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ADDRESSING HIGH DIMENSIONALITY IN WATER QUALITY MODELLING IN WATER DISTRIBUTION NETWORKS

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DECLARATION

I hereby declare that:

- This research project represents my original work.
- Except as properly attributed through thorough and correct referencing, this research project contains no information that has already been published or authored by a third party.
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- I have correctly acknowledged all direct quotations and paraphrased content appropriately.
- The work was done under the guidance and supervision of Professor Akpofure Taigbenu at the University of the Witwatersrand, Johannesburg.

10 January 2024

Signature (Morongwa Machweu)   Date

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ABSTRACT

Water quality models are the most effective tools for characterizing water quality conditions, assessing the effects of water pollution, and supporting decision-makers with water quality management. They can be utilised for detecting the variations in the water quality parameters. Despite the usefulness of water quality models, an appropriate and simple water quality descriptor for a particular application, considering the high dimensionality of various water quality parameters, remains a challenge (Chapman, 1992). To address this high dimensionality, a single dimensionless index is commonly used to describe water quality for a particular application. While pollution loads at various points in a river reach have been widely assessed by studies using water quality indices, little research has been done on water distribution networks with service reservoirs and a variation of loading conditions. In a water distribution network, service reservoirs function similarly to rivers in that they have complicated mixing mechanisms, are subject to a variety of water quality factors, and are sized and located differently. The most common water quality indices require the formation of sub-indices and weights to avoid ambiguity, eclipsing and rigidity. The Canadian Council of Ministers of the Environment Water Quality Index (CCME WQI) does not require the formation of sub-indices and weights, thus providing a simplified way of describing water quality. This study investigates the use of the CCME WQI to address high dimensionality in water quality modelling of water distribution networks, taking into consideration the locations of multiple service reservoirs. This study was carried out primarily for decision-making and design optimization purposes only. Using EPANET 2.2, four hydraulically optimised solutions (which satisfied minimum pressure requirements) were further analysed for water quality performance. This was achieved by incorporating simulated data on three water quality variables (chlorine residual, water age and THM concentration) into the CCME WQI for a hypothetical water distribution network, Anytown. The results indicate that two of the four hydraulically optimised solutions achieved excellent water quality levels. This study has demonstrated the usefulness of a dimensionless index as a proxy for multiple water quality variables of a water distribution system in facilitating decision-making.

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LIST OF ABBREVIATIONS

ADD	Average Day Demand
CCME	Canadian Council of Ministers of the Environment
DBP	Disinfection By-Products
DWAF	Department of Water Affairs and Forestry
EPA	Environmental Protection Agency
EPS	Extended Period Simulation
IPF	Instantaneous Peak Flow
NSF	National Sanitation Foundation
OWQI	Oregon Water Quality Index
PFMOEA	Penalty-Free Multi-Objective Evolutionary Approach
SSS	Steady-State Simulation
THM	Trihalomethane
WDN	Water Distribution Network
WDS	Water Distribution System
WHO	World Health Organisation
WQI	Water Quality Index
WQM	Water Quality Model

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CHAPTER 1: INTRODUCTION

1.1. Background of the study

In the management of water resources, one of the most critical components is its quality. It is important to assess not only the quantity of water available, but also its quality (Swamee and Tyagee, 2000). It is vital to explore both the quantity and quality of water to match water supply and demand. The three major categorisation of water quality are the physical, biological, and chemical. Monitoring these three categories commonly involves sampling water at different points and times within a water system. This process can be expensive, time consuming and labour-intensive. Researchers have identified water quality models (WQMs) as suitable and cost-effective tools for predicting the variations of these categories in a water system. A water quality model is a mathematical description of how pollutants are distributed, moved, and degraded in a body of water. To some extent, these models can play an important role in reducing the costs involved in water quality assessments. However, water quality models are not exempt from challenges and/or constraints.

There are various sources of errors that could result in unreliability in water quality models: how the inputs are measured and the response unreliability (Rode & Suhr, 2007), unreliability in parameters, structural error caused by a model's inability to accurately reproduce physical mechanisms (Montanari, 2004). It is also vital to select a water quality model based on its intended use. Some models have limitations because they are only suitable for specific water bodies, are able to simulate selected water quality variables, require skilled users, and necessitate a large volume of data.

Water quality modelling results can be difficult to analyse due to the multiple variables involved. As a result, effective water resource management decisions are made more challenging for policymakers and water resource engineers. One of the ways of overcoming these challenges is the use of a single aggregated value, namely a water quality index (WQI), to represent the pollutant loadings at various points in the water system. It is important to note that parameters and variables are used interchangeably throughout the report.

1.2. Problem Statement

The aggregation of sub-indices and weights in attaining a given WQI can result in ambiguity, eclipsing and rigidity (Abbasi and Abbasi, 2012). Ambiguity is a feature of linear and root sum square aggregation forms that might cause some concern. Ambiguity exists when all of the sub-indices meet the standard for a specific use or application, but the overall index fails. As a result, the overall water quality may be deemed poor, even when it is truly of acceptable level for that application (Sutadian et al, 2016). Eclipsing is a feature of weighted root mean square aggregation that can offer the impression of safety. When one or more sub-indices have lower values but are dominated by higher values for other sub-indices or vice versa, this is referred to as eclipsing and results in the final index value being inaccurately indicative of the overall water quality status (Swamee and Tyagi, 2000; Liou et al, 2004).

When more important variables are required in an index when addressing certain water quality issues, but the user is unable to add the additional variables in the aggregated index in such a way that those issues are correctly incorporated, rigidity results (Swamee and Tyagi, 2007). A WQI which does not require the formation of sub-indices and weights provides a simplified way for water quality monitoring and addressing high dimensionality in water distribution networks. The CCME WQI is preferred mainly for this reason. Glen (2013) states that dimensionality differs based on the context it is being used for. In statistics, dimensionality refers to “how many variables a dataset has”, whereas in mathematics and physics it is expressed differently. In the field of physics, the concept of dimensionality is often quantified by employing fundamental dimensions such as mass, time, or length. Glen (2023) also states that “in matrix algebra, two units of measure have the same dimensionality if both statements are true: (1) A function exists that maps one variable onto another variable and (2) the inverse of the function in (1) does the reverse”. High dimensionality in this study refers to the number of parameters and their variations over time to describe the water quality of a water distribution system (WDS). Three parameters are used and assessed with a simulation interval of 1 minute over a 24-hour period in this study. It is important to note that a Water Distribution Network (WDN) and a Water Distribution System (WDS) are the same and the names are used interchangeably in this report.

1.3. Rationale

The term “water quality index” (WQI) refers to a unique method of assessing water quality that has been utilised to examine temporal and spatial water variations in South African river basins and the global water cycle (Banda and Kumarasamy, 2020). The creation of a single value from a variety of complex water quality data is made easy and repeatable using water quality indices (WQIs). Which explains why WQIs have been used successfully since the 1960s (Hamlat et al, 2017) since they enable the collection of a lot of scientific data and the dissemination of the status of water quality to the general public and policymakers using a simple, dimensionless score. When classes are presented as “poor”, “fair”, “medium”, “good”, and “excellent”, the water quality evaluation ratings will be clear to even non-technical stakeholders (Banda and Kumarasamy, 2020). This enables effective decision making in water quality monitoring. When not only hydraulic performance but water quality is included in design optimisation, water engineers stand a better chance of producing multi-objective and sustainable designs. Incorporating water quality into design optimisation of water distribution networks can be a challenging, but important task. Comparing the water quality values to the local standards has historically been utilised as a technique to assess the quality of water. The spatial and temporal variations of the overall quality, however, are not provided by this technique. Consequently, modern approaches have been created, such as the WQI. The use of WQMs and WQIs provides water engineers with a simplified way of taking water quality assessment into consideration during the design process.

This is meant to assist water engineers with effective decision-making during design optimisation stage. Water engineers can gain a better understanding of where to locate service reservoirs depending on how water quality changes during water quality modelling. Although this paper does not include the design of a new water distribution network, it will assess a few design optimisation solutions found in literature where service reservoirs are sited at multiple locations within the network. A few authors (Walters et al, 1999; Siew et al, 2016; Prasad, 2010; Vamvakeridou-Lyroudia et al, 2005) provided design optimisation simulation solutions to the Anytown network (Walski et al, 1987) and located service reservoirs at different locations within the network. These service reservoirs will be assessed from a water quality point of view through water quality modelling using the EPANET model. The CCME WQI has primarily been utilised up to this point in assessing of the water quality in river reaches and across existing water distribution systems where water samples are taken at specific points for a certain period (usually not less than a year). In all the cases investigated, design optimisation for service reservoirs was not taken into consideration.

1.4. Research Question

How efficient is the CCME WQI in analysing and incorporating water quality for design solution optimisation in water distribution networks that contain various loading conditions and service reservoirs?

1.5. Aim and Objectives

The primary objective of the study is to assess the usefulness of the CCME WQI to address the high dimensionality of water quality in water distribution networks for the purposes of design optimization and decision-making.

The objectives of the study are:

- To obtain spatial and temporal water quality variations of water quality parameters for efficient water quality analysis.
- To calculate a single, non-dimensional value using the water quality modelling results and the CCME WQI.
- To assess the usefulness of the CCME WQI to address high dimensionality in water quality modelling by comparing it with other results found in literature.

1.6. Methodological approach

The Anytown Network has been selected for this study because it is a good representation of a network that is still in the design optimisation stage. This network is subjected to five different loading conditions and has multiple service reservoirs at different locations within the network. More details on this network have been provided in Chapter 3 of this study. A more detailed methodology can also be found in Chapter 3 of this study.

Objective 1:

EPANET 2.2 (Rossman, 2000) was used to achieve objective 1. Hydraulic and water quality analysis was conducted to collect network data that could be utilised in determining the final water quality index. This process is further explained in Chapter 3 and the results are discussed in Chapter 4 of this study. The results obtained from the water quality modelling were compared against drinking water quality standards.

Objective 2:

The CCME WQI was used to calculate the final index score at each of the service reservoirs and the water quality status was interpreted using the water classification tables that can be found in Chapter 3 of this study.

Objective 3:

The results obtained in objective 2 were compared to those obtained using the Oregon WQI (OWQI). Any challenges, limitations and assumptions encountered in using the CCME WQI on this network have been discussed in chapter 5. The advantages and disadvantages of such an index have been discussed in comparison to the other indices that require the formation of sub-indices and weights.

1.7. Structure of the research report

Below is a summary of how the study is structured:

Chapter 1: The study's background and significance are discussed in this chapter. It highlights the aims and objectives of the study.

Chapter 2: In this chapter, previous studies on the use of water quality modelling and indices in water distribution systems are summarised. It highlights the gaps found in literature and goes into more detail about the significance of integrating water quality analysis into the design optimisation process.

Chapter 3: Outlines the methodological approach used to accomplish the study's objectives. It provides more information on the area of study and describes how the hydraulic and water quality simulations were carried out.

Chapter 4: The outcomes of the hydraulic and water quality modelling are covered in this chapter. The chapter also covers the condition of water quality in the studied network, and the process of determining the CCME WQI score.

Chapter 5: A synopsis of the research results is presented, along with an examination of the study's constraints and recommendations for future investigations.

CHAPTER 2: LITERATURE REVIEW

2.1. Overview

From the literature, a substantial amount of effort has been dedicated to water quality modelling and the application of water quality indices. These studies have been conducted primarily on raw water quality and not water distribution networks. Optimal design solutions that incorporate water quality analysis are important for ensuring that the end consumer receives the same quality of water as from the water treatment plant. Research indicates that there are several changes that occur during transportation in a WDS that could result in the deterioration of water quality (Rossman et al, 1994). Some of these changes include odour and taste development, bacterial re-growth due to a loss of disinfection residuals (Clark & Haught, 2005), development of potentially carcinogenic disinfection by-products (DBPs) due to reaction of organic and inorganic substances with the disinfectant (Rodriguez et al, 2004) (figure 1.1), and corrosion.

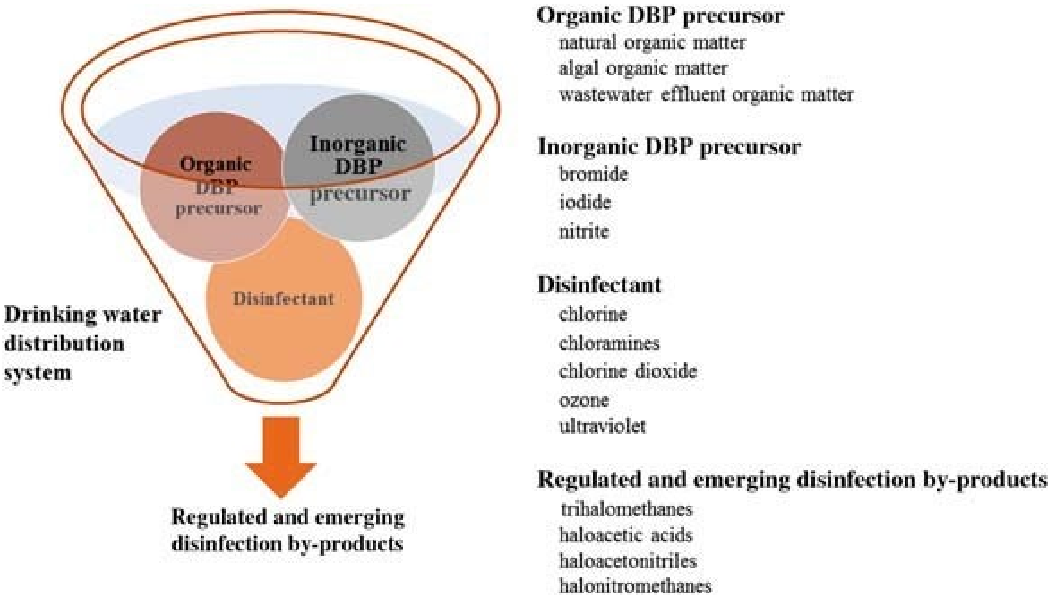


Figure 1.1: Diagram showing how disinfectants react with organic and inorganic disinfection by-product (DBP) precursors in drinking water distribution systems to create regulated and emergent DBPs (Dubey et al, 2020).

Before it reaches the user, finished water may undergo chemical or biological changes, lose system integrity, or experience other quality changes. Water distribution systems usually use a combination of wells, diverse surface sources, or both to get their supply of water. The hydraulic behaviour of the WDS leads to the mixing of water from different sources within the system. Chemical instability can occur when the waters come from diverse sources and have varied properties. The system's tendency to distribute water of varying quality to consumers over time and space may result from all the aforementioned factors.

Water providers are under pressure to better understand the dynamics of water quality processes and to manage them to conform to the norms of regulatory bodies. One cost-effective way of achieving this is through the use of water quality models.

2.2. Defining water quality modelling

A water quality model (WQM) is defined as a mathematical representation of pollutant fate, transport and degradation within a water body (Kebede, 2009). To some extent, these models can play a significant role in reducing the costs involved in water quality assessments. However, water quality models are not exempt from challenges and/or constraints. There are various sources of errors that could result in unreliability in water quality models: how the inputs are measured and the response unreliability (Rode & Suhr, 2007), unreliability in parameters, structural error caused by a model's inability to accurately reproduce physical mechanisms (Montanari, 2004).

It is also vital to select a water quality model based on its intended use. Some models have limitations because they are only suitable for specific waterbodies, they select which water quality parameter to simulate, require skilled users, necessitate a large volume of data and are not commercially available. Water quality modelling results can be difficult to analyse due to the multiple variables involved. This makes it difficult for policy makers and water resource engineers to make effective water resource management decisions. As a result of the high dimensionality in water quality modelling, water quality indices are then used to quantify water pollutants at various points in the water system by using a single aggregated value. Water Quality Indices are a functional aggregation of data on water quality parameters and produce a single value that is representative of the water quality, thereby consolidating a huge amount of information into an understandable and simple descriptor.

2.2.1. Constitutive equations for water quality modelling

A method that considers both advection and dispersion is presented by Tzatchkov et al. (2002). WQMs frequently consider reactions at the pipe wall and in the bulk flow while assuming advective-reactive transport. The governing equations consider mass conservation, reaction kinetics, and frequently assume that water at nodes, junctions, and storage facilities is instantaneously and completely mixed (Monteiro et al, 2015; Rossman, 2000; Grayman et al, 1988; Rossman and Boulos 1996). A quick summary of the mass conservation equations for pipes, junctions, and storage facilities as provided by Rossman (2000) is as follows:

The equation for mass conservation in pipes is:

$$\frac{\partial C_i}{\partial t} = -u_i \frac{\partial C_i}{\partial x} + r_i; \qquad \forall_i \tag{1.1}$$

Where $u_i \equiv u(i,t)$ is the mean flow velocity in pipe i at time t;

$C_i \equiv C(i,x,t)$ is the reactant concentration in pipe i at time t and location x; and

$r_i \equiv r(C_i)$ is the reaction rate.

The equation assumes that there is negligible longitudinal dispersion in the pipes.

The equation for mass balance at node n is:

$$\left(\sum_{j\in I_n} Q_{pj} + Q_e\right) C(i,x,t)_{x=0} = \sum_{j\in I_n} \left(Q_{pj} C(j,x,t)_{x=L_j}\right) + Q_e C_e; \qquad \forall i \in O_n,\ \forall n \tag{1.2}$$

Where j is the flow entering the node n and i is the flow exiting node n;

L_j = pipe length;

I_n = links with flow entering and;

O_n = links with flow exiting node n;

Q_{pj} = volume flow rate in pipe j

Q_e = volume flow rate and

C_e = the reactant concentration for any external flow entering node n.

The equation for mass balance at the S^{th} storage facility/tank/service reservoir is:

$$\frac{\partial(V_s C_s)}{\partial t} = \sum_{i \in I_s} \left(Q_{pi} C(i,x,t)_{x=L_i}\right) - \sum_{j \in O_s} Q_{pj} C_s + r_s; \qquad \forall s \tag{1.3}$$

Where $C_s \equiv C(s,t)$ is reactant concentration at time t;
O_s indicates links receiving flow from the storage facility;
$V_s \equiv V(s,t)$ is storage volume at time t;
$r_s \equiv r(C_s)$ refers to the reaction rate.
I_s refers to the set of links that provide flow to the storage facility;
In addition to bulk flow reactions, pipe walls could undergo corrosion and reactions with biofilm and other substances (Seyoum and Tanyimboh, 2017)

EPANET 2.2 (Rossman, 2000; Powell et al, 2004) uses the reaction rate coefficient for pipe wall reactions as:

$$k_{w*} = \frac{k_w k_f}{R\left(k_w + k_f\right)} \tag{1.4}$$

Where k_{w*} is the reaction rate coefficient of the wall (time^{-1});
k_w is reactivity coefficient of the wall (length/time);
k_f is determined by the flow turbulence and the molecular diffusivity of the reactive species.

EPANET 2.2 is a single-species, fast-reacting kinetic model that was utilized for modelling disinfection by-product, THM, water age and the disinfectant, chlorine. A distribution system's insufficient chlorine residual could cause bacterial regrowth and, as a result, water-borne diseases (Clark and Haught, 2005). According to Ghebremichael et al. (2008), chlorine interactions with natural organic molecules in water produce disinfection by-products that have a negative impact on human health (Nieuwenhuijsen and Toledano, 2000).

The first-order kinetic model for chlorine decay (Rossman, 2000) is:

$$r(C_C) \equiv \frac{\partial C_C}{\partial t} = -kC_C \tag{1.5}$$

C_C is the chlorine concentration at the demand node or junction at time t, recalling that the reactant concentrations vary with time and space as in Equations 1.1 to 1.3.
k is the reaction rate constant.
It is important to stress the connection between the results for the water quality and the completely developed operational cycle (This is typically 24 hours).

The first-order reaction model (Rossman, 2000) for trihalomethanes is:

$$r(C_{TTHM}) \equiv \frac{\partial C_{TTHM}}{\partial t} = k(C_L - C_{TTHM}) \tag{1.6}$$

Where C_L is the final concentration and C_{TTHM} is the time-varying total concentration of trihalomethanes (Rossman, 2000).

2.3. Water quality indices

2.3.1. Historical background

The concept of characterizing water quality based on the extent of pollution and degree of purity originated in Germany in 1848 (Medeiros et al, 2017; Lumb et al, 2011a). The “saprobic system” was created as a biological paradigm for measuring water quality in the 19th century (Kolkwitz and Marsson,1909 as cited in Banda, 2020). The method computes a saprophytic index based on the organic degradable content of water sources (Medeiros et al, 2017; Sládeček, 1973; Rolauffs et al, 2004; Cairns, 1974; Lindegaard, 1995; Hawkes, 1998). The saprobic indexing technique focuses on relative proportions of a selection of aquatic biological organisms and the distribution pattern; however, such a non-chemical study is incapable of addressing present water quality concerns. The saprobic system, which assesses the presence of specific aquatic organisms to determine if water meets certain minimum quality requirements, has been widely accepted by the public and remains a traditional method of evaluating the suitability of water for various applications (Rolauffs et al, 2004; Cairns, 1974, as cited in Banda, 2020).

The first parameter-based numerical indexing system was created by Horton (1965), more than a century after the invention of the saprobic index. In this method, selected physical and chemical water quality variables are rated and aggregated using a mathematical model. After Horton proposed the original water quality index (WQI) in 1965, many other indices were developed to build on this initial idea.

Information gathered from WQIs can help with the following purposes:

a) Describing the quality status of water bodies to the community, regulatory bodies/policy makers and researchers (Ocampo-Duque et al, 2006).

b) Examining how environmental quality is affected by regulations and policies (Swamee & Tyagi, 2007).

c) Reducing the cost of comparison on the quality data from different water sources without conducting highly technical assessments. (Sarkar and Abbasi, 2006).

d) To help the public and policy makers in preventing intuitive assessments and prejudiced opinions (Štambuk-Giljanović, 2003).

2.3.2. Common steps in formulation of WQIs

Since Horton's initial index in 1965, a sizable number of other indices have been created, however, despite these attempts, there is still no method for creating water quality indices that is universally accepted (Banda and Kumarasamy, 2020; Sutadian et al, 2016). However, Figure 2.1 shows a discernible and true trend that can be identified by the following common steps:

a) Parameter selection

This involves selecting and identifying the most important water quality variables or parameters that will give the water quality index a real-world meaning. Setting just the right number of parameters — neither too little nor too much — requires proficiency. Statistical methods or expert judgment (individually or collectively) can both be used to choose the parameters (Banda, 2020).

b) Formation of sub-indices

It becomes necessary to convert different water quality parameters into a single common scale because they have different scientific units. This process is accomplished by creating sub-indices.

c) Determination of weights

The coefficients used to weigh the input parameters are established by assessing the possible impact of each parameter, particularly if its concentration exceeds the permissible limits. These coefficients are allocated based on the significance of each parameter.

d) Aggregation of the final index

Consequently, it is considered as the last step to obtain an absolute index value. The sub-indices are combined into a single index number using mathematical models that take into account the assigned weights. There are several aggregation techniques, but three basic models are frequently employed. These have logical, additive, and multiplicative properties.

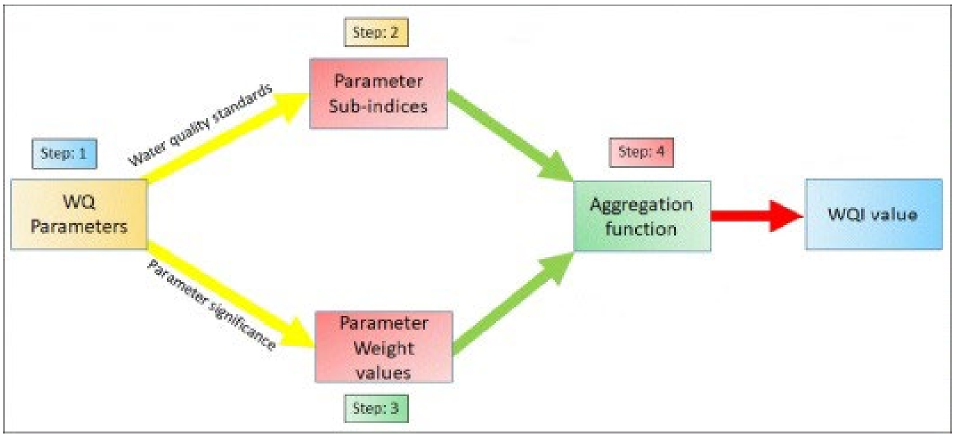


Figure 2.1: Common steps for formulation of WQIs (Uddin et al, 2021)

2.3.3. Common water quality indices

2.3.3.1. Canadian Council of Ministers of the Environment Water Quality Index

The British Colombia WQI was modified by the Canadian Council of Ministers to develop the CCME WQI. This index provides a mathematical framework that is straightforward and suitable for the purpose of calculating the final index value. The water quality status or the health of the water body can then be evaluated by users with the help of this index (Tirkey et al, 2013). The CCME WQI was developed by the Canadian Council of Ministers of the Environment as a tool to evaluate and disseminate information about water quality to management organisations and the general public (CCME, 2001).

This WQI has been applied in various literature for various purposes. Currently, it can be utilised for examining the state of the water quality in multiple river catchments (Khan et al, 2003; Davies, 2006; Boyacioglu, 2007; Lumb et al, 2006), drinking water quality in a WDS (Khan et al, 2004 and Hurley et al, 2012), and metal mines water quality (de Rosemond et al, 2009) as cited in Islam et al (2016). The CCME WQI offers flexibility in parameter selection, allowing users to quickly adapt to local conditions and problems. This index is adaptable regarding the number and type of water quality parameters to be checked, the application period, and the type of water body (Canadian Council of Ministers of the Environment, 2017; Islam et al, 2016).

The index does not require sub-indices to be formed, weights to be established and contains no conventional index aggregation. Magnitude, scope, and frequency are the three significant factors required by the CCME WQI to generate a single non-dimensional value that is used to characterize the overall water quality status. The details of magnitude, scope and frequency are elaborated below. Dimensionless scores ranging between 0 and 100 are used to interpret the results, where 100 denotes that the water characteristics are similar to the benchmarks set, which means the water quality is excellent. (Tirkey et al, 2013).

The CCME WQI uses the harmonic square mean formula to calculate the final aggregate score.

The index is calculated in the following way:

$$CCME\ WQI = 100 - \frac{\sqrt{F_1^2 + F_2^2 + F_3^2}}{1.732} \qquad (2.1)$$

The three significant factors are calculated using the equations below:

F_1 (Scope) indicates the number of failed variables (variables with unmet goals), in relation to the overall number of variables assessed:

$$F_1 = \left(\frac{Number\ of\ failed\ variables}{Total\ Number\ of\ Variables}\right) x\ 100 \qquad (2.2)$$

F_2 (frequency) describes the number of occasions that the goals were not met:

$$F_2 = \left(\frac{Number\ of\ failed\ tests}{Total\ Number\ of\ Tests}\right)\ x\ 100 \tag{2.3}$$

F_3 (Amplitude) is calculated in three steps and shows the percentage of time when the objectives have not been met.

Calculation of F_3 follows three steps:

Step 1: Calculating the excursions:

When the objective must not be exceeded by the test value:

$$Excursion = \left(\frac{Failed\ Test\ Value}{Objective}\right) - 1 \tag{2.4}$$

When the objective must not exceed the test value:

$$Excursion = \left(\frac{Objective}{Failed\ Test\ Value}\right) - 1 \tag{2.5}$$

Step 2: Calculating the normalised sum excursion value (nse).

$$nse = \frac{\sum_i^n Excursion_i}{Total\ number\ of\ tests} \tag{2.6}$$

Step 3: Calculating the F_3 value:

$$F_3 = \left(\frac{nse}{0.01 \times nse + 0.01}\right) = \left(\frac{nse}{nse + 1}\right) \times 100 \tag{2.7}$$

There are five quality classes with ranges from 0 – 100 used to describe the WQI (Dao et al, 2020).

Table 2.1: CCME WQI Classification (Source: Khan et al, 2004)

Ranges	Classification	Explanation
0 – 44	Poor	Poor water quality
45 – 64	Marginal	Average water quality
65 – 79	Fair	Water quality is fair
80 – 94	Good	Water quality is good
95 – 100	Excellent	Excellent water quality

2.3.3.2. Improved Oregon Water Quality Index

The improved Oregon Water Quality Index (OWQI) was created to eliminate ambiguity and eclipsing. The OWQI helps to compare the temporal and geospatial characteristics at various locations in the water body. The Oregon department of environmental quality developed sub-index charts which inform the aggregation of nine sub-index values. The OQWI uses the unweighted harmonic square mean formula which allows for a value between 10 and 100 to be calculated depending on the water quality variables which have been measured. This method involves the formation of sub-indices for common water quality indicators (Brown et al, 1970). (Nyirenda and Tanyimboh, 2020) used this method to assess water quality on the Anytown Network. Prior to their study, the formulation of these sub-indices for usage in service reservoirs had never been done before, to the authors' knowledge. The authors found this approach to be appropriate for use in water distribution networks and service reservoirs.

The unweighted harmonic square mean formula is given as:

$$WQI = \sqrt{\frac{n}{\sum_{i=1}^{n} S_i^{-2}}} \tag{2.8}$$

Where S_i refers to the sub-index value;

WQI refers to the water quality index;

and *n* refers to the number of water quality variables.

(Swamee and Tyagi, 2000) proposed an improved way to eliminate ambiguity and eclipsing from the aggregation method. The method is similar to the OWQI in that the measured parameters are described using sub-indices for each river reach. The equation for aggregating the sub-indices is as follows:

$$WQI = \left[1 - n + \sum_{i+1}^{n} S_i^{-1/k}\right]^{-k} \tag{2.9}$$

Where the variables description is the same in equation (2.8) and *k* = 0.4

Equation (2.9) was further improved to propose another water quality index (Swamee and Tyagi, 2000). The improved index replaces the constant parameter of *k* and claims to eliminate rigidity.

Below is the formula for the proposed index:

$$WQI = \left[1 - n + \sum_{i+1}^{n} S_i^{-\log_2(n-1)}\right]^{-1/\log_2(n-1)} \tag{2.10}$$

Where the variables description is similar to equation (2.9). Water quality is described using values ranging between 0 and 100 as seen in Table 2.2.

Table 2.2: Water Quality Classification for OWQI

WQI Range	WQI Description
90 – 100	Water quality is excellent
85 – 89	Water quality is good
80 – 84	Water quality is medium/average
60 – 79	Water quality is fair
0 – 59	Water quality is poor

2.3.3.3. The National Sanitation Foundation Water Quality Index

One of the first WQIs, the National Sanitation Foundation (NSF) WQI, was created in the early 1970s (Brown et al, 1970). Since more than 100 water quality specialists from throughout the US were consulted during the construction of this index, the index gained credibility among other WQIs (Sutadian et al, 2016). Although it was developed in the US, the World Quality Index (WQI) or a modified version of it has been adopted by several countries, including Iran (MPCB, 2014), Brazil (Simões et al, 2008), and India (Mojahedi and Attari, 2009) (Sutadian et al, 2016). The Delphi method was employed by the NSF WQI to settle on a fixed set of parameters. Nine parameters (total solids (TS), turbidity, nitrates (NO_3), total phosphorus (TP), temperature, biochemical oxygen demand-5 days (BOD_5), pH, faecal coliform (FC) and dissolved oxygen (DO)) were chosen based on the agreement of water quality experts from around the US. The nine parameters were then expanded by two additional ones (pesticides and hazardous elements). The Delphi method was also used to establish the NSF WQI sub-indices. With the exception of pesticides and harmful substances, this data was then utilized to create "an average curve" that indicated the overall trend of all sub-indices. These two sub-indices were created using categorical 0 and 1 scaling. If both values are over the allowable limits, the water quality status is recorded as zero (the worst level) (Sutadian et al, 2016).

Another questionnaire was created using the Delphi method to determine specific weights for the chosen parameters. This process resulted in the following final weights, which are listed in brackets: TS (0.07), turbidity (0.08), NO_3 (0.10), TP (0.10), temperature (0.10), BOD_5 (0.11), pH (0.11), FC (0.16) and DO (0.17). All separate weights added together equal one (Sutadian et al, 2016). The additive method was developed to aggregate the sub-indices in the index that Brown et al. first proposed in 1970. Using the index revealed that, despite being simple to comprehend and compute, its mathematical formulation lacked sensitivity with respect to the impact of a single incorrect parameter value on the WQI (Lumb et al, 2011b). This prompted Brown et al. (1973) to suggest a modification of the NSF WQI that takes advantage of multiplicative aggregation. Table 2.3 indicates the interpretation of the final index value.

Table 2.3: Classification of water quality based on the NSF WQI

WQI Range	WQI Description
91 – 100	Water quality is excellent
71 – 90	Water quality is good
51 – 70	Water quality is medium/average
26 – 50	Water quality is fair
0 – 25	Water quality is poor

2.3.3.4. Weighted arithmetic mean WQI

The weighted arithmetic method for water quality index classification uses the most commonly measured water quality factors to assess the cleanliness of water. Several scientists (Balan et al, 2012; Rao et al, 2010; Chowdhury et al, 2012; Chauhan and Singh, 2010) have applied this approach extensively, and the WQI was calculated using equation (2.11) below:

$$WQI = \frac{\sum Q_i W_i}{\sum W_i} \tag{2.11}$$

Where Q_i is calculated using equation (2.12) and it represents the quality rating scale for each parameter in the water being analysed:

$$Q_i = 100\ [(V_i - V_o)/(S_i - V_o)] \tag{2.12}$$

where V_o is the parameter's ideal value in water,

V_i is the estimated concentration of the i^{th} parameter,

S_i is the i^{th} parameter's recommended standard value.

The following formula is used to calculate each water quality parameter's unit weight (W_i):

$$W_i = K/S_i \tag{2.13}$$

Where the following equation can be used to calculate the proportionality constant, K:

$$K = \frac{1}{\sum(1/S_i)} \tag{2.14}$$

The results are then interpreted using Table 2.4.

Table 2.4: Water quality classification for Weighted Arithmetic Mean WQI

WQI Range	WQI Description	Grading
91 – 100	Water quality is unsuitable for drinking purpose	E
71 – 90	Water quality is very poor	D
51 – 70	Water quality is poor	C
26 – 50	Water quality is good	B
0 – 25	Water quality is excellent	A

2.4. Water quality in water distribution networks

2.4.1. Overview of water quality in WDNs

The deterioration of water quality in WDNs is a major concern for the water industry today (Rossman et al, 1993; Clark and Grayman, 1998 as cited in Seyoum, 2015). Historically, assessing the quality of water was accomplished by comparing its values to the applicable local standards (Damo and Icka, 2013). Water samples are collected and tested at the laboratory and these results are compared to the standards. However, this process does not indicate temporal and spatial water quality variations within the WDS. It is essential to be able to evaluate the water quality within a WDS in real-time using the proper instruments. This is why the use of WQMs and WQIs is vital.

2.4.2. Water contamination in water distribution networks

During the operation of the water supply system, increased concentrations of various organometallic, inorganic and organic pollutants and chemicals with various microorganisms may result in deteriorated water quality, reduced flow or pressure, and water losses (Rakić, 2018). All these variations could be the result of the interactions of the water with the tubular and reinforced wall components as well as its own diverse physical, chemical, and biological reactions as it travels to the user from the water supply system. In order to reduce the impacts of contamination and to provide a form of protection against microbial growth while the water is being transported through the distribution system, a disinfectant is often applied after the water has been treated

(Rakić, 2018). Unfortunately, certain drinking water supplies have become polluted in several countries around the world with the source of contamination arising from multiple sources (Figure 2.2).

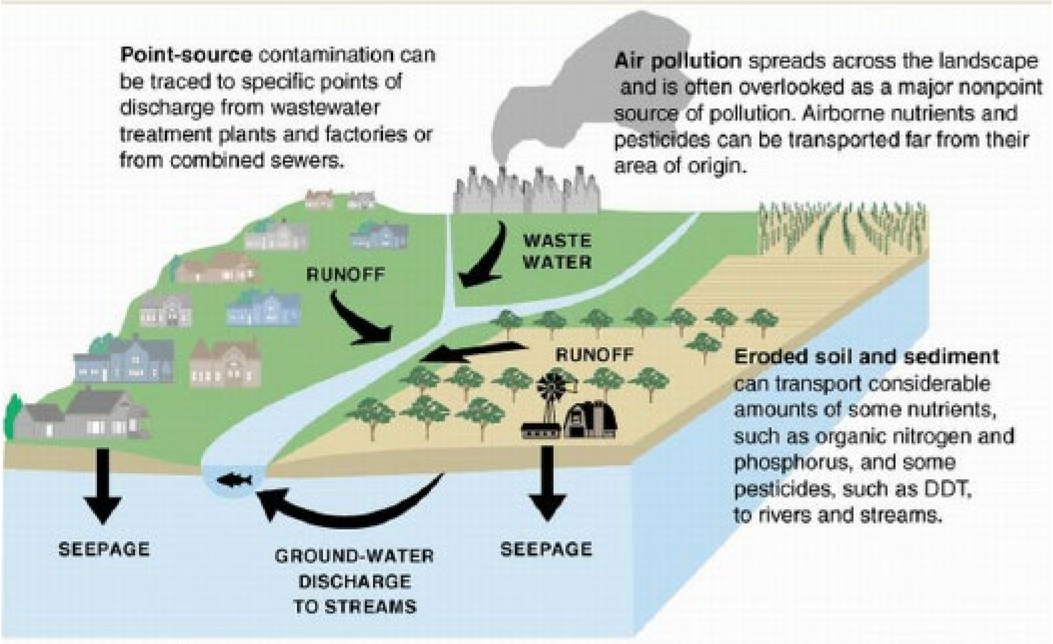
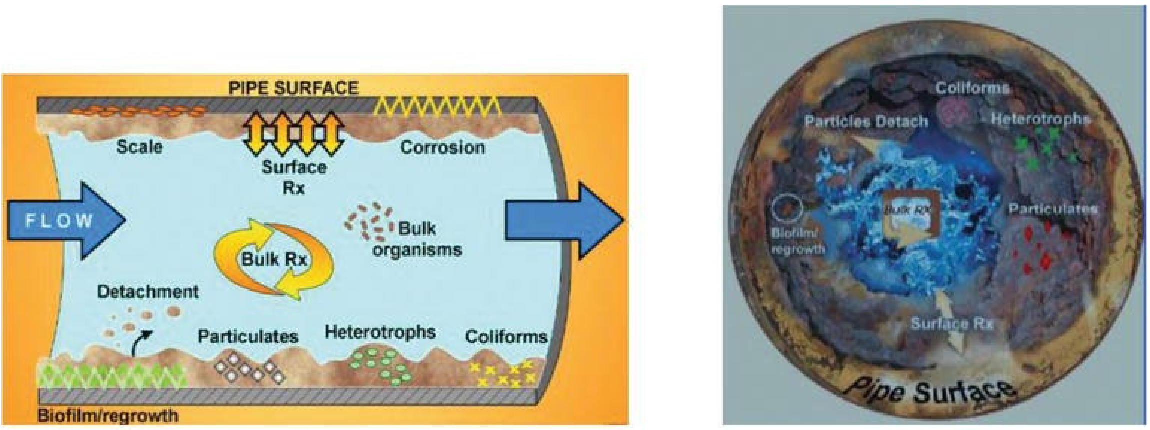


Figure 2.2: Potential sources of water contamination (adapted from Shober, 2009)

Changes in water quality can range in severity, such as the accumulation of deposits or sludge, which produces taste and foul smell that makes the water potentially unsafe to drink (Wang et al, 2014; Enning and Garrelfs, 2014). These phenomena depend on multiple variables, including sediment deposits on the walls (organic sediments, water solids and different oxide and oxyhydroxide products of corrosion), disinfectant, hardness, pH, temperature, usage dynamics, age and flow of the water, among other factors (Chu et al, 2005). Different corrosion products and deposits are formed inside the pipeline over time. The factors that have the greatest impact on these occurrences include the water's hardness, pH, temperature, and other chemical nutrients, as well as the pipeline material of the water distribution network (Bédard et al, 2015). Because of this, the water distribution network should be seen as a special biocenosis, an extremely complicated chemical-biological reactor where multiple connected reactions take place (Figure 2.3a and b) (Rakić, 2018).



(a) b)

Figure 2.3: Schematic illustration of a water pipe used as a chemical and biological reactor in longitudinal (a) and cross section (b) (EPA, 2008)

Corrosion increases the surface area that is available for microbial colonisation and biofilm formation in the water delivery system. The water age after leaving the treatment plant should be kept to a minimum, a range of locally predetermined disinfectant residual concentrations should be maintained, and pollutants should be prevented from entering the water supply network (Rakić, 2018). Water age, chlorine residual and Trihalomethane (THM) are the variables that are of primary interest in this study. The most common factors in the water supply system that lead to drinking water contamination include corrosion of fittings and metal pipes (Little et al, 2014) and biofilm precipitation on the interior of water pipes (Wang, 2015). This is why it is important to consider global bulk wall coefficients during the water quality modelling process.

2.4.3. Water quality indices in water distribution networks

To quantify pollution loads in a river or stream, scientists employ water quality indices, which have traditionally been represented by a single aggregated value (Rathnayake & Tanyimboh, 2015; Madalina & Gabriela, 2014; Cude, 2001; CCME, 2001). In a water distribution network, service reservoirs function similarly to rivers in that they have complicated mixing mechanisms, are subject to a variety of water quality factors, and are sized and located differently. The water quality index can be used to characterise the water quality status in a service reservoir by adding the values of all water quality variables that have been measured or calculated (Nyirenda and Tanyimboh, 2020).

This is beneficial in situations when competing water distribution network designs are given and must be evaluated. At each step of an optimization process, the water quality index can be utilised for assessing the adequacy of the service reservoirs. Otherwise, as the number of water quality parameters increases, many candidate solutions may become non-dominated too early in the optimization process, which might increase the risk of premature convergence and computing complexity (Saleh & Tanyimboh 2014, 2016; Sinha et al, 2013; Saxena et al, 2013 as cited by Nyirenda and Tanyimboh, 2020). For assessing water quality in each reservoir, Nyirenda and Tanyimboh (2020) developed sub-indices for three widely used water quality parameters on the Anytown Network (Walski et al, 1987). Sub-index charts for THM, chlorine and water age, the three most common water quality indicators in WDNs, were developed by the authors.

2.4.4. CCME WQI in water distribution networks

Similar to the OWQI and the NSF WQI, the CCME WQI has been widely applied to evaluating the water quality of rivers and streams (Shah and Joshi, 2017). To the author's knowledge, this index has never been used in water quality modelling for design optimisation using models like EPANET. Literature shows that this index has been used in existing WDNs, where samples have to be taken at specific points within the WDN over a specific period of time. It has not been used in cases where quantitative data must be obtained from a WQM like EPANET and be utilised for calculating the final index score. Complex water quality data can be aggregated in a simplified manner provided by the CCME WQI. It does not require the formulation of sub-indices, aggregation of sub-indices and development of weights. This means that water quality variables can be used as they are with their varying dimensions and units, without the user having to create sub-indices and weights for each water quality parameter. This simplifies the process of addressing high dimensionality in water quality modelling, as opposed to the other methods used by Nyirenda and Tanyimboh (2020).

The CCME WQI allows flexibility in the number of variables used, making it suitable for use in water quality modelling by using water quality variations obtained from the EPANET software. Unlike physical water sampling, WQMs are remote and can be used during design stage where actual water quality data is not yet available.

This is the part of the reasons why the Anytown Network is suitable for this study. This network is explained further in Chapter 3 of this study. The Anytown network is a hypothetical network which was created by Walski et al (1987) for design optimisation purposes. The design optimisation solution should meet future demands while not violating the given pressure requirements. This network is to supply water at the specified requirements while being subjected to 5 different loading conditions. The Anytown network is a good representation of a network that is still under design stage and thus provides a baseline for water quality decisions taken during the design stage. This will assist with design optimization.

CHAPTER 3: METHODOLODY

3.1. Area of study

The “Anytown” network (Figure 3.1) was selected for this study. Walski et al (1987) created this hypothetical network for design optimization purposes. At the battle of the network models workshop, participants were given a hypothetical water distribution system (The Anytown network) to solve, with the main challenge being to optimise the network to meet future demands at the lowest possible cost. The participants also had to ensure that the system does not violate the minimum pressure requirements. Water quality was not considered at that time. The Anytown network is most likely the only benchmark that can be found in the known body of research that involves several loadings in addition to various storage tanks and pumps. While these are all common characteristics of real-world WDSs, few works have been published that simultaneously combine demand variations, multiple operating conditions and the sizing and functioning of tanks and pumps. For completeness, a quick summary of the network is provided here, however, additional information has been included in the supporting documents (Appendix A).

The network was subjected to five loading conditions (Appendix A-3), namely, instantaneous peak flow, average day demand and three fire scenarios. The average day flow and the instantaneous peak flow, which is 1.8 times the average day flow, must be provided at all nodes at a minimum pressure of 28.12 m. For the three crucial fire scenarios, a minimum pressure of 14.06m should be satisfied at all nodes. The network consists of two existing tanks (nodes 14 and 17), three parallel pumps (node 20) with identical characteristics and forty-one pipe variables that are available for pipe paralleling, cleaning, and lining or replacement. Through three identical pumps connected in parallel, with characteristics of each pump presented in Appendix A-5a, water is pumped into the network from the treatment plant located at node 40 (Figure 3.1). To put out a 2-hour fire and satisfy peak flow demands that are 1.3 times the daily average flow, all tanks must start at their lowest operational levels with one pump out of service. The optimised design allows for the addition of up to two more tanks. New tanks can be located at any of the available nodes, provided that the node does not have an existing tank. Two extra diameter decision variables are provided by the riser that connects the tanks to the node; this riser has a variable diameter but a known length of 30.78 m. Additional pumping stations are not permitted; however, the allowable maximum upgrades to an existing pumping station could consist of the addition of two new pumps that have the same characteristics as the existing pumps.

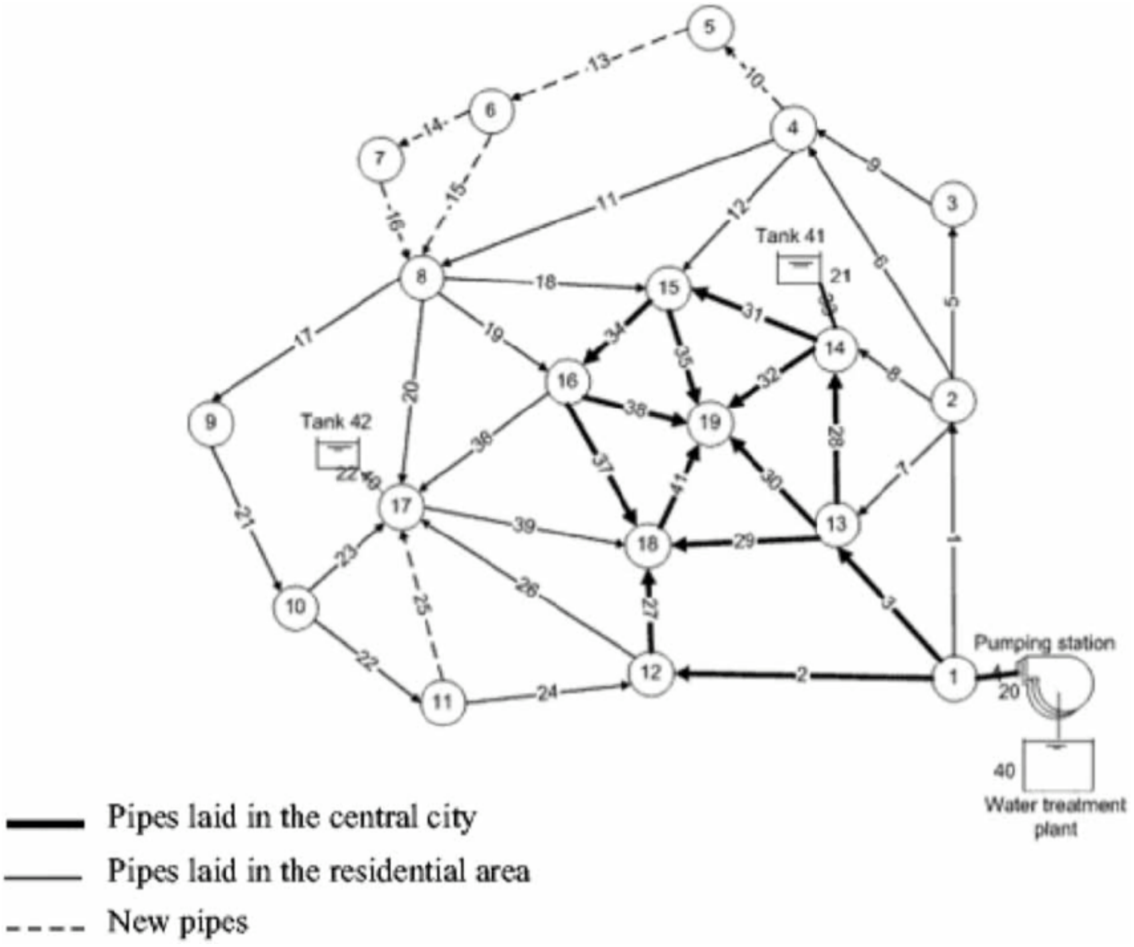


Figure 3.1: Anytown network (Walski et al, 1987)

3.2. Hydraulic Analysis

Since the inception of the battle of the network models workshop, more authors have attempted to solve and optimise the Anytown network. The cheapest solutions (Prasad, 2010; Siew et al, 2016; Walters et al, 1999; Vamvakeridou-Lyroudia et al, 2005) were adopted for this study. To ensure correlation with the results obtained by the authors mentioned, hydraulic analysis was performed on the Anytown network. Hydraulic simulations were performed on the Anytown network using the EPANET 2.2 model that incorporated a 1-minute time step. The Extended Period Simulation (EPS) was carried out over a duration of 24hours, the Steady-State Simulation (SSS) was carried out for the instantaneous peak flow and an EPS of 2 hours was adopted for the fire flow scenarios. The EPANET simulations took an average of 3 seconds to run the hydraulic analysis. Hazen-Williams formula was used for the hydraulic calculations where the coefficient of friction (C) was 130 for all new pipes, 125 for cleaning and lining and 120 for all existing pipes. The actual daily water use pattern is given in Appendix A-2.

3.2.1. Solution 1 (Siew et. al, 2016)

Siew (2011) created a Penalty-Free Multi-Objective Evolutionary Approach (PFMOEA) to solve the Anytown network which was later improved by Siew et al (2016). Their solution, denoted as Solution 1, is reproduced in Table 3.1.

Table 3.1: Solution 1 Rehabilitation and Upgrade (Siew et al, 2016)

Recommendation	Pipe ID	Diameter (m)	Length (m)
Pipe paralleling	1	0.3556	3657.6
	2	0.6096	3657.6
	20	0.4064	1828.8
	23	0.3556	1828.8
	26	0.6096	1828.8
Pipe cleaning and lining	40		304.8
New Pipes	10	0.3556	1828.8
	13	0.1524	1828.8
	14	0.1524	1828.8
	15	0.254	1828.8
	16	0.3556	1828.8
	25	0.1524	1828.8
	Riser for Tank 7N	0.4064	30.78

To prevent misunderstandings, it is important to note that the original "Anytown" network problem was developed using imperial units. Therefore, the diameters in Table 3.1 have been converted from inches to metres, and do not necessarily reflect the use of standard pipe diameters. This information was required to carry out the hydraulic simulation of Solution 1 by Siew et al (2016).

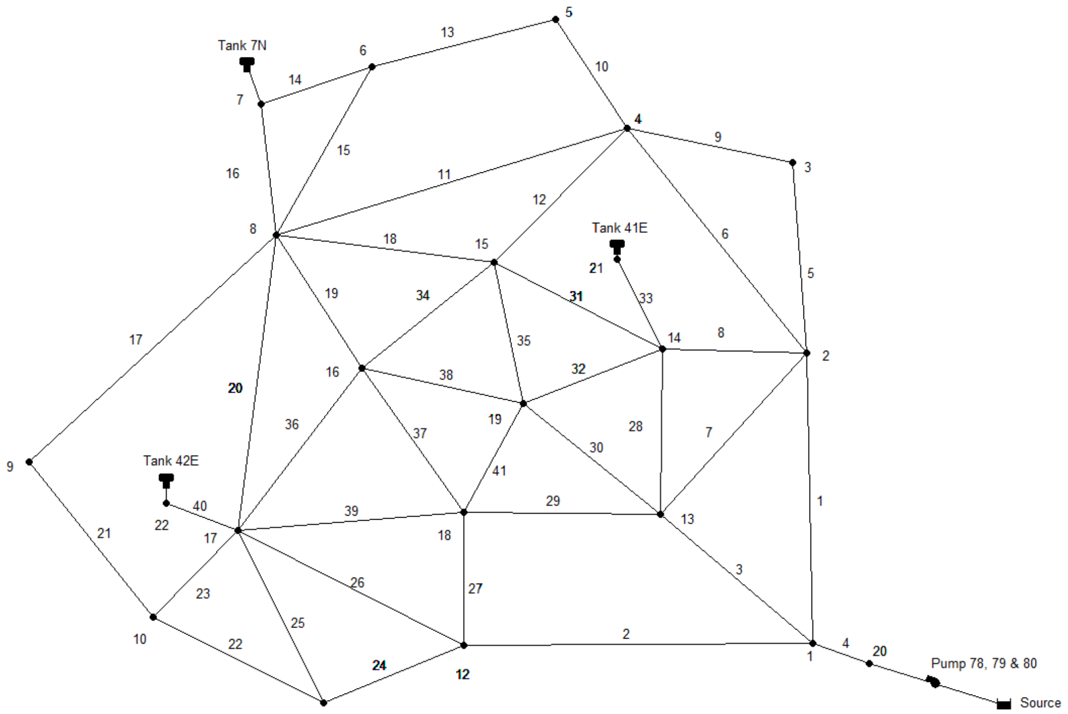


Figure 3.2: Solution 1 (Siew et al, 2016)

Table 3.2: Tank 7N Properties

Tank Properties	
Minimum operating level (m)	67.18
Maximum operating level (m)	72.98
Bottom level (m)	60.96
Top level (m)	74.31
Diameter (m)	18.67
Tank location (Node ID)	7
Volume (m^3)	3409.188

Table 3.3: Pump operation schedule for Solution 1 (Siew et al, 2016)

Number of pumps operating				Energy consumed (kWh/day)
06:00 – 09:00	09:00 – 15:00	15:00 – 18:00	18:00 – 06:00	
2	3	3	2	18 733.50

Table 3.4: Recommendations for solution 2 (Siew et al, 2016)

Recommendation	Pipe ID	Diameter (m)
Pipe paralleling	2	0.6096
	4	0.2032
	17	0.2032
	20	0.6096
	26	0.6096
Pipe cleaning and lining	3	
New Pipes	10	0.1524
	13	0.254
	14	0.2032
	15	0.4572
	16	0.2032
	25	0.2032
	Riser for Tank 6N	0.3048

The simulated results for this design optimization solution are presented and discussed in Chapter 4 of this report.

3.2.2. Solution 2 (Siew et al, 2016)

Siew et al (2016) further proposed another solution which incorporates a tank depletion strategy. The new tank for this solution is located at node 6. The information used for hydraulic simulation of this network is as detailed below:

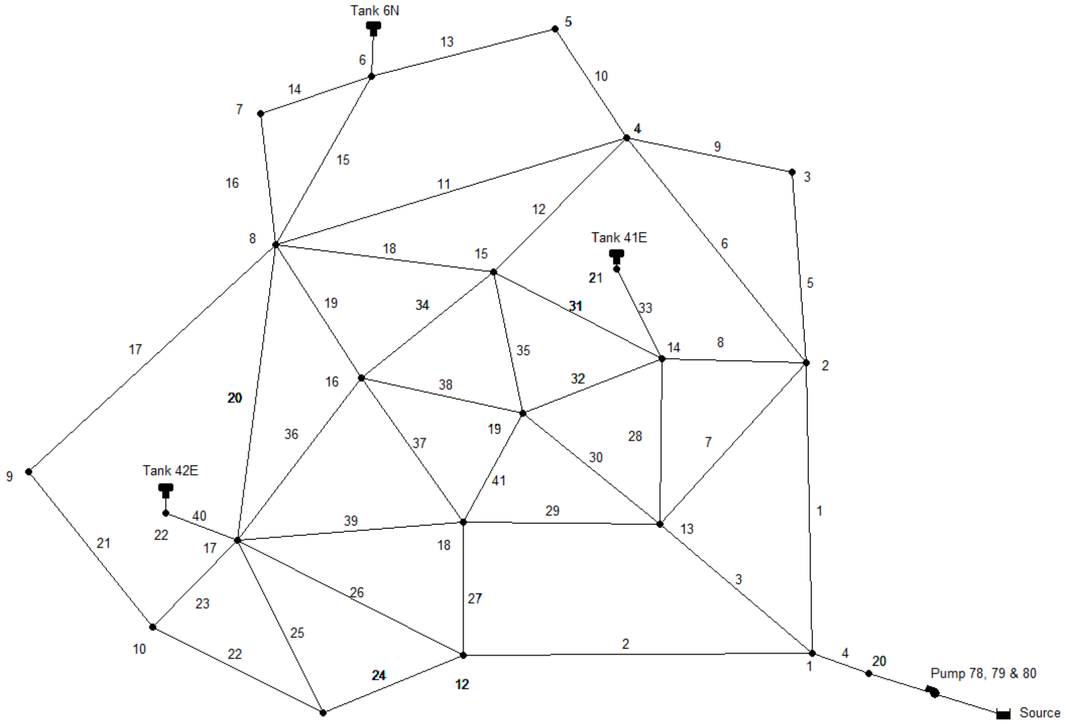


Figure 3.3: Solution 2 (Siew et al, 2016)

Table 3.5: Tank 6N Properties

Tank Properties	
Minimum operating level (m)	66.56
Maximum operating level (m)	72.98
Bottom level (m)	60.96
Top level (m)	74.31
Diameter (m)	18.67
Tank location (Node ID)	6
Volume (m^3)	3409.188

Table 3.6: Pump operation schedule for Solution 2 (Siew et al, 2016)

Number of pumps operating				Energy consumed (kWh/day)
06:00 – 09:00	09:00 – 15:00	15:00 – 18:00	18:00 – 06:00	
3	3	2	2	19 017.92

3.2.3. Prasad (2010) solution

One of the cost-effective solutions to the Anytown network problem was provided by Prasad (2010). The Non-dominated Sorting Genetic Algorithm (NSGA II) was utilised by Prasad (2010) for design optimisation of the Anytown network. The information used to perform the hydraulic simulation for this network is as stipulated below:

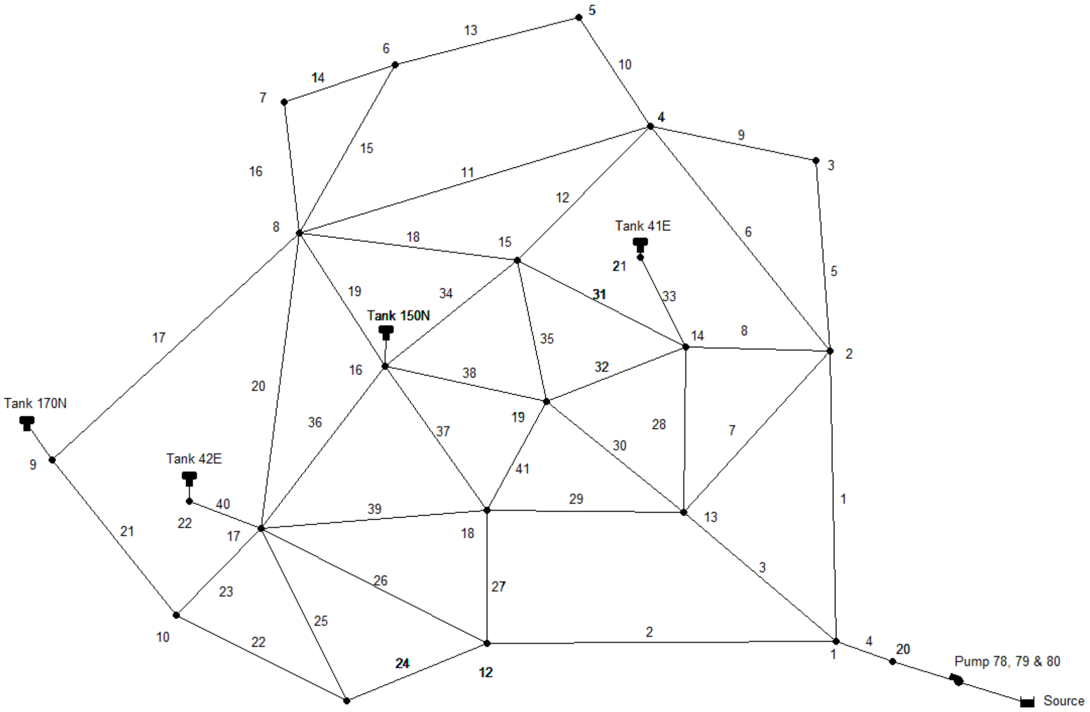


Figure 3.4: Prasad (2010) solution

Table 3.7: Pump operation schedule for Prasad (2010) solution

Number of pumps operating			
06:00 – 09:00	09:00 – 15:00	15:00 – 18:00	18:00 – 06:00
3	3	3	2

Table 3.8: New Tank properties (Prasad, 2010)

Tank Properties		
	Tank 150N	Tank 170N
Minimum operating level (m)	69.93	67.33
Maximum operating level (m)	74.34	72.02
Bottom level (m)	63.40	63.70
Top level (m)	75.56	72.93
Diameter (m)	14.51	12.06
Tank location (Node ID)	16	9

Table 3.9: Recommendations for Prasad (2010) solution

Recommendation	Pipe ID	Diameter (m)	Length (m)
Pipe paralleling	1	0.4064	3657.6
	2	0.6096	3657.6
	20	0.4572	1828.8
	24	0.254	1828.8
	26	0.508	1828.8
	40	0.1524	30.48
New Pipes	10	0.3048	1828.8
	13	0.1524	1828.8
	14	0.2032	1828.8
	15	0.254	1828.8
	16	0.3048	1828.8
	25	0.1524	1828.8
	Riser for Tank 170N	0.254	30.78
	Riser for Tank 150N	0.3556	30.78

3.2.4. Walters et al (1999) solution

The Structured Messy Genetic Algorithm (SMGA) was utilised by Walters et al (1999) to optimise and solve the Anytown network problem. To carry out the hydraulic simulation for this solution, the following information was adopted:

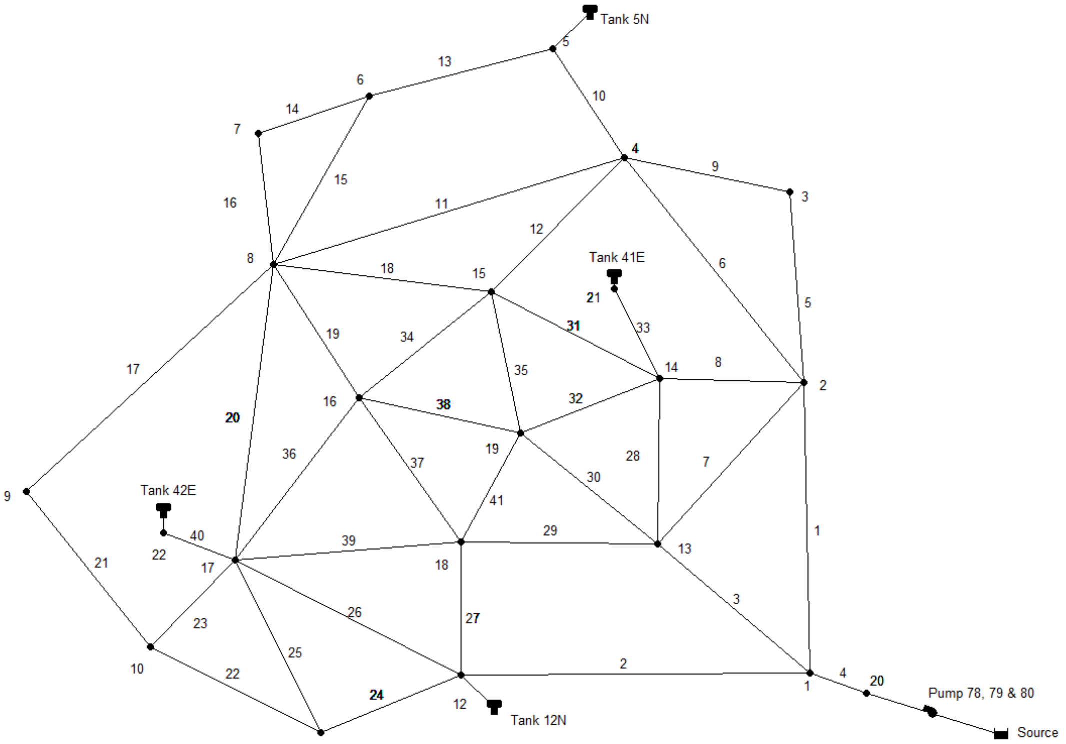


Figure 3.5: Walters et al (1999) solution

Table 3.10: Pump operation schedule for Walters et al (1999) solution

Number of pumps operating			
06:00 – 09:00	09:00 – 15:00	15:00 – 18:00	18:00 – 06:00
3	3	3	2

Table 3.11: Recommendations for Walters et al (1999) solution

Recommendation	Pipe ID	Diameter (m)	Length (m)
Pipe paralleling	2	0.6096	3657.6
	4	0.762	30.48
	20	0.508	1828.8
	21	0.3048	1828.8
	23	0.4064	1828.8
	26	0.508	1828.8

	33	0.2032	30.48
	39	0.254	1828.8
	40	0.4572	304.8
	Riser for Tank 41E	0.2032	30.48
	Riser for Tank 42E	0.4572	30.48
New Pipes	10	0.3048	1828.8
	13	0.254	1828.8
	14	0.2032	1828.8
	15	0.1524	1828.8
	16	0.3556	1828.8
	25	0.1524	1828.8
	Riser for Tank 55N	0.4572	30.48
	Riser for Tank 110N	0.254	30.48

N = New Tank; E = Existing Tank

Table 3.12: New Tank Properties (Walters et al, 1999)

Tank Properties		
	Tank 5N	Tank 12N
Minimum operating level (m)	57.61	73.46
Maximum operating level (m)	62.18	76.50
Bottom level (m)	51.51	72.24
Diameter (m)	19.48	21.79
Tank location (Node ID)	55	110

3.2.5. Vamvakeridou-Lyroudia et al (2005) Solution

Vamvakeridou-Lyroudia et al (2005) proposed new tanks at nodes 9 and 16, and a solution similar to that of Prasad (2010) solution. The information required to carry out the hydraulic simulation is presented below:

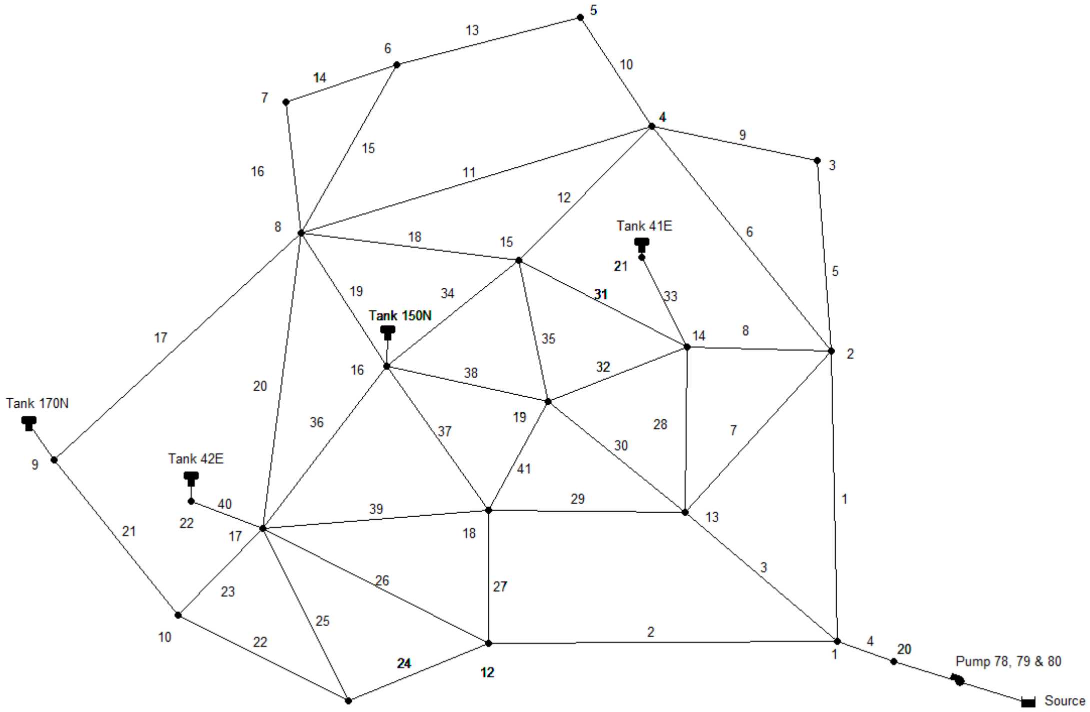


Figure 3.6: Vamvakeridou-Lyroudia et al (2005) solution

Table 3.13: New Tank properties (Vamvakeridou-Lyroudia et al, 2005)

Tank Properties		
	Tank 150N	Tank 170N
Minimum operating level (m)	70.25	66.00
Maximum operating level (m)	74.35	70.05
Bottom level (m)	65.55	62.10
Top level (m)	75.80	71.10
Diameter (m)	15.94	9.85
Tank location (Node ID)	16	9

Table 3.14: Recommendations of Vamvakeridou-Lyroudia et al (2005) solution

Recommendation	Pipe ID	Diameter (m)	Length (m)
Pipe paralleling	2	0.762	3657.6
	20	0.457	1828.8
	21	0.203	1828.8
	26	0.508	1828.8

	27	0.254	1828.8
	33	0.203	30.48
	40	0.356	30.48
Pipe cleaning and lining	3		3657.6
New Pipes	10	0.305	1828.8
	13	0.152	1828.8
	14	0.152	1828.8
	15	0.305	1828.8
	16	0.254	1828.8
	25	0.152	1828.8
	Riser for Tank 170N	0.305	30.48
	Riser for Tank 150N	0.305	30.48

Table 3.15: Pump operation schedule for Vamvakeridou-Lyridou et al (2005) solution

Number of pumps operating				Energy consumed (kWh/day)
06:00 – 09:00	09:00 – 15:00	15:00 – 00:00	00:00 – 06:00	
2	3	2	2	19 431.81

3.3. Water Quality Analysis

The most feasible solutions from the hydraulic analysis were carried over to the water quality simulations. One of the solutions (Vamvakeridou-Lyroudia et al, 2005) was found to be infeasible as it violated the minimum pressure requirements. These results are further discussed in Chapter 4 of this report. Water quality simulations were carried out on four of the most feasible solutions (Solutions 1 and 2, Siew et al, 2016; Prasad, 2010 and Walters, 1999). The water quality simulations were performed on the same computer as the hydraulic analysis in 3.2. The preferred software for the water quality simulation was EPANET 2.2. Other similar software (e.g. EPANET MSX) could be substituted readily if required. While EPANET 2.2 can only simulate one parameter at a time, EPANET-MSX (Shang et al, 2008) can model THM, water age and chlorine residual content simultaneously. It should be noted that EPANET-MSX does not possess hydraulic modelling abilities due to its network analysis model being interdependent. It instead utilises the typical EPANET dynamic link library for the required hydraulic assessments, the results of which are not readily available. EPANET 2.2 was selected based on its user-friendly interface. A 1-minute time step was used for both the hydraulic and water quality simulations. A duration of 72 hours was used for the water quality EPS to allow the results to stabilise and a pattern to be established.

Chlorine, THM and water age are the three water quality parameters that were simulated. There are two main reasons for selecting these variables, one of them being the limitations of the EPANET model. Only age, trace and chemical properties can be simulated using the EPANET model. In treated water, chlorine residual is of paramount importance as it is a measure of the potability of water. With the use of chlorine as a disinfectant comes the need to assess disinfection by-products like THM in the system. EPANET does not have the ability to simulate physical and biological parameters. It is also important to note that the results for physical parameters can change based on the physical conditions on site. In terms of chemical properties, it was possible to simulate chlorine because the initial concentration could be assumed at the source. A study to confirm what other chemical properties can be simulated using EPANET would have to be conducted as current literature shows chlorine simulation only. The EPANET 2.2 manual also focuses on how to simulate chlorine in a WDS.

The second reason the study continued with these three parameters is that the use of three variables did not prohibit the achievement of the study objectives. The three parameters and their associated variations over time represent multiple dimensions that have been evaluated by the CCME WQI as a single dimensionless value without converting any of the dimensions or creating weights and/or sub-indices. Should more parameters be added, the same process for calculating the CCME WQI would still be followed.

Chapter 4 of this report focuses solely on the outcomes obtained during the final 24 hours. Based on first-order kinetics model, the global bulk wall coefficient was taken as 0,5 day^{-1} and the global wall coefficient was taken as 0,1 m/day (Seyoum et al, 2014). In order to maintain a minimal chlorine residual concentration of 0,2 mg/L (WHO, 2017) at the network's most remote sites, it was presumed that the treatment plant would maintain a steady chlorine concentration of 0.6 mg/L. In a “real” network, it might be important to find out what chlorine concentration is applied by the authorities at the source/ water treatment plant. This might have to change each time based on the area for which the analysis is being carried out and the water source used. DWAF (1996) recommends a minimum chlorine residual level between 0.3mg/L and 0.6mg/L. At each node, the initial THM concentration was taken to be zero. A limiting concentration of 100µg/L was adopted as per the UK and EU standards for drinking water and South African water quality guidelines (DWAF, 1996). The US EPA and WHO standards (WHO, 2017) are stricter; with a maximum THM concentration of 80 µg/L and 60µg/L, respectively. In fact, the EU and US regulations recommend aiming for a lower value whenever possible without compromising disinfection. It is uncertain whether a desirable water age range exists as it is mainly used as an indirect indicator of water quality. 48 hours was adopted as the maximum allowable water age for this study. According to the guidelines of the United States Environmental Protection Agency, the water age must not exceed three days (AWWA, 2002). Within the four results that the optimisation algorithms offered, a total of 14 reservoirs were investigated. EPANET 2.2 required 4 seconds on average to simulate chlorine and 3 seconds to analyse water age and THM. Since chlorine decay contains wall reaction component that requires a longer simulation period, chlorine has an overall longer simulation duration.

3.4. Calculation of the CCME WQI

Chlorine residual, THM and water age are the three variables that were assessed in this study. To determine the CCME WQI for each service reservoir, the water quality simulation results for the last 24 hours were used. The results for each hour in the last 24 hours of the analysis were recorded and used to calculate the CCME WQI for each reservoir. Although the CCME WQI was not calculated for each demand node, the results for all the demand nodes have been reported in Chapter 4 and these results have been compared to the standards applicable for drinking water (WHO, 2017; US and EU drinking water standards). For each service reservoir, all three variables were analysed, spatial and temporal variations were noted and all key factors for calculating the CCME WQI were recorded.

To calculate the CCME WQI, three significant factors were required (F_1, F_2, and F_3); which were discussed in detail in chapter 2. With a reporting time step of 1 hour over a period of 24 hours for each of the variables analysed, this amounts to a total of 72 tests carried out. The results at each hour were compared to the drinking water standards/limits available to record failed variables and ultimately calculate the values of F_1 and F_2 and F_3. Table 3.16 indicates the objectives applied on this study based on the water quality guidelines/standards available. Any value outside of these limits is considered a failure. The appendices (Appendices B – E) contain the outcomes of the water quality simulation and the mathematical calculation of the CCME WQI. These results are further discussed in Chapter 4 of this study.

Table 3.16: Objectives for the three measured variables

Variable	Objective	Reference
Chlorine residual	0,2 mg/L	WHO, 2017
THM	100 µg/L	DWAF (1996); EC (1998) and HMG (2001; 2010)
Water age	48 hours	Conservative estimate based on AWWA (2002)

CHAPTER 4: RESULTS AND DISCUSSION

4.1. Overview

The results from the hydraulic simulations were used to determine which solutions were feasible and could be further processed for the water quality simulations. A correlation of $R^2 = 0.999999$ was achieved between the simulated results from this study and those obtained by the other authors found in the literature. This was important to validate the hydraulic simulation results and to ensure that the applied data are as specified by the authors. Any solution that violated the minimum pressure requirements was disqualified and there was no water quality analysis carried out on that solution. As mentioned in the previous chapter, a minimum pressure of 28.12m was required for the average flow and instantaneous peak flows; and 14.06m was the required minimum pressure for the fire flow conditions. When designing and optimizing a network, there are other factors to be considered besides residual head. Design guidelines usually specify velocity constraints to be adhered to within the network, however, this was not part of this study and velocity constraints were therefore not considered. This chapter gives a summary of the outcomes for discussion; however, the full results are provided in detail in the appendices (Appendix B – E).

4.2. Hydraulic analysis results

4.2.1. Results for Solution 1 (Siew et al, 2016)

Tables 4.1 shows the minimum pressures obtained from this solution as well as the critical nodes. This is the cheapest solution found in literature to date and it satisfied all pressure requirements. One of the disadvantages of this solution is that it does not provide a tank depletion strategy, which results in water in the tanks not draining completely throughout the day. This is not good from a water quality perspective. The results show that the hydraulic performance for this solution is good.

Table 4.1: Minimum pressures applicable to the different loading conditions

	Residual heads at the critical nodes (m)				
	Average day flow	Instantaneous peak flow	Fire flow 1	Fire Flow 2	Fire flow 3
Solution 1 (Siew et. al, 2016)	28.96 (16)	28.19 (9)	15.16 (16)	16.7 (16)	22.5 (11)
Achieved results (current study)	28.95 (16)	28.20 (9)	15.16 (16)	16.74 (16)	22.58 (11)

4.2.2. Results for Solution 2 (Siew et al, 2016)

This solution contains a tank depletion strategy and is the preferred solution in terms of hydraulic performance. According to the literature, it is one of the most economical alternatives available for the Anytown Network. However, it was also important to assess how it would perform in a water quality analysis to qualify as the most preferred solution based on both hydraulic and water quality performance. Table 4.2 indicates the minimum pressures obtained from this solution and the critical nodes.

Table 4.2: Minimum pressures applicable to the different loading conditions

	Residual heads at the critical nodes (m)				
	Average day flow	Instantaneous peak flow	Fire flow 1	Fire Flow 2	Fire flow 3
Solution 2 (Siew et. al, 2016)	28.29 (16)	29.91 (9)	17.10 (16)	16.61 (7)	21.66 (9)
Achieved results (current study)	28.29 (16)	29.91 (9)	17.10 (16)	16.61 (7)	21.66 (9)

As can be observed from the results above, there was correlation between the results obtained by Siew et al, (2016) and those obtained from the current study. This solution was assessed further in terms of water quality analysis and calculation of the CCME WQI.

4.2.3. Results for Prasad (2010) solution

Prasad (2010) did not originally publish the results obtained from the hydraulic simulation, however, the results obtained from this current study have been summarised below. This solution also met the minimum pressure requirements and was analysed further for water quality analysis.

Table 4.3: Minimum pressures applicable to the different loading conditions

	Residual heads at the critical nodes (m)				
	Average day flow	Instantaneous peak flow	Fire flow 1	Fire Flow 2	Fire flow 3
(Prasad, 2010)	No results published.				
Achieved results (current study)	29.52 (9)	28.51 (5)	27.75 (16)	15.86 (6)	28.28 (10)

4.2.4. Results for Walters et al (1999) solution

This was the final feasible solution in terms of hydraulic performance out of the 5 solutions that were analysed. The outcomes thereof are indicated in Table 4.4.

Table 4.4: Minimum pressures applicable to the different loading conditions

	Residual heads at the critical nodes (m)				
	Average day flow	Instantaneous peak flow	Fire 1	Fire 2	Fire 3
(Walters, 1999)		28.58 (11)	29.35 (16)	26.44 (6)	25.57 (11)
Achieved results (current study)	29.31 (9)	28.58 (11)	29.36 (16)	26.44 (6)	25.60 (11)

4.2.5. Results for Vamvakeridou-Lyroudia et al (2005)

This solution was found to be infeasible because it violated the pressure requirements for the Anytown network. Nodes 5 and 11 had minor pressure deficiencies when analysed for average day flow. This was attributed to the fact that the original design optimization for this solution was carried out using a larger time step of 3 hours. Node 11 also has a new tank connected to it based on the optimization plan. The current study used a time step of 1 minute. Vamvakeridou-Lyroudia et al (2005) also did not assess the performance of the network during average day loading conditions because they assumed that if the network performed well during instantaneous peak flow conditions, then it should be feasible even during average day flow conditions. Prasad (2010) proved that this assumption was incorrect in their study. This solution was not analysed further for water quality performance.

Table 4.5: Minimum pressures for the various loading conditions

	Residual heads at the critical nodes (m)				
	Average day flow	Instantaneous peak flow	Fire flow 1	Fire Flow 2	Fire flow 3
(Vamvakeridou-Lyroudia et al., 2005)					
Achieved results (current study)	28.08 (11)	14.16 (16)	24.08 (11)	24.01 (11)	23.80 (11)

4.3. Results for water quality analysis

4.3.1 Overview

The variations in disinfection by-products, water age, chlorine residual from the EPANET model were investigated. The water quality simulations were conducted for 72 hours, assuming that complete mixing takes place in all tanks. The results for the last 24 hours are discussed herein. This study was primarily for design optimization and decision-making purposes with the focus being mainly on water quality in the service reservoirs. Even though the water quality at the demand nodes is mentioned here, the CCME WQI was not calculated at the demand nodes. Siew et al (2016) and Walters et al (1999) sited new tanks further from the demand centre while Prasad (2010) placed new tanks closer to the existing tanks.

4.3.2. Analysis of water quality at demand nodes

A summary of the worst case observed from the four solutions is presented in Table 4.6. From the results, all the solutions except Walters et al (1999) achieved acceptable water quality levels. Chlorine concentrations were above 0,2mg/L at all the nodes for all the solutions, except for Walters et al (1999) which recorded values of 0,18 and 0,19 for two consecutive hours. Node 5 had the worst performing results in the Walters et al (1999) solution and this could be attributed to the new Tank 5N that is connected to this node. This node is also remote from the treatment plant. The nodes in Solution 2 achieved the lowest values for THM and water age, and the highest value for chlorine. Figures 4.1 – 4.12 show the daily variations in water quality at demand nodes for the four solutions.

Table 4.6: Water quality outcomes at demand nodes (worst-case scenario)

Worst Case Results								
	Solution 1 (Siew et al., 2016)		Solution 2 (Siew et al., 2016)		Prasad (2010)		Walters et al. (1999)	
	Value	Node	Value	Node	Value	Node	Value	Node
Min. Chlorine (mg/L)	0.26	7	0.31	9	0.25	9	0.18	5
Max. THM (µg/L)	45.03	7	28.6	6	37.1	16	53.7	5
Max. Water age (hours)	31.1	7	21.2	6	26.0	16	38.7	5

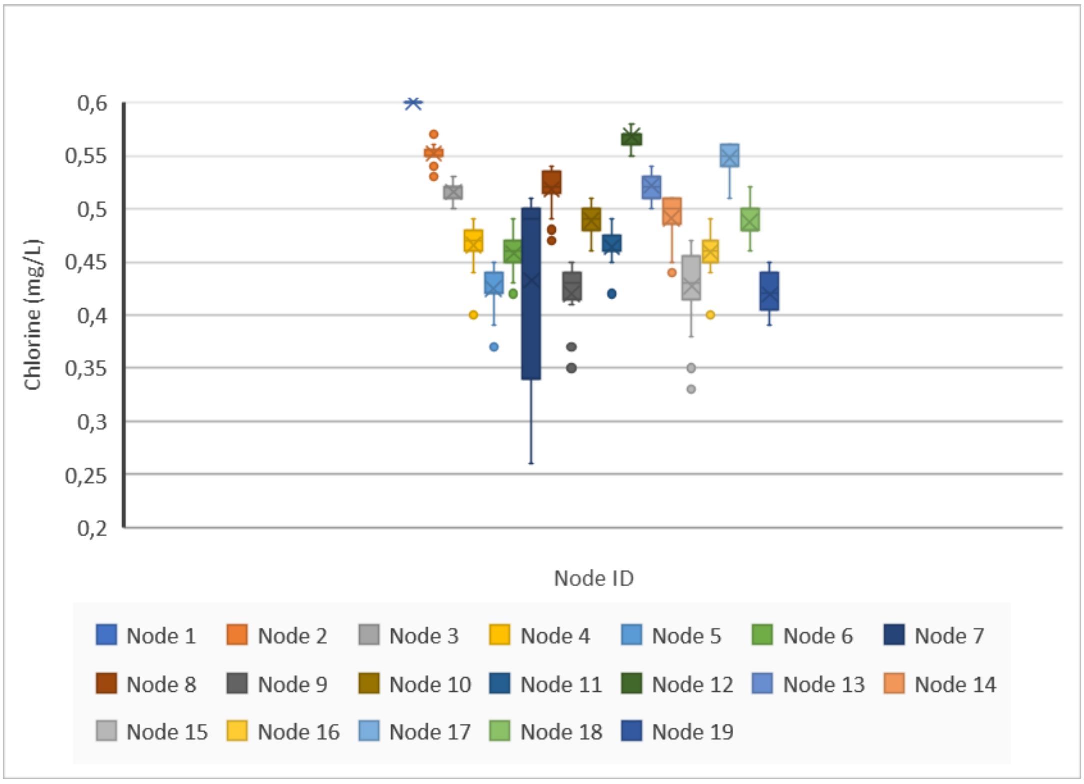


Figure 4.1: Daily chlorine concentration variations at demand nodes (Solution 1)

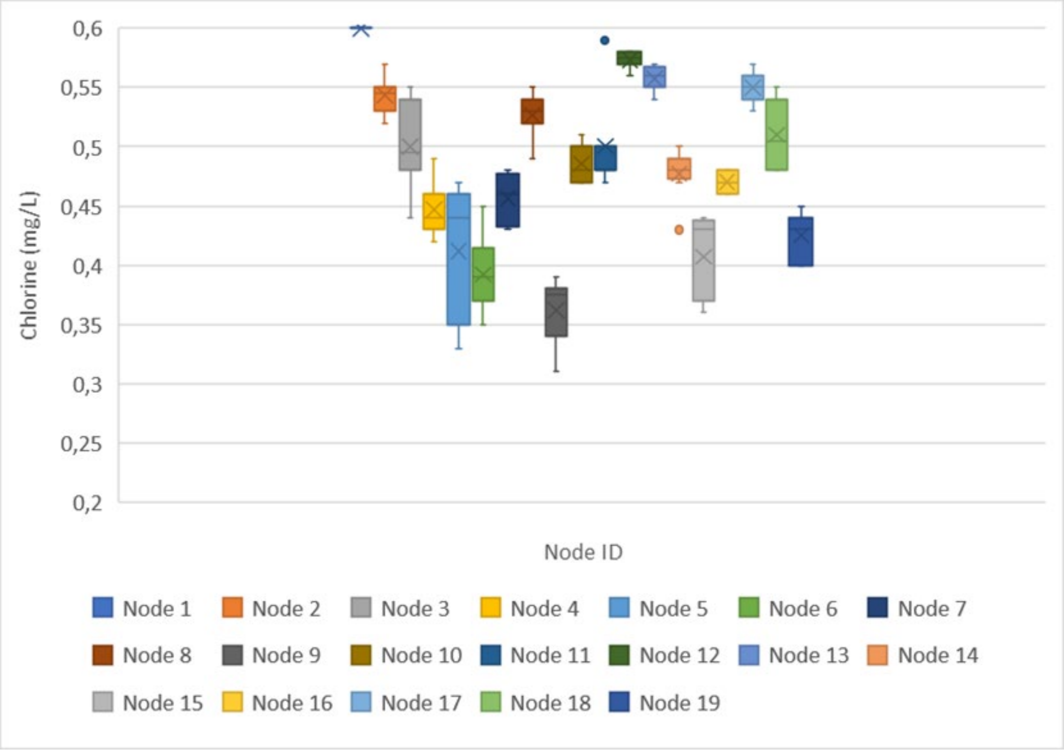


Figure 4.2: Daily chlorine concentration variations at demand nodes (Solution 2)

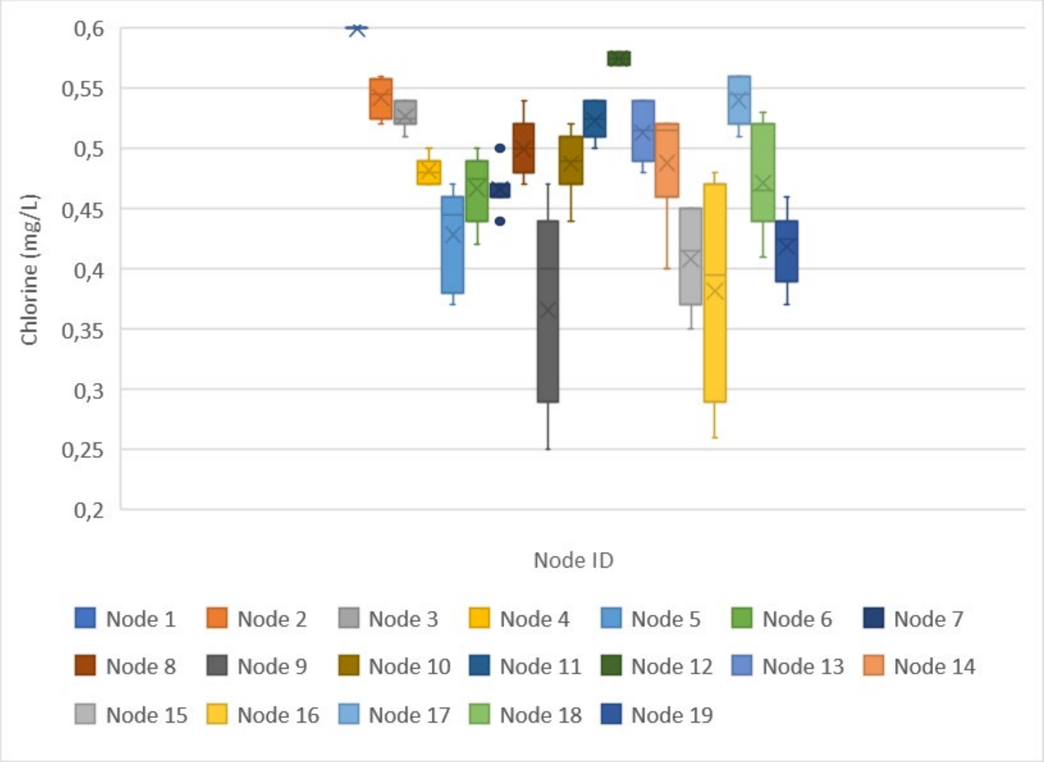


Figure 4.3: Daily chlorine concentration variations at demand nodes (Prasad, 2010)

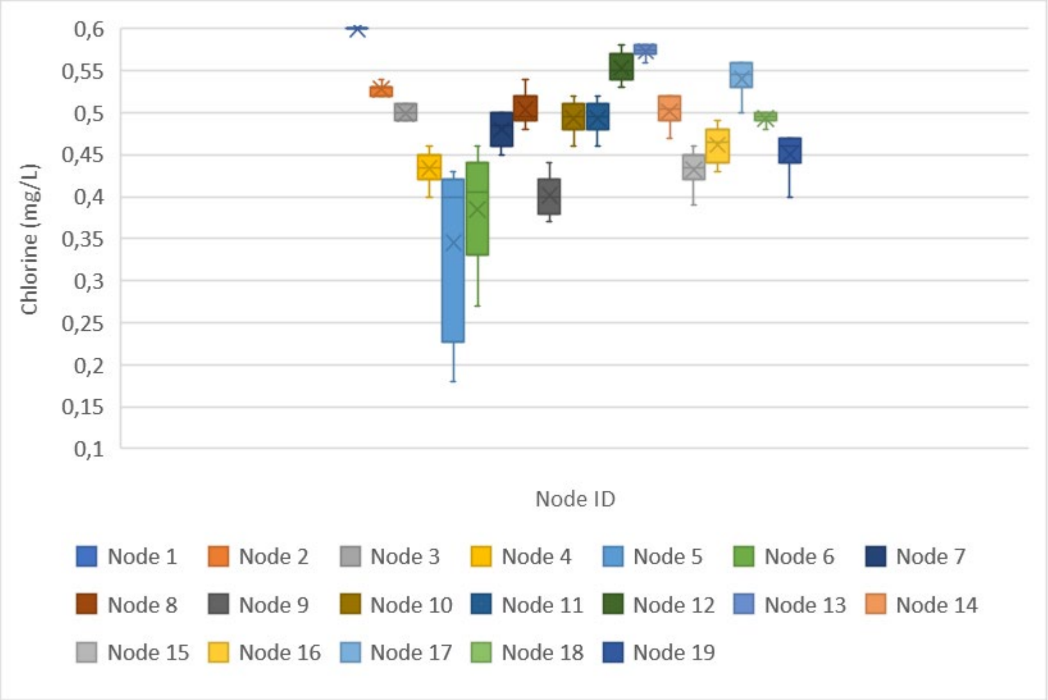


Figure 4.4: Daily chlorine concentration variations at demand nodes (Walters et al., 1999)

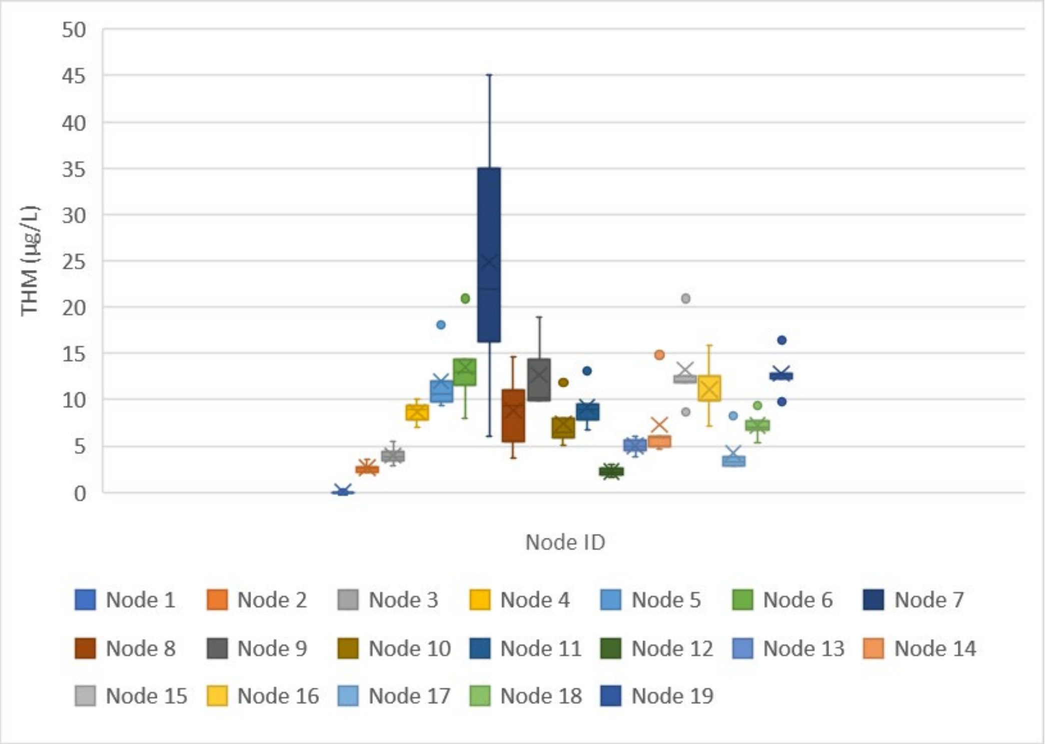


Figure 4.5: Daily THM concentration variations at demand nodes (Solution 1)

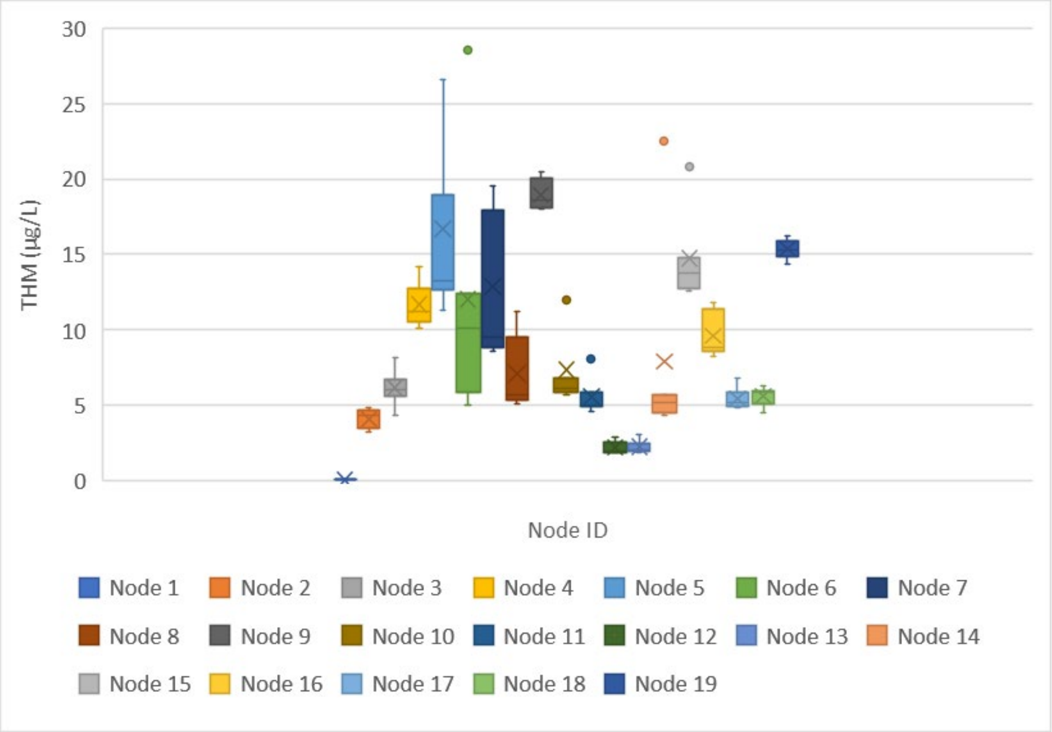


Figure 4.6: Daily THM concentration variations at demand nodes (Solution 2)

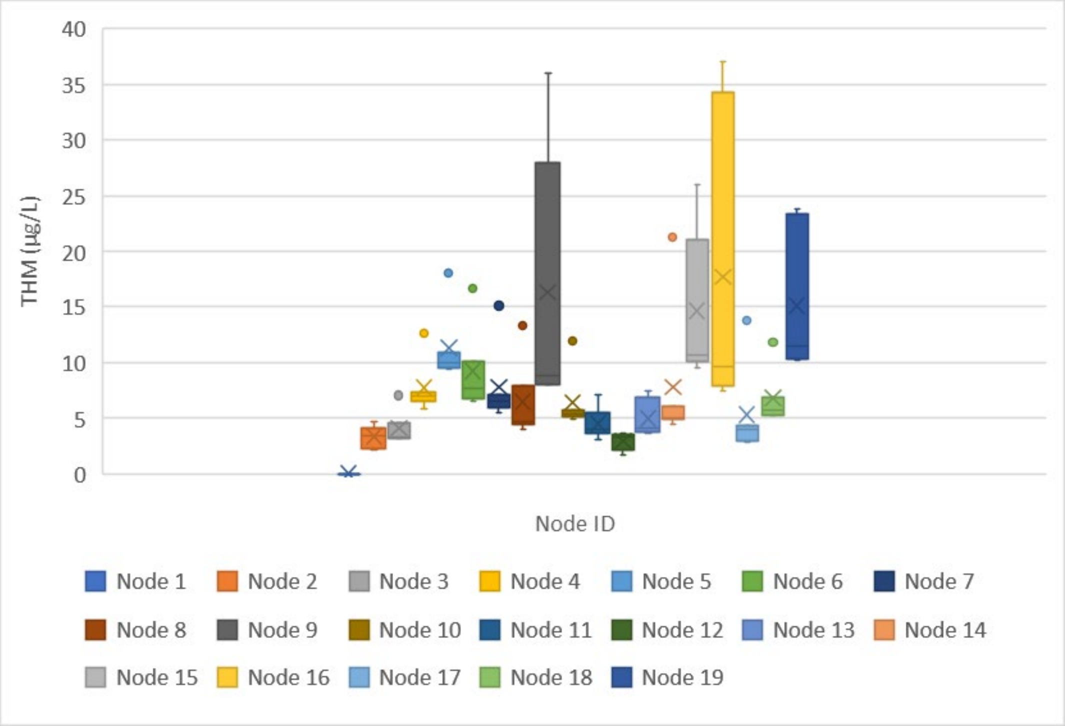


Figure 4.7: Daily THM concentration variations at demand nodes (Prasad, 2010)

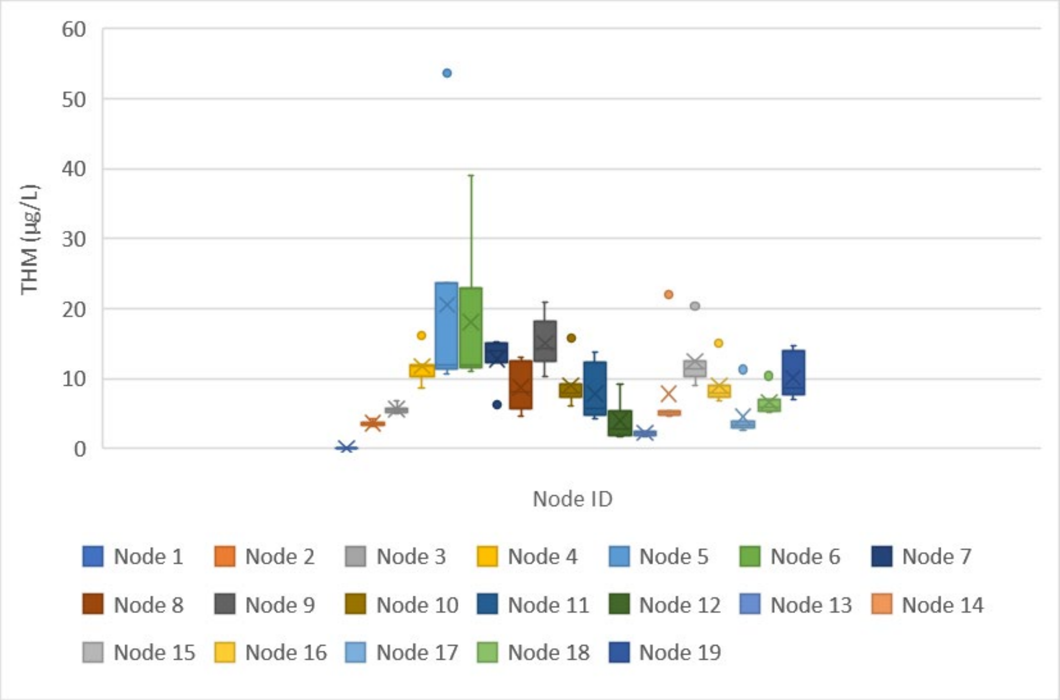


Figure 4.8: Daily THM concentration variations at demand nodes (Walters et al, 1999)

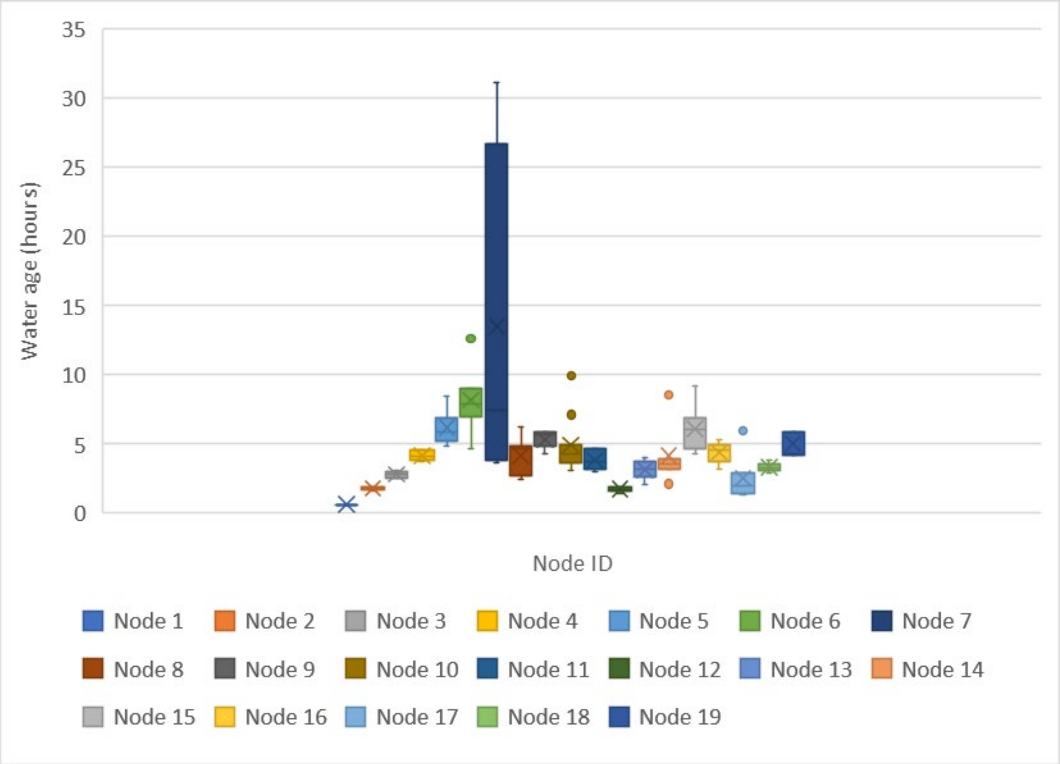


Figure 4.9: Daily water age variations at demand nodes (Solution 1)

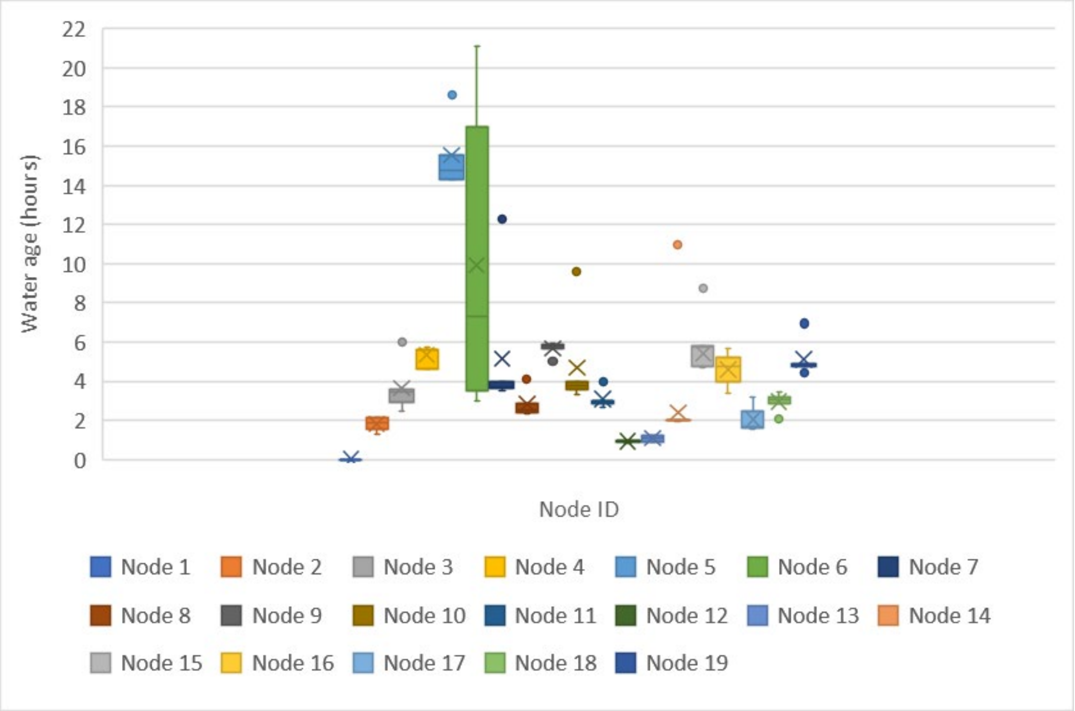


Figure 4.10: Daily water age variations at demand nodes (Solution 2)

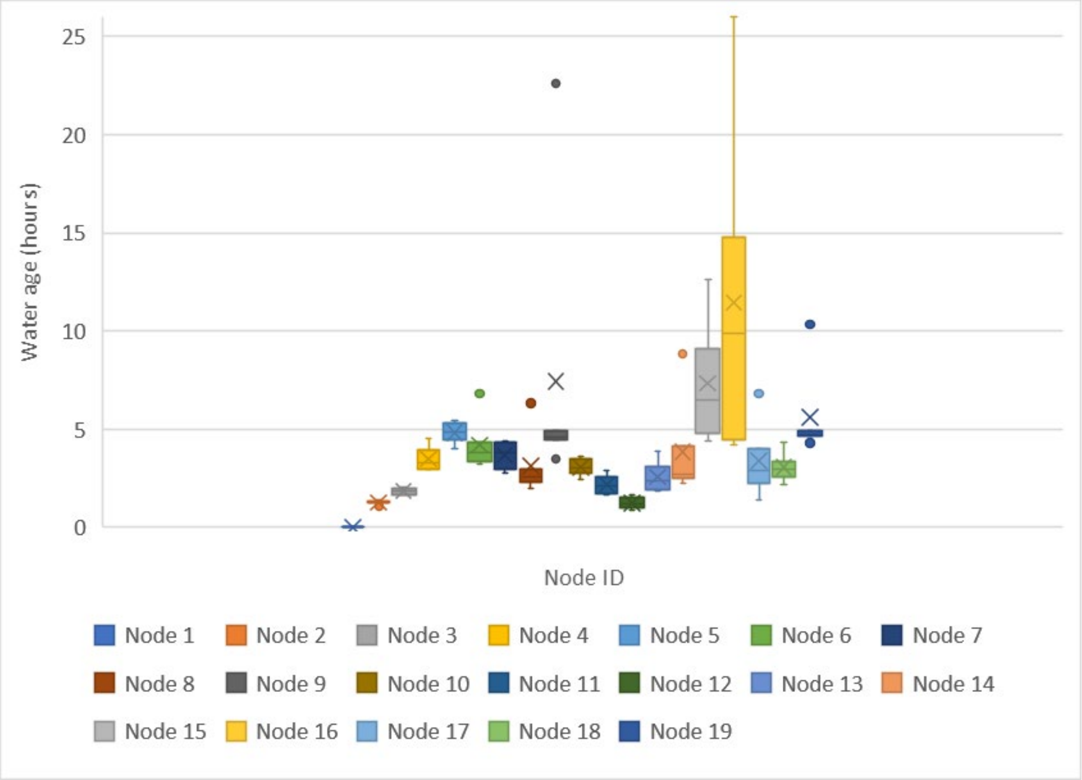


Figure 4.11: Daily water age variations at demand nodes (Prasad, 2010)

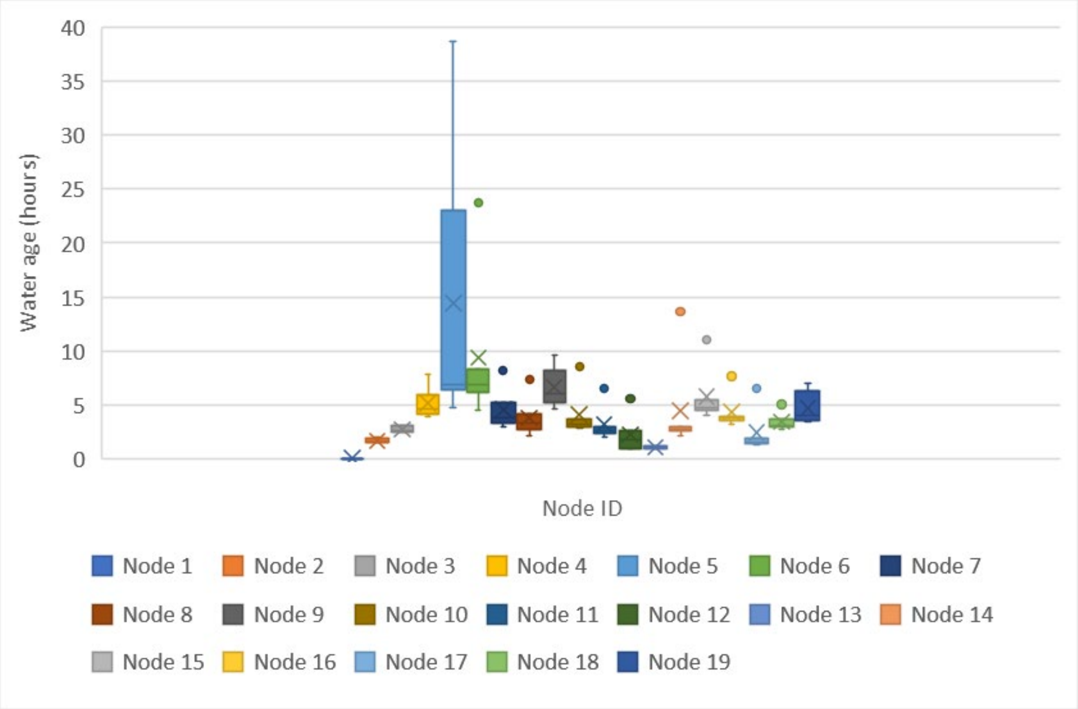


Figure 4.12: Daily water age variations at demand nodes (Walters et al, 1999)

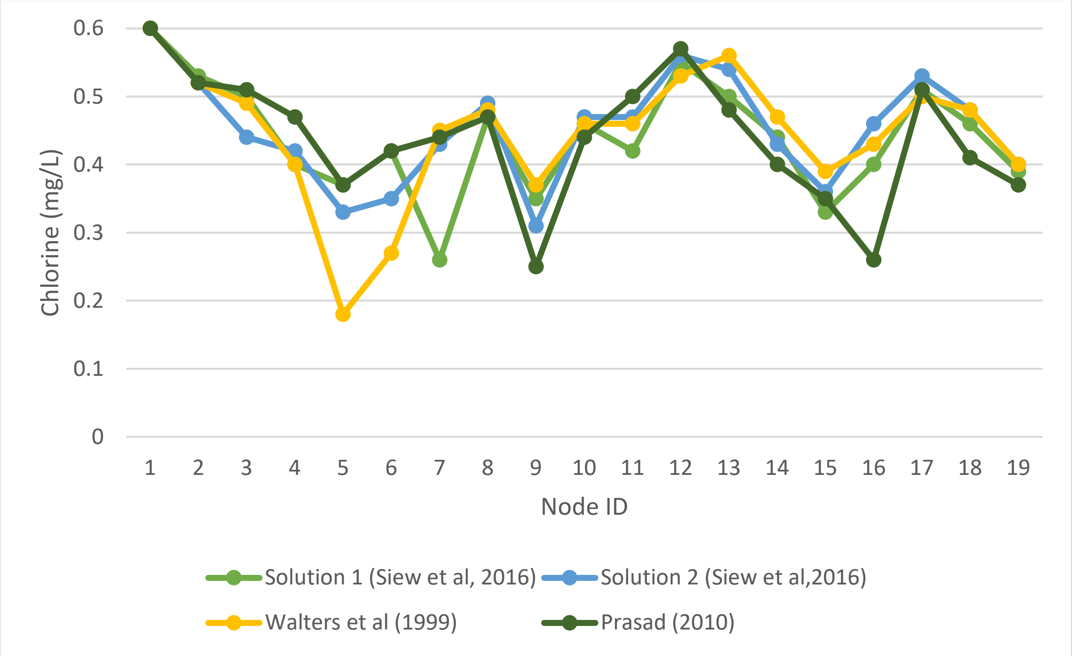


Figure 4.13: Minimum chlorine concentration at demand nodes

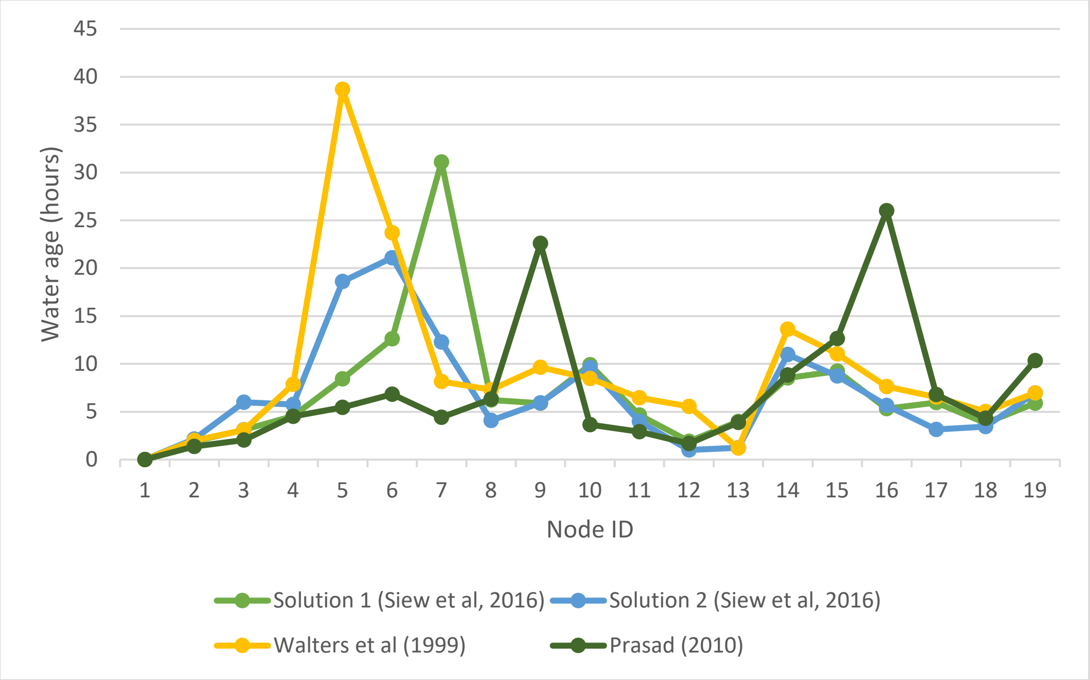


Figure 4.14: Maximum water age concentration at demand nodes

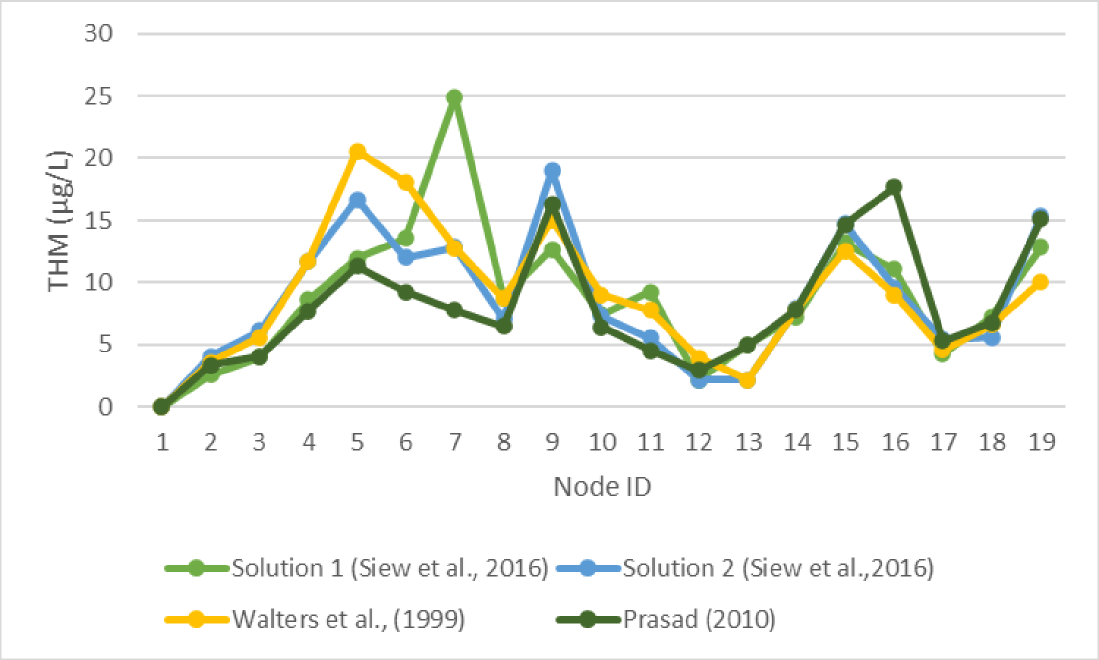


Figure 4.15: Maximum THM concentration at the demand nodes

4.3.3. Water quality analysis at the existing tanks

4.3.3.1. Water age

The results of the solutions with respect to water age were within the maximum allowable water age of 48 hours, with the exception of the Walters et al (1999) solution. The water age values were highest for Tank 41E and 42E. The maximum water age recorded at the existing tanks was 72 hours, which is above the maximum allowable water age of 48 hours. Solutions 1 and 2 had the lowest values for water age at the existing tanks. The maximum water age for Tank 41E is 18 hours in Solution 1 and 27,67 hours for Tank 42E in Solution 2.

4.3.3.2. Residual Chlorine

Both Solution 2 and Walters et al (1999) had the lowest values for chlorine residual in the existing tanks. Although Solution 2 had low chlorine residual concentrations in only one of the existing tanks (41E) and for only a period of 1 hour, Walters et al (1999) had values below 0,2 mg/L for the entire 24-hour simulation period and for both the existing tanks. Based on the water quality results for the demand nodes, it was observed that node 5 was the only node in the Walters et al (1999) solution that had chlorine concentration levels below 0,2mg/L. Araya and Sanchez (2018) state that “once chlorinated, water with a short contact time in the supply sources remains a long time (several hours to several days) in the pipelines and tanks before it is used. This time can be considered as a second contact time, also known as water age, which requires a higher chlorine demand compared to the chlorine demand at the point of application”. This explains the low level of chlorine residual in the existing tanks in the Walters et al (1999) solution. The chlorine results at 20:00 for Solution 2 is an outlier and the anomaly may be associated with the inherent approximations of water quality relations in the EPANET model. Seyoum (2015) states that sometimes, when there is extreme flow, EPANET gives unrealistic results. It has been recommended that a different water quality model be used for future studies. Prasad (2010) had the lowest values for chlorine concentration, recording 0,35mg/L for Tank 41E and 0,33mg/L for Tank 42E.

4.3.3.3. THM Concentration

None of the solutions had THM concentrations above the allowable limit of 100µg/L. Solution 1 had the lowest concentrations of THM, with the maximum value being 30,15µg/L for Tank 41E and 43,17µg/L for Tank 42E.

The results are inconclusive as to which solution performed best in terms of water quality at the existing tanks, however, Solution 1 (Siew et al, 2016) and Prasad (2010) did not have any failed variables in all the existing tanks. The water quality variations at the existing tanks are presented in Table 4.7.

Table 4.7: Water quality variations at the existing tanks for all solutions

	Solution 1 Siew et al (2016)		**Solution 2 Siew et al (2016)**		**Prasad (2010)**		**Walters et al (1999)**	
	Tank 41E	Tank 42E	Tank 41E	Tank 42E	Tank 41E	Tank 42E	Tank 41E	Tank 42E
Minimum chlorine concentration (mg/L)	0,34	0,31	0,00	0,32	0,35	0,33	0,00	0,00
Duration under 0,2mg/L	0 hours	0 hours	1 hour	0 hours	0 hours	0 hours	24 hours	24 hours
Maximum THM concentration (µg/L)	30,15	43,17	47,42	41,20	44,62	43,18	78,1	78,1
Duration over 100µg/L (hours)	0 hours	0 hours	0 hours	0 hours	0 hours	0 hours	0 hours	0 hours
Maximum water age (hours)	18.00	31,55	32,52	27,67	31,16	29,02	72	71
Duration over 48 hours	0 hours	0 hours	0 hours	0 hours	0 hours	0 hours	24 hours	23 hours

4.3.4. Analysis of water quality at the new tanks

4.3.4.1. Water age

The Prasad (2010) solution recorded the highest values for water age at the new tanks. Prasad (2010) introduced new tanks to the Anytown network which are near the existing tanks. Tank 150N, which is located at node 16, had a maximum water age of 48,82 hours and Tank 170N which is located at node 11 had a maximum water age of 53,5 hours. The maximum water age for both tanks is above the maximum allowable water age of 48 hours. Solutions 1 and 2 (Siew et al, 2016) performed well with maximum water age of 39,32 hours and 39,74 hours at Tank 7N and Tank 6N, respectively.

4.3.4.2. Residual chlorine

Similar to water age, the Prasad (2010) solution yielded the most unfavourable results in terms of residual chlorine. Tank 150N had a minimum chlorine residual of 0,15mg/L while Tank 170N had a minimum chlorine residual of 0,22mg/L. Tank 150N had chlorine residuals which are below 0,2mg/L for most of the simulation period. Walters et al (1999) proposed new tanks at nodes 5 (Tank 5N) and 12 (Tank 12N), and the values at Tank 5N were below 0,2 mg/L for most of the simulation period. Tank 5N is remote to the treatment plant while Tank 12N is closer to the treatment plant. Solution 1 and 2 (Siew et al, 2016) yielded the most favourable results in relation to residual chlorine. Both solutions had no values below 0,2mg/L for the entire simulation period.

4.3.4.3. THM Concentration

None of the solutions had THM concentrations exceeding 100µg/L, although Prasad had the highest values of 60,1µg/L and 65,5µg/L at Tank 150N and Tank 170N, respectively. Tank 12N in the Walters et al (1999) solution had the lowest maximum value of 43,68µg/L.

Table 4.8 shows the results for water quality at the new tanks. The calculation of the CCME WQI now follows to determine the overall water quality status for all solutions.

Table 4.8: Water quality at the new tanks for all solutions

	Solution 1 Siew et al, (2016)	**Solution 2 Siew et al, (2016)**	**Prasad (2010)**		**Walters (1999)**	
	Tank 7N	Tank 6N	Tank 150N	Tank 170N	Tank 5N	Tank 12N
Minimum chlorine concentration (mg/L)	0,23	0,22	0,17	0,22	0,16	0,33
Duration under 0,2mg/L	0 hours	0 hours	11 hours	0 hours	16 hours	0 hours
Maximum THM concentration (µg/L)	51,30	53,03	61,33	53,56	57,4	42,68
Duration over 100µg/L (hours)	0 hours	0 hours	0 hours	0 hours	0 hours	0 hours
Maximum water age (hours)	39,32	39,74	48,82	51,87	43,99	28,77
Duration over 48 hours	0 hours	0 hours	1 hour	2 hours	0 hours	0 hours

4.4. CCME WQI Formulation

The water quality results in section 4.3 were utilised to calculate the CCME WQI. These calculations are indicated in detail in Appendices B - E. A summary of the results is included here for discussion. The CCME WQI makes it easier for the excessive data to be aggregated into one single value for simplified decision-making. Since the study was aimed at incorporating water quality into design optimization solutions, taking into consideration tank siting, the CCME WQI was calculated for the tanks only. Demand nodes were not considered in these calculations. A total of 14 tanks were assessed from the 4 optimization solutions found in literature. Table 4.9 shows the CCME WQI for all the tanks. The CCME WQI results indicate that although all solutions were optimal in hydraulic performance and cost, not all the solutions displayed excellent water quality. The results further indicate that the solutions with only one additional tank at the periphery of the network are the preferred solutions.

Table 4.9: CCME WQI for the service reservoirs in all the solutions

TANK ID	SOLUTION 1	SOLUTION 2	PRASAD (2010)	WALTERS ET AL (1999)
41E^{A}	100	77,28	100	25,93
42E	100	100	100	26,35
7N^{B}	100			
6N		100		
150N			60,31	
170N			80,69	
5N				76,82
12N				100

E^{A} refers to existing tank; N^{B} refers to new tank

4.4.1. Solution 1 (Siew et al, 2016)

Solution 1 had the best water quality performance with all service reservoirs achieving an index score of 100. This indicates that the water in all the three tanks in this solution is of excellent quality. This solution would be the recommended solution as it is also the cheapest solution found in literature to date.

4.4.2. Solution 2 (Siew et al, 2016)

This was the second-best solution in terms of water quality performance. Tanks 42E and 6N achieved an index score of 100, but Tank 41E had an index score of 77,28 due to the low chlorine residual concentration for a 1-hour period. The water quality in Tank 41E is classified as fair according to the CCME WQI classification because it falls within the range of 65 and 79. This solution can be ranked second in terms of water quality performance.

4.4.3. Prasad (2010)

The two new tanks which Prasad (2010) introduced to the Anytown network had low CCME WQI scores. The existing tanks in this solution had excellent water quality. Tank 170N which is located at the periphery of the network had a CCME WQI score of 80,69. Tank 150N had a score of 60,31 which means that the water quality in this tank is marginal. This solution is not recommended in terms of water quality performance.

4.4.4. Walters et al (1999)

This is the only solution that had failure not only at the tanks but at the demand nodes too. Node 5 had chlorine residual levels below 0,2mg/L. Node 5N is linked to Tank 5N, and these low chlorine residual levels resulted in the tank achieving an index score of 76,82. Tank 41E and 42E obtained the lowest index scores of 25,93 and 26,35, respectively. This means the water quality in these two tanks is of poor quality. This solution is the worst performing solution in terms of water quality and would not be recommended for implementation.

4.5. Comparison with other WQI scores in literature

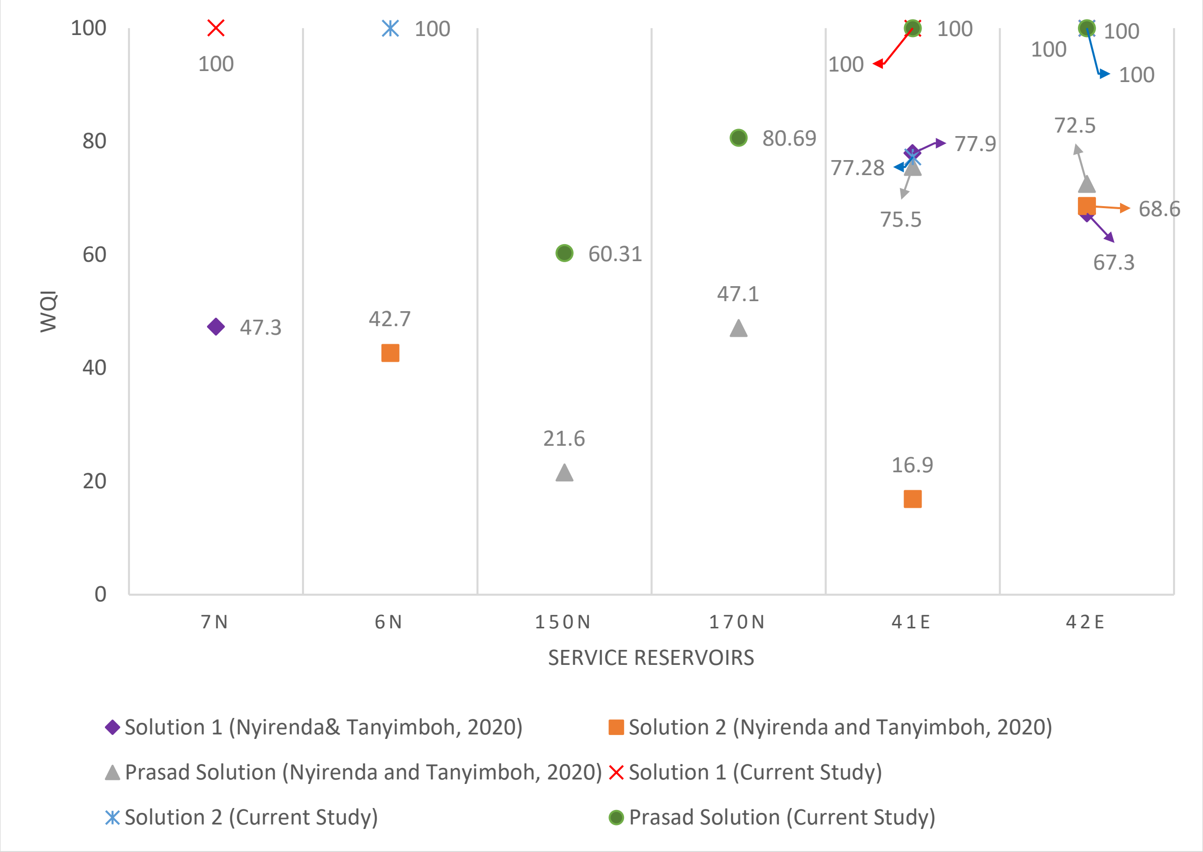


Figure 4.16: Comparison between CCME WQI and Minimum water quality index scores based on Equation (2.8), adapted from Nyirenda and Tanyimboh (2020)

Nyirenda and Tanyimboh (2020) calculated the index scores for these solutions using the unweighted harmonic square mean formula associated with the Oregon WQI. The authors included the solution by Vamvakeridou-Lyroudia et al (2005) in their calculations, although it was stated in their study that this solution was not feasible in terms of hydraulic performance. It is unclear why the authors analysed the solution further for water quality performance. This solution is not included in Figure 4.16 because the current study did not evaluate it further after it was found to be infeasible. The Walters et al (1999) solution is also not included on the comparison chart as Nyirenda and Tanyimboh (2020) did not assess that solution. The authors also indicated that Tank 7N of Solution 1 (Siew et al, 2016) had no failed test values, however, their calculated WQI score was 47,3 for this tank. This score, according to the Oregon WQI classification, indicates that the water in this tank is of fair quality.

This classification is just above the poor-quality classification according to the OWQI. This indicates a discrepancy in the water quality variations and the actual calculation of the index score. Figure 4.16 shows the results obtained from the study by Nyirenda and Tanyimboh (2020) in comparison to the results obtained from the current study.

The authors created sub-index charts (Figures 4.17 – 4.19) for the most common water quality indicators in WDNs. The distributions of these sub-index values in those figures underpin the discrepancy. For instance, according to the water age sub-index chart, a desirable value for water age is zero hours and this value is allocated the maximum index value of 1. However, any test value above zero hours would achieve a lower index value. This presents a challenge when the water age is within acceptable range of between 0 and 48 hours. For instance, when the water age is 40 hours, the index value is just above 0,2, suggesting that the water is of poor water quality, though it is within the acceptable range. This results in ambiguity. As stated in the problem statement, ambiguity exists when all the sub-indices meet the standard for a particular use or application, but the overall index fails. As a result, the overall water quality may be deemed poor, even when it is truly of acceptable level for that application (Sutadian et al, 2016). The same concern can also be noted on the THM sub-index chart. Any test value above 60µg/L would achieve a sub-index value of 0,1, although the limiting concentration for THM was taken to be 80µg/L in their study. Tank 150N in the Prasad (2010) solution achieved a maximum THM concentration of 61,33µg/L and this would result in a sub-index value of 0,1 which is the lowest achievable value. This sub-index value indicates poor water quality although the THM concentration is within the acceptable range.

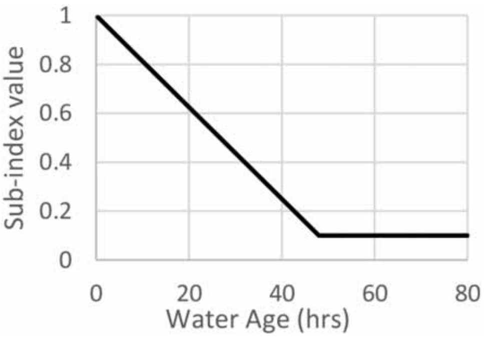


Figure 4.17: Water age sub-index

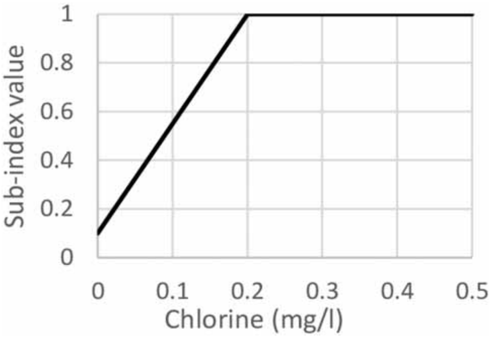


Figure 4.18: Chlorine sub-index

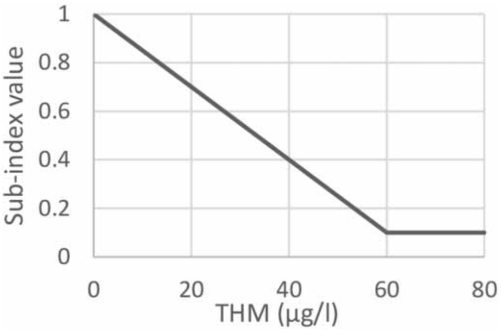


Figure 4.19: THM sub-index

According to Nyirenda and Tanyimboh (2020), “existing reservoirs 41E and new reservoir 150N have the lowest indices, indicating poor water quality states”. This statement contradicts the finding from solution 2 that Tank 41E had only 1-hour period when the water quality was below 0,2mg/L. This cannot cause the overall water quality status in this service reservoir to be deemed poor. The authors noted that these sub-index charts represented an estimate and might need to be refined further. The authors also indicated that according to their knowledge, these sub-index charts had not been used in service reservoirs before. The original OWQI method requires sub-index values to be calculated and aggregated using equation 2.9. The sub-index charts created by Nyirenda and Tanyimboh (2020) were estimated, and they are not consistent with the water quality modelling results. The calculated WQI should be congruent with the water quality variations that were obtained from the water quality modelling. The use of the CCME WQI has shown to be consistent with the water quality modelling results.

4.6. Summary of findings

This section is dedicated to discussing the findings of the study. The study had three main objectives identified with the aim of integrating water quality and hydraulic performance into the design optimisation process in a simplified manner. A discussion on the finding of the objectives is addressed herein.

4.6.1. To obtain spatial and temporal water quality variations for efficient water quality analysis.

Water quality variations were obtained using EPANET 2.2 over a 72-hour simulation period as discussed in Chapters 3 and 4 of this study. Through this process, the water quality variations at every minute of the simulation were studied. The findings were compared to the standards available for water quality. Since the water quality varies at every hour, it becomes a challenging task to present the overall water quality in the system in a simplified manner. This was proven by the amount of data which was obtained from the water quality modelling. This process becomes even more challenging with the increase of water quality variables. Without investigating water quality during the design optimisation process, solutions like Walters et al (1999) would be considered to be optimal based on the hydraulic performance, however, water quality modelling showed that service reservoirs in this solution have high water age which resulted in undetectable chlorine concentration levels. The absence of chlorine residuals increases the chances of bacteria forming and the water being contaminated.

Solution 2 (Siew et al, 2016) was the most optimal solution in terms of hydraulic performance. However, the water quality modelling indicated that this solution had low chlorine concentration levels in existing tank 41E for an hour. Based on the water quality modelling results, it was evident that Solution 1 had the most optimal water quality performance. The results also indicated that the Walters et al (1999) was the worst performing solution. It could be argued that these results indicate the optimal solution and the worst solution without the introduction of the CCME WQI. Although this is the case for this network in which Solution 1 did not have any failed variables, an optimal solution would be difficult to identity without the use of the CCME WQI where all the solutions had failed variables. This can be observed with the WQM results for the demand nodes. Based on these results, Solution 2 was the optimal solution in terms of water quality performance but the results for the service reservoirs indicated otherwise. The results of the new tanks presented in Table 4.8 show that both Prasad and Walters et al solutions failed some objectives, yet Prasad had one of the best solutions according to Table 4.7. This goes to show that water quality variations are subject to change at any given time step, and without calculating the overall WQI it can become challenging to decide which solution performed better based off the water quality variations only, especially where all the solutions have failed variables. The process becomes more tedious should more variables be added, and the simulation period increased. A reduction in the time step also increases the amount of data obtained from the water quality modelling, making the process even more challenging.

4.6.2. To calculate a single, non-dimensional value using the water quality modelling results and the CCME WQI.

The results obtained from the water quality modelling were used to calculate the final index value.

a) Determination of F_1

The failed variables were identified as any variables that did not meet the objective. For example, for Tank 150N in the Prasad (2010) solution, chlorine and water age were the failed variables as they failed the objectives of 0.2mg/l and 48hours, respectively. These results are shown in Appendix D-5. The total number of variables represents the variables that were tested in total. This means the total number of variables for this study were three, namely chlorine, THM and water age.

b) Determination of F_2

The number of failed tests represented how many times the failed variable did not meet its objective over the last 24-hour period. Using the same example that was used in (a) above, chlorine failed its objectives eleven times between 12:00 and 22:00. Water age failed its objective once at 24:00. This brought the total number of failed tests to twelve. The total number of tests were 72 to represent the total number of readings for all three variables over the 24-hour period.

c) Determination of F_3

Using the same Prasad (2010) solution for Tank 150N example, all the failed test values were as obtained from EPANET 2.2 and as recorded in Appendix D-5. The failed test value for chlorine at 12:00 was 0,19mg/l and the objective was 0,2mg/l. All the failed test values had to be measured against their respective objective to ultimately arrive at the value for the normalised sum of excursions as per the calculations attached in Appendix D-5.

It may be argued that the inclusion of this step was extraneous for this network, given the findings derived from the WQM had already demonstrated that solution 1 did not exhibit any variable failures, hence establishing it as the favoured option. Solution 2 would be ranked second, based on the excellent water quality classification in Tank 42E and 6N, and the good water quality classification in Tank 41E. Although the CCME WQI was not required to identify the most optimal solution in this network, Table 4.9 indicates that the use of the CCME WQI assists with reporting the results in a simplified manner. When analysing a network using a 1 minute time step and a 72 hour extended period simulation, there are 4320 data points to be analysed. Although this study only focused on the last 24-hours of the EPS, this process still resulted in a high number of data points that had to be analysed. Calculating the CCME WQI simplified the process by not requiring the determination of sub-indices and weights. The values obtained from the WQM were used in their original form and were used in the CCME WQI equations without any need for additional steps to be followed.

Solutions with only one additional tank performed better than the solutions with two additional tanks. These most optimal solutions are also the solutions with the lowest implementation cost compared to the other two solutions with two additional tanks.

4.6.3. To assess the usefulness of the CCME WQI to address high dimensionality in water quality modelling by comparing it with other results found in literature.

The CCME WQI method proved to be easy to apply and provided a simplified way of reporting water quality results found in the water quality modelling for the Anytown network. With respect to addressing high dimensionality, although there were only three variables that were tested, these variables contained different units of measurement however, the CCME WQI did not require the formation of sub-indices prior to calculating the final index value. The process would remain the same even with the addition of more variables. The variables were used with their respective units, and it was not mandatory to assign weights to each of the variables. The process of assigning weights and formation of sub-indices often leads to ambiguity as can be seen in the case of Tanyimboh and Nyirenda (2020). Other challenges that could arise from the formation of sub-indices and assigning weights to the parameters include eclipsing and rigidity as was discussed in chapter 2 of this study.

This study did not go further into investigating whether the CCME WQI eliminates ambiguity, rigidity and eclipsing, however, the method was found to be simple and user-friendly. Although the CCME WQI was only calculated for the service reservoirs, the results indicate that this method can be used as a surrogate for water quality performance to assist with reporting and decision-making. The use of the CCME WQI for design optimisation does not replace the need for water quality modelling as the results from the WQM are needed to calculate the final index value. The same can be said about the use of the CCME WQI in a WDN that has been commissioned. It does not replace the need for water testing or taking of samples as the values obtained from water sampling are needed to calculate the final index value. This process of incorporating water quality in design optimisation does not replace the need for water quality testing once the network has been commissioned. These are two different processes which serve different purposes.

Based on this study, it was established that the CCME WQI assists with reporting of the results from the WQM in a simplified manner. The CCME WQI, however, fell short in terms of assisting with selecting the most optimal solution because the WQM results had already indicated that Solution 1 was the only solution with no failed variables, making it the preferred solution. These results could change in other WDNs and a recommendation for the need to assess other WDNs using this same method is made in Chapter 5. In a case where all the solutions for a particular WDN had failed variables, the CCME WQI could assist with making the final decision and selecting the most optimal solution based on the final index values for these solutions. This study was not able to demonstrate that based on the reason stated above. It can be proposed that the CCME WQI be used as a surrogate for design optimisation only in cases where the WQM results were inconclusive. It can also be used when a need arises to report water quality results in a simpler manner.

CHAPTER 5: CONCLUSION AND RECOMMENDATIONS

5.1. Conclusion

This study has demonstrated that the CCME WQI is efficient in addressing high dimensionality in water quality modelling of WDSs as described in section 4.6.3. A large amount of quantitative data with different units of measurement were obtained from the water quality simulation and this data was aggregated into a single non-dimensional value for each service reservoir. This simplified the process of analysing results obtained from water quality modelling and further allowed water quality to be explained in a simplified manner. Chlorine residual, THM and water age are the variables that were analysed in this study and even with only these three parameters, the amount of data to be interpreted for water quality purposes was enormous. This process becomes harder with an introduction of more parameters with varying dimensions and units. Without the use of WQIs, water quality modelling becomes a strenuous exercise with a large amount of data that is hard to interpret, thus making it difficult to arrive at an optimal solution. The CCME WQI did not require any additional processes to be carried out to change the different dimensions into a non-dimensional value. The same process can be followed even with the addition of more parameters.

It is important to state that the study was not meant to eliminate the process of water quality modelling and water quality monitoring. Upon the completion of the system's commissioning, it remains imperative to continue conducting water quality testing in accordance with the relevant criteria specific to the given region. The study was mainly for design optimisation and decision making only. It does not constitute a holistic approach to water quality analysis. The study was not meant to address water quality in WDSs. Section 3.3 of this report indicated that other parameters like pH, temperature, colour, taste, odour, etc would still have to be tested once the system is commissioned but that was not the main purpose of the study. Based on the limitations of EPANET 2.2 and for the purpose of assessing the influence of the CCME WQI on design optimisation, only three parameters were chosen. EPANET2.2 can only simulate age, trace and chemical properties. Chlorine residual (chemical property), being one of the most important parameters in the transportation of treated water, THM (trace), being a disinfection by-product, and water age (age), being another water quality indicator, are water quality parameters that can be modelled by EPANET 2.2.

Haloacetic acids (trace) can be added, however, they are also disinfection by-products and would thus serve the same purpose as THM in terms of the results obtained. At this stage, it was not possible to simulate other parameters, especially the physical and biological parameters.

If hydraulic performance was the only factor considered when selecting an optimal solution, the solution proposed by Walters et al (1999) would have been one of those to consider for implementation. However, the water quality consideration has shown the shortcoming of this solution which would lead to the end user receiving water of poor quality. Once implemented, the solution would require other measures such as chlorine booster stations to be installed within the network, which would result in increased cost. Alternatively, the pipe upgrade and rehabilitation measures would have to be reconsidered for this solution and this has a cost implication attached to it too. A design solution which was selected because of its low-cost implications could end up being an expensive solution due to the maintenance and/or upgrade costs resulting from poor water quality.

The index score served as a measure for the outcomes of the 24-hour water quality modelling. For example, Tanks 41E and 42E in the Walters et al (1999) solution failed test values for chlorine residual and water age for the entire 24-hour simulation period. It would be expected that these findings would result in poor water quality classification. The efficiency of the CCME WQI was demonstrated by the obtained index score values below 44 for the aforementioned tanks. Similarly, the index scores for all the tanks in Solution 1 (Siew et al, 2016) were synonymous with the water quality variations in these tanks. The sub-index values, which were created by Nyirenda and Tanyimboh (2020) and aggregated using the OWQI method, resulted in a score of 47,3 for Tank 7N which is synonymous with poor water quality, although this tank had no failed test values or parameters. This indicates a discrepancy in the index score calculated. It is important to note that the OWQI method was not used in this study and the use of this method cannot be discredited or confirmed by this study. However, it is recommended for the sub-index charts created by Nyirenda and Tanyimboh (2020) to be refined or for new sub index values to be calculated and aggregated using equation 2.8, 2.9 or 2.10 for a proper comparison with this study.

5.2. Implications of the study

The findings of this study indicate that water quality can be integrated into the design optimisation process and the results can be reported in a simplified manner using the CCME WQI. Based on this study, it was demonstrated that the CCME WQI can assist with addressing high dimensionality, however, it fell short in terms of assisting with selecting the most optimal solution as Solution 1 was found to be favourable even prior to calculating the CCME WQI. This should be viewed within the context of the Anytown network. Water engineers can consider using the CCME WQI to select the most optimal design solution for cases where the results from the WQM are inconclusive. This could assist water engineers with integrating water quality into design optimisation solutions thus providing designs that are more economically sustainable. The findings of this study further demonstrate the need for reliable water quality models to avoid modelling errors and a false representation of water quality variations within the network. This has been further confirmed by the results obtained in Solution 2 were there was a chlorine concentration below 0,2mg/l for only an hour in Tank 41E. This result proved to be an outlier and the anomaly could be attributed to the limitations of the model being used.

5.3. Study limitations

1) EPANET 2.2 simulates only one variable at time. This insinuates that each variable exists on its own within the WDN. This is a flawed representation of the mixing mechanisms involved in a WDN. EPANET-MSX is a multi-species extension of EPANET and it simulates all variables concurrently, however, it does not have a user-friendly interface and challenges may arise when attempting to run/install the programme.

2) Another restriction with EPANET 2.2 relates to modelling THM as it requires the modeller to establish the limiting THM concentration. An alternate model for THM that does not require the specification of a limiting concentration was proposed by Sohn et al (2004). The example for this can be found in Seyoum and Tanyimboh (2013).

3) The CCME WQI recommends using a minimum of 4 variables to avoid sensitivity on the value of F_1. This is achievable in the analysis of WDNs that have been commissioned where sampling can be carried out. In a study such as this where the

designs are still in the optimization stages, this can be difficult or impossible to achieve. In a live network, water samples are taken and tested in a laboratory to obtain the test values for each parameter. Drinking water is often tested based on multiple parameters as per the drinking water regulations so it would be possible to have test values for more than 4 water quality parameters.

4) The CCME WQI calculation for excursions in cases where the test value must not fall below the objective could result in division by zero, which would in turn result in an error in the calculations. The following formula is used to determine excursions when the test result is not below the objective:

$$Excursion = \left(\frac{Objective}{Failed\ Test\ Value}\right) - 1$$

The above formula indicates that should the failed test value be equal to zero, this calculation would be invalid.

5) The study is based on a benchmark WDN. Results for a real WDN might differ and other limitations may arise from the use of a real life WDN. For instance, in a network that contains thousands of demand nodes and more service reservoirs than the ones presented in this study, the process could be even more challenging.

6) The process for calculating the CCME WQI for design optimisation of WDNs relies on the data obtained from water quality modelling using EPANET or any other model which has water quality analysis capabilities. Any errors resulting from the use of such a model could result in wrong data being obtained, thus affecting the final CCME WQI calculations.

5.4. Study assumptions

There were no assumptions made when calculating the CCME WQI, however, there were assumptions made when performing water quality modelling using EPANET2.2. The initial chlorine concentration at the source had to be assumed. This value could change based on the water quality standards for the area being investigated.

5.5. Recommendations for future studies

It is recommended that a different simulation model which allows multi-species analysis and addition of more parameters be used for future studies. Future studies should also consider adding another parameter to the calculations. Haloacetic acids (HAAs) could be explored as an additional parameter for water quality modelling and calculation of the CCME WQI. Future studies should also consider applying the CCME WQI to other distribution networks that are being designed to assess its usefulness in design optimisation from both hydraulic and water quality perspectives.

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APPENDICES

APPENDIX A: EXISTING DATA FOR ANYTOWN NETWORK

A-1: Input data for Anytown network

NODE ID	ELEVATION (m)	ADD (L/s)
1	6.096	31.545
2	15.24	12.618
3	15.24	12.618
4	15.24	37.854
5	24.384	37.854
6	24.384	37.854
7	24.384	37.854
8	24.384	25.236
9	36.576	25.236
10	36.576	25.236
11	36.576	25.236
12	15.24	31.545
13	15.24	31.545
14	15.24	31.545
15	15.24	31.545
16	36.576	25.236
17	36.576	63.090
18	15.24	31.545
19	15.24	63.090
20	6.096	0
21	15.24	0
22	36.576	0

A-2: Demand Pattern

Time	Actual Water Use
00:00-03:00	0.7
03:00-06:00	0.6
06:00-09:00	1.2
09:00-12:00	1.3
12:00-15:00	1.2
15:00-18:00	1.1
18:00-21:00	1
21:00-24:00	0.9

A-3: Loading Conditions for Anytown Network

NODE ID	ADD (L/s)	IPF (L/s)	FIRE 1 (L/s)	FIRE 2 (L/s)	FIRE 3 (L/s)
1	31.545	56.781	41.009	41.009	41.009
2	12.618	22.712	16.403	16.403	16.403
3	12.618	22.712	16.403	16.403	16.403
4	37.854	68.137	49.210	49.210	49.210
5	37.854	68.137	49.210	94.635	49.210
6	37.854	68.137	49.210	94.635	49.210
7	37.854	68.137	49.210	94.635	49.210
8	25.236	45.424	32.807	32.807	32.807
9	25.236	45.424	32.807	32.807	32.807
10	25.236	45.424	32.807	32.807	32.807
11	25.236	45.424	32.807	32.807	63.090
12	31.545	56.781	41.009	41.009	41.009
13	31.545	56.781	41.009	41.009	41.009
14	31.545	56.781	41.009	41.009	41.009
15	31.545	56.781	41.009	41.009	41.009
16	25.236	45.424	32.807	32.807	32.807
17	63.090	113.562	82.017	82.017	63.090
18	31.545	56.781	41.009	41.009	41.009
19	63.090	113.562	157.725	82.017	82.017

A-4: Anytown Network Pipe Data

PIPE	START NODE	END NODE	LENGTH (m)	DIAMETER (mm)	ROUGHNESS COEFFICIENT	LOCATION
1	1	2	3657.6	304.8	120	Residential
2	1	12	3657.6	304.8	70	City
3	1	13	3657.6	406.4	70	City
4	1	20	30.48	762	130	City
5	2	3	1828.8	254	120	Residential
6	2	4	2743.2	254	120	Residential
7	2	13	2743.2	304.8	70	Residential
8	2	14	1828.8	254	120	Residential
9	3	4	1828.8	254	120	Residential
11	4	8	3657.6	203.2	120	Residential
12	4	15	1828.8	254	120	Residential
17	8	9	3657.6	203.2	120	Residential
18	8	15	1828.8	254	120	Residential
19	8	16	1828.8	203.2	120	Residential
20	8	17	1828.8	203.2	120	Residential
21	9	10	1828.8	203.2	120	Residential
22	10	11	1828.8	203.2	120	Residential
23	10	17	1828.8	254	120	Residential
24	11	12	1828.8	203.2	120	Residential
26	12	17	1828.8	254	120	Residential
27	12	18	1828.8	203.2	70	City
28	13	14	1828.8	304.8	70	City
29	13	18	1828.8	304.8	70	City
30	13	19	1828.8	254	70	City
31	14	15	1828.8	304.8	70	City
32	14	19	1828.8	254	70	City
33	14	21	30.48	304.8	120	City
34	15	16	1828.8	254	70	City
35	15	19	1828.8	254	70	City
36	16	17	1828.8	203.2	120	Residential
37	16	18	1828.8	304.8	70	City
38	16	19	1828.8	254	70	City
39	17	18	1828.8	203.2	120	Residential
40	17	22	30.48	304.8	120	Residential
41	18	19	1828.8	254	70	City
142	21	41	0.3048	304.8	120	City
143	22	42	0.3048	304.8	120	Residential

A-5a: Pump Characteristics

Discharge (L/s)	Head (m)	Efficiency (%)
0	91.44	0
126.18	89	50
252.36	82.296	65
378.54	70.104	55
504.72	55.169	40

A-5b: Tank Properties

Properties	Tank 41E	Tank 42E
Minimum level (m)	68.58	68.58
Maximum level (m)	76.2	76.2
Top level (m)	77.72	77.72
Bottom level (m)	65.53	65.53
Diameter (m)	10.892	10.892
Location	Node 14	Node 17

A-5c: Reservoir Properties

Location	Head (m)
Node 40	3.05m

APPENDIX B: WATER QUALITY PERFORMANCE FOR SOLUTION 1 (SIEW ET AL, 2016)

B-1: Water quality variations in Tank 41E

Parameters	Objective
Water Age (hours)	<48
Chlorine (mg/L)	≥0.2
THM (µg/L)	<100

Time Period	Chlorine (mg/L)	Water age (hours)	THM (µg/L)
00:00	0.41	12.89	20.82
01:00	0.45	8.64	14.35
02:00	0.46	7.89	13.33
03:00	0.46	7.76	13.3
04:00	0.46	7.48	12.99
05:00	0.45	8	13.97
06:00	0.44	9	15.75
07:00	0.44	10	17.48
08:00	0.43	11	19.18
09:00	0.42	12	20.85
10:00	0.41	13	22.48
11:00	0.4	14	24.08
12:00	0.39	15	25.65
13:00	0.38	16	27.18
14:00	0.38	17	28.68
15:00	0.34	18	30.15
16:00	0.49	5.46	9.35
17:00	0.49	4.82	8.41
18:00	0.49	5.05	8.93
19:00	0.48	6.05	10.78
20:00	0.47	7.05	12.62
21:00	0.46	8.05	14.43
22:00	0.49	4.27	7.74
23:00	0.48	4.55	8.29
24:00	0.48	5.03	9.21

B-2: CCME WQI CALCULATION FOR TANK 41E IN SOLUTION 1

	CODE	CCME Data
NO. OF FAILED VARIABLES (Paramaters)	X	0
TOTAL NUMBER OF VARIABLES (Parameters Studied)	Y	3
TOTAL NUMBER OF TESTS	Z	72
TOTAL NUMBER OF FAILED TESTS (ALL PARAMETERS EXCEEDING)	E	0

Step 1: Calculate F_1

$$F_1 = \left(\frac{Number\ of\ failed\ variable}{Total\ Number\ of\ Variable}\right) x\ 100$$

F_1	0

Step 2: Calculate F_2

$$F_2 = \left(\frac{Number\ of\ failed\ test}{Total\ Number\ of\ Tests}\right) x\ 100$$

F_2	0.00

Step 3.1: Calculate Excursions

Excursion

$$Excursion\ i = \left(\frac{Failed\ test\ i}{Objective\ j}\right) - 1$$

FAILED TEST VALUE	**A**
OBJECTIVE	**B**
	A/B
C = Excursion	**C = A/B MINUS 1**
	C = 0

Step 3.2: Calculate Nse

Normalised sum excursion (Nse)

$$nse = \frac{\sum_{i=1}^{n} Excursion\ i}{Total\ number\ of\ tests}$$

Nse	∑ [C]/Z	0

Step 3.3: Calculate F_3

$$F_3 = \left(\frac{nse}{0.01\ x\ nse + 0.01}\right) = \left(\frac{nse}{nse + 1}\right)\ x\ 100$$

Nse value	0.01 * Nse	0.01*Nse + 0.01	F_3
0	0	0.01	0

Step 4: Overall CCME Calculation

$$CCME\ WQI = 100 - \frac{\sqrt{F_1^2 + F_2^2 + F_3^2}}{1.732}$$

Component of CCME WQI	Value	Square Value
F_1	0	0
F_2	0.00	0
F_3	0	0
	Sum	0
	Square root value	0
	Divide by 1.732 (D)	0

CCME WQI for Tank 41E in Solution 1	100 - D	100

CCME WQI Value for Tank 41E in Solution 1 = 100

B-3: WATER QUALITY VARIATIONS FOR TANK 42E

Parameters	**Objective**
Water Age (hours)	<48
Chlorine (mg/L)	≥0.2
THM (μg/L)	<100

Time Period	**Chlorine (mg/L)**	**Water age (hours)**	**THM (μg/L)**
00:00	0.31	26.27	38.4
01:00	0.35	22.84	33.32
02:00	0.36	21.72	31.66
03:00	0.36	22.43	32.67
04:00	0.35	23.43	34.06
05:00	0.34	24.43	35.42
06:00	0.33	25.43	36.75
07:00	0.33	26.43	38.05
08:00	0.32	27.43	39.33
09:00	0.31	28.43	40.58
10:00	0.33	26.56	37.8
11:00	0.33	27.49	38.99
12:00	0.32	28.49	40.25
13:00	0.33	28.09	39.53
14:00	0.32	28.8	40.38
15:00	0.32	29.8	41.61
16:00	0.33	28.1	39.11
17:00	0.33	28.05	38.94
18:00	0.33	28.55	39.51
19:00	0.32	29.55	40.75
20:00	0.32	30.55	41.97
21:00	0.31	31.55	43.17
22:00	0.33	30	40.95
23:00	0.33	29.81	40.58
24:00	0.33	30.08	40.84

B-4: CCME WQI CALCULATION FOR TANK 42E IN SOLUTION 1

	CODE	CCME Data
NO. OF FAILED VARIABLES (Paramaters)	**X**	0
TOTAL NUMBER OF VARIABLES (Parameters Studied)	**Y**	3
TOTAL NUMBER OF TESTS	**Z**	72
TOTAL NUMBER OF FAILED TESTS (ALL PARAMETERS EXCEEDING)	**E**	0

Step 1: Calculate F_1

$$F_1 = \left(\frac{Number\ of\ failed\ variable}{Total\ Number\ of\ Variable}\right) x\ 100$$

F_1	0

Step 2: Calculate F_2

$$F_2 = \left(\frac{Number\ of\ failed\ test}{Total\ Number\ of\ Tests}\right) x\ 100$$

F_2	0.00

Step 3.1: Calculate Excursions

Excursion

$$Excursion\ i = \left(\frac{Failed\ test\ i}{Objective\ j}\right) - 1$$

FAILED TEST VALUE	**A**
OBJECTIVE	**B**
	A/B
C = Excursion	**C = A/B MINUS 1**
	C = 0

Step 3.2: Calculate Nse

Normalised sum excursion (Nse)

$$nse = \frac{\sum_{i=1}^{n} Excursion\ i}{Total\ number\ of\ tests}$$

Nse	∑[C]/Z	0

Step 3.3: Calculate F_3

$$F_3 = \left(\frac{nse}{0.01\ x\ nse + 0.01}\right) = \left(\frac{nse}{nse + 1}\right)\ x\ 100$$

Nse value	0.01 * Nse	0.01*Nse + 0.01	F_3
0	0	0.01	0

Step 4: Overall CCME Calculation

$$CCME\ WQI = 100 - \frac{\sqrt{F_1^2 + F_2^2 + F_3^2}}{1.732}$$

Component of CCME WQI	Value	Square Value
F_1	0	0
F_2	0.00	0
F_3	0	0
	Sum	0
	Square root value	0
	Divide by 1.732 (D)	0
	D	0

CCME WQI for Tank 42E in Solution 1

	100 - D	100

CCME WQI Value for Tank 42E in Solution 1 = 100

B-5: WATER QUALITY VARIATIONS FOR TANK 7N IN SOLUTION 1

Parameters	Objective
Water Age (hours)	<48
Chlorine (mg/L)	≥0.2
THM (µg/L)	<100

Time Period	Chlorine (mg/L)	Water age (hours)	THM (µg/L)
00:00	0.23	33.35	47.5
01:00	0.23	32.02	45.43
02:00	0.25	30.63	43.34
03:00	0.27	29.27	41.36
04:00	0.28	27.64	39.07
05:00	0.3	26.31	37.24
06:00	0.31	25.69	36.41
07:00	0.3	26.69	37.72
08:00	0.29	27.69	39
09:00	0.29	28.69	40.26
10:00	0.28	29.69	41.49
11:00	0.28	30.69	42.7
12:00	0.27	31.69	43.88
13:00	0.26	32.69	45.04
14:00	0.26	33.69	46.17
15:00	0.25	34.69	47.28
16:00	0.25	35.45	48.05
17:00	0.25	35.86	48.36
18:00	0.25	36.2	48.58
19:00	0.25	36.6	48.88
20:00	0.24	37.58	49.91
21:00	0.24	38.58	50.94
22:00	0.24	39.05	51.3
23:00	0.24	39.22	51.27
24:00	0.24	39.32	51.17

B-6: CCME WQI CALCULATION FOR TANK 7N IN SOLUTION 1

	CODE	CCME Data
NO. OF FAILED VARIABLES (Paramaters)	X	0
TOTAL NUMBER OF VARIABLES (Parameters Studied)	Y	3
TOTAL NUMBER OF TESTS	Z	72
TOTAL NUMBER OF FAILED TESTS (ALL PARAMETERS EXCEEDING)	E	0

Step 1: Calculate F_1

$$F_1 = \left(\frac{Number\ of\ failed\ variable}{Total\ Number\ of\ Variable}\right) x\ 100$$

F_1	0

Step 2: Calculate F_2

$$F_2 = \left(\frac{Number\ of\ failed\ test}{Total\ Number\ of\ Tests}\right) x\ 100$$

F_2	0.00

Step 3.1: Calculate Excursions

Excursion

$$Excursion\ i = \left(\frac{Failed\ test\ i}{Objective\ j}\right) - 1$$

FAILED TEST VALUE	**A**
OBJECTIVE	**B**
	A/B
C = Excursion	**C = A/B MINUS 1**
	C = 0

Step 3.2: Calculate Nse

Normalised sum excursion (Nse)

$$nse = \frac{\sum_{i=1}^{n} Excursion\ i}{Total\ number\ of\ tests}$$

Nse	∑ [C]/Z	0

Step 3.3: Calculate F_3

$$F_3 = \left(\frac{nse}{0.01\ x\ nse + 0.01}\right) = \left(\frac{nse}{nse + 1}\right)\ x\ 100$$

Nse value	0.01 * Nse	0.01*Nse + 0.01	F_3
0	0	0.01	0

Step 4: Overall CCME Calculation

$$CCME\ WQI = 100 - \frac{\sqrt{F_1^2 + F_2^2 + F_3^2}}{1.732}$$

Component of CCME WQI	Value	Square Value
F_1	0	0
F_2	0.00	0
F_3	0	0
	Sum	0
	Square root value	0
	Divide by 1.732 (D)	0
	D	0

CCME WQI for Tank 7N

100 - D	100

CCME WQI Value for Tank 7N = 100

APPENDIX C: WATER QUALITY PERFORMANCE FOR SOLUTION 2 (SIEW ET AL, 2016)

C-1: WATER QUALITY VARIATIONS FOR TANK 41E

Parameters	Objective
Water Age (hours)	<48
Chlorine (mg/L)	≥0.2
THM (µg/L)	<100

Highlighted values indicate failed test values

Time Period	Chlorine (mg/L)	Water age (hours)	THM (µg/L)
00:00	0.25	30.57	46.21
01:00	0.33	22.39	34.23
02:00	0.36	19.03	29.38
03:00	0.37	17.51	27.25
04:00	0.39	15.62	24.64
05:00	0.39	14.61	23.35
06:00	0.39	14.52	23.49
07:00	0.39	15.52	25.07
08:00	0.38	16.52	26.61
09:00	0.37	17.52	28.12
10:00	0.36	18.52	29.61
11:00	0.35	19.52	31.06
12:00	0.35	20.52	32.48
13:00	0.34	21.52	33.87
14:00	0.33	22.52	35.24
15:00	0.33	23.52	36.57
16:00	0.32	24.52	37.88
17:00	0.31	25.52	39.16
18:00	0.27	26.52	40.41
19:00	0.2	27.52	41.64
20:00	0.01	28.52	42.85
21:00	0.2	29.52	44.03
22:00	0.28	30.52	45.18
23:00	0.28	31.52	46.31
24:00	0.27	32.52	47.42

C-2: CCME WQI CALCULATION FOR TANK 41E IN SOLUTION 2

	CODE	CCME Data
NO. OF FAILED VARIABLES (Paramaters)	**X**	1
TOTAL NUMBER OF VARIABLES (Parameters Studied)	**Y**	3
TOTAL NUMBER OF TESTS	**Z**	72
TOTAL NUMBER OF FAILED TESTS (ALL PARAMETERS EXCEEDING)	**E**	1

Step 1: Calculate F_1

$$F_1 = \left(\frac{Number\ of\ failed\ variable}{Total\ Number\ of\ Variable}\right) x\ 100$$

F_1	33.33333333

Step 2: Calculate F_2

$$F_2 = \left(\frac{Number\ of\ failed\ test}{Total\ Number\ of\ Tests}\right) x\ 100$$

F_2	1.39

Step 3.1: Calculate Excursions

Excursion

For cases when the test value must not exceed the objective:

$$Excursion\ i = \left(\frac{Failed\ test\ i}{Objective\ j}\right) - 1$$

For cases when the test value must not fall below the objective

$$Excursion\ i = \left(\frac{Objective\ j}{Failed\ test\ Value\ i}\right) - 1$$

		Chlorine
FAILED TEST VALUE	**A**	0.01
OBJECTIVE	**B**	0.2
	B/A	20
C = Excursion	**C = B/A MINUS 1**	19
	C = 19	

Step 3.2: Calculate Nse

Normalised sum excursion (nse)

$$nse = \frac{\sum_{i=1}^{n} Excursion\ i}{Total\ number\ of\ tests}$$

Nse	$\sum$ [C]/Z	0.263888889

Step 3.3: Calculate F_3

$$F_3 = \left(\frac{nse}{0.01\ x\ nse + 0.01}\right) = \left(\frac{nse}{nse + 1}\right) x\ 100$$

Nse value	0.01 * Nse	0.01*Nse + 0.01	F_3
0.264	0.003	0.013	20.879

Step 4: Overall CCME Calculation

$$CCME\ WQI = 100 - \frac{\sqrt{F_1^2 + F_2^2 + F_3^2}}{1.732}$$

Component of CCME WQI	Value	Square Value
F_1	33.33333333	1111.111
F_2	1.39	1.929
F_3	20.87912088	435.938
	Sum	1548.978
	Square root value	39.357
	Divide by 1.732 (D)	22.723
	D	22.723

CCME WQI for Tank 41E in Solution 2	100 - D	77.28

CCME WQI Value for Tank 41E in Solution 2 = 77.28

C-3: WATER QUALITY VARIATIONS FOR TANK 42E IN SOLUTION 2

Parameters	**Objective**
Water Age (hours)	<48
Chlorine (mg/L)	≥0.2
THM (µg/L)	<100

Time Period	**Chlorine (mg/L)**	**Water age (hours)**	**THM (µg/L)**
00:00	0.36	21.09	32.31
01:00	0.4	17.45	26.89
02:00	0.41	16.4	25.38
03:00	0.42	15.97	24.81
04:00	0.43	14.91	23.32
05:00	0.43	15.12	23.74
06:00	0.42	16.12	25.31
07:00	0.41	17.12	26.85
08:00	0.4	18.12	28.36
09:00	0.39	19.12	29.84
10:00	0.38	20.12	31.29
11:00	0.38	21.12	32.7
12:00	0.37	22.12	34.09
13:00	0.36	22.65	34.74
14:00	0.36	23.65	36.09
15:00	0.35	24.65	37.41
16:00	0.34	25.65	38.7
17:00	0.34	26.65	39.96
18:00	0.32	27.65	41.2
19:00	0.35	25.86	38.43
20:00	0.34	26.67	39.43
21:00	0.33	27.67	40.68
22:00	0.37	23.74	34.93
23:00	0.37	22.87	33.64
24:00	0.38	22.66	33.3

C-4: CCME WQI CALCULATION FOR TANK 42E IN SOLUTION 2

	CODE	CCME Data
NO. OF FAILED VARIABLES (Paramaters)	**X**	0
TOTAL NUMBER OF VARIABLES (Parameters Studied)	**Y**	3
TOTAL NUMBER OF TESTS	**Z**	72
TOTAL NUMBER OF FAILED TESTS (ALL PARAMETERS EXCEEDING)	**E**	0

Step 1: Calculate F_1

$$F_1 = \left(\frac{Number\ of\ failed\ variable}{Total\ Number\ of\ Variable}\right) x\ 100$$

F_1	0

Step 2: Calculate F2

$$F_2 = \left(\frac{Number\ of\ failed\ test}{Total\ Number\ of\ Tests}\right) x\ 100$$

F_2	0.00

Step 3.1: Calculate Excursions

Excursion

$$Excursion\ i = \left(\frac{Failed\ test\ i}{Objective\ j}\right) - 1$$

FAILED TEST VALUE	**A**
OBJECTIVE	**B**
	A/B
C = Excursion	**C = A/B MINUS 1**
	C = 0

Step 3.2: Calculate Nse

Normalised sum excursion (Nse)

$$nse = \frac{\sum_{i=1}^{n} Excursion\ i}{Total\ number\ of\ tests}$$

Nse	∑[C]/Z	0

Step 3.3: Calculate F_3

$$F_3 = \left(\frac{nse}{0.01\ x\ nse + 0.01}\right) = \left(\frac{nse}{nse + 1}\right) x\ 100$$

Nse value	0.01 * Nse	0.01*Nse + 0.01	F_3
0	0	0.01	0

Step 4: Overall CCME Calculation

$$CCME\ WQI = 100 - \frac{\sqrt{F_1^2 + F_2^2 + F_3^2}}{1.732}$$

Component of CCME WQI	Value	Square Value
F_1	0	0
F_2	0.00	0
F_3	0	0
	Sum	0
	Square root value	0
	Divide by 1.732 (D)	0
	D	0

CCME WQI for Tank 42E in Solution 2	100 - D	100

CCME WQI Value for Tank 42E in Solution 2 = 100

C-5: WATER QUALITY VARIATIONS FOR TANK 6N IN SOLUTION 2

Parameters	**Objective**
Water Age (hours)	<48
Chlorine (mg/L)	≥0.2
THM (µg/L)	<100

Time Period	**Chlorine (mg/L)**	**Water age (hours)**	**THM (µg/L)**
00:00	0.24	31.67	45.67
01:00	0.26	29.6	42.64
02:00	0.28	27.74	39.88
03:00	0.3	26.36	38.02
04:00	0.31	24.9	36.01
05:00	0.33	23.74	34.45
06:00	0.32	24.74	35.8
07:00	0.31	25.74	37.12
08:00	0.31	26.74	38.42
09:00	0.3	27.74	39.69
10:00	0.29	28.74	40.93
11:00	0.29	29.74	42.15
12:00	0.28	30.74	43.35
13:00	0.28	31.74	44.51
14:00	0.27	32.74	45.66
15:00	0.26	33.74	46.78
16:00	0.26	34.74	47.88
17:00	0.25	35.74	48.95
18:00	0.25	36.74	50
19:00	0.24	37.74	51.03
20:00	0.24	38.74	52.04
21:00	0.22	39.74	53.03
22:00	0.24	39.02	51.85
23:00	0.25	37.97	50.3
24:00	0.26	37.15	49.07

C-6: CCME WQI CALCULATION FOR TANK 6N IN SOLUTION 2

	CODE	CCME Data
NO. OF FAILED VARIABLES (Paramaters)	**X**	0
TOTAL NUMBER OF VARIABLES (Parameters Studied)	**Y**	3
TOTAL NUMBER OF TESTS	**Z**	72
TOTAL NUMBER OF FAILED TESTS (ALL PARAMETERS EXCEEDING)	**E**	0

Step 1: Calculate F_1

$$F_1 = \left(\frac{Number\ of\ failed\ variable}{Total\ Number\ of\ Variable}\right) x\ 100$$

F_1	0

Step 2: Calculate F_2

$$F_2 = \left(\frac{Number\ of\ failed\ test}{Total\ Number\ of\ Tests}\right) x\ 100$$

F_2	0.00

Step 3.1: Calculate Excursions

Excursion

$$Excursion\ i = \left(\frac{Failed\ test\ i}{Objective\ j}\right) - 1$$

FAILED TEST VALUE	**A**
OBJECTIVE	**B**
	A/B
C = Excursion	**C = A/B MINUS 1**
	C = 0

Step 3.2: Calculate Nse

Normalised sum excursion (Nse)

$$nse = \frac{\sum_{i=1}^{n} Excursion\ i}{Total\ number\ of\ tests}$$

Nse	∑ [C]/Z	0

Step 3.3: Calculate F_3

$$F_3 = \left(\frac{nse}{0.01\ x\ nse + 0.01}\right) = \left(\frac{nse}{nse + 1}\right) x\ 100$$

Nse value	0.01 * Nse	0.01*Nse + 0.01	F_3
0	0	0.01	0

Step 4: Overall CCME Calculation

$$CCME\ WQI = 100 - \frac{\sqrt{F_1^2 + F_2^2 + F_3^2}}{1.732}$$

Component of CCME WQI	Value	Square Value
F_1	0	0
F_2	0.00	0
F_3	0	0
	Sum	0
	Square root value	0
	Divide by 1.732 (D)	0
	D	0

CCME WQI for Tank 6N in Solution 2	100 - D	100

CCME WQI Value for Tank 6N in Solution 2 = 100

APPENDIX D: WATER QUALITY PERFORMANCE FOR PRASAD (2010) SOLUTION

D-1: WATER QUALITY VARIATIONS FOR TANK 41E

Parameters	Objective
Water Age (hours)	<48
Chlorine (mg/L)	≥0.2
THM (µg/L)	<100

Time Period	Chlorine (mg/L)	Water age (hours)	THM (µg/L)
00:00	0.39	24.24	36.85
01:00	0.4	17.4	26.98
02:00	0.4	14.62	23.06
03:00	0.39	13.62	21.74
04:00	0.38	14.16	22.7
05:00	0.38	15.16	24.3
06:00	0.37	16.16	25.86
07:00	0.36	17.16	27.39
08:00	0.35	18.16	28.88
09:00	0.35	19.16	30.35
10:00	0.35	20.16	31.79
11:00	0.35	21.16	33.19
12:00	0.35	22.16	34.57
13:00	0.35	23.16	35.92
14:00	0.35	24.16	37.24
15:00	0.35	25.16	38.53
16:00	0.35	26.16	39.8
17:00	0.35	27.16	41.04
18:00	0.35	28.16	42.26
19:00	0.36	29.16	43.45
20:00	0.36	30.16	44.62
21:00	0.37	31.16	45.76
22:00	0.39	30.44	44.49
23:00	0.39	27.9	40.76
24:00	0.4	25.92	37.91

D-2: CCME WQI CALCULATION FOR TANK 41E IN PRASAD (2010)

	CODE	CCME Data
NO. OF FAILED VARIABLES (Paramaters)	**X**	0
TOTAL NUMBER OF VARIABLES (Parameters Studied)	**Y**	3
TOTAL NUMBER OF TESTS	**Z**	72
TOTAL NUMBER OF FAILED TESTS (ALL PARAMETERS EXCEEDING)	**E**	0

Step 1: Calculate F_1

$$F_1 = \left(\frac{Number\ of\ failed\ variable}{Total\ Number\ of\ Variable}\right) x\ 100$$

F_1	0

Step 2: Calculate F_2

$$F_2 = \left(\frac{Number\ of\ failed\ test}{Total\ Number\ of\ Tests}\right) x\ 100$$

F_2	0.00

Step 3.1: Calculate Excursions

Excursion

$$Excursion\ i = \left(\frac{Failed\ test\ i}{Objective\ j}\right) - 1$$

FAILED TEST VALUE	**A**
OBJECTIVE	**B**
	A/B
C = Excursion	**C = A/B MINUS 1**
	C = 0

Step 3.2: Calculate Nse

Normalised sum excursion (Nse)

$$nse = \frac{\sum_{i=1}^{n} Excursion\ i}{Total\ number\ of\ tests}$$

Nse =	∑ [C]/Z	0

Step 3.3: Calculate F_3

$$F_3 = \left(\frac{nse}{0.01\ x\ nse + 0.01}\right) = \left(\frac{nse}{nse + 1}\right)\ x\ 100$$

Nse value	0.01 * Nse	0.01*Nse + 0.01	F_3
0	0	0.01	0

Step 4: Overall CCME Calculation

$$CCME\ WQI = 100 - \frac{\sqrt{F_1^2 + F_2^2 + F_3^2}}{1.732}$$

Component of CCME WQI	Value	Square Value
F_1	0	0
F_2	0.00	0
F_3	0	0
	Sum	0
	Square root value	0
	Divide by 1.732 (D)	0
	D	0

CCME WQI for Tank 41E in Prasad (2010)	100 - D	100

CCME WQI Value for Tank 41E in Prasad (2010) = 100

D-3: WATER QUALITY VARIATIONS FOR TANK 42E IN PRASAD (2010)

Parameters	Objective
Water Age (hours)	<48
Chlorine (mg/L)	≥0.2
THM (µg/L)	<100

Time Period	Chlorine (mg/L)	Water age (hours)	THM (µg/L)
00:00	0.43	17.51	27.08
01:00	0.42	12.59	19.88
02:00	0.41	13.02	20.7
03:00	0.4	14.02	22.34
04:00	0.39	15.02	23.94
05:00	0.39	16.02	25.51
06:00	0.38	17.02	27.04
07:00	0.37	18.02	28.55
08:00	0.36	19.02	30.02
09:00	0.36	20.02	31.47
10:00	0.35	21.02	32.88
11:00	0.34	22.02	34.26
12:00	0.34	23.02	35.62
13:00	0.34	24.02	36.95
14:00	0.33	25.02	38.25
15:00	0.33	26.02	39.52
16:00	0.33	27.02	40.77
17:00	0.33	28.02	41.99
18:00	0.33	29.02	43.18
19:00	0.43	28.57	42.29
20:00	0.45	28.5	41.96
21:00	0.45	28.82	42.19
22:00	0.45	21.17	31.24
23:00	0.45	19.16	28.43
24:00	0.44	18.68	27.83

D-4: CCME WQI CALCULATION FOR TANK 42E IN PRASAD (2010)

	CODE	CCME Data
NO. OF FAILED VARIABLES (Paramaters)	**X**	0
TOTAL NUMBER OF VARIABLES (Parameters Studied)	**Y**	3
TOTAL NUMBER OF TESTS	**Z**	72
TOTAL NUMBER OF FAILED TESTS (ALL PARAMETERS EXCEEDING)	**E**	0

Step 1: Calculate F_1

$$F_1 = \left(\frac{Number\ of\ failed\ variable}{Total\ Number\ of\ Variable}\right) x\ 100$$

F_1	0

Step 2: Calculate F_2

$$F_2 = \left(\frac{Number\ of\ failed\ test}{Total\ Number\ of\ Tests}\right) x\ 100$$

F_2	0.00

Step 3.1: Calculate Excursions

Excursion

$$Excursion\ i = \left(\frac{Failed\ test\ i}{Objective\ j}\right) - 1$$

FAILED TEST VALUE	**A**
OBJECTIVE	**B**
	A/B
C = Excursion	**C = A/B MINUS 1**
	C = 0

Step 3.2: Calculate Nse

Normalised sum excursion (Nse)

$$nse = \frac{\sum_{i=1}^{n} Excursion\ i}{Total\ number\ of\ tests}$$

0

Step 3.3: Calculate F_3

$$F_3 = \left(\frac{nse}{0.01\ x\ nse + 0.01}\right) = \left(\frac{nse}{nse + 1}\right)\ x\ 100$$

Nse value	0.01 * Nse	0.01*Nse + 0.01	F_3
0	0	0.01	0

Step 4: Overall CCME Calculation

$$CCME\ WQI = 100 - \frac{\sqrt{F_1^2 + F_2^2 + F_3^2}}{1.732}$$

Component of CCME WQI	Value	Square Value
F_1	0	0
F_2	0.00	0
F_3	0	0
	Sum	0
	Square root value	0
	Divide by 1.732 (D)	0

D 0

CCME WQI for Tank 42E in Prasad (2010)	100 - D	100

CCME WQI Value for Tank 42E in Prasad (2010) = 100

D-5: WATER QUALITY VARIATIONS FOR TANK 150N IN PRASAD (2010)

Parameters	**Objective**
Water Age (hours)	<48
Chlorine (mg/L)	≥0.2
THM (µg/L)	<100

Highlighted values indicate failed test values

Time Period	**Chlorine (mg/L)**	**Water age (hours)**	**THM (µg/L)**
00:00	0.2	39.51	55.47
01:00	0.23	36	50.65
02:00	0.24	31.67	44.95
03:00	0.23	28.79	41.3
04:00	0.23	28.82	41.33
05:00	0.22	29.82	42.54
06:00	0.22	30.82	43.73
07:00	0.22	31.82	44.89
08:00	0.21	32.82	46.02
09:00	0.21	33.82	47.14
10:00	0.2	34.82	48.23
11:00	0.2	35.82	49.29
12:00	0.19	36.82	50.34
13:00	0.19	37.82	51.36
14:00	0.19	38.82	52.37
15:00	0.18	39.82	53.35
16:00	0.18	40.82	54.31
17:00	0.17	41.82	55.25
18:00	0.17	42.82	56.18
19:00	0.17	43.82	57.08
20:00	0.16	44.82	57.96
21:00	0.17	45.82	58.83
22:00	0.19	46.82	59.68
23:00	0.21	47.82	60.51
24:00	0.23	48.82	61.33

D-6: CCME WQI CALCULATION FOR TANK 150N IN PRASAD (2010)

	CODE	CCME Data
NO. OF FAILED VARIABLES (Paramaters)	X	2
TOTAL NUMBER OF VARIABLES (Parameters Studied)	Y	3
TOTAL NUMBER OF TESTS	Z	72
TOTAL NUMBER OF FAILED TESTS (ALL PARAMETERS EXCEEDING)	E	12

Step 1: Calculate F_1

$$F_1 = \left(\frac{Number\ of\ failed\ variable}{Total\ Number\ of\ Variable}\right) x\ 100$$

F_1	66.66666667

Step 2: Calculate F_2

$$F_2 = \left(\frac{Number\ of\ failed\ test}{Total\ Number\ of\ Tests}\right) x\ 100$$

F_2	16.67

Step 3.1: Calculate Excursions

Excursion

Step 3.1: For cases when the test value must not exceed the objective:

$$Excursion\ i = \left(\frac{Failed\ test\ i}{Objective\ j}\right) - 1$$

Water Age			
FAILED TEST VALUE (A)	**OBJECTIVE (B)**	**A/B**	**C = A/B MINUS 1**
48.82	48	1.017	0.017
		∑C =	0.017

For cases when the test value must not fall below the objective

$$Excursion\ i = \left(\frac{Objective\ j}{Failed\ test\ Value\ i}\right) - 1$$

Chlorine			
FAILED TEST VALUE (A)	**OBJECTIVE (B)**	**B/A**	**C = B/A MINUS 1**
0.19	0.2	1.053	0.053
0.19	0.2	1.053	0.053
0.19	0.2	1.053	0.053
0.18	0.2	1.111	0.111
0.18	0.2	1.111	0.111
0.17	0.2	1.176	0.176
0.17	0.2	1.176	0.176
0.17	0.2	1.176	0.176
0.16	0.2	1.250	0.250
0.17	0.2	1.176	0.176
0.19	0.2	1.053	0.053
		∑C =	1.389
		∑C (Water age & Chlorine)	1.406

Step 3.2: Calculate Nse

Normalised sum excursion (nse)

$$nse = \frac{\sum_{i=1}^{n} Excursion\ i}{Total\ number\ of\ tests}$$

Nse = ∑ [C]/Z 0.0195

Step 3.3: Calculate F_3

$$F_3 = \left(\frac{nse}{0.01\ x\ nse + 0.01}\right) = \left(\frac{nse}{nse + 1}\right)\ x\ 100$$

Nse value	0.01 * Nse	0.01*Nse + 0.01	F_3
0.0195	0.0002	0.0102	1.9150

Step 4: Overall CCME Calculation

$$CCME\ WQI = 100 - \frac{\sqrt{F_1^2 + F_2^2 + F_3^2}}{1.732}$$

Component of CCME WQI	Value	Square Value
F_1	66.6667	4444.444
F_2	16.6667	277.778
F_3	1.9150	3.667
	Sum	4725.889
	Square root value	68.745
	Divide by 1.732 (D)	39.691
	D	39.691

CCME WQI for Tank 150N in Prasad (2010) Solution	100 - D	60.309

CCME WQI Value for Tank 150N in Prasad (2010) = 60.31

D-7: WATER QUALITY VARIATIONS FOR TANK 170N IN PRASAD (2010)

Parameters	**Objective**
Water Age (hours)	<48
Chlorine (mg/L)	≥0.2
THM (µg/L)	<100

Highlighted values indicate failed test values

Time Period	Chlorine (mg/L)	Water age (hours)	THM (µg/L)
00:00	0.25	32.68	50.5
01:00	0.27	33.71	46.36
02:00	0.29	34.22	41.56
03:00	0.31	35.62	37.98
04:00	0.31	36.53	33.78
05:00	0.3	37.87	31.57
06:00	0.3	38.87	32.98
07:00	0.29	39.87	34.36
08:00	0.28	40.87	35.72
09:00	0.28	41.87	37.04
10:00	0.27	42.87	38.34
11:00	0.27	43.87	39.61
12:00	0.26	44.87	40.86
13:00	0.26	45.87	42.08
14:00	0.25	46.87	43.27
15:00	0.24	47.87	44.44
16:00	0.24	49.97	45.59
17:00	0.23	51.87	46.71
18:00	0.23	47.87	47.81
19:00	0.23	46.87	48.88
20:00	0.22	45.87	49.94
21:00	0.23	44.87	50.97
22:00	0.24	43.87	51.98
23:00	0.26	41.87	52.97
24:00	0.27	39.55	53.56

D-8: CCME WQI CALCULATION FOR TANK 170N IN PRASAD (2010)

	CODE	CCME Data
NO. OF FAILED VARIABLES (Paramaters)	**X**	1
TOTAL NUMBER OF VARIABLES (Parameters Studied)	**Y**	3
TOTAL NUMBER OF TESTS	**Z**	72
TOTAL NUMBER OF FAILED TESTS (ALL PARAMETERS EXCEEDING)	**E**	2

Step 1: Calculate F_1

$$F_1 = \left(\frac{Number\ of\ failed\ variable}{Total\ Number\ of\ Variable}\right) x\ 100$$

F_1	33.33333333

Step 2: Calculate F_2

$$F_2 = \left(\frac{Number\ of\ failed\ test}{Total\ Number\ of\ Tests}\right) x\ 100$$

F_2	2.78

Step 3.1: Calculate Excursions

Excursion

Step 3.1: For cases when the test value must not exceed the objective:

$$Excursion\ i = \left(\frac{Failed\ test\ i}{Objective\ j}\right) - 1$$

		Water age	
FAILED TEST VALUE	**A**	49.97	51.87
OBJECTIVE	**B**	48	48
	A/B	1.0410	1.0806
C = Excursion	**C = A/B MINUS 1**	0.0410	0.0806
	C =	0.0410	0.0806

For cases when the test value must not fall below the objective

$$Excursion\ i = \left(\frac{Objective\ j}{Failed\ test\ Value\ i}\right) - 1$$

FAILED TEST VALUE	**A**
OBJECTIVE	**B**
	B/A
C = Excursion	**C = B/A MINUS 1**

∑C =	0.1217

Step 3.2: Calculate Nse

Normalised sum excursion (Nse)

$$nse = \frac{\sum_{i=1}^{n} Excursion\ i}{Total\ number\ of\ tests}$$

Nse =	∑ [C]/Z	0.0017

Step 3.3: Calculate F_3

$$F_3 = \left(\frac{nse}{0.01\ x\ nse + 0.01}\right) = \left(\frac{nse}{nse + 1}\right)\ x\ 100$$

Nse value	0.01 * Nse	0.01*Nse + 0.01	F3
0.0017	0.0000	0.0100	0.1687

Step 4: Overall CCME Calculation

$$CCME\ WQI = 100 - \frac{\sqrt{F_1^2 + F_2^2 + F_3^2}}{1.732}$$

Component of CCME WQI	Value	Square Value
F1	33.3333	1111.111
F2	2.7778	7.716
F3	0.1687	0.028
	Sum	1118.856
	Square root value	33.449
	Divide by 1.732 (D)	19.313
	D	19.313

CCME WQI for Tank 170N in Prasad (2010) Solution	100 - D	80.6875

CCME WQI Value for Tank 170N in Prasad (2010) = 80.69

APPENDIX E: WATER QUALTITY PERFORMANCE FOR WALTERS ET AL (1999) SOLUTION

E-1: WATER QUALITY VARIATIONS FOR TANK 41E IN WALTERS ET AL (1999)

Parameters	Objective
Water Age (hours)	<48
Chlorine (mg/L)	≥0.2
THM (µg/L)	<100

Highlighted values indicate failed test values

Time Period	Chlorine (mg/L)	Water age (hours)	THM (µg/L)
00:00	0.01	48.93	66.49
01:00	0.01	49.22	65.18
02:00	0.01	50.86	61.62
03:00	0.01	51.98	68.84
04:00	0.01	52.6	54.09
05:00	0.01	53.19	49.47
06:00	0.01	54.01	45.47
07:00	0.01	68.01	47.01
08:00	0.01	59.01	48.52
09:00	0.01	60.01	49.99
10:00	0.01	61.01	51.43
11:00	0.01	62.01	52.85
12:00	0.01	63.01	54.23
13:00	0.01	64.01	65.59
14:00	0.01	65.01	66.92
15:00	0.01	66.01	68.22
16:00	0.01	66.51	69.49
17:00	0.01	66.92	70.74
18:00	0.01	67.01	71.96
19:00	0.01	67.91	73.16
20:00	0.01	68.01	74.33
21:00	0.01	69.01	75.48
22:00	0.01	70.01	76.6
23:00	0.01	71.01	77.7
24:00	0.01	72	78.1

E-2: CCME WQI CALCULATION FOR TANK 41E IN WALTERS ET AL (1999)

	CODE	CCME Data
NO. OF FAILED VARIABLES (Paramaters)	**X**	2
TOTAL NUMBER OF VARIABLES (Parameters Studied)	**Y**	3
TOTAL NUMBER OF TESTS	**Z**	72
TOTAL NUMBER OF FAILED TESTS (ALL PARAMETERS EXCEEDING)	**E**	48

Step 1: Calculate F_1

$$F_1 = \left(\frac{Number\ of\ failed\ variable}{Total\ Number\ of\ Variable}\right) x\ 100$$

F_1	66.66666667

Step 2: Calculate F_2

$$F_2 = \left(\frac{Number\ of\ failed\ test}{Total\ Number\ of\ Tests}\right) x\ 100$$

F_2	66.67

Step 3.1: Calculate Excursions

Excursion

Step 3.1: For cases when the test value must not exceed the objective:

$$Excursion\ i = \left(\frac{Failed\ test\ i}{Objective\ j}\right) - 1$$

Water Age			
FAILED TEST VALUE (A)	**OBJECTIVE (B)**	**A/B**	**C = A/B MINUS 1**
48.93	48	1.019	0.019
49.22	48	1.025	0.025
50.86	48	1.060	0.060
51.98	48	1.083	0.083
52.6	48	1.096	0.096
53.19	48	1.108	0.108
54.01	48	1.125	0.125
68.01	48	1.417	0.417
59.01	48	1.229	0.229
60.01	48	1.250	0.250
61.01	48	1.271	0.271
62.01	48	1.292	0.292
63.01	48	1.313	0.313
64.01	48	1.334	0.334
65.01	48	1.354	0.354
66.01	48	1.375	0.375
66.51	48	1.386	0.386
66.92	48	1.394	0.394
67.01	48	1.396	0.396
67.91	48	1.415	0.415
68.01	48	1.417	0.417
69.01	48	1.438	0.438
70.01	48	1.459	0.459
71.01	48	1.479	0.479
72	48	1.500	0.500
			∑C = 7.235

For cases when the test value must not fall below the objective

$$Excursion\ i = \left(\frac{Objective\ j}{Failed\ test\ Value\ i}\right) - 1$$

Chlorine			
FAILED TEST VALUE (A)	OBJECTIVE (B)	B/A	C = B/A MINUS 1
0.01	0.2	20	19
0.01	0.2	20	19
0.01	0.2	20	19
0.01	0.2	20	19
0.01	0.2	20	19
0.01	0.2	20	19
0.01	0.2	20	19
0.01	0.2	20	19
0.01	0.2	20	19
0.01	0.2	20	19
0.01	0.2	20	19
0.01	0.2	20	19
0.01	0.2	20	19
0.01	0.2	20	19
0.01	0.2	20	19
0.01	0.2	20	19
0.01	0.2	20	19
0.01	0.2	20	19
0.01	0.2	20	19
0.01	0.2	20	19
0.01	0.2	20	19
0.01	0.2	20	19
0.01	0.2	20	19
0.01	0.2	20	19
0.01	0.2	20	19
		∑C =	475
		∑C (water age & chlorine)	482.235

Step 3.2: Calculate Nse

Normalised sum excursion (nse)

$$nse = \frac{\sum_{i=1}^{n} Excursion\ i}{Total\ number\ of\ tests}$$

Nse = ∑ [C]/Z 6.6977

Step 3.3: Calculate F_3

$$F_3 = \left(\frac{nse}{0.01\ x\ nse + 0.01}\right) = \left(\frac{nse}{nse + 1}\right)\ x\ 100$$

Nse value	0.01 * Nse	0.01*Nse + 0.01	F_3
6.6977	0.0670	0.0770	87.0091

Step 4: Overall CCME Calculation

$$CCME\ WQI = 100 - \frac{\sqrt{F_1^2 + F_2^2 + F_3^2}}{1.732}$$

Component of CCME WQI	Value	Square Value
F_1	66.6667	4444.444
F_2	66.6667	4444.444
F_3	87.0091	7570.587
	Sum	16459.476
	Square root value	128.294
	Divide by 1.732 (D)	74.073
	D	74.073

CCME WQI for Tank 41E in Walters (1999) Solution	100 - D	25.927

CCME WQI Value for Tank 41E in Walters (1999) = 25.93

E-3: WATER QUALITY VARIATIONS FOR TANK 42E IN WALTERS ET AL (1999)

Parameters	Objective
Water Age (hours)	<48
Chlorine (mg/L)	≥0.2
THM (µg/L)	<100

Highlighted values indicate failed test values

Time Period	Chlorine (mg/L)	Water age (hours)	THM (µg/L)
00:00	0.01	47.98	69.37
01:00	0.01	48.92	66
02:00	0.01	49.86	67.22
03:00	0.01	50.68	68.48
04:00	0.01	51.5	69.32
05:00	0.01	52.67	69.88
06:00	0.01	53.01	71.01
07:00	0.01	54.01	72.64
08:00	0.01	55.01	74.23
09:00	0.01	56.01	75.79
10:00	0.01	57.01	77.32
11:00	0.01	58.01	77.90
12:00	0.01	59.01	70.29
13:00	0.01	60.01	71.73
14:00	0.01	61.01	73.14
15:00	0.01	62.01	74.52
16:00	0.01	63.51	75.87
17:00	0.01	64.92	77.19
18:00	0.01	65.01	78.1
19:00	0.01	66.91	69.75
20:00	0.01	67.01	68.99
21:00	0.01	68.01	66.21
22:00	0.01	69.01	66.26
23:00	0.01	70.01	61.65
24:00	0.01	71	60.77

E-4: CCME WQI CALCULATION FOR TANK 42E IN WALTERS ET AL (1999)

	CODE	CCME Data
NO. OF FAILED VARIABLES (Paramaters)	X	2
TOTAL NUMBER OF VARIABLES (Parameters Studied)	Y	3
TOTAL NUMBER OF TESTS	Z	72
TOTAL NUMBER OF FAILED TESTS (ALL PARAMETERS EXCEEDING)	E	47

Step 1: Calculate F_1

$$F_1 = \left(\frac{Number\ of\ failed\ variable}{Total\ Number\ of\ Variable}\right) x\ 100$$

F_1	66.66666667

Step 2: Calculate F_2

$$F_2 = \left(\frac{Number\ of\ failed\ test}{Total\ Number\ of\ Tests}\right) x\ 100$$

F_2	65.28

Step 3.1: Calculate Excursions

Excursion

Step 3.1: For cases when the test value must not exceed the objective:

$$Excursion\ i = \left(\frac{Failed\ test\ i}{Objective\ j}\right) - 1$$

Water Age			
FAILED TEST VALUE (A)	**OBJECTIVE (B)**	**A/B**	**C = A/B MINUS 1**
48.92	48	1.019	0.019
49.86	48	1.039	0.039
50.68	48	1.056	0.056
51.5	48	1.073	0.073
52.67	48	1.097	0.097
53.01	48	1.104	0.104
54.01	48	1.125	0.125
55.01	48	1.146	0.146
56.01	48	1.167	0.167
57.01	48	1.188	0.188
58.01	48	1.209	0.209
59.01	48	1.229	0.229
60.01	48	1.250	0.250
61.01	48	1.271	0.271
62.01	48	1.292	0.292
63.51	48	1.323	0.323
64.92	48	1.353	0.353
65.01	48	1.354	0.354
66.91	48	1.394	0.394
67.01	48	1.396	0.396
68.01	48	1.417	0.417
69.01	48	1.438	0.438
70.01	48	1.459	0.459
71	48	1.479	0.479
			∑C = 5.878

For cases when the test value must not fall below the objective

$$Excursion\ i = \left(\frac{Objective\ j}{Failed\ test\ Value\ i}\right) - 1$$

Chlorine			
FAILED TEST VALUE (A)	**OBJECTIVE (B)**	**B/A**	**C = B/A MINUS 1**
0.01	0.2	20	19
0.01	0.2	20	19
0.01	0.2	20	19
0.01	0.2	20	19
0.01	0.2	20	19
0.01	0.2	20	19
0.01	0.2	20	19
0.01	0.2	20	19
0.01	0.2	20	19
0.01	0.2	20	19
0.01	0.2	20	19
0.01	0.2	20	19
0.01	0.2	20	19
0.01	0.2	20	19
0.01	0.2	20	19
0.01	0.2	20	19
0.01	0.2	20	19
0.01	0.2	20	19
0.01	0.2	20	19
0.01	0.2	20	19
0.01	0.2	20	19
0.01	0.2	20	19
0.01	0.2	20	19
0.01	0.2	20	19
0.01	0.2	20	19
			∑ C = 475
	∑C (Water age & Chlorine)		480.8775

Step 3.2: Calculate Nse

Normalised sum excursion (Nse)

$$nse = \frac{\sum_{i=1}^{n} Excursion\ i}{Total\ number\ of\ tests}$$

Nse = ∑ (C)/Z 6.6789

Step 3.3: Calculate F_3

$$F_3 = \left(\frac{nse}{0.01\ x\ nse + 0.01}\right) = \left(\frac{nse}{nse + 1}\right)\ x\ 100$$

Nse value	0.01 * Nse	0.01*Nse + 0.01	F_3
6.6789	0.0668	0.0768	86.9772

Step 4: Overall CCME Calculation

$$CCME\ WQI = 100 - \frac{\sqrt{F_1^2 + F_2^2 + F_3^2}}{1.732}$$

Component of CCME WQI	Value	Square Value
F_1	66.6667	4444.444
F_2	65.2778	4261.188
F_3	86.9772	7565.037
	Sum	16270.670
	Square root value	127.557
	Divide by 1.732 (D)	73.647
	D	73.647

CCME WQI for Tank 41E in Walters (1999) Solution	100 - D	26.353

CCME WQI Value for Tank 41E in Walters (1999) = 26.35

E-5: WATER QUALITY VARIATIONS FOR TANK 5N IN WALTERS ET AL. (1999)

Parameters	Objective
Water Age (hours)	<48
Chlorine (mg/L)	≥0.2
THM (µg/L)	<100

Highlighted values indicate failed test values

Time Period	Chlorine (mg/L)	Water age (hours)	THM (µg/L)
00:00	0.17	33.58	48.52
01:00	0.18	32.31	46.9
02:00	0.2	31.23	45.43
03:00	0.21	30.4	44.3
04:00	0.22	29.44	43.05
05:00	0.22	29.77	43.4
06:00	0.22	30.77	44.57
07:00	0.21	31.77	45.71
08:00	0.21	32.77	46.83
09:00	0.2	33.77	47.93
10:00	0.2	34.77	49
11:00	0.19	35.77	50.06
12:00	0.19	36.77	51.09
13:00	0.19	37.77	52.09
14:00	0.18	38.77	53.08
15:00	0.18	39.77	54.05
16:00	0.18	40.77	55
17:00	0.17	41.77	55.93
18:00	0.16	42.77	56.83
19:00	0.17	43.21	57.05
20:00	0.17	43.58	57.2
21:00	0.17	43.99	57.4
22:00	0.18	42.66	55.59
23:00	0.19	41.67	54.25
24:00	0.19	40.95	53.3

E-6: CCME WQI CALCULATION FOR TANK 5N IN WALTERS ET AL. (1999)

	CODE	CCME Data
NO. OF FAILED VARIABLES (Paramaters)	**X**	1
TOTAL NUMBER OF VARIABLES (Parameters Studied)	**Y**	3
TOTAL NUMBER OF TESTS	**Z**	72
TOTAL NUMBER OF FAILED TESTS (ALL PARAMETERS EXCEEDING)	**E**	16

Step 1: Calculate F_1

$$F_1 = \left(\frac{Number\ of\ failed\ variable}{Total\ Number\ of\ Variable}\right) x\ 100$$

F_1	33.33333333

Step 2: Calculate F_2

$$F_2 = \left(\frac{Number\ of\ failed\ test}{Total\ Number\ of\ Tests}\right) x\ 100$$

F_2	22.22

Step 3.1: Calculate Excursions

Excursion

Step 3.1: For cases when the test value must not exceed the objective:

$$Excursion\ i = \left(\frac{Failed\ test\ i}{Objective\ j}\right) - 1$$

FAILED TEST VALUE	**A**
OBJECTIVE	**B**
	A/B
C = Excursion	**C = A/B MINUS 1**
	C = 0

For cases when the test value must not fall below the objective

$$Excursion\ i = \left(\frac{Objective\ j}{Failed\ test\ Value\ i}\right) - 1$$

Chlorine			
FAILED TEST VALUE (A)	**OBJECTIVE (B)**	**B/A**	**C = B/A MINUS 1**
0.17	0.2	1.176	0.176
0.18	0.2	1.111	0.111
0.19	0.2	1.053	0.053
0.19	0.2	1.053	0.053
0.19	0.2	1.053	0.053
0.18	0.2	1.111	0.111
0.18	0.2	1.111	0.111
0.18	0.2	1.111	0.111
0.17	0.2	1.176	0.176
0.16	0.2	1.250	0.250
0.17	0.2	1.176	0.176
0.17	0.2	1.176	0.176
0.17	0.2	1.176	0.176
0.18	0.2	1.111	0.111
0.19	0.2	1.053	0.053
0.19	0.2	1.053	0.053
		∑ C =	1.951

Step 3.2: Calculate Nse

Normalised sum excursion (Nse)

$$nse = \frac{\sum_{i=1}^{n} Excursion\ i}{Total\ number\ of\ tests}$$

Nse = ∑ (C)/Z 0.0271

Step 3.3: Calculate F_3

$$F_3 = \left(\frac{nse}{0.01\ x\ nse + 0.01}\right) = \left(\frac{nse}{nse + 1}\right)\ x\ 100$$

Nse value	0.01 * Nse	0.01*Nse + 0.01	F_3
0.0271	0.0003	0.0103	2.6383

Step 4: Overall CCME Calculation

$$CCME\ WQI = 100 - \frac{\sqrt{F_1^2 + F_2^2 + F_3^2}}{1.732}$$

Component of CCME WQI	Value	Square Value
F_1	33.333	1111.111
F_2	22.222	493.827
F_3	2.638	6.961
	Sum	1611.899
	Square root value	40.148
	Divide by 1.732 (D)	23.180
	D	23.180

CCME WQI for Tank 5N in Walters (1999) Solution	100 - D	76.820

CCME WQI Value for Tank 5N in Walters (1999) = 76.820

E-7: WATER QUALITY VARIATIONS FOR TANK 12N IN WALTERS ET AL (1999)

Parameters	Objective
Water Age (hours)	<48
Chlorine (mg/L)	≥0.2
THM (µg/L)	<100

Time Period	Chlorine (mg/L)	Water age (hours)	THM (µg/L)
00:00	0.34	24.84	37.75
01:00	0.39	20.02	30.43
02:00	0.42	16.64	25.43
03:00	0.44	14.83	22.81
04:00	0.45	13.34	20.71
05:00	0.46	12.77	20
06:00	0.45	13.77	21.65
07:00	0.44	14.77	23.26
08:00	0.43	15.77	24.84
09:00	0.42	16.77	26.39
10:00	0.41	17.77	27.91
11:00	0.40	18.77	29.40
12:00	0.40	19.77	30.85
13:00	0.39	20.77	32.28
14:00	0.38	21.77	33.68
15:00	0.37	22.77	35.04
16:00	0.36	23.77	36.38
17:00	0.36	24.77	37.70
18:00	0.35	25.77	38.98
19:00	0.34	26.77	40.24
20:00	0.34	27.77	41.47
21:00	0.33	28.77	42.68
22:00	0.34	27.77	40.98
23:00	0.35	27.08	39.78
24:00	0.35	26.51	38.78

E-8: CCME WQI CALCULATION FOR TANK 12N IN WALTERS ET AL (1999)

	CODE	CCME Data
NO. OF FAILED VARIABLES (Paramaters)	**X**	0
TOTAL NUMBER OF VARIABLES (Parameters Studied)	**Y**	3
TOTAL NUMBER OF TESTS	**Z**	72
TOTAL NUMBER OF FAILED TESTS (ALL PARAMETERS EXCEEDING)	**E**	0

Step 1: Calculate F_1

$$F_1 = \left(\frac{Number\ of\ failed\ variable}{Total\ Number\ of\ Variable}\right) x\ 100$$

F_1	0

Step 2: Calculate F_2

$$F_2 = \left(\frac{Number\ of\ failed\ test}{Total\ Number\ of\ Tests}\right) x\ 100$$

F_2	0.00

Step 3.1: Calculate Excursions

Excursion

$$Excursion\ i = \left(\frac{Failed\ test\ i}{Objective\ j}\right) - 1$$

FAILED TEST VALUE	**A**
OBJECTIVE	**B**
	A/B
C = Excursion	**C = A/B MINUS 1**
	C = 0

Step 3.2: Calculate Nse

Normalised sum excursion (Nse)

$$nse = \frac{\sum_{i=1}^{n} Excursion\ i}{Total\ number\ of\ tests}$$

Nse = ∑ [C]/Z 0

Step 3.3: Calculate F_3

$$F_3 = \left(\frac{nse}{0.01\ x\ nse + 0.01}\right) = \left(\frac{nse}{nse + 1}\right) x\ 100$$

Nse value	0.01 * Nse	0.01*Nse + 0.01	F_3
0	0	0.01	0

Step 4: Overall CCME Calculation

$$CCME\ WQI = 100 - \frac{\sqrt{F_1^2 + F_2^2 + F_3^2}}{1.732}$$

Component of CCME WQI	Value	Square Value
F_1	0	0
F_2	0.00	0
F_3	0	0
	Sum	0
	Square root value	0
	Divide by 1.732 (D)	0
	D	0

CCME WQI for Tank 12N in Walters et al (1999)	100 - D	100

CCME WQI Value for Tank 12N in Walters et al (1999) = 100