Developing a new model to predict the diurnal water demand pattern for residential areas subjected to formal intermittent supply in South Africa

By

Thembela Lumumba Msutu

Thesis presented in partial fulfilment of the requirements for the degree of Master of Engineering in the Faculty of Engineering at Stellenbosch University.

Supervisor: Mr. Carlo Loubser

Faculty of Engineering

Department of Civil Engineering

Division of Water and Environmental Engineering

March 2020

Plagiarism declaration

By submitting this thesis electronically, I declare that the entirety of the work contained therein is my own, original work, that I am the sole author thereof (save to the extent explicitly otherwise stated), that reproduction and publication thereof by Stellenbosch University will not infringe any third party rights and that I have not previously in its entirety or in part submitted it for obtaining any qualification.

Date:...................................

Summary

Intermittent water supply (IWS) is one of the most common methods of water demand control under water scarcity conditions, however the implementation of IWS has many negative impacts and is not recommended for planned implementation (Vairavamoorthy et al., 2001; Mckenzie et al., 2014). However, in many water scarce countries, IWS is still being implemented out of necessity rather than choice (Totsuka & Trifunovic, 2004). An important step in the planning, design and analysis of water distribution systems (WDSs) to ensure adequate system performance and levels of service (LOS) is determining the peak water demand of the supply area. The peak water demand, along with the demand patterns, are used to validate the WDS capacity (CSIR, 2003; Van Zyl et al., 2008). Extensive research on the estimation of water demand and the derivation of diurnal water demand patterns for continuous water supply (CWS) WDSs has been conducted. However, there has been little similar research conducted for WDSs subjected to IWS.

The purpose of this study was to develop a new theoretical model to predict the diurnal water demand patterns for residential areas subjected to formal IWS, by taking into account the WDS filling process, as well as the crucial parameters that impact the WDS during IWS. An attempt was made to define the typical form and characteristics of the diurnal water demand patterns associated with IWS conditions, and how these patterns can be estimated when logged field data is not available.

This research consisted of a non-empirical study and an empirical study. The non-empirical study consisted of a literature review of key topics, an investigation of crucial influence parameters, and the development of the model. The empirical study consisted of the collection, processing and analysis of actual water consumption data from an area subjected to IWS, which in turn, was used for the model validation and calibration.

The developed model was based on the results of the parameters investigation and the assumption of a three phase WDS filling process, using the theory from the literature review. From the parameters investigation, ten parameters were found to be the most influential parameters to water demand and WDS performance during IWS. These ten parameters are supply duration, network hydraulic capacity, the available pressure head at the outlet, pressure within the water distribution network (WDN), topography, flow rate within the WDN, climate, supply area population, network filling time, and supply area size. Out of these parameters, supply duration was found to be the most influential parameter. The most influential parameters for IWS systems were found to be different from those associated with CWS

systems. IWS systems are driven by the operational and structural parameters, compared to the CWS systems which are mainly driven by the socio-economic parameters.

The outcomes of the parameters investigation were combined with a three phase calculation process, where each phase represented a section of the pattern shape. Phase 1 represents the initial flow spike related to the filling of the bulk supply pipe from the reservoir to the reticulation network. Phase 2 is related to the reticulation network filling, and phase 3 is related to the water demand characteristics after the filling process is complete. As a result of the research, a hypothetical 15 minute peak factors diurnal water demand pattern model was derived. This pattern yielded a high similarity to the shape of the water demand patterns derived from the actual data collected, based on the Kolmogorov-Smirnov goodness of fit test. Furthermore, the peak factors determined from the model, were also similar to the peak factors derived from the actual logged data.

The results of the study show that in order to estimate water demand and derive water demand patterns related to IWS, the parameters that impact water demand and WDS performance have to be identified, and the nature of the impact of these parameters has to be understood. By combining the crucial parameters with the governing hydraulic principles related to the WDS filling process, an estimate of the diurnal water demand pattern for residential areas subjected to IWS can be derived. This diurnal demand pattern model can be used as input for the reliability analysis of existing WDSs in a water scarce area. It can assist design engineers in identifying and planning the most effective ways of implementing IWS, in the process improving the resilience of the water supply during periods of water scarcity.

Acknowledgements

I wish to express my appreciation to the following persons and organisations who made this thesis possible:

a) Thanksgiving to God for the blessing of having this opportunity, without Him I can do nothing.

b) To my family, for the support and encouragement, especially to my mother, Nontyatyambo Msutu, no words can describe how thankful I am.

c) To my friends, for the constant encouragement and support.

d) To my supervisor, Mr. Carlo Loubser for his invaluable guidance and constant words of encouragement throughout the study.

e) To the Stellenbosch University Department of Civil Engineering, and Aurecon for the financial support granted to undertake my postgraduate studies.

f) To GLS Consulting for the assistance and use of Wadiso® during the course of the study.

g) To Silulumanzi (Pty) Ltd, Tabateq (Pty) Ltd, and Metasphere UK for the provision of data.

Table of contents

Plagiarism declaration ... **i**

Summary ... **ii**

Acknowledgements ... **iv**

Table of contents ... **v**

List of figures ... **viii**

List of tables ... **x**

List of abbreviations and acronyms ... **xii**

List of nomenclature ... **xiv**

1 Introduction ... **1**

- 1.1 Background ... 1
- 1.2 Study purpose ... 3
- 1.3 Study methodology ... 3
- 1.4 Possible contributions of the study ... 5
  - 1.4.1 Theoretical contribution ... 5
  - 1.4.2 Practical contribution ... 5
- 1.5 Scope of the study ... 5
- 1.6 Organisation of the study ... 7

2 Literature review ... **8**

- 2.1 Introduction ... 8
- 2.2 An overview of IWS ... 8
  - 2.2.1 Definition of IWS ... 8
  - 2.2.2 Prevalence of IWS ... 10
  - 2.2.3 The causes of IWS ... 12
  - 2.2.4 The impacts of IWS ... 14
- 2.3 Fundamental water demand concepts ... 17
  - 2.3.1 Defining water demand ... 17

2.3.2 Water demand patterns 18
2.3.3 Peak factors 24
2.4 Water demand estimation and influence parameters 27
2.4.1 Estimating water demand 27
2.4.2 Past water demand estimation studies 31
2.4.3 Parameters affecting water demand 36
2.5 Water distribution systems during intermittent supply 39
2.5.1 Domestic water consumption during IWS 39
2.5.2 Network filling process during IWS 40
2.5.3 Influence parameters during IWS 45
2.5.4 Modelling water distribution systems 47
3 Methodology 51
3.1 Introduction 51
3.2 Overall research methodology 51
4 Model development 54
4.1 Introduction 54
4.2 Model development concept 54
4.3 Parameters investigation 55
4.4 Model structure 57
4.4.1 Excel input requirements 57
4.4.2 CWS water demand scenario 59
4.4.3 IWS water demand scenario 63
4.4.4 Phase 1 - Bulk supply pipe filling 65
4.4.5 Phase 2 - Network filling 67
4.4.6 Phase 3 - Network full 70
5 Model validation 73
5.1 Introduction 73
5.2 Data collection 73
5.3 Data sorting and filtering 74
5.4 Data analysis 75
5.5 Model comparison and calibration 76
5.6 Limitations and challenges 79

6 Results and discussion of results 80

6.1 Parameters investigation 80

6.1.1 Parameter interrelationships and effects 80

6.1.2 Identifying the crucial parameter 85

6.2 Model development 88

6.2.1 IWS diurnal water demand pattern model 88

6.2.2 KaNyamazane diurnal water demand patterns 102

6.2.3 Model validation and calibration 105

6.2.4 Discussion of the model results 111

7 Conclusion and Recommendations 114

7.1 Conclusion 114

7.2 Recommendations 116

8 References 117

Appendix A Model development 124

Appendix B Results 133

Appendix C Study site flow readings 135

List of figures

Figure 2-1 Instantaneous water demand pattern for large consumer groups (Trifunovic, 2006). 19

Figure 2-2 Instantaneous diurnal water demand pattern averaged by the peak hour factors (Trifunovic, 2006). 20

Figure 2-3 Typical diurnal water demand patterns for residential areas (Mayer & DeOreo, 1999). 22

Figure 2-4 Diurnal water demand patterns for residential areas in South Africa (Pretorius, 2016; GLS Consulting). 23

Figure 2-5 Weekend and holiday diurnal demand pattern for the City of Tshwane LCH areas (Loubser et al., 2018). 24

Figure 2-6 Typical logged flows for a residential area subjected to IWS (McKenzie et al., 2014). 41

Figure 2-7 Water column velocity at the initial stage of the pipe filling process (Liou & Hunt, 1996). 43

Figure 2-8 Comparison of the experimental flow rate and the simulated flow rate during pipe filling (Hou et al., 2014). 43

Figure 2-9 Comparison of the measured and simulated pressure head at the nodes during network filling (De Marchis et al., 2011). 45

Figure 3-1 Research design flowchart 53

Figure 4-1 Model development concept. 55

Figure 4-2 Model calculation procedure. 58

Figure 4-3 Assumed WDN configuration. 58

Figure 4-4 WDN model of the KaNyamazane Toad Street supply area. 60

Figure 4-5 Assumed CWS diurnal demand patterns derived by GLS Consulting and Loubser et al., (2018). 62

Figure 4-6 Pressure head-time relationship (De Marchis et al., 2011; Hou et al., 2014). 69

Figure 4-7 Concept of the assumed IWS demand pattern shape during phase 3. 71

Figure 5-1 Typical daily flow readings from Site 2 74

Figure 5-2 K-S statistic for two cumulative distribution functions (Rojas-Lima et al., 2019).. 78

Figure 6-1 Correlation between the ten most influential parameters to IWS systems. 86

Figure 6-2 Chosen initial CWS diurnal water demand pattern. 91

Figure 6-3 Determining the 15 minute peak flows for phase 2. 96

Figure 6-4 Phase 3 peak flows calculations and CWS pattern conversion for the first supply period... 99

Figure 6-5 Phase 3 peak flows calculations and CWS pattern conversion for the second supply period. 100

Figure 6-6 Predicted IWS demand pattern for one supply period per day for 27/7/2019.... 101

Figure 6-7 Predicted IWS demand pattern for two supply periods per day for 27/7/2019. . 101

Figure 6-8 Predicted IWS demand pattern for three supply periods per day for 27/7/2019.102

Figure 6-9 Comparison of the 5 minute, 15 minute and 1 hour PF diurnal demand patterns derived from actual data. 103

Figure 6-10 First comparison of the IWS model prediction with the actual IWS demand pattern. 105

Figure 6-11 Comparison of the model prediction with the actual demand pattern after the first calibration step.. 107

Figure 6-12 The result of the second calibration step of the model prediction. 108

Figure 6-13 Model prediction after the third calibration step. 109

Figure 6-14 The final result of the model calibration. 110

Figure 6-15 Comparison of the CDFs for the model and the actual demand patterns........ 111

List of tables

Table 2-1 Number of countries and utilities reported to experience IWS (Kumpel & Nelson, 2016). 11

Table 2-2 Distribution of IWS systems in selected regions (Klingel, 2012). 11

Table 2-3 Recommended peak hour, day and week factors derived by GLS Consulting (Department of Human Settlements, 2019). 27

Table 2-4 Typical breakdown of internal residential water use (Department of Human Settlements, 2019). 29

Table 2-5 Typical outdoor water use as percentage of AADD (Department of Human Settlements, 2019). 30

Table 2-6 ILI benchmark values (Department of Human Settlements, 2019). 30

Table 2-7 Parameters influencing domestic water demand (Botha & Jacobs, 2017). 37

Table 2-8 Parameters influencing domestic water demand and WDS performance during IWS. 46

Table 2-9 Pressure dependent flow conditions during intermittent supply (Mohapatra et al., 2014). 50

Table 4-1 Residential area size based on AADD (adapted from Vorster et al., 1995) 60

Table 4-2 Generic peak hour factors as derived by GLS Consulting and Loubser et al., (2018). 61

Table 4-3 Compiled demand multiplier table 65

Table 4-4 Pressure dependent flow conditions during intermittent supply. 68

Table 5-1 Water meter sizes and site locations. 73

Table 5-2 Critical K-S values, $d\alpha$ for a sample size larger than 35 (adapted from Rojas-Lima et al., 2019) 78

Table 6-1 Parameter correlations table. 81

Table 6-2 Summary of the parameters applicability to the CWS and IWS systems. 84

Table 6-3 Parameter correlations analysis. 85

Table 6-4 Comparison of water distribution systems. 87

Table 6-5 KaNyamazane WDN and supply area characteristics. 88

Table 6-6 Characteristics of the chosen CWS diurnal water demand pattern. 89

Table 6-7 Peak hour factors and peak water demand calculated for the chosen CWS demand pattern. 90

Table 6-8 Step 2 IWS scenario for one supply period per day. 92

Table 6-9 Step 3 IWS scenario for two supply periods per day. 92

Table 6-10 Step 4 IWS scenario for three supply periods per day. 92

Table 6-11 Determining the bulk supply pipe filling flow rate. 93

Table 6-12 Summary of the peak factors calculated for each supply period scenario. 94

Table 6-13 Determining the available flow rate for network filling and the filling time. 95

Table 6-14 Phase 3 calculations to determine the peak hour flows. 97

Table 6-15 Check of the phase 3 peak hour flow values. 98

Table 6-16 Calculated 15 minute peak factors from actual data for 27/7/2019. 104

Table 6-17 Calculated 15 minute PFs for the predicted demand patterns for each comparison and calibration step. 106

List of abbreviations and acronyms

AAD	Average Annual Demand
AADD	Average Annual Daily Demand
ADD	Average Daily Demand
AZP	Average Zone Pressure
BCI	Business/Commercial/Industrial
CARL	Current Annual Real Losses
CBD	Central Business District
CDF	Cumulative Distribution Function
CSIR	Council for Scientific and Industrial Research
CWS	Continuous Water Supply
DMA	District Metered Area
EDF	Empirical Distribution Function
EPS	Extended Period Simulation
IBNET	International Benchmarking Network
ILI	Infrastructure Leakage Index
IWDS	Intermittent Water Distribution System
IWS	Intermittent Water Supply
K-S	Kolmogorov-Smirnov
LCH	Low Cost Housing
LIH	Low Income Housing
LOS	Level of Service
LRG	Large Single User
MAE	Mean Annual Evaporation
MAP	Mean Annual Precipitation
MDD	Maximum Day Demand

MMD	Maximum Month Demand
NIA	No Information Available
PDA	Pressure Dependent Analysis
PDF	Peak Day Factor
PDLS	People Driven Levels of Service
PF	Peak Factor
PHD	Peak Hourly Demand
PHF	Peak Hour Factor
PWD	Peak Weekly Demand
PWF	Peak Week Factor
RES	Residential
SDDRF	Supply Duration Demand Reduction Factor
TAADD	Total Average Annual Daily Demand
UARL	Unavoidable Annual Real Losses
UNESCO	United Nations Educational, Scientific and Cultural Organization
UNICEF	United Nations Children's Fund
WDN	Water Distribution Network
WDS	Water Distribution System
WHO	World Health Organization

List of nomenclature

Q	Flow rate
Q_t	Peak flow rate averaged over time interval
Q_i	Demand at a node
$Q_{max,IWS}$	Maximum demand for IWS
Q_{max}	Maximum demand
Q_{avg}	Average demand
m^3	Cubic metres
s	Seconds
ha	Hectares
L	Litres
kL	Kilolitres
d	Day
km	Kilometres
m	Metres
L_m	Length of the water mains
N_c	Number of service connections
L_p	Total length of underground pipe between street edge to the user meter
V	Velocity of the water
L	Length of the water column in a pipeline
L_{min}	Length of the water column in a pipeline at time zero
θ	Angle of the pipe from the horizontal
H_R	Available head at the reservoir
K	Entrance and valve losses coefficient

g	Gravitational acceleration
D	Pipe diameter
t	Time
f	Darcy-Weisbach friction factor
L_o	Local flow acceleration of the flow approaching the pipe inlet from the reservoir
c	Celerity of the pressure waves
H	Water pressure head
J	Head loss per unit length of pipe according to the Darcy-Weisbach equation
H_i	Pressure head at a node
H_{max}	Maximum pressure head at a node
H_{min}	Minimum pressure head at a node
Δt	Time interval
n	Total number of subintervals
K	Subinterval number
a	Lower interval limit
b	Upper interval limit
$f(x_K)$	Graph curve function
A	Pipe cross sectional area
m^2	Square metres
P	Pressure
ρ	Density
z	Elevation above datum
kg	Kilograms
N	Newton
$F_X(x)$	Theoretical cumulative distribution function

$S_N(x)$	Sample cumulative distribution function
D_{KS}	K-S statistic value
α	Level of significance
$c(\alpha)$	Inverse of the Kolmogorov distribution at significance level α
d_α	K-S critical value
h	Hour

1 Introduction

1.1 Background

The amount of water resources available to meet the demands of the world population are becoming depleted and the rate of depletion might be accelerated due to an increase in the growth rate of the world population (Vairavamoorthy et al., 2001). The United Nations world water development report estimates that between 2011 and 2050 the world population will increase by 33%, increasing the pressure on the available water resources. Furthermore, the majority of this population growth will occur in areas of severe water stress, such as Northern and Southern Africa, and South and Central Asia (UNESCO, 2016). For these regions water conservation through proper planning and management becomes crucial for the sustainable use of the available water resources.

The growth rate of the world population is however not the only main factor that contributes to the water shortage problem; there are a number of other factors that contribute as well. These factors can be broadly categorised into natural factors and human activities. The natural factors include factors such as climatic drought conditions, while the human activities include the mismanagement of the water distribution networks (WDN), leading to an increase in leakages, and the inefficient use of the water resources. The duration, severity and negative impact of water shortage on the people and on the economy of a country depends on the characteristics of the climatic drought conditions and the ability of the human systems to adapt to the new environmental conditions (De Marchis et al., 2011).

The water distribution systems (WDS) are one of the main human systems that have to be adapted to the new conditions. This is accomplished through the design and modification of the water systems and by implementing methods of water demand control. Intermittent water supply (IWS) is one of the most common methods of controlling water demand under water shortage conditions, and it is widely adopted in both developing and developed countries (Vairavamoorthy et al., 2001). IWS is typically used as a short term reactive measure to the water shortage problem, and also as one of the methods of water demand control that requires the least amount of financial effort to implement (De Marchis et al., 2011). This especially applies to countries that have no choice but to implement intermittent supply on WDSs that were originally designed for continuous supply.

South Africa has in recent years been awakened to the possibility of water shortages mainly due to physical water scarcity. Winter (2018) mentioned that South Africa is ranked as being the 30th driest country in the world, based on the low average annual rainfall of 500 mm compared to the world average of 860 mm. Furthermore, the recurrence of drought events and the poor regulation and management of the water usage, constantly threatens the country's water security. As a result, some of the municipalities in the smaller towns are resorting to IWS in an attempt to reduce water losses and the water consumption (Mckenzie et al., 2014).

The implementation of IWS has many documented negative impacts and is fittingly not recommended for planned implementation (Mckenzie et al., 2014). However, in many water scarce countries, IWS is still being implemented out of necessity rather than choice (Totsuka & Trifunovic, 2004). It is the responsibility of the local water services authorities and water services providers to ensure efficient WDNs and adequate levels of service (LOS) to the consumers during conditions of both continuous water supply (CWS) and during conditions of limited water supply. This is in compliance with the South African Water Services Act (No. 108 of 1997) which indicates that everyone has a right to have access to basic water supply. Every water services institution must take the necessary measures to ensure these rights are adhered to, and in cases when the water supply has to be limited, it is indicated that the limitation of the water services must be "fair and equitable".

An important step in the planning, design and analysis of WDSs to ensure adequate system performance and LOS, is determining the peak flows or peak water demand of the supply area (CSIR, 2003; Van Zyl et al., 2008). The peak flows determined represent the maximum demand conditions that the WDS has to satisfy. Therefore, the sizing and capacity of the WDS is dependent on the determination of the peak flows of the supply area (Scheepers, 2012). These peak flows, along with the demand patterns which describe the variation of the water demand with time, are used to validate the WDS capacity. This is achieved via the modelling and analysing of the WDSs using the available WDN modelling software packages such as EPANET and WADISO, among others.

In South Africa, the peak water demand and the water demand patterns are typically derived using the logged water consumption data that has been collected from the municipal or private databases that store the measured readings from installed water meters. This logged water consumption data is collected from the relevant databases, analysed and used to derive the peak factors and the diurnal water demand patterns. However, Van Zyl et al (2008) mentioned that the water consumption data as measured by the municipalities is not always readily available. In such instances, guidelines such as The Neighbourhood Planning and Design Guide (also referred to as the new Red book) are generally used, where the peak flows are obtained by multiplying the estimated average annual daily demand (AADD) by the instantaneous, hourly, daily or seasonal peak factors recommended. According to Scheepers (2012) this is the most prevalent method of estimating the peak flows. However, the new Red book method is applicable to CWS systems only, no provision has been made for the estimation of peak flows during the IWS scenario.

1.2 Study purpose

There has been extensive research conducted on CWS systems both in South Africa and internationally. This includes research related to water demand estimation, water demand patterns, and the parameters that influence domestic water consumption. However, there has been little similar research conducted for WDSs subjected to IWS, and for the modelling of IWS diurnal water demand patterns.

Due to the general lack of IWS research, when IWS is implemented as a method of water demand control, it is typically implemented and operated with little technical criteria. In most cases the implementation of IWS mainly depends upon the experience of the technical personnel, and a process of trial and error (Ilaya-Ayza et al., 2016). This often leads to inadequate supply pressures in the system, supply inequality and numerous other problems as discussed in Section 2.2.4. Thus, there is some benefit in investigating the mechanisms that drive IWS systems and also the effects of IWS on the domestic water consumption pattern. This is required to better understand the possibility of planning and managing the WDSs more effectively during periods of water scarcity.

Therefore, the purpose of the current study was to develop a new theoretical model to predict the diurnal water demand pattern for residential areas subjected to formal IWS, by taking into account the WDS filling process and the crucial parameters that impact the WDS during intermittent supply. This study serves as an initial exploratory attempt to define the typical form and characteristics of diurnal water demand patterns related to IWS, and to estimate these demand patterns for cases when actual water consumption data related to IWS is not available.

1.3 Study methodology

The study followed a deductive research approach to achieve the study purpose, which is a process of moving from the application of theory to the validation of the theory application via the use of data. To develop the model, the theory, results and concepts from various past research sources were applied. Following the development, the theoretical model was validated by comparing the model to diurnal water demand patterns derived from actual data.

The research design consists of two parts, which are the non-empirical study and the empirical study. The non-empirical study consists mainly of the literature review and the model development. The empirical study consists of the collection, processing and analysis of actual water consumption data, which is used for the model validation and calibration steps of this part of the research design.

The following objectives were formulated to guide the study according to the research design:

1. Conduct a literature review which forms the theory base upon which the assumptions for the development of the model are made, focussing on the following topics:

    a. Understanding the definition and characteristics of IWS systems in order to investigate how IWS differs from CWS.

    b. Understanding the fundamental concepts related to water demand and reviewing the methods of estimating water demand and deriving the diurnal water demand patterns.

    c. Investigating the parameters that affect domestic water demand and WDS performance.

    d. Investigating the effects of IWS on domestic water consumption and WDS characteristics during IWS conditions.

2. Conduct an investigation to identify the parameters that influence intermittent water distribution systems (IWDS), and to establish the correlation of these parameters with the WDS performance and the water demand. This would assist in determining what the most crucial parameters are for systems operated under IWS conditions. The crucial parameters identified become the “building blocks” used to develop the model.

3. By using the identified crucial parameters and correlations, along with the established theory from the literature review, develop the IWS diurnal water demand pattern estimation model by:

    a. Constructing the model in a Microsoft Excel spreadsheet using the first principles obtained from the literature review, and the outcomes from the parameters investigation as constituting components of the model.

b. Validating and calibrating the model using the collected, processed and analysed water consumption data from a residential area subjected to formal IWS.

1.4 Possible contributions of the study

1.4.1 Theoretical contribution

The study provides new insights into the typical form of a diurnal water demand pattern associated with IWS in residential areas. The aim is to provide an initial attempt to define and understand what components constitute the IWS diurnal water demand pattern, and to determine the typical shape of the pattern. Factors considered include the filling spike, network filling process, and the consumer demand. Another potential study outcome includes determining what crucial parameters affect the IWS diurnal water demand pattern, and how these parameters affect the pattern and the WDS performance during the filling process and also after the network is full.

1.4.2 Practical contribution

The study aims to present a software based model that can be used to predict the domestic IWS diurnal water demand pattern with the estimated peak factors and peak flows generated during the initial network filling process and after the network filling process. This makes it possible to analyse the current or designed WDN's modelled response to such peak flows using the extended time period simulation (EPS) in modelling software such as EPANET and WADISO.

Therefore, a reliability based analysis of the WDN can be conducted to determine how reliable the WDN would be under water scarce or IWS conditions, and also to be able to investigate whether every user in the network would receive water supply during IWS. This can be achieved by determining which areas of the network have inadequate pressures and flows caused by the high IWS peak flows. Such an analysis tool can assist design engineers during the planning phase to identify the most effective ways of distributing water during periods of water scarcity.

1.5 Scope of the study

The study was predominantly limited to a desk top investigation. This was mainly due to a lack of published research related to IWS diurnal water demand patterns, and also due to a lack of readily available water consumption data as measured by the municipalities for areas subjected to IWS. A novel method of investigation was developed to achieve the purpose of the study as described in Section 1.3 and in more detail in Chapter 3 of this document.

The following scope and limitations apply to the study:

- The study is limited to a first principles investigation of the WDS filling process, the crucial influence parameters and the diurnal water demand patterns. Aristotle defines a first principle as “the first basis from which a thing is known” (Irwin, 2002). Therefore the study is limited to the assumptions and concepts obtained from the reviewed literature sources.

- For the development of the model, the study is limited to residential land use developments (specifically low cost housing areas) with per stand water connections and water-borne sanitation. The industrial and commercial land use types are not considered.

- The South African conditions of IWS for residential areas is considered, where IWS is implemented as a method of water demand control on an existing CWS designed system. The majority of WDNs in South Africa are designed for CWS, and IWS is typically implemented on these kinds of WDNs during periods of water scarcity. Therefore, the term ‘formal IWS’ in the study refers to planned IWS implementation for the purposes of water demand control and water resource management during a water scarcity scenario.

- Only the case of municipal water supply to the households is considered, supply from alternative water sources such as groundwater and rainwater is not considered. Furthermore, both indoor and outdoor household water use is considered in the study of the domestic water demand. The water demand for fire flow events is not considered in the model development.

- The developed model would have a certain level of uncertainty and variation, it is not meant to be an exact prediction of the IWS diurnal water demand pattern. The model rather serves to present an estimated range of the magnitude of the initial filling spike, the water consumed during the supply period, along with the simplified form of the diurnal water demand pattern. Van Zyl et al. (2007) noted that the process of measuring water consumption has a variable nature, hence the level of uncertainty anticipated from the model predictions.

- The study does not aim to promote IWS as a method of water demand control. According to Mckenzie et al. (2014), IWS should be the last resort to be considered in cases of water shortage.

1.6 Organisation of the study

Chapter 1 - This chapter is an introduction to the study, presenting the background to the current study, the study purpose, the overall study methodology, which in turn includes the study objectives, the possible contributions of the study and the study scope.

Chapter 2 – This chapter contains a literature review of previous research conducted on the definition, prevalence and characteristics of IWS systems, the fundamental concepts of water demand patterns and water demand estimation, the parameters that affect water demand and WDS performance, and the modelling of WDS for the IWS scenario.

Chapter 3 – This chapter presents the overall research methodology of the study which consists of a discussion of the research approach chosen, the nature of the study and the overall research design structure.

Chapter 4 – This chapter presents the detailed methodology of the development of the theoretical model. It consists of two main sections, (1) the influence parameters investigation to determine the crucial parameters for the model development. And, (2) the theoretical model construction for the prediction of the IWS diurnal water demand patterns.

Chapter 5 – This chapter describes the steps followed for the validation and calibration of the developed model. It includes a description of the study site, water consumption data collection, data processing and data analysis. A description of the calibration of the model is also included. Finally, the limitations and challenges encountered during the model validation process are discussed in this chapter.

Chapter 6 – This chapter presents the study findings and a discussion of the study findings.

Chapter 7 – This chapter concludes the study and provides recommendations for future research.

Appendices – Appendix A contains figures and tables depicting the calculation worksheets for the model development. Appendix B contains figures and tables depicting the calculation worksheets for the results and the discussion of results chapter. Appendix C contains the daily flow readings for each of the study sites over the entire record period.

2 Literature review

2.1 Introduction

This chapter is the first step of the research objectives in order to establish the context and relevance of the current study and to provide an overview of the following identified key topics and concepts:

- The overall definition, prevalence and characteristics of IWS systems;
- The fundamental water demand concepts and the methods currently used to estimate water demand;
- The parameters that affect water demand and WDS performance;
- Water consumption and WDS characteristics during IWS conditions;
- The modelling of WDSs for the IWS scenario.

The findings from the research of the aforementioned topics provided a theory base for the investigation of the most critical influence parameters and also for the assumptions used in the development of the IWS diurnal water demand prediction model

2.2 An overview of IWS

2.2.1 Definition of IWS

The term “intermittent water supply” can be broadly defined as water supply that is unavailable to the users for certain periods of time, usually periods that are less than 24 hours per day (Galaitsi et al., 2016). It is one of the most common methods used to control water demand during conditions of water scarcity. It works on the basis of interrupting the water supply to an area for certain periods of time during the day, in order to control and reduce water consumption and minimize water losses (Vairavamoorthy et al., 2008; Mckenzie et al., 2014). The implementation of this water demand control method is often used as an emergency measure; as a last resort when the water supply resources are close to empty and there is no other option but to begin rationing the available water resources. However, according to Mckenzie et al. (2014), there are also cases when intermittent water supply is used as a quick and simple solution for reducing water losses in the water supply systems even when there is no water shortage. Intermittent supply can also be related to the temporary water stoppages due to pipe network maintenance, however, this definition of intermittency is not considered in the current study.

Traditionally, water supply systems are designed and optimized to provide sufficient volumes of water to meet the system demand at adequate pressures and at the least cost. However, during periods of water scarcity, the goal of the water supply system is to distribute the available water resources as fairly, equally and predictable as possible to every user in the network within the supply period (Totsuka & Trifunovic, 2004). For a system designed for continuous water supply, this requirement or goal is automatically met, but when the same system is operated intermittently this goal cannot be achieved due to water availability, system pressures and supply duration constraints. As a result Vairavamoorthy et al. (2008) noted that during water scarcity conditions, the focus should shift from supply based management to demand management. Demand management is the implementation of strategies or techniques designed to influence the water demand and usage in order to improve the efficient use of the limited water resources. This is often achieved at below satisfactory service levels. Intermittent water supply is one of the techniques used in demand management programmes, together with leak detection and repair strategies, water metering, changes in water pricing, installation of water saving devices, wastewater recycling, and public awareness and educational campaigns (Vairavamoorthy et al., 2008).

Galaitsi et al. (2016) further expanded on the definition of intermittent systems by proposing three different types or levels of intermittency: predictable intermittency, irregular intermittency and unreliable intermittency. Predictable intermittency refers to supply that is predictable and scheduled with constant pressures during delivery and sufficient water storage and water volumes. Irregular intermittency is supply that arrives at unknown time periods (usually short time periods of no more than a few days) when users know the quantity of water to expect during the short time period of supply, but they cannot predict when the supply will arrive. Unreliable intermittency refers to intermittent supply that has an unknown and inconsistent schedule of delivery, often with long periods of no supply and inconsistent water pressures in the system. During this level of intermittency, the users experience insufficient water quantities and must adapt their behavior to cope with the shortage in supply. Furthermore, intermittent supply can also be categorized as regular IWS through daily scheduled supply, seasonal IWS due to the variation in the water demand caused by the change in the seasons and occasional IWS when there are issues with the water source (e.g. low water resource levels, and poor water quality) (Charalambous & Liemberger, 2016).

2.2.2 Prevalence of IWS

In 2007 Vairavamoorthy et al. (2007) reported that there were at least 30 countries worldwide considered water stressed, and 20 of these countries were water scarce. By 2020 the number of water scarce countries is predicted to approach 35 worldwide. It was mentioned that the majority of these water stressed countries were developing countries, which are experiencing rapid population growth and economic development, leading to the rapid urbanization of these regions. This rapid urbanization, along with the changes in the climate, have caused added strain to the global freshwater resources contributing to the physical and to the economic water scarcity in those regions (Kumpel & Nelson, 2016). The United Nations world water development report estimates that between 2011 and 2050 the world population will increase by 33%, increasing from 7 billion to 9.3 billion. The population of people living in urban areas will almost double, from 3.6 billion to 6.3 billion, in this period of time, increasing strain on the water resources available. Furthermore, the majority of the additional 2.3 billion people are expected to live in areas of severe water stress, such as Northern and Southern Africa and South and Central Asia (Unesco, 2016). This implies that the problem of water shortage will have the greatest impact on the urban areas of developing countries, and the prevalence of intermittent water supply may increase in these regions.

Accurately determining the global prevalence of intermittent water supply is a challenge, due to the lack of available data on these types of systems. Kumpel and Nelson (2016) noted that the available data on IWS systems is scattered, and some of the data for certain regions is missing. However, using the data collected by the International Benchmarking Network (IBNET), it was estimated that not less than 309 million people experience intermittent water supply in the regions analysed. Refer to Table 2-1 for a summary of the regions reported to experience IWS.

Table 2-1 Number of countries and utilities reported to experience IWS (Kumpel & Nelson, 2016).

Region	Countries with IWS (IWS/Total)	Utilities (IWS/Total)	Population with IWS (millions)	Supply Duration (mean hours [range])
East Asia and Pacific	9/32	54/479	15	16.7 [1-23]
Europe and Central Asia	17/41	162/960	25.4	13 [0.2-23.7]
Latin America, North America, Caribbean	8/21	79/1403	28.4	16 [2-24]
Middle East and Northern Africa	1/2	12/13	4.6	3 [3-3]
South Asia	5/6	104/107	116.6	7.2 [0.3-23]
Sub-Saharan Africa	19/40	249/314	118.8	12.8 [1-23.5]
Total	59/142	660/3276	308.9	12.5 [0.2-24]

Klingel (2012) similarly estimated that approximately one third of the water supply systems in Africa are operated intermittently, and over half of the supply systems in Asia are IWS, and approximately two thirds of the systems in Latin America do not operate continuously. Furthermore, it was also reported that over 90% of the water supply systems in Southeast Asia and in India are operated intermittently. Table 2-2 summarizes the distribution of IWS in various regions of the world.

Table 2-2 Distribution of IWS systems in selected regions (Klingel, 2012).

Region	Intermittent water distribution	Reference
Africa	Approximately 30%	WHO & UNICEF (2000)
Asia	Approximately 50%	WHO & UNICEF (2000)
Latin America	Approximately 60%	Lee &Schwab (2005)
Southeast Asia	Approximately 90%	Vairavamoorthy et al. (2001)
India	Approximately 100%	Vairavamoorthy et al. (2001)

In a study by Galaitsi et al. (2016), a global literature search on IWS water case studies was conducted and from the results of the search it was observed that the majority of the literature found was located in Africa, Southeast Asia, and South Europe including the Middle East. The volume and concentration of intermittent water supply case studies in a certain region, implies that intermittent water supply is a topic of concern or importance in those areas. From the different sources reporting on the global prevalence of IWS, it can be observed that the regions where IWS is most prevalent are Africa, Latin America, the Middle East and Asia (mainly Southeast Asia).

2.2.3 The causes of IWS

According to Klingel (2012), it is vitally important that solutions be developed for the IWS problems experienced by many developing countries. In order to achieve this, there is a need for a better and more precise understanding of the causes and the consequences related to IWS systems. IWS is the result of various factors such as deteriorating infrastructure due to poor maintenance, increased demand due to population growth and urbanisation, water scarcity due to drought and the growth of the demand beyond the network design limit due to poor forward planning. Totsuka and Trifunovic (2004) grouped the factors that cause intermittent supply into three main categories: management causes, economic causes and absolute/physical causes. Each category of causes results in a certain type of water scarcity such as scarcity due to poor management, economic scarcity and physical water scarcity.

Poor systems management

Apart from physical water shortages, the most common cause of IWS in developing countries is the poor planning and the mismanagement of the WDSs (Klingel, 2012; Totsuka & Trifunovic, 2004). In research conducted by Klingel (2012), it is noted that the WDSs in many developing countries are designed and operated without any overarching principle and basic plan. This leads to complex water supply systems without a properly defined arrangement of the main system components such as the feeders, pumps and the supply zones. It also leads to poorly planned system expansions and incorrectly dimensioned system components. Poorly planned supply systems are complex to operate and maintain. Furthermore, collection of data, which is crucial for the monitoring, operation and maintenance of the water supply systems, is complicated. The planning, analysis and operation of WDSs requires a detailed knowledge of the water supply network infrastructure and condition. This is possible only when there is proper data collection and data management. A lack in proper data management, therefore, can lead to poor performing supply systems and unsatisfactory levels of service to the consumers. Totsuka and Trifunovic (2004) mentioned that good water systems management and administration are critical to achieving efficient water supply system performance.

Economic scarcity

Economic water scarcity is experienced when the water provider or government does not have the financial resources to enlarge or expand the capacity of the existing water distribution infrastructure (Totsuka & Trifunovic, 2004). This type of scarcity is possibly exacerbated by poor planning and poor water demand forecasting, which are related to system management. An increase in the population of a certain area causes an increase in the overall water demand, leading to increased pressure on the available water resources. As a result, the consumers could experience IWS, due to the demand exceeding the existing network hydraulic capacity, and eventually exceeding the capacity of the water sources. A solution may be to expand the existing infrastructure, however, some developing countries might not have the financial resources to further develop their existing water distribution networks. As a result, economic water scarcity is experienced. Totsuka and Trifunovic (2004) further mentioned that an improvement in the management of the water supply systems, would aid in the improvement of the level of service. However, 24 hour supply would not be restored without the physical expansion of the water infrastructure. The inability of the water providers to further develop the existing supply networks, would also contribute to an increase in the water losses in the system. This is due to the deteriorating infrastructure and the damage caused by IWS on the pipes in the network, resulting in leakages, which could be a further cause for the implementation of IWS on the water supply network in the area (Klingel, 2012).

Restricted water availability (physical/absolute water scarcity)

Absolute or physical water scarcity, due to the inadequate volume of water resources available to meet the demand, is typically considered the main reason for IWS. This is considered as the most challenging cause of IWS to solve, because it is mainly a result of climatic or natural factors that are often beyond human control and influence (Klingel, 2012; Totsuka & Trifunovic, 2004). During periods of physical water scarcity, methods of water demand control are implemented to manage and conserve the available water resources. In addition, alternative water sources, which are not always readily available and accessible are explored. De Marchis et al. (2011) commented that the duration, severity and negative impact of physical water shortage on the people and on the economy of a country, depends on the characteristics of the climatic drought conditions, and the ability of the human systems to adapt to the new environmental conditions. In this case, both the water providers and the water consumers have an obligation to adapt to the new conditions (Totsuka & Trifunovic, 2004).

2.2.4 The impacts of IWS

Generally, IWS is not considered as a water supply option during the initial design of water supply systems. As a result, when IWS is implemented on such systems, the negative consequences caused by IWS overshadow any advantages that come with IWS. According to Klingel (2012), Totsuka and Trifunovic (2004), and Charalambous and Liemberger (2016), the main consequences related to IWS are supply inequality, poor water quality, increased consumer and water provider coping costs, increased water losses, damage to water meters, and negative hydraulic and physical impacts on the network.

Supply Inequality

One of the main consequences of IWS, is the inequitable supply of water to the consumers in the network. This occurs due to the pressure dependent flow conditions found during IWS. The volume of water supplied to the consumers during IWS, depends on the WDN hydraulic capacity and the available pressures in the system, rather than the demand of the users (Totsuka & Trifunovic, 2004). During the network filling process at the beginning of each supply period, the network will have higher peak flows than usual, resulting in low supply pressures in the pipe network and inadequate pressure heads at the outlets (Charalambous & Liemberger, 2016). As a result, the consumers who are located at the extremes of the network and those situated at higher elevations in the supply area, will have less time to draw water from the system and will experience lower levels of water quality and water quantity supplied to them, compared to the consumers closest to the supply reservoirs or water source (Klingel, 2012).

Water contamination/poor water quality

Since water does not continuously flow in the pipe system during periods of IWS, the probability of water contamination greatly increases and the health of the consumers is potentially jeopardised. Totsuka and Trifunovic (2004), and Charalambous and Liemberger (2016) commented that during periods of IWS, there is a high risk of water contamination through damaged pipes from polluted rainwater, groundwater and sewage during non-supply hours. The stagnant water in the damaged pipes when the supply is off, also creates an opportunity for pathogens to settle in the pipe system. This risk is increased in areas with high temperatures. Contamination can also be caused by the low or negative system pressures in the pipes, which form a vacuum that can draw contaminants into the pipe system when the water supply is flowing. According to Klingel (2012), water contamination during periods of intermittency, is an inevitable consequence of IWS.

Consumer and water provider coping costs

When IWS is implemented, both the water providers and the consumers have to adapt and find ways of coping with the new conditions. During IWS, the consumers often have to pay a high price for private household tanks and private pumps. These devices are required to ensure that they will be able to withdraw an adequate amount of water to meet their demand during the supply periods. The addition of these items creates even more expenses for the consumers, due to the energy and running costs of the pumps, and the assigning of household members who have to spend their time watching and filling the tanks during the supply periods. This, in turn, may affect the overall household income, due to the costs of treating the water for contamination, or the loss of income due to the illnesses related to water-borne diseases. Often, when the volume of water supplied is not enough, the consumers may also have to find alternative sources of supply such as bottled water, private wells, public taps and water tankers which also come at a cost (Klingel, 2012; Totsuka & Trifunovic, 2004). Charalambous and Liemberger (2016) found that the higher income consumers cope by spending money buying private tanks, pumps and water treatment facilities. The middle income consumers spend less on these facilities, and more on the time needed to collect water and the energy needed to run the pumps installed. The low income consumers cope by spending their productive time collecting water. The low income consumers cannot afford the tanks, pumps and treatment facilities, so they have to spend the most time fetching water at the public taps and at water selling points. They often have to travel long distances and spend time waiting in queues to get their water supply.

It is not only the consumers that experience increased costs, the water utilities also bear the extra costs related to IWS systems. During IWS, the water providers experience an increase in the labour costs due to the more frequent operation, maintenance and replacement of the valves in the system. In addition, system maintenance costs also increase, due to the increased rate of damage to the pipe network caused by the repetitive variation of pressures associated with IWS. Moreover, the costs to clean and chlorinate the pipes increase, due to a higher risk of water contamination during IWS periods. All these extra costs incurred by the water providers eventually result in an increase in cost of the water, which the consumers have to pay (Totsuka & Trifunovic, 2004).

Water wastage

Although IWS is typically implemented as a method of water conservation and saving, Klingel (2012), Totsuka and Trifunovic (2004), and Charalambous and Liemberger (2016) have observed that the extent of water wastage during periods of IWS is more than the periods of

24 hour supply. According to Klingel (2012), this a consequence of supplying large volumes of water within a short period of time. During IWS, the consumers keep their taps constantly open and also tend to remove the control valves installed, so that they can withdraw and store as much water as possible in the private household tanks during the supply period. As a result, the tanks tend to overflow during the supply hours causing water wastage. Furthermore, it is also observed that the consumers have shown a tendency of emptying their tanks before the next supply period to create capacity for the fresh water about to be received, which further contributes to the increased water wastage during these periods (Totsuka & Trifunovic, 2004). It can be reasonably assumed that the consumers closest to the water source, and the consumers who are financially advantaged, contribute the most to water wastage during IWS. The financially disadvantaged consumers are not able to buy and install large private tanks, and the consumers located further away from the water source receive smaller quantities of water supply.

Meters malfunctioning

The repeated filling and draining of the pipe network also affects the ability to collect accurate flow data from the WDNs. The vacuum conditions and the air in the pipes cause the water meters to malfunction, producing inaccurate readings. During the pipe filling process, air trapped in the pipes has to be discharged. During this process of water filling and air discharge, the meters operate at higher speeds, producing inaccurate readings and increasing the rate of deterioration of the meters (Totsuka & Trifunovic, 2004).

Hydraulic and infrastructure impact

When IWS is implemented on a WDS as a long term solution, the negative impacts of IWS on the water distribution infrastructure begin to become evident mainly due to the pressure surges and pressure variations caused by the repeated filling and draining of the network. The pressure surges accelerate the rate of deterioration of the infrastructure. Since most WDNs are designed to be operated in the continuous mode of supply, the high volumes of water supplied in a short period of time during IWS, produce load factors that are up to three times higher than what the WDS was designed to bear (Klingel, 2012). This, in turn, results in inadequate pressures in the system due to high head losses, pipe zones with stagnant water increasing the risk of water contamination and unequal supply over the supply area. This often leads to an increase in illegal connections, causing more stress on the existing water distribution infrastructure (Klingel, 2012). All these consequences can further be exacerbated by the lack in proper maintenance of the WDS.

The causes and consequences of IWS listed above, are the main identified factors observed by Klingel (2012), Totsuka and Trifunovic (2004), and Charalambous and Liemberger (2016). However, these are not the only causes and consequences related to IWS. Galaitsi et al. (2016) conducted a multidisciplinary and global investigation on the causes and the consequences of IWS by reviewing 129 IWS case studies from different regions worldwide. As a result of the study, 47 causes of IWS were identified and in total 106 cause to consequence links were made.

2.3 Fundamental water demand concepts

2.3.1 Defining water demand

In Section 2.2.1, it is indicated that the main objective of a WDN is to supply an adequate volume of water to the supply area, in order to meet the system water demand. The system water demand refers to the total volume of water required to meet all the water needs of the consumers in the supply area, within a certain supply duration (Billings & Jones, 2011). This is the definition of water demand considered in the current study. In broader terms, water demand includes not only the consumer demand in the supply area, but also the water required for firefighting, system flushing and for the proper operation of the water treatment facilities (Arunkumar & Nethaji Mariappan, 2011). Additionally, the volume of water leakage in the pipe network is also conventionally included in the total water demand (Arunkumar & Nethaji Mariappan, 2011). The leakage in the pipe network is also considered as a component of the water demand in the current study, however, the water demand for firefighting and for the operation of the water treatment facilities is not considered. According to Arunkumar and Nethaji Mariappan (2011) water demand can be further described in the following terms:

- The Average Annual Demand (AAD) – The total volume of water supplied to the WDS in a year, averaged out over several years, if there is an annual variation in the total amount supplied value, expressed in units of litres (L).

- The Average Daily Demand (ADD) – The total volume of water supplied to the WDS within the period of a year divided by the 365 days of a full year, expressed in units of litres per day (L/day).

- The Maximum Month Demand (MMD) – The average water usage per day for the month with the highest water demand, expressed in units of litres per day (L/day).

- The Peak Weekly Demand (PWD) – The highest average weekly demand that occurs within a year for a certain WDS, expressed in units of litres per day (L/day)

- The Maximum Day Demand (MDD) – The highest volume of water supplied to the WDS in one day, expressed in units of litres per day (L/day).
- The Peak Hourly Demand (PHD) – The highest volume of water supplied to the WDS in one hour, expressed in units of litres per day (L/day).

The water demand described above is mostly expressed in litres per day and is referred to as the average demand. When this demand is divided by the population of the supply area, it becomes the specific demand expressed in litres per capita per day (L/c/d) (Trifunovic, 2006).

2.3.2 Water demand patterns

Definition of water demand patterns

Water demand varies according to the type of land use of the supply area, the geographical location of the supply area and according to the time-related demand (or consumption) of the consumers in the supply area (Trifunovic, 2006). The consumer water demand variation caused by the differing types of land use, is typically categorised into residential (domestic) and non-residential (non-domestic) water demand. Non-residential water demand refers to the volume of water required or used by the commercial, industrial and agricultural sectors, which include hospitals, schools, business offices, parks, farms and sports facilities. Residential water demand, however, refers to the indoor and outdoor water end-use requirements for a household which includes activities such as showering, toilet flushing, dishwashing, laundry, cooking, gardening and car washing. The current study only considers the residential water demand.

Furthermore, Trifunovic (2006) also ascribed the variations in water demand to the differing spatial (location) and temporal (time-related) characteristics of the supply areas. The spatial variation of the water demand is mainly caused by the differing climatic factors of each supply region, and the temporal variation in the water demand is caused by the hourly, daily, weekly and seasonal consumer water use trends. It is noted by Trifunovic (2006) that human activities tend to have cyclic characteristics, and this observation is also applicable to water consumption. As a result, it is possible to predict the consumer water consumption behaviour and schedule in a supply region. Whenever the typical domestic water activities such as toilet flushing, showering, cooking, and laundry occur in a household, the sum of the measured flows of the different water activities occurring at the same time produces the instantaneous flows or demand of that particular house. By assessing the instantaneous demands over a period of time, a demand pattern for each house can be derived. Eventually, a demand pattern for an entire supply region can be derived, as more households are considered over an extended period of time (Scheepers, 2012). According to Trifunovic (2006), there are various types of water demand patterns that can be derived for supply areas, and the distinction between the patterns can be made based on the time period over which the flows are observed. The types of water demand patterns identified are, instantaneous, hourly, daily (diurnal), weekly and annual (seasonal).

Types of water demand patterns

The instantaneous water demand pattern is derived by monitoring the flows of a defined number of consumers over a short period of time of a few seconds or minutes. Determining the instantaneous demand pattern, is the initial step in the derivation of the water demand pattern of an entire supply region. This type of demand pattern may be used for the design of house installations during the WDN design of small residential areas. The instantaneous water demand patterns, are the least predictable patterns of consumer water demand, due to the small number of consumers monitored and the short time period of observation. The larger the number of consumers considered, the clearer and more predictable the water demand pattern for a supply region becomes. When the instantaneous water demand of large groups of consumers is considered, the observed water demand is uniformly distributed over a duration longer than a few seconds or minutes, as illustrated by Figure 2.1.

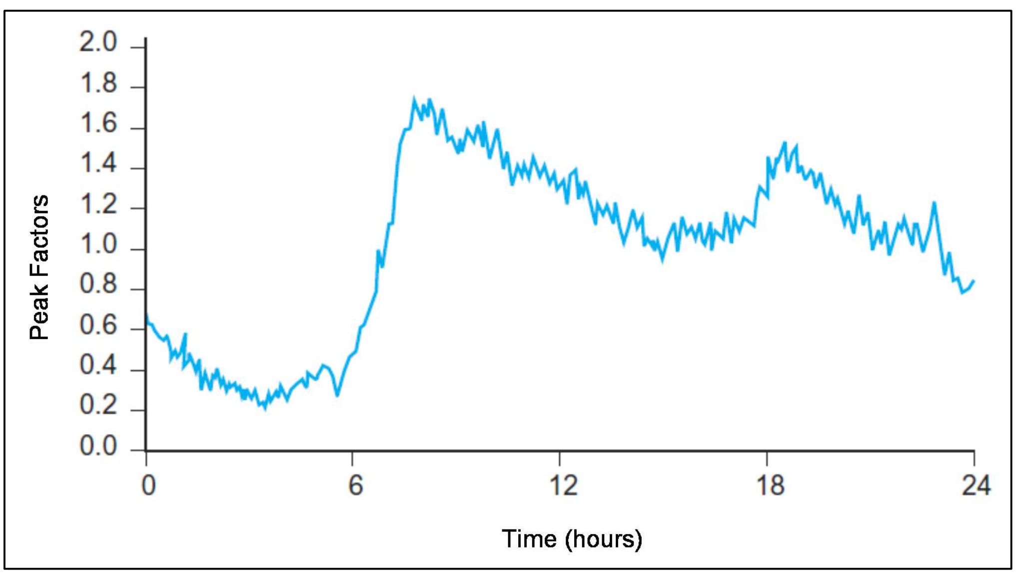


Figure 2.1 Instantaneous water demand pattern for large consumer groups (Trifunovic, 2006).

For most practical applications, such as the design and sizing of the primary and secondary WDNs, the water demand over an hour duration is generally used, expressed as the peak hour factor (PHF). The peak hour factor is a ratio between the maximum instantaneous demand measured within the hour duration and the average water demand over the entire pattern duration considered. When these peak hour factors are derived from the instantaneous demand of a large group of consumers for every hour over a 24 hour observation period, the daily (diurnal) water demand pattern can be derived. See Figure 2.2 for an example of a daily water demand pattern.

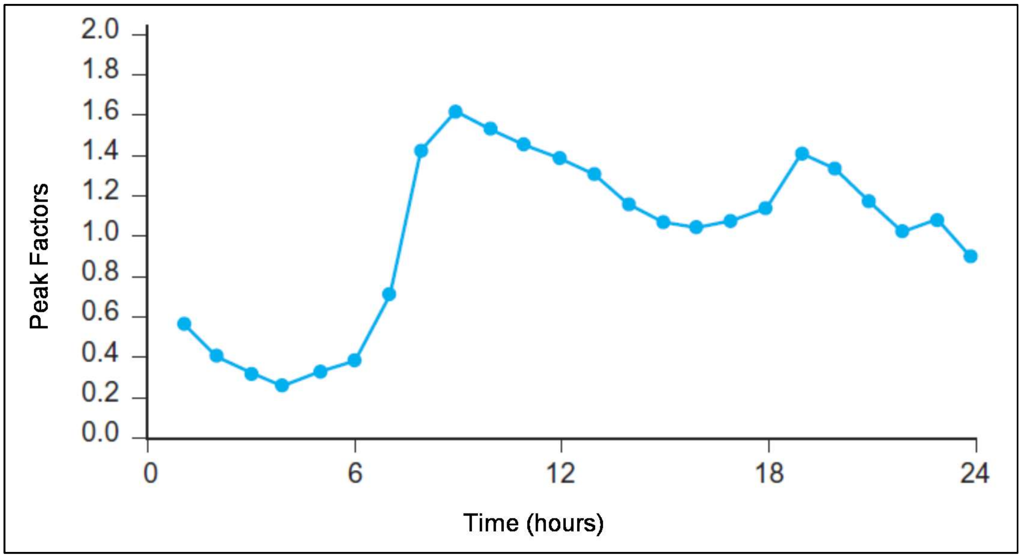


Figure 2.2 Instantaneous diurnal water demand pattern averaged by the peak hour factors (Trifunovic, 2006).

Additionally, the weekly demand patterns describe the demand variation in a supply region for working and non-working days in a week, which may also include occasional days such as public holidays. Scheepers (2012) defined the working days as Monday to Friday and the non-working days as Saturday and Sunday. It was observed that the water demand during the working days has a distinct pattern compared to the non-working days, and also that the peak demands are higher during the working days. The non-working days, however, have a more evenly distributed water demand throughout each day, due to the consumers being at home for longer periods of time during the day producing lower demand peaks, compared to the working days. Then finally, the annual water demand pattern describes the water demand variation over the period of a year. The annual water demand variation is mainly caused by the change in seasons during the year, and is also referred to as the seasonal water demand. Pretorius (2016) noted that the seasonal variation refers to only the summer and the winter seasons. In the summer season, the water demand is observed to increase mainly due to the increase of outdoor water use related to higher temperatures, and also due to an increase in the number of water consumers in a particular supply region such as holiday locations. The water demand in the winter season, however, is mentioned to have higher morning peaks than in the summer months and lower afternoon or evening peaks than the summer months.

Diurnal water demand patterns

The focus of the current study is on the daily (diurnal) water demand pattern. Mayer and DeOreo (1999) conducted a study of 1188 households across 12 locations in the U.S. and in Canada, investigating the residential water use of single family households across North America. It was observed that the overall or total hourly water use pattern monitored over a period of 24 hours for all the 12 study locations followed the same diurnal pattern, consisting of four noticeable periods:

- The first period is generally between 11 p.m. to 5 a.m. during the night time, when the lowest water consumption occurs;
- The second period is generally between 5 a.m. to 11 a.m. in the morning when the highest water consumption occurs;
- The third period is between 11 a.m. to 6 p.m. when there is moderate water consumption during the middle of the day;
- The fourth period is generally between 6 p.m. to 11 p.m. in the evening when there is high water consumption.

Mayer and DeOreo (1999) described the form and characteristics of the resulting total hourly residential demand pattern as “classic” and “typical” as illustrated in Figure 2.3. For residential areas, this is the standard or accepted form of the diurnal water demand pattern.

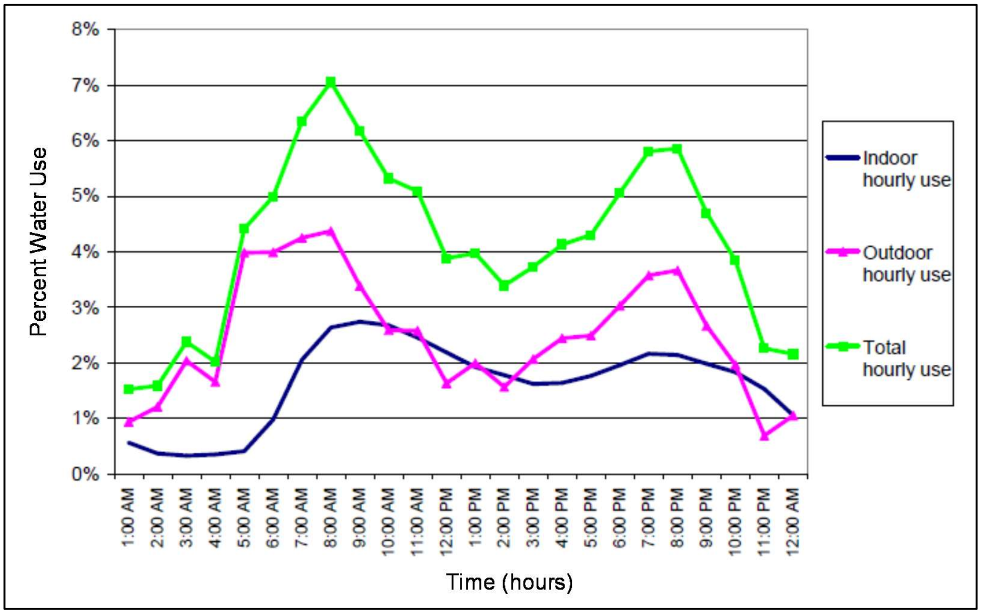


Figure 2.3 Typical diurnal water demand patterns for residential areas (Mayer & DeOreo, 1999).

The two highest peaks in the four identified periods of a typical residential diurnal water demand pattern, are related to the high water demand in the mornings when the consumers wake up and prepare for the day, and the high demand in the evenings when the consumers come back from work or school. Furthermore, the lowest water demand period refers to the period of time when the consumers are asleep in the evening (Scheepers, 2012). The type of lifestyle of the residential consumers in a supply region, can be identified by examining the diurnal demand pattern of that region. In South Africa, similar residential diurnal water demand patterns have been derived by GLS Consulting for small, medium, large and low cost housing (LCH) residential areas, as illustrated in Figure 2.4. These diurnal water demand patterns have also been included in the new Red book guidelines, “The Neighbourhood Planning and Design Guide.” It can be noted from Figure 2.4, that the demand pattern for the LCH development areas does not follow the typical diurnal demand pattern for residential areas; instead of two identifiable demand peaks in the morning and in the evening, there is one demand peak in the afternoon.

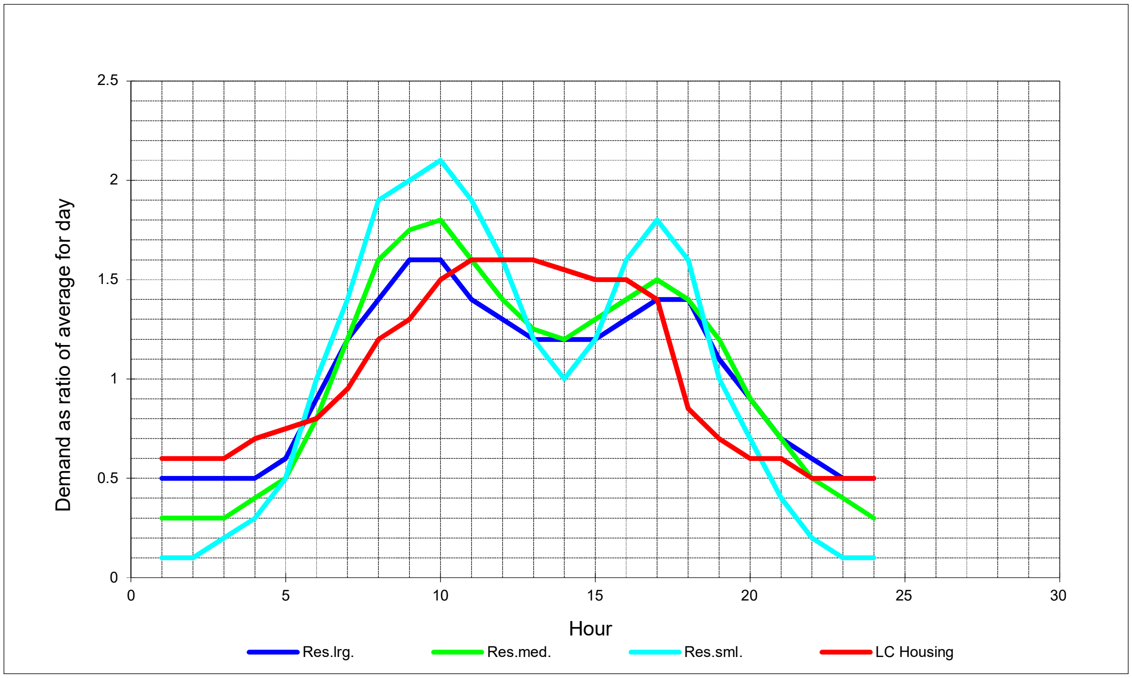


Figure 2.4 Diurnal water demand patterns for residential areas in South Africa (Pretorius, 2016; GLS Consulting).

Loubser et al. (2018) also conducted a study to derive the typical diurnal water demand patterns for LCH areas in South Africa. In the investigation Loubser et al. (2018) identified and analysed the logged flow data for three low cost housing (LCH) water distribution zones in the City of Tshwane. From the analysis of the data, it was observed that for the LCH areas the water demand over weekends and public holidays was significantly higher than for the normal work days. Three diurnal pattern types were derived for each of the LCH zones, in order to identify the water demand pattern that was most representative of the critical water demand scenario for a typical LCH area in Tshwane. The first diurnal pattern represents the week day demand, the second pattern represents the weekend or holiday demand and the last pattern was the combined average pattern of the water demand of each of the LCH zones. From the three derived diurnal demand pattern types, it was determined that the weekend and holiday patterns were the most critical water demand scenario for LCH areas in Tshwane. Therefore, the investigation concluded that the weekend and holiday diurnal water demand pattern was the most appropriate pattern to use for the modelling of WDNs for LCH areas. Refer to Figure 2.5 to view the resulting weekend and holiday diurnal demand patterns for LCH areas. In the current study, the LCH diurnal demand patterns derived by Loubser et al. (2018) were considered.

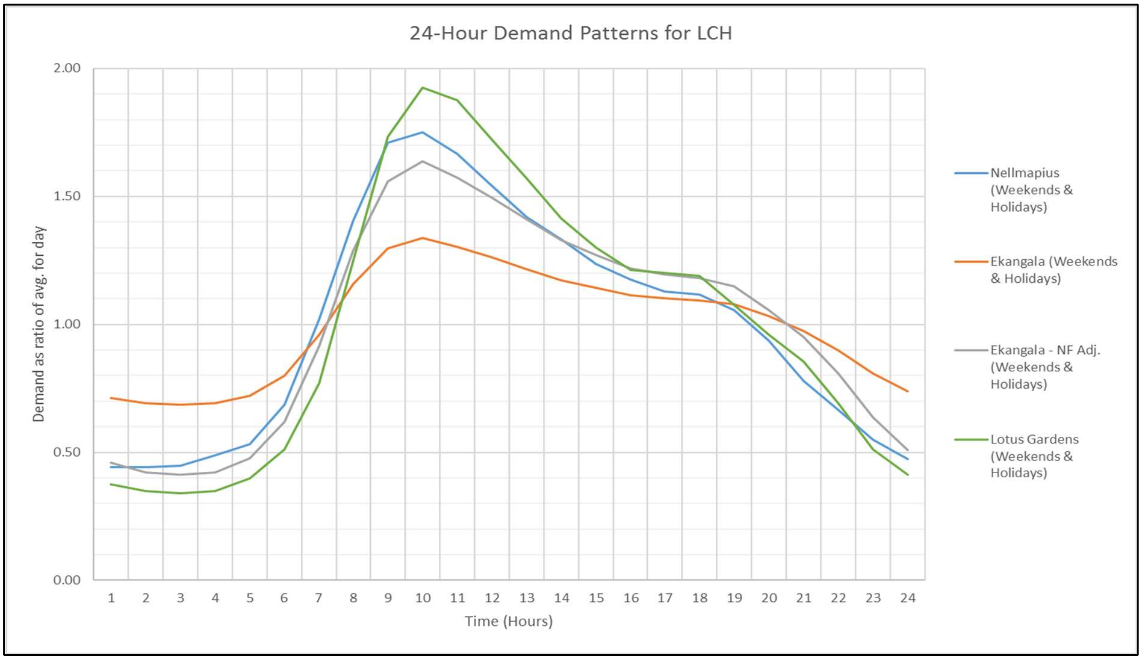


Figure 2.5 Weekend and holiday diurnal demand pattern for the City of Tshwane LCH areas (Loubser et al., 2018).

2.3.3 Peak factors

Definition and importance of PFs

Peak factors, also referred to as peak coefficients or demand multipliers, are the dimensionless ratio between the maximum demand observed over a specific time period and the average demand during the observed extended time period which could be hours, days, weeks or years (Trifunovic, 2006). In South Africa, the average annual daily demand (AADD) is generally used as the average demand over the extended period of time when the PFs are being determined as described by Equation (2-1) (Department of Human Settlements, 2019; Scheepers, 2012).

$$PF = {Q_{max}}/{Q_{avg}} \qquad (2\text{-}1)$$

where:

PF = dimensionless peak factor.

Q_{max} = the maximum demand or peak demand observed over a specific time period (δt) (${m^3}/{s}$, or any other unit of flow rate).

Q_{avg} = the average demand observed during an extended time period, often represented by the AADD ($^{m^3}/_s$, or any other unit of flow rate).

The peak factors describe how the demand of a household or supply region varies with time, as discussed in Section 2.3.2, and are the main component in the derivation of the diurnal water demand patterns. More specifically, the peak factors describe the peak flows (Q_{max}) of a household or supply region, which are the maximum flow rates that occur during the observed period (Scheepers, 2012). The peak flows are obtained by multiplying the AADD by a PF. According to Scheepers (2012), this is the most prevalent method of estimating the peak flows. Determining the peak flows is an essential step in the design of WDSs, because the peak flows represent the maximum demand conditions that the WDS has to meet. Therefore, the sizing and capacity of the pipe network is dependent on the determination of the peak flows of the supply region.

Characteristics of PFs

According to Scheepers (2012) and Diao et al. (2010), there are three main parameters that influence the characteristics of peak factors, namely the time interval for the calculation of the peak factors, the number of consumers considered, and the size of the supply area. Scheepers (2012) investigated the effects of different time intervals (δt) on the PFs for different residential area sizes, and compared the results with the old Guidelines for Human Settlement Planning and Design (the old "Red book"). When considering the old Red book guidelines, Scheepers (2012) noted that the definition of the instantaneous PFs was incomplete, since it is not specified over what time interval the peak flows were measured for the derivation of the instantaneous PFs. Therefore, it is unclear what time interval (δt) constitutes an 'instantaneous' PF. In the study, Scheepers (2012) mentioned that peak factors should be defined in terms of the time interval over which the peak flows are measured. This is because the magnitude of the peak factor for a certain diurnal water demand pattern will vary depending on the chosen time interval (δt). When the time interval over which the peak flows are measured is reduced, the value of the peak factors will increase, therefore the time interval (δt) is inversely proportional to the magnitude of the peak factor (Johnson, 1999). Based on this relationship, it can be deduced that the instantaneous PFs, which are based on the instantaneous flows, would be larger than the peak hour factors (PHF) based on the hourly flows. Similarly, the PHFs would be larger than the peak day factors (PDF) based on the daily flows measured. The average peak flow rate determined over a fifteen second time interval, may be higher than the average peak flow rate determined over a 10 minute time interval.

Furthermore, Diao et al. (2010) mentioned that the number of consumers monitored also has an influence on the magnitude of the peak factors. The higher the number of consumers considered, the lower the value of the peak factors will become. This relationship is based on the observation that when a smaller number of consumers (for example a population of 100 people) is considered, the probability of the consumers using water at the same moment in time is high. This would result in high flows being measured within a short time interval, producing high instantaneous peak flows in comparison to the average flows of the observed extended time period. The resulting peak factors determined using Equation (2-1) would be significantly high. When a larger number of consumers is considered (for example a population of 10 000 people), the probability of the population using water at the same moment in time is lower. Therefore, the resulting instantaneous flows would be lower and the average flows measured over the extended time period would be higher. In this case, the peak factors determined would be lower. Therefore, the magnitude of the peak factors is dependent on the population size of a supply area. This same relationship also applies to the size of the supply area - as the supply area size increases the PFs decrease. A smaller or larger supply area implies a smaller or larger population size. In a study conducted by Vorster et al. (1995) for the East Rand, Gauteng region, the AADD was used as a substitute for the population size when determining the peak factors to be used in the computer model for the analysis of the WDS.

From the results of the PF-time interval investigation by Scheepers (2012), it was found that for a small number of consumers, the maximum PFs (instantaneous PFs) were relatively large, and as the number of consumers increased the PFs decreased. This was due to the decrease of the water demand variation when the water demand of a large number of consumers is combined. Furthermore, it was also found that a small time interval (δt) produced high PFs. As the time interval used to determine the PFs increased, the value of the PFs decreased, because when longer time intervals are used the variation in the measured flows is averaged out, and the value of the ratio between the peak flows and the average flows decreases.

Different methods to derive PFs

Peak factors are normally derived from the logged flows of a particular supply region, and each supply region tends to have particular traits that are unique to that region. This implies that the flow characteristics will also differ from region to region, resulting in the derivation of differing PFs for every supply region (Trifunovic, 2006). Similarly, the methods used to calculate PFs also vary from region to region. Scheepers (2012) identified three types of methods used to derive peak factors; namely deriving peak factors using the logged water consumption data of a supply region, deriving peak factors using empirical equations or figures, and the derivation of peak factors using probability theory. In addition to these methods, Scheepers (2012) developed a probability based end-use model. In South Africa, peak factors are typically derived from the logged water consumption data of the supply region in focus as was done in the study conducted by (Loubser, et al., 2018). Scheepers (2012) commented that there are cases when engineering consultants develop their own “in-house” PFs, such as in the case of GLS Consulting. The peak factors derived by GLS Consulting have been included in the new Red book guidelines (The Neighbourhood Planning and Design Guide). For the calculation of the peak hour demand values, refer to Table 2-3 for the recommended peak factors as derived by GLS Consulting.

Table 2-3 Recommended peak hour, day and week factors derived by GLS Consulting (Department of Human Settlements, 2019).

Predominant land use	AADD (kL/d)	PWF	PDF	PHF
Low cost housing	<1000	1.50	1.90	3.60
	1000 - 5000	1.40	1.80	3.40
	5000 - 10000	1.35	1.70	3.30
	10000 - 15000	1.30	1.50	3.20
	15000 - 20000	1.25	1.40	3.10
	>20000	1.25	1.40	3.00
Residential	<1000	1.80	2.20	4.60
	1000 - 5000	1.65	2.00	4.00
	5000 - 10000	1.50	1.80	3.60
	10000 - 15000	1.40	1.60	3.50
	15000 - 20000	1.35	1.50	3.30
	>20000	1.30	1.50	3.00
Business/Commercial/Industrial	<5000	1.45	1.70	3.30
	5000 - 10000	1.30	1.60	3.15
	>10000	1.25	1.50	3.00
Large single users	>500	1.45	1.70	2.50
Inner city	<5000	1.30	1.60	2.00

2.4 Water demand estimation and influence parameters

2.4.1 Estimating water demand

Determining the water demand, especially the future water demand of a certain supply area, is one of the crucial steps in the planning and design of water distribution systems (CSIR, 2003). Once the water demand for a certain supply area is known, the sizing and capacity requirements of the different components of the WDS can be determined. For example, Arunkumar and Nethaji Mariappan (2011) recommended that the water treatment facilities, storage reservoirs and the water distribution pipe network should be designed to have sufficient capacity to handle the peak hour demand, or the maximum day demand with the demand for firefighting included. In order to determine the water demand, the recorded historical water consumption data of the supply area has to be used and in cases when the recorded data is not available, the Guidelines for Human Settlement Planning and Design recommends that the water consumers in the supply area be consulted to estimate the current water demand. Alternatively, the water demand of the nearby WDS can also be used in the estimation of the water demand of the focus supply area.

The water consumption data used to determine the water demand, and ultimately the water demand pattern, is usually obtained by monitoring and measuring the flows at supply points (treatment plants), specific points within the distribution network (reservoirs, control points) or at the property of the consumer (Trifunovic, 2006). However, Van Zyl (2008) mentioned that in South Africa, the logged water consumption data of the supply areas as measured by the municipalities, is not always readily available. Therefore, the domestic water demand is generally estimated using the available design guidelines.

The water demand during CWS conditions, for when the logged water consumption data is not available, can be estimated using the method recommended in The Neighbourhood Planning and Design Guide (Department of Human Settlements, 2019). In order to calculate the AADD for the scenario when the detailed layout of the development is not known, the area based demand method is recommended, by using Equation (2-2) and Equation (2-3).

$$AADD = Area\ Water\ Demand \times Gross\ Area, \qquad (2\text{-}2)$$

$$AADD = Area\ Water\ Demand \times Net\ Area \div Net\ Area\ Factor, \qquad (2\text{-}3)$$

where:

$AADD$ = average annual daily demand (kL/d).

$Area\ Water\ Demand$ = in kL/ha/d.

$Gross\ Area$ = in hectares.

$Net\ Area$ = in hectares.

For the scenario when the land use, and the number and size of the stands in the supply area are known the unit demand method is recommended to calculate the AADD, by using Equation (2-4).

$$AADD = Unit\ Water\ Demand \times number\ of\ units, \qquad (2\text{-}4)$$

where:

$Unit\ Water\ Demand$ = in kL/unit/d

In cases when the area based method and the unit demand method are not applicable, such as the case of rural water supply or the supply of backyard dwellings in low cost housing areas, the per capita method of determining the AADD is recommended, as described by Equation (2-5). This method is based on the supply infrastructure type, and requires an estimate of the supply area population.

$$AADD = Unit\ Water\ Demand\ per\ capita \times population \div 1000, \qquad (2\text{-}5)$$

where:

$Unit\ Water\ Demand\ per\ capita$ = in L/capita/d

Provision has also been made for the impact of alternative water sources such as rainwater, greywater and groundwater on the AADD. The use of these alternative sources reduces the AADD requirements on the municipal water supply according to the percentages indicated in Table 2-4 and Table 2-5.

Table 2-4 Typical breakdown of internal residential water use (Department of Human Settlements, 2019).

Point of use	Proportion of indoor demand
Toilet	25%
Shower/bath	30%
Washing machine	25%
Tap	18%
Dishwasher	2%

Table 2-5 Typical outdoor water use as percentage of AADD (Department of Human Settlements, 2019).

Land use category		Outdoor use
Low-income housing		0 - 15%
Single residential stands	<500m^2	0 - 20%
	≥500 - ≤1 000m^2	0 - 30%
	>1000 - ≤1 600m^2	0 - 40%
	>1 500 - ≤2 000m^2	0 - 50%
	>2 000m^2	0 - 60%
Cluster housing		0 - 10%
Flats and low-income walk-ups		0 - 5%

The impact of the water losses on the AADD is taken into consideration by using either an estimated percentage range of 15% to 25% of the AADD as the real water losses in the WDS (method 1), or determining the water losses using a target infrastructure leakage index (ILI) obtained from Table 2-6 (method 2).

Table 2-6 ILI benchmark values (Department of Human Settlements, 2019).

Anticipated level of infrastructure leakage	Typical ILI range for developing countries
Excellent	1-4
Good	4-8
Average	8-16
Poor	>16

In order to use method 2, the unavoidable annual real losses (UARL) have to be calculated by using Equation (2-6). Then, the determined UARL is multiplied by the ILI, to calculate the current annual real losses (CARL) as described in Equation (2-7).

$$UARL = ((18 \times L_m) + (0.8 \times N_c) + \left(25 \times L_p\right)) \times AZP, \qquad (2\text{-}6)$$

$$CARL = UARL \times ILI, \qquad (2\text{-}7)$$

where:

$UARL$ = unavoidable annual real losses (L/d).

L_m = length of mains (km)

N_c = number of service connections

L_p = total length of underground pipe between street edge to the user meter (km)

AZP = average zone pressure (m)

$CARL$ = current annual real losses (L/d)

ILI = infrastructure leakage index

The total AADD (TAADD) is then determined using Equation (2-8) or Equation (2-9), depending on the method used to determine the water losses. The peak water demand is then calculated as the product of the TAADD and the recommended peak factors (PFs), as listed in Table 2-3.

$$TAADD = AADD/(1 - Real\ Losses)\ (\ for\ Method\ 1), \qquad (2\text{-}8)$$

$$TAADD = \ AADD + Real\ Losses\ (for\ Method\ 2), \qquad (2\text{-}9)$$

where:

$TAADD$ = total average annual daily demand

2.4.2 Past water demand estimation studies

In this section, a brief review of notable past studies related to domestic water demand estimation for CWS conditions is conducted. Studies by Garlipp (1979), Stephenson and Turner (1996), Van Vuuren and Van Beek (1997), Van Zyl et al (2003), Jacobs et al (2004), Husselmann and Van Zyl (2006), and Griffioen and Van Zyl (2014) are discussed.

Garlipp (1979)

Garlipp (1979) investigated the domestic water demand and the factors influencing the water demand for Pretoria, Bloemfontein, Cape Town, Port Elizabeth and Durban. The investigation was carried out using the logged water consumption data from the meter readings, the water meter books from individual customers, and customer surveys. The results of this investigation indicated that the most influential parameters in these five cities were household size, extended periods of high temperatures, the stand area size, and the area income level. Amongst these parameters the household size was found to be the single most influential parameter to the domestic water consumption. It was observed that the per capita water consumption of these areas increased with an increase in the stand area size (applicable only to the outdoor domestic water use) and also increased with an increase in the income level of the area, but decreased with an increase in the household size. However, Van Zyl et al. (2008)

pointed out that the study surveys used in the investigation were inadequate and biased. As such, it did not represent the true representative water consumption trends in the country, because the surveys were mainly conducted amongst the engineering fraternity in South Africa.

Stephenson and Turner (1996)

In the study conducted by Stephenson and Turner (1996), the logged water consumption of consumers from different income levels in the Gauteng area was investigated. The study focussed on one high income residential area of 242 stands, seven medium income residential areas of 7119 stands in total and two low income residential areas with 2370 stands in total. The income level for the different areas considered, was defined according to the income per annum per household. The results of the study confirmed that the stand area size was the most influential parameter to the domestic water consumption. This is underlined in the CSIR guidelines (CSIR, 1994) which specified a stand size based water demand estimation method. The population density, area income level, housing development type and the level of the water services were also found to influence the water demand of the supply areas. The limitations of the study were the use of the average stand area size for all the stands in the supply area. This, in turn, led to an inaccurate representation of stand area and high AADD values when compared to the actual domestic water consumption of the respective areas (Van Zyl et al., 2008).

Van Vuuren and Van Beek (1997)

In order to update the existing guidelines for urban water demand, Van Vuuren and Van Beek (1997) investigated the domestic water consumption and the non-domestic water consumption of 69 areas in the Pretoria supply region. Measured water consumption data over a 12 year period, from March 1982 to October 1994, was used in the study. Distinction was made between the low, middle and high income areas. The results of the study confirmed the relationship between domestic water consumption and the income level of the households. It was found that the high income households used more water than the middle and low income households, and that the climate also had a considerable impact on the water demand pattern. The water demand pattern of high income households was reported to be more susceptible to the change in climate than for the low income households, due to the higher outdoor water demand associated with high income households. The effects of climate were found to be minimal for low income areas, since the outdoor water demand in these areas was considerably less than the high income areas. Similarly, the effects of climate on the non-domestic water consumption were found to be negligible. Some of the limitations of the study were the accuracy of meter readings, land use characteristics, and the defining of the household income levels based on property rates. In addition, the study only considered formal residential areas with house connections and water borne sanitation (Van Zyl et al., 2008). Van Zyl et al. (2008) also noted that the domestic AADD for all the income levels in the study area determined by Van Vuuren and Van Beek were lower than the AADD estimated by the old Red book guidelines (Guidelines for Human Settlement and Design).

Van Zyl et al. (2003)

Van Zyl et al. (2003) conducted research on the benefits and disadvantages of using end-use modelling to predict water demand. In the study, the consumer water end-uses were categorised into indoor consumption, outdoor consumption and leakage. The influence of variables such as water price, household income level, stand area size, and water pressure on the water consumption of more than 110 000 consumers in the Gauteng area was investigated. Data from consumer surveys, treasury data and data from past research was analysed to determine the elasticity values of the influence variables. Once these values were determined, a sensitivity analysis was conducted to evaluate the effects of the elasticity values on the water demand. The water price was discovered to have the biggest influence on the water consumption patterns. Furthermore, the household income level, stand area size and water pressure were found to have positive elasticity of demand, implying that when any of these variables increase the water demand also increases. According to Van Zyl et al. (2008), this study was beneficial in terms of the type of developments that were investigated, by including both suburbs and townships. However, Van Zyl et al. (2008) pointed out the shortcomings of the study through the exclusion of potential influence parameters such as climate, geographic location, level of service and the age of the infrastructure.

Jacobs et al 2004

In the research conducted by Jacobs et al. (2004), new guidelines for domestic water demand estimation in Southern Africa were proposed. The study was based on a single coefficient model, evaluating the relationship between water demand and stand area size. The water consumption data of 582 997 domestic water consumers from various Southern African towns and cities was analysed over a period of at least 12 months in a period from December 1999 to July 2003. The study only considered stands that use less than 20 kL/day and measure between 50 m^2 and 2050 m^2 in size in both suburb and township areas. It was found that there was a strong correlation between the domestic water demand and stand area size, and three new stand area based models were developed to estimate the residential water demand for three different geographic and climatic areas. It was acknowledged however, that there are numerous parameters that influence water demand. Therefore, it was recommended that the single-coefficient model approach based on stand size should only be used when there are no better alternative methods available. When the water demand versus stand area curves from the three newly developed models were compared to the old Red book guidelines, it was discovered that the old guidelines were overly conservative and that updating of the guidelines should be considered.

Husselmann and Van Zyl (2006)

In another study, Husselmann and Van Zyl (2006) investigated the individual effects of stand area size and area income level on water consumption, using the measured data of 769 393 residential stands from various towns and cities in the Gauteng area. In the study, the stand value was used as a substitute for the area income level and the stands considered were organised into six stand area groups and six stand value groups, each group representing an approximately equal number of data points. The study found a strong connection between water consumption and income level. It was found that an increase in the stand value and the stand area size, results in an increase in the water demand. The authors also noted that stand value was too variable, and that stand area provided a better basis for water demand estimation. Lastly, the authors compared the AADD versus stand area curve results from the investigation with the old Red book guidelines. It was found that for stand area sizes between 300 m^2 and 700 m^2 the AADD in the old Red book was underestimated, and for stand area sizes greater than 700 m^2 the AADD was overestimated.

Griffioen and Van Zyl (2014)

Griffioen and Van Zyl (2014) conducted a study to determine the most important influencing factors on the water demand of residential areas in South Africa, and proposed new updated water demand estimation guidelines. The water consumption data used for the study was obtained from 48 municipal treasury databases considering only the suburban residential areas. The information on the possible factors affecting water demand, was collected from various sources such as the South African municipal demarcation board and the South African weather services. There were 12 influence parameters considered in the study, and these influence parameters were linked to the demand data to allow correlations to be investigated. By using a regression analysis, the importance of the different parameters on the domestic water demand was investigated for 459 residential areas. The study found that stand area size was the most influential parameter on the residential water consumption, justifying the use of stand size as the basis for the water demand estimation method used by the old Red book guidelines. After processing of the collected data from the databases, the AADD values of 739 suburb areas throughout South Africa were analysed and plotted against the related average stand area sizes. A new envelope curve was thus developed, and it was found that the range of AADD values considered by the old Red book guidelines excluded a large number of data from a majority of the suburb areas, resulting in a narrow lower and upper envelope of AADD points when the AADD was plotted against the stand area. Therefore, a new envelope curve was proposed which included the AADD values of these previously excluded areas.

Summary

Overall, the studies reviewed in this section, outline a continual investigation of water demand estimation in South Africa and provide important insight into the different parameters that need to be considered, when investigating methods of water demand estimation. The common approach taken by the studies included two main components, namely the measured water consumption data of the study area, and investigating the important parameters that influence the water consumption or demand. In order to obtain the water consumption data private company databases, the municipal treasury databases and consumer surveys were used by the majority of the studies. The water consumption data sets collected and used in the studies were large and were from diverse sample areas in terms of geographic location and area size. The data was also measured over extended periods of time (most of the studies measured data over at least 12 months). The influence parameters were obtained from various sources, such as the South African municipal demarcation board, past research, the municipal treasury data and the South African weather services. The important parameters were identified and correlated to the water consumption data collected. In order to determine the importance and the level of influence of the parameters on the water demand, sensitivity analyses, and single and multiple regression analyses were performed.

Four out of the seven studies reviewed concluded that stand area is the parameter that has the biggest influence on domestic water demand. Garlipp (1979) reported that household size is the most important parameter, while Van Vuuren and Van Beek (1997) indicated that household income is the most important parameter. Van Zyl et al. (2003) indicated that water price is the most important parameter. When the authors in the four studies that consider stand area to be the most important parameter plotted stand area against water consumption data, it was found that the old Red book guidelines were too conservative. Van Zyl (2006) noted that the water demand estimation method based on stand size, has remained unchanged since the compilation of the “Blue book” (Department of Community Development, 1983) which was the first published guideline in South Africa to provide information for engineering services in residential areas. Since the 1983 Blue book guidelines, the only notable changes that were made in the 2003 Guidelines for human settlement and design (CSIR, 2003) was distinguishing between the domestic water demand estimation for developed and developing areas (Van Zyl, 2008). Therefore, Griffioen and Van Zyl (2014) and Jacobs et al. (2004) proposed multiple parameter based guidelines, which take into consideration the combined effect of all the relevant influence parameters, instead of investigating the influence parameters individually.

2.4.3 Parameters affecting water demand

In order to estimate the water demand of an area, there needs to be a good understanding of the parameters that affect the water demand in that particular supply region (Day & Howe, 2003). Most of the water demand estimation methods in South Africa, are based on these influence parameters that drive the WDSs. Day and Howe (2003) conducted a literature review to identify the main non-climatic parameters that influence domestic water demand, and to determine how these parameters affect the forecasting of the peak water demand. Similarly, Botha and Jacobs (2017) conducted a desktop study, to identify the potential crucial parameters that affect domestic water demand. In the study by Botha and Jacobs (2017), a total of 31 influence parameters were identified, which includes the parameters identified by Day and Howe (2003). Table 2-7 lists the identified influence parameters, as summarised by Botha and Jacobs (2017).

Table 2-7 Parameters influencing domestic water demand (Botha & Jacobs, 2017).

Influence parameters	Literature reference
Appliance water use	Whitford (1972); Hall et al. (1988)
Changes in technology	Agthe and Billings (2002); Day and Howe (2003)
Climate	Foster and Beattie (1979); Metzner (1989); Weber (1989); Tamada et al. (1993); Martinez-Espineira (2002); Zhou et al. (2002); Goodchild (2003); de Lourdes Fernandes Neto et al. (2005)
Conservation attitudes	Syme et al. (2004)
Day of the week	Edwards and Martin (1995); Letpalangsunti et al. (1999)
Demography	Day and Howe (2003); Jacobs et al. (2004)
Distance from city	Durga Rao (2005)
Economy	Bradley (2004)
Employment	Huei (1990); Bradley (2004); Koo et al. (2005)
Garden presence and size	Billings and Jones (1996); Day and Howe (2003); Syme et al. (2004); Fox et al. (2009)
Household income	Foster and Beattie (1979); Billings and Jones (1996); Clarke et al. (1997); Liu et al. (2003); van Zyl et al. (2003); Syme et al. (2004); Husselmann and van Zyl (2006)
Household size (people per household)	Metzner (1989); Russac et al. (1991); Martinez-Espineira (2002); Liu et al. (2003); Bradley (2004)
Housing patterns	Whitford (1972)
Land use	Day and Howe (2003); Durga Rao (2005)
Lifestyle	Syme et al. (2004)
Number of rooms	Huei (1990); Agthe and Billings (2002)
Number of persons	Foster and Beattie (1979); Huei (1990)
Occupancy	Martinez-Espineira (2002); Kowalski and Marshalsay (2005)
Property type	Russac et al. (1991); Clarke et al. (1997); Bradley (2004); Troy and Holloway (2004); Kowalski and Marshalsay (2005); Fox et al. (2009)
Population	Koo et al. (2005); Durga Rao (2005)
Socio-economic	Day and Howe (2003); Kowalski and Marshalsay (2005)
Soils	Durga Rao (2005)
Stand size	Clarke et al. (1997); CSIR (2003); van Zyl et al. (2003); Jacobs et al. (2004); Husselmann and van Zyl (2006); Fox et al. (2009)
Swimming pools	Agthe and Billings (2002); Fisher-Jeffes et al. (2015)
Tenure	Clarke et al. (1997)
Vacancy rates	Agthe and Billings (2002)
Value per bedroom	Agthe and Billings (2002)
Water pressure	van Zyl et al. (2003)
Water price	Whitford (1972); Döckel (1973); Foster and Beattie (1979); Howe (1982); Weber (1989); Dandy et al. (1997); Veck and Bill (2000); Agthe and Billings (2002); Liu et al. (2003); van Zyl et al. (2003); De Lourdes Fernandes Neto et al. (2005)
Water use behaviour	Herrington (1996); Day and Howe (2003)
Water using appliance ownership	Power et al. (1981); Hall et al. (1988); Russac et al. (1991); Herrington (1996)

For water demand forecasting, the National Academy of Sciences (1999) noted the difficulty of accurately predicting the future water consumption of a supply area, due to the many unknowns, ill-defined variables and inconsistency of the water consumption behaviour of the consumers. Identifying and understanding the factors that affect the water consumption does not only assist in the water demand estimation process, but also is useful in the development and assessment of water resources management strategies during the planning phase. It is important to understand which of the parameters can be managed and regulated to improve water use efficiency (National Academy of Sciences, 1999). The volume of water used by the consumers is determined by the available water supply, and by the water demand for the various consumer water activities. The water supply and water demand are further determined by the various influence parameters (National Academy of Sciences, 1999).

According to National Academy of Sciences (1999) there are two categories of factors that determine municipal water supply and water demand. The two categories are the general factors that are independent of location, and the regional factors that are determined by the location specific variables. The general influence factors include:

- Population size and distribution;
- Water use and water saving technology;
- The economic status of the area;
- Environmental factors such as the rainfall, evaporation and temperatures in the area.

The regional factors include:

- Local climate;
- Population size;
- Number of people per household;
- Household water use and water saving technology;
- Household income;
- Water prices;
- Accurate water metering and reporting.

2.5 Water distribution systems during intermittent supply

2.5.1 Domestic water consumption during IWS

In a study conducted in a similar manner to the water demand estimation studies reviewed in Section 2.4.2, Fan et al (2014) investigated the influence and importance of intermittent water supply duration on the domestic water usage activities and patterns of the Wei River region in rural China. The study aim was to explore what effect restricting the water supply time would have on the hygiene related household water use activities and the domestic water consumption patterns. The study region consisted of both intermittent and continuous modes of water supply. Water consumption data was collected from 225 households using surveys.

The study found that by implementing IWS, domestic water consumption can be reduced. However, this mode of supply also had an impact on the hygiene behaviour of the users. A supply duration of six hours or more per day did not affect the indoor domestic water consumption for the rural area, however, when the supply duration was reduced to six hours per day the outdoor domestic water consumption reduced. The indoor domestic water consumption only began to be affected when the supply duration was reduced to be between 1.5 and 6 hours of supply per day. The personal hygiene activities such as face, hands and feet washing became less for these supply durations. The minimum supply duration per day, so that all users were supplied with the minimum required volume of 33.6 – 34.7 L/capita/day, was found to be 1.5 hours. It was observed that the water consumption did not change for supply durations between 1 and 1.5 hours for the different villages. The volume of water collected by the consumers was enough to meet their basic needs. Therefore, a supply duration of 1.5 hours per day was considered to be the minimum allowable supply time that should be supplied for domestic water use.

According to Fan et al. (2014) the effects of restricted water supply on domestic water consumption and water use activities need to be understood, in order to determine an appropriate daily water supply duration when IWS is implemented. The outcomes of the study by Fan et al. (2014) could allow for improved management of domestic water consumption, and can also assist in determining suitable intermittent supply durations for rural areas in developing countries.

Andey and Kelkar (2009) also collected data through water meter readings for four cities in India subjected to both continuous and intermittent supply, in order to investigate the impact of these two modes of supply on the domestic water consumption. The mode of supply for the identified WDSs in each city was switched from IWS to CWS, and the water consumption during each mode was measured. The study found that the domestic water consumption during IWS conditions was mainly dependent on the adequacy of the water supply, rather than the supply duration. It was noted that when the supply duration is long enough, and the pressures in WDS are adequate during intermittent supply, there was no notable increase in the water consumption when changing from IWS to CWS. This was due to people being able to collect an adequate volume of water to meet their needs. However, supply duration and the timing of supply, were still recognised to have a notable impact on the domestic water consumption. Andey and Kelkar (2009) argued that a short supply duration with adequate system pressures can meet the user demand, in the same way that a long supply duration with lower system pressures can meet the user demand.

In another similar study by Andey and Kelkar (2007), the WDSs of the same four cities in India were investigated. The focus of the study was on the pressures, flow patterns, water quality, water leakage, and the water consumption during IWS. The same outcome was achieved, namely that supply duration and the timing of supply had a significant impact on the aforementioned factors. The water consumption during CWS was observed to be 10 to 30% more than for IWS. The peak factors during IWS varied between 2.00 and 6.40, and from 1.66 to 3.00 for CWS. The water losses due to leakage increased by a range of 19.5 to 47.8% during CWS, when compared to IWS.

From the outcomes of the studies discussed, it can be noted that domestic water consumption and peak factors during IWS are mainly reliant on the supply duration, timing of supply and the adequacy of the volume of water supply, which is dependent on the WDS pressures and the WDS hydraulic capacity. Also, an interesting finding is that the reduction of the supply duration does not necessarily result in a reduction in the water consumption.

2.5.2 Network filling process during IWS

There are few published studies that have investigated the domestic water consumption during IWS, and the possible typical water demand pattern during IWS in South Africa. As a result, the typical diurnal water consumption pattern for domestic water consumption during IWS is not as clearly defined as the diurnal water demand pattern during CWS. However, there is a general understanding of the typical logged water consumption data in areas subjected to IWS as seen in Figure 2.6 presented in a study by McKenzie et al. (2014) depicting the measured flows for a residential area subjected to IWS.

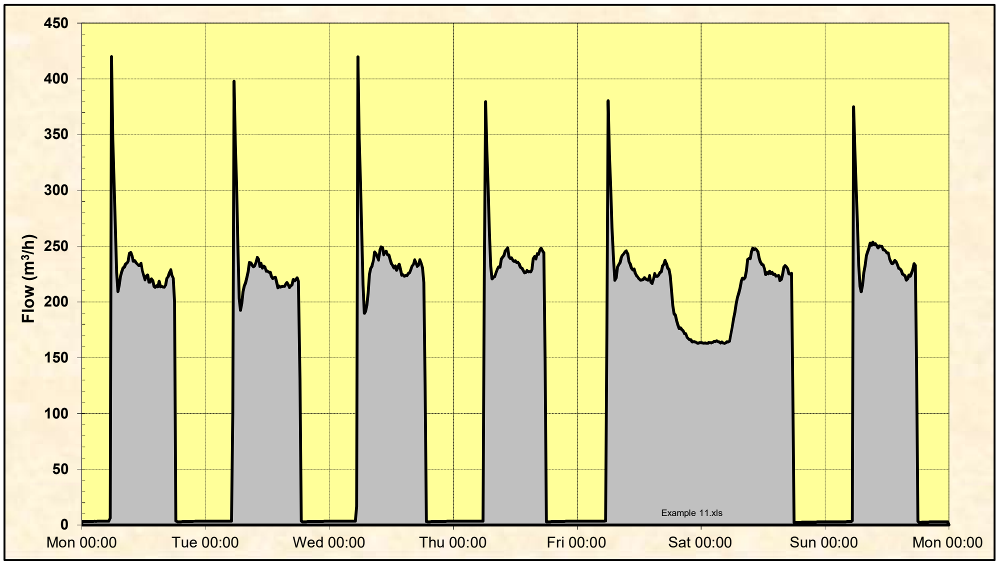


Figure 2.6 Typical logged flows for a residential area subjected to IWS (McKenzie et al., 2014).

A prominent feature in the water flows data related to IWS is the initial spike at the beginning of every supply period. Mckenzie et al. (2014) attributed this initial spike to the rapid refilling of the empty pipes. The spike is caused by the flow of the initial volume of water required to fill the empty pipes after a period of zero pressures in the pipeline. Mckenzie et al. (2014) also mentioned that the sudden high flow rates in the pipeline can cause damage to the water meters and in the long term would cause an increase in the number of leakages in the reticulation network. Furthermore, the air in the pipes would also cause a problem if there are no air outlet points in the network. Eventually the use of IWS would cause a complete breakdown of the WDN.

Liou and Hunt (1996) developed a model to describe the filling of empty pipelines with changing elevation profiles. Equation (2-10) and Equation (2-11) were developed to model the lengthening of a rigid water column filling an empty pipeline, and determining the water column length, velocity and pressure over time (Liou & Hunt, 1996).

$$\frac{dV}{dt} = \frac{\frac{g}{L}\left(H_R - K\frac{V^2}{2g}\right) + g\sin\theta - \frac{fV^2}{2D}}{1 + \frac{Lo}{L}} \qquad (2\text{-}10)$$

$$L = L_{min} + \int_0^t V\,\delta t \qquad (2\text{-}11)$$

where:

V = velocity of the water column ($^{m}/_{s}$).

L = length of the water column in the pipeline being filled (m).

L_{min} = length of the water colume at t=0, (m).

θ = downward angle of the pipe from the horizontal (degrees)

H_R = the available head at the reservoir (m).

K = entrance and valve losses coefficient

g = gravitational acceleration $(^{m}/_{s^2})$.

D = pipe diameter (m).

t = time taken for the pipeline to fill (seconds)

f = Darcy-Weisbach friction factor

Lo = local flow acceleration of the flow approaching the pipe inlet from the reservoir (m).

Four main assumptions were made in the development of Equation (2-10) and Equation (2-11). The first assumption was that the pipe is always full, which means that there will be a well-defined water column front. The second assumption was that the pressure at the front surface of the water column was assumed to be atmospheric. The third assumption was that the water in the pipe is incompressible, which means the water column was assumed to be rigid, implying that there was no air entrapped in the water column (therefore there is no air intrusion). The fourth assumption was the use of the frictional flow resistance for steady state flows. The results obtained from the developed equations for the beginning phase of the filling process, were verified with the results from laboratory experiments of the pipeline filling process. It was found that the model compared well with the experimental results. The findings show that during the initial phase of pipeline filling, there would be rapid acceleration of the water column for a short time period, followed by gradual deceleration, as illustrated in Figure 2.7.

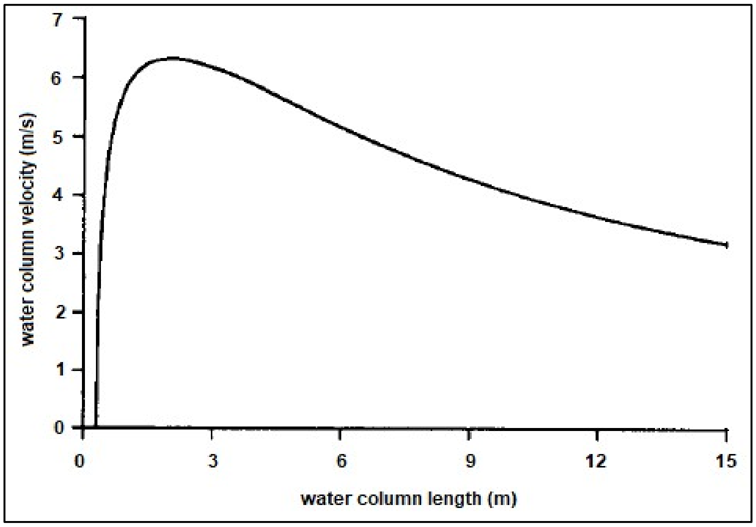


Figure 2.7 Water column velocity at the initial stage of the pipe filling process (Liou & Hunt, 1996).

These finding were confirmed by Hou et al (2014), who conducted experiments to investigate the water and air relationship during the rapid filling of pipelines, as well as the overall behaviour of the water column during pipe filling. The observations from the experiments were compared to the model simulation results obtained from using the rigid column model developed by Liou and Hunt (1996). It was found that the experimental results and the model simulation results had similar trends, as illustrated in Figure 2.8. The results of the one dimensional rigid column theory calculation, were compared to the flow rates observed in the experiments.

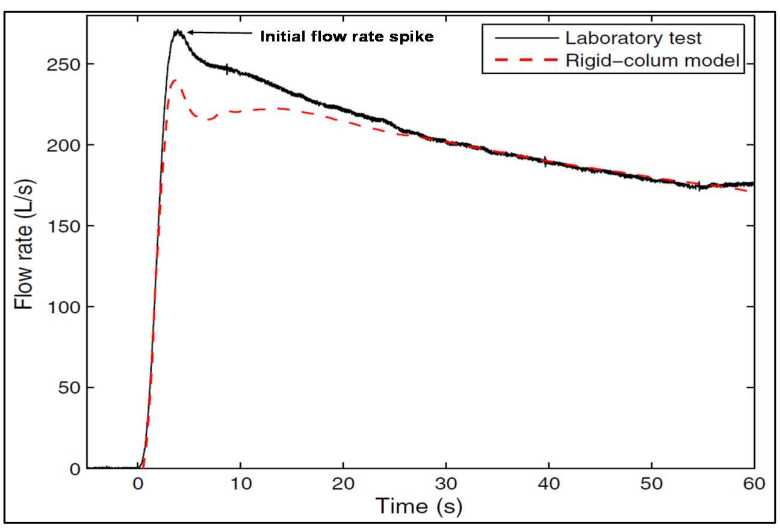


Figure 2.8 Comparison of the experimental flow rate and the simulated flow rate during pipe filling (Hou et al., 2014).

Hou et al. (2014) explained that the initial spike in the flow rate in Figure 2.8, is a result of the driving pressure in the pipeline. The deviation in the maximum flow rate, is due to the free overflow and air intrusion in the pipeline. Hou et al. (2014) concluded that the similarity between the rigid column theory and the experimental results, implied that the effects of air intrusion on the filling process during the rapid filling of pipelines were minor.

By using the established theory related to the rapid filling of pipelines, and the pressure dependent analysis methods, De Marchis et al. (2011) developed a numerical model to describe the network filling process for the IWS scenario, making provision for the private tanks often installed by the users during IWS conditions. The model was then applied to a real case study, and the supply inequality in the network associated with IWS was investigated. To simplify the complex multiphase filling process, the same assumptions as Liou and Hunt (1996) were applied in the model development. Similar to Liou and Hunt (1996), it was assumed that the water surface at the front of the water column had atmospheric pressure and that the water column front coincided with the pipe cross section. This implies that the pipe sections were either entirely wet or entirely dry. Furthermore, the steady state equations were used to calculate the frictional resistance. However, De Marchis et al. (2011) noted that the rigid column theory, developed by Liou and Hunt (1996), was not suitable for the modelling of complex WDNs. It is more suitable for simple or singular pipeline systems. Therefore, the method of characteristics was used to simulate the network filling process, as described by Equation (2-12) and Equation (2-13). These equations are modified versions of the momentum and continuity partial differential equations, and can be solved using finite-difference methods

$$\frac{dV}{dt} + \frac{g}{c}\frac{dH}{dt} + gJ + Vsen\theta = 0, \quad (2\text{-}12)$$

$$\frac{dV}{dt} + \frac{g}{c}\frac{dH}{dt} + gJ - Vsen\theta = 0, \quad (2\text{-}13)$$

where:

V = velocity averaged over the pipe section ($^{m}/_{s}$).

t = time ($seconds$).

c = celerity of the pressure waves ($^{m}/_{s}$).

H = water pressure head (m).

g = gravitational acceleration ($^{m}/_{s^2}$)

J = head loss per unit length of pipe according to the Darcy-Weisbach equation (m).

θ = pipe slope

This numerical model was applied to the Palermo, Italy WDS case study. The model adequately simulated how each node received water supply depending on the node distance from the network inlet node. It was also observed that there is a rapid pressure spike in the network when the supply begins, until the private tanks start to be supplied. As the private tanks are filling, the network pressures decline and then gradually rise again as the private tanks become full. Refer to Figure 2.9 for an example of the pressure profile for a node in the WDN during the filling of the Palermo WDS. Figure 2.9 illustrates the comparison between the actual measured water pressures in the WDN for a particular node, represented by the dotted points, and the numerically modelled pressures in the network for the same node, represented by the line plot.

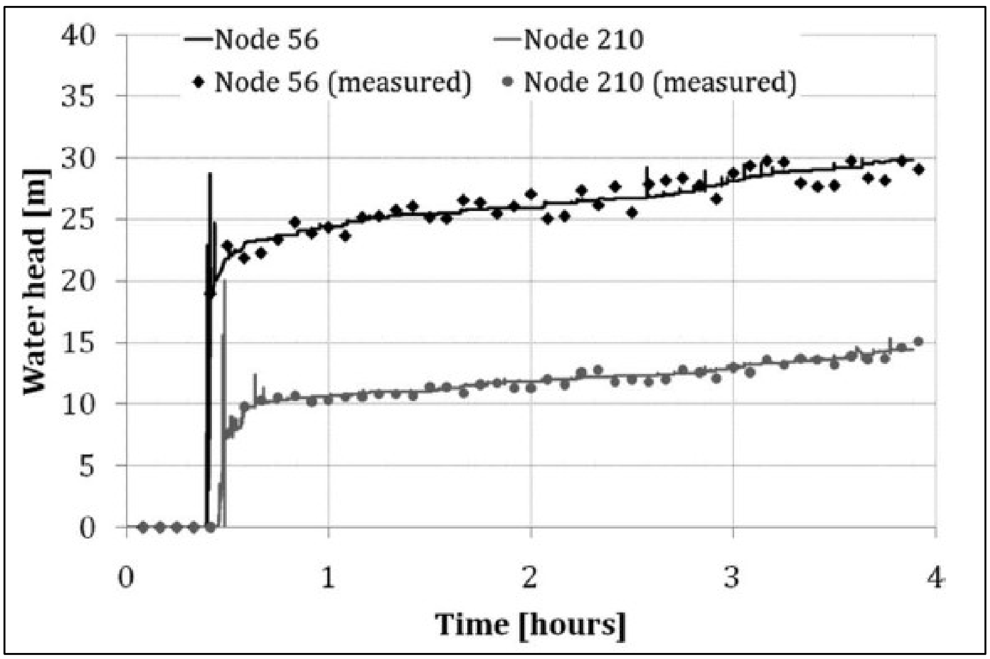


Figure 2.9 Comparison of the measured and simulated pressure head at the nodes during network filling (De Marchis et al., 2011).

2.5.3 Influence parameters during IWS

Studies focussing solely on the factors or parameters affecting water demand during IWS conditions are limited. However, certain IWS influence factors have been highlighted in sections of various studies, providing a good indication of the most influential parameters that affect the water demand of WDSs subjected to IWS. In Table 2-8, the influence parameters identified in the reviewed literature sources are listed.

Table 2-8 Parameters influencing domestic water demand and WDS performance during IWS.

IWS influence parameter	Literature reference
Development	Galaitsi et al. (2016)
Population	Galaitsi et al. (2016); Totsuka and Trifunovic (2004)
Water price	Galaitsi et al. (2016); Abu-Madi and Trifunovic (2013)
Pressures in the system	Galaitsi et al. (2016); Totsuka and Trifunovic (2004); Ilaya-Ayza et al. (2017)
Private tanks/storage	Galaitsi et al. (2016); Abu-Madi and Trifunovic (2013); De Marchis et al. (2011); Vairavamoorthy et al. (2001)
Illegal connections	Galaitsi et al. (2016); Ilaya-Ayza et al. (2017)
Pressure at the outlet	Totsuka and Trifunovic (2004)
Supply duration	Totsuka and Trifunovic (2004); Ilaya-Ayza et al. (2017); Fan et al. (2014); Abu-Madi and Trifunovic (2013); Andey and Kelkar (2007); Andey and Kelkar (2009)
Timing of supply	Totsuka and Trifunovic (2004); Andey and Kelkar (2007); Andey and Kelkar (2009); Ilaya-Ayza et al. (2017)
Flow rate at the outlet	Totsuka and Trifunovic (2004); Ilaya-Ayza et al. (2017)
House connection type	Totsuka and Trifunovic (2004); Ilaya-Ayza et al. (2017)
Location of the connection	Totsuka and Trifunovic (2004); Ilaya-Ayza et al. (2017)
Topography	Abu-Madi and Trifunovic (2013); Ilaya-Ayza et al. (2017)
Water availability at the source	Abu-Madi and Trifunovic (2013)
Supply area size	Ilaya-Ayza et al. (2017)
Network topology	Ilaya-Ayza et al. (2017); Gottipati and Nanduri (2014)
Supply source location	Ilaya-Ayza et al. (2017)
Network capacity	Ilaya-Ayza et al. (2017)

In a study by Totsuka and Trifunovic (2004), new design guidelines were developed to assist in optimising WDSs subjected to IWS, thus mitigating some of the negative consequences associated with IWS. The guidelines include modified network analysis simulation and design tools, which are based on the objectives of achieving equity in supply and people driven levels of service (PDLS), at the lowest cost. In the guidelines, four design parameters were identified as the crucial parameters that affect supply inequality and the levels of service of WDSs subjected to IWS. These parameters were supply duration, timing of the supply, pressure at the outlet (or flow rate at the outlet), and the type of connection required (which also included the location of the connection). According to Totsuka and Trifunovic (2004), the requirements related to these parameters are automatically satisfied during the design of CWS systems. For the design of IWDS, these parameters are important variables that have to be considered.

The calculations of the four crucial parameters in the guidelines, were based on the relationship between the outflow at the water connection, and the available pressure at the connection outlet. During IWS conditions, the demand for water at the nodes in the network are not based on the diurnal variations of demand related to the consumers' behaviour as with CWS systems. Instead, the demand is related to the maximum volume of water that can be collected during the supply period. The volume of water collected by the consumers, is completely dependent on the driving pressure heads at the outlets. Therefore, the relationship between the pressures in the system and the demands are important. Unlike CWS systems, it cannot be assumed that demand will be satisfied under all conditions.

In order to make provision for the pressure dependent outflow conditions, functions to model the outflow were developed by Totsuka and Trifunovic (2004). The functions consisted of three components, which include: the demand model, the secondary network model and the modified network analysis model. The demand model used queuing theory and reservoir routing to predict the end-users demand profile during IWS, which includes determining the intensity and distribution of the water consumption during the supply period. The type of connection, timing of supply, supply duration, and the pressure regime were the input data used for this model. The secondary network model lumped pressure dependent outflows for a group of nodes, into one primary node. The modified network analysis model used the gradient algorithm of Todini and Pilati (1987) to solve the governing equations.

2.5.4 Modelling water distribution systems

The use of software models to design WDSs according to recommended design criteria, to ensure an adequate level of service is common practice. A well designed WDS can often result in not only satisfactory levels of service to the end users, but also significant cost savings. Bhoyer and Mane (2017) noted that the WDS should be designed to adequately supply the water demand for the domestic, commercial, and industrial water use activities, also making provision for the firefighting water demand. This is achieved by ensuring adequate pressures and flow velocities in the WDS. The models simulate the effect of various water demand scenarios on the pressures and flows throughout the WDS (Wurbs, 1994). Several modelling software programs have been developed to perform hydraulic analysis of WDSs. With the rapid development of technology, new mathematical models are becoming available with EPANET, WaterCAD, WaterGEMS, and LOOP being among the most commonly used software programs (Bhoyar & Mane, 2017; Wurbs, 1994). According to Trifunovic (2006), most of the available WDN modelling software share common features and have similar concepts such as:

- Demand driven calculation modules or steady state analysis simulations;
- Extended period hydraulic simulations;
- Integrated modules for water quality simulations;
- Graphical user interfaces for the presentation of results;
- Almost unlimited capacity to handle large network sizes and to compute complex network configurations.

From the basic steady state and extended period simulations, more advanced simulations could be added, such as fire flow analyses, and transient analyses (Bistriceanu, 2006).

According to Trifunovic (2006), the entire modelling process consists of the following steps:

1. The collection of input data, where information related to the layout of the system, the water demand, and the operation and maintenance of the system is gathered.
2. Network schematization, which is the simplification of the complex network system characteristics to a certain level when the model accuracy is not substantially affected, enabling faster processing of the software calculations.
3. The model building process, which is the process of converting the real-world WDS scenario to a model that represents the actual WDN as links and nodes. The WDN models have many types of elements, which include the junction nodes where pipes connect, storage tank and reservoir nodes, pump nodes, and control valve nodes. Models use these elements to describe the pipes connecting these nodes. The nodal demands are determined and assigned to the nodes.
4. Model testing, when the various types of simulations are performed on the defined WDN model. The results of the simulation can be validated and calibrated to match reality.
5. Problem analysis, which is the final step of the modelling process. This step is the optimization of the pipe diameters, pumps, reservoirs and valves. Additionally simulation of fire events can also be investigated along with the analysis of water quality in the system.

In most available modelling software simulations, only the demand driven analysis of CWS systems is considered. Limited provision is made for the IWS scenario. Under the assumption of demand driven analysis, the model nodes are assigned fixed demand values, and the problem is to determine the system pipe flows and nodal pressures that meet the nodal demand (Mohapatra et al., 2014). However, this assumption is not applicable to IWS systems, because WDNs under IWS conditions are pressure head driven rather than demand driven. This implies that the volume of water drawn by the users is dependent on the WDN hydraulic capacity and the available pressure head in the network system, rather than the nodal demand as in the CWS systems (Totsuka &Trifunovic, 2004).

There is limited research related to the modification of modelling software to simulate IWS, but in one of the few studies found by Mohapatra et al. (2014), it was proposed that the demand nodes be replaced by reservoirs that represent the pressure head at a particular node. This modification adjusts the WDN model to simulate the pressure dependence of the system. In this case, the hydraulic grade line of the demand nodes is set equal to the desired pressure head at the node. The pressure dependent demand at each node was explained using Equation (2-14) (Mohapatra et al., 2014).

$$Q_i = Q_{max}\sqrt{\frac{H_i - H_{min}}{H_{max} - H_{min}}} \qquad (2\text{-}14)$$

where:

Q_i = the demand or flow at a node i, ($m^3/_s$, or any other unit of flow rate).

Q_{max} = the maximum or desired demand or flow at a node i, ($m^3/_s$, or any other unit of flow rate).

H_i = the pressure head at a node i, (m).

H_{max} = the maximum or desired pressure head at a node i, (m).

H_{min} = the minimum required pressure head at a node i, (m).

This equation describes the head-flow relationship used to determine the flows at the demand nodes, based on the pressure head available at the nodes. The demand at each node (Q_i) is satisfied when the available pressure head at the node (H_i) is above the desired head (H_{max}). When the available head at the node falls below the minimum head, there is no flow at the node (Q_i=0). The nodes with an available pressure head above the minimum head, but below the desired head, would have the demand partially satisfied (Siew & Tanyimboh, 2011; Wagner et al., 1988). Refer to Table 2-9 for a summary of the flow conditions during intermittent supply in relation to Equation (2-14).

Table 2-9 Pressure dependent flow conditions during intermittent supply (Mohapatra et al., 2014).

Pressure head condition	Demand at the node	Flow condition at the node
$H_i \geq H_{max}$	$Q_i = Q_{max}$	Maximum flow
$H_{min} < H_i < H_{max}$	$0 < Q_i < Q_{max}$	Partial flow
$H_i \leq H_{min}$	$Q_i = 0$	No flow

3 Methodology

3.1 Introduction

This chapter provides a description of the overall research methodology followed. The research approach chosen for the study to achieve the study purpose, the nature of the study and the research design structure of the study are discussed. Also highlighted in this chapter is the justification of the chosen research approach.

3.2 Overall research methodology

According to Blaikie (2000) research can be conducted using four possible approaches which are: deductive, inductive, abductive and retroductive. The approaches mentioned provide different ways to investigate a research topic or answer a research question. The goal of the inductive approach is to derive a theory or generalization from data or observations that have been collected and analysed and to use these derived theories to explain further observations. Then, the deductive approach is often used to test existing theories or hypotheses with actual collected data, checking how these hypotheses match the actual data. This approach begins with provisional theories that are applied to a certain observed phenomenon that needs to be explained and hypotheses are deduced from these theories and tested using actual data which has been collected, as a result the applied theories would be either approved or disproved. Similarly, the retroductive approach also begins with an existing theory or observed regularity, but this is followed by the development of a hypothetical model that can possibly explain the reason for the existing theory then through observation and experimentation an investigation is conducted to test if the explanatory hypothetical model is true. Lastly, the abductive approach explores the possible factors or data related to a certain phenomenon in order to understand and describe the phenomenon with the goal of developing a theory and testing the theory iteratively. The research approach chosen for the current study is a deductive approach.

To develop a theoretical model of the IWS diurnal water demand pattern the theories and results of existing research literature related to IWS network filling, water demand estimation, water demand patterns and influence factors were collected and reviewed. Currently there is limited research on IWS water demand estimation and IWS diurnal water demand modelling, however the combined application of the different outcomes, principles and theories from the existing IWS studies reviewed provided a theory base for the assumptions made to estimate the IWS diurnal water demand pattern. Most of the research papers found related to the derivation of diurnal water demand patterns considered only the CWS systems, there were no similar studies and methodologies found investigating the derivation and modelling of the IWS diurnal water demand pattern in South Africa. Internationally, one published article was found by Totsuka and Trifunovic (2004). In the study a method of estimating the end-users water demand profile for pressure dependent WDSs is described. This method uses queuing theory and reservoir routing to forecast the intensity and distribution of the user water consumption over a certain supply period, but no methodology describing the derivation of this method is presented because it was not the focus of the research.

Therefore, it was decided that a new method of investigating the IWS diurnal water demand patterns had to be developed for the current study. Traditionally, the water demand and the water demand patterns of a supply area are derived using the measured flow data from the water meters installed on site. However, for the current study there was no readily available IWS field data for a supply area where IWS is implemented formally. Therefore, the traditional approach of deriving demand patterns could not be used, and also the inductive study approach could not be considered. Water meter loggers had to be installed at an identified study site where formal IWS is being implemented, in order to obtain actual water consumption data for the model validation phase of the study. The developed water demand pattern prediction model was validated by comparing the model to the derived water demand patterns, by using the actual field data to either approve or disprove the model prediction. The current study serves as an initial exploratory investigation and attempt to better understand and define the typical form and characteristics of IWS diurnal patterns, and how these patterns can possibly be predicted or modelled in cases when logged field data is not available.

The chosen research approach also influenced the choice of the research design. There are two types of research design: quantitative research design and qualitative research design. A quantitative research design was chosen for the current study. A quantitative research design is used to test theories or assumptions and emphasises the statistical, mathematical or numerical analysis of the data collected. For this type of research design, the interpretation of the results is mainly expressed in numbers, graphs and tables (Babbie, 2010). The research design for the study consists of two parts. The first part consists of a non-empirical study of the concepts related to IWS network filling, water demand estimation, and the WDS influence factors in order to derive a theoretical model of the IWS diurnal demand pattern. This part of the study includes identifying the important components for the structuring of the model, and understanding these important components. This was achieved by investigating and establishing the nature of the relationships among the component variables using the theory, formulae, and principles found in the existing literature sources reviewed. The second part of the study design is an empirical investigation of the validity and accuracy of the theoretical model, by conducting a case study. The case study aimed to compare the model prediction of the IWS diurnal water demand pattern to the water demand patterns derived from the actual field data measured in the study area, which is subjected to formal IWS. Then, based on the results of the comparison, a calibration of the model was conducted for a better representation of the real-world scenario. Refer to Figure 3.1 for an illustrative summary of the overall research design.

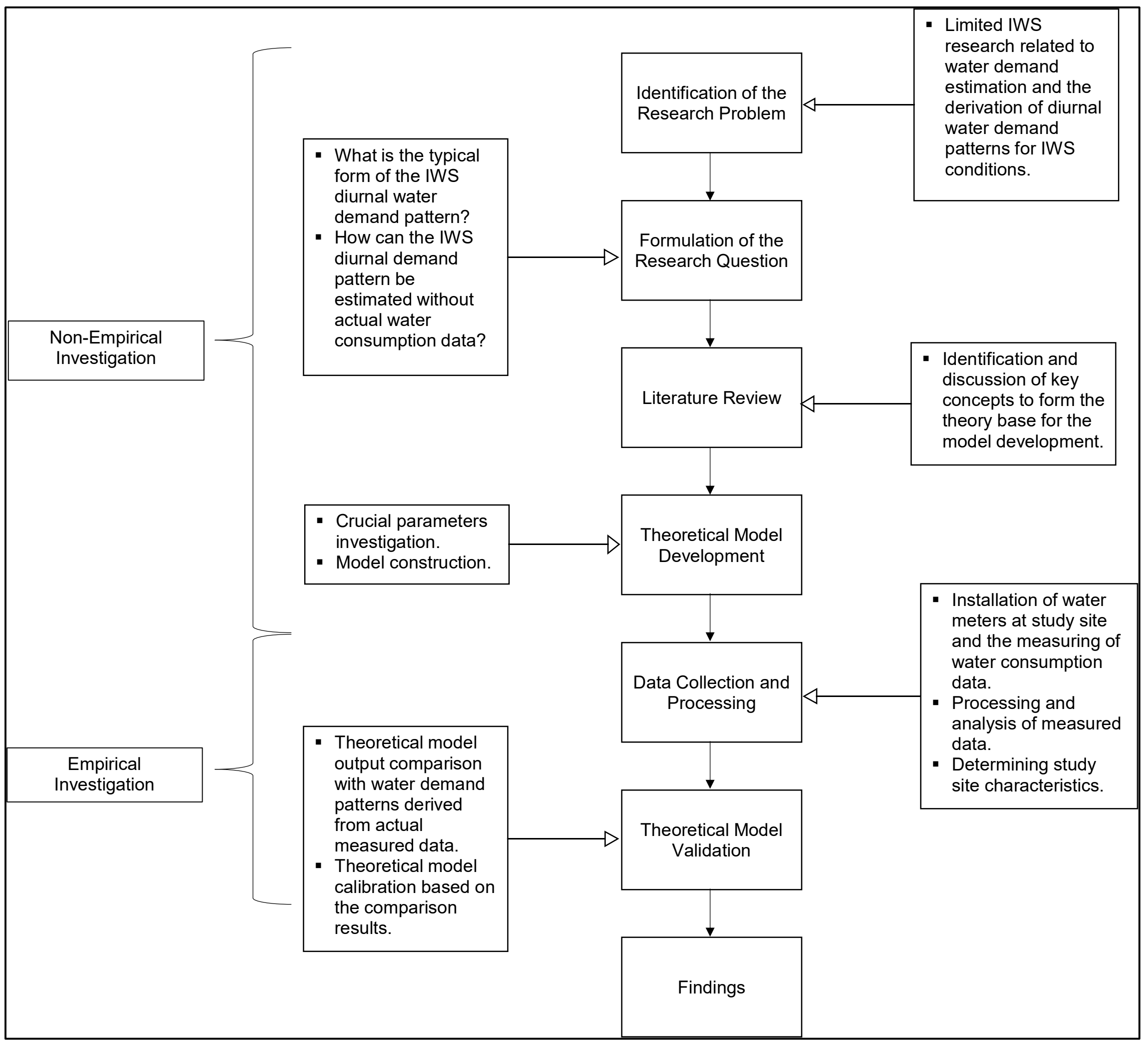


Figure 3.1 Research design flowchart

4 Model development

4.1 Introduction

This chapter covers the second and third step of the research objectives which includes: The influence parameters investigation to find the most important influence parameters, and the development of the IWS diurnal water demand model. These two main objectives are discussed in this chapter, describing the equations, principles, theory and steps used to achieve the study purpose.

4.2 Model development concept

According to Scheepers (2012), Totsuka and Trifunovic (2004), the demand for networks under IWS is not driven by the consumer water use pattern as in CWS, but rather by the maximum volume of water that can be drawn from the network or can be supplied by the network within the supply duration. The volume of water drawn by the users is dependent on and driven by the WDN hydraulic capacity, and the available pressure head in the network system, rather than the user demand as in the CWS systems.

Since IWS systems are more dependent on the physical hydraulic capacity of the system and the pressures in the system than the consumer water use behaviour, it is possible to describe and make an initial prediction of the behaviour of the water consumption in the network using the different hydraulic principles and equations developed, and driving parameters identified as seen in the literature review. Apart from actual field measurements, it is difficult to attempt to model the human behaviour related to water use, since there are numerous variables and factors to consider. While human behaviour varies according to region, modelling the flow in a WDN is achievable through hydraulic principles. The theories and equations used to estimate and describe such flows exist, and most of the influence factors have been investigated, identified and described. Furthermore, the research and equations related to water demand estimation associated with CWS are well established.

Therefore the concept in the development of the model was to take what is known of the CWS water demand patterns, and convert this information to a hypothetical IWS water demand pattern equivalent. By using the outcomes of the parameters investigation combined with theory, an Excel based mathematical worksheet was created. This spreadsheet can be used to investigate the effects of varying the parameters of the WDS hydraulic characteristics and the CWS water demand pattern. Due to the exploratory nature of the study Excel was considered to be the most suitable software application to use in the model development due to its versatility, availability and ease of use. In Figure 4.1 the diagrammatic illustration of the overall model development concept is presented.

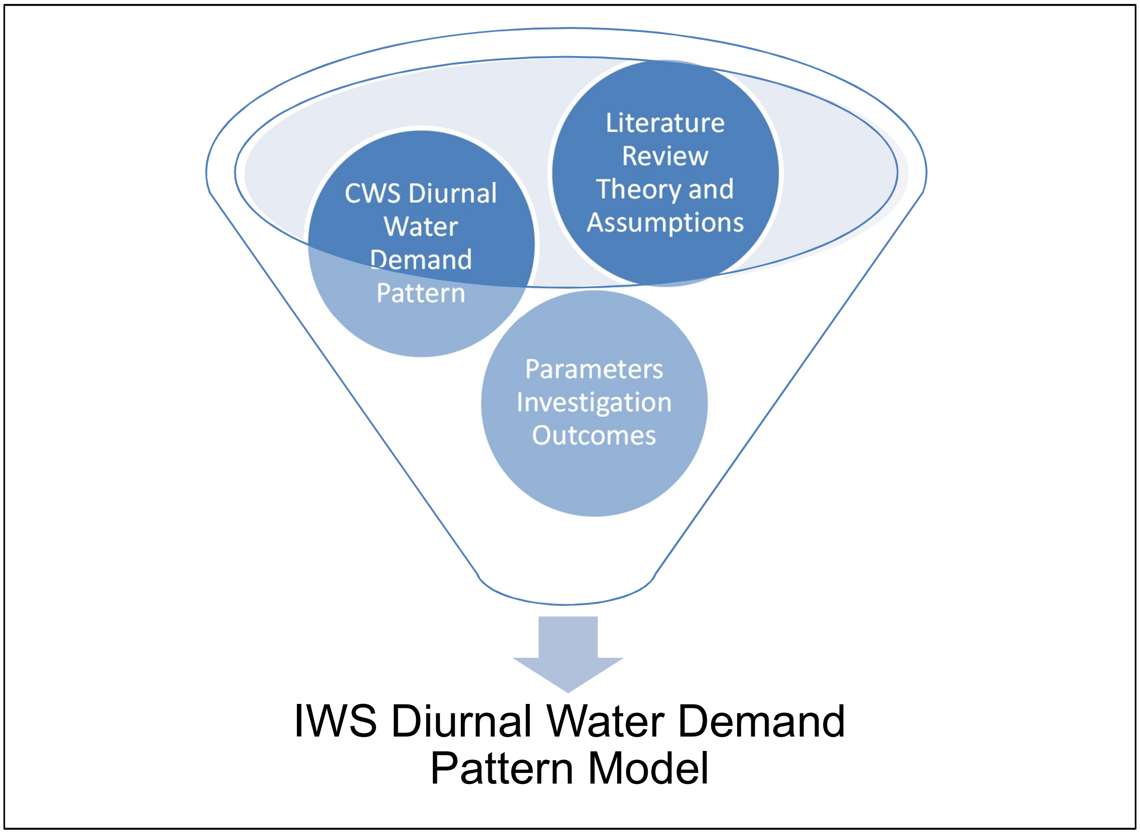


Figure 4.1 Model development concept.

4.3 Parameters investigation

To begin the process of developing the model structure, a parameters investigation was conducted in order to identify the crucial driving parameters that affect WDSs. The aim was to establish the correlation of these influence parameters with the water consumption of the supply area and the performance of the WDN. The driving parameters were envisaged to form part of the building blocks of the model structure. To derive water demand patterns, each component of Equation (4-1) has to be known:

$$PF = {}^{Q_{max}}/_{Q_{avg}} \tag{4-1}$$

where:

PF = dimensionless peak factor.

Q_{max} = the maximum demand or peak demand observed over a specific time period (δt) ($^{m^3}/_{s}$, or any other unit of flow rate).

Q_{avg} = the average demand observed during an extended time period, often represented by the AADD ($^{m^3}/_{s}$, or any other unit of flow rate).

Each component of Equation (4-1) is derived from actual measured water consumption data that is obtained either from the municipal databases, or directly from the installed water meters as discussed in Section 2.3.3 and Section 2.4.1 of the literature review. This data, however, is not always readily available and as a result there has been a number of studies conducted in South Africa to estimate the water demand as reviewed in Section 2.4.2. Through review of these past water demand estimation studies, it was noticed that the methodology used always included an investigation of the factors or parameters that affect water consumption. The various influence parameters are often identified and collected from existing databases, such as the South African municipal demarcation board.

Then, the correlation of these parameters with the water supply systems performance and the water demand is investigated using correlation and multiple regression analysis. The correlation analysis is used to determine the association between two or more variables, and the multiple regression analysis is used to determine the type of relationship between the variables, by considering the cause and effect relationships of the variables. Once the nature of the parameters and water consumption relationship is established and better understood, the development of various methods of predicting water demand becomes possible. Therefore, in the development of the IWS diurnal water demand pattern estimation model, a parameters investigation was considered to be imperative.

Due to a lack of readily available water consumption data and parameters information for areas subjected to IWS in South Africa, the standard methodology from the previous studies could not be used. Therefore, a literature based approach was chosen, in order to identify the relevant influence parameters. A total of 18 literature sources were consulted to identify the relevant parameters that influence water demand and system performance. To establish which parameters were the most important from the identified influence parameters, a combination of frequency analysis and scenario building was conducted. Once the key driving parameters were identified, the interrelationships between the parameters, water consumption, and the WDS performance were investigated, along with the effects of the parameters on the water consumption and on the WDS performance. Each of the literature sources contained further description of the parameters considered, enabling the study of the different possible correlations and relationships between the parameters, water consumption and system performance. From this investigation a table was compiled (refer to Table 6-1) where the parameters were categorised and correlations were made between the parameters, water consumption and system performance.

From Table 6-1, important parameter effects and relationships could be identified. This was achieved by firstly determining the number of parameters affected by a specific driving parameter, then determining the number of parameters that affect this specific driving parameter. The final step involved was determining whether a link exists between the driving parameter, and the water consumption and system performance.

4.4 Model structure

With the crucial parameters identified and the correlations and effects of the parameters on the water consumption and the WDS performance determined, it became possible to investigate the form of an IWS diurnal water demand pattern. By taking what is known of the WDS characteristics, and the CWS water demand pattern, it was possible to deduce a hypothetical IWS equivalent demand pattern. This was achieved by varying the crucial parameters with an Excel based mathematical investigation as discussed in the following sub-sections.

4.4.1 Excel input requirements

For the Excel based mathematical investigation, the overarching assumption was that the IWS model pattern form is based on three phases of the WDS filling process. Phase 1 represents the filling of the bulk supply pipe between the storage reservoir and the reticulation network, phase 2 represents the filling process of the water reticulation network, and phase 3 represents the water demand when the network is full. The three phases were defined according to the application of the different equations and principles related to pipeline filling, WDN filling using pressure dependent analysis (PDA), WDS characteristics during IWS, and water demand estimation theory. The Excel worksheet was structured in 4 steps as presented in Figure 4.2

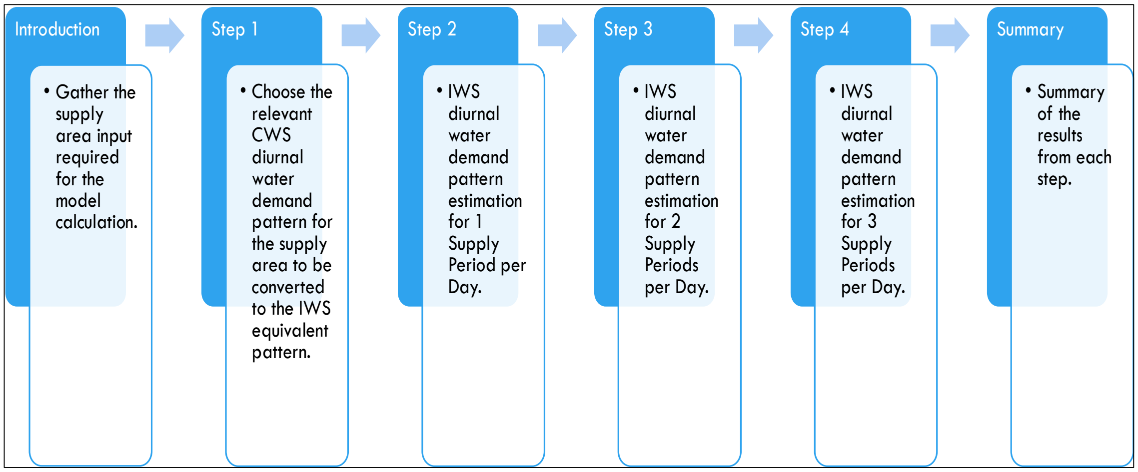


Figure 4.2 Model calculation procedure.

For each supply period considered in steps 2 to 4 in Figure 4.2, the calculations for the three phases had to be performed. Table A.1, Table A.2, Table A.3 and Table A.4 were included in Appendix A for an illustration of the Excel calculation worksheet with the calculations for each of the three phases.

The assumed WDN configuration in the Excel based calculations according to the three phase assumption is presented in Figure 4.3. For this configuration certain WDN characteristics needed to be assigned or defined as input in the Excel worksheet. Similarly, the supply area characteristics related to the assumed WDN configuration needed to be defined.

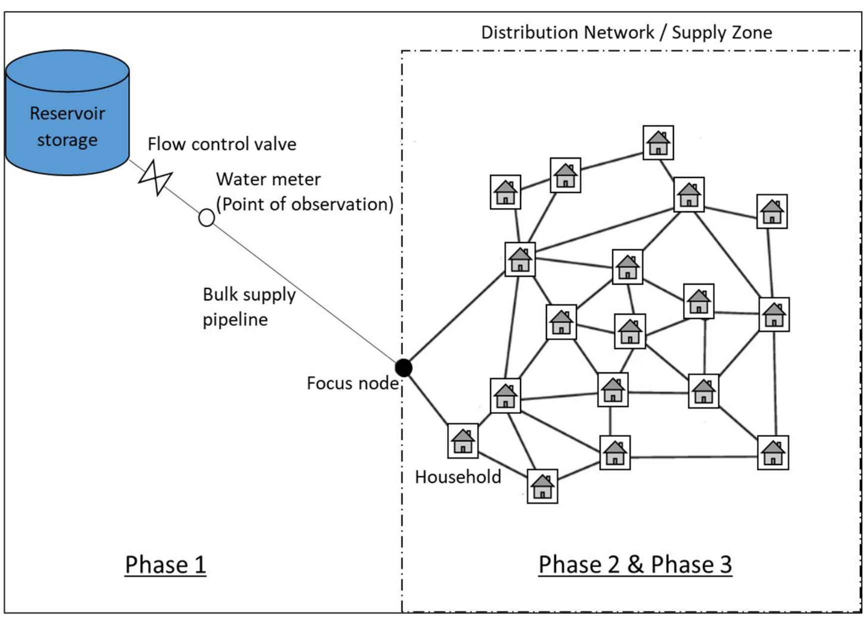


Figure 4.3 Assumed WDN configuration.

The WDN characteristics required for the calculation of the theoretical equations were:

- Total WDN length and volume;
- Pipe materials constituting the network;
- Range of pipe diameters in the WDN;
- Maximum and minimum allowable flow rates and velocities for the various pipe diameters and pipe material.

The supply area characteristics required were:

- Supply area population;
- AADD of the supply area;
- Supply area topography;
- Supply area size based on the size of the AADD;
- Assumed initial CWS diurnal water demand pattern based on the generic PHFs derived by GLS Consulting, who performed the network modelling for the study area.

The WDN and supply area characteristics were obtained from the WDS model of the KaNyamazane study site, which was made available by GLS Consulting. KaNyamazane is one of the urban areas located in the city of Mbombela municipal area, situated in the Mpumalanga province of South Africa. KaNyamazane was identified as having formal IWS implemented.

Once this information was collected for the study site, the calculations to derive the hypothetical diurnal water demand pattern for IWS were conducted according to the assumed three phase WDS filling structure. The calculations were performed for one supply period per day, two supply periods per day and three supply periods per day.

4.4.2 CWS water demand scenario

In step 1 of the Excel based calculations as indicated in Figure 4.2, the assumed CWS diurnal water demand pattern for the study site had to be determined. In order to determine the CWS demand pattern for the KaNyamazane, Toad Street supply area, the AADD and the PFs related to the supply area characteristics had to be established. The AADD was obtained from the node data table in the KaNyamazane model as presented in Figure 4.4, where the

Theoretical AADD for each node in the supply area (Th AADD column) were summed to obtain the total actual water demand. Similarly, the output for the nodes in the supply area (output column) were summed to determine the total design water demand (peak water demand) for the supply area.

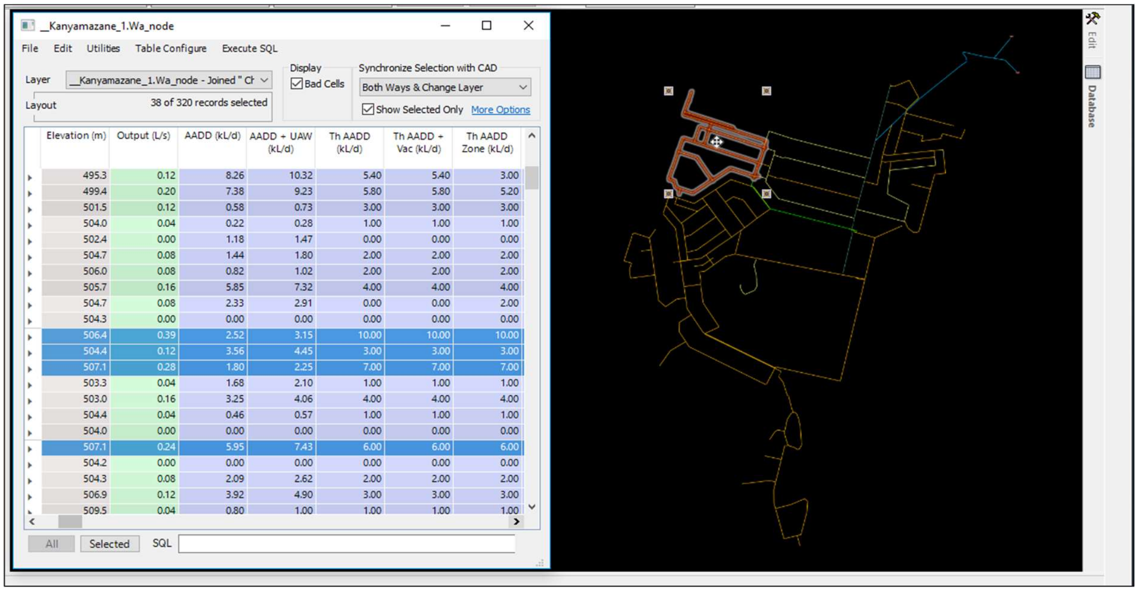


Figure 4.4 WDN model of the KaNyamazane Toad Street supply area.

From the determined AADD, the supply area size was derived using Table 4-1 which is an adapted table from the study conducted by Vorster et al. (1995) for residential land use supply areas. Furthermore, the base flows due to leakage losses were assumed to be a fixed percentage of 30% of the AADD in the calculations.

Table 4-1 Residential area size based on AADD (adapted from Vorster et al., 1995)

Predominant land use	Area size	AADD (kl/d)
Low cost housing	Small	<1000
		1000 - 5000
	Medium	5000 - 10000
		10000 - 15000
	Large	15000 - 20000
		>20000
Residential	Small	<1000
		1000 - 5000
	Medium	5000 - 10000
		10000 - 15000
	Large	15000 - 20000
		>20000

To determine the PFs in deriving the CWS water demand pattern, the PHFs derived by Loubser et al., (2018) were used. Loubser et al., (2018) derived diurnal demand patterns specifically for low cost housing residential areas using actual logged flow data. KaNyamazane is classified as a low cost housing area, therefore the low cost housing PHFs and the related diurnal water demand pattern were used in the calculations. Refer to Table 4-2 and Figure 4.5 to view the PHFs and the related diurnal demand patterns.

Table 4-2 Generic peak hour factors as derived by GLS Consulting and Loubser et al., (2018).

Hour	GLS Consulting				Loubser et al. (2018)
	Residential large	Residential medium	Residential small	Low Cost Housing	Low Cost Housing
	PHF	PHF	PHF	PHF	PHF
1	0.5	0.3	0.1	0.6	0.45
2	0.5	0.3	0.1	0.6	0.46
3	0.5	0.3	0.2	0.6	0.49
4	0.5	0.4	0.3	0.7	0.51
5	0.6	0.5	0.5	0.75	0.61
6	0.9	0.8	1	0.8	0.84
7	1.2	1.2	1.4	0.95	1.20
8	1.4	1.6	1.9	1.2	1.55
9	1.6	1.75	2	1.3	1.71
10	1.6	1.8	2.1	1.5	1.70
11	1.4	1.6	1.9	1.6	1.60
12	1.3	1.4	1.6	1.6	1.49
13	1.2	1.25	1.2	1.6	1.38
14	1.2	1.2	1	1.55	1.28
15	1.2	1.3	1.2	1.5	1.20
16	1.3	1.4	1.6	1.5	1.15
17	1.4	1.5	1.8	1.4	1.11
18	1.4	1.4	1.6	0.85	1.09
19	1.1	1.2	1	0.7	1.00
20	0.9	0.9	0.7	0.6	0.84
21	0.7	0.7	0.4	0.6	0.71
22	0.6	0.5	0.2	0.5	0.61
23	0.5	0.4	0.1	0.5	0.51
24	0.5	0.3	0.1	0.5	0.42
Total	24.00	24.00	24.00	24.00	24.00

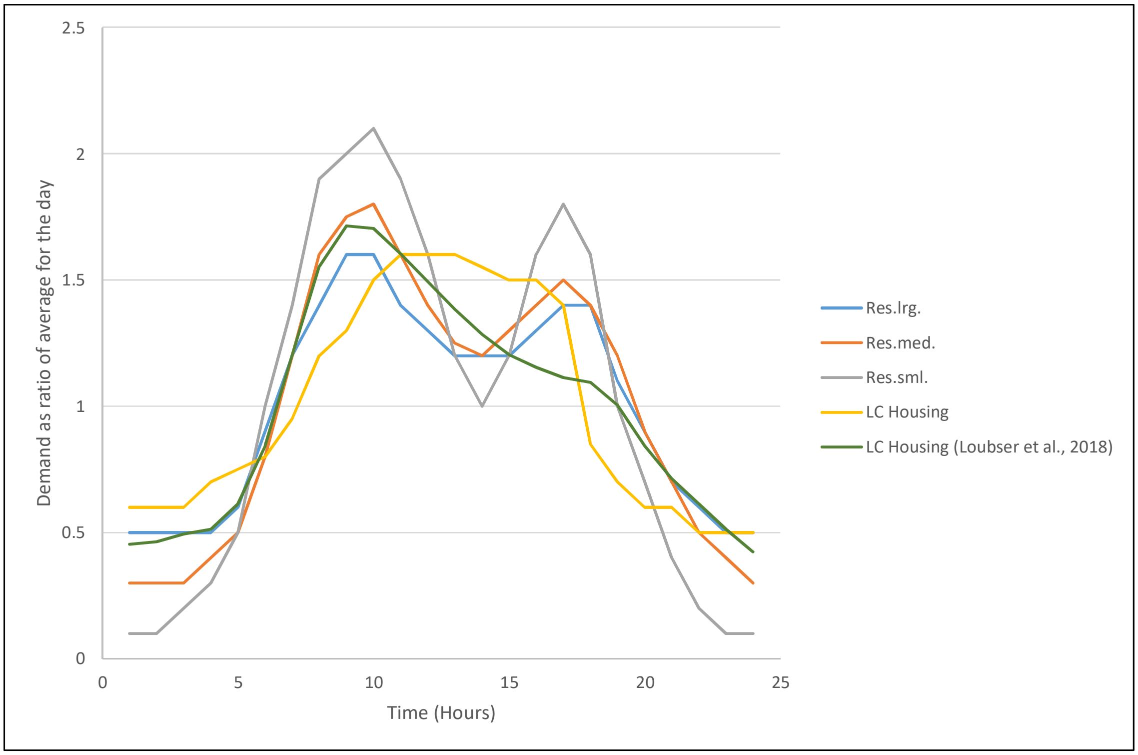


Figure 4.5 Assumed CWS diurnal demand patterns derived by GLS Consulting and Loubser et al., (2018).

The peak flows per hour were then calculated according to Equation (4-1). The estimated volume of water demand per hour was calculated using the Riemann sum, which estimates the area under the graph curve according to Equation (4-2). Refer to Figure A.1 in Appendix A to view an example of the Excel worksheet calculations for this step.

$$Area = \sum_{K=1}^{n} f(x_K)\Delta x \ for \ closed \ interval \ [a, b] \quad (4\text{-}2)$$

where:

$f(x_K)$ = graph curve function.

$\Delta x = \frac{b-a}{n}$.

n = total number of subintervals.

K = subinterval number.

a = lower interval limit.

b = upper interval limit.

4.4.3 IWS water demand scenario

In this section, the theory related to WDS characteristics during IWS was applied to the Excel calculation process, in order to determine what happens to the AADD, PFs, peak flows, and the volume of water consumption when the water is supplied intermittently. The calculations for this step of the investigation were based on Equation (4-1), as well as the effects of supply duration on the components of Equation (4-1) as reported by De Marchis et al. (2011), Fan et al. (2014), Andey and Kelkar (2009), and Abu-Madi & Trifunovic (2013).

Abu-Madi and Trifunovic (2013) stated that supply duration has a significant effect on the peak factors, according to the relationship described by:

$$PF = 24/t, \tag{4-3}$$

where:

PF = dimensionless peak factor or demand multiplier.

t = water supply duration (hours per day).

Additionally, from the study conducted by Andey and Kelkar (2009), it was found that there is no significant change in the water consumption during IWS conditions compared to the CWS consumption as long as the water demand was satisfied under IWS. In addition, De Marchis et al. (2011) found that the domestic water demand or domestic water consumption behaviour only started to be affected by the reduction in supply duration after the supply duration became less than 20 hours per day. Once the supply duration becomes less than 20 hours per day, the water demand would not be satisfied for some of the consumers that are disadvantaged in terms of position in the network. Based on the outcomes of these studies, it was assumed that when the supply duration is greater than 20 hours per day the consumer water demand for the IWS scenario remains the same as under CWS. This assumption was applied for the calculations related to the case of one supply period per day.

The lowest supply duration considered for a supply period in the calculations was a supply duration of 1.5 hours. According to Fan et al. (2014), a supply duration of 1.5 hours should be the minimum allowable supply time to the consumers in order to avoid consumer health issues and to supply the minimum water needed per person. For the AADD related to supply durations less than 20 hours per day, the following series of equations were developed for the relationships between AADD, peak demand and the peak factors during IWS:

$$AADD_{IWS} = AADD_{CWS} \times SDDRF, \quad (4\text{-}4)$$

$$PF_{IWS} = PF_{CWS} \times Demand\ multiplier, \quad (4\text{-}5)$$

$$SDDRF = {}^{t}/_{24} + 0.25\ for\ 1 \leq t \leq 17, \quad (4\text{-}6)$$

$$SDDRF = 1\ for\ 18 \leq t \leq 24, \quad (4\text{-}7)$$

$$Q_{max,IWS} = AADD_{IWS} \times PF_{IWS}, \quad (4\text{-}8)$$

where:

$AADD_{IWS}$ = average annual daily demand for IWS conditions (${}^{m^3}/_{s}$, or any other unit of flow rate).

$AADD_{CWS}$ = average annual daily demand for CWS conditions (${}^{m^3}/_{s}$, or any other unit of flow rate).

$SDDRF$ = supply duration demand reduction factor (%).

PF_{IWS} = dimensionless peak factor during IWS conditions.

PF_{CWS} = dimensionless peak factor during CWS conditions.

t = water supply duration (hours per day).

$Demand\ multiplier$ = ${}^{24}/_{t}$

$Q_{max,IWS}$ = maximum demand or peak demand for the IWS scenario (${}^{m^3}/_{s}$, or any other unit of flow rate).

By combining the findings of Abu-Madi and Trifunovic (2013) and the assumed SDDRF in Equation (4-6) and Equation (4-7), Table 4-3 was compiled for use in the calculations.

Table 4-3 Compiled demand multiplier table

Supply Duration (t)	Demand multiplier (PHF=24/t)	Supply duration demand reduction factor (%)
24	1.00	100
23	1.04	100
22	1.09	100
21	1.14	100
20	1.20	100
19	1.26	100
18	1.33	100
17	1.41	96
16	1.50	92
15	1.60	88
14	1.71	83
13	1.85	79
12	2.00	75
11	2.18	71
10	2.40	67
9	2.67	63
8	3.00	58
7	3.43	54
6	4.00	50
5	4.80	46
4	6.00	42
3	8.00	38
2	12.00	33
1	24.00	29

4.4.4 Phase 1 - Bulk supply pipe filling

To investigate and mathematically model the peak flows during the filling of the bulk supply pipeline from the storage reservoir to the water reticulation network, two approaches were adopted. These were compared to determine which approach was a more accurate representation of the real-world scenario. The first approach was by using Equation (4-9) and Equation (4-10) derived by Liou and Hunt (1996) to model the length of a water column with time when an empty pipe is filling.

$$\frac{dV}{dt} = \frac{\frac{g}{L}\cdot\left(H_R - K\frac{V^2}{2g}\right) + g\sin \quad \frac{fV^2}{2D}}{1+\frac{Lo}{L}}, \tag{4-9}$$

$$L = L_{min} + \int_0^t V\,\delta t, \tag{4-10}$$

where:

V = velocity of the water column ($^m/_s$).

L = length of the water column in the pipeline being filled (m).

L_{min} = length of the water colume at t=0, (m).

θ = downward angle of the pipe from the horizontal (degrees)

H_R = the available head at the reservoir (m).

K = entrance and valve losses coefficient

g = gravitational acceleration ($^m/_{s^2}$).

D = pipe diameter (m).

t = time taken for the pipeline to fill (seconds)

f = Darcy-Weisbach friction factor

Lo = local flow acceleration of the flow approaching the pipe inlet from the reservoir (m).

By multiplying the velocities obtained from (4-9) with the pipe cross sectional area according to the continuity equation (4-11), the peak flows during the pipe filling process were calculated.

$$Q = V \times A, \tag{4-11}$$

where:

$Q =$ flow rate ($^{m^3}/_s$).

$V =$ velocity ($^m/_s$).

$A =$ pipe cross sectional area (m^2).

In order to apply Equation (4-9) and Equation (4-10), the reservoir and bulk supply pipe characteristics such as the water level in the reservoir, the angle of the connection of the bulk supply pipe with the reservoir, and the position of the flow control valve from the reservoir, had to be known. Refer to Figure A.2 in Appendix A for an example of the Excel calculation.

The second approach was based on the pipe filling volume, where the volume of the bulk supply pipeline was determined using the known pipe diameter and pipe length. The Riemann sum equation was then used to calculate the 15 minute peak flow value related to the volume.

The determined 15 minute peak flow value was then used in Equation (4-8) to determine the 15 minute IWS PF for the phase 1 calculation, which was then entered into a plotter table for the phase 1 time interval of the IWS diurnal water demand pattern. The plotter table was created to compile all the peak flow values derived for every 15 minute interval, in order to calculate the 15 minute IWS peak factors required to plot the IWS diurnal water demand pattern. Refer to Appendix A, Table A.1 to view an example of the Excel calculation for phase 1. Table A.5 contains an extract of the plotter table.

4.4.5 Phase 2 - Network filling

To determine the peak flow values to populate the plotter table for the phase 2 calculations, the equations and theory related to the pressure dependent analysis of WDNs were used. Since WDSs subjected to IWS are pressure head driven, Equation (4-12) was used to model the network filling process during IWS:

$$Q_i = Q_{max} \sqrt{\frac{H_i - H_{min}}{H_{max} - H_{min}}} \qquad (4\text{-}12)$$

where:

Q_i = the demand or flow at a node i, (m^3/s, or any other unit of flow rate).

Q_{max} = the maximum or desired demand or flow at a node i, (m^3/s, or any other unit of flow rate).

H_i = the pressure head at a node i, (m).

H_{max} = the maximum or desired pressure head at a node i, (m).

H_{min} = the minimum required pressure head at a node i, (m).

Equation (4-12) was developed by Wagner et al. (1988) to describe the head-flow relationship at a demand node during the WDN filling process. Using Equation (4-12) the available flow at the demand nodes could be calculated as the pressure heads at the nodes change during the filling process. The demand at the node was considered to be satisfied when the pressure head at the demand node was greater than or equal to the desired pressure head. When the pressure head at the demand node (H_i) was less than the desired pressure head, but more than the minimum required pressure head, the demand at the node was considered partially satisfied. When the pressure head at the node was less than the minimum required pressure head, the demand at the node was considered to be zero. Refer to Table 4-4 for a summary of the different flow conditions during intermittent supply.

Table 4-4 Pressure dependent flow conditions during intermittent supply.

Pressure head condition	Demand at the node	Flow condition at the node
$H_i \geq H_{max}$	$Q_i = Q_{max}$	Maximum flow
$H_{min} < H_i < H_{max}$	$0 < Q_i < Q_{max}$	Partial flow
$H_i \leq H_{min}$	$Q_i = 0$	No flow

To simplify the calculations the first node of the WDN, the node where the pipe from the reservoir connects to the WDN was chosen as representative of the entire network. The total demand of the whole network was lumped on this representative node, which is illustrated as the focus node in Figure 4.3. When the nodal demand was satisfied (Q_i=Q_{max}), the whole network was assumed to be full.

For the calculation of Equation (4-12), the minimum allowable pressure head (H_{min}) at the demand node was assumed to be a value between 5 m and 8 m. The maximum pressure head or the desired pressure head (H_{max}) at the node was assumed to be a value equal to or greater than 24 m, but lower than 60 m according to the new Red book guidelines. In order to determine the head available (H_i) for the focus node at time zero, the Bernoulli Equation (4-13) was used. Before the flow begins at the node (at time zero) the pressure head as well as the velocity head would be zero because there is no flow. Therefore, the available head at the focus node in this instance would be equal to the elevation head of the node.

$$\frac{P}{\rho g} + \frac{v^2}{2g} + z = constant, \qquad (4\text{-}13)$$

where:

P = pressure ($^N/_{m^2}$).

g = gravitational acceleration ($^m/_{s^2}$).

ρ = liquid density ($^{kg}/_{m^3}$).

v = velocity ($^m/_s$).

z = elevation above datum (m).

By assuming there is already partial or full flow at the focus node at time zero, the Bernoulli Equation (4-13) could be used to determine the head at the node. This was achieved by determining the total head available at the storage reservoir, and then determining the friction losses and any minor energy losses up until the position of the node.

Furthermore, a relationship of the change in the pressure head with time at the focus node also had to be determined, in order to establish the rate at which the desired demand was satisfied. This is assumed to be the rate at which the initial head at the demand node approaches the desired head, and the rate at which the flows at the node reach the required flow rate. To determine this pressure head-time relationship, the results from the studies conducted by De Marchis et al. (2011) and Hou et al. (2014) were used. These two studies defined the pressure head-time relationship during the network filling process according to Figure 4.6.

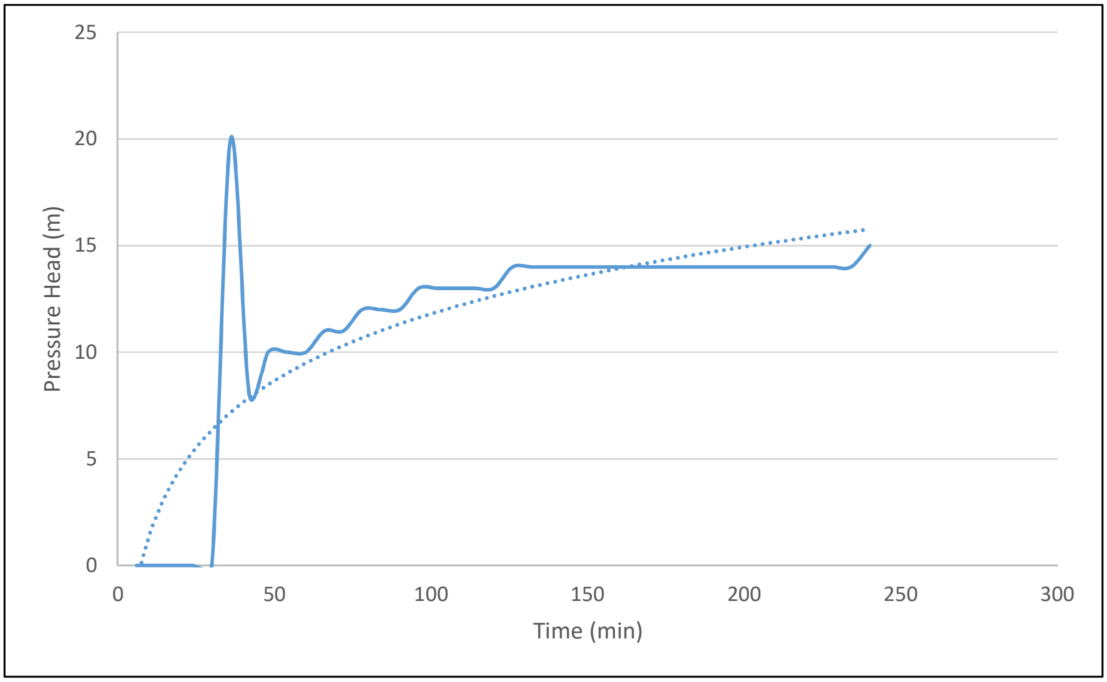


Figure 4.6 Pressure head-time relationship (De Marchis et al., 2011; Hou et al., 2014).

Combining the results of these studies with Equation (4-12), the progression of the flows (Q_i) at the node with time was determined. When the flow value at the focus node (Q_i) became the same as the required flow rate (Q_{max}) it was assumed the whole network was full. This process of the WDN filling was assumed to occur over the network filling time (t_{max}).

The known WDN characteristics were used to determine the network filling time at the maximum allowable flow rate, by dividing the total network volume with the maximum allowable flow rate for the determined average network pipe diameter. The assumption was that the WDN would fill at the maximum flow rate the pipe could handle. However, this assumption was adjusted for the effects of the user water demand during the network filling. As the network is filling, users start abstracting water from the system as soon as the available pressure head at the outlets is high enough, which increases the network filling time. During IWS, users tend to leave their taps open in order to collect as much water as possible during the supply period. Therefore, the network filling time was recalculated by determining the user water consumption during the filling process. The user peak water consumption was assumed to be equal to the volume of water that would be consumed during CWS conditions over the network filling time interval of phase 2. The user peak water consumption rate was then subtracted from the maximum allowable flow rate, to obtain the available flow rate for network filling (Q_{max}). The network filling time (t_{max}) at the available flow rate was then determined by dividing the total network filling volume with the available flow rate for network filling. The calculated available flow rate for network filling, and the network filling time at the available flow rate were used as input for Equation (4-12), in order to determine the 15 minute peak flow values to populate the plotter table. Table A.2 and Figure A.3 in Appendix A contain examples of the Excel calculations related to phase 2.

4.4.6 Phase 3 - Network full

For the phase 3 calculations, the network was assumed to be full. Therefore a further assumption was made, that the user water consumption behaviour during this phase would be similar to the CWS case. This assumption implied that the shape of the IWS diurnal water demand pattern after the network filling process is complete, would be similar to the CWS diurnal water demand pattern. Even though the shape of the patterns were assumed to be similar, the magnitude of the peak flow values and the peak factors related to IWS, however, were different. The peak flow values for IWS were higher than for the CWS scenario, due to the shorter supply duration. The IWS peak flows and the related peak factors for phase 3 were calculated based on Equation (4-8) and Equation (4-5) respectively.

The IWS AADD used for the phase 3 calculations takes into consideration the user peak water consumption during the network filling process in phase 2. If it was assumed that the users start consuming water after the network filling process is complete, the IWS AADD value would be the full IWS AADD as determined using Equation (4-4) and Table 4-3. The remaining IWS AADD after making provision for the phase 2 user water consumption, was multiplied by the IWS PHFs determined for the time interval that corresponds to the start and end time of phase 3, in order to obtain the IWS peak hour demand values according to Equation (4-8). In determining the IWS PHFs according to Equation (4-5) the demand multiplier for each supply period was obtained from Table 4-3 according to the total supply duration per day. The CWS PHFs from Table 4-2, that are within the phase 3 time interval, were subsequently chosen and multiplied by the demand multiplier to determine the IWS PHFs. The result of using the CWS PHFs in the determination of the IWS PHFs, was that the shape of the IWS diurnal water demand pattern over the phase 3 time interval would be similar to the CWS diurnal water demand pattern for the same time interval. This concept is illustrated in Figure 4.7, where it is illustrated that the shape of the IWS pattern depends on the position and duration of the phase 3 supply period. In Figure 4.7, there is a typical low cost housing CWS pattern overlaid by two IWS supply periods of 5 hours each (shaded area). The portion of the low cost housing CWS pattern that is within the shaded area represents the general shape of the IWS diurnal demand pattern during the phase 3 time interval.

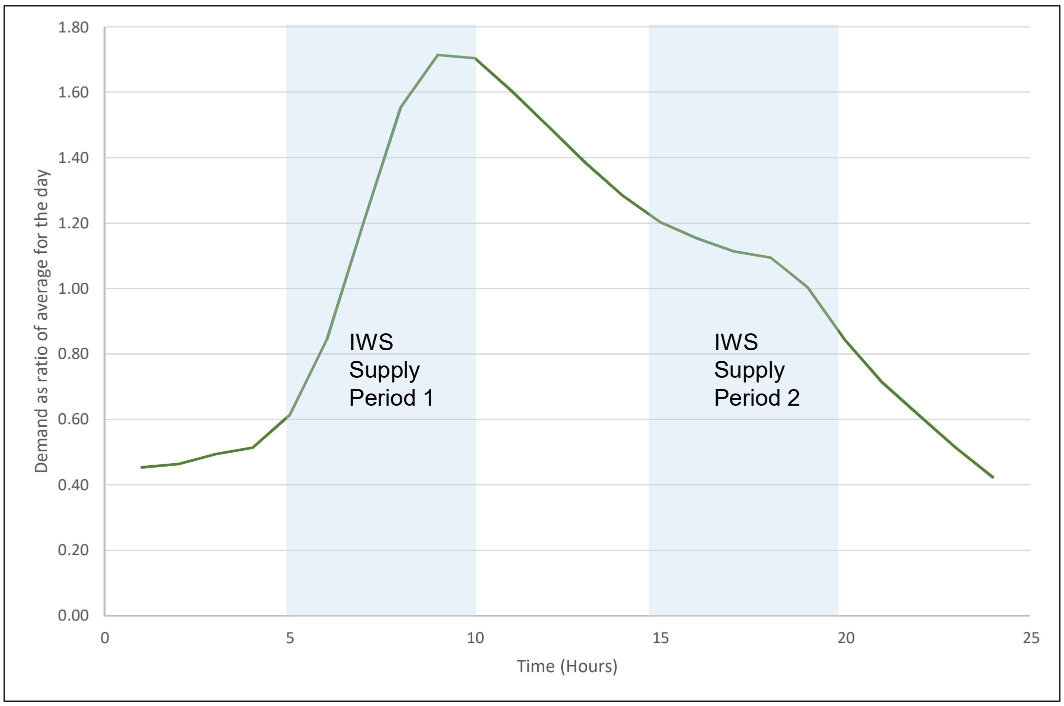


Figure 4.7 Concept of the assumed IWS demand pattern shape during phase 3.

The determined IWS peak hour flow values were then checked to ensure that the values were below the WDN hydraulic capacity demand rate, which is the maximum flow rate that the WDS can supply. It was assumed that the WDS could not supply water at flow rates beyond the designed maximum demand without further structural modifications to the WDS. Therefore, any IWS peak demand values calculated that were beyond this maximum demand value were limited to be equal to the maximum design demand for the WDN. Furthermore, it was assumed that the WDS is fully drained during each supply period, which means there are no base flows during the non-supply periods, as no water remains in the pipeline after the supply period. Therefore, the base flows related to the CWS PHFs were redistributed throughout the supply area. Then, by interpolating the determined IWS peak hour flows the IWS peak 15 minute flows were determined to populate the plotter table. Refer to Table A.3 and Figure A.4 in Appendix A for an example of the phase 3 Excel worksheets.

5 Model validation

5.1 Introduction

This chapter describes the steps followed to validate the hypothetical model, by comparing and calibrating the model for a better representation of the study site data. The limitations and challenges encountered during the model validation process are also discussed in this chapter.

5.2 Data collection

To collect water consumption data for the model validation process three Point Orange 3G water loggers were installed at three different site locations. The loggers were installed on one water meter with a size of 250 mm in Matsulu, and on two water meters with sizes of 80 mm and 150 mm at two locations in KaNyamazane. Both Matsulu and KaNyamazane are towns situated in the Mbombela municipality, in the Mpumalanga province in South Africa, where planned or 'formal' IWS is implemented as a strategy of supplying water to the people. These two towns consist of both formal and informal settlement areas where the levels of water and sanitation infrastructure vary (Bender & Gibson, 2010). The specific site names where the loggers were installed and the sizes of the relevant meters at those sites, are summarised in Table 5-1.

Table 5-1 Water meter sizes and site locations.

Site Number	Town	Site Name	Meter Size (mm)	Pulse value (m^3/pulse)
1	Matsulu	Vodacom	250	1
2	KaNyamazane	Toad Street	80	0.1
3	KaNyamazane	Thembeka	150	1

The water meters were positioned to measure the water consumption of distinct and isolated supply zones where each site is a district metered area (DMA). The flows were logged for the period starting 19/07/2019 and ending 26/08/2019. The flow data recorded by the loggers was directly uploaded onto the Metasphere online server. Metasphere developed an online-based remote monitoring dashboard that is directly connected to the loggers, which convey the measured flow data to the website in real time. Through the Metasphere website the recorded data was readily available for collection, and could be accessed and downloaded in Excel format.

5.3 Data sorting and filtering

The water meter data from the three sites was collected from the Metasphere online-based remote monitoring dashboard. Based on the observed flow profiles, it was decided that only the data from Site 2, which is the KaNyamazane Toad Street area, was used for the data analysis phase. The data from Site 3, KaNyamazane Thembeka Street was excluded, because the observed flow pattern was typical of one where leaks were so high, that the system probably never pressurised.

Furthermore, for Site 3 it was also observed that between 19/07/2019 and 26/08/2019 there was still flow during the non-supply periods for some of the days, which could be attributed to the pipeline not being fully shut off. In the current study one of the assumptions was that the WDN is fully drained during non-supply periods, therefore there should be no flow observed during the non-supply time. Compared to the Site 1 and Site 3 data, the data from Site 2 had the best fit to the IWS flow data patterns observed by Mckenzie et al. (2014), as presented in Figure 2.6. Furthermore, no flows were measured during non-supply periods between 22/7/2019 and 25/8/2019 for Site 2. Therefore, the 35 day data set chosen for the analysis and model validation was obtained from the Site 2 data. Refer to Figure 5.1 for an example of the typical daily flow readings at Site 2. Refer also to Appendix C for the daily flow readings for Site 1, Site 2, and Site 3.

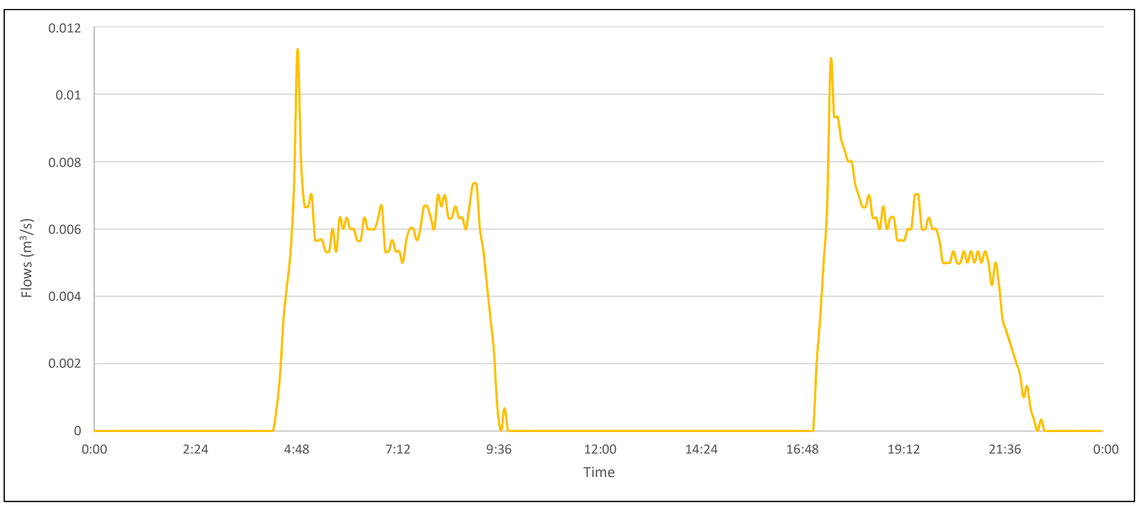


Figure 5.1 Typical daily flow readings from Site 2

The data extracted from the Metasphere online server was downloaded in Excel format, therefore no conversion of the data format was needed. In the Excel worksheet, the data related to the reservoir level was filtered out leaving only the pulse readings of the metered flow rates. The loggers recorded the flows as the total number of pulses in every 5 minute interval, and each pulse represents a volume of one cubic meter, as indicated in Table 5-1. Thus, by using the filtered and sorted data for the KaNyamazane Toad Street site for the record period 22/7/2019 to 25/8/2019, the diurnal water demand patterns were derived.

5.4 Data analysis

In the derivation of the diurnal water demand patterns using actual water consumption readings from the KaNyamanzane, Toad Street site the diurnal flow volume variations recorded by the water meter in 5 minute intervals were converted to flow rates using (5-1):

$$Q_t = {}^{Volume}/_{\delta t} \tag{5-1}$$

where:

Q_t = peak flow rate averaged over time interval δt $({}^{m^3}/_{s})$.

δt = time interval over which the flow rates are averaged (seconds).

$Volume$ = number of pulses in 5 minutes multiplied by the pulse volume $(1\ m^3)$

The flow rates calculated were the peak flow rates averaged over 5 minutes. The peak flow rates over the 5 minute time interval, a 15 minute time interval and an hour time interval for each day in the total record were determined. This was done to determine the most suitable time interval (δt) for the derivation of the peak factors. The magnitude of the peak factors depend on the length of the time interval used in the calculations, as explained by Scheepers (2012) in Section 2.3.3. If the chosen time interval is too long, the determined peak factors would not be a suitable representation of the peak water demand for that particular day. Therefore the diurnal water demand patterns for the 5 minute time interval, the 15 minute time interval and the 1 hour time interval for each day in the total record period were determined using the following steps:

- Calculate a set of 288 values containing the average peak flow rate of each 5 minute time interval in a 24 hour day for every day in the record period. Then repeat the same procedure for the 15 minute time interval by calculating 96 values containing the average peak flow rates of each 15 minute time interval in a day and also for the 1 hour time interval by determining a set of 24 values containing the average peak flow rates of each 1 hour time interval in a day.

- Determine the average demand per day (ADD) of the entire record period.
- Determine the peak factors (PF) for each 5 minute, 15 minute and 1 hour time interval period for each day in the record period.
- Plot the dimensionless PFs determined (y-axis) for each of the different time interval periods per day (x-axis) for the entire record period.

The 288 values containing the average peak flow rates of each 5 minute time interval were determined using Equation (5-1) as described above. The same applies to the 96 values related to the 15 minute time interval and the 24 values related to the 1 hour time interval. The ADD was determined by calculating the total volume of water consumed in the entire record period and dividing that value with the total number of days in the record period.

To determine the PFs for each day Equation (4-1) was used. In South Africa the PFs are generally calculated using the AADD as the average demand of the extended time period. Since the flow data for a whole year was not available for the study, the ADD for the record period of 35 days was used instead. The dimensionless PFs were then plotted against time for each day of the record period, in order to obtain the diurnal demand patterns. Therefore, each day in the record period had three types of diurnal demand patterns based on the three types of PFs derived.

5.5 Model comparison and calibration

In order to investigate the validity and accuracy of the proposed model, the diurnal water demand patterns derived using actual water consumption data from the study site were compared to the hypothetical IWS diurnal water demand pattern. The hypothetical IWS diurnal water demand patterns were determined using the same WDS characteristics and supply characteristics as the area where the actual data was recorded, which in this study is the KaNyamazane, Toad Street site (Site 2). Furthermore, the hypothetical IWS diurnal water demand pattern were also determined to be for the same supply durations and number of supply periods per day as the actual derived diurnal water demand pattern for a particular day. For example, if the demand pattern derived from actual data for the date 22/7/2019 had two supply periods per day where the first supply period has a supply duration of 6 hours from 4:00 until 10:00 and the second supply period has a supply duration of 5 hours from 17:00 until 22:00, the IWS demand pattern derived using the Excel calculations would be determined with the same supply period and supply duration characteristics over the same time intervals for that particular day.

The accuracy of the model was determined based on how precise the hypothetical IWS demand pattern estimates the shape of the actual water demand pattern. The Kolmogorov-Smirnov goodness of fit test (K-S test) was conducted to determine the similarity between the two patterns. The two sample K-S test is a nonparametric hypothesis test, which determines the probability that a certain univariate data set is derived from the same parent population as a second data set. The test is based on a K-S statistic value (D in Equation (5-2)), that quantifies the distance between the empirical distribution function (EDF) or cumulative distribution function (CDF) of a univariate dataset, and the CDF of a second dataset. The null hypothesis for this test is that both data sets come from a population with the same distribution. If the K-S statistic value (D) is greater than the critical K-S value (d_α in Equation (5-3)), the EDF or CDF of the univariate dataset is considered not to come from the CDF of the second dataset, and the null hypothesis is rejected at a certain significance level (α). This means there is a significant difference between the distributions. However, if the K-S statistic value (D) is less than the critical K-S value (d_α), there is no significant difference between the distributions (Kolmogorov, 1933; Smirnov, 1948; Rojas-Lima et al., 2019). Refer also to Table 5-2 and Figure 5.2.

$$D_{KS} = max|F_X(x) - S_N(x)|, \tag{5-2}$$

$$d_\alpha = c(\alpha)\sqrt{\frac{m+n}{mn}}, \tag{5-3}$$

where:

D_{KS} = K-S statistic value.

$F_X(x)$ = theoretical cumulative distribution function.

$S_N(x)$ = sample cumulative distribution function.

d_α = K-S critical value of the maximum absolute difference between the sample and theoretical cumulative distribution functions.

$c(\alpha)$ = inverse of the Kolmogorov distribution at significance level α.

m = sample size for the first sample.

n = sample size for the second sample.

α = level of significance.

Table 5-2 Critical K-S values, d_α for a sample size larger than 35 (adapted from Rojas-Lima et al., 2019).

Sample size (N)	Level of significance (α)						
	0.20	0.15	0.10	0.05	0.01	0.005	0.001
Over 35	$\frac{1.07}{\sqrt{N}}$	$\frac{1.14}{\sqrt{N}}$	$\frac{1.22}{\sqrt{N}}$	$\frac{1.36}{\sqrt{N}}$	$\frac{1.63}{\sqrt{N}}$	$\frac{1.73}{\sqrt{N}}$	$\frac{1.95}{\sqrt{N}}$

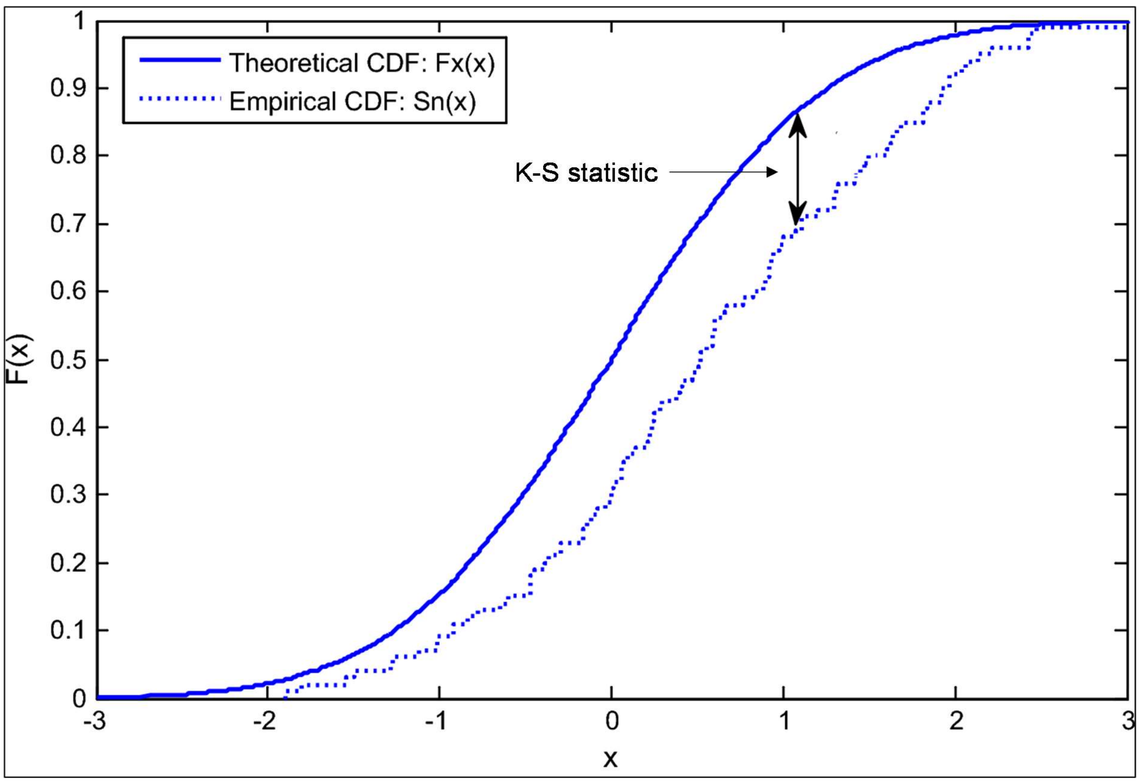


Figure 5.2 K-S statistic for two cumulative distribution functions (Rojas-Lima et al., 2019).

Cumulative distribution functions (CDF) for the model demand pattern and the actual demand pattern were derived using the PF values as the data sets. The K-S statistic value (D) was calculated as the maximum distance between the two CDFs. The smaller the K-S statistic value (D), the more similar the patterns are considered to be, provided that the K-S statistic value (D) is less than the critical K-S value (d_α). Based on the comparison results, the model was calibrated by re-evaluating the assumptions made in the Excel calculations. Where there were major differences and deviations, each assumption was revisited in order to identify the cause of the differences. The model assumptions were subsequently adapted to match the actual diurnal water demand pattern.

5.6 Limitations and challenges

Identifying an applicable study site and collecting IWS data that is appropriate for the comparison section of the study were the main challenges during the model validation process. The ideal study site had to meet the following requirements:

- It had to be a supply area where IWS is formally implemented and where the supply zones are clearly defined;
- The supply area for which the data was obtained, preferably had to have an EPANET or WADISO model of the WDN available, in order to determine the system and supply characteristics;
- The study site should ideally have logged IWS flow data for a record period of at least 12 months as has been done for the water demand estimation studies discussed in the literature review for the determination of the AADD.

There are not many water supply zones in South Africa that satisfy these conditions, but the Matsulu and KaNyamazane areas were identified to meet the first two requirements. The two identified areas did not have readily available water supply data for the IWS conditions implemented in these areas. Therefore, loggers had to be installed on the water meters for the purposes of collecting relevant water supply data for this study. Due to the time sensitivity of the study, the data collected was for a short record period of 38 days only; it was not possible to record the water consumption for a period of 12 months or longer. As a result, the ADD instead of the AADD was used in the calculations for the peak factors for the derivation of the diurnal water demand patterns. Furthermore, the remote location of the study site meant that the investigation of any anomalies was not viable.

6 Results and discussion of results

6.1 Parameters investigation

6.1.1 Parameter interrelationships and effects

In the derivation of the IWS diurnal water demand pattern model a parameters investigation was conducted using a literature based approach. The influence parameters were identified and correlated to water demand and WDS performance using the findings of past research. The findings of the past research papers and the related citations were used to compile Table 6-1, which would be used to consolidate and organise the information collected for the parameters analysis.

Table 6-1 consists of 20 identified driving parameters that influence domestic water demand. These are presented in five categories which are geographic, climatic, socio-economic, structural and operational. From Table 6-1, the description of the nature of the interrelationships between the parameters is presented. In addition, the correlations and the nature of the correlations of these parameters with the domestic water demand and the WDS performance are also presented. The blank spaces in the table indicate correlation gaps in the research related to those particular parameters. However, for some of the unknown or undefined parameters suggestions were made of the interrelationships and effects of these parameters on IWS systems. These are based on empirical and logical argument, and hydraulic principles that emanate from the literature sources reviewed in the table. All the suggestions made are indicated in blue italic font in the table and can be considered for further research.

Table 6-1 Parameter correlations table.

Parameter Categories	Driving Parameters	Parameter Interrelationships and Effects: Directly affected by the following parameters:	Nature of the relationship	How is it linked to water demand?	How is it linked to system performance?	CWS, IWS or Both?	Reference
Geographic	Location: Coastal/Inland	*Climate*		Inland stands generally use more water than coastal stands for the same stand area and stand value (there are exceptions, for example it has been found that stands in Cape Town have higher water consumption than some inland stands and this is due mainly to outdoor demand and climatic conditions). *The different climates and rainfall seasons could also be a reason for the difference in the water usage between coastal and inland areas.*	*No available research defining the link.*	Both	(Jacobs et al., 2004); (Van Zyl et al., 2007); (Van Zyl, 2006)
Geographic	Topography			The advantaged users in terms of position and topography tend to have high water consumption in IWS conditions as these users will experience adequate pressure and flow rates to fill their tanks.	Topography affects the pressures and the pressure distribution of a network. Elevation differences are one of the main influencing factors for supply equity. Not only does the area topography affect system performance but also the network configuration and the location of the supply source.	Both	(De Marchis et al., 2011); (A. Ilaya-Ayza et al., 2017)
Geographic	Supply Area Size			As a function of population density (number of people per unit area). The more people in the supply area the more the water consumption. The supply area size also affects the peak factors.	The supply area size is one of the main influencing factors for supply equity. The larger the supply area the longer it takes for supply to reach the users at the far ends of the network resulting in unequal supply when the supply duration is short. The users at the network ends will get less time to draw from the water supply resulting in less quantity of water supplied to them before the end of the supply duration.	Both	(Van Zyl et al., 2007); (U.S. National Academy of Science et al., 1999); (Kumpel & Nelson, 2016); (A. Ilaya-Ayza et al., 2017)
Climatic	Climate: MAE, MAP, Min. and Max. Temperature	Location	The climate (rainfall, evaporation and temperature) varies with the geographical location.	The influence of climate on the water consumption is far less in low income areas than high income areas since these areas have a much smaller outdoor water demand. Referenced as one of the most influential factors. Decrease in water consumption after a prolonged rain period and a decrease in the evaporation rate can also lead to a decrease in water consumption. With an increase in temperature there's an increase in water consumption (prolonged high temperatures) and the evaporation rate also increases. Seasonal variation in daily demand may vary from +-10% to 30% of the average daily demand for the year.	*No available research defining the link.*	Both	(Van Zyl et al., 2007); (U.S. National Academy of Science et al., 1999); (Andey & Kelkar, 2009); (Van Zyl, 2006)
Operational	Network Filling Time	Topography, Private Tanks, Supply Area Size.	The high consumption of positionally and topographically advantaged users who are able to collect water as soon as supply begins protracts the time needed to fill the network completely and the larger the tanks the longer it will take to fill them. The larger the distribution network and area that has to be supplied the longer it will take to fill the water distribution network.	The longer it takes for the water to fill the network the less water is collected by the users at the tail end of the network as they have less time to collect water during the supply hours (sometimes even when the network is full the pressure levels are too low to allow supply and it takes time for the pressure levels to become adequate). The high consumption of positionally and topographically advantaged users who are able to collect water as soon as supply begins protracts the time needed to fill the network completely.	*The pressures and the pressure distribution within the network improve towards the end of the network filling time and as a result the supply equity also improves with time. Therefore the longer the network filling time the longer it will take for the pressures in the system to be adequate and for the pressure distribution to be uniform.*	IWS	(Vairavamoorthy et al., 2001); (De Marchis et al., 2011)
Operational	Supply Duration	Supply Area Size, Network Filling Time, Available Pressure Head at the Outlet, Area Population Size, Network Hydraulic Capacity, Climate, Pressure within the WDN, Topography	The size of the supply area also determines the type of intermittent supply method used to distribute the water. For example large distribution systems for large supply areas would usually require rotational intermittent supply where zones are supplied with water sequentially for a certain supply duration (usually in days, e.g. 3-4 days per zone). However for smaller supply areas water can be supplied to cover the whole area at the same time and the supply duration and frequency can be shorter and more frequent When considering the length of the supply duration the network filling time should be factored in (the number of users supplied increases with time). Even when the network is full the hydraulic head at the outlet is sometimes too low to allow supply for the positionally and topographically disadvantaged users and it takes time for the pressure levels to become adequate for all the users in the network. Supply duration also changes based on population size and pipe configuration. Larger population areas are more sensitive to a reduction in supply duration. The larger the population the longer the supply duration has to be in order to ensure that every user has adequate supply of water. There can also be variation in the supply duration depending on the seasons or climate of a certain area (e.g. 10-12 hour daily supply during the dry season and then switch to 24h supply during the rainy season). Users in topographically advantageous areas in the network get water almost immediately after supply starts however users in less advantageous areas must wait longer for supply. The amount of time required for the water supply to reach all the users in the network also depends on the network hydraulic capacity (specifically related to the pipe diameters and the reservoir capacity). If the pipe diameters are large enough to handle the initial peak flows and pressure then the water supply to the users will be delivered at a faster rate. Also the availability of water in the reservoirs affects how long users have to wait in order get water supply.	The timing of supply together with the supply duration has a significant impact on the water usage per capita and the peak factors. It has also been observed that there is no significant increase in water consumption when the duration of supply is long enough. The water demand for IWS is based on the maximum amount of water that can be collected during the supply hours. Therefore the longer the supply duration the more water can be collected (the supply duration should be long enough to allow all the users in the network to receive the allocated ration of water for the day) and vice versa. IWS of 1-6 hours per day result in lower water consumption than 24 hour CWS. The determining of the supply duration depends on the local conditions (i.e. the indoor and outdoor usage, household water usage activities, the socio-economic characteristics and network filling time). The supply duration also has a strong effect on the peak factor (PF=24/t, t=supply duration). A decrease in the supply duration increases the peak factor (the magnitude of the peak factors depends on the supply duration and the system carrying capacity). Note however that an increase in water usage during IWS conditions is mainly due to the adequacy of the water supply than it is due to the duration of supply.	Arranging the supply hours to be staggered (by dividing the supply area into smaller supply zones where some zones are supplied for a certain duration and other zones are closed for the supply duration) can be an effective way to improve the supply equity. The level of pressure distribution over the network is also affected by the length of the supply duration (i.e. are the supply hours long enough to allow the pressure distribution in the network to be adequate for all users to collect the allocated ration of water?). Reducing the supply duration to less than 12hrs per day significantly increases hydraulic losses and energy costs. Maintaining the supply duration at more than 12 hours per day reduces the chances of hydraulic failure in the water distribution networks. Water distribution systems in large localities are more sensitive to reductions in water supply duration than those in small towns and villages. Similarly, intermittent-supply conditions have more impact on the main transmission pipes than on the distribution network. A reduction of the supply duration results in insufficient water availability to the users and unreliability of the WDN.	IWS	(Vairavamoorthy et al., 2008); (Vairavamoorthy et al., 2001); (De Marchis et al., 2011); (Fan et al., 2014); (Abu-Madi & Trifunovic, 2013); (Andey & Kelkar, 2007); (Kumpel & Nelson, 2016); (A. Ilaya-Ayza et al., 2017); (Gottipati & Nanduri, 2014); (Andey & Kelkar, 2009)

Parameter Categories	Driving Parameters	Parameter Interrelationships and Effects					
		Directly affected by the following parameters:	**Nature of the relationship**	**How is it linked to water demand?**	**How is it linked to system performance?**	**CWS, IWS or Both?**	**Reference**
Structural	House Connection Type: LOS for water supply and sanitation	Area Income Level, Area Development Type	The high/medium income level areas will usually have house connection water supply and full waterborne sanitation systems. The low income areas will mostly have partial waterborne sanitation systems and chemical/pit systems which require no water. The higher the income level, the better the connection and sanitation systems used.	The type of connection determines the typical amount of water consumption in a certain area. Standpipe is the least amount and house connection is the highest amount of consumption with yard connection in-between. Also the better the sanitation system in an area, the higher the water consumption in an area.	Connection type and location also affects supply equity.	Both	(CSIR, 2003); (Van Zyl et al., 2007); (Van Zyl, 2006); (A. Ilaya-Ayza et al., 2017)
	Pressure within the WDN	Flow rate within the WDN, Supply Duration, Network Hydraulic Capacity, Topography	As the pipeline fills up and the flow rate within the WDN decreases due to the frictional losses the pressure continues to increase until a steady-state condition is reached. Increased flow rates within the pipeline result in reduced network pressure. Therefore the water supply service will not reach the users located in higher elevated areas of the network. Topography and the network configuration affect the pressures and the pressure distribution of a network.	Low pressures within the pipe network decreases the water consumption	To improve the supply equity in the IWS system the difference between maximum and minimum pressures in the system must be small. It is recommended that the pressure difference between the highest and the lowest pressure points in the system should not exceed 5m to ensure supply equity throughout the system.	Both	(De Marchis et al., 2011); (Charalambous & Laspidou, 2017); (Abu-Madi & Trifunovic, 2013); (A. Ilaya-Ayza et al., 2017)
	Flow within the WDN	Pressure within the WDN, Network Hydraulic Capacity, Supply Duration, Topography	In the initial hours of supply there are larger than expected flow rates in the pipeline which leads to low pressures in the network (the large flow rates happen during the pipeline filling process). The larger flow rates will require larger pipe diameters in the network in order to accommodate the larger flows and to supply water at adequate pressures and flow rates. A decrease in the supply duration results in an increase in the flow rate within the WDN for certain fixed pipe diameters.	High flow rates within the pipeline leads to low pressures in the network which decreases the water consumption. The users further away from the supply point (and those in high ground areas) won't be able to draw sufficient quantities of water during the supply period due to the low pressure. The flow variation between the large initial flows in the first minutes of supply and the reduced flows towards the end of the supply period ranges from 20% to 30% (i.e. flow rate reduces by 20 to 30% between initial flows and end flows).	The high initial flow rates within the pipeline leads to low pressures in the network which cause poor supply equity throughout the network.	Both	(Vairavamoorthy et al., 2001); (Charalambous & Laspidou, 2017); (A. E. Ilaya-Ayza et al., 2017); (Abu-Madi & Trifunovic, 2013); (A. Ilaya-Ayza et al., 2017)
	Network Hydraulic Capacity	Supply Duration, Flow rate within the WDN, Pressure within the WDN	Reducing the supply duration will require an increase in the network pipe diameters in order for the network to have a reliable and even distribution of water to meet the total water demand of the region. The impact of supply duration on the pipe sizes is low for small networks (e.g. supplying 5000 people) and it increases as the network gets larger. During the network filling process there will be larger flow rates which will require larger pipe diameters in the network in order to accommodate the larger flows and to supply water at adequate pressures and flow rates. When supply duration is reduced from 24 hours to 12 hours the effects are minor but once the supply duration is reduced to below 12 hours per day the required pipe diameters drastically increase.	The magnitude of the peak factors depends on the supply duration and the system carrying capacity. An increase in the network pipe diameters results in a reduction in the initial peaks and an improvement in the even water distribution which would result in an increase in the water consumption.	For the service reservoirs there's a minimum water level required for adequate supply equity. An increase in the reservoir water levels results in an improvement in the supply equity up to a certain point where the supply equity remains constant (increasing the water level or elevation after this point has no effect on supply equity). The location of the service reservoir and the network configuration also have a significant influence on the supply equity.	Both	(Abu-Madi & Trifunovic, 2013); (Andey & Kelkar, 2007); (A. E. Ilaya-Ayza et al., 2017); (Gottipati & Nanduri, 2014); (A. Ilaya-Ayza et al., 2017)
	Private Tanks	Available Pressure Head at the Outlet	The advantaged users in terms of position and topography, who are able to collect water as soon as supply begins will have larger tank capacity to ensure that they can get the maximum amount of water during the supply hours as long as there is adequate pressure available at the outlets.	User tanks modify the demand pattern of users during CWS conditions. The diurnal pattern for CWS will have reduced peaks. User demand/consumption is much higher at the beginning of the supply hours than if a user doesn't have a tank. This reduces the pressure levels in the network and this affects the positionally disadvantaged users.	The presence of tanks reduces the initial pressure levels in the network and the users downstream are negatively affected due to inadequate/low pressures. As the supply hours continue the pressure increases slightly as the tanks get full. Therefore the presence of tanks creates competition for the water supply because each user aims to collect as much water as possible in a short period of time which creates supply inequality in the system.	IWS *(Both)*	(De Marchis et al., 2011); (A. Ilaya-Ayza et al., 2017)
	Available Pressure Head at the Outlet	Flow rate within the WDN, Pressure within the WDN, Topography, Private Tanks, Network Hydraulic Capacity	Low pressures within the pipeline result in a low pressure head at the outlet. Too high flow rates within the pipeline also result in low available pressure head at the outlet. The position/elevation of the user tanks also affects the outflow the user experiences. When there is a small elevation difference between the tap/outlet in the house and the storage tank the user will experience low water pressure regardless of the adequate pressures within the pipeline or distribution network. The presence of tanks causes low pressures in the network.	Referenced as one of the most influential factors. A reduction in pressure leads to a decrease in the water consumption (and vice versa). The quantity of water collected is totally dependent on the available pressure at the outlet for IWS. For CWS pressure has a small but significant effect on domestic water usage (it mainly affects the amount of leakage in CWS system).	*No available research defining the link.*	Both	(Vairavamoorthy et al., 2008); (Vairavamoorthy et al., 2001); (De Marchis et al., 2011); (Van Zyl, 2006); (Abu-Madi & Trifunovic, 2013)
	Flow at the Outlet	Available Pressure Head at the Outlet, Topography, Network Hydraulic Capacity	An increase in the available pressure head at the outlet increases the flow rate/outflow at the outlet. The discharge at the outlet is directly proportional to the pressure head available at the outlet. The flow rate at the outlet depends on the geometric and hydraulic features and the pressure at the outlet.	An adequate flow rate at the outlet the results in an increase in the water consumption.	*No available research defining the link.*	Both	(De Marchis et al., 2011); (Gottipati & Nanduri, 2014); (A. Ilaya-Ayza et al., 2017)

Parameter Categories	Driving Parameters	Parameter Interrelationships and Effects: Directly affected by the following parameters:	Nature of the relationship	How is it linked to water demand?	How is it linked to system performance?	CWS, IWS or Both?	Reference
Socio-Economic	Land Use: Residential			Referenced as one of the most influential factors. The water demand varies according to the type of land use of the supply area being considered. Land use is categorized into domestic and non-domestic, domestic refers to residential areas and non-domestic refers to commercial, industrial and agricultural areas.	*No available research defining the link.*	CWS *(Both)*	(Van Zyl et al., 2007)
	Area Population Size			Population density (number of people per unit area) and size can substantially influence water consumption. The more the population and the higher the population density, the more the water consumption will be. The population size also affects the peak factors.	*The larger the population size being supplied at the same time, the more difficult it becomes to achieve an acceptable level of supply equity. The larger the number of users that are drawing water from the system at the same time the less adequate the pressures will become in the system. That is why it is common during IWS to divide the supply area into smaller supply zones (decreasing the number of people being supplied at the same time) so that the users in each zone will be able to experience better supply equity.*	Both	(Van Zyl et al., 2007); (Vairavamoorthy et al., 2008); (U.S. National Academy of Science et al., 1999); (Kumpel & Nelson, 2016); (Totsuka & Trifunovic, 2004.)
	Area Development Type: Suburb/Township	Area Income Level, Average Stand Size	The better the level or type of development, the higher the income level of people living there. The stand sizes in the suburbs tend to be bigger than the stands in the townships (there are exceptions).	Referenced as one of the most influential factors. Suburbs generally have higher water consumption per capita compared to the township areas.	*Since the type of development is mainly based on the area income level it can be argued that most of the higher income area houses (suburbs) will have storage tanks and pumps to assist the users draw as much water as possible during the supply period. The presence of tanks and pumps affects the pressures and the pressure distribution in the network. Township households are less likely to have pumps and storage tanks because these are usually low income areas and the users cannot afford them. Also the type of water supply and sanitation connections that the users have affects the supply equity in the network. Suburbs tend to have house connections and full waterborne sanitation. Townships usually have a combination of standpipe and yard connections along with septic tank and pit sanitation connections. The type of connections in each type of development affects the system performance.*	CWS *(Both)*	(Van Zyl et al., 2007); (Jacobs et al., 2004)
	Water Price	Location, Area Income Level	There are varying tariff structures for the different regions/locations. The price elasticity of indoor domestic water use stays more or less the same for all the income groups, but the price elasticity for outdoor domestic water use is higher (more negative) for high and middle income users compared to low income users. The price elasticity for low income users for both indoor and outdoor water use is similar.	Referenced as one of the most influential factors. An increase in the water tariffs decreases the water consumption (this occurs when done in conjunction with proper metering and management of the tariff structure and collection system). Outdoor water use will reduce more than indoor use when there's a price increase (outdoor water use is more price elastic). Residential water demand tends to have a negative price elasticity (i.e. water demand decreases with an increase in the water price). The price elasticity is also dependent on time (short term and long term elasticity). A price increase will have an immediate effect on the water usage pattern (short term elasticity) and in the long term users will also start installing water saving plumbing fixtures. This results in a higher elasticity value (i.e. the long term effect of the price increase is even more which means there is more decrease in water consumption).	*No available research defining the link.* *The higher the cost of water, the lower the demand becomes during CWS conditions. As a result of lower demand there will be lower peak flows, and the WDS can be operated at lower pressures thus reducing leaks in the system.*	CWS *(Both)*	(Van Zyl et al., 2007); (Jacobs et al., 2004); (Van Zyl, 2006); (Abu-Madi & Trifunovic, 2013)
	Area Income Level: Low/Middle/High	Area Development Type	A high area income level is typically related to a suburb type of development area.	Water demand of high income users is more sensitive to climate than middle and low income users. Referenced as one of the most influential factors. The higher the income level, the higher the water consumption per capita (follows an S-curve pattern of influence). It is widely accepted that water consumption is directly proportional to income level per person. The income status of a household impacts mostly the outdoor water usage due to higher income users having bigger yards.	*Higher income areas will be able to afford storage tanks and pumps which affect the way the IWS system performs*	Both	(Van Zyl et al., 2007); (U.S. National Academy of Science et al., 1999); (Jacobs et al., 2004); (Van Zyl, 2006)
	Average Stand Size	Area Income Level, Climate	It is argued that high income users seldom have small stands (there are exceptions where low income areas have larger stand areas and high income users have smaller stands). Therefore it is assumed that the higher the income level the bigger the stand size and when estimating water consumption stand size and income level should always be taken into account.	The water usage of high income users is more sensitive to climate due to the larger stand sizes which cause more outdoor water usage. For low income users this is almost negligible because the outdoor water usage is much less. Referenced as one of the most influential factors. The bigger the stand size the more the water consumption (AADD) (for stand sizes 0-600m^2 the demand is constant then it increases for bigger stands) also as the stand size decreases the water consumption also decreases. There's an exception to this when there's densification, where there's an increased demand in a smaller stand area (e.g. apartment complexes). The influence of the stand size is mostly applicable to outdoor water usage.	*No available research defining the link.*	CWS	(Van Zyl et al., 2007); (Jacobs et al., 2004); (Van Zyl, 2006)
	Number of People per Household			Referenced as the most significant parameter influencing water consumption. As the household size increases, the water consumption per capita decreases.	*No available research defining the link.*	CWS	(Van Zyl et al., 2007); (U.S. National Academy of Science et al., 1999); (Van Zyl, 2006)

The interrelationships and effects of the identified parameters were investigated for both the CWS and IWS scenario. From the compiled description of the parameter interrelationships, effects and correlations for 13 out of the 20 parameters were found to be applicable to both the CWS and the IWS scenarios. Two parameters were found to be exclusively applicable to the IWS scenario and the last five parameters were found to be exclusively applicable to the CWS scenario. Refer to Table 6-2 for a summary of the applicability of the parameters to CWS and IWS.

Table 6-2 Summary of the parameters applicability to the CWS and IWS systems.

Total number of parameters investigated		20
Number of parameters applicable to both CWS and to IWS	13	Geographic location, area topography, supply area size, climate, area population size, area income level, house connection type, pressure within the WDN, flow within the WDN, available pressure head at the outlet, network hydraulic capacity, flow at the outlet, private tanks.
Number of parameters applicable to CWS only	5	Land use, area development type, number of people per household, average stand size, water price.
Number of parameters applicable to IWS only	2	Supply duration, network filling time.

A common theme in the methodologies used to develop water demand estimation methods as reviewed in Section 2.4.2, is that each study commenced with a parameters investigation. The main parameters influencing the domestic water demand were identified and correlated to the measured water demand data of the supply area. This was done to establish the nature of the relationship between the parameters and the water demand. In order to develop an accurate representation of the real-world scenario of the domestic water demand during IWS conditions, there was the need to investigate and understand the different driving parameters that affect the water demand pattern. The outcomes presented in Table 6-1 provided insight into the complex relationships of the parameters that drive the domestic water demand and the performance of WDSs during IWS conditions.

6.1.2 Identifying the crucial parameter

The information in Table 6-1 was analysed in order to determine which driving parameter or parameters were the most influential to WDSs subjected to IWS. This analysis was conducted by determining the number of parameters affected by a driving parameter, and then determining the number of parameters that affect that driving parameter. This was done for each of the 20 parameters, and the results of the analysis are presented in Table 6-3.

Table 6-3 Parameter correlations analysis.

Driving Parameters:	How many affected by parameter	How many affecting parameter	Sum of links to other parameters	Link to water demand	Link to system performance
Supply Duration	3	8	11	Yes	Yes
Network Hydraulic Capacity	6	3	9	Yes	Yes
Available Pressure Head at the Outlet	3	5	8	Yes	Yes
Pressure within the Pipeline	4	4	8	Yes	Yes
Topography	7	0	7	Yes	Yes
Flow Rate within the Pipeline	3	4	7	Yes	Yes
Income Level	4	1	5	Yes	NIA
Development Type	2	2	4	Yes	NIA
Private Tanks	3	1	4	Yes	Yes
Network Filling Time	1	3	4	Yes	NIA
Supply Area Size	3	0	3	Yes	Yes
Climate	2	1	3	Yes	NIA
Average Size of Stands	1	2	3	Yes	NIA
Flow Rate at the Outlet	0	3	3	Yes	Yes
Location	2	0	2	Yes	NIA
Type of Connection	0	2	2	Yes	Yes
Water price	0	2	2	Yes	NIA
Population Size	1	0	1	Yes	NIA
Land Use	0	0	0	Yes	NIA
Number of People per Household	0	0	0	Yes	NIA

From Table 6-3 it is evident that supply duration, network hydraulic capacity, the available pressure head at the outlet, pressure within the WDN, topography and flow rate within the WDN have the most correlations with other parameters. However, supply duration had the highest number of correlations, with 11 correlations, as presented in Table 6-3. Based on the highest number of correlations, supply duration was determined to be the most crucial parameter in WDSs subjected to IWS. The 11 correlations with supply duration consist of nine different parameters as indicated in Figure 6.1. In Figure 6.1 it is indicated that network hydraulic capacity and pressure within the WDN do not only affect supply duration but are affected by supply duration (indicated in green). The flow rate within the pipeline was the only parameter to be affected by supply duration (indicated in orange). The remaining six parameters (indicated in blue in Figure 6.1) had single correlations with supply duration, all affecting supply duration. The description of the nature of the correlations have been included in Table 6-1. The ten influence parameters, together with the determined correlations, became the building blocks of the IWS water demand pattern model.

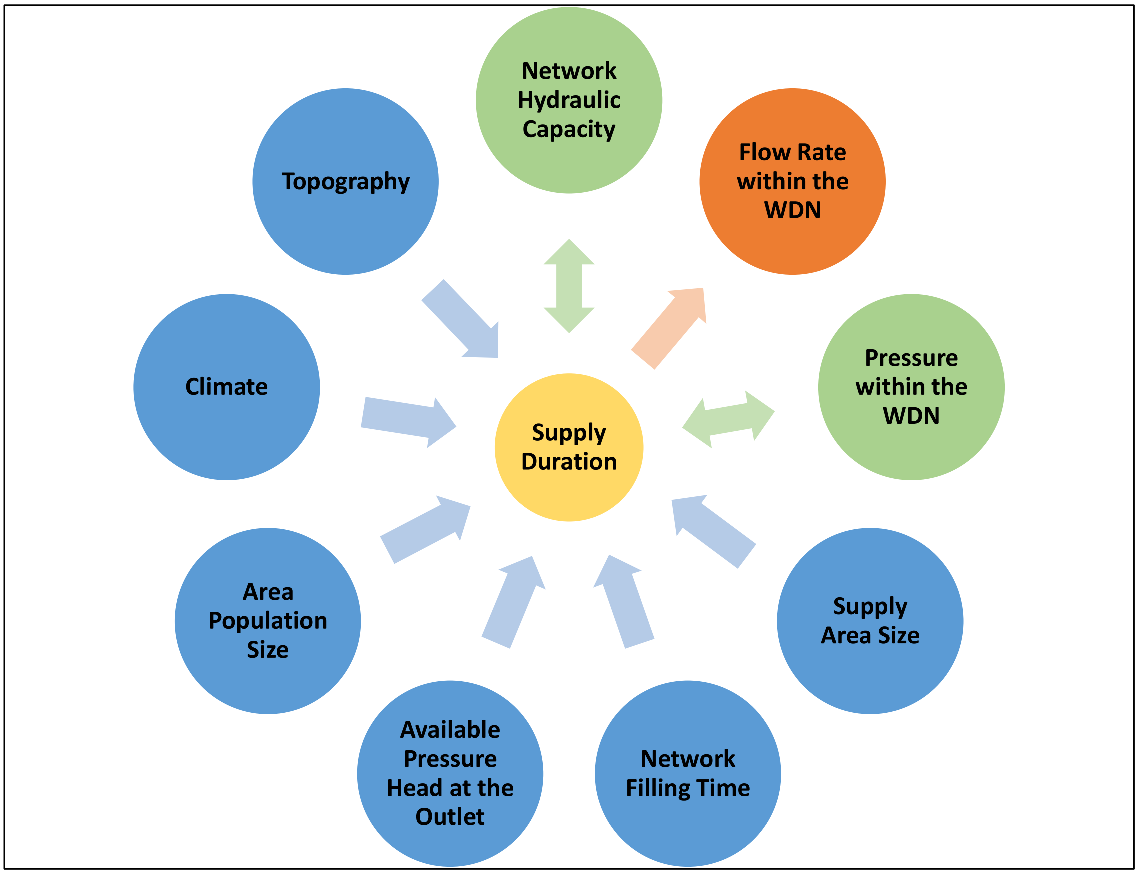


Figure 6.1 Correlation between the ten most influential parameters to IWS systems.

In Table 6-3 there were a number of parameters recorded to not have any correlations with other parameters and with system performance. These parameters are marked with a zero value and with the 'NIA' abbreviation which means no information available. This was a result of a lack of available research related to the correlation of these parameters with the other parameters and with system performance.

From the ten most influential parameters for IWDSs only climate, area population size, and the pressure within the WDN were listed amongst the most influential parameters for CWS systems, as mentioned in Section 2.4.3 of the literature review. From this result, it can be deduced that CWS and IWS are vastly different systems, which probably have to be considered and dealt with in vastly different ways. Scheepers (2012) and Totsuka and Trifunovic (2004) mention that the water demand for WDNs under IWS is not dependent on user demand as in CWS systems, but rather on the WDN hydraulic capacity and the available pressure head in the network system. Therefore, the important parameters for CWS systems are not necessarily the same important parameters for IWS, since the characteristics of these two forms of water supply are distinct. CWS is characterised by a fully pressurised WDS, with the objective of providing a sufficient volume of water to meet the consumer demand at adequate pressures and at the least cost. IWS is characterised by mostly partially full flows in the WDS, where the objective is to distribute the available water resources to the consumers as fairly, equally and predictable as possible within the supply period. The comparison of the two water supply methods, based on the literature review and the results of the parameters investigation is summarised in Table 6-4.

Table 6-4 Comparison of water distribution systems.

	Continuous Water Supply	Intermittent Water Supply
Definition	A 24 hours per day continuous water supply that is delivered every day of the year to meet the consumer demand.	A demand management/control technique where the water supply is made available and delivered in intervals of less than 24 hours per day to the consumers.
System type	Demand Driven	Pressure Driven
System objectives	To provide a sufficient amount of water to meet the consumer demand at adequate pressures and at the least cost.	To distribute the available water resources to the consumers as fairly, equally and predictable as possible within the supply period.
Most influential parameters	Stand Area Size Household Income Water Price Available Pressure Type of Development Climate	Supply duration Network Hydraulic Capacity Available Pressure Head at the Outlet Pressure within the WDN Topography Flow rate within the WDN Climate Supply Area Population Network Filling Time Supply Area Size
Crucial driving parameter(s)	Stand Area Size	Supply duration

6.2 Model development

6.2.1 IWS diurnal water demand pattern model

This study set out with the purpose of developing a new model to predict the domestic diurnal water demand pattern for areas subjected to IWS. This was attempted by considering the WDS filling process and the crucial parameters that impact WDSs, for the case when logged water demand data is not readily available and when a reliability analysis of the existing WDS is needed. The concept in the development of the IWS diurnal water demand pattern model was to take and use what is known of the CWS water demand patterns as well as the WDS characteristics of a certain supply area. This information would be converted into a predicted IWS diurnal water demand pattern, by creating a spreadsheet containing the identified dominant influence parameters, and the theory, assumptions, and equations discussed in Chapter 4. By using the spreadsheet a mathematical investigation was conducted using the steps presented in Figure 4-2.

Defining the WDN and supply area characteristics

The WDN and supply characteristics were defined in order to perform the calculations. These characteristics were determined using the provided WDN model for the KaNyamazane Toad Street supply area. The results of this step of the investigation are summarised in Table 6-5.

Table 6-5 KaNyamazane WDN and supply area characteristics.

Supply Area Characteristics	
Design demand	518.4 kL/day
Actual demand (AADD)	150.4 kL/day
Base flows (30% of AADD)	45.12 kL/day
Area size	Small
PHF	LCH
Focus node	Node 331585
Area topography	Flat
Water Distribution Network Characteristics	
Reticulation network pipes:	
Pipe material	uPVC
WDN volume	2134.3 m^3
Pipe diameter range	50-141.8 mm
Max. allowable flow rate for pipe diameter	28.8 m^3/h
Max. allowable velocity for pipe diameter	2.08 m/s
Bulk supply pipe:	
Pipe material	uPVC
Pipe diameter	141.8 mm
Pipe length	17.13 m
Max. allowable flow rate for pipe diameter	93.6 m^3/h
Max. allowable velocity for pipe diameter	2.08 m/s

CWS water demand scenario

The representative CWS diurnal demand pattern for the KaNyamazane supply area was determined using the actual demand (AADD) and the PHFs obtained from the study by Loubser et al. (2018). An AADD of 150.4 kL/day was determined and the base flows were calculated to be 45.12 kL/day which is 30% of the AADD. The KaNyamazane Toad Street area was found to be a small low cost housing area (determined by using Table 4-1). Therefore, the generic low cost housing CWS diurnal water demand pattern was chosen (using Figure 4-5) along with the related PHFs (using Table 4-2) as the initial assumed CWS demand pattern. This pattern would be converted to the IWS water demand pattern equivalent in the following steps. Refer to Table 6-6 for the resulting Excel calculations, and also refer to Table 6-7 and Figure 6.2 for the PHFs, peak water demand, and related CWS diurnal water demand pattern.

Table 6-6 Characteristics of the chosen CWS diurnal water demand pattern.

Specify CWS AADD	150.40	kL/day
Specify CWS Base Flows	45.12	kL/day
AADD	6.27	m^3/h
Highest Morning PHF	1.71	
Highest Evening PHF	1.49	
Peak Morning Demand	10.74	m^3/h
Peak Evening Demand	9.36	m^3/h
Total Estimated Volume Water Demand per Day	147.65	kL/day
Estimated AADD	6.15	m^3/h
Base Flow	1.85	m^3/h
Total Base Flow Volume	44.30	kL/day
Base Flow Percentage of the Total Water Demand	30.00	%

Table 6-7 Peak hour factors and peak water demand calculated for the chosen CWS demand pattern.

	LC Housing	Peak Water Demand (m^3/h)	Area under curve (Reimann sum)
	Pattern4		
Hour	PHF		
1	0.45	2.84	2.87
2	0.46	2.91	3.00
3	0.49	3.09	3.16
4	0.51	3.22	3.53
5	0.61	3.85	4.57
6	0.84	5.29	6.42
7	1.20	7.54	8.64
8	1.55	9.74	10.24
9	1.71	10.74	10.71
10	1.70	10.68	10.36
11	1.60	10.05	9.71
12	1.49	9.36	9.02
13	1.38	8.67	8.36
14	1.28	8.04	7.79
15	1.20	7.54	7.39
16	1.15	7.23	7.10
17	1.11	6.98	6.92
18	1.09	6.85	6.57
19	1.00	6.29	5.79
20	0.84	5.29	4.88
21	0.71	4.47	4.16
22	0.61	3.85	3.53
23	0.51	3.22	2.94
24	0.42	2.66	
Total		150.40	147.65

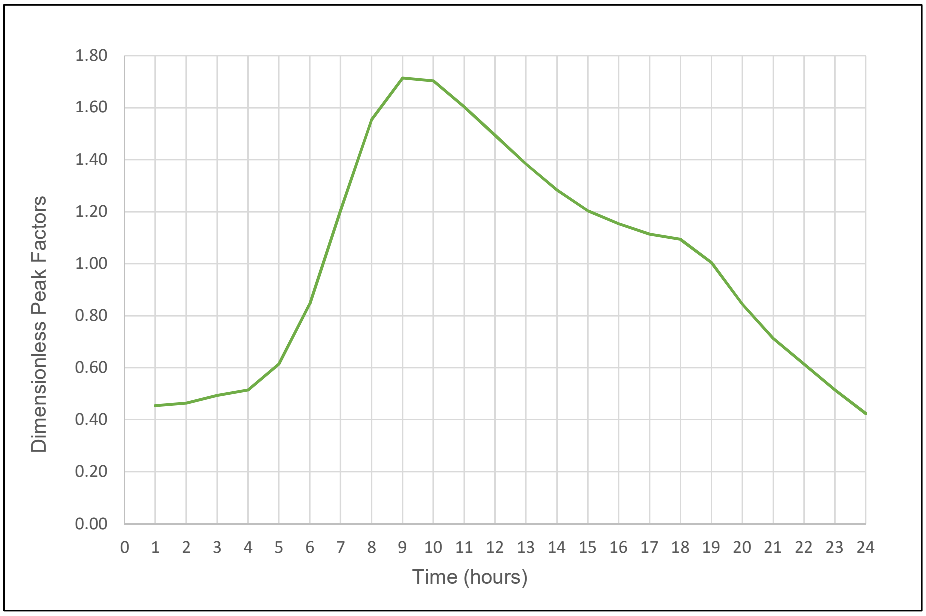


Figure 6.2 Chosen initial CWS diurnal water demand pattern.

IWS water demand scenario

From step 2 to step 4 of the Excel calculation procedure, three IWS scenarios of one, two and three supply periods per day for the KaNyamazane study site were investigated. For the scenario of one supply period per day, a total supply duration of 20 hours was assumed starting from 3:00 and ending at 23:00. For the scenario of two supply periods per day, a total supply duration of 11.5 hours was used, with the first supply period starting at 4:00 and ending at 10:30 and the second supply period starting at 16:45 and ending at 21:45. This scenario was based on the actual KaNyamazane Toad Street conditions where there are two supply periods per day. For this particular day (27/7/2019) the total supply duration was 11.5 hours. Then, for the scenario of three supply periods per day a total supply duration of 15 hours was assumed with supply period 1 starting at 3:00 and ending at 8:00, supply period 2 starting at 10:00 and ending at 15:00, and supply period 3 starting at 17:00 and ending at 22:00. Since it was assumed that a total supply duration of 20 hours per day did not affect the AADD of a supply area, a SDDRF of 100% was chosen according to Table 4-3 for the one supply period per day scenario. Similarly, for the three supply periods per day scenario, a SDDRF of 88% was chosen from Table 4-3. Then for the two supply periods per day scenario, a SDDRF of 73% was determined. By using Equation (4-3), the IWS equivalent AADDs were calculated for all three scenarios. Refer to Table 6-8, Table 6-9, Table 6-10 for the Excel calculations of the three IWS scenarios investigated.

Table 6-8 Step 2 IWS scenario for one supply period per day.

CWS AADD		147.65	kL/day
		6.15	m^3/h
Supply duration demand reduction factor		100	%
New AADD		147.65	kL/day
		6.15	m^3/h
For Total Supply Duration >= 20 hours per Day			
1 Supply Period per Day			
No.	Start	End	Duration (hrs)
1	3:00	23:00	20:00

Table 6-9 Step 3 IWS scenario for two supply periods per day.

CWS AADD		147.65	kL/day
		6.15	m^3/h
Supply duration demand reduction factor		73	%
New IWS AADD		107.78	kL/day
		4.49	m^3/h
For Total Supply Duration >= 1.5 but < 20 hours per Day			
2 Supply Periods per Day			
No.	Start	End	Duration (hrs)
1	4:00	10:30	6:30
2	16:45	21:45	5:00

Table 6-10 Step 4 IWS scenario for three supply periods per day.

CWS AADD		147.65	kL/day
		6.15	m^3/h
Supply duration demand reduction factor		88	%
New AADD		129.93	kL/day
		5.41	m^3/h
For Total Supply Duration >= 1.5 but < 20 hours per Day			
3 Supply Periods per Day			
No.	Start	End	Duration (hrs)
1	3:00	8:00	5:00
2	10:00	15:00	5:00
3	17:00	22:00	5:00

Phase 1 – Bulk supply pipe filling

After determining the IWS equivalent AADDs, the phase 1 calculations were conducted for each IWS scenario. This calculation was based on the pipe filling volume, which was discovered to produce a better representation of the real-world scenario than using Equation (4-9) and Equation (4-10). It was found that the phase 1 calculations are the same for every supply period. A 15 minute peak flow value of 4 m^3/h was determined for each supply period in each step. Then, by using Equation (4-8), the corresponding 15 minute peak factors were calculated. Table 6-11 contains the related Excel calculations. Also refer to Table 6-12 for the determined PFs.

Table 6-11 Determining the bulk supply pipe filling flow rate.

Phase 1:		
Bulk Supply Pipe between Storage Reservoir and Supply Area Network		
Total Pipe Length	17.13	m
Average Pipe Internal Diameter	0.14	m
Pipe Cross Sectional Area	0.02	m^2
Pipe Filling Volume	1.00	m^3
Initial Filling Flow Rate	4.00	m^3/h
Max Allowable Flow Rate for Pipe Diameter and Pipe Material	93.60	m^3/h
Recommended Maximum Allowable Velocities in the Pipe	2.08	m/s
Total Pipe Filling Time at Maximum Velocity	0.64	min
	38.46	sec

Table 6-12 Summary of the peak factors calculated for each supply period scenario.

1 Supply Period Per Day						2 Supply Period Per Day						3 Supply Period Per Day					
Time	PF15min	Time	PF15min	Time	PF15min	Time	PF15min	Time	PF15min	Time	PF15min	Time	PF15min	Time	PF15min	Time	PF15min
0:00	0.00	8:15	1.48	16:30	1.38	0:00	0.00	8:15	2.73	16:30	0.00	0:00	0.00	8:15	0.00	16:30	0.00
0:15	0.00	8:30	1.59	16:45	1.36	0:15	0.00	8:30	2.94	16:45	0.00	0:15	0.00	8:30	0.00	16:45	0.00
0:30	0.00	8:45	1.70	17:00	1.35	0:30	0.00	8:45	3.15	17:00	5.49	0:30	0.00	8:45	0.00	17:00	0.00
0:45	0.00	9:00	1.81	17:15	1.34	0:45	0.00	9:00	3.29	17:15	4.76	0:45	0.00	9:00	0.00	17:15	4.55
1:00	0.00	9:15	1.88	17:30	1.33	1:00	0.00	9:15	3.38	17:30	4.09	1:00	0.00	9:15	0.00	17:30	3.95
1:15	0.00	9:30	1.94	17:45	1.32	1:15	0.00	9:30	3.48	17:45	3.41	1:15	0.00	9:30	0.00	17:45	3.13
1:30	0.00	9:45	1.99	18:00	1.31	1:30	0.00	9:45	3.55	18:00	2.74	1:30	0.00	9:45	0.00	18:00	2.32
1:45	0.00	10:00	2.04	18:15	1.31	1:45	0.00	10:00	3.55	18:15	2.29	1:45	0.00	10:00	0.00	18:15	1.50
2:00	0.00	10:15	2.04	18:30	1.29	2:00	0.00	10:15	3.54	18:30	2.28	2:00	0.00	10:15	4.19	18:30	1.45
2:15	0.00	10:30	2.04	18:45	1.26	2:15	0.00	10:30	3.54	18:45	2.27	2:15	0.00	10:30	3.57	18:45	1.44
2:30	0.00	10:45	2.03	19:00	1.24	2:30	0.00	10:45	0.00	19:00	2.25	2:30	0.00	10:45	3.12	19:00	1.43
2:45	0.00	11:00	2.02	19:15	1.21	2:45	0.00	11:00	0.00	19:15	2.22	2:45	0.00	11:00	2.67	19:15	1.40
3:00	0.00	11:15	1.99	19:30	1.16	3:00	0.00	11:15	0.00	19:30	2.17	3:00	0.00	11:15	2.22	19:30	1.36
3:15	4.01	11:30	1.96	19:45	1.11	3:15	0.00	11:30	0.00	19:45	2.11	3:15	4.55	11:30	2.17	19:45	1.32
3:30	3.48	11:45	1.93	20:00	1.06	3:30	0.00	11:45	0.00	20:00	2.06	3:30	3.95	11:45	2.13	20:00	1.26
3:45	2.70	12:00	1.90	20:15	1.01	3:45	0.00	12:00	0.00	20:15	1.97	3:45	2.87	12:00	2.09	20:15	1.19
4:00	1.93	12:15	1.86	20:30	0.97	4:00	0.00	12:15	0.00	20:30	1.88	4:00	1.79	12:15	2.04	20:30	1.12
4:15	1.15	12:30	1.83	20:45	0.93	4:15	5.49	12:30	0.00	20:45	1.79	4:15	0.70	12:30	1.99	20:45	1.06
4:30	0.59	12:45	1.79	21:00	0.88	4:30	4.76	12:45	0.00	21:00	1.71	4:30	0.66	12:45	1.95	21:00	1.00
4:45	0.60	13:00	1.75	21:15	0.85	4:45	3.70	13:00	0.00	21:15	1.64	4:45	0.67	13:00	1.90	21:15	0.95
5:00	0.60	13:15	1.72	21:30	0.81	5:00	2.63	13:15	0.00	21:30	1.56	5:00	0.68	13:15	1.85	21:30	0.90
5:15	0.61	13:30	1.68	21:45	0.78	5:15	1.57	13:30	0.00	21:45	1.49	5:15	0.72	13:30	1.80	21:45	0.86
5:30	0.63	13:45	1.65	22:00	0.75	5:30	1.10	13:45	0.00	22:00	0.00	5:30	0.76	13:45	1.76	22:00	0.82
5:45	0.66	14:00	1.62	22:15	0.72	5:45	1.16	14:00	0.00	22:15	0.00	5:45	0.80	14:00	1.72	22:15	0.00
6:00	0.69	14:15	1.59	22:30	0.69	6:00	1.22	14:15	0.00	22:30	0.00	6:00	0.90	14:15	1.67	22:30	0.00
6:15	0.73	14:30	1.55	22:45	0.65	6:15	1.28	14:30	0.00	22:45	0.00	6:15	1.00	14:30	1.64	22:45	0.00
6:30	0.79	14:45	1.52	23:00	0.62	6:30	1.42	14:45	0.00	23:00	0.00	6:30	1.10	14:45	1.60	23:00	0.00
6:45	0.87	15:00	1.50	23:15	0.00	6:45	1.56	15:00	0.00	23:15	0.00	6:45	1.24	15:00	1.57	23:15	0.00
7:00	0.94	15:15	1.47	23:30	0.00	7:00	1.69	15:15	0.00	23:30	0.00	7:00	1.40	15:15	0.00	23:30	0.00
7:15	1.02	15:30	1.45	23:45	0.00	7:15	1.88	15:30	0.00	23:45	0.00	7:15	1.55	15:30	0.00	23:45	0.00
7:30	1.13	15:45	1.43	0:00	0.00	7:30	2.09	15:45	0.00	0:00	0.00	7:30	1.71	15:45	0.00	0:00	0.00
7:45	1.25	16:00	1.41			7:45	2.31	16:00	0.00			7:45	1.86	16:00	0.00		
8:00	1.36	16:15	1.39			8:00	2.52	16:15	0.00			8:00	2.01	16:15	0.00		

Phase 2 – Network filling

In the phase 2 calculations, the WDS characteristics, total network length and the total network filling volume were used to determine the available flow rate for network filling (Q_{max}) and the network filling time (t_{max}). This calculation is presented in the Excel calculation in Table 6-13.

Table 6-13 Determining the available flow rate for network filling and the filling time.

Phase 2:			
Network Filling Process:			
Total Network Length		2134.29	m
Average Network Pipe Internal Diameter		0.07	m
Pipe Cross Sectional Area		0.004	m^2
Total Network Filling Volume		9.00	m^3
Max Allowable Flow Rate for Pipe Diameter		28.80	m^3/h
Network filling time at maximum allowable flow rate		0.31	hrs
		19.00	min
User water consumption begins after time =		0.64	min
CWS user demand rate during the network filling time interval		7.20	m^3/h
Supply duration demand reduction factor		73	%
Adjusted CWS user demand rate during the network filling time interval		5.25	m^3/h
Total user water consumption during Phase 2		1.64	m^3
Available flow rate for network filling	Qmax =	23.55	m^3/h
Network filling time at available flow rate	tmax =	0.38	hrs
		22.93	min
Check: Is the flow rate for network filling within the range of the designed network demand?			
WDN design demand		21.60	m^3/h
Flow rate during network filling is beyond designed network capacity, therefore use WDN design demand to determine Qmax			
Recalculated flow rate in the pipes (Qmax)		21.60	m^3/h
Recalculated network filling time (tmax)		0.42	hrs
		25.00	min
Check: Is the total supply duration per supply period > total network filling time?			
Total Network Filling Time		0.42	hrs

The calculated available flow rate for network filling of 23.55 m^3/h was found to be greater than the designed maximum network demand capacity of 21.60 m^3/h, which was determined using the node data table of the KaNyamazane model. The available flow rate for network filling was the result of subtracting the user demand during network filling from the maximum allowable flow rate for the pipe. Therefore, the value for the available flow rate for network filling was limited to the designed maximum network demand. The network filling time was calculated to be 25 minutes; refer to Table 6-13. By using Equation (4-12), the 15 minute peak flows for the phase 2 time interval were determined. The minimum required pressure head (H_{min}) was chosen to be 6 m. The Stellenbosch Municipality Design Guidelines and Minimum Standards for Civil Engineering Services recommend pressure heads between 6m and 8m as the minimum working pressure at a household connection. The pressure head at the focus node (H_i) was obtained from the WADISO model (37.8 m). The maximum desired pressure head at the focus node (H_{max}) was chosen to be a value greater than the available head at the focus node (H_i) in order to simulate maximum flow conditions at the focus node. Refer to Figure 6.3 for the Excel worksheet used to determine the 15 minute peak flow values. The phase 2 calculations were repeated for every supply period, due to supply periods being at different times of the day, which affects the user water demand during the network filling process. By using Equation (4-8), the corresponding 15 minute peak factors were calculated for phase 2. Refer to Table 6-12 for the determined 15 minute PFs.

Phase 2 - Water Distribution Network Filling

Representing the whole network as one node that has all the demand lumped on it

Determining ΔHavl:			
ΔH	3	tmax (min)	27.6
tmin (min)	0	ti =	t(min)

Determining Qavl at the node:			
Qmax (m^3/min)	0.326	Hmax (m)	40.8
Hmin (m)	6	Hi (m)	37.8

time increments	0.46

t(hrs)	t(min)	Δhi (m)	Hi (m)	Qi(m^3/min)	Qi(m^3/h)		Vol.(m^3)	
0	0	0	37.8	0.311631635	18.69789809		0.1440733	
0.007682	0.46	0.3872983	38.1873	0.313523603	18.81141615		0.1446891	
0.015364	0.92	0.5477226	38.3477	0.314303946	18.85823673		0.1450067	
0.023046	1.38	0.6708204	38.4708	0.314901413	18.89408476		0.1452602	
0.030728	1.84	0.7745967	38.5746	0.315404221	18.92425328		0.1454781	
0.03841	2.3	0.8660254	38.666	0.315846542	18.95079249		0.145672	
0.046092	2.76	0.9486833	38.7487	0.316245897	18.97475382		0.1458486	
0.053774	3.22	1.0246951	38.8247	0.316612698	18.99676187		0.1460117	
0.061456	3.68	1.0954451	38.8954	0.316953726	19.01722357		0.1461641	
0.069138	4.14	1.161895	38.9619	0.317273693	19.03642159		0.1463075	
0.07682	4.6	1.2247449	39.0247	0.317576029	19.05456173		0.1464434	
0.084502	5.06	1.2845233	39.0845	0.317863322	19.07179935		0.1465728	
0.092184	5.52	1.3416408	39.1416	0.318137586	19.08825514		0.1466965	
0.099866	5.98	1.396424	39.1964	0.318400418	19.1040251		0.1468154	
0.107548	6.44	1.4491377	39.2491	0.318653117	19.11918703		0.1469297	
0.11523	6.9	1.5	39.3	0.318896751	19.13380507		0.1470402	
0.122912	7.36	1.5491933	39.3492	0.319132214	19.14793283		0.147147	
0.130594	7.82	1.5968719	39.3969	0.31936026	19.16161563		0.1472505	
0.138276	8.28	1.6431677	39.4432	0.319581537	19.17489222		0.1473511	
0.145958	8.74	1.6881943	39.4882	0.319796601	19.18779606		0.1474489	
0.15364	9.2	1.7320508	39.5321	0.320005937	19.20035622		0.1475442	
0.161322	9.66	1.7748239	39.5748	0.32020997	19.21259821		0.1476371	
0.169004	10.12	1.8165902	39.6166	0.320409075	19.2245445		0.1477278	
0.176686	10.58	1.8574176	39.6574	0.320603584	19.23621506		0.1478164	
0.184368	11.04	1.8973666	39.6974	0.320793795	19.24762771		0.1479032	
0.19205	11.5	1.9364917	39.7365	0.320979973	19.25879841		0.1479881	
0.199732	11.96	1.9748418	39.7748	0.321162359	19.26974156		0.1480714	
0.207414	12.42	2.0124612	39.8125	0.32134117	19.28047017		0.148153	
0.215096	12.88	2.0493902	39.8494	0.321516601	19.29099608		0.1482331	
0.222778	13.34	2.0856654	39.8857	0.321688834	19.30133006		0.1483118	
0.23046	13.8	2.1213203	39.9213	0.321858033	19.31148195		0.1483891	
0.238142	14.26	2.1563859	39.9564	0.322024347	19.32146081		0.1484652	
0.245824	14.72	2.1908902	39.9909	0.322187916	19.33127496		0.1485399	
0.253506	15.18	2.2248595	40.0249	0.322348867	19.34093205		0.1486136	
0.261188	15.64	2.258318	40.0583	0.32250732	19.35043918		0.148686	
0.26887	16.1	2.2912878	40.0913	0.322663382	19.35980294		0.1487574	
0.276552	16.56	2.32379	40.1238	0.322817157	19.36902942		0.1488278	
0.284234	17.02	2.3558438	40.1558	0.322968739	19.37812432		0.1488972	
0.291916	17.48	2.3874673	40.1875	0.323118216	19.38709295		0.1489656	
0.299598	17.94	2.4186773	40.2187	0.323265671	19.39594026		0.1490331	
0.30728	18.4	2.4494897	40.2495	0.323411181	19.40467089		0.1490998	
0.314962	18.86	2.4799194	40.2799	0.32355482	19.4132892		0.1491656	
0.322644	19.32	2.5099801	40.31	0.323696655	19.42179928		0.1492305	
0.330326	19.78	2.539685	40.3397	0.32383675	19.43020497		0.1492947	
0.338008	20.24	2.5690465	40.369	0.323975165	19.43850991		0.1493582	
0.34569	20.7	2.5980762	40.3981	0.324111959	19.44671752		0.1494208	
0.353372	21.16	2.6267851	40.4268	0.324247184	19.45483101		0.1494828	
0.361054	21.62	2.6551836	40.4552	0.324380891	19.46285346		0.1495441	
0.368736	22.08	2.6832816	40.4833	0.324513129	19.47078775		0.1496047	
0.376418	22.54	2.7110883	40.5111	0.324643944	19.47863664		0.1496647	
0.3841	23	2.7386128	40.5386	0.324773379	19.48640272		0.1497241	
0.391782	23.46	2.7658633	40.5659	0.324901475	19.49408847		0.1497828	
0.399464	23.92	2.792848	40.5928	0.325028271	19.50169625		0.149841	
0.407146	24.38	2.8195744	40.6196	0.325153805	19.5092283		0.1498985	
0.414828	24.84	2.8460499	40.646	0.325278113	19.51668676		0.1499556	
0.42251	25.3	2.8722813	40.6723	0.325401228	19.52407365		0.150012	
0.430192	25.76	2.8982753	40.6983	0.325523182	19.53139094		0.150068	
0.437874	26.22	2.9240383	40.724	0.325644008	19.53864048		0.1501234	
0.445556	26.68	2.9495762	40.7496	0.325763734	19.54582405		0.1501784	
0.453238	27.14	2.974895	40.7749	0.325882389	19.55294334		0.1502328	
0.46092	27.6	3	40.8	0.326	19.56			
							8.8844504	Total volume

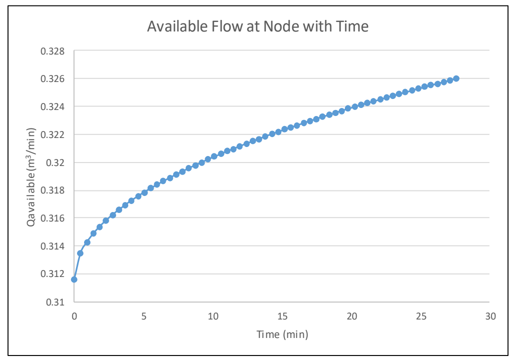


Figure 6.3 Determining the 15 minute peak flows for phase 2.

Phase 3 – Network full

The phase 3 calculations were performed for each supply period using Equation (4-5) and Equation (4-8) in order to determine the peak hour flows, refer to Table 6-14. The determined peak hour flows were also verified to ensure that these are lower than the designed maximum network demand capacity of 21.60 m^3/h, as presented in Table 6-15. These peak hour flows were then redistributed taking into account the base flows during supply and non-supply hours, resulting in a demand pattern that was converted from the CWS pattern to the IWS equivalent shape. The 15 minute peak flow values were subsequently determined, by interpolating the peak hour flow values. Figure 6.4 and Figure 6.5 present the Excel worksheet of the 15 minute PFs calculation and pattern conversion for the two supply periods per day scenario. By using Equation (4-8), the corresponding 15 minute peak factors were calculated. Refer to Table 6-12 to view the determined 15 minute PFs.

Table 6-14 Phase 3 calculations to determine the peak hour flows.

Determining the peak flows after the network filling process is complete:						
Phase 3:						
New IWS AADD		107.78	kL/day			
		4.49	m^3/h			
Supply Period 1:						
Total user water consumption during Phase 2		0.82	kL			
New AADD Remaining Supply Period 1		106.96	kL			
AADD Supply Period 1		4.46	m^3/h			
Start time =		4:25	End time =	10:30		
Demand multiplier		2.09	for	6.07	hour supply duration	
- Enter the time interval hours in the table below						
Supply Period 2:						
AADD supplied during Supply Period 1		75.88	kL			
AADD Remaining before Supply Period 2 begins		31.90	kL			
Return to Phase 1 and Phase 2 Calculations						
Total user water consumption during Phase 2		1.64	kL			
New AADD Remaining Supply Period 2		106.14	kL			
AADD Supply Period 2		4.42	m^3/h			
Start time =		17:10	End time =	21:45		
Demand multiplier		2.09	for	4.57	hour supply duration	
- Enter the time interval hours in the table below						
Supply Period 1			Supply Period 2			
Hour	PHF	Peak Water Demand (m^3/h)	Hour	PHF	Peak Water Demand (m^3/h)	
4	1.07	4.79	17	2.33	10.29	
5	1.28	5.72	18	2.29	10.11	
6	1.76	7.86	19	2.10	9.28	
7	2.52	11.21	20	1.76	7.80	
8	3.25	14.47	21	1.49	6.60	
9	3.58	15.96		0.00	0.00	
10	3.56	15.87		0.00	0.00	
	0.00	0.00		0.00	0.00	
	0.00	0.00		0.00	0.00	
	0.00	0.00		0.00	0.00	
	0.00	0.00		0.00	0.00	
	0.00	0.00		0.00	0.00	
	0.00	0.00		0.00	0.00	
	0.00	0.00		0.00	0.00	
	0.00	0.00		0.00	0.00	
	0.00	0.00		0.00	0.00	
	0.00	0.00		0.00	0.00	
	0.00	0.00		0.00	0.00	
	0.00	0.00		0.00	0.00	
	0.00	0.00		0.00	0.00	
	0.00	0.00		0.00	0.00	
	0.00	0.00		0.00	0.00	
	0.00	0.00		0.00	0.00	
	0.00	0.00		0.00	0.00	
	Total Water Volume =	75.88	m^3	Total Water Volume =	44.08	m^3
	New Base Flows =	22.76	m^3	New Base Flows =	13.22	m^3
		3.75	m^3/h		2.89	m^3/h

Table 6-15 Check of the phase 3 peak hour flow values.

Check: Are the peak water demand values within the range of the max. network demand?

Supply Period 1:

WDN design demand		21.6	m^3/h
75.88	m^3 supplied in	6.07	hours

Therefore:

70.40 % of the IWS water demand can be supplied during the supply period 1

Increase supply duration, increase network capacity and/or decrease water demand

Supply Period 2:

WDN design demand		21.60	m^3/h
44.08	m^3 supplied in	4.57	hours

Therefore:

40.90 % of the IWS water demand can be supplied during the supply period 2

Increase supply duration, increase network capacity and/or decrease water demand

Supply Period 1	Adjusted Peak Water Demand	Supply Period 2	Adjusted Peak Water Demand
4.79	4.79	10.29	10.29
5.72	5.72	10.11	10.11
7.86	7.86	9.28	9.28
11.21	11.21	7.80	7.80
14.47	14.47	6.60	6.60
15.96	15.96	0.00	0.00
15.87	15.87	0.00	0.00
0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.00

Recalculated Base Flows		Recalculated Base Flows	
22.76	m^3	13.22	m^3
3.75	m^3/h	2.89	m^3/h

Phase 3 - Full Distribution Network

Peak Water Demand (After the Filling Process)

New base flows (m^3/h)	3.75		
Time interval for CWS data (hrs)	7	IWS time interval required (hrs)	6.07
Value of last peak flow from phase 2 (m^3/h)	21.39	Total plotting interval	6.07

Hrs	IWS Peak flows (m^3/h)	Peak flows (without base flows)	redistribute base flows	Interval redistribution(hrs)	Minutes equivalent	Phase 3 Peak flows (m^3/h)
				0.867	0	21
1	5	1	5	0.867	52	5
2	6	2	6	1.734	104	6
3	8	4	8	2.601	156	8
4	11	7	11	3.469	208	11
5	14	11	14	4.336	260	14
6	16	12	16	5.203	312	16
7	16	12	16	6.070	364	16
8		0	0	6.070	364	0
9		0	0	6.070	364	0
10		0	0	6.070	364	0
11		0	0	6.070	364	0
12		0	0	6.070	364	0
13		0	0	6.070	364	0
14		0	0	6.070	364	0
15		0	0	6.070	364	0
16		0	0	6.070	364	0
17		0	0	6.070	364	0
18		0	0	6.070	364	0
19		0	0	6.070	364	0
20		0	0	6.070	364	0
21		0	0	6.070	364	0
22		0	0	6.070	364	0
23		0	0	6.070	364	0
24		0	0	6.070	364	0

Sum of peak flows for the column	Sum of peak flows for the column	Sum of peak flows for the column
76	50	76

Base flow for redistribution	3.75	m^3/h
% of Base flow for redistribution	100	
Base flow during supply period	3.75	m^3/h
Base flow during non-supply period	0	m^3/h

Consider the Supply Area Slope:

Area topography	% redistribution of base flows	
Flat	100	% redistributed during supply period
	0	% redistributed during off supply periods
Hilly/Moderate	50	% redistributed during supply period
	50	% redistributed during off supply periods
Steep	90	% redistributed during supply period
	10	% redistributed during off supply periods

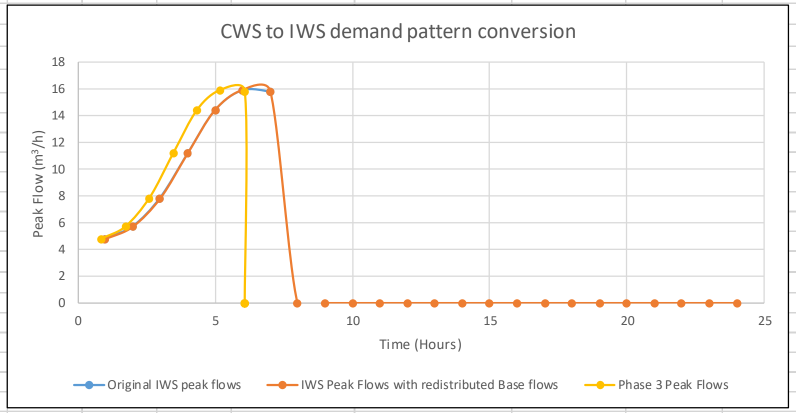


Figure 6-4 Phase 3 peak flows calculations and CWS pattern conversion for the first supply period.

Phase 3 - Full Distribution Network

		Peak Water Demand (After the Filling Process)				
		New base flows (m^3/h)	2.89			
		Time interval for CWS data (hrs)	5	IWS time interval required (hrs)	4.57	
		Value of last peak flow from phase 2 (m^3/h)	21.39	Total plotting interval	4.57	
				Interval redistribution(hrs)	Minutes equivalent	Phase 3 Peak flows (m^3/h)
Hrs	IWS Peak flows (m^3/h)	Peak flows (without base flows)	redistribute base flows	0.914	0	21
1	10	7	10	0.914	55	10
2	10	7	10	1.828	110	10
3	9	6	9	2.742	165	9
4	8	5	8	3.656	219	8
5	7	4	7	4.570	274	7
6		0	0	4.570	274	0
7		0	0	4.570	274	0
8		0	0	4.570	274	0
9		0	0	4.570	274	0
10		0	0	4.570	274	0
11		0	0	4.570	274	0
12		0	0	4.570	274	0
13		0	0	4.570	274	0
14		0	0	4.570	274	0
15		0	0	4.570	274	0
16		0	0	4.570	274	0
17		0	0	4.570	274	0
18		0	0	4.570	274	0
19		0	0	4.570	274	0
20		0	0	4.570	274	0
21		0	0	4.570	274	0
22		0	0	4.570	274	0
23		0	0	4.570	274	0
24		0	0	4.570	274	0

Sum of peak flows for the column	Sum of peak flows for the column	Sum of peak flows for the column
44	30	44

Base flow for redistribution	2.89	m^3/h
% of Base flow for redistribution	100	
Base flow during supply period	2.89	m^3/h
Base flow during non-supply period	0	m^3/h

Consider the Supply Area Slope:

Area topography	% redistribution of base flows	
Flat	100	% redistributed during supply period
	0	% redistributed during off supply periods
Hilly/Moderate	50	% redistributed during supply period
	50	% redistributed during off supply periods
Steep	90	% redistributed during supply period
	10	% redistributed during off supply periods

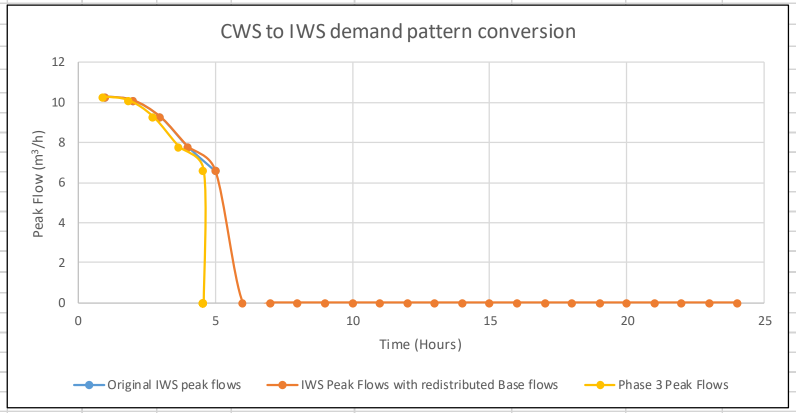


Figure 6-5 Phase 3 peak flows calculations and CWS pattern conversion for the second supply period.

IWS diurnal water demand patterns

The 15 minute PFs in Table 6-12 were used to plot the predicted IWS diurnal water demand patterns for the three IWS supply period scenarios considered, based on the chosen CWS demand pattern, supply area characteristics and WDS characteristics. Refer to Figure 6.6, Figure 6.7 and Figure 6.8 for the derived IWS diurnal water demand patterns for the KaNyamazane Toad Street supply area for the one supply period per day scenario, the two supply periods per day scenario and the three supply periods per day scenario, respectively.

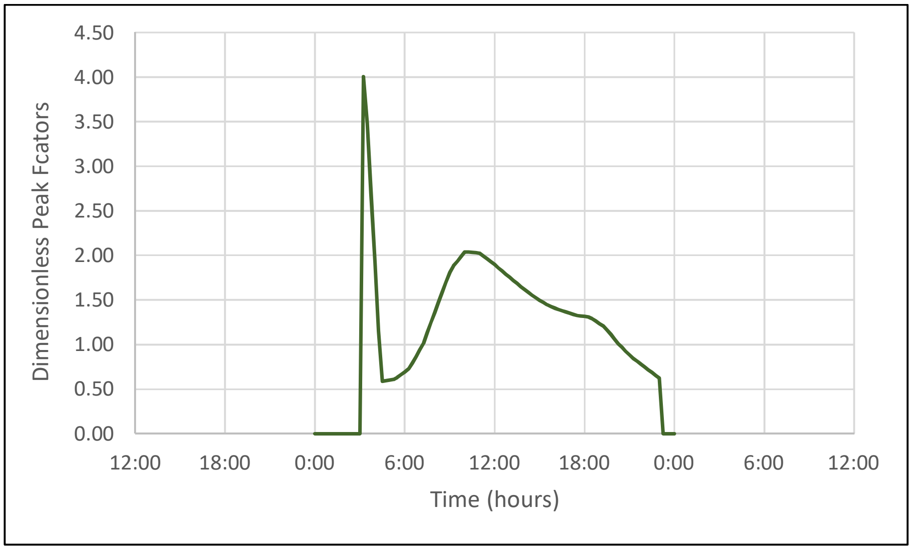


Figure 6.6 Predicted IWS demand pattern for one supply period per day for 27/7/2019.

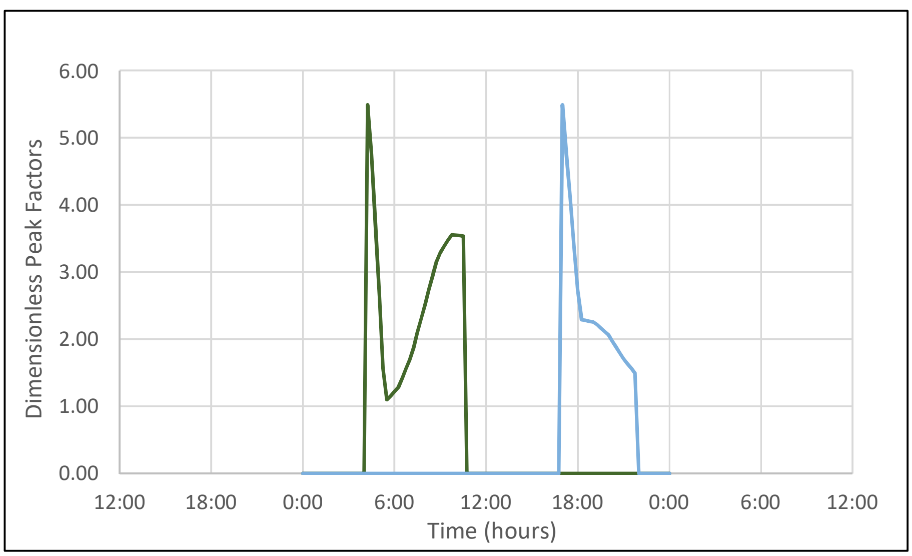


Figure 6.7 Predicted IWS demand pattern for two supply periods per day for 27/7/2019.

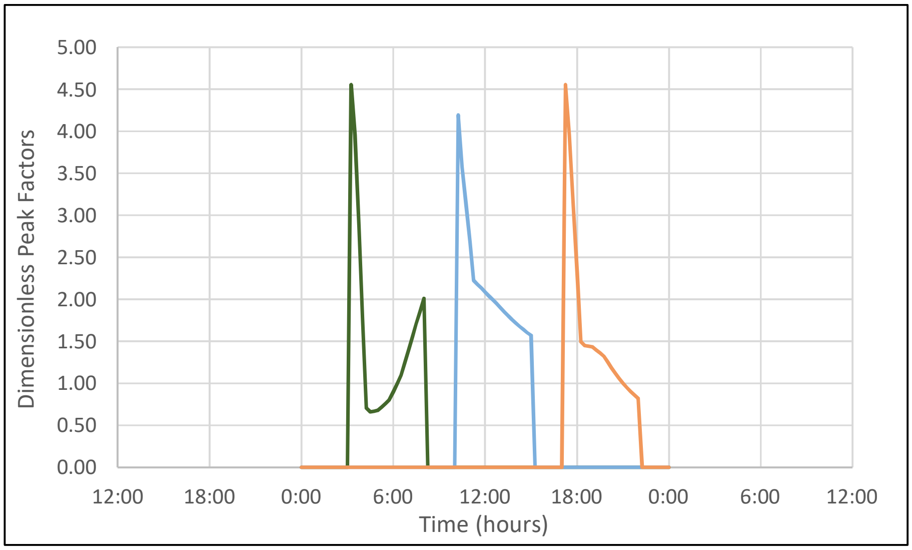


Figure 6.8 Predicted IWS demand pattern for three supply periods per day for 27/7/2019.

6.2.2 KaNyamazane diurnal water demand patterns

From the logged data of the KaNyamazane Toad Street supply area, for the period between 22/07/2019 and 25/08/2019, diurnal water demand patterns were derived for each day of the record period as described in Section 5.4. The demand patterns for the 5 minute, 15 minute and 1 hour time intervals were determined for each day, in order to determine the most suitable time interval for the derivation of the PFs. By comparing the three types of diurnal demand patterns, it was concluded that the diurnal demand patterns derived using the 15 minute peak factors (PF15min) were the most suitable for the comparison phase of the study. The demand patterns derived using the 1 hour peak factors did not effectively represent the filling spike as shown in Figure 6.9. To consolidate the calculations, the 15 minute time interval was considered to be more manageable than the 5 minute time interval due to the number of intervals per day which would have to be considered in the calculations (a set of 288 values for the 5 minute interval compared to 96 values for the 15 minute interval). Therefore, the 15 minute PF diurnal water demand patterns were used for the comparison with the IWS demand pattern model.

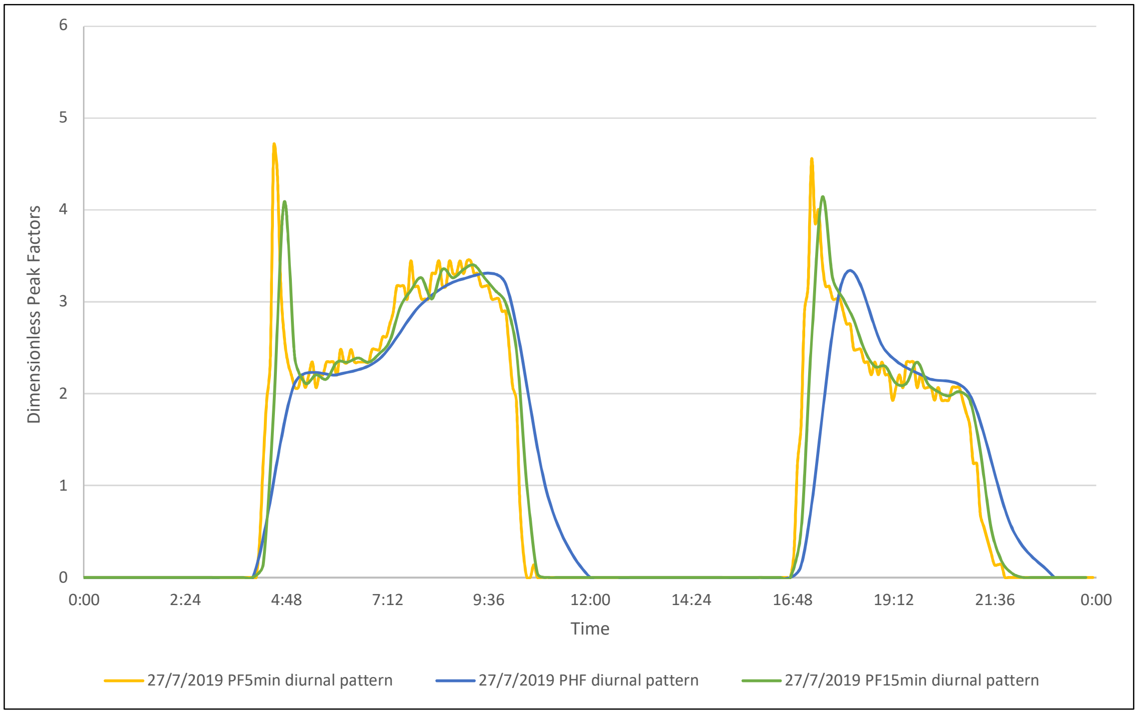


Figure 6.9 Comparison of the 5 minute, 15 minute and 1 hour PF diurnal demand patterns derived from actual data.

For the derivation of the diurnal demand patterns, an ADD of 208.93 kL/day was calculated for the entire record period. By using Equation (4-1), the PFs for each day were determined. The diurnal demand pattern for the 27/7/2019 was chosen as a representative pattern for the comparison; see Figure 6.9 for the 15 minute PF diurnal water demand pattern. For this day, there were two supply periods with a total supply duration of 11.5 hours per day. The first supply period started at 4:15 and ended at 10:45, then the second supply period started at 17:00 and ended at 22:00. The highest 15 minute PF determined was 4.13 (associated with the second supply period initial demand spike), followed by a 15 minute PF of 4.09 (associated with the first supply period initial demand spike). Refer to Table 6-16 for the calculated 15 minute PFs for the date 27/7/2019.

Table 6-16 Calculated 15 minute peak factors from actual data for 27/7/2019.

Time	PF15min	Time	PF15min	Time	PF15min
0:00	0.00	8:15	3.03	16:30	0.00
0:15	0.00	8:30	3.35	16:45	0.00
0:30	0.00	8:45	3.26	17:00	0.51
0:45	0.00	9:00	3.35	17:15	2.57
1:00	0.00	9:15	3.40	17:30	4.13
1:15	0.00	9:30	3.26	17:45	3.26
1:30	0.00	9:45	3.12	18:00	3.03
1:45	0.00	10:00	2.99	18:15	2.80
2:00	0.00	10:15	2.48	18:30	2.48
2:15	0.00	10:30	1.01	18:45	2.30
2:30	0.00	10:45	0.05	19:00	2.30
2:45	0.00	11:00	0.00	19:15	2.11
3:00	0.00	11:15	0.00	19:30	2.11
3:15	0.00	11:30	0.00	19:45	2.34
3:30	0.00	11:45	0.00	20:00	2.11
3:45	0.00	12:00	0.00	20:15	2.02
4:00	0.00	12:15	0.00	20:30	1.98
4:15	0.14	12:30	0.00	20:45	2.02
4:30	1.84	12:45	0.00	21:00	1.93
4:45	4.09	13:00	0.00	21:15	1.38
5:00	2.39	13:15	0.00	21:30	0.55
5:15	2.11	13:30	0.00	21:45	0.18
5:30	2.21	13:45	0.00	22:00	0.05
5:45	2.16	14:00	0.00	22:15	0.00
6:00	2.34	14:15	0.00	22:30	0.00
6:15	2.34	14:30	0.00	22:45	0.00
6:30	2.39	14:45	0.00	23:00	0.00
6:45	2.34	15:00	0.00	23:15	0.00
7:00	2.44	15:15	0.00	23:30	0.00
7:15	2.57	15:30	0.00	23:45	0.00
7:30	2.94	15:45	0.00	0:00	
7:45	3.12	16:00	0.00		
8:00	3.26	16:15	0.00		

6.2.3 Model validation and calibration

The actual 15 minute PF diurnal water demand pattern for KaNyamazane Toad Street, as presented in Figure 6.9, was compared to the predicted equivalent IWS diurnal water demand pattern model for the date 27/7/2019. The first comparison of the actual demand pattern (blue curve in Figure 6.10) with the predicted IWS demand pattern model revealed an overestimation of the initial water demand spike, as presented in Figure 6.10. However, the shape of the two patterns was observed to be very similar. The PFs calculated for the predicted demand patterns for each comparison step are presented in Table 6-17.

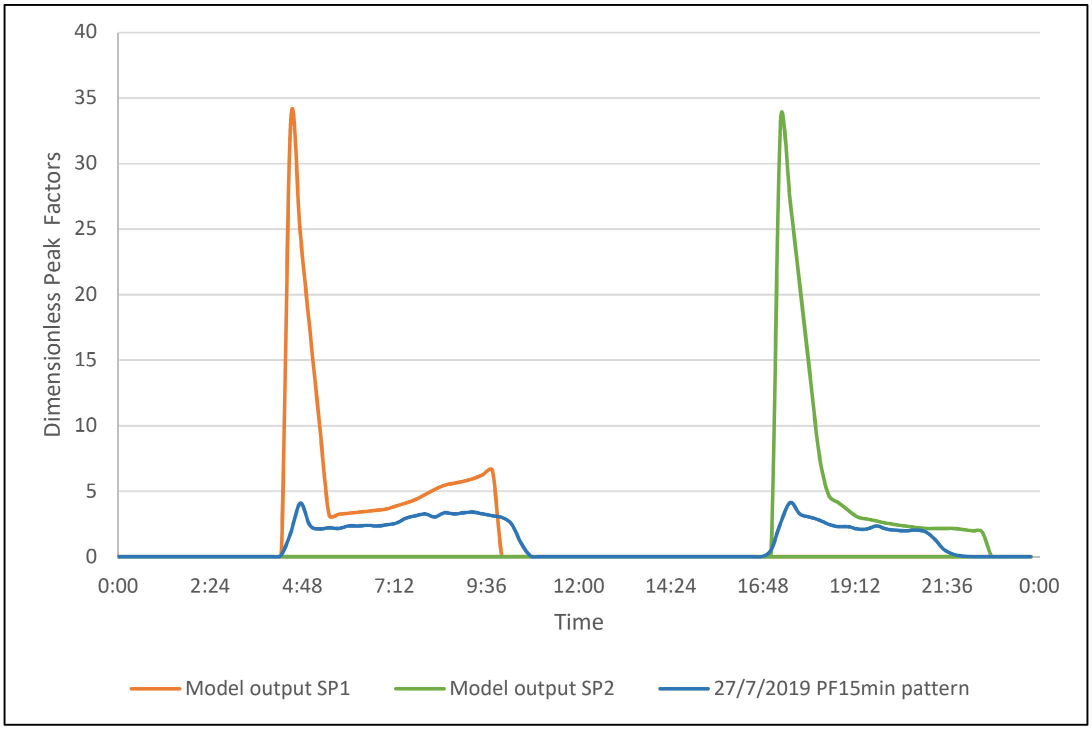


Figure 6.10 First comparison of the IWS model prediction with the actual IWS demand pattern.

Based on this first comparison, the predicted IWS water demand pattern model was calibrated for a more accurate prediction of the actual demand pattern. Four calibration steps were followed. The calibration of the model was an iterative process, whereby the original assumptions made during the development of the model were revisited.

Table 6-17 Calculated 15 minute PFs for the predicted demand patterns for each comparison and calibration step.

First Model Prediction					
Time	PF15min	Time	PF15min	Time	PF15min
0:00	0.00	8:15	5.13	16:30	0.00
0:15	0.00	8:30	5.46	16:45	0.00
0:30	0.00	8:45	5.61	17:00	0.00
0:45	0.00	9:00	5.76	17:15	33.23
1:00	0.00	9:15	5.96	17:30	27.16
1:15	0.00	9:30	6.26	17:45	20.74
1:30	0.00	9:45	6.57	18:00	14.32
1:45	0.00	10:00	0.00	18:15	7.90
2:00	0.00	10:15	0.00	18:30	4.71
2:15	0.00	10:30	0.00	18:45	4.14
2:30	0.00	10:45	0.00	19:00	3.57
2:45	0.00	11:00	0.00	19:15	3.04
3:00	0.00	11:15	0.00	19:30	2.88
3:15	0.00	11:30	0.00	19:45	2.72
3:30	0.00	11:45	0.00	20:00	2.57
3:45	0.00	12:00	0.00	20:15	2.45
4:00	0.00	12:15	0.00	20:30	2.34
4:15	0.00	12:30	0.00	20:45	2.24
4:30	33.23	12:45	0.00	21:00	2.16
4:45	24.62	13:00	0.00	21:15	2.16
5:00	17.28	13:15	0.00	21:30	2.16
5:15	10.05	13:30	0.00	21:45	2.16
5:30	3.17	13:45	0.00	22:00	2.08
5:45	3.24	14:00	0.00	22:15	1.97
6:00	3.32	14:15	0.00	22:30	1.87
6:15	3.40	14:30	0.00	22:45	0.00
6:30	3.47	14:45	0.00	23:00	0.00
6:45	3.55	15:00	0.00	23:15	0.00
7:00	3.65	15:15	0.00	23:30	0.00
7:15	3.88	15:30	0.00	23:45	0.00
7:30	4.11	15:45	0.00	0:00	0.00
7:45	4.36	16:00	0.00		
8:00	4.74	16:15	0.00		

First Calibration Step					
Time	PF15min	Time	PF15min	Time	PF15min
0:00	0.00	8:15	4.36	16:30	0.00
0:15	0.00	8:30	4.69	16:45	0.00
0:30	0.00	8:45	5.02	17:00	0.00
0:45	0.00	9:00	5.35	17:15	1.42
1:00	0.00	9:15	5.52	17:30	13.03
1:15	0.00	9:30	5.66	17:45	11.26
1:30	0.00	9:45	5.79	18:00	9.50
1:45	0.00	10:00	0.00	18:15	7.73
2:00	0.00	10:15	0.00	18:30	6.50
2:15	0.00	10:30	0.00	18:45	5.71
2:30	0.00	10:45	0.00	19:00	4.92
2:45	0.00	11:00	0.00	19:15	4.19
3:00	0.00	11:15	0.00	19:30	3.98
3:15	0.00	11:30	0.00	19:45	3.76
3:30	0.00	11:45	0.00	20:00	3.54
3:45	0.00	12:00	0.00	20:15	3.38
4:00	0.00	12:15	0.00	20:30	3.23
4:15	0.00	12:30	0.00	20:45	3.09
4:30	1.42	12:45	0.00	21:00	2.98
4:45	12.53	13:00	0.00	21:15	2.98
5:00	9.81	13:15	0.00	21:30	2.98
5:15	7.10	13:30	0.00	21:45	2.98
5:30	4.38	13:45	0.00	22:00	2.86
5:45	3.20	14:00	0.00	22:15	2.72
6:00	3.27	14:15	0.00	22:30	2.58
6:15	3.33	14:30	0.00	22:45	0.00
6:30	3.40	14:45	0.00	23:00	0.00
6:45	3.46	15:00	0.00	23:15	0.00
7:00	3.53	15:15	0.00	23:30	0.00
7:15	3.59	15:30	0.00	23:45	0.00
7:30	3.75	15:45	0.00	0:00	0.00
7:45	3.94	16:00	0.00		
8:00	4.14	16:15	0.00		

Second Calibration Step					
Time	PF15min	Time	PF15min	Time	PF15min
0:00	0.00	8:15	4.18	16:30	0.00
0:15	0.00	8:30	4.49	16:45	0.00
0:30	0.00	8:45	4.81	17:00	0.00
0:45	0.00	9:00	5.12	17:15	6.70
1:00	0.00	9:15	5.29	17:30	5.49
1:15	0.00	9:30	5.42	17:45	5.93
1:30	0.00	9:45	5.54	18:00	6.37
1:45	0.00	10:00	0.00	18:15	6.82
2:00	0.00	10:15	0.00	18:30	6.57
2:15	0.00	10:30	0.00	18:45	5.77
2:30	0.00	10:45	0.00	19:00	4.97
2:45	0.00	11:00	0.00	19:15	4.23
3:00	0.00	11:15	0.00	19:30	4.02
3:15	0.00	11:30	0.00	19:45	3.80
3:30	0.00	11:45	0.00	20:00	3.58
3:45	0.00	12:00	0.00	20:15	3.41
4:00	0.00	12:15	0.00	20:30	3.27
4:15	0.00	12:30	0.00	20:45	3.12
4:30	6.70	12:45	0.00	21:00	3.01
4:45	5.49	13:00	0.00	21:15	3.01
5:00	4.78	13:15	0.00	21:30	3.01
5:15	4.06	13:30	0.00	21:45	3.01
5:30	3.35	13:45	0.00	22:00	2.89
5:45	3.06	14:00	0.00	22:15	2.75
6:00	3.13	14:15	0.00	22:30	2.60
6:15	3.19	14:30	0.00	22:45	0.00
6:30	3.25	14:45	0.00	23:00	0.00
6:45	3.32	15:00	0.00	23:15	0.00
7:00	3.38	15:15	0.00	23:30	0.00
7:15	3.44	15:30	0.00	23:45	0.00
7:30	3.59	15:45	0.00	0:00	0.00
7:45	3.78	16:00	0.00		
8:00	3.96	16:15	0.00		

Third Calibration Step					
Time	PF15min	Time	PF15min	Time	PF15min
0:00	0.00	8:15	2.71	16:30	0.00
0:15	0.00	8:30	2.91	16:45	0.00
0:30	0.00	8:45	3.11	17:00	0.00
0:45	0.00	9:00	3.32	17:15	4.34
1:00	0.00	9:15	3.43	17:30	3.56
1:15	0.00	9:30	3.51	17:45	3.84
1:30	0.00	9:45	3.59	18:00	4.13
1:45	0.00	10:00	0.00	18:15	4.42
2:00	0.00	10:15	0.00	18:30	4.26
2:15	0.00	10:30	0.00	18:45	3.74
2:30	0.00	10:45	0.00	19:00	3.22
2:45	0.00	11:00	0.00	19:15	2.74
3:00	0.00	11:15	0.00	19:30	2.60
3:15	0.00	11:30	0.00	19:45	2.46
3:30	0.00	11:45	0.00	20:00	2.32
3:45	0.00	12:00	0.00	20:15	2.21
4:00	0.00	12:15	0.00	20:30	2.12
4:15	0.00	12:30	0.00	20:45	2.02
4:30	4.34	12:45	0.00	21:00	1.95
4:45	3.56	13:00	0.00	21:15	1.95
5:00	3.09	13:15	0.00	21:30	1.95
5:15	2.63	13:30	0.00	21:45	1.95
5:30	2.17	13:45	0.00	22:00	1.87
5:45	1.99	14:00	0.00	22:15	1.78
6:00	2.03	14:15	0.00	22:30	1.69
6:15	2.07	14:30	0.00	22:45	0.00
6:30	2.11	14:45	0.00	23:00	0.00
6:45	2.15	15:00	0.00	23:15	0.00
7:00	2.19	15:15	0.00	23:30	0.00
7:15	2.23	15:30	0.00	23:45	0.00
7:30	2.32	15:45	0.00	0:00	0.00
7:45	2.45	16:00	0.00		
8:00	2.57	16:15	0.00		

As a first step of calibration, it was noted that the use of Equation (4-9) and Equation (4-10) in the phase 1 calculations resulted in an overestimation of the water volume during the filling of the bulk supply pipeline. Therefore, the bulk supply pipeline filling calculations were adjusted to reflect the bulk supply pipeline filling volume as determined by using the Riemann sum Equation (4-2). Refer to Figure 6.11 to view the effects of this calibration step on the initial model prediction.

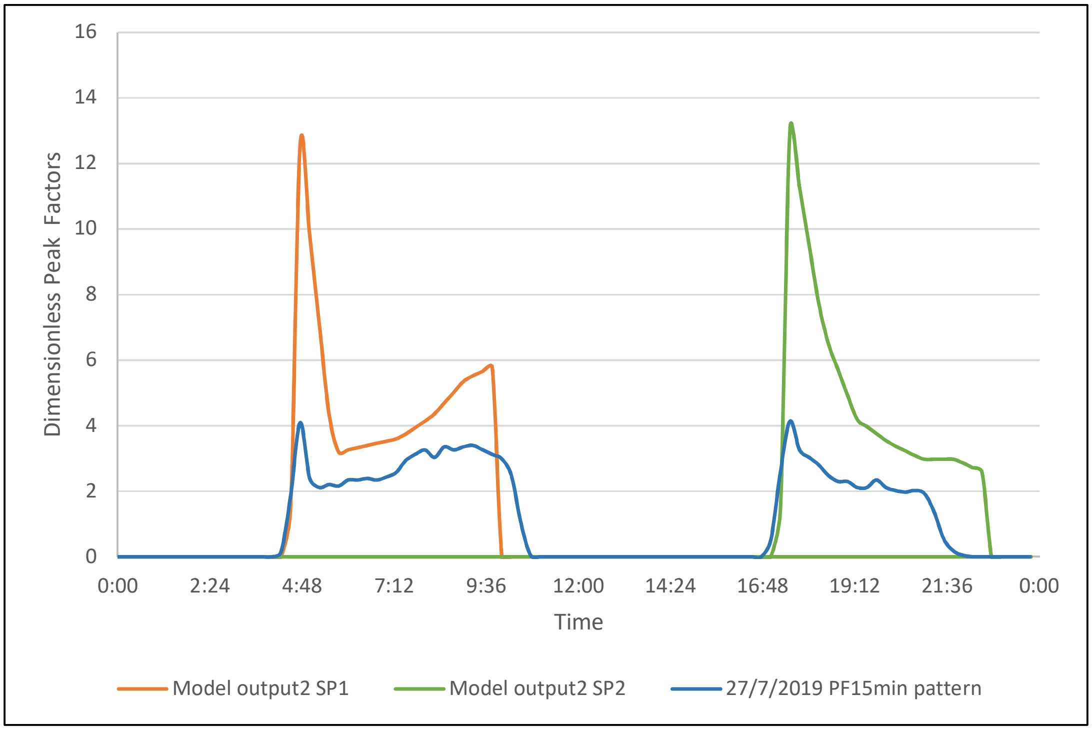


Figure 6.11 Comparison of the model prediction with the actual demand pattern after the first calibration step.

During the second calibration step, phase 2 was modified to better represent the network filling phase. In the phase 2 calculation, the assumption that the user water demand during the network filling process formed part of the available flow rate for network filling (Q_{max}) at the focus node was removed. Therefore Q_{max} represents solely the flows in the pipes. Due to the usage of water during the filling of the network, the network filling rate was found to be lower than the maximum allowable network filling rate determined using the maximum allowable flow rate for the pipe. Furthermore, the user water demand during the network filling phase was assumed to not change significantly from the water demand during CWS conditions for that specific time interval. This is because, during IWS conditions, the users will attempt to draw as much water as possible, unless the users are specifically asked not to use the water until the network is full. The users may not always be available to collect water from the taps at the onset of the supply period, therefore the users may leave the taps open in order to collect as much water as possible when the water becomes available during the supply period. This collection of water by the users that are advantaged in terms of position and topography, prolongs the rate at which the WDN fills. The phase 2 calculations were modified and calibrated to better model this scenario, and by doing so the IWS model pattern became a more accurate prediction of the actual demand pattern, refer to Figure 6.12.

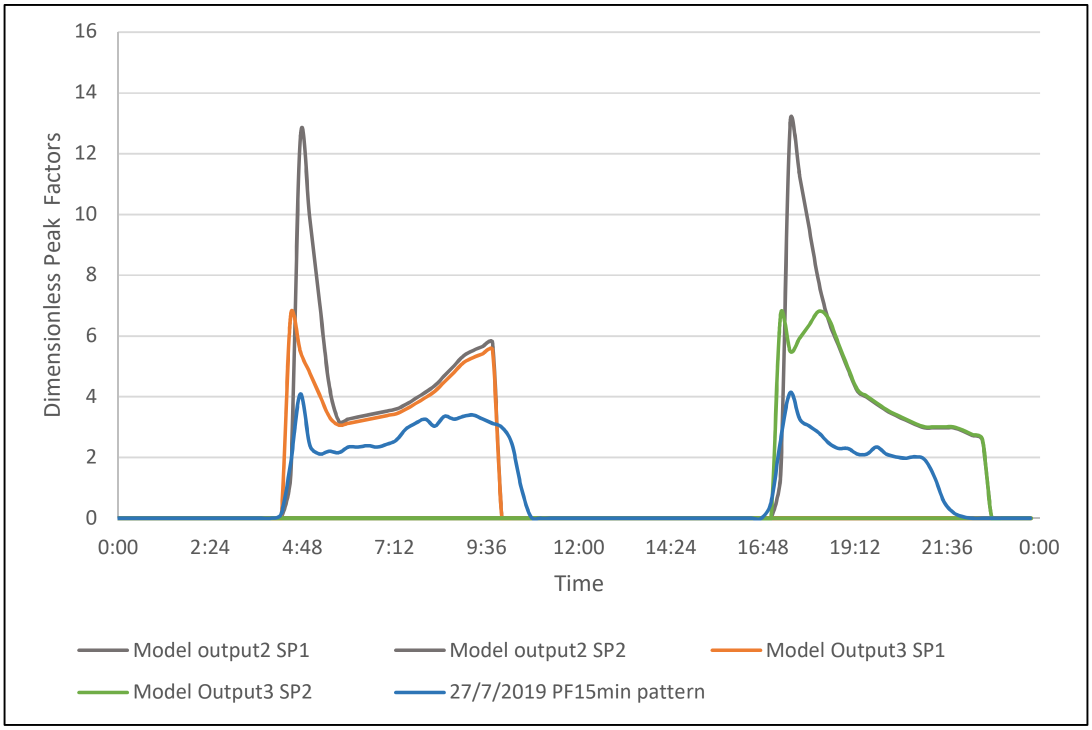


Figure 6.12 The result of the second calibration step of the model prediction.

After the second calibration step, there was still a difference in the volume as predicted by the model and the actual demand pattern. After verifying the accuracy of the CWS AADD value obtained from the WADISO model it was discovered that the calculation of the IWS PFs according to Equation (4-8) resulted in an overestimation. The IWS PFs were calculated using the IWS AADD. The IWS AADD, in turn was determined by applying a reduction factor (SDDRF) to the CWS AADD, with the assumption that the AADD decreases when the supply duration decreases due to there being reduced time for water consumption. Therefore, the overestimation could be explained by the overestimation of the assumed reduction factors (SDDRF). There is a lack of research in the field of determining the percentage by which the AADD decreases when the supply duration is shortened. Therefore, an assumption of an AADD reduction factor (SDDRF) was made in the model according to Equation (4-6) and Equation (4-7). For the prediction of the 27/7/2019 IWS demand pattern the SDDRF was adjusted from 48% to 73% for the two supply periods per day scenario. This means that when there is 11.5 hours of water supply per day, the AADD is reduced by 27% instead of 52%. After this step of calibration the model produced a more accurate prediction according to Figure 6.13.

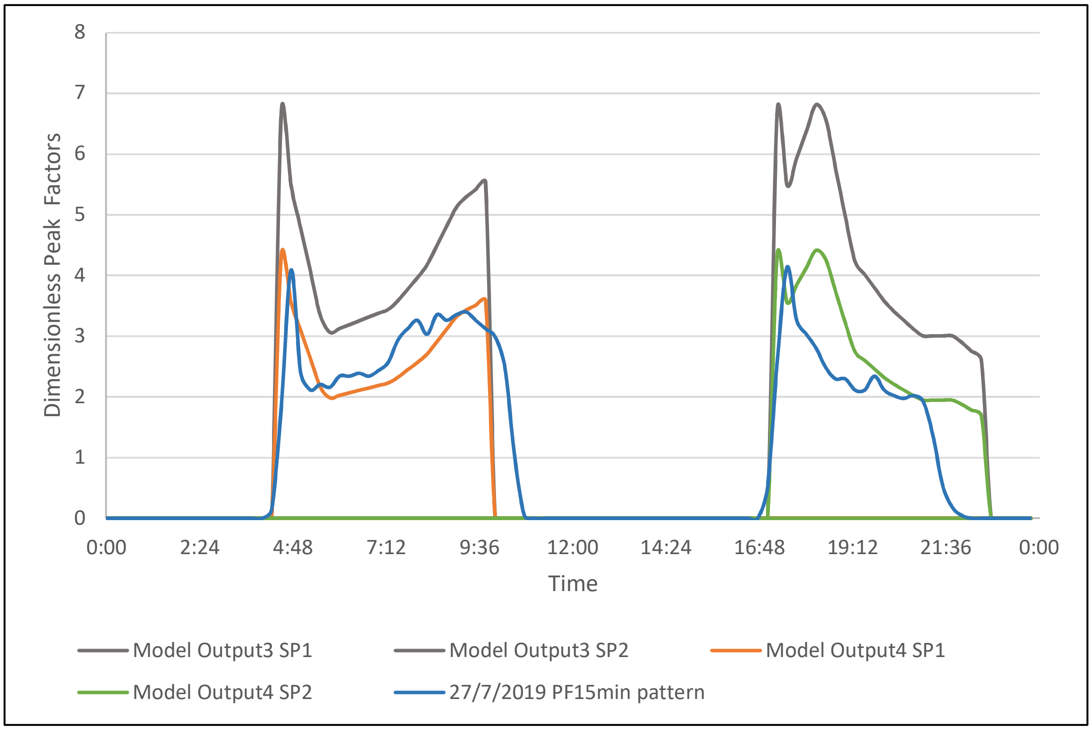


Figure 6.13 Model prediction after the third calibration step.

In the last calibration step the phase 3 assumption that the IWS AADD during the second supply period will be lower than the IWS AADD for the first supply period was noticed to cause an underestimation of the IWS AADD during the second supply period. It was discovered that by assuming the IWS AADD to be equally distributed between supply periods of similar duration, produced results that were more representative of the collected field data. The demand multiplier used to determine the IWS PFs during phase 3 for each supply period, was discovered to be based on the overall supply duration per day instead of the supply duration for each supply period. Therefore, the same demand multiplier was applied to every supply period. After these adjustments were made, there was an improvement in the prediction accuracy of the model. The final model prediction of the actual IWS diurnal demand pattern is presented in Figure 6.14.

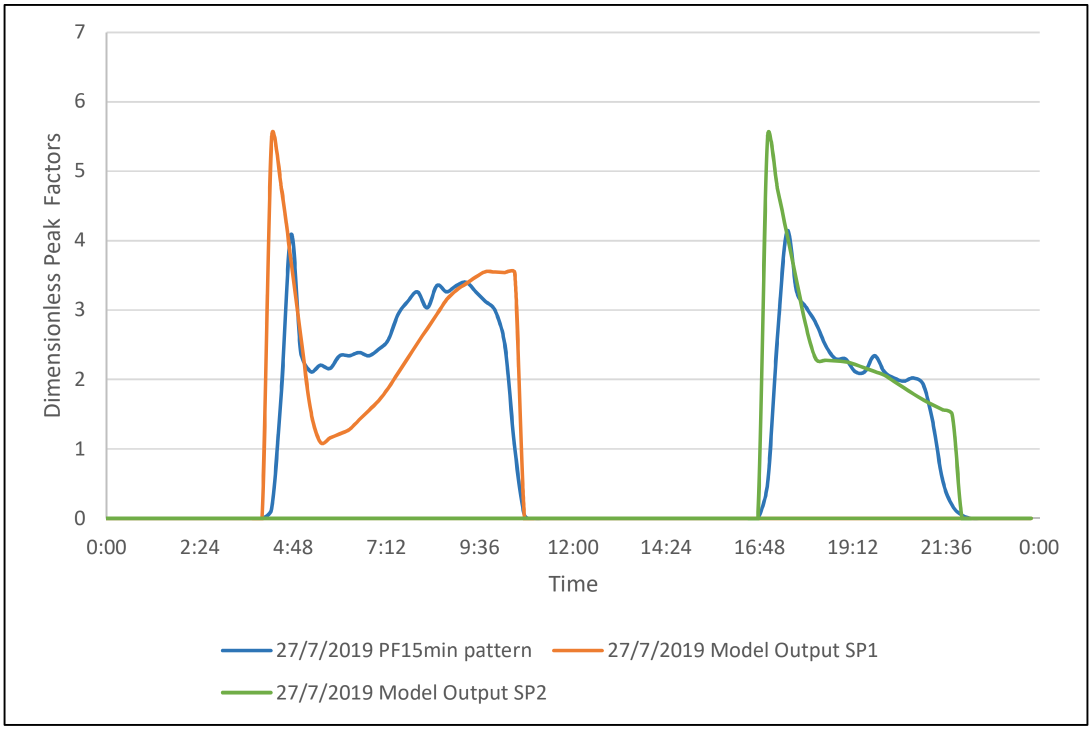


Figure 6.14 The final result of the model calibration.

For the final comparison of the predicted IWS diurnal water demand pattern, it was found that there was a high similarity between the shape of the model prediction and the shape of the actual IWS water demand pattern for the KaNyamazane Toad Street supply area. This result was based on the Kolmogorov-Smirnov goodness of fit test. The test was conducted using the 15 minute peak factors for the two supply periods per day scenario as presented in Table 6-12, and the 15 minute peak factors obtained from the actual field data for 27/7/2019, as presented in Table 6-16. Cumulative distribution functions (CDF) were determined for each pattern using the relevant 15 minute peak factors. The K-S statistic value (D) was calculated as the maximum distance between the two CDFs, as illustrated in Figure 6.15. A K-S statistic value of 0.07 was determined, using Equation (5-2), as well as the critical K-S value (d_α) of 0.14 using Table 5-2. The K-S statistic value was found to be less than the critical K-S value, which means that there is no significant difference between the model prediction and the actual IWS water demand pattern, at a level of significance of 5%.The lower the K-S statistic value, the higher the level of similarity is between the two patterns. Refer also to Table B.1 in Appendix B to view the calculations for the K-S goodness of fit test.

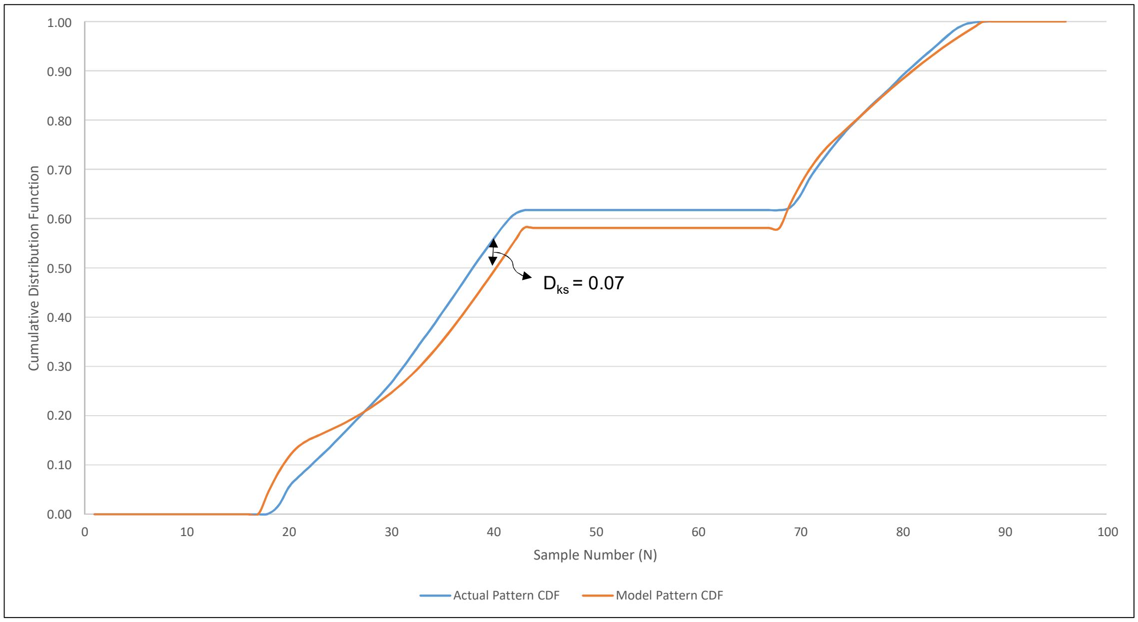


Figure 6.15 Comparison of the CDFs for the model and the actual demand patterns.

6.2.4 Discussion of the model results

The purpose of the IWS diurnal water demand pattern model developed in this study was not to be an exact prediction of the domestic water demand during IWS conditions. Instead, the development of the model was an initial attempt to understand and define the typical form of a diurnal demand pattern of residential areas subjected to IWS conditions, and to investigate how these patterns can potentially be derived in cases when the logged water consumption data is not available.

From the results presented it can be noted that it is possible to predict the domestic water demand of a supply area by using certain overarching hydraulic principles. In this study these were related to the WDS filling process associated with IWS conditions, together with the identification and correlation of the important parameters that influence WDSs. By using this novel approach, it was found that there was a high similarity between the shape of the model prediction and the shape of the actual IWS water demand pattern for the KaNyamazane Toad Street supply area. However, according to Van Zyl et al. (2007) there will always be a level of variability and uncertainty during the process of determining the water demand of an area.

The assumptions in the development of the model resulted in an IWS diurnal water demand pattern consisting of three phases which determine the shape of the pattern. Phase 1 was related to a portion of the initial water demand spike, phase 2 was related to both the initial water demand spike and the reticulation network filling, and phase 3 was related to the water demand characteristics after the filling process was completed and when the shape of the IWS demand pattern becomes similar to the shape of the CWS pattern.

From the results of the derivation of the actual water demand patterns in Section 6.2.2, it was observed that a diurnal water demand pattern related to IWS consists of an initial spike in the water demand at the beginning of every supply period. In this study, the initial spike was assumed to be caused by the filling of the bulk supply pipeline and the filling of the reticulation network, which is consistent with the research by Mckenzie et al. (2014). From the mathematical investigation in this study, it was discovered that the initial filling spike probably consisted of both the filling of the bulk supply pipeline and the filling of the reticulation network. The filling of the reticulation network was found to contribute the main portion of the value of the magnitude of the spike.

The phase 1 and phase 2 assumptions in the development of the model, provide initial insight regarding the magnitude of the initial spike in the water demand. The definition of the exact relationship or factors that determine the magnitude and duration of the initial spike in the demand can be the subject for further research. Furthermore, the assumption that the shape of the IWS diurnal demand pattern is similar to the CWS diurnal water demand pattern after the reticulation network filling time was proven to have some level of validity. Even though the matching of the shapes of the predicted model and the actual demand pattern was not exact, the assumption provided a reasonable estimation of the actual IWS demand pattern shape. This provides clearer insight into the typical shape of a diurnal demand pattern associated with IWS, and how it potentially relates to the CWS diurnal demand pattern.

Practically, the model developed in this study may assist in the analysis of the capacity of existing WDSs for the water scarcity scenario. The diurnal water demand patterns are typically used to determine the water needs of the people in a supply area, and to investigate the hydraulic capacity of the existing or designed WDS to meet that need. Normally, the steady state analysis and the extended period simulation (EPS) features in modelling software such as WADISO or EPANET are used for this purpose. The extended period simulation analysis requires the diurnal demand pattern related to the WDN being analysed as input for the computation of the flows, pressures and operational attributes of the system over an extended period of time. The modelled IWS diurnal demand pattern can be used as input for the extended period simulation analysis of an existing WDN system to verify what the WDS response would be should IWS be implemented.

By using the model and following the model calculation procedure, various IWS demand pattern scenarios for a supply area can be derived or predicted to use as input in the EPS analysis. The design engineers can investigate the effects of different hydraulic demand scenarios associated with IWS on an existing WDS, based on the relationships of the important influence parameters identified in this study. For example, it would be possible to investigate the effects of increasing or decreasing the number of supply periods and the supply duration per day on the peak factors, water demand and on the flows and pressures in the WDS. This could assist the design engineers to determine whether the network operated under IWS would satisfy the consumer demand. Moreover, design engineers will be in a better position to plan the distribution of water more effectively during periods of water scarcity.

7 Conclusion and Recommendations

7.1 Conclusion

The use of IWS as a method of water demand control is still prevalent worldwide, even though there are so many negative consequences related to this water supply method. Therefore, there is some benefit to understanding these systems, in order to possibly improve the WDS performance and LOS experienced by the consumers during conditions of IWS. One of the most important steps in the planning, analysis and design of WDNs to achieve these objectives, is determining the peak water demand, the peak factors and the water demand patterns for a supply area. These parameters are key inputs in the process of determining the WDN capacity requirements.

The main purpose of the current study was to develop a new model to predict the diurnal water demand pattern for residential areas subjected to IWS in South Africa. This was achieved by taking into account the theory related to the WDS filling process and the crucial parameters that impact WDSs during intermittent supply. There has been little research done related to diurnal demand patterns associated with IWS. Due to the lack of existing research, a novel method of investigation was developed to achieve the study objective.

The novel method consisted of a non-empirical investigation and an empirical investigation. The non-empirical investigation included conducting a literature review of the key concepts related to water demand and WDSs to form a theory base for the model development. A parameters investigation was conducted to identify the most influential parameters that impact water demand and WDSs. These influential parameters became the building blocks of the model. The empirical investigation comprised the validation of the developed model. This was achieved by comparing the proposed hypothetical model, with actual diurnal demand patterns obtained from logged water demand data for a residential area subjected to IWS. Based on the results of the comparison, the hypothetical model was calibrated.

It was found that, in order to estimate water demand and derive water demand patterns, the parameters that impact water demand and WDS performance have to be identified and the nature of the impact of these parameters has to be understood. Out of a total of 20 driving parameters, ten parameters were identified as the most influential parameters to water demand and WDS performance during IWS. These ten parameters are supply duration, network hydraulic capacity, the available pressure head at the outlet, pressure within the WDN, topography, flow rate within the WDN, climate, supply area population, network filling time and supply area size. Out of these parameters, supply duration was found to be the most influential parameter, followed by the hydraulic capacity of the network. Some of the identified crucial parameters for IWDSs were different to the parameters considered crucial for the CWS scenario. It was deduced that IWS systems were mainly driven by the operational and structural parameters, compared to the CWS systems which are mainly driven by the socio-economic parameters. This implies that CWS and IWS should be considered as two separate supply regimes, with distinct parameters related to each regime. This result was attributed to WDSs subjected to IWS being pressure head dependent compared to CWS systems that are demand dependent.

By combining the most influential parameters with the governing hydraulic principles related to the WDS filling process, an Excel based IWS diurnal water demand pattern model for the 15 minute peak factors was developed. The hypothetical IWS demand pattern model was derived based on a three phase calculation process, where each phase represented a section of the pattern shape. Phase 1 represents the initial water demand spike related to the filling of the bulk supply pipeline from the reservoir to the reticulation network. Phase 2 is related to the reticulation network filling, and phase 3 is related to the water demand characteristics after the filling process is complete. The results indicated that the model shape had a high similarity to the shape of the water demand patterns obtained from actual data, based on the Kolmogorov-Smirnov goodness of fit test. The similarity of the hypothetical model to the actual demand patterns, provides new insight into the typical form of a diurnal water demand pattern associated with IWS in residential areas. It also provides an understanding of what constitutes the IWS diurnal demand pattern, and which parameters are crucial to the peak factors and the form of the demand pattern.

The outcomes of this study provide a domestic diurnal water demand pattern model for areas subjected to IWS. This model can be used as input for the reliability analysis of existing WDSs in a water scarce area. By using this derived Excel based model as an analysis tool, the design engineers can investigate the WDS response to the high peak factors associated with IWS. It can therefore assist design engineers in identifying and planning the most effective ways of implementing IWS, in the process improving the resilience of water supply during periods of water scarcity. The model can also be a useful reference source to investigate the effects of changing the crucial parameters on the peak factors, the diurnal demand pattern and on the WDS performance of a certain supply area that is subjected to IWS.

7.2 Recommendations

For further research, more in-depth individual studies of the undefined relationships and effects of the parameters identified in Table 6-1 and Table 6-3 can be conducted. There were ten parameters marked 'NIA' in Table 6-3 that did not have any information available in relation to the water demand and WDS performance during conditions of IWS. Also a more exhaustive parameter investigation can be conducted, focussing solely on the parameters that influence water demand and WDS performance of systems subjected to IWS.

Furthermore, a more extensive and longer logging exercise of the domestic water demand during IWS conditions for residential areas subjected to IWS in South Africa can also be conducted. The typical diurnal water demand patterns, the seasonal water demand patterns, the weekday water demand patterns, and the weekend water demand patterns associated with IWS, can potentially be derived through the availability of longer period data sets. It is recommended that the logged record period be longer than 12 months and the areas logged should be varied in terms of the area characteristics such as location, area size, development type, and income level. This investigation would provide more real-world based demand patterns, and would also allow the accuracy and applicability of the IWS diurnal demand pattern model developed in this study to be tested for residential areas with varying characteristics. Similarly, research and development of typical diurnal water demand patterns for commercial, institutional and industrial land use areas subjected to IWS can also possibly be conducted.

Lastly, a reliability analysis of existing WDSs using the developed IWS water demand pattern model can be conducted, in order to investigate the effects of different demand scenarios associated with IWS on the performance of the WDSs.

8 References

Abu-Madi, M. & Trifunovic, N. 2013. Impacts of supply duration on the design and performance of intermittent water distribution systems in the West Bank. Water International, 38(3): 263–282.

Agathokleous, A. & Christodoulou, S. 2016. The Impact of Intermittent Water Supply Policies on Urban Water Distribution Networks. Procedia Engineering, 162: 204–211.

Andey, S.P. & Kelkar, P.S. 2007. Performance of water distribution systems during intermittent versus continuous water supply. Journal - American Water Works Association, 99(8): 99–106.

Andey, S.P. & Kelkar, P.S. 2009. Influence of Intermittent and Continuous Modes of Water Supply on Domestic Water Consumption. Water Resources Management, 23(12): 2555–2566.

Arunkumar, M. & Nethaji Mariappan, V.E. 2011. Water Demand Analysis of Municipal Water Supply Using Epanet Software. International Journal on Applied Bio-Engineering, 5(1): 9–19.

Babbie, E.R. 2010. The Practice of Social Research. 12 ed. Belmont, CA: Wadsworth Cengage.

Bender, P. & Gibson, S. 2010. Mbombela (Nelspruit) Water and Sanitation Concession South Africa. : 59.

Bhoyar, R.D. & Mane, M.S.J. 2017. Modelling and Optimization of Water Distribution System Site: Nagpur. International Journal of Advance Engineering and Research Development, 4(7): 12.

Billings, R. B. & Jones, C. V. 2011. Forecasting Urban Water Demand. American Water Works Association.

Bistriceanu, I. 2006. Applications of the hydraulic and water quality network simulations. : 10.

Blaikie, N. 2000. Designing Social Research. Polity Press.

Botha, B.E. & Jacobs, H. 2017. Estimating household groundwater abstraction by means of a probability model. : 127.

Charalambous, B. & Liemberger, R. 2016. Dealing with the Complex Interrelation of Intermittent Supply and Water Losses. Water Intelligence Online, 16.

Day, D. & Howe, C. 2003. Forecasting peak demand - what do we need to know? Water Science and Technology: Water Supply, 3(3): 177–184.

De Marchis, M., Fontanazza, C.M., Freni, G., La Loggia, G., Napoli, E. & Notaro, V. 2010. A model of the filling process of an intermittent distribution network. Urban Water Journal, 7(6): 321–333.

De Marchis, M., Fontanazza, C.M., Freni, G., Loggia, G.L., Napoli, E. & Notaro, V. 2011. Analysis of the impact of intermittent distribution by modelling the network-filling process. Journal of Hydroinformatics, 13(3): 358–373.

Department of Community Development. 1983. Guidelines for the provision of engineering services for residential townships. The Blue Book. Pretoria.

Department of Human Settlements. 2019. The Neighbourhood Planning and Design Guide. South African Government.

Diao, K., Barjenbruch, M. & Bracklow, U. 2010. Study on the impacts of peaking factors on a water distribution system in Germany. Water Science & Technology: Water Supply, pp. 165-172.

Fan, L., Liu, G., Wang, F., Ritsema, C.J. & Geissen, V. 2014. Domestic Water Consumption under Intermittent and Continuous Modes of Water Supply. Water Resources Management, 28(3): 853–865.

Galaitsi, S., Russell, R., Bishara, A., Durant, J., Bogle, J. & Huber-Lee, A. 2016. Intermittent Domestic Water Supply: A Critical Review and Analysis of Causal-Consequential Pathways. Water, 8(7): 274.

Garlipp, K. 1979. Water consumption patterns in urban areas. University of Pretoria.

Gottipati, P.V.K.S.V. & Nanduri, U.V. 2014. Equity in water supply in intermittent water distribution networks: Equity in water supply. Water and Environment Journal, 28(4): 509–515.

Griffioen, M.L. & van Zyl, J.E. 2014. Journal of the South African Institution of Civil Engineering. Journal of the South African Institution of Civil Engineering, 56(1): 6.

Gupta, R. & Bhave, P.R. 1996. Comparison of Methods for Predicting Deficient-Network Performance. Journal of Water Resources Planning and Management, 122(3): 214–217.

Gupta, R., Nair, A.G.R. & Ormsbee, L. 2016. Leakage as Pressure-Driven Demand in Design of Water Distribution Networks. Journal of Water Resources Planning and Management, 142(6): 04016005.

Hou, Q., Tijsseling, A.S., Laanearu, J., Annus, I., Koppel, T., Bergant, A., Vučković, S., Anderson, A. & van't Westende, J.M.C. 2014. Experimental Investigation on Rapid Filling of a Large-Scale Pipeline. Journal of Hydraulic Engineering, 140(11): 04014053.

Husselmann, M. & Van Zyl, J. 2006. Effect of stand size and income on residential water demand. Journal of the South African Institution of Civil Engineering, Volume 48, pp. 12-16.

Ilaya-Ayza, A., Martins, C., Campbell, E. & Izquierdo, J. 2017. Implementation of DMAs in Intermittent Water Supply Networks Based on Equity Criteria. Water, 9(11): 851.

Ilaya-Ayza, A.E., Benítez, J., Izquierdo, J. & Pérez-García, R. 2017. Multi-criteria optimization of supply schedules in intermittent water supply systems. Journal of Computational and Applied Mathematics, 309: 695–703.

Irwin, T. 2002. Aristotle's first principles. Repr. Oxford: Clarendon Press.

Jacobs, H., LC, G., Loubser, B. & Van der Merwe, B. 2004. Estimating residential water demand in Southern Africa. Journal of the South African Institution of Civil Engineering, Volume 46, pp. 2-13.

Jacobs, H.E., Geustyn, L.C. & Loubser, B.F. 2004. Estimating residential water demand in southern Africa. Journal of the South African Institution of Civil Engineering: 12.

Johnson, E. 1999. Degree of utilisation - The reciprocal of the peak factor. Its application in the operation of a water supply and distribution system. Water SA, pp. 111-114.

Kansal, M.L., Kumar, A. & Sharma, P.B. 1995. Reliability analysis of water distribution systems under uncertainty. Reliability Engineering & System Safety, 50(1): 51–59.

King, T., 2015. Design Guidelines and Minimum Standards for Civil Engineering Services, Stellenbosch: Stellenbosch Municipality.

Klingel, P. 2012. Technical causes and impacts of intermittent water distribution. Water Science and Technology: Water Supply, 12(4): 504–512.

Kolmogorov, A. 1933. Sulla determinazione empirica di una legge di distribuzione. G. Ist. Ital. Attuari, pp. 83-91.

Kumpel, E. & Nelson, K.L. 2016. Intermittent Water Supply: Prevalence, Practice, and Microbial Water Quality. Environmental Science & Technology, 50(2): 542–553.

Letting, L., Hamam, Y. & Abu-Mahfouz, A. 2017. Estimation of Water Demand in Water Distribution Systems Using Particle Swarm Optimization. Water, 9(8): 593.

Liou, C.P. & Hunt, W.A. 1996. Filling of Pipelines with Undulating Elevation Profiles. Journal of Hydraulic Engineering, 122(10): 534–539.

Loubser, C., Loubser, E., Moult, N. & Strydom, R. 2018. Using real-time demand patterns to improve hydraulic modelling accuracy in water reticulation networks. Cape Town.

Malekpour, A. & Karney, B.W. 2011. Rapid Filling Analysis of Pipelines with Undulating Profiles by the Method of Characteristics. ISRN Applied Mathematics, 2011: 1–16.

Mane, D., Dhumal, J., Shetage, A. & Bamanikar, A. 2018. Study on implementing 24x7 water supply schemes. International Journal for Research in Applied Science and Engineering Technology, 6(2).

Mayer, P.W. & DeOreo, W.B. 1999. Residential end uses of water. : 18.

McKenzie, R.S., South Africa & Water Research Commission. 2014. Guidelines for reducing water losses in South African municipalities: report to the Water Research Commission.

Mohapatra, S., Sargaonkar, A. & Labhasetwar, P.K. 2014. Distribution Network Assessment using EPANET for Intermittent and Continuous Water Supply. Water Resources Management, 28(11): 3745–3759.

National Academy of Sciences. 1999. Factors affecting patterns of water use. In: Water for the future: The West Bank and Gaza Strip, Israel, and Jordan. Washington, DC: The National Academies Press, pp. 54-65.

Nauges, C. & Whittington, D. 2010. Estimation of Water Demand in Developing Countries: An Overview. The World Bank Research Observer, 25(2): 263–294.

Ntombela, C., Funke, N., Meissner, R., Steyn, M. & Masangane, W. 2016. A critical look at South Africa's Green Drop Programme. Water SA, 42(4): 703.

Pathirana, A. 2011. EPANET2 Desktop Application for Pressure Driven Demand Modeling. In Water Distribution Systems Analysis 2010. 12th Annual Conference on Water Distribution Systems Analysis (WDSA). Tucson, Arizona, United States: American Society of Civil Engineers: 65–74. http://ascelibrary.org/doi/10.1061/41203%28425%298 27 September 2019.

Pretorius, D. 2016. Investigation into low cost housing water use patterns and peak factors. Stellenbosch: Stellenbosch Unviersity.

Ramana, G.V., Sudheer, Ch.V.S.S. & Rajasekhar, B. 2015. Network Analysis of Water Distribution System in Rural Areas using EPANET. Procedia Engineering, 119: 496–505.

Republic of South Africa. 1997. Water Services Act No. 108, 1997. Republic of South Africa Government Gazette, Volume 390.

Rojas-Lima, J.E., Domínguez-Pacheco, F.A., Hernández-Aguilar, C., Hernández-Simón, L.M. & Cruz-Orea, A. 2019. Kolmogorov–Smirnov Test for Statistical Characterization of Photopyroelectric Signals Obtained from Maize Seeds. International Journal of Thermophysics, 40(1). http://link.springer.com/10.1007/s10765-018-2462-4 15 November 2019.

Scheepers, H. M. 2012. Deriving peak factors for residential indoor water demand by means of a probability based end-use model. Stellenbosch: Stellenbosch University.

Siew, C. & Tanyimboh, T.T. 2011. Pressure-Dependent EPANET Extension: Extended Period Simulation. In Water Distribution Systems Analysis 2010. 12th Annual Conference on Water Distribution Systems Analysis (WDSA). Tucson, Arizona, United States: American Society of Civil Engineers: 85–95. http://ascelibrary.org/doi/10.1061/41203%28425%2910 19 May 2019.

Smirnov, N. 1948. Table for estimating the goodness of fit of empirical distributions. Annals of Mathematical Statistics, 19(2), pp. 279-281.

Sonaje, N.P. & Joshi, M.G. 2015. A review of modelling and application of water distribution networks (WDN) software, 3(5): 5.

South African Council for Scientific and Industrial Research, Division of Building and Construction Technology, South Africa & Department of Housing. 2003. Guidelines for human settlement planning and design. Pretoria: CSIR Building and Construction Technology.

Stephenson, D. & Turner, K. 1996. Water demand patterns in Gauteng. IMIESA, 21(1), pp. 11-16.

Strijdom, J.L. & Jacobs, H.E. 2016. Evaluation of minimum pressure head during peak flow as design criterion for water distribution systems. : 145.

Todini, E. & Pilati, S. 1987. A gradient algorithm for the analysis of pipe networks. Computer applications in water supply, Volume 1, pp. 1-20.

Totsuka, N. & Trifunovic, N. 2004. Intermittent urban water supply under water starving situations. : 8.

Trifunovic, N. 2006. Introduction to Urban Water Distribution: Unesco-IHE Lecture Note Series. 0 ed. CRC Press. https://www.taylorfrancis.com/books/9781134158102 29 May 2019.

Unesco ed. 2016. Water and jobs. Paris: UNESCO.

Vairavamoorthy, K. & Elango, K. 2002. Guidelines for the design and control of intermittent water distribution systems. Waterlines, 21(1), pp. 19-21.

Vairavamoorthy, K., Akinpelu, E., Lin, Z. & Ali, M. 2001. Design of Sustainable Water Distribution Systems in Developing Countries. In Bridging the Gap. World Water and Environmental Resources Congress 2001. The Rosen Plaza Hotel, Orlando, Florida, United States: American Society of Civil Engineers: 1–10. http://ascelibrary.org/doi/abs/10.1061/40569%282001%29378 22 March 2019.

Vairavamoorthy, K., Gorantiwar, S.D. & Mohan, S. 2007. Intermittent Water Supply under Water Scarcity Situations. Water International, 32(1): 121–132.

Vairavamoorthy, K., Gorantiwar, S.D. & Pathirana, A. 2008. Managing urban water supplies in developing countries – Climate change and water scarcity scenarios. Physics and Chemistry of the Earth, Parts A/B/C, 33(5): 330–339.

Van Vuuren, S. & Van Beek, J. 1997. Her-Evaluering van die Bestaande Riglyne vir Stedelike en Industriele Watervoorsiening gebaseer op Gemente Waterverbruike Fase 1: Pretoria Voorsieningsgebied. Report for the Water Research Commission.

Van Zyl, H. J. 2006. Evaluation of municipal water demand and related parameters. Johannesburg.

Van Zyl, H., Ilemobade, A. & van Zyl, J. 2008. An improved area-based guideline for domestic water demand estimation in South Africa. Water SA, 34(3): 12.

Van Zyl, H.J., South Africa & Water Research Commission. 2007. Water consumption levels in selected South African cities.

Van Zyl, J., Haarhoff, J. & Husselman, M. 2003. Potential application of end-use demand modelling in South Africa. Journal of the South African Institution of Civil Engineering, Volume 45, pp. 9-19.

Vorster, J. et al. 1995. A strategy and master plan for water supply, storage and distribution in the East Rand Region. Journal of the South African Institution of Civil Engineers, pp. 1-5.

Wagner, J.M., Shamir, U. & Marks, D.H. 1988. Water Distribution Reliability: Simulation Methods. Journal of Water Resources Planning and Management, 114(3): 276–294.

Winter, K. 2018. Crisis proofing South Africa's water security. [Online] Available at: https://theconversation.com/crisis-proofing-south-africas-water-security-106261#

Wurbs, R.A. 1994. Computer models for water resources planning and management. : 228.

Appendix A Model development

A.1 Model development calculation worksheets

Table A.1 Phase 1 – Bulk supply pipe filling Excel calculation worksheet.

Phase 1:		
Bulk Supply Pipe between Storage Reservoir and Supply Area Network		
Total Pipe Length	17.13	m
Average Pipe Internal Diameter	0.14	m
Pipe Cross Sectional Area	0.02	m^2
Pipe Filling Volume	1.00	m^3
Initial Filling Flow Rate	4.00	m^3/h
Max Allowable Flow Rate for Pipe Diameter and Pipe Material	93.60	m^3/h
Recommended Maximum Allowable Velocities in the Pipe	2.08	m/s
Total Pipe Filling Time at Maximum Velocity	0.64	min
	38.46	sec

Table A.2 Phase 2 – Network filling Excel calculation worksheet.

Phase 2:			
Network Filling Process:			
Total Network Length		2134.29	m
Average Network Pipe Internal Diameter		0.07	m
Pipe Cross Sectional Area		0.004	m^2
Total Network Filling Volume		9.00	m^3
Max Allowable Flow Rate for Pipe Diameter		28.80	m^3/h
Network filling time at maximum allowable flow rate		0.31	hrs
		19.00	min
User water consumption begins after time =		0.64	min
CWS user demand rate during the network filling time interval		7.20	m^3/h
Supply duration demand reduction factor		73	%
Adjusted CWS user demand rate during the network filling time interval		5.25	m^3/h
Total user water consumption during Phase 2		1.64	m^3
Available flow rate for network filling	Qmax =	23.55	m^3/h
Network filling time at available flow rate	tmax =	0.38	hrs
		22.93	min
Check: Is the flow rate for network filling within the range of the designed network demand?			
WDN design demand		21.60	m^3/h
Flow rate during network filling is beyond designed network capacity, therefore use WDN design demand to determine Qmax			
Recalculated flow rate in the pipes (Qmax)		21.60	m^3/h
Recalculated network filling time (tmax)		0.42	hrs
		25.00	min
Check: Is the total supply duration per supply period > total network filling time?			
Total Network Filling Time		0.42	hrs

Table A.3 Phase 3 – Full network Excel calculation worksheet.

Determining the peak flows after the network filling process is complete:						
Phase 3:						
New IWS AADD	107.78	kL/day				
	4.49	m^3/h				
Supply Period 1:						
Total user water consumption during Phase 2		0.82	kL			
New AADD Remaining Supply Period 1		106.96	kL			
AADD Supply Period 1		4.46	m^3/h			
Start time =		4:25	End time =	10:30		
Demand multiplier		2.09	for	6.07	hour supply duration	
- Enter the time interval hours in the table below						
Supply Period 2:						
AADD supplied during Supply Period 1		75.88	kL			
AADD Remaining before Supply Period 2 begins		31.90	kL			
Return to Phase 1 and Phase 2 Calculations						
Total user water consumption during Phase 2		1.64	kL			
New AADD Remaining Supply Period 2		106.14	kL			
AADD Supply Period 2		4.42	m^3/h			
Start time =		17:10	End time =	21:45		
Demand multiplier		2.09	for	4.57	hour supply duration	
- Enter the time interval hours in the table below						
Supply Period 1			Supply Period 2			
Hour	PHF	Peak Water Demand (m^3/h)	Hour	PHF	Peak Water Demand (m^3/h)	
4	1.07	4.79	17	2.33	10.29	
5	1.28	5.72	18	2.29	10.11	
6	1.76	7.86	19	2.10	9.28	
7	2.52	11.21	20	1.76	7.80	
8	3.25	14.47	21	1.49	6.60	
9	3.58	15.96		0.00	0.00	
10	3.56	15.87		0.00	0.00	
	0.00	0.00		0.00	0.00	
	0.00	0.00		0.00	0.00	
	0.00	0.00		0.00	0.00	
	0.00	0.00		0.00	0.00	
	0.00	0.00		0.00	0.00	
	0.00	0.00		0.00	0.00	
	0.00	0.00		0.00	0.00	
	0.00	0.00		0.00	0.00	
	0.00	0.00		0.00	0.00	
	0.00	0.00		0.00	0.00	
	0.00	0.00		0.00	0.00	
	0.00	0.00		0.00	0.00	
	0.00	0.00		0.00	0.00	
	0.00	0.00		0.00	0.00	
	0.00	0.00		0.00	0.00	
	0.00	0.00		0.00	0.00	
	0.00	0.00		0.00	0.00	
	0.00	0.00		0.00	0.00	
	Total Water Volume =	75.88	m^3	Total Water Volume =	44.08	m^3
	New Base Flows =	22.76	m^3	New Base Flows =	13.22	m^3
		3.75	m^3/h		2.89	m^3/h

Table A.4 Phase 3 – Full network check Excel calculation worksheet.

Check: Are the peak water demand values within the range of the max. network demand?

Supply Period 1:

WDN design demand		21.6	m^3/h
75.88	m^3 supplied in	6.07	hours

Therefore:

70.40 % of the IWS water demand can be supplied during the supply period 1

Increase supply duration, increase network capacity and/or decrease water demand

Supply Period 2:

WDN design demand		21.60	m^3/h
44.08	m^3 supplied in	4.57	hours

Therefore:

40.90 % of the IWS water demand can be supplied during the supply period 2

Increase supply duration, increase network capacity and/or decrease water demand

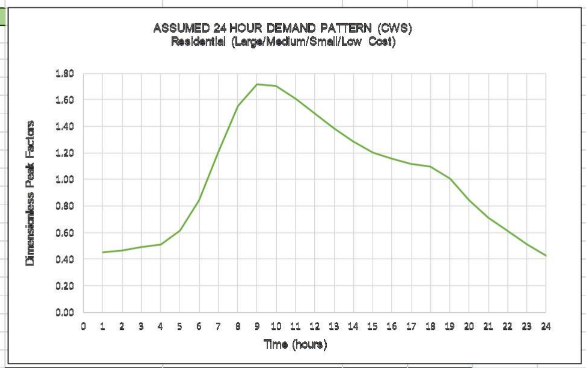
Supply Period 1	Adjusted Peak Water Demand	Supply Period 2	Adjusted Peak Water Demand
4.79	4.79	10.29	10.29
5.72	5.72	10.11	10.11
7.86	7.86	9.28	9.28
11.21	11.21	7.80	7.80
14.47	14.47	6.60	6.60
15.96	15.96	0.00	0.00
15.87	15.87	0.00	0.00
0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.00
0.00	0.00	0.00	0.00

Recalculated Base Flows		Recalculated Base Flows	
22.76	m^3	13.22	m^3
3.75	m^3/h	2.89	m^3/h

Step 1: Choose the assumed CWS diurnal water demand pattern for the estimation

Select Pattern:

- ○ ResidentialLarge
- ○ ResidentialMedium
- ○ ResidentialSmall
- ◉ Low Cost Housing



Characteristics of the chosen CWS diurnal water demand pattern:		
Specify CWS AADD	150.40	kL/day
Specify CWS Base Flows	45.12	kL/day
AADD	6.27	m^3/h
Highest Morning PHF	1.71	
Highest Evening PHF	1.49	
Peak Morning Demand	10.74	m^3/h
Peak Evening Demand	9.36	m^3/h
Total Estimated Volume Water Demand per Day	147.65	kL/day
Estimated AADD	6.15	m^3/h
Base Flow	1.85	m^3/h
Total Base Flow Volume	44.30	kL/day
Base Flow Percentage of the Total Water Demand	30.00	%

Selected Pattern:	LC Housing	Peak Water	SPECIFY ASSUMED PATTERNS	Res.lrg.	Res.med.	Res.sml.	LC Housing	Area under curve
4	Pattern4	Demand	Hour	Pattern1	Pattern2	Pattern3	Pattern4	(Reimanns sum)
Hour	PHF	(m^3/h)						
1	0.45	2.84	1	0.5	0.3	0.1	0.45	2.87
2	0.46	2.91	2	0.5	0.3	0.1	0.46	3.00
3	0.49	3.09	3	0.5	0.3	0.2	0.49	3.16
4	0.51	3.22	4	0.5	0.4	0.3	0.51	3.53
5	0.61	3.85	5	0.6	0.5	0.5	0.61	4.57
6	0.84	5.29	6	0.9	0.8	1	0.84	6.42
7	1.20	7.54	7	1.2	1.2	1.4	1.20	8.64
8	1.55	9.74	8	1.4	1.6	1.9	1.55	10.24
9	1.71	10.74	9	1.6	1.75	2	1.71	10.71
10	1.70	10.68	10	1.6	1.8	2.1	1.70	10.36
11	1.60	10.05	11	1.4	1.6	1.9	1.60	9.71
12	1.49	9.36	12	1.3	1.4	1.6	1.49	9.02
13	1.38	8.67	13	1.2	1.25	1.2	1.38	8.36
14	1.28	8.04	14	1.2	1.2	1	1.28	7.79
15	1.20	7.54	15	1.2	1.3	1.2	1.20	7.39
16	1.15	7.23	16	1.3	1.4	1.6	1.15	7.10
17	1.11	6.98	17	1.4	1.5	1.8	1.11	6.92
18	1.09	6.85	18	1.4	1.4	1.6	1.09	6.57
19	1.00	6.29	19	1.1	1.2	1	1.00	5.79
20	0.84	5.29	20	0.9	0.9	0.7	0.84	4.88
21	0.71	4.47	21	0.7	0.7	0.4	0.71	4.16
22	0.61	3.85	22	0.6	0.5	0.2	0.61	3.53
23	0.51	3.22	23	0.5	0.4	0.1	0.51	2.94
24	0.42	2.66	24	0.5	0.3	0.1	0.42	
		150.40		24.00	24.00	24.00	24.00	147.65

Figure A.1 Step 1 - Choosing the CWS diurnal demand pattern.

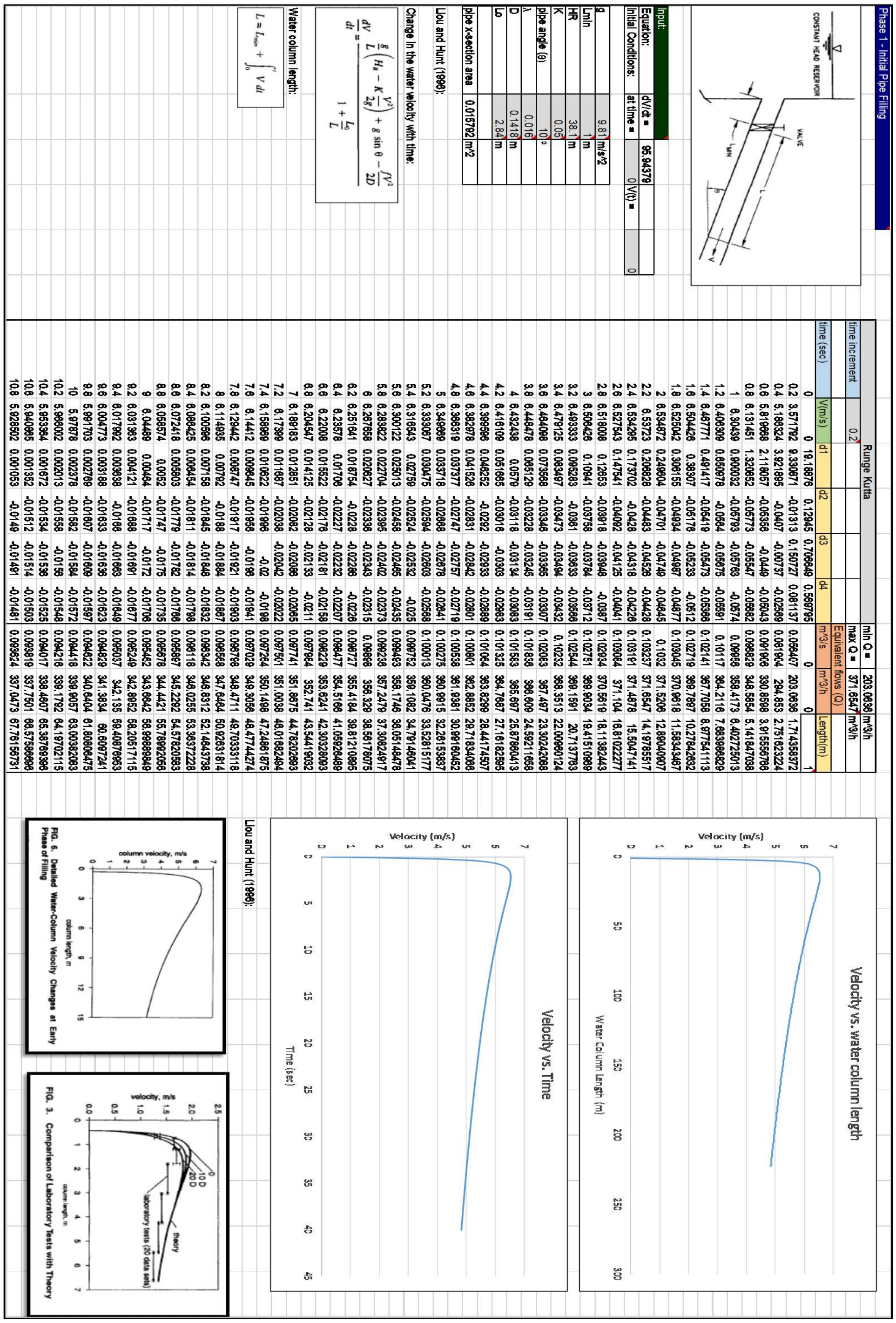


Figure A.2 First approach to determining the bulk supply pipe filling flow rate in Phase 1.

Table A.5 Extract of the plotter table used to plot the IWS diurnal water demand pattern.

Plotter:																
	3 Phases of the IWS Water Demand Pattern															
	Peak Water Demand (Initial Pipe Filling)				Peak Water Demand (During Network Filling)				Peak Water Demand (After Network Filling)					15 min. Peak Water Demand (m^3/h)	Reimann Sum	IWS PF(15min)
	Duration (h:mm:ss)	0:00:38			Duration (h:mm:ss)	0:25:00			Duration (h:mm:ss)	19:34:22						
	Start	3:00	End	3:00	Start	3:00	End	3:25	Start	3:25	End	23:00				
Time(hrs)	Q (m^3/h)-Supply Period 1				Q (m^3/h)-Supply Period 1				Q (m^3/h)-Supply Period 1				Time(hrs)			
0:00													0	0.00	0.00	0.00
0:15													0.25	0.00	0.00	0.00
0:30													0.5	0.00	0.00	0.00
0:45													0.75	0.00	0.00	0.00
1:00													1	0.00	0.00	0.00
1:15													1.25	0.00	0.00	0.00
1:30													1.5	0.00	0.00	0.00
1:45													1.75	0.00	0.00	0.00
2:00													2	0.00	0.00	0.00
2:15													2.25	0.00	0.00	0.00
2:30													2.5	0.00	0.00	0.00
2:45													2.75	0.00	0.00	0.00
3:00													3	0.00	3.08	0.00
3:15	4				21								3.25	24.65	5.75	4.01
3:30					21								3.5	21.39	4.75	3.48
3:45									17				3.75	16.62	3.56	2.70
4:00									12				4	11.86	2.37	1.93
4:15									7				4.25	7.09	1.34	1.15
4:30									4				4.5	3.63	0.91	0.59
4:45									4				4.75	3.67	0.92	0.60
5:00									4				5	3.71	0.93	0.60
5:15									4				5.25	3.75	0.95	0.61
5:30									4				5.5	3.87	0.99	0.63
5:45									4				5.75	4.07	1.04	0.66
6:00									4				6	4.27	1.09	0.69
6:15									4				6.25	4.46	1.17	0.73
6:30									5				6.5	4.87	1.27	0.79
6:45									5				6.75	5.32	1.39	0.87
7:00									6				7	5.77	1.50	0.94
7:15									6				7.25	6.25	1.65	1.02
7:30									7				7.5	6.96	1.83	1.13
7:45									8				7.75	7.67	2.01	1.25
8:00									8				8	8.37	2.18	1.36
8:15									9				8.25	9.08	2.35	1.48
8:30									10				8.5	9.76	2.53	1.59
8:45									10				8.75	10.45	2.70	1.70
9:00									11				9	11.14	2.84	1.81
9:15									12				9.25	11.59	2.94	1.88
9:30									12				9.5	11.91	3.02	1.94
9:45									12				9.75	12.22	3.09	1.99
10:00									13				10	12.54	3.14	2.04
10:15									13				10.25	12.55	3.13	2.04
10:30									13				10.5	12.53	3.13	2.04
10:45									13				10.75	12.51	3.12	2.03
11:00									12				11	12.46	3.09	2.02
11:15									12				11.25	12.26	3.04	1.99
11:30									12				11.5	12.06	2.99	1.96
11:45									12				11.75	11.87	2.94	1.93
12:00									12				12	11.66	2.89	1.90

Phase 2 - Water Distribution Network Filling

Representing the whole network as one node that has all the demand lumped on it

Determining ΔHavl:			
ΔH	3	tmax (min)	27.6
tmin (min)	0	ti =	t(min)

Determining Qavl at the node:			
Qmax (m^3/min)	0.326	Hmax (m)	40.8
Hmin (m)	6	Hi (m)	37.8

time increments	0.46						
t(hrs)	t(min)	Δhi (m)	Hi (m)	Qi(m^3/min)	Qi(m^3/h)		Vol.(m^3)
0	0	0	37.8	0.311631635	18.69789809		0.1440733
0.007682	0.46	0.3872983	38.1873	0.313523603	18.81141615		0.1446891
0.015364	0.92	0.5477226	38.3477	0.314303946	18.85823673		0.1450067
0.023046	1.38	0.6708204	38.4708	0.314901413	18.89408476		0.1452602
0.030728	1.84	0.7745967	38.5746	0.315404221	18.92425328		0.1454781
0.03841	2.3	0.8660254	38.666	0.315846542	18.95079249		0.145672
0.046092	2.76	0.9486833	38.7487	0.316245897	18.97475382		0.1458486
0.053774	3.22	1.0246951	38.8247	0.316612698	18.99676187		0.1460117
0.061456	3.68	1.0954451	38.8954	0.316953726	19.01722357		0.1461641
0.069138	4.14	1.161895	38.9619	0.317273693	19.03642159		0.1463075
0.07682	4.6	1.2247449	39.0247	0.317576029	19.05456173		0.1464434
0.084502	5.06	1.2845233	39.0845	0.317863322	19.07179935		0.1465728
0.092184	5.52	1.3416408	39.1416	0.318137586	19.08825514		0.1466965
0.099866	5.98	1.396424	39.1964	0.318400418	19.1040251		0.1468154
0.107548	6.44	1.4491377	39.2491	0.318653117	19.11918703		0.1469297
0.11523	6.9	1.5	39.3	0.318896751	19.13380507		0.1470402
0.122912	7.36	1.5491933	39.3492	0.319132214	19.14793283		0.147147
0.130594	7.82	1.5968719	39.3969	0.31936026	19.16161563		0.1472505
0.138276	8.28	1.6431677	39.4432	0.319581537	19.17489222		0.1473511
0.145958	8.74	1.6881943	39.4882	0.319796601	19.18779606		0.1474489
0.15364	9.2	1.7320508	39.5321	0.320005937	19.20035622		0.1475442
0.161322	9.66	1.7748239	39.5748	0.32020997	19.21259821		0.1476371
0.169004	10.12	1.8165902	39.6166	0.320409075	19.2245445		0.1477278
0.176686	10.58	1.8574176	39.6574	0.320603584	19.23621506		0.1478164
0.184368	11.04	1.8973666	39.6974	0.320793795	19.24762771		0.1479032
0.19205	11.5	1.9364917	39.7365	0.320979973	19.25879841		0.1479881
0.199732	11.96	1.9748418	39.7748	0.321162359	19.26974156		0.1480714
0.207414	12.42	2.0124612	39.8125	0.32134117	19.28047017		0.148153
0.215096	12.88	2.0493902	39.8494	0.321516601	19.29099608		0.1482331
0.222778	13.34	2.0856654	39.8857	0.321688834	19.30133006		0.1483118
0.23046	13.8	2.1213203	39.9213	0.321858033	19.31148195		0.1483891
0.238142	14.26	2.1563859	39.9564	0.322024347	19.32146081		0.1484652
0.245824	14.72	2.1908902	39.9909	0.322187916	19.33127496		0.1485399
0.253506	15.18	2.2248595	40.0249	0.322348867	19.34093205		0.1486136
0.261188	15.64	2.258318	40.0583	0.32250732	19.35043918		0.148686
0.26887	16.1	2.2912878	40.0913	0.322663382	19.35980294		0.1487574
0.276552	16.56	2.32379	40.1238	0.322817157	19.36902942		0.1488278
0.284234	17.02	2.3558438	40.1558	0.322968739	19.37812432		0.1488972
0.291916	17.48	2.3874673	40.1875	0.323118216	19.38709295		0.1489656
0.299598	17.94	2.4186773	40.2187	0.323265671	19.39594026		0.1490331
0.30728	18.4	2.4494897	40.2495	0.323411181	19.40467089		0.1490998
0.314962	18.86	2.4799194	40.2799	0.32355482	19.4132892		0.1491656
0.322644	19.32	2.5099801	40.31	0.323696655	19.42179928		0.1492305
0.330326	19.78	2.539685	40.3397	0.32383675	19.43020497		0.1492947
0.338008	20.24	2.5690465	40.369	0.323975165	19.43850991		0.1493582
0.34569	20.7	2.5980762	40.3981	0.324111959	19.44671752		0.1494208
0.353372	21.16	2.6267851	40.4268	0.324247184	19.45483101		0.1494828
0.361054	21.62	2.6551836	40.4552	0.324380891	19.46285346		0.1495441
0.368736	22.08	2.6832816	40.4833	0.324513129	19.47078775		0.1496047
0.376418	22.54	2.7110883	40.5111	0.324643944	19.47863664		0.1496647
0.3841	23	2.7386128	40.5386	0.324773379	19.48640272		0.1497241
0.391782	23.46	2.7658633	40.5659	0.324901475	19.49408847		0.1497828
0.399464	23.92	2.792848	40.5928	0.325028271	19.50169625		0.149841
0.407146	24.38	2.8195744	40.6196	0.325153805	19.5092283		0.1498985
0.414828	24.84	2.8460499	40.646	0.325278113	19.51668676		0.1499556
0.42251	25.3	2.8722813	40.6723	0.325401228	19.52407365		0.150012
0.430192	25.76	2.8982753	40.6983	0.325523182	19.53139094		0.150068
0.437874	26.22	2.9240383	40.724	0.325644008	19.53864048		0.1501234
0.445556	26.68	2.9495762	40.7496	0.325763734	19.54582405		0.1501784
0.453238	27.14	2.974895	40.7749	0.325882389	19.55294334		0.1502328
0.46092	27.6	3	40.8	0.326	19.56		
							8.8844504 Total volume

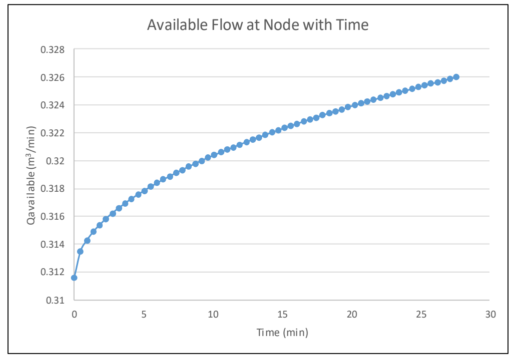


Figure A.3 Phase 2 Excel calculation worksheet to determine peak flows.

Phase 3 - Full Distribution Network

		Peak Water Demand (After the Filling Process)				
		New base flows (m³/h)	3.75			
		Time interval for CWS data (hrs)	7	IWS time interval required (hrs)	6.07	
		Value of last peak flow from phase 2 (m³/h)	21.39	Total plotting interval	6.07	
Hrs	IWS Peak flows (m³/h)	Peak flows (without base flows)	redistribute base flows	Interval redistribution(hrs)	Minutes equivalent	Phase 3 Peak flows (m³/h)
				0.867	0	21
1	5	1	5	0.867	52	5
2	6	2	6	1.734	104	6
3	8	4	8	2.601	156	8
4	11	7	11	3.469	208	11
5	14	11	14	4.336	260	14
6	16	12	16	5.203	312	16
7	16	12	16	6.070	364	16
8		0	0	6.070	364	0
9		0	0	6.070	364	0
10		0	0	6.070	364	0
11		0	0	6.070	364	0
12		0	0	6.070	364	0
13		0	0	6.070	364	0
14		0	0	6.070	364	0
15		0	0	6.070	364	0
16		0	0	6.070	364	0
17		0	0	6.070	364	0
18		0	0	6.070	364	0
19		0	0	6.070	364	0
20		0	0	6.070	364	0
21		0	0	6.070	364	0
22		0	0	6.070	364	0
23		0	0	6.070	364	0
24		0	0	6.070	364	0

Sum of peak flows for the column	Sum of peak flows for the column	Sum of peak flows for the column
76	50	76

Base flow for redistribution	3.75	m³/h
% of Base flow for redistribution	100	
Base flow during supply period	3.75	m³/h
Base flow during non-supply period	0	m³/h

Consider the Supply Area Slope:

Area topography	% redistribution of base flows	
Flat	100	% redistributed during supply period
	0	% redistributed during off supply periods
Hilly/Moderate	50	% redistributed during supply period
	50	% redistributed during off supply periods
Steep	90	% redistributed during supply period
	10	% redistributed during off supply periods

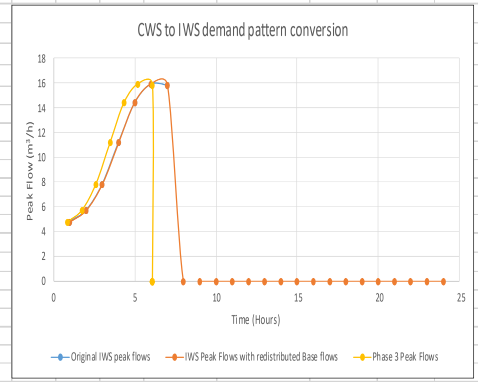


Figure A.4 Phase 3 Excel calculation worksheet to determine peak flows.

Appendix B Results

B.1 Results and discussion of results calculation worksheets

Table B.1 K-S goodness of fit test calculations.

Total N =	96	alpha =	0.05					
		Actual Pattern (Y1)		Model Pattern (Y2)				
		PF(15min)	Determining CDF	PF(15min)			Determining CDF	
No.	Time		Cum. %	SP1	SP2	Combined	Cum. %	Difference
1	0:00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
2	0:15	0.00	0.00	0.00	0.00	0.00	0.00	0.00
3	0:30	0.00	0.00	0.00	0.00	0.00	0.00	0.00
4	0:45	0.00	0.00	0.00	0.00	0.00	0.00	0.00
5	1:00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
6	1:15	0.00	0.00	0.00	0.00	0.00	0.00	0.00
7	1:30	0.00	0.00	0.00	0.00	0.00	0.00	0.00
8	1:45	0.00	0.00	0.00	0.00	0.00	0.00	0.00
9	2:00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
10	2:15	0.00	0.00	0.00	0.00	0.00	0.00	0.00
11	2:30	0.00	0.00	0.00	0.00	0.00	0.00	0.00
12	2:45	0.00	0.00	0.00	0.00	0.00	0.00	0.00
13	3:00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
14	3:15	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15	3:30	0.00	0.00	0.00	0.00	0.00	0.00	0.00
16	3:45	0.00	0.00	0.00	0.00	0.00	0.00	0.00
17	4:00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
18	4:15	0.14	0.00	5.49	0.00	5.49	0.05	0.04
19	4:30	1.84	0.02	4.76	0.00	4.76	0.09	0.07
20	4:45	4.09	0.06	3.70	0.00	3.70	0.12	0.06
21	5:00	2.39	0.08	2.63	0.00	2.63	0.14	0.06
22	5:15	2.11	0.10	1.57	0.00	1.57	0.15	0.06
23	5:30	2.21	0.12	1.10	0.00	1.10	0.16	0.04
24	5:45	2.16	0.14	1.16	0.00	1.16	0.17	0.03
25	6:00	2.34	0.16	1.22	0.00	1.22	0.18	0.02
26	6:15	2.34	0.18	1.28	0.00	1.28	0.19	0.01
27	6:30	2.39	0.20	1.42	0.00	1.42	0.20	0.00
28	6:45	2.34	0.22	1.56	0.00	1.56	0.22	0.00
29	7:00	2.44	0.24	1.69	0.00	1.69	0.23	0.01
30	7:15	2.57	0.27	1.88	0.00	1.88	0.25	0.02
31	7:30	2.94	0.29	2.09	0.00	2.09	0.26	0.03
32	7:45	3.12	0.32	2.31	0.00	2.31	0.28	0.04
33	8:00	3.26	0.35	2.52	0.00	2.52	0.30	0.05
34	8:15	3.03	0.38	2.73	0.00	2.73	0.33	0.05
35	8:30	3.35	0.41	2.94	0.00	2.94	0.35	0.06
36	8:45	3.26	0.44	3.15	0.00	3.15	0.38	0.06
37	9:00	3.35	0.47	3.29	0.00	3.29	0.41	0.06
38	9:15	3.40	0.50	3.38	0.00	3.38	0.43	0.07
39	9:30	3.26	0.53	3.48	0.00	3.48	0.46	0.07
40	9:45	3.12	0.56	3.55	0.00	3.55	0.49	0.07
41	10:00	2.99	0.59	3.55	0.00	3.55	0.52	0.06
42	10:15	2.48	0.61	3.54	0.00	3.54	0.55	0.06
43	10:30	1.01	0.62	3.54	0.00	3.54	0.58	0.04
44	10:45	0.05	0.62	0.00	0.00	0.00	0.58	0.04
45	11:00	0.00	0.62	0.00	0.00	0.00	0.58	0.04
46	11:15	0.00	0.62	0.00	0.00	0.00	0.58	0.04
47	11:30	0.00	0.62	0.00	0.00	0.00	0.58	0.04
48	11:45	0.00	0.62	0.00	0.00	0.00	0.58	0.04
49	12:00	0.00	0.62	0.00	0.00	0.00	0.58	0.04
50	12:15	0.00	0.62	0.00	0.00	0.00	0.58	0.04
51	12:30	0.00	0.62	0.00	0.00	0.00	0.58	0.04
52	12:45	0.00	0.62	0.00	0.00	0.00	0.58	0.04
53	13:00	0.00	0.62	0.00	0.00	0.00	0.58	0.04
54	13:15	0.00	0.62	0.00	0.00	0.00	0.58	0.04
55	13:30	0.00	0.62	0.00	0.00	0.00	0.58	0.04
56	13:45	0.00	0.62	0.00	0.00	0.00	0.58	0.04
57	14:00	0.00	0.62	0.00	0.00	0.00	0.58	0.04
58	14:15	0.00	0.62	0.00	0.00	0.00	0.58	0.04
59	14:30	0.00	0.62	0.00	0.00	0.00	0.58	0.04
60	14:45	0.00	0.62	0.00	0.00	0.00	0.58	0.04
61	15:00	0.00	0.62	0.00	0.00	0.00	0.58	0.04
62	15:15	0.00	0.62	0.00	0.00	0.00	0.58	0.04
63	15:30	0.00	0.62	0.00	0.00	0.00	0.58	0.04
64	15:45	0.00	0.62	0.00	0.00	0.00	0.58	0.04
65	16:00	0.00	0.62	0.00	0.00	0.00	0.58	0.04
66	16:15	0.00	0.62	0.00	0.00	0.00	0.58	0.04
67	16:30	0.00	0.62	0.00	0.00	0.00	0.58	0.04
68	16:45	0.00	0.62	0.00	0.00	0.00	0.58	0.04
69	17:00	0.51	0.62	0.00	5.49	5.49	0.63	0.00
70	17:15	2.57	0.65	0.00	4.76	4.76	0.67	0.02
71	17:30	4.13	0.68	0.00	4.09	4.09	0.70	0.02
72	17:45	3.26	0.71	0.00	3.41	3.41	0.73	0.02
73	18:00	3.03	0.74	0.00	2.74	2.74	0.75	0.01
74	18:15	2.80	0.77	0.00	2.29	2.29	0.77	0.01
75	18:30	2.48	0.79	0.00	2.28	2.28	0.79	0.00
76	18:45	2.30	0.81	0.00	2.27	2.27	0.81	0.00
77	19:00	2.30	0.83	0.00	2.25	2.25	0.83	0.00
78	19:15	2.11	0.85	0.00	2.22	2.22	0.85	0.00
79	19:30	2.11	0.87	0.00	2.17	2.17	0.86	0.00
80	19:45	2.34	0.89	0.00	2.11	2.11	0.88	0.01
81	20:00	2.11	0.91	0.00	2.06	2.06	0.90	0.01
82	20:15	2.02	0.93	0.00	1.97	1.97	0.92	0.01
83	20:30	1.98	0.94	0.00	1.88	1.88	0.93	0.01
84	20:45	2.02	0.96	0.00	1.79	1.79	0.95	0.02
85	21:00	1.93	0.98	0.00	1.71	1.71	0.96	0.02
86	21:15	1.38	0.99	0.00	1.64	1.64	0.97	0.02
87	21:30	0.55	1.00	0.00	1.56	1.56	0.99	0.01
88	21:45	0.18	1.00	0.00	1.49	1.49	1.00	0.00
89	22:00	0.05	1.00	0.00	0.00	0.00	1.00	0.00
90	22:15	0.00	1.00	0.00	0.00	0.00	1.00	0.00
91	22:30	0.00	1.00	0.00	0.00	0.00	1.00	0.00
92	22:45	0.00	1.00	0.00	0.00	0.00	1.00	0.00
93	23:00	0.00	1.00	0.00	0.00	0.00	1.00	0.00
94	23:15	0.00	1.00	0.00	0.00	0.00	1.00	0.00
95	23:30	0.00	1.00	0.00	0.00	0.00	1.00	0.00
96	23:45	0.00	1.00	0.00	0.00	0.00	1.00	0.00
		sum				sum		
		110.17				119.69		

K-S statistic (D):
0.07

K-S critical value (dα):
0.14

Significant difference (Y/N)
No

Appendix C Study site flow readings

C.1 Daily flow readings for Site 1, Site 2, and Site 3

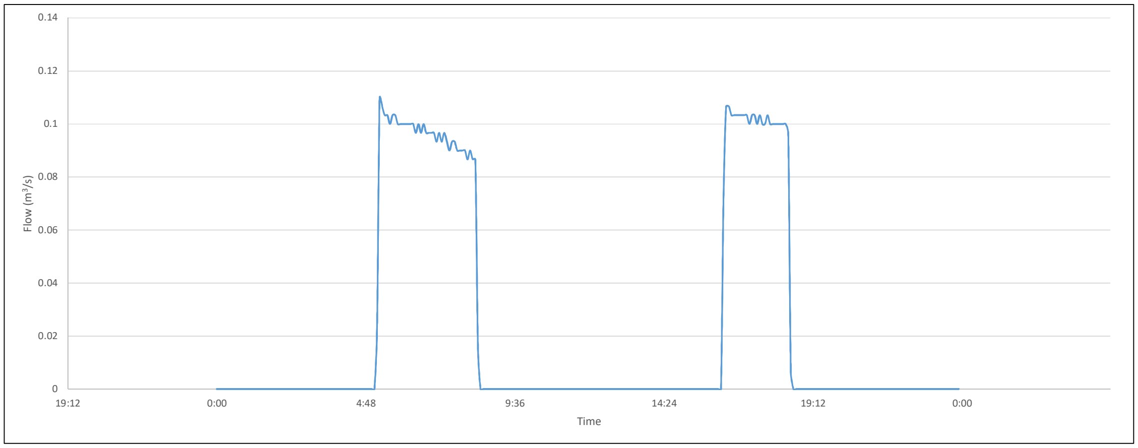


Figure C.1 Typical daily flow reading from Site 1 (Vodacom).

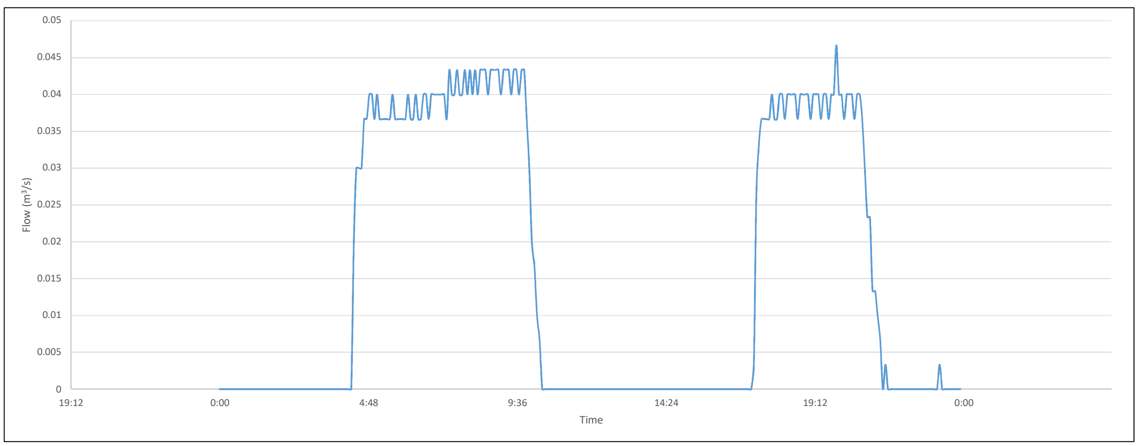


Figure C.2 Typical daily flow reading from Site 3 (Thembeka).

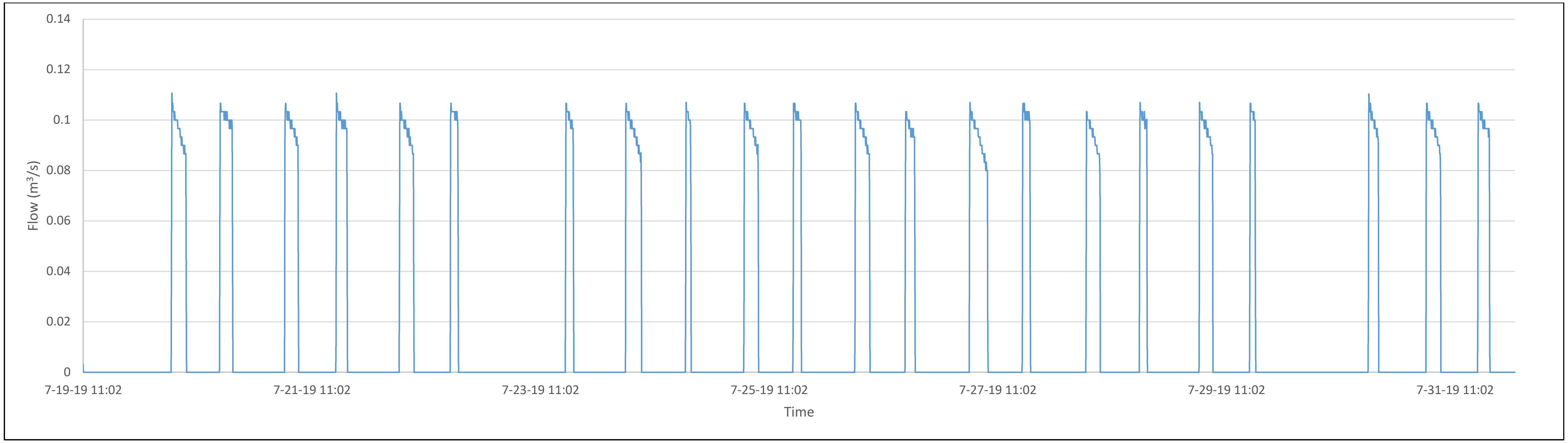


Figure C.3 Daily flow record for July for Site 1 (Vodacom).

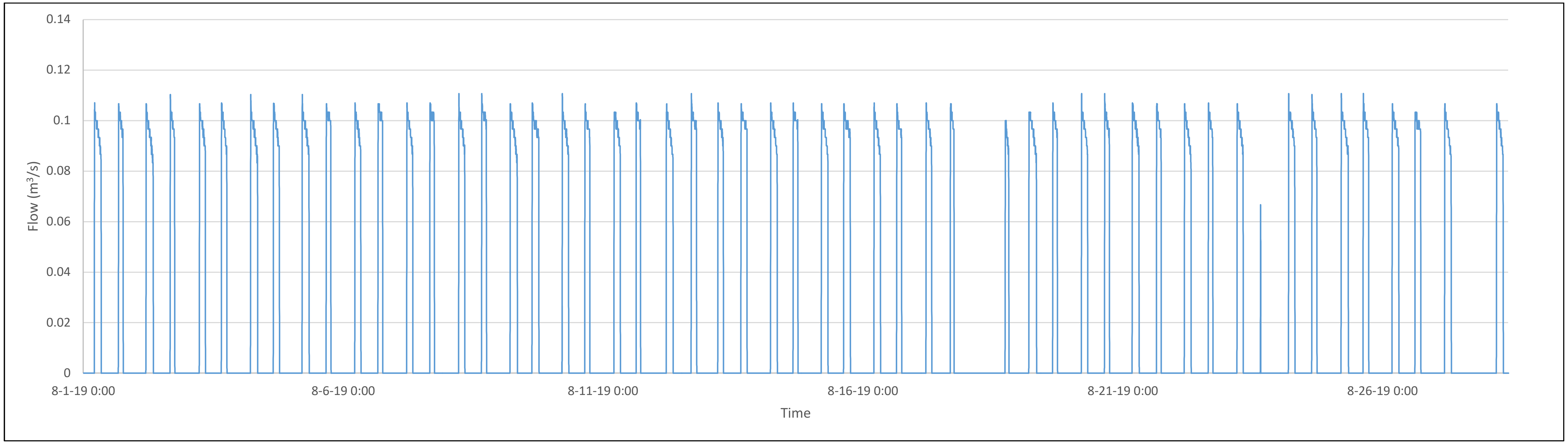


Figure C.4 Daily flow record for August for Site 1 (Vodacom).

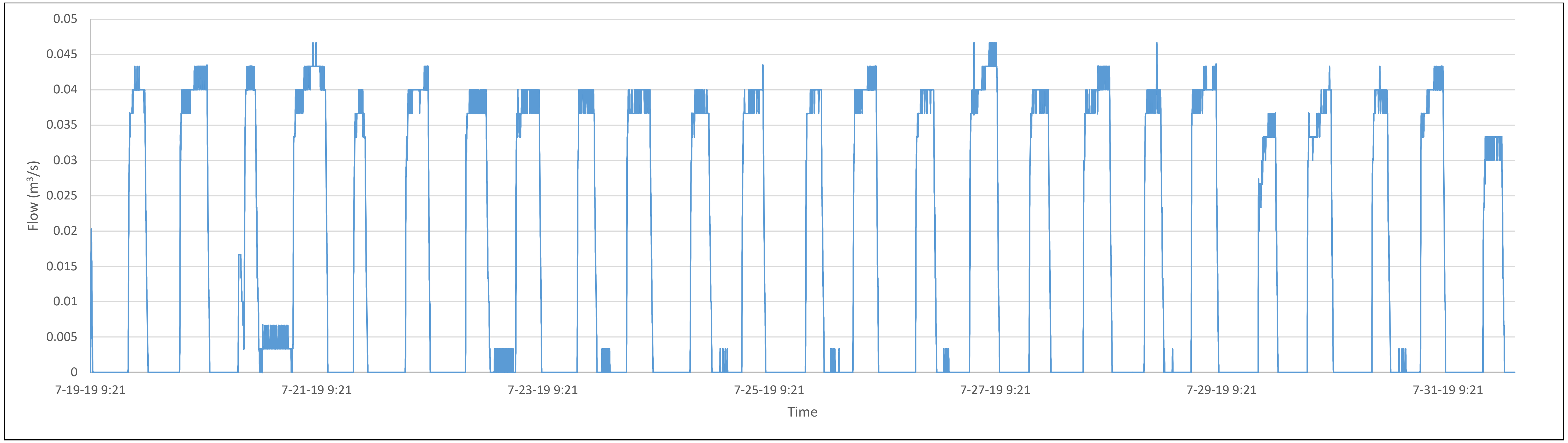


Figure C.5 Daily flow record for July for Site 3 (Thembeka).

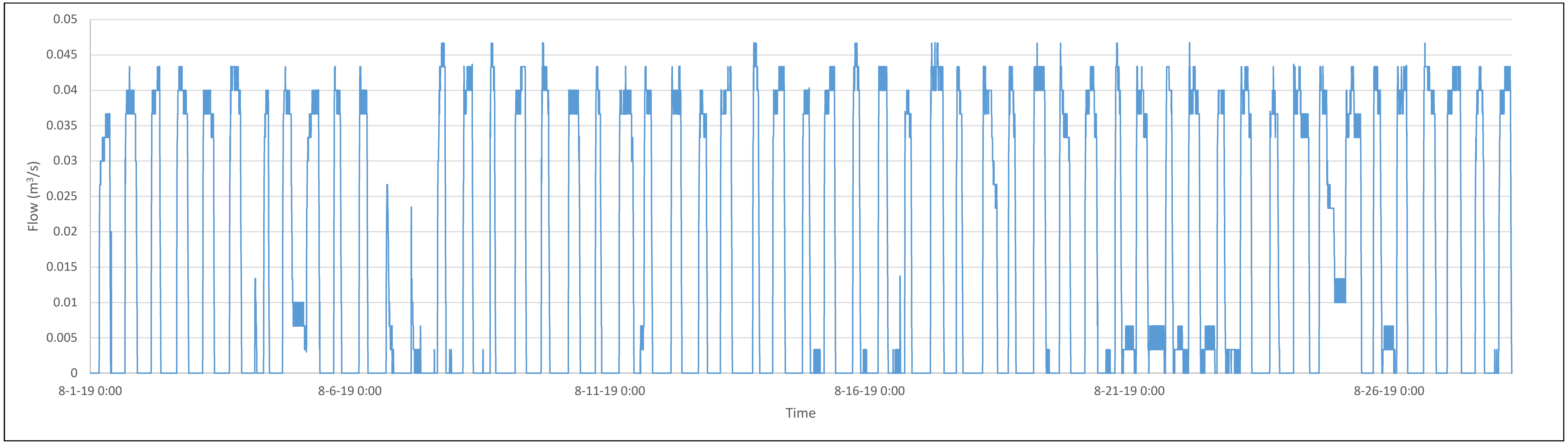


Figure C.6 Daily flow record for August for Site 3 (Thembeka).

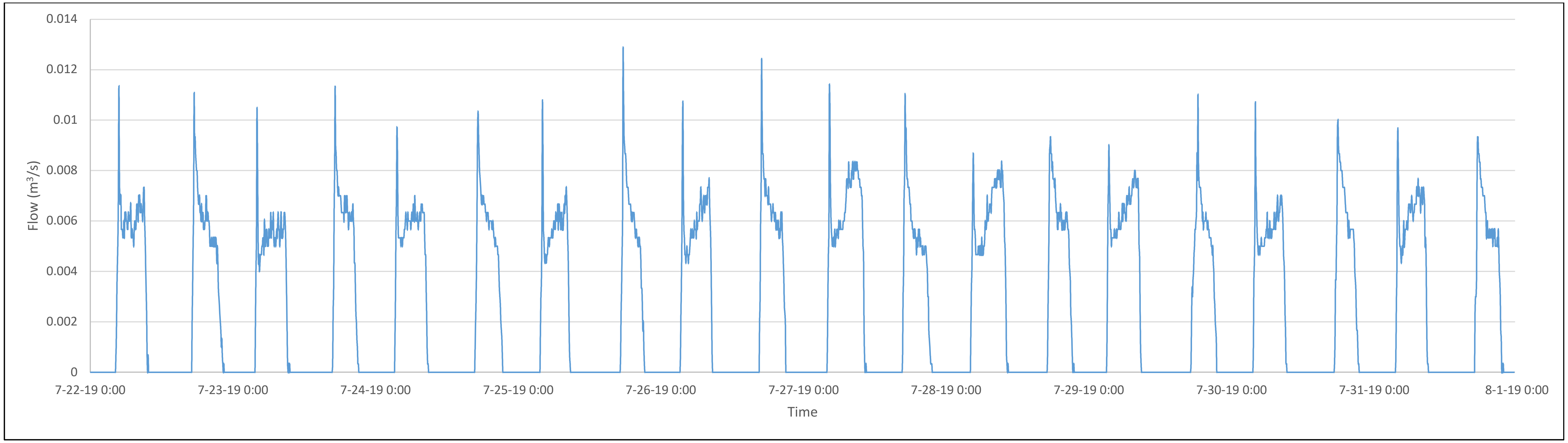


Figure C.7 Daily flow record for July for Site 2 (Toad Street).

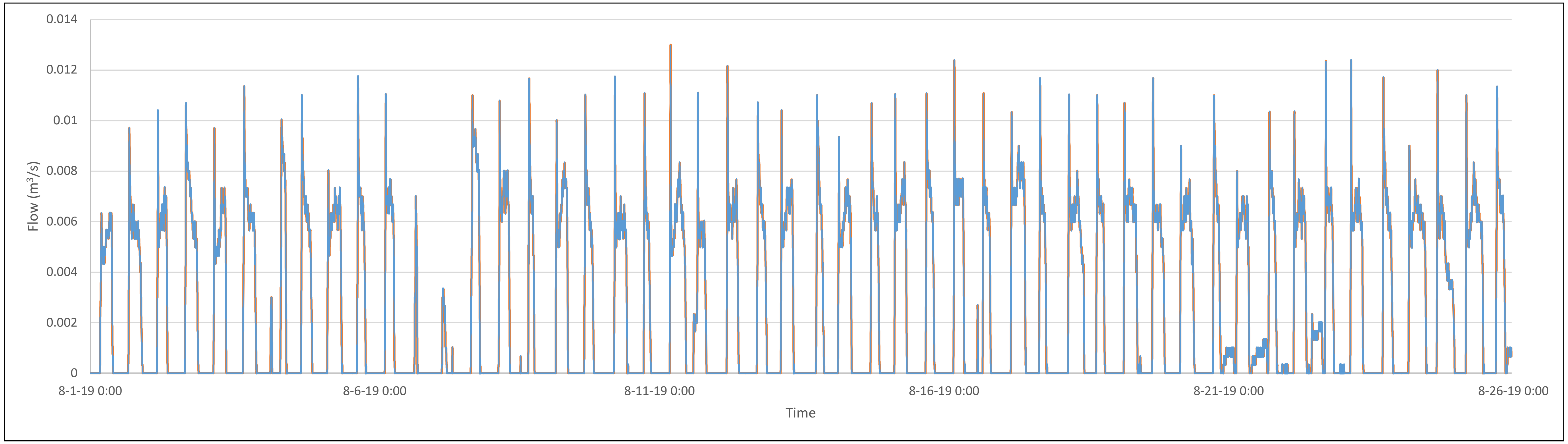


Figure C.8 Daily flow record for August for Site 2 (Toad Street).