |
| No100 | 0.001358632 |
| No101 | 0.002276288 |
| No102 | 0.000938111 |
| No103 | 0.004078688 |
| No104 | 0.00068066 |
| No105 | 0.004815588 |
| No106 | 0.001627342 |
| No107 | 0.007085469 |
| No108 | 0.005548544 |
| No109 | 0.00030769 |
| No110 | 0.009300671 |
| No111 | 0.001044297 |
| No113 | 0.004742827 |
| No112 | 0.00298491 |
| No114 | 0.000941348 |
| No115 | 0.00258357 |
| No116 | 0.004964055 |
| No117 | 0.003190451 |
| No118 | 0.005088514 |
| No119 | 0.004817675 |
| No120 | 0.005540827 |
| No121 | 0.002757067 |
| No122 | 0.001290534 |
| No123 | 0.002086659 |
| No124 | 0.000452397 |
| No125 | 0.0047438 |
| No126 | 0.0008761 |
| No127 | 0.002505188 |
| No128 | 0.003033717 |
| No129 | 0.001998873 |
| No130 | 0.001183815 |
| No131 | 0.002375846 |
| No132 | 0.004589793 |
| No133 | 0.00349747 |
| No134 | 0.001810112 |
| No135 | 0.002144773 |
| No136 | 0.001733437 |
| No137 | 0.003737966 |
| No138 | 0.000675442 |

| | |
|---|---|
| No140 | 0.008420902 |
| No141 | 0.000575273 |
| No142 | 0.004000537 |

[QUALITY]

| ;Node | InitQual |
|---|---|

[SOURCES]

| ;Node | Type | Quality | Pattern |
|---|---|---|---|

[REACTIONS]

| ;Type | Pipe/Tank | Coefficient |
|---|---|---|

[REACTIONS]

| | |
|---|---|
| Order Bulk | 1 |
| Order Tank | 1 |
| Order Wall | 1 |
| Global Bulk | 0 |
| Global Wall | 0 |
| Limiting Potential | 0 |
| Roughness Correlation | 0 |

[MIXING]

| ;Tank | Model |
|---|---|

[TIMES]

| | |
|---|---|
| Duration | 23:00 |
| Hydraulic Timestep | 01:00 |
| Quality Timestep | 00:03 |
| Pattern Timestep | 01:00 |
| Pattern Start | 00:00 |
| Report Timestep | 01:00 |
| Report Start | 00:00 |
| Start ClockTime | 00:00 |
| Statistic | NONE |

[REPORT]

| | |
|---|---|
| Status | Yes |
| Summary | No |
| Page | 0 |

[OPTIONS]

| | |
|---|---|
| Units | CMH |
| Headloss | D-W |
| Specific Gravity | 1 |
| Viscosity | 1.106364 |
| Trials | 100 |
| Accuracy | 0.001 |
| CHECKFREQ | 2 |
| MAXCHECK | 10 |
| DAMPLIMIT | 0.1 |
| Unbalanced | Continue 10 |
| Pattern | ~FLAT |
| Demand Multiplier | 1 |
| Emitter Exponent | 0.983005506 |
| Quality | Nonemg/L mg/L |
| Diffusivity | 1 |
| Tolerance | 0.01 |

[COORDINATES]

| ;Node | X-Coord | Y-Coord |
|---|---|---|
| No1 | 795920.78 | 628995.79 |
| No2 | 795708.95 | 629012.2 |
| No3 | 795911.39 | 628560.28 |
| No4 | 796412.05 | 629633.68 |
| No5 | 795960.33 | 630024.24 |
| No6 | 796301.48 | 630112.2 |
| No7 | 796438.48 | 630476.94 |
| No8 | 796213.55 | 630280.34 |
| No9 | 796967.24 | 630178.31 |
| No10 | 796566.93 | 629819.02 |
| No11 | 797318.71 | 629108.41 |
| No12 | 797315.77 | 628913.54 |
| No13 | 797577.14 | 629101 |
| No14 | 797307.04 | 629410.49 |
| No15 | 797208.04 | 629401.36 |
| No16 | 797288.52 | 630162.71 |
| No17 | 797296.33 | 630382.18 |
| No18 | 797625.04 | 630107.13 |
| No19 | 796948.82 | 629863.99 |
| No20 | 797258.4 | 630363.23 |
| No21 | 797450.8 | 630458.96 |
| No22 | 797432.68 | 630592.63 |

| | | |
|---|---|---|
| No23 | 797432.94 | 630755.91 |
| No24 | 797210.01 | 630596.58 |
| No25 | 797738.9 | 630633.85 |
| No26 | 796971.1 | 630508.9 |
| No27 | 797926.62 | 630451.08 |
| No28 | 798154.21 | 630265.31 |
| No29 | 797888.97 | 630264.66 |
| No30 | 797934.08 | 630232.19 |
| No31 | 797769.79 | 630051.68 |
| No32 | 798060.83 | 630634.76 |
| No33 | 798166.99 | 630621.17 |
| No34 | 798156.31 | 630741.18 |
| No36 | 798016.17 | 630988.06 |
| No37 | 797875.12 | 630804.9 |
| No38 | 798381.93 | 630186.8 |
| No39 | 797781.21 | 630686.26 |
| No40 | 798011.72 | 630981.69 |
| No41 | 797941.55 | 630925.63 |
| No42 | 797907.15 | 630885.63 |
| No43 | 797809.86 | 630923.76 |
| No44 | 798200.64 | 631617.37 |
| No45 | 797775.04 | 631448.01 |
| No46 | 798055.84 | 631044.82 |
| No47 | 798185.12 | 630925.47 |
| No48 | 798301.23 | 631256.22 |
| No49 | 797651.05 | 631868.57 |
| No50 | 798181.64 | 632169.58 |
| No51 | 798116.26 | 632178.94 |
| No52 | 798346.53 | 632252.41 |
| No54 | 798276.76 | 629770.72 |
| No55 | 798332.46 | 629608.55 |
| No56 | 798419.55 | 630994.32 |
| No57 | 799016.21 | 630035.17 |
| No58 | 798387.98 | 629899.04 |
| No59 | 798441.38 | 630280.69 |
| No60 | 798398.88 | 630841.95 |
| No61 | 798384.77 | 630727.62 |
| No62 | 798414.12 | 631047.81 |
| No63 | 798392.82 | 631333.33 |
| No64 | 798530.67 | 631401.01 |
| No65 | 798779.14 | 631543.45 |
| No66 | 798563.34 | 631119.09 |

| | | |
|---|---|---|
| No67 | 798576.82 | 630997.89 |
| No68 | 798694.26 | 631130.04 |
| No69 | 798775.07 | 630961.71 |
| No70 | 798650.22 | 631270.07 |
| No71 | 798659.01 | 631885 |
| No72 | 798352.74 | 632242.49 |
| No73 | 798567.2 | 631935.45 |
| No74 | 798459.25 | 631942.62 |
| No75 | 800395.93 | 629869.62 |
| No76 | 799214.26 | 629732.78 |
| No77 | 799411.43 | 629762.93 |
| No78 | 799111.07 | 630041.6 |
| No81 | 799272.49 | 629593.53 |
| No82 | 799470.41 | 629617.27 |
| No83 | 799070.89 | 629855.98 |
| No84 | 799290.84 | 630179.25 |
| No85 | 799441.3 | 630258.37 |
| No86 | 799055.75 | 630386.46 |
| No87 | 799267.18 | 630973.96 |
| No88 | 799187.2 | 630863.8 |
| No89 | 799275.87 | 630966.95 |
| No90 | 799563.23 | 630727.29 |
| No91 | 799200.97 | 630584.46 |
| No92 | 799263.25 | 631198.03 |
| No93 | 799234.84 | 630521.5 |
| No94 | 799371.96 | 630667.69 |
| No96 | 798805.62 | 631483.48 |
| No95 | 798823.9 | 631487.12 |
| No97 | 798834.61 | 631472.88 |
| No98 | 798910.1 | 631640.05 |
| No99 | 798942.52 | 631402.25 |
| No100 | 799096.47 | 631558.45 |
| No101 | 799005.26 | 631375.8 |
| No102 | 798885.58 | 631329.37 |
| No103 | 799151.25 | 631315.55 |
| No104 | 799171.27 | 631428.6 |
| No105 | 799355.35 | 631058.99 |
| No106 | 799539.34 | 630856.8 |
| No107 | 800066.23 | 629279.18 |
| No108 | 799929.44 | 628944.76 |
| No109 | 799912.93 | 628895.48 |
| No110 | 799634.24 | 629873.72 |

| | | | |
|---|---|---|---|
| No111 | | 799761.02 | 629225.98 |
| No113 | | 799711.82 | 629395.36 |
| No112 | | 799805.22 | 629457.78 |
| No114 | | 799728.49 | 629597.03 |
| No115 | | 800023.08 | 629475.22 |
| No116 | | 799983.86 | 629780.43 |
| No117 | | 799857.65 | 629630.51 |
| No118 | | 799583.18 | 630348.47 |
| No119 | | 799639.56 | 630383.32 |
| No120 | | 799935.78 | 629944.57 |
| No121 | | 800034.85 | 629811.01 |
| No122 | | 799821.36 | 630503.57 |
| No123 | | 800031.93 | 629462.76 |
| No124 | | 800139.82 | 629299.74 |
| No125 | | 800134.98 | 629298.23 |
| No126 | | 800179.37 | 629157.08 |
| No127 | | 800192.92 | 629630.54 |
| No128 | | 800282.07 | 629738.92 |
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[LABELS]

;X-Coord Y-Coord Label & Anchor Node

[BACKDROP]

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[END]

## Appendix J: Recommendations for Districts GWCL

*Recommendations for Amasaman District*

Water demand is growing rapidly in the Amasaman District. With 25,000 potential connections currently and over 1000 applications filed, there is a big need for water to be supplied into the district. In order to be able to provide continuous water supply, transport mains have to be constructed within the district. It is therefore recommended to create a backbone structure, which transports water into the district fetching from the 315mm on the north-east side. These transport lines need to be looped, so they can be fed from different points. From this backbone structure, offtakes can be made going into the different communities.

It is highly recommended to invest in creating sub-DMA's within the District by combining a group of connections (500-2000 ideally), isolating them hydraulically, and having one feeding point from the backbone transport pipeline. Planning of these DMA's should be done before waiting customers are connected to the network to optimize design and engineering. This should result in accurate analysis of NRW levels from which reduction strategies can be deduced.

Currently, offtakes from the 315mm line have a small diameter (100-150mm HDPE). Hydraulics should be run to determine optimal diameters. A hydraulic model enhances decision making with regards to optimal quantity, pressures and flows as well as energy requirements and economic diameters of pipelines. Furthermore, the network extension proposals that have been made by the district and regional office are hydraulically limited and require oversight and advice from the head office.

Finally, it is recommended to evaluate the rehabilitation of the old treatment works at Nsawam. The plan and proposals were there, funding was covered, however the government decided to not take on this project.

*Recommendations Santo District*

In order to improve future supply conditions for Santo district, the following solutions are proposed:

- Construct reservoir at boundary Adenta – Santo for future growth, fed from 1200mm from Dodowa which will supply the district by gravity.
- Create backbone structure to transport water throughout district
- Decide where water is going into Santo (Dodowa or/and Tema)
- Make a hydraulic analysis of pressures throughout the system for these options and make decision.

*Recommendations Adenta District*

To improve supply conditions towards the future, different solutions are proposed:

- All decisions for Adenta should be made with the supply conditions/effects for Adenta reservoir in mind.
- It is important to follow the design of the GAMA Masterplan 2016. It proposes a 2000 dedicated steel line to be constructed between Kpong phase 1 and Accra reservoir. The 1200mm line will then be used to only feed reservoirs within the districts and the reservoirs will feed the network.
- Since Adenta reaches over 10,000 connections, it is recommended to create another GWCL district within the original district. Plans are to make Agbogba (Adenta West) a district. It is important to note that this district can be hydraulically isolated relatively easy. It needs two pipes to be disconnected and two lines to be constructed (BOI offtake and Ecowash road west) and it only requires one meter to be installed.
- Construct and rehabilitate reservoirs throughout the district to increase supply conditions as well as regulating pressures throughout the network. An ideal location would be the border between Adenta and Santo, where the reservoir can be shared by both districts and be fed from the 1200mm line from Dodowa.
- A solution to isolate the Ecowash road East could be to change the southern boundary of Adenta. This way none of the take offs have to be cut, and no additional pipeline might be required. The southern boundary will then run alongside a river/stream within Madina and serving about 4000 connections. This will also enhance the capacity of the current Adenta office, since further growth is expected in the north-east part of Adenta.

## Appendix K: Conclusions and Recommendations Re IWS

Research on IWS is gaining momentum in the scientific community. It is complex in nature, which may be a reason the topic is still ill-defined. Although Galaitsi, et al. provided useful definitions and insights in the topic, some themes are not covered, like the different supply states, the positive and negative feedback loops for both causes and effects of IWS and a more detailed analysis of IWS by theme, like social-economic, politics & governance, management, technical, internal and external. The definition and understanding of IWS must be expanded and refined to develop better solutions. It is recommended to take a more integral approach. From there each element should be described in detail from which solutions can be developed. To enhance the current understanding of IWS, it is recommended to use and extend the overview developed in this study which describes the different water supply states based on supply frame and quantity, shown in figure 59.

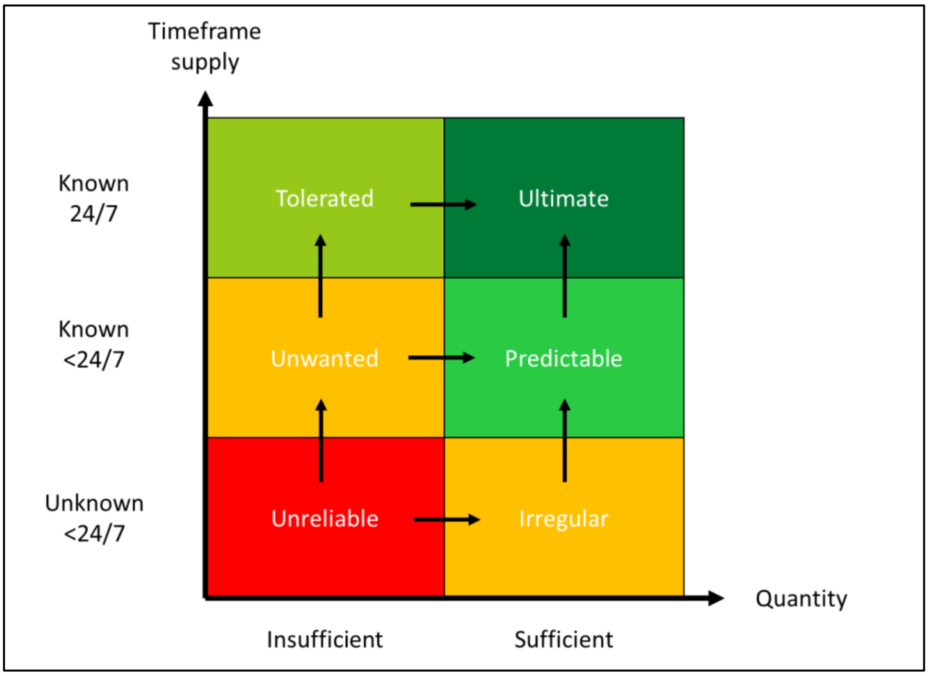



*Figure 53 Overview of different supply states. Red colour indicates a bad state, yellow is not preferred and dark green is preferred over light green and yellow. Arrows indicate how the transitions can be made to improve supply conditions.*

Graphs and figures should be developed to clarify the interdependent nature of IWS and each sub-theme. Figure 60 and 61 show, graphs and figures that were created in this research that could be further developed and deployed in ongoing research into the topic of IWS.

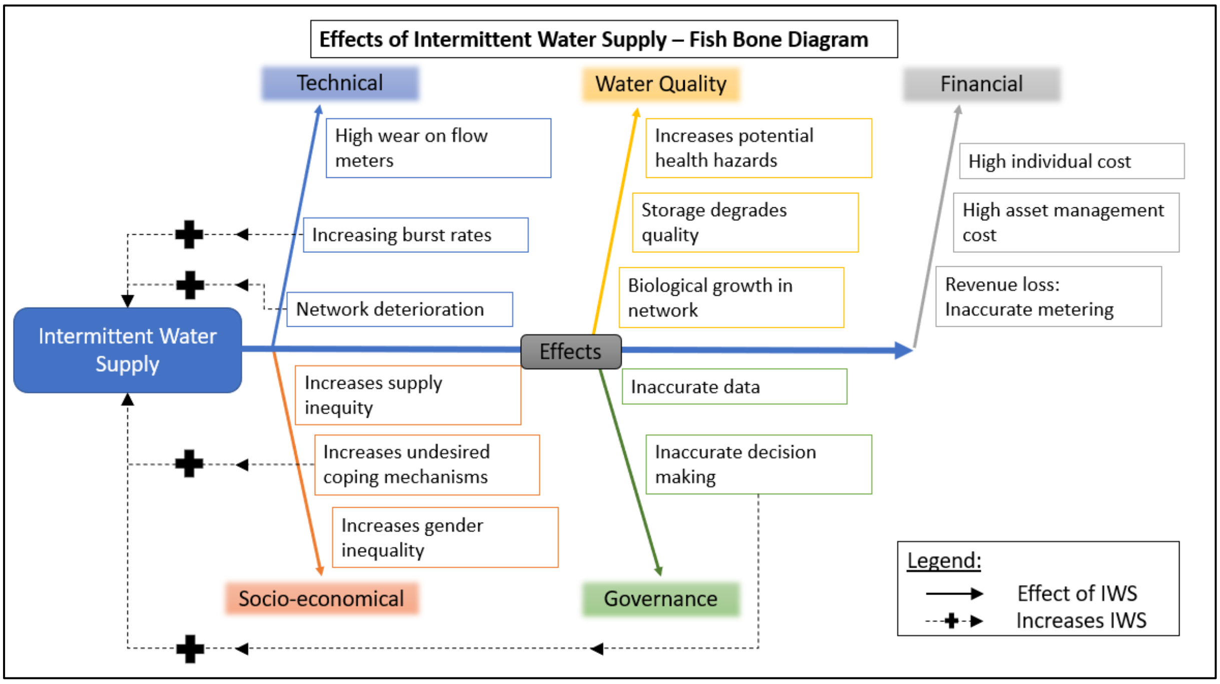



*Figure 54 Fishbone diagram - Effects of Intermittent Water Supply and positive feedback loops (Design by author, 2019)*

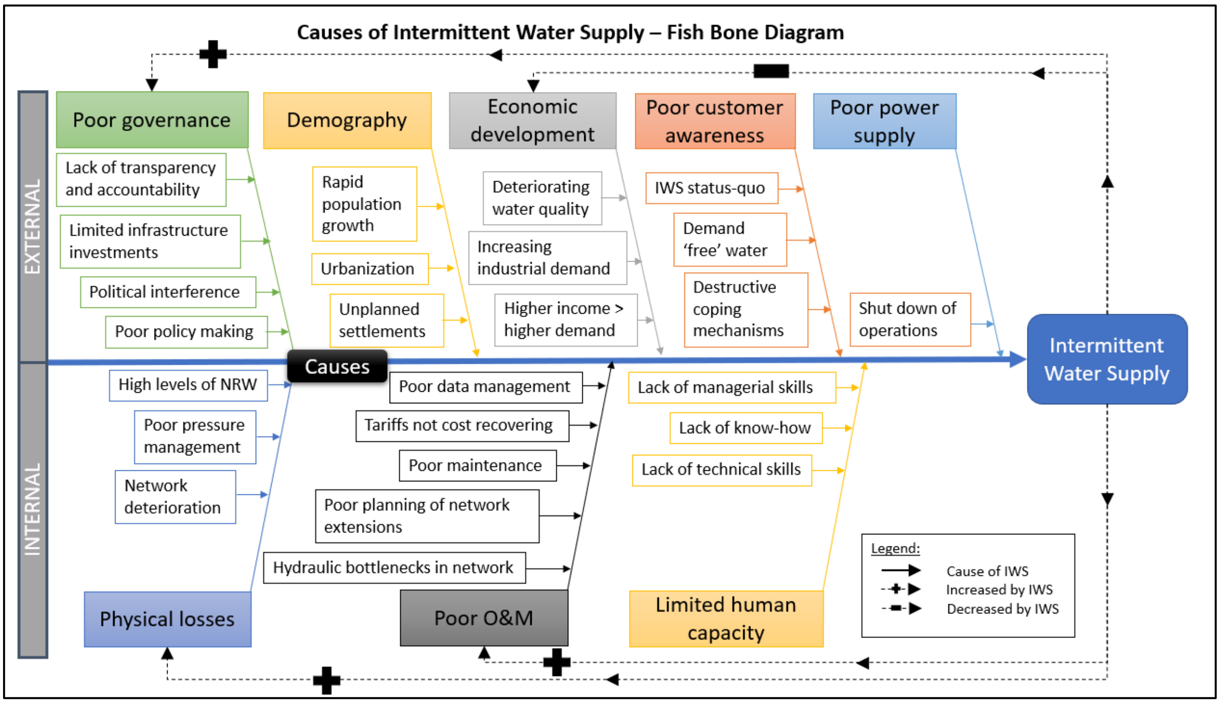



*Figure 55 Fishbone diagram- Causes of Intermittent Water Supply with positive and negative feedback loops (Design by author, 2019)*

There is a need for an integrated assessment on the causes of IWS in each country/ utility. Based on these assessments appropriate and more suitable interventions can be proposed and transition pathways developed towards CWS. Furthermore, it is recommended to develop interventions for elements within IWS, not only the technical factors. Customer awareness, staff awareness, government policies, management decision making are some important examples.

**Conclusions and Recommendations to the Scientific Community**

Develop opensource software that can model pressure dependent demands for both domestic demand and real losses, that is easily accessible and updated regularly with the state-of-the-art research. This is an opportunity because interest in IWS is gaining momentum in the scientific community only recently.

Develop a precise method to measure, calculate and verify the different parameters for burst and background leakage modelling with pressure dependent demand, this would enhance and increase the reliability of the outcomes and would better facilitate decision making towards CWS.

The Water Research Group developed a systematic tool to assess real loss components, however this tool is not available to us. It is recommended to make this software and research open source so everyone that is interested can use and apply this knowledge and tools and contribute to the continuous improvement of the matter.

The EasyCalc tool is not pressure dependent for apparent losses. One can see the impact of pressure changes in the 'What If' – tab however, this impact is only connected to real losses and not to the illegal connection's component of apparent losses. After conversing with the developer of the tool, Roland Liemberger[4], he acknowledged the error, but deemed it 'minor'. However, when the ratio of apparent losses over real losses is high, and the illegal connection component is large, its impact can be much higher than currently estimated and should not be reckoned as minor.

[4] Liemberger, Roland (Liemberger CC, World Bank and IWA) email correspondence with author, August 2019

# Appendix L: HtH Survey Amasaman District

## HtH survey for Amasaman District

### Start

**Select "Current Location"**

**Customer access**

◯ House open
◯ House closed / empty - Ask neighbours when they are most likely available.

*If Customer access is House open:*

### General data

**Name of interviewee**

**Number of people in the household (adults and children)**

☐ Don't Know

**Telephone number (of the one responsible to deal with water issues)**

☐ Not Applicable

**Home ownership**

◯ Owner of the house
◯ Tenant of the house

**With water connection?**

◯ Yes
◯ No

*If With water connection? is Yes:*

**Other households / families connected to the water connection?**

◯ Yes
◯ No

*If Other households / families connected to the water connection? is Yes:*

**Number of people in other household/families**

*If With water connection? is No:*

**Are you interested to get a water connection?**

**Home ownership**

◯ Owner of the house
◯ Tenant of the house

**With water connection?**

◯ Yes
◯ No

*If With water connection? is Yes:*
**Other households / families connected to the water connection?**

◯ Yes
◯ No

*If Other households / families connected to the water connection? is Yes:*
**Number of people in other household/families**

*If With water connection? is No:*
**Are you interested to get a water connection?**
*Hint: Refer to the house you are visiting now.*

◯ Yes
◯ No

☐ Don't Know

*If With water connection? is No:*
**Why don't you have a water connection?**

◯ There is no network of GWCL
◯ Not enough money for a contract
◯ Tenant of the house
◯ No response from GWCL on the request for connection

☐ Don't Know

**What is your main water source?**

*Hint: Or in case you have a water connection, what is your alternative in case of no water supply?*

◯ No need because 24/7 water
◯ Well
◯ Tap of the neighbour
◯ Water Kiosk
◯ Handpump / borehole
◯ Water truck
◯ Lake / River / dam
◯ GWCL water from another area
◯ Reservoir or tanque at house
◯ Other (please specify)

*If With water connection? is Yes:*

## Client data

**Client's status**

◯ Active
◯ Disconnected
◯ Turn-off
◯ Illegal connection - client not able to demonstrate invoice or contract, while water connection is present
◯ Not regularized - e.g. not in Customer Database yet

**Type of connection**

◯ Yard tap (domestic)
◯ In house (domestic)
◯ Commercial
◯ Industrial
◯ Bottling - e.g. Pepsi, CocaCola, breweries
◯ Institutional - e.g. hospital, church, municipial buildings, army, etc.
◯ Water kiosk
◯ Other (please specify)

*If Client's status isn't one of Illegal connection, Not regularized , Disconnected:*

**Name of client's account**

*Hint: Copy complete account name from invoice or contract*

**Street name**

*Hint: or use nearest street name*

☐ Don't Know

**Any other reference to localize the house**

*Hint: use an adequate reference which can be localized*

*If Client's status isn't one of Illegal connection, Not regularized , Disconnected:*

**Account number or control number**

*Hint: Preferably account number, otherwise Control number. In case of Control number, include the month.*

Comments...

**Take picture of invoice and (if possible) of the water meter**

**Is the client receiving invoices regularly?**

◯ Yes
◯ No
◯ Sometimes

☐ Not Applicable

Comments...

*If Client's status isn't one of Illegal connection, Disconnected, Turn-off:*

**Is the water supply reliable?**

*Hint: more than 20 hours per day with good pressure*

◯ Yes
◯ No

*If Is the water supply reliable? isn't Yes:*

**On AVERAGE, how many hours per week do you NOT have water?**

☐ Don't Know

*If Is the water supply reliable? is No:*

**For how long didn't you have water supply?**

◯ Less than a day
◯ Less than a week
◯ Less than a month
◯ 1-3 months
◯ 3-6 months
◯ more than 6 months
◯ no water supply at all

☐ Don't Know

**Do you have a storage tank to store water?**

◯ Yes, and in use
◯ Yes, but not in use
◯ No, would like one but cannot afford
◯ No, don not need one
◯ Other (please specify)

☐ Don't Know

*If Do you have a storage tank to store water? is Yes, and in use and Do you have a storage tank to store water? is Yes, but not in use:*

**What volume of water can be stored?**

*Hint: Volume in Liters (1m3 = 1000L)*

☐ Don't Know

Comments...

**Is there a control mechanism to stop water flow into tank?**

◯ Yes, a float valve.
◯ No, I close tap manually
◯ Other (please specify)

☐ Don't Know

**Status of the water meter system**

◯ Complete - All fittings and valve are present. If only water meter is missing, consider the system as complete. See next question.
◯ Incomplete - Part of fitting or valve is missing.
◯ Damaged
◯ Buried
◯ Taken out by client
◯ Without water meter system

☐ Not Applicable

Comments...

*If Status of the water meter system isn't one of Without water meter system, Not Applicable:*

**Leakage at water meter system**

☐ Without leakages
☐ Leakage before water meter - didn't passed the water meter
☐ Leakage after water meter - passed the water meter
☐ Leakage at the water meter

*If Status of the water meter system isn't one of Without water meter system, Not Applicable:*

**Status of the water meter**

◯ Good water meter with all seals - Check if the meter is running.
◯ Water meter without seal(s) - Check if the meter is running.
◯ No water meter
◯ Not readable / visible
◯ Broken water meter - = non-working water meter
◯ Water meter reversed - Water meter is installed in opposite direction (see arrow on meter)
◯ Water meter in a locked box - ...which can not be opened.
◯ Water meter taken out by client - Water meter still available at the house

☐ Not Applicable

Comments...

*If Status of the water meter isn't one of No water meter, Water meter taken out by client, Not Applicable:*

**Is the water meter installed horizontally?**

◯ Yes
◯ No

*If Status of the water meter isn't one of No water meter, Water meter taken out by client, Not Applicable:*

**Is the water meter installed with enough distance to the bends or valves?**

◯ Yes - at least 10 times the diameter before and 5 times after the meter
◯ No

*If Status of the water meter isn't one of No water meter, Water meter in a locked box, Not Applicable:*

**Water meter number**

*If Status of the water meter isn't one of No water meter, Not Applicable:*

**Diameter of the water meter**

◯ 15 mm (1/2")
◯ 20 mm (3/4")
◯ 25 mm (1")
◯ 32 mm (1,5")
◯ 50 mm (2")
◯ Other (please specify)

*If Status of the water meter isn't one of No water meter, Not readable / visible, Water meter in a locked box, Not Applicable:*

**Actual reading of the water meter**

**Is the client aware of the Call Centre telephone number to report leakages or complaints?**

◯ Yes
◯ No

☐ Not Applicable

**General satisfaction of the client**

*Hint: Try to avoid pleasing answers*

◯ Very good
◯ Good
◯ Medium
◯ Bad
◯ Very bad

☐ Not Applicable

**Connected to sewer or simplified sewer?**

◯ Yes
◯ No

☐ Don't Know

*If Connected to sewer or simplified sewer? is No:*

**Why not connected to sewer?**

◯ No sewer network
◯ Initial connection costs not affordable.
◯ Monthly fee not affordable.
◯ Client not aware - Client doesn't know the existance of sewer system
◯ Other (please specify)

## Comments

**Any other comments or observations?**

☐ Not Applicable

## Appendix M: Effects of IWS systems

| | Effects of IWS systems |
|---|---|
| **Technical** | • Degradation of network due to pressure surges. (Simukonda, Farmani, & Butler, 2018)<br>• Increasing burst rates (Klingel & Nestmann, 2014)<br>• More difficult and expensive to locate and repair leaks<br>• Higher wear on flow meters<br>• Inaccurate customer metering |
| **Water quality** | • Biological growth in the network (Kumpel & Nelson, 2016)<br>• Water should be assumed to be contaminated (WHO, 2005)<br>• Customer storage facilities deteriorate water quality further (Kumpel & Nelson, 2016)<br>• Increases potential health hazards |
| **Financial** | • Loss of revenues due to inaccurate metering<br>• High costs of physically lost water<br>• Lost revenues due to affluent customers transition to private borehole supply<br>• High asset management costs (Totsuka, Trifunović, & Vairavamoorthy, 2004)<br>• High individual coping costs (customer storage capacity and household water treatment) |
| **Socio-economical** | • Increases supply inequity (Klingel, 2012a)<br>• Affects the poor and disadvantaged (Totsuka et al., 2004)<br>• Affects women and girls in extreme poverty, since they are the ones fetching water in absence of local storage capacity (Blair, 2005; Klingel, 2012b)<br>• Increases undesired coping mechanisms like corruption, illegal connections and meter tampering.<br>• Reduces individual productivity and disrupts everyday life of families<br>• Wastage of 'old' water when fresh water is available for storage (A. O. Lambert, 2003) |
| **Management** | • Analysis is hard due to inaccurate data<br>• Uninformed & poor decision making<br>• Creates chaos and stress for management, due to all the different facets of IWS |

*Table 14 Overview of effects of Intermittent Water Supply*

## Appendix N: Causes of IWS

| | Causes of IWS |
|---|---|
| **Poor external governance** | • Lack of transparency, equity, responsive institutions, rule of law and accountability (Rogers & Hall, 2003)<br>• Limited investments in infrastructure development (Bruggen & Borghgraef, 2010)<br>• Affected due to political interference (Mckenzie & Ray, 2009) |
| **Demographics (increasing demand)** | • Rapid population growth<br>• Urbanization (Bruggen & Borghgraef, 2010)<br>• Unplanned settlements |
| **Economic development** | • Deteriorating water quality (mining, industrial effluents)<br>• Increasing industrial demand<br>• Water demand increases when consumers become more affluent (Totsuka et al., 2004) |
| **Climate change and land-use** | • Occurrence of extreme droughts and floods (Kala Vairavamoorthy, Gorantiwar, & Mohan, 2007)<br>• Depletion of natural water sources (Bruggen & Borghgraef, 2010)<br>• Deforestation influences hydrological conditions and influences usability of surface and ground waters negatively (Bruggen & Borghgraef, 2010) |
| **Poor Operations and Management** | • High levels of non-revenue water (NRW) (Charalambous et al., 2016)<br>• Poor data management<br>• Tariffs below cost-recovery level (van den Berg & Danilenko, 2017)<br>• Poor maintenance of infrastructure<br>• Poor planning of network extension (Mcintosh, 2003)<br>• Hydraulic bottlenecks within the network<br>• Lack of control over distribution network and lack of steering capacity (flows, pressures) |
| **Limited human capacity** | • Lack of know-how and importance of databases (Guth & Klingel, 2012)<br>• Lack of technical and managerial skills (Blair, 2005) |
| **Lack of customer awareness** | • IWS has become status-quo<br>• Demand for 'free-water' (Hunter, MacDonald, & Carter, 2010)<br>• Unwillingness to cooperate to CWS measures (Franceys & Jalakam, 2010) |
| **Poor power supply** | • Power outages resulting in shut down of operations |

*Table 15 Overview of causes of Intermittent Water Supply.*

**Primary Causes of IWS for Ghana Water Company Limited.**

According to Galaitsi, et al., most important causes of IWS are prioritization of network extension, effects of coping mechanisms and inadequate policies (Galaitsi et al., 2016). These causes are elaborated in detail below to build a perspective on the vicious consequences of each cause.

**Prioritization of network extension**

Oftentimes utilities attempt to connect as many residents as possible to a distribution network. This is true especially where 'pro-poor' initiatives and 'last-mile service' require it and the utility depends on funding from organizations demanding maximization of availability This creates issues with increasing block tariffs (IBT), where the poor get highly subsidized water and large consumers pay a high tariffs. (Baisa, Davis, Salant, & Wilcox, 2010; Kala Vairavamoorthy, Gorantiwar, & Mohan, 2007). Studies show that prioritization of network extension can cause water rationing (Stoler et al., 2012), reduced prices (Anjum Altaf, 1994), network stretch (Mcintosh, 2003) and, generally intermittent supply (Elala, Labhasetwar, & Tyrrel, 2011; Ingeduld, Pradhan, Svitak, & Terrai, 2008; Totsuka, Trifunović, & Vairavamoorthy, 2004). Prioritization compromises service to individual households and has corollary effects such as the necessity for household water storage and increasing entrenchment of the intermittency. Finally, as access to piped water is increases worldwide, intermittent supply is becoming the more prevalent mode of supply.

**Effects of coping mechanisms**

Citizens that do not regularly receive water tend to cope by making illicit connections to the distribution network, increasing non-revenue water (NRW) levels. These illegal connections extend the network further, reducing service levels for legitimate customers (Klingel, 2012a). Illicit connections contribute to apparent losses, poor data management of the utility and poor water quality (Elala et al., 2011; Lee & Schwab, 2005). Furthermore, customers cope with IWS individually by creating buffer capacity with storage tanks. This creates availability when the distribution network is not pressurized. But private storage has negative side effects. First, it is a costly private investment, unnecessary under CWS conditions (Choe, Varley, & Bijlani, 1996; Klingel, 2012b). Most importantly, it causes the water quality to deteriorate, fomenting serious health risks (Evison & Sunna, 2001; Tokajian & Hashwa, 2003). Finally it generates network pressure surges increasing network deterioration (Al-Ghamdi, 2011; Fontanazza, Freni, & La Loggia, 2007) and water wastage by consumer overdraw and tank spillage (Batish, 2003; Coelho, James, Sunna, Abu Jaish, & Chatiia, 2003; Kumar, 1997; Rabah & Jarada, 2012). These coping mechanisms stem from unreliable supply conditions caused, in part, by the imbalance between network expansion and network service.

**Inadequate policies**

Public policy and political prioritizations can reduce a network's ability to supply continuous water, entrench intermittent supply and increase its negative effects (Galaitsi et al., 2016). Causes include poor data management and government corruption (Klingel, 2012; Nganyanyuka, Martinez, Wesselink, Lungo, & Georgiadou, 2014). Policies include low prices for the poor by IBT, encouraging resource depletion caused by administrative decisions to over-pump aquifers (Choe et al., 1996; Zérah, 2000). Furthermore, bad policy undermines water utilities by denying sufficient funding to perform its public duties (Galaitsi et al., 2016). More fundamental are policies regarding the provision of broad water access for additional consumers. This can cause diminished supply service for existing customers, and inadequate service for new consumers who tend to appear at network edges.

For Sustainable Development Goal (SDG) 6, reliability of supply is not emphasized, whereas access to water is a crucial part of the goal (Renata, Ortigara, Kay, & Uhlenbrook, 2018; WHO, 2017). Policies that target improvement of supply conditions can include restructuring water tariffs, awareness programs, or a phase-out of agricultural subsidies (Klassert, Sigel, Gawel, & Klauer, 2015). Commercial and technical innovations can help extend coverage and institutional creativity needed for future growth (Criqui, 2015). Decentralized planning may reach more people and distribute power to lower levels for creative, innovative and responsive provision programs (Cherunya, Janezic, & Leuchner, 2015). On a larger scale, Vaidya examines governance and management of local storage to support community resilience (Vaidya, 2015). Understanding the pathways between conditions of water supply intermittency makes interdisciplinary analysis even more important, because what may appear to be engineering constraints may actually be governance or management constraints. The conditions need to be understood as a structure to characterize the whole system and demonstrate that decisions or actions in one area can affect options in another. Accounting for this structure can facilitate implementing water access improvements through multiple interventions.

## Appendix O: Case Studies

Different case studies have been conducted and described on transitioning to CWS, with each focusing on different causes. McIntosh (2003) focused on governance and tariffs, Klingel and Nestmann (2013) focused on database management and Dahasahasra (2018) focused on GIS mapping and hydraulic modelling. Other studies focus on policy improvement and water sector reform (D. Mckenzie & Ray, 2009).

In the case study of Karnatanka, India, Franceys, et al. found that it is achievable to transition to a CWS, while decreasing total water demand by 10 percent, revenue billing by five times and revenue collection by a factor of seven (Franceys & Jalakam, 2010). This was accomplished in India through a management contract with Veolia. Cronk, et al, found that water quality and year-round availability were more influential than management variables such as the availability of external technical support and funds to rehabilitate the system (Cronk & Bartram, 2018).

All studies recommended a phased transition. However, the selection of the different zones to be converted to CWS varied and the selection criteria were unclear in these studies (Simukonda et al., 2018). The differences within each of these studies showed a need for integral solutions for transitioning (Simukonda et al., 2018).

Reducing water pressures to below acceptable norms to maintain a 24/7 pressurized supply is preferable to introducing an IWS. Intermittent supply should be the last option (McKenzie, 2016). This was evident during the water crisis in Cape Town in 2017, where considerable effort was exerted to avoid IWS (Loubser, 2019).

## Appendix P: Non-Revenue Water

**Real Losses - $Q_{RL}$**

Water pressure affects real or physical losses in a network. At high pressures, pipes break more easily, and systems can lose large volumes from leaky components. Pressure surges, a water hammer for example, can seriously damage a distribution system. And soil conditions cause pipes to break due to settlements or sudden temperature changes. Another significant factor is the quality of a water system's pipes and joints, design, age, and maintenance history. Pipe materials also contribute to leakages. For example, asbestos cement (AC) pipes can leach internally and externally depending on the chemical stability and conditions of the (ground)water. In 2003, the IWA Water Loss Task Force defined the four principal intervention methods to combat real water losses as illustrated in figure 7 (Ziegler et al., 2011):

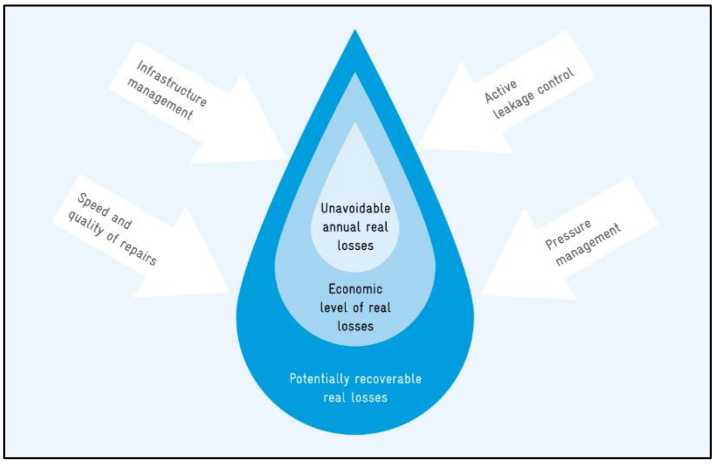


*Figure 56 Measures to reduce real losses. (Ziegler et al., 2011)*

- Pressure Management (PM)
  - Reduces the pressures in the network, causing water losses to be reduced as well as a decrease in pipe burst rates.
- Asset Management (AM)
  - Increases the lifetime of a system and causes water losses to be reduced and prevented in the future. Furthermore, spare parts are readily available to make repairs.
- Active Leakage Control (ALC)
  - Increases leakages detection and reduces real losses when leaks fixed.
- Speed and quality of repairs (SQR)
  - Reduces leakage volume from bursts and reduces future burst rates.

An IWA Task Force developed and tested the Infrastructure Leakage Index (ILI) as an accurate water loss performance indicator (A. O. Lambert et al., 1999). The ILI recognizes that real losses will always exist, even in best managed distribution systems. The international performance indicator (PI) can provide the best technical standards for determining water losses by utilities. Operators can use these standards to determine the effectiveness at their attempts to mitigate water loss. (Winarni, 2009).

ILI is the ratio of Current Annual Real Losses (CARL) to Unavoidable Annual Real Losses (UARL) (A. O. Lambert et al., 1999):

$$ILI = \frac{CARL}{UARL} \tag{1}$$

CARL equals the minimum night flow ($Q_{MNF}$) adjusted by a pressure factor, since pressures change due to the diurnal pattern of demand. $Q_{MNF}$ is determined by making district metered areas (DMA's), where the inflow and pressures are known, further explained in the next paragraph. CARL is the volume of water lost per connection per day (l/con/day).

UARL is the lowest technically workable volume of real losses at the current operating pressure, calculated using the following formula (A. O. Lambert et al., 1999):

$$UARL = 18 \cdot L_p + 0.8 \cdot N_{SC} + 25 \cdot L_{SC} \cdot P_{avg} \tag{2}$$

With:

UARL Unavoidable Annual Real Losses [l/day]

$L_p$ length of network (without service connection pipes) [km]

$N_{SC}$ number of service connections [-]

$L_{SC}$ length of property line till customer meter [km]

$P_{avg}$ average zonal pressure [mwc]

The World Bank provides performance ranges for different ILI bands. It distinguished between developed and developing countries, or high-income countries (HIC) and medium to low-income countries (MIC and LIC). Table 4 shows the performance range of different ILI bands.

| WBI band | ILI range | | Guideline description of real loss management performance categories |
|---|---|---|---|
| | Developed countries | Developing countries | |
| A | < 2.0 | < 4.0 | Further loss reduction may be uneconomic unless there are shortages; careful analysis needed to identify cost-effective leakage management |
| B | 2.0 to < 4.0 | 4.0 to < 8.0 | Possibilities for further improvement; consider pressure management, better active leakage control, better maintenance |
| C | 4.0 to < 8.0 | 8.0 to < 16.0 | Poor leakage management, tolerable only if plentiful cheap resources; even then, analyse level and nature of leakage, intensify reduction efforts |
| D | 8.0 or more | 16.0 or more | Very inefficient use of resources, indicative of poor maintenance and system condition in general, leakage reduction programs imperative and high priority |

*Table 16 ILI performance ranges and focal points (Liemberger, Brothers, & Lambert, 2007)*

Real losses must be valued at the total cost of water, including market price including production, transmission and distribution costs (Ziegler et al., 2011).

**Apparent Losses - $Q_{AL}$**

Apparent losses or commercial losses are caused by illegal connections, accounting errors, meter inaccuracy, measuring errors and corruption. Methods to reduce apparent losses include the reduction of (IWA, 2007):

- Data acquirement errors
- Water meter errors
- Unmetered consumption estimates
- Unauthorized consumption

Apparent loss is water successfully delivered to the customer, but not metered or accurately recorded, resulting customer consumption errors (Ziegler et al., 2011). These losses should be valued at retail price because it reaches the customer, but not paid for. Reducing apparent loss is often a good starting point to reduce NRW, because of relatively low cost of targeting apparent loss reductions.

*Wastage*

Wastage is the water that lost after the customer meter. Wastage is unaccounted for in the IWA model, but can represent substantial volumes of water that are spilled out of household storage tanks, leaking taps  and running toilets (Ziegler et al., 2011). Water supplies can be increased in regions with IWS and frequent network high pressure surges that cause leaking joints.  These system weaknesses are readily identified and remedied. Awareness campaigns for utility workers consumers can help decrease this wastage with no-to-low cost measures.

**NRW Assessment**

Two methods are used calculate water volumes losses: the top-down annual water balance and bottom-up real loss assessment. Both methodologies are described in the next chapter.

**Top-down Water Balance**

An annual water balance audit is performed to determine the loss per segment of the IWA water balance. The results of the water balance rely heavily on accurate measurements, careful estimates to achieve the difficult confidence limit of less than 15% of real losses. (A. O. Lambert, 2003). In order to develop an appropriate water loss reduction strategy with the top-down water balance audit, a bottom-up assessment is required (Charalambous & Hamilton, 2011).

**Bottom-up Assessment**

The bottom-up real loss assessment is used to cross check the real losses calculated in the water balance. This way, the estimated apparent losses can be adjusted by more accurate volumes. Real losses are often determined through a minimum night flow (MNF) analysis factoring diurnal system pressure variations. DMAs are often used to determine MNF. Furthermore, DMAs are employed for leakage reduction due to easier and faster location of leaks, and the creation of a permanent pressure control system which enabling low levels of leakage to be maintained (Ferrari et al., 2014).

**District Metered Areas**
Leakage monitoring requires the installation of flow and pressure meters at strategic locations in the distribution system. In order to increase accuracy and operational efficiency, the network is sectorized into districts. A DMA is defined as a discrete area of a distribution system. It is typically created by the closure of valves or complete disconnection of pipe works in which the quantities of water entering and leaving the area are metered (Morrison et al., 2007). The DMAs enable the presence of unreported bursts to be identified and leakage to be calculated with confidence.

A distinction needs to be made between a DMA and a pressure management area (PMA). A DMA is a discrete area where inflows and outflows are measured, but without active pressure management (Ziegler et al., 2011). A pressure management area (PMA) is also a discrete area with measured inflows and outflows but has active pressure management (Morrison et al., 2007). A DMA can be upgraded to a PMA by installing pressure reducing valves (PRVs) at the inlet points.

**DMA Requirements**
Depending on the characteristics of the network, a DMA is preferably (Morrison et al., 2007):

- Supplied via single main;
  - Resulting in increased measuring accuracy, since flow meters wear over time. Therefore, only one flow meter is preferred.
- A discrete area, hydraulically isolated (i.e. no flow into adjacent DMAs);
  - Whenever water flows outside of the DMA that is not metered, NRW levels will be inaccurate (too high).
- Geographically even
  - Minimal variation in ground elevation within the district, this will enhance optimal pressure management.

An effective permanent leakage control system within a DMA will:

- Maximize the accuracy of measurement of leakage within DMAs;
- Facilitate the location of the leaks;
- Limit or eliminate the number of closed valves;
- Minimize the changes to the hydraulic and qualitative operation of the existing network.

**Design Requirements**
Hydraulic, operational, economical and practical factors must be considered when designing DMAs. Small zones generally have higher installation and maintenance costs per connection. However, new leaks can be discovered earlier and easier, and it is possible to distinguish small leaks from customer night use and background leakage. Small DMAs can economically achieve a lower level of leakage than large DMAs. IWA strongly recommends that DMAs in urban areas should have 500 and 3,000 service connections (Morrison et al., 2007).

Instructive manuals are available to implement and operate DMAs, by Farley, the American Water Works Association (AWWA), the Asian Development Bank (ADB) German Development Cooperation (GDC) and IWA (American Water Works Association, 2003; Farley & Trow, 2003; Frauendorfer & Liemberger, 2010; A. O. Lambert, 2003; Ziegler et al., 2011) However, most of these manuals are not designed for IWS systems. Some mention IWS and recommend to adjust calculations for when the system is pressurized (Farley, 2001; Frauendorfer & Liemberger, 2010; Morrison et al., 2007).

**DMA Real Loss Assessment**

Real losses can be analyzed real time. Flow and pressure measurements can be transferred real time to the utility's control center (CC) by using a supervisory control and data acquisition (SCADA) system. However, many utilities in developing countries do not have these capabilities yet, or do not utilize them properly.

The most common way to assess real losses in a DMA is to perform night flow analysis. This is done by analyzing the period of minimum night flow (MNF), which usually occurs between 02:00 am and 04:00 am. It is the minimum of all recorded inflows and outflows. Customer consumption is at a minimum during this period and leakage thus represents the maximum of net inflow into the DMA.

**MNF assessment**

In order to perform an MNF analysis, the DMA inflows and outflows are measured throughout the night. Furthermore, the night consumption of bigger consumes (hospitals, industries operating 24/7 and hotels/resorts) must be accounted for, i.e. their night demand is assumed. Then the leakage flow can be calculated according to Eq(3).

$$Q_{NNF} = Q_{MNF} - Q_{CNF} \tag{3}$$

With:

$Q_{NNF}$ Net night flow (leakage) inside DMA [$m^3/h$]

$Q_{MNF}$ Minimum night flow inside DMA [$m^3/h$]

$Q_{LNC}$ Legitimate nighttime consumption inside DMA [$m^3/h$]

$Q_{LNC}$ should be estimated accurately case by case, but can be roughly based on the assumption that 6% of the population are active and water use for toilet flushing and other use is in the order of 10 L/hour (Stuart Hamilton & McKenzie, 2014). The real losses in the DMA are, however, not equal to $Q_{NNF}$, because pressures are not factorized.

**Effect of Pressure**

The pressure in a network varies relative to the flow. At peak demands, when flow is high, pressure decreases as shown in figure 8 (AL-Washali et al., 2018). Leakage rates are not constant throughout the day but increase with increasing pressures at night.

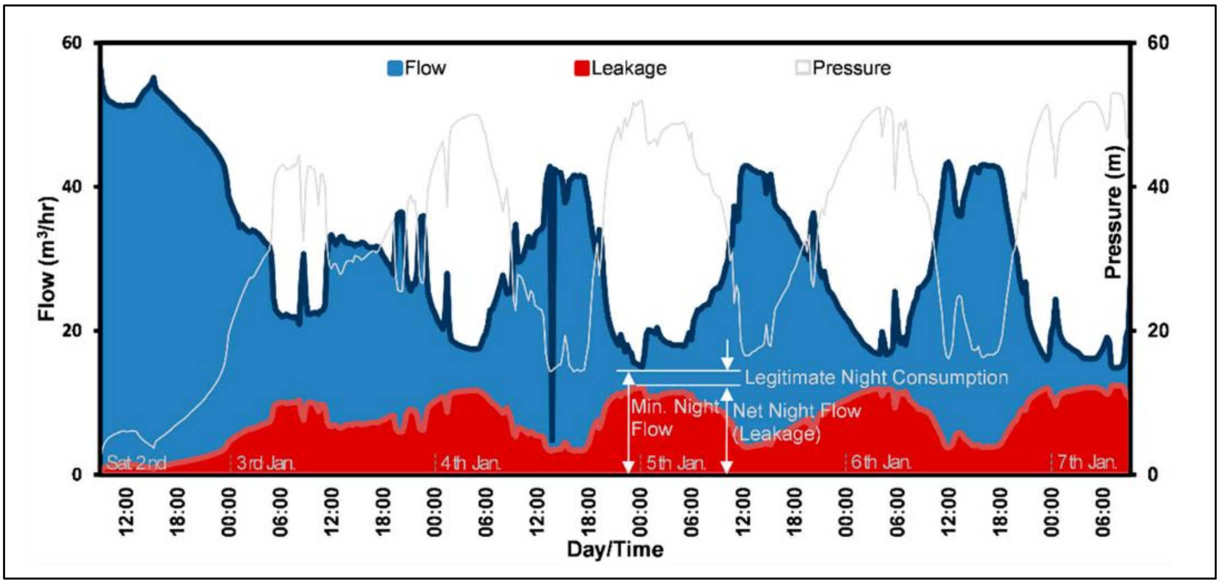



*Figure 57 Typical DMA flow-pressure profile (AL-Washali et al., 2018)*

$Q_{NNF}$ represents the real losses in the timeframe of minimum flow at a specific pressure and not for the entire day. In order to calculate daily real losses, pressure variation must be accounted for. Not until recently has the effect of pressure been appreciated in leakage management both in terms of reducing and maintaining a low level of leakage in a distribution network (Morrison et al., 2007). It is important to be aware of the pressure dependent relationships to leakage and demand.

**Average Zonal Pressure**

Pressures in a WDS are not distributed equally. Pressure variations occur due to elevation differences, demand variance within the district and pipe characteristics. Therefore, the concept of average zonal pressure (AZP) is introduced to account for the variation. AZP is an important parameter in the real loss assessment, MNF analysis, and crucial to assess impacts of different interventions. There is no standardized approach to determine the AZP, since every WDS is unique. However, the best practice is to measure pressures at different transport mains within the DMA in a variety of places. Recently, Halkijevic, et al., developed an approach to assess AZP (Halkijevic, Vouk, & Posavcic, 2018). LEAKSSuite developed a tool with limited availability to assess AZP.

## Appendix Q: Economic Leakage

As mentioned, leakage volumes can be physically reduced to the level of UARL, unavoidable real losses. However, reducing real losses to this point can be costly, when the cost of real loss reduction measures and maintenance of the real loss level is higher than the gains in terms of water volume and value. Therefore, the economic leakage level (ELL) was introduced by Farley and Trow (Farley & Trow, 2003). ELL can be split between short term and long term, based on the time required to plan and implement interventions and generate and monitor the results. Background leakage requires long term ELL since interventions are costly and require a long time to perform (Kanakoudis & Gonelas, 2016). The ELL of unreported breaks is calculated by finding the optimal duration of ACL. The ELL of reported bursts is calculated by finding the optimal speed or repairs for distribution pipes and transport mains. Both require a short term ELL (SRELL) (Kanakoudis & Gonelas, 2016).

**SRELL Methodologies**

Two methods can be used to determine ELL of unreported breaks. EIF-BABE calculates the economic frequency of intervention (EIF) with the breaks and background estimation (BABE) method. This method assumes a system is in 'steady state' condition, with no backlog of unreported breaks, except those that have occurred since the last survey (Fanner et al., 2007). Therefore, this method is not suitable for utilities that are starting with ALC, but it can be considered after ALC has been implemented.

Rate of rise (RR) method is another method, which can be used in non 'steady state' systems. It calculates the EIF at which the marginal cost of ALC equals the variable cost of water lost, according to Equation 10 (Fanner et al., 2007).

$$EIF = \sqrt{0.789 + \frac{CI}{CV * RR}} \tag{10}$$

With:

EIF Economic Intervention Frequency [months]

CI Cost of Intervention [€]

CV Variable cost of water [€/m$^3$]

RR Rate of Rise of unreported leakage flow [m$^3$/day/year]

The RR of an area or DMA can be calculated from active leak surveys done at several times throughout the years, or by (occasional) MNF measurements. It is important that the RR should be corrected for pressure at the time of measurement. Figure 14 shows the natural rate of rise for unreported leakage and figure 15 depicts the EIF for a range of RR and CI/CV values (A. Lambert & Lalonde, 2005).

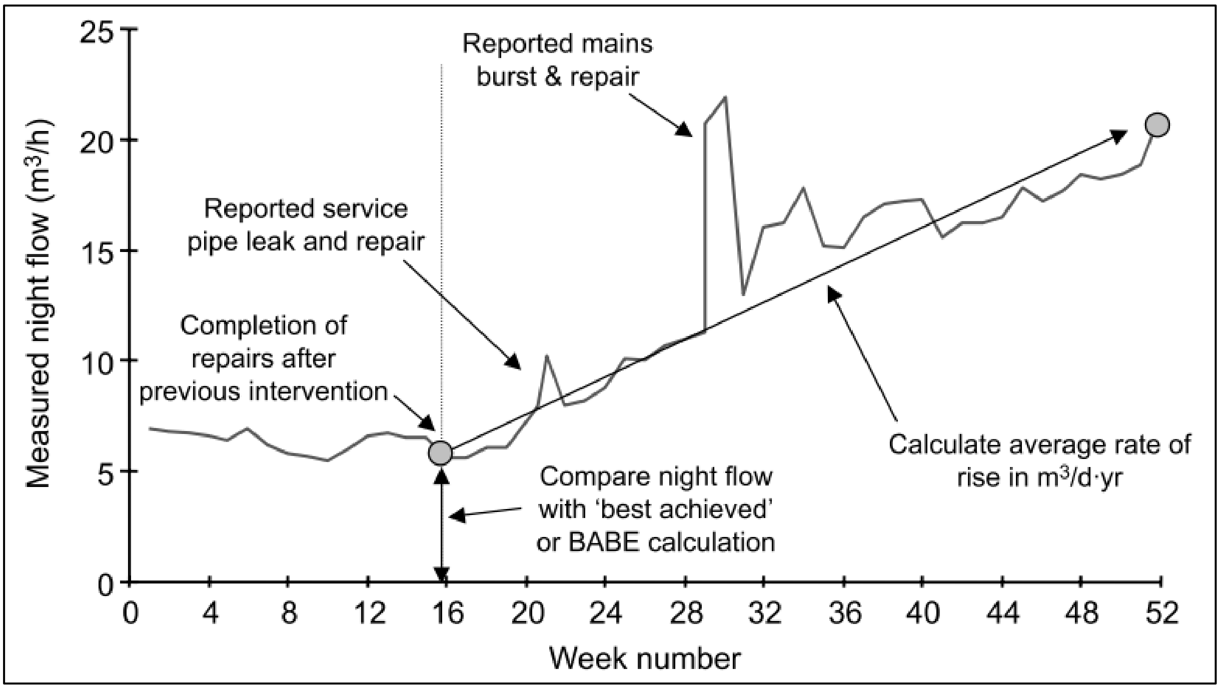



*Figure 58 Natural rate of rise of unreported leakage throughout the year* (A. Lambert & Lalonde, 2005)

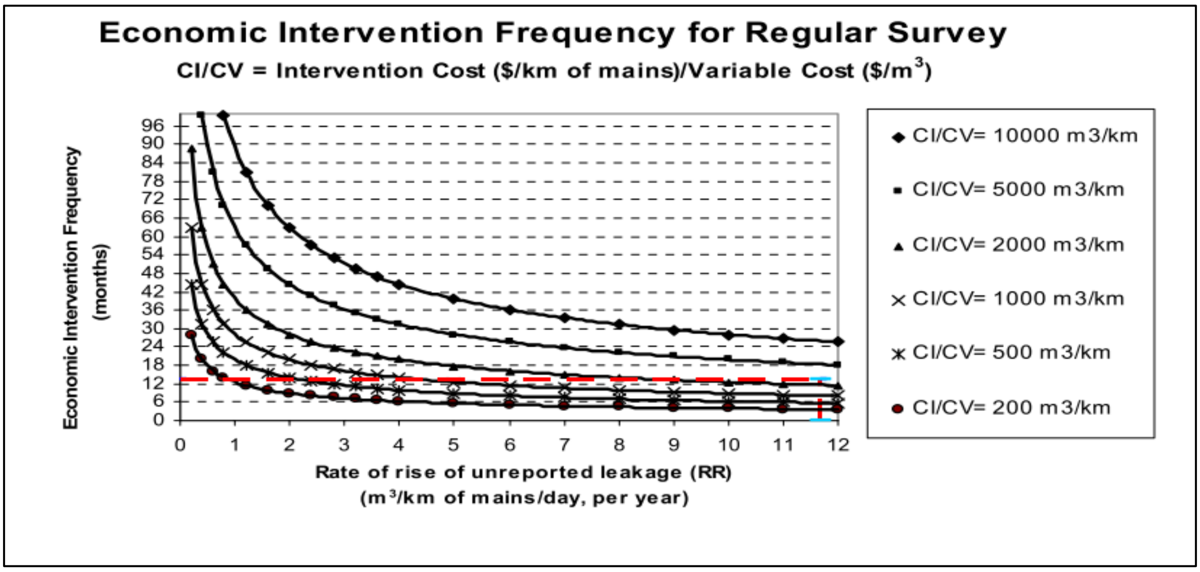



*Figure 59 Predicting Economic Intervention Frequency (EIF) for Regular Survey. (A. Lambert & Lalonde, 2005)*

This method does enable any size of system to obtain an initial assessment of the EIF for ALC and a budget in order to justify the first stages of a real loss reduction programme.

To determine the economic level of service for repairs a similar approach is used. If the repair time is reduced for leaks by improved work planning and prioritization, lower real loss levels will be achieved. However, reducing the repair time will become expensive when employees have to work overtime, additional staff need to be hired and equipment bought. For very short repair times, repair personnel cannot be efficiently utilized, due to the peaks and throughs in workload (Fanner et al., 2007). This results in a relationship between additional cost per repair to meet the shorter repair time and the actual repair time, shown in figure 16 (Pearson & Trow, 2005).

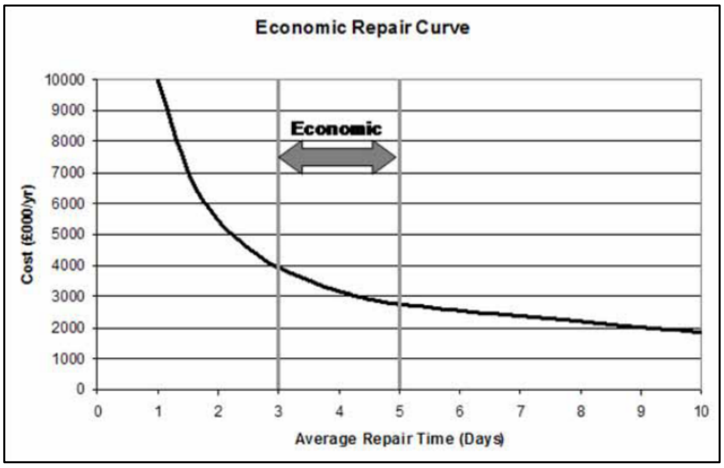



*Figure 60 Economic level of service repairs.*(Pearson & Trow, 2005)

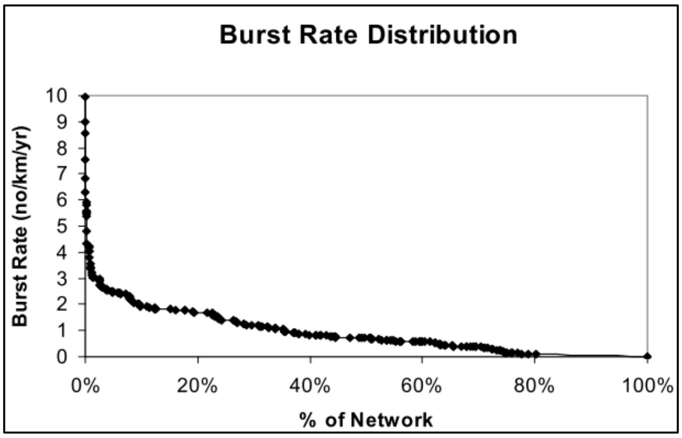
SRELL has a short planning horizon and are therefore assessed using a 5 to 10 year net present value (NPV) calculation (A. Lambert & Lalonde, 2005).

**LRELL Methodologies**

Long run economic leakage (LRELL) has a payback longer than the SRELL and consist of interventions like pressure management, sectorization and infrastructure renewal. After implementation of these interventions, SRELL will also be reduced.

Pressure management will reduce real losses by reducing both background -, unreported- and reported leakage. However, PM also reduces break frequencies, since it reduces stress on the network. This reduces repair costs, customer service costs, inspection costs for reported breaks, interruptions in supply and compensation payments (Fanner et al., 2007).

Sectorization has the benefit of having different rates of rise in different sectors or DMAs. The calculations establish an economic breakpoint that would give the economic level of sectorization and the optimum size of the sectors.




Infrastructure renewal reduces background losses as well as the burst frequency. It is a suitable intervention for distribution or transport sections with high levels of burst frequency and/or background leakage. In most cases only a small number of mains suffer from high break frequencies, as shown in figure 17 (Fanner et al., 2007; Pearson & Trow, 2005).

*Figure 61 Typical distribution of break rates across network infrastructure* (Pearson & Trow, 2005)

Long run interventions have a long planning horizon and should therefore, be assessed with a 25 to 30 year net present value (NPV) calculation (A. Lambert & Lalonde, 2005).

## Appendix R: Multi Criteria Analysis Interventions

To determine the most optimal intervention per district for the whole of GAMA, a multi criteria analysis (MCA) is deployed. MCA is a widely used method to assess the value of different options more objectively. A method is posed, that can be used by practitioners and utilities to determine suitable interventions per district. The main criterion is the increase in supply time per district [hours/week] that each intervention promotes. Furthermore, the economic cost-benefit criterion represented in the net present value (NPV) is used as well as the flow of water saved [$m^3$/month]. These criteria are important since the first goal is to increase service levels for the whole of GAMA which needs to be done in a cost effective and efficient way, represented by the NPV. Furthermore, the flow of water saved can be further used to provide districts where SIV is too low, to serve customers that currently experience low service levels (intermittent supply). The value of each intervention for each criterion is scaled from zero to one by using a minimum and maximum value. An overview of this process is shown in figure 21. Weights could then be added for each criterion, to indicate the relative preference of one over the other, with the total weight adding up to one. In that case the grading does not necessarily need to be linear but could be exponential depending on the preference.

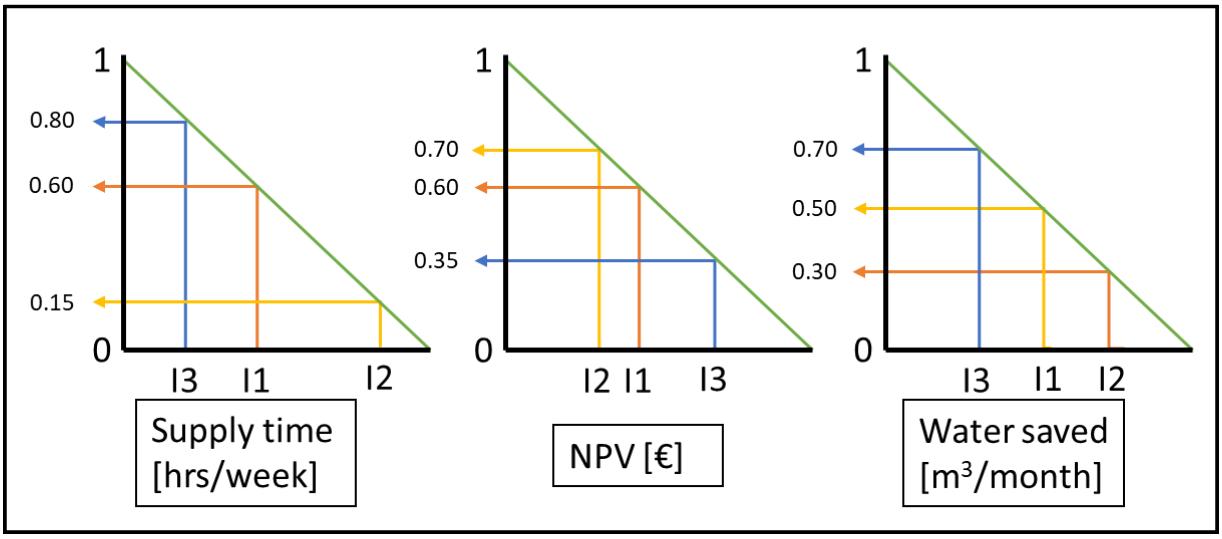



*Figure 62 Example of grading each intervention (I1, I2, I3) for each criterion (Supply time, NPV and water saved).*

## Appendix S: Questionnaire Applicability DMA approach to transition to CWS

1/24/2020 Applicability DMA approach to transition to Continuous Water Supply

# Applicability DMA approach to transition to Continuous Water Supply

This survey is conducted to gain insight in the applicability of the DMA approach that enables the transition to continuous water supply.

The DMA approach consists of the following:

Requirements:
1) Divide region or area into smaller districts. (max. 3000 connections, level elevation, clear boundaries, (preferably) one inlet.)

2) Insight and availability of network data in GIS, customer billing, System Input Volume and Pressures, Minimum Night Flow measurements.

Approach:
- Assess the current state of districts based on this data in terms of NRW, real losses, apparent losses, domestic demand, pressure and intermittency levels.

- Develop a strategy for which districts to focus on. (High pressures and high supply time first, then high pressures and lower supply times, then high real losses and high supply time and then the others)

- Based on the current state of the district, propose interventions like Pressure Management (PRV and VSP), Active Leakage Control, Speed and Quality of Repairs and creating storage within the district (when fed from a pumped transport main).

- Model impact of intervention with (hydraulic) software

- Implement and maintain intervention

.

What utility and country do you represent? *

Jouw antwoord

Does your utility experience Intermittent Water Supply in (some of) their regions?

- Yes
- No

Does your utility meet the requirements, stated above? *

- Yes
- No

If no: What is needed to meet requirements?

Jouw antwoord

If no, build they be willing to meet the requirements?

- Yes
- No
- Maybe

Based on your experience and expertise, do you think the DMA approach can be applied to transition to Continuous Water Supply for your utility? *

- Yes
- No

Why do you think so? *

Jouw antwoord

What boundary conditions need to be considered when implementing this approach?

Jouw antwoord

Do you have any critique, feedback or recommendations on the DMA approach to transition to CWS? *

Jouw antwoord