Evaluation of Water Quality in Urban Drinking Water Distribution Networks: A Case Study in Johannesburg, South Africa

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Abstract

In the Republic of South Africa (RSA) access to drinking water is a constitutionally guaranteed human right. The supply of safe drinking water to consumers is a legal requirement and the numerical limits for drinking water quality are described in SANS 241:2015. However, in 2023 over 30% of RSAs Water Services Providers (WSPs) supplied water that was of poor quality, with breakouts of cholera becoming more frequent. Poor drinking water quality is a result of a complex combination of factors requiring different interventions, one of the complex factors is drinking water quality management in a distribution network. What makes drinking water quality management in a distribution network complex is *inter alia* the fact that the water quality deteriorates within the distribution network. This presents a need for tools that can assist WSPs to better understand and manage drinking water quality in a distribution network.

The evaluation of widely used, credible and freely available modelling tools, that can assist WSPs with limited resources (financial and human capital) for drinking water quality management in their distribution networks, was considered worth exploring. The main objective of the study was therefore to evaluate water quality in an urban drinking water distribution network, considering a case study in RSA, utilising a widely used and freely available modelling tool. The study calibrated (hydraulic and water quality) and validated (water quality) the distribution network following international best-practice, prior to commencing with the evaluation. To ensure practical relevance for WSPs, the study focused on the key sources of uncertainty in drinking water quality modelling in water distribution networks. Drinking water quality (considering various specific determinants) was the dependent variable, with the independent variables being: hydraulic definition, level of calibration, pipe age, pipe material, water demand pattern, load shedding and tank (reservoir) mixing model. The study also considered the practical usefulness of a free and widely used tool (EPANET 2.0) in optimising a network drinking water quality sampling programme.

To optimally evaluate the sources of uncertainty two independent models were developed through skeletonisation and reduction. These were the Medium-Level Detail Model – MLDM (reduced all pipes model) and the Low-Level Detail System – LLDS (significantly reduced and skeletonised). It is reasonable to assume that utilities will not always have all the distribution network hydraulic data, but they may still need to model the water quality of these networks with incomplete data (in the interest of protecting public health). A consideration of various water quality determinants (physical, chemical and biochemical), modelled on the MLDM and the LLDS showed that distribution network simplification (through reduction and skeletonisation) does not compromise water quality modelling accuracy. However, the MLDM proved accurate for more determinants (dissolved oxygen, total chlorine, chloramine and dissolved organic carbon) than those of the LLDS (free chlorine, biodegradable organic carbon). It was therefore concluded that there is some benefit in investing in additional hydraulic detail (hydraulic definition) as the returns are higher levels of water quality modelling accuracy, for certain determinants.

The study then considered the impacts of the level of calibration on water quality modelling accuracy for both the LLDS and the MLDM. As expected, the level of calibration was shown to correlate directly with water quality modelling accuracy. However, the investigation showed that hydraulic definition was more important than the level of calibration, in extended period simulation. This was because the second best calibrated MLDM (known control status), produced more accurate results than the most calibrated LLDS (variable control status) when considering total chlorine. This further highlighted the need to prioritise hydraulic definition as far as practically possible.

Distribution network pipes are generally underground as such their material, age and or condition is often not reliably known, resulting in a need to estimate the roughness coefficient (C-value). For both models (LLDS and MLDM) it was clear that the pipe material and age were significant parameters as there was notable variance in total chlorine concentrations (up to 34% for the MLDM and up to 95% for the LLDS) with changes in the roughness coefficient (C-value).When considering the same C-value, the MLDM was generally more accurate (range was between 10% and 20%) than the LLDS, as observed from the lower errors (difference between model outputs and field measurements). Both models followed the same pattern of water quality deterioration, meaning they were both useful for modelling purposes. For both models, the most significant adverse impacts (on water quality) were for badly corroded steel, iron and clay pipes. This meant that special care to determine pipe age (condition) must be taken when modelling networks with these materials, otherwise the model will produce errors that render the model useless. Pipe age, condition and material were found to have the most significant influence on water quality modelling accuracy. Therefore, the most effort should be expanded on this source of uncertainty, as incorrect C-values can render the water quality model outputs useless, thus undermining the entire exercise.

Water demand patterns are another source of uncertainty in practise, as they are not always exhaustively known. In this study demand patterns were found to have a very minimal impact on water quality, with an absolute maximum difference of 4% when considering total chlorine. However, demand patterns are critical for hydraulic and operational considerations. South Africa has been struggling with rolling electricity blackouts (load-shedding) for well over a decade. Load-shedding has a direct impact on water supply as the study area relied on pumps (additionally, the absence of electricity influences water demand patterns). Load-shedding was shown to have the most pronounced adverse water quality impact during stages 5 (five) to 8 (eight), with water quality precautions (additional dosing or boil notices) needed during stages 7 (seven) and 8 (eight); this was because water ages increased to a point (well above 70 days) where free chlorine was well below 0.2mg/l. Tank (reservoir) mixing models were shown to influence water quality most significantly under conditions of short-circuiting (First-In-First-Out), presenting a need to understand under what conditions tank short-circuiting was likely and how it could be prevented.

The identification of optimal critical sampling points is a key facet of drinking water quality management (adhering to a preventive risk management philosophy), and the selected modelling software was shown to be one of the options that can be considered for this exercise. The importance of both hydraulic and water quality parameters was observed in all scenarios, underscoring the need to understand both independently and jointly (the impact one has on the other); as the end objective is ensuring safe drinking water is delivered to consumers, to preserve public health. It was concluded that EPANET 2.0 was a useful software, that could add significant practical value to WSPs in managing drinking water quality in a distribution network to protect and preserve public health.

Dedication

For Tanatana Nocingile Dzila

You held my hand and walked me to the beginning of my formal education. You were there, always.

For Zuziwe Nosakhale Dzila Luyaba

All that is good that I know, I learned from you.

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1. Introduction

1.1 Background to investigation

Water is and has always been an integral part of human life. To that end, the South African Constitution legislates access to safe drinking water as a basic human right in the Republic of South Africa (RSA Constitution, 1996). Through the Constitution, the national government is the custodian of all water resources in South Africa, on behalf of all South Africans. To effectively achieve this function, the Department of Water and Sanitation (DWS) was set up and it is primarily responsible for the formulation and implementation of policy governing the sector, while striving to ensure that all in South Africa gain access to clean water and dignified sanitation (DWAF, 2005).

Schedule 4 Part B of the South African Constitution allocates the provision of drinking water to municipalities. Furthermore, through the Municipal Structures Act (Act 117 of 1998), municipalities (District, Local and Metropolitan) are given the power to act as Water Services Authorities (WSAs) and the roles and responsibilities of the WSAs are clearly defined in the Water Services Act (Act 108 of 1997) and the Municipal Systems Act (Act 32 of 2000). Only municipalities can act as WSAs (in South Africa 144 of the 257 municipalities are WSAs, as at 2023). A WSA has two legislated functions: Governance and Provision of all water services. Governance pertains to: ensuring access to water and sanitation services, regulation through the creation of bylaws, planning through the water services development planning (WSDP) framework and deciding on a Water Services Provider (WSP) mechanism (Act 108 of 1997).

Post democratic South Africa has been focused on “backlog eradication”, which centres (almost exclusively) around providing access to water. This is vital, but access to water that is not safe nor reliable falls short of fulfilling the Constitutional obligation of the State. In recognition of failures in the provision of safe, reliable and sustainable access to water; many policies and strategies have been developed. Most pertinent in this context is the Blue Drop Initiative (BDI). The BDI is a DWS led incentive-based regulation developed in 2008. The initiative encourages the proactive management and regulation of drinking water quality; in line with legislated norms and standards. The BDI provides a qualitative and quantitative assessment of drinking water (quality) management in South Africa. The BDI presents (Blue Drop Report, 2014):

- Drinking water compliance reports, with respect to the national drinking water standard (SANS 241:2015 preceded by SANS 241:2011);
- Results from an assessment of drinking water safety planning practices, whose intention is drinking water quality management through risk management and mitigation;
- A review of asset management practises (operation and maintenance, process audit reporting, asset registers, design and operational capacity); and
- An assessment of human capital development (operator skill and management support).

The 2023 Blue Drop results were extremely alarming; with only 2.7% of the 958 water supply systems assessed achieving a “blue drop” status (water supply considered excellent). There was a steady deterioration from 2014 to 2023, with the number of water supply systems in a poor or critical state increasing from 41% in 2014 to 47% in 2023. If the identified areas of vulnerability are not urgently addressed, these poor and critical systems will ultimately not supply safe drinking water. Table 1-1 below presents national performance from 2009 to 2023 (DWS, 2023).

Table 1-1: Blue Drop comparative performance from 2009 to 2023 (DWS, 2014 & 2023)

Performance Category	2009	2010	2011	2012	2014	2023
National Blue Drop Score	51.4%	67.2%	72.9%	87.6%	79.6%	51.3%[*θ]
No. of Water Services Authorities Assessed	107	153	162	153	152	144
Number of water supply systems assessed	402	787	914	931	1036	958
Number of systems achieving BD status	25	38	66	98	44	26
Percentage of systems achieving BD Status	6%	4.8%	7.2%	10.5%	4.2%[*+]	2.7%

[*+] The percentage drop between 2012 and 2014, is partly explained by the addition of the water loss management element of water conservation and demand management also referred to as “No-Drop” (DWS, 2014).

[*θ] Calculated using National Summary of the 2023 Blue Drop Audit Key Performance Areas (Table 2), applying weightings provided in the 2023 Blue Drop Audit Requirements (DWS, 2023).

In 2023 DWS undertook a smaller version of the Blue Drop assessment, titled the “Blue Drop Watch Report 2023”. This assessment considered: a technical site assessment, microbiological compliance and chemical compliance. A total of 151 water supply systems were assessed, with 15% (23) systems found to be in a poor and critical physical state, 51% (77) of the systems were found to have poor to bad microbiological compliance and a concerning 71% (107) were determined to have failed to achieve chemical compliance (DWS, 2023). The standard used for microbiological and chemical compliance was SANS 241:2015. This suggests a pattern of deteriorating drinking water quality and a general inability to provide safe drinking water by municipalities (as Water Services Providers – WSPs).

National government, DWS, municipalities, water utilities and water users in South Africa are increasingly realising that water quality management is a complex undertaking, requiring a wide array of expertise and resources to successfully undertake. The Blue Drop results suggest that South Africa is currently failing in this endeavour, thus endangering public health, ecological life and the environment at large. In response to this challenge, South Africa has developed extensive policy and guidelines to create a legislative environment conducive for effective water quality management. To complement the legislative framework, the Water Research Commission (WRC), research institutions, water utilities and other stakeholders have invested in research initiatives to address challenges of water resource and quality management.

The continued drinking water quality deterioration despite collaboration between various stakeholders is testament to the fact that drinking water quality management is a complex field requiring: an understanding of the chemical, biological and physical (includes but not limited to hydrology and hydraulics) characteristics of water and the relationship and interaction between these. This relationship exists and is important from catchment, abstraction, through treatment, distribution, use, and eventually discharge of the used water (AWWA, 2005; Shang *et al.*, 2008; Biyela, 2010; Rust, 2014; Paluszczyszyn, 2015; Culligan, 201; Lui, 2017). Thus, there exists a need to contribute to drinking water quality management in South Africa. This contribution will not only ease the work of WSAs and WSPs, but will also assist national and local government in fulfilling their legislated mandate of providing safe drinking water to all in South Africa. There are many areas of potential contribution, one of the most important being: tools to assist water utilities and practitioners in the management, monitoring and prediction of water quality within drinking water distribution networks.

1.2 Problem Statement

In South Africa the supply and delivery of safe drinking water is a legal requirement and the numerical limits are described in SANS 241:2015. However, a notable and ever-increasing number of South African Water Services Authorities (WSAs) have Water Services Providers (WSPs) that supply water that is of poor quality. This number has been steadily increasing from 2008 and was 30% in 2023 (DWS, 2014; 2023). The (poor) water quality is a result of a complex combination of factors requiring different interventions, but one of the factors is the complexity of drinking water quality management (in a distribution network). One dimension of this complexity is the fact that a proportion of the water quality deterioration occurs within the drinking water distribution network (Woolschlager, 2000; Lahlou, 2002; AWWA, 2005; EPA, 2005; Shang *et al.*, 2008; Biyela, 2010; Useh, 2017). Drinking water quality monitoring is a very complex and resource (financial and level of skill and expertise) intensive undertaking. The evaluation of widely used and freely available tools, that can assist water utilities in the management of water quality deterioration within the distribution network, is therefore worth investigating, as it would contribute to the realisation of a basic human right (access to safe drinking water).

1.3 Goal and Objectives of investigation

The main goal of this study was to model the drinking water quality in a portion of Johannesburg Water's drinking water distribution network and evaluate various water quality considerations in the selected network.

The specific objectives of this investigation were therefore to:

- Incorporate the readily available physical, structural (design and layout) and operational characteristics, of a selected portion of Johannesburg Water's (JW's) drinking water distribution network, into the EPANET 2.0 hydraulic software i.e. hydraulic model calibration and validation to acceptable industry norms for network modelling purposes;

- Through skeletonisation and reduction, develop two independent hydraulic network models with decreasing definition and detail (simplified models with decreasing levels of hydraulic definition) for hydraulic and water quality calibration and water quality validation;
- Model water quality for the two network definitions by comparing modelled water quality outputs with field measurements over the same period;
- Use the water quality field data collected in a 2016 and 2017 study (Useh, 2017) of the selected portion of JW's distribution network to test various scenarios that are common in practice;
- Perform a sensitivity analysis, specifically testing model sensitivity to: model hydraulic definition (level of detail in the model), level of calibration, pipe material and age, water demand changes, water age, load-shedding, tank mixing model and
- Assess the functionality of the EPANET 2.0 software for optimal water quality sampling location determination and general modelling of water quality and make recommendations.

1.4 Scope of investigation

A single public domain (freely available) software that is widely used by researchers and practitioners was selected (EPANET 2.0). The project was limited to the calibration (hydraulic and water quality) and validation (water quality) of a water distribution network and use of the calibrated and validated models as inputs to the EPANET 2.0 software, to test various water quality monitoring and management scenarios and to make observations on these.

2. Literature review

This literature review aimed to identify the state of knowledge in the complex field of drinking water quality management in a distribution network. This was achieved through a review of relevant research material i.e. structural factors, operational factors, water quality factors, water quality modelling and a consideration of relevant previous studies.

The findings of a review of relevant published research material is presented, and demonstrates the importance and complexity of hydraulic calibration in the development and application of deterministic water quality models.

2.1 Structural factors

To get water from source to the end-user, a complex network of water conveyance infrastructure is required. This network will vary from system to system but typically includes: dams, water treatment works, pump stations, bulk pipe lines, reservoirs, valves, meters, connections and reticulation pipe work. Thus, the structural elements of a distribution network can be considered to be the fixed or physical elements. This infrastructure must be in place before water can be conveyed.

Research has shown that the structural elements of water infrastructure have a significant impact on water quality (Lahlou, 2002; DWA, 2009; EPA, 2010; Lindhe, 2010; Rubulis *et al.*, 2007; WHO, 2014). These range from: inappropriately located abstraction points, poorly designed water treatment plants that produce water of sub-standard quality, oversized systems with long retention times that result in a loss of residual chlorine, poor pipe routing maximising opportunities for hazardous ingress during low or negative pressure events to poor material selection resulting in increased risks of corrosion and or biofilm accumulation (e.g. pipeline going through an area with contaminated soil). This section will discuss some structural factors that adversely affect water quality and consequently pose a hazard to the end-user’s health. The first aspect to be discussed is water supply source, then water treatment and finally distribution network design and material (pipe) selection.

2.1.1 Water supply source

Drinking water can be obtained from a wide array of sources, namely: ground, surface (lakes, rivers and streams), sea, wastewater effluent etc. Depending on water availability, demand, intended end-use and costs (capital expenditure and operating), one or a combination of these sources can be utilised (Van Duuren, 1997; Edzwald, 2011). The selected treatment process (conventional or advanced) is significantly influenced by raw water quality, e.g. sea water can only be treated through advanced treatment processes (reverse or forward osmosis), while surface water can be adequately treated with conventional (dosing, mixing, sedimentation, filtration and disinfection) treatment technology. The selected treatment technology is continually optimised (and upgraded where necessary), to ensure the desired treatment efficacy is achieved.

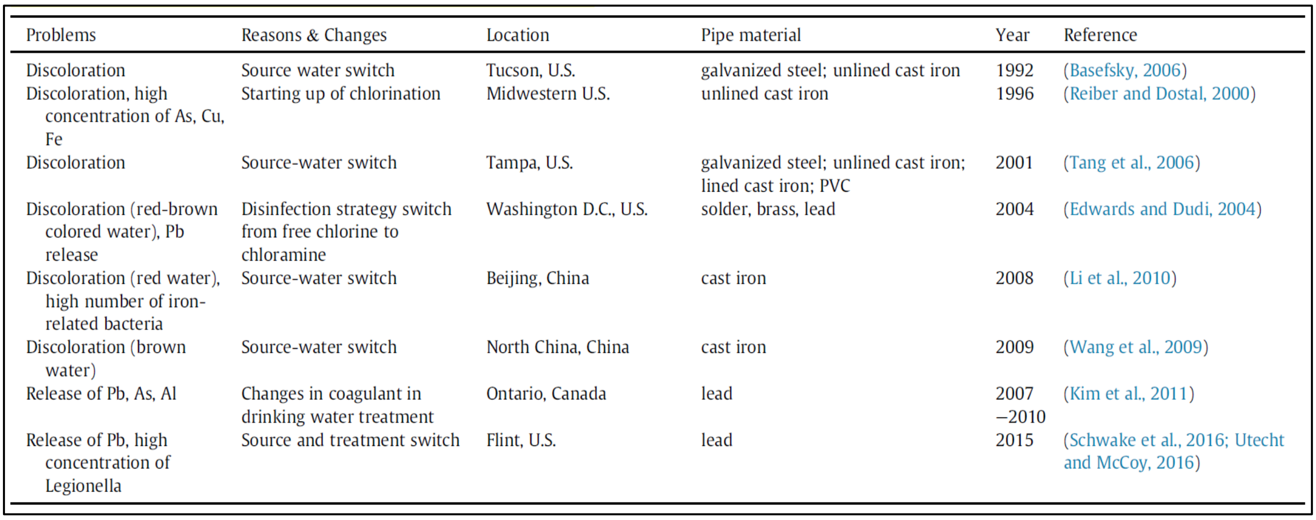
The continuous protection and monitoring of water sources is advocated for in all countries with comprehensive drinking water standards (AWWA, 2007; DWA, 2009; EPA, 2010; Umwelt Bundesamt, 2014; WHO, 2014). In South Africa the Department of Water and Sanitation (DWS) established Catchment Management Agencies (CMA's) to fulfil this critical role (DWAF, 2005). These efforts are as a result of increasing realisation that good and consistent water quality are corelated to quality supply.

Research has shown that raw water quality supply has implications for operations at the treatment works and in the distribution network (Van Duuren, 1997; Spellman, 2003; Schutte, 2006). Operations at treatments works are often standardised, with respect to chemical dosages, retention times, flow velocities, backwash rates, production volumes etc. (Schutte, 2006). This consistency and predictability allows for the optimisation of operations, though raw water readings are taken daily. The deterioration and variability in supply source quality necessitates changes to the established operations and in that time of "transition"; the quality of the effluent is often adversely affected (EPA, 2010; Edzwald, 2011; DWS, 2013). This adverse effect is amplified in areas with: poorly maintained infrastructure, limited operating budgets and low operator skill levels (Lawless, 2005, 2007, 2016; Macleod, 2013).

Research has shown that these adverse effects are not limited to the treatment works, and can have a ripple effect on the entire downstream distribution network. Adverse "transition effects" are often assumed (incorrectly so) to be a result (almost exclusively) of deteriorating raw water supply, but these can be a result of changes in raw water characteristics, treatment method / process or chemicals (Liu, 2017). Table 2-1 below presents some of these changes and the effect they have had on water quality.

Liu (2017) argues that: "Irregular changes in supply-water quality may cause physiochemical and microbiological de-stabilisation of pipe material, biofilms and loose deposits in the distribution system that have been established over decades and may harbour components that cause health or aesthetic issues". While these potential impacts may pose a material hazard to human health, more studies need to be undertaken to objectively quantify these risks. Their recommendation of a more systematic approach to these changes is noteworthy and should be considered by law makers and regulators. It is however clear that careful attention needs to be paid to the source quality, as changes in it have a notable impact on final water quality.

Table 2-1: **Recorded instances of adverse effects on drinking water distribution systems** (Liu, 2017)



Problems	Reasons & Changes	Location	Pipe material	Year	Reference
Discoloration	Source water switch	Tucson, U.S.	galvanized steel; unlined cast iron	1992	(Basefsky, 2006)
Discoloration, high concentration of As, Cu, Fe	Starting up of chlorination	Midwestern U.S.	unlined cast iron	1996	(Reiber and Dostal, 2000)
Discoloration	Source-water switch	Tampa, U.S.	galvanized steel; unlined cast iron; lined cast iron; PVC	2001	(Tang et al., 2006)
Discoloration (red-brown colored water), Pb release	Disinfection strategy switch from free chlorine to chloramine	Washington D.C., U.S.	solder, brass, lead	2004	(Edwards and Dudi, 2004)
Discoloration (red water), high number of iron-related bacteria	Source-water switch	Beijing, China	cast iron	2008	(Li et al., 2010)
Discoloration (brown water)	Source-water switch	North China, China	cast iron	2009	(Wang et al., 2009)
Release of Pb, As, Al	Changes in coagulant in drinking water treatment	Ontario, Canada	lead	2007 −2010	(Kim et al., 2011)
Release of Pb, high concentration of Legionella	Source and treatment switch	Flint, U.S.	lead	2015	(Schwake et al., 2016; Utecht and McCoy, 2016)

2.1.2 Water treatment

The objective of water treatment is to purify raw water (from whatever source), to a point where it can be safely utilised for its intended end-use. This project is concerned with drinking water, thus "safe" is with respect to the microbiological, physical, aesthetic and chemical characteristics as set out in various drinking water standards (SANS 241:2015 in the South African context). The selected method of treatment is most influenced by: source quality, available resources (financial and human capital) and drinking water regulation (Van Duuren, 1997; Schutte, 2006, Edzwald, 2011). Water treatment is generally separated into "conventional treatment" and "advanced treatment", with the former typically cheaper and the latter more expensive (Edzwald; 2011). The conventional treatment process entails four or more of the following treatment steps: coagulation, flocculation, sedimentation, flotation, sand filtration, disinfection and stabilization. Where "advanced treatment" typically uses membrane processes such as: reverse osmosis (RO), nanofiltration (NF), ultrafiltration (UF), microfiltration (MF) or electrodialysis (ED); typically preceded by carbon filtration i.e. granular activated carbon (GAC) and biologically activated carbon (BAC) (Cheremisinoff, 2002; Schutte, 2006).

The performance of treatment plants is the first step in ensuring safe potable water is received by the end users, thus the design, operation and maintenance of treatment works becomes important (EPA, 2010). If poorly treated water leaves the plant, the only recourse is disinfection. An over-reliance on disinfection is not recommended as high doses of chlorine (if it is the selected disinfectant) are then required, and this comes with its own problems. If there is too much residual chlorine the water will have an undesirable odour. The other risk associated with poorly treated water (generally translating to higher concentrations of organic compounds) is the formation of disinfectant by-products, in the case of chlorine these are trihalomethanes (THM's) and haloacetic acid. These include bromoform ($CHBr_3$), chlorodibromomethane ($CHClBr_2$, chloroform ($CHCl_3$) and dichlorobromomethane ($CHCl_2Br$) (Clark, 1998; Schutte, 2006; Edzwald, 2011). Another concern with organic material leaving the plant is the formation of biofilms within the distribution system. These films adversely affect biostability within the distribution network and subsequently increase the risk to the end-user's health. Thus, the optimal design, operation, maintenance and performance of the treatment works is a critical component in the safe supply of drinking water; as the consequences of failure in this step are often almost impossible to undo (DWS, 2013).

2.1.3 Distribution network design

The final step in water supply is the distribution network, this step is typically preceded by some form of water treatment (even if it is just disinfection and or the addition/removal of some mineral or nutrients. The potable water distribution network is typically comprised of (CSIR, 2005):

- Bulk water distribution network (typically provincial / city level infrastructure)
- Bulk water storage (typically provincial / city level infrastructure)
- Intermediate water storage reservoirs (typically town / suburb level infrastructure)
- Reticulation / distribution network (typically street and village level infrastructure)
- End-user installations.

The network is characterised by a complex set of individual civil, electrical and mechanical units of varying size, diameter and function. These typically include: pipes, pumps, valves, meters, hydrants, chambers etc.

The main objective of water distribution networks is to provide access to water. In South Africa "access to water" is clearly defined as: "a reliable supply of 25 litres of potable water per person per day (6kl per household per month), within 200m of a household. At a minimum flow rate of 10 litres per minute and the water should be available on a daily basis" (RDP, 1994). In 2019 this definition was expanded upon to further define basic water supply as: "the provision of a basic water supply facility, the sustainable operation of the facility (available for at least 350 days per year and not interrupted for more than 48 consecutive hours per incident) and the communication of good water use, hygiene and related practices" (DHS, 2019). These definitions give clear guidance on what the bare and legislated minimum is. All South African Water Services Authorities (WSAs) are legally mandated to provide this minimum level of service for free to those who are unable to pay (indigent consumers), but only up to 6kl per household.

The first consideration in the design of any network is the demand. In rural areas and informal settlements, designing to meet demands of 25 to 75 litres per person per day (*l/c/d)* is acceptable. However, in developed areas (typified by house connections) demand can vary from 60*l/c/d* to be as high as 400*l/c/d* (DHS, 2019). Various tools can be used to estimate water demand, Table 2-2 below is Department of Human Settlements (DHS) resource for estimating demand from consumption. Demand is a key input in system sizing, thus it becomes very important to understand the current and future characteristics of the area being serviced.

The use of incorrect demands adversely impacts service levels and has financial implications. If the demand is understated, the system will be undersized. Under sizing results in increasingly poor levels of service (assuming the area continues to develop), these include low pressures and interrupted supplies of water, with the most severe consequence being water shortages for firefighting. In the long term the situation deteriorates and may result in a need for capital investment, that could have been avoided through the use of appropriate demands. The inverse is overstating the demand, in this instance consumers would enjoy good levels of service, but this may come at a high capital cost for utilities (as the utility will have invested in infrastructure that is not all used). The more severe consequences of an oversized network will be long retention times and potentially low velocities. Low velocities result in settlement and long retention times contribute to a high-water age. A key issue with a high-water age is the loss of residual disinfectant; this can in turn adversely affect water quality. From this it is clear that engineers can no longer continue to design systems, without a consideration for water quality (Clark, 1998; 2010; 2012; Blokker, 2008; 2013; AWWA, 2014).

Table 2-2: Typical consumption used to estimate domestic demand in South Africa (DHS, 2019)

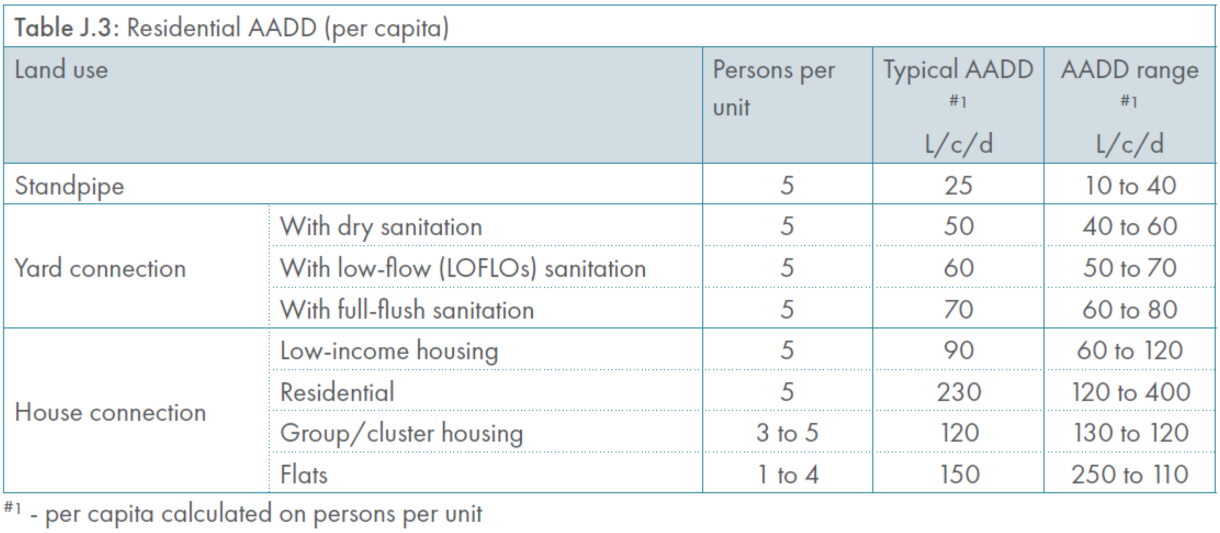


Table J.3: Residential AADD (per capita)

Land use		Persons per unit	Typical AADD #1 L/c/d	AADD range #1 L/c/d
Standpipe		5	25	10 to 40
Yard connection	With dry sanitation	5	50	40 to 60
	With low-flow (LOFLOs) sanitation	5	60	50 to 70
	With full-flush sanitation	5	70	60 to 80
House connection	Low-income housing	5	90	60 to 120
	Residential	5	230	120 to 400
	Group/cluster housing	3 to 5	120	130 to 120
	Flats	1 to 4	150	250 to 110

#1 - per capita calculated on persons per unit

The basis for pipeline design is the conservation of flow and energy, the most commonly utilized mathematical expressions for hydraulic design of pressurized pipelines are the conservation of flow (Equation 2.1) and the Bernoulli Equation (equation 2.2). The conservation of flow is expressed as shown below:

$$Q = vA \qquad (2.1)$$

where:

- $Q = flow\ in\ m^3/s$
- $A = Area\ in\ m^2$
- $v = velocity\ in\ m/s$

The Equation (2.2) is based on the principle of conservation of energy and is outline below.

$$\frac{P_1}{\gamma} + \frac{v_1^2}{2g} + z_1 = \frac{P_2}{\gamma} + \frac{v_2^2}{2g} + z_2 + h_f + h_m \qquad (2.2)$$

where:

- $\frac{P}{\gamma} = pressure\ head$
- $\frac{v^2}{2g} = velocity\ head$
- $z = elevation\ head$
- $h_f = friction\ loss$
- $h_m = minor\ losses$

Knowing the current and future demand the flow (Q) can be determined, the next item is velocity. Velocity must not be so low that sedimentation can occur (not typically a problem in pressurised systems), but also must not be excessively high as this places strain (energy demand. higher capital expenditure on pipe material, major losses when pipe leaks etc.) on the system. The recommended velocity is between 0.6m/s and 1.5m/s (DPW, 2004; CSIR, 2005; DHS, 2019). Once the desired velocity is set, the area can be solved for. Once this is complete, the Bernoulli equation can be used to check and optimize the pipeline.

Part of the pipeline optimization includes catering for some of the following:

- Pipe restraint through thrust blocks where necessary.
- Air valves to keep system operation at optimum levels.
- Scour valves for operation and maintenance related issues.
- Pipe material selection, to minimize opportunity for corrosion and fatigue, whilst considering the following:
  - Cost of pipe material (and cost of installation)
  - availability of selected / potential material
  - fittings and bends (price and compatibility to minimize sacrificial corrosion)
  - International acceptance of proposed material (e.g. asbestos is now illegal)
  - Constructability (special installation requirements, special handling and availability of local skills to perform such functions)
  - Maintenance (testing procedures, availability of local support/expertise for the operation teams
  - Design life of the material, relative to other components of the system (factoring required external long-term protection and maintenance
- Pipe wall thickness, to ensure that both internal and external pressures can be handled by the selected pipe.

- Surge and water hammer mitigation.
- Pipe routing to optimize design.
- Reservoir sizing and location to optimize network design.

Once the design is completed, a review should be undertaken to review risks that typically undermine the hydraulic integrity of the distribution network, and a check undertaken to confirm that satisfactory prevention and or mitigation measures have been taken. These risks are (Van Zyl, 2014):

- Excessive demands
- Reduction in system capacity
- Negative pressures
- Pressure transients
- Excessive pressures
- Low velocities
- Air pockets in the system
- Operational setting errors

Not adequately addressing one of more of the above typically results in one or more of the following: end-user dissatisfaction, damage to the distribution network infrastructure, ingress resulting in contamination of the network water, sediment accumulation and residual disinfectant depletion.

2.1.4 Pipe material

Pipe material is an important structural consideration as it affects: design life, cost, operability and maintenance of the system. Pipe material selection is important for water quality as each material will respond uniquely to: pH, alkalinity, temperature, chlorine and changes in treatment (WHO, 2014). Pipe material selection is influenced by a wide range of factors including (but not limited to): cost of pipe, availability locally, familiarity with material, diameter required, durability and expected service life, material compatibility with other materials to be used in the system, client specification and local standards. In South Africa the most commonly used materials in distribution networks are: asbestos cement (AC) /fibre cement, bitumen, cement and concrete, copper and brass, iron and steel, modified / unplasticised / orientated / polyvinyl chloride (mPVC/uPVC/oPVC/PVC), high / low density polyethylene (HDPE/LDPE), glass reinforced plastic (GRP) and fibre reinforced plastic (FRP) (Van Zyl, 2014).

Pipe material preferences have changed over time, these preferences are influenced mostly by: cost, constructability, research and service life. In recent years, PVC pipes have become more popular for diameters up to 400mm and steel is generally preferred from 450mm to 1800mm (Van Zyl, 2014). Utilities have become increasingly aware of the impacts of corrosion on: hydraulic, physical and water quality integrity of the network. It is through corrosion that: leaching, pipe wall collapse, ingress, increased pipe wall roughness and excessive adsorption and storage of nutrients takes place (Lahlou, 2002; Mains, 2008; Van Zyl, 2014). Tables (such Table

2-3) have become increasingly popular and are part of the set of tools available to assist designers make informed material selection decisions.

Table 2-3: Corrosion properties of common water distribution system materials (Lahlou, 2002)

Distribution Material	Corrosion Resistance	Potential Contaminants
Copper	Resists corrosion well, but is subject to corrosive attack from high velocities, soft water, chlorine, dissolved oxygen and low pH.	Copper
Lead	Corrodes in soft water with low pH.	Lead, arsenic and cadmium
Mild Steel	Subject to uniform corrosion, particularly sensitive to high dissolved oxygen levels.	Iron, resulting in turbidity and red water complaints
Cast or Ductile Iron	Aggressive waters can cause surface erosion.	Iron, resulting in turbidity and red water complaints
Galvanised Iron	Aggressive waters can cause galvanic corrosion of zinc.	Zinc and iron
Asbestos Cement	Good corrosion resistance; aggressive waters can leach calcium from cement.	Asbestos fibres
Plastic	Resistant to corrosion	Source: Larry Mays, 2000

2.2 Operational factors

Once the design and construction are complete, the network is then handed over to the operation and maintenance (O&M) team; and the provision of safe and reliable water becomes their job for the design life of the system. The design and construction of a network are important and have a direct impact on operation and maintenance, but once these are done the O&M team must find ways of ensuring continued success of the network. Hence, a lot of responsibility rests on the O&M team and their actions have a direct impact on the continued supply of safe and reliable water. The team must balance a lot of competing interests and consider the consequences of all their actions on system efficiency (cost and otherwise), water quality and network design life preservation.

O&M teams oversee: water detention time, maintenance of positive pressures in network, optimal flow velocities and pressures, system cleaning/scouring, water quality monitoring, water conservation and demand management, development and adherence to sound operating procedures and incident management and response (CSIR, 2005; Van Zyl, 2014; DHS, 2019). This section will explore some of these items by covering: operation and maintenance, flow velocity and water age.

2.2.1 Operation and maintenance

Optimal operation and maintenance is not a forgone conclusion, and a lot of intentional thought and planning are needed to do this successfully. The American Water Works Association (AWWA) has a five-step process for the effective operation and maintenance of distribution systems. The process involves the following (AWWA, 2014):

a) Intimate knowledge of the distribution system and the definition of potential problems
b) Clear water quality goals complemented by comprehensive performance objectives. Reflected in a water quality monitoring programme that:
    - Continuously monitors source water quality;
    - Compliant product water released from the treatment works;
    - Active monitoring of the distribution network, with an adequate number of monitoring points, an adequate number of samples must be collected at these points and tested at accredited laboratories. The loop must be closed with a clear plan of how the test results are interpreted and used as in input to improve / optimise operation; and
    - Monitoring program must, as a minimum, monitor the most relevant / important determinants of water quality.
c) Constant evaluation, assessment and improvement of programme using information from step 1 and 2 above, to ensure the O&M plans and procedures are up to date and mitigate all significant risks and hazards
d) Implementation and monitoring of the results from step 3 above. Real change, effective management and continuous improvement can only be achieved when plans and procedures are implemented
e) Development of standard operating procedures (SOPs) once the performance standards are finalised. Network specific SOPs are vital, as no two networks are identical, hence context sensitive SOPs are most likely to be successful as they have the system knowledge imbedded in them. Typical SOPs for water distribution networks include but are not limited to (WHO, 2014):
    - Flow maintenance, positive pressure maintenance and surge protection
    - Maintenance of disinfectant residuals to end-user
    - Intermittent supply and flow management
    - Raw water mixing (from differing sources) methodology
    - Reservoir / storage tank maintenance, inspection and cleaning
    - Mechanical fitting (valves, pumps etc) inspection and maintenance
    - Water leakage and management
    - Corrosion protection / prevention
    - Customer connection procedures (including material and fittings selection)
    - Pipeline construction and commissioning

- Water sampling procedure (who, what, where, how, when and frequency)
- Equipment calibration (who, what, where, when, how and frequency)
- Incident management plans
- Incident response plans.

Through the Blue Drop System (BDS), operation and maintenance has been identified as a key area of improvement, in the supply of quality drinking water in South Africa (DWS, 2014). DWS reports that some of the key problems with O&M in South Africa are staff (operators and management skill levels) and the absence of SOPs. When this toxic combination is at play, the proper design of water supply networks is not adequate to ensure quality water supply, thus the necessary attention must be paid to O&M as it is critical in the supply of safe, reliable and good quality water.

2.2.2 Flow velocity

Bernoulli's equation relates flow velocity and pressure. Though flow velocity is a hydraulic parameter, it has a direct impact on water quality. As previously discussed, ideal flow velocities are between 0.8 m/s and 1.2 m/s (DPW, 2004). Very low velocities have various negative implications; the most common being sediment deposition (Schneider *et al.*, 2010). At velocities lower than 0.6 m/s, heavy particles can start to settle in a pipeline. Depending on the characteristics of the sediment, the condition of the pipe and how long this lasts for; a loss in pipe area can be experienced. This would in turn reduce the hydraulic capacity of the pipe and erode system efficiency. The biological consequences of low velocities relate to biofilm accumulation. Research has shown that most biofilm accumulation takes place in low velocity conditions, this may be due to the fact that higher flow result in higher sheer stresses on the pipe surface, thus limiting sedimentation and biofilm thickness (DPW, 2004; Chu, 2010; Clark, 2012; AWWA, 2014). Lower velocities are often linked to lower pressures and a higher water age, utilities need to make sure that these do not adversely affect water quality.

The other end of the spectrum is excessively high flow velocities. High velocities are often incorrectly assumed to have no adverse impacts on water quality, this is not completely true; as systems with higher velocities often experience negative pressures in the event of pump trips. Negative pressures are undesirable as they pose a risk to the pipe's integrity and also create opportunities for ingress into the system, particularly around joints and weaker elements of the system (Schneider *et al.*, 2010). Another issue associated with very high velocities are the corresponding high energy requirements (this is not an issue for gravity systems). Lifecycle costing and operation and maintenance evaluations show that energy and labour are among the highest costs in the operational life of infrastructure, thus any savings on these would be to the benefit of the asset owner and users. The other significant issue is that of high pressures in the system. These increase the risk and consequences of leaks and pipe bursts/breakages. Pipe joints are the weakest points in points in network, if restraints are not adequate for the resultant forces (due to the pressure), leaks may occur. Another concern is the quantity of water that will be lost in the event of a leak; as the network starts to age. Many utilities have resorted to pressure management programs to mitigate these risks and these have yielded positive results (DPW, 2004; CSIR, 2005; DHS, 2019).

2.2.3 Water age

Water age refers to the time it takes water to get from the treatment works to the end-user. Water age of the same system can vary as it is affected by: distribution network operation, system design, water demand and water production rate (Blokker, 2008; Cruickshank, 2010; Blokker *et al.* 2013). Distribution networks with a high-water age are typically over-designed; to accommodate future demand, provide sufficient time for repairs without supply interruptions and provide sufficient water for firefighting. Though these are beneficial and good considerations, they can negatively affect water quality, as a high-water age is typically associated with: residual chlorine losses, increased risk of bacterial regrowth, higher levels of biofilm growth, increased risk of disinfectant by-product formation and a generally greater risk of contamination (Lahlou, 2002; EPA, 2005; Mains, 2008; WHO, 2014; AWWA, 2015). The American Water Works Association (AWWA, 2015) recommends a water age of less than seven days.

Water residence time and hydraulic design considerations, though often in conflict, can be managed to ensure the sustainable delivery of quality water. This requires the careful operation and maintenance of the system. This typically includes (but is not limited to): careful planning to achieve mixing; this can be done by scheduling pumping to ensure deep cycling and ensuring that all storage facilities are optimally configured to ensure high water turnover rates and continued mixing. The water chemistry also requires careful consideration and the use of innovation, utilities can implement shock chlorination, increase chlorine residual (by increased dosage or dosage points) and provide aeration where possible (Lahlou, 2002; EPA, 2005; Mains, 2008; WHO, 2014; AWWA, 2015).

2.3 Water quality factors

Water quality is affected by biological, chemical and physical factors. These affect water quality individually (in isolation), but also in combination. An example would be over-sized storage facilities (physical), with extended storage times, resulting in a high-water age; resulting in the reduction of residual chlorine (chemical), which in-turn results in bacterial and or pathogen growth (biological), all of which adversely affect water quality. Though the over-sized storage facilities may have been designed with good intentions e.g. looking at future population growth and trying to maximise savings by minimising future expenditure / obviating the need for future upgrades. This well intended physical action would result in unintended chemical and biological consequences. This complex subject area is covered by looking at: SANS 241: 2015, micro biological parameters, temperature, pH, disinfection and the aesthetics of treated drinking water.

2.3.1 SANS 241:2015

SANS 241 (2015) is the national drinking water standard in South Africa and derives its power and authority from its recognition by the Water Services Act (Act 108 of 1997). The standard is split into two parts, part one deals with (SANS 241, 2015):

- The provision of a quantitative specification of acceptable drinking water, with respect to: chemical, physical, microbiological and aesthetic parameters at the point of delivery to the end-user
- The qualitative definition of what acceptable water is, outlined as water that poses an acceptable health risk over the lifetime of consumption. Lifetime consumption is defined as an average consumption of two litres per day for 70 years, by individual weighing 60 kilograms.

Part two of the standard intends to prescribe how the numerical limits provided in part one can be achieved. By describing programmes for routine monitoring and risk assessment. Part two highlights the following as key management actions, that are a minimum if safe drinking water is to be provided (SANS 241, 2015):

a) “Water quality risk assessment: assessment of risk from intake water to treatment works to point of delivery;
b) Routine monitoring;
c) Response monitoring;
d) Verification of water quality;
e) Water safety planning: incorporating outcomes of all the above”.

The standard is comprehensive and in-line with international norms and standards, but water quality compliance is still a challenge in South Africa. The standard clear outlines is scope and further states that SANS 241 (2015) is: “not intended to provide a comprehensive water management plan, which is required for the implementation of a water safety plan that addresses related issues such as water quantity, finance and maintenance”. It seems to be unclear who’s responsibility it is to provide: guidance, financing, monitoring and evaluation of these comprehensive water management plans. The poor scores in the blue drop assessment, are largely attributed to a failure in this regard (DWS, 2014).

2.3.2 Micro-biological considerations

Nutrients can never be completely removed in treatment and in the distribution network, but the objective is to limit the available concentration; so as to avoid the challenges associated with high nutrient concentrations in water. The presence of nutrients such as phosphorous, nitrogen and organic carbon, in drinking water results in a loss of biostability and the formation of biofilms (Momba *et al.*, 1999, 2002; Kerr, 2003; AWWA, 2005). Nutrients in the distribution network have varying sources, consequences and implications for water quality.

Research has shown that micro-organisms are a result of break-through in treatment or survivors of primary disinfection. Organisms that have survived primary treatment (typically chlorination), have been shown to have the potential to become more resistant to secondary disinfection, than strains of the same species that have not gone through primary disinfection. The disinfectant resistant species are then likely to reproduce in the distribution system (Kruger, 2001; AWWA, 2005; Berry *et al.*, 2006; Mains, 2008). Other sources of micro-organisms in the distribution system are: cross connections with contaminated networks, ingress as a result of negative pressures or physical breaks in the network (loss of physical integrity) or growth due to inadequate disinfectant residuals, high temperatures or excessive retention times (Hallam, 2001; Melo *et al.*, 1997; Mains, 2008).

A growing concern for water utilities is the formation of biofilms. Biofilms are a collection of organic and inorganic bacteria / material collected on a surface (LeChevallier *et al.*, 1987; Mains, 2008). Biofilms present various water quality and operational challenges for water utilities. With respect to water quality, biofilms can be the primary cause of disinfectant residual losses, loss of biostability evidenced by high micro-biological growth (leading to pathogenic and bacterial growth and possibly viral attachment post contamination events), losses in dissolved oxygen and taste and odour problems (Hallam, 2001; Kerr, 2003;)

From an operational perspective, biofilms make the development of reliable chlorine decay rates very difficult, resulting in constantly under or over-dosing of chlorine. Biofilms have also been shown to be more aggressive towards certain types of material, cast iron for instance has been shown to display increased corrosion, roughness and loss of physical strength (Kerr, 20003), this has serious implications for distribution networks, as many fittings (valves and pipe specials) are still made from cast iron, though pipe are generally concrete, steel or plastics. Another significant impact of biofilms is on pipe roughness and the velocity profile. The optimal design of pipelines requires a reasonable estimation of friction co-efficients, to accommodate for roughness over the design life of the pipe network. The unpredictability brought on by biofilms has made most of these empirical co-efficients highly inaccurate. Energy is among the highest costs over the service life of a pipeline, thus errors in this regard have significant financial costs for utilities, owners and ultimately end-users as they must pay for the water (Characklis, 1973; Lethola *et al.*, 2006; Lambert *et al*, 2009).

2.3.3 Temperature and pH

The temperature of water has a direct influence on the quality of water. Temperatures will vary daily and seasonally. Hence, the necessary care to mitigate the impacts of temperature variations on water quality are necessary. The rate of chemical reactions, solubility and viscosity are severely affected by temperature (Schutte, 2006). Higher temperatures are associated with higher chemical reaction rates, translating to a more rapid consumption of the disinfectant residual, hence disinfectant dosages should be higher in summer and lowered again in winter (Schutte, 2006). Furthermore, higher temperatures typically favour the development of biofilms. Research suggests that this is not only true for the quantity of biofilm, but also the diversity thereof

(AWWA, 2014). Table 2-4 below presents a quantitative assessment of the relationship between temperature, pH and disinfectant residual requirements. Temperature and pH are closely related and have an impact on each other, with regards to disinfectant residual they are directly proportional.

The pH is a measure of the concentration of hydrogen ions in water, and it gives an indication of how acidic or basic the water is. In addition to disinfectant residual, pH also affects the network material and hydraulic integrity. Very low pH is acidic, leading to corrosion and very high pH is basic, leading to scale formations and hydraulic capacity reduction in the system. Thus, pH and temperature need to be carefully monitored in the delivery of quality drinking water (Van Duuren, 1997; Schutte, 2006; Edzwald, 2011).

Table 2-4: Disinfectant concentration values required to achieve inactivation of *Gardi Lambia* with free residual chlorine at different temperatures and pH values (Schutte, 2006)

Free available chlorine – 2*mg/l*	**pH**	**Temperature ºC**			
		0.5	5	10	15
		Ct values			
	6	60	40	30	20
	7	90	60	40	30
	8	130	90	60	50
	9	170	120	90	60

2.3.4 Disinfection

The provision of drinking water that is: microbiologically, physically, chemically and aesthetically compliant requires a multi-barrier approach. The last barrier in this approach is disinfection. Disinfection is concerned with the inactivation or destruction of pathogenic micro-organisms, with the intention of reducing the probability of infection from water-borne diseases. Disinfection should not be confused with sterilization, the objective is sterilization is the destruction of all forms of life in water (Schutte, 2006). Research has shown that; regardless of how well the water is treated at the water treatment works, the water quality will deteriorate within the drinking water distribution network (Kruger, 2001; Lahlou, 2002; Helbling, 2006; Biyela, 2010; Edzwald, 2010). Thus, meaningful disinfection is concerned with the mechanisms and drivers of this deterioration. The problem of water quality deterioration can be approached in many ways, one of the more well documented and pragmatic ways is through; residual disinfectant maintenance and disinfectant by-product formation, analysis.

Residual disinfectant maintenance is critical in ensuring that pathogen free drinking water is supplied to the end-user. Chlorine decay is influenced by a wide range of factors, the most significant being: microbiological activity, water age, pH and temperature (Kruger, 2001; Schutte, 2006, Edzwald, 2010). Temperature is noted as having the most severe impact on disinfectant decay, as higher temperatures corelate with increased bacterial growth and disinfectant residual decay (AWWA, 2005; Schutte, 2006). Increased water age and pH are also linked disinfectant decay (Lahlou, 2002; Edzwald, 2010). As aforementioned, disinfection is an integral part of the multi-barrier approach in the supply of safe drinking water, but is not a substitute for the other treatment steps. Thus, if proper treatment has not been undertaken, disinfection efficacy will be significantly reduced (AWWA, 2005). Research has shown that chlorine demand is a suitable surrogate indicator for microbiological activity. In simple terms there is a direct correlation between chlorine demand and micro-organism concentration (Helbling, 2006). What is peculiar is that chlorine resistant organisms also exert a chlorine demand, it hypothesized that these organisms defend against oxidation by allowing for oxidation only at the cell wall, thus lowering disinfectant concentration at the periphery, whilst the nucleus is intact (Helbling, 2006).

Poor treatment, as evidenced by the presence of higher than desirable concentrations of nutrients, has other undesirable consequences, besides residual disinfectant decay; namely disinfectant by-product (DBP) formation. DBP are organic compounds that are undesirable, but produced as a side-effect of drinking water disinfection and oxidation (Spellman, 2003; Schutte, 2006; Edzwald, 2010). The exact nature and characteristics of the DBP is influenced by the type of disinfectant used. There is a growing list of disinfectants being used, but for the purposes of residual disinfection, chloramines are most commonly used. This is because chloramines offer the greatest level of efficacy as they are the most stable and enduring in the distribution system, though other mechanisms are more potent, they are not as stable. Thus, the most commonly observed DBP's are: trihalomethanes (THM's), haloacetic acids (HAAs) and haloacetonitriles (Schutte, 2006; Edzwald, 2010).

2.3.5 Aesthetics

Acceptable drinking water quality is not only defined in terms of microbiological, physical and chemical determinants but also aesthetics (SANS 241, 2011). Aesthetics is a broad term used for: water colour, conductivity, odour, taste and total dissolved solids. Though these aesthetic indicators are not necessarily indicative of any health risk to the consumer, they are often indicators or early warnings of problems in the system (Edzwald, 2010; AWWA, 2014).

End users provide valuable feedback on the aesthetic quality of water, this feedback should be recorded, patterns noted and used as an input in planning and operations activities (Spellman, 2003, Schutte, 2006). The aesthetic indicators have varying root causes and each of these must be understood if the issue is to be resolved. The most significant aesthetic indicators are discussed below.

One of the most easily observed aesthetic indicators is colour. The colour of water is influenced by various factors, but it is helpful to distinguish between apparent and true colour. Apparent colour is a result of suspended colloids, where true colour is the result of dissolved chemicals. Filtration is used to distinguish between true and apparent colour. The other easily observable aesthetic parameters are taste and odour. Taste and odour are generally grouped together, as one can have an influence on the other. A general broad characterization is that taste is influenced by inorganic constituents, while odour is influenced by organic constituents (Spellman, 2003; Edzwald, 2010). Taste and odour are very difficult to measure, as such SANS 241:2015 sets the standard limit as “inoffensive”, which is highly subjective. A common issue around odour is chlorine, excessive chlorination results in a “sharp” odour, this feedback can be used to utilities to confirm that they are not over-dosing chlorine. A change in taste and odour could also be caused by backflow in cross connections or general contamination in the system; this feedback can be used by utilities to ensure that the water quality is not compromised (AWWA, 2014).

2.4 Water quality modelling

Water quality parameters are generally divided into chemical, microbiological and physical, but water quality is influenced by all these individually and in combination. Effective management of water quality in distribution systems, requires a firm grasp of the key water quality determinants and how they interact with each other. For simple distribution networks, the task is manageable with a robust operation and maintenance programme; the challenge comes with larger complex networks, where the number of variables for consideration makes effective water quality management almost impossible without some strategy or plan. Water quality modelling is invaluable in instances where the number of variables affecting water quality can only be optimally managed through the use of modelling (Woolschlager, 2000; Zhang *et al.*, 2004; Technau, 2006; Culligan, 2015). Many models have been developed over the years and they have provided invaluable aid to utilities and operators.

Modelling is a complex craft, requiring a good grasp of applied mathematics, statistics and other relevant sciences (microbiology, chemistry, hydraulics etc). Because water is influenced by a combination of factors, the most helpful models are those that consider complex interactions and combinations between various elements and determinants and express these relationships using mathematical equations with some co-efficients, state variables and constants (Reichert, 1994; Bergdoll *et al.*, 1995; Technau, 2006). Hydraulic modelling has been utilised for decades, but water quality modelling is not as advanced (Rossman, 2000; Woolschlager, 2000; Biyela, 2010). It is not practical to model all variables, thus surrogate indicators are most commonly used and bacterial growth is a commonly used indicator, as there is direct correlation between water quality deterioration and bacterial growth in distribution systems (Woolschlager, 2000; Technau, 2006; Biyela, 2010). A couple of the most commonly used biological regrowth models are presented below (Rubulis *et al.*, 2007).

2.4.1 Biological Accumulation Model - BAM

Montana State University's Centre for Biofilm Engineering developed the Biofilm Accumulation Model (BAM). The basis of the model is conservation of mass; which is applied to bulk liquid and biofilm (Stewart, 1994). The model uses assimilable organic carbon without the addition of inorganic nutrients (AOC) as an input parameter; model outputs are heterotrophic plate count (HPC) bacteria and coliforms AOC and biofilm thickness. The selected growth limiting nutrient is carbon. The model is not linked with any hydraulic model; thus, some work would need to be done before the model could be used to model water quality in distribution systems. BAM also appears to be proprietary intellectual property of the Centre for Biofilm Engineering at the University of Montana (Camper, 1994; Steward, 1994; Camper, 1996; Rubulis *et al.*, 2007).

2.4.2 Computer Program for the Identification and Simulation of Aquatic Systems - AQUASIM

AQUASIM was developed by the Swiss Federal Institute for Environmental Science and Technology (EAWAG). The primary objective of the program is to aid practitioners in the identification and simulation of aquatic systems. The program is a suitable tool for biofilm simulation as it includes a one-dimensional multi-substrate and multispecies biofilm. AQUASIM is capable of substrate removal calculation in biofilm, biofilm thickness development and the development of any other substrates and microbial species in the bulk fluid and biofilm over time. The program is linked to a hydrological program (SWIM), but no hydraulic model that would enable water distribution network related analysis. AQUASIM is open source and has been extensively calibrated and validated. However, this calibration and validation has only been on: reactor compartments (mixed, biofilm and advective-diffusive), saturated soil columns, river sections and lakes, but not drinking water distribution networks (Reichert, 1994; Reichert, 1998; Wanner *et al.*, 2004).

2.4.3 SANCHO Model

The SANCHO model was developed on the basis of experimental microbiological studies on French distribution systems. The model considers the dynamics of bacteria and biodegradable organic carbon (BDOC) on distribution networks as the growth limiting nutrient. Temperature, chlorine, BDOC are classified in three classes of biodegradability and free bacteria are the input parameters along with nineteen (19) constants that are obtained experimentally. The output parameters are: chlorine profile, BDOC profile, reduction thresholds for BDOC and fixed and free bacteria. The model calibration and validation were undertaken on two full-scale distribution systems in French suburbia and later four Canadian and three French distribution networks. The model is proprietary and is thus, not available as free-ware and is not linked to any hydraulic model (Servais *et al.*, 1995; Rubulis *et al.*, 2007).

2.4.4 PICCOBIO Model

The PICCOBIO model was developed in France by Dukan, Levi, Piriou, Guyon and Villon. The model proposes a mechanism for the study of the behaviour of bacterial biomasses in drinking water distribution networks. The model utilises temperature, chlorine and nutrients as input parameters, with carbon as the growth limiting nutrient. The output parameters are fixed and free bacteria and the identification of high-risk zones. The software is proprietary, which partly explains the slow up take by industry. PICCOBIO is linked with the hydraulic model PICCOLO, which is capable of predictive mapping of water quality. The two models used together provide a dynamic forecast of water quality by considering the physiochemical and biological variations (at the intake to the distribution network) and residence time (Dukan *et al.*, 1996; Rubulis *et al.*, 2007).

2.4.5 Zhang *et al* Model

Zhang *et al.* (2004) are among a long list of researchers who have developed a bacterial regrowth model. Their model caters for microbial processes of: free and attached growth, detachment, endogenous respiration and chlorine accomplished inactivation (Zhang *et al.*, 2004). The model also considers hydraulics and its results were comparable to those obtained through EPANET. Model input parameters are: temperature, chlorine, BDOC, free bacteria and biofilm bacteria; with the growth limiting nutrient being Carbon (Technau, 2006). The model calibration and validation were undertaken through experimental data and experiments. The model is linked with a hydraulic model and is open source.

2.4.6 Comprehensive Disinfection and Water Quality Models - CDWQ

The Comprehensive Disinfection and Water Quality Model (CDWQ) was first developed by Woolschlager (2000). The CDWQ was unique in that it modelled the hydraulic, chemical and microbiological processes affecting water quality in water distribution networks. This in a time where most models considered one or two of those processes or had other shortcomings, that ultimately affected the reliability of the model (Woolschlager, 2000). The model offered a complete hydraulic solution for a full-scale distribution network; accounted for disinfectant decay through chloramine and free chlorine chemistry subroutine and biological growth processes were accounted for through consideration of fixed and suspended heterotrophs and nitrifiers (Woolschlager, 2000).

Further work was done on the model, by Biyela (2010); Expanding the Comprehensive Water Quality and Disinfection Model, by accounting for nitrification, denitrification, oxygen depletion and *N. fowleri* within drinking water network and biofilms (Biyela, 2010). This work led to the creation of the CDWQ-E (Expanded Comprehensive Disinfection and Water Quality) model.

In 2015 Culligan undertook further work on the CDWQ-E model, developing version 2 of the model (CDWQ-E$_2$). Version 2 updates the microbial growth and residual disinfectant decay processes and automates some of the manual processes required to run the original CDWQ and CDWQ-E models (Culligan, 2015).

However, none of the previous research undertook a comprehensive hydraulic calibration of the model. Woolschlager (2000) performed a very high-level calibration, due to time and data constraints. Hydraulic calibration was outside the scope of Biyela's (2010) development of the CDWQ-E and Culligan (2015) used data from Woolschager (2000) to merely demonstrate that the CDWQ-E2 model was capable of analysing a real distribution network. Both Woolschlager (2000) and Culligan (2015) recommend that an in-depth hydraulic calibration of the model should be undertaken.

2.4.7 EPANET 2.0 and EPANET Multi-Species Extension (MSX)

The EPANET 2.0 hydraulic modelling tool is generally widely known and utilised for various applications, it however has limitations as it can only track one water quality parameter at a time (EPA, 2008). The tracking of a single parameter is not optimal as water quality is affected by multiple parameters acting individually and or jointly (EPA, 2008; Biyela, 2010). Noting this limitation, the Environmental Protection Agency (EPA) developed the EPANET Multi-Species Extension, which has the capability of modelling multiple interacting chemical species (EPA, 2008).

2.5 Previous studies

The view that: former and current practices of water quality monitoring, though having served society well, are in need of a review and enhancement is not a local idea. This is evidenced by studies conducted in South Africa, on the African continent and abroad. Selected cases are presented below.

2.5.1 Local (South African) studies

A 2001 Water Research Commission (WRC) study investigated water quality deterioration in potable water reservoirs relative to chlorine decay (Kruger, 2001). Kruger (2001) notes that water quality deteriorates within the distribution network and asserts that reservoirs are the last and most strategic points at which water quality can be modified before reaching the end-user. The study notes that chlorine is a useful surrogate indicator for water quality, thus investigation into the causes of its decay would prove useful in water quality maintenance undertakings. The study concluded the following (only significant items noted):

- Hydraulic flow patterns play a major role in chlorine loss and water age. Thus, reservoirs should be carefully designed and operated to ensure plug flow conditions are maintained;
- Chlorine loss / decay is influenced by a variety of factors, but the most significant were found to be temperature and water age (detention time); and
- Some organisms survive disinfection, these organisms then become inoculum for aftergrowth in the distribution system and biofilms. Such organisms can become resistant to disinfection.

The study made the following recommendations for future work:

- Development of a more reliable chlorine decay test method and model, which incorporates influence of temperature and initial chlorine concentration;
- Use of computational fluid dynamics (CFD) models to optimise design and operation of reservoirs;
- Development of guides that optimise operations of distribution networks, that minimise water quality deterioration and are cost-effective; and
- Investigation into differences between small and large distribution networks and small rural storage tanks and large reservoirs and how water quality can be optimised in each of these.

In his MSc Thesis titled: “Predicting Water Quality in Bulk Distribution Systems”, Rust (2014) notes that: current master planning and design standards (conventional wisdom) could have adverse effects on water quality. This is because little to no attention is paid to the adverse effects of extended residence times on water quality. He further notes that additional design criteria, specifically focused on water quality, needs to be added to the existing South African design standards / codes. The research is specifically focused on disinfectant decay and is a first order attempt at disinfectant dosage and location optimisation. A case study is undertaken by developing a detailed design for disinfection of the Umgeni Water Central and Coastal Regions, looking at seven (7) supply systems. Rust (2014) makes the following recommendations for future work:

- Computational Fluid Dynamics (CFD) modelling can be used to more accurately model reservoir hydraulics and thus improve accuracy of the hydraulic model;
- The development of a model that not only predicts disinfectant decay, but also the potential of contamination. This integrated risk approach could be utilised in identifying high risk areas and the risk could be proactively managed; and
- Development of models with additional modelling capabilities, specifically offering more accuracy and flexibility in the determination of disinfectant decay rates.

2.5.2 Continental (African) studies

In the study: “Water Distribution System Modelling for Water Quality Management: A Case Study of Kumasi in Ghana”, Nyarko *et al.* (2006); recognised the importance of mathematical models in the effective management of drinking water quality in distribution networks. The

EPANET software was utilised to model and predict spatial and time variations in water quality, with the ultimate goal being identifying the most optimal location for booster chlorination. At the time of the study; Kumasi was spread over a land area of 65km^2, with a networking serving approximately one million people, with a water works with a production capacity of 94 500m^3 per day.

The study was a success, as the model was able to reliably predict water age and residual chlorine concentrations at selected testing/sampling points. Verified results from the model were used to delineate water quality zones; each zone was associated with a unique chlorine residual concentration. Zones with lower than desirable chlorine residual concentrations (0.2mg/l), were identified as high priority areas and were recommended for more frequent testing and flushing; to ensure acceptable water quality was maintained. The study also noted the potential benefits of using water quality model outputs, as inputs to water quality monitoring programs and other similar water quality improvement initiatives.

In their study: Monitoring Residual Chlorine Decay and Coliform Contamination in a Water Distribution Network of Kampala, Uganda; Ecuru *et al.* (2011) showed significant water quality deterioration in a small portion of a network. The study objective was the evaluation of residual chlorine decay and relating this decay to recontamination risk. The study assessed the water quality of five storage reservoirs and four consumer taps; monitoring: temperature, pH, turbidity, colour, ammonia and iron levels. Results showed notable chlorine decay in the distribution network, with two reservoirs and most consumer taps having residual chlorine concentrations that are lower than the WHO guidelines. The study concluded that recontamination risk increased with: long water retention times in reservoirs, supply/flow interruptions and infrastructure age (reservoir and distribution network); subsequently the study recommends the remediation of these risk factors through improved operation and maintenance practices (Ecuru, 2011).

In their study: Assessment of Changes in Drinking Water Quality During Distribution: A Case Study of Area 25 Township in Lilongwe, Malawi; Kosamu et. al (2013) studied the water quality at the tail of works, one storage tank and eight consumer taps in a section of Lilongwe. The study tested: turbidity, pH, faecal coliforms, manganese, lead, zinc and residual chlorine. Tests were taken three times over a period of five months (July 2011 to November 2011), this period coincided with the dry season. All results from all samples showed that all tested determinants were within acceptable limits (World Health Organisation [WHO] and Malawi Bureau of Standards [MBS]), though there were changes in the water quality. The study acknowledges that more tests could have been taken and that it would be instructive to take samples in the wet season too (Kosamu, 2013).

In their study: Assessment of Physio-Chemical and Bacteriological Quality of Drinking Water at Sources and Households in Adama Town, Oromia Regional State, Ethiopia; Eliku and Sulaiman (2015) undertook a total of 107 samples. The 107 triplicate samples were from: 52 pipe water samples, 52 strategically selected households; with the remainder coming from the head and tail of the treatment plant and the other from a reservoir. The study focused on: total coliform (TC), faecal coliform (FC), faecal streptococci (FS), temperature, pH and turbidity. Results showed that water quality deteriorated within the network, with 82.7 and 92.3% of pipeline samples complying and 55.8 and 71.1% of household samples complying. The study showed a positive correlation between TC and temperature, as sample compliance dropped with increasing temperatures. The study was undertaken between May 2008 to July 2008, but acknowledges the need for sampling in both the dry and wet season. Adama is one of the largest and most populated towns in its district (Oromia National Regional State), with a land area of approximately 13 000 hectares and a network of approximately 323km (Eliku and Sulaiman, 2015). These water quality failures will only become worse, with increasing populations and ageing infrastructure; a water supply system such as this could benefit immensely from drinking water quality monitoring and prediction models.

2.5.3 Global studies

In their study: "Online Modelling of Water Distribution Systems: A UK Case Study", Machell *et al.* (2010), note that modelling capabilities haves been underutilised; often used as tools that aid in reactive interventions. Through their case study they show how up to date asset information would assist in using models for proactive distribution network management, and how this proactive approach would in turn assist in minimising any adverse impacts on the end user.

Machell *et al.* (2010) notes that modelling capability has historically been stifled by the cost of continuously monitoring and reporting distribution network data, but argues that this is changing and communication technology is making it feasible to transfer data directly from distribution network instrumentation to the network operators. The study showed the value of online monitoring, as it allows utilities to proactively manage distribution networks, improving efficiencies, optimising operation, monitoring service levels. All these advances ultimately benefit the end user and are vital for any utility that esteems continuous improvement.

In their study: "Residual Chlorine Decay Simulation in Water Distribution System". Nagatani *et al.* (2008), simulated chlorine decay for section of the water distribution system of Osaka Municipal Waterworks Bureau in Japan. The study used field sampling data collected over an extended period, to develop an EPANET 2 simulation algorithm. First order bulk decay coefficients and their relationship with temperature along with zero order pipe wall reaction coefficients, were investigated through tests and data. Results showed that the model was reliable and the modelled residual chlorine concentrations were closely correlated to those from tests at specific temperatures. The study noted that an application of the model to the entire Osaka distribution network would require further studies. These studies would be used to develop the relationship between: water temperature, wall reaction coefficients, pipe friction coefficients for pipes of different ages and lining material(s).

2.6 Summary and need for further research

From the literature reviewed the following research opportunities are identified:

a) Water quality can and does deteriorate within drinking water distribution networks. This deterioration is a result of: structural, operational and water quality factors acting independently and in combination. The characteristics of this deterioration are therefore unique for every network to the degree that the factors vary between networks.

b) The complex and multi-variable nature of water quality management makes it very difficult to effectively manage water quality without the use of deterministic mathematical modelling tools. However, for these tools to provide reliable (useful) results, calibration and validation are of critical importance.

c) Previous studies have looked at the various elements of water quality modelling, but very few models have attempted to develop a comprehensive water quality modelling tool, that looks at multiple water quality vectors. Of the few comprehensive models that have been developed, even fewer have been calibrated at a pilot scale and then used as a tool in water quality management (using field data).

From the above, it is reasonable to conclude that further work in this area may add value to all.

3. Method

3.1 Introduction

In this study the water quality modelling of a drinking water distribution network was undertaken, for a portion of a utility's network. The methodology gives a brief outline of the activities undertaken and their sequencing, towards the realisation of the study objectives as summarised in Figure 3-1 below.

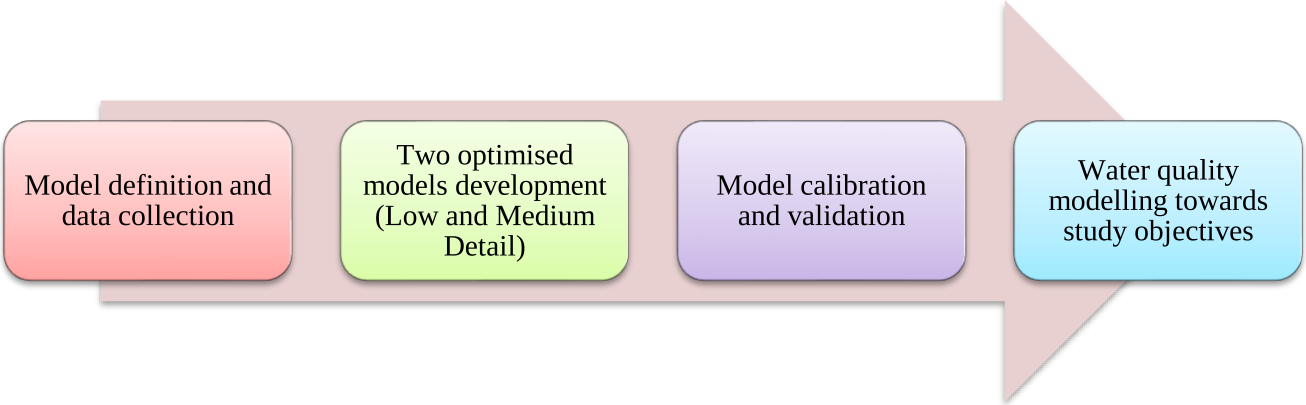


Figure 3-1: Overview of study methodology

The complexity of drinking water quality modelling necessitated the meticulous undertaking of a calibration and validation exercise. The calibration was undertaken following international best-practice, and a great deal of effort was invested in this step as, a properly calibrated model is a perquisite for meaningful water quality modelling. South Africa has no drinking water distribution network hydraulic model calibration standards or guidelines (at the time of submission of this research report), thus tested methodologies and best practise guidelines from other regions were sought out and used.

The detailed hydraulic calibration followed Ormsbee *et al.* (1997) and the Water NZ (2021) guideline, as presented in Figure 3-2 below. A key objective in distribution network calibration for water quality modelling is: the development of an instrument that will improve the management and operation of the network and associated infrastructure, by creating a model that closely represents the system being modelled (Hirrel, 2008; AWWA, 2012; Ostfeld, 2012).

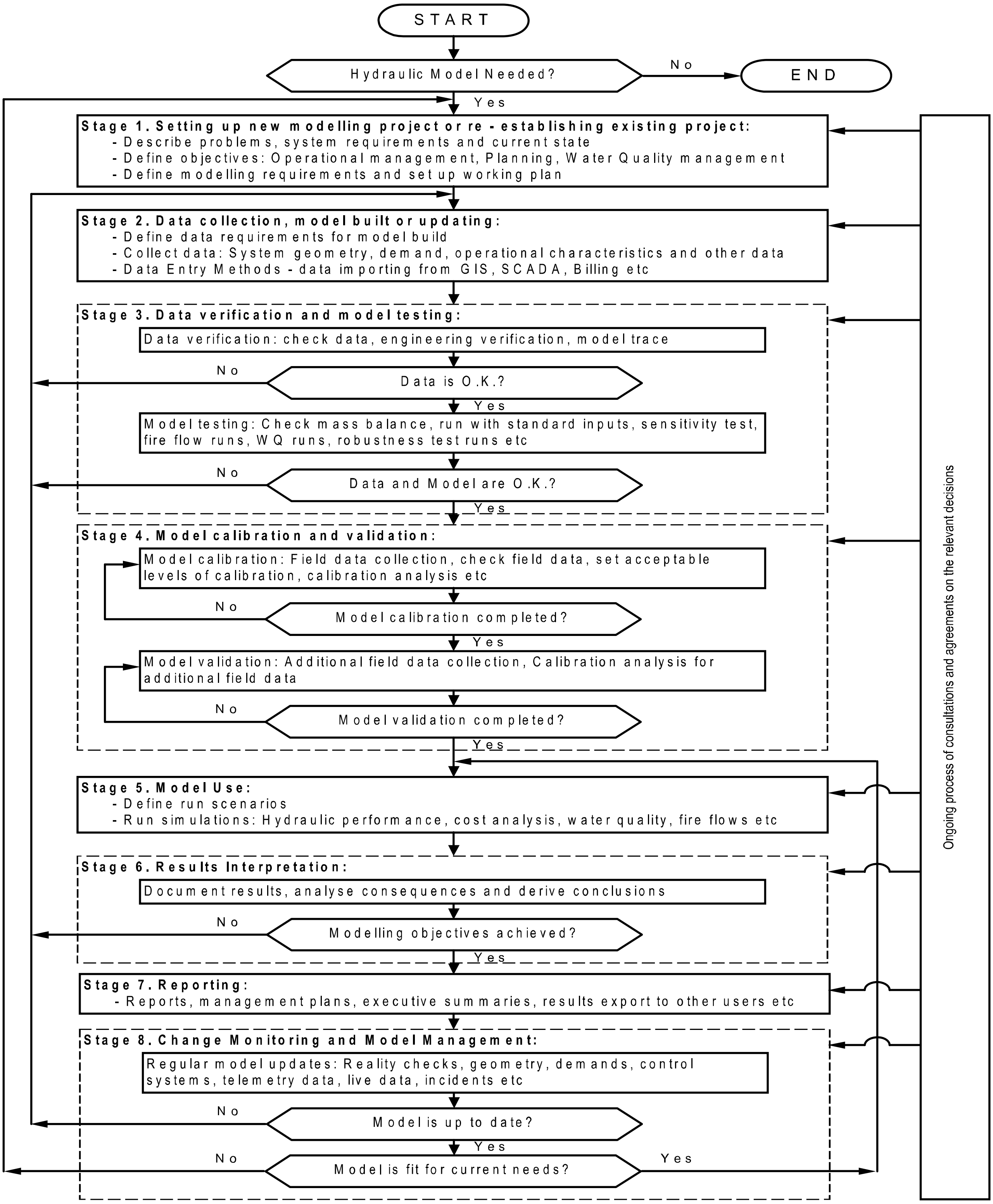


Figure 3-2: Overview of hydraulic model calibration methodology (Water NZ, 2021)

3.2 Model definition

The first step in the calibration of the model was to determine the intended use of the model. This exercise was important as calibration can be a cumbersome undertaking, thus clarity of purpose from the outset was vital and saved valuable resources, namely: time, effort and money. Models are typically used for: planning (short and long term), design, operations and water quality assessments (Ormsbee *et al.*, 1997; Trasky, 2008; Rivera, *et al.*, 2010; Seyoum *et al.*, 2017; AWWA, 2012; Walski, 2017). As stated in the research objectives, this study sought to undertake an evaluation of the EPANET 2.0 for the purposes of meaningfully modelling a selected drinking water distribution network.

As far as hydraulic modelling goes, there are three types of hydraulic analysis namely: Steady State (normal flow conditions), Fire Flow (high flow conditions) and Extended Period Simulation (EPS). Selection of any these is guided by the intended purpose of the model; Table 3-1 below provides a brief summary of some key considerations when selecting an analysis type and the associated levels of input data accuracy.

Table 3-1: Summary of hydraulic simulation options (Adapted from: ECAC, 1999)

Hydraulic Analysis Type	**Input Data Volume**	**Input Data Accuracy Requirement**	**Water Quality Modelling Suitability**
Steady State	Medium	± 10%	No
Fire flow	Low	± 10%	No
Extended Period Simulation (EPS)	High	± 5%	Yes

From the table above, it was clear that an EPS was the most suitable type of analysis for this exercise. This was appropriate, as the model’s intended use and hydraulic analysis type are often directly related (Ormsbee *et al.*, 1997; Woolschlager, 2000). Related to the use and analysis type are the data requirements; this also speaks to the type, quantity (volume) and accuracy of data required. These insights were considered in the subsequent steps and activities.

3.2.1 Study area definition

The project study area was influenced by the research undertaken by Useh (2017) from a water quality data perspective and the available hydraulic input data from Johannesburg Water. Applying this criterion the selected area was the Northcliff Zone, as depicted in Figure 3-3 below.

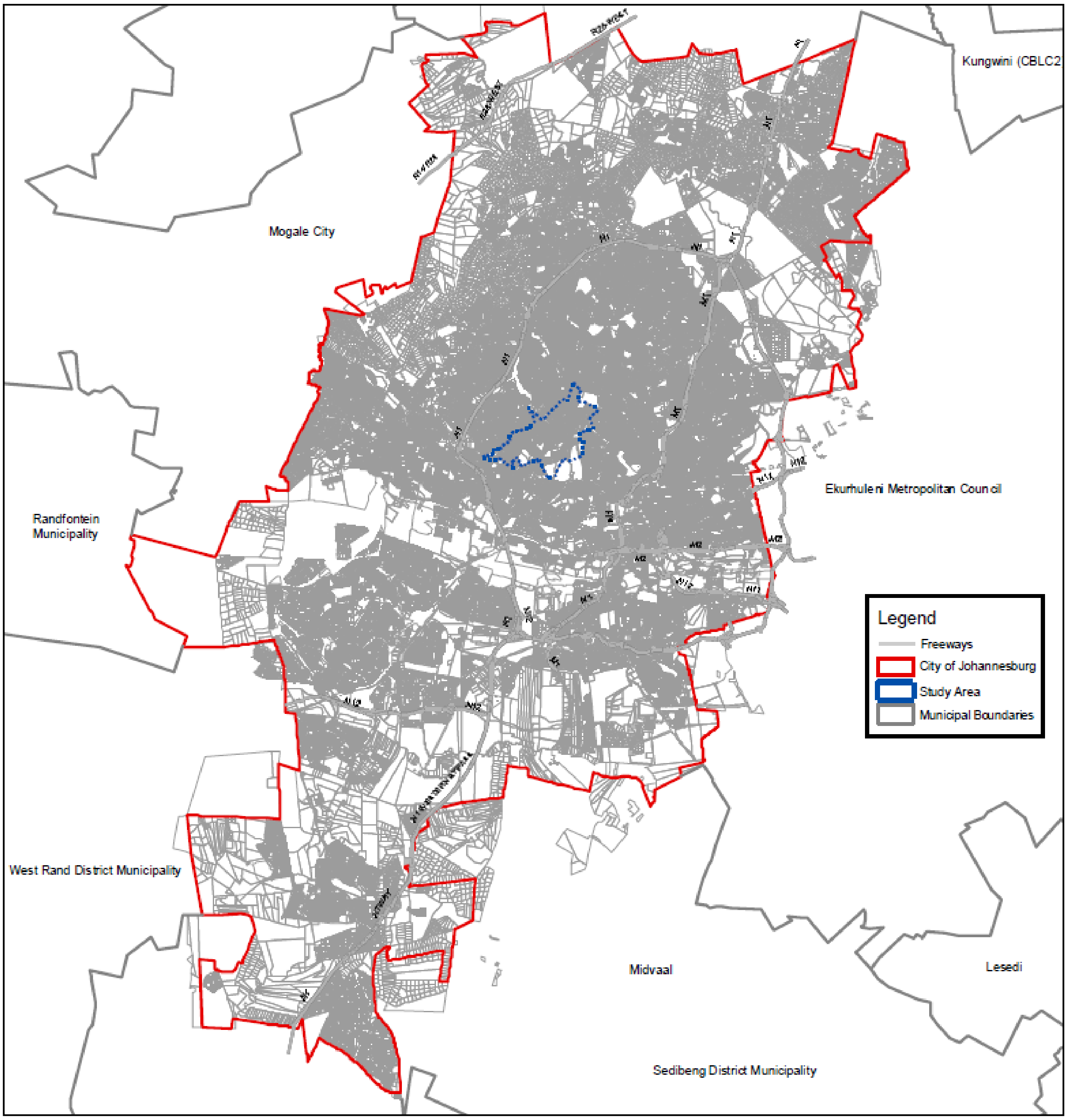


Figure 3-3: Study area (Northcliff zone) outline (Source: JW, 2005)

3.3 Model inputs: data and parameters

Once the model use, analysis type and study area were defined, the next step was the selection model inputs. This process started with the selection of appropriate input data and their corresponding input parameters; followed by the methodology for estimating and measuring the dynamic data.

3.3.1 Input data and parameters

Input data can be separated into two groups, namely static and dynamic. Static data refers to data that does not change in the short-term, as such is typically the physical components of the distribution network such as: pipes, reservoirs, valves, pumps, network topography and structure etc. The dynamic input data refers mainly to: demand and flow, these change (to varying degrees) in both the short and long term and have a notable impact on model accuracy and reliability. Table 3-2 and Table 3-3 below summarise input data and input parameter considerations.

Table 3-2: Distribution network input data break-down

	Data Category	Components
Input Data	Static	Reservoirs, storage tanks, pipes, valves, pumps etc.
	Dynamic	Demand, pressure and flow

Table 3-3: Distribution network input parameter break down (Source: Rossman, 2000)

	Distribution Network Feature	Inputs and Considerations
Input Parameters	Reservoirs	Location, elevation, total head, head pattern, initial quality, source quality, construction material, storage volume
	Tanks	Location, volume, elevation, storage time, demand pattern, maximum and minimum water levels, supply rate, geometry (tank volume curve), material, mixing model (fully mixed, two-compartment mixing, first-in-first-out plug flow [FIFO], last-in-first-out plug flow [LIFO], Mixing Fraction, Reaction Co-efficient, Initial quality, Source quality.
	Pipes	Diameter, length, roughness, geometry, levels (invert, soffit, crown), material, coating and type of coating, wall thickness, age, structural integrity, friction

Input Parameters		equation (Hazen-Williams, Darcy Weisbach or Chezy-Manning), bulk coefficient, wall coefficient.
	Valve	Valve type (PRV, PSV, PBV, FCV, TCV OR GPV), material, function, operational data/settings, elevation, minor loss co-efficient
	Pumps	Pump curves, power rating (kW), speed, efficiency, Net Positive Suction Head (NPSH), Required NPSH, constant or variable speed pump type, operating data (when is it on, for how long etc)
	Nodes/Junctions	Location, elevation, base demand, demand pattern, demand categories, initial quality, source quality,
	Demand	Demand pattern, demand clusters, demand calculation methodology, flows, velocities, pressure and pressure zones etc.

3.3.2 Dynamic input parameter estimation

All input data and input parameters are important as they have an impact on the accuracy and precision of model outputs and ultimately usefulness, but of all such inputs none are trickier than water demand and pipe roughness (AWWA, 2012; Ostfeld *et al.*, 2012). This is because this data is often difficult to obtain, due to poor record keeping (demand) and access (pipe roughness) and the cost associated with both (Hirrel, 2008; Macleod, 2013). Researchers, scientists and engineers often rely on estimation techniques, which can result in good or bad results. In this study both demand data and pipe roughness had to be estimated. Below are details of the principles followed in estimating both during the hydraulic calibration process.

Demand Estimation

A key input in hydraulic modelling is water demand. Hydraulic nomenclature refers to demand rather than use/consumption, as the two are directly related, though not exactly the same thing (example: in instances of drought and other such cases; the use is lower than demand due to supply constraints). Demand is important as it ultimately influences: flow velocities, pressure, water age and indirectly influences water quality (the degree to which this is the case was examined in this study). There are various ways of calculating and predicting demand, the most common being the use of historic consumption data (DHS, 2019). Water demand can be separated between domestic and non-domestic use. This study considered both domestic and non-domestic (schools, clinics, shopping centres and offices) water demand, as the calibration was undertaken for a residential supply area (Northcliff), with both domestic and non-domestic users. Domestic water demand (significant majority of demand in this study) is influenced by a combination of the following (CSIR, 2005; Van Zyl, 2006; Jacobs, 2007; 2008; 2013; DHS,2019):

- Type of development (rural, township, suburb)
- Stand area
- Price of water
- Household income levels
- Static pressure of water
- Climate of area

In South Africa the Neighbourhood Planning and Design Guide (Section J – Water Supply) is the national design guideline for water demand estimation (DHS, 2019). This guideline accounts for all of the aforementioned influencing variables. The accuracy and reliability of the design guideline is generally acceptable. The previous national guideline (CSIR, 2005) was found to be conservative for users with higher consumption and other such special instances (Jacobs *et al.* 2004; Van Zyl, 2007; Jacobs et al. 2013; Griffioen, 2014).

As aforementioned, water demand can be estimated from consumption records (for existing areas). The City of Johannesburg is one of the most developed metropolises on the African continent, hence historic demand data was readily available. This limited the need to use estimates in this study, thus decreasing the probability of input errors. However, in line with the research objectives, the impact of water demand patterns on water quality was studies through the use of four different demand patterns.

Pipe Roughness Estimation

The pipe wall-fluid interaction results in friction. This friction will vary with pipe age, material, diameter, pipe wall material accumulation/loss and water source (Spellman, 2003; Van Vuuren and Van Dijk, 2012). This friction is important as it increases turbidity near the pipe wall and this in turn results in increased energy losses. This loss in energy must be considered as it has material effect on system hydraulics and subsequently water quality. Pipe surface roughness can only be estimated as it varies from pipe to pipe and along the length of the pipe. Bioaccumulation further complicates this estimation, as biological growth in internal pipe walls is highly variable (EPA, 2005; Lambert *et al.*, 2009; Van Vuuren and Van Dijk, 2012). Studies have shown that biological growth (biofilm) inside pipes, results in a variation from the historic pipe roughness estimates/coefficients (Characklis, 1973; Stoodley *et al.*; 1994; Lambert *et al.*, 2009).

Materials used in the Northcliff water distribution system were: steel (coated/lined), uPVC (majority of pipes below 355mm diameter), cast iron (fittings) and concrete (reservoirs). The general roughness estimates that were used for these are listed below in Tables 3-4 and 3-5, but as aforementioned; these values are very difficult to estimate accurately and hence they formed part of the sensitivity analysis in this study.

Table 3-4: Empirical pipe roughness coefficients applicable to this study (Sources: Van Vuuren & Van Dijk, 2006; SAPPMA, 2009)

Pipe Material	**New**	**25 Years Old**	**50 Years Old**	**Badly Corroded**
PE, PP, PVC	150	140	140	130
Smooth Concrete & FRC	150	130	120	100
Steel – Bitumen Lined/Galvanised	150	130	100	60
Cast Iron	130	110	90	50
Vitrified Clay	120	100	80	45

Table 3-5: Absolute roughness values applicable to this study (Sources: Van Vuuren & Van Dijk, 2006; SAPPMA, 2009)

Material	**K (mm)**
PE, PP, PVC	0.002-0.030
GRP	0.01
Steel, new	0.05
Galvanised Iron, new	0.15
Ductile Iron, new	0.5-1.0
Ductile Iron, corroded	1.0-1.5

3.4 Calibration data collection

One of the key steps in this study was the hydraulic and water quality calibration of the Northcliff Zone in EPANET 2.0. Hydraulic calibration is important because water quality in a distribution system cannot be understood or predicted; without an understanding of the hydraulics (Ormsbee *et al.*, 1997; Woolschlager, 2000; AWWA, 2005; Culligan, 2015; Seyoum *et al.,* 2017). This section describes the collection of hydraulic and water quality data.

3.4.1 Hydraulic data collection

A detailed description of the data required for hydraulic calibration in EPANET is presented in Tables 3-2 and 3-3 above. All areas within the JW metro boundary are supplied by the Rand Water Board. The Northcliff zone is supplied from a single 600mm diameter connection point equipped with a 550mm bulk meter, with a static head of approximately 55m. From this connection point the water flows to a 45 mega litre reservoir, which supplies the seven (7) nodes within the study area and one zone outside the study area (Claremont direct feed).

The Northcliff Reservoir supplies all seven nodes within the study area. The topography of the area is such that the six zones east of the reservoir are supplied directly from the Northcliff reservoir. These six zones are low lying and the reservoir is adequately elevated such that five of the six zones require pressure reducing valves (PRVs). The five zones are named after their PRVs and they are: Franklin Roosevelt Park PRV, Montgomery Park PRV, Northcliff Reservoir PRV, Risidale PRV and Victory Park PRV, the Northcliff Reservoir Sub-District is the sixth zone. The last zone is the Northcliff Tower Zone, this area is above the reservoir, hence a 1.1 mega litre elevated water tower was constructed. Water is pumped from the Northcliff Reservoir to the tower and the tower supplies the Northcliff Tower Zone. Table 3-6 below summarises all the nodes, zones and the corresponding supply source for each of these. This data was collected from various as-built drawings and a detailed network analysis report. The network analysis report entailed the following:

- Description of the existing system:
- Present land use and water demand:
- Future land use and water demand:
- Evaluation of existing system:
- Master planning:

The report contained key tables summarising the following information that was critical for the calibration (confirmed as latest available and most accurate):

- Summary of existing infrastructure – Northcliff;
- Land use and zoning categories;
- Actual demand per suburb category and land use;
- List of large users (consumption greater than 20 000L/day), of which there were 24 with a joint consumption of 910 000L/day;
- Existing water sub-districts / zones AADD, as included on Table 3-10 below;
- Future development areas and water demand;
- Flow and pressure logging results
- Present and future sub-district / zone AADD as summarised on Table 3-10 below.

The next piece of critical information was the drawings (and their primary use in model development and calibration), as detailed below:

- Locality plan for the Northcliff Reservoir sub-district as shown on Figure 3-3 above:
    - Used to locate study area.
- Existing water distribution system:
    - Used to draw system on EPANET as it also contains the pipe diameters.
- Existing water sub-districts / zones as shown on Figure 3-8 below:
    - Used to draw boundary conditions as it clearly shows the positions of all system valves (and type of valve).
- Contour plan:
    - Used to obtain natural ground level (NGL) and nodal elevations.
- Suburbs in Northcliff Reservoir sub-district:
    - Use for orientation purposes.
- Existing land use per stand:
    - Used in demand estimation and verification.
- Bulk water demand as measured by Rand Water (supplier to JW):
    - Used in diurnal demand pattern estimation and verification.
- Proposed development densities:
    - Used in demand estimation and verification
- Potential future land developments:
    - Used in future demand rationalisation and estimation.
- Pressure and flow logging positions:
    - Used in model micro and macro calibration.
- Graphs of pressure and flow measurements:
    - Used in model trial simulation, micro and macro calibration.
- Existing system – static pressure:
    - Used in model trial simulation, micro and macro calibration.
- Existing system – peak hour pressures:
    - Used in model trial simulation, micro and macro calibration.
- Existing system – flow velocities at peak hour demand:
    - Used in model trial simulation, micro and macro calibration.
- Proposed water sub-districts / zones:
    - Used in model trial simulation, micro and macro calibration.
- Future system – static pressures:
    - Used in model trial simulation, micro and macro calibration.
- Future system – peak hour pressures:
    - Used in model trial simulation, micro and macro calibration.
- Layout at Northcliff Reservoir:
    - EPANET model input and set-up.
- Layout at Northcliff Water Tower:

- EPANET model input and set-up.

Table 3-6: Summary of study area nodes, zones and corresponding supply sources
(Source: JW, 2005)

NODE	WATER SUB-DISTRICTS / ZONES	SUPPLY SOURCE
1	Northcliff Reservoir	Northcliff Reservoir
2	Northcliff Reservoir PRV	
3	Risidale PRV	
4	Montgomery Park PRV	
5	Franklin Roosevelt Park PRV	
6	Victory Park PRV	
7	Northcliff Tower	Northcliff Water Tower
8*	Claremont Direct Feed	RW Claremont Direct

*Outside study area.

The study area is isolated from surrounding areas with district valves. The seven nodes are supplied through a distribution network that is approximately 172km long, with pipes of various diameters. As part of a water conservation and demand management strategy implementation in 2000, JW automated all mechanical control equipment within this system. This means that the pumps and PRVs can be remotely controlled and all readings (levels, pressures and flow) are automated through telemetry. Table 3-7 below summarises the pipe data for the supply area (pipe material was not accurately know for various reasons); with Figure 3-4 presenting a graphical representation of: the study area, pressure zones and the distribution network and its key features. Table 3-8 below presents the water demands for each water district (zone).

Table 3-7: Summary of the pipeline network within the study area (Source: JW, 2005)

Diam (mm)	Material	Length (m)	% of TOTAL NETWORK
80	uPVC & Asbestos Cement (AC)	5615	3.3%
110	uPVC and AC	116160	67.5%
160	uPVC and AC	27045	15.7%
200	uPVC and AC	10030	5.8%
250	uPVC and AC	10	0.0%
300	uPVC and AC	2840	1.6%
375	Steel, uPVC and AC	4710	2.7%
400	N/A	0	0.0%
450	Steel	675	0.4%
500	Steel	5	0.0%
600	Steel	5080	3.0%
Total		**172170**	**100%**

Table 3-8: Water districts / zones with their corresponding water demands (Source: JW, 2005)

NODE	WATER SUB-DISTRICTS / ZONES	PRESENT AADD* (Ml/day)	5 YEAR AADD (Ml/day)	ULTIMATE AADD (Ml/day)
1	Northcliff Reservoir	1.467	1.567	2.554
2	Northcliff Reservoir PRV	4.551	4.677	4.855
3	Risidale PRV	1.409	1.869	2.17
4	Montgomery Park PRV	1.107	1.142	1.152
5	Franklin Roosevelt Park PRV	1.207	1.512	1.733
6	Victory Park PRV	2.144	2.307	2.576
7	Northcliff Tower	4.105	4.513	4.760
8	Claremont Direct Feed	0.886	0.900	0.900
TOTAL		**16.876**	**18.487**	**20.700**

*Confirmed for the period 2015 and 2016, coinciding with the water quality data for the same period. This is because the area was fully developed with negligible development and improvement in water use efficiency (keeping demand in check).

Noting the available data and its accuracy, pressure was selected as the hydraulic calibration parameter. Figure 3-5 below presents more detail of the study area detailing: pressure logging points, flow logging points and valves (pressure reducing and closed). Figure 3-5 also shows a reasonable spatial distribution of the pressure loggers, with each of the pressure zones being monitored. While it was beyond the scope of the study, consideration of spatial representation in pressure logger placement is important for hydraulic calibration and model use (Yi Wu and Song, 2012; Nejjari *et al.*, 2015; Yi Wu *et al.*, 2015; Cao *et al.*, 2019;).

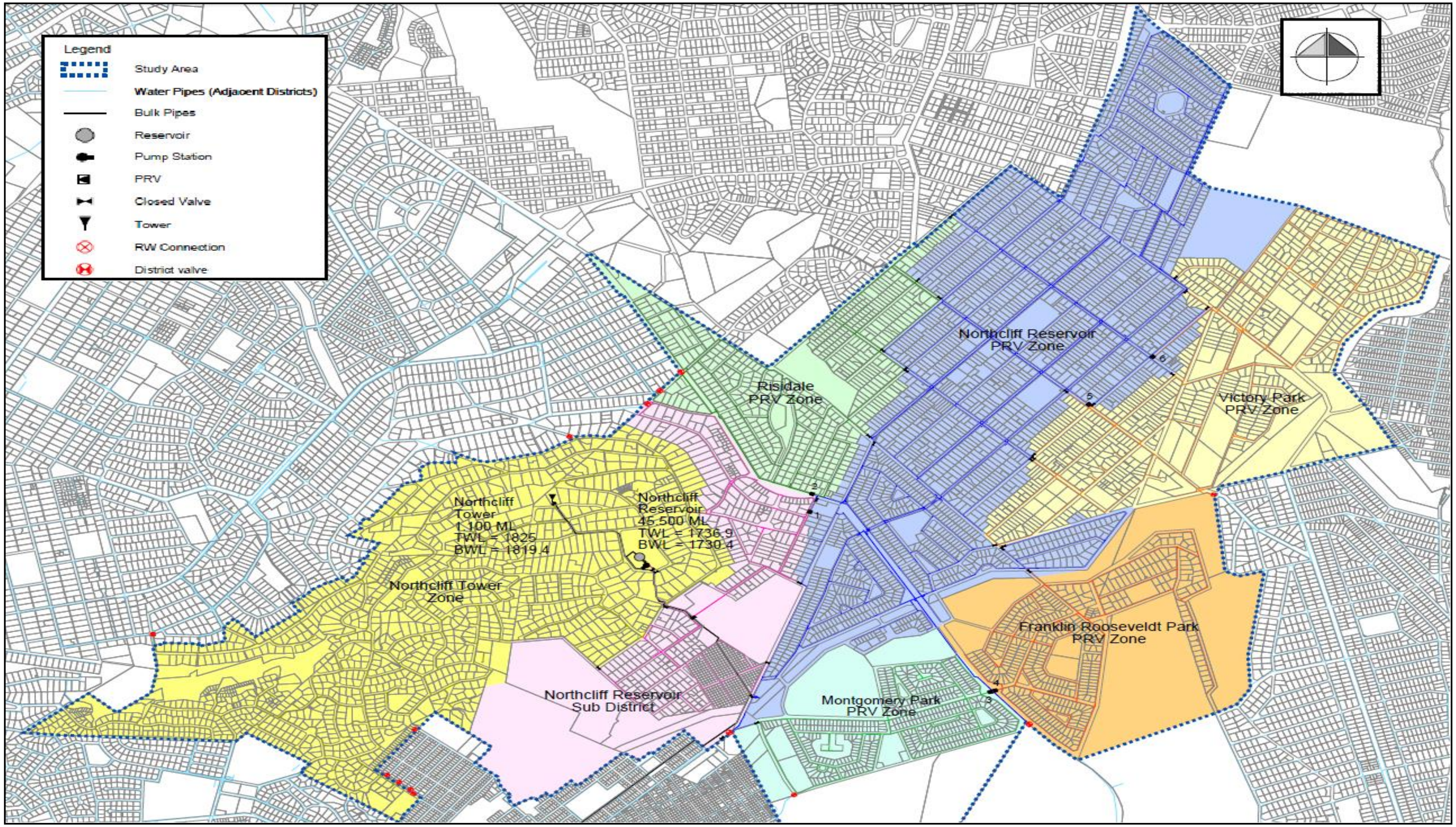


Figure 3-4: Study area boundary, pressure zones and key hydraulic features (Source: JW, 2005)

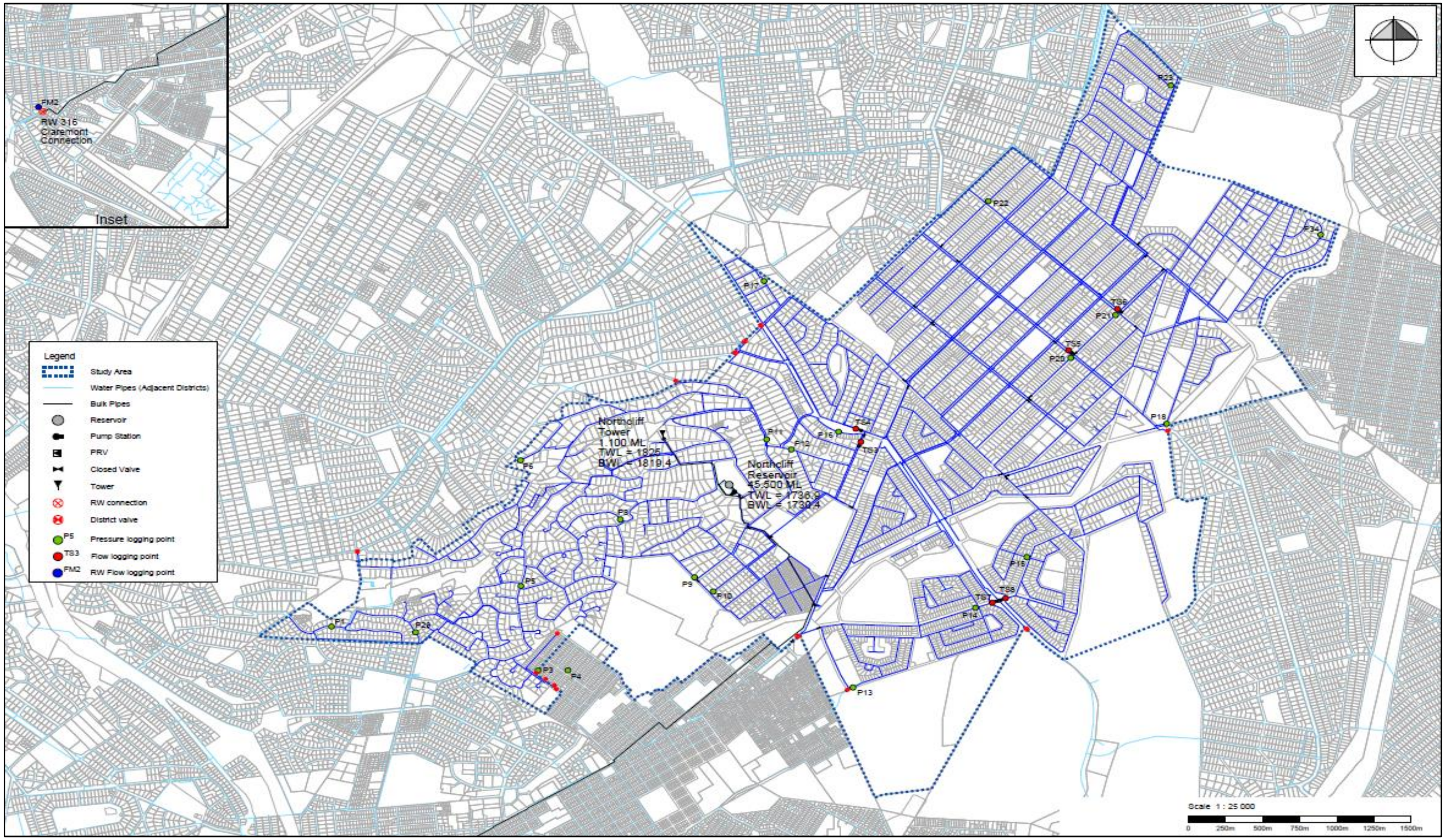


Figure 3-5: Study area pressure logging points and other hydraulic controls (Source: JW, 2005)

3.4.2 Water quality data collection

The EPANET 2.0 software utilises both hydraulic and water quality parameters as inputs. The collection of hydraulic data was followed by the collection of water quality data. The water quality data collection stage was completed by Useh (2017). The study monitored water quality parameters for an area supplied by JW, and bi-weekly samples were collected at 13 (thirteen) sites from September 2015 to August 2016 (Useh, 2017); Table 3-9 below presents all thirteen sites. Due to the large size of the area studied by Useh (2017), only areas within the Northcliff reservoir area were considered for this study, these were: RW_0080, RW_0081, RW_0082, RW_0083 and RW_0084. The water quality data coincides with the pressure logger data, as above.

Table 3-9: Water quality sample collection points (Source: Useh, 2017)

Sites	Latitude			Longitude			Code	Description	Address
1	26	8	53	27	58	24	RW_0080	Northcliff Reservoir Inlet	14 Bernard Lane Northcliff
2	26	8	53	27	58	24	RW_0081	Northcliff Reservoir Outlet	14 Bernard Lane Northcliff
3	26	8	9	27	58	58	RW_0082	Northcliff Reservoir Distr - Linden Bowling Club	Emma Park 1st St Linden
4	26	8	40	27	58	10	RW_0083	Northcliff Tower Outlet	Lucky St Northcliff
5	26	8	40	27	58	10	RW_0084	Northcliff Tower Inlet	Lucky St Northcliff
6	26	9	22	27	56	49	RW_0104	Corriemoor Reservoir Inlet	Washington Dr, Northcliff
7	26	9	22	27	56	49	RW_0105	Corriemoor Reservoir Outlet	Washington Dr, Northcliff
8	26	7	39	27	56	41	RW_0106	Corriemoor Reservoir Distr - Fairland	c/r 4th & Market Fairland
9	26	8	20	27	56	27	RW_0107	Fairland Reservoir Inlet	8th Ave., off Kessel St Fairland
10	26	8	15	27	56	31	RW_0108	Fairland Reservoir Outlet	6th Ave., off Kessel St
11	26	7	56.76	27	57	49.31	RW_0109	Fairland Reservoir Distr - Fairland	273 Castle Hill St Northcliff
12	26	6	36	28	0	20	RW_0251	Blairgowrie Reservoir Inlet	Equity & Susman
13	26	6	59	28	1	35	RW_0253	Blairgowrie Reservoir Distr Old Parks	Old Parktonians Sports Club

Over the study period, Useh (2017) monitored: temperature, conductivity, pH, alkalinity, dissolved oxygen (DO), nitrogen species (ammonium, total ammonia, nitrate and total nitrogen), chlorine species (free chlorine, total chlorine and chloramine), Heterotrophic Plate Count (HPC), total coliform species (average total and average faecal), dissolved organic carbon (DOC) and biodegradable dissolved organic carbon (BDOC) at all sites. Sample handling, collection frequency, preparation and measured parameter was in conformity with the requirements outlined in SANS 241-2015. The said data was then used as the water quality input data for model calibration purposes.

3.5 Trial simulation

Hydraulic calibration is not new and practical approaches / methodologies have been developed and these are briefly outlined below (Ormsbee *et al.*, 1997, 2012; Ostfeld *et al.*, 2012):

Skeletonisation: this would involve a simplification of the network; once simplified inputs can be adjusted until an acceptable fit is obtained. This simplification is referred to as skeletonization. Once the desired pipes roughness and demands are obtained from the skeleton; detail can be added to the model again until the complete network is used, this approach can also be described as an analytical approach.

Simulation: This is a mathematical approach, based on the premise of developing additional equations and solving for those and then using those outputs to solve for the original unknowns in the model.

Optimization: This approach is more commonly known as Genetic Algorithm (GA) development, but is not limited to GA development. This non-deterministic approach is generally used for solving optimization problems, that are not easily solved with other conventional optimization techniques. GAs are based on Darwinian evolutionary principles of natural evolution and biological reproduction, but require experience and careful setting up (Van Zyl, 2000; Yi Wu *et al.*, 2002; Borzi *et al.*, 2005).

After considering the available data, time and the desired applications of this research; it was decided that the use of the analytical approach was most optimal.

The objective of this step was the initial setup of the hydraulic model in EPANET, using the collected data. In order to meet the broader objectives of the study, two models were set up. For the purposes of the trial simulation, the models were calibrated for steady state normal flow condition. A more detailed account of the procedure and rationale for this is given in the sections below.

3.5.1 EPANET hydraulic model set up

One of the objectives of the study was a sensitivity analysis, to achieve this objective in this investigation two independent models were set up in EPANET. The two models differed in the level of hydraulic detail they contained, as one was skeletonized and reduced ("low" level of hydraulic detail) and the other reduced but not skeletonized ("medium" level of hydraulic detail). This was done to investigate whether or not the level of hydraulic detail provided in a model was material for the EPANET 2.0 model outputs. This was important as utilities in developing countries often do not have exhaustive hydraulic data, but still need to undertake hydraulic calibration.

The second key consideration in model set up and calibration was the types of analyses to be undertaken. In keeping with the Ormsbee *et al.* (1997) and Water NZ (2021) methodology and extensive guidance from the AWWA (2012) M32 manual and Walski (2017), the calibration methodology included both steady state (SS) and extended period simulation (EPS). These two types of analyses are defined in the sections below.

The development of the two models had to be undertaken within the confines of industry best-practise (American Water Works Association). To achieve this, the well-established methods of skeletonisation and reduction were employed, following guidelines available in literature (Ormsbee *et al.*, 1997, 2012; AWWA, 2012; Perelman *et al.*, 2012; Paluszczyszyn, 2015; Walski, 2017). The skeletonised and reduced model was named the low-level detail system (LLDS) and the more detailed model was named the medium-level detail model (MLDM). The LLDS and MLDM models were both fully calibrated in each step, as outlined below.

Skeletonisation and reduction

The objective of skeletonization and reduction is the simplification of a distribution network, as much as possible, without compromising the modelled outputs (Ormsbee *et al.*, 1997). Though skeletonization and reduction have the same end-goal, and are often used in combination, they are not the same thing (AWWA, 2012; Paluszczyszyn, 2015). In the reduced and skeletonized model the following was undertaken, to simplify the model:

a) Parallel pipes with insignificant diameters (smaller than 90mm) were aggregated;
b) Nodes that were in place for structure, but had no demands were aggregated;
c) Nodes with insignificant demands were aggregated;
d) Nodes with similar pressure heads, that were closely clustered were combined, and
e) Pipes and nodes at dead ends, where no significant demand was present and no field data (from SCADA) was available were combined.

The utilisation of skeletonisation and reduction techniques, as summarized in Figure 3-6 below, allowed for the creation of two graphically (visually) distinct models, for the same study area. Furthermore, skeletonisation and reduction allow for the creation of four model types, for the same area. These models are differentiated by type and size, a brief summary of these is presented below (AWWA, 2012):

- All-Pipes Model: the model is an actual representation of all hydraulic elements in the ground, often making model large in size and requiring longer computation times.
- All-Pipes Reduced Model: the model has fewer pipes (up to 50% less) than the all-pipe model, thus requiring less storage space and a reduced computation time.
- Skeletonized Model: the model omits small diameter pipes (80 to 90% less), leaving only the transmission mains. The model is much smaller and has shorted computation times, but neighbourhood and street level detail is generally inaccurate or not possible.
- Skeletonized Reduced Model: the model is similar to the skeletonized model, but also has a significant number of nodes omitted.

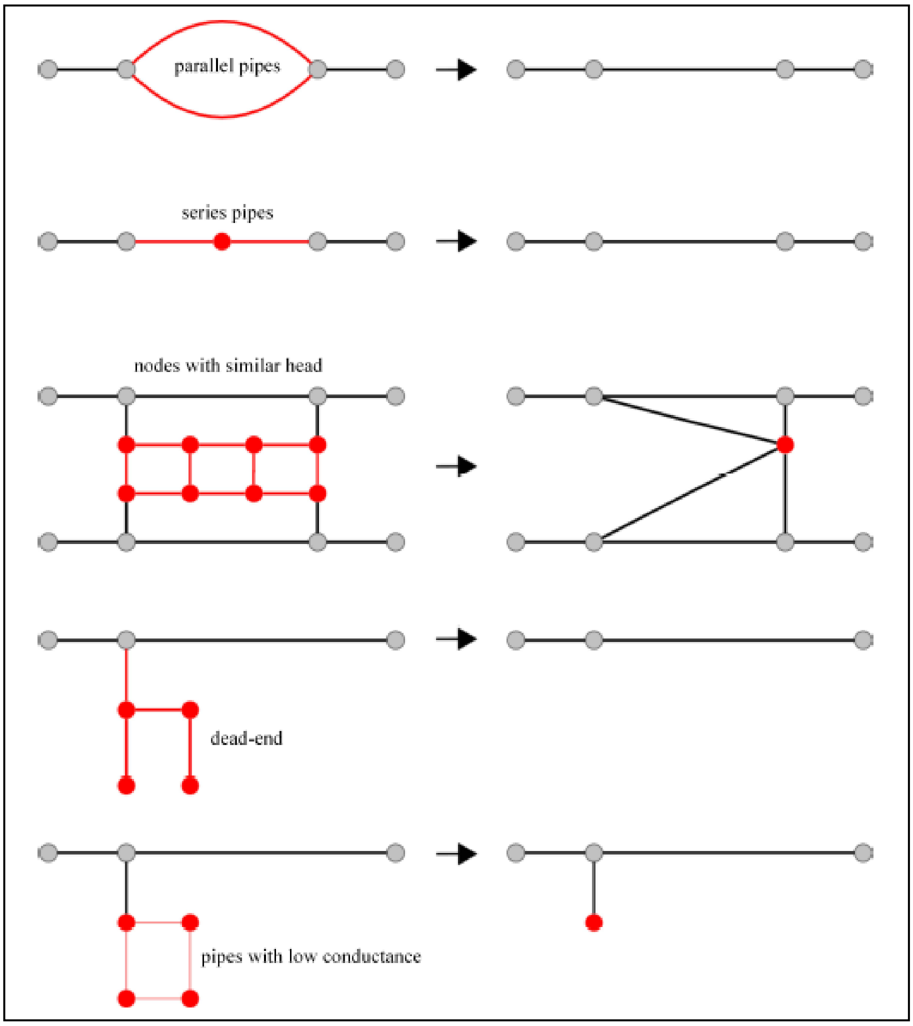


Figure 3-6: Overview of reduction and skeletonization techniques used in model development (Source: Paluszczyszyn, 2015)

The aforementioned techniques, as summarized in Figure 3-7 below, were utilised to create two structurally distinct models for the study area. The two models were the:

- Skeletonised and reduced model, referred to as the Low-Level Detail System (LLDS);
- All-pipes model, referred to as the Medium-Level Detail Model (MLDM).

A brief description of each of the models is given below:

Hydraulic model 1: skeletonised and reduced model - LLDS

This model was developed to test the impact of hydraulic definition on water quality outputs, as such the model was significantly reduced and skeletonized. This was done by the aggregation and deletion of hydraulically insignificant pipes, combining of nodes and demands.

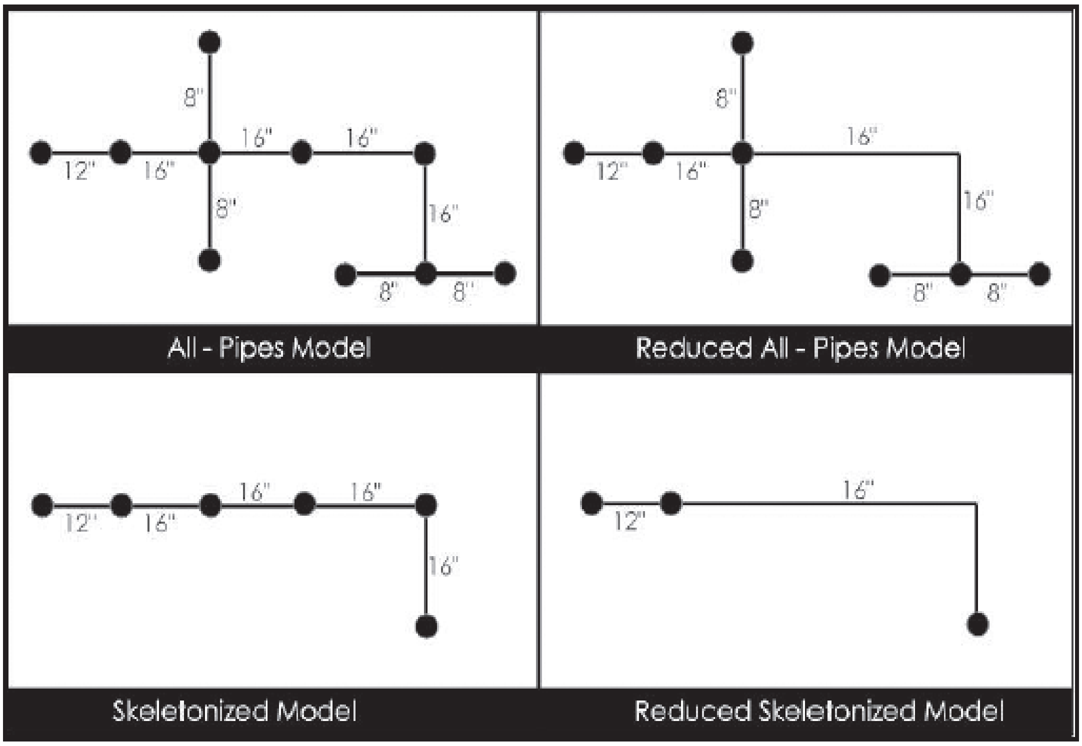


Figure 3-7: Overview of the four main model structures (Source: AWWA, 2012)

Hydraulic model 2: detailed model - MLDM

This model was developed, to be contrasted to the LLDS. The Medium-Level Detail Model (MLDM) represented the system as accurately as possible, by including: all pipes, *most* nodes and all demands.

Model verification, validation and acceptance

Model verification and validation refer to the steps that follow once all data has been captured on the model and the model has no obvious simulation output errors. These steps were concerned with verifying model outputs through comparison with field data, preferably from a period not used during model calibration. If agreement is poor, the modeller must find reasons for this difference and address the underlying issues. At this stage the modeller must be careful not to arbitrarily adjust inputs for the sake of a good fit with collected data (Hirrel, 2007).

In determining whether or not there was sufficient agreement between field data and modelled outputs, a deviation tolerance approach informed by the ECAC (1999) guidelines was followed. If the modelled deviation was determined to be tolerable (less than 10% for demands and 5% for flows), calibration was appropriate and acceptable; however, if errors were greater than the said margins, a process of reconciliation was undertaken.

3.5.2 Simulation 1: steady state – normal flow conditions

Hydraulic calibration can become overwhelming, thus a systematic and considered approach is vital (Ormsbee *et al.*, 1997; AWWA, 2012; Walski, 2017). Once the model was set up, the first simulation to be run was the: steady state – normal flow conditions (SS-NFC) simulation.

Steady state in hydraulic modelling refers to an analysis at a particular (fixed) point in time in the operation of the network, this means that no hydraulic parameters vary (Van Zyl, 2001; AWWA, 2012; Paluszczyszyn, 2015, Walski, 2017). This “snapshot” allows for the use of the laws of conservation of mass or energy equations as outlined above. Steady state analysis is not realistic, as hydraulic conditions in a network vary with time, but it is a useful approach in the systematic calibration of a network.

Steady State – Normal Flow Conditions (SS_NFC) model

The Steady State – Normal Flow Conditions (NFC) simulation, was the first to be undertaken. The NFC refers to periods of normal and low flow, for the selected site these periods would be: late morning to lunch (9:00AM to 11:30AM), early afternoon (2:00PM to 4:30PM) and late evening to very early morning (9:00PM to 4:00AM). During the normal flow conditions the hydraulic grade line (HGL) is flat, velocities are low and head losses are negligible. These conditions were ideal to check the following errors (AWWA, 2012, Walski 2017):

- Incorrect water levels in reservoirs and tanks;
- Incorrect pressure reducing valve (PRV) settings;
- Incorrect elevations at nodes;
- Interconnections between pressures zones that should be isolated;
- Incorrect pressure zone boundaries;
- System calculated HGL levels at nodes that are similar to node elevation (typically 1.5m below natural ground level); and
- Incorrect pump status (open / closed) and speed.

These checks were undertaken for both the LLDS and the MLDM, validation was undertaken by comparing outputs to available field data. The models were accepted after all issues arising in validation were addressed and the models were considered to be calibrated for steady state normal flow conditions.

3.6 Macro-level calibration

Once the initial simulation was undertaken and the outputs compared to the collected field data, and it was determined that calibration and not reconciliation was required (because the errors are statistically acceptable), macro-level calibration then commenced.

3.6.1 Simulation 2: steady state – high flow conditions

Steady State – High Flow Conditions (HFC) simulation was the second undertaking. The HFC refers to periods of high demand and thus high flow, for the selected site these were: morning (5:00AM to 9:00AM), lunch (12:00PM to 2:00PM) and evening (5:00PM to 9:00PM).

Steady State – High Flow Condition (SS_HFC) model

In steady state high flow conditions, system demands are at a peak and the effects of frictional losses are amplified and thus easy to assess. The following errors were checked in this simulation (Ormsbee *et al.*, 1997, 2012; AWWA, 2012; Walski, 2017):

- Valves that should be open but are closed;
- Pipe roughness (linked to mistaken pipe age and or material);
- Incorrect demand locations;
- Incorrect demand magnitude;
- Incorrect network connectivity; and
- Incorrect pipe diameters.

The above-mentioned checks were undertaken for both the LLDS and the MLDS with validation being undertaken by comparing modelled outputs to available field data. After verification and validation discrepancies were addressed the model was considered calibrated for steady state – high flow conditions

3.7 Sensitivity analysis

This step was a pre-cursor to the micro-level calibration. The objective of this step was to assess which inputs have the greatest influence on the hydraulic model outputs and accuracy. The input data and parameters were then split into two groups: a) sensitive data and b) resilient data. Sensitive data was used in the micro-level calibration, as part of the extended period simulation. The resilient data is important, because it tells us which data is not sensitive and thus, does not need a high level of accuracy. This is important as limited resources are available (time and financial), and they must be used strategically. The outputs of this step were part of simulation 3, hence they are not reported on separately. But it is worth noting that pipe properties (roughness, age and material) were part of the resilient data set; while demands (including diurnal patterns) and operating conditions were part of the sensitive data set. These conclusions were reached by making global changes to the said variables and observing their impact on the system. The sections that follow focus on this sensitive data, to ultimately achieve what is referred to as micro-calibration.

3.7.1 Simulation 3: extended period simulation – known control status

Steady state analysis was followed by extended period simulation (EPS). EPS refers to a grouping of steady simulations, undertaken over an extended period of time to simulate system behavioural changes over time (Van Zyl, 2001; AWWA, 2012). The changes in the system are as a result of changes in water demand and operating conditions, EPS is thus concerned with demand patterns and the conditions under which the system is operated. Due to these changes over time, EPS is also referred to as dynamic simulation (Van Zyl, 2001).

EPS – Known Control Status (EPS_KCS) model

Extended Period Simulation – Known Control Status (KCS) simulation was the third of the four simulations undertaken. The KCS can refer to many variables, but for this study it was limited to a consideration of the temporal variations in demand and the changes in settings and the control of pumps and valves (AWWA, 2012; Walski, 2017). These focus areas were identified in the sensitivity analysis and are confirmed by literature and best-practise manuals as the most sensitive and appropriate for the final steps in calibration (Ormsbee *et al.*, 1997; ECAC, 1999; Hirrel, 2008; AWWA, 2012; Ostfeld, *et al*. 2012).

The modelling process included the following (Walski, 2017):

- Comparing and matching of modelled reservoir and tank tower level fluctuations with field data, paying attention to both the direction and slope of the graphs (generated from the patterns) over a 24- and 48-hour period;
- Adjusting pump controls as much as possible, to try align modelled reservoir and tank level patterns with field data. This was achieved through the use of control statements in EPANET (also known as rule-based controls);
- Once the pump control related issues were all resolved, the accuracy of the diurnal demand patterns was checked. This was done using available flow metering data;
- Checking the reservoir and elevated water tower dimensions.

Expecting an exact match between modelled outputs and field data is both unreasonable and unlikely to be achieved. The objective of this simulation was to obtain patterns that are as close as possible to filed data. The aforementioned checks and validation were undertaken for both the LLDS and the MLDM. The models were accepted and considered validated for the extend period simulation – known control status.

3.8 Micro-level calibration

Micro-level calibration is concerned with the calibration of the most sensitive model characteristics, as such it was logical for this to be the final step in the process.

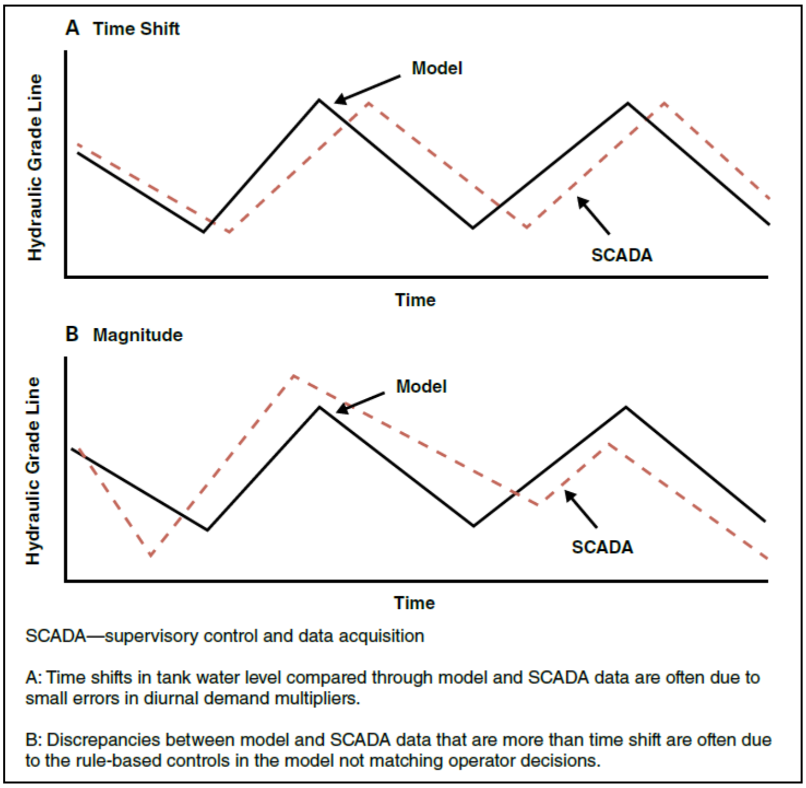
3.8.1 Simulation 4: extended period simulation – varying control status

The fourth and final simulation was the Extended Period Simulation – Varying Control Status (VCS). This simulation is primarily concerned with the validation of time-based controls over a period of time, typically 7 to 21 days (Walski, 2017).

EPS –Varying Control Status (EPS_VCS) model

As this was the final hydraulic simulation, its key objective was to undertake analysis that had not been undertaken in the previous simulations. The work undertaken at this stage was:

- Checking for time shift errors, these would be as a result of using incorrect demand patterns and or errors in control settings. Time shift errors can be extremely difficult to resolve, but do not render a hydraulic model useless (Walski, 2017).
- Checking for magnitude errors, these would be due to:
  - Operators not following system control rules;
  - Input data errors, not identified in the preceding simulations;
  - EPANET model errors, not identified in the preceding simulations.



SCADA—supervisory control and data acquisition

A: Time shifts in tank water level compared through model and SCADA data are often due to small errors in diurnal demand multipliers.

B: Discrepancies between model and SCADA data that are more than time shift are often due to the rule-based controls in the model not matching operator decisions.

Figure 3-8: Graphical illustration of the main errors in varying control status models
(Source: Walski, 2017)

Time shift errors are not fatal, but magnitude errors can render the model useless. Figure 3-8 above gives a graphical representation of the two errors that this simulation sought to resolve.

Once the aforementioned checks and validation were undertaken for both the LLDS and the MLDM, the models were accepted.

3.8.2 Model verification, validation and acceptance

Model validation (and verification) was undertaken for all four hydraulic simulations. This was done by comparing selected modelled outputs to field data (hydraulic and water quality). If the outputs were within the acceptable deviation tolerances, the model was considered validated (and verified) and could thus be accepted.

It must be noted that model acceptance entails a degree of subjectivity, this is because model acceptability must be assessed with the intended model use in mind. Thus, a model that is acceptable for one context and end use, might not be acceptable for another end use. Additionally, a model can never be perfectly calibrated as there are too many unknowns. It is rather helpful to think of calibration along a continuum that goes from poorly and completely uncalibrated to a very well calibrated model, still bearing in mind that perfection is neither possible nor realistic (Walski, 2017). It thus becomes obvious that calibration can be an endless pursuit, but the modeller should stop when the cost of attaining greater accuracy, exceeds the benefit (imbedded in the model endues).

3.9 Water quality modelling towards study objectives

One of the objectives of the study was to perform a sensitivity analysis, specifically testing model output sensitivity to the input parameters that are often estimated. The sensitivity analysis looked at what effect varying one of the parameters had, on the accuracy of the modelled outputs (a selected water quality parameter). The accuracy was determined by comparing the modelled water quality indicator to its field measured equivalent. Table 3-10 below presents the selected parameters, the justification for selection and the water quality indicator that was monitored in the modelling.

Table 3-10: Sensitivity analysis parameters with corresponding indicator and justification for selection.

PARAMETER	JUSTIFICATION / EXPLANATION	INDICATOR
Hydraulic definition	Utilities have varying degrees of data and information of their water distribution networks. It is important to understand how much hydraulic definition is required for meaningful water quality modelling.	Physical, chemical and biochemical determinants

Level of calibration	There are differing levels of calibration, it was therefore important to understand what impact the differing levels had on water quality modelling accuracy.	Chlorine (mg/l)
Pipe material and age	Poor recording keeping often results in pipe characteristics being unknown, thus needing to be estimated (pipe roughness c-value was surrogate indicator)	Chlorine (mg/l)
Demand pattern	Requires a notable capital and operational investment and technical expertise to measure reliably. It is also not simple, as demand pattern varies with end-users and changes hourly and seasonally. It not exact, but is often poorly estimated.	Water age (hours) and Chlorine (mg/l)
Load-shedding	South Africa has been affected by load-shedding for over ten years. It is important to understand what impact the loss of electricity has on water quality in a system	Water age (hours) and Chlorine (mg/l
Tank mixing model	Water retaining structures (reservoirs / tanks) are not always designed to ensure mixing. It is therefore important to understand what impact the different mixing models have on water quality	Chlorine (mg/l)

The sensitivity analysis was important as it would provide a decision-making tool for the Utility. In a world of limited resources, utilities need to priorities in order to save time and money; outputs from this analysis could guide the Utility when deciding on which parameters to prioritise. This was particularly important for utilities in developing countries, as they often have limited resources. The sensitivity analysis was undertaken in a systematic manner, where one variable was varied and its effects measured (via the indicator), while the other variables were kept constant.

4. Results and discussion

4.1 Introduction

This section presents and discusses the results of the different hydraulic and water quality evaluations undertaken; these were a result of following the foregoing methodology to achieve the set objectives. For the study results and analysis to be credible, the results of the calibration (hydraulic and water quality) and validation (water quality), which follow the procedure outlined in the Water New Zealand (2021) guidelines, are presented first. Building on the confidence of appropriately calibrated and validated models, results and discussion of the following analysis is presented:

- The impact of hydraulic definition on water quality (various parameters);
- The impact of the level of calibration on water quality (chlorine);
- The significance of pipe age and material on water quality (chlorine);
- Effect of demand pattern on water quality (chlorine and water age);
- The impact of the different stages of load-shedding on water quality (chlorine and water age);
- The impact of the different tank (reservoir) mixing models on water quality (chlorine and water age); and
- Water quality monitoring improvements (improved risk management) that EPANET 2.0 enables, specifically considering sampling location optimisation.

4.2 Hydraulic and water quality calibration results

In line with the objectives of the study, two independent models were built manually in EPANET 2.0 and calibrated (following the same methodology for both). The two models are the Medium Level Detail Model (MLDM) and the Low-Level Details System (LLDS). Table 4-1 below summarises the difference between the actual network and the two models.

Table 4-1: A summary comparison of the three systems under consideration

Parameter	Actual / Real Network	MLDM (Medium-Level)	LLDS (Low-Level)
Total Pipe Length (km)	172.170	157.032 (91% of actual)	68.221 (40% of actual)
Nodes	6160 linked stands	524 (9% of actual)	121 (2% of actual)
Pressure Reducing Valves	7	7	7
Water Sub-districts	7	7	7
Pumps	1	1	1
Number of Pipes	Unknown	674	116
Pipe Diameters Used	10 (80mm to 600mm) no 400mm used.	Same as actual	Same as actual
Reservoirs	1	1	1
Elevated Tanks	1	1	1
Annual Average Daily Demand (Ml/d)	2.846	2.846	2.846
Large Water Users	24	24	24

The calibration results of both models are presented and discussed below. The dual model approach was designed to allow for a detailed evaluation of the models independently and against each other, as detailed in the sections below.

4.2.1 Low-Level Detail System calibration

Following international best-practice guidelines for hydraulic calibration, several models were developed as outlined in the methodology (AWWA, 1999; Hirrel, 2008; EPA, 2005; Walski, 2017; NZ Water, 2021). Table 4-2 below summarises the results for the Low-Level Detail System (LLDS) when considering the pressure at all monitored nodes (9 of): J26, J36, J92, J151, J98, J102, J108, J82, J78. As part of the micro-calibration process and noting operational notes from the data collection, further refinement was under taken and faulty pressure loggers omitted. As shown in Table 4-2 below, this resulted in only the following loggers being considered (5 of): J26, J36, J82, J102, J108 (shown in brackets below).

Table 4-2: Low-Level Detail System pressure calibration results

No	Model Name	Mean Error	RMS Error	Correlation Between Means
1	Steady State – Normal Flow Conditions (SS_NFC_LLDS)	14.478 (9.895)	16.675 (11.315)	0.897 (0.926)
2	Steady State – High Flow Conditions (SS_HFC_LLDS)	10.448 (4.643)	13.392 (6.589)	0.922 (0.964)
3	Extended Period Simulation - Known Control Status (EPS_KCS_LLDS)	16.146 (4.885)	26.375 (6.576)	0.742 (0.981)
4	Extended Period Simulation – Variable Control Status (EPS_VCS_LLDS)	11.148 (3.619)	13.966 (4.706)	0.921 (0.992)

From the above, it was clear that there was a direct correlation between increased levels of calibration and the accuracy of the model for the selected hydraulic parameter (pressure). This is evident from the reduction in the Mean Error Root Mean Square (RMS) Error and the increased correlation between means as presented. Furthermore, these results were within the acceptable range to warrant moving from model hydraulic calibration to water quality calibration.

As the intended purpose of the exercise was water quality modelling; water quality calibration had to be undertaken. Data collected by Useh in her 2017 study of the area was utilised, the parameter considered was total chlorine using averaged field data collected for November 2015. As the modelling was undertaken in EPANET 2.0, the key system characteristics to estimate were: Bulk Reaction Order, Global Bulk Coefficient, Limiting Concentration and the Global Wall Coefficient. These had to be estimated as they were not collected by Useh (2017). A manual trial and error approach was adopted (as no field data existed), guided by literature (Rossman, 2000; Georgescu *et al.*, 2012; Mostafa *et al.*, 2013). Table 4-3 below summarises the results at four sites, with the Bulk reaction order kept at 1.0 and the Limiting concentration kept at 0.0.

Table 4-3: Low-Level Detail System total chlorine initial calibration results

Test No: 1	**Global Bulk Coefficient = -0.25**		**Global Wall Coefficient = 0**	
Node:	J10 (Site 1)	J12 (Site 2)	J67 (Site 3)	J6 (Site 5)
*Error (Field: Model)	-1%	-2%	52%	-1%
Test No: 2	**Global Bulk Coefficient = -0.5**		**Global Wall Coefficient = 0**	
Node:	J10 (Site 1)	J12 (Site 2)	J67 (Site 3)	J6 (Site 5)
*Error (Field: Model)	1%	20%	81%	21%
Test No: 3	**Global Bulk Coefficient = -0.85**		**Global Wall Coefficient = - 0.013**	
Node:	J10 (Site 1)	J12 (Site 2)	J67 (Site 3)	J6 (Site 5)
*Error (Field: Model)	1%	42%	97%	42%

Test No: 4	**Global Bulk Coefficient = -0.15**		**Global Wall Coefficient = 0**	
Node:	J10 (Site 1)	J12 (Site 2)	J67 (Site 3)	J6 (Site 5)
*Error (Field: Model)	1%	-9%	33%	-9%
Test No: 5	**Global Bulk Coefficient = -1.0**		**Global Wall Coefficient = 0**	
Node:	J10 (Site 1)	J12 (Site 2)	J67 (Site 3)	J6 (Site 5)
*Error (Field: Model)	0%	48%	97%	49%
Test No: 6	**Global Bulk Coefficient = -0.20**		**Global Wall Coefficient = -**	
Node:	J10 (Site 1)	J12 (Site 2)	J67 (Site 3)	J6 (Site 5)
*Error (Field: Model)	0%	-5%	43%	-5%

*Error (Field: Model) represents the percentage difference between actual field data and EPANET model output.

Based on the above, Test No 4 was selected and the final calibrated LLDS model had the following model parameters: Bulk Reaction Order = 1, Limiting Concentration = 0, Global Bulk Coefficient = -0.15; Global Wall Coefficient = 0. These parameters were then used as inputs to the four LLDS models and the results were as presented in Table 4-4 below.

Table 4-4: Low-Level Detail System total chlorine final calibration results

Model: Steady State - Normal Flow Conditions (LLDS_SS_NFC)					
Nodes:	J10 (Site 1)	J12 (Site 2)	J67 (Site 3)	J6 (Site 5)	J7 (Site 4)
Error (Model: Field)	0%	2%	53%	9%	52%
Model: Steady State - High Flow Conditions (LLDS_SS_HFC)					
Nodes:	J10 (Site 1)	J12 (Site 2)	J67 (Site 3)	J6 (Site 5)	J7 (Site 4)
Error (Model: Field)	0%	1%	53%	10%	53%
Model: Extended Period Simulation – Known Control Status (LLDS_EPS_KCS)					
Nodes:	J10 (Site 1)	J12 (Site 2)	J67 (Site 3)	J6 (Site 5)	J7 (Site 4)
Error (Model: Field)	0%	-9%	26%	-9%	-11%
Model: Extended Period Simulation – Variable Control Status (LLDS_EPS_VCS)					
Nodes:	J10 (Site 1)	J12 (Site 2)	J67 (Site 3)	J6 (Site 5)	J7 (Site 4)
Error (Model: Field)	0%	-9%	26%	-9%	-11%

Error (Field: Model) represents the percentage difference between actual field data and EPANET model output.

From the above it was clear that an improvement in the level of calibration from the LLDS_SS_NFC (LLDS steady state, normal flow conditions) to the LLDS_EPS_VCS (LLDS extended period simulation, variable control status) corelates with an improvement in the accuracy of the model as the gap between the modelled and field data closes at all observation points. The site furthest from the reservoir (node J67) produced the greatest error, though this decreased from 53% to 26% it was still not within the acceptable norms for credible modelling. Observing the principle of diminishing returns in the calibration process, no further calibration was undertaken but this particular result was carefully noted (Hirrel, 2008).

4.2.2 Medium-Level Detail Model calibration

Following international best-practice guidelines for hydraulic calibration, several models were developed as outlined in the methodology. Table 4-5 below summarises the results for the Medium-Level Detail Model (MLDM) when considering the pressure at all monitored nodes (21 of): J131, J146, J158, J203, J208, J170, J92, J51, J98, J235, J102, J108, J394, J363, J272, J276, J78, J501, J458, J135 and J310. As part of the micro-calibration process and noting operational notes from the data collection, further refinement was under taken and faulty pressure loggers omitted. This resulted in only the following loggers being considered (9 of): J8, J131, J135, J158, J208, J363, J272, J276 and J310 (shown in brackets below)

Table 4-5: Medium-Level Detail Model pressure calibration results

No	Model Name	Mean Error	RMS Error	Correlation Between Means
1	Steady State – Normal Flow Conditions (SS_NFC_MLDM)	12.172 (6.018)	17.606 (8.003)	0.876 (0.944)
2	Steady State – High Flow Conditions (SS_HFC_MLDM)	12.146 (6.738)	17.533 (8.558)	0.886 (0.938)
3	Extended Period Simulation - Known Control Status (EPS_KCS_MLDM)	12.196 (5.184)	17.438 (6.404)	0.885 (0.973)
4	Extended Period Simulation – Variable Control Status (EPS_VCS_MLDM)	12.183 (4.626)	17.405 (5.830)	0.885 (0.988)

From the above, it was clear that there was a direct correlation between increased levels of calibration and the accuracy of the model for the selected hydraulic parameter (pressure). Furthermore, these results were within the acceptable range to progress from hydraulic model calibration to water quality calibration, following the approached described for the LLDS above.

Table 4-6: Medium-Level Detail Model total chlorine initial calibration results

Test No: 1	**Global Bulk Coefficient = -1.0**		**Global Wall Coefficient = 0**	
Node:	J10 (Site 1)	J12 (Site 2)	J67 (Site 3)	J6 (Site 5)
Error (Field: Model)	2%	31%	47%	34%
Test No: 2	**Global Bulk Coefficient = -0.5**		**Global Wall Coefficient = 0**	
	J10 (Site 1)	J12 (Site 2)	J67 (Site 3)	J6 (Site 5)
Error (Field: Model)	1%	7%	19%	10%
Test No: 3	**Global Bulk Coefficient = -0.85**		**Global Wall Coefficient = -0.013**	
Node:	J10 (Site 1)	J12 (Site 2)	J67 (Site 3)	J6 (Site 5)
Error (Field: Model)	2%	25%	46%	28%
Test No: 4	**Global Bulk Coefficient = -0.5**		**Global Wall Coefficient = 0.013**	
Node:	J10 (Site 1)	J12 (Site 2)	J67 (Site 3)	J6 (Site 5)
Error (Field: Model)	1%	7%	27%	10%

Test No: 5	**Global Bulk Coefficient = -0.4**		**Global Wall Coefficient = 0**	
Node:	J10 (Site 1)	J12 (Site 2)	J67 (Site 3)	J6 (Site 5)
Error (Field: Model)	1%	2%	12%	4%

Error (Field: Model) represents the percentage difference between actual field data and EPANET model output.

Table 4-6 above presents trial and error results from estimating the most appropriate water quality coefficients (guided by literature as there was no field data). Based on the above, Test No. 5 was selected and the final calibrated MLDM had the following model parameters: Bulk Reaction Order = 1.0, Limiting Concentration = 0, Global Bulk Coefficient = -0.4 and a Global Wall Coefficient = 0. The same calibration process was followed for all four MLDM models and the same model water quality parameters used. It could therefore be concluded that the level of calibration increases water quality model accuracy for total chlorine as is evident from Table 4-7 below.

Table 4-7: Medium-Level Detail Model total chlorine final calibration results

Model: Steady State - Normal Flow Conditions (MLDM_SS_NFC)					
Nodes:	J10 (Site 1)	J12 (Site 2)	J67 (Site 3)	J6 (Site 5)	J7 (Site 4)
Error (Model: Field)	2%	31%	83%	100%	100%
Model: Steady State - High Flow Conditions (MLDM_SS_HFC)					
Nodes:	J10 (Site 1)	J12 (Site 2)	J67 (Site 3)	J6 (Site 5)	J7 (Site 4)
Error (Model: Field)	2%	29%	48%	67%	85%
Model: Extended Period Simulation – Known Control Status (MLDM_EPS_KCS)					
Nodes:	J10 (Site 1)	J12 (Site 2)	J67 (Site 3)	J6 (Site 5)	J7 (Site 4)
Error (Model: Field)	1%	3%	25%	5%	8%
Model: Extended Period Simulation – Variable Control Status (MLDM_EPS_VCS)					
Nodes:	J10 (Site 1)	J12 (Site 2)	J67 (Site 3)	J6 (Site 5)	J7 (Site 4)
Error (Model: Field)	1%	2%	8%	4%	7%

From the above it was clear that the level of accuracy when comparing model and field data increased as the level of calibration increased from the base SS_NFC model to the EPS_VCS model, to within acceptable international norms for water quality modelling. This meant that the process could proceed to the final stage of validation before the scenario testing. The levels of accuracy between the LLDS and the MLDM are also already clear when Table 4-4 (LLDS) and Table 4-7 (MLDM) are compared at each stage of calibration, this was further examined in the sections below.

4.3 Water quality validation results

To have an increased level confidence in a model, calibration followed by validation must be undertaken. The primary purpose of validation is to ensure that the model is well calibrated, this is achieved through the use of an independent set of data (EPA, 2005; Mukuve, 2010, NZ Water, 2021). For validation to be effective, the modelled data must be compared to the field data; noting the lack of additional independent hydraulic data, independent water quality data was used for model validation. For this purpose, dissolved oxygen (DO) field data from 23 May 2016 was utilised. This ensured that there was independence on two levels (date of data collection and parameter considered) between the water quality field data used for calibration (total chlorine from November 2015) versus that used for validation (dissolved oxygen from 23 May 2016). All four models (SS_NFC, SS_HFC, EPS_KCS and EPS_VCS) were validated for both the LLDS and the MLDM. This is despite knowing that only the EPS_VCS models would have meaningful utility (as the most calibrated model) in other aspects of the investigation.

4.3.1 Low-Level Detail System validation

Table 4-8 below presents results from the Low-Level Detail System (LLDS) validation using dissolved oxygen (field data from 23 May 2016) as the modelled parameter. As expected, there was an improvement in model accuracy from the SS_NFC model to the EPS_VCS. When comparing the calibration and validation results, there was also a marginal improvement in the accuracy of the modelled outputs. From this it was concluded that the validation had served its purpose of providing assurance of the model credibility under an independent set of conditions and data. The four LLDS models were taken as reasonably calibrated and validated and thus acceptable for further analysis.

Table 4-8: Low-Level Detail System validation using dissolved oxygen

Model: Steady State - Normal Flow Conditions (LLDS_SS_NFC)					
Nodes:	J10 (Site 1)	J12 (Site 2)	J67 (Site 3)	J6 (Site 5)	J7 (Site 4)
Error (Model: Field)	-0.13%	-7.35%	-42.54%	-7.23%	-54.15%
Model: Steady State - High Flow Conditions (LLDS_SS_HFC)					
Nodes:	J10 (Site 1)	J12 (Site 2)	J67 (Site 3)	J6 (Site 5)	J7 (Site 4)
Error (Model: Field)	-0.10%	-7.22%	-42.33%	-6.69%	-54.45%
Model: Extended Period Simulation – Known Control Status (LLDS_EPS_KCS)					
Nodes:	J10 (Site 1)	J12 (Site 2)	J67 (Site 3)	J6 (Site 5)	J7 (Site 4)
Error (Model: Field)	0.01%	-4.99%	-27.56%	0.98%	7.38%
Model: Extended Period Simulation – Variable Control Status (LLDS_EPS_VCS)					
Nodes:	J10 (Site 1)	J12 (Site 2)	J67 (Site 3)	J6 (Site 5)	J7 (Site 4)
Error (Model: Field)	0.01%	-5.08%	-27.29%	0.83%	7.03%

Error (Field: Model) represents the percentage difference between actual field data and EPANET model output.

4.3.2 Medium-Level Detail Model validation

Table 4-9 below presents results from the Medium-Level Detail Model (MLDM) validation using dissolved oxygen (from 23 May 2016) as the modelled parameter. In contrast to the trend observed in the LLDS, the validation results were marginally less accurate than those of the calibration exercise, but were still within an acceptable range for the purposes of validation. The MLDM results were also more accurate than those of the LLDS, meaning the model (MLDM) was still more credible than the LLDS as was observed at the calibration stage.

Table 4-9: Medium-Level Detail Model validation using dissolved oxygen

Model: Steady State - Normal Flow Conditions (MLDM_SS_NFC)					
Nodes:	J10 (Site 1)	J12 (Site 2)	J67 (Site 3)	J6 (Site 5)	J7 (Site 4)
Error (Model: Field)	-0.57%	-43.23%	-85.68%	-100%	-100%
Model: Steady State - High Flow Conditions (MLDM_SS_HFC)					
Nodes:	J10 (Site 1)	J12 (Site 2)	J67 (Site 3)	J6 (Site 5)	J7 (Site 4)
Error (Model: Field)	-0.41%	-41.62%	-55.52%	-69.37%	-85.96%
Model: Extended Period Simulation – Known Control Status (MLDM_EPS_KCS)					
Nodes:	J10 (Site 1)	J12 (Site 2)	J67 (Site 3)	J6 (Site 5)	J7 (Site 4)
Error (Model: Field)	0.03%	-16.64%	-33.35%	-13.81%	-16.11%
Model: Extended Period Simulation – Variable Control Status (MLDM_EPS_VCS)					
Nodes:	J10 (Site 1)	J12 (Site 2)	J67 (Site 3)	J6 (Site 5)	J7 (Site 4)
Error (Model: Field)	0.03%	-15.20%	-14.37%	-12.90%	-14.14%

Error (Field: Model) represents the percentage difference between actual field data and EPANET model output.

The purpose of the validation was to attain an increased level of confidence in the model through the use of an independent set of data. The use of the 23 May 2016 dissolved oxygen data achieved this purpose, as the variance between the modelled and field data was within an acceptable range. Noting the aforementioned, the MLDM was considered calibrated and validated and ready for use in the subsequent analysis in line with the research objectives.

4.4 Scenario analysis 1 – water quality and hydraulic definition

The purpose of this scenario was to examine the extent to which water quality was affected by how well defined a model is (hydraulic definition). This exercise was important in establishing the extent to which skeletonisation and sectorisation affect the accuracy of water quality modelling. This was relevant as these are techniques that are commonly applied and may be of particular utility in instances where the network is not fully known but important decisions must be made and taken based on models or the funds to fully model the network are not available. Table 4-10 below provides a summary of the differences in the pipe length, between the three systems (real/actual, MLDM and LLDS). This table should be considered in conjunction with Table 4-1 above that further described the physical differences between the systems. In undertaking skeletonisation and sectorisation, best practice as recommended in literature was followed (EPA, 2005, Paluszezyszyn, 2015; Armand *et al.*, 2018).

Table 4-10: Pipe length comparison for corresponding diameters across the three systems

Real System			**Medium Level Model**		**Low-Level Detail System**	
Diameter (mm)	Length (m)	% of Total	Length (m)	% of Total	Length (m)	% of Total
80 - 160	148 820	86.4%	124 295	79.2%	21030	35.5%
200 - 300	12 880	7.5%	15386	9.8%	16309	27.5%
375 - 600	10 470	6.1%	17 292	11%	21973	37%
Total	**172 170**	**100%**	**156 973**	**100%**	**59311**	**100%**
Percentage of real system length in model			**91.2%**		**34.4%**	

A total of seven water quality determinants were analysed, these were: water age, dissolved oxygen (DO), total chlorine, free chlorine, chloramine, dissolved organic carbon (DOC), biodegradable dissolved organic carbon (BDOC). This was done by comparing the EPANET 2.0 model outputs with field data from Useh (2017), where there was no field data (water age) a comparison between the LLDS and the MLDM was undertaken. To better structure the results and discussion, the seven parameters were grouped into three clusters: physical, chemical and biochemical determinants. Table 4-11 below summarises this functional grouping. It must be noted that the clustering was only for analysis purposes and should not be understood to mean that the three clusters are unrelated. In reality, complex relationships and interdependence between all these exists, as detailed in the literature review.

Table 4-11: Overview of the clustering used for the seven water quality parameters

Clustering	Modelled Determinants
Physical	Water age and DO
Chemical	Chlorines (total, free and chloramine)
Biochemical	DOC and BDOC

Results from the analysis of the three functional groups is presented and discussed below. A wide range of determinants was considered, to ensure reasonable conclusions could be drawn from an analysis that was reasonably thorough. It must be noted that modelling each of these separately was not efficient and failed to consider the interactions between the seven parameters.

4.4.1 Physical determinants

As detailed above water age and dissolved oxygen (DO) were classified (only for purposes of this analysis) as “physical determinants”. Water age was an important determinant to consider as it regulates reaction times in a system. This is because water age measures how long water is retained (in hours) in a network (reservoir to end point), the exact quantity of time is largely dependent on: the physical design, operating rules, water demand and the water production rate. The water age is thus a direct measure of how much time reactions have to be completed; it is thus reasonable to associate high water ages with poor water quality.

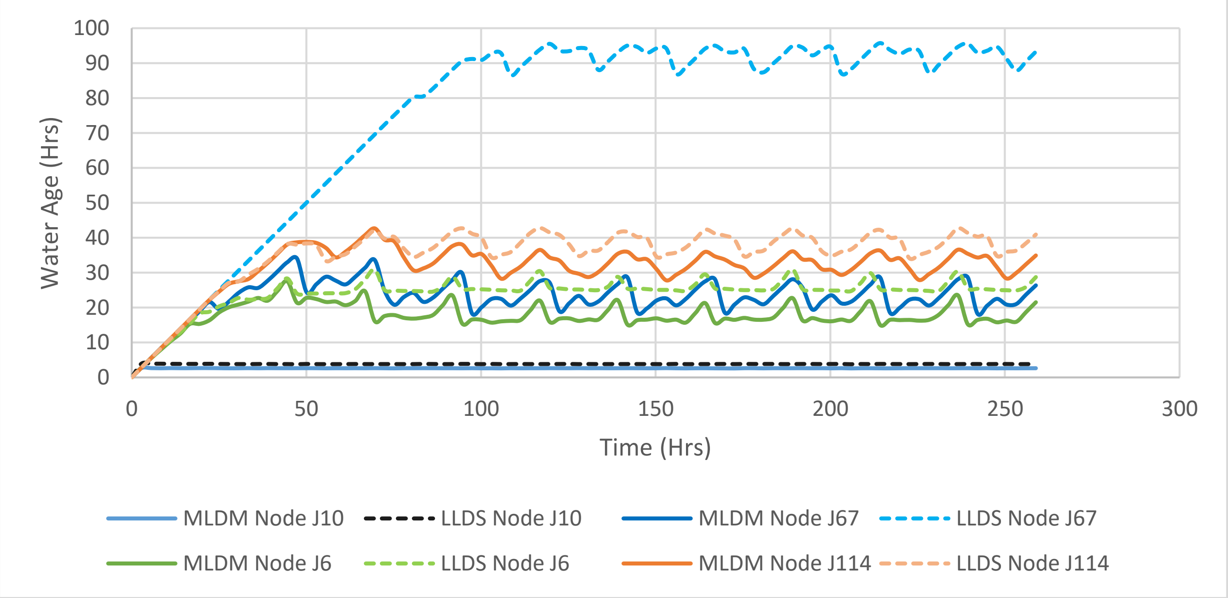


Figure 4-1: LLDS and MLDM water age comparison at nodes J6, J10, J67 and J114

Figure 4-1 above presents a comparison of water age at four points (sites), in the LLDS and the MLDM. There was no field data for water age collected, the comparison is thus only limited to the two models.

The retention times near the reservoir (node J10) were practically identical in both models, with a slight, but reasonable increase observed further from the reservoir as seen at nodes J6 and J114. The LLDS node J67 however, was an outlier with the average time in the MLDM of around 25-hours compared to 90-hours in the LLDS. The reason for this was not immediately clear as node J114 was approximately nearly identically distant from node J10 as node J67. With this considered, the LLDS node J67 results were considered an outlier, with an anomaly for further investigation and this was borne in mind for all subsequent analysis. A pattern that was consistent between both the LLDS and the MLDM, was that the retention times of the LLDS were all (J10, J6, J67 and J114) higher than those of the MLDM. This is expected as there were more pipes and nodes (representing demand points – 6160 connections in real system) in the MLDM (524 – 9% of actual) than the LLDS (121 – 2% of actual). More nodes likely reduce retention time across the system as there are more points for water to exit, all thing being equal in a closed system.

The next determinant that was considered was dissolved oxygen (DO). DO is an important proxy indicator for both chemical and biological water quality determinants as it is an indicator of which microorganisms may be present, affects corrosion chemistry and provides an indication on temperature levels (LeChevallier, 1998; Sarin, 2004; Kumar, 2012; Useh, 2017). Figure 4-2 below presents a comparison of DO levels between the two models and field data. The outlier that is node J67 in the LLDS is immediately visible as the abnormally high water age results in the lowest DO levels across all systems. LLDS node J67 had a modelled average DO of 4,9mg/l which was 14% lower than the field average of 5,6mg/l, contrasted with the MLDM node J67 that modelled at an average of 6.1mg/l, which was 9% higher than the field average.

Similar to the water age analysis, node J10 of both systems was identical at 8,518mg/l compared to a field average of 8.52mg/l this translated to an error of 0,02%. Further movement from the reservoir resulted in a slightly higher error as observed at node J67 of both models and node J6 (8.7% for the MLDM and -13,5% for the LLDS). With the exception of node J67, the MLDM produced consistently lower DO levels than the LLDS; when this is considered context of the water age analysis, the opposite was expected. The assumption was that the model with the higher water ages, would have lower DO levels as there is more reaction time, the fact that this is not the case warranted further investigation by considering the chemical and biochemical determinants as presented below. A key take-away from the exercise was that both the MLDM and the LLDS produced acceptable results when compared to the field data, suggesting that the EPS VCS models were both capable of modelling DO.

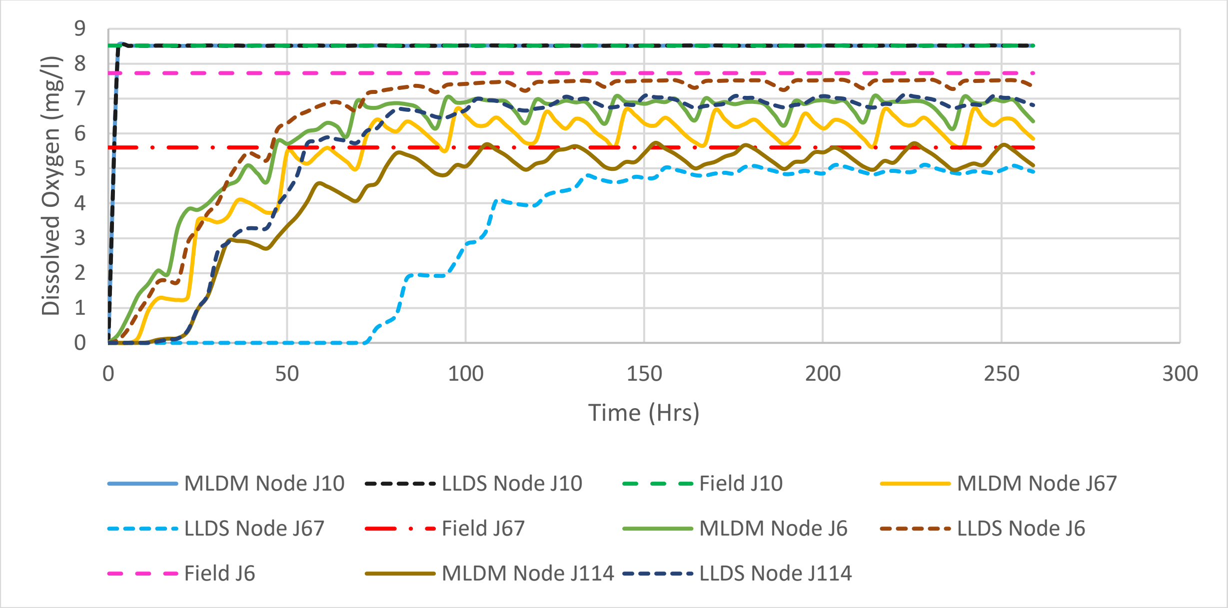


Figure 4-2: LLDS and MLDM dissolved oxygen levels comparison at nodes J6, J10, J67 and J114

4.4.2 Chemical determinants

In analysing the chemical determinants, the following were considered: total chlorine and chlorine residuals (free chlorine and chloramine). Chlorine was considered as it is a common disinfectant that is recognised and used across the world for primary and secondary disinfection (Woolschlager, 2000; AWWA, 2007; WHO, 2017). The system under consideration used and still uses chlorine for disinfection. Being so commonly used, the South African national standard (SANS241, 2015) also recognises chlorine (free chlorine and chloramines) as a disinfectant and prescribes lower and upper limits.

<u>Total chlorine</u>

Total chlorine is not directly regulated in the South African drinking water standard (SANS241, 2015), but is a combination of free available chlorine and the reactive chloramines (Schutte, 2006). The monitoring of total chlorine is valuable as it gives an indication of whether or not there are high levels of ammonia or ammonia-based compounds that are hazardous to humans. Figure 4-3 below presents the MLDM and LLDS total chlorine model results. The water age was a clear predictor for total chlorine levels for node J67. As expected, node J67 (highest water age), had the lowest total chlorine levels. However, this pattern did not hold for the other observations as the LLDS model produced comparatively higher total chlorine levels than the MLDM (though the LLDS had slightly higher water ages).

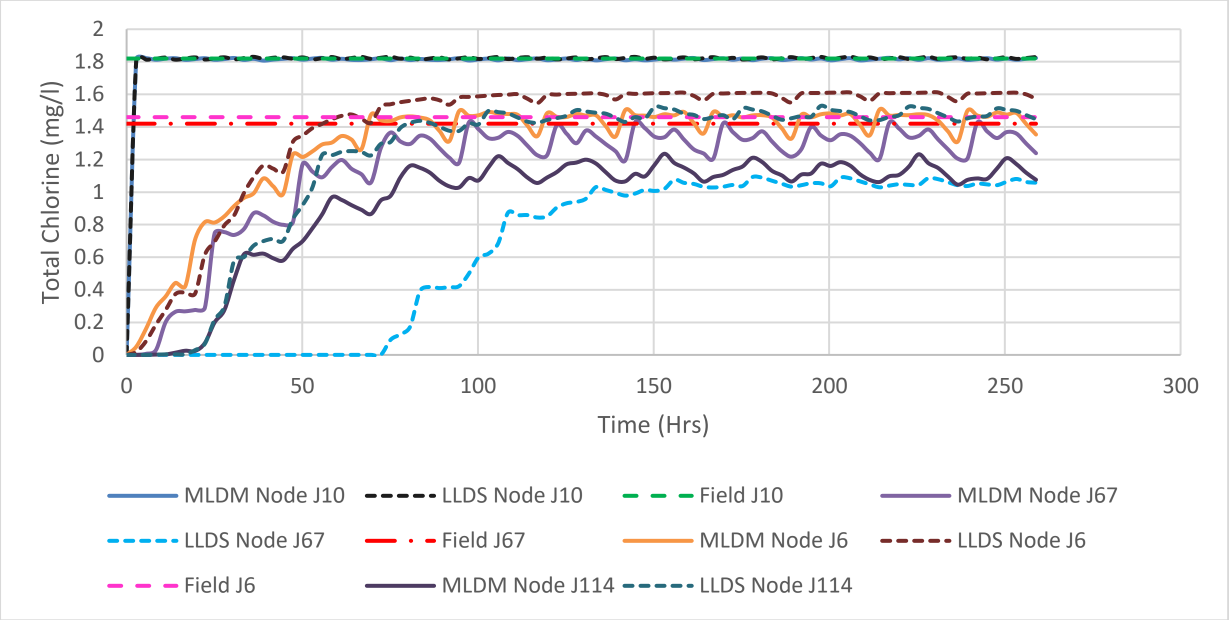


Figure 4-3: LLDS and MLDM total chlorine comparison at nodes J6, J10, J67 and J114

Free chlorine

Free chlorine generally represents residual chlorine in a system and is used as an indication of how safe the water is from pathogens. SANS (241:2015) prescribes a maximum free chlorine concentration of no more than 5mg/l but does not prescribe a minimum. However, its predecessor (SANS241:2011) prescribed a minimum of 0.2mg/l in line with international guidelines (WHO, 2011). From Figure 4-4 below, a majority of the nodes have free chlorine levels that are marginally higher than the 0.2mg/l minimum. The LLDS node J67 was the most accurate (error of 0.86%) and shows the free chlorine concentrations as dangerously low. The local regulator (DWS) publishes compliance results for this sampling point and they have often shown non-compliance, it was therefore assuring to observe that the model results correlated with what was happening in reality, from the perspective of the regulator.

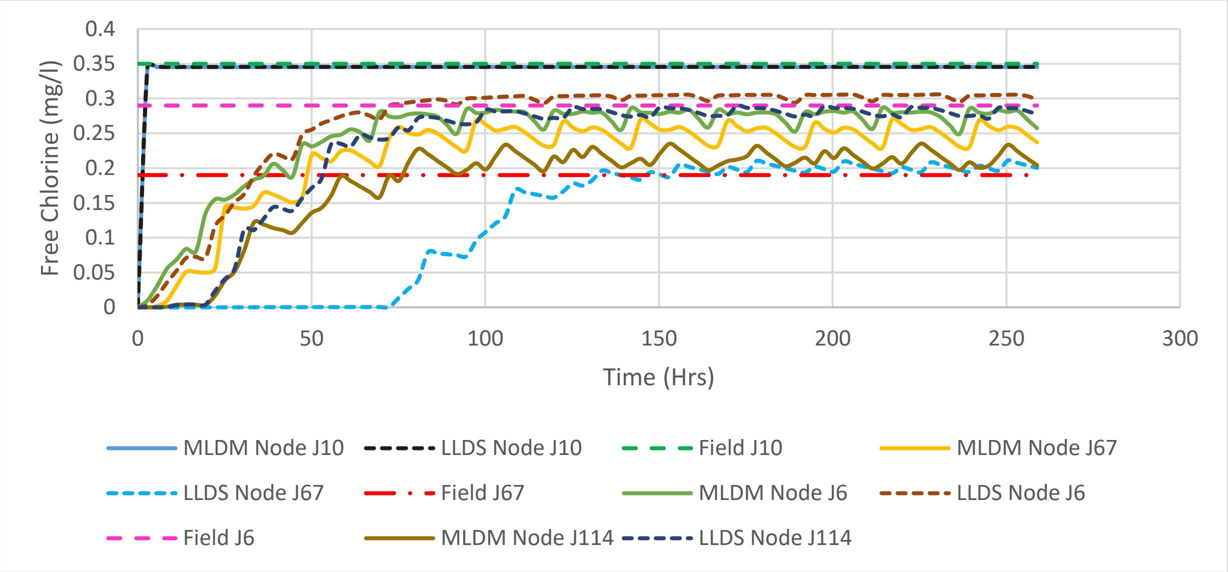


Figure 4-4: LLDS and MLDM free chlorine comparison at nodes J6, J10, J67 and J114

Chloramine

Monochloramines are used for secondary disinfection as they are less reactive than chlorine (Culligan, 2015). Chloramines were not measured on the field by Useh (2017), but the values were obtained from the difference between total chlorine and free chlorine as prescribed in literature (Schutte, 2006). Figure 4-5 below presents the chloramine levels in the system at various nodes. The modelled results were acceptably close to the field data (calculated as the difference between total chlorine and free chlorine).

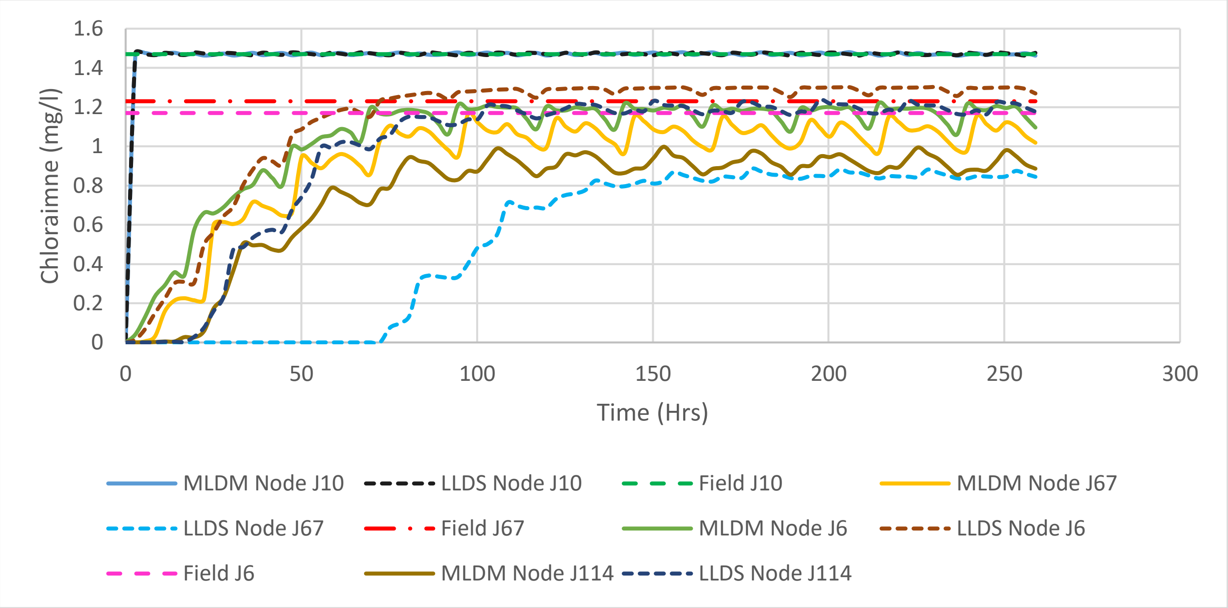


Figure 4-5: LLDS and MLDM chloramine comparison at nodes J6, J10, J67 and J114

Summary

Table 4-12 below presents a summary of the chlorine analysis. For all three chlorine related determinants the LLDS on average produced higher concentrations than the MLDM. The MLDM was on average more accurate than the LLDS, as the MLDM produced better results (total chlorine and chloramine) than the LLDS (free chlorine). The levels of accuracy produced by the LLDS in the free chlorine modelling were not matched by any other model analysed, this suggested that the LLDS model was very suitable for free chlorine analysis in this specific system. While the LLDS model had longer water age (hours), it had lower chlorine consumption (decay) across the board. This is possibly due to the fact that the LLDS Global Bulk Coefficient (-0.15) was lower than that of the MLDM (-0.40); and this was important as the Global Bulk Coefficient is the rate at which chlorine decays due to reactions in the bulk flow in all pipes (Rossman, 2000).

Table 4-12: Chlorines analysis summary model to field errors for nodes J6, J10, J67 and J114

Total chlorine				
Nodes:	**J10**	**J67**	**J6**	**J114**
LLDS Error (Model: Field)	0.12%	-26.32%	6.14%	N/A
MLDM Error (Model: Field)	-0.21%	-9.04%	-4.16%	N/A
Free chlorine				
LLDS Error (Model: Field)	-1.26%	-0.86%	0.93%	N/A
MLDM Error (Model: Field)	-1.33%	29.13%	-7.82%	N/A
Chloramine				
LLDS Error (Model: Field)	0.12%	-33.01%	6.96%	N/A
MLDM Error (Model: Field)	0.06%	-14.53%	-2.36%	N/A

Error (Model: Field) represents the percentage difference between EPANET model output and actual field data.

Overall, all the models produced acceptable results in general. The low levels of free chlorine were below expected international norms and a cause for concern as consumers in and around that area may have been receiving water with inadequate levels of residual pathogen protection (WHO, 2011).

4.4.3 Biochemical determinants

An understanding of the quantities of organic matter in a distribution system is important, as this is related to biostability and ultimately is a proxy for whether or not the water is safe for consumption (Schutte, 2006; Biyela 2010). There are many ways to measure the quantities of organic matter in water, but for this study dissolved organic carbon (DOC) and biodegradable organic carbon (BDOC) were used, as this was the data collected from the field. Measuring DOC and BDOC does not tell us anything specific about the type of organic matter present, and the tests are also expensive to conduct, for these reasons they (DOC and BDOC) are not part of the SANS 241 (2015) standard (only considers total organic carbon as C). Presented below are results for the DOC and BDOC analyses, primarily presented for the purposes of determining the extent to which the LLDS and the MLDM can model these (as they are not monitored in RSA).

Dissolved organic carbon (DOC) and biodegradable organic carbon (BDOC)

The DOC results became less accurate the further the testing site from the reservoir as can be seen in Figure 4-6 below. This pattern was also observed with the BDOC results as seen in Figure 4-7 below. Table 4-13 below summarises the results for both models as seen below, with concentrations for DOC at node J67 being concerningly inaccurate. This is because the reaction coefficients used were not appropriate for the DOC and the BDOC, as these are typically derived from field tests and are thus network specific. DOC and BDOC are also significantly influenced by the biofilm inside pipelines and react to and with other parameters, all of these considerations were beyond the scope of this study.

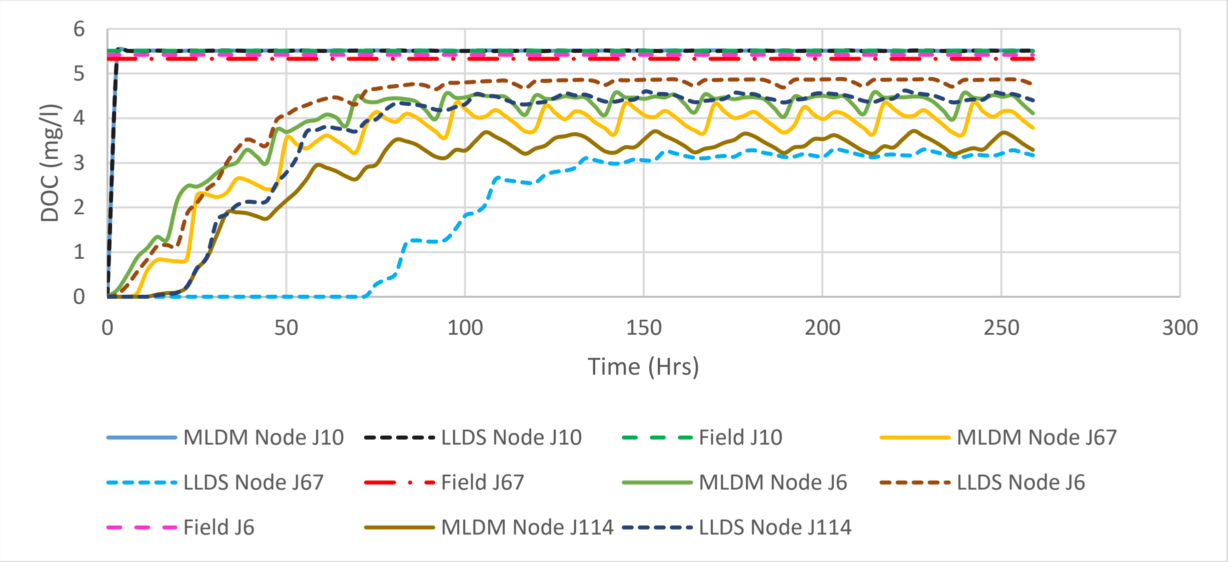


Figure 4-6: LLDS and MLDM DOC comparison at nodes J6, J10, J67 and J114

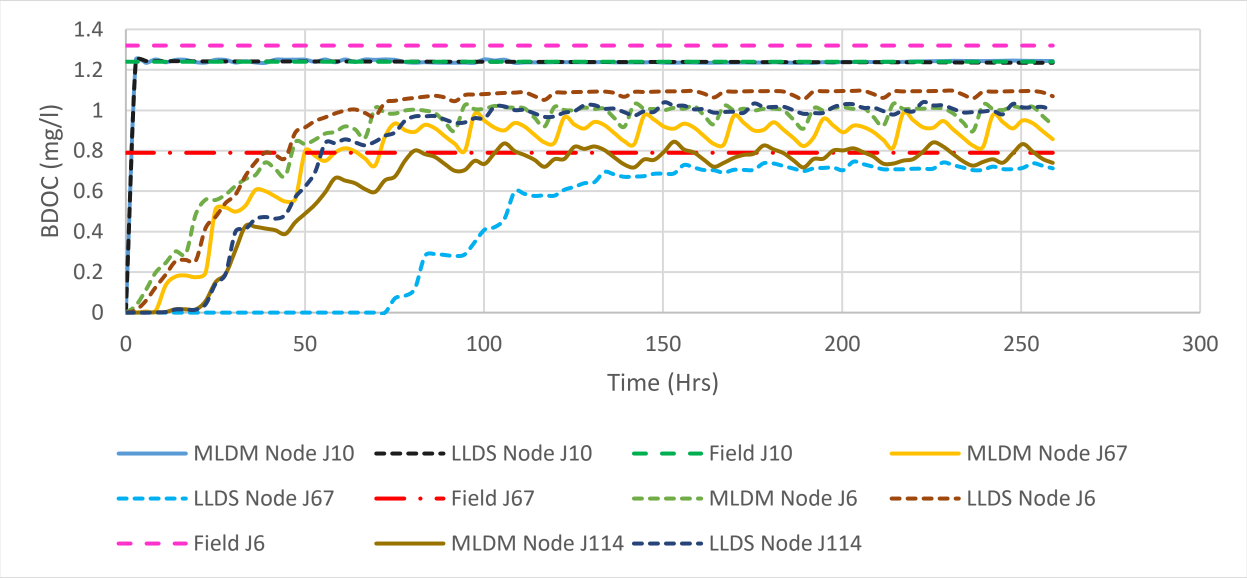


Figure 4-7: LLDS and MLDM BDOC comparison at nodes J6, J10, J67 and J114

Table 4-13: Organic carbons model to field error summary analysis

Dissolved Organic Carbon (DOC)				
Nodes:	**J10**	**J67**	**J6**	**J114**
LLDS Error (Model: Field)	-0.02%	-40.64%	-11.69%	N/A
MLDM Error (Model: Field)	-0.03%	-26.11%	-18.88%	N/A
Biodegradable Organic Carbon (BDOC)				
LLDS Error (Model: Field)	0.06%	-10.25%	-18.55%	N/A
MLDM Error (Model: Field)	-0.06%	12.57%	-25.16%	N/A

Error (Model: Field) represents the percentage difference between EPANET model output and actual field data.

The key observation from the errors in Table 4-13 above was that the level of hydraulic definition did not have a direct correlation to the accuracy of the models. This could be observed with the DOC model at nodes J67 and J6 respectively, where neither model was outright more accurate. In the instance of the BDOC, the LLDS was marginally more accurate than the MLDM. The complexity of modelling organic matter in a drinking water network, as detailed in literature, was also noted and provided one explanation for these results (Woolschlager, 2000; Biyela, 2010; Culligan, 2015). A proven solution to manage this complexity is the determination of system specific water quality reaction co-efficients from field data.

4.5 Scenario analysis 2 – water quality and level of calibration

The previous chapter compared the LLDS with the MLDM; this chapter was concerned with internal comparison within the respective models. As outlined in the calibration and validation procedure, model development progressed from steady state (SS) models to extended period simulations (EPS). The model progression was as outlined below for both LLDS and MLDM:

a) Steady State – Normal Flow Conditions – SS_NFC

b) Steady State – High Flow Conditions – SS_HFC

c) Extended Period Simulation – Known Control Status – EPS_KCS

d) Extended Period Simulation – Variable Control Status – EPS_VCS

From literature it was known that an increase in the level of calibration from SS_NFC to EPS_VCS, should translate to more accurate model outputs across the board (Ormsbee *et al.*, 1997; Hirrel, 2008; Water NZ, 2021). The purpose of this section was to merely confirm this for the models developed. The analysis utilised Total Chlorine from November 2015. The effect of model calibration was examined at two locations (nodes J6 and J67), as these were the furthest from reservoirs and would produce useful results for comparison with field data.

4.5.1 LLDS model outputs

Figures 4-8 and 4-9 below present the results of the LLDS analysis.

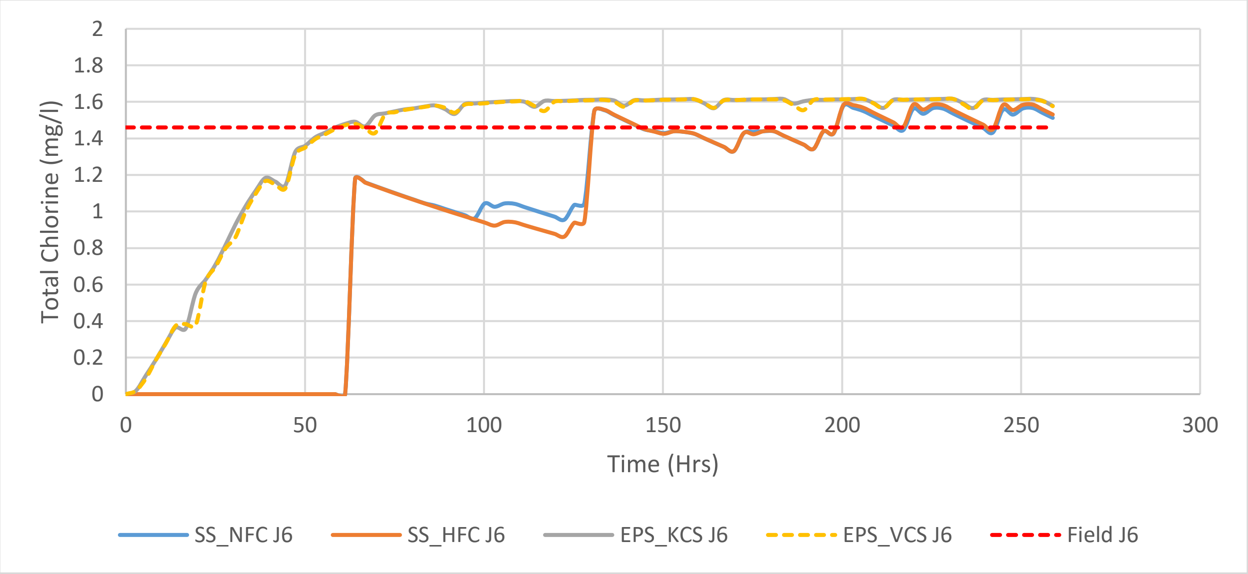


Figure 4-8: Total chlorine at Node J6 for differing levels of LLDS model calibration

At node J6 there was no significant variance in the error as it moved from -9% to +9%. Node J67 produced the most significant improvement; starting at -40% and gradually improving to -26%, with slight improvement with each model as expected from literature.

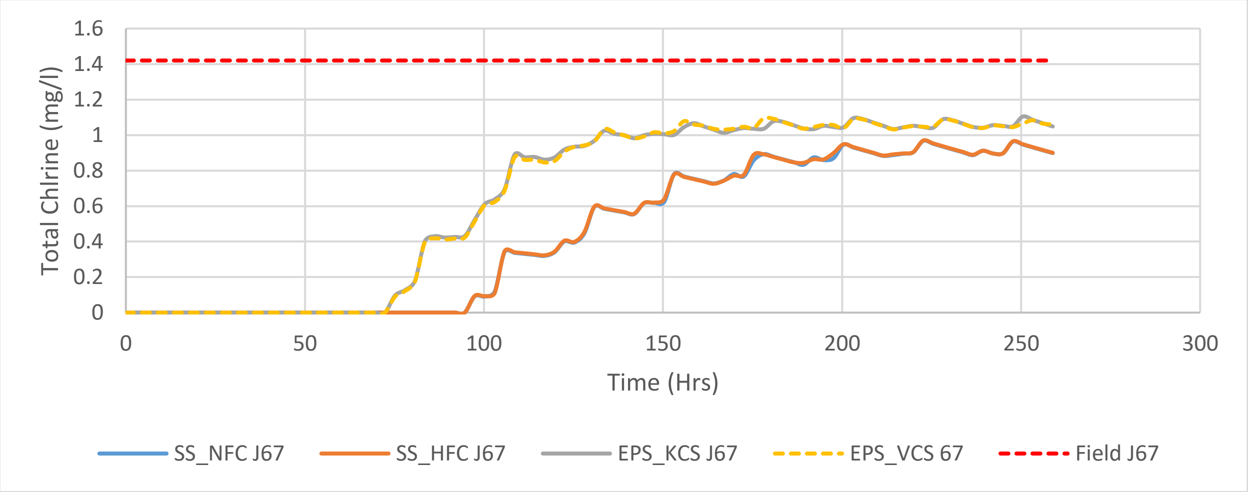


Figure 4-9: Total chlorine at Node J67 for differing levels of LLDS model calibration

4.5.2 MLDM outputs

Figures 4-10 and 4-11 below present the results of the MLDM analysis. A significant improvement at both sites was observed through the percentage error reduction. For node J6, there was an improvement from a 100% (SS_NFC) to an error of -0.82% (EPS_VCS). For node J67, there was an improvement from -79% (SS_NFC) to -7% (EPS_VCS). These results were more closely aligned to the expected outputs, as guided by literature.

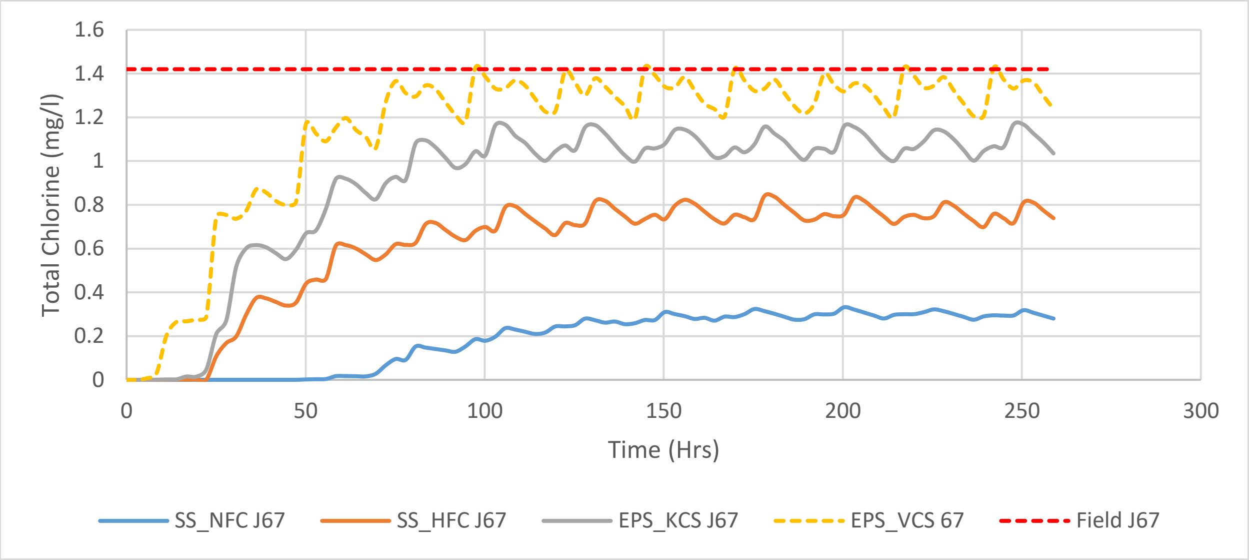


Figure 4-10: Total chlorine at Node J67 for differing levels of MLDM model calibration

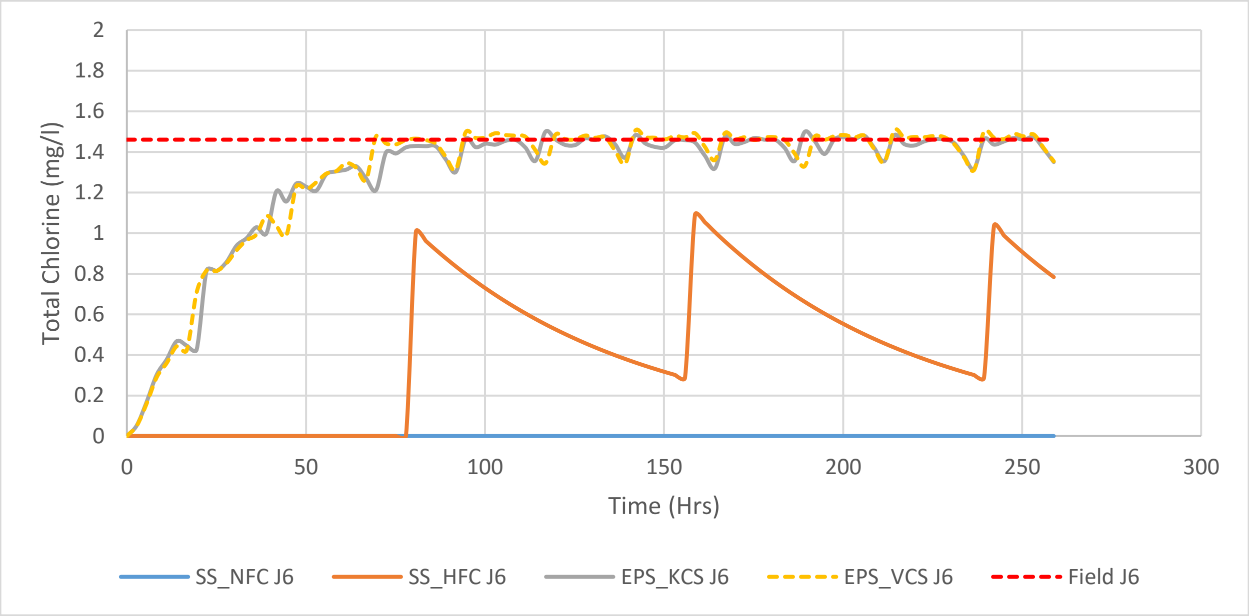


Figure 4-11: Total chlorine at Node J6 for differing levels of MLDM model calibration

4.5.3 Summary

Table 4-14 below summarises the results for both models, and it was clear that the level of calibration played a direct role in how accurately water quality (total chlorine in this case) could be modelled. It was also clear that the steady state models (SS_NFC and SS_HFC) were not suitable for water quality modelling, as stated in literature (EPA, 2005; Water NZ 2021).

Table 4-14: Summary overview of model accuracy at different levels of calibration

LLDS at Node J6				
Model:	SS_NFC	SS_HFC	EPS_KCS	EPS_VCS
Error (Model: Field)	-8.92%	-9.61%	9.24%	8.93%
LLDS at Node J67				
Model:	SS_NFC	SS_HFC	EPS_KCS	EPS_VCS
Error (Model: Field)	-39.86%	-39.68%	-26.37%	-26.13%
MLDM at Node J6				
Model:	SS_NFC	SS_HFC	EPS_KCS	EPS_VCS
Error (Model: Field)	100%	-57.34%	-1.72%	-0.82%
MLDM at Node J67				
Model:	SS_NFC	SS_HFC	EPS_KCS	EPS_VCS
Error (Model: Field)	-79.12%	-46.21%	-24.04%	-7.18%

Error (Model: Field) represents the percentage difference between EPANET model output and actual field data.

The combined effect of model level of calibration and hydraulic detail are most clearly shown in Figure 4-12 and 4-13 below, where it is clear that both level of hydraulic detail and level of calibration contribute to water quality modelling accuracy. At node J67 it could be observed that the level of hydraulic detail was more significant than the level of calibration. This was because the second best calibrated MLDM (EPS_KCS) was more accurate (with respect to error), than the most calibrated LLDS model (EPS_VCS). While this effect was not equally exaggerated at node J6, this relationship was still clear, with an error of -1.72% for the MLDM_EPS_KCS (second most calibrated MLDM model after the EPS_VCS) compared with the LLDS_EPS_VCS (most calibrated LLDS model) error of 8.93%.

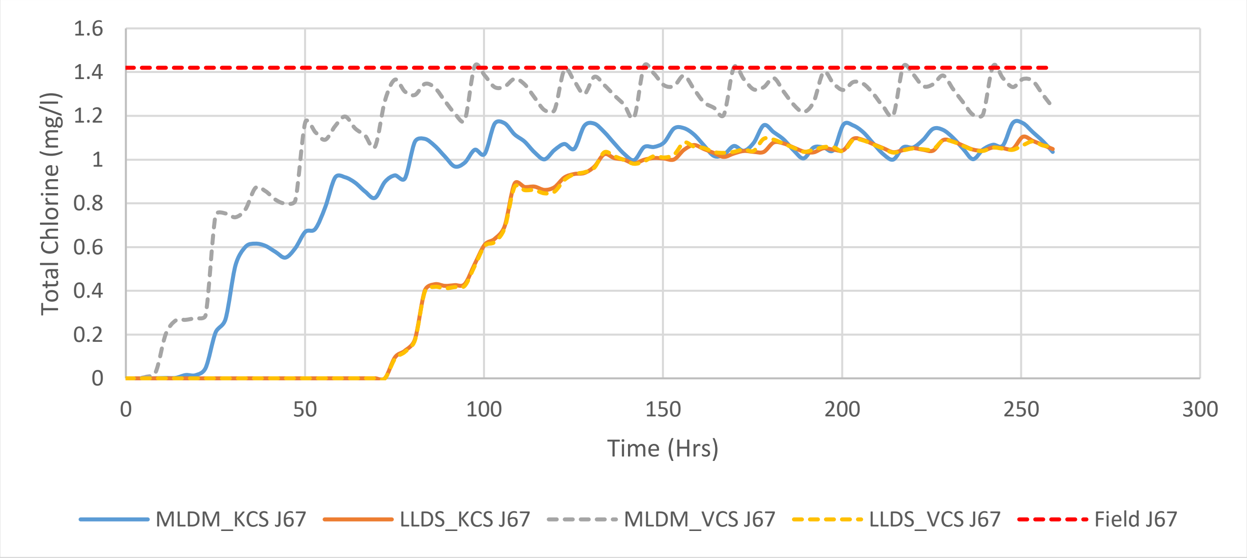


Figure 4-12: LLDS and MLDM EPS model accuracy comparison at node J67

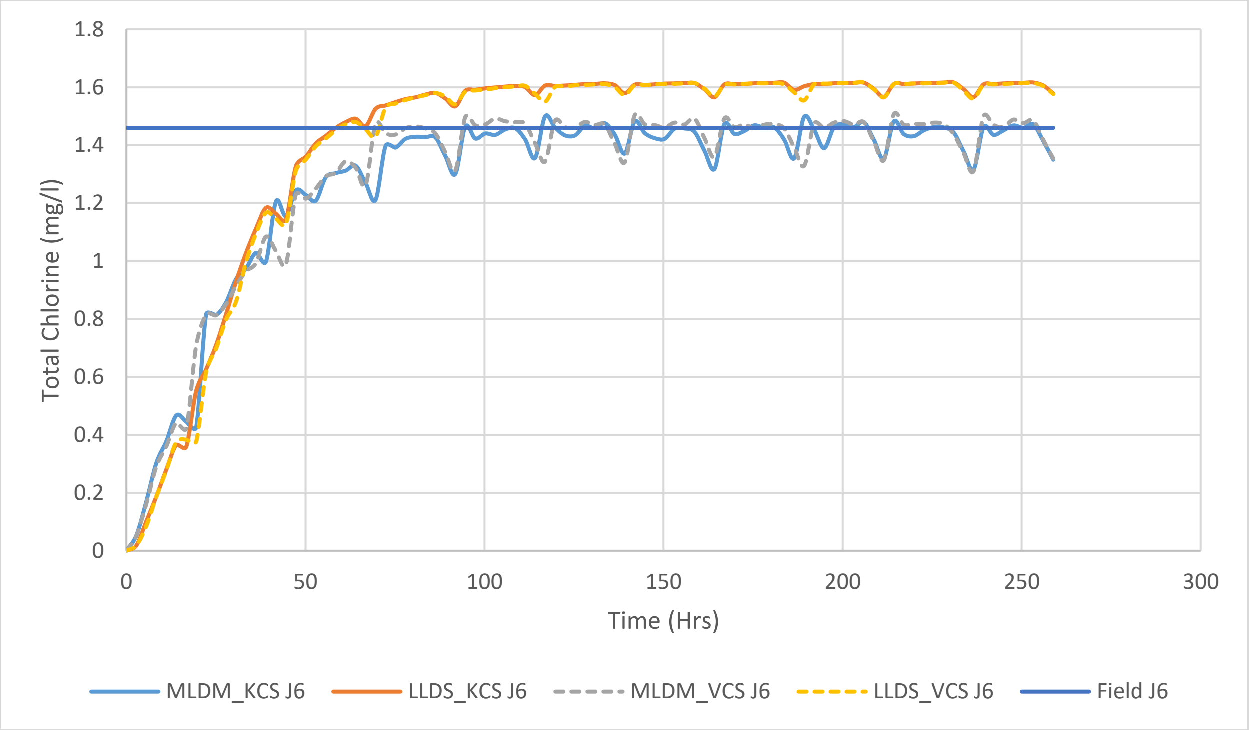


Figure 4-13: LLDS and MLDM EPS model accuracy comparison at node J67

4.6 Scenario analysis 3 – water quality and pipe age and material

This section considered the impact of pipe age on water quality (total chlorine levels) in both systems (LLDS and MLDM). In EPANET 2.0 pipe age and pipe material are both represented by the roughness co-efficient (C-value). In undertaking these tests pipe materials that were used in the real system were considered and these are: iron, plastics (PE, PP and PVC) and steel. Table 3-4 above shows the C-values for a range of corresponding pipe materials and ages. The typical range was between 45 (badly corroded pipe) and 150 (new pipe). Noting this, a range between 50 (badly corroded cast iron) and 150 (new plastics, new concrete, new cement and new steel) was used and this reasonably covered the common pipe materials and their age range.

4.6.1 LLDS analysis of pipe age and water quality

Analysis of the LLDS model was undertaken at nodes J67 and J6 as these were the two furthest points from the reservoir that had field data collected. Figures 4-14 and 4-15 below present the change in total chlorine levels with increasing pipe age (decreasing c-value). As seen below, pipe age (and by extension material) had a limited impact on water quality for plastic, smooth concrete and FRC pipes, as the c-value for new and badly cored pipes varies between 150 and 100; this equated to a variance that was not more than 6% as seen in Table 4-15 below. It could also be seen that all select pipe materials (plastics, smooth concrete, cement, steel and iron) were relatively similar up to the age of 50 years (C-value range between 150 and 90); where material difference was visible was in the instance of badly corroded steel, iron and clay pipes.

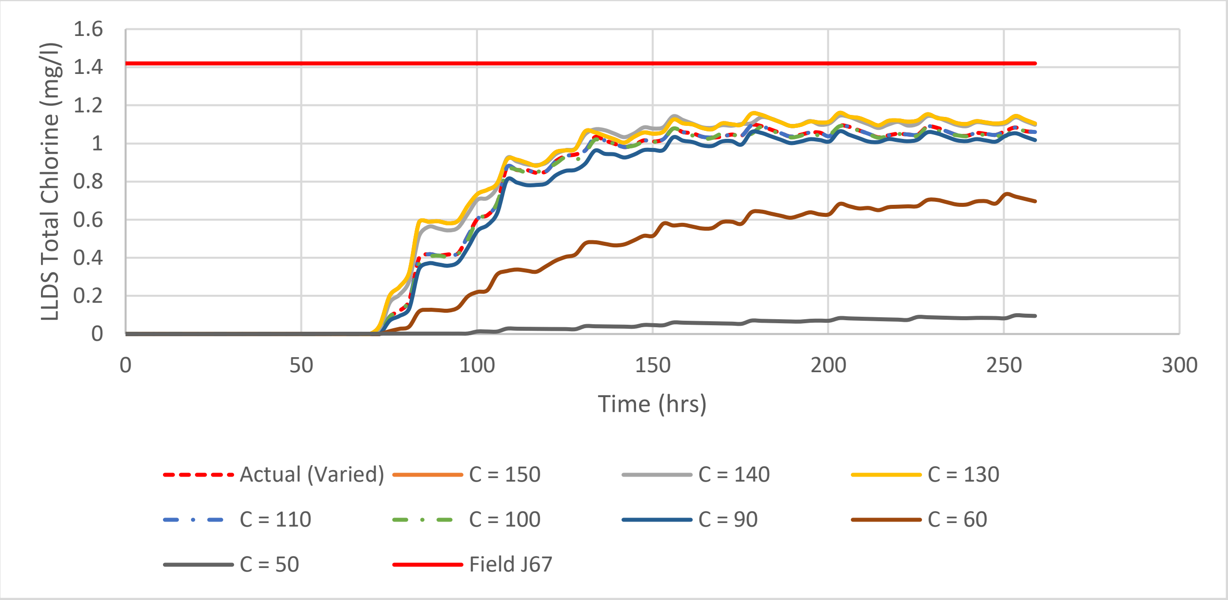


Figure 4-14: LLDS total chlorine change with pipe age at node J67

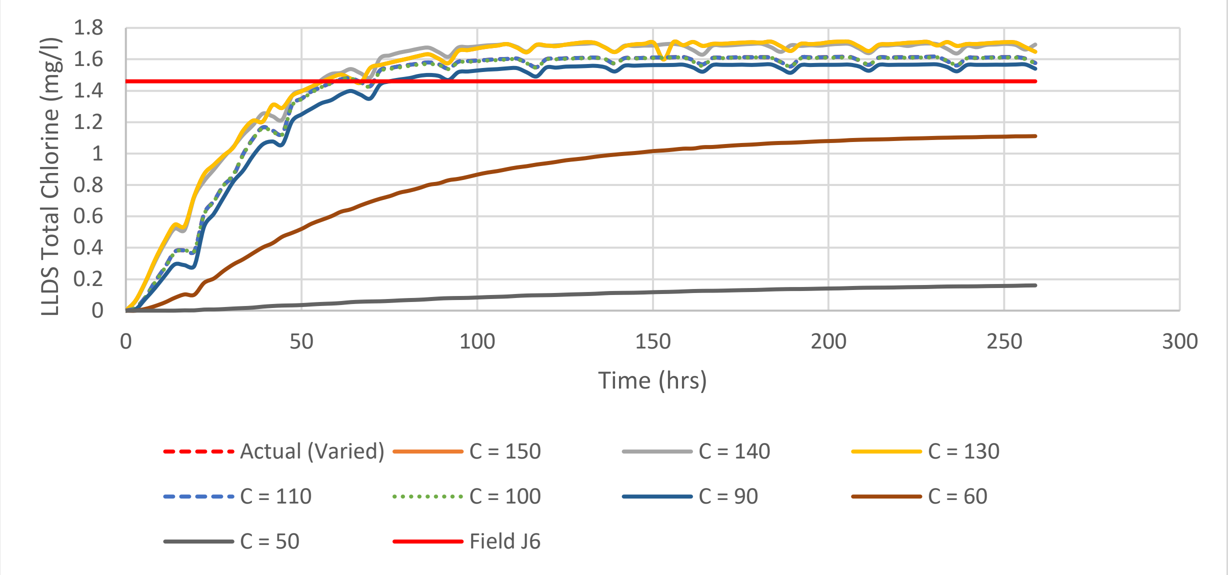


Figure 4-15: LLDS total chlorine change with pipe age at node J6

Table 4-15 below quantifies the variances that are visually presented in Figures 4-14 and 4-15 above. The first consideration (represented by "Error 1") was the difference between a particular C-value (representing a specific pipe material and age) and the field measurement at the two nodes (J67 and J6). No large (greater than 10%) changes were observed for materials up to the age of 50 years; with material differences only visible for badly corroded steel and cast iron.

Table 4-15: Summary overview of C-value change impact on LLDS model accuracy

LLDS at Node J67									
Roughness	**C: Actual**	**C:150**	**C:140**	**C:130**	**C:110**	**C:100**	**C:90**	**C:60**	**C:50**
Error 1*	-26.1%	-22.3%	-22.4%	-22.3%	-26.1%	-26.5%	-28.8%	-56.2%	-95.1%
Error 2**	0.0%	5.2%	5.1%	5.2%	0.0%	-0.51%	-3.7%	-40.7%	-93.4%
LLDS at Node J6									
Error 1*	9.9%	15.9%	15.3%	15.9%	9.9%	9.7%	6.7%	-27.0%	-90.6%
Error 2**	0.0%	5.5%	4.9%	5.5%	0.0%	0.2%	-2.9%	-33.6%	-91.4%

*Error 1 = difference between modelled results and field data

**Error 2 = difference between calibrated and validated C-value and selected C-value

The second consideration (represented by "Error 2") was the difference between the actual C-value (a combination of C-values from the calibrated and validated model, that represent the real pipe network) and specific C-values ranging from 150 to 50 as seen in the table and figures above. The Error 2 analysis confirmed the following:

- There was no significant difference (Error 2 is less than 5.5%) between new and badly corroded pipes for plastics (PE, PP and PVC) and smooth concrete / cement pipes.
- Badly corroded pipes for steel, cast iron and clay have a material impact on the chlorine levels as seen in the table above for C-values of 60 and 50; Clay is C=45 and would be expected to have an even greater error than C=50 and was therefore not modelled.
- Error 2 values for C-values ranging between 110 and 90 produce more accurate results, this suggests that the real pipe network has materials and roughness that more closely correspond with these values. This suggests that the real pipe network has the following: majority roughness 100 (badly corroded smooth concrete and FRC and 50 years old steel) and cast iron that is between 25 and 50 years old.

4.6.2 MLDM analysis of pipe age and water quality

In the analysis of the MLDM, the same approach as that followed in the LLDS analysis was followed, to ensure consistency and enable comparison between the two model results. The nodes considered were J67 and J6 (two furthest from the reservoir, with field data available). Figures 4-16 and 4-17 below present the modelled outputs, showing a decrease in total chlorine levels, as the pipe roughness increases (represented through a decrease in C-value). In contrast to the LLDS, changes in the C-value resulted in noteworthy changes (greater than 10%) in total chlorine concentrations as quantified through Error 1 and Error 2, presented in Table 4-16 below.

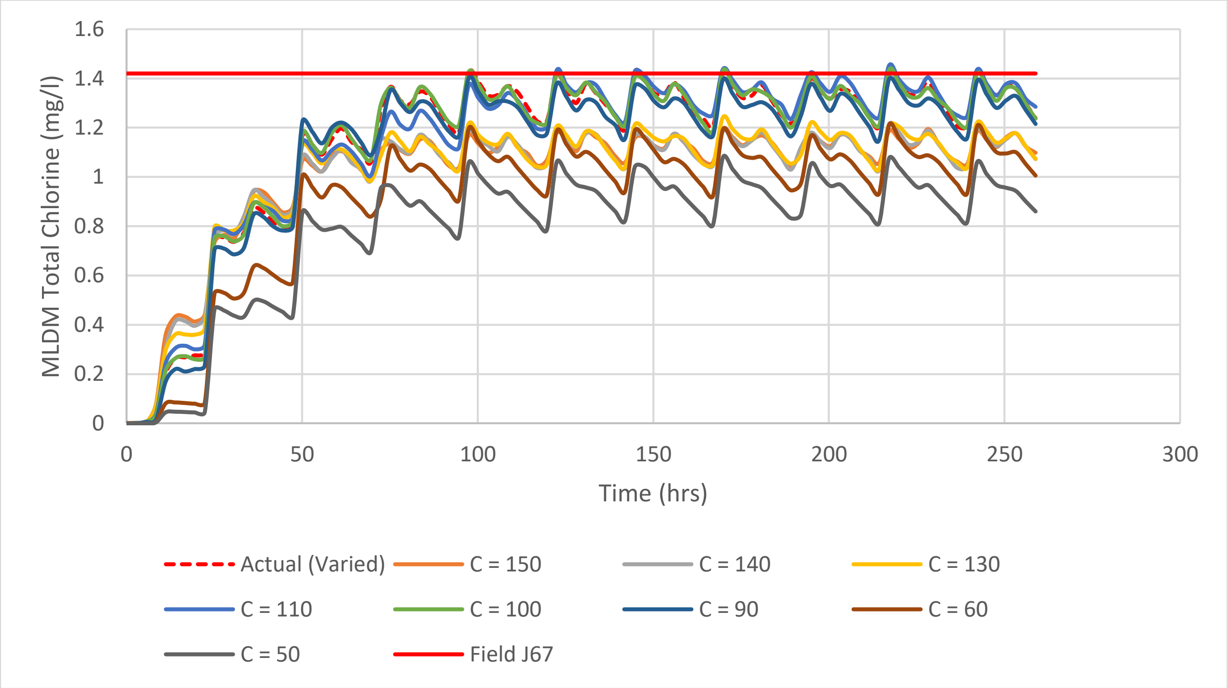


Figure 4-16: MLDM total chlorine change with pipe age at node J67

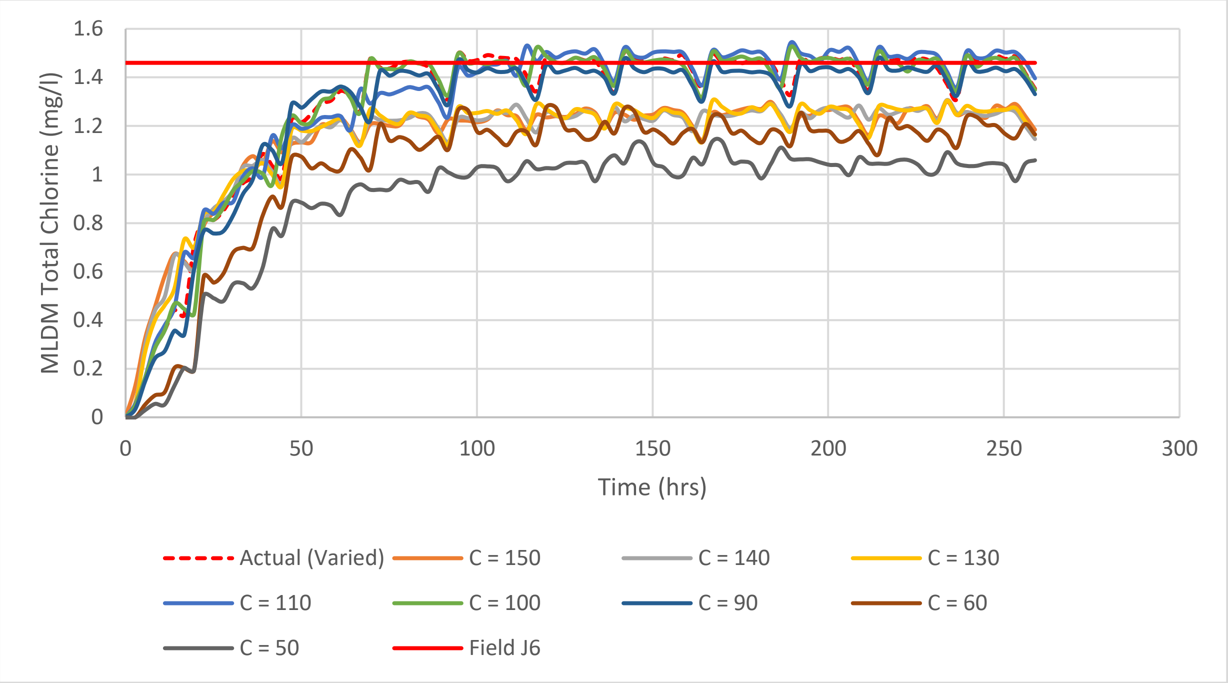


Figure 4-17: MLDM total chlorine change with pipe age at node J6

A closer analysis of Figures 4-16 and 4-17 above and Table 4-16 below shows that the actual network contains few: new and 25 years old plastic (PE, PP and PVC), smooth concrete and Fibre Reinforced Cement (FRC) as Error 2 at both nodes 67 and 6 is greater than 13%. The new to 25 years old materials are represented by C-values that range between 150 and 130.

For C-values between 110 and 90 at both nodes, Error 2 was not significant (less than 3%). This meant that the actual age (and material) of the real network was within this range, as there was a strong correlation between these results and the actual C-value used. Consistent with the LLDS model, badly corroded pipes (C-values of 60 and below) produced significant Error 1 and Error 2 outputs, indicating that the actual network likely had few pipes that were badly corroded of steel, cast iron and clay material. The above inferences were valid as all other variables were held constant.

Table 4-16: Summary overview of C-value change impact on MLDM model accuracy

MLDM at Node J67									
Roughness	**C: Actual**	**C:150**	**C:140**	**C:130**	**C:110**	**C:100**	**C:90**	**C:60**	**C:50**
% Error 1*	-7.2%	-20.5%	-20.7%	19.8%	-5.6%	-7.3%	-9.9%	-25,2%	-33.9%
% Error 2**	0%	-14.4%	-14.6%	-13.6%	-1.7%	-0.1%	-2.9%	-19.4%	-28.9%
MLDM at Node J6									
% Error 1*	-0.8%	-14,7%	-14.7%	-14.4%	1.23%	-0.6%	-3.2%	-19.3%	-28.2%
% Error 2**	0%	-13.9%	-13.9%	-13.6%	2.1%	0.2%	-2.4%	-18.6%	-27.6%

*Error 1 = difference between modelled results and field data
**Error 2 = difference between calibrated and validated C-value and selected C-value

4.6.3 Summary: model definition, pipe age and water quality

Figures 4-18 and 4-19 below present both the LLDS and MLDM results at both nodes (J67 and J6) for easier comparison. A careful consideration of the results of both the LLDS and the MLDM shows that the C-value (representing both pipe age and material) is an important parameter to consider in water quality modelling. The LLDS model showed less sensitivity to new, 25 years old and 50 years old pipes than the MLDM model; this was easily observed through the scale of Error 2 values. One possible reason for this, was that the LLDS Error 1 was already much higher than that of the MLDM.

The second critical observation was that the model results could be used to estimate the pipe age and material of the real network, by observing the errors (1 and 2) as the C-value is carefully changed. Both the LLDS and the MLDM showed the lowest Error 2 at C-values: 110, 100 and 90. This meant that a majority of the pipes in the real network could be described by these c-values i.e. badly corroded smooth concrete, fibre reinforced cement, steel that was nearing 50 years of age and cast iron between 25 and 50 years of age. Knowing the pipe materials and estimated age from the responsible utility, this was in fact the case. The MLDM produced more accurate results than the LLDS, but the LLDS results shows similar patterns and trends as the MLDM, suggesting that the model was useful for this purpose. Only the EPS_VCS models were used and the Steady Stade models were not considered, as high levels of accuracy were required as guided but literature (Ormsbee *et al.*, 1997; ECAC, 1999; EPA, 2005; Water NZ, 2021).

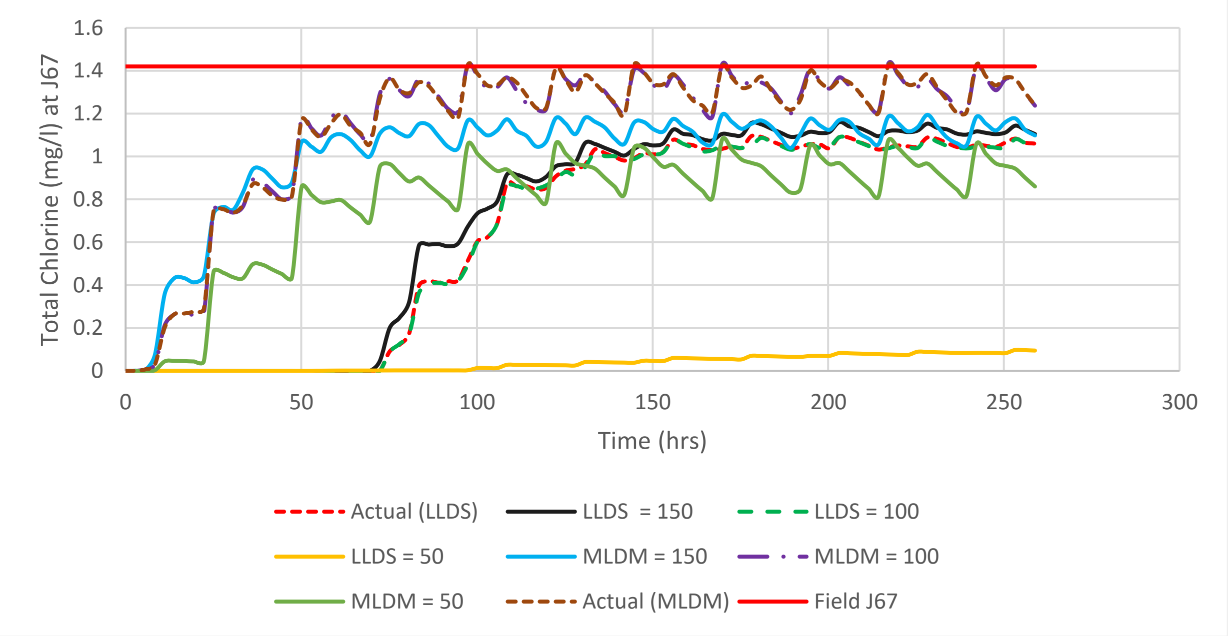


Figure 4-18: LLDS and MLDM total chlorine change with pipe age at node J67

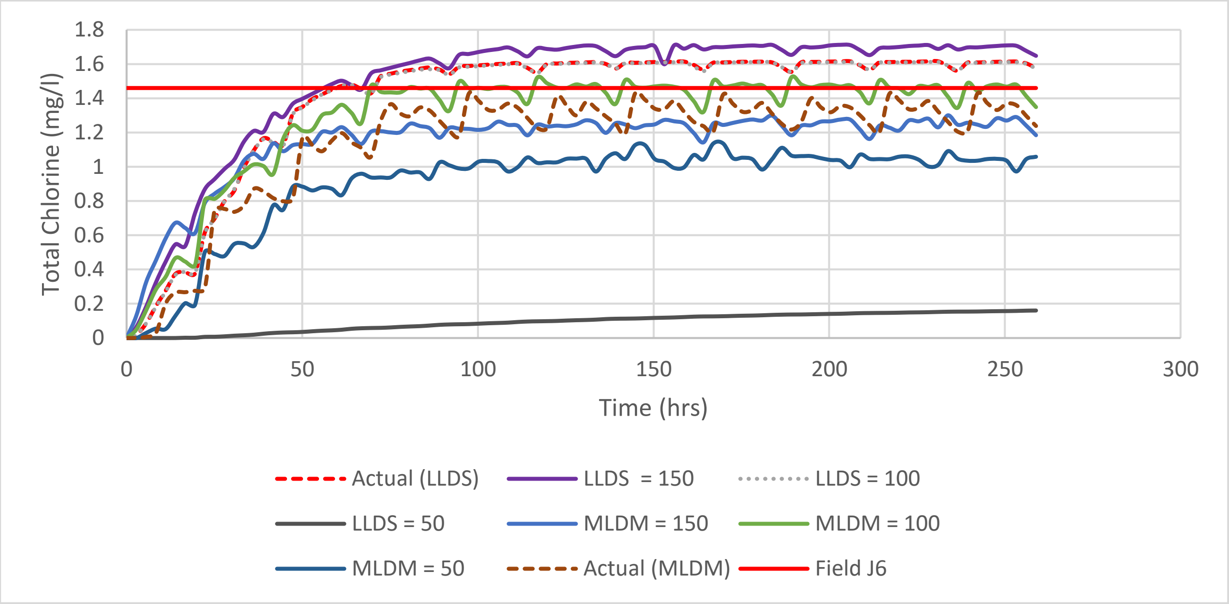


Figure 4-19: LLDS and MLDM total chlorine change with pipe age at node J6

4.7 Scenario analysis 4 – water quality and water demand pattern

This section examined water quality (total chlorine) sensitivity to changes in the demand pattern. This was done to achieve one of the research objectives. Demand patterns represent the general consumption patterns of users in a particular supply area. Like with other hydraulic and water quality parameters, the extent to which the demand patterns can be precisely known is dependent on a lot of factors, chief among which is available resourcing and the purpose of the collection of the demand information; in either case it is important to establish the extent to which demand patterns affect water quality.

For this study, four demand patterns were considered as listed below:

- Pattern 1 = P1 = Van Zyl, 1996 (this is the demand pattern used in the calibrated models)
- Pattern 2 = P2 = Loureiro *et al.*, 2006
- Pattern 3 = P3 = Van Zyl, 2001
- Pattern 4 = P4 = Default pattern

Figure 4-20 below shows the peak factors for each of these demand patterns.

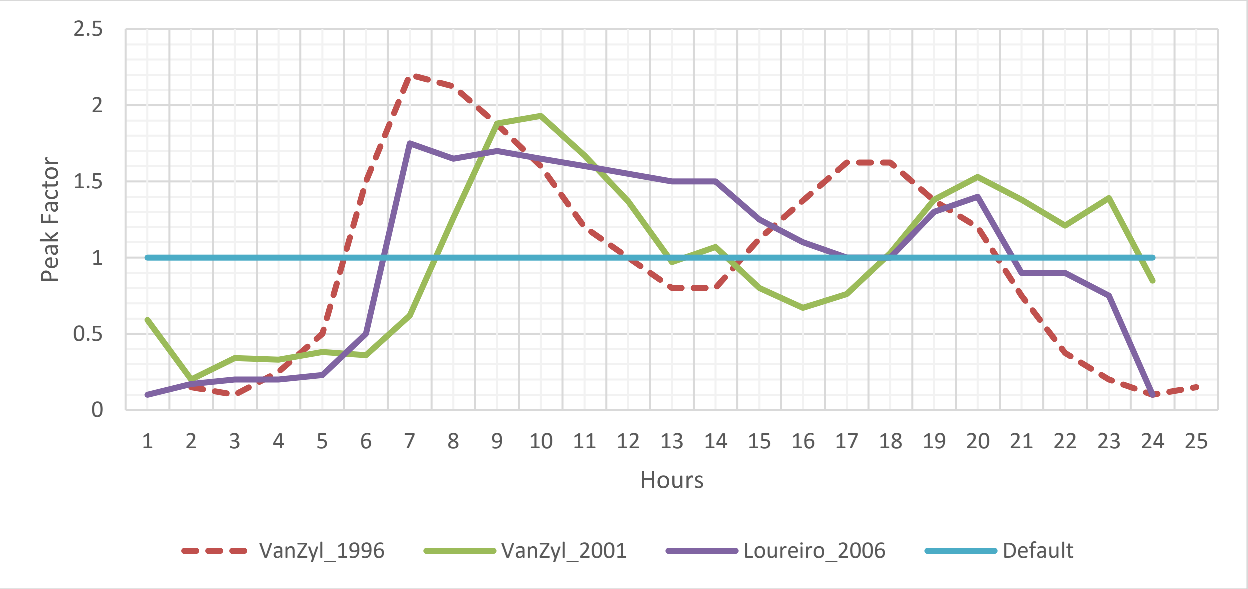


Figure 4-20: Four peak factors used for water demand patterns

4.7.1 Impact of demand pattern on water age

Water age is a critical factor in water quality modelling, Figures 4-21 and 4-22 below present the modelled water ages for both the LLDS and MLDM at nodes J67 and J6, for all four demand patterns. From these figures it is clear that the selected demand pattern has minimal impact on the water age. This is expected in a pressurised system as the individual and varied demands are met (and replaced) almost instantly and this prevents significant changes to water age. It is also important to note that the demand pattern is for a 24-hour period (diurnal), while water quality analysis is undertaken over several days (extended period simulations), the pronounced changes in peak demand at different hours of the day can appear insignificant, as is the case with Figures 4-21 and 4-22 below, but should not be assumed to have no impact. To further explore this, analysis was undertaken as presented below.

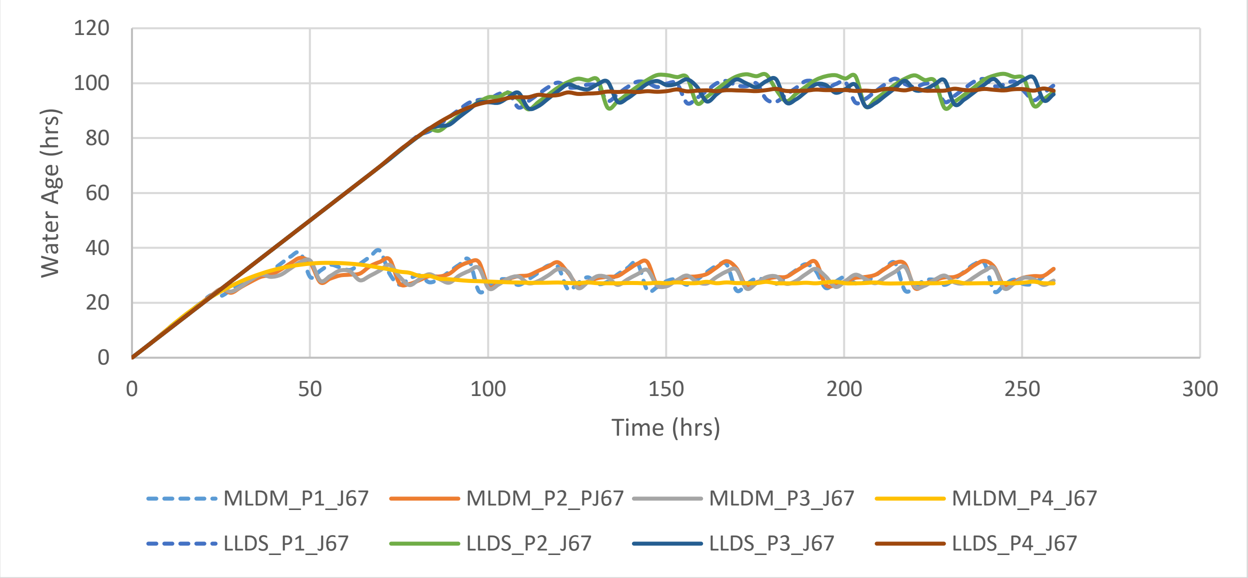


Figure 4-21: LLDS and MLDM water ages for corresponding water demand patterns at node J67

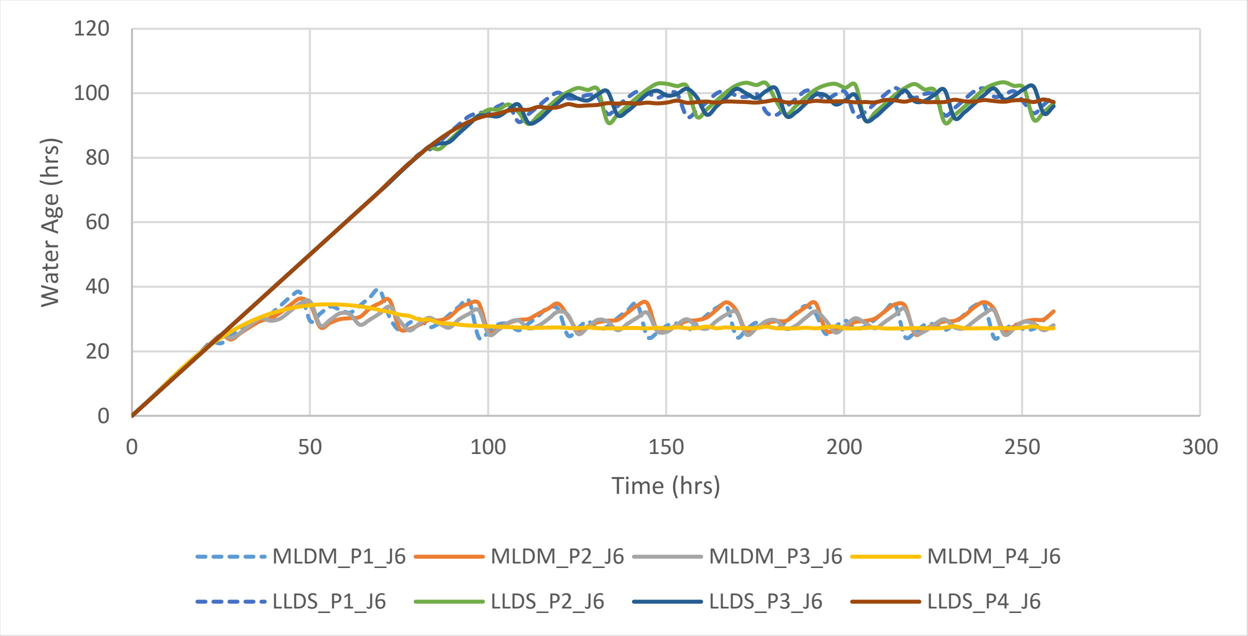


Figure 4-22: LLDS and MLDM water ages for corresponding demand patterns at node J6

4.7.2 LLDS analysis of demand pattern impact on water quality

Analysis of the LLDS was undertaken at nodes J67 and J6 (furthest points from reservoir, with available field data). Figure 4-23 below presents the analysis results and shows very minimal difference between the four demand patterns at both J67 and J6. This observation is consistent with the negligible changes in water age for the same nodes and the same demand patterns as presented in Figure 4-21 and 4-22 above.

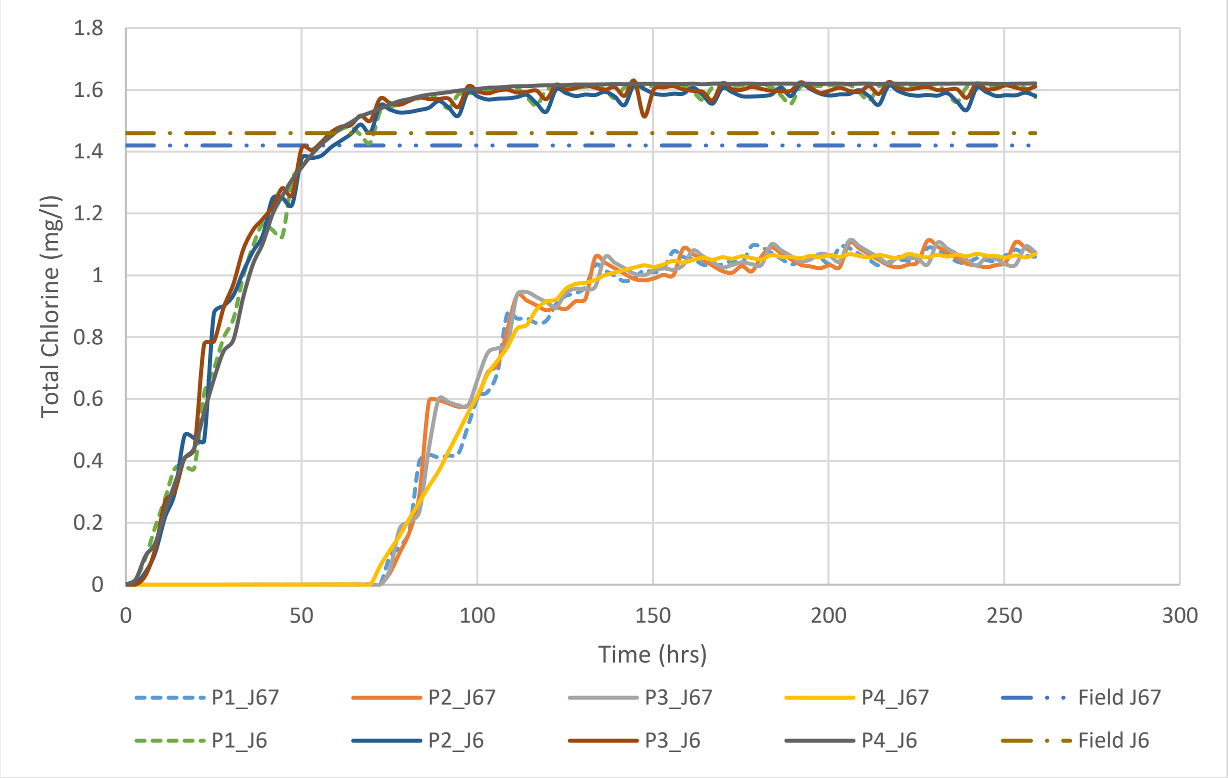


Figure 4-23: LLDS total chlorine for corresponding demand patterns at nodes J6 and J67

One explanation for these minimal changes was the fact that water quality in a drinking water system does not (significantly) fluctuate hourly, thus any changes that occur are very gradual over time and the impacts of hourly changes in demand are minimised. It must still be noted that the demand pattern is an important hydraulic modelling parameter, despite variations in the demand pattern having limited impact on water quality in the LLDS model.

4.7.3 MLDM analysis of demand pattern impact on water quality

A similar approach to the LLDS was taken in analysing the MLDM, such that comparison between the two models was possible at a later stage. Analysis was therefore undertaken at nodes

J67 and J6 and Figures 4-24 and 4-25 below present the results from this analysis. As with the LLDS the variance between the patterns appears negligible, with the most significant visual difference being the "smoothness" of Pattern 4 (P4 which has a default peak factor of 1), but despite this, all four demand patterns follow the same general trend.

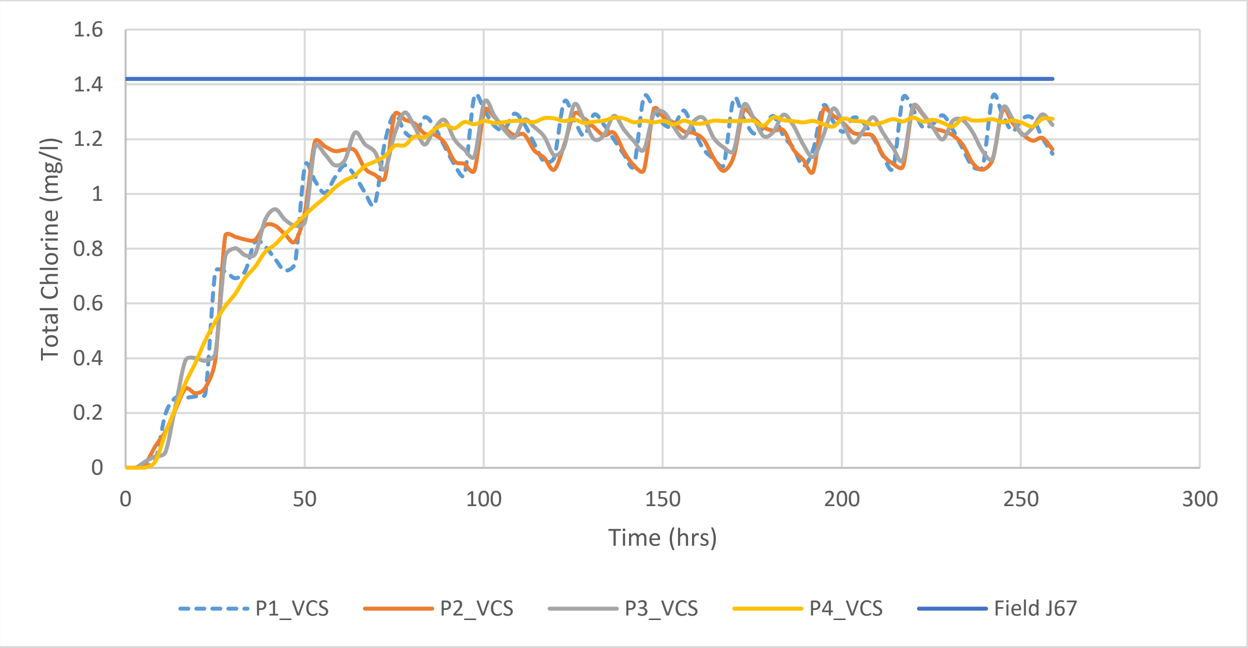


Figure 4-24: MLDM total chlorine for corresponding demand patterns at node J67

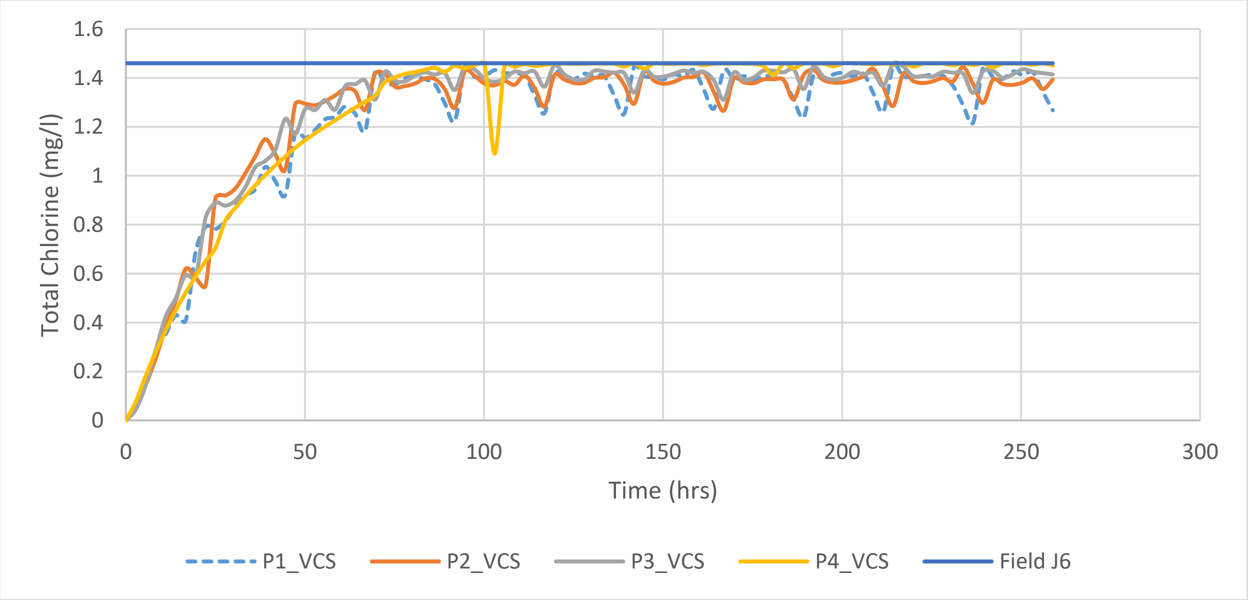


Figure 4-25: MLDM total chlorine for corresponding demand patterns at node J6

4.7.4 Summary: model definition, demand pattern and water quality

To conclude the analysis all four models from both the LLDS and the MLDM were compared and the results are presented in Figures 4-26 and 4-27 and Table 4-17 below. At node J67 total chlorine concentrations were consistently lower in the LLDS models as seen in Figure 4-26, this was expected as the water age for the LLDS models was generally higher (roughly three times more) than that of the MLDM as seen in Figures 4-21 and 4-22 above. Noting that the water stays longer in the system (higher water age) for the LLDS; it is logical that the concentrations in the LLDS would be lower.

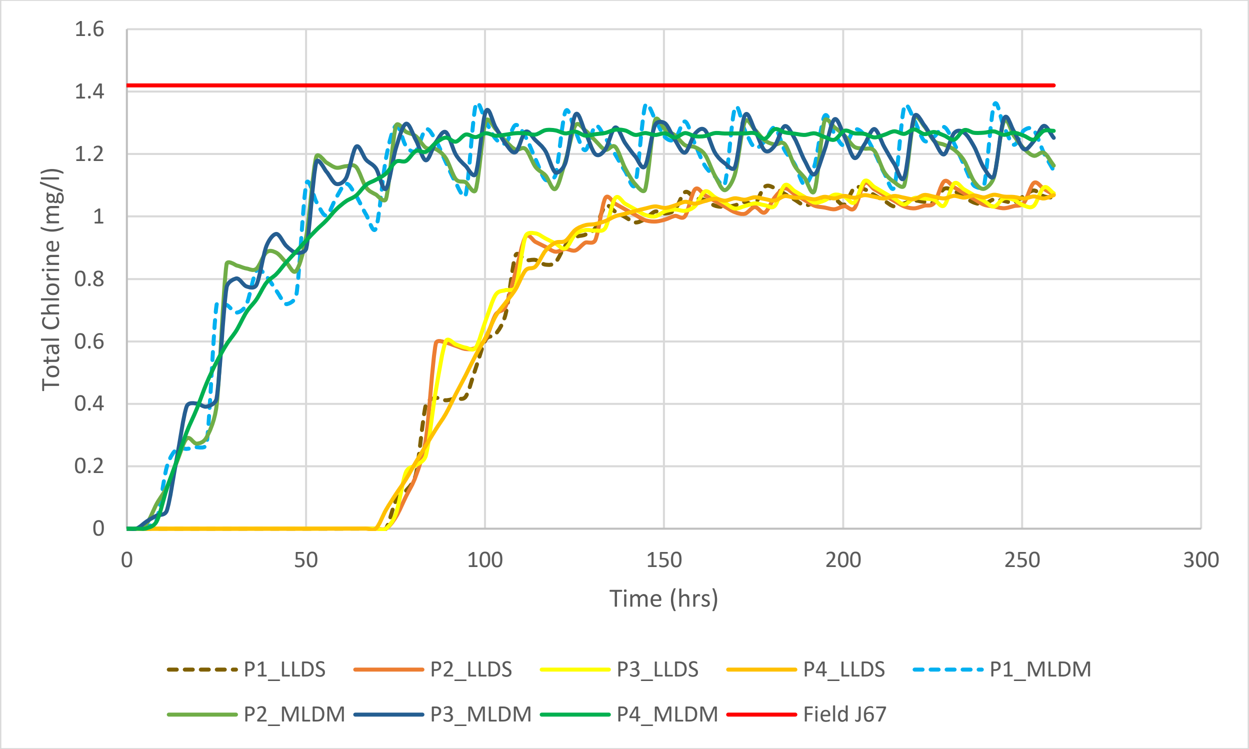


Figure 4-26: LLDS and MLDM total chlorine comparison for corresponding demand patterns at J67

Total chlorine concentration at node J6 was not as straight-forward to explain, as the LLDS had a higher water age, but still produced higher concentrations of total chlorine, as seen in Figure 4-27 below. Further investigation was needed to explain this occurrence; what was important however is that the MLDM was more accurate in absolute terms, as seen by its proximity to the field concentration of 1.46 mg/l at J6.

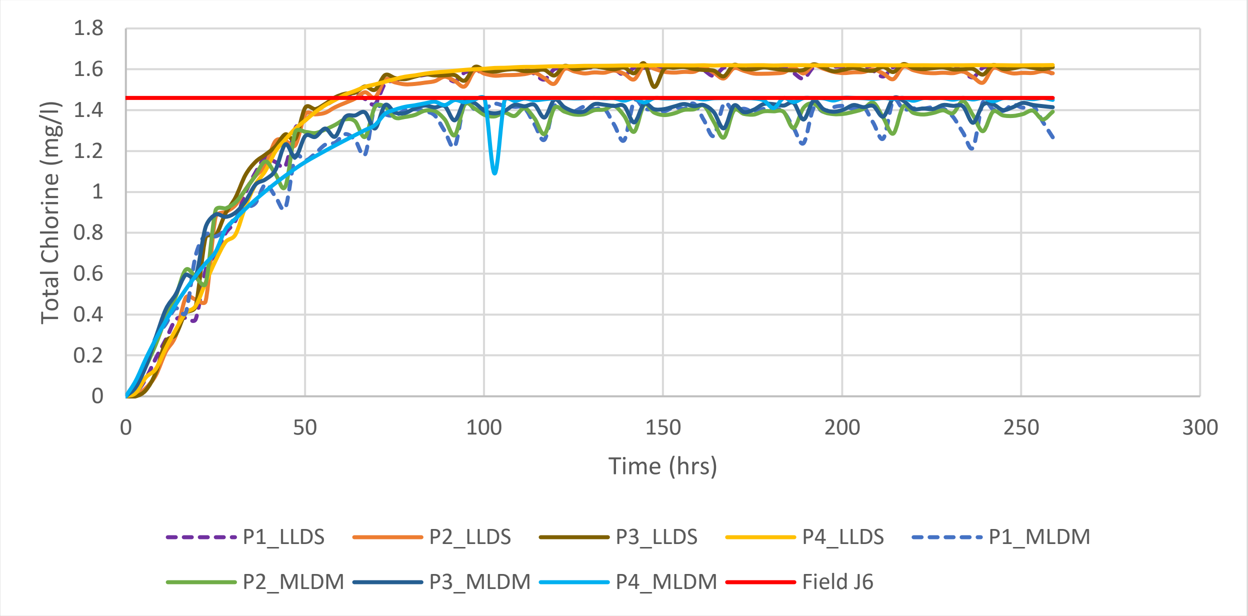


Figure 4-27: LLDS and MLDM total chlorine comparison for corresponding demand patterns at J6

Table 4-17 below summarises all the scenarios considered and shows that the MLDM was more accurate at both J67 and J6. It was evident that demand pattern changes had a limited impact on water quality as seen in the percentage error 2 values. Error 2 was the average difference between the pattern used in model calibration (P1_VanZyl 1996) and other demand patterns. Applying the EAC (1999) guidelines, all the errors were within an acceptable range. While the demand pattern impact is diminished for water quality modelling; demand pattern accuracy is very important for hydraulic calibration. Additionally, it was clear that if the demand pattern produces acceptable results in the hydraulic calibration stage, it should not be tampered with in the water quality calibration and validation stage. Furthermore, demand patterns from literature can prove useful, thus the amount of resourcing in demand pattern determination can be managed.

Table 4-17: LLDS and MLDM accuracy summary comparison for corresponding demand patterns at nodes J67 and J6

LLDS and MLDM at Node J67								
Pattern	**P1_LL**	**P2_LL**	**P3_LL**	**P4_LL**	**P1_ML**	**P2_ML**	**P3_ML**	**P4_ML**
% Error 1*	-26.13%	-26.45%	-26.15%	-26.0%	-13.62%	-15.37%	-13.12%	-10.92%
% Error 2**	0.0%	0.32%	0.02%	0.13%	0.0%	1.75%	-0.49%	-2.69%
LLDS and MLDM at Node J6								
% Error 1*	8.93%	7.75%	9.01%	10.12%	5.75%	-5.55%	-3.57%	-1.43%
% Error 2**	0.0%	1.18%	-0.08%	1.18%	0.0%	-0.20%	-2.18%	-4.32%

*Error 1 = difference between modelled results and field data

**Error 2 = difference between calibrated and validated C-value and selected C-value

4.8 Scenario analysis 5 – water quality and load-shedding

This section presents the impact of controlled intermittent electricity power supply on water quality. This scenario was important to undertake as electrical power interruption has been an ongoing challenge in South Africa for over a decade and appears set to continue (Eskom, 2023).

4.8.1 What is load-shedding in South Africa

Eskom is a South African State-Owned Entity (SOE) that is primarily responsible for the generation, transmission and reticulation of electricity in South Africa (Eskom, 2023). For various reasons, Eskom has not been able to meet the electricity demand of South Africa at various periods between 2007 and 2023 (date of submission of this research report), and the challenge was forecast to continue for the coming years (Eskom, 2023). To protect the electricity power supply system in instances where demand is greater than supply, managed load reduction is implemented, and this is done in one of two ways (Eskom 2023):

- Load Curtailment: large industrial users are requested to reduce their consumption.
- Load-Shedding: consumption is reduced by cutting supply to certain areas at the utility level.

Load-shedding is generally scheduled and is set up in stages one (1) to eight (8), with each stage having a corresponding time of no electrical power. Table 4-18 below provides a summary of stage one to eight load-shedding (Eskom, 2023).

Table 4-18: Load-shedding stages and corresponding frequency and downtime

Stage	**Implementation**	**Number of times per day**	**Total downtime (worst case)**
1	2.5-hour blocks	0 or 1	2.5 Hours
2	2.5-hour blocks	1 or 2	5 hours
3	2.5-hour blocks	2 or 3	7.5 Hours
4	2.5-hour blocks	3 or 4	10 Hours
5	5-hour blocks	4 or 5	12.5 Hours
6	5-hour blocks	5 or 6	15 Hours
7	5-hour blocks	6 or 7 times	17.5 Hours
8	5-hour blocks	7 or 8 times	20 Hours

The design of the water supply system in the project area did not foresee load-shedding, the system is thus affected by these power cuts. The most significant impact of the power cuts (load-shedding) is the loss in pumping time and the associated hydraulic challenges that comes with a loss of pressure (from the pumps). For the purpose of this study, the focus was on simulating the impacts of load-shedding on water quality only. This was done through the use of simple control statements for the pump within EPANET 2.0.

To effectively undertake this analysis, a determination of the time when load-shedding impacts are most severe had to be made. This was done through a 24-hour analysis of load-shedding at stage four. Figures 4-28 and 4-29 below present the impact of load-shedding, in a 24-hour cycle on total chlorine levels at nodes J67 and J132.

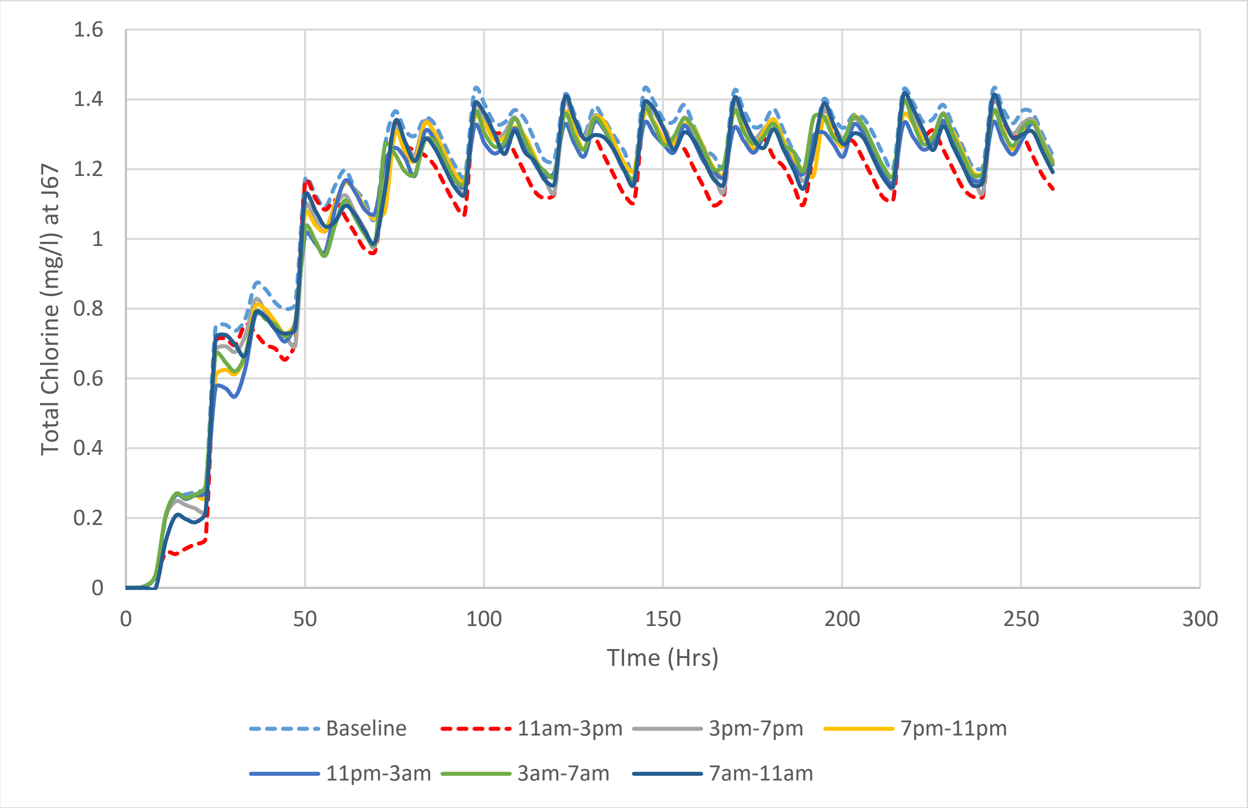


Figure 4-28: MLDM 24-hour cycle impact of load-shedding on total chlorine at node J67

As seen in Figure 4-28 above for node J67, load-shedding did not result in an extreme decrease in chlorine concentrations. While minimal, the impact of load-shedding was most pronounced in the 11am to 3pm window, which coincides with low water demand as this is after the morning water consumption peak. As seen in Figure 4-29 below, total chlorine levels at node J132 displayed more variance, as the node is supplied from an elevated reservoir that is supplied from the main reservoir. This meant that node J132 had a greater water age, as the water passed through an additional elevated reservoir before reaching the node. The lowest chlorine levels at node J132 occurred in the 3pm to 7pm slot, which coincides with the afternoon/early evening peak demand. The second lowest concentrations occurred in the 7am to 11am slot; which coincides with the second half of the morning peak consumption.

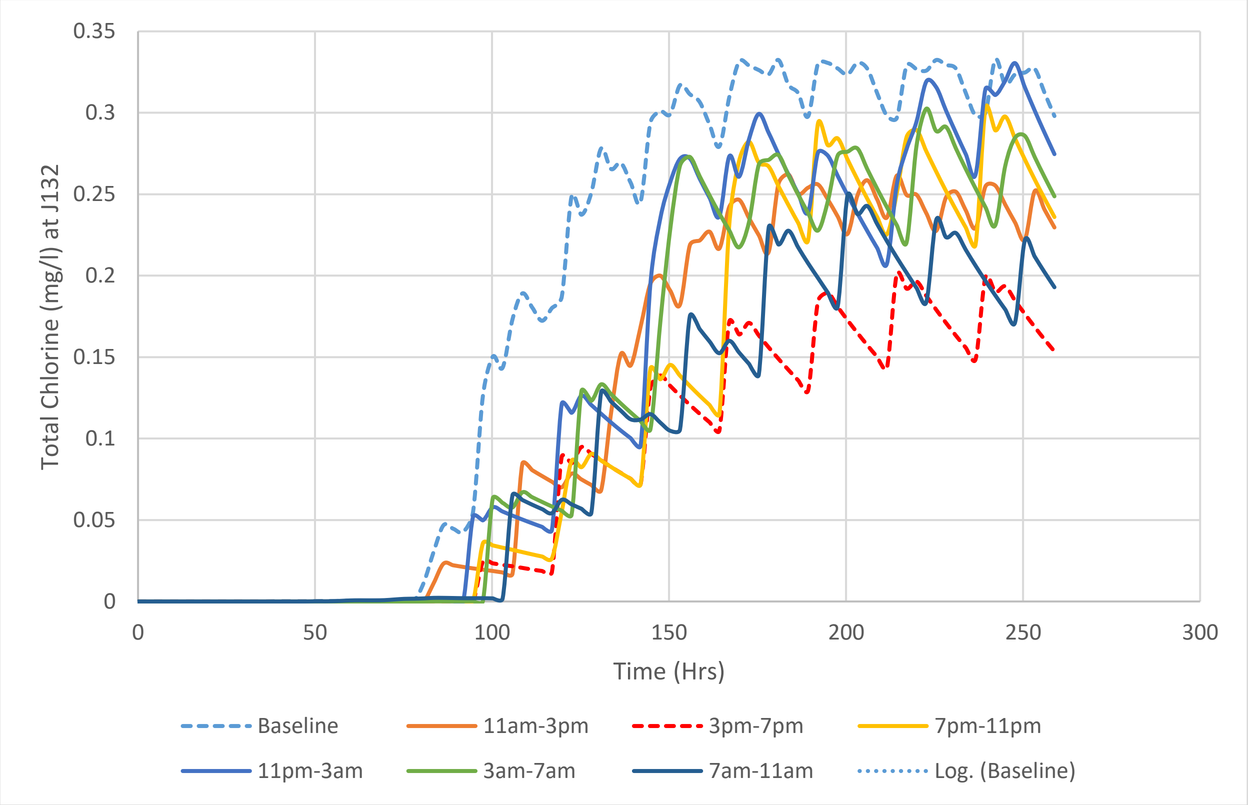


Figure 4-29: MLDM 24-hour cycle impact of load-shedding on total chlorine at node J132

Considering the above, and reality that only a limited number of simulations were feasible; further analysis was undertaken in the post morning peak slot for stages one (1) to eight (8) as presented below. This was done as this post morning slot was shown to have lower total chlorine levels at both nodes J67 and J132.

4.8.2 Impact of load-shedding on water age

Water age is a key surrogate indicator for water quality. Knowing this, an analysis of the impact of load-shedding on water age was undertaken. The analysis was setup to show the impacts of load-shedding across the different stages. The analysis thus considered the following stages: Stage 0 (S0), Stage 1 (S1), Stage 5 (S5) and Stage 8 (S8). Figures 4-30 and 4-31 below present the results at Nodes J67 and J6 respectively.

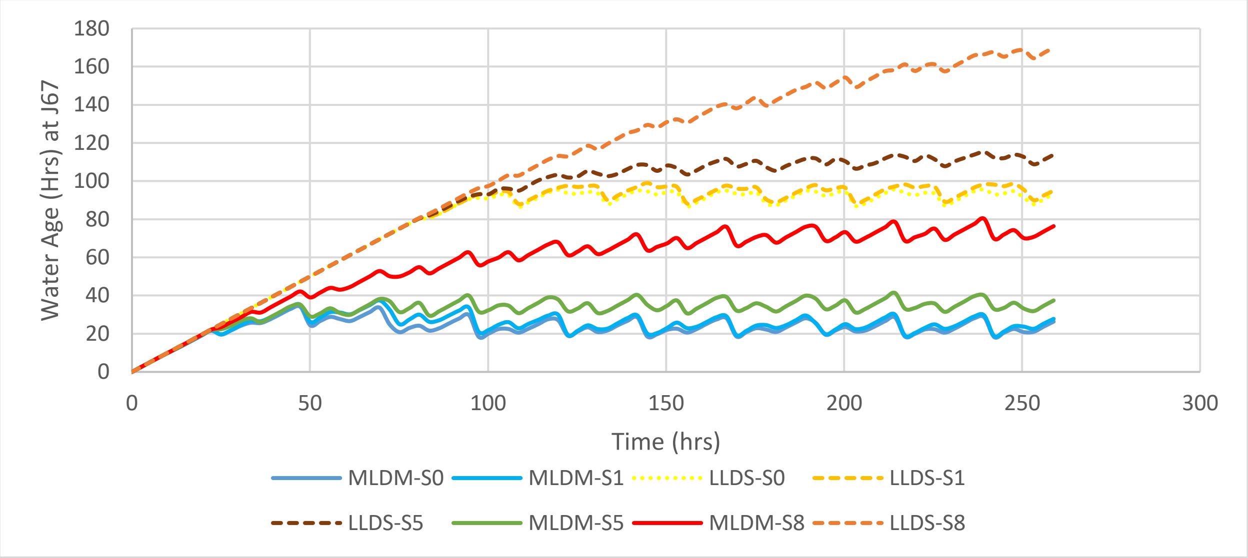


Figure 4-30: Impact of different load-shedding stages on water age at node J67

Figure 4-30 above presents node J67 results that were expected: load-shedding stage is directly correlated with water age. As a result water age was highest during stage 8 load-shedding and lowest during Stage 0 and 1 respectively. Figure 4-31 below presents results for node J6, where a similar general correlation between load-shedding stage and wate age was observed. Node J6 however, presented a slight difference between the MLDM and the LLDS; where the MLDM Stage 8 water age was greater than the LLDS Stage 5 water age. This was inconsistent with the node J67 results where the LLDS water ages were all greater than the MLDM water ages; to an extent where the LLDS Stage 0 water age was greater than the MLDM Stage 8 water age.

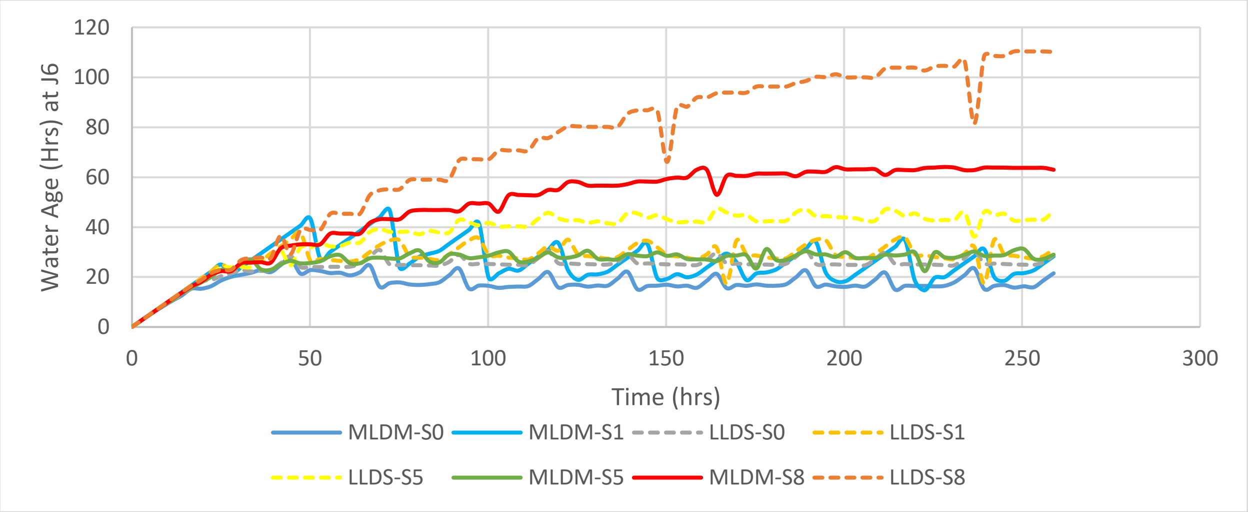


Figure 4-31: Impact of different load-shedding stages on water age at node J6

The node J6 model results therefore presented results that better aligned with the expected results. The high variance at node J67 required additional investigation that was beyond the scope of this study. It must also be noted that further one moved from the reservoir, the higher the water age and the higher the chances of variance in the result as the water has more time to interact with the system. The next step was to look at the LLDS and MLDM individually as presented below.

4.8.3 LLDS analysis of load-shedding impact on water quality

This analysis presents the results of an evaluation of load-shedding impacts on water quality, with the considered water quality parameter being total chlorine (mg/l). Figures 4-32 and 4-33 below present the results of this analysis, at nodes J67 and J6.

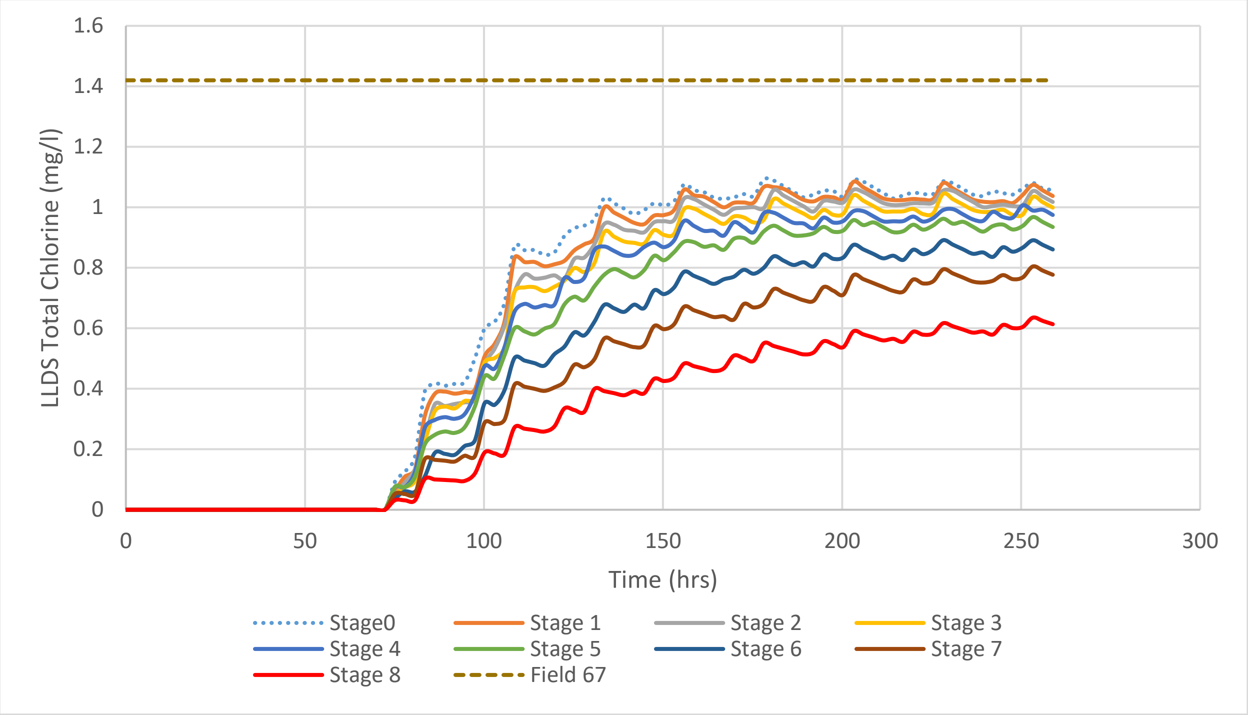


Figure 4-32: LLDS impact of different load-shedding stages on total chlorine at node J67

As seen in Figure 4-32 above and Figure 4-33 below, total chlorine levels decrease with increasing stage of load-shedding, this is consistent with the water age analysis results presented above and the LLDS model produced results that were expected. Not only is the relationship as expected, but the rate of chlorine loss is also proportional to the stage of load-shedding, this can be seen in how pronounced the decreases are between stages 5 to stage 8.

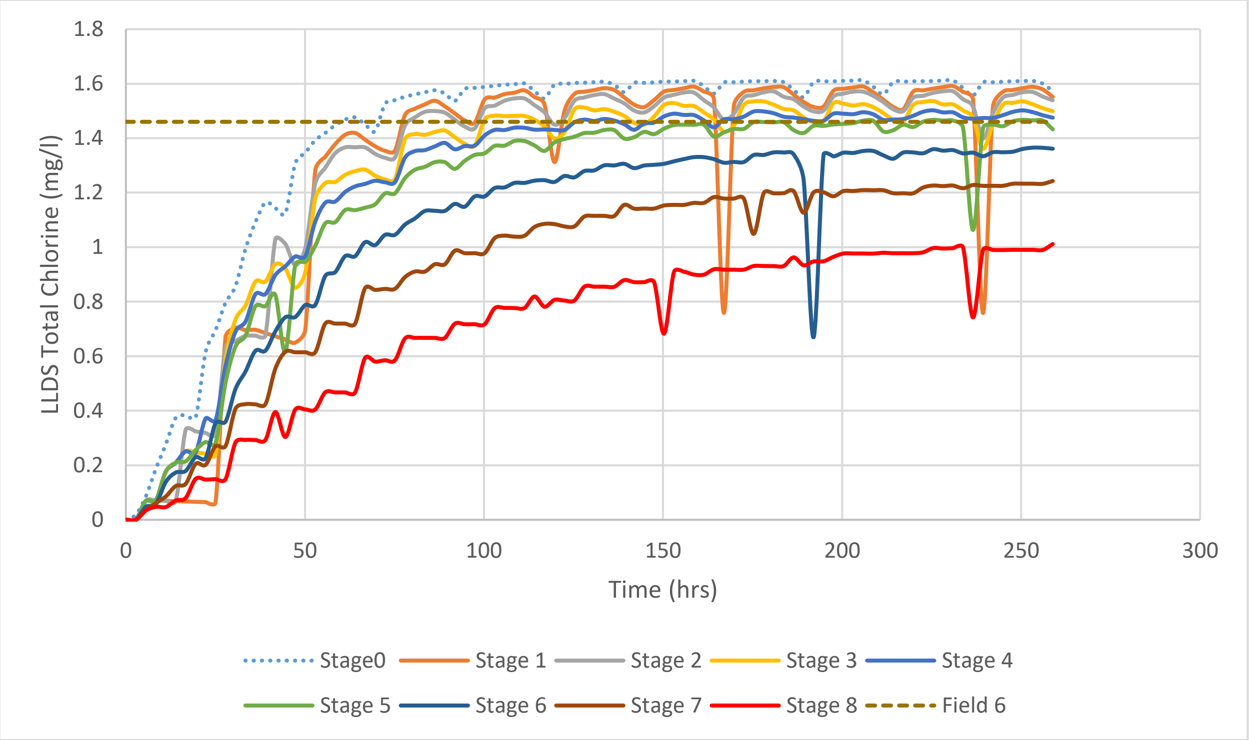


Figure 4-33: LLDS impact of different load-shedding stages on total chlorine at node J6

4.8.4 MLDM analysis of load-shedding impact on water quality

This section presents the results of the analysis of load-shedding on water quality (total chlorine) for the MLDM, following on from the analysis on the LLDS. Noting that there was a slight variance in the expected water age results for the MLDM model, an initial analysis of water age for stages 0 to 8 was undertaken for nodes J67 and J6 and the results are presented below in Figures 4-34 and 4-35. The water age analysis for node J67 presents expected results between stages 0 and 8, with nothing irregular that required further investigation being visible as is the case in Figure 4-30 above. Figures 4-34 and 4-35 below show an increase in water age from Stage 0 to 8, but stage 1 to 4 display some inconsistent variability from one hour to the next, but the general average shows an increase in water age from one load-shedding stage to the next.

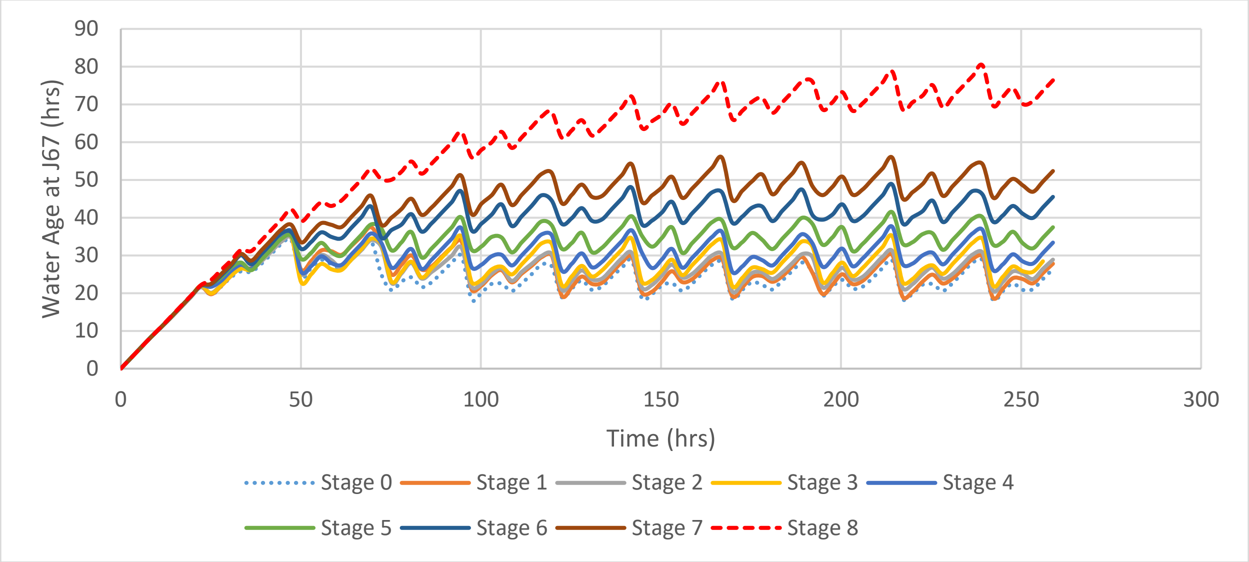


Figure 4-34: MLDM impact of different load-shedding stages on water age at node J67

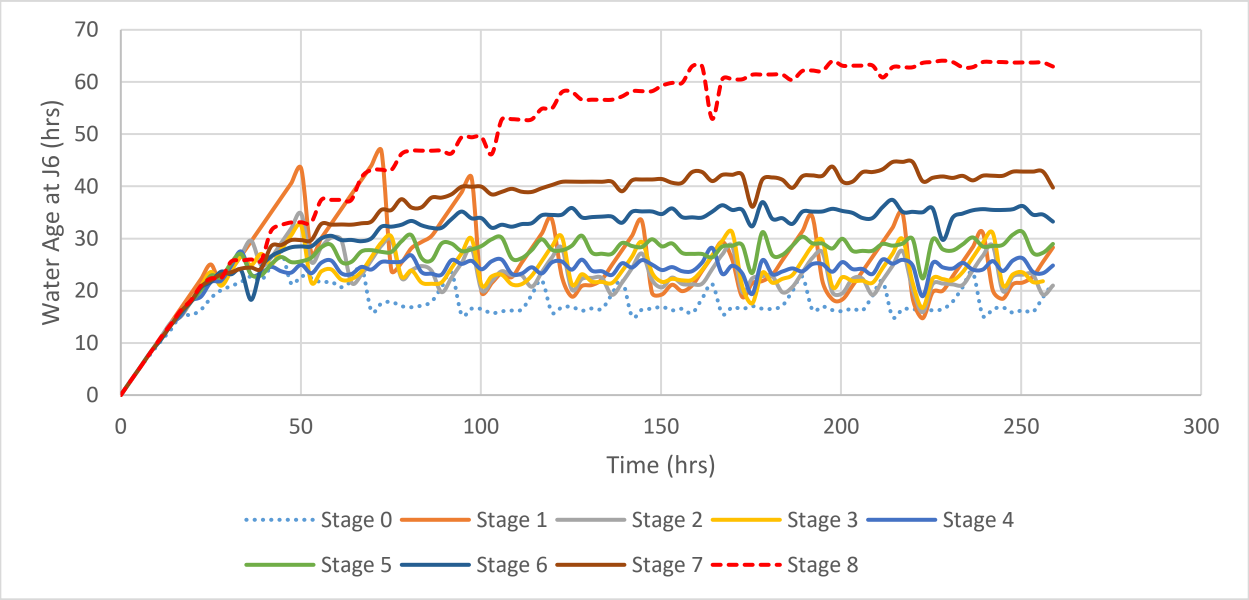


Figure 4-35: MLDM impact of different load-shedding stages on water age at node J6

Notwithstanding the slight inconsistencies in stages 1 to 4 at node J6 as discussed above, there was nothing that suggested that the results were not usable. Noting this, the analysis proceeded to consider load-shedding impacts (Stage 0 to 8) on water quality (total chlorine) at nodes J67 and J6. The results of this analysis are presented below in Figures 4-36 and 4-37 for J67 and J6 respectively.

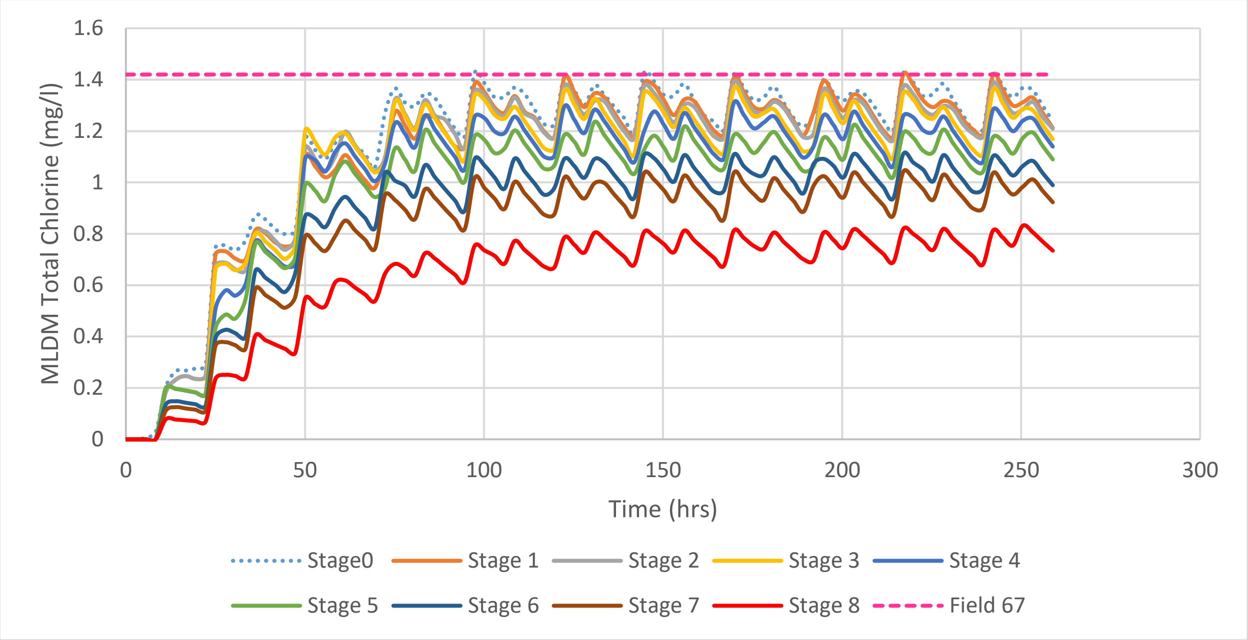


Figure 4-36: MLDM impact of different load-shedding stages on total chlorine at node J67

Figure 4-36 above presents the results at node J67 and the total chlorine levels were very similar to the water age; with total chlorine concentrations decreasing at the same rate as the water age increased.

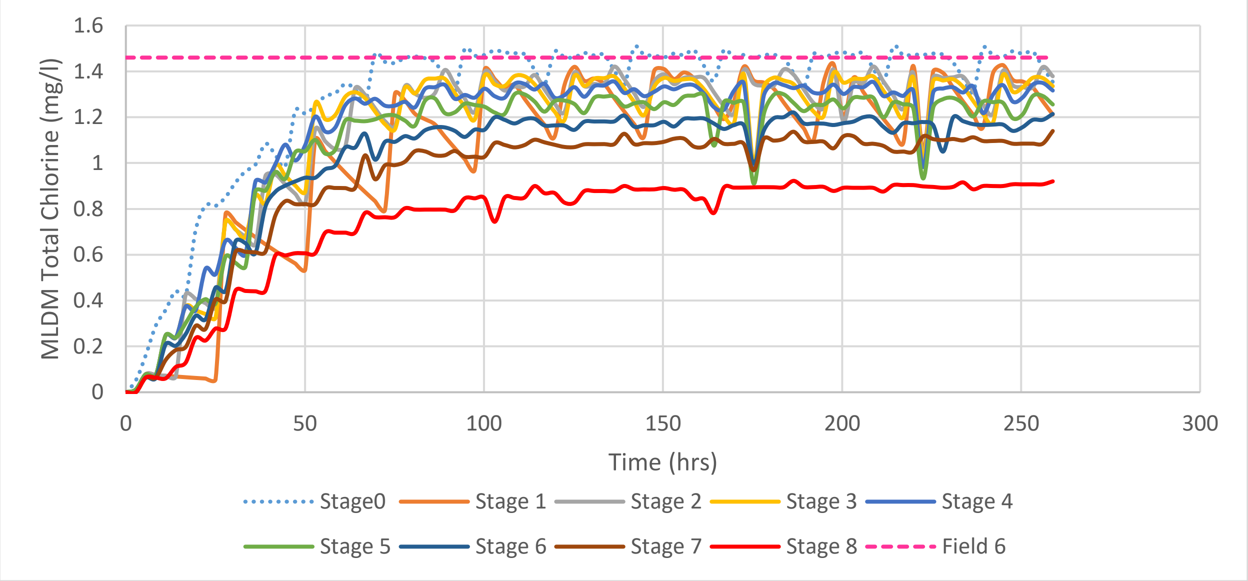


Figure 4-37: MLDM impact of different load-shedding stages on total chlorine at node J6

Figure 4-37 above presented the results at node J6 and the total chlorine decrease followed a similar pattern to water age; with stages 1 to 5 having interval inconsistencies, but the general average showing that total chlorine concentration was inversely proportional to load-shedding stage (and by extension water age). Both MLDM analysis showed that water age was a good proxy for water quality and despite the minor variations, this pattern was clear at both nodes.

4.8.5 Summary: model definition, load-shedding and water quality

This section considered the impact of load-shedding on water quality as measured by total chlorine concentrations at nodes J67 and J6, with the results summarised in Table 4-19 below. Both the LLDS and the MLDM were analysed, with all analysis showing that load-shedding had an impact on water quality. This impact was best observed through water age, as load-shedding has no direct impact on the water quality but only increases the residence time (water age) of the water in the system. The analysis showed a direct corelation between water age and the different stages of load-shedding. The rate of increase in the load-shedding stages, translated directly to water age, which correlated directly with a decrease in total chlorine concentrations, this phenomenon was most clear between stages 5 to stage 8 of load-shedding, as shown in Table 4-19 below. Stage 5 represented a difference of more than 10% from the baseline model (Error 2), at both nodes J6 and J67 in the LLDS and the MLDM. Table 4-19 also shows that while the difference between the field results and the modelled results (as quantified through Error 1) varied; the absolute difference from the baseline (S-0) and the respective stages (as quantified through Error 2), was comparable. The Error 2 trend becomes most consistent from load-shedding stages 5 to 8.

Table 4-19: Summary overview of load-shedding impacts on LLDS and MLDM total chlorine levels and model accuracy at nodes J6 and J67

LLDS at Node J6									
Stage	**S_0**	**S_1**	**S_2**	**S_3**	**S_4**	**S_5**	**S_6**	**S_7**	**S_8**
% Error 1*	-8.6%	-3.7%	-3.6%	-0.4%	-1.3%	-4.6%	-13.4%	-23.7%	-41.2%
% Error 2**	0.0%	4.9%	5%	8.2%	9.9%	13.3%	22.1%	32.3%	49.8%
MLDM at Node J6									
% Error 1*	-1.1%	-14.5%	-9.4%	-10.7%	-10.9%	-15.1%	-20.6%	-26.5%	-40.8%
% Error 2**	0.0%	13.4%	8.3%	9.6%	9.9%	14%	19.5%	25.4%	39.7%
LLDS at Node J67									
% Error 1*	-26.3%	-27.8%	-29.3%	-31.3%	-33.5%	-36.4%	-43.2%	-50.7%	-62.7%
% Error 2**	0.0%	1.4%	2.9%	5%	7.2%	10.1%	16.9%	24.4%	36.4%
MLDM at Node J67									
% Error 1*	-7.2%	-8.9%	-10.4%	-12.8%	-15.8%	-20.5%	-26.8%	-32.1%	-46.5%
% Error 2**	0.0%	1.8%	3.3%	5.6%	8.7%	13.3%	19.6%	24.9%	39.3%

*Error 1: represents the percentage difference between field data and modelled output.

*Error 2: represents the percentage difference between S0 (baseline model) and that specific stage model outputs.

The percentages above also represent a decrease to total chlorine concentrations. Most concerning was the extent to which the concentrations decreased in stages 7 and 8, this posed a risk that residual/free chlorine (which was already very low as seen in Figure 4-4) was depleted in stages 7 and 8 of load-shedding, to below the acceptable concentration of 0.2mg/l.

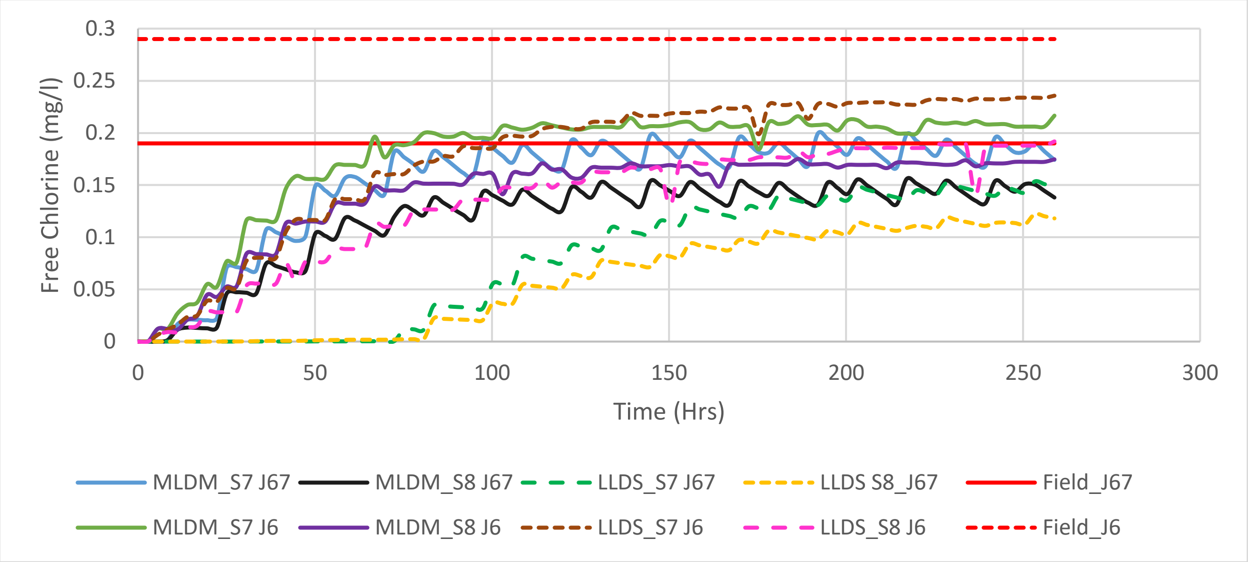


Figure 4-38: LLDS and MLDM free chlorine concentrations in load-shedding stages 7 and 8 at nodes J6 and J67

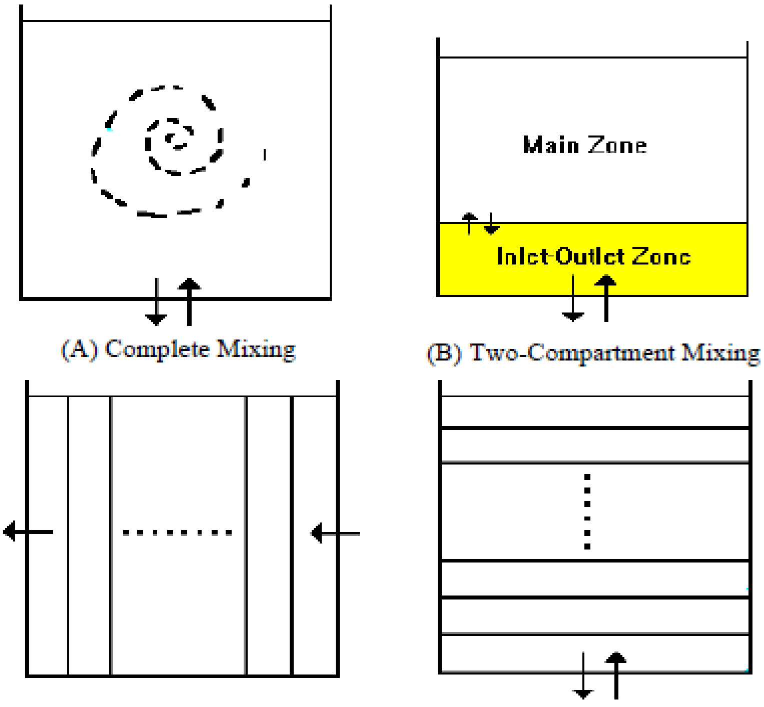
The concern that the already low free chlorine (as seen in the field data) might decrease to below 0.2mg/l was tested in both the MLDM and the LLDS at nodes J6 and J67 and Figure 4-38 above presents the results. As seen in Figure 4-38 above, free chlorine at node J67 was below the minimum of 0.2mg/l at both J6 and J67, in both stage 7 and 8. This meant that during stage 7 and 8 load-shedding consumers at node J67 received water that had no residual chlorine. There were only two exceptions in the analysis, that being node J6 in Stage 7 for both the LLDS and the MLDM. The above also provided confirmation that the both the LLDS and the MLDM could be utilised to model the impacts of load-shedding on water quality (total chlorine and free chlorine). This was important for the broader study objectives.

4.9 Water quality management insights

This section considered some general potentially value-adding water quality management insights that EPANET 2.0 presents for users. While there are many such applications, only two were considered in this study: reservoir (tank) mixing models and optimal sample location for water quality monitoring.

4.9.1 Storage tank mixing models and water quality

Fluid dynamics are applicable to water distribution networks, with the impacts being most pronounced in stored bodies of water (reservoirs or tanks). Chemical reactions in water retaining structures take place due to *inter alia* flow patterns and heat transfer, which can be best modelled through computational fluid dynamics (CFD) models (Kruger, 2001; EPA, 2005). While EPANET does not have full CFD modelling capabilities, it does have four mixing models, these are: complete mixing, two-compartment mixing, first-in/first-out (FIFO) plug flow and last-in/first-out (LIFO) plug flow (short circuiting). The four models are depicted in Figure 4-39 below, with complete mixing having been used in this study.



(A) Complete Mixing (B) Two-Compartment Mixing

(C) Plug Flow - FIFO (D) Plug Flow - LIFO

Figure 4-39: EPANET tank mixing model options (Rossman, 2000)

Noting the potential for mixing models to impact water quality, the study investigated this phenomenon at nodes J67 and node J7. Node J7 is immediately downstream of the elevated tank in the system. Figure 4-40 below presents the results for node J67, while Figure 4-41 below presents the results for node J7. It was clear from both models (LLDS and MLDM) that the mixing model had an impact on water quality (total chlorine in this instance).

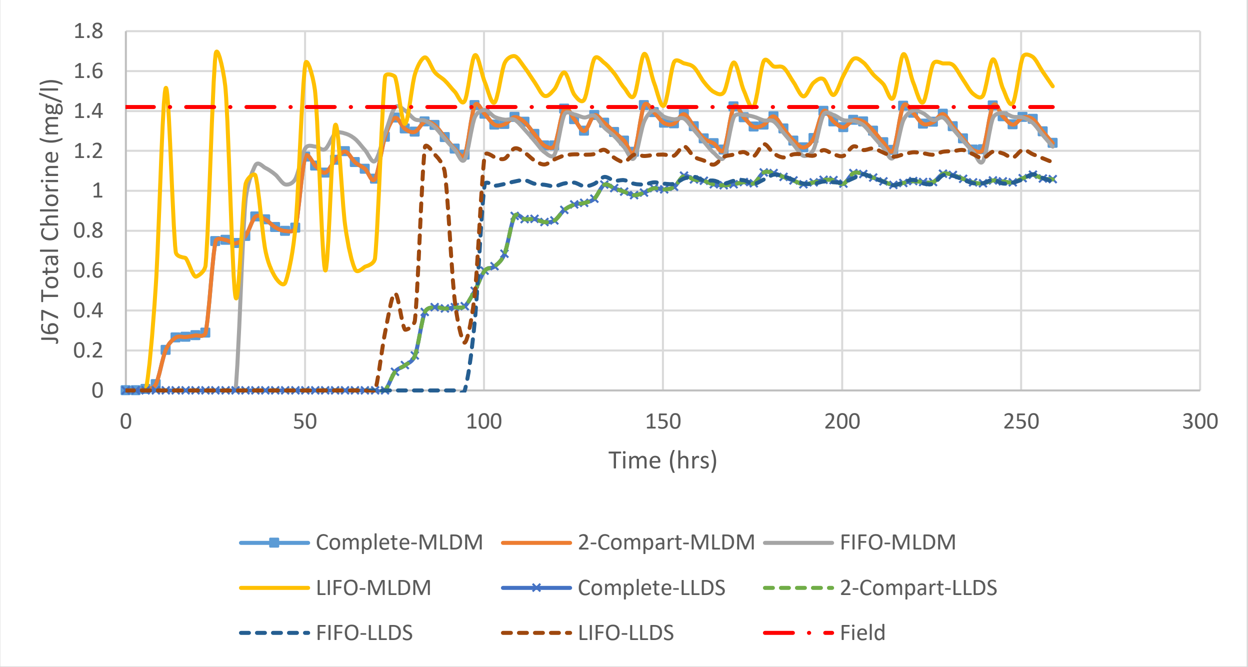


Figure 4-40: LLDS and MLDM Tank mixing model impact on total chlorine at node J67

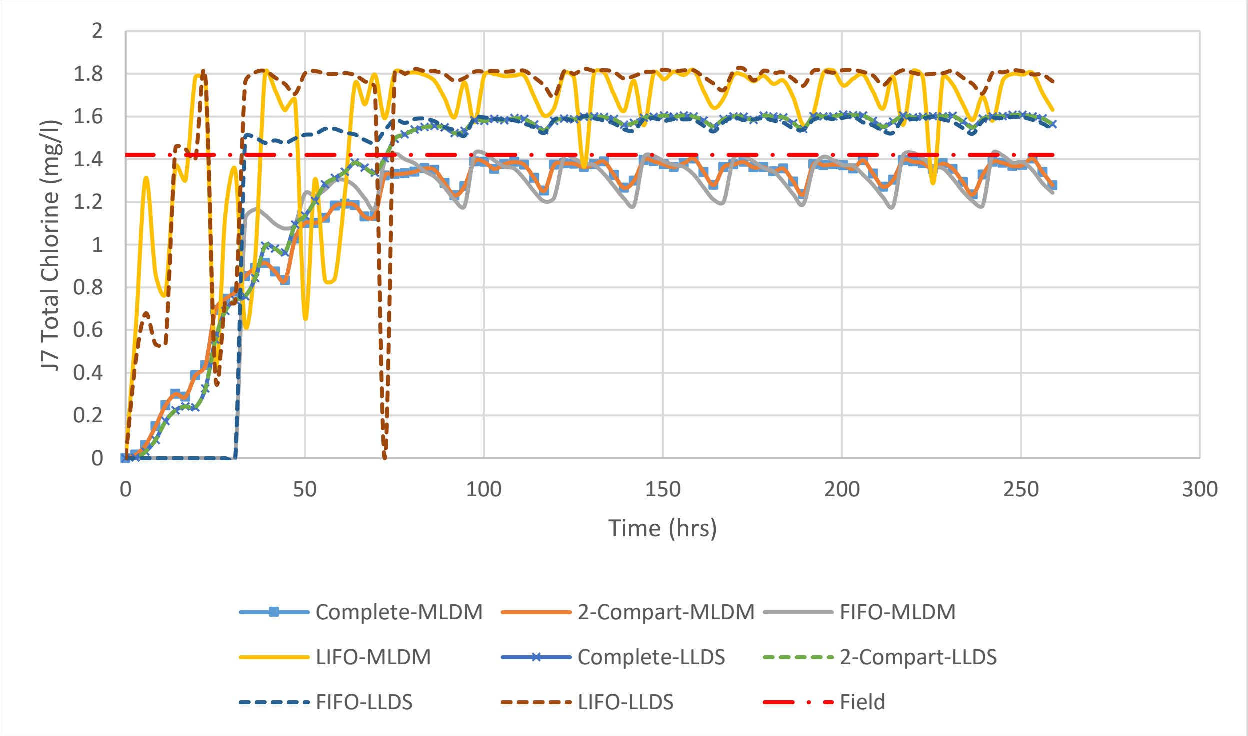


Figure 4-41: LLDS and MLDM Tank mixing model impact on total chlorine at node J7

The extent of the impact of each of the models is quantified in Table 4-20 below. The results for the completely mixed models were nearly identical at both nodes, with a minor difference (0.9% and 1.1%) for the FIFO plug flow model. The LIFO plug flow model (short-circuiting) presented a notable variance in both models and at all nodes, suggesting that short circuiting cannot be ignored as it has a material impact on water quality.

Table 4-20: LLDS and MLDM Summary overview of tank mixing model impact on model accuracy

LLDS and MLDM at Node J67								
Model	**A_LLD**	**B_LLD**	**C_LLD**	**D_LLD**	**A_ML**	**B_ML**	**C_ML**	**D_ML**
% Error 1*	-27%	-27%	-26%	-16.7%	-7.2%	-7.2%	-7.7%	-9.9%
% Error 2**	0.0%	0.0%	-0.9%	-10.3%	0.0%	0.0%	0.6%	-17.1%
LLDS and MLDM at Node J7								
% Error 1*	12%	12%	10.9%	26.5%	-5.1%	-5.1%	-6.5%	21.2%
% Error 2**	0.0%	0.0%	1.1%	-14.5%	0.0%	0.0%	1.4%	-26.4%

*Error 1: represents the percentage difference between field data and modelled output.

*Error 2: represents the percentage difference between A (complete mixing) and that specific mixing model outputs.

Where: A = Complete Mixing; B = Two Compartment Mixing; C = FIFO Plug Flow; D = LIFO Plug Flow.

The materiality of the consequences of short-circuiting (LIFO plug flow model) means that utilities that have reservoirs (tanks) that are likely to short-circuit, need to consider counter measures such as baffle walls, this is supported by Kruger (2001) in his investigation of water quality deterioration in reservoirs.

4.9.2 Water Quality Sampling Locations

The sections above have demonstrated the complexity in drinking water quality management and how EPANET 2.0 can potentially add value in the management of water quality in a distribution network. A prescribed action in drinking water quality management is sampling, as part of the water safety planning practice (WHO, 2009; SANS, 2011; SANS 2015; Simone *et al.,*2016). In the sample design programme, care must be taken to sample at the appropriate locations and at the appropriate frequencies; to provide reasonable assurance that the supplied water is in fact safe for human consumption (potable). Sampling is not cheap, therefore the number of samples that can be taken typically needs to be optimised from the general distribution sample points. In the study EPANET was used to approximate two optimal critical distribution sample points, the criteria for these were that they should indicate the most critical points for spatial and temporal water quality risks.

As will be shown by the results below, the location of these limited number of points is not reasonably or easily achievable without the aid of some modelling and simulation software. In the study the exercise of determining the critical distribution sample regions was undertaken by considering three parameters for both the LLDS and the MLDM, these parameters were: water age (under normal conditions and during stage 8 load-shedding), total chlorine and free chlorine. The figures below present the EPANET models, demonstrating the extent to which EPANET 2.0 could add value to the utility concerned.

Water age (Under Normal Conditions)

Water age is an important surrogate indicator as it can be used to predict specific water quality determinants, it was thus selected as one of the three criteria to monitor. Figures 4-42 and 4-43 below present the network water age at 175.21 hours into the simulation. The exact time was selected for all other simulations, such that a snapshot is taken at the exact same time to enable comparison. The 175.21 hours simulation time is significant as it represents the recommended maximum water age of seven (7) days. The 175.21 snapshot is also one of many possible time slots where the system had normalised (see Figure 4-1).

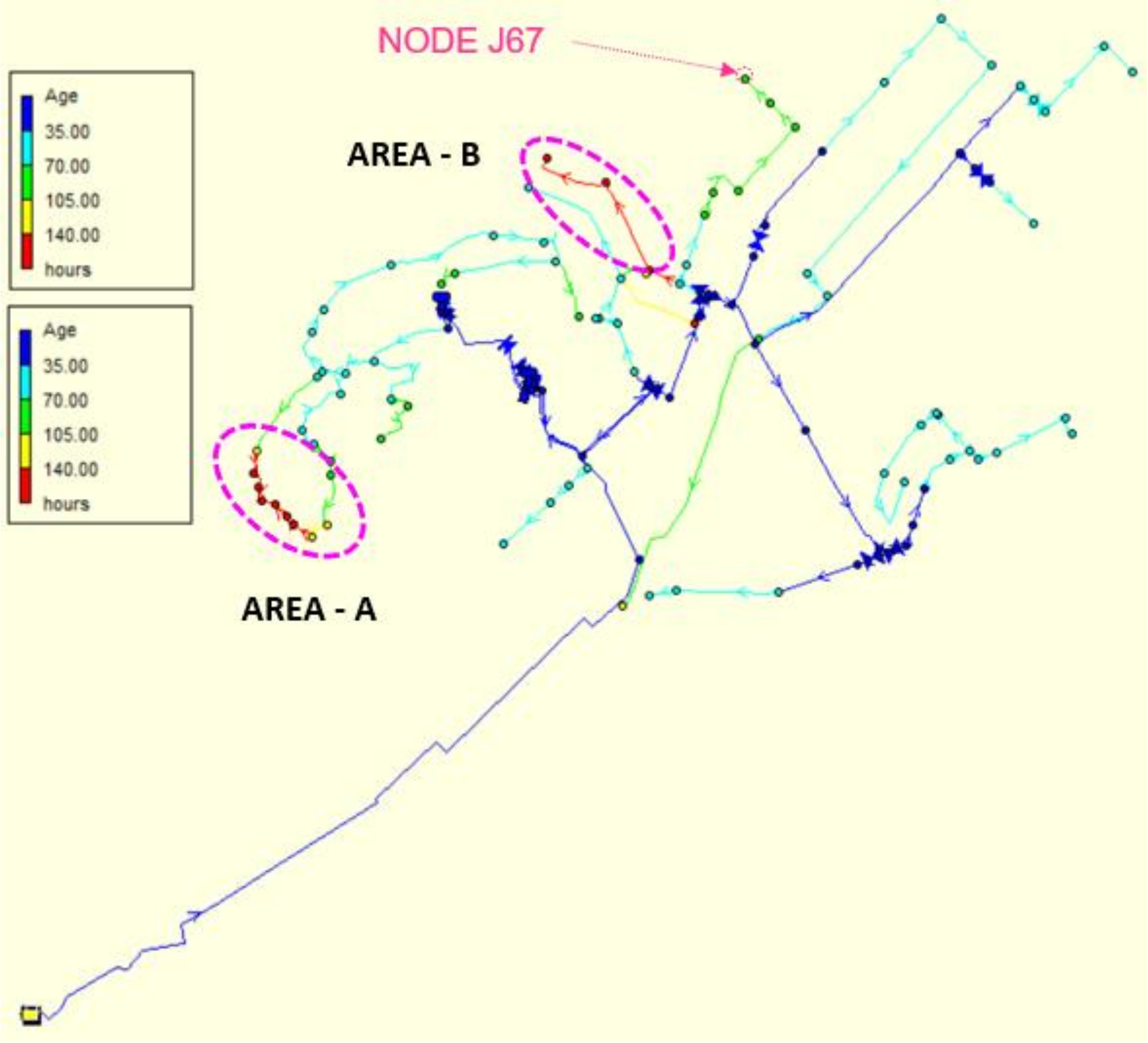


Figure 4-42: LLDS water age output under normal conditions

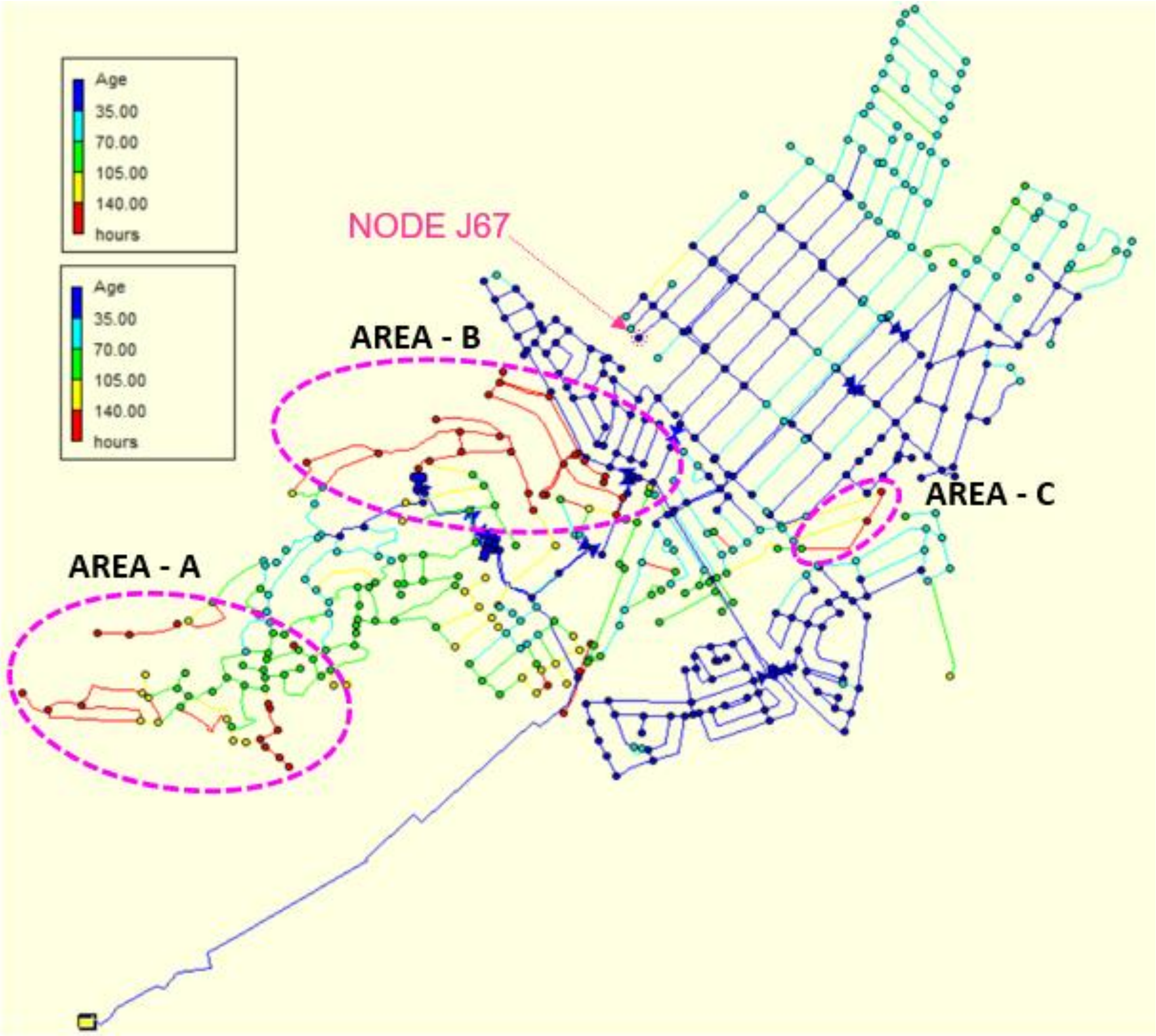


Figure 4-43: MLDM water age output under normal conditions

Both Figures 4-42 and 4-43 above showed that the highest water age was found in areas A and B. The indicated areas are identical for both the LLDS and the MLDM. As expected, the MLDM had more detail, this was evident in the number of pipes contained in its areas A and B. Additionally, the MLDM has a small area C, which is not visible in the LLDS because it is already small (two pipes and two nodes) in the MLDM. This served as a caution for the loss of important detail that can be lost due to reduction and skeletonisation, of the magnitude and scale implemented in the LLDS.

What was most concerning was that the sampling point (Node J67) did not fall under either of these three areas on both the LLDS and the MLDM. The concern was warranted as literature and the results from sections above show a correlation between high water age and water quality deterioration (Kruger, 2001; Lahlou, 2002; EPA, 2005; AWWA, 2015). It was therefore possible that the testing location (Node J67) was not optimal (optimal critical distribution point sample).

Water age (under stage 8 load-shedding)

Load-shedding has been a reality in South Africa since 2007, it was therefore appropriate to consider its impact on drinking water quality in a distribution network. Figures 4-44 and 4-45 below show water age under stage 8 load-shedding for the LLDS and the MLDM respectively. Stage 8 was selected as it was the worst-case scenario, and it had the highest modelled water ages. Utilising the worst-case was an appropriate risk mitigation modelling approach as stages 1 to 7 were covered by stage 8, as they were less of a risk.

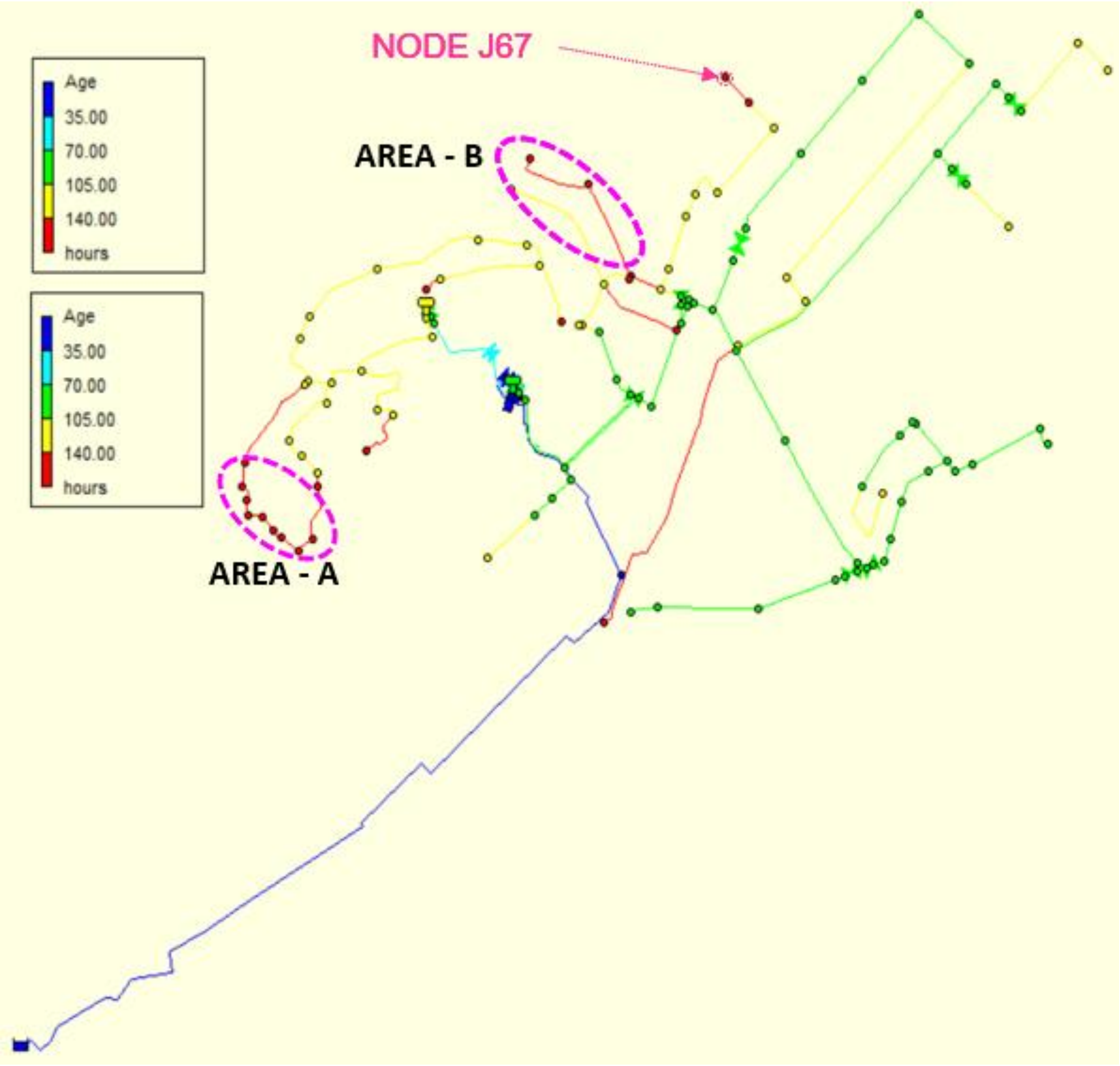


Figure 4-44: LLDS modelled water age during stage 8 load-shedding

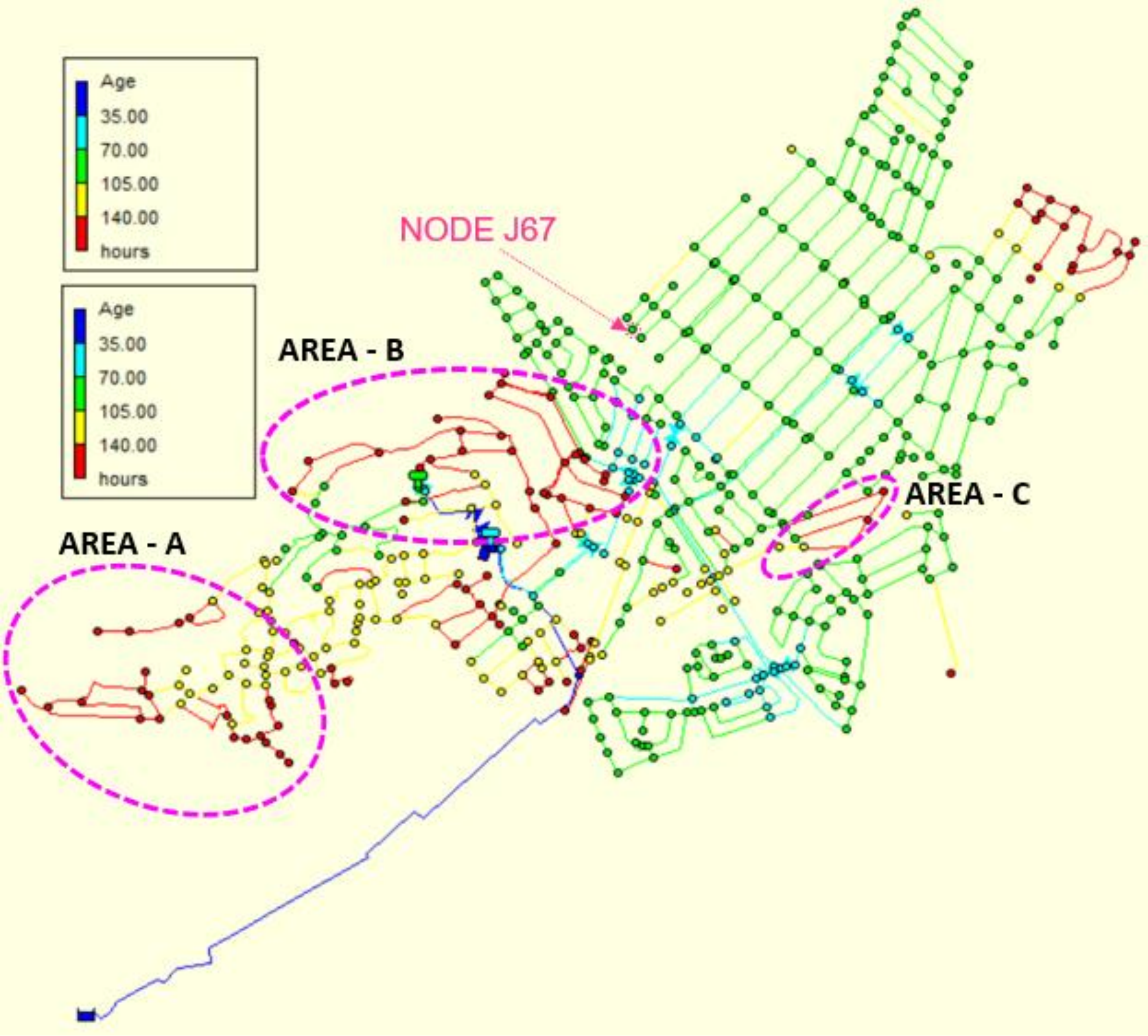


Figure 4-45: MLDM modelled water age during stage 8 load-shedding

Water age under stage 8 load-shedding produced expected results; where Areas A, B and C were still at the same locations, although they were marginally larger (in terms of the affected number of pipes and nodes) as seen in Figures 4-44 and 4-45 above. As anticipated new areas (over and above Areas A, B and C) with water ages exceeding 140 hours emerged. Noteworthy was that node J67 (current sampling point) appeared as critical (water age above 140 hours) on the LLDS, but not on the MLDM. This further confirmed that the sampling point selected by the utility, at the time of the data collection was indeed not the most critical or optimal. On the basis of water age, the optimal critical distribution sampling point should have been in areas A or B, as these areas are identified in both the LLDS and the MLDM and also appear under the extreme conditions of stage 8 load-shedding. The consistency of areas A and B in both the LLDS and MLDM also suggested that both the LLDS and the MLDM could be utilised in the modelling of water age. In the context of this study, it was therefore reasonable to note that level of hydraulic definition was not absolutely detrimental to the modelling of water quality.

Total chlorine (under normal conditions)

The next variable that was considered was total chlorine for both the LLDS and the MLDM at time 175.21, as shown in Figures 4-46 and 4-47 below. The total chlorine levels followed the pattern of water age and as such Areas A, B and C were at identical for the LLDS (as seen on Figures 4-42 and 4-46) and the MLDM (as see on Figures 4-43 and 4-47). Noteworthy was that the utility sampling point (Node J67) was not in the high-risk areas (Areas A, B and C). This was problematic, as there was a possibility of the optimal critical distribution sample location not being appropriate, as the most critical areas are A, B and C.

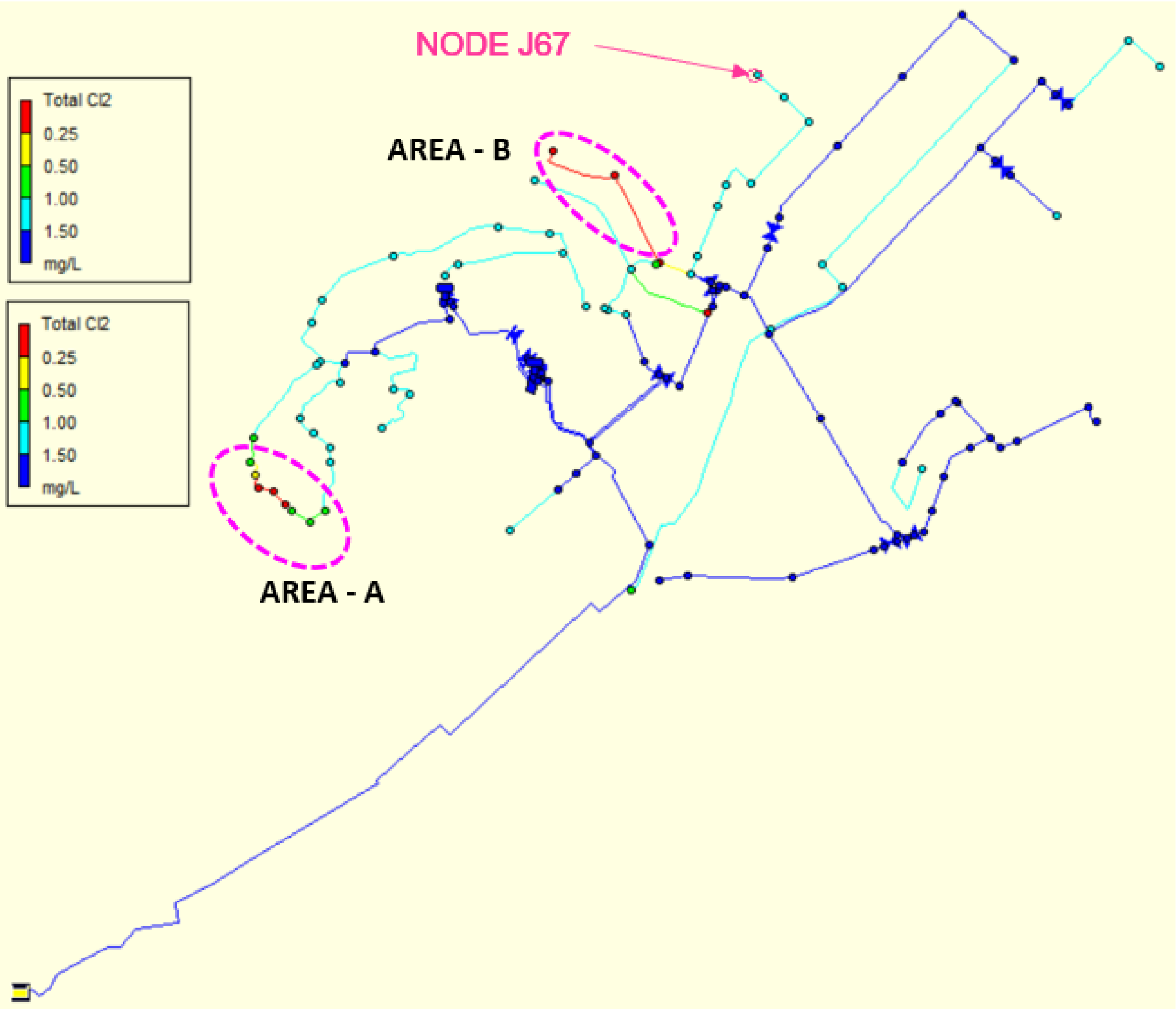


Figure 4-46: LLDS total chlorine concentrations in the study area

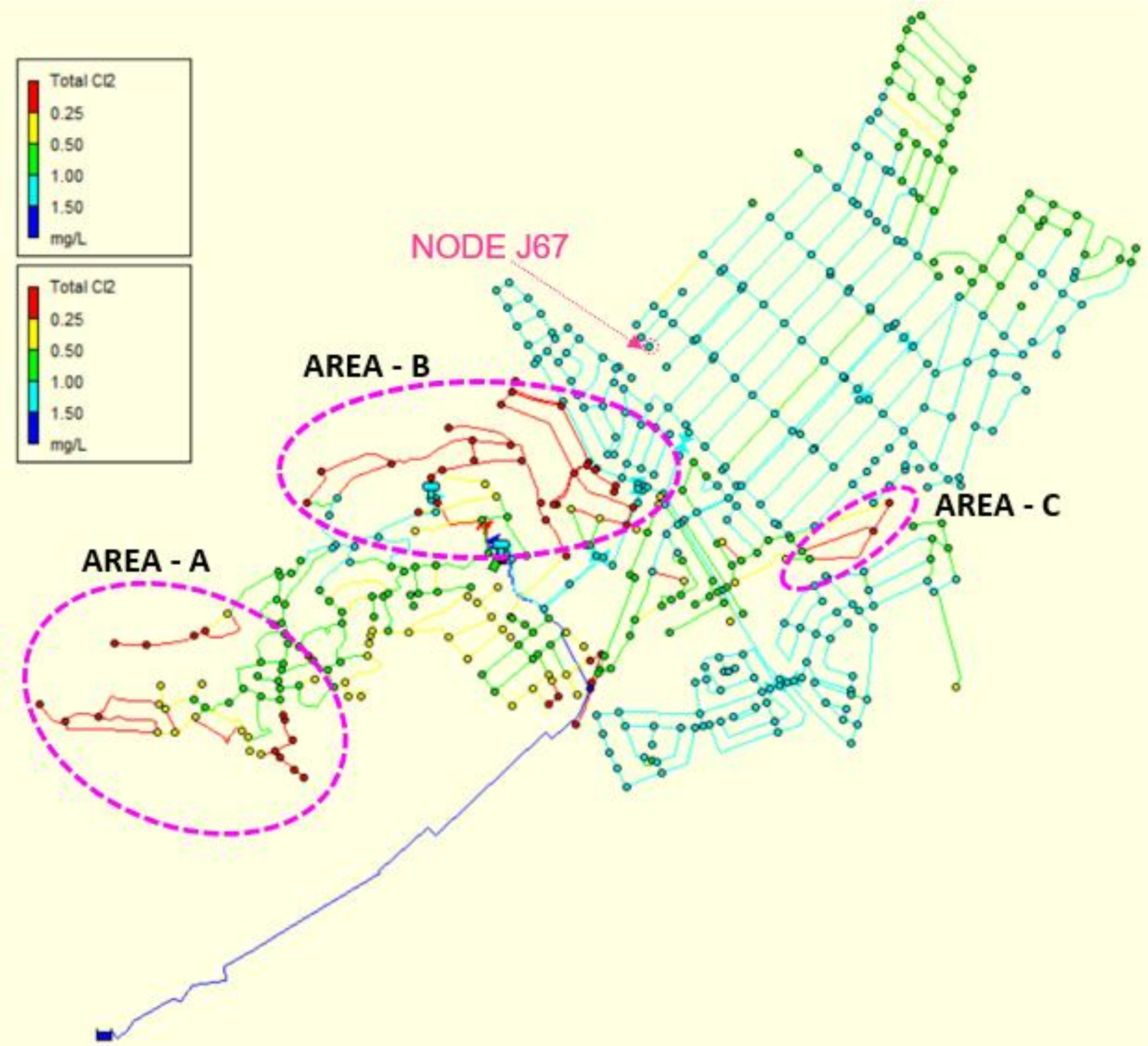


Figure 4-47: MLDM total chlorine concentrations in the study area

While total chlorine is not monitored, it is a useful indicator as it quantifies all chlorine in the system. Notwithstanding the likelihood of the utility sampling point being sub-optimally located, the above confirmed that water age can be used as a proxy indicator for total chlorine and this finding was consistent with literature (EPA, 2002; EPA, 2005; WHO, 2014).

Free chlorine (under normal conditions)

Free chlorine is important as a minimum residual is required to ensure that drinking water is safe for human consumption, it was thus considered as one of the determinants in the analysis. Figures 4-48 and 4-49 below present free chlorine analysis results for the LLDS and the MLDM respectively at time 175.21. The 175.21 time point was used to ensure consistency and allow for reasonable comparison. As was the case with total chlorine, free chlorine levels showed a direct correlation with water age; as such the areas with high water age (Areas A. B and C) translated directly for free chlorine as seen below. This further confirmed that water age can be used as a proxy for free chlorine, and this is also consistent with literature.

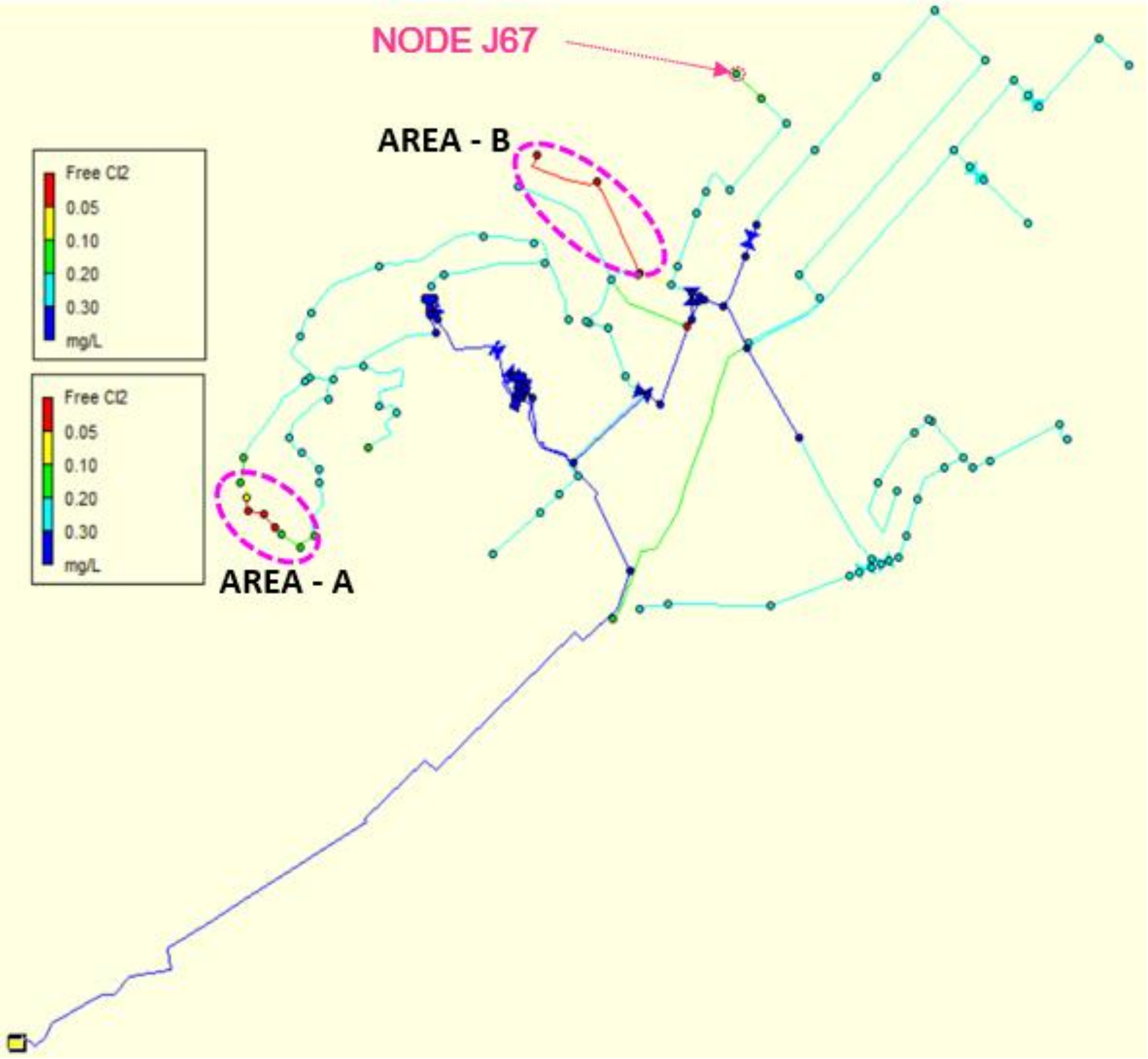


Figure 4-48: LLDS free chlorine concentrations in the study area

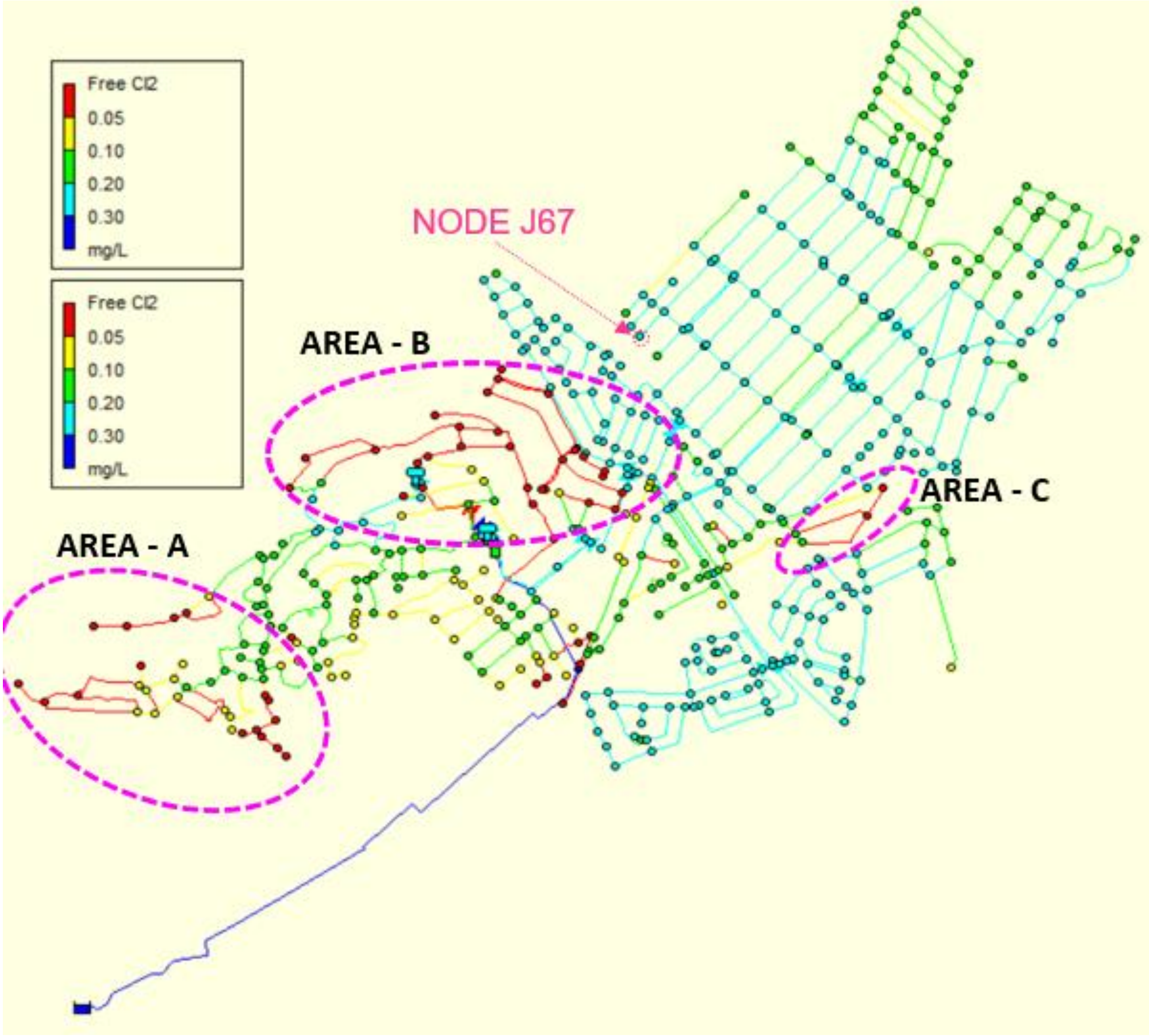


Figure 4-49: MLDM free chlorine concentrations in the study area

The free chlorine levels were extremely concerning as the expected minimum of 0.2mg/l was not in place for large areas in the system. Consistent with previous analysis, the utility sampling point was not identified (in the modelling exercise) as the most critical, though it was below 0.2mg/l in the LLDS (but not in the MLDM). Most alarming was that Areas A, B and C virtually had no free chlorine as the levels were below 0.05 mg/l. Such low levels are not acceptable and require an increase in the chlorine dosage to ensure safe drinking water is supplied. The consistency between the two models (LLDS and MLDM) also confirmed that lowering hydraulic definition (skeletonisation and reduction) was not necessarily detrimental to effective water quality modelling for free chlorine.

4.10 Chapter summary

This chapter presented the results of an evaluation of various considerations for water quality management improvement for a specific Johannesburg Water drinking water distribution network, utilising a free and widely used modelling tool. A prerequisite for the analysis was a hydraulic (pressure) and water quality (total chlorine) calibration. The calibration produced results that were acceptable and was followed by model validation. The validation was undertaken using water quality data, as this was the only data available that was independent of the data used for model calibration. The parameter considered in the model validation was dissolved oxygen. The validation of both the LLDS and MLDM produced acceptable results. This meant that the actual analysis work on the models could be undertaken. The following scenarios were considered:

- The impact of hydraulic definition on water quality;
- The impact of the level of calibration on water quality;
- The significance of pipe age and material on water quality;
- The effects of demand pattern on water quality;
- The impacts of the various stages of load-shedding on water quality;
- The impacts of the four tank (reservoir) mixing models on water quality; and
- Evaluating optimal water quality sample location for water quality risk management.

An analysis of the above-mentioned scenarios produced the following key observations:

- A simplification (reduction in hydraulic definition) of an EPANET 2.0 model was possible through a systematic reduction and skeletonisation. This was undertaken to produce the LLDS and the MLDM for this study. The simplified models were able to model physical (water age and dissolved oxygen), chemical (total chlorine, free chorine and chloramine) and biochemical (dissolved organic carbon and biodegradable organic carbon) determinants. Both the LLDS and the MLDM produced relatively similar results and the results were comparable to the field data for the corresponding nodes.
- The study results concurred with literature on the impropriety of using steady state (normal flow conditions and high flow conditions) models for water quality modelling, as these models produced results that were not useful. The results further concurred with literature, as increased accuracy was obtained through the use of the most calibrated models (variable control status) for both the LLDS and the MLDM. The difference between the LLDS known control status and variable control status models was not as pronounced as that of the two MLDMs. It was therefore clear that the level of calibration was positively correlated to water quality modelling accuracy, but hydraulic definition was more significant.

- The results showed that pipe age and material (both represented by the roughness co-efficient) had a notable impact on water quality. Older pipes (more corroded) presented higher total chlorine loss, but the severity of this loss varied across materials. Plastic pipes demonstrated the greatest utility as a badly corroded plastic pipe produced the same performance (all things being equal) as a new cast iron or vitrified clay pipe. When optimising for water quality, the most suitable materials can be ranked as follows: plastics, smooth concrete and then steel. The results showed that badly corroded steel, cast iron and vitrified clay are not conducive in the maintenance of high chlorine levels. As with other results, the analysis of the LLDS and the MLDM were comparable in this scenario.
- The impact of a variance in demand pattern on water quality was evaluated. Though the difference in the four demand patterns utilised was significant, there was minimal observed impact on water quality. This suggested that the diurnal demand patterns had a greater impact on steady state models (hydraulic), than extended period simulations (water quality). The MLDM and the LLDS produced comparable results in this scenario.
- The analysis showed that load-shedding impacted water quality negatively. The adverse impact of load-shedding on water quality was evident across all stages, but was most pronounced from stages five (5) to eight (8). The mechanism through which load-shedding impacted water quality was water age, as the increase in water age correlated to the reduction in water quality (loss of total chlorine). The results of both the LLDS and the MLDM were comparable in this scenario.
- Tank (reservoir in South African terminology) mixing model variations were shown to have an impact on water quality. Complete mixing and two compartment mixing performed identically, with FIFO plug flow resulting in a minor change. LIFO plug flow (short-circuiting) resulted in the greatest variance for all simulation scenarios. The results of both the LLDS and the MLDM were comparable in this scenario.
- Locating the critical distribution sample point can be made easier by using EPANET 2.0. For the study area under consideration, the Utility's selected sample point (Node J67) did not correlate with the modelled high-risk areas (considering water age and chlorine levels). From the analysis it also appeared that two sample points are required and not the one that the Utility currently samples.

5. Summary, conclusions and recommendations

5.1 Summary

In South Africa municipalities (through Water Services Providers) are legally responsible for the provision of drinking water to end-users, this is primarily achieved through drinking water distribution networks. It is expected that the water supplied will be potable in line with local standards (SANS241:2015), at the point of delivery to the end-user. However, research has shown that managing drinking water quality in a distribution network is a complex undertaking. Chapter 2 explored this complexity by considering structural, operational and water quality contributors to drinking water quality variations in a distribution network. An overview of available water quality modelling software, that seeks to aid utilities in drinking water quality management was also provided. Relevant drinking water quality management, local (South African), continental (African) and global case studies were also reviewed.

Chapter 3 presents the methodology followed in modelling the water quality of a drinking water distribution network, for a portion of the selected WSPs network. This involved: model definition, data collection, model development, calibration, validation and water quality modelling towards the study objectives.

Chapter 4 presents the analysis and discusses the results of the impact of: hydraulic definition, level of calibration, pipe age, pipe material, water demand pattern, load-shedding and tank (reservoir) mixing model on water quality. Consideration of optimal critical sampling point identification for water quality risk management is also made. Water quality in the study is represented through the grouping of three determinants, namely: physical (water age and dissolved oxygen), chemical (total chlorine, free chlorine and chloramine) and biochemical (dissolved organic carbon and biodegradable dissolved organic carbon). Chapter 5 concludes the research report by presenting the summary, conclusions and recommendations for future research.

5.2 Conclusions

The primary objective for all drinking water provision utilities is to supply end users with water that is fit for consumption (in line with local and or international standards). The literature review demonstrated that water quality can deteriorate at all points along the distribution network infrastructure (reservoirs, tanks and pipes), and there are various water quality software models that can simulate this deterioration, with EPANET 2.0 being one of these (and the one selected for this study). This dissertation investigated drinking water quality in a portion of a South African WSP's drinking water distribution network, evaluating various considerations for water quality management improvement.

A key objective of the study was to ascertain whether or not network simplification (through skeletonisation and reduction) undermined water quality modelling in EPANET 2.0. To assess this, two models were developed: the Low-Level Detail System (LLDS) and the Medium-Level Detail Model (MLDM). All other aspects of the study were undertaken for both models, starting with calibration and validation. The Water NZ national modelling guidelines for water distribution network modelling (Water NZ, 2021) were followed in the calibration and validation of the LLDS and the MLDM. The analysis was undertaken through various scenarios.

The first scenario considered the simplification of complex networks through skeletonisation and reduction, which can save utilities time and money. The study considered the extent to which this network simplification affected the accuracy of the LLDS and the MLDM. When considering physical, chemical and biochemical determinants, both the LLDS and MLDM produced useful outputs. However, the MLDM proved accurate for more determinants (dissolved oxygen, total chlorine, chloramine and dissolved organic carbon) than those of the LLDS (free chlorine, biodegradable organic carbon). It was therefore concluded that there is some benefit in investing in additional hydraulic detail as the returns are higher levels of water quality modelling accuracy.

The second scenario considered the extent to which the level of calibration affected the accuracy of the LLDS and the MLDM. As expected, the steady state models did not produce useful outputs and it could therefore be concluded that these were indeed inappropriate for water quality modelling. Accuracy increased with an increase in the level of calibration, as expected. The results of this analysis showed that the level of hydraulic definition was more important than the level of calibration in extended period simulation models. This was because the second best calibrated MLDM (known control status), produced more accurate results than the most calibrated LLDS (variable control status) when considering total chlorine.

The third scenario considered the impacts of pipe material and age on water quality. For both models it was clear that the pipe material and age were significant parameters as there was notable variance in water quality (total chlorine concentrations) with changes in the roughness coefficient (C-value). The total number and length of pipes in the MLDM made it more sensitive to C-value changes, when compared with the LLDS. When considering the same C-value, the MLDM was generally more accurate than the LLDS, as observed from the lower errors (difference from field measurement). Both models followed the same pattern of deterioration, meaning they were both useful for modelling purposes. For both models, the most significant impacts were for badly corroded steel, iron and clay pipes. This meant that special care to determine pipe age (condition) must be taken when modelling networks with these materials, otherwise the model will produce errors that are difficult to explain.

The fourth scenario considered the impact of water demand patterns on water quality. An examination of four notably different patterns showed that the impact of demand patterns on water quality was negligible. Demand pattern impacts are clearer on steady state models and less so on extended period simulations. Practically this means that once a satisfactory level of hydraulic calibration (steady state) has been achieved, the demand pattern should not be altered for the water quality simulation (extended period simulation). Both the LLDS and the MLDM produced acceptable (and similar) results, which meant that both models could be used for this purpose.

The fifth scenario considered the impacts of load-shedding on water quality. The models showed the impacts of load-shedding to be most pronounced from stages 5 to 8. Load-shedding resulted in increased water age, which reduced chlorine levels and thus compromised water quality. Free chlorine concentrations were alarmingly low in load-shedding stages 7 and 8, warranting special precautions to ensure the safety of drinking water for end-users in the study area. Both the LLDS and the MLDM produced similar results, meaning both models were appropriate for this modelling exercise.

The sixth scenario considered the extent to which tank (reservoir) mixing models affected water quality. The simulations showed that complete mixing, two compartment mixing and FIFO plug flow had negligible impact on water quality, as changing between these resulted in little to no difference. The LIFO plug flow (short-circuiting) resulted in significant changes to water quality. This meant that care should be taken to minimise opportunity for short-circuiting in reservoirs as this occurrence may result in adverse impacts on water quality and the reliability of model results. Both the LLDS and MLDM produced comparable results in this analysis, it could therefore be concluded that both models were suitable for this specific analysis.

The final observation considered the utility of EPANET in water quality management through the identification of optimal critical distribution sample points. The spatial visualisation of the network showed that the current sampling location was not optimal on both the LLDS and the MLDM. The poor location of the sampling points undermines the very objective of sampling and needed immediate attention. Both the LLDS and the MLDM produced similar results, indicating their suitability for this modelling purpose.

The study showed that EPANET 2.0 can be used to meaningfully model drinking water quality in the selected study area. Furthermore, considered and cautious network simplification did not undermine modelling objectives. The study also showed the interdependence between hydraulics and water quality. No decision on the one should be taken without a consideration of the impact on the other. This also spoke to the value of simulation software that considers both hydraulic and water quality aspects.

5.3 Recommendations for future research

After a careful consideration of this study’s findings and limitations; potential areas of future research are outlined below:

a) This study utilised EPANET 2.0 only and did not venture into the utilisation of EPANET-MSX. It would therefore be appropriate to undertake a study where the results of EPANET 2.0 and EPANET-MSX are compared. This would assist in determining whether or not the additional effort of modelling using EPANET-MSX is necessary for utilities. The benefits of EPANET-MSX are clear for research purposes, but work should be done to determine if the same holds for utilities and practitioners.

b) The study demonstrated that a reduced and skeletonised model can still be used to meaningfully model water quality. However, stricter guidance to determine the limits of skeletonisation and reduction is needed. Further research should therefore be undertaken to set these guidelines and limits. This work should be accompanied by a guideline of the minimum standards for water distribution network information required to undertake meaningful hydraulic and water quality modelling.

c) The LLDS and MLDM ability to predict water quality with minimal re-calibration should be tested. This can be easily achieved by using present-day hydraulic and water quality information and comparing modelled outputs and field data.

d) EPANET requires useful water quality reaction coefficients (Global Reaction Order, Global Bulk Coefficient, Limiting Concentration and Global Wall Coefficient) for meaningful water quality modelling. Further research should be undertaken to determine these (common ranges) for the South African context; this should be accompanied by guidelines for practitioners with limited chemical and biochemical competencies.

e) Pipe material and age (condition) are both estimated through the use of the roughness coefficient (C-value) in EPANET. If the pipe material is known, C only represents the pipe age (condition), it should therefore be theoretically possible to reasonably estimate pipe condition when field data (hydraulic and water quality) are known. Work to test this theory may prove useful for low-cost continuous pipe condition monitoring.

f) The study only considered the four tank mixing models available in EPANET 2.0. These may be overly simplistic when considering the site-specific characteristics of the reservoirs in the study area. Chlorine decay in reservoirs is also influenced by temperature; as temperature can play a significant role in hydraulic flows (Kruger, 2001). It would therefore be appropriate to undertake a more comprehensive investigation of mixing, using Computational Fluid Dynamics (CFD) software that considers water temperature. The two objectives would be to assess the extent to which the EPANET 2.0 simplified mixing models are useful; the second would be to determine under which conditions short-circuiting occurs and how its impact can be prevented or minimised.

g) The study used water quality data from a previous study (Useh, 2017), in the analysis it was clear that the locations of the water quality sampling points were not representative of the area. A future study should be undertaken with more representative water quality sampling locations.

All of the above should culminate in a South African municipal drinking water network modelling guideline, that considers international best-practise and the local context.

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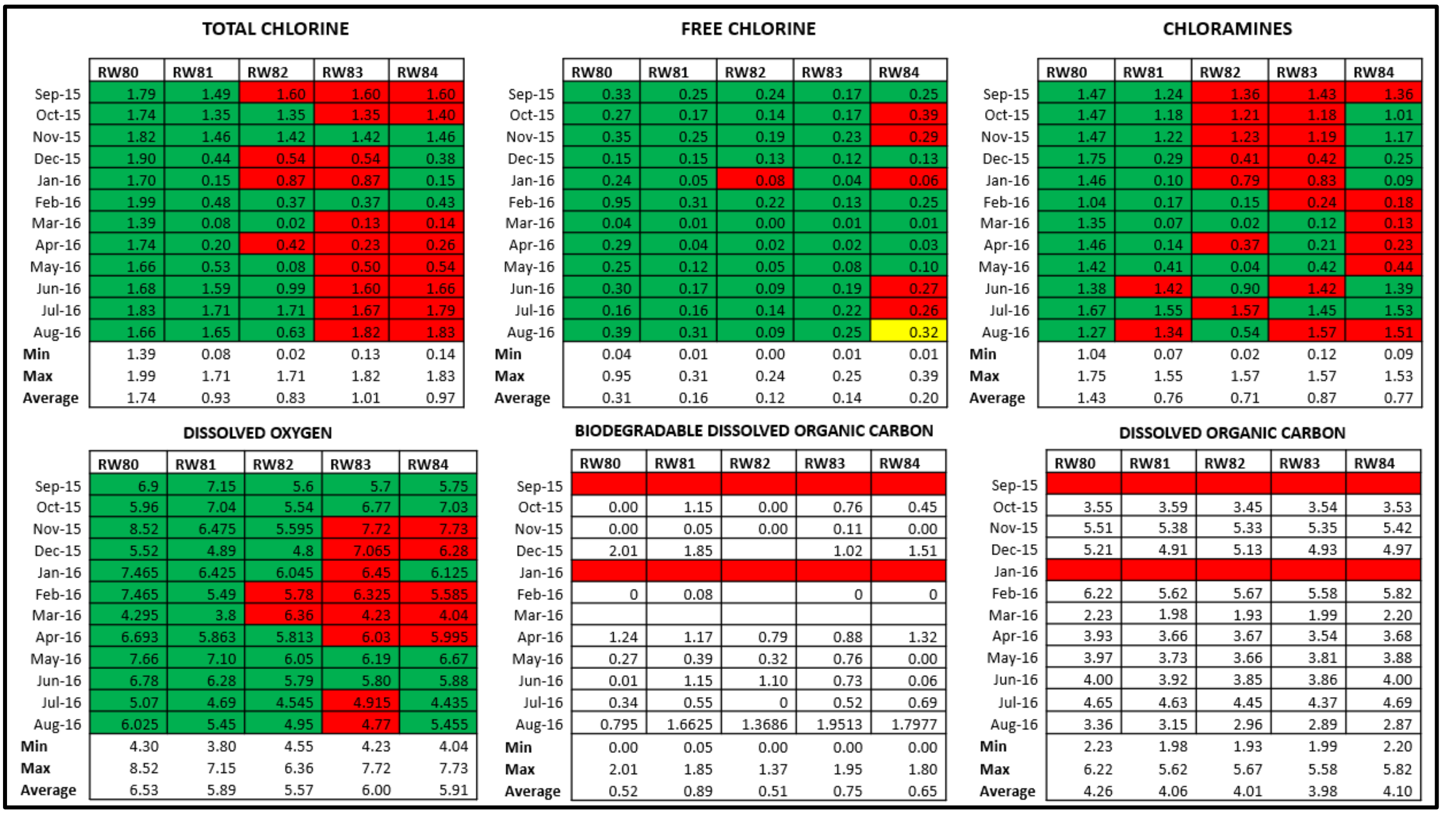
7. Appendices

A Calibration and validation results and data

A.1 Water quality field data

App A1-A: Total chlorine field data (Useh, 2017)



TOTAL CHLORINE

	RW80	RW81	RW82	RW83	RW84
Sep-15	1.79	1.49	1.60	1.60	1.60
Oct-15	1.74	1.35	1.35	1.35	1.40
Nov-15	1.82	1.46	1.42	1.42	1.46
Dec-15	1.90	0.44	0.54	0.54	0.38
Jan-16	1.70	0.15	0.87	0.87	0.15
Feb-16	1.99	0.48	0.37	0.37	0.43
Mar-16	1.39	0.08	0.02	0.13	0.14
Apr-16	1.74	0.20	0.42	0.23	0.26
May-16	1.66	0.53	0.08	0.50	0.54
Jun-16	1.68	1.59	0.99	1.60	1.66
Jul-16	1.83	1.71	1.71	1.67	1.79
Aug-16	1.66	1.65	0.63	1.82	1.83
Min	1.39	0.08	0.02	0.13	0.14
Max	1.99	1.71	1.71	1.82	1.83
Average	1.74	0.93	0.83	1.01	0.97

FREE CHLORINE

	RW80	RW81	RW82	RW83	RW84
Sep-15	0.33	0.25	0.24	0.17	0.25
Oct-15	0.27	0.17	0.14	0.17	0.39
Nov-15	0.35	0.25	0.19	0.23	0.29
Dec-15	0.15	0.15	0.13	0.12	0.13
Jan-16	0.24	0.05	0.08	0.04	0.06
Feb-16	0.95	0.31	0.22	0.13	0.25
Mar-16	0.04	0.01	0.00	0.01	0.01
Apr-16	0.29	0.04	0.02	0.02	0.03
May-16	0.25	0.12	0.05	0.08	0.10
Jun-16	0.30	0.17	0.09	0.19	0.27
Jul-16	0.16	0.16	0.14	0.22	0.26
Aug-16	0.39	0.31	0.09	0.25	0.32
Min	0.04	0.01	0.00	0.01	0.01
Max	0.95	0.31	0.24	0.25	0.39
Average	0.31	0.16	0.12	0.14	0.20

CHLORAMINES

	RW80	RW81	RW82	RW83	RW84
Sep-15	1.47	1.24	1.36	1.43	1.36
Oct-15	1.47	1.18	1.21	1.18	1.01
Nov-15	1.47	1.22	1.23	1.19	1.17
Dec-15	1.75	0.29	0.41	0.42	0.25
Jan-16	1.46	0.10	0.79	0.83	0.09
Feb-16	1.04	0.17	0.15	0.24	0.18
Mar-16	1.35	0.07	0.02	0.12	0.13
Apr-16	1.46	0.14	0.37	0.21	0.23
May-16	1.42	0.41	0.04	0.42	0.44
Jun-16	1.38	1.42	0.90	1.42	1.39
Jul-16	1.67	1.55	1.57	1.45	1.53
Aug-16	1.27	1.34	0.54	1.57	1.51
Min	1.04	0.07	0.02	0.12	0.09
Max	1.75	1.55	1.57	1.57	1.53
Average	1.43	0.76	0.71	0.87	0.77

DISSOLVED OXYGEN

	RW80	RW81	RW82	RW83	RW84
Sep-15	6.9	7.15	5.6	5.7	5.75
Oct-15	5.96	7.04	5.54	6.77	7.03
Nov-15	8.52	6.475	5.595	7.72	7.73
Dec-15	5.52	4.89	4.8	7.065	6.28
Jan-16	7.465	6.425	6.045	6.45	6.125
Feb-16	7.465	5.49	5.78	6.325	5.585
Mar-16	4.295	3.8	6.36	4.23	4.04
Apr-16	6.693	5.863	5.813	6.03	5.995
May-16	7.66	7.10	6.05	6.19	6.67
Jun-16	6.78	6.28	5.79	5.80	5.88
Jul-16	5.07	4.69	4.545	4.915	4.435
Aug-16	6.025	5.45	4.95	4.77	5.455
Min	4.30	3.80	4.55	4.23	4.04
Max	8.52	7.15	6.36	7.72	7.73
Average	6.53	5.89	5.57	6.00	5.91

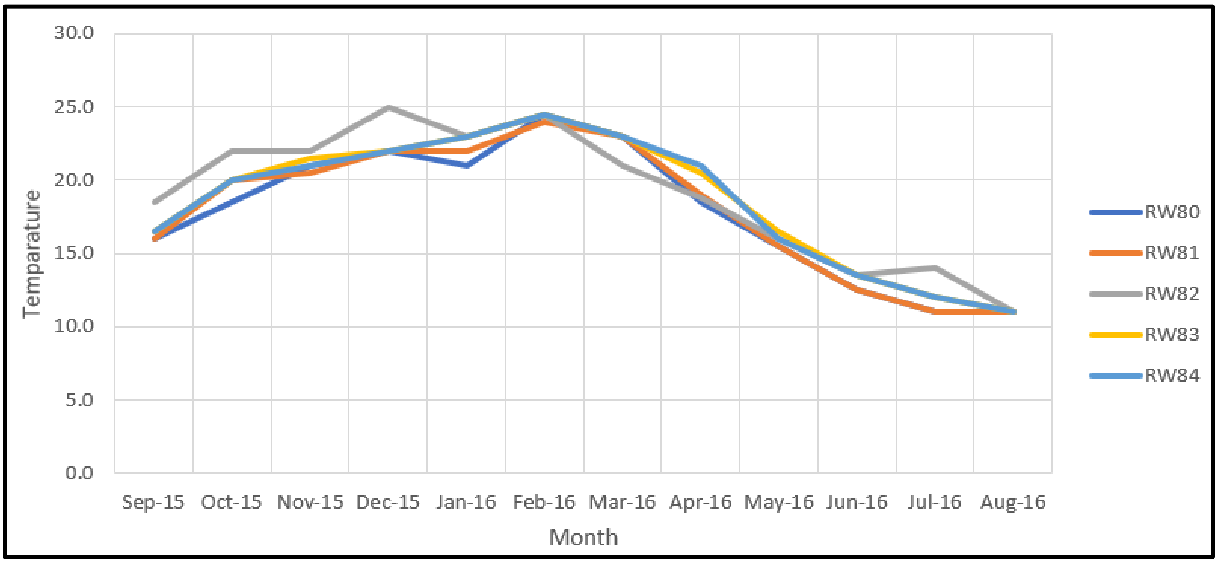
BIODEGRADABLE DISSOLVED ORGANIC CARBON

	RW80	RW81	RW82	RW83	RW84
Sep-15	🟥	🟥	🟥	🟥	🟥
Oct-15	0.00	1.15	0.00	0.76	0.45
Nov-15	0.00	0.05	0.00	0.11	0.00
Dec-15	2.01	1.85		1.02	1.51
Jan-16	🟥	🟥	🟥	🟥	🟥
Feb-16	0	0.08		0	0
Mar-16					
Apr-16	1.24	1.17	0.79	0.88	1.32
May-16	0.27	0.39	0.32	0.76	0.00
Jun-16	0.01	1.15	1.10	0.73	0.06
Jul-16	0.34	0.55	0	0.52	0.69
Aug-16	0.795	1.6625	1.3686	1.9513	1.7977
Min	0.00	0.05	0.00	0.00	0.00
Max	2.01	1.85	1.37	1.95	1.80
Average	0.52	0.89	0.51	0.75	0.65

DISSOLVED ORGANIC CARBON

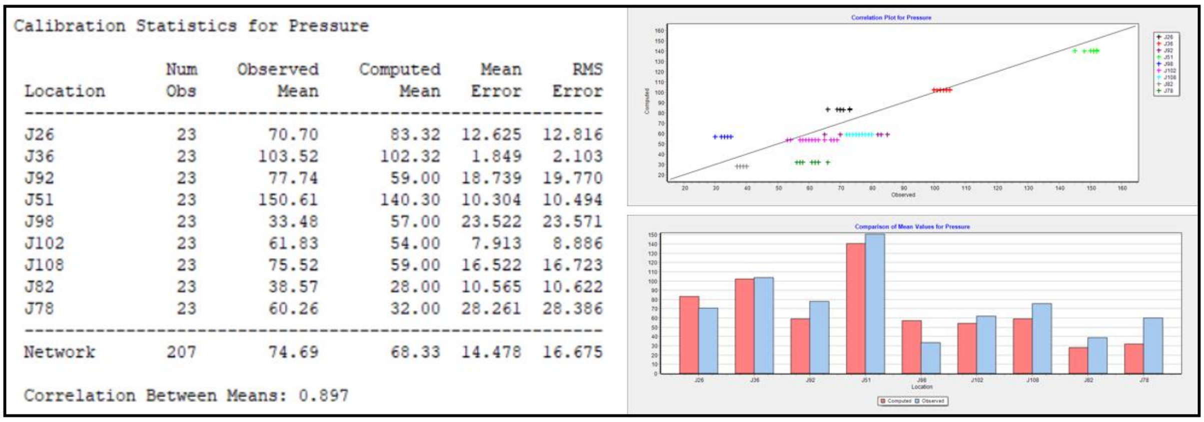
	RW80	RW81	RW82	RW83	RW84
Sep-15	🟥	🟥	🟥	🟥	🟥
Oct-15	3.55	3.59	3.45	3.54	3.53
Nov-15	5.51	5.38	5.33	5.35	5.42
Dec-15	5.21	4.91	5.13	4.93	4.97
Jan-16	🟥	🟥	🟥	🟥	🟥
Feb-16	6.22	5.62	5.67	5.58	5.82
Mar-16	2.23	1.98	1.93	1.99	2.20
Apr-16	3.93	3.66	3.67	3.54	3.68
May-16	3.97	3.73	3.66	3.81	3.88
Jun-16	4.00	3.92	3.85	3.86	4.00
Jul-16	4.65	4.63	4.45	4.37	4.69
Aug-16	3.36	3.15	2.96	2.89	2.87
Min	2.23	1.98	1.93	1.99	2.20
Max	6.22	5.62	5.67	5.58	5.82
Average	4.26	4.06	4.01	3.98	4.10

App A1-B: Average temperatures at field sample points (Useh, 2017)



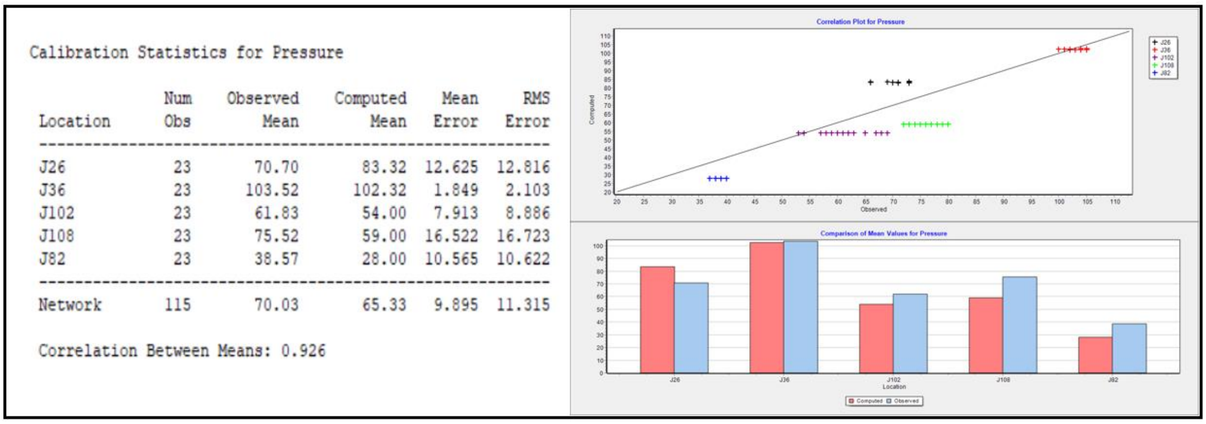
A.2 LLDS calibration and validation data

App A2-A: SS_NFC_LLDS pressure calibration (all nodes)



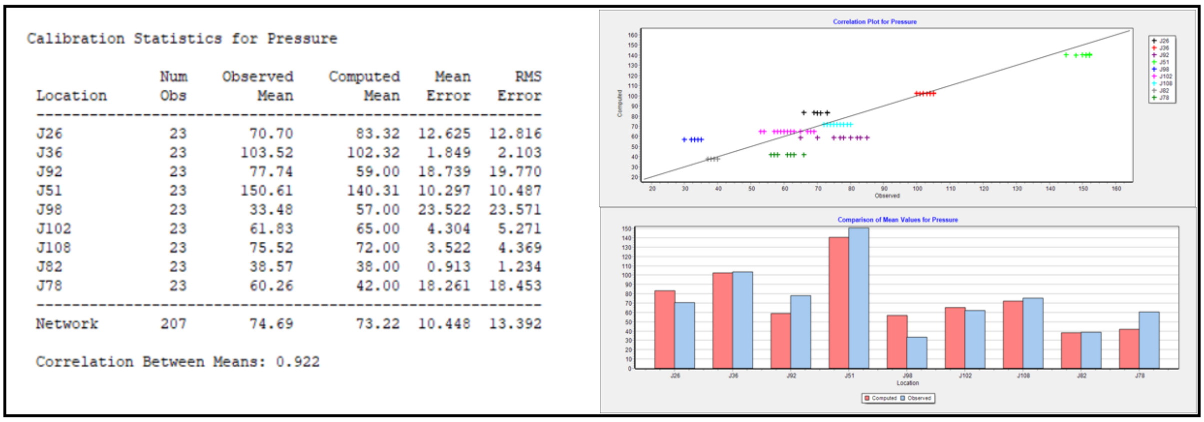
```
Calibration Statistics for Pressure

                 Num    Observed    Computed    Mean     RMS
 Location        Obs        Mean        Mean   Error   Error
 -----------------------------------------------------------
 J26              23       70.70       83.32  12.625  12.816
 J36              23      103.52      102.32   1.849   2.103
 J92              23       77.74       59.00  18.739  19.770
 J51              23      150.61      140.30  10.304  10.494
 J98              23       33.48       57.00  23.522  23.571
 J102             23       61.83       54.00   7.913   8.886
 J108             23       75.52       59.00  16.522  16.723
 J82              23       38.57       28.00  10.565  10.622
 J78              23       60.26       32.00  28.261  28.386
 -----------------------------------------------------------
 Network         207       74.69       68.33  14.478  16.675

 Correlation Between Means: 0.897
```

App A2-B: SS_NFC_LLDS pressure calibration graph (selected nodes)



```
Calibration Statistics for Pressure

                 Num    Observed    Computed    Mean     RMS
 Location        Obs        Mean        Mean   Error   Error
 -----------------------------------------------------------
 J26              23       70.70       83.32  12.625  12.816
 J36              23      103.52      102.32   1.849   2.103
 J102             23       61.83       54.00   7.913   8.886
 J108             23       75.52       59.00  16.522  16.723
 J82              23       38.57       28.00  10.565  10.622
 -----------------------------------------------------------
 Network         115       70.03       65.33   9.895  11.315

 Correlation Between Means: 0.926
```

App A2-C: SS_HFC_LLDS pressure calibration graph (all nodes)

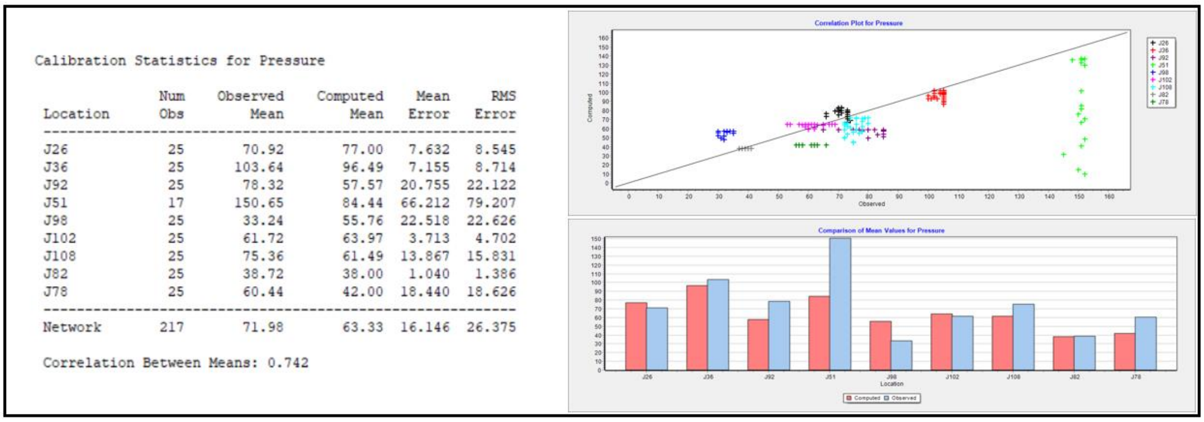


```
Calibration Statistics for Pressure

                 Num    Observed    Computed    Mean     RMS
 Location        Obs        Mean        Mean   Error   Error
 -----------------------------------------------------------
 J26              23       70.70       83.32  12.625  12.816
 J36              23      103.52      102.32   1.849   2.103
 J92              23       77.74       59.00  18.739  19.770
 J51              23      150.61      140.31  10.297  10.487
 J98              23       33.48       57.00  23.522  23.571
 J102             23       61.83       65.00   4.304   5.271
 J108             23       75.52       72.00   3.522   4.369
 J82              23       38.57       38.00   0.913   1.234
 J78              23       60.26       42.00  18.261  18.453
 -----------------------------------------------------------
 Network         207       74.69       73.22  10.448  13.392

 Correlation Between Means: 0.922
```

App A2-D: SS_HFC_LLDS pressure calibration (selected nodes)

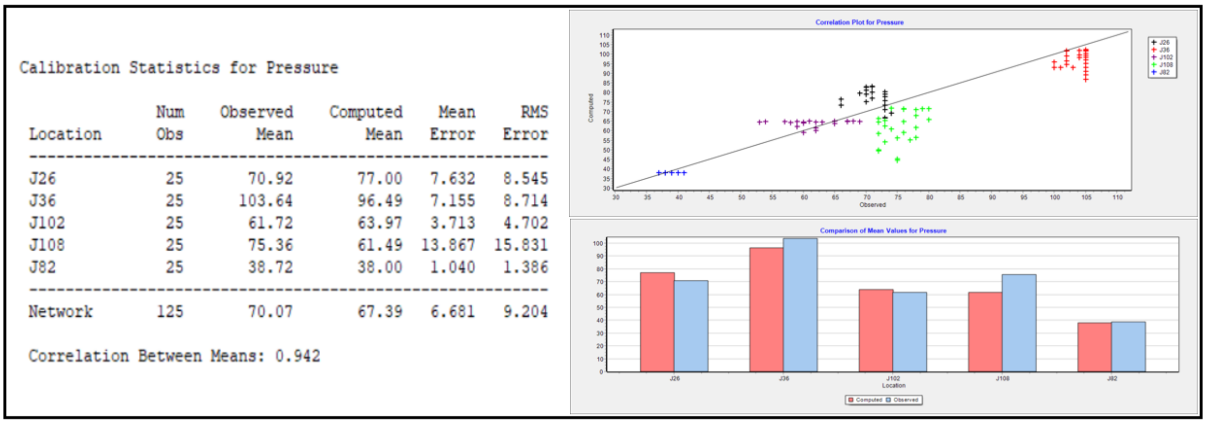
```
Calibration Statistics for Pressure

                Num    Observed    Computed    Mean     RMS
Location        Obs        Mean        Mean   Error   Error
------------------------------------------------------------
J26              23       70.70       83.32  12.625  12.816
J36              23      103.52      102.32   1.849   2.103
J102             23       61.83       65.00   4.304   5.271
J108             23       75.52       72.00   3.522   4.369
J82              23       38.57       38.00   0.913   1.234
------------------------------------------------------------
Network         115       70.03       72.13   4.643   6.589

Correlation Between Means: 0.964
```

App A2-E: EPS_KCS_LLDS pressure calibration graph (all nodes)



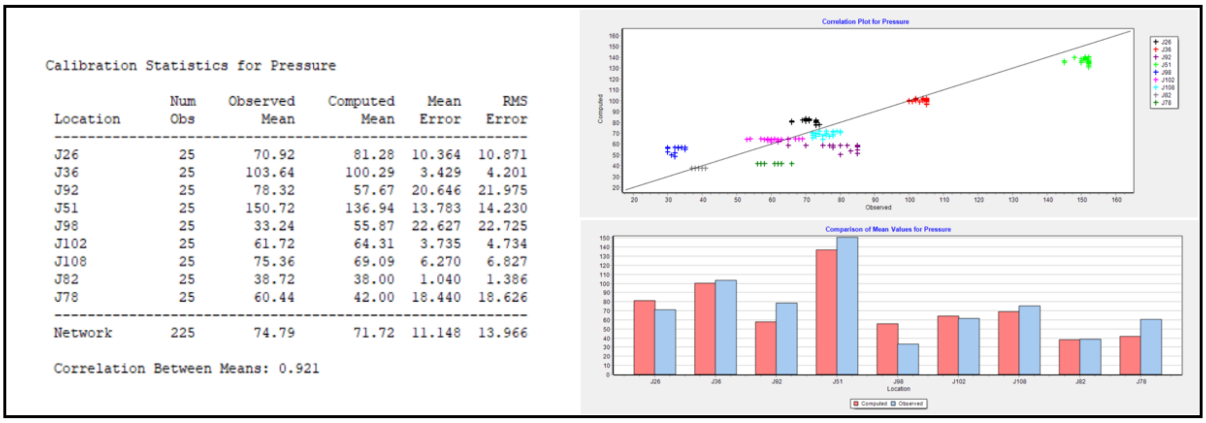
```
Calibration Statistics for Pressure

                Num    Observed    Computed    Mean     RMS
Location        Obs        Mean        Mean   Error   Error
------------------------------------------------------------
J26              25       70.92       77.00   7.632   8.545
J36              25      103.64       96.49   7.155   8.714
J92              25       78.32       57.57  20.755  22.122
J51              17      150.65       84.44  66.212  79.207
J98              25       33.24       55.76  22.518  22.626
J102             25       61.72       63.97   3.713   4.702
J108             25       75.36       61.49  13.867  15.831
J82              25       38.72       38.00   1.040   1.386
J78              25       60.44       42.00  18.440  18.626
------------------------------------------------------------
Network         217       71.98       63.33  16.146  26.375

Correlation Between Means: 0.742
```

App A2-F: EPS_KCS_LLDS pressure calibration graph (selected nodes)



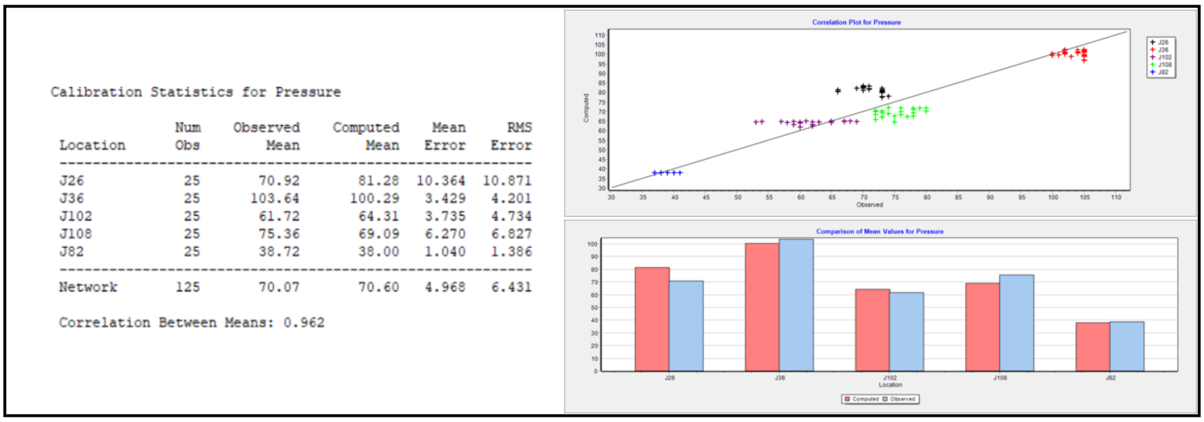
```
Calibration Statistics for Pressure

                Num    Observed    Computed    Mean     RMS
Location        Obs        Mean        Mean   Error   Error
------------------------------------------------------------
J26              25       70.92       77.00   7.632   8.545
J36              25      103.64       96.49   7.155   8.714
J102             25       61.72       63.97   3.713   4.702
J108             25       75.36       61.49  13.867  15.831
J82              25       38.72       38.00   1.040   1.386
------------------------------------------------------------
Network         125       70.07       67.39   6.681   9.204

Correlation Between Means: 0.942
```

App A2-G: EPS_VCS_LLDS pressure calibration graph (all nodes)



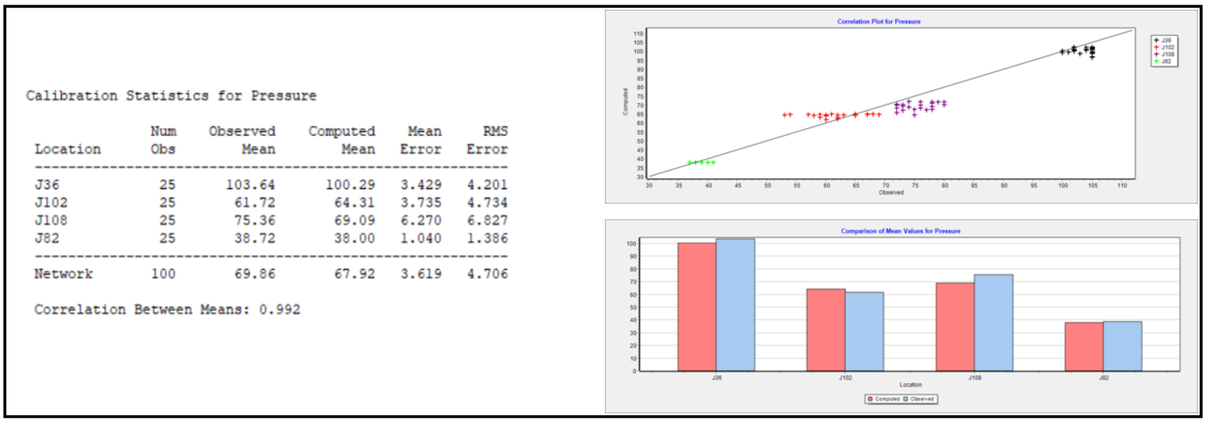
```
Calibration Statistics for Pressure

                 Num    Observed    Computed    Mean     RMS
 Location        Obs        Mean        Mean   Error   Error
 ---------------------------------------------------------
 J26              25       70.92       81.28  10.364  10.871
 J36              25      103.64      100.29   3.429   4.201
 J92              25       78.32       57.67  20.646  21.975
 J51              25      150.72      136.94  13.783  14.230
 J98              25       33.24       55.87  22.627  22.725
 J102             25       61.72       64.31   3.735   4.734
 J108             25       75.36       69.09   6.270   6.827
 J82              25       38.72       38.00   1.040   1.386
 J78              25       60.44       42.00  18.440  18.626
 ---------------------------------------------------------
 Network         225       74.79       71.72  11.148  13.966

 Correlation Between Means: 0.921
```

App A2-H: EPS_VCS_LLDS pressure calibration results (selected nodes 1)



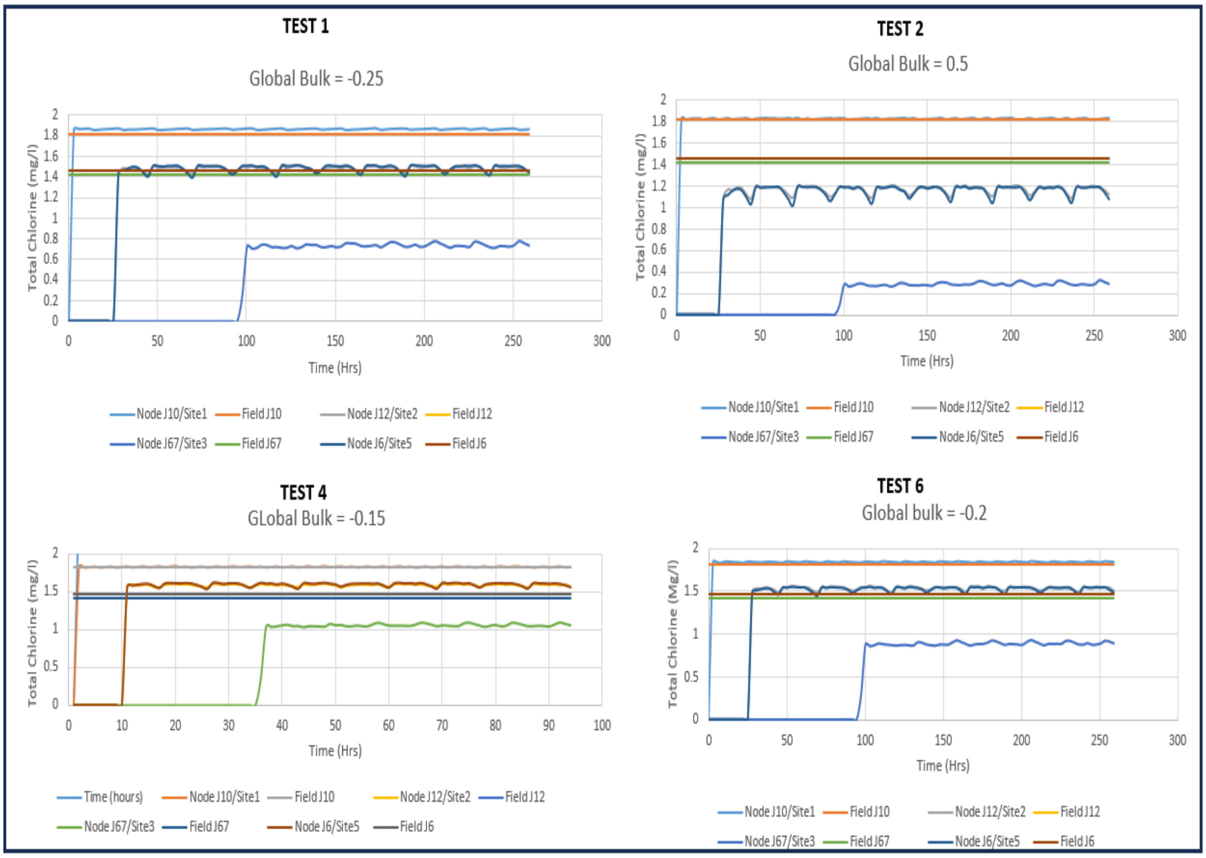
```
Calibration Statistics for Pressure

                 Num    Observed    Computed    Mean     RMS
 Location        Obs        Mean        Mean   Error   Error
 ---------------------------------------------------------
 J26              25       70.92       81.28  10.364  10.871
 J36              25      103.64      100.29   3.429   4.201
 J102             25       61.72       64.31   3.735   4.734
 J108             25       75.36       69.09   6.270   6.827
 J82              25       38.72       38.00   1.040   1.386
 ---------------------------------------------------------
 Network         125       70.07       70.60   4.968   6.431

 Correlation Between Means: 0.962
```

App A2-I: EPS_VCS_LLDS pressure calibration results (selected nodes 2)

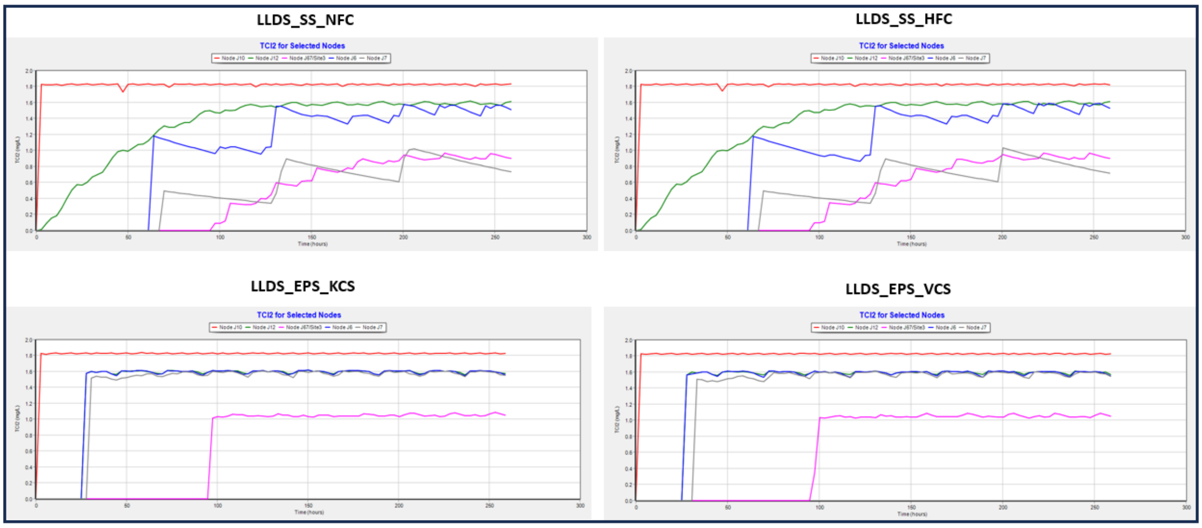


```
Calibration Statistics for Pressure

                 Num    Observed    Computed    Mean     RMS
 Location        Obs        Mean        Mean   Error   Error
 ---------------------------------------------------------
 J36              25      103.64      100.29   3.429   4.201
 J102             25       61.72       64.31   3.735   4.734
 J108             25       75.36       69.09   6.270   6.827
 J82              25       38.72       38.00   1.040   1.386
 ---------------------------------------------------------
 Network         100       69.86       67.92   3.619   4.706

 Correlation Between Means: 0.992
```

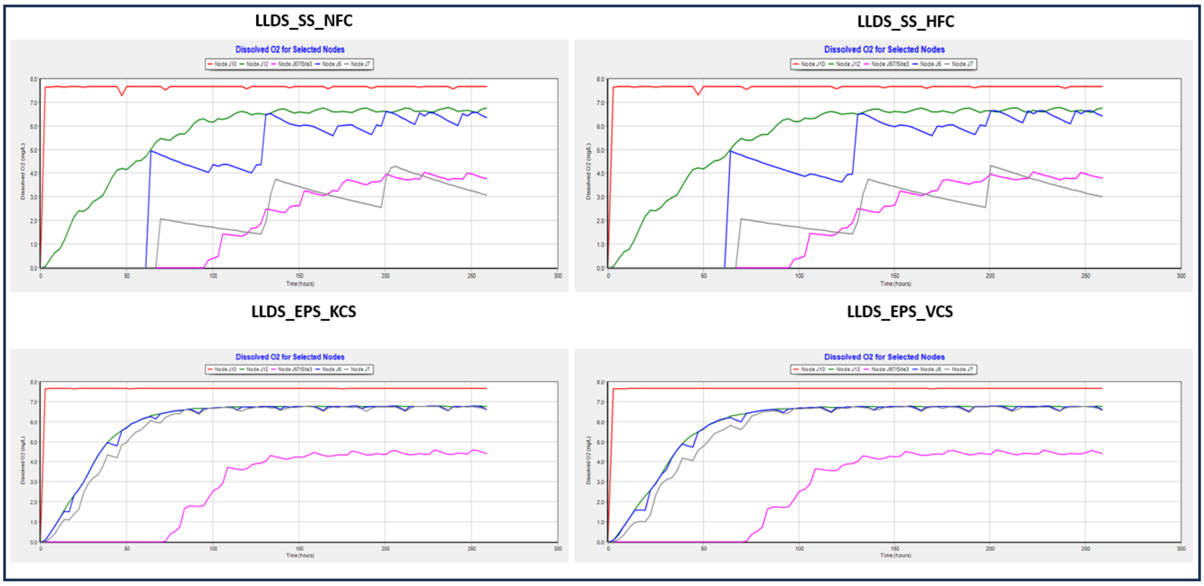
App A2-J: LLDS water quality calibration results (initial)



App A2-K: LLDS water quality calibration results (final)

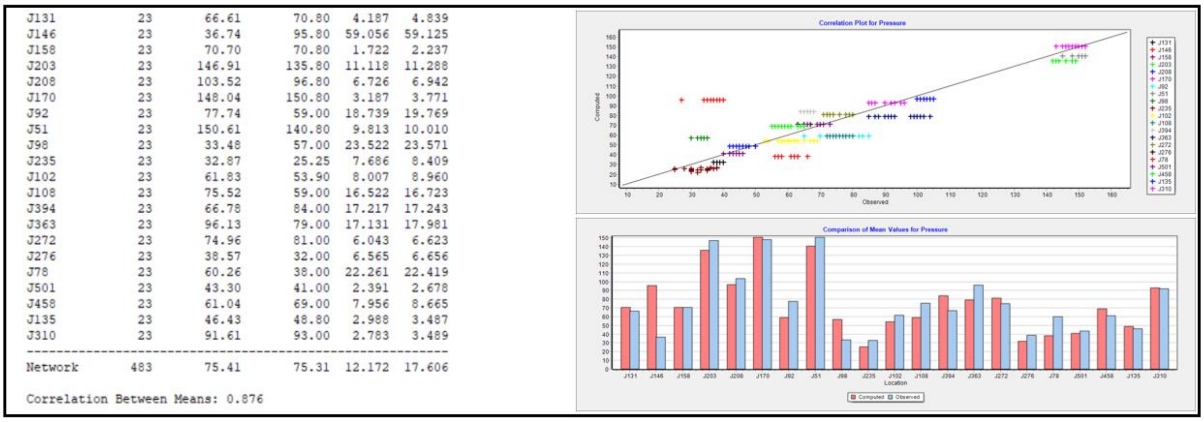


App A2-L: LLDS water quality validation results (selected nodes)



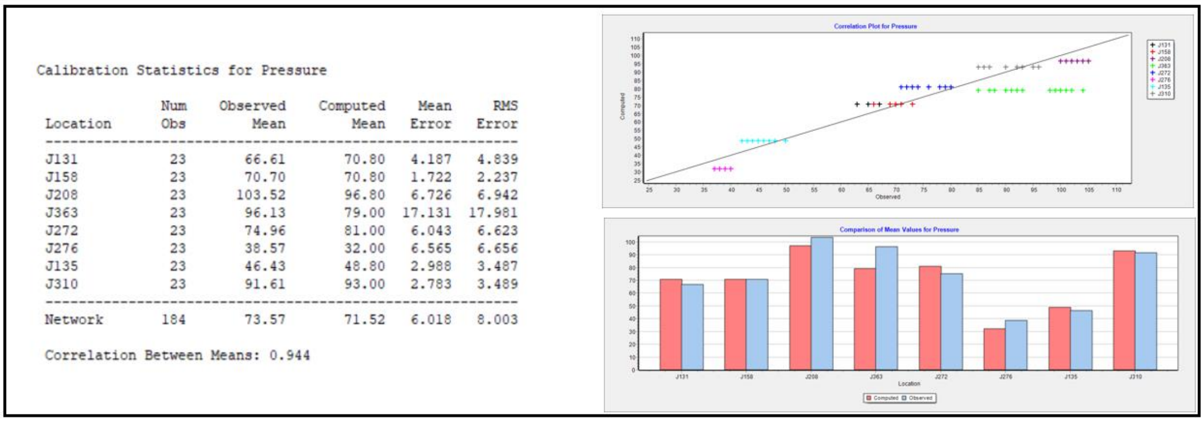
A.3 MLDM calibration and validation data

App A3-A: SS_NFC_MLDM pressure calibration (all nodes)



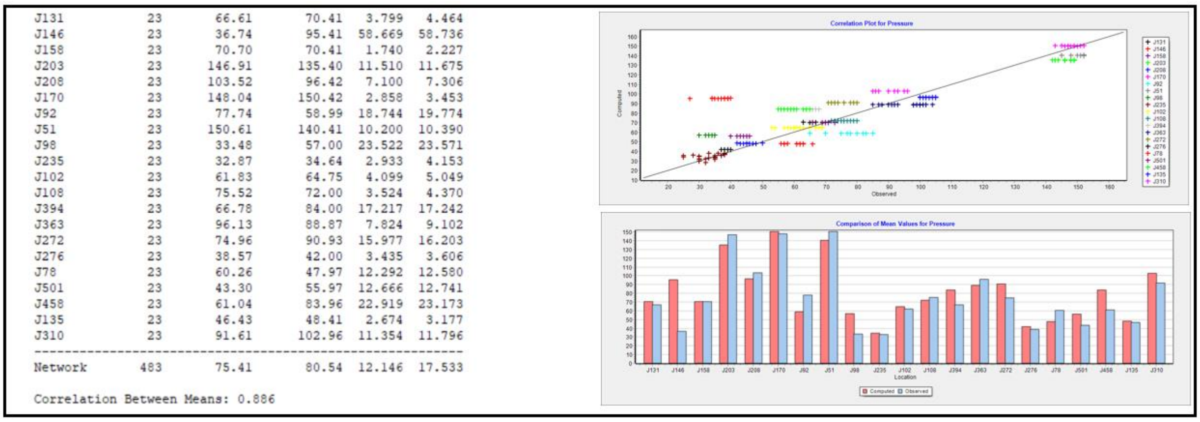
```
J131          23      66.61      70.80   4.187   4.839
J146          23      36.74      95.80  59.056  59.125
J158          23      70.70      70.80   1.722   2.237
J203          23     146.91     135.80  11.118  11.288
J208          23     103.52      96.80   6.726   6.942
J170          23     148.04     150.80   3.187   3.771
J92           23      77.74      59.00  18.739  19.769
J51           23     150.61     140.80   9.813  10.010
J98           23      33.48      57.00  23.522  23.571
J235          23      32.87      25.25   7.686   8.409
J102          23      61.83      53.90   8.007   8.960
J108          23      75.52      59.00  16.522  16.723
J394          23      66.78      84.00  17.217  17.243
J363          23      96.13      79.00  17.131  17.981
J272          23      74.96      81.00   6.043   6.623
J276          23      38.57      32.00   6.565   6.656
J78           23      60.26      38.00  22.261  22.419
J501          23      43.30      41.00   2.391   2.678
J458          23      61.04      69.00   7.956   8.665
J135          23      46.43      48.80   2.988   3.487
J310          23      91.61      93.00   2.783   3.489
-------------------------------------------------------
Network      483      75.41      75.31  12.172  17.606

Correlation Between Means: 0.876
```

pp A3-B: SS_NFC_MLDM pressure calibration (selected nodes)



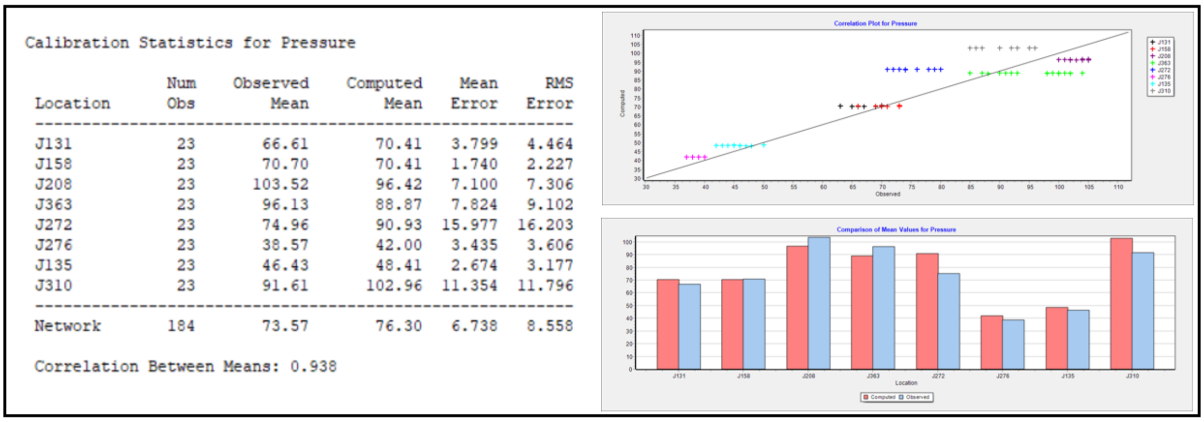
```
Calibration Statistics for Pressure

                Num   Observed    Computed    Mean     RMS
 Location       Obs       Mean        Mean   Error   Error
 ----------------------------------------------------------
 J131            23      66.61       70.80   4.187   4.839
 J158            23      70.70       70.80   1.722   2.237
 J208            23     103.52       96.80   6.726   6.942
 J363            23      96.13       79.00  17.131  17.981
 J272            23      74.96       81.00   6.043   6.623
 J276            23      38.57       32.00   6.565   6.656
 J135            23      46.43       48.80   2.988   3.487
 J310            23      91.61       93.00   2.783   3.489
 ----------------------------------------------------------
 Network        184      73.57       71.52   6.018   8.003

 Correlation Between Means: 0.944
```

App A3-C: SS_HFC_MLDM pressure calibration (all nodes)



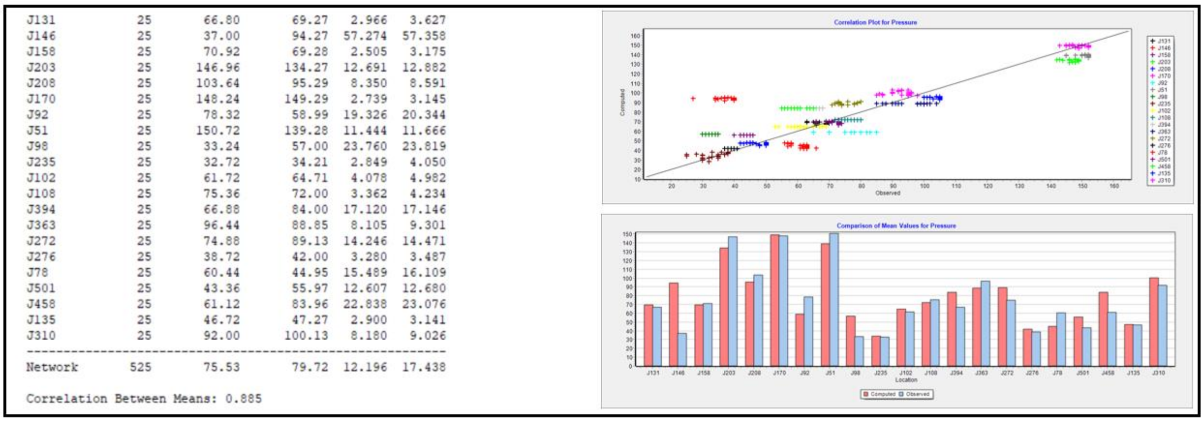
```
J131          23      66.61      70.41   3.799   4.464
J146          23      36.74      95.41  58.669  58.736
J158          23      70.70      70.41   1.740   2.227
J203          23     146.91     135.40  11.510  11.675
J208          23     103.52      96.42   7.100   7.306
J170          23     148.04     150.42   2.858   3.453
J92           23      77.74      58.99  18.744  19.774
J51           23     150.61     140.41  10.200  10.390
J98           23      33.48      57.00  23.522  23.571
J235          23      32.87      34.64   2.933   4.153
J102          23      61.83      64.75   4.099   5.049
J108          23      75.52      72.00   3.524   4.370
J394          23      66.78      84.00  17.217  17.242
J363          23      96.13      88.87   7.824   9.102
J272          23      74.96      90.93  15.977  16.203
J276          23      38.57      42.00   3.435   3.606
J78           23      60.26      47.97  12.292  12.580
J501          23      43.30      55.97  12.666  12.741
J458          23      61.04      83.96  22.919  23.173
J135          23      46.43      48.41   2.674   3.177
J310          23      91.61     102.96  11.354  11.796
------------------------------------------------------
Network      483      75.41      80.54  12.146  17.533

Correlation Between Means: 0.886
```

App A3-D: SS_HFC_MLDM pressure calibration (selected nodes)



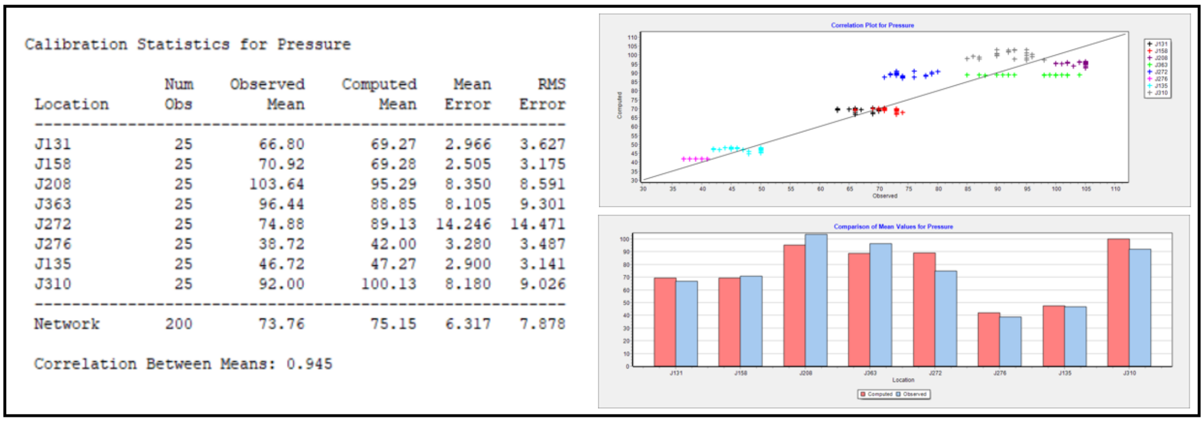
```
Calibration Statistics for Pressure

                Num   Observed    Computed    Mean    RMS
 Location       Obs       Mean        Mean   Error  Error
 -------------------------------------------------------
 J131            23      66.61       70.41   3.799   4.464
 J158            23      70.70       70.41   1.740   2.227
 J208            23     103.52       96.42   7.100   7.306
 J363            23      96.13       88.87   7.824   9.102
 J272            23      74.96       90.93  15.977  16.203
 J276            23      38.57       42.00   3.435   3.606
 J135            23      46.43       48.41   2.674   3.177
 J310            23      91.61      102.96  11.354  11.796
 -------------------------------------------------------
 Network        184      73.57       76.30   6.738   8.558

 Correlation Between Means: 0.938
```

App A3-E: EPS_KCS_MLDM pressure calibration (all nodes)



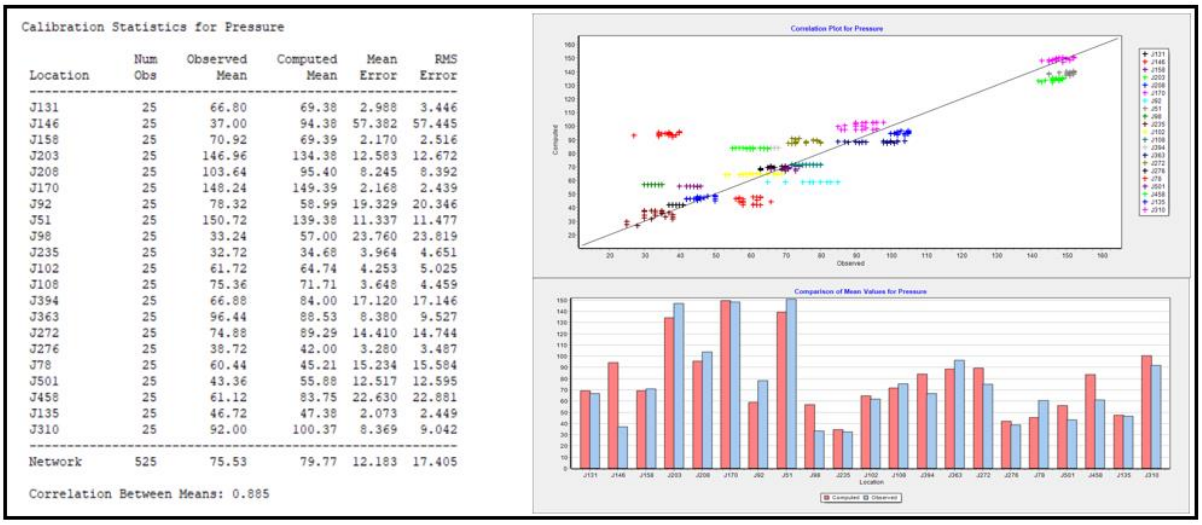
```
J131          25      66.80      69.27   2.966   3.627
J146          25      37.00      94.27  57.274  57.358
J158          25      70.92      69.28   2.505   3.175
J203          25     146.96     134.27  12.691  12.882
J208          25     103.64      95.29   8.350   8.591
J170          25     148.24     149.29   2.739   3.145
J92           25      78.32      58.99  19.326  20.344
J51           25     150.72     139.28  11.444  11.666
J98           25      33.24      57.00  23.760  23.819
J235          25      32.72      34.21   2.849   4.050
J102          25      61.72      64.71   4.078   4.982
J108          25      75.36      72.00   3.362   4.234
J394          25      66.88      84.00  17.120  17.146
J363          25      96.44      88.85   8.105   9.301
J272          25      74.88      89.13  14.246  14.471
J276          25      38.72      42.00   3.280   3.487
J78           25      60.44      44.95  15.489  16.109
J501          25      43.36      55.97  12.607  12.680
J458          25      61.12      83.96  22.838  23.076
J135          25      46.72      47.27   2.900   3.141
J310          25      92.00     100.13   8.180   9.026
------------------------------------------------------
Network      525      75.53      79.72  12.196  17.438

Correlation Between Means: 0.885
```

App A3-F: EPS_KCS_MLDM pressure calibration (selected nodes)



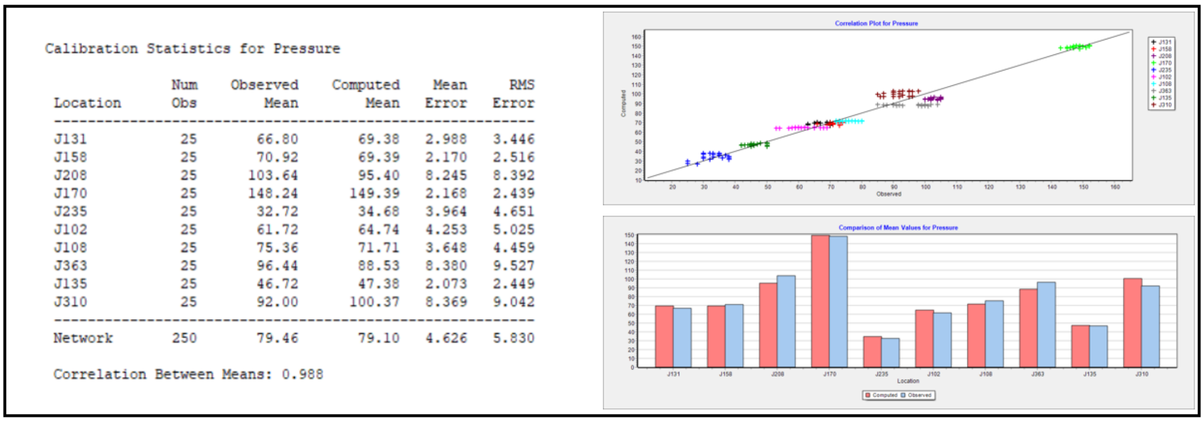
```
Calibration Statistics for Pressure

                 Num    Observed    Computed    Mean     RMS
Location         Obs        Mean        Mean   Error   Error
-----------------------------------------------------------
J131              25       66.80       69.27   2.966   3.627
J158              25       70.92       69.28   2.505   3.175
J208              25      103.64       95.29   8.350   8.591
J363              25       96.44       88.85   8.105   9.301
J272              25       74.88       89.13  14.246  14.471
J276              25       38.72       42.00   3.280   3.487
J135              25       46.72       47.27   2.900   3.141
J310              25       92.00      100.13   8.180   9.026
-----------------------------------------------------------
Network          200       73.76       75.15   6.317   7.878

Correlation Between Means: 0.945
```

App A3-G: EPS_VCS_MLDM pressure calibration (all nodes)



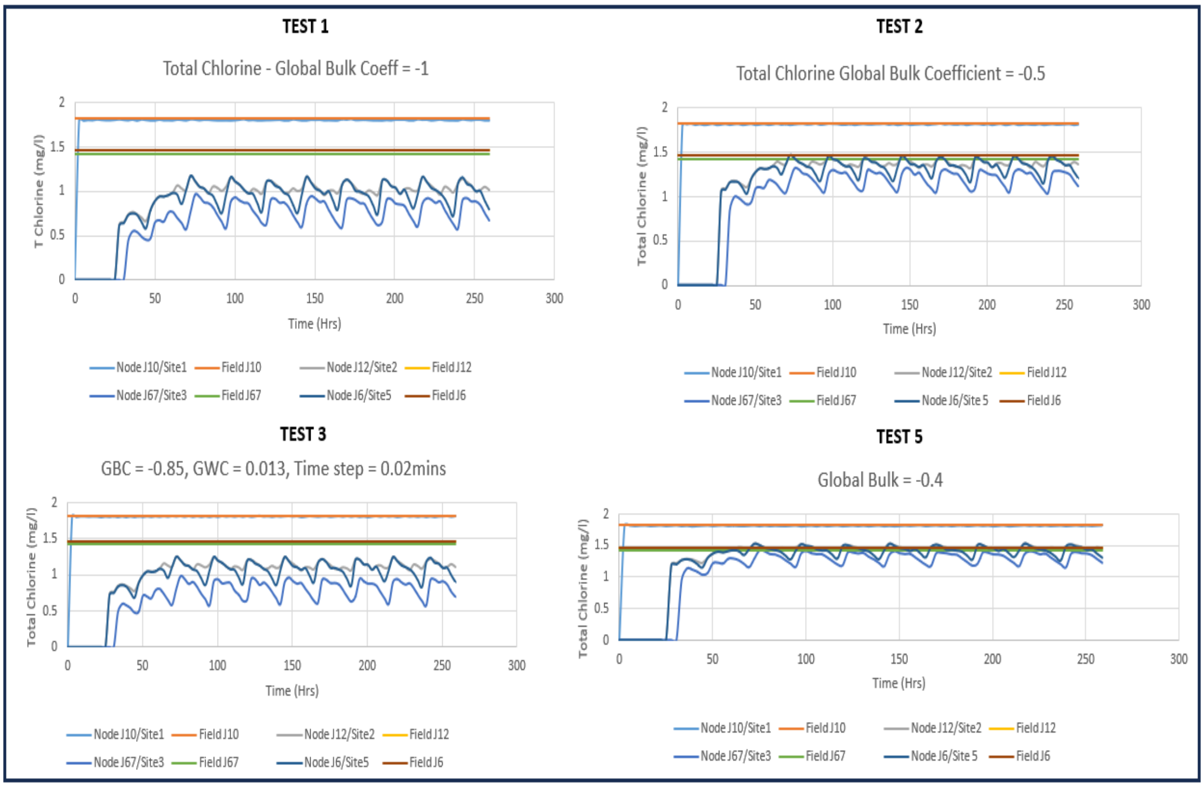
```
Calibration Statistics for Pressure

              Num   Observed   Computed    Mean     RMS
Location      Obs       Mean       Mean   Error   Error
-------------------------------------------------------
J131           25      66.80      69.38   2.988   3.446
J146           25      37.00      94.38  57.382  57.445
J158           25      70.92      69.39   2.170   2.516
J203           25     146.96     134.38  12.583  12.672
J208           25     103.64      95.40   8.245   8.392
J170           25     148.24     149.39   2.168   2.439
J92            25      78.32      58.99  19.329  20.346
J51            25     150.72     139.38  11.337  11.477
J98            25      33.24      57.00  23.760  23.819
J235           25      32.72      34.68   3.964   4.651
J102           25      61.72      64.74   4.253   5.025
J108           25      75.36      71.71   3.648   4.459
J394           25      66.88      84.00  17.120  17.146
J363           25      96.44      88.53   8.380   9.527
J272           25      74.88      89.29  14.410  14.744
J276           25      38.72      42.00   3.280   3.487
J78            25      60.44      45.21  15.234  15.584
J501           25      43.36      55.88  12.517  12.595
J458           25      61.12      83.75  22.630  22.881
J135           25      46.72      47.38   2.073   2.449
J310           25      92.00     100.37   8.369   9.042
-------------------------------------------------------
Network       525      75.53      79.77  12.183  17.405

Correlation Between Means: 0.885
```

App A3-H: EPS_VCS_MLDM pressure calibration (selected nodes)

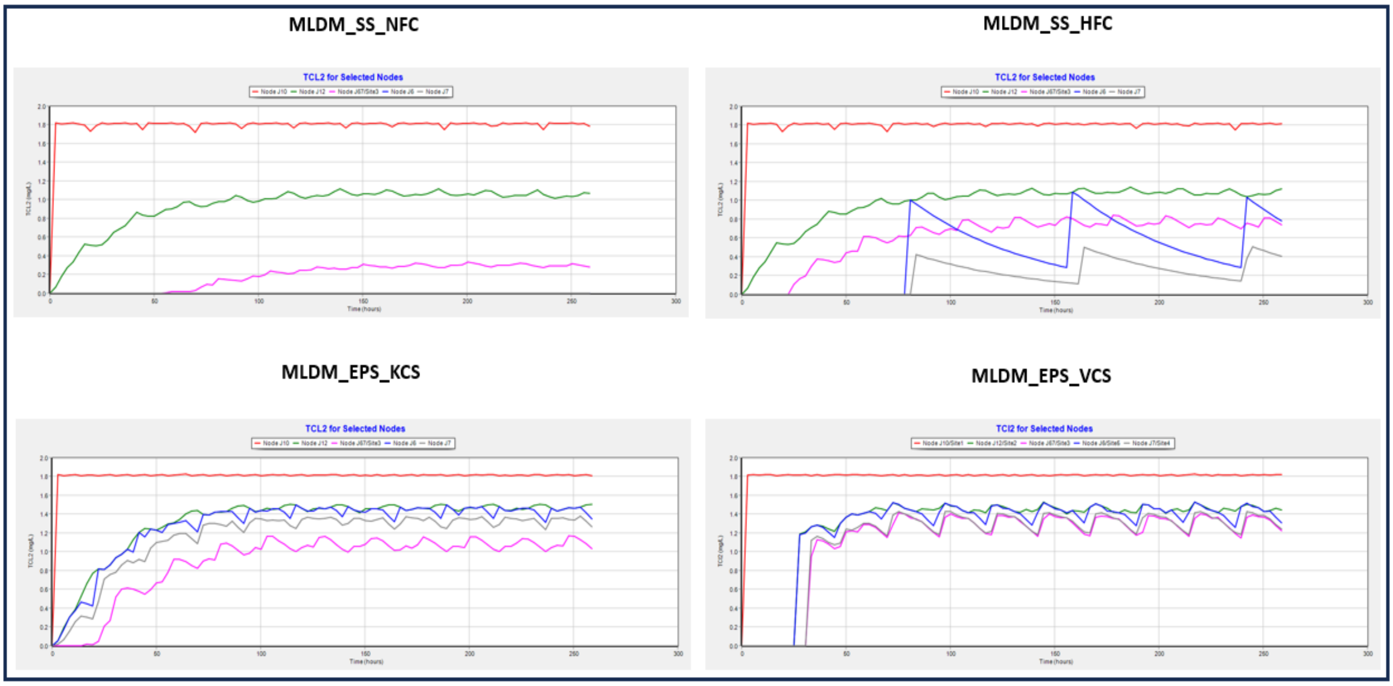


```
Calibration Statistics for Pressure

               Num    Observed    Computed    Mean     RMS
Location       Obs        Mean        Mean   Error   Error
----------------------------------------------------------
J131            25       66.80       69.38   2.988   3.446
J158            25       70.92       69.39   2.170   2.516
J208            25      103.64       95.40   8.245   8.392
J170            25      148.24      149.39   2.168   2.439
J235            25       32.72       34.68   3.964   4.651
J102            25       61.72       64.74   4.253   5.025
J108            25       75.36       71.71   3.648   4.459
J363            25       96.44       88.53   8.380   9.527
J135            25       46.72       47.38   2.073   2.449
J310            25       92.00      100.37   8.369   9.042
----------------------------------------------------------
Network        250       79.46       79.10   4.626   5.830

Correlation Between Means: 0.988
```

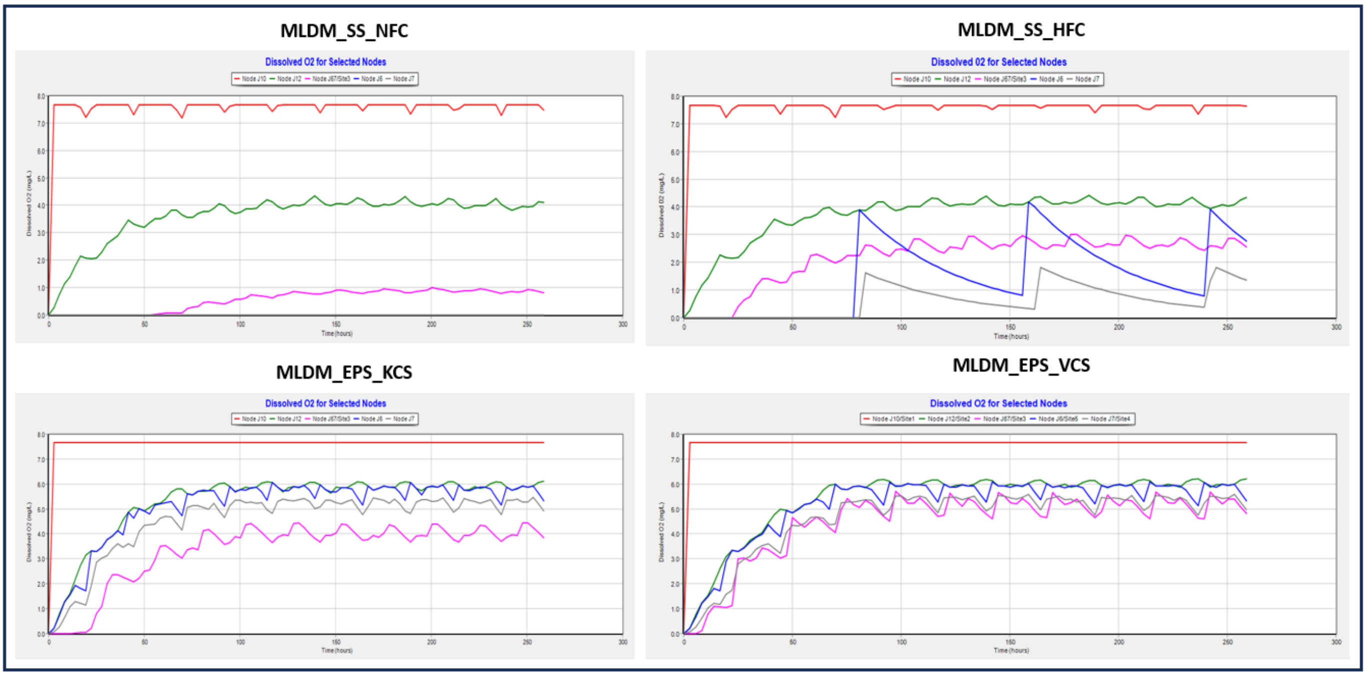
App A3-I: MLDM water quality calibration results (initial)



App A3-J: MLDM water quality calibration results (final)

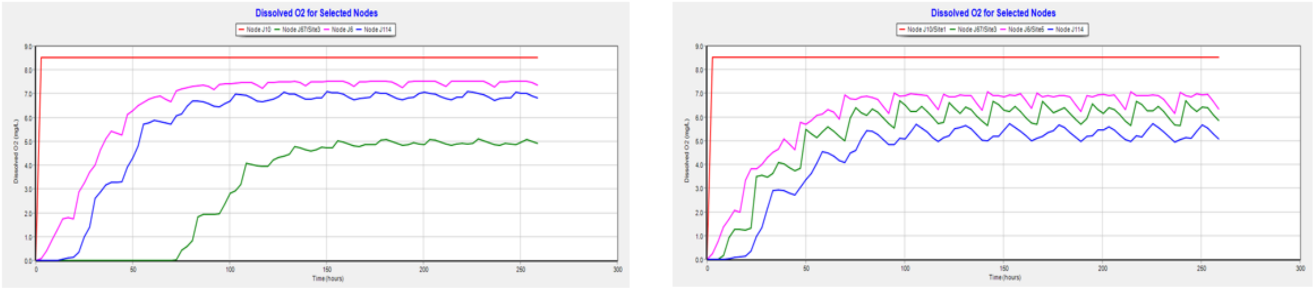


App A3-K: MLDM water quality validation results



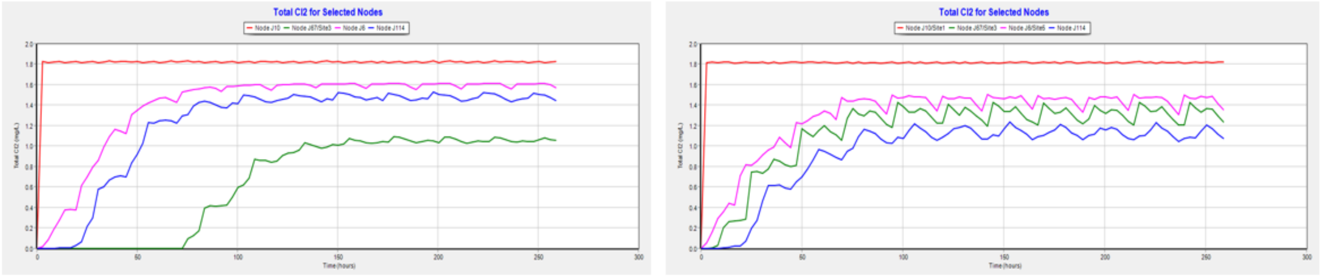
B Water quality and hydraulic definition data

App B-1: LLDS and MLDM dissolved oxygen and total chlorine analysis results



LLDS_DO ANALYSIS

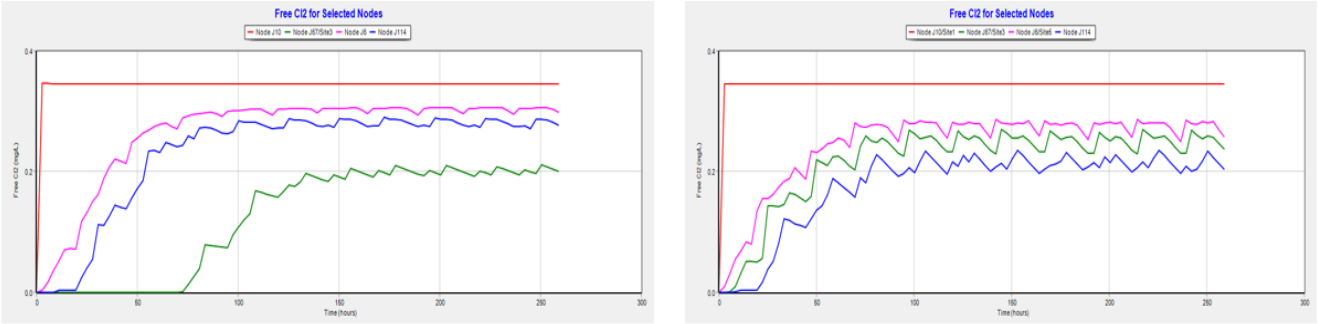
MLDM_DO ANALYSIS



LLDS TOTAL CHLORINE ANALYSIS

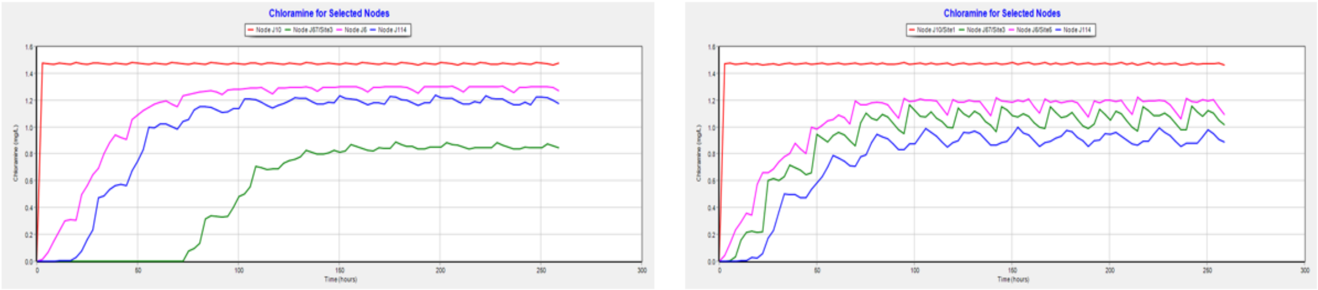
MLDM TOTAL CHLORINE ANALYSIS

App B-2: LLDS and MLDM free chlorine and chloramine analysis results



LLDS_FREE CHLORINE ANALYSIS

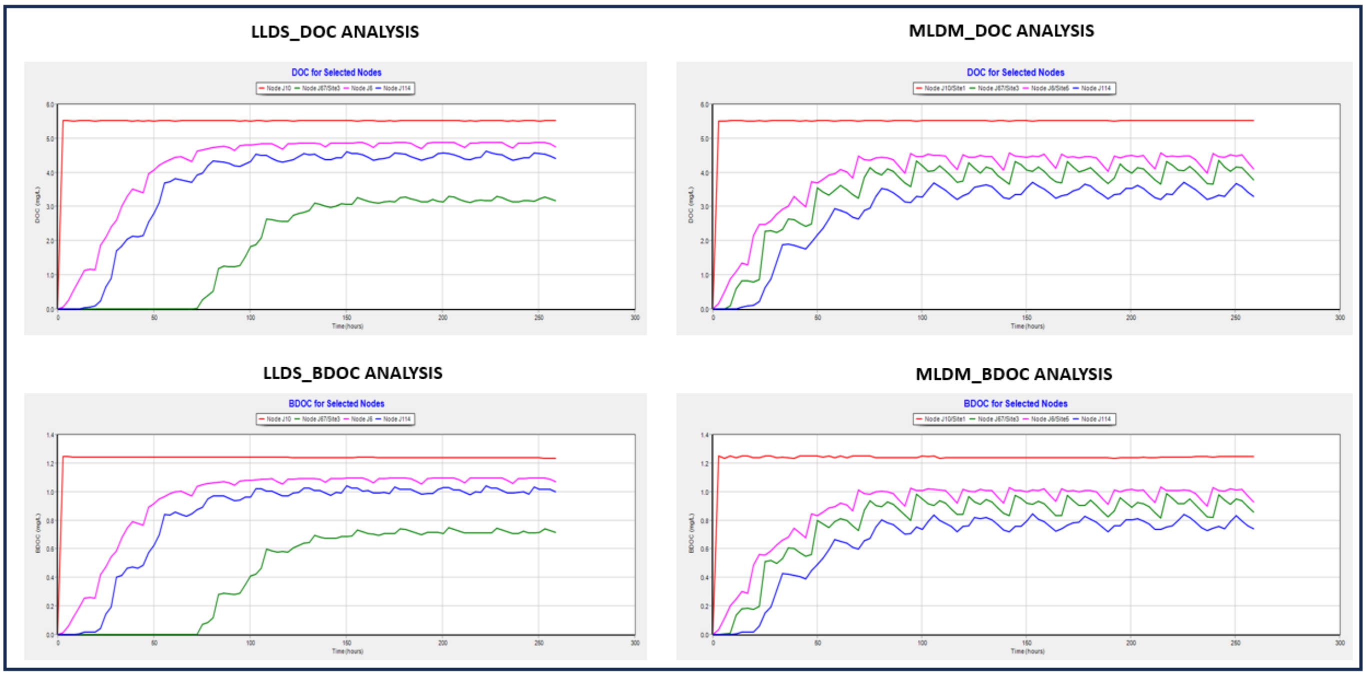
MLDM_FREE CHLORINE ANALYSIS



LLDS_CHLORAMINE ANALYSIS

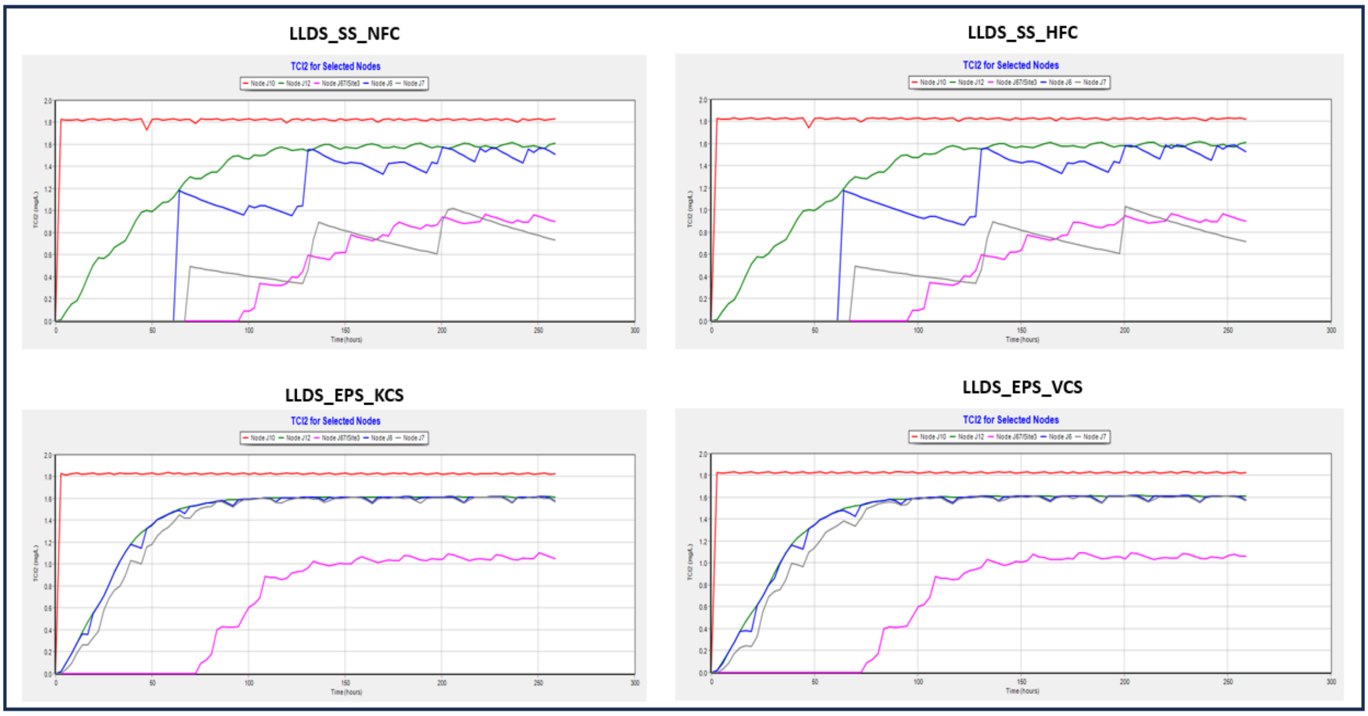
MLDM_CHLORAMINE ANALYSIS

App B-3: LLDS and MLDM DOC and BDOC analysis results

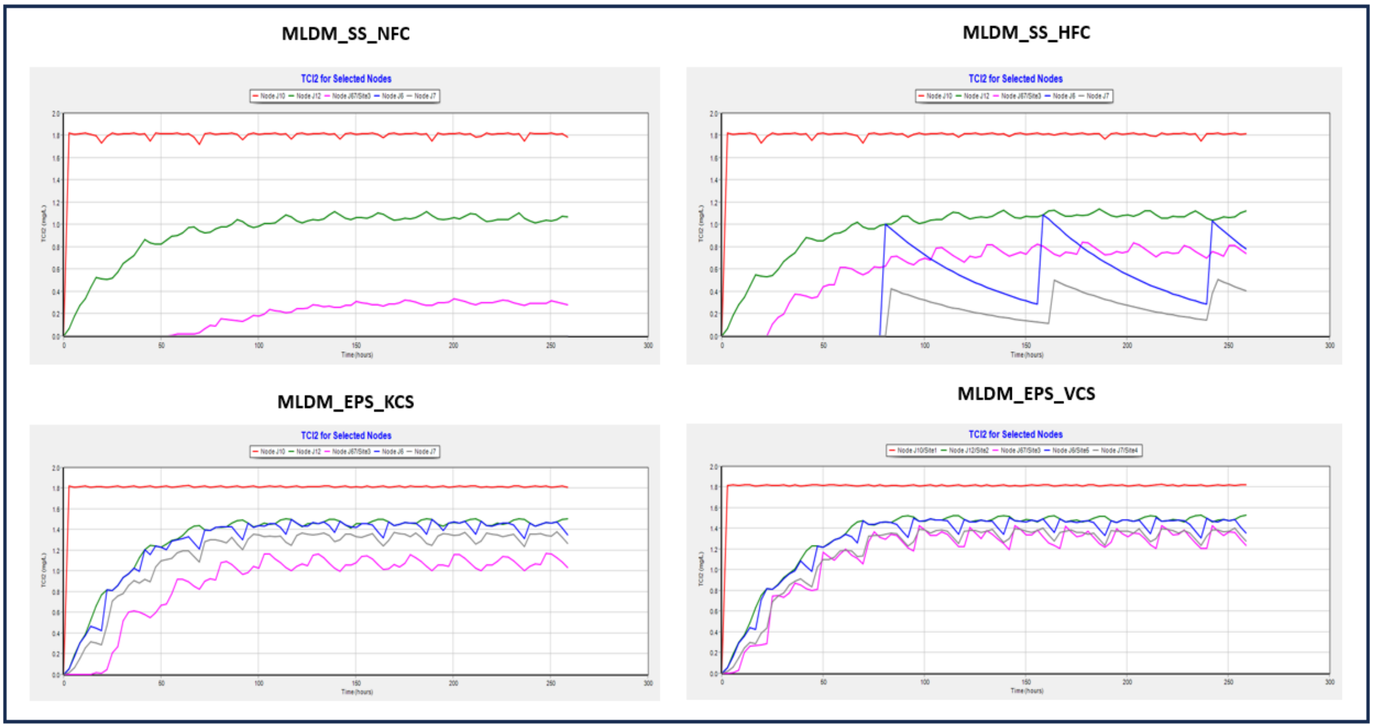


C Water quality and level of calibration data

App C-1: LLDS total chlorine analysis results at different levels of calibration

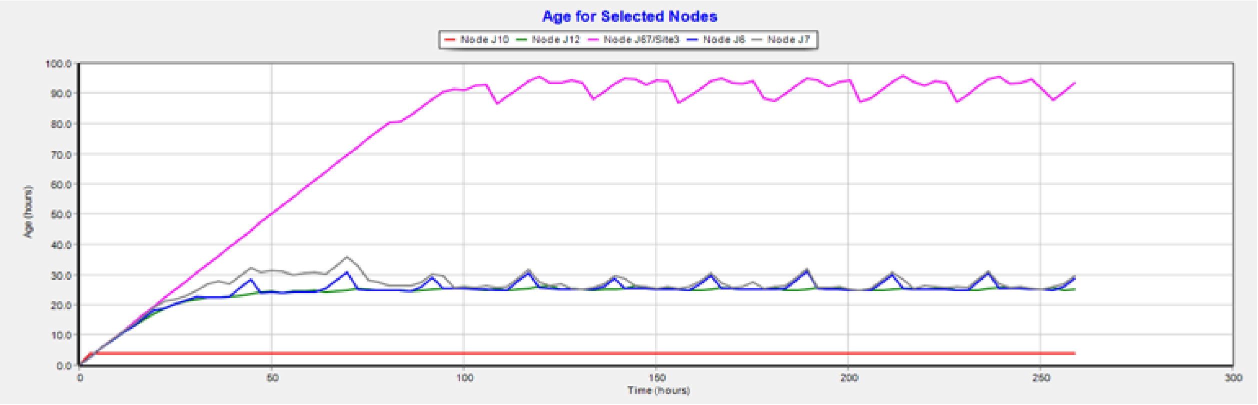


App C-2: MLDM total chlorine analysis results at different levels of calibration

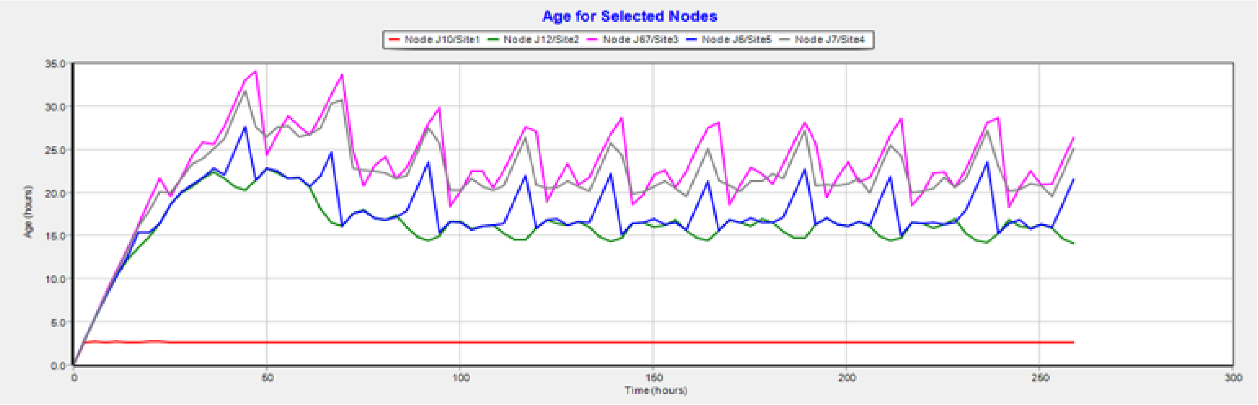


App C-3: LLDS and MLDM EPS VCS water age analysis results

LLDS_EPS_VCS



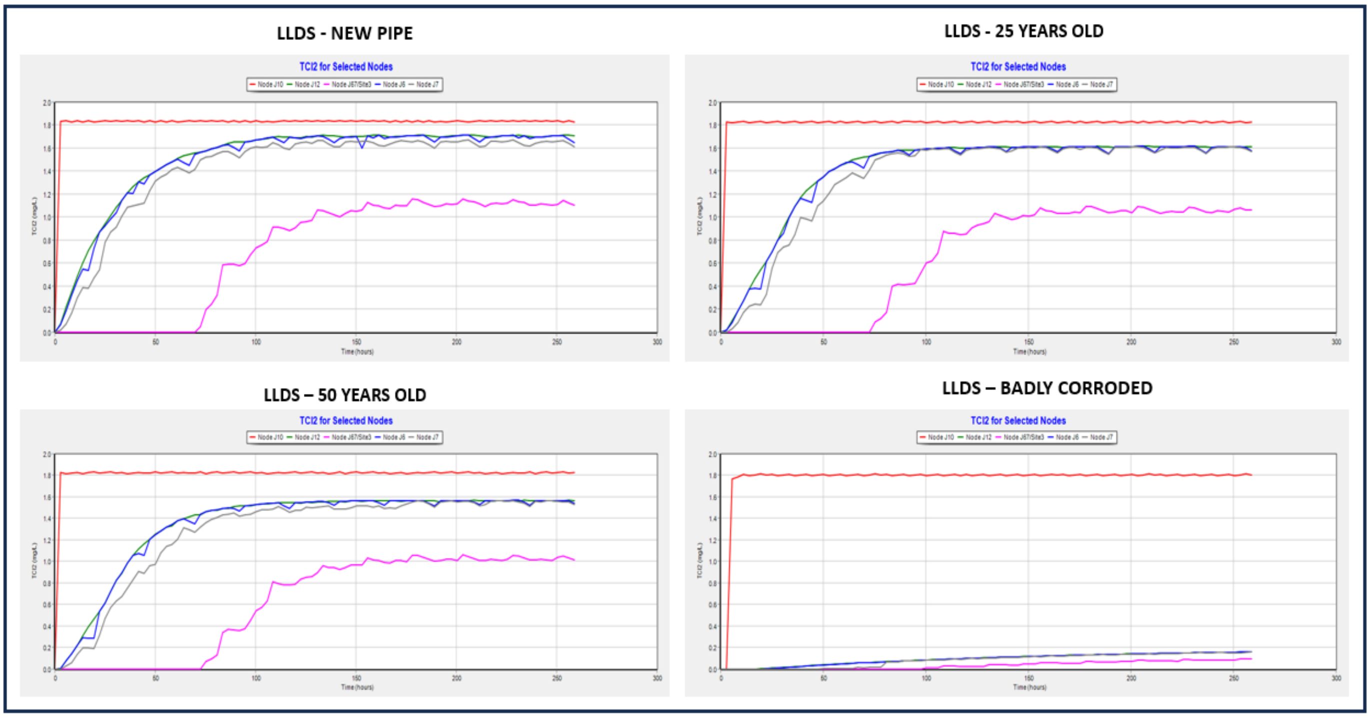
MLDM_EPS_VCS



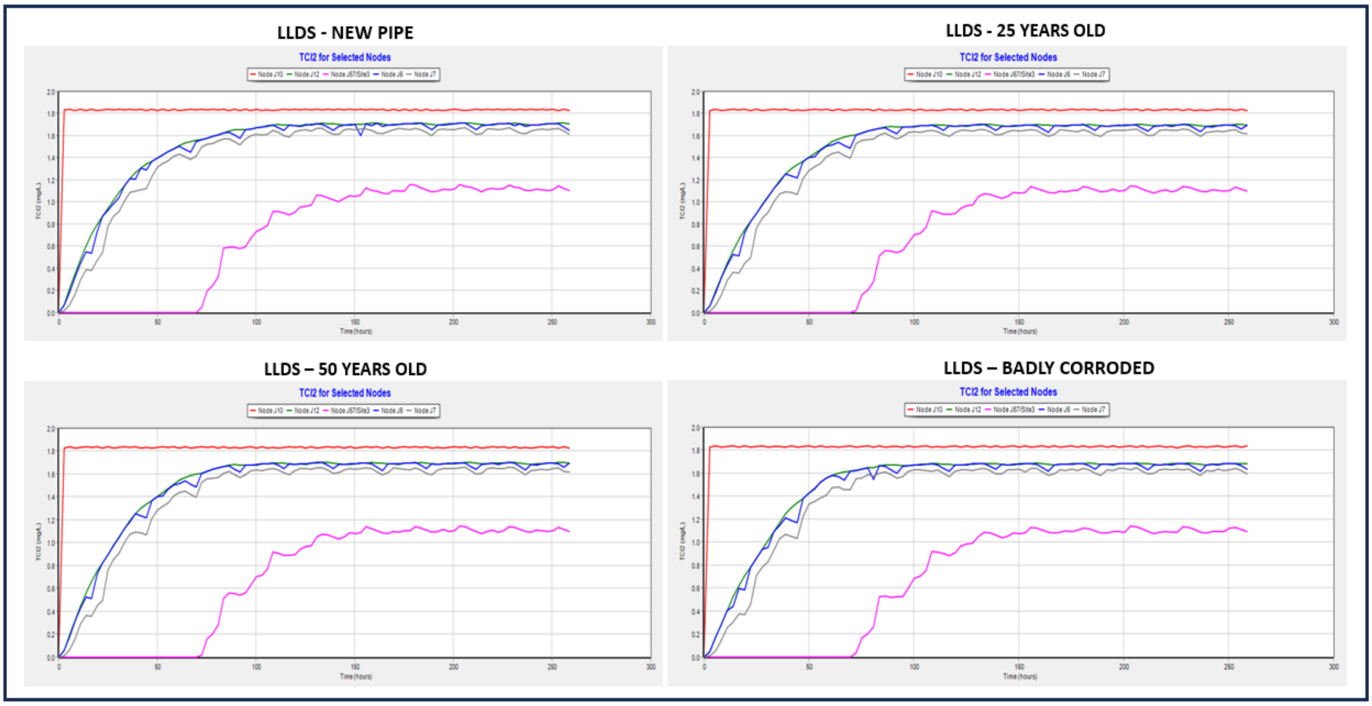
D Water quality and pipe age and material data

D.1 LLDS pipe age and material analysis model data

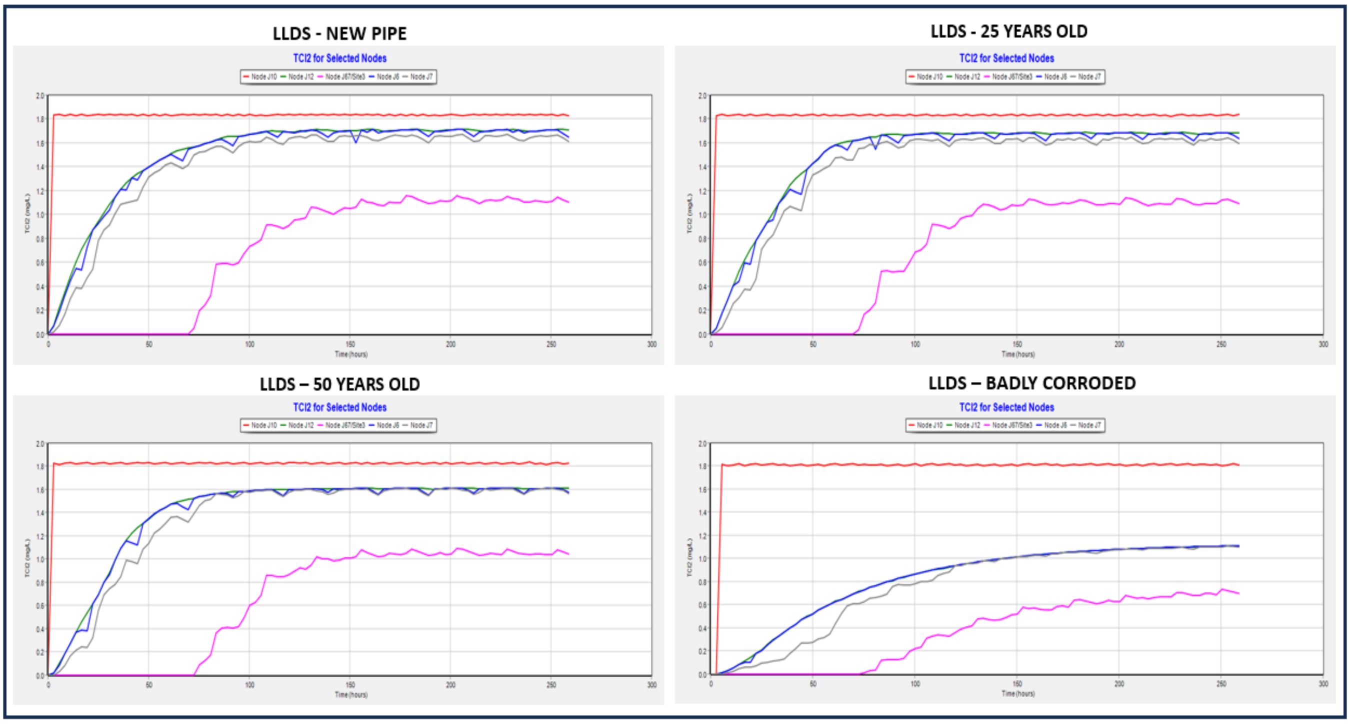
App D1-A: Iron pipes total chlorine analysis results



App D1-B: Plastic pipes total chlorine analysis results

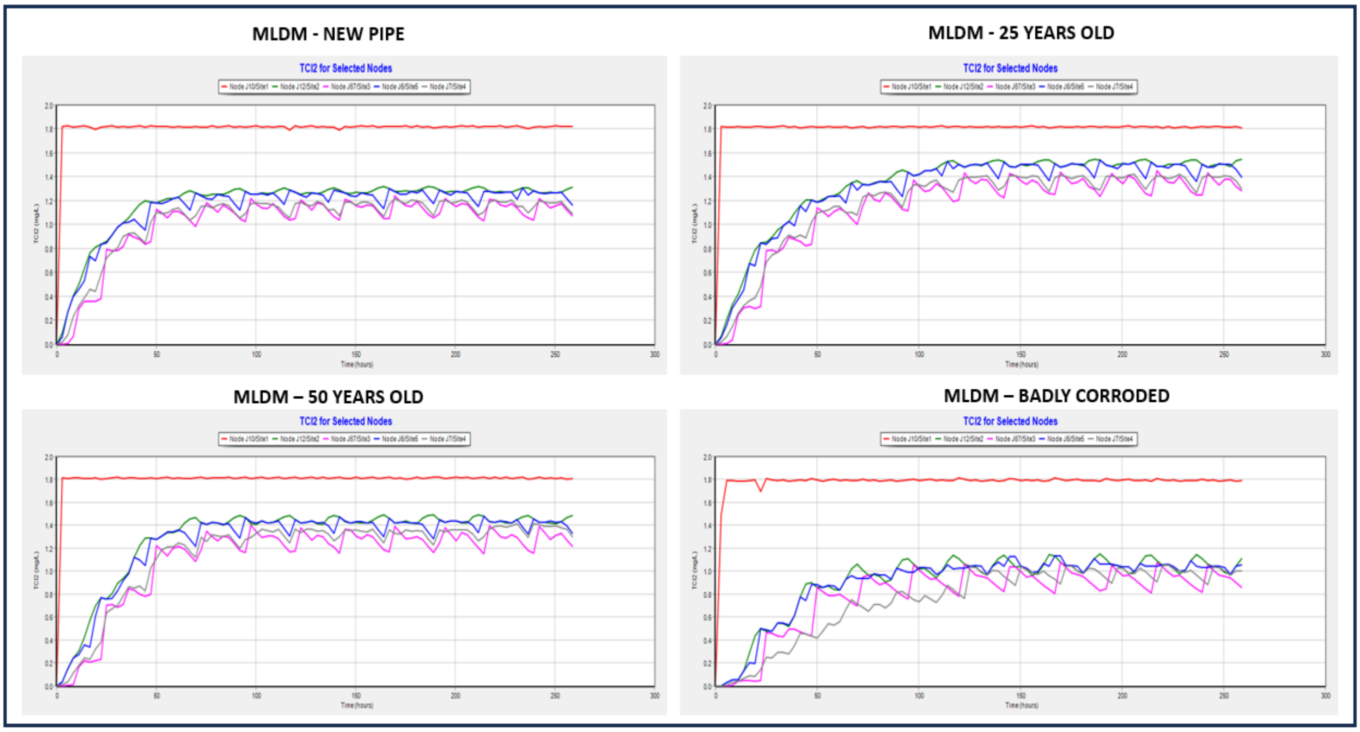


App D1-C: Steel pipes total chlorine analysis results



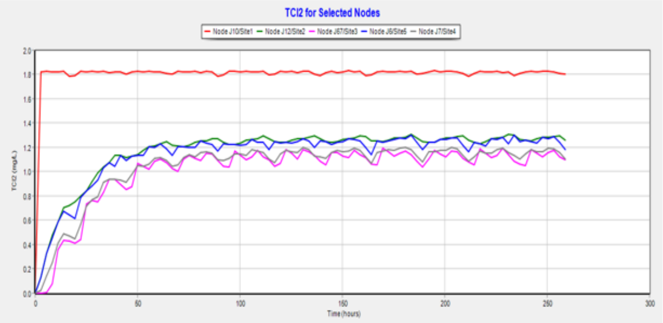
D.2 MLDM pipe age and material analysis model data

App D2-A: Iron pipes total chlorine analysis results

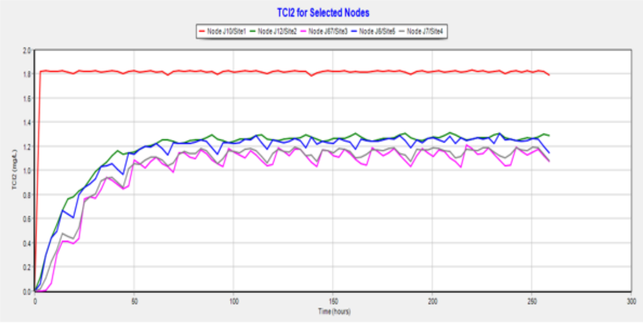


App D2-B: Plastic pipes total chlorine analysis results

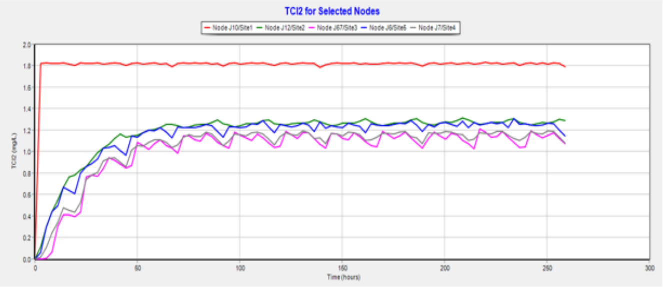
MLDM - NEW PIPE



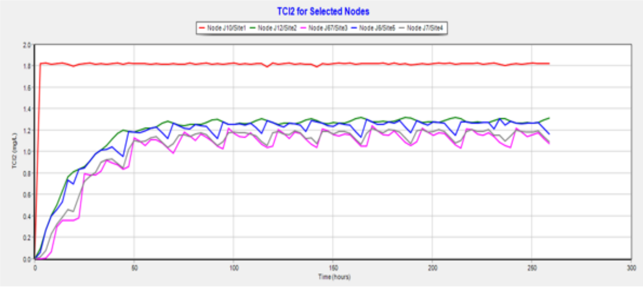
MLDM - 25 YEARS OLD



MLDM – 50 YEARS OLD

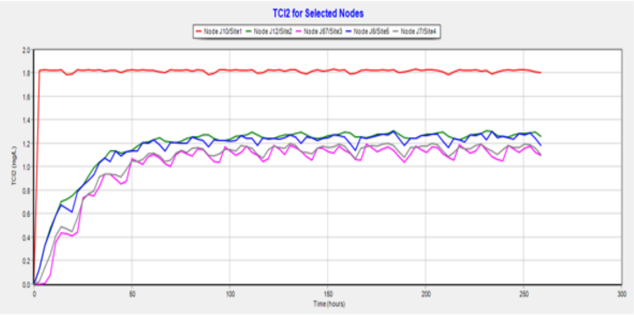


MLDM – BADLY CORRODED

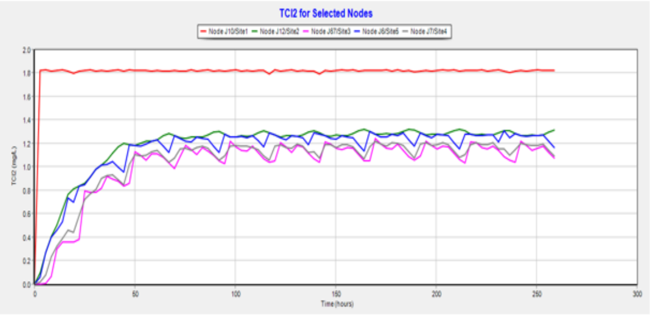


App D2-C: Steel pipes total chlorine analysis results

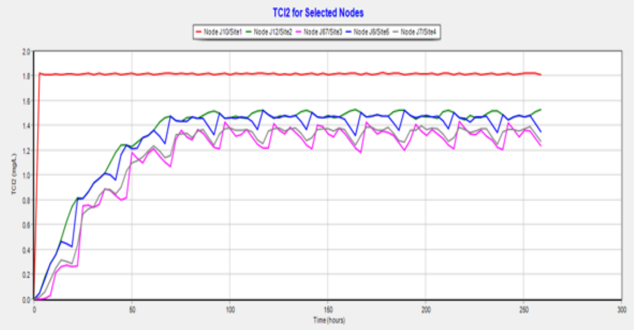
MLDM - NEW PIPE



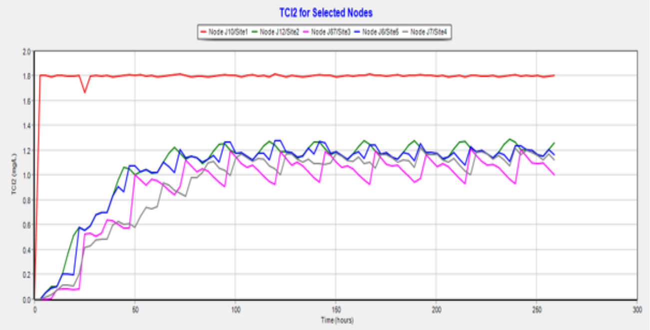
MLDM - 25 YEARS OLD



MLDM – 50 YEARS OLD



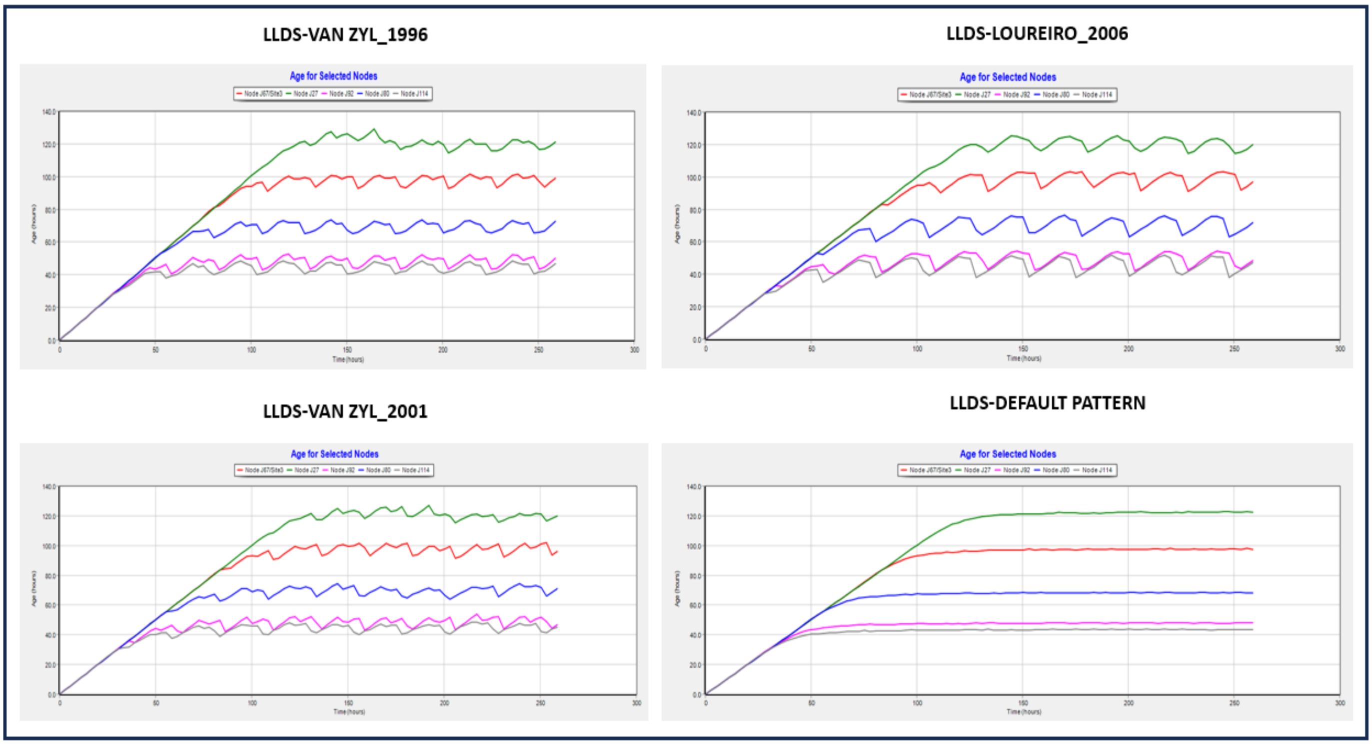
MLDM – BADLY CORRODED



E Water quality and water demand data

E.1 LLDS water demand pattern analysis model data

App E1-A: LLDS water age analysis results for different pattern



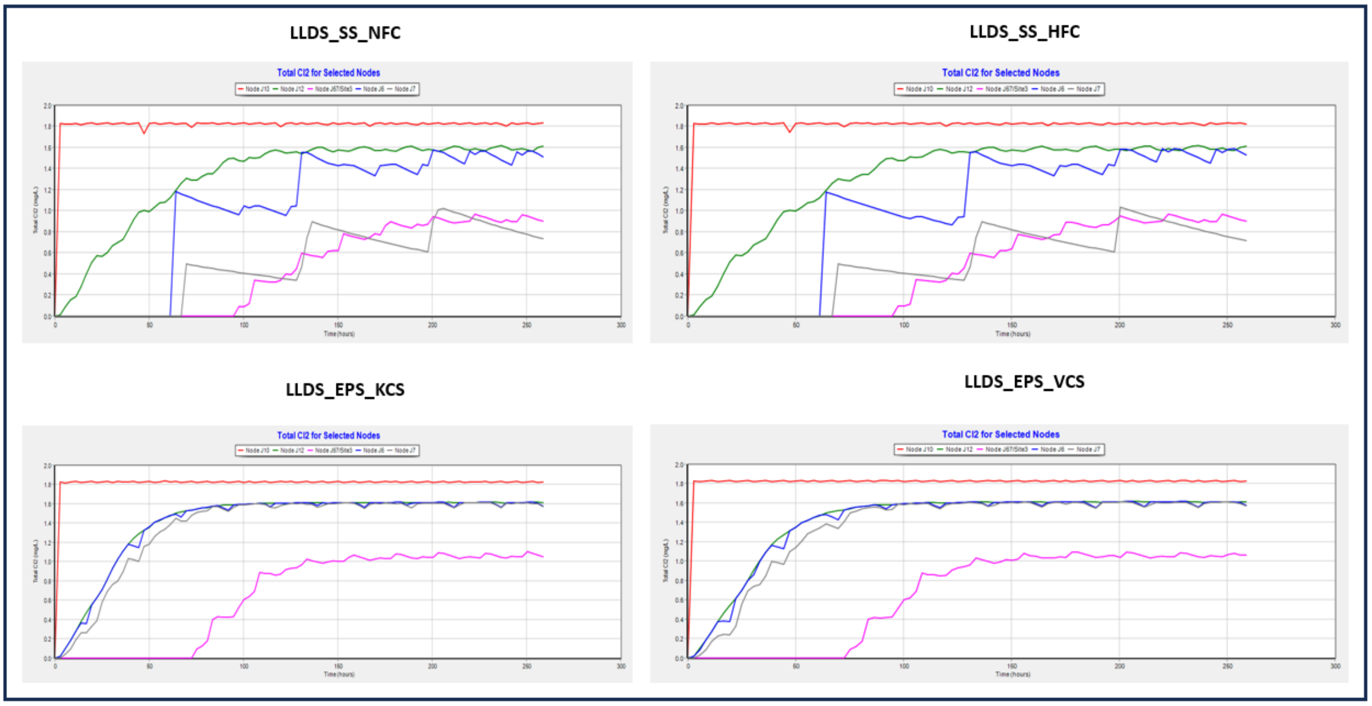
LLDS-VAN ZYL_1996

LLDS-LOUREIRO_2006

LLDS-VAN ZYL_2001

LLDS-DEFAULT PATTERN

App E1-B: LLDS total chlorine and pattern 1(Van Zyl_1996) analysis results



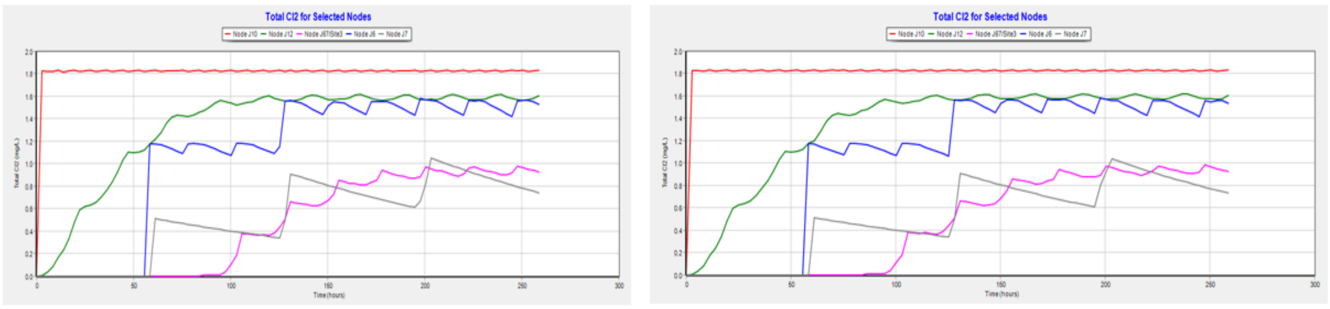
LLDS_SS_NFC

LLDS_SS_HFC

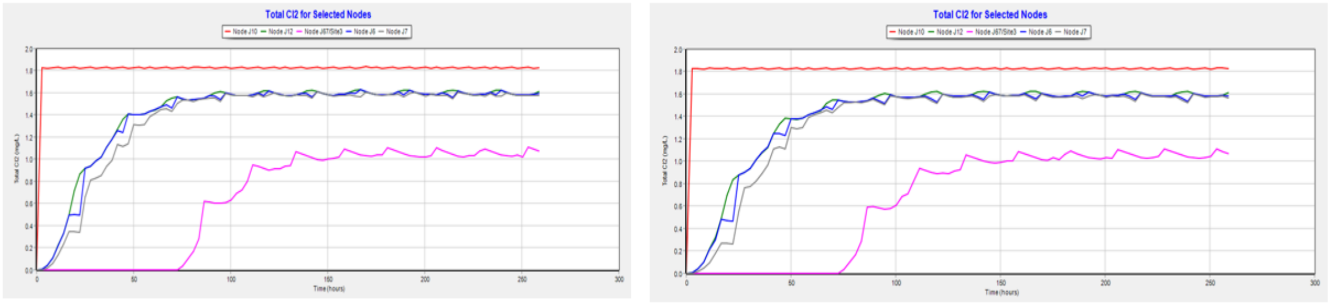
LLDS_EPS_KCS

LLDS_EPS_VCS

App E1-C: LLDS total chlorine and pattern 2 (Loureiro_2006) analysis results

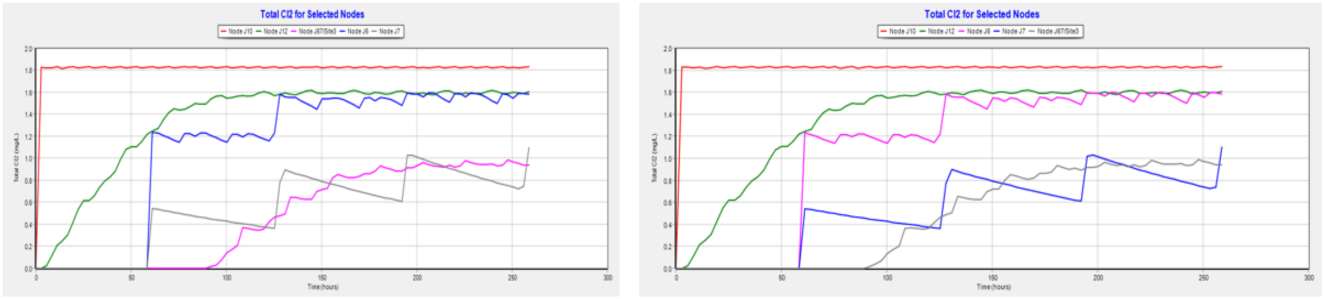

LLDS_SS_NFC

LLDS_SS_HFC

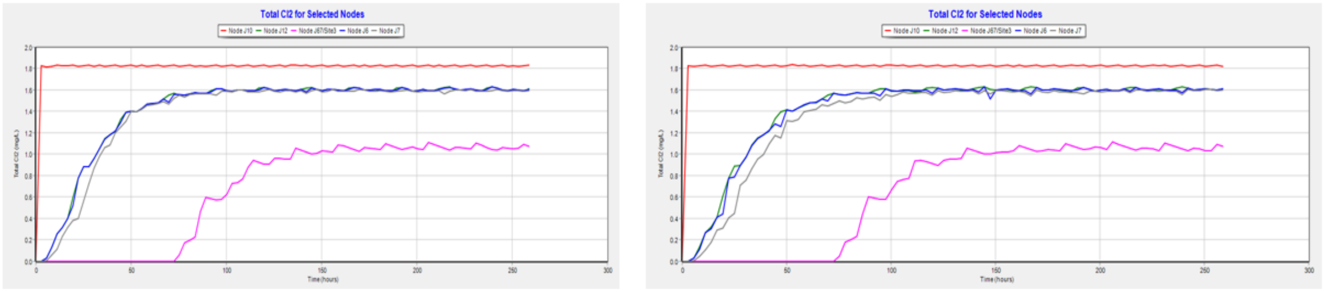

LLDS_EPS_KCS

LLDS_EPS_VCS

App E1-D: LLDS total chlorine and pattern 3 (Van Zyl_2001) analysis results

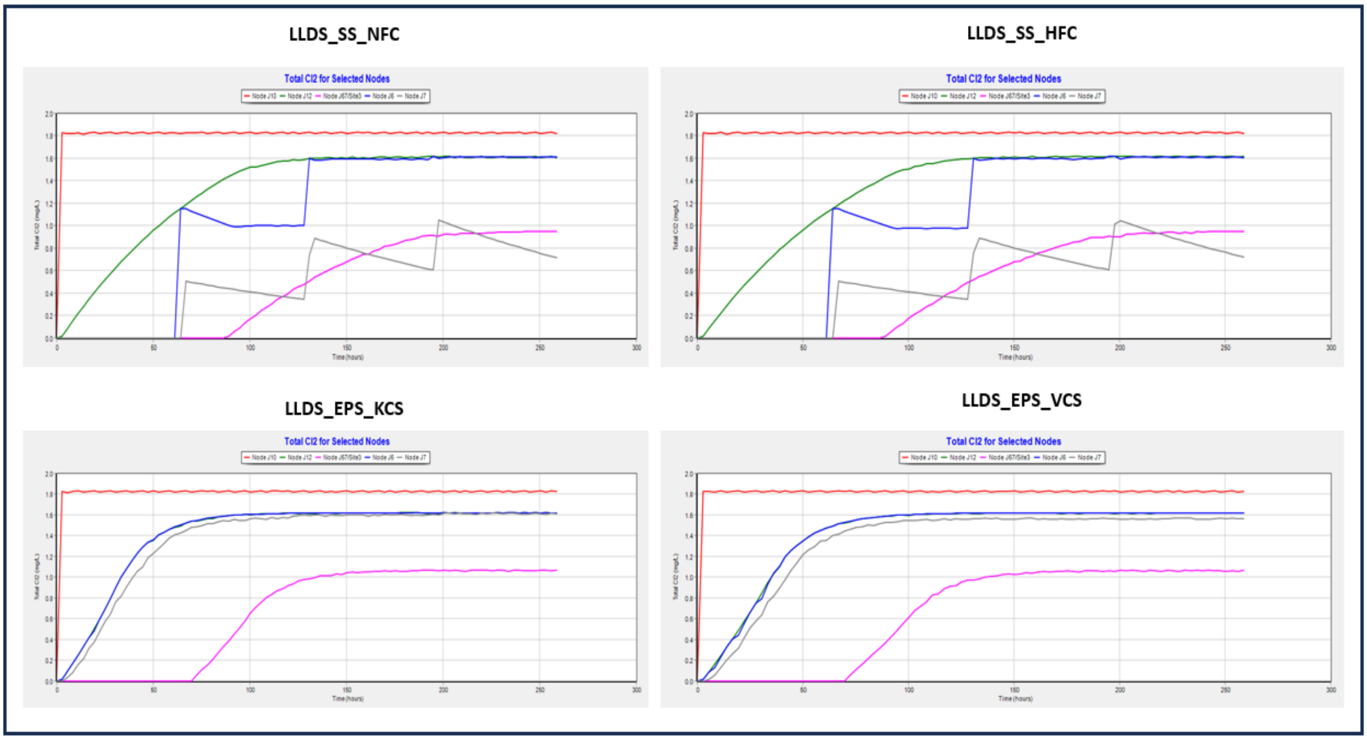

LLDS_SS_NFC

LLDS_SS_HFC


LLDS_EPS_KCS

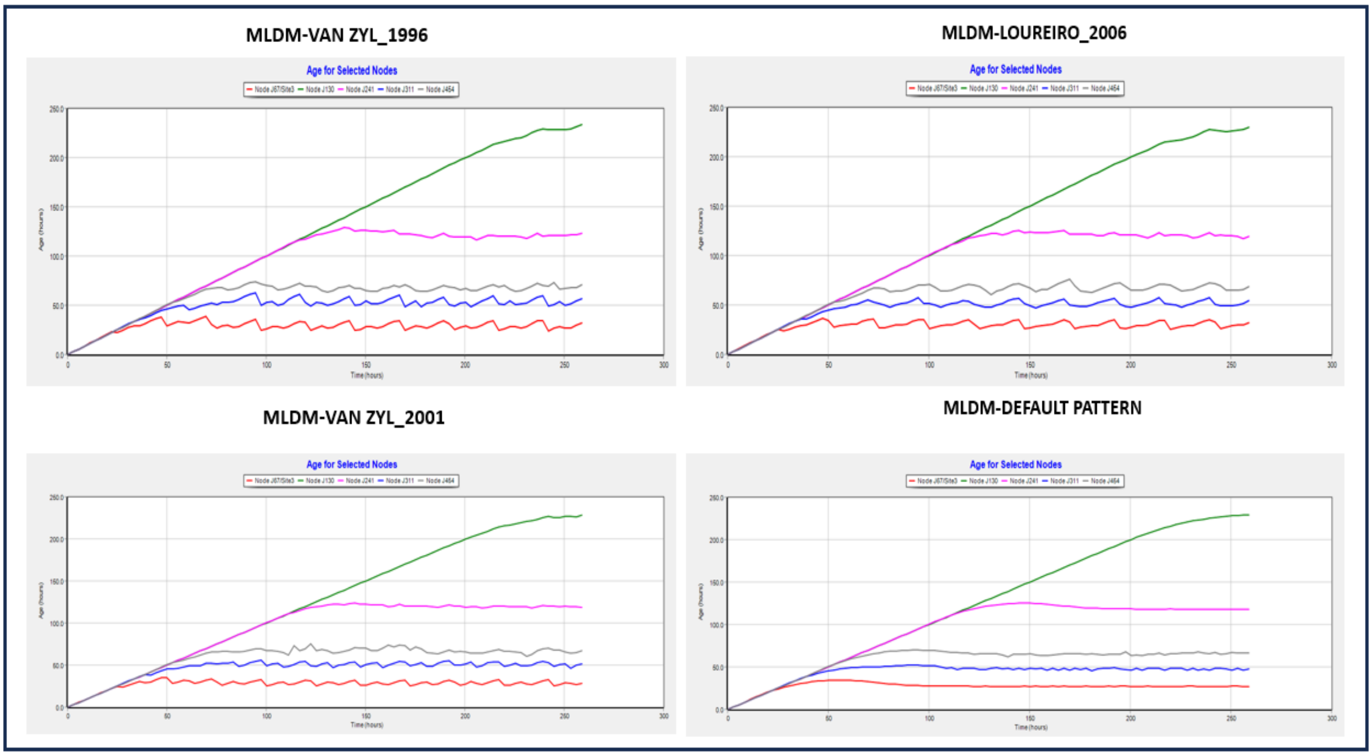
LLDS_EPS_VCS

App E1-E: LLDS total chlorine and pattern 4 (Default Pattern) analysis results

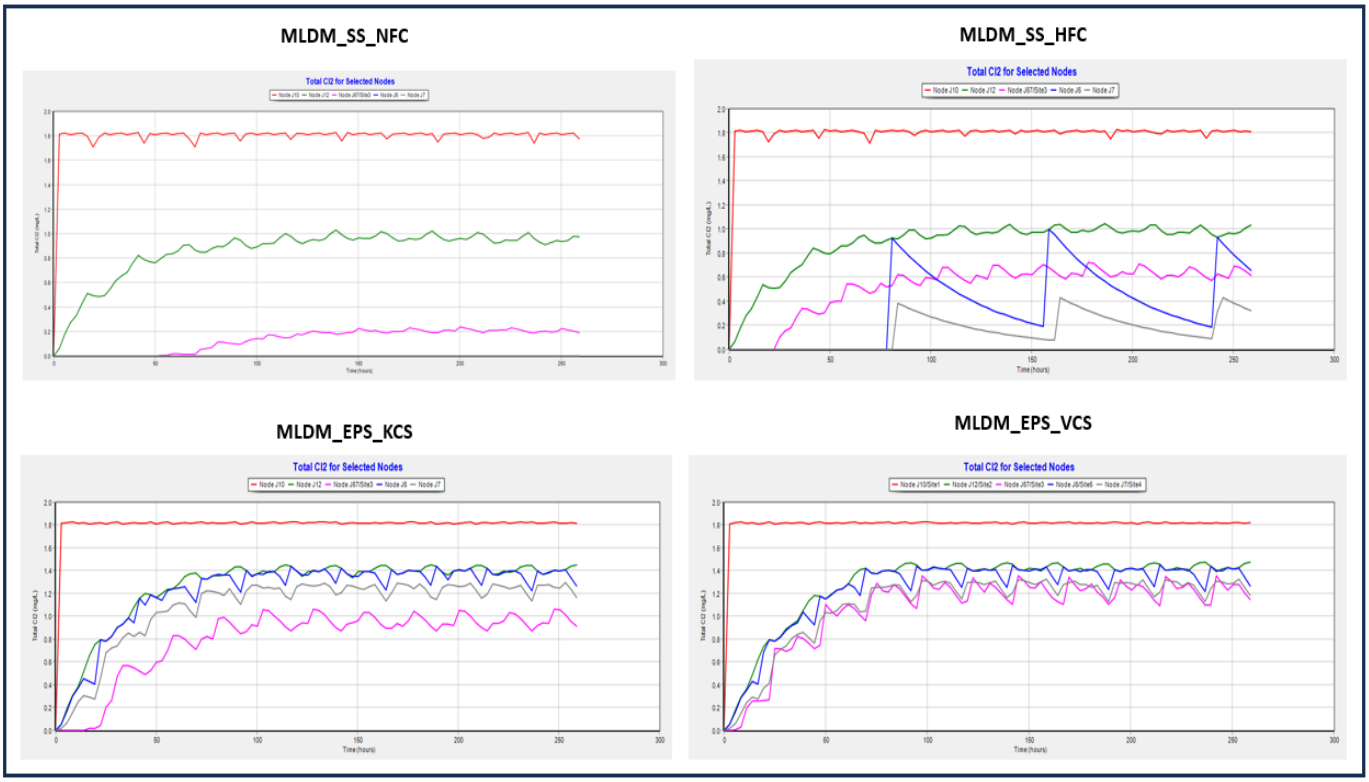


E.2 MLDM water demand pattern analysis model data

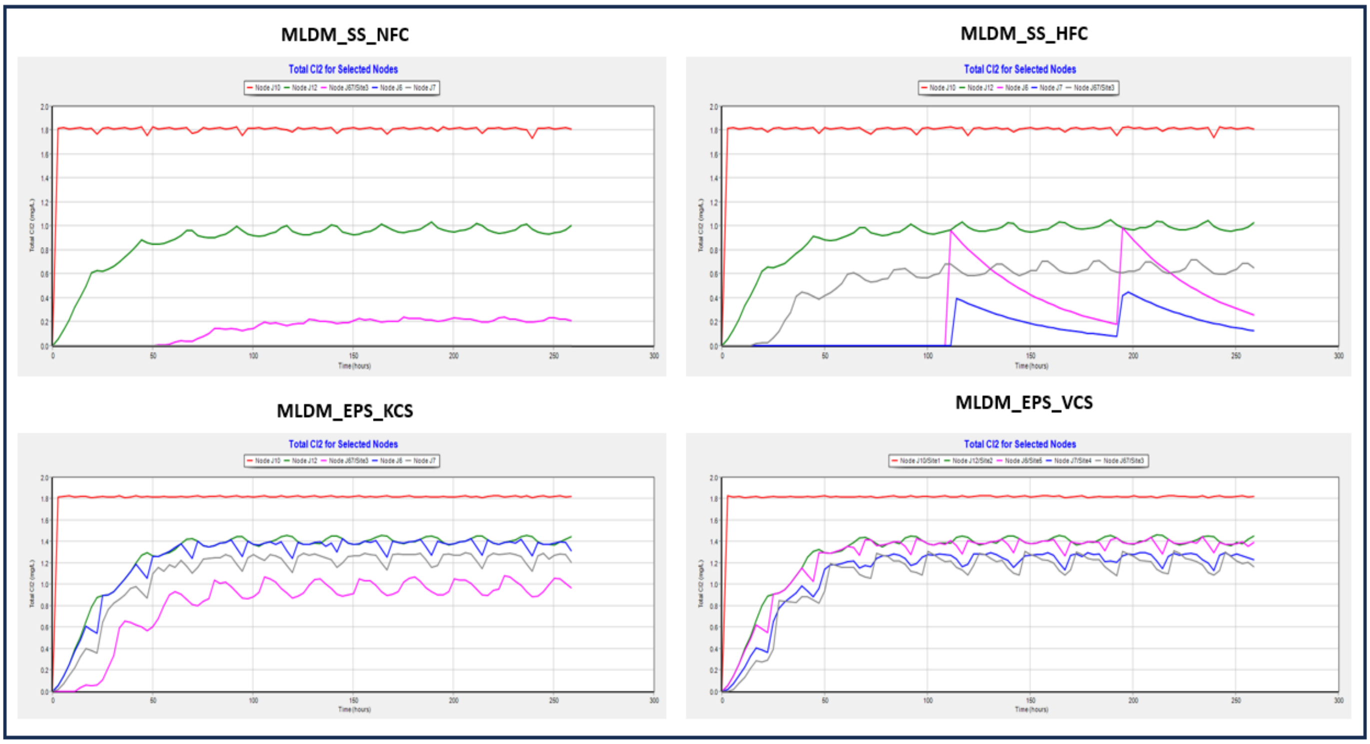
App E2-A: MLDM water age analysis results for different patterns



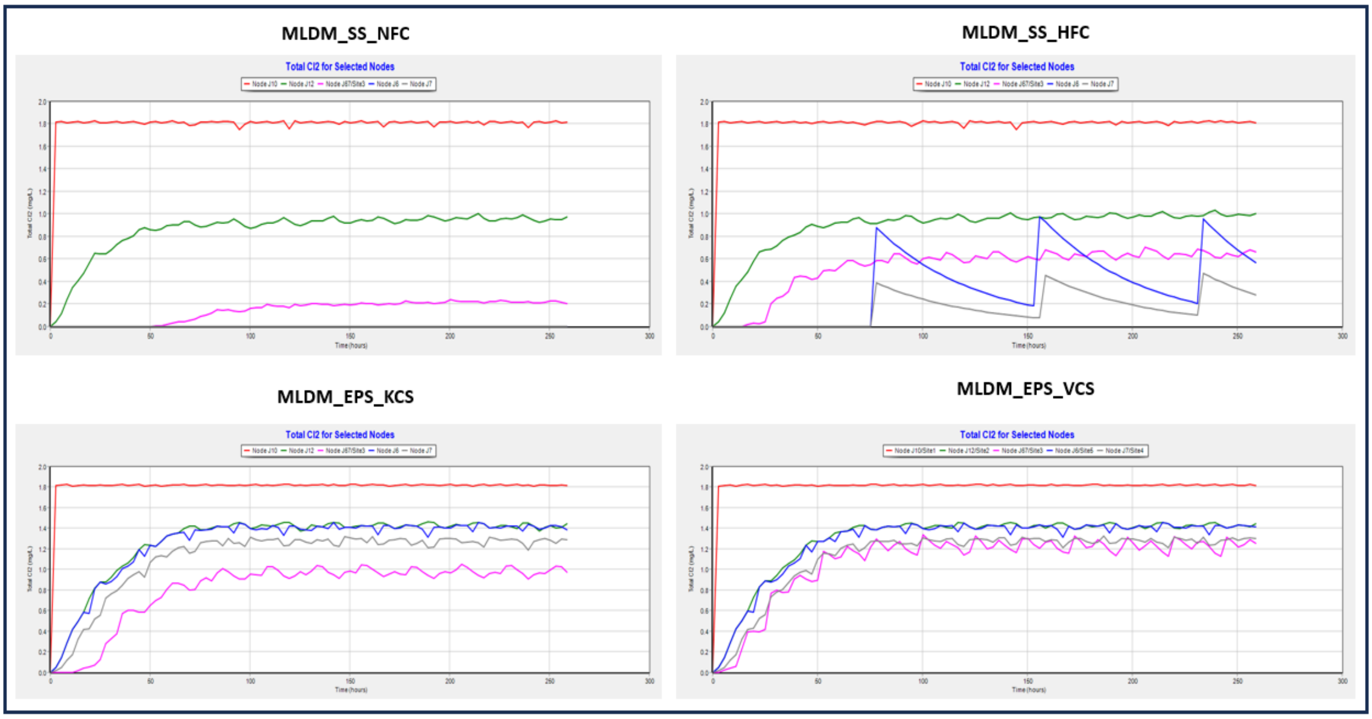
App E2-B: MLDM total chlorine and pattern 1 (Van Zyl_1996) analysis results



App E2-C: MLDM total chlorine and pattern 2 (Loureiro_2006) analysis results



App E2-D: MLDM total chlorine and pattern 3 (Van Zyl_2001) analysis results



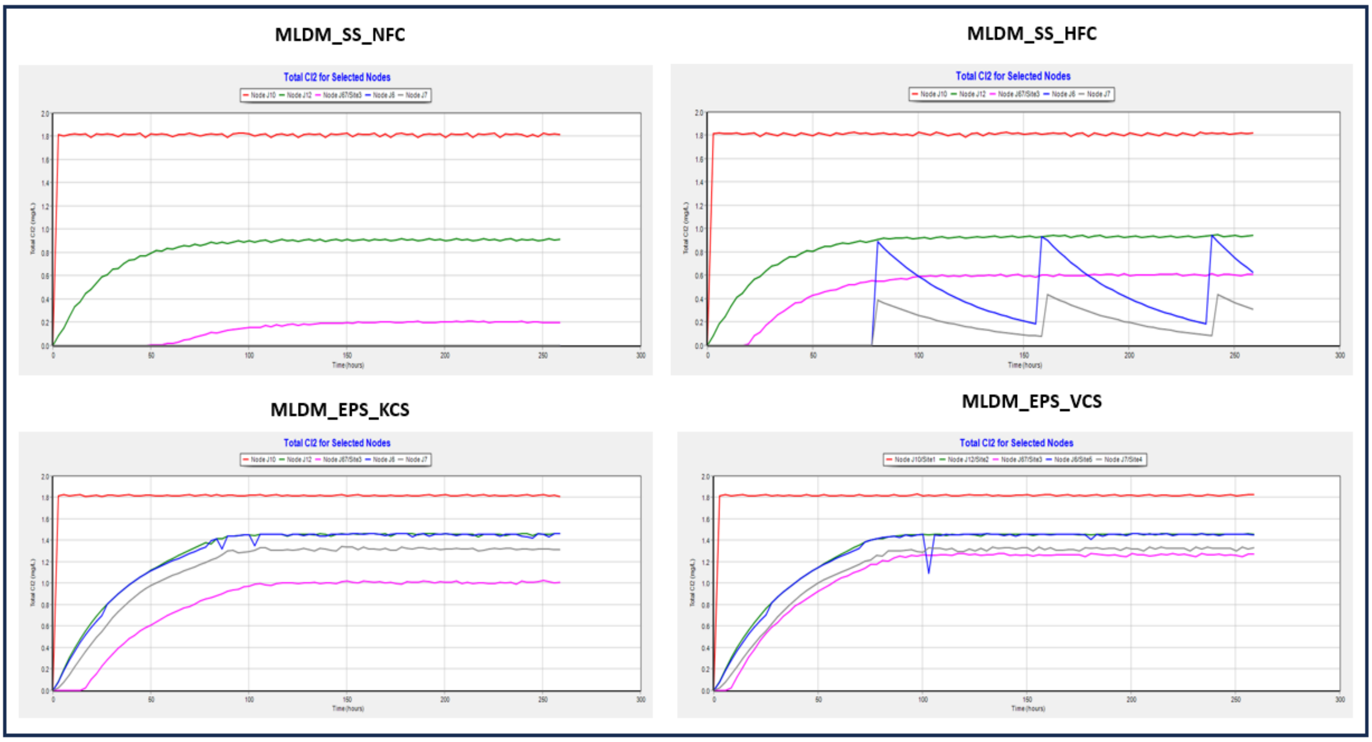
MLDM_SS_NFC

MLDM_SS_HFC

MLDM_EPS_KCS

MLDM_EPS_VCS

App E2-E: MLDM total chlorine and pattern 4 (Default Pattern) analysis results



MLDM_SS_NFC

MLDM_SS_HFC

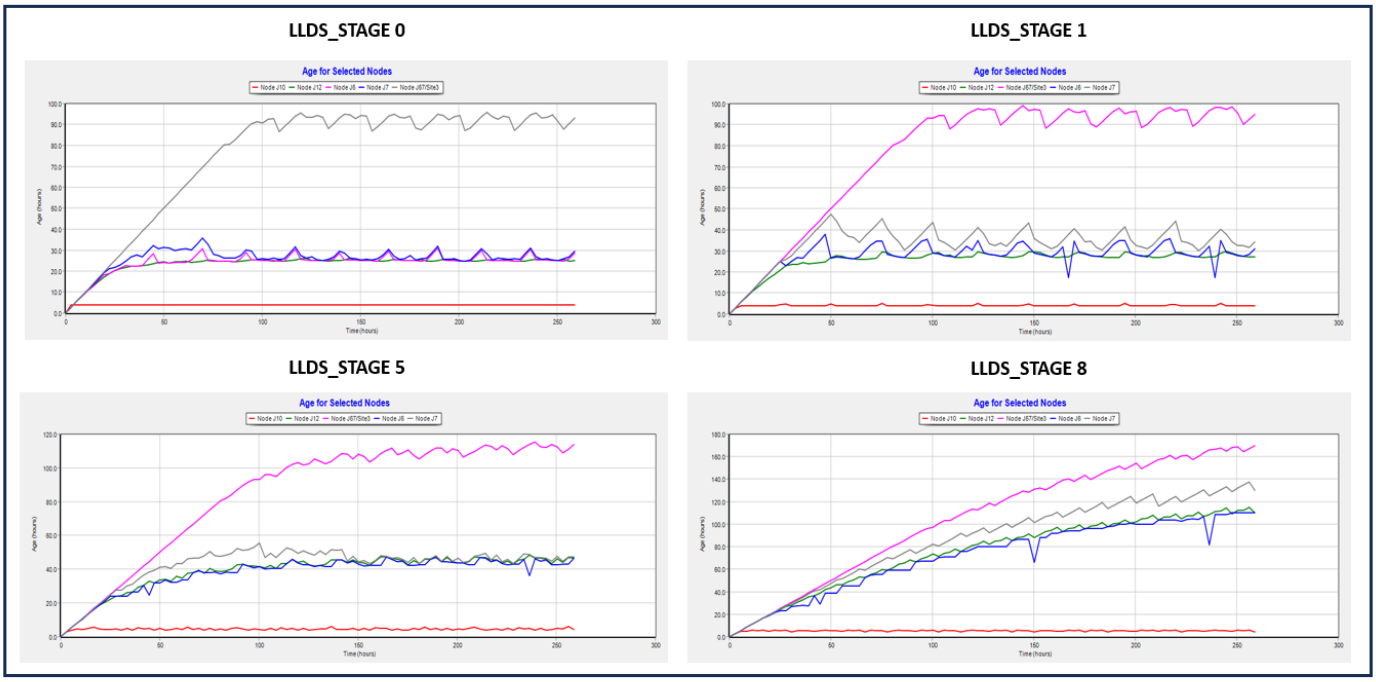
MLDM_EPS_KCS

MLDM_EPS_VCS

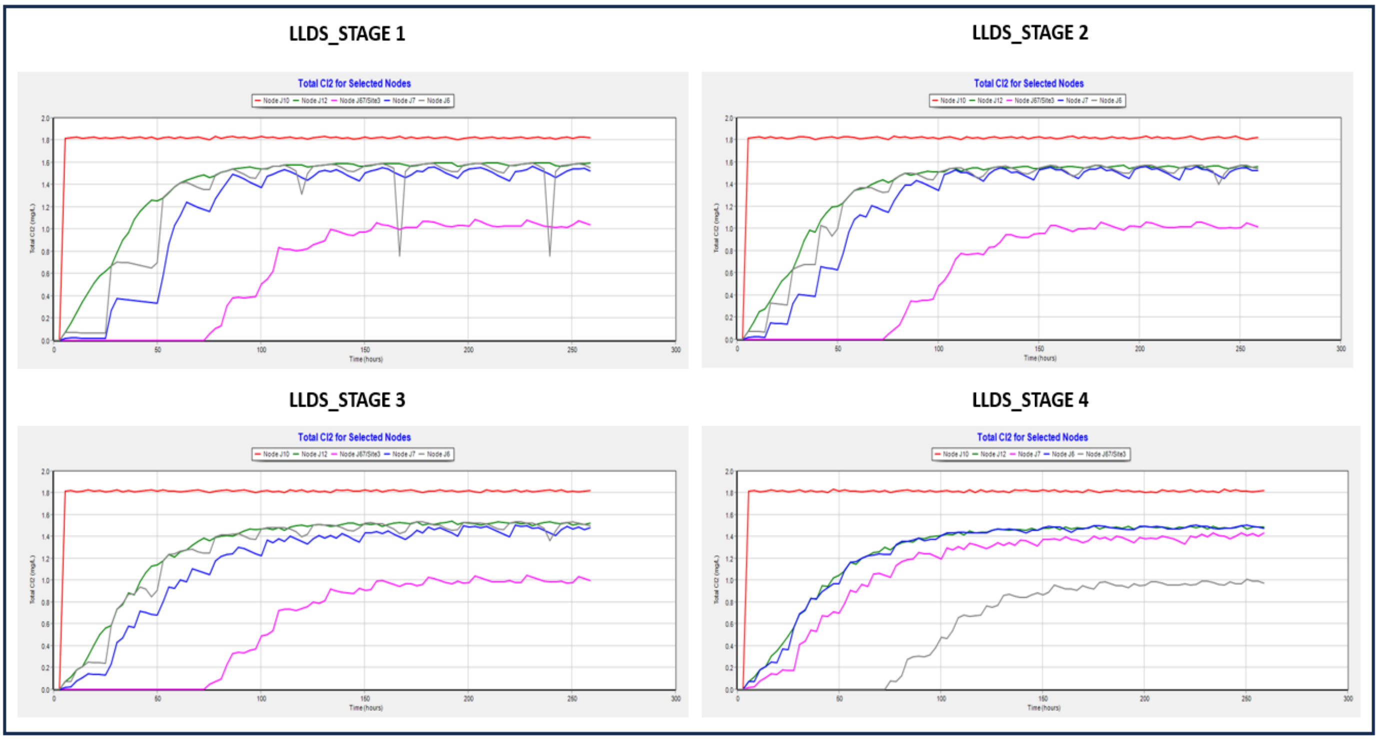
F Water quality and load-shedding data

F.1 LLDS load-shedding analysis model data

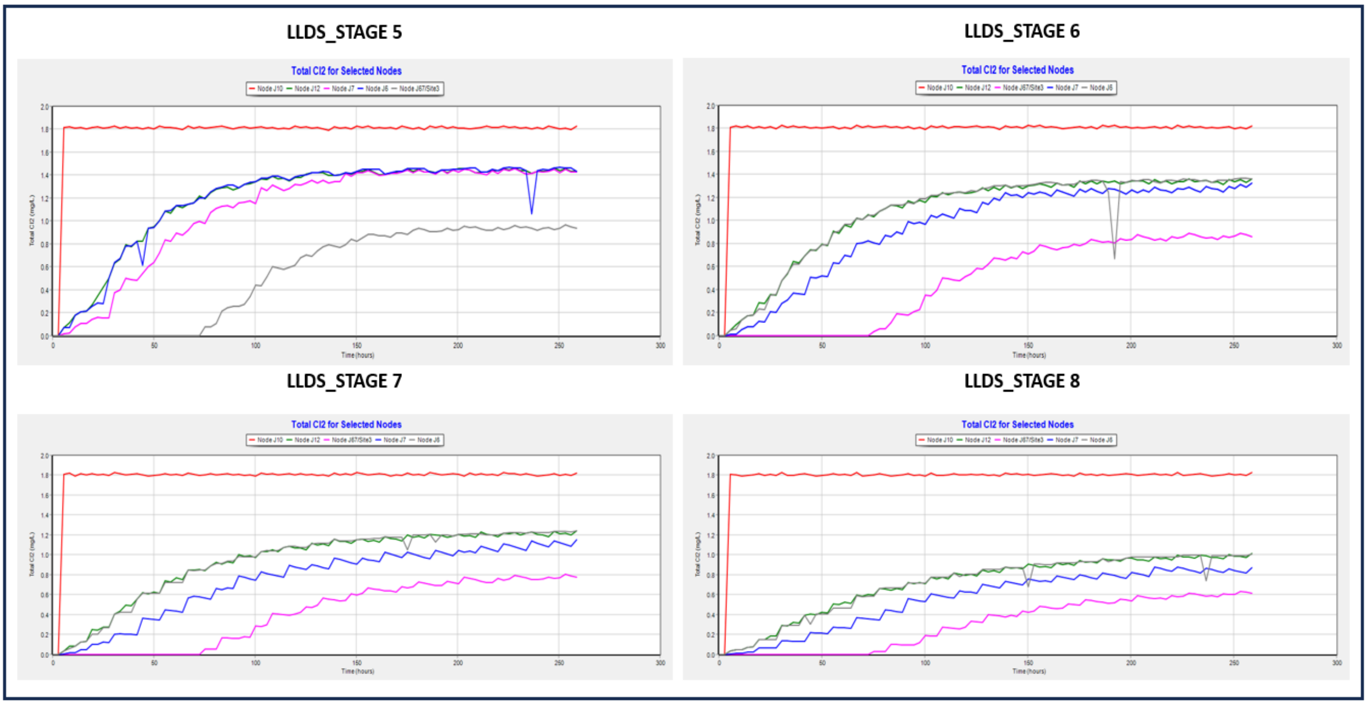
App F1-A: LLDS load-shedding impact on water age



App F1-B: LLDS load-shedding (stage 1 to 4) impact on total chlorine

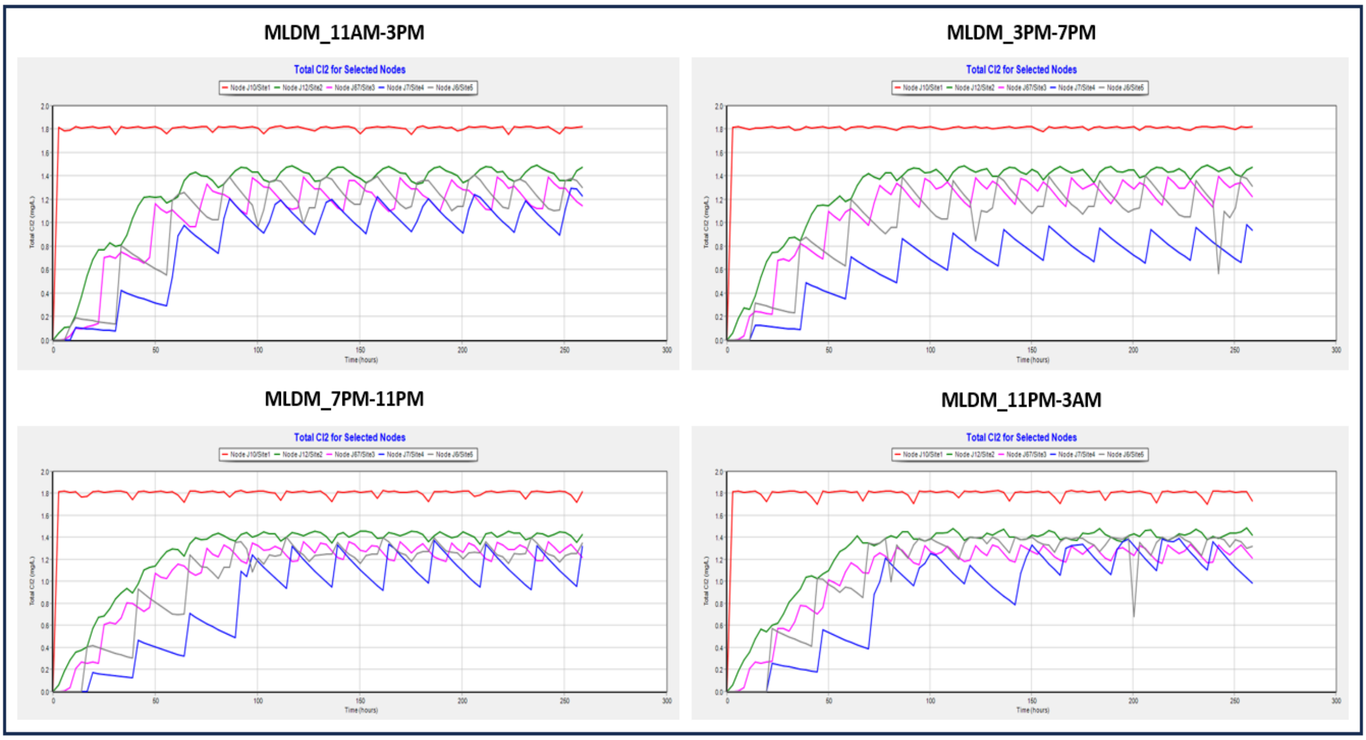


App F1-C: LLDS load-shedding (stage 5 to 8) impact on total chlorine

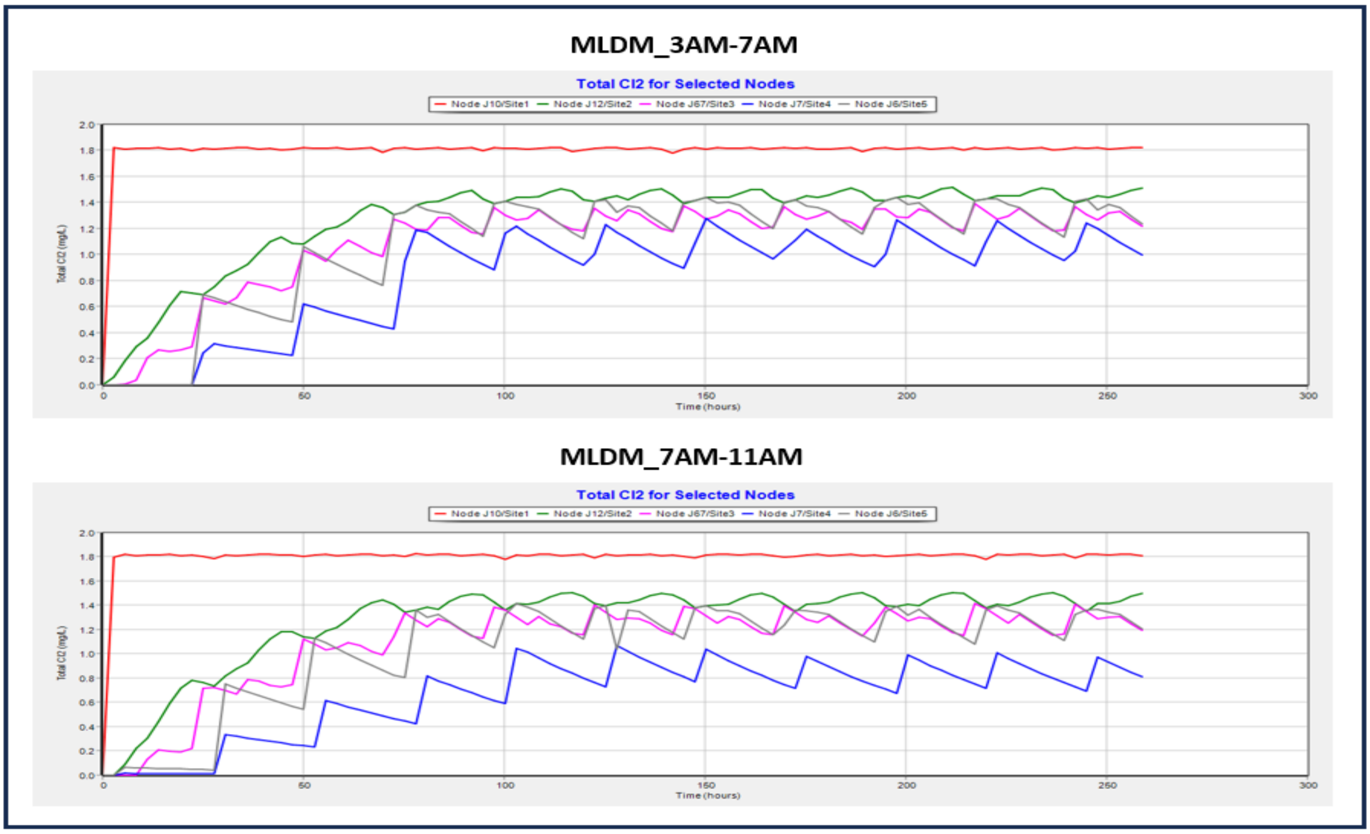


F.2 MLDM load-shedding analysis model data

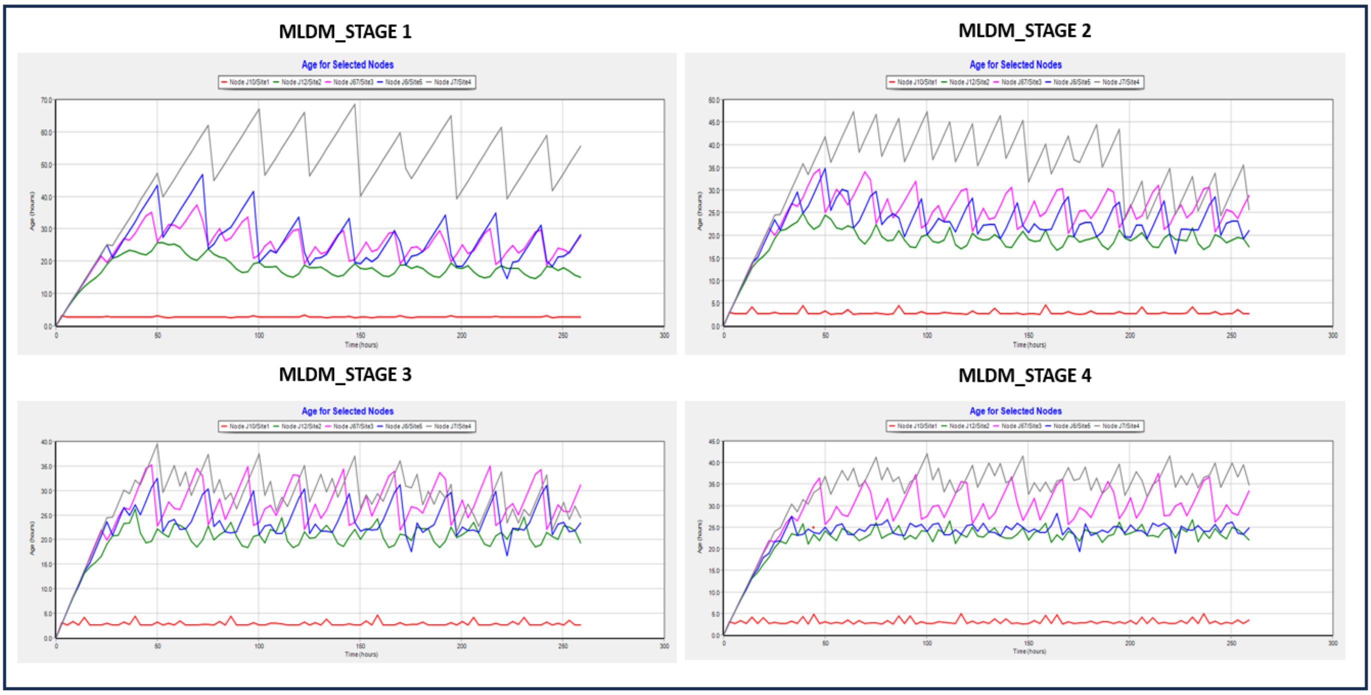
App F2-A: MLDM load-shedding stage x impact on total chlorine (11am to 3am)



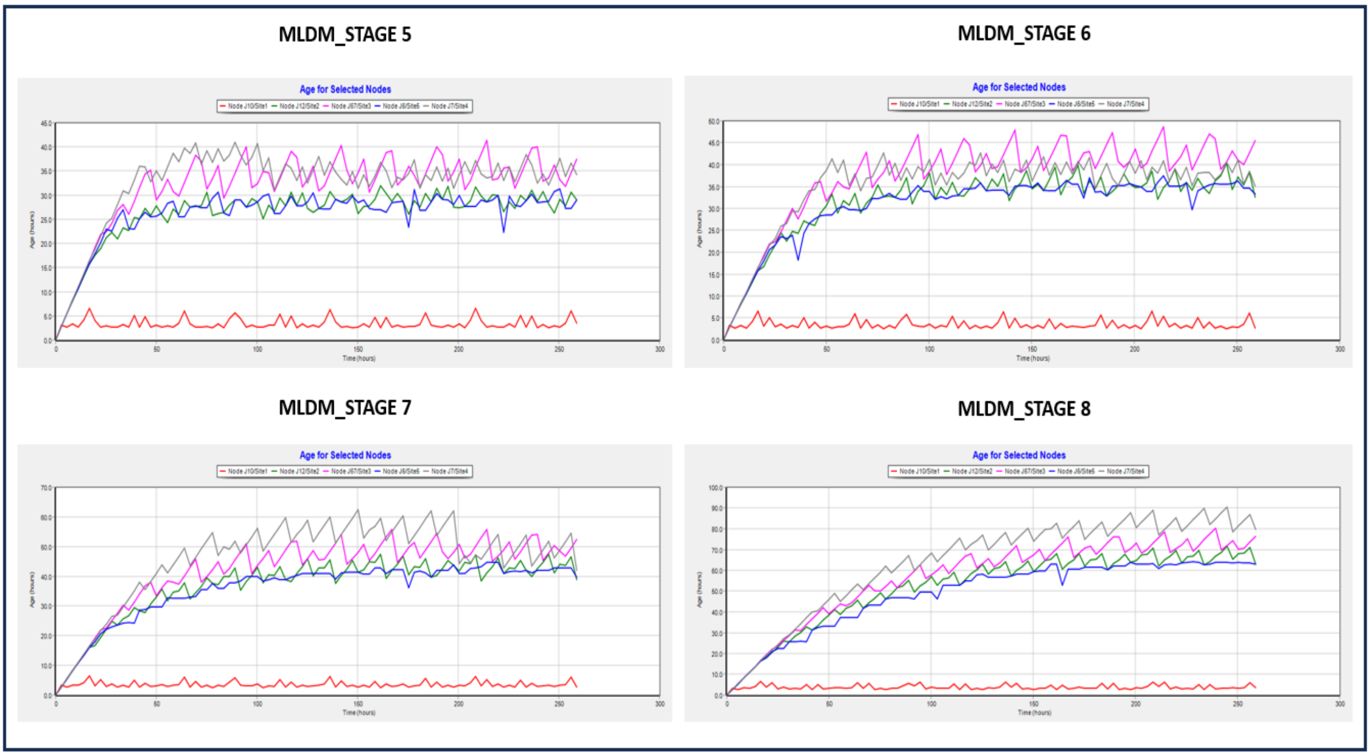
App F2-B: MLDM load-shedding stage x impact on total chlorine (3am to 11am)



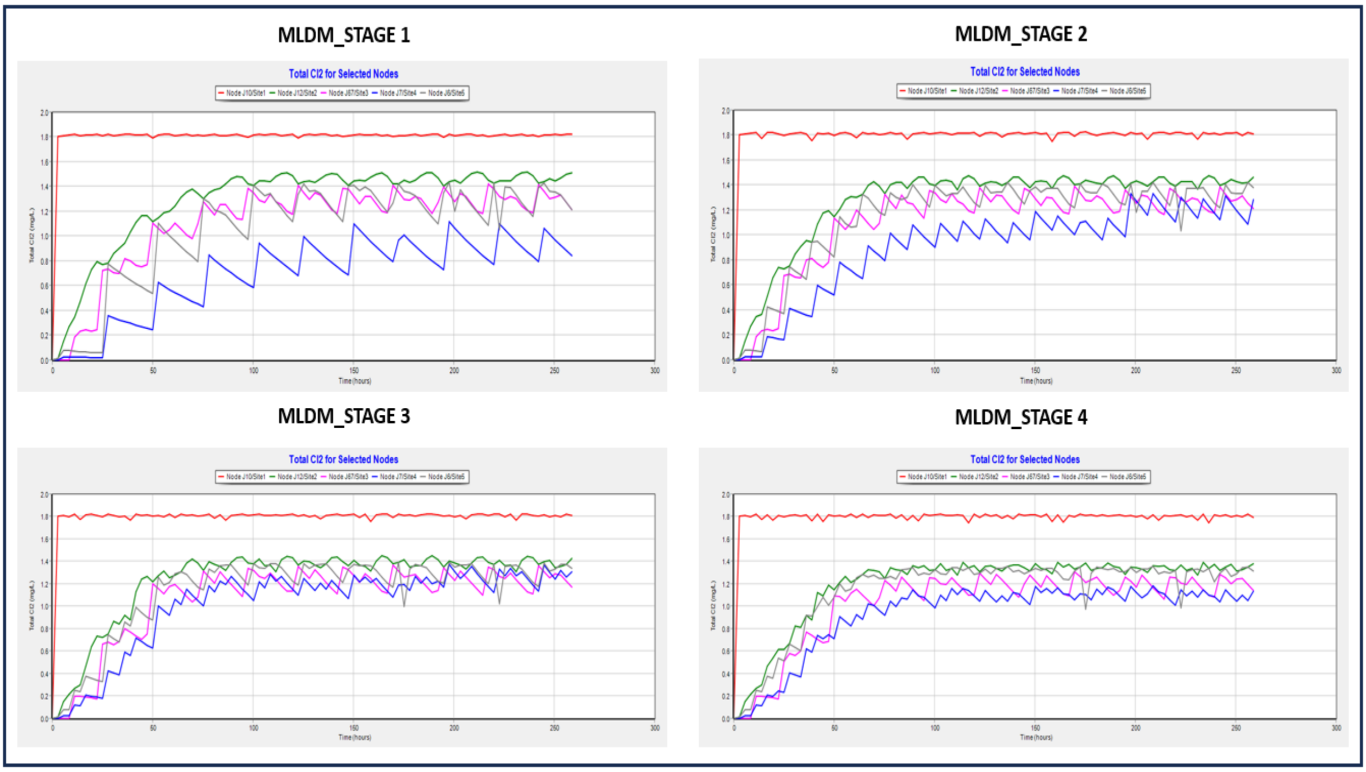
App F2-C: MLDM load-shedding (stages 1 to 4) impact on water age



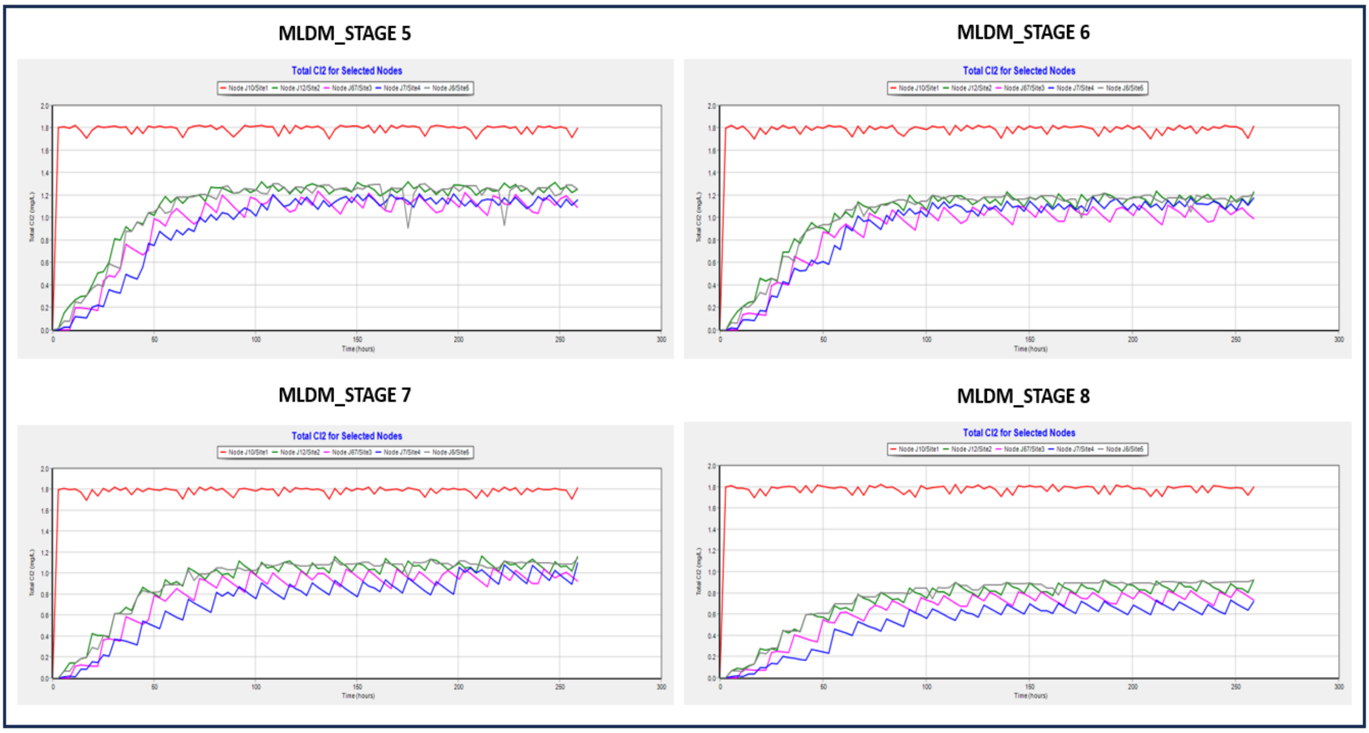
App F2-D: MLDM load-shedding (stages 5 to 8) impact on water age



App F2-E: MLDM load-shedding (stage 1 to 4) impact on total chlorine



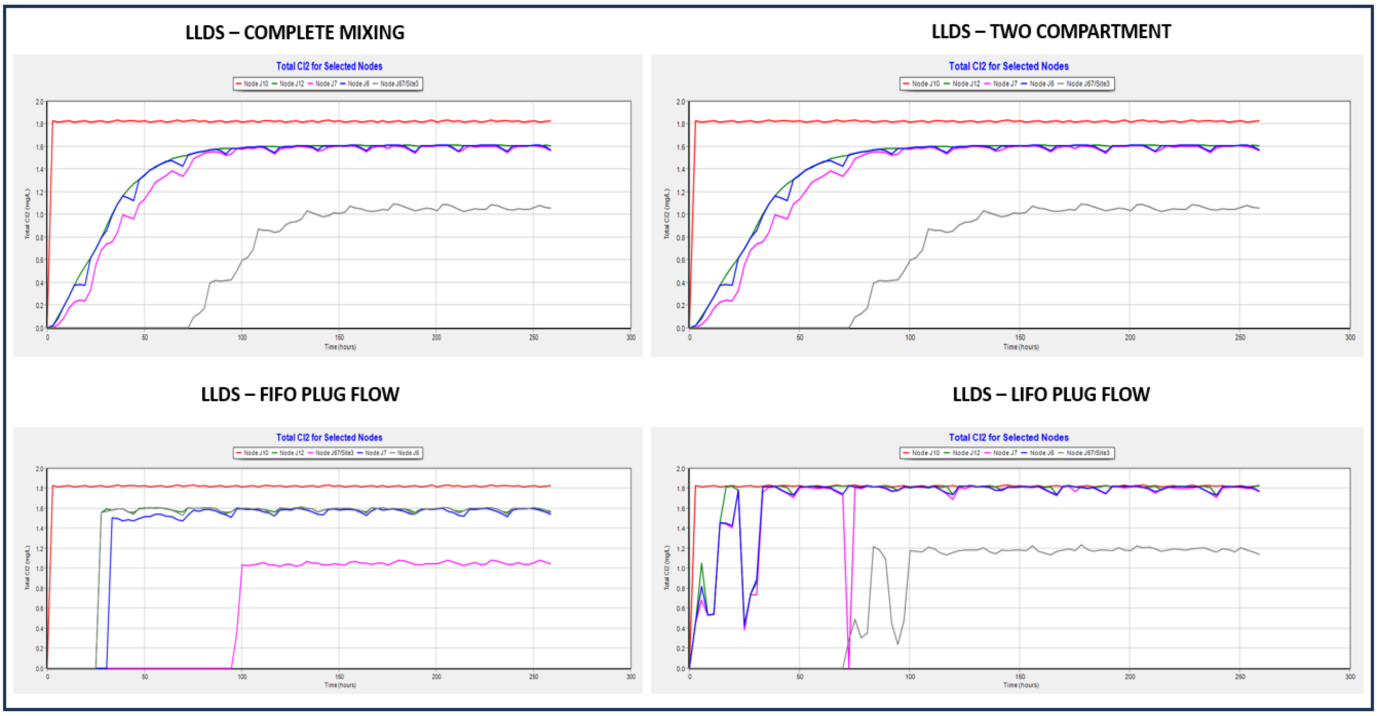
App F2-F: MLDM load-shedding (stage 5 to 8) impact on total chlorine



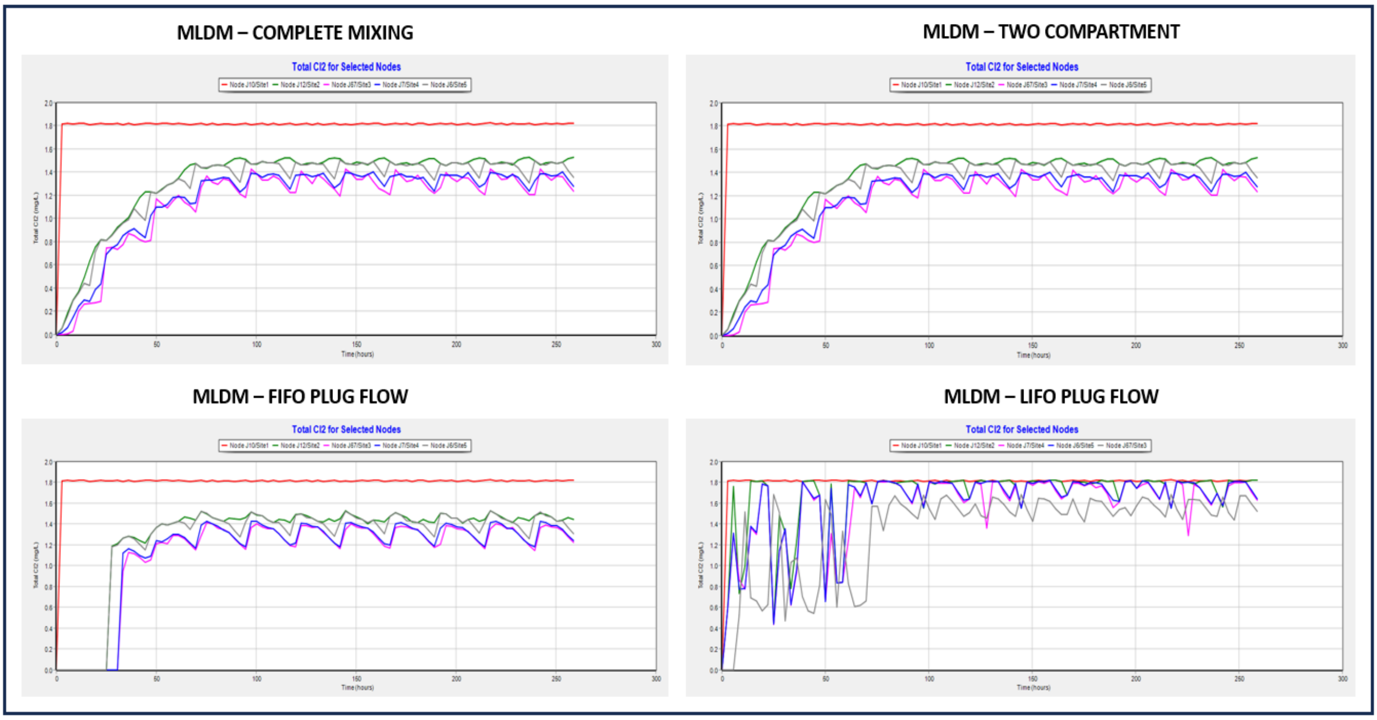
G Water quality management insight data

G.1 Water quality and tank mixing analysis model data

App G1-A: LLDS tank mixing model impact on water quality data

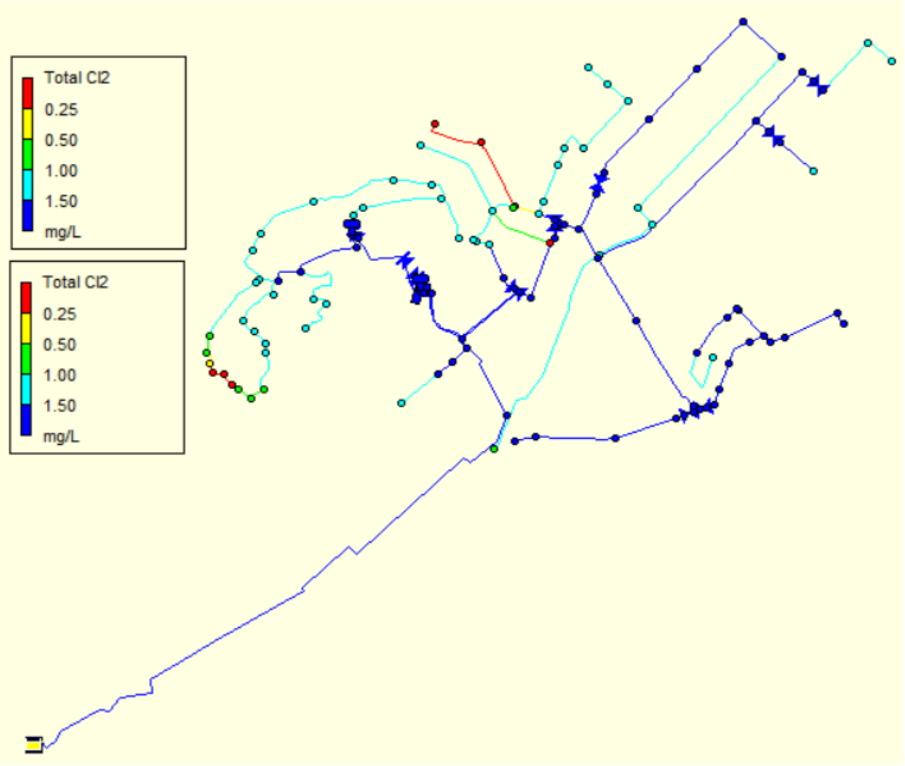


App G1-B: MLDM tank mixing model impact on water quality data

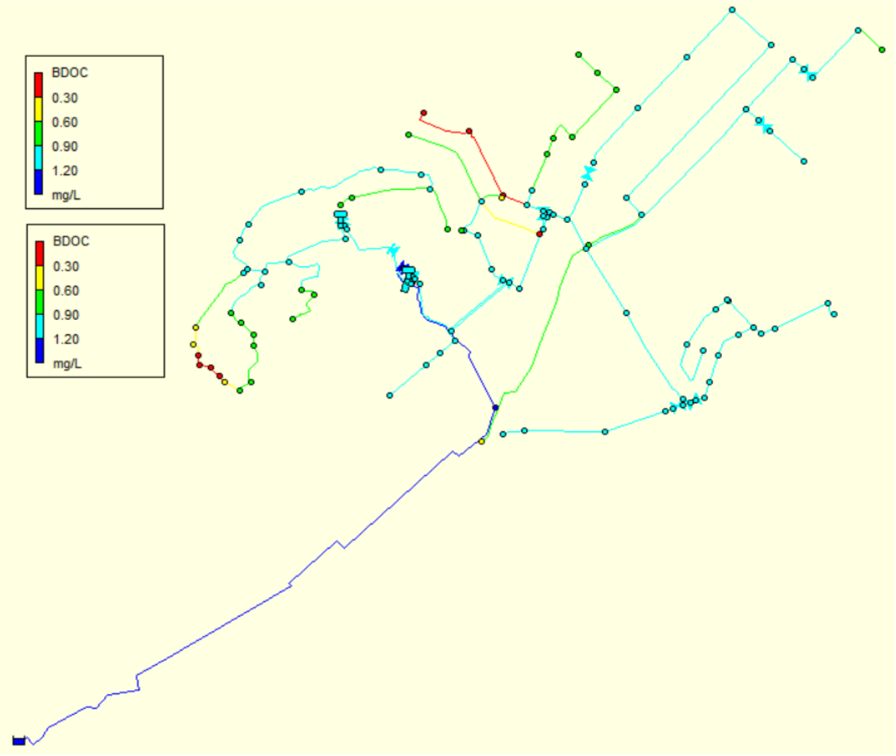


G.2 Water quality sampling locations analysis model data

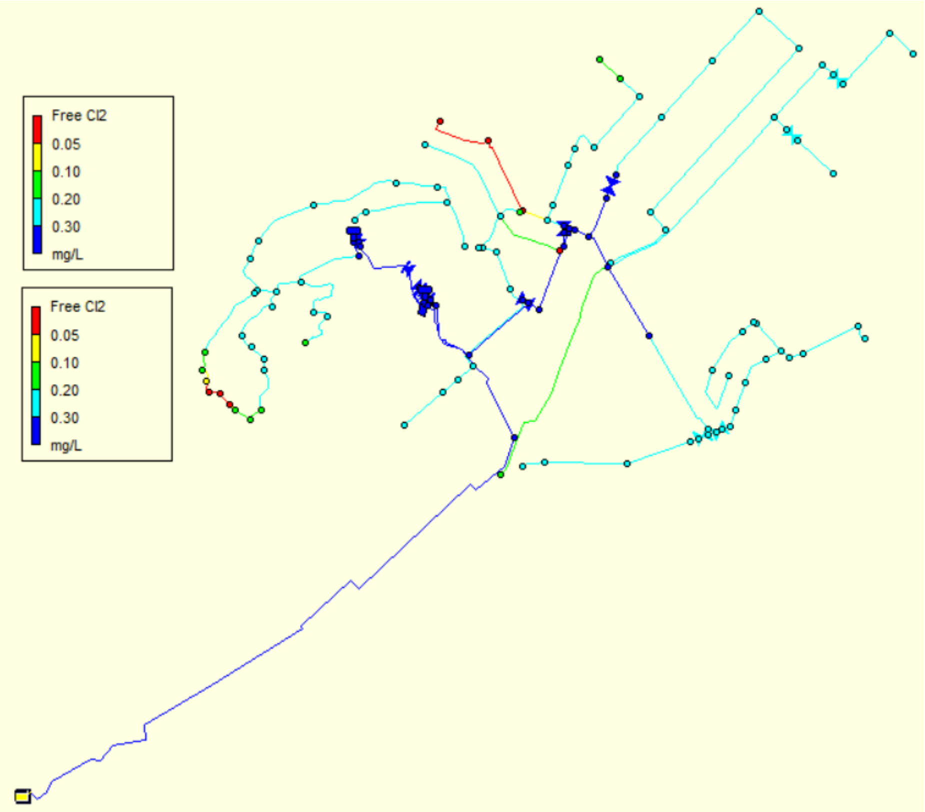
App G2-A: LLDS_EPS_VCS total chlorine model at 175.21 hours



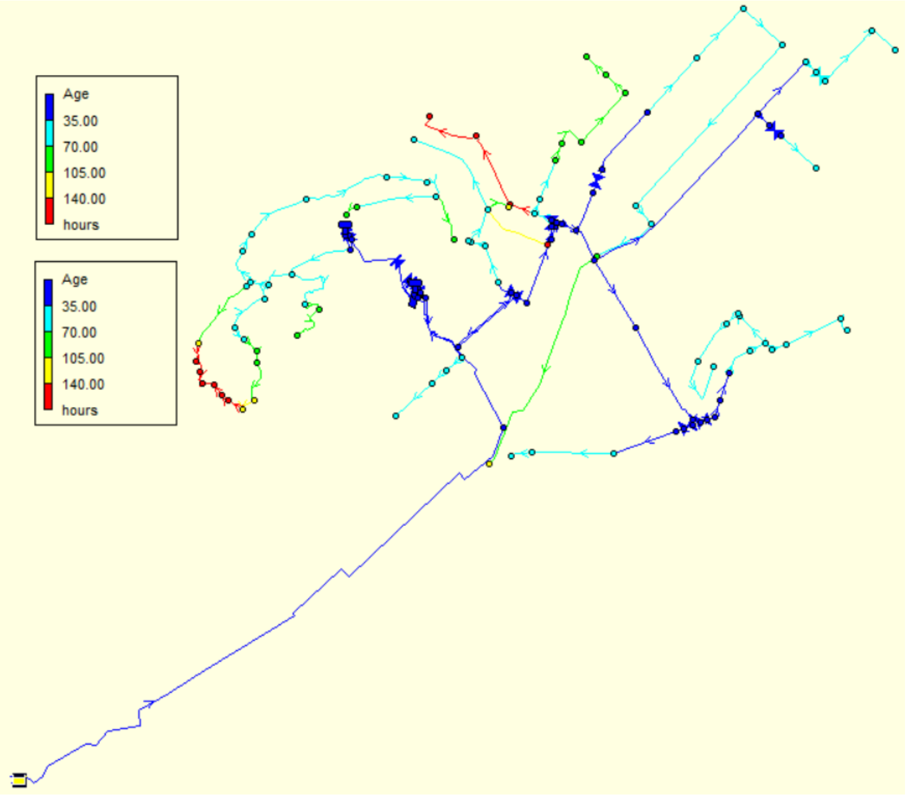
App G2-B: LLDS_EPS_VCS BDOC model at 175.21 hours



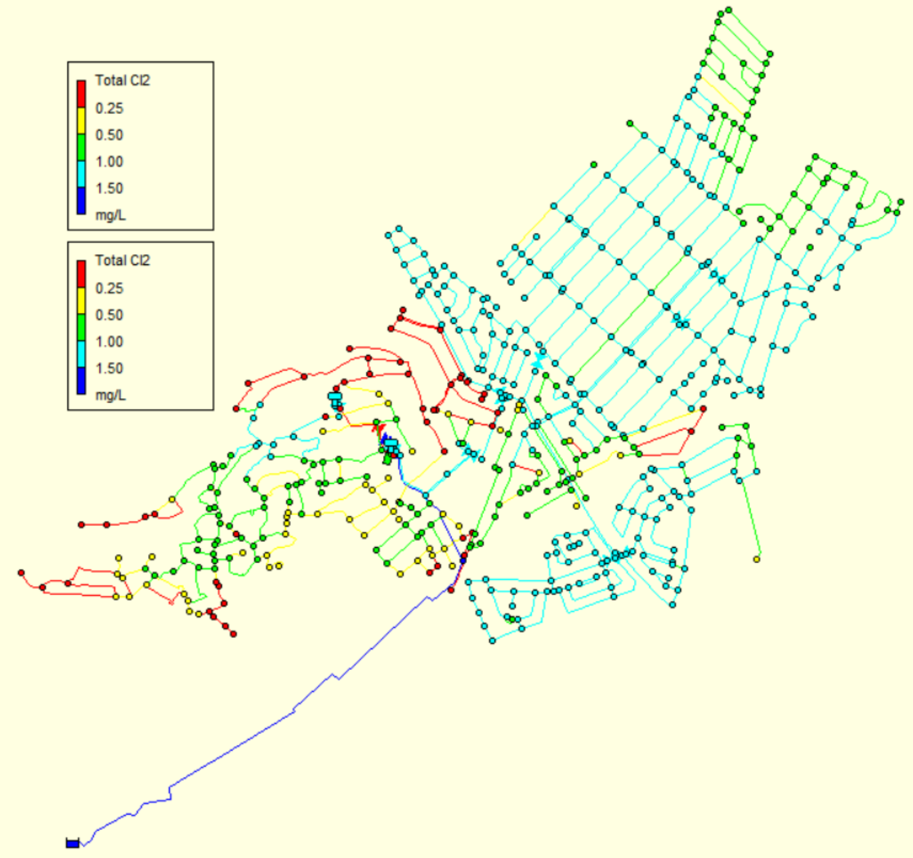
App G2-C: LLDS_EPS_VCS free chlorine model at 175.21 hours



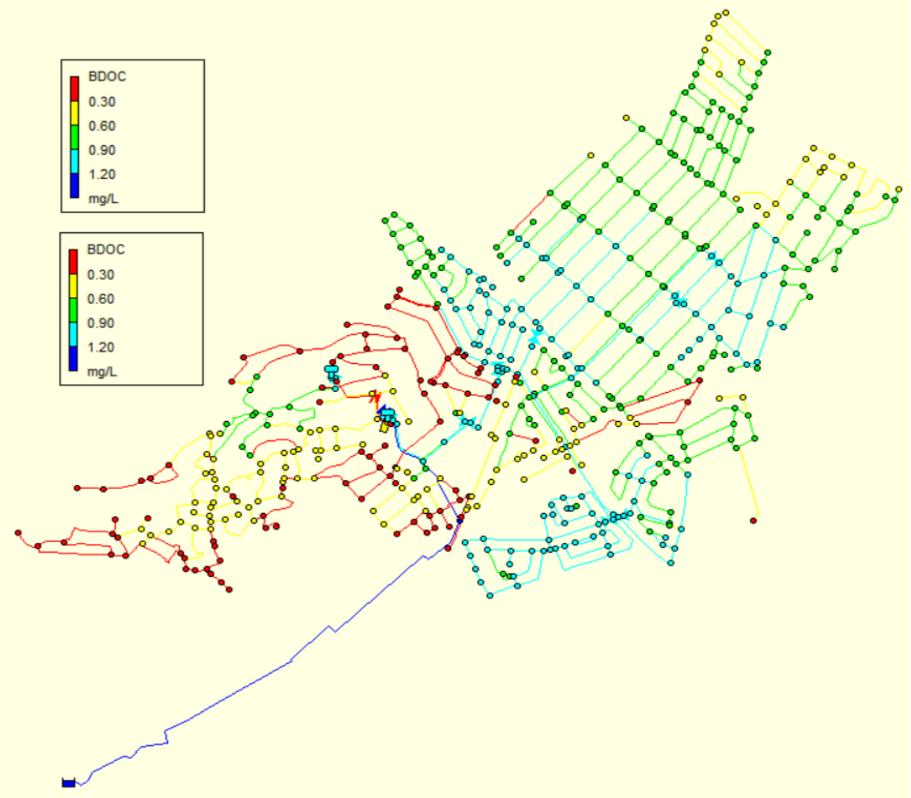
App G2-D: LLDS_EPS_VCS water age model at 175.21 hours



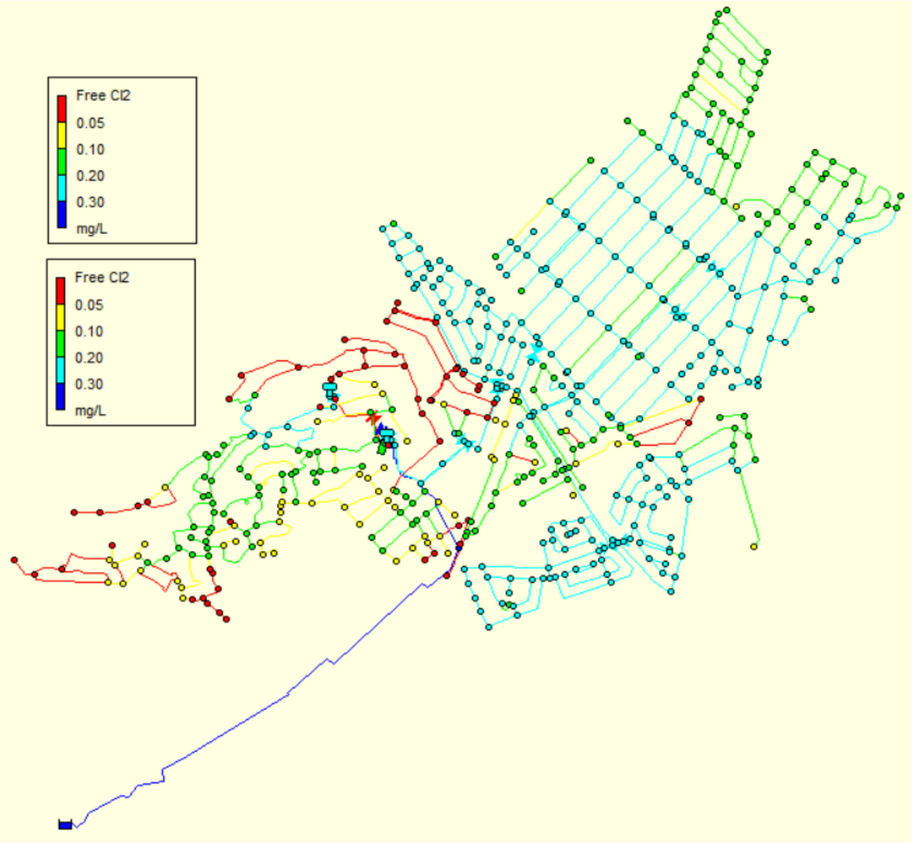
App G2-E: MLDM_EPS_VCS total chlorine model at 175.21 hours



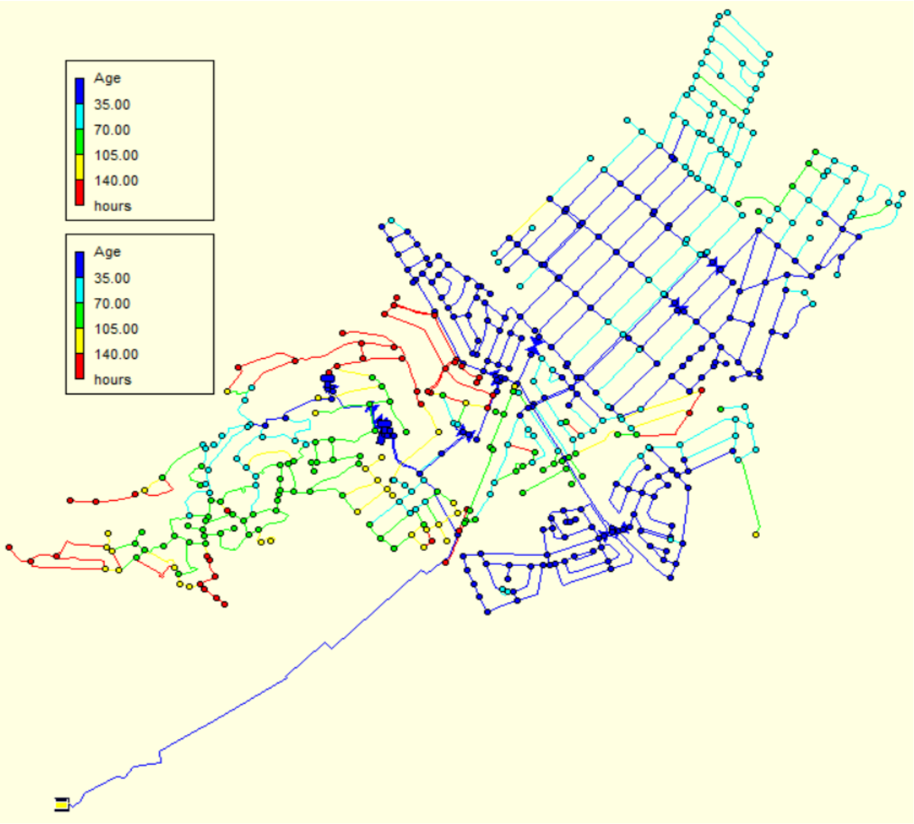
App G2-F: MLDM_EPS_VCS BDOC model at 175.21 hours



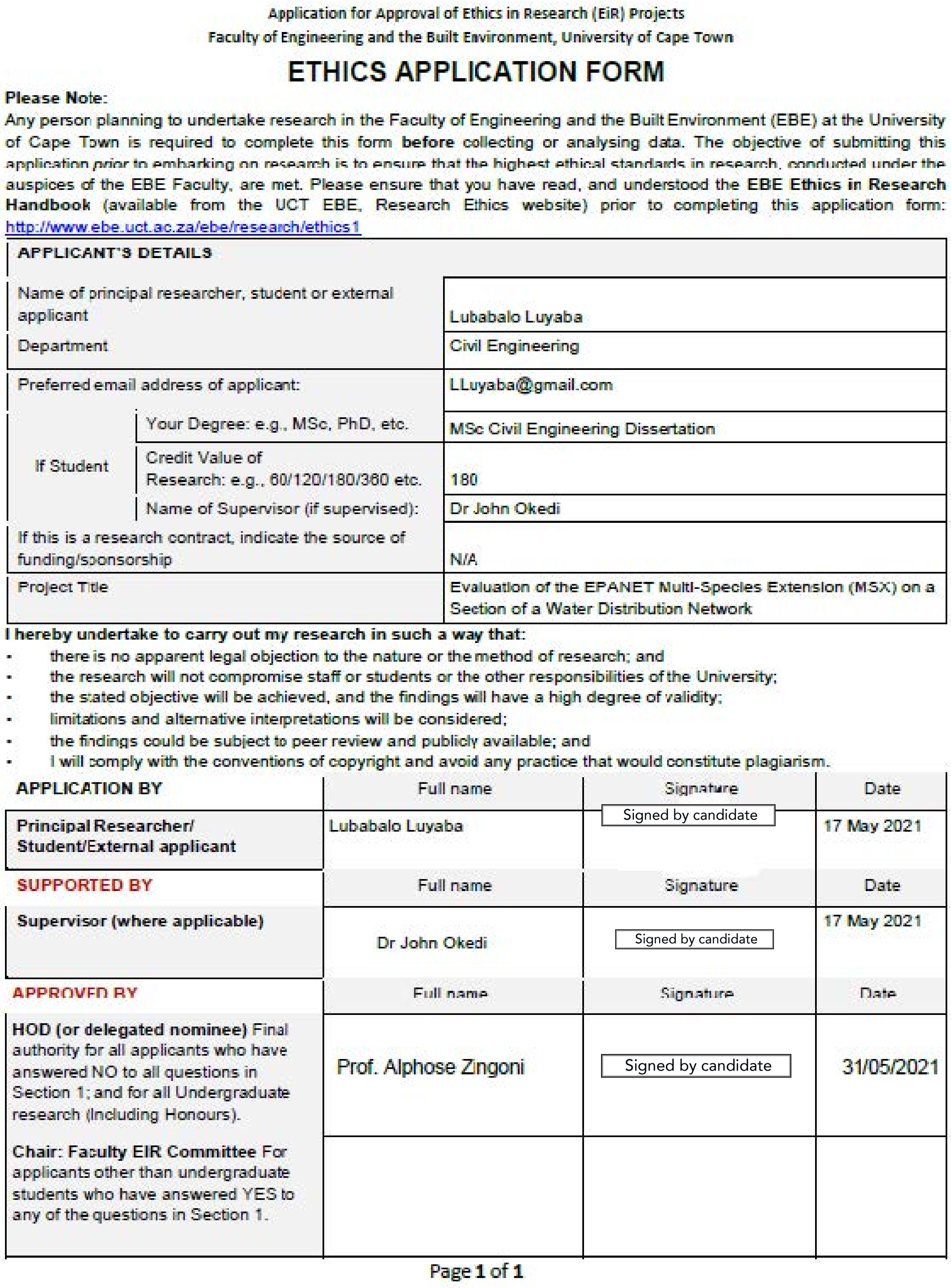
App G2-G: MLDM_EPS_VCS free chlorine model at 175.21 hours



App G2-H: MLDM_EPS_VCS water age model at 175.21 hours



H Approval of ethics in research projects



Application for Approval of Ethics in Research (EiR) Projects
Faculty of Engineering and the Built Environment, University of Cape Town

ETHICS APPLICATION FORM

Please Note:
Any person planning to undertake research in the Faculty of Engineering and the Built Environment (EBE) at the University of Cape Town is required to complete this form **before** collecting or analysing data. The objective of submitting this application *prior* to embarking on research is to ensure that the highest ethical standards in research, conducted under the auspices of the EBE Faculty, are met. Please ensure that you have read, and understood the **EBE Ethics in Research Handbook** (available from the UCT EBE, Research Ethics website) prior to completing this application form: http://www.ebe.uct.ac.za/ebe/research/ethics1

APPLICANT'S DETAILS		
Name of principal researcher, student or external applicant		Lubabalo Luyaba
Department		Civil Engineering
Preferred email address of applicant:		LLuyaba@gmail.com
If Student	Your Degree: e.g., MSc, PhD, etc.	MSc Civil Engineering Dissertation
	Credit Value of Research: e.g., 60/120/180/360 etc.	180
	Name of Supervisor (if supervised):	Dr John Okedi
If this is a research contract, indicate the source of funding/sponsorship		N/A
Project Title		Evaluation of the EPANET Multi-Species Extension (MSX) on a Section of a Water Distribution Network

I hereby undertake to carry out my research in such a way that:

- there is no apparent legal objection to the nature or the method of research; and
- the research will not compromise staff or students or the other responsibilities of the University;
- the stated objective will be achieved, and the findings will have a high degree of validity;
- limitations and alternative interpretations will be considered;
- the findings could be subject to peer review and publicly available; and
- I will comply with the conventions of copyright and avoid any practice that would constitute plagiarism.

APPLICATION BY	Full name	Signature	Date
Principal Researcher/ Student/External applicant	Lubabalo Luyaba	Signed by candidate	17 May 2021
SUPPORTED BY	Full name	Signature	Date
Supervisor (where applicable)	Dr John Okedi	Signed by candidate	17 May 2021
APPROVED BY	Full name	Signature	Date
HOD (or delegated nominee) Final authority for all applicants who have answered NO to all questions in Section 1; and for all Undergraduate research (Including Honours).	Prof. Alphose Zingoni	Signed by candidate	31/05/2021
Chair: Faculty EIR Committee For applicants other than undergraduate students who have answered YES to any of the questions in Section 1.			

Page **1** of **1**