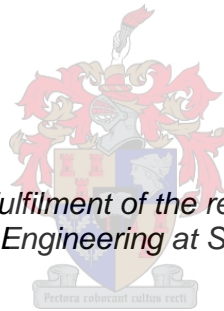


Predicting Water Quality in Bulk Distribution Systems

by
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*Thesis presented in partial fulfilment of the requirements for the degree of
Master of Science in Engineering at Stellenbosch University*



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Abstract

The increased water demand to be supplied by municipal water distribution systems, and subsequent increased storage period of reserve water, may have implications with regards to water ageing and subsequently may have an impact on health and safety.

Current master planning design standards could have a negative effect on water residence time. The decay of the disinfectant potential is a function of the residence time in the distribution system. The objective of this study is to identify and measure existing systems to optimally increase water quality in a distribution system while supplying an increase in demand, dealing with the deterioration of pipe infrastructure and the introduction of alternative water sources.

To do this, one must understand the dynamics of water networks and the parameters that affect water quality. The foundation of a water quality model is based on the construction of an accurate hydraulic model. To identify and measure these systems, one must understand the aspects of water purification and the techniques used to achieve water standards in a distribution system. These techniques and standards play a huge role in the prediction of water quality. In this paper the fundamentals and techniques used to determine and measure such a model are discussed.

Consequently, additional design parameters to assess water quality must be incorporated into current master planning practice to optimally design water networks. These models are used to determine the appropriate levels of disinfectant at strategic locations in a system.

To illustrate these design parameters and systems currently used in practice, a case study involving Umgeni Water (UW) and EThekweni municipality (EWS) was used to determine the most suitable disinfectant strategy for a municipality's distribution system. Future scenarios and the impact of disinfectant mixing and increased residence time of the water in the system were also determined. The use of this water quality model in a distribution system will ultimately provide a sustainable platform for a risk monitoring procedure.

Opsomming

Die verhoogde aanvraag na water in munisipale voorsieningstelsels, en die daaropvolgende verhoogde stoortydperk van reserwe water, kan implikasies inhou met betrekking tot water veroudering waarna dit 'n impak op gesondheid en veiligheid kan hê.

Huidige meesterbeplanning ontwerpstandaarde kan 'n noemenswaardige uitwerking op water retensietyd hê, veral omdat chloor se vervaltyd op sy beurt 'n funksie van water retensietyd is. Die doel van hierdie studie is om 'n prosedure te identifiseer om watergehalte optimaal te verhoog in 'n waterverspreidingstelsel, terwyl die toename in water aanvraag voortduur.

Om dit te kan doen moet die dinamika van water netwerke en die parameters wat die gehalte van water beïnvloed, bestudeer word. Die opstel van 'n waterkwaliteit model is gebaseer op die bou van 'n akkurate hidrouliese model. Om uiteindelik die ontwerp van 'n waterkwaliteit oplossing suksesvol uit te voer, moet 'n mens al die aspekte van watersuiwering en die tegnieke wat gebruik word om waterstandaarde te handhaaf in 'n verspreidingstelsel verstaan. In hierdie verslag word die beginsels en tegnieke wat gebruik word om so 'n model op te stel, bespreek.

Bykomende waterkwaliteit ontwerpparameters moet by huidige meesterbeplanning gevoeg word om waternetwerke optimaal te ontwerp. Hierdie modelle word gebruik om die geskikte vlakke van ontsmettingsmiddel op strategiese plekke in 'n stelsel te bepaal.

'n Gevallestudie van Umgeni Water (UW) en eThekweni-munisipaliteit (EWS) is gebruik om die mees geskikte ontsmettingsmiddel strategie vir 'n munisipaliteit se verspreiding te illustreer. Toekomstige scenario's en die impak van ontsmettingsmiddelvermenging en verhoogde retensietyd van die water in die stelsel sal ook bepaal kan word. Die gebruik van hierdie gehalte-watermodel in 'n verspreidingstelsel sal uiteindelik 'n volhoubare platform vir 'n risiko moniteringstelsel inhou.

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List of Abbreviations

Asbestos Cement (AC)
Average Annual Daily Demand (AADD)
Bottom Water Level (BWL)
Business/Commercial/Industrial (BCI)
Computational Fluid Dynamics (CFD)
Computer Aided Design (CAD)
eThekweni Municipality (EWS)
Digital Terrain Model (DTM)
Disinfectant by products (DBP)
Durban Heights (DBH)
Geographic Information System (GIS)
Halocacetic Acids (HAA)
Inner City (CBD)
Large Single Consumer (LRG)
Low Cost Housing (LCH)
Peak Daily Factor (PDF)
Peak Hour Factor (PHF)
Peak Week Factor (PWF)
Pilot Distribution System (PDS)
Polyvinyl Chloride Pipe (PVC)
Residential (RES)
Shuttle Radar Topography Mission (SRTM)
South Coast Aqueduct (SCA)
Steady State (SS)
Top Water Level (TWL)
Trihalomethane (THM)
Ultraviolet radiation (UV)
Umgeni Water Utility (UW)
Unaccounted Water (UAW)
Water Safety Plan (WSP)
Water Treatment Works (WTW)

1. Introduction

1.1 Background

It is estimated that there will be water scarcity problems in 118 countries over the period of 1990-2025. Approximately 1 billion people will face absolute water scarcity because they lack sufficient water sources. More than 348 million people face severe water scarcity because they live in regions where existing water resources require high-cost to water acquisition (Bruins, 2000). The global water demand has more than tripled since 1950 mainly based on the availability of surface water resources such as rivers, lakes and ground water. The need for domestic, agricultural and industrial water has increased enormously (Clark, 2011).

The amount of resources invested into conservation of water and to have water available for various uses can have a huge effect on an economy and ultimately dire consequences. These investments can be vulnerable to droughts, earthquakes and terrorism. Drought affects more people than any other natural hazard (Bruins, 2000).

In the early part of the 1930s, water distribution systems were expanded with huge investments into pipe infrastructure. This huge investment period brought water distribution systems to where it is today. The techniques used for reticulation relied on asbestos cement (AC) pipes that could potentially last 50 years, but later the introduction of PVC pipes resulted in the replacement of old pipe infrastructure. The consequence will have huge capital investment requirements on municipalities based on the fact that the majority of these pipes will require replacement in the next couple of years. Pipe deterioration will result in water quality reduction based on the increase in the risk of contamination through pipe replacement.

In 1974 the Safe Drinking Water Act was established to determine the maximum levels for each contamination and the influence it has on human health. Adequate safety margins were implemented for which no known or anticipated adverse effects on human health would occur. These water acts can impose huge challenges on municipalities due to the lack of funds to sample and ensure healthy water at each household. If a sustained water quality approach were imposed, adequate planning and prevention can be implemented and consequently reduce the risk of water quality problems.

In the 1996 Safe Drinking Water Act and its amendments it indicated that not only is it necessary to measure the maximum contamination levels at the Water Treatment Works (WTW), but also at the consumer's tap. Factors that can influence the quality of water in a distribution system are as follows:

- Chemical and biological quality of source water;
- Effectiveness and efficiency of the treatment processes, adequacy of treatment facilities, storage facilities, distribution systems;
- Age, type, design and maintenance of the distribution network.

1.2 Focus of Study

Distribution systems are extremely complex and difficult to study in the field. To fully understand the complexity of predicting water quality, we must first understand the effect that the distribution system can have on water quality. The increase in the stringency of rules and regulations indicates the growing awareness of the possibility that the quality of drinking water can deteriorate between the treatment works and the consumer. Chlorine or other disinfectants interact with natural organic matter and form disinfection by-products. Water Safety Plans (WSP) are currently implemented to reduce the risk of contamination of drinking water and water quality modelling could form part of such a risk analyses system. It is a popular assumption that chlorine levels in drinking water are good indicators of water quality. The purpose of this study is to predict chlorine levels in existing water systems and the deterioration of these levels based on infrastructure upgrades as the demand increases in the future.

1.3 Objectives of Study

As a result of the aforementioned, interest has been growing in the use of hydraulic and water-quality models as a tool for evaluating the various factors that influence the deterioration of water quality in drinking-water distribution systems. Research into disinfectant methods, and the impact that these methods have on water quality, is being done at an increasing rate. The objective of this report is to obtain knowledge in the evaluation of water quality and the methods that are used to predict it.

This thesis involves a case study regarding water quality modelling for **Umgeni Water (UW) and EThekweni Municipality (EWS)** using Wadiso 5 as modelling algorithm to illustrate the methods used to determine water quality variations in water networks.

2. Literature Review:

The following literature review involves examining the corpus of water quality modelling.

The purpose of this review is to indicate the theories, concepts and issues that already exist in water quality modelling. These relationships were then used to establish a methodology to implement a procedure on predicting water disinfectant potential in a large water utility. These issues were divided into sections based on a particular part of water quality modelling. Indicated in **Figure 2-1** is a small illustration of a typical water utility flow diagram from the source to the tap.

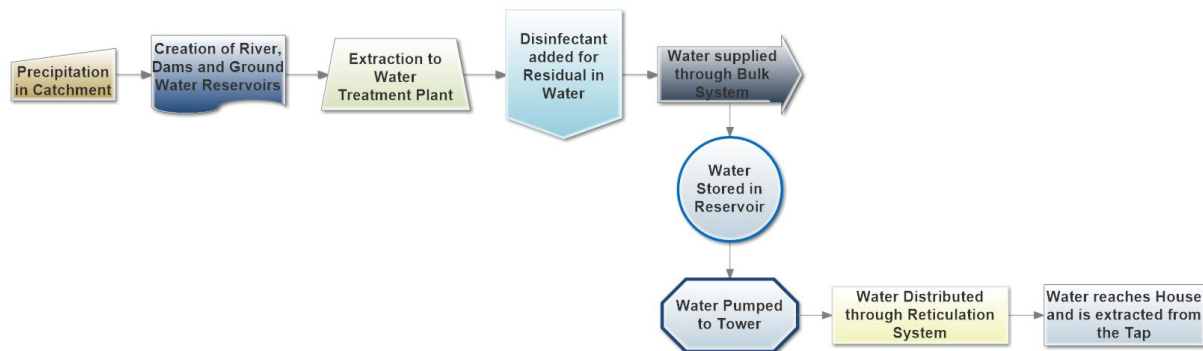


Figure 2-1 Water Utility Flow Diagram

2.1 Disinfectant Methods

Preserving water quality throughout a distribution system is one of the most challenging technological issues for suppliers. This is done by choosing the disinfectant method in operating the water distribution system and then predicting the consumer water usage to minimize water retention times.

There are several disinfectant methods which can be used to disinfect high water flows. They can be divided into three main divisions (Castro & Mario, 2003):

- UV Radiation;
- Ozone;
- Chlorine.

It is important to evaluate each disinfectant method to understand their effects on potential disinfectant reduction throughout the distribution system. UV and Ozone are usually only used in the primary or initial disinfectant process; however, in areas where water contamination is expected along the distribution network, additional disinfectant would be

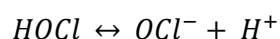
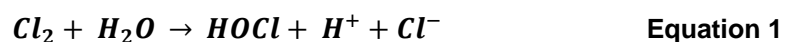
required in the distribution system where contamination is expected. For example the Netherlands do not use chlorine as a disinfectant, neither for the primary disinfection nor to maintain a residual disinfectant in the distribution network. The Dutch approach that allows production and distribution of drinking water without the use of chlorine, while not compromising microbial safety at the tap, is mainly due to their ability to use good sources, use preferred physical treatment, prevent microbial growth in the distribution system and monitor for detection of any failures in the system (Kinsbergen, 2014).

Using laboratory tests to show the effect on water quality is not sufficient and thus understanding the results based on additional literature will help with predicting possible solutions to water quality problems.

2.1.1 Chloramination/Chlorination

When chlorine gas dissolves in water, it forms part of a hypochlorous acid (HOCl) which in an alkaline solution turns into hypochlorite (OCl⁻). Hypochlorite is the active form of chlorine and exists as a free ion in solution. As indicated in the equation below the reference typically used for water quality concentration is free chlorine. Common bleach contains sodium hypochlorite, the same active ingredient as the free chlorine from chlorine gas (IBT, 2003).

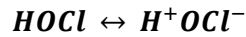
Chlorine is added to water in a gaseous form or as sodium or calcium hypochlorite. The chlorine gas rapidly hydrolyses to hypochlorous acid as indicated in equation 1 (Harp, 2002).



Similarly to Equation 1, the solution of sodium or calcium hyperchlorite would hydrolyse according to equation 2:



The two chemical species formed by adding chlorine to water, hypochlorous acids (HOCl) and hypochlorite ion (OCl⁻), are commonly referred to as “Free available chlorine”. Note that gas chlorination tends to acidify the water by adding hydrochloric acid, while hypochlorination increases the pH by the addition of sodium hydroxide (ATI, n.d.). Hypochlorous acid will dissociate based on weak acid construction shown in Equation 3 and **Figure 2-2** (Harp, 2002).



Equation 3

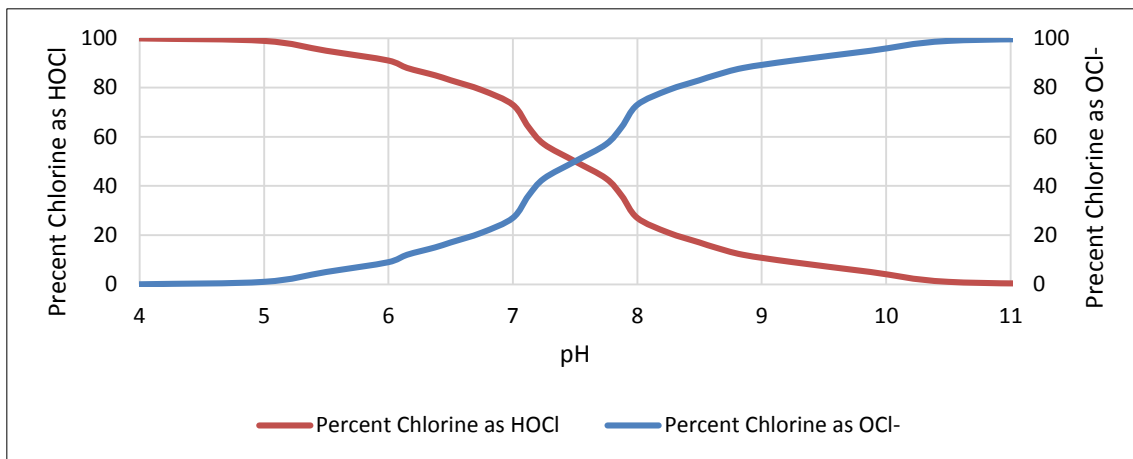


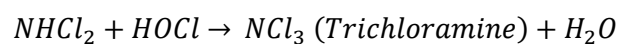
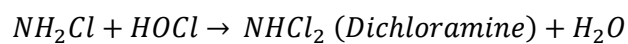
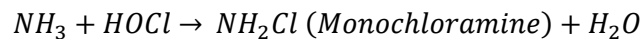
Figure 2-2 Relative Amounts of Chlorine as HOCl and OCl⁻ at 20°C versus pH

Water with a pH between 6.5 and 8.5 presumes a reversible reaction and both species indicated in **Figure 2-2** will be present, where hypochlorous acid is the more effective of the two (Harp, 2002).

Free chlorine has a short life span and is more easily destroyed when it comes into contact with blood. Chlorine is eliminated through blood components like antioxidants. These antioxidants serve as an oxidant/free radical scavengers and protect the body against the formation of cancer. Chronic exposure to chlorine would increase the risk of oxidative stress and reduce the body’s ability to fight cancer.

Chlorine is a relatively strong oxidation agent and can react with a wide range of compounds. Of particular importance is the chlorine reaction with nitrogenous compounds such as ammonia, nitrites and amino acids. Ammonia, commonly present in natural waters, will react with hypochlorous acid or hypochlorite ion to form monochloramine, dichloramine or trichloramines depending on several factors, including pH and temperature of the water.

Chloramine is the product of the chemical reaction between chlorine and an amine. The term chloramine is short for chloro-amine. Amine compounds are organic chemical compounds of the amine group. Chloro-amine exists in several different forms of which the most common group is ammonia with the chemical structure of NH_3 (IBT, 2003). Chloramination is the process of combining chlorine and ammonia to create a “combined” form of chlorine for drinking water disinfection. Typical reactions are indicated below:



Since chloramine is less reactive than free chlorine, it will not form other chlorinated compounds such as tri-chloromethane (CHCl_3) which would occur where high levels of free chlorine are available. Trihalomethane (THM) is a cancer causing agent (IBT, 2003). Consequently chloramines were used more regularly during the 1930s and 1940s, but due to the shortage of ammonia during the second World War its popularity declined (Girard, 2013).

In the past, high concentrations of chlorine were applied to a distribution system to achieve disinfection in long water retention systems. This process caused harmful disinfection by-products (DBPs) and current regulations are implemented to control the limits on chemical disinfectants that can be used in treating water which is meant for human consumption (USEPA, 2003).

Balancing these two chlorine based disinfectants to control THM's and Haloacetic acids (HAA's) is essential in the supply of safe drinking water. While chloramination reduces THM and HAA formation, it triggers nitrification events that may support algal blooms in reservoirs and increase overall bacterial activities in the distribution system (Woolschlager, et al., 2005).

Booster chlorination stations

A booster chlorination station is the reintroduction of chlorine into a distribution system when the current disinfectant potential has been depleted. This process is used to increase the concentration of chlorine to such an extent that within the time required for the water to reach its end destination there will be sufficient chlorine available. This process is widely used because of its easy implementation and low cost associated with it (GLS, 2012).

Breakpoint Chlorination Phenomenon

Chlorination, to the extent that all the ammonia is converted to trichloramine or oxidized to nitrogen or other gases, is referred to as “breakpoint chlorination”. The breakpoint reactions are important in water disinfection since the chloramines are potent biocides, but not as effective as hypochlorous acid or the hypochlorite ion.

Figure 2.3 indicates the typical breakpoint chlorination curve where the combined chlorine (monochloramine and dichloramine) is predominant. If chlorine dosage is increased to a molar ratio of 1.5:1 to 2:1 a breakpoint dose is observed and free chlorine residual is present. The killing power of chlorine is 25 times higher than the combined residual. Thus, the presence of a free chlorine residual is sufficient disinfection. The shape of the breakpoint curve is very dependent on the contact time, water temperature, concentrations of ammonia and chlorine and pH ((Harp, 2002) & (Viljoen, et al., 1997)).

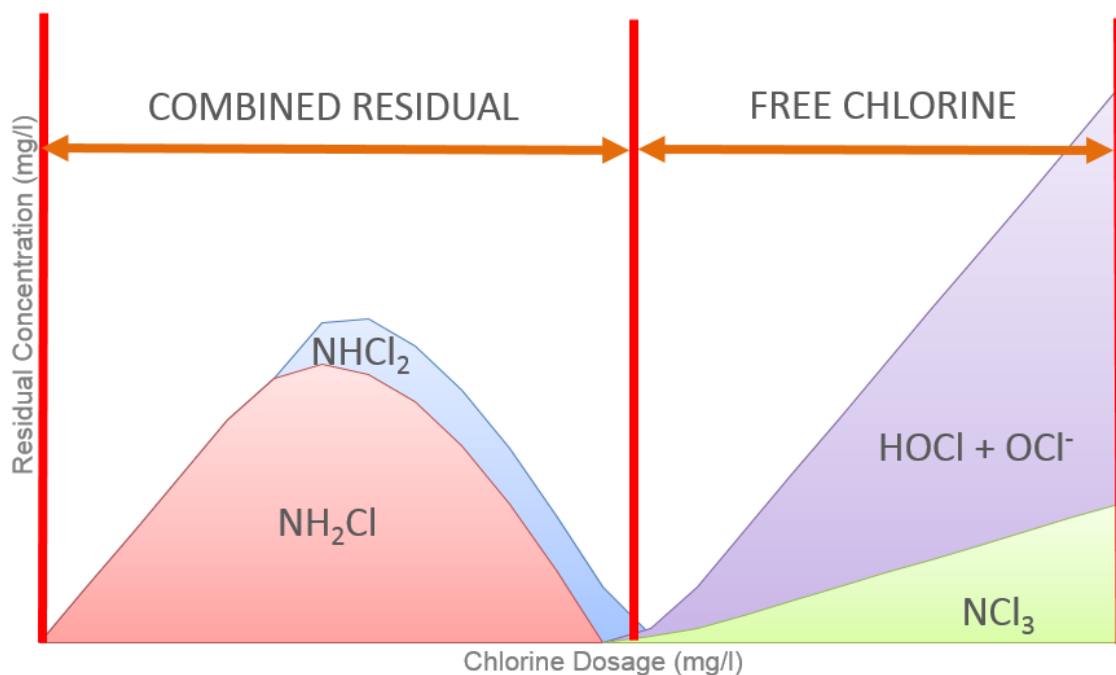


Figure 2-3 Breakpoint Chlorination Curve

Sampling Disinfectant Potential

Monitoring residual chlorine concentration in distribution systems is of particular increasing importance, due to the implementation of the safe drinking water standards and under the Clean Water Act in America. In addition to this, accurate measuring is required for calibration and ultimately validating a water quality prediction tool. The measurement required to monitor residual chlorine will depend on the type of chlorination being practiced in the isolated water system. As indicated, chlorinated water is the most commonly used disinfectant, thus free chlorine sampling being the average measuring tool used. However, the need to have the benefit of longer disinfection potential resulted in chloramine treatment which necessitated the need to measure combined chlorine in order to properly measure the mono-chloramine in the system (ATI, n.d.).

There are two commonly used laboratory procedures for measuring residual chlorine, namely amperometric titration and DPD colorimetric. Amperometric titration is the standard to which all the other methods are compared. The DPD colorimetric is a much simpler procedure and is suitable for quick field tests. Both laboratory procedures produce similar results in most measurements with an accuracy of approximately ± 0.1 PPM. Electronic monitoring systems can increase accuracy to ± 0.01 PPM. However, when monochloramine concentration is significant, the DPD free chlorine testing could result in an over measurement of free chlorine. This is due to the interference with the free measurement caused by monochloramine (ATI, n.d.). Alternative measuring tools are available and should be used in chloraminated water systems.

In the following flow chart the disinfectant potential is calculated in conjunction with **Figure 2-4** Different flow potential based on chlorinated and chloraminated water.

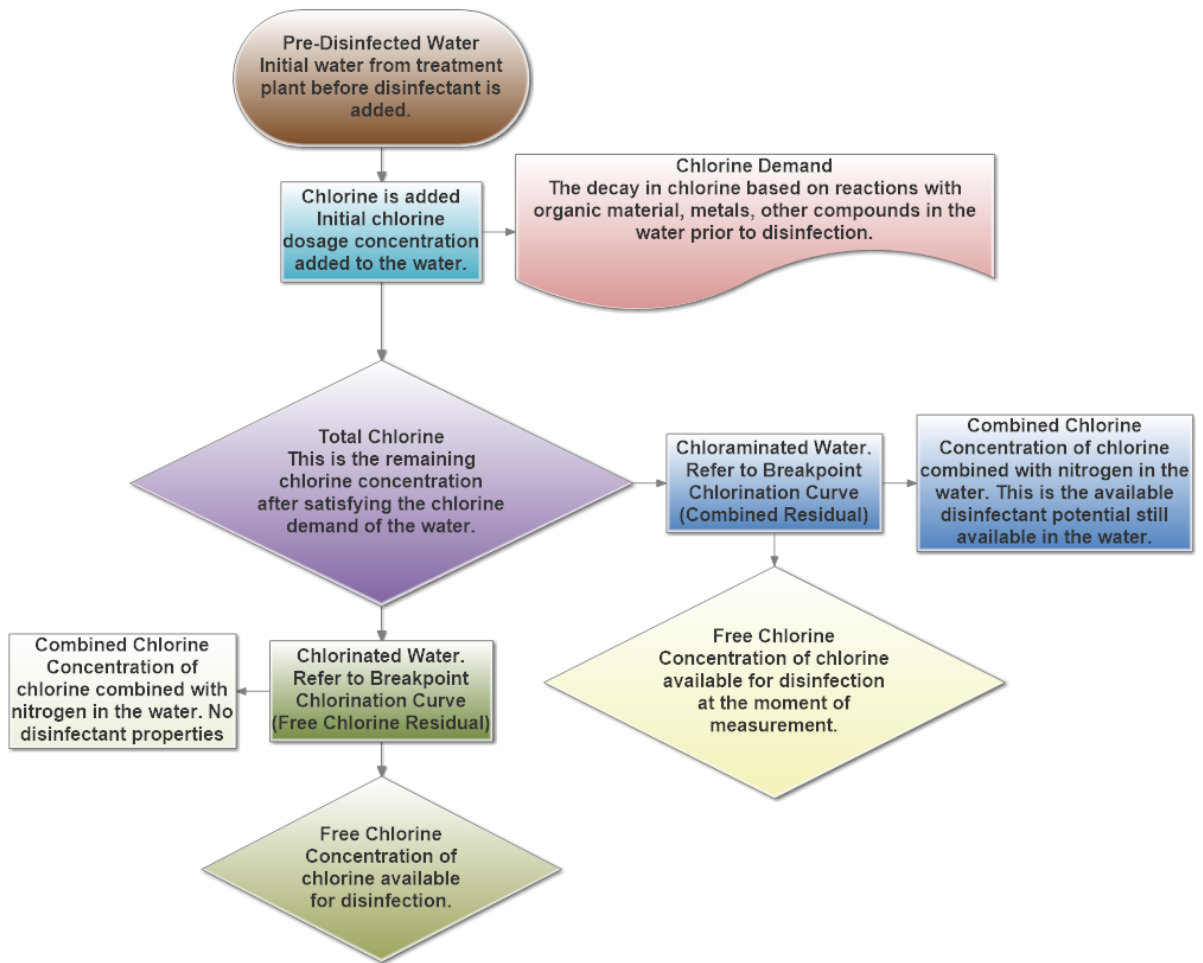


Figure 2-4 Different flow potential based on chlorinated and chloraminated water

* The disinfection potential is dependent on the area in which the reaction is taking place as indicated in **Figure 2.3**.

2.1.2 Ultraviolet radiation

Unlike chlorinated disinfectants, UV radiation does not inactivate microorganisms by chemical reactions. UV radiation inactivates organisms by absorption of the light which causes a photochemical reaction that alters molecular components essential to cell function. As the rays penetrate the cell wall of the microorganism, the energy reacts with nucleic acids and other vital cell components, resulting in damage or death of the exposed cell. The use of this method has raised the conclusion that if sufficient dosage of UV energy reaches an organism, UV radiation can disinfect water to whatever extent is necessary.

The purpose of this thesis is not to determine the efficiency of UV radiation to disinfect potable water, but to model the water quality in a distribution system. The use of UV radiation can have a specific reaction on the decay rate of disinfectants used in a distribution system (USEPA, 1999).

Previous literature shows that UV radiation is exceptionally efficient for disinfection of small microorganisms such as bacteria and viruses. Thus, UV radiation is often considered in conjunction with ozone and/or hydrogen peroxide to enhance the disinfection effectiveness of UV or for ground water where *Giardia* and *Cryptosporidium* are not expected to occur (USEPA, 1999).

The chemistry regarding UV radiation is based on the dissipation of electromagnetic waves into water to be absorbed and reflected off material in water, thus no residual is produced. Although this is attractive from a disinfectant by-products (DBP) standpoint, a secondary chemical disinfectant is required to maintain a residual through the distribution system. UV radiation causes a photochemical reaction in the organism RNA and DNA, which can suggest that UV radiation of water can result in the formation of ozone or radical oxidants (Ellis & Wells, 1941).

Before additional disinfectants are required UV lamps are used in much the same way as fluorescent lamps. UV radiation is emitted from electron flow through ionized mercury vapour to produce UV energy in most units. The main difference between these two lamps is that the fluorescent lamp bulb is coated with phosphorous, which converts the UV radiation into visible light. The UV lamp does not have this coating, thus it transmits the UV radiation generated by the arc of the lamp (White, 1992).

DBP formation with Chlorination and Chloramination following UV radiation is very important in the prediction of water quality within a distribution system. UV radiation does not directly form DBPs or alter the concentration or species of DBPs formed by post-disinfection (Snicer, et al., 1996).

However, the question of whether UV radiation influences the rate of DBP formation by post-disinfection is important for water quality prediction. According to the studies it was found that UV radiation does not significantly affect the rate of DBP formation (Snicer, et al., 1996).

2.1.3 Ozone

Ozone (O_3) is found in nature. Ozone is the product of an electrical discharge in air of which the process is explained below:

Air is made up of diatomic oxygen (O_2) and nitrogen (N_2). Diatomic oxygen is a molecule composed of two oxygen atoms held together by four equally shared electrons. As electricity is discharged into the air some of these molecules are split into atoms. These atoms then quickly react with the oxygen molecules to form ozone ($O_1 + O_2 = O_3$) (USEPA, 1999).

Ozone is a very reactive oxidant and unstable gas. This gives it a short life span before it reverts back to oxygen. The unstable gas is known as a free radical of oxygen. It will readily give up one atom of oxygen providing a powerful oxidising agent which is toxic to most water borne organism such as bacteria, mould and yeast spores, viruses or harmful protozoa that form cysts. The single oxygen atom binds with these organisms and causes them to oxidize (USEPA, 1999).

For this study no further work was done regarding ozone treatment because it has limited residual qualities that could impact water quality over time.

2.2 Hydraulic Modelling

Throughout history people have been trying to control floods, irrigation and water supply. This resulted in an enormous amount of progress in the field of hydraulics. However, the nature of hydraulics is still somewhat empirical, with many indeterminate factors. These problems have been resolved by using powerful mathematical techniques, but the practical hydraulics problem still does not have theoretical solutions. With the expansion of urban development large volumes of water need to be transported to sufficiently supply everyone, resulting in a need to determine the energy required to move water from one point to another (Clark, 2011). These techniques is discussed in this section.

There are different types of computer-based mathematical hydraulic models for water distribution systems that have been developed to be used to assess the movement of contaminants within the distribution system. These hydraulic models can be split into three main categories, viz. the basic hydraulic model that simulates flow volumes, direction and pressures in pipes, and in addition, the more advanced steady state water quality and flow tracing models that determine the movement of contaminants, flow path and travel times under steady-state and operational and demand conditions. The model that indicates the most properties of water quality modelling, is the dynamic water quality model. This model determines the movement of contaminants and flow patterns under dynamic conditions.

The main objective of a water distribution system is to supply water to the consumer in proper quality and pressure, with acceptable quality in terms of flavour, odour, appearance and sanitary security (Castro & Mario, 2003).

To accurately determine the water quality in a distribution system one must calibrate and ultimately validate the hydraulic model. Simulation of pressure and demand with accurate pipe properties is essential to ultimately obtain an accurate water quality model.

Hydraulic Principles

Hydraulic principles are divided into three basic relationships. These relationships will only be discussed briefly as it is assumed that basic hydraulic knowledge is used throughout the thesis. The relationships used to calculate fluid flow in pipe networks are:

- Conservation of Mass: (density of fluid) x (volume of fluid entering per second) hence:

$$Q = u_1 A_1 = u_2 A_2$$

u = flow velocity (m/s)

A = Area perpendicular to the direction of flow (m²)

The principle is based on the fact that the sum of the mass flows in all pipes entering a junction must equal the sum of all mass flows leaving the junction. Water is essentially an incompressible fluid, thus conservation of mass can also be seen as conservation of volume (Chadwick, et al., 2004).

- Conservation of Energy: (energy entering = energy leaving) hence

$$\frac{p_1}{\rho g_1} + \frac{u_1^2}{2g} + z_1 = \frac{p_2}{\rho g_2} + \frac{u_2^2}{2g} + z_2 = H = \text{Constant}$$

u = flow velocity (m/s)

p = pressure (N/m²)

ρ = density (kg/m³)

g = gravity (N/kg)

z = potential or elevation head (m)

Hydraulic systems typically only have three main types of energy. Kinetic energy is associated with the movement and potential energy is associated with the elevation. Head is referred to as energy and energy losses within a network are associated primarily with friction along pipe walls and turbulence (Chadwick, et al., 2004).

- Pipe Friction Head Loss: Darcy-Weisbach equation (Chadwick, et al., 2004)

$$h_f = \frac{\lambda L V^2}{2gD}$$

h_f = head loss due to pipe friction (m)

λ = pipe friction factor ($\lambda = 0.316/Re^{0.25}$)

Re = Reynolds number (dimensionless)

D = pipe diameter (m)

V = flow velocity (m/s)

The key in determining the flow through a pipe is based on the capacity to calculate the friction head loss. Three empirical equations are used namely; Darcy-Weisbach, Hazen-Williams, and Manning equations. All three these equations relate head loss in pipes to velocity, length of pipe, pipe diameter and pipe roughness (Clark, 2011).

Water Quality Modelling

Although the thesis is based on water quality modelling, it is essential to remember that all water quality simulations are based on hydraulic modelling. This includes, as mentioned, the three basic relationships in hydraulics and the calibration and validation processes of hydraulic modelling.

A water quality model is based on the output of the hydraulic model in conjunction with additional inputs to predict the temporal and spatial distribution of a variety of constituents within a distribution system (Clark, 2011).

These elements are as follows:

- The fraction of water originating from a particular source;
- The age of the water;
- The concentration of a nonreactive tracer compound either added to or removed from the system;
- The concentration of a reactive compound, including the concentration of a secondary disinfectant with additional input of its loss rate (Chlorine or Chloramines) and the concentration of DBP with their growth rate.

Different water quality processes occur in water distribution systems that can lead to the introduction of contaminants and water quality transformation as water moves through the network. Water quality can be degraded by cross connections, failure at the treatment barrier, and transformation in the bulk phase. Corrosion, leaching of pipe material, biofilm formation, and scour can occur at the pipe wall to degrade water quality as it moves through the water pipe system. Additional to this, bacteriological quality changes may cause aesthetic problems involving taste and odour development, discoloured water, and other adverse impacts (NRC, 2006).

In addition to the basic water hydraulics indicated in this section, the mathematical equations indicated in the next chapter are based on conservation of constituent mass. These models represent the following phenomena occurring in a distribution system:

- Advection, transport of mass within pipes;
- Mixing of mass at pipe junctions;
- Mixing of mass within storage tanks;
- Reaction within pipes and storage tanks.

This is explained further in the following **Chapter 2.3**, Mass Transfer.

2.3 Mass Transfer

The fundamentals of hydraulic and water quality modelling are not mutually exclusive. This is due to continuity, conservation of momentum and mass transport as indicated in this chapter.

2.3.1 Pollutant Transfer Processes

Pollutants can be transported in two ways, namely advection or diffusion. Advection is the transport of bulk fluid motion without any disturbance in the physical structure of the particle along the flow path. Diffusion is the movement of pollutants due to random motion and disruption of the physical structure as indicated in **Figure 2-5**.

A dissolved substance can be transported along the length of the pipe by bulk fluid motion (average velocity) or by dispersion. In turbulent flow mass transport is dominated by advection. Mass transport due to dispersion may be important at dead ends or when flow conditions are laminar. Turbulent flow normally occurs in transmission systems (Chadwick, et al., 2004). These two transport processes are indicated in **Figure 2-5**.

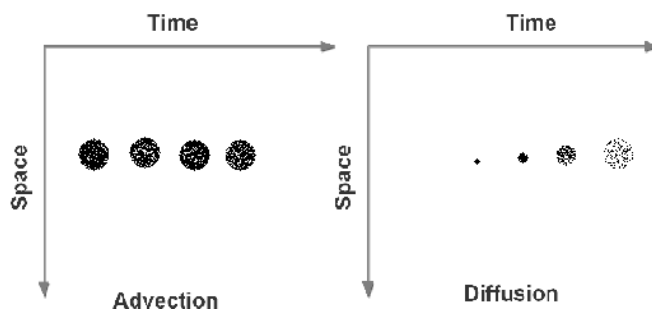


Figure 2-5 Advection Vs Diffusion

2.3.2 Transport Equations

Transport equations represent the constituent transport of mass in 3-dimensions. The following section shows the two mechanisms by which mass is transported through advection and dispersion.

Transport Mechanisms

The equation shows the fundamental transport of mass in 3-dimensions. The equation represents the two mechanisms by which mass is transported (Bird, et al., 2007).

3-Dimensional Mass Transport

$$\frac{\partial C}{\partial t} = -u \frac{\partial C}{\partial x} - v \frac{\partial C}{\partial y} - w \frac{\partial C}{\partial z} + D \left[\frac{\partial^2 C}{\partial x^2} + \frac{\partial^2 C}{\partial y^2} + \frac{\partial^2 C}{\partial z^2} \right] + r$$

In the modelling of non-conservative water quality constituents, the rate of reaction is normally governed by a reaction rate as indicated as a constant, which is dependent on temperature. The equation represents the point of departure in quantifying the concentration profiles in water networks (Bird, et al., 1960).

As discussed the following section of the equation is referred to advection transport:

$$-u \frac{\partial C}{\partial x} - v \frac{\partial C}{\partial y} - w \frac{\partial C}{\partial z} \quad \text{Equation 4}$$

Diffusive transport is presented in equation 5 as indicated below:

$$D \left[\frac{\partial^2 C}{\partial x^2} + \frac{\partial^2 C}{\partial y^2} + \frac{\partial^2 C}{\partial z^2} \right] \quad \text{Equation 5}$$

Source or Sink term:

$$+r$$

Plug Flow Conditions

To attain plug flow conditions, mass transport must be by advection alone. To illustrate this, Fick's first law describes the diffusion of a chemical specie as indicated below (Freeze & Cherry, 1979).

$$J = -D_0 \tau_a \theta \frac{\partial c}{\partial x}$$

Where

τ_a = Tortuosity factor

θ = Volumetric water content

D_0 = Free- solution diffusion coefficient

As indicated in the equation above, diffusive transport of tortuosity factors cannot be measured independently. Therefore, it is convenient to define an effective diffusion coefficient, D^* , as follows (Bear, 1972):

$$D^* = D_0 \tau_a$$

This is then substituted into Fick's first law equation to get a description of a steady state diffusive flux solutes ((Freeze & Cherry, 1979) and (Bear, 1972)).

$$J = -D^* \theta \frac{\partial c}{\partial x}$$

Thus for time dependant transport of nonreactive solutes, Fick's second law is assumed to apply (Freeze & Cherry, 1979) (Bear, 1972),

$$\frac{\partial c}{\partial t} = D^* \frac{\partial^2 c}{\partial x^2}$$

By implementing a one dimensional equation with no advection and no reaction as indicated by Fick's second law of diffusion, the following equation can be derived (Bird, et al., 1960):

$$\frac{\partial c_i}{\partial t} = \frac{\partial Q_i \partial c_i}{A_i \partial x} + \theta (C_i), i = 1$$

2.3.3 Reactions in Pipes

Many reactions in the pipe network are physical and chemical. Most of these reactions are not understood and thus not modelled due to its complexity. Bulk reactions occurring in the bulk flow of the water mass are the reactions between water molecules. Additionally, in pipe networks surface reactions occur, such as biofilm reacts with the disinfectant in the water.

A visual illustration of the reactions experienced in a pipe is indicated in **Figure 2-6** below. The equations shown thereafter calculate the decay of a specific disinfectant in the water. The decay at the pipe wall is a function of the transport to the wall and the decay rate at the wall.

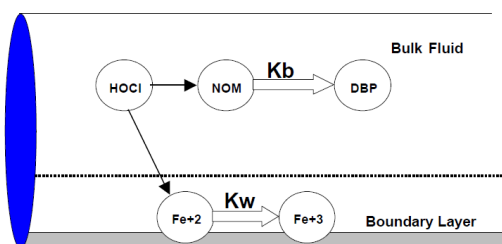


Figure 2-6 Disinfectant Reaction in Pipes (Rossman, 2000)

Where,

- HOCL = Disinfectant
- NOM = Natural Organic Matter
- DBP = Disinfectant-By Products
- Fe⁺² = Oxidized Iron (Corrosion)
- K_b = Bulk Reaction rate coefficient

K_w = Pipe Wall Reaction rate coefficient

By using previous studies, a first-order decay model was used as reaction kinetic model for chlorine in the bulk liquid, and zero-order decay model was used as wall reaction kinetic model, thus a decay reaction can be calculated as indicated below (Nagatani, et al., 2008).

$$\frac{dC}{dt} = k_b C + \frac{4}{d} k_w$$

k_b = First-order bulk decay coefficient (1/day)

d = Pipe diameter (mm)

k_w = Zero-order wall reaction decay coefficient (mg/m²/day)

2.4 Water Quality Modelling (Disinfectant Decay)

The decay measured in pipes is affected by many functions including the properties of the pipe and the source water. Reservoirs and other storage units have their own properties as well and can affect the potential decay of disinfectant. By using different coefficients that take into account the properties of these factors, one can determine an accurate decay rate for every disinfectant in an isolated pipe, water source or storage segment.

2.4.1 Decay Rates in Pipes

As indicated in the previous chapter the decay rates of a disinfectant in a pipe's section is a function of the bulk water and the pipe wall. The bulk water (source water) has its own properties and will be explained independently of the pipe decay rates.

The following study was performed by the Civil and Environmental Engineering Department, University of Central Florida, to illustrate the rate of chloramine decay in a pilot distribution system (PDS) receiving corrosion inhibitors. Eighteen pilot distribution systems were created to demonstrate the decay rate obtained from the wall of the pipe. This included Cast Iron, PVC, Ductile Lined Iron, Galvanized Steel and 14 hybrid lines. The study used four ground water sources treated with different treatment methods and two surface water sources with ozonation and nanofiltration as treatment method. The PDS was used to mimic dead-end conditions, thus the water was left for two days and then measured (Arevalo, et al., n.d.).

The data was grouped together by pipe material and thus showed different decay rates based on the same water source, disinfectant and temperature as indicated in **Figure 2-7** below. This only illustrates the different decay patterns for different pipe materials and was not used for any additional purpose other than informational purposes based on the fact that it also includes bulk decay as a constant for all the materials.

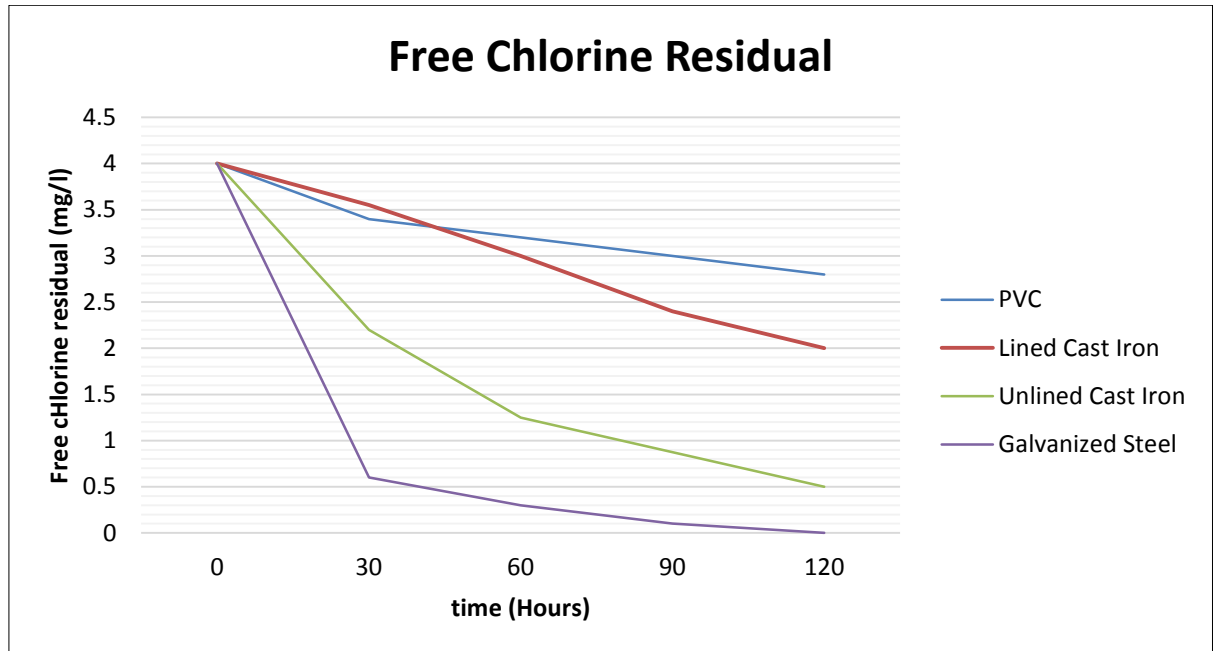


Figure 2-7 Material Vs Free Chlorine Residual

The study further investigated the impact that pipe diameter and flow velocity has on the overall decay rate of chlorine. The following equation was developed based on the Reynolds Number for chlorine decay.

$$K = 0.01 \times R_e^{(0.2209)}$$

K is the overall decay constant per hour and R_e the Reynolds number. The aforementioned indicated a method that could be used to determine decay rates for chlorine decay (Arevalo, et al., n.d.). Additional modelling techniques are indicated in **Chapter 2.4.3**.

As indicated, this study only shows the dramatic changes in decay rates that could be observed between pipes based on specific pipe condition and material.

The following factors influence water quality in a distribution system based on pipe characteristics (AWWA, 2006):

- Presence and density of biofilm inside the distribution system;
- Pipe Material and presence of corrosion in pipes as indicated in the study above;
- Size, age and configuration of network, including pipe diameter.

2.4.2 Decay Rates in Reservoirs

The reduction in disinfectant potential in reservoirs is a function of the bulk water and the characteristic of the reservoir. Reservoirs have different shapes and sizes, and are designed for storage of reserve water. For this specific reason long retention time (retention time is the time a solute takes to travel through a column) can be expected of reservoirs supplying large areas. With longer retention time more disinfectant potential is lost. The characteristics of mixing fresh and aged water in a reservoir are also essential and must be modelled accordingly.

2.4.3 Disinfectant Decay Behaviour

Each disinfectant has its own decay rate depending on the source water characteristics. These characteristics affect the bulk decay rate and is indicated below (AWWA, 2006):

- Treatment processes used at the plant;
- Type of disinfectant and the doses used at the treatment plant or in the distribution system as a booster station;
- Physical and chemical characteristics of the water, which includes temperature, PH, dissolved oxygen concentrations, etc.;
- Type and quantity of NOM and nutrients present in the water body;
- Type and concentration of microorganisms;
- Presence of sediments introduction in pipe during installation or repairs.

Bulk water decay can also be influenced by operations and not only by the water source alone. This includes hydraulic conditions such as water velocity, pressure, and residence time, and ultimately the distribution system operation and maintenance procedures form an important role in bulk water decay.

As indicated by Chapra (Chapra, 2008) the residence time τ_E of a substance E represents the mean amount of time that a molecule or particle of E would stay or “reside” in a system. For a steady state it is defined for a constant – volume system as (Stumm & Morgan, 1981):

$$\tau_E = \frac{E}{|dE/dt|_{\pm}}$$

Where E = Quantity of E in the volume (M)
 $|dE/dt|$ = absolute value of either the source or the sinks (MT^{-1})

2.4.3.1 Laboratory batch test decay rates

The following section is based on the results obtained from the World Research Commission (WRC) thesis on the prediction for chlorine decay from potable water in pipeline systems. The objectives of that report was to develop a standard test procedure to calculate an accurate bulk water decay rate, to select the most suitable mathematical model, to measure the effects of temperature on chlorine and to establish typical ranges for model calibration constants regarding decay rates (Viljoen, et al., 1997).

The tests were divided into free chlorine decay and monochloramine decay.

Free Chlorine Decay:

Work was conducted over a period of 48 hours with an added chlorine concentration of breakpoint + 1mg/l (Breakpoint as explained in **Chapter 2.3**). The breakpoint was determined after 20 minutes. Four water samples were tested every 15 minutes for the first half hour, on the first hour and every second hour thereafter for a period of 48 hours. These sample testing included all chlorine species (Viljoen, et al., 1997).

The data was then graphed over the 48 hour time frame and several curve fittings were done by applying the least squared method for varying time intervals to produce the best fit for the shortest time possible (Viljoen, et al., 1997).

Two types of curve fitting was done after the data was graphed. Initially, the data points were maintained and curve fittings were done with increasing time intervals. Then, the data points were kept constant while the initial points were increased. This resulted in a graph that was used to see the effects of different time intervals on the chlorine decay rate constant. Based on these results that during the first hour of testing an initial fast reaction takes place which did not show the long-term decay rate. Thus, by analysing the graphs it was clear that the testing of samples should not commence within the first hour due to the initial reactions. It appeared as if the reactions had stabilised after the first hour (Viljoen, et al., 1997).

Figure 2-8 represents the effect of the test duration on the first order decay rate constant for free chlorine, shown for the four different raw water types. The data points represents a decay rate constant obtained from the curve fitting mentioned earlier. It is evident that for the time period of 8 hours or longer the decay rate constants stabilised for all four raw water sources (Viljoen, et al., 1997).

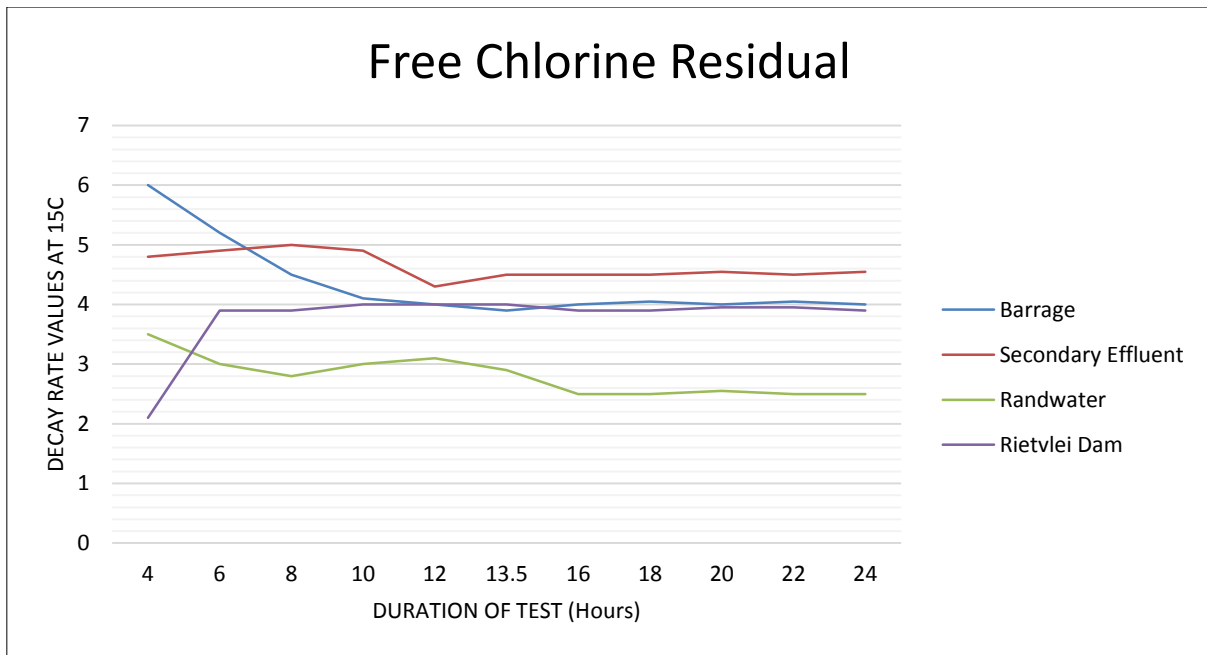


Figure 2-8 Effects of the test duration on the first order decay rate constant for free chlorine, shown for the four different raw water source

In summary to determine chlorine decay rate constant for free chlorine, these laboratory experiments done by (Viljoen, et al., 1997) show that the overall time period of 8 hours should be conducted. Measurements should be taken every hour except for the first hour (Viljoen, et al., 1997).

Monochloramine Decay:

As indicated in the previous test for free chlorine, a 48 hour time period was selected to conduct tests for monochloramine. The original chlorine concentration added was 1mg/l, all four water types were tests for all chlorine species at pre-determined time intervals. Based on the longer disinfection abilities of monochloramines, measurements were taken every second hour for the total period of 48hours (Viljoen, et al., 1997).

The data was then graphed over the 48 hour time frame and several curve fittings were done by applying the least squared method for varying time intervals to produce the best fit for the shortest time possible (Viljoen, et al., 1997).

After analysing these graphs it was clear that monochloramine also has an initial fast reaction that took place. This initial reaction takes around four hours. Thus, no measurements should be taken during the first four hours.

Figure 2-9 Effects of the test duration on the first order decay rate constant for monochloramine, shown for the four different raw water source below represents the effect of the test duration on the first order decay rate constant for monochloramine, shown for the four different raw water types. The data points represents a decay rate constant obtained from the curve fitting mentioned earlier. It is evident that for the time period of 24 hours or longer the decay rate constants stabilises for all four raw water sources (Viljoen, et al., 1997).

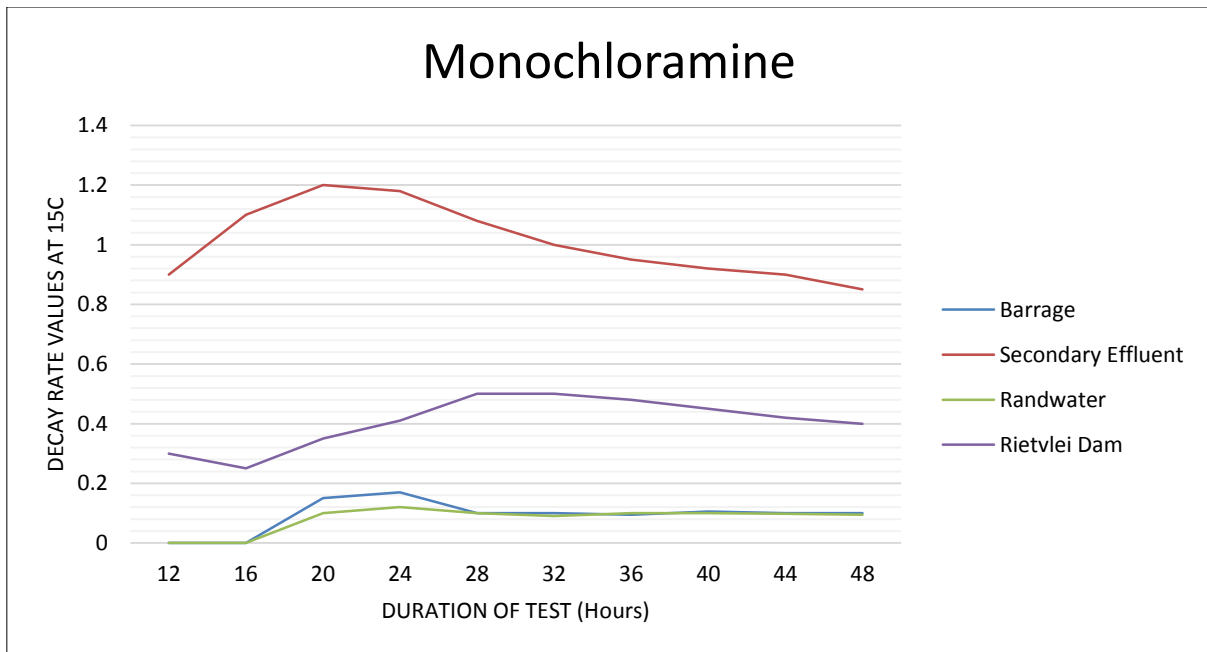


Figure 2-9 Effects of the test duration on the first order decay rate constant for monochloramine, shown for the four different raw water source

In summary, to determine the chlorine decay rate constant for monochloramine, laboratory experiments should be conducted over a total time period of 24 hours, taking measurements every 4 hours. No measurements should be taken during the first 4 hours of the experimental tests (Viljoen, et al., 1997).

The decay rates observed between different water sources and different disinfectant products vary enormously. Thus, emphasis is put on the need to sample and test individual sources of water for a decay constant. Four water sources were used in this report obtained from the WRC. The results indicated that to determine the disinfectant decay rate constant the following steps must be taken (Viljoen, et al., 1997):

1. First hour has an unrealistic decay rate and must not be used in the decay calculations, this is due to the initial disinfectant contact time required and the original chlorine demand of the water;

2. During the following seven hours the decay constant stabilizes and can be used for water quality modelling.

This section was used to determine a pre-calibration decay rate when laboratory batch testing was not available for a specific source water as in the case study conducted in this thesis. The theoretical decay rates for chlorinated water is assumed to be close to -0.55/day and -0.185/day for chloraminated water.

2.4.4 Most suitable mathematical model:

Various mathematical models are available to the water industry for modelling chlorine decay in distribution system. Three different mathematical models were tested in the Viljoen (Viljoen, et al., 1997) report. These models must be examined to determine the reaction order of chlorine decay in the bulk liquid phase and the number of parameters considered when modelling pipelines (bulk water flow, pipe wall reactions and radial diffusion from the bulk water flow to the pipe wall). Mathematical models for the determination of chlorine concentration decay were divided into the following models:

1. The first - order, 1 parameter model

This model contains only one parameter, the bulk water in the pipeline. It's the most basic of all the models and accounts only for the decay in chlorine concentration in the bulk liquid phase. The influence of the pipe wall, and the radial diffusion are ignored. The model could be represented as:

$$\begin{aligned}\frac{dC}{dt} &= -kC \\ \therefore \frac{dC}{C} &= -kdt \\ \therefore \int_{C_0}^C \frac{dC}{C} &= -k \int_0^t dt \\ \therefore \ln C - \ln C_0 &= -kt \\ \therefore \ln \frac{C}{C_0} &= -kt \\ \therefore C(t) &= C_0 e^{-kt}\end{aligned}$$

Thus, by using this equation the concentration of the chlorine can be calculated at any point at time t ((Viljoen, et al., 1997) and (Levenspiel, 1972)).

2. The n^{th} – order, one parameter model

The n^{th} – order can be described in exactly the same manner as the first-order, one parameter model except that the reaction rate is of the n^{th} -order. Thus, the assumption that the reaction rate of $k = 1$ is not made. k is instead obtained from the chlorine loss in a batch system test calculated from laboratory measurements (Viljoen, et al., 1997).

$$\begin{aligned} \frac{dC}{dt} &= -kC^n \\ \therefore \frac{dC}{C^n} &= -kdt \\ \therefore \int_{C_0}^C \frac{dC}{C^n} &= -k \int_0^t dt \\ \therefore \left[\frac{C^{(-n+1)}}{-n+1} \right] - \left[\frac{C_0^{(-n+1)}}{-n+1} \right] &= -kt \\ \therefore C^{-n+1} - C_0^{-n+1} &= -kt(-n+1) \\ \therefore C &= \left[kt(n-1) + \frac{1}{C_0} \right]^{\frac{1}{-n+1}} \\ \therefore C &= \left[kt(n-1) + \frac{1}{C_0} \right]^{\frac{1}{n-1}} \end{aligned}$$

Where:

- $C(t)$ = chlorine concentration at time t (mg/l)
- C_0 = initial chlorine concentration (mg/l)
- k = n^{th} -order chlorine decay rate constant (1/s)
- t = residence time in the pipe
- n = order coefficient

3. A revised model of the nth- order, one parameter model

The revised third model is based on the nth-order, one parameter model. To assure that every graph obtained during data analysis approaches a certain value, an arbitrary value, denoted by X, was introduced in the previous equation. Thus, the revised nth-order, one parameter model could be presented as (Viljoen, et al., 1997):

$$C(t) = \left[kt(n-1) + \frac{1}{C_0 + X} \right]^{(n-1)^{\left(\frac{1}{n-1}\right)}} + X$$

Where:

- C(t) = chlorine concentration at time t (mg/l)
- C₀ = initial chlorine concentration (mg/l)
- k = nth-order chlorine decay rate constant (1/s)
- t = residence time in the pipe (s)
- n = order coefficient
- X = variable to assure that every graph strives to a certain non-zero value

Thus, by including the variable X, the chlorine decays at a certain rate until a minimum concentration and not zero. This was used as most suited mathematical model and incorporated into the water quality modelling tool (**Chapter 2.5**).

2.4.5 Findings from literature review based on Disinfectant Decay

The following observations can be made based on the research obtained in this section regarding disinfection decay behaviour:

1. Chlorine decay in a distribution system could occur based on both bulk fluid and from reactions with materials associated with the pipe wall (Arevalo, et al., n.d.);
2. There is no established method for directly determining the kinetics of chlorine decay due to the pipe wall reactions, as indicated in the study from (Arevalo, et al., n.d.);
3. Mass transfer expression based on a two dimensional model containing radial diffusion could produce more accurate results than using a mass transfer coefficient over the total system;
4. Analysis of chlorine decay test showed that an n^{th} –order, or parallel first order kinetic models, provided better fits to the data than did first order models (Viljoen, et al., 1997).;
5. Pipe wall reactions related to corrosion of ferrous pipes material consume more chlorine than those related to biofilm;
6. The rates associated with reaction of chlorine at the pipe wall is inversely related to pipe diameter and can be limited by the rate of mass transfer of chlorine to that wall;
7. Calibration of the hydraulic model is a requirement for attempting the modelling of a water quality model in a distribution system;
8. Bottle testing could be used to determine the bulk water chlorine decay coefficient based on a first order kinetic constant. First or zero order kinetic constants can be used to determine the pipe wall reaction coefficient if the pipe roughness coefficient is inversely related to the wall kinetic constant (Viljoen, et al., 1997)

2.5 Water Quality Modelling (Mathematical Modelling)

EPANET (USEPA, 2009), is a simple hydraulic computer program that models distribution systems (pipe, pumps, and reservoirs). The program has the ability to calculate the eventual age of the water at any given point as well as concentration of chemicals. Decay rates of a disinfectant can be entered into the model and this provides a method of back calculating the initial required concentration at the bulk supply connection point of a distribution system. A set of rules could also be applied to control pumps, reservoirs and valves to mimic real life scenarios (van Der Walt, et al., 2002).

Mathematical modelling of disinfectant decay within a water supply system is a problematic solution which is not yet completely mastered. Decay simulation regarding bulk fluid and pipe walls can be conducted within EPANET. The phenomena of chlorine reaction with chemical species is introduced into the program by means of a first order kinetics constant, k_B . The contribution made by the pipe wall reactions to the decay of the disinfectant is introduced into the program through another constant K_f . K_f meaning is more complex. These constants were explained in **Chapter 2.3.3**, Reaction in Pipes, but are included below for comprehensiveness.

$$\frac{dC}{dt} = k_b C + \frac{4}{d} k_w$$

Based on research done by (Westbrook & Digiano, 2009), water quality models such as EPANET (USEPA, 2009) can be successfully implemented to predict disinfectant residuals if the rates for disinfectant loss, both in the bulk water and at the pipe wall, are known. Wadiso is based on EPANET and was used as the modelling tool in this thesis (GLS, 2013). Wadiso has numerous additional tools to simulate water quality aspects and is useful in the data collection phase of a project.

When the water distribution systems of municipalities are modelled, huge amounts of data are gathered and data integration can become problematic. Wadiso has a multi-program-platform that makes data collection more structured. Thus, in areas where model validation requires additional investigation to increase reliability Wadiso can track integrity updates with ease.

2.5.1 Hydraulics (EPANET)

As indicated, Wadiso is based on EPANET and was used as the modelling tool in this thesis to calculate disinfectant potential in large water utilities.

The method used in EPANET/Wadiso to solve the flow continuity and head loss equations that characterize the hydraulic state of the pipe network at any given time can be termed a *hybrid node-loop approach*.

In addition, EPANET water quality solver is based on the principals of conservation of mass, coupled with reaction kinetics. This phenomena and reactions regarding bulk and wall, together with water and source tracing were explained in the **Chapter 2.3** and **2.4** (Rossman, 2000).

2.5.1.1 Gradient Method

Todini and Pilati (Todini & Pilati, 1987) and later Salgado (Salgado, et al., 1988) chose to call the hybrid node-loop approach the gradient method. Different approaches have also been developed, the only difference between these methods is the way in which link flows are updated after a new trial solution for nodal head has been calculated. Todini's approach was chosen for EPANET because it was simpler (Rossman, 2000).

2.5.1.2 Bulk Reactions

Modelling reactions occurring in EPANET based on the bulk flow with n^{th} order kinetics, where the instantaneous rate of reaction are assumed to be concentration-dependent. This is according to the following equation (Rossman, 2000).

$$R = K_b C^n$$

Where,

R = Instantaneous rate of reaction (mass/volume/time)

K_b = a bulk reaction rate coefficient, this is positive for growth reactions and negative for decay reactions

n = reaction order

C = reactant concentration (mass/volume)

As indicated in **Chapter 2.4.4**, Most Suitable Mathematical Model, an additional reaction is required where a limiting concentration exists. EPANET can also consider

these limiting concentrations based on ultimate growth or loss of the substance. In this case the rate expression is as follows (Rossman, 2000):

For $n > 0$, $K_b > 0$

$$R = K_b (C_L - C)C^{(n-1)}$$

For $n < 0$, $K_b < 0$

$$R = K_b (C - C_L)C^{(n-1)}$$

Where C_L is the limiting concentration as indicated in the revised model of the n^{th} -order, 1 parameter model. The three parameters (K_b , C_L and n) are used to characterize bulk reaction rate (Rossman, 2000).

2.5.1.3 Wall Reactions

The mathematics regarding the most suitable model for wall reaction was explained in **Chapter 2.4**. EPANET uses the following expression to calculate the reaction occurring at or near the pipe wall (Rossman, 2000):

$$R = \left(\frac{A}{V}\right) K_w C^n$$

Where K_w is the wall reaction rate coefficient and (A/V) is the surface area per unit volume (m^2) within a pipe (as previously indicated in **Chapter 2.4.1**, equal to four divided by the pipe diameter). EPANET limits the choice of wall reaction order to either 0 or 1, so to indicate that K_w is either mass/area/time or length/time, respectively. As with K_b , K_w must be supplied to the program by the modeller (Rossman, 2000).

K_w needs to be adjusted to account for any mass transfer limits in moving reactants and product between the bulk flow and the wall. EPANET does this automatically, by basing the adjustment on the molecular diffusivity of the substance being modelled and on the flow's Reynolds number. Setting the molecular diffusivity to zero will remove the mass transfer effects (Rossman, 2000).

The wall reaction coefficient has many parameters that can be dependent on temperature and can also be correlated to pipe age and material. As the roughness increase is based on encrustation and tuberculation of corrosion over years so does the friction head loss of the flow in the pipe (Rossman, 2000).

Based on the conclusion in EPANET 2 User's Manual (Rossman, 2000) there is some evidence that suggests that the same processes that increase a pipe's roughness with age also tend to increase the reactivity of its wall with some chemical species, especially chlorine and other disinfectants. EPANET can be used to make each pipe's K_w be a function of the coefficient used to describe its roughness. The different functions are dependent on the formula used to calculate headloss through the pipe (Rossman, 2000):

Headloss Formula	Wall Reaction Formula
Hazen-Williams	$K_w = F/C$
Darcy-Weisbach	$K_w = -F/\log(e/d)$
Chezy-Manning	$K_w = F n$

Where C = Hazen-Williams C-factor, e = Darcy-Weisbach roughness, d = pipe diameter, n = Manning roughness coefficient, and F = wall reaction – pipe roughness coefficient. The last coefficient must be calculated based on site-specific field measurements (Rossman, 2000).

2.5.1.4 Water and Source Tracing

Additional to chemical transport, EPANET also models the changes in the age of the water throughout a distribution system. Water age is linked to a parcel of water based on the time occupied in the network. Water is indicated as new when it enters into the system through reservoirs or where source nodes are when the modeller indicated that the age is zero. Water age provides a simple, non-specific measurement of the overall quality of the drinking water. EPANET calculates age reaction as a zero-order kinetics with a rate constant of one (Rossman, 2000).

EPANET can also perform source tracing. This enables the modeller to track, over time, what percentage of water, reaching any node in the network, had its origin at a particular node. EPANET can track any node and treats this node as a constant source of a non-reacting constituent that enters the network with a concentration of 100. This is a helpful tool in determining the percentage source water mixed with an additional source (Rossman, 2000).

2.6 Demand Patterns

Demand patterns are the ratio allocated to each user to simulate an actual dynamic flow pattern of a specific user. This was then extended to a 168 hour pattern based on a 24 hour land use pattern indicated in the Table 2-1. **Table 2-1** shows the peak factors used on the average annual daily demand (AADD) per land use. Pattern 1 to 3 indicate residential land use for a large to small holding, Pattern 4 shows the typical theoretical demand pattern for a low cost house, Pattern 5 and 6 show the pattern for business, commercial and industrial land use and Pattern 7 indicates the flow for unaccounted water (UAW) and is set as one times AADD.

Table 2-1 Peak Factors for Land use (24hours)

	Residential Large	Residential Medium	Residential Small	Low Cost Housing	Business, Commercial Industrial Medium	Business, Commercial Industrial Large
Hour	Pattern1	Pattern2	Pattern3	Pattern4	Pattern5	Pattern6
1	0.5	0.3	0.1	0.6	0.4	0.4
2	0.5	0.3	0.1	0.6	0.4	0.4
3	0.5	0.3	0.2	0.6	0.4	0.45
4	0.5	0.4	0.3	0.7	0.5	0.5
5	0.6	0.5	0.5	0.75	0.6	0.6
6	0.9	0.8	1	0.8	0.7	0.85
7	1.2	1.2	1.4	0.95	1	1.1
8	1.4	1.6	1.9	1.2	1.3	1.3
9	1.6	1.75	2	1.3	1.4	1.4
10	1.6	1.8	2.1	1.5	1.6	1.5
11	1.4	1.6	1.9	1.6	1.7	1.4
12	1.3	1.4	1.6	1.6	1.8	1.35
13	1.2	1.25	1.2	1.6	1.8	1.35
14	1.2	1.2	1	1.55	1.7	1.3
15	1.2	1.3	1.2	1.5	1.5	1.4
16	1.3	1.4	1.6	1.5	1.4	1.45
17	1.4	1.5	1.8	1.4	1.3	1.5
18	1.4	1.4	1.6	0.85	1.1	1.4
19	1.1	1.2	1	0.7	0.8	1.2
20	0.9	0.9	0.7	0.6	0.7	1
21	0.7	0.7	0.4	0.6	0.6	0.75
22	0.6	0.5	0.2	0.5	0.5	0.55
23	0.5	0.4	0.1	0.5	0.4	0.45
24	0.5	0.3	0.1	0.5	0.4	0.4

For each land use the AADD for that area is computed and used to calculate a Peak Week Factor (PWF), Peak Day Factor (PDF) and Peak Hour Factor (PHF). Eighteen different time patterns were developed by using five different predominant land uses namely Low Cost Housing (LCH), Residential (RES), Business/Commercial/ Industrial (BCI), large single consumer (LRG) and Inner City CBD (CBD). Additionally, each of the land uses were divided into water demand zones. These zones were then given a PHF, PWF and PDF that were integrated over a 168 hour time pattern (one week).

Table 2-2 indicates the different peak factors as a function of land use and AADD per region. Furthermore, by using reservoir levels over time, these patterns can be calibrated to simulate an accurate water quality model.

A combination of water demand and land occupation profiles were used to determine both a peak hour and peak week factor for each stand. These factors were analysed by means of dynamic simulation over a total modelling period of 168 hours with one hour increments in order to accurately simulate the residence time in each model element as an indication of water quality.

Table 2-2 Peak Factors for Water (GLS, 2013)

Predominant land use	AADD (kl/d)*	PWF	PDF	PHF
Low cost housing (LCH)	<1000	1.5	1.9	3.6
	1000 - 5000	1.4	1.8	3.4
	5000 - 10000	1.35	1.7	3.3
	10000 - 15000	1.3	1.5	3.2
	15000 - 20000	1.25	1.4	3.1
	>20000	1.25	1.4	3
Residential (RES)	<1000	1.8	2.2	4.6
	1000 - 5000	1.65	2	4
	5000 - 10000	1.5	1.8	3.6
	10000 - 15000	1.4	1.6	3.5
	15000 - 20000	1.35	1.5	3.3
	>20000	1.3	1.5	3
Business/Commercial/Industrial (BCI)	<5000	1.45	1.7	3.3
	5000 - 10000	1.3	1.6	3.15
	>10000	1.25	1.5	3
Large single consumers (LRG)	>500	1.45	1.7	2.5
Inner City CBD (CBD)	<5000	1.3	1.6	2

The patterns in **Figure 2-10** indicate the ratios used for a peak week scenario for each land use and supply area. It was designed to mimic an average week ratio simulation of the demand in the specific area. These were allocated to each node with an allocated demand, after which the predominant land use pattern is allocated with that demand.

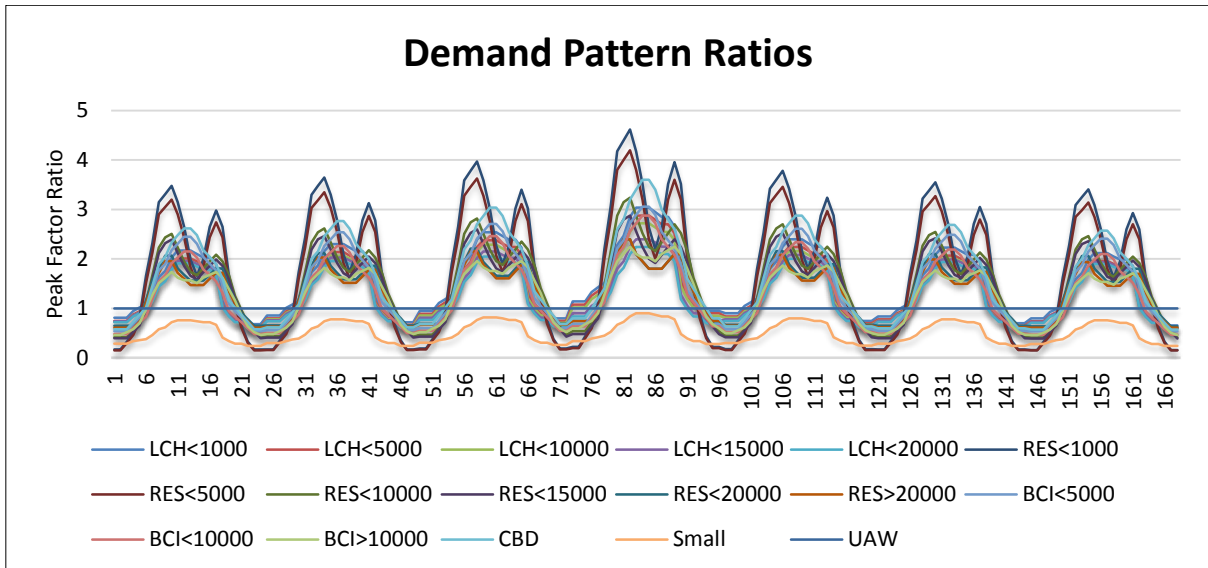


Figure 2-10 Peak Week Ratio for Land Use AADD

2.7 Calibration Procedure

In order to use a hydraulic model with certainty, one must first calibrate and validate your findings and then additional assumptions can be added to use the model for prediction and forecasting scenarios. These calibration and validation processes are explained in this section.

2.7.1 Metering of Water Quality

The process of determining the most suitable sampling scheme for a specific catchment must be handled with care. The optimization of a sampling scheme is very important because of the cost linked to it and the interpretation of the results obtained.

At every step of a water quality assessment one must determine the value added regarding money and time spent. To measure each and every reservoir, pipe connection, pump station and values would be impractical and irrational. To fully optimize a sampling scheme one must first predict the critical areas in the system. This implies including areas that would have the longest retention times and also could be used for additional validation properties.

2.7.1.1 *How to measure disinfectant potential*

The disinfectant potential is the amount of free chlorine or combined chlorine available in the water. This is explained in **Chapter 2.1**.

The method used to determine the amount of chloramine in the water is a combination of total chlorine and free chlorine. There is no direct chemical method for measuring chloramine. Since total chlorine is the sum of free chlorine and chloramine, the chloramine, therefore, is total chlorine less free chlorine as indicated in **chapter 2.1**.

2.7.1.2 *Determine an optimized water sampling scheme.*

After the initial hydraulic model is compiled and results can be obtained from the model, additional validation can be done. Calibration is done based on laboratory testing on the source water and the decay rates related to different pipe characteristics. These characteristics include pipe size, material and age. Once the basic parameters are obtained from laboratory testing and the model is compiled based on actual flow demands, validation of the results can be done. Validation is based on the assumption that for the time period of sampling no drastic alterations were done in the system. This includes flushing of the system or rezoning of water districts. Thus, for accurate validation, a short period of data is required to determine the water quality for the specific model designed. If alternative scenarios need to be determined, then the validation can also be used as calibration for more accurate boundary conditions to predict possible concentrations.

2.7.2 Water Flow Scenario

To determine the most realistic water flow scenario, a low flow simulation must be done. This means that the demand patterns determined as indicated in **Chapter 2.6**, would be used with a reduced AADD to simulate a lower flow and to increase the retention time in the system. Thus, when validation is done on the model, results of the sampling data must be adjusted accordingly. Determining the below average disinfectant sampling results must be calculated. This was done by determining the average measurements obtained from a specific location, then determining the standard deviation. This, with the minimum observed measurement, can indicate and validate the model result and ranges.

This validation and calibration of the model could be adjusted by changing the pipe wall coefficients for certain pipe diameters and pipe material as explained in **Chapter 2.4**.

2.8 Master Planning

Water quality modelling does not form part of the main master planning procedure and is often under-estimated. This then leads to water quality problems that, in return, lead to additional feasibility studies to resolve water quality problems. These aspects could have been avoided if water quality formed part of the initial master planning frame work.

2.8.1 Forecasting Water Quality

Forecasting the potential water quality in a distribution network is based on a large amount of assumptions and is generally regarded as a futile exercise because of the expected accuracy that is involved in the initial assumptions. This process can be implemented with great success if critical assumptions are identified and resolved using accurate and logical procedures.

3 Methodology

The methodology indicated in this section is constructed based on the case study in **Chapter 4**. Nine steps were used to divide the methodology into understandable portions. The first eight steps are indicated below, followed by the last step containing its own **Chapter 3.8**, integrated decision making approach.

3.1 Collection of data for Steady State and Dynamic Model

To ultimately compute a Water Quality model, a Steady State (SS) model is required. In order to establish a SS model it was essential to obtain the following data:

- An EPANET model if available that include portions of the bulk system. This would be used as basis for the steady state model.
- Geographic Information System (GIS) data that includes pipe location, diameter, age and material.
- GIS data that includes reservoir and Water Treatment Plant (WTW) information.
- Digital Terrain Model (DTM) obtained from the Shuttle Radar Topography Mission (SRTM).
- Computer Aided Design (CAD) drawings indicating reservoir layouts plans and reservoir design drawings.
- Pump details regarding locations and pump curves are required.
- Top water levels and volumes of reservoirs are required.
- Operational process based on pump, reservoir and valves control settings.
- Demands and flow meter readings for isolated zones.
- WTW supply and operational controls.

3.2 Compilation of Hydraulic Steady State Model

The EPANET model was converted into a Wadiso (Consulting, 2013) model and thus can be easily used in conjunction with GIS, CAD, DTM and any relevant data. The data obtained in the collection phase of this study was converted into a hydraulic model.

3.3 Collection of Boundary Conditions for Dynamic and Water Quality Models

In order to convert the steady state model into an extended dynamic model, it is essential to obtain the system controls, such as reservoir operating levels, switching on and off of pumps, opening and closing of valves, inline or on reservoir feeders and possible adjusting of control valve settings as accurately as possible. It is important that any changes in control procedures related to different supply conditions, such as seasonal changes, also be recorded. It was envisaged that substantial time would be spent with operational staff in order to obtain a thorough knowledge of the system operation, control procedures and typical system behaviour currently being experienced.

Water demand patterns should be established from operations and using metering data in the field. Since metering data is not necessarily available or would not be over a long enough period to identify typical and extreme conditions, reliance would in some cases be made on typical patterns obtained from previous results and literature applicable to the relevant consumer profile.

Most reservoirs are cylindrical, and dynamic simulation was enabled by using the min/max levels and a cross-sectional area. In the cases of non-cylindrical reservoirs, dimensions were obtained in order to establish the water level-volume relationships.

Existing information on water quality sampling was obtained. Furthermore, the total chlorine and free chlorine concentrations at all the connections were obtained or interpolated from available water quality stations to be used as boundary conditions for the water quality model.

Available literature and information on the bulk- and wall reaction coefficients were obtained based on the fact that water utilities do not necessarily have the capacity to calculate specific source water qualities.

No allowance was made for field surveys of system components or water quality sampling, but the requirement for surveys and sampling was identified if deemed necessary, to be performed by water utilities and municipalities themselves.

In summary the following data was required:

- Water quality sampling data as indicated above;
- Available telemetry data at each reservoir inlet and outlet as well as water levels;
- Available bulk meter data (monthly) for each operational reservoir inlet/outlet bulk meter;
- Operational controls for each reservoir to be work through with operations;
- Current control valve settings or locality of any orifices.

3.4 Analysis of Water Quality Sampling Data

The available water quality sampling points was geo-referenced and the data analysed to confirm the minimum and maximum disinfectant potential and coliform counts at each sampling point. The level of compliance (as per Blue Drop requirements) was calculated and plotted to identify critical points in the system.

3.5 Compilation of Dynamic Model

The steady state (Wadiso) model was enhanced to an extended dynamic model through applying the system control rules, reservoir attributes and water demand patterns. The water demands as per existing steady state models were adjusted, representing a number of supply scenarios, such as average, low and high demand extremes.

3.6 Hydraulic Calibration of the Model

The hydraulic calibration of the dynamic model is required to validate the water quality analysis parameters.

An assessment of the accuracy with which the model simulates the prototype was made based on available water quality measurements and observations to assess the requirement for additional monitoring. This was also the stage at which potential misrepresentation of the model with respect to connectivity or abnormalities, such as closed valves should be identified and cleared with the system operators and managers.

It was assumed that the available flow and water quality measurements would provide sufficient information to calibrate the model and model characteristics. However, additional measurements may be required.

The retention time (or water age) through selected systems was calculated by inserting a trace element (sodium chloride) into the system and then recording the electrical conductivity at a downstream location.

3.7 Evaluation of Existing and Future Scenarios

The calibrated hydraulic model was used to model the decay of chloramines and chlorine through the system for the low flow scenario, water age (including retention times in reservoirs), mixing and source water trace.

Critical reservoirs and distribution systems, in terms of residual chlorine, as identified from the hydraulic Water Quality model were validated against available water quality sampling data. Recommendations for optimum ranges of monochloramine at each bulk meter connection were calculated using the calibrated hydraulic model. Reservoirs with excessive retention times were identified from the Water Quality model and recommendations to reduce retention times were made.

3.8 Integrated decision approach model

This Feasibility Study considered all practical variables related to the conversion of chlorination to chloramination, or vice versa, as the method of disinfection at the relevant Water Treatment Works (WTW).

The Feasibility Study was closely guided by the Integrated Decision Approach Model as delineated in the AWWA publication– Optimizing Chloramine Treatment – 1st Edition (Kirmeyer, et al., 1993). This model formed the basis of the Feasibility Study and was used as a guideline by the working group (see **Figure 3-1**).

The study investigated the feasibility of chloramination at certain pipeline systems. The study looked at both the water utility and the municipality water services systems to assist in the decision of whether, and to what extent, chloramination could provide an improved disinfectant in water with high residence times. The study also makes recommendations to minimize negative impacts of the mixing of water.

In addition, the study included the hydraulic and water quality modelling of the distribution infrastructure to assist in the determination of the probable effect of chloramination on disinfectant residuals within these systems.

A stepped approach was used and the following **key evaluating criteria** formed part of this stepped approach:

3.8.1 Regulatory Compliance

There are three main regulatory drivers for considering chloramines in lieu of free chlorine as a final disinfectant: (1) better disinfectant residual maintenance throughout the distribution system, (2) lower bacterial counts or coliform occurrence, and (3) lower disinfection by-products.

3.8.2 Compatibility with Other Treatment Processes

The final disinfection process must be fully integrated in the existing treatment processes. Also, chloramines should not be used as a substitute for inadequate treatment processes, but are an adjunct to good water treatment.

3.8.3 Compatibility with the Distribution System

The study made a thorough assessment of the receiving distribution systems to ensure that the systems' current and future designs could be operated to be compatible with chloramines. Examples of distribution issues are water age, potential mixing with local sources that are not chloraminated, number and type of reservoirs, ability to flush water from distribution system pipes, plus others.

3.8.4 Nitrification Potential

The largest operational drawback with chloramines is the potential for nitrification. The main factors contributing to nitrification include excess ammonia, warm water temperatures, long detention times, presence of nitrifiers, and low disinfectant residuals. The consequences of a nitrification episode can be serious and include loss of disinfectant residual, proliferation of bacteria, and possible regulatory compliance problems.

3.8.5 Capital and Operating Costs

The capital costs are usually not large by comparison with other treatment components; however, the operating costs can be substantial especially if one considers the hidden elements, such as additional monitoring, enhanced distribution system operation, nitrification control plans, and other operating procedures.

3.8.6 Customer Issues

Some customers have special needs or use the water for special purposes and these customers would need to be notified of the change from free chlorine to chloramines. For example, kidney dialysis centres or those on home dialysis would need to provide additional treatment to remove chloramines or serious injury can occur. Customers using the water in aquariums or fishponds may need to take precautions to remove chloramines and possibly ammonia to preclude harming the fish or aquatic life.

3.8.7 Environmental and Safety Concerns

Discharge of chloraminated water to surface waters may require special care as discharges have been observed to kill fish and aquatic life and harm the stream environment. Also, ammonia can be stored in various forms including gas, liquid and solid and each has its own special safety and hazard mitigation requirements.

3.8.8 Taste, Odour and Other Aesthetic Considerations

When a switch is made to chloramines, the customers would very likely notice some difference in their water and some type of customer interface would be required. Chloramines should have less taste and odour when compared with free chlorine.

3.8.9 Ease of Implementation

Most of the criteria mentioned above relate to the technical aspects of implementing chloramination. The study assessed the technical issues and those that require political involvement or customer relations.

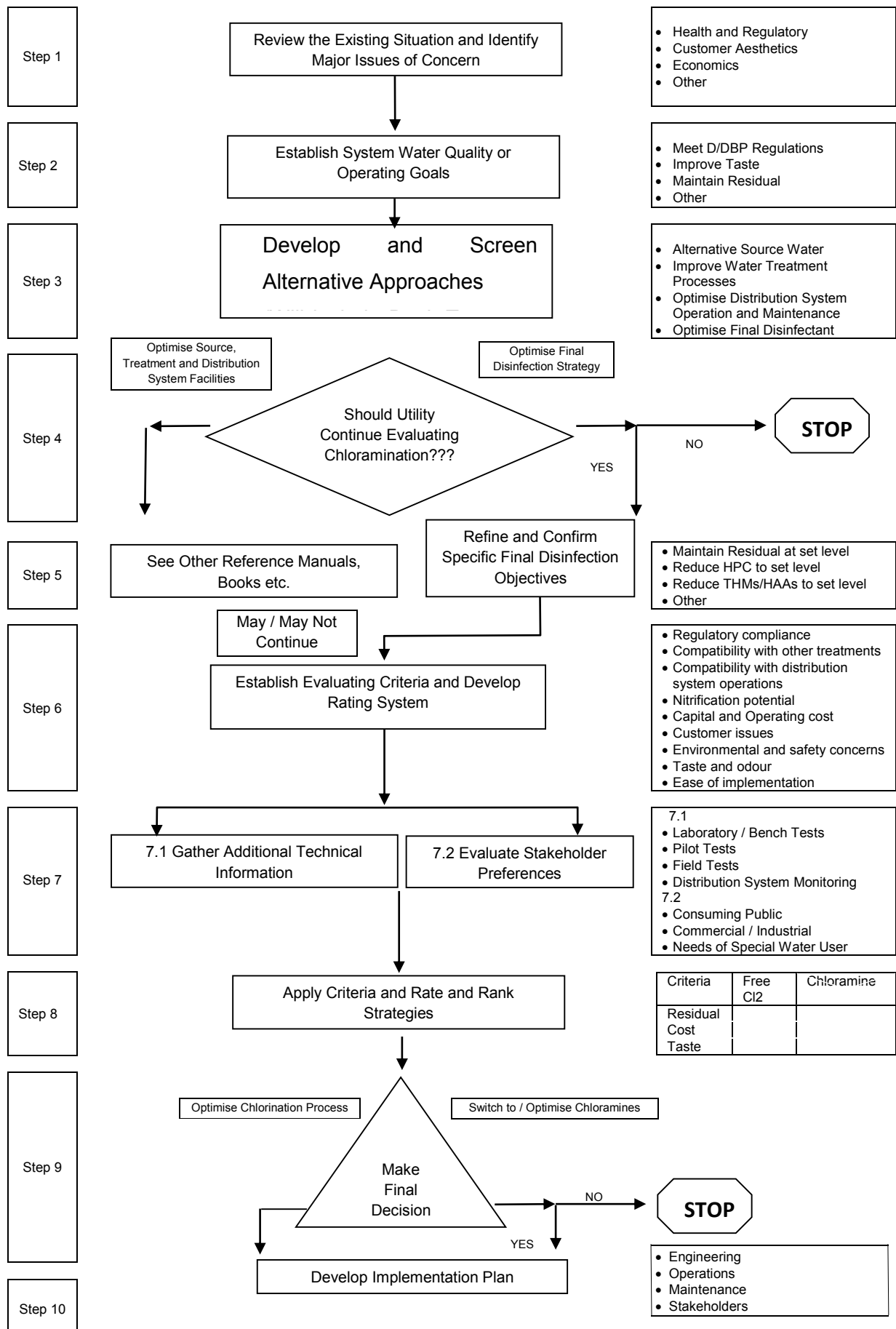


Figure 3-1 Integrated Decision Process (AWWA, n.d.)

4 Case Study: Detailed Feasibility Study and Detailed design for chloramination of the Umgeni Water Central and Coastal Regions

Due to the large number of people affected by this feasibility study, only relevant material was made available for this thesis.

4.1 Introduction

Umgeni Water (UW) indicated during a meeting held in April 2012 that they require a proposal for the compilation of a Water Quality model of their bulk water supply. As indicated in **Chapter 3**, the methodology used to establish such a model was explained. The literature review was used as a basis to:

- Calculate an increase in the water demand;
- Determine the effect of master plan guidelines on water residence time;
- Measure decay rates based on disinfectant reduction;
- Measure the effects of alternative water sources on water quality;
- Calculate the effect of pipe infrastructure deterioration and replacement;
- Obtain a Water Safety plan to combat health and safety risks;
- Determine the primary water quality indicators in a system;
- Predict water quality based on mathematical models;
- Identifying risk areas in the system;
- Provide comprehensive recommendations to resolve existing and future water quality issues.

4.1.1 Background

Umgeni water supplies water to municipalities in the KwaZulu-Natal area, however this thesis only focused on EThekweni (EWS) municipality.

UW owns almost all the treatment works supplying to EWS, thus this study was identified as a joint venture between the two parties to determine the best practise for water disinfectant.

The problem statement is based on the fact that EWS has chlorine WTWs and booster stations, but UW has many chloramine WTWs and this can result in mixing between these disinfectant chemicals.

Water is supplied from Midmar WTW (1) to the western part of EWS. Based on the fact that long retention times were expected in this system, chloramines are used for disinfectant. Hazelmere WTW (2) is also identified as using chloramines as disinfectant. This water can possibly mix with water supplied from Tongaat WTW supply area.

Water & Wastewater Engineering / Project Assignments, in conjunction with GLS Consulting, has been appointed to conduct the Detailed Feasibility Study for the Chloramination Feasibility Investigation. Umgeni Water (UW), together with eThekweni Water Services (EWS), wants to investigate the implementation of chloramines as a disinfectant in the central area systems. This is driven by the large residence times in the central system, which results in chlorine decay. In addition, future plans indicate the possibility of mixing chloraminated water from the inland systems and Hazelmere WTW with chlorinated water from the Durban Heights WTW and Wiggins WTW. This mixing could result in the formation of unwanted by-products and reduction in disinfection potential as indicated in the chapters following up to the case study.

4.2 Purpose of the study

The purpose of the project was to investigate and report on the viability, extent, applicability, and optimum method of implementation of utilizing chloramines as a disinfectant in the central area systems as indicated **Figure 4-1**. Additionally this case study was used to supply information relative to the overall theme of this thesis (Water and WasteWater Engineering & GLS Consulting, 2012).



Figure 4-1 Catchment of Investigation

4.3 Hydraulic Modelling

A dynamic water quality model was constructed of the bulk distribution systems to determine the most suitable combined disinfectant strategy for UW and EWS.

The steps followed in undertaking this study (including data assumptions/integrity and model extent/results) were fully work-shopped with officials from both UW and EWS in seven monthly progress reports.

The extent of the bulk systems to be modelled culminated in to the compilation of one steady state model and six separate dynamic models (see **Figure 4-2**):

- A. Central South (Durban Heights and Wiggins WTWs to Amanzimtoti WTW)
- B. Western (Midmar WTW and DV Harris WTW including Kloof WTW)
- C. Central North (Durban Heights and Hazelmere WTWs)
- D. Central (Durban Heights WTW)
- E. North (Hazelmere WTW to Honolulu Reservoir)
- F. South (Amanzimtoti WTW to Mathulini Reservoir)

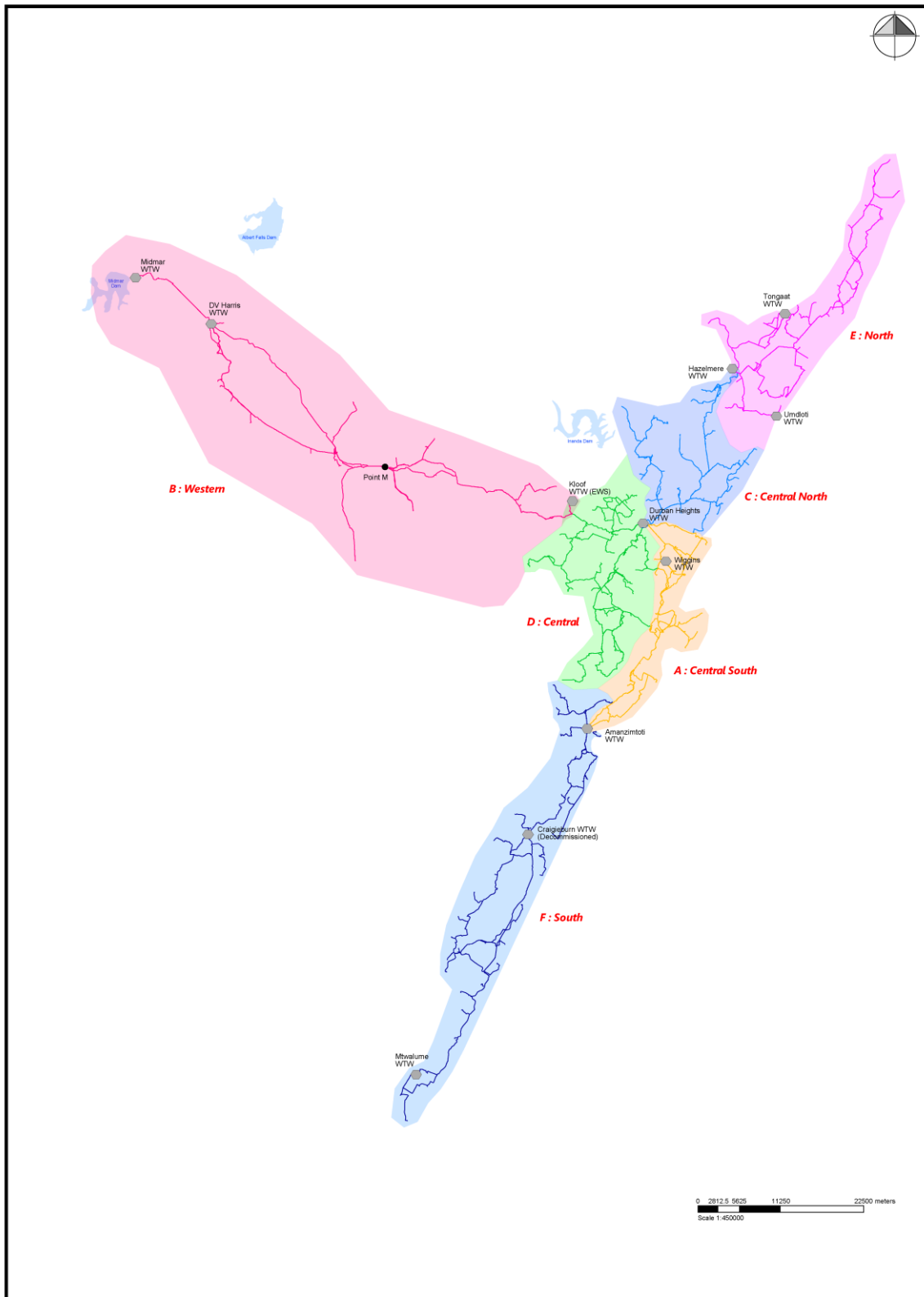


Figure 4-2: Revised Study Area and Dynamic Model Extents

4.3.1 Data Collection

Data was obtained from EWS and UW based on the methodology approach indicated in **Chapter 3.1**.

4.3.1.1 *Data obtained from EWS*

Data obtained from EWS included confidential information and these data portions were not used in this case study. The following GIS information was available for model calculation:

- EWS pipe trunk system (including nominal diameter and pipe material);
- EWS Facilities (locality of facility extent only – no internal pipe layouts available);
- EWS Water Quality sampling points;
- EWS reticulation pipes;

Other data that was used for model calculation:

- Visio Schematic Maps of the EWS bulk water system;
- Spread sheet of pump stations details (including approximate GPS coordinates);
- Secondary booster chlorination points (current operational and de-commissioned);
- EWS bulk meter data for the Northern, Southern and Western portions;
- Operational system control and operating rules;
- Measured disinfectant concentrations at certain EWS reservoir sites.

The following assumptions and data enhancements were made in order to establish the base data for the compilation of the hydraulic model:

- There were many “loose” or unconnected trunk mains which were either deleted where considered irrelevant or connected through interpretation of the connectivity as illustrated in the Visio Schematic Maps. Many assumptions (including pipe connections, closed valves, reservoir connection points, reservoir and tower locations, and pump station connection points) were required in this regard due to the incomplete connectivity information.
- The total reservoir volumes and top water levels (TWLs) as indicated on the Visio Schematic Maps were cross-referenced to the name in the EWS Facility GIS file in order to determine the geographic position of each reservoir. No GIS information of individual reservoir sites was available, but only a shape file indicating the location of the reservoir site and not the individual reservoirs and their connectivity.

- The GPS coordinates as contained in the pump station spread sheet provided by EWS were used to generate a shape file of all pump stations. However, the coordinates were only approximate and in most instances did not plot directly on the EWS trunk system. Assumptions were required as to how each pump station connects to the EWS trunk system as well as pump directions, which were estimated through interpretation of the Visio Schematic Maps and were later confirmed with EWS operational personnel.
- No pump curve information was provided. GLS used the details in the Pump Station spread sheet to identify the relevant pump manufacturer and contacted them or used the applicable catalogues to obtain the pump curve. Not all pump curves could be obtained due to insufficient pump data.
- The geographic locations of the Water Treatment Plants were not available and were estimated by using aerial photographs (provided by UW).
- Bulk meter data was assessed and divided into usable and non-usable meters through inspection of the meter records to determine any obvious irregularities or gaps.
- Treatment facility disinfectants dosage volumes and methods were obtained and incorporated into the model.
- Missing pump curves were estimated assuming pumps can deliver the current demand.
- Bottom Water Levels (BWL) were assumed to be the same as the ground level where no BWL information was available.
- Top Water Levels (TWL) of reservoirs were assumed to be 5m above the BWL where no TWL data was available.
- Flow control valves were modelled on trunk mains supplying more than one reservoir located at different TWLs to avoid low level reservoirs drawing down the hydraulic gradient.
- Demands were populated from meter data readings obtained from EWS as well as field data obtained from JOAT. Where demand data was insufficient, the demands were calculated from the reservoir volume assuming a 48h retention time for that reservoir.
- Reservoirs were assumed to be 80% full before pumps switch on. However, certain important reservoir controls were obtained from EWS.
- The calculated reservoir input volumes for each specific reservoir zone were calculated as included in the model database.

- An average Hazen Williams roughness coefficient of 120 was assumed (General assumption based on operations experience). This coefficient was adjusted for certain pump pipelines to meet water demands and or head loss requirements.
- Reservoir supply zones and reticulation zones were updated as required.

4.3.1.2 Data obtained from UW

The initial steady state model compilation has been completed using available base data. Compilation of a steady state hydraulic model entailed the following steps:

The available EPANET model was converted to WADISO format (see **Figure 4-3**). The integrity and assumptions of the original EPANET model were discussed with UW personnel. The UW model was extended, using the UW and EWS bulk system shape file, to compile a total steady state model for the study area (existing and future scenario). The EWS trunk system GIS data had many gaps and could not be used as-is to build the required model topology due to incomplete information / pipe connectivity. Therefore, the model was compiled using the GIS trunk data in relation with the Visio schematic maps to ensure similar model schematisation is achieved. Many assumptions were required for pipe connections (especially for those on the EWS facilities), closed valves, reservoir connection points, reservoir and tower locations, and pump station connection points.

The following points are important:

- Potential mixing areas could then be identified using the extended model;
- The extent of the systems to be modelled (study area) was then updated;
- EWS reservoir data (volume and TWL) was captured into the model using the Visio schematic maps and EWS Reservoir layout sketches;
- Pump stations were geo-referenced using the GPS coordinates in the spreadsheet provided and captured into the model in their approximate positions. Aerial photos were inspected to confirm possible inaccuracies in the plotted pump station locations;
- Pump curves were obtained for 105 EWS pumps either from pump catalogues or by contacting relevant pump suppliers and entered into the model. The pump curves for many pumps could not be obtained due to insufficient information;
- The position of closed valves and other control valves were identified from the Visio schematic maps and captured into the hydraulic model, but were later confirmed with EWS Operations;
- Water quality sampling data was obtained and geo-referenced with the level of compliance plotted against the disinfectant used;
- A DTM was compiled and the model node elevations were updated;

- The above progress had equipped the modellers with sufficient understanding of the system operation to identify areas needing further clarification as discussed below;
- Feedback received during meetings with EWS Operations was incorporated into the model;
- Follow-up discussions were held to obtain missing data;
- Verification of the model topology and operational system controls were done through various workshops with EWS Operations;
- Expected water supply zones were created to smooth the population process once demands were released from EWS;
- The initial steady state model was completed and handed to UW for validation. This model included assumptions with regards to pump curves, PRV settings, reservoir control valve locations and demands for EWS and Umgeni reservoirs;
- Schematic and spatial layouts of the complete steady state model were generated and are not included in this thesis based on a confidentiality agreement made with EWS.

4.3.2 Model Compilation

The UW EPANET model was converted to WADISO and extended using EWS/UW trunk system shape files and Visio Maps:

- Reservoir volumes and TWLs were captured from Visio Maps;
- Reservoir connectivity was captured from EWS A3 reservoir layout plans
- Bulk meter data assessed and AADDs entered;
- The position of closed valves and other control valves were identified from the Visio schematic maps and captured into the hydraulic model, but also confirmed with EWS Operations.

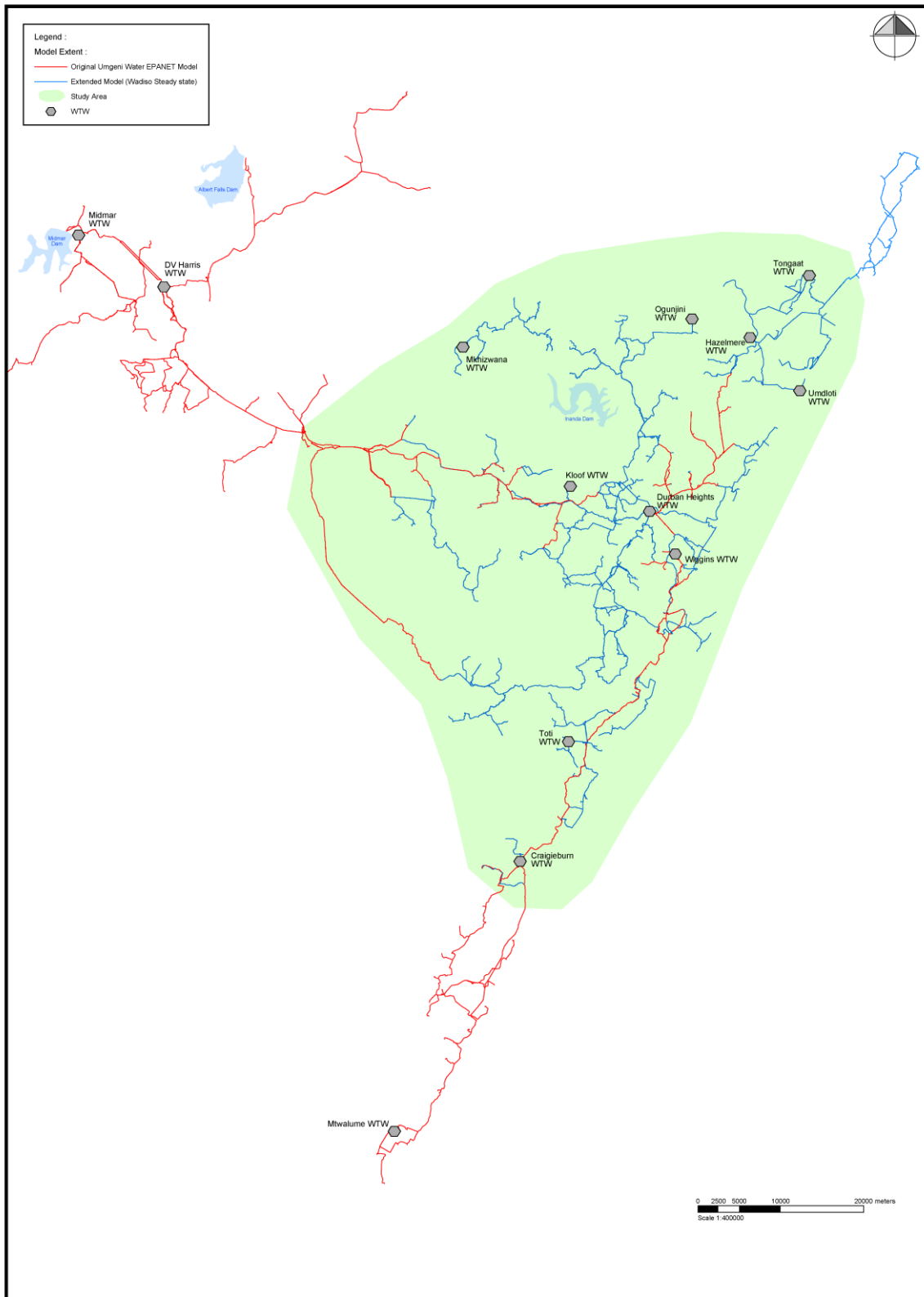


Figure 4-3 EPANET Model Extent

4.3.3 Assumptions

As indicated in **Chapter 2.7** and **Chapter 2.8** the need to indicate critical assumptions is required to reduce the risk of instability in the model. This was done in various ways throughout the assumption process.

4.3.3.1 *Steady State and Dynamic Simulation Assumptions*

- All pump curves are critical for accurate modelling of a hydraulic model. The missing pump curves were determined through investigation of pump data. Additional missing pump curves were estimated, assuming pumps can deliver the current demand and flow.
- Bottom water levels of reservoir were not supplied and for additional upgrading of the model is required but as a critical assumption a 5m height of all reservoirs not specified were used.
- Where reservoir demand data was insufficient, the demands were calculated assuming a 48h retention time for that reservoir. This process was only used in reservoirs that don't have an influence on residence times downstream.
- Reservoirs were assumed to be 80% full before pumps switch on or valves open to fill up reservoirs. However, certain important reservoir controls were obtained from UW/EWS and adjusted accordingly. This reservoir water level fluctuation is indicated in **Figure 4-5** below based on a low week simulation of 168h and top water level of 5m. Additionally the corresponding retention times for that reservoir's operations settings are indicated in **Figure 4-6**.
- For the dynamic simulation hydraulic setup no land-use categories were identified and no hourly data was obtained. The indicated time patterns were used according to pipe sizing, pump sizing and reservoir capacity. For the purpose of this study it was determined that the time pattern used at each reservoir demand is not critical to the overall project deliverables. This is further validated by the facts that the more conservative flow patterns were used to illustrate low flow as indicated in **Figure 4-4**.

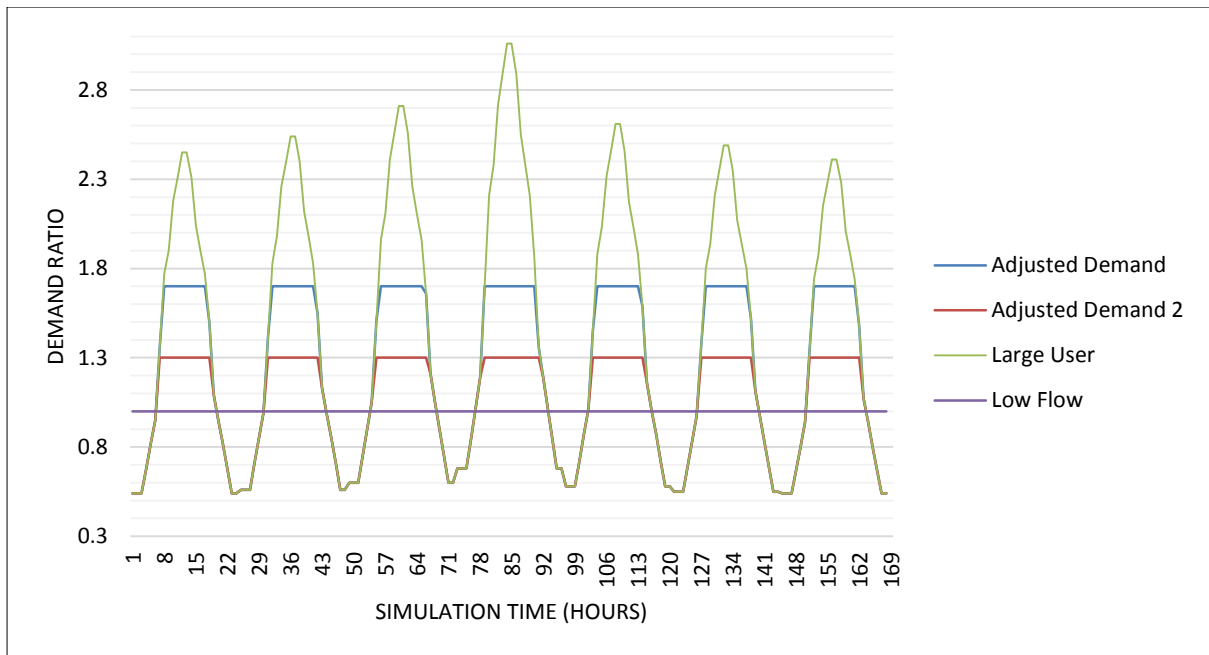


Figure 4-4 Time Patterns

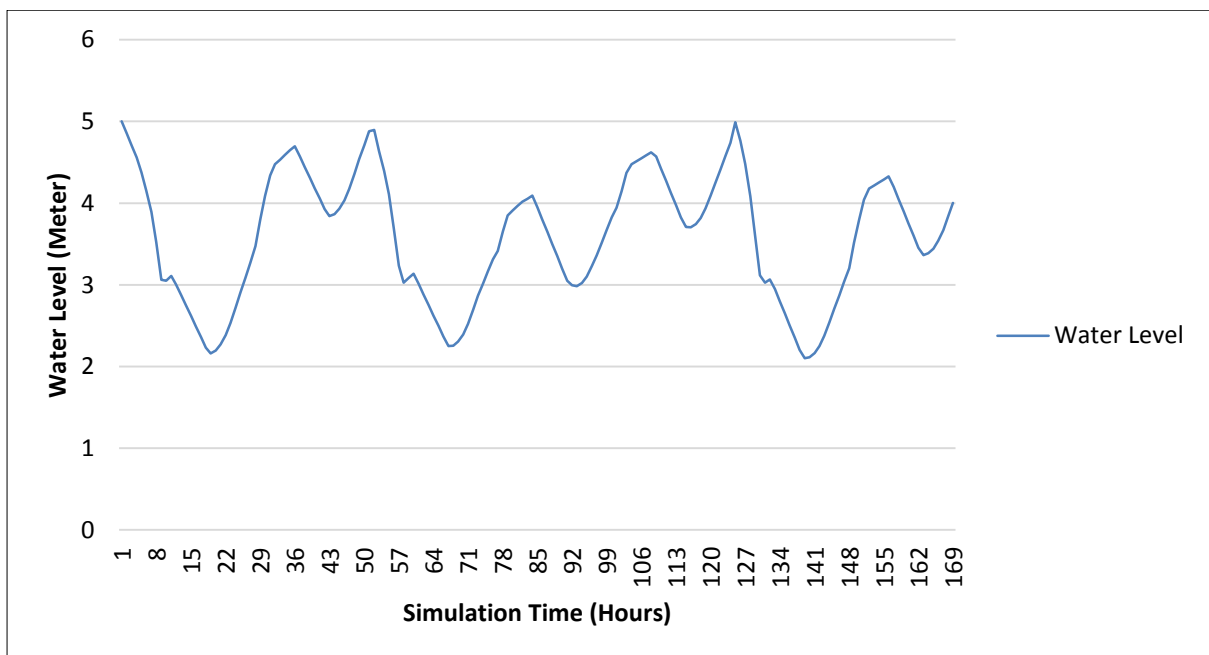


Figure 4-5 Reservoir Water Level Durban North HL

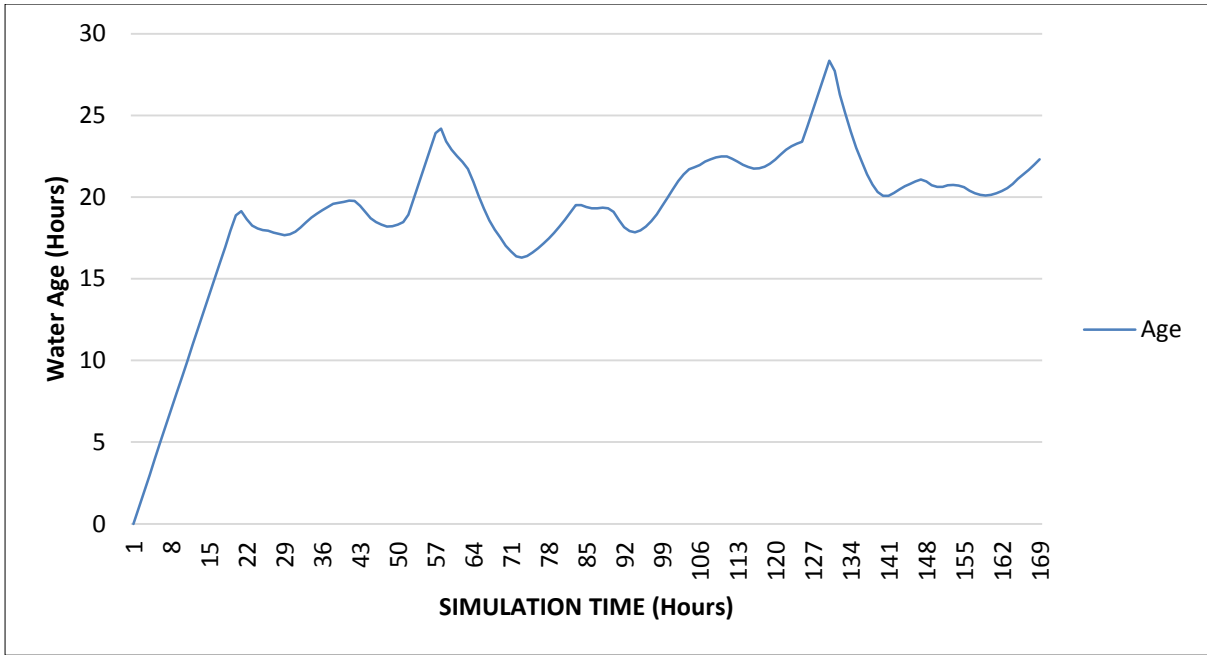


Figure 4-6 Reservoir Age Durban North HL

4.3.3.2 Water Quality Assumptions:

Aspects that influence the water quality of the model are indicated in the following section. It is assumed that a concentration of 1mg/l chlorine is available when leaving each booster chlorination station. This assumption is relative to the operational control settings and was discussed with operators of Water Treatment works.

Concentration of water leaving water treatment plants were assumed to be as per control levels supplied by operations. In addition to this assumption mixing of reservoir water on one reservoir site regarding old and new water is assumed to be mixing sufficiently. This assumption was made based on the software limitations regarding 3-dimensional modelling. (See Chapter 2 for additional information regarding 3-dimensional modelling and the implications of this assumption).

The initial disinfectant concentrations for each reservoir were given based on a first iteration process where the model was run and a mean concentration was given to each reservoir. This process only decreases the warm-up period of the model and has little impact on the resulting concentration of each reservoir.

Decay rates for chloramines and chlorines were determined by using previous studies and the rates were not specifically calibrated to the source water used. For further information regarding decay coefficients please refer to **Chapter 2.4.1**.

4.3.4 Steady State and Dynamic Models

The dynamic model set-up was completed using available data and assumptions as discussed in Chapter 3, and the steady state model that was sub-divided into six hydraulic models for purposes of dynamic modelling. These models indicated potential disinfectant mixing areas (See **Figure 4-7** and **Figure 4-8**). The criteria for splitting the models were:

- Potential mixing areas (chloraminated and chlorinated water);
- Operational boundaries;
- Integrity of information;
- Logical water boundary;

The six dynamic models produced representing the current scenario:

4.3.4.1 Central South Model (Durban Heights and Wiggins WTWs to Amanzimtoti WTW):

The model starts from Durban Heights WTW to Wiggins WTW straight through to the South Coast Aqueduct (SCA) pump station. This system comprises only chlorinated water from Wiggins, Durban Heights and Amanzimtoti WTWs.

4.3.4.2 Western Model (Midmar WTW and DV Harris WTW including Kloof WTW)

This system stretches from Midmar WTW past DV Harris to Umlaas Road Reservoir (chloramines) and along the West Coast Aqueduct to Abelia Reservoir where mixing occurs with chlorinated water from Kloof WTW. The integrity of this model was regarded as good mainly because reliable data was obtained from UW and EWS Western Operations. Detailed studies were undertaken to model and determine chloramine decay and retention times upstream of Kloof WTW with Umlaas Road Reservoir chosen for validation of the chloramine concentration / decay rates to be used for water quality modelling of the rest of the system.

4.3.4.3 Central Northern Model (Durban Heights and Hazelmere WTWs)

The system includes the northern aqueduct from Durban Heights WTW to Hazelmere WTW. This model includes mixing of chlorinated and chloraminated water at Grange / Mount View Reservoirs.

4.3.4.4 Central Model (Durban Heights WTW)

The model starts at Abelia Reservoir through to Durban Heights WTW down to Floweni Reservoir. Potential mixing can occur at Abelia / Jerome Drive Reservoirs. It was difficult to achieve model stability for this area, which could be due to bottlenecks in the system as demonstrated through the dynamic model. The model indicated that certain reservoirs could run empty for the current demand scenario, which was confirmed by operational personnel to be the case. This situation is managed through the opening of valves that should normally be closed resulting in mixing of chlorinated water from Kloof WTW (EWS) as well as Floweni WTW (EWS) and chloraminated water from Umlaas Road supply (UW). The model was therefore simplified by excluding unstable reservoirs or through making further assumptions.

4.3.4.5 Northern Model (Hazelmere WTW to Honolulu Reservoir)

The system includes the Hazelmere WTW straight through to Tongaat WTW and Mvoti WTW. Mixing occurs at Umdhloti as well as Belvedere Reservoirs. Water quality modelling for large parts of the northern system was excluded as insufficient information was available.

4.3.4.6 Southern Model (Amanzimtoti WTW to Mathulini WTW)

The southern system comprises chlorinated water from Amanzimtoti and Mathulini WTW, which feeds the SCA pipeline supplying the southern part of EWS over long distances with long retention times. This model was used to evaluate the need for booster chlorination requirements.

The dynamic model produced representing the future scenario:

4.3.4.7 Future Western Aqueduct Model (Simplified):

The planned extension of the western aqueduct will introduce additional chloraminated water in the EWS Central Region. The impact of this future scenario on water quality was evaluated by compiling a simplified model of the western aqueduct system including only draw-off points and balancing reservoirs on the aqueduct. The demands modelled at the take-offs were obtained from Knight Piesold who was involved in the compilation of the original master plan for the western aqueduct. This model was used to model water age and disinfectant concentrations along the future aqueduct to evaluate the need for booster chlorination/chloramination. The full details of the master plan (as well as the model from Knight Piesold) indicating the future infrastructure requirements to supply chloraminated water from the proposed Western Aqueduct extension into the existing EWS chlorinated systems were not made available. This information is required to model water age and disinfectant concentration from the western aqueduct into the EWS bulk systems for this future scenario, but a detailed analysis of the master plan was not part of the scope of works.

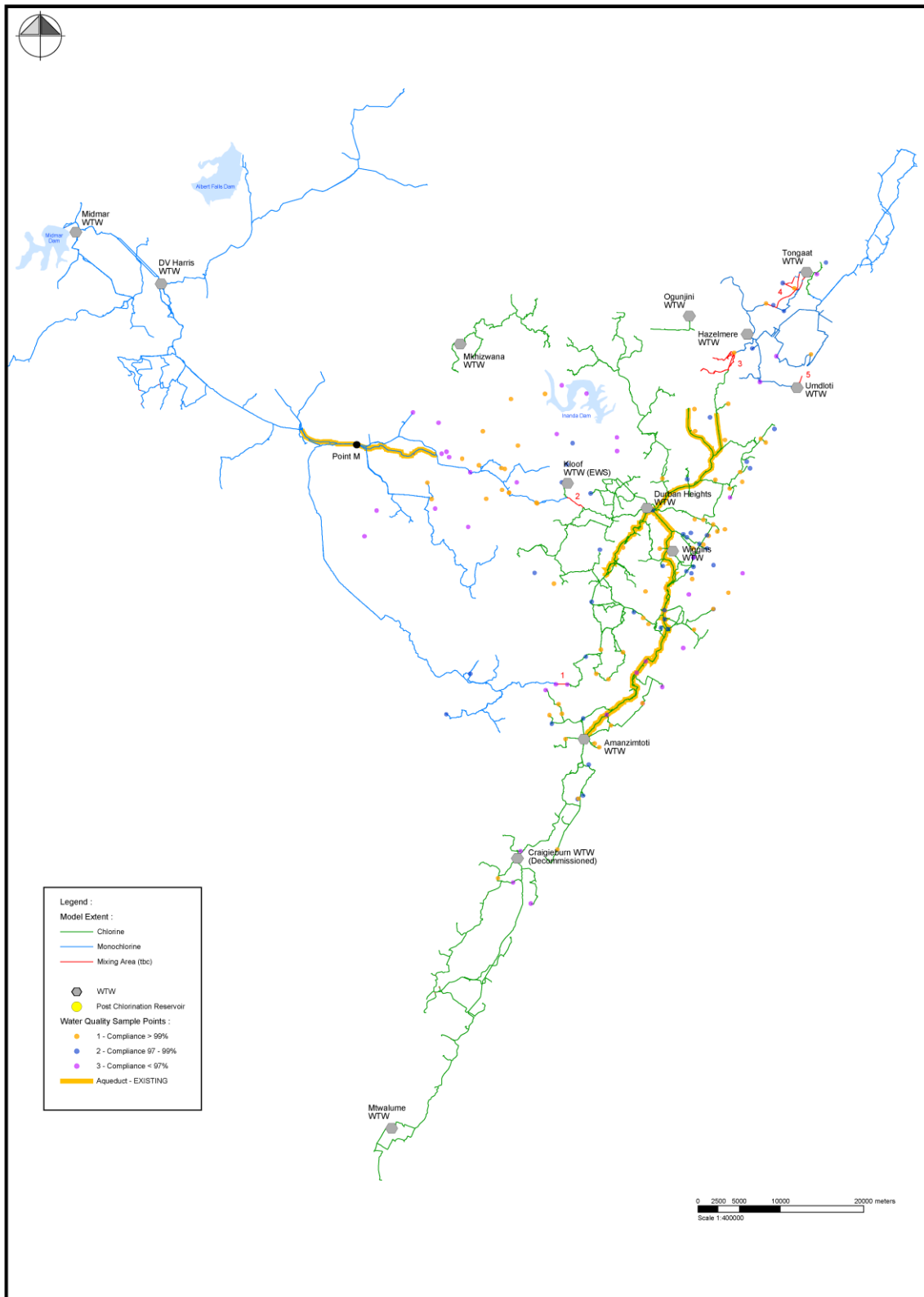


Figure 4-7 Potential Mixing Areas Existing

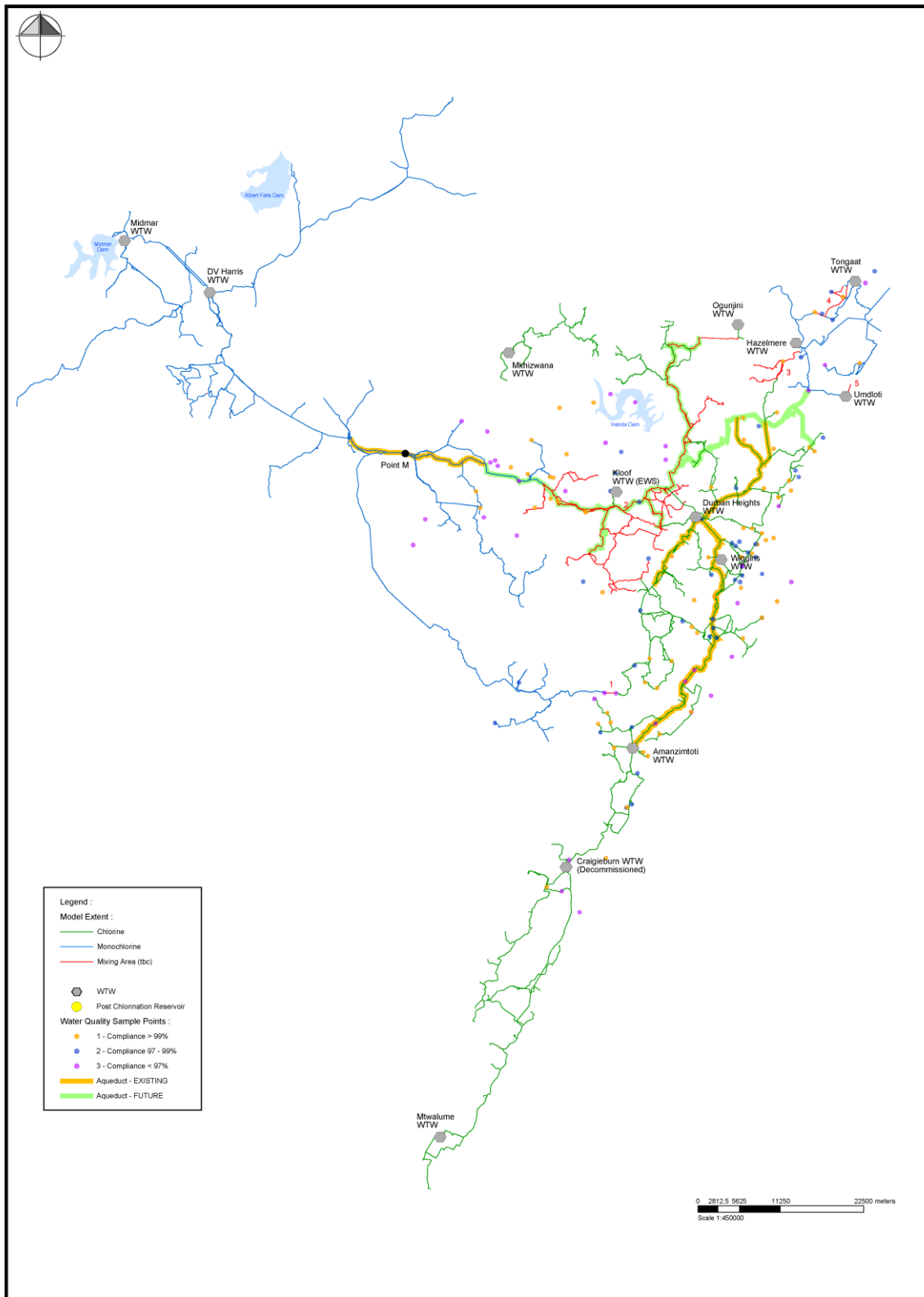


Figure 4-8 Potential Mixing Areas Future

4.3.5 Water Quality Results and Validation

Calibration and validation of the Water Quality model was done by using sampling readings from Umlaas Road Reservoir and determining the residence time of the water from DV Harris to Umlaas road.

4.3.5.1 Calibration

The first results indicate the time that the allocated tracer reached the specific reservoir (Umlaas Road Res) from DV Harris WTW. As indicated in **Figure 4-9** Sampling Umlaas Residual Chlorine, the spike (Red Line) in conductivity was observed after 9 hours had elapsed. The difference in conductivity (Δ conductivity) is indicated in the secondary y axes.

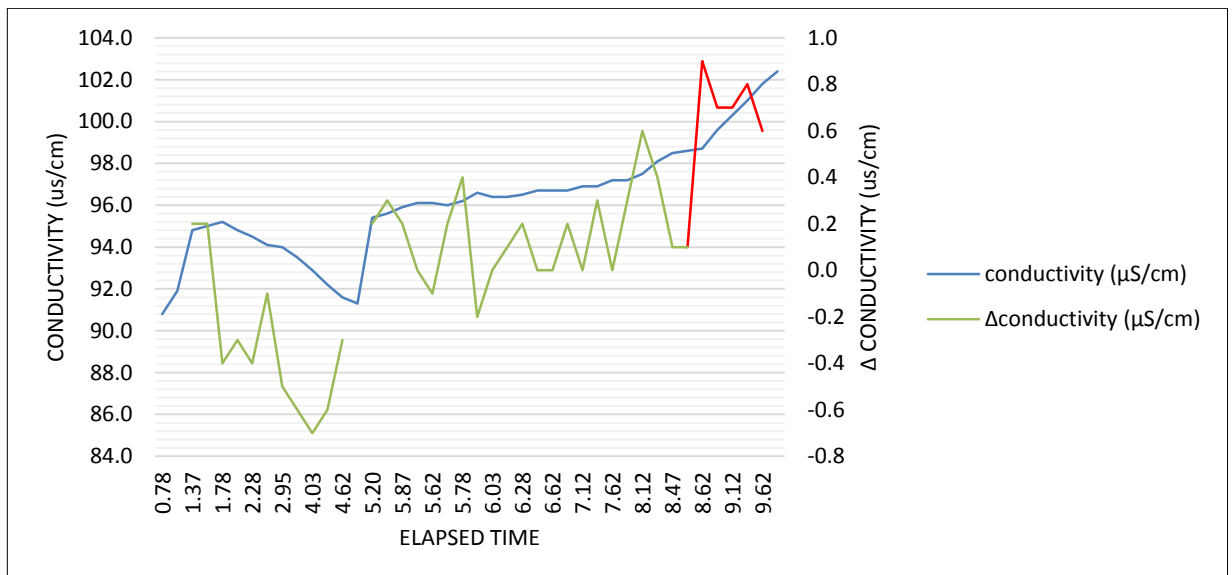


Figure 4-9 Sampling Umlaas Residual Chlorine

The results obtained from the conductivity test **Figure 4-9** Sampling Umlaas Residual Chlorine were used to validate the retention time in the Water Quality model. The results obtained from the Water Quality model corresponded with the conductivity test and were relatively accurate as indicated in the **Figure 4-11** Water Age at Umlaas Res from DV Harris WT.

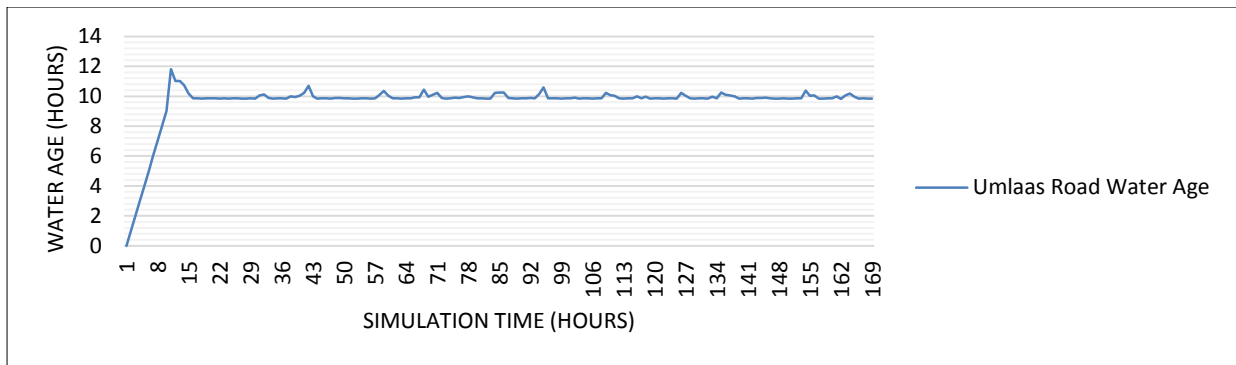


Figure 4-10 Retention Time DV Harris to Umlaas Sampling

Historical information obtained from UW based on sampling data at Umlaas Road Reservoir indicated that the monochlorine concentration readings of the past year are 2mg/l as indicated in the following **Figure 4-11** Water Age at Umlaas Res from DV Harris WT.

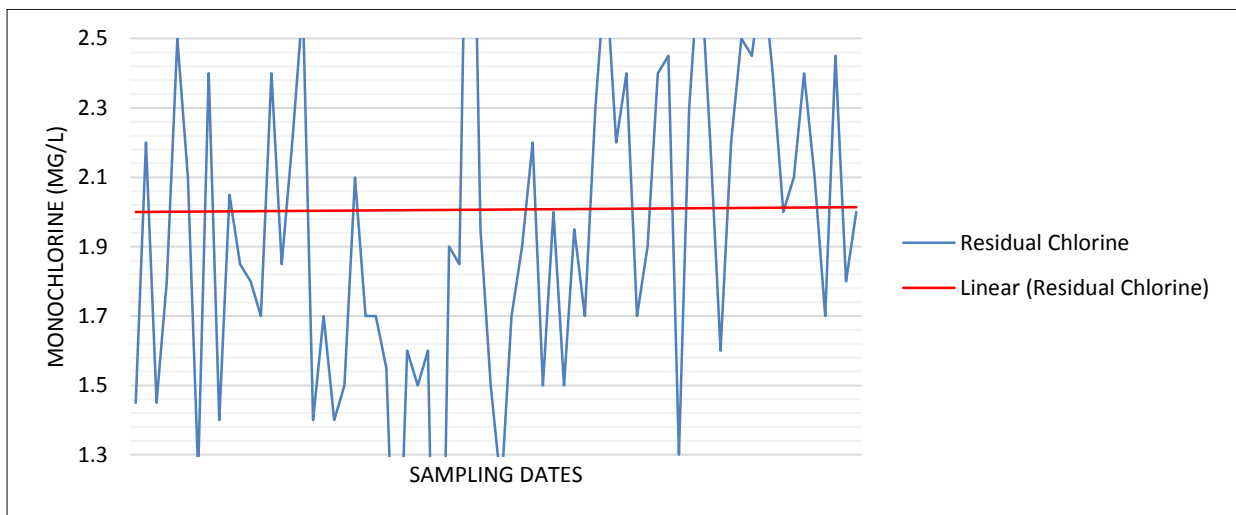


Figure 4-11 Water Age at Umlaas Res from DV Harris WTW

After the Water Quality model decay rates were used, based on literature studies, the results obtained at Umlaas Road Reservoir were calculated as shown in **Figure 4-12** Monochlorine Concentration at Umlaas Rd Reservoir. It can be seen from **Figure 4-12** that the average disinfectant concentration is just below 1.9mg/l (red line) which is sufficient based on the low flow scenario that is simulated.

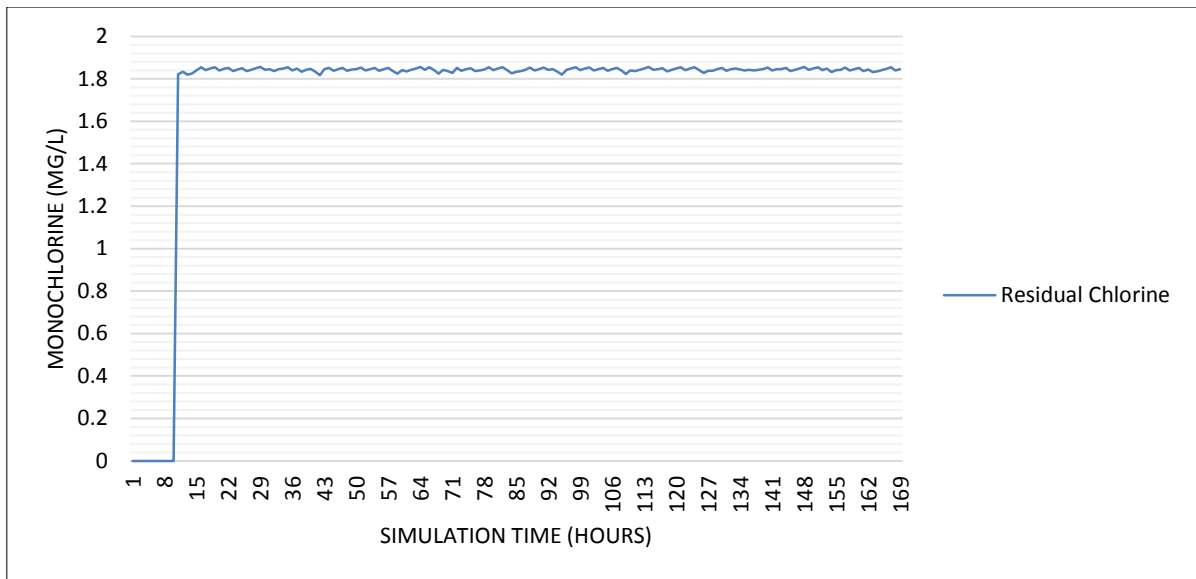


Figure 4-12 Monochlorine Concentration at Umlaas Rd Reservoir

Reservoir sampling data was used for further validation of the model. The results indicated in **Table 4-1A** and **B** obtained from EWS laboratory. Reservoir names were removed based on confidentiality agreement signed. For the purpose of this thesis no reservoir names were required - only the results obtained from the Water Quality model and the test result were important. The minimum, average and maximum concentration of disinfectant were used and correlated with the results obtained from the Water Quality model. As the results indicate, the model is more conservative than the results of the sampling data. This was expected, based on the low flow used as modelling scenario criteria.

Table 4-1A Reservoir Sampling

Model	Reservoir Name*	Minimum**	Average**	Maximum**	Model***
Central South: A	A Res	0.18	0.74	1.2	0.26-0.3
	B Res	0.05	0.194	0.46	0.16-0.22
	C Res	0.04	0.26	0.63	0.18-0.22
	D Res	0.04	0.24	0.72	0.2-0.46
	E Res	0.04	0.9	1.3	0.5-0.85
	F Res	0.17	0.49	1.1	0.45-0.6
	G Res	0.1	0.62	1.2	0.35-0.55
	H Res	0	0.24	0.55	0.2-0.26
	I Res	0	0.422	1.3	0.22-0.34
	J Res	0.09	0.27	0.71	0.09-0.16
	K Res	0	0.16	1.2	0.08-0.12
	L Res	0	0.13	0.29	0.06-0.14
Western: B	M Res 4	0.14	1.04	2	0.24-0.28
	N Res 7	0.15	0.77	1.7	0.18-0.22
Central Northern: C	O Res 3	0.44	0.71	0.98	0.38-0.5
	P Res 5	0.1	0.55	0.9	0.1-0.3
Central: D	Q 1 Res	0.04	0.62	0.99	0.15-0.23
	Q 2 Res	0	0.47	0.95	0.25-0.36
	Q 3 Res	0.19	0.39	0.69	0.07-0.16
	Q 4 Res	0.08	0.395	0.86	0.1-0.14
	R 1 Res	0.04	0.36	1	0.17-0.23
	R 2 Res	0.04	0.67	1.2	0.2-0.26
Northern: E	S Res	0.34	1.2	2.6	1.2-1.4
	T Res	0.19	1.23	1.7	0.7-0.85
	V Res	0.22	0.86	1.8	0.8-1.2
	W Res	0.23	0.63	1.8	0.15-0.2
	X Res	0.17	0.68	1.2	0.2-0.26
	Y Res	0.13	1.09	1.7	0.35-0.55
	Z Res	0.03	0.58	1.2	0.3-0.5
	AA Res	0.21	0.87	1.8	0.28-0.5

*Reservoir Concentrations are inlet or outlet

** Disinfectant concentration (mg/l) over two years

*** Model disinfectant concentration (mg/l)

Table 4-2B Reservoir Sampling

Model	Reservoir Name*	Minimum**	Average**	Maximum**	Model***
Southern: F	BB 2 Res	0.03	0.35	0.69	0.22-0.46
	BB 3 Res	0.03	0.44	0.75	0.3-0.75
	BB 4 Res	0.03	0.26	0.61	0.22-0.58
	BB 5 Res	0.02	0.23	1.5	0.12-0.28
	CC Rd Res	0.09	0.37	1.4	0.2-0.5
	DD Res	0.15	0.85	1.5	0.6-0.8
	EE Res	0.31	0.75	1.2	0.75-0.85
	FF Res	0.2	0.58	1.7	0.36
	GG Res	0.56	0.93	1.4	0.26-0.4

*Reservoir Concentrations are inlet or outlet

** Disinfectant concentration (mg/l) over two years

*** Model disinfectant concentration (mg/l)

The results indicate that the model had an accurate representation of the actual readings and could be used for additional modelling.

4.4 Results of Case Study

This Chapter presents the analysis results for water age and disinfectant concentration for each of the six dynamic models. A spatial layout indicating water age for the total study area (all six models combined) are provided in the following section.

The results of the water age and disinfectant concentration analysis for each of the models described are presented in schematic format in **Figure 4-13 to 4-24**. These figures indicate:

- Ranges for water age and disinfectant concentrations through a colour coded legend at each reservoir site as per dynamic simulation results. The results for some reservoirs are not shown as marked in grey hatch, which is due to mixing taking place (which cannot be modelled), poor model stability, insufficient information or because it was considered irrelevant / less important;
- Potential mixing areas by means of pink blocks;
- Existing WTW and booster chlorination points;
- Flow direction, closed valves, pump lines, gravity lines, emergency lines and the connectivity of the bulk system;
- TWL (m.asl) and Reservoir volumes (Mega litres as Megs);
- Reservoirs where demand data is considered unreliable, and thus also the water age, are marked with an asterisk (*). The demands for these reservoir sites need further validation.
- The water age and disinfectant concentration analysis results, for each of the six dynamic models produced, are discussed below with a summary of the measured versus modelled disinfectant concentrations provided in Table 4-1A. As the models represent the low flow scenario with low assumed peak factors, the modelled disinfection concentration needs to be compared with the minimum to average measured concentrations ranges.

4.4.1 Central South Model (Durban Heights and Wiggins WTWs to Amanzimtoti WTW):

Residence time in the South Coast Aqueduct (SCA) could be manipulated by changing the operational levels of Woodlands 3&4 Reservoirs. With this operational control the water age at the end of the SCA reaching Amanzimtoti is between 90-110 hours. No mixing of chloraminated and chlorinated water occurs. Chlorine concentrations were at acceptable levels. This was mainly due to booster chlorination at Woodlands Reservoir 3&4. Low chlorine concentrations were mainly due to operational procedures at reservoirs and not because of the long retention times in the SCA.

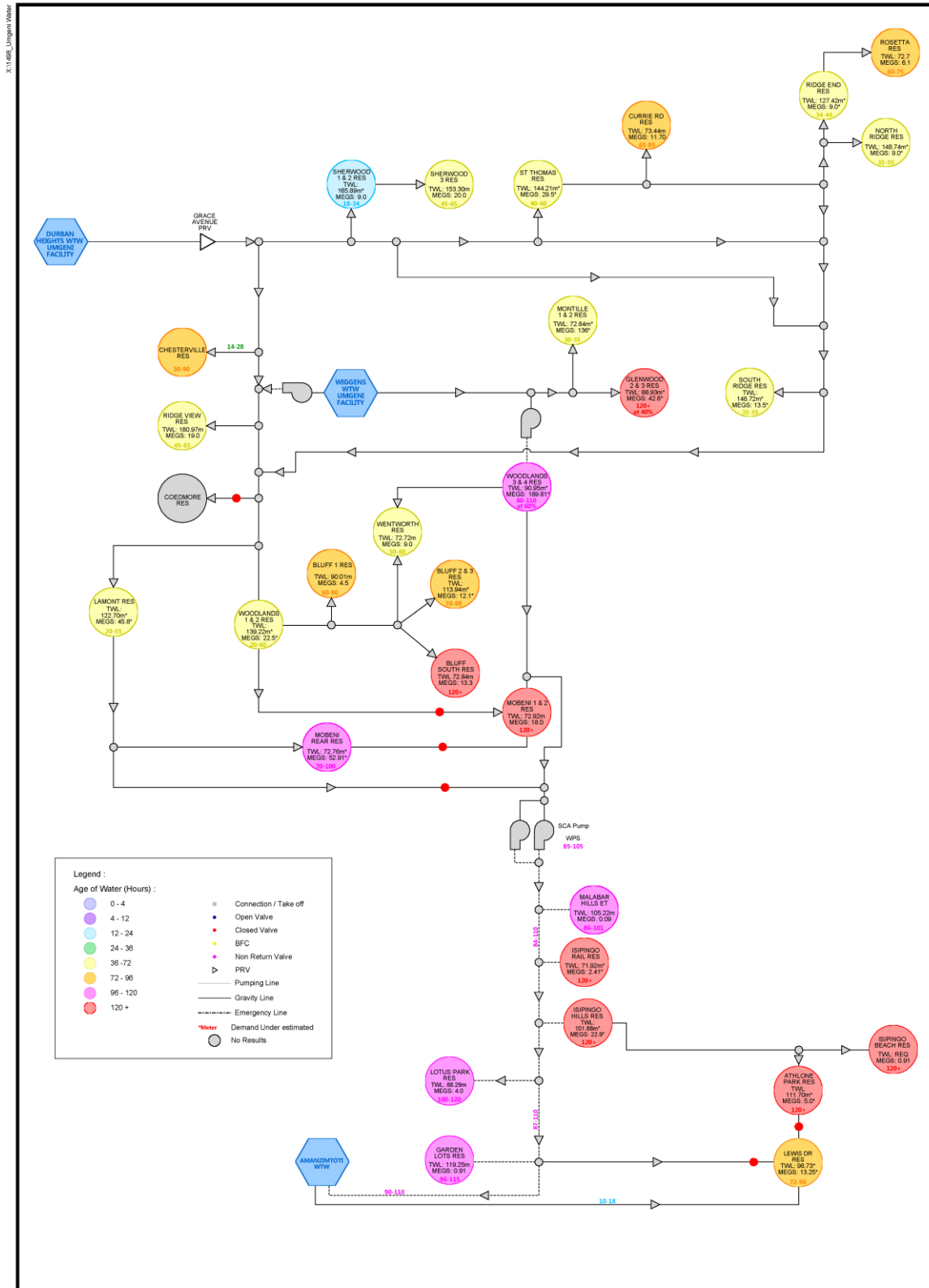


Figure 4-13 Water Age Central South

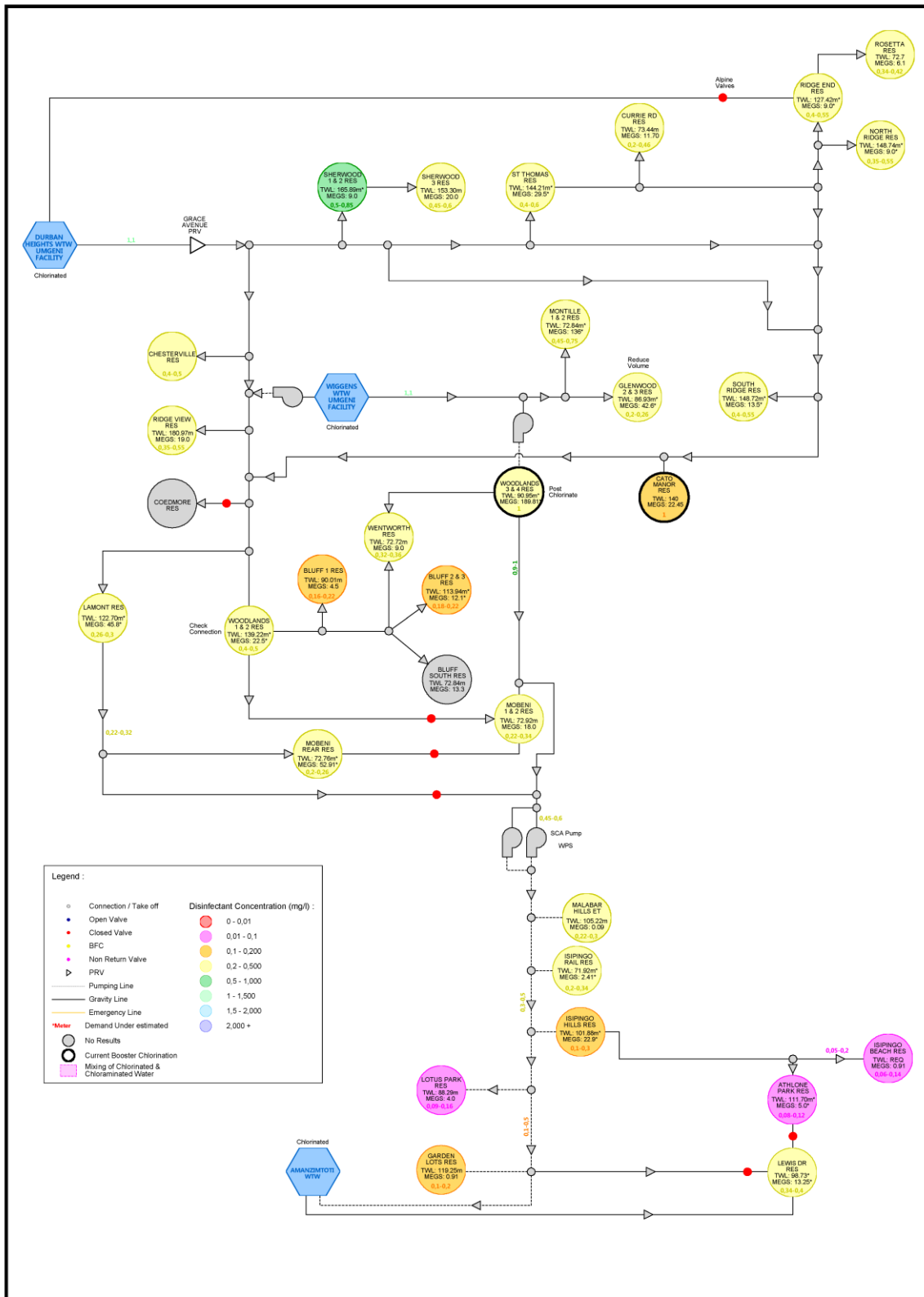


Figure 4-14 Disinfectant Concentration Central South

4.4.2 Western Model (Midmar WTW and DV Harris WTW including Kloof WTW):

Modelled water age at Umlaas Road is between 28-43 h due to fresh water supplied from DV Harris WTW that mixes with older water from Midmar WTW. After Umlaas Road the water is fed to Point M and Umbumbullu Reservoirs including UW Eston reservoir. The age of the water in this system increases considerably from Umlaas Road to Umbumbullu Res. This could be due to the oversized bulk pipe lines (600mm and 450mm Ø) that feed only 8 Ml/day resulting in pipe velocities less than 0.33m/s. The model indicated that residence time from Midmar WTWs to Abelia Reservoir was less than 70 hours. The water age in the current Western Aqueduct is not the critical point in the system, but rather the Umbumbullu reservoir system. The mixing of chloraminated and chlorinated water from Adams Mission 6 Reservoir to Floweni 1 Reservoir was indicated by operations.

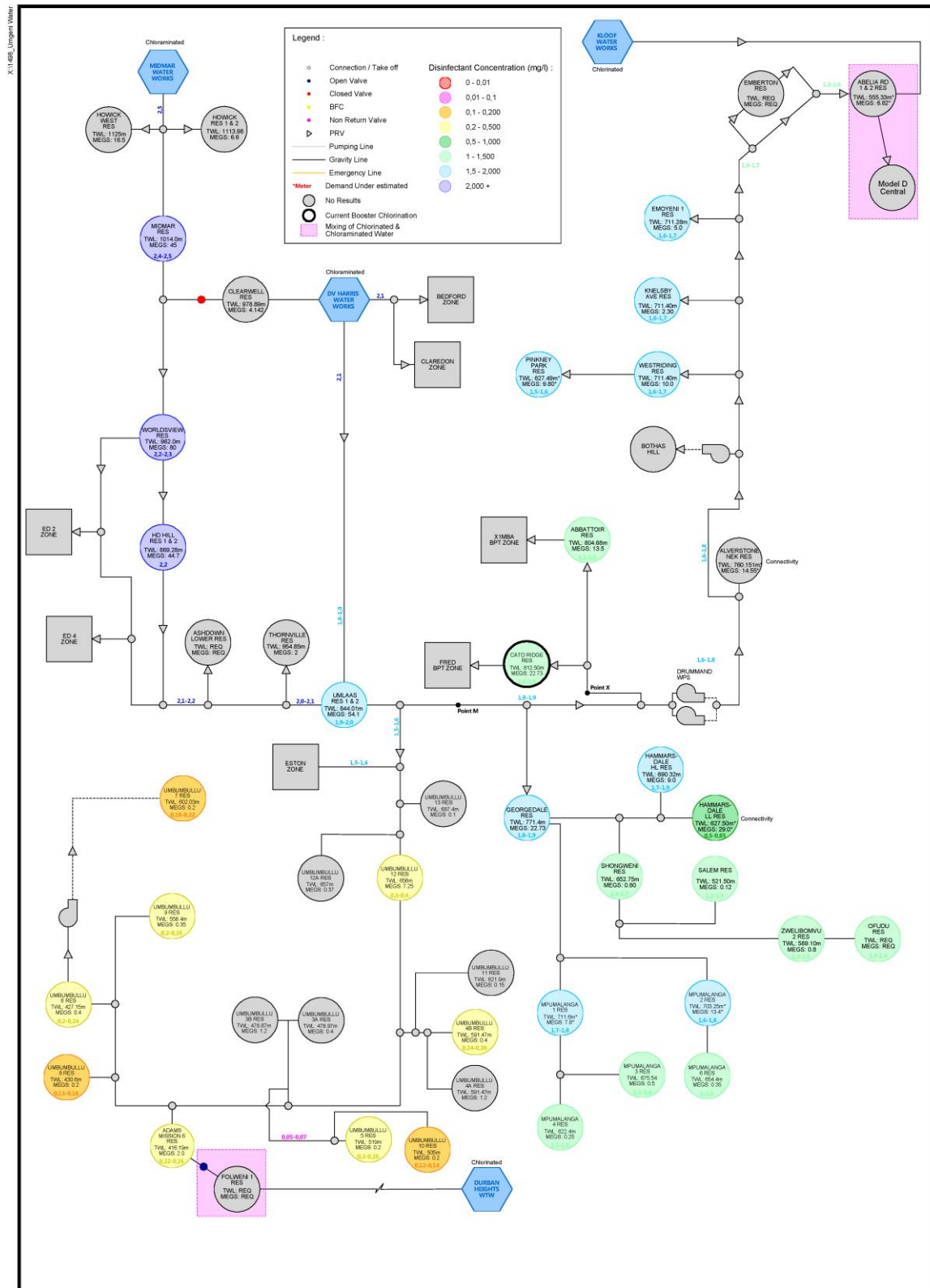


Figure 4-15 Disinfectant Concentration Western

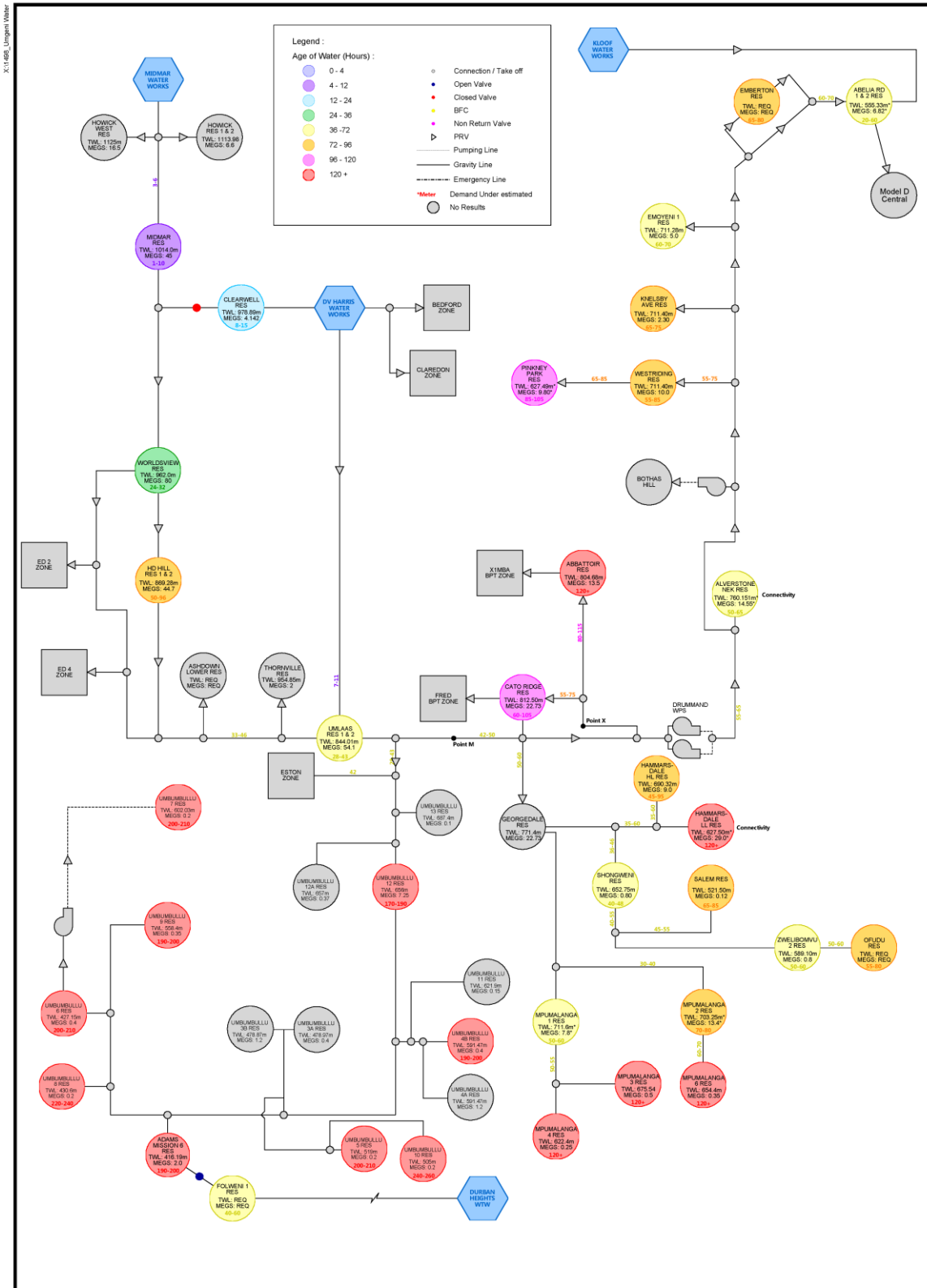


Figure 4-16 Water Age Western

4.4.3 Central Northern Model (Durban Heights and Hazelmere WTWs):

The model indicates that residence time in the system from Durban Heights WTW (DBH) to Mount View Reservoir is greater than 36 h, whilst the modelled water age reaching Grange Reservoir from DBH is 35-65 h. Water is supplied from DBH to the Mgangeni system through Ntuzuma Reservoir 2. The retention time in this reservoir was low and no problems were expected in this area. The critical points in the system are Ambleside Reservoir and the mixing areas.

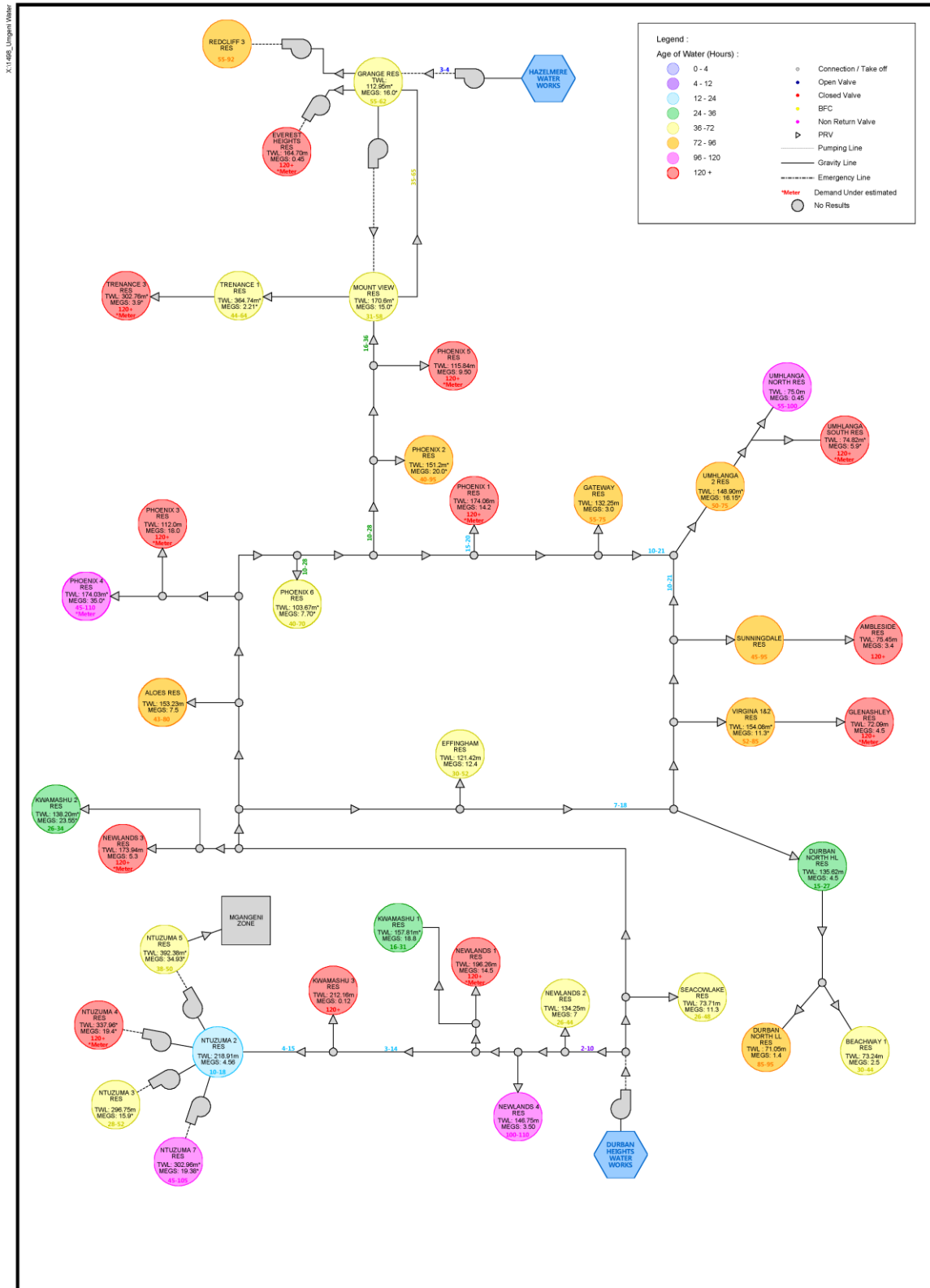


Figure 4-17 Water Age Central Northern

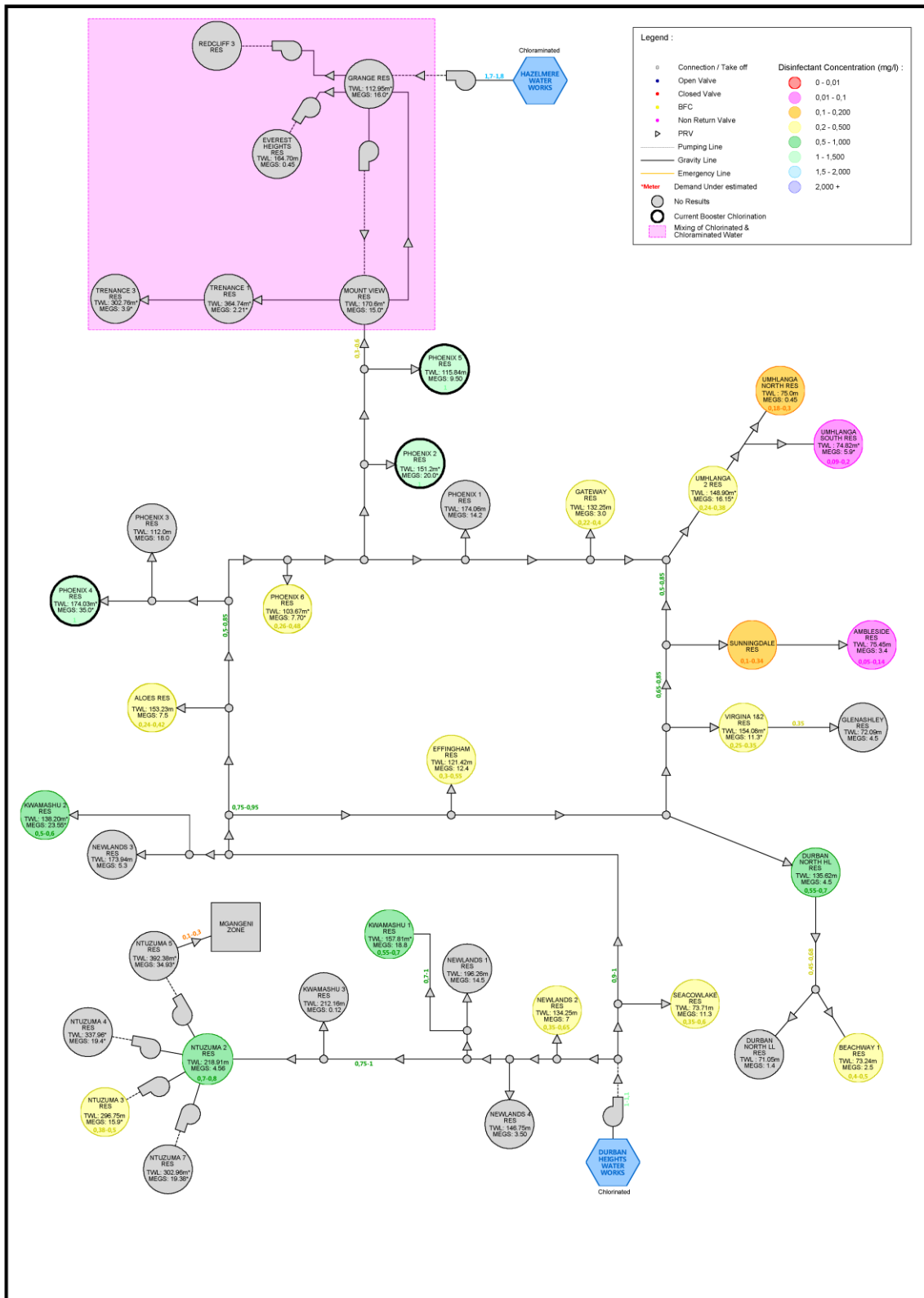


Figure 4-18 Disinfectant Concentration Central Northern

4.4.4 Central Model (Durban Heights WTW):

No critically low chlorine levels were indicated. However, Hocking Place, Clubhouse Place and Haygarth Road Reservoir could have water quality problems. The need for booster chlorination at these locations needs to be discussed. Two mixing areas were identified for this model as indicated on **Figure 4-20**. The model shows that acceptable chlorine levels were entering these areas. Sampling done by EWS on specific reservoirs sites indicated that the model results were in the low to average concentration range as expected for the low flow scenario. When chlorine was replaced by monochlorine decay no booster chlorine pumps were required. Current booster stations are at the correct locations.

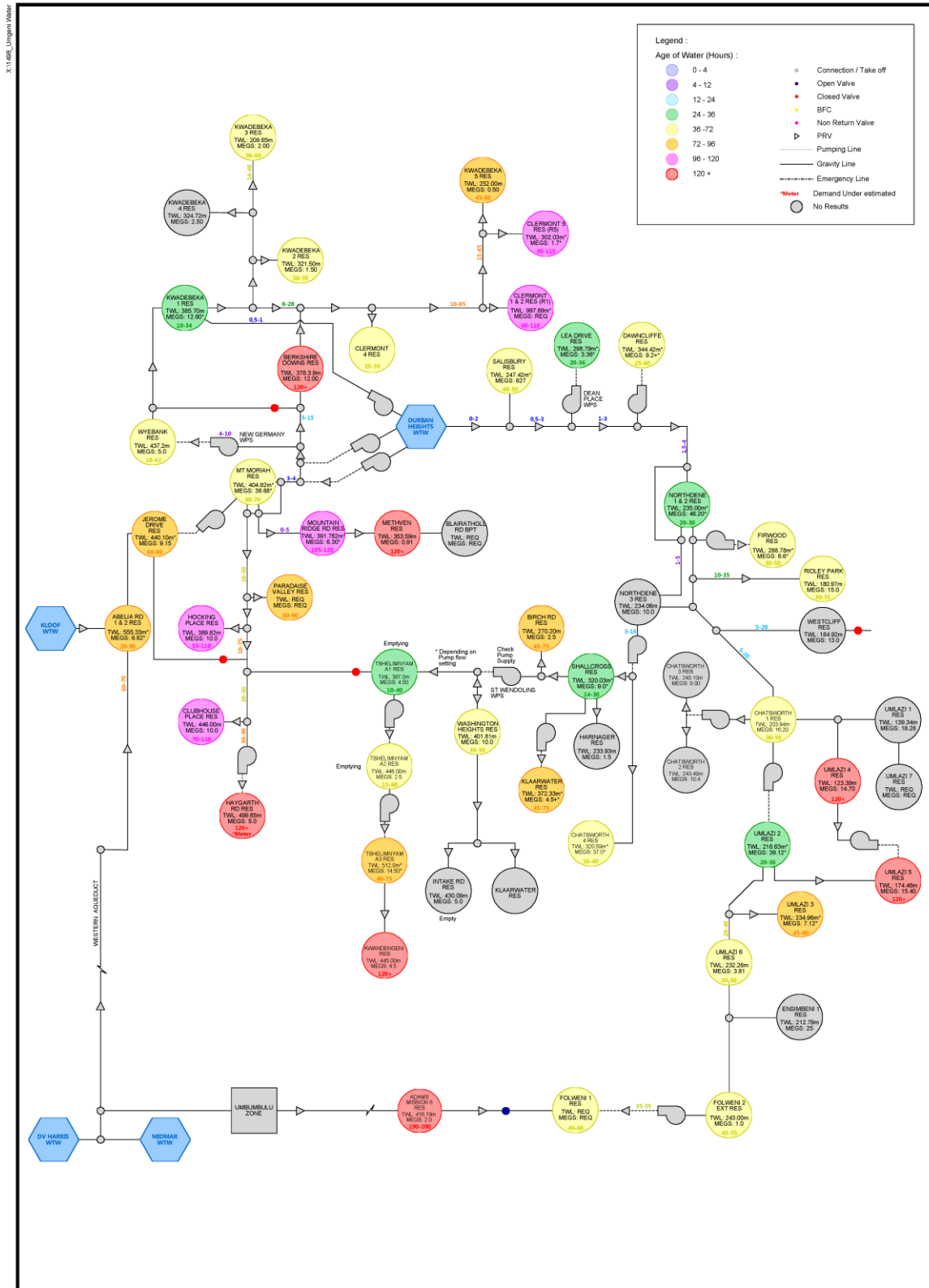


Figure 4-19 Water Age Central Model

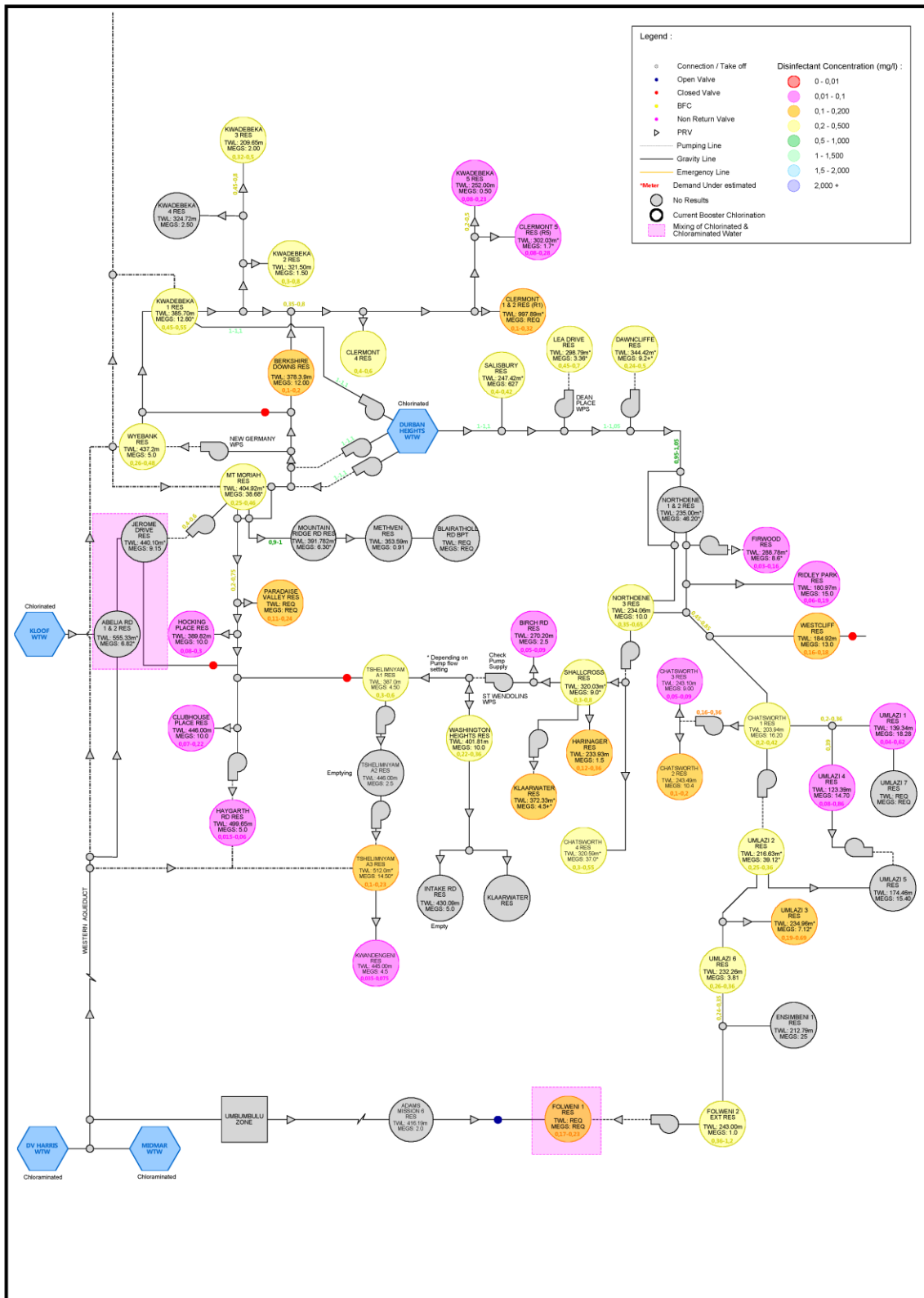
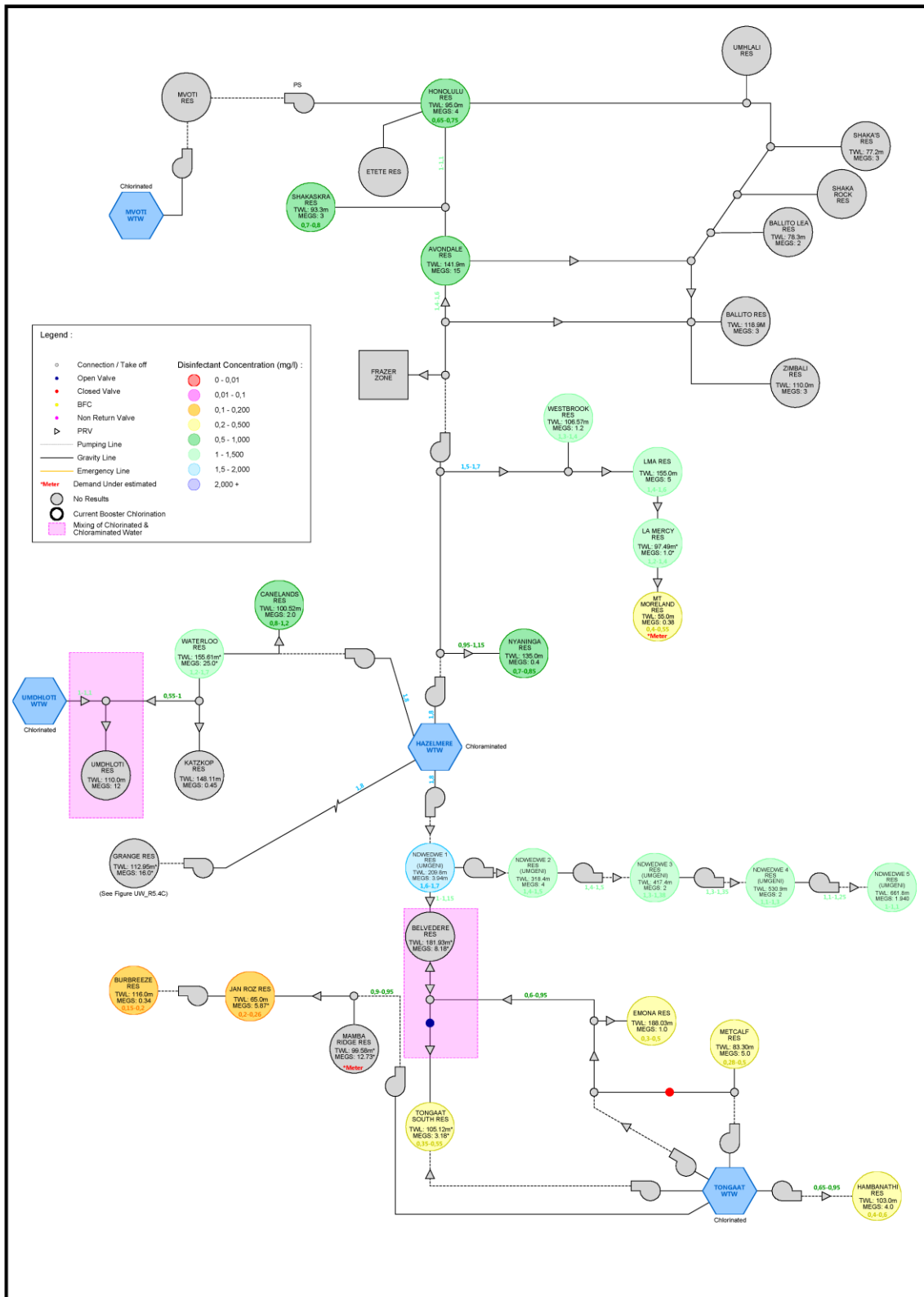


Figure 4-20 Disinfectant Concentration Central

4.4.5 Northern Model (Hazelmere WTW to Honolulu Reservoir):

The northern system has three WTWs, viz. Hazelmere WTW (Chloraminated), Umdhloti WTW (Chlorinated) and Tongaat WTW (Chlorinated), thus mixing occurs. The overall disinfectant concentration in the northern model was high. The residence time for the systems supplied from the Hazelmere WTW, with the exception of the Honolulu system, was relative short. Therefore, water quality problems would most likely be experienced because of the break point phenomenon as the disinfectant concentrations from Durban Heights WTW (chlorine) and Hazelmere WTW (chloramines) are both high in the mixing areas.



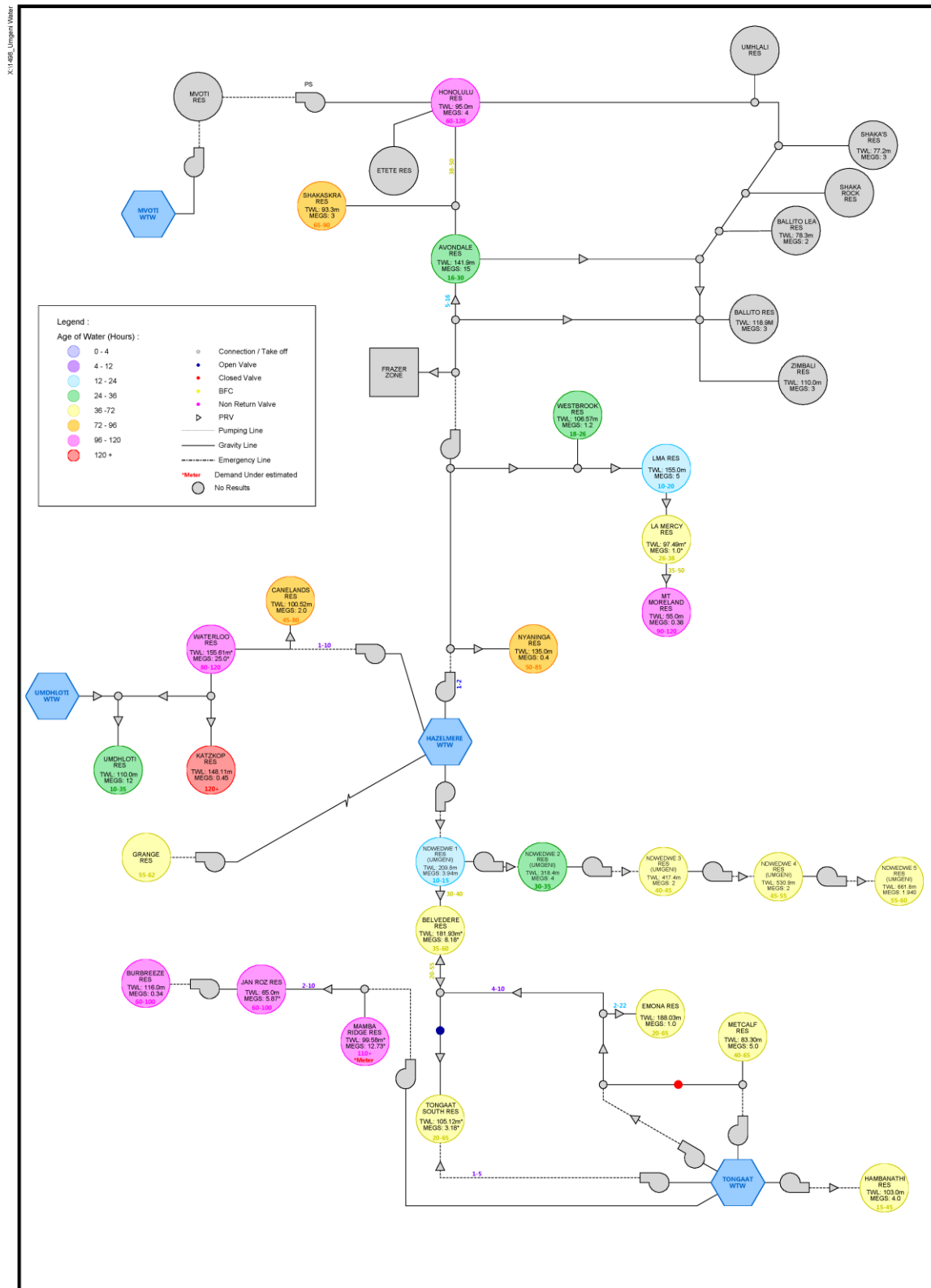
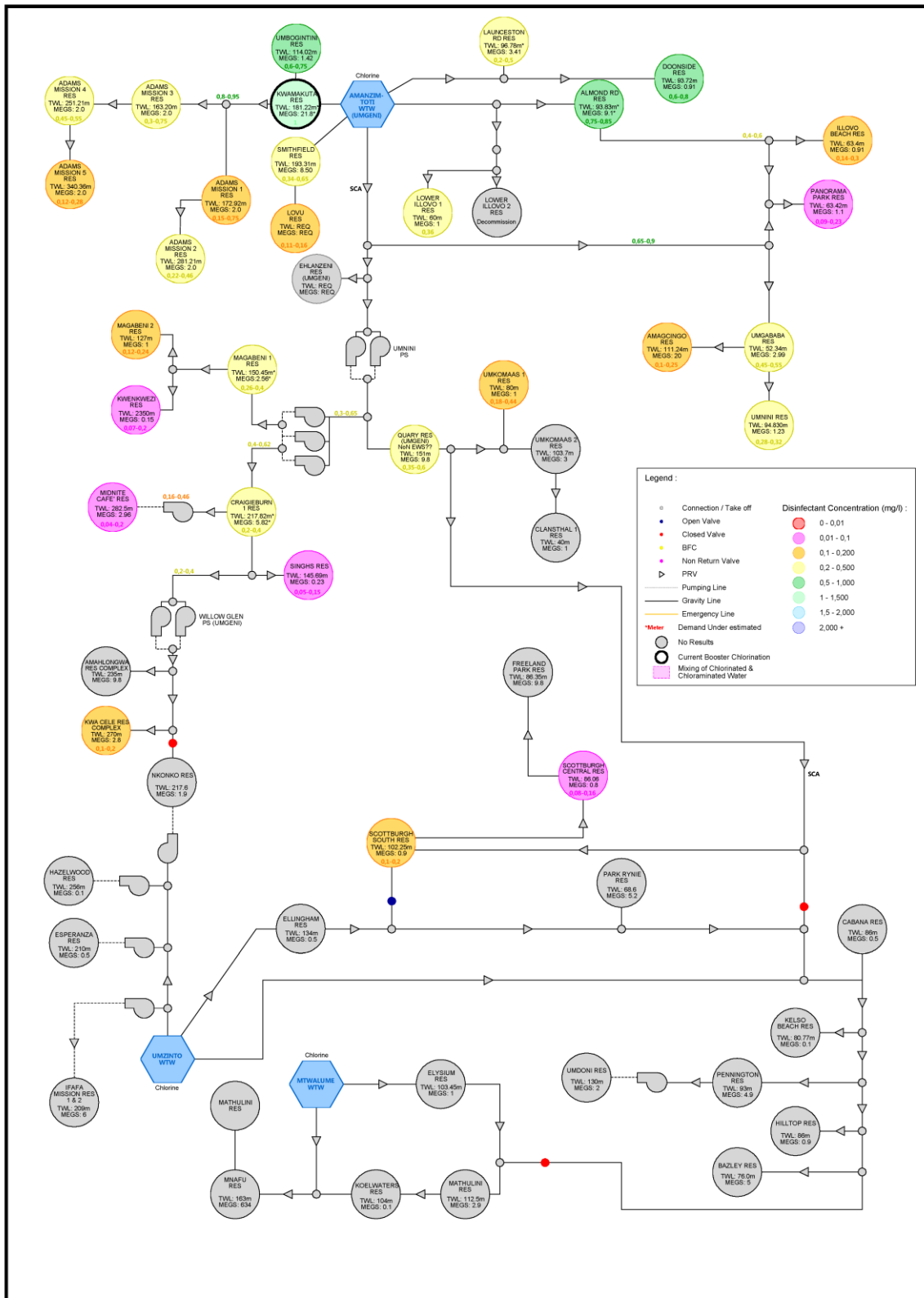


Figure 4-22 Water Age Northern Model

4.4.6 Southern Model (Amanzimtoti WTW to Mathulini WTW):

No mixing of disinfectants occurred in this area. Freeland Park Reservoir is at the end of the Amanzimtoti WTW system and could have water quality problems. A need for booster chlorination at Scottburgh Central and Craigieburn Pump Station was discussed. According to the model only reservoir retention times was a problem and not residence times in the bulk system. The implementation of a second booster chlorination point at Umkomaas Reservoir needs to be confirmed through sampling. Possible mixing of source water from Amanzimtoti WTW and Umzinto WTW. Chloramination of Amanzimtoti WTW would require no booster chlorination but mixing would most possibly occur to the south.



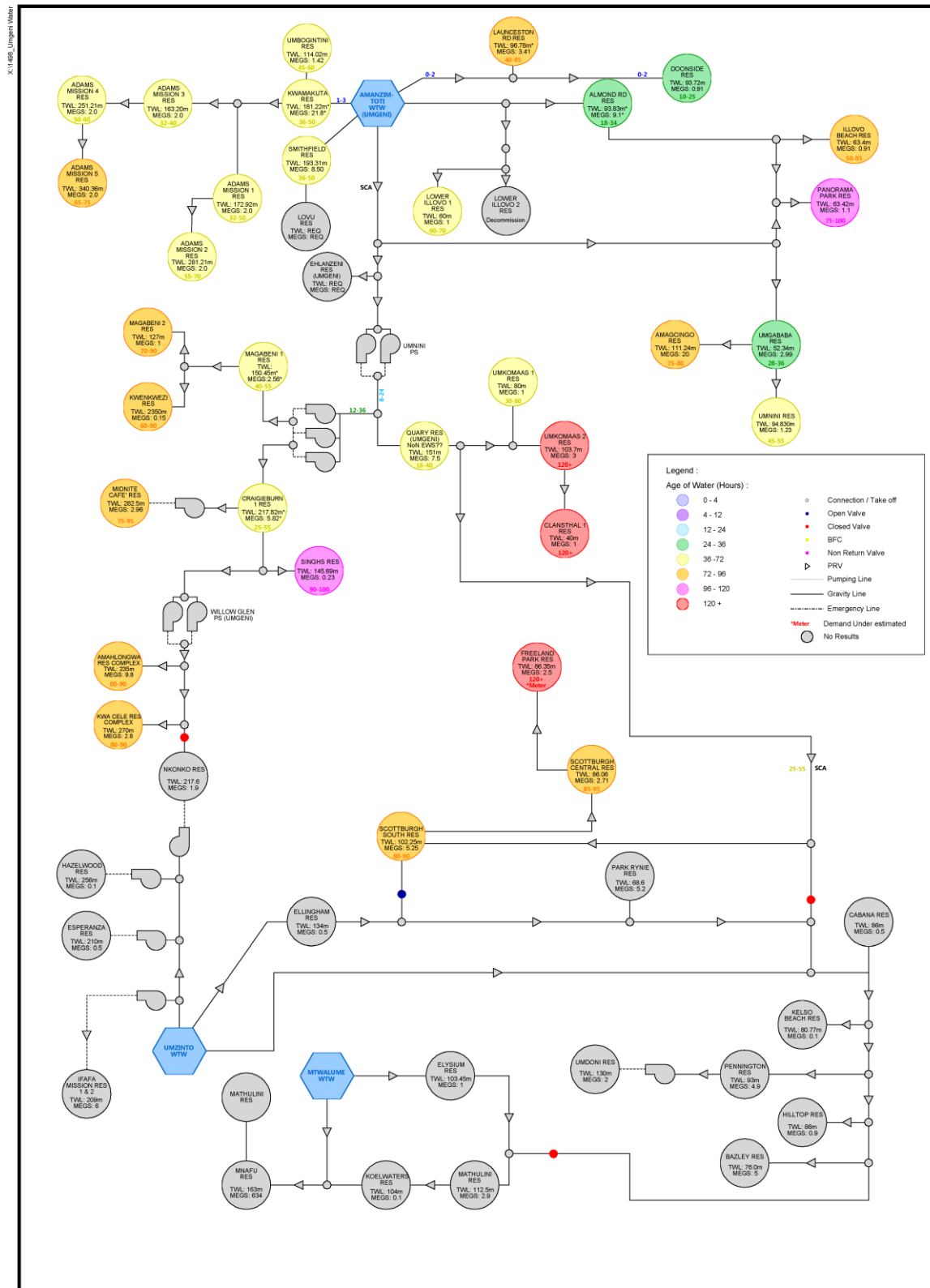


Figure 4-24 Water Age Southern Area

4.4.7 Future Western Aqueduct Model (Simplified):

Chloraminated water from Umlaas Road Res would mix with the chlorinated water from Durban Heights WTW. The concentration of disinfectant reaching Ashley BPT was high because the Western Aqueduct is not the critical point in the Midmar system.

4.5 Case Study Conclusion

Based on the preceding results the following conclusions were made.

The water quality model provides an extensive understanding of the operation of the system, providing a basis for a robust disinfectant strategy recommendation. This knowledge, together with the water quality modelling results as well as the outcomes of the Decision Support Tool (AWWA, n.d.), provided a sound basis for the recommendations.

4.6 Recommendations

4.6.1 Proposed Hazelmere WTW Disinfectant Operation:

The Hazelmere WTW is currently operated with chloramines with planned upgrading to the disinfection facilities currently in the design stage. In order to avoid mixing of reasonably freshly chloraminated water from Hazelmere WTW with chlorinated water from Durban Heights WTW, it is proposed to move the ammonia dosage point at Hazelmere WTW so that only the line to Honolulu (Northern System) be operated with chloramines. This is to counter the long retention times in this system for which chloramines are better suited and thus avoid the need for booster chlorination.

The other three lines from Hazelmere WTW to Belvedere, Waterloo and Grange Reservoirs respectively should then be switched to chlorination so that mixing is thus avoided.

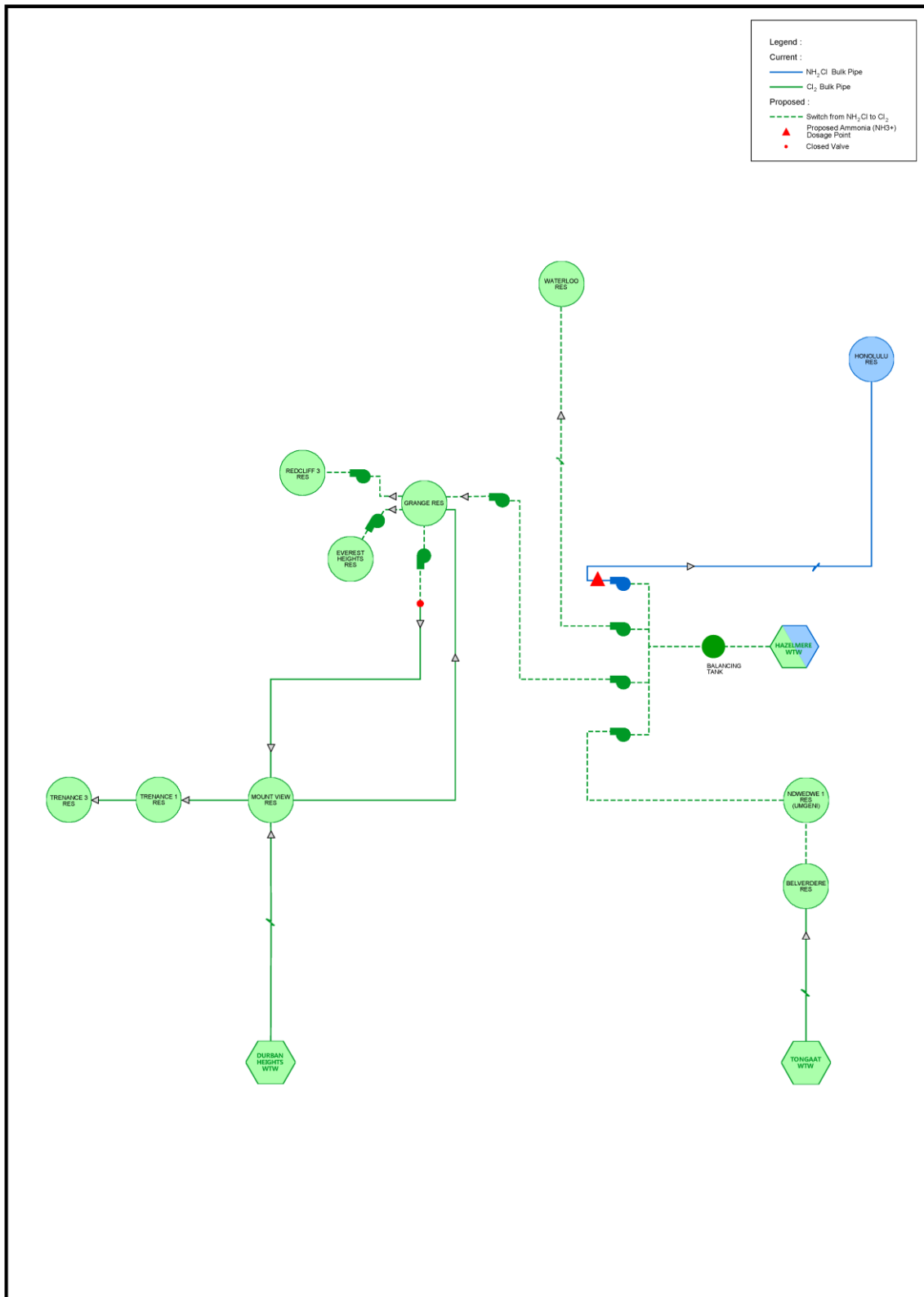


Figure 4-25 Proposed Disinfectant Operation at Hazelmere WTW

4.6.2 For implementation when the Western Aqueduct is extended:

It is proposed to implement breakpoint chlorination at the proposed Ashley BPT to be installed as part of the planned Western aqueduct extension, in order to avoid the extensive future mixing that would otherwise occur (see **Figure 4-26**). Current chloramine concentration at Ashley BPT is estimated at 1.2 mg/l – this can be reduced through lower dosage at Midmar WTW if booster breakpoint chlorination is implemented for Umbumbulu supply, depending on demand growth to minimise breakpoint chlorination dosage at Ashley. The pipe connecting Abelia Road Reservoir to Emberson Reservoir can then be closed off, thus totally eliminating mixing that currently occurs as depicted in **Figure 4-20**. The supply to Abelia Road Reservoir will then be via the proposed Western Aqueduct after Ashley BPT.

A possible booster chlorination station may be required on the Western Aqueduct upstream of Lindelani BPT. More detailed modelling of the future system /master plan is needed to confirm this requirement.

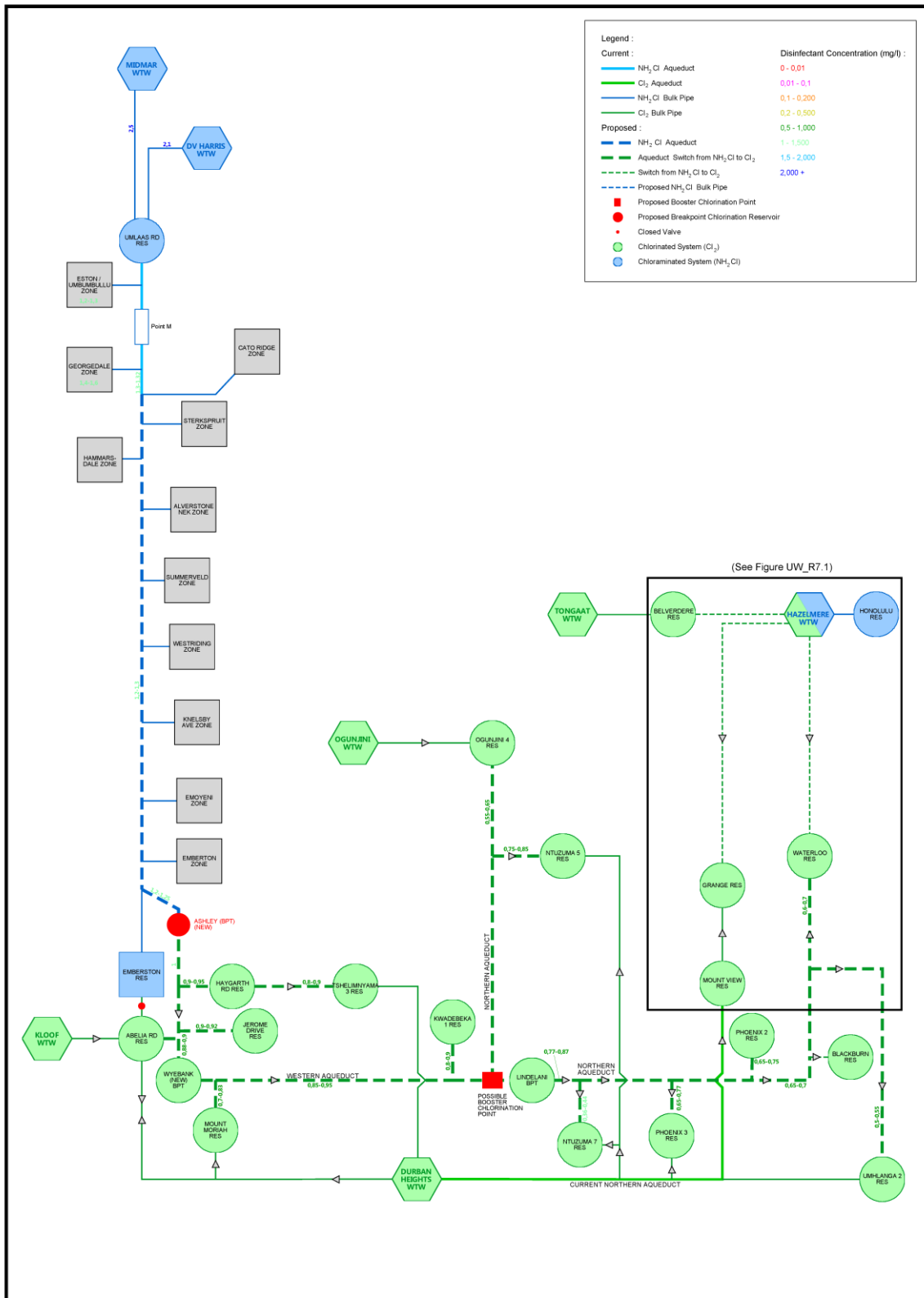


Figure 4-26 Proposed Disinfectant Operation Future Western Aqueduct

4.6.3 South Coast System Booster Chlorination Requirements:

It is proposed that booster chlorination stations be installed on the south coast system at Craigeburn Pump Station (Craigeburn Decommissioned WTW) and Scottburgh Central Reservoir due to low chlorine concentrations downstream of the proposed points (see **Figure 4-27**). However, further water quality sampling is recommended to confirm this requirement, as Scottburgh Central Reservoir is situated in the Ugu District Municipality, which is outside the study area of this thesis.

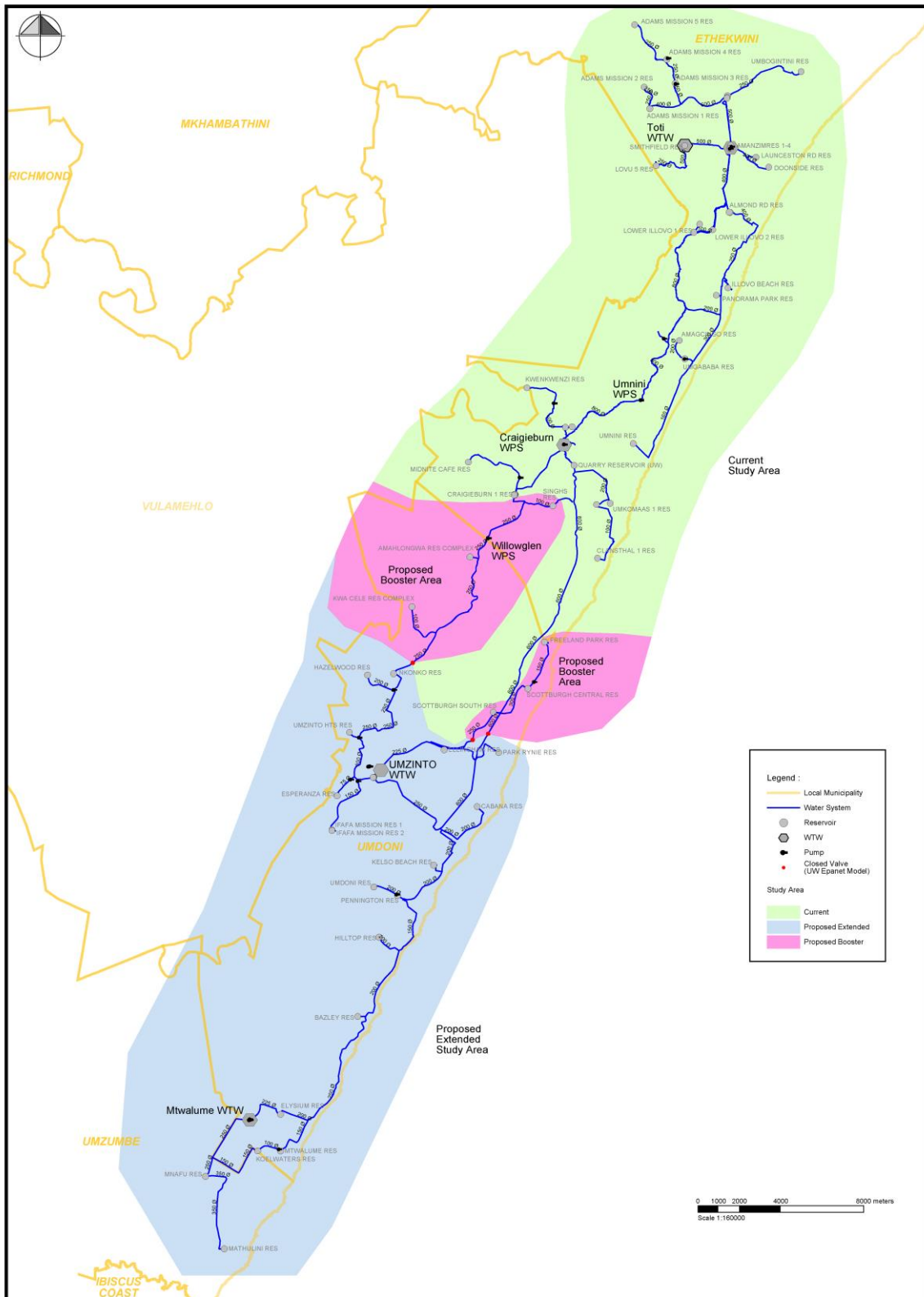


Figure 4-27 South Coast Booster

4.6.4 Possible Operational Improvements:

Reservoirs with long retention times of typically exceeding 120 hours and which have storage capacities exceeding current requirements (typically 36 to 48h x AADD) should be operated at lower water levels to reduce retention times and thus disinfectant decay through the system. This should be confirmed by accurate flow measurements and sampling over a long term at the inlets and outlets to confirm potential problematic reservoirs. Additional sampling is required where low disinfectant concentration were observed.

4.6.5 Future Model Improvements:

As indicated, the model could still be improved, which will present additional benefits, especially if it is to be used for purposes other than water quality modelling, in the following areas:

- Internal pipe layout /connectivity for all reservoir sites are to be captured;
- Actual reservoir operating levels should be obtained to improve stability and determine actual retention times;
- Missing pump information (including curve data) and reservoir TWL, volumes and actual BWLs should be obtained and entered into the model;
- Demand patterns can be calibrated;
- It is proposed to model the full extent of all reservoirs and bulk systems affected by the proposed Western Aqueduct to confirm retention times and the need for a booster chlorination station. This essentially means incorporating all master plan items for the future EWS system to establish a future water quality model;
- It will also be beneficial to model reticulation systems supplied from reservoirs indicated as potentially having long retention times to confirm critical points in the system and disinfectant concentrations / water quality problem areas;
- The model could be expanded to obtain results for less important reservoirs/ bulk systems or those for which model stability could not be achieved. This would require additional effort to capture and validate pipe connectivity on reservoir sites etc;
- Reservoir sites with more than one interconnected reservoir could have flow problems and need additional 3D modelling (CFD).

5 Conclusion

In summary the thesis objectives were to predict and determine the water quality in large water utilities. This was completed by researching the following:

- The effects of increased water demand worldwide;
- Increase in residence time of water in distribution systems based on current master planning guidelines;
- The aspects that affect the reduction in disinfectant potential;
- Water source characteristics based on the decline in available water sources;
- Pipe infrastructure deterioration and the effect this may pose on water quality;
- Health risks associated with alternative methods used to resolving water quality;
- How water quality is measured in a distribution system;
- Potential recommendations available to improve water quality in a distribution system.

Based on the **literature study** reviewed in this thesis the following main conclusions can be made:

- The increase in water demand has a huge impact on water residence time, this could be negative or positive depending on current master planning and operational procedures used in water utilities;
- The need for water quality models has increased in popularity and is very useful in determining a disinfectant strategy for large water utilities as indicated in the case study;
- Disinfectant decay coefficients can differ immensely based on source water and disinfectant used, but previous studies can be used for a sensitivity analyses;
- Pipe wall decay coefficients can be used to calibrate Water Quality models and can be a function of pipe condition and the Reynolds number of the flow through the pipe section;
- Bulk water decay coefficients was the main influence of decay in a bulk distribution systems;
- Master Planning design requirements need to be revised to address long retention times in future scenario planning;
- The general assumption is that chlorinated water can still be used as disinfectant up to five days;
- Chlorinated water is 25 times more reactive than chloraminated water, thus a better disinfectant which requires less contact time for disinfection;

- The general assumption is that chloraminated water has a life span of 14-21 days;
- Based on breakpoint chlorination, mixing of chlorine and chloramines is unfavourable.

Based on the **case study** done in this thesis the following conclusions were made:

- Data collection regarding model setup is the main limitation on the stability of a Water Quality model and not the hydraulic modelling tool itself;
- The main contribution to model integrity is the confidence in the operational rules used to operate reservoirs, pumps and valves, thus, establishing an accurate dynamic simulation;
- Wadiso can be used to predict disinfectant potential even if limited information is available. In this case a sensitivity analysis can be used to use results as an estimate for scenario testing;
- Water Quality modelling is a useful tool and provides great insight into a distribution system predicted water quality;
- Water quality modelling is not a substitute for sampling, but can be used as a tool to predict possible water quality risk areas and where additional sampling is required;
- Water quality modelling is most efficient in calculating expected disinfectant dosage levels at water treatment plants to sustain disinfectant potential throughout the system and optimize a water quality sampling scheme to regulate water quality sufficiently;
- Structural and operational recommendations can be made based on isolating potential high risk areas obtained by water quality modelling.

In summary, chlorine concentration can be used as a sufficient indicator for water quality. The reduction in chlorine concentration can be predicted and modelled throughout a distribution system by using Wadiso (EPANET) solver. These reductions in chlorine concentration were based on certain characteristics of the water and must be calculated individually. In the case where limited data is available due to a lack of testing, previous studies can be incorporated into the model to determine decay rates, but results must be used accordingly. The main problem with large water utilities is the aspect of obtaining useable data. Thus, it is recommended that when water quality modelling is attempted in large water utilities a management system is used that is capable of data organisation and tracking for integrity and reduction in data overloading. This could potentially be the most critical aspect in large water utilities. Additional future work is required to fully utilize the process of water quality modelling and is indicated in **Chapter 6**.

6 Recommendations for Future work

In this section a short description of future work is indicated that will provide valuable benefits to the existing process of water quality modelling.

6.1 CFD Modelling Regarding Reservoir Retention Time

As previously explained the accuracy of a hydraulic water quality modelling is based on critical assumptions. These assumption include reservoir mixing and flow within a specific reservoir. Computational Fluid Dynamics (CFD) modelling can be used to reduce the risk of incorrect assumptions through modelling the identified reservoirs in 3D. This information can then be reintroduced into the hydraulic model as boundary conditions.

6.2 Water Quality Risk Analysis Approach

As indicated in the conclusion, the existing water quality model only indicates the predicted disinfectant potential and not the risk of potential contamination. Thus, by combining the consequence of failure and the likelihood of failure, an integrated risk analysis can be determined. Indicated in the thesis is the empirical consequences of a pipe failure to contamination. By incorporating this and the probability of contamination, one could obtain an area with high health risk. This could be used in a Water Safety Plan to reduce potential health issues.

6.3 Additional modelling capabilities

In a hydraulic model pipes were preselected with a defined decay rate associated with the expected source water that will be supplied to this area. When multiple water sources were used in an isolated system there is a need to change the disinfectant coefficients based on the percentage of water available in each pipe section. In other words, when chloraminated and chlorinated water are mixed, the disinfectant decay is changed. This will require laboratory testing to establish a baseline for decay rates based on percentage variations in source water concentrations. As indicated, mixing of chlorinated and chloraminated water is regarded as a high risk to public health and must be avoided as far as possible. Utilities spend huge amounts of time and money in avoiding mixing of disinfectants in water districts and by using this modelling tool utilities could be able to manage mixing areas instead of avoiding them.

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