

DEVELOPMENT OF A WATER NETWORK MODELLING SYSTEM TO EVALUATE EXISTING MUNICIPAL WATER INFRASTRUCTURE IN DEVELOPING COUNTRIES: CASE STUDY OF SWAKOPMUND, NAMIBIA

by

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DECLARATION

I, the undersigned, declare that the dissertation hereby submitted by me for the degree *Master of Engineering in Civil Engineering* at the Central University of Technology, Free State, is my own independent work and has not been submitted by me to another University and/or Faculty in order to obtain a degree. I further cede copyright of this dissertation in favour of the Central University of Technology, Free State.

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ABSTRACT

Geographical Information Systems (GIS) have become more easily accessible with the advent of open-source software such as QGIS. In combination with algorithms, referred to as plugins, written in the Python programming code, the conversion and processing of GIS data sets for use or further processing in other software programs have become less perplexing. The latter can be attributed to the fact that the computing software has become more efficient in processing large data sets with less manual user-input required.

The main purpose of this study is to develop a methodological approach based on the integration of open-source software, *e.g.*, QGIS and EPANET, and engineering practices associated with water distribution network design to enable practitioners to create a water distribution network (WDN) model of an existing network. A major contributing reason for the necessity to develop such methodological guidelines, is that of value-added engineering. By reducing the time required to set up a WDN model, time can be more effectively allocated to the actual technical analyses pertained to hydraulics, water loss management, water quality management and evaluation of network-specific challenges.

The Swakopmund Municipality in Namibia has been chosen as pilot case study. In other words, the methodology adopted focussed on the compilation of a WDN model for Swakopmund by: (i) compiling methodological procedures for the acquisition and processing of data, (ii) identifying various data sources which can be utilised in the absence of traditional data sets, (iii) conducting a sensitivity analysis to determine the effect of incorrect/inaccurate data on hydraulic results within a WDN model, and (iv) deriving equations from graphical sources to enable the automated implementation thereof in data processing software instead of relying on the manual interpretation and reading thereof from graphs and/or nomograms.

The use of QGIS and EPANET, as well as the integration thereof with sound engineering practices and judgement, proved to be both viable and practical. The methodological approach and associated process flow diagrams developed,

will not only provide guidance to practitioners, but ultimately, it could also assist municipalities in Southern Africa facing challenges in terms of outdated records and budget constraints.

Hence, it is envisaged that the implementation of the methodology and the recommendations for future research will contribute to improved water infrastructure planning, design and management, especially in the Southern African context where water resources are scarce and service delivery remains a pressing issue.

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LIST OF ABBREVIATIONS

AADD	Average Annual Daily Demand
AC	Asbestos Cement
CAD	Computer Aided Design/Computer Aided Draughting
CSIR	Council for Scientific and Industrial Research
CSV	Comma Separated Values
DBF	Database File
DEM	Digital Elevation Model
EPANET	Open-source Hydraulic Modelling Software
FL	Floor Level
FSL	Full Storage Level
GIS	Geographical Information Systems
GPS	Global Positioning Service
HPP	Harambee Prosperity Plan
ID	Identification
INP	File Extension associated with EPANET
NSL	Normal Service Level
RGB	Red, Green and Blue
SCADA	Supervisory Control and Data Acquisition
SG	Surveyor-General
SRTM	Shuttle Radar and Topography Mission
QGIS	Open-source GIS Processing Software
uPVC	Unplasticised Polyvinyl Chloride
USA	United States of America
USEPA	United States Environmental Protection Agency
USGS	United States Geological Survey
WDN	Water Distribution Network

CHAPTER 1 : INTRODUCTION

This chapter provides some background on the use of computer modelling systems and open-source software to develop a methodology and guidelines for the implementation, analysis and simulation of a water network modelling system. Thereafter the problem statement is discussed, followed by the purpose of the study, the study assumptions and the specific objectives. The outline of the dissertation structure is provided in Section 1.4.

1.1 Background

Water resources management, planning and supply form an integral part of the functionality of water utilities (e.g., local authorities, municipalities and/or water boards) throughout the world (Ormsbee, 2006). This is particularly the case in semi-arid and arid regions where rainfall is low and water resources are scarce. The use of computer modelling systems for the simulation of water distribution networks (WDNs) can aid water utilities with the identification of problem areas within a water supply system, as well as enable the planning processes required for the ever-changing urban landscape. It remains an ominous challenge to the practitioner to make provision for rapid expanding cities, urbanisation, development and population growth in the 21st century (McKenzie *et al.*, 2003). Availability of data and the format in which the data are available, pose significant challenges to the effective management of WDNs. By developing a more standardised method in which data can be stored, evaluated and processed to enable the modelling of a WDN, better management practices could be implemented, specifically within the Namibian and Southern African context.

Various studies (e.g., Saminu *et al.*, 2013; Gama *et al.*, 2015; Ramana *et al.*, 2015; Georgescu *et al.*, 2016; Kara *et al.*, 2016; Shital *et al.*, 2016; Iglesias-Rey *et al.*, 2017) focused on the actual analysis of network models; however, this study aims to develop methodological guidelines for the implementation of a water network modelling system by means of open-source software, *i.e.*, EPANET and QGIS. EPANET is a water distribution system modelling platform to enable information sharing between different technical departments (Gama *et al.*, 2015), while QGIS

offers many common GIS (Geographic Information Systems) functionalities provided by core features and plugins (QGIS Development Team, 2016).

1.2 Problem Statement

Internationally, numerous research has been published on the relevance and applicability of WDN models implemented using EPANET (Saminu *et al.*, 2013; Gama *et al.*, 2015; Ramana *et al.*, 2015; Georgescu *et al.*, 2016; Kara *et al.*, 2016; Shital *et al.*, 2016). Detailed guidelines on the compilation of a WDN model from existing GIS and end-user data are generally not available, especially within the Southern African context. According to Berardi (2014), data availability is a major issue in many water utilities. Although, some water utilities have network data in GIS format, the accuracy is not always acceptable and some data are missing. In addition, the lack of efficient decision-support systems, that might exploit detailed information of the WDN, further reduces the motivation for improving the accuracy of data collection (Berardi, 2014).

Saminu (2013) and Shital (2016) describe the compilation of a WDN model by manually entering the data into the EPANET interface. However, the utilisation of different methods by means of which large data sets are processed in QGIS, and subsequently exported to EPANET, have not been fully implemented in a single study or method. The use of a transitional package, *i.e.*, plugin, to convert GIS data into EPANET readable file formats may be required. An example of such a plugin is *GHydraulics*, which is also an open-source plugin available in QGIS (Macke, 2014). In addition, no comprehensive methodologies for the development of a WDN model based on existing GIS data are available. Methodologies that do exist, *e.g.*, Saminu (2013) and Shital (2016), do not address the extensive acquisition and/or processing of data and the tedious development of a WDN model.

Advances in open-source software have resulted in the development of numerous add-in/plugin packages being developed for a wide range of applications within QGIS. The majority of these add-ins in the public domain are made available free of charge and are quite frequently updated. Therefore, it is necessary to realise

that the development of new WDN modelling tools is an on-going process that evolves along with advances in technology on almost a daily basis. In addition to these add-in packages, there are a variety of tools currently available in QGIS, which enables the practitioner to process large amounts of data and perform topological analyses on GIS-based feature classes (shapefiles) and data; all with the aim of creating a representative WDN model.

A notable challenge faced by the practitioner, are the technical capabilities of the end-user of the WDN model. It is thus necessary for the model developer to ensure that the functionality and the interface of the software are explained in adequate detail to the end-user. Practical issues that may arise during the development of the WDN model include, but are not limited to, the following (Paulsson, 1992; Cooper, 1993; Pickering *et al.*, 1993; McKenzie, 2014):

- (a) Shortfall, backlog or inaccuracy of metering data;
- (b) Incomplete or incorrect cadastral (base map) data;
- (c) Unreferenced metering systems (*i.e.*, no link between cadastral data and metered records);
- (d) Out-dated digitised data;
- (e) Incorrect digitised data;
- (f) Digitised data stored in an incompatible format (*i.e.*, text objects cannot be converted into attributes for a shapefile or text indicating diameter and material was converted to geometry objects);
- (g) Poorly digitised data (*i.e.*, duplicate geometry objects); and
- (h) Different geographical projections applied to different network components.

Technical problems that may affect the compilation and eventually the accuracy of the WDN model include, but are not limited to, the following (Pickering *et al.*, 1993; Mays, 2000; Gama *et al.*, 2015; Sanz and Pérez, 2015; Trifunovic, 2015):

- (a) Pipe materials are not specified, resulting in unknown friction loss coefficients;
- (b) Pipe diameters are not specified;
- (c) Status of valves is unknown (*i.e.*, open/closed valves);
- (d) Configuration of crossing pipes is unknown;

- (e) Connectivity between parallel and/or crossing pipes is unknown;
- (f) Water supply zones are not defined;
- (g) Main supply lines are not defined;
- (h) Decommissioned pipelines are not identified; and
- (i) Bulk metering data are not available.

1.3 Purpose of Study

The main purpose of this study is to develop a methodological approach based on the integration of open-source software, *e.g.*, QGIS and EPANET, and engineering practices associated with water distribution network design to enable practitioners to create a WDN model of an existing network. Such a methodological approach will not only provide guidance to practitioners, but ultimately, it possibly could also assist municipalities in Southern Africa facing challenges in terms of outdated records and budget constraints. Hence, the motivation or need for such a methodology based on software that does not require annual licensing or additional capital expenditure.

The study focusses on details regarding the connection between different databases, *e.g.*, linking financial account data with the available cadastral data in order to determine demand allocations to nodes. The latter approach is required for instances where metered data is not linked to Global Positioning Service (GPS) or other location data. Furthermore, demands have to be allocated to nodes within the network and the allocation should be such that an acceptable approach is utilised.

A major contributing reason for the necessity to develop such methodological guidelines is that of value-added engineering. Time saved during the setup of a WDN model, can be more effectively allocated to the actual technical analyses pertained to hydraulics, water loss management, water quality management and evaluation of network-specific challenges. WDN models for larger cities should no longer be seen as a tedious exercise due to the advances in GIS software to process large data sets; provided that, the data sets are up-to-date and well maintained.

1.3.1 Assumptions

The study is based on the following assumptions:

- (a) **Assumption 1:** A WDN is a closed system with known inflows and outflows.
- (b) **Assumption 2:** All valves in the network are open, unless specifically indicated to be closed by confirmed accounts.

1.3.2 Study Objectives

To achieve the research aims, the specific objectives are to:

- (a) Compile methodological guidelines for the acquisition and processing of data for the compilation of a WDN model;
- (b) Identify various data sources which can be utilised in the absence of traditional data sets;
- (c) Conduct a sensitivity analysis to determine the effect of incorrect/inaccurate data on hydraulic results within a WDN model; and
- (d) Derive equations from graphical sources to enable the automated implementation thereof in data processing software instead of relying on the manual interpretation and reading thereof from graphs and/or nomograms.

1.4 Outline of Dissertation Structure

The remainder of this dissertation is organised as follows:

Chapter 2 contains a comprehensive literature review of the open-source software used in this study, the administrative processes typically followed by water utilities in terms of records and asset management, and an overview of the technical data requirements to compile a WDN model.

Chapter 3 provides an overview of the location and general characteristics of the study area, *i.e.*, Swakopmund, Namibia.

In Chapter 4, the detailed methodology adopted in meeting the specific study objectives is discussed. This chapter focuses on the compilation of a WDN model and the associated data acquisition processes. Chapter 5 presents the results based on the methodology followed in Chapter 4, with some further discussions included in both Chapters 5 and 6, respectively.

The final conclusions and general recommendations for future research are included in Chapter 6.

CHAPTER 2 : LITERATURE REVIEW

The literature review in this chapter provides a background and overview of the open-source software packages used in this study. Furthermore, the social and legal responsibilities of water utilities are discussed along with practical considerations pertained to record-keeping by the water utilities. The literature substantiating the proposed methodology to be developed in this study, as well as the steps to be followed to compile a WDN model, are discussed as part of the literature review.

2.1 Applicable Legislation in Water Demand Management and Supply

Service delivery to disadvantaged communities has been identified as a high priority in order to address all the social-economic inequalities of the past; and this is also in line with the South African Constitution Section 27(1) (b) (Constitution of the Republic of South Africa, 1996). In addition to the constitutional obligation, the South African Water Services Act requires of every South African municipality to have an asset management plan for both their water and sanitation infrastructure, respectively (Blom *et al.*, 2018).

Tjiramba (2009) stated that: 'The Namibian Constitution does not explicitly guarantee the right to water. However, Article 95(j) enjoins the Government to actively promote and maintain the welfare of the people by adopting, *inter alia*, policies aimed at the consistent planning to raise and maintain an acceptable level of nutrition and standard of living of the Namibian people and to improve public health.' Although the Namibian Constitution does not make specific mention of water service delivery, programmes such as the Harambee Prosperity Plan (HPP), contains milestones in terms of service delivery that were set in order to uplift communities with no basic services such as water and sanitation (Government of Namibia, 2016).

Bearing in mind that the responsibility of water utilities is not merely an administrative and financial function, but also directly and indirectly a constitutional responsibility, it is of utmost importance that adequate record-keeping is maintained in order for sufficient and responsible service delivery to be guaranteed.

2.2 Obligation by Water Supply Utilities to Maintain Adequate Records

According to Pickering *et al.* (1993), no sensible development, upgrading of infrastructure or maintenance can be done in urban areas without sufficient record-keeping and the management thereof. Over years, various methods of record-keeping, specifically related to infrastructure and underground services, have been adopted by water utilities. These include, but are not limited to, normal grain paper drawings, sepia drawings, micro-film records and of course in more recent years, digital drawings. The updating of records is usually neglected where the media of data differs vastly. Especially where data overlap, with only minor upgrades or alterations indicated on different drawings for the same area, the updating and maintaining of the records become a tedious process; however, digital mapping can alleviate most of these obstacles (Pickering *et al.*, 1993). With the utilisation of GIS, digital mapping advances to a more comprehensive level. Digitised components can be converted into entities that do not merely serve as a visual representation of objects, but also have an underlying group of attributes containing a multitude of data bearing reference to the visually represented entities.

For example, as-built data formed an integral part of leakage or water loss reduction projects as implemented by the Ekurhuleni Metropolitan Municipality in South Africa (McKenzie, 2014). A municipality or water utility should maintain records from their maintenance teams regarding the extent and location of pipe bursts, replacements, valve leakages and pressure issues. In some municipalities, *e.g.*, Tshwane Municipality, the final payment for the construction of water services is dependent on the prior submission of as-built drawings to the technical department (Tshwane Water and Sanitation Division, 2010). This is a notable practice that should strongly be considered by water utilities where the latter is not already enforced.

As stated by Mays (2000): 'WDNs do not exist in a vacuum'; consequently, WDNs form part of an integrated data system which relies on data from different departments within the water utility which could include engineering, GIS, financial, customer service, maintenance and monitoring departments. With the combination

of data from all the aforementioned departments, it becomes possible to manage a water utility effectively and with a broad spectrum of information at hand to evaluate shortfalls and areas that require attention. One such area may include a pipe replacement schedule. Over the years, the focus of the majority of water utilities was on the expansion of infrastructure to meet the ever-increasing water demands, while the design life of pipe materials was ignored. Many water utilities are caught in a predicament between the maintenance/replacement of old pipe infrastructure and the expansion requirements of the WDN (Mays, 2000). According to Jamer (2015), a water utility has an efficient asset management programme if sufficient funding is available to maintain assets throughout their life cycle, and ultimately, to replace assets at the end thereof. Replacement may be postponed based on situational analyses of the infrastructure by means of inspections and testing.

2.3 Digitising of Records

In practice, especially in South Africa, numerous situations are experienced where technical personnel with vast technical experience and knowledge, reach retirement age without any of this information being properly documented in any archives, recorded on any systems or transferred to an apprentice. Pickering *et al.* (1993), states that it is common to see these same persons being brought back on a sub-consultancy basis to assist with the updating of records and location of services. However, it is also noted that no amount of technology can entirely replace the in-house knowledge of skilled staff members with in-depth field experience. Subsequently, the need for sustained record-keeping cannot be over-emphasised.

Due to the continuous need to keep WDN models up-to-date, water utilities typically become overwhelmed by the huge task and associated processes (Gama *et al.*, 2015). GIS adds the advantage to maintain large data sets in attribute tables which are linked to the spatial features. In addition to the attributes that can be added to the physical infrastructure entities, the database information can also be visualised and linked to GIS entities. Maps visualising high-maintenance areas, high-pressure zones, pipe burst events, high consumptive use

or infrastructure approaching its design life can be compiled. It provides a versatile tool to communicate technical information across the entire spectrum of departments within the water utility. The application of GIS for the digitisation of as-built data and records is discussed in more detail in Section 2.9.4.

2.4 Geographical Information Systems

In South Africa, the Council for Scientific and Industrial Research (CSIR) is regarded as pioneers in the South African GIS fraternity (Van Riet and Vosloo, 1990). According to Heywood (1998), GIS are essentially digital models containing data representative of the real world. Since the 1960s with the advent of computers, both Computer Aided Design (CAD) and GIS systems have been used to present spatial data (Van Riet and Vosloo, 1990). GIS utilise features consisting of vector and/or raster data. According to Van Riet and Vosloo (1990) and Cooper (1993), vector data could be defined as spatial data recorded as co-ordinate tuples forming nodes and chains, while raster data are spatial data expressed as a tessellation of cells (usually rectangular), with the spatial position implicit in the ordering of the cells. The most common source of raster data is remote sensing, *e.g.*, satellite imagery, aerial photography, and topographical data (Van Riet and Vosloo, 1990; Cooper, 1993; QGIS Development Team, 2016).

In the context of data acquisition as to be discussed later in this dissertation, scanned municipal drawings that have been georeferenced will form part of raster data. It is possible to convert vector data to raster data by means of automated processes. The contrary is also true and the process is referred to as vectorising; however, the process tends to be sensitive as it relies strongly on pixel density and quality and can be adversely affected by ‘noise’ in the raster data (Cooper, 1993). Digitising of scanned municipal drawings is in most cases done by tracing over a georeferenced drawing; thus, creating separate vector layers in which features are created and attributes populated by the operator based on manual input. Typical examples thereof are shown in Figures 2.1 and 2.2, respectively.

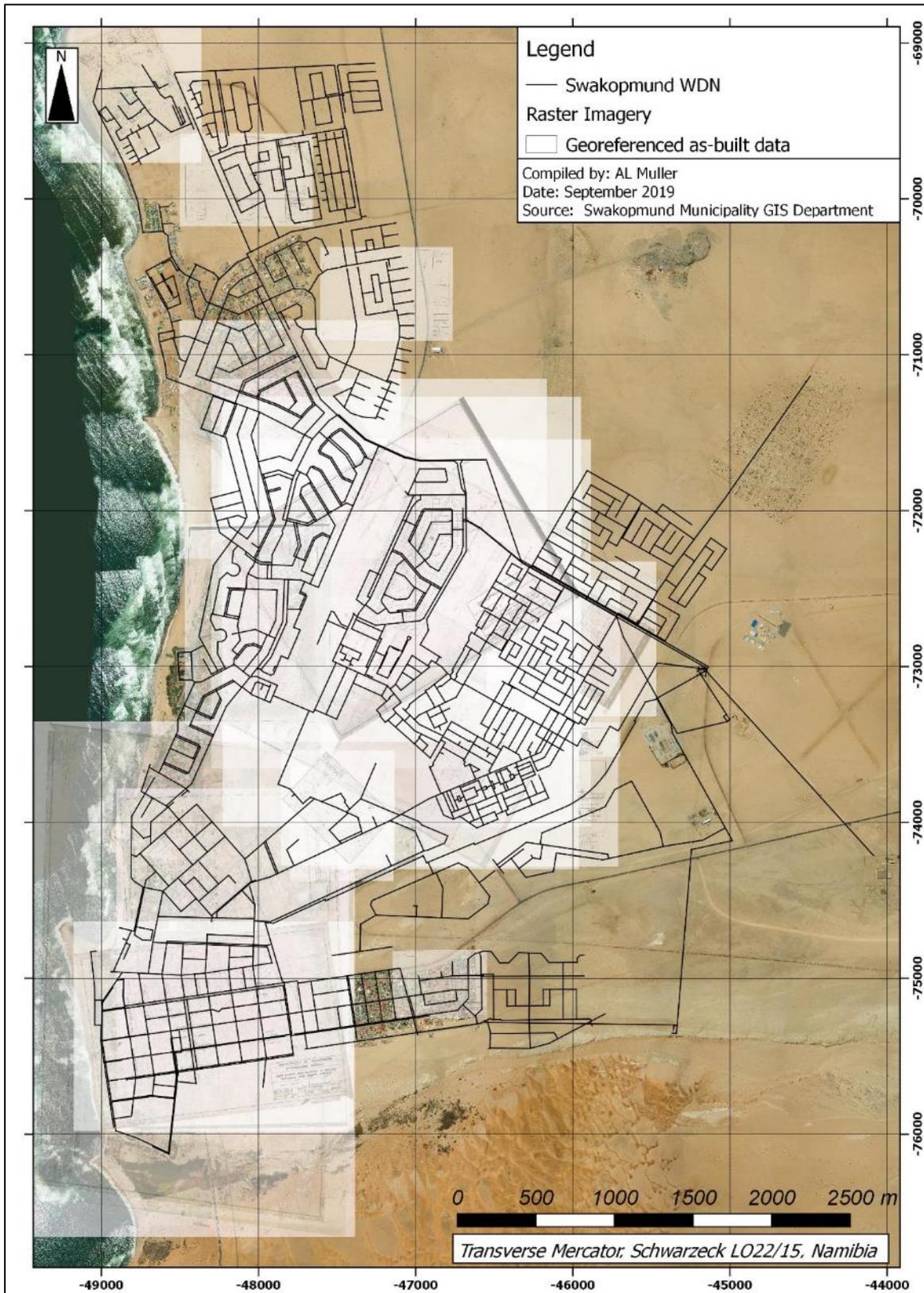


Figure 2.1: Georeferenced drawings for Swakopmund Municipality



Figure 2.2: Vector data digitised from georeferenced drawings for Swakopmund Municipality

Typically, in the context of municipal services, vector data are used to represent infrastructure in a GIS environment and will most likely originate from scanned (and possibly georeferenced) municipal drawings as shown in Figure 2.2. Pumps, reservoirs, tanks, junctions and valves are classified as point (node) data, while lines and polylines are used to represent pipes within a WDN model (Rossman, 2000). Point data refer to a single x- and y-coordinate reference point with attributes. Unlike point data, lines have a start and an end coordinate and in the case of polylines, intermediate changes in direction can also be presented. The three basic types of vector data are illustrated in Figure 2.3.

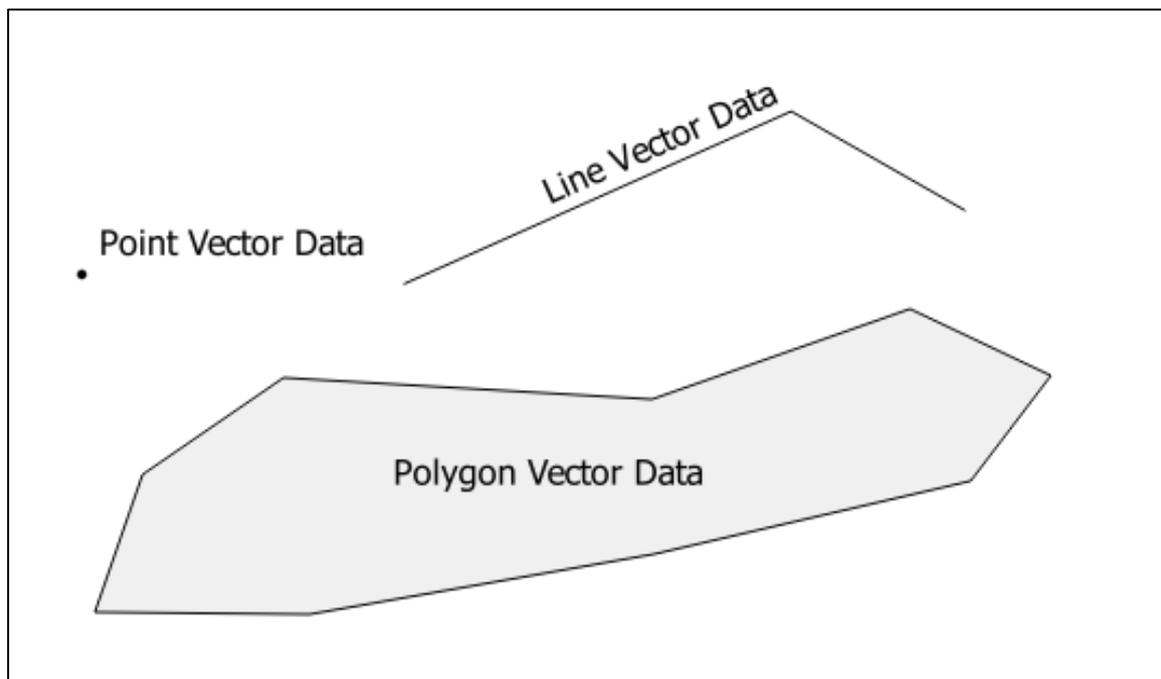


Figure 2.3: The three basic types of vector data

QGIS is open-source software freely available in the public domain and was released for the first time in 2002 (QGIS Development Team, 2016). The latest (as well as previous versions) of the software can be downloaded at the following link: <https://www.qgis.org/en/site/forusers/download.html>. In this study, QGIS Version 2.18 was used. The software package, with the option of free additional plugin packages, provides the practitioner with a comprehensive and versatile tool to process large data sets and manage information. Conversion into other file formats is possible; hence, QGIS acts as a platform for information management

and processing of data in order to enable the exportation of information to other software interfaces such as EPANET.

2.5 Phases of GIS Project Design and Management

The phases in which most GIS projects will manifest and be managed are linked to the actual systems' analysis and can be divided into four (4) stages (Heywood *et al.*, 1998):

- (a) **Lexical Phase:** A problem within a specific geographical area is identified and entities that influence, or are influenced by the problem, are defined. Attributes for these entities are then identified. Typically, this would originate from a client stating specific needs and requirements pertained to either the technical or administrative functionality within the organisational structures. In terms of this study, this phase may typically consist of a client identifying the need for the compilation of a GIS database of WDN components and subsequently the creation of a WDN model. The need for the compilation of such a database and/or model may originate from obligatory requirements (*cf.* Section 2.2, Chapter 2), legal requirements (*cf.* Section 2.1, Chapter 2) or from technical reasons as discussed in Section 2.5, Chapter 2.
- (b) **Parsing Phase:** Relationships and interconnectivity of entities are defined during this phase. With the development of GIS software, the compilation, maintenance and processing of large data sets have become increasingly more versatile; hence, the parsing phase is expedited in terms of processing. In practice, this phase would typically include obtaining data from the client, acquisition of data by means of field surveys, digitisation of paper records (as-built drawings, survey diagrams, *etc.*) and other means of data gathering. Methods for assigning attribute values to the entities include, but are not limited to, linking of records (*e.g.*, census data) or using GIS to establish geometric properties (*e.g.*, length, area, *etc.*). During the compilation of a WDN component database and a WDN model, the parsing phase also comprises the linking of nodes (digitised point data) with lines (digitised pipe data), ensuring that the correct geographical projection is used, obtaining and linking consumer data with nodes and the execution of topological validation.

- (c) **Modelling Phase:** Phases (a) and (b) above provide the foundation for the compilation of a GIS model. The entities and the information that were identified during the parsing phase are combined to create a digital model. Different scenarios that influence entities can be modelled by means of GIS, as well as other resultant behaviours. The execution of any EPANET analysis is dependent on both the data exported from GIS and the modelling phase as described in (c). Other actions such as Heatmap generation, *i.e.*, for the identification of high demand consumers, are usually built-in as part of the GIS software and are not reliant on external modelling packages.
- (d) **Analysis Phase:** This phase can be summarised as the calibration or control of the results and how accurately information relates to the real-world data and/or scenarios.

2.6 Information Interchange between QGIS and EPANET

The conversion from GIS to EPANET is accomplished by means of the *GHydraulics* plugin, which is also open-source software written in the Python programming language (Macke, 2014).

2.7 The Need for Water Distribution Network Models

WDN models are a mathematical representation of real-world entities that comprise a typical water distribution network (Walski *et al.*, 2003). Hardy Cross (1936) illustrated that a systematic mathematical approach may be followed to determine the flow distribution within complex networks. The impact of this latter systematic mathematical approach had a significant impact on subsequent developments pertained to water distribution modelling in the water industry and formed the baseline for initial computer modelling software (Cheng *et al.*, 2014). The versatile interface improved the usability of modelling software packages upon the release of open-source software in the public domain such as EPANET and other commercial WDN modelling software packages (Rossman, 2000). With variation in pressure and demands throughout the day, WDN modelling software needs to simulate the system behaviour under varying time periods and

demands; whereas initial models, could only perform a steady-state analysis. Typical applications of the software include system design, master planning and fire flow analyses (Panguluri *et al.*, 2005; Cheng *et al.*, 2014).

A mathematical model can be manipulated to simulate different what-if scenarios. As highlighted by Iglesias-Rey *et al.* (2017), the number of academic citations that make reference to the EPANET User Manual (Rossman, 2000), is remarkable. Today, almost 25 years after the first version of EPANET was released (Ormsbee, 2006), it is becoming even more evident that WDN models are essential for the effective management (both technical and administrative) of a successful and effective water utility (Iglesias-Rey *et al.*, 2017). Decision-making and planning are augmented by the better understanding of the system dynamics, as well as the predictive capability of a WDN model (Mays *et al.*, 2000). Furthermore, optimisation of existing protocols is made possible with a reduction in operation costs and adequate planning for future upgrades and/or expansions (Mays *et al.*, 2000). WDN models are used by water utilities and municipalities for a broad spectrum of applications (Sanz and Pérez, 2015), which typically could include:

- (a) **Water loss management:** Couvelis and Van Zyl (2012) stated that: ‘Water losses from water distribution systems are reaching alarming levels in many towns and cities throughout the world.’ WDN models can offer invaluable tools to mitigate water losses. For example, McKenzie (2014) indicated that the Drakenstein Municipality in South Africa uses EPANET to identify areas of high pressure and subsequently to establish pressure management zones. Upon the establishment of pressure management zones, existing and alternative operating configurations could be analysed to achieve reduction in excessive pressures in the network. In addition, the number and distribution of pressure reducing valves were also optimised based on the pressure zones. Although pressure is not the only contributing factor to water losses, it is the main driving mechanism behind water losses. Hence, the reduction or effective management of pressures within a WDN is often amongst the most cost-effective measures that can be implemented for reducing water losses

(McKenzie, 2014). In the Municipality of Castelo Branco, a similar approach was followed to identify high pressure zones and several mitigation measures were recommended (Alves *et al.*, 2014).

- (b) **Optimisation of pump scheduling:** The water utility that has a WDN model at its disposal, has the ability to evaluate alternative operation sequences and pump scheduling options (Gama *et al.*, 2015). A reduction in energy requirements with regard to pump operation leads to a direct saving in costs for any water utility. Optimised pump scheduling limits pump operation mainly to off-peak hours, during which the electricity costs are usually lower. Critical supply levels of reservoirs, specifically in terms of emergency storage volumes for fire-fighting purposes, can also be considered when scheduling is done. Once again, the WDN model forms the backbone of these operational procedures.
- (c) **Visualisation and modelling of a WDN from source to supply:** By making use of GIS software, it is possible to visualise, and model the entire cycle of a water supply network from source to household (Ramesh *et al.*, 2012). The latter functionality is a key management tool to communicate the intrinsic operation of a WDN to various departments within the water utility, even to those with no inherent technical background. Technical interdependencies of infrastructure components are frequently explained by visual representations during community awareness campaigns; GIS images in many cases form part of the visual material and brochures that are distributed to communities.
- (d) **Safety, risk identification and disaster management planning:** With more and more industries and major contributors to the economy being reliant on water supply, the importance of an uninterrupted water supply to these businesses should not be underestimated. According to Krocová (2016), except for scheduled maintenance activities, interruptions in water supply from water utilities cannot be foreseen. While this may be the case, it remains important for water utilities to have risk identification and disaster management plans in place. Such plans prepare supervisors and maintenance personnel to cope with untimely infrastructure or equipment failures, thus keeping the down-time in water supply to a minimum in such events (Salmivirta, 2015).

- (e) **System trouble shooting:** The interdependency of components and distribution zones are better understood when different scenarios could be simulated. For example, a WDN model provided a more intricate understanding of the water supply system's functionality for three neighbourhoods in the Municipality of Castelo Branco (Alves *et al.*, 2014). By utilising a series of simulations, areas of concern can be identified and technical personnel may be dispatched to the areas to confirm the *status quo* on site, after which, records may be updated (Alves *et al.*, 2014; Salmivirta, 2015).
- (f) **Evaluation of operational settings:** Operational settings (*i.e.*, closure of valves) can be evaluated within a simulation model without compromising service delivery. In other words, the operation of the actual system is not compromised to witness the effect of changes in the operational settings (Gama *et al.*, 2015).
- (g) **Network management:** WDN models provide effective expedients for managing complex data sets (Boano *et al.*, 2015; Kara *et al.*, 2016). A calibrated WDN model which presents the on-site infrastructure components accurately, serves as an indispensable decision support tool for managers (Alves *et al.*, 2014).
- (h) **Energy recovery:** Blom *et al.* (2018) stated that: 'Municipal infrastructure asset registers and asset management plans which contain water supply and distribution system data, should be leveraged to identify preliminary potential for energy recovery and leakage reduction.' Hence, WDN models can be used to identify high pressure zones in a network to enable energy recovery. Typically, energy recovery is done by means of an installed turbine generator at a point in the network where high pressure is present (Blom *et al.*, 2018). In essence, the turbine and generator act in a similar fashion as a pressure reducing station, except for the fact that both energy recovery and leak reduction is achieved (Blom *et al.*, 2018).
- (i) **Research:** WDN models may be made available for joint research (with the necessary modifications for security purposes) and academic applications to enhance knowledge sharing and further development of analysis software (Gama *et al.*, 2015).

2.8 Hydraulic Modelling using EPANET

Applications for the design of pipe networks were developed as early as the 1960s with the advent of the computer era. Since then, the calculation capability and technology has advanced to such a point where complex water distribution networks consisting of numerous pipes of various lengths, diameters and different materials, can be analysed in mere seconds (Mays *et al.*, 2000). As stated by Walski (2006), practitioners take for granted the fact that complex hydraulic analyses of WDNs can be done by a single 'click of a button'. Ramesh (2016) indicated that the use of EPANET and QGIS in combination saves time when the hydraulic parameters in WDN models are simulated. The latter point of view is also supported and substantiated by Shital (2012).

For the purposes of this study, EPANET Version 2.0 was used. The software can be downloaded at the following link: <https://www.epa.gov/water-research/epanet>. EPANET was developed by the United States Environmental Protection Agency (USEPA) in order to better understand the movement and behaviour of potable water during conveyance through distribution systems (Rossman, 2000). The software furthermore enables the practitioner to simulate and evaluate the hydraulics and water quality of a WDN using variable time periods. Upon the successful model set-up and associated test runs, the practitioner has the flexibility of analysing different scenarios that could affect the hydraulics and/or water quality in the system, *e.g.*, different diameters, isolation of distribution zones, addition of possible demands, *etc.* EPANET is open-source software available in the public domain for commercial, educational, institutional and private use (Ormsbee, 2006). The software is used worldwide for the comprehensive analysis of both intricate, as well as unsophisticated networks (Ramana *et al.*, 2015).

EPANET is widely regarded as the world standard in terms of hydraulic and water quality modelling software for the analysis of WDNs (Gomes *et al.*, 2009). EPANET enables the tracking of the flow rate in each pipe, the pressure at each node, the water levels in each reservoir, and the concentration of chemicals throughout the network for multiple time steps (Ramesh *et al.*, 2012; Ramana *et al.*, 2015). An EPANET model, as with a real-world WDN, consists of

various components as shown in Figure 2.4 and includes pipes, nodes, reservoirs, tanks, pumps and valves (Rossman, 2000). Each of the above-mentioned components have associated input parameters that influence the hydraulic modelling and results.

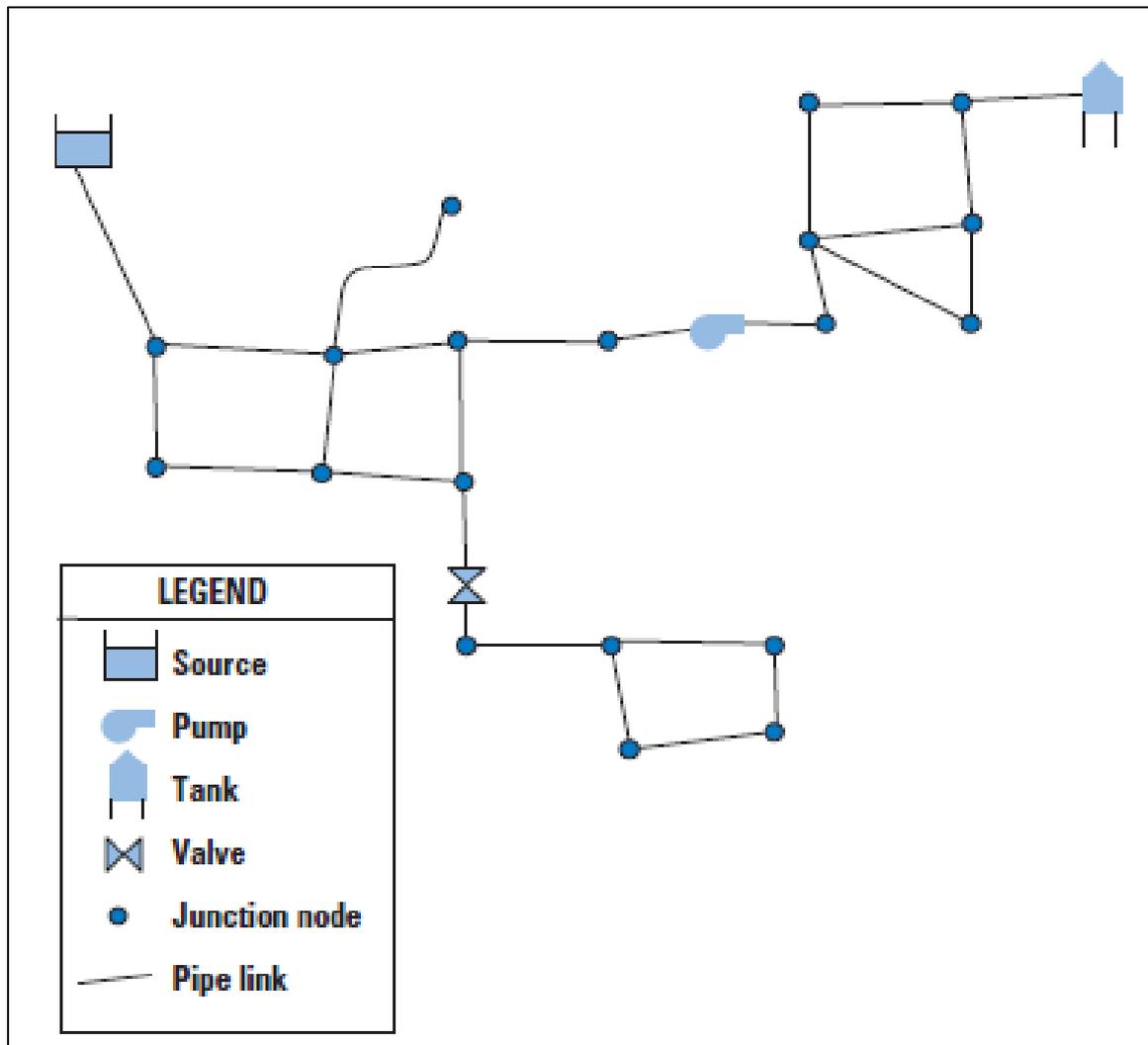


Figure 2.4: Simple link-node representation of a WDN in EPANET (Panguluri *et al.*, 2005)

It is important to note that some terminology used in EPANET is contradictive to the common use in Southern Africa, e.g., reservoir versus tank. Variables include, but are not limited to, the variables as included in Eqs. (2.1) to (2.3). Subsequently, elevation data added to nodes enable the calculation of static pressure differences throughout the WDN model. The chemical characteristics and concentrations of

chemicals in the water could also be included in a WDN model; although, the chemical data are not a prerequisite for any hydraulic analysis (Rossman, 2000).

In EPANET, friction losses within the WDN are calculated by means of either one of the following equations (Mays, 2000; Rossman, 2000; Panguluri *et al.*, 2005): (i) Darcy-Weisbach [Eq. (2.1)], (ii) Hazen-Williams [Eq. (2.2)], or (iii) Manning [Eq. (2.3)]. In essence, the Swaimee and Jain equation is used in EPANET to determine the Darcy-Weisbach friction coefficient as shown in Eq. (2.1a) (Mays, 2000).

$$h_f = f \frac{LV^2}{2gD} = 0.0826 \frac{Q^2}{D^5} Lf \quad (2.1)$$

$$f = \frac{0.25}{\left[\log \left(\frac{e}{3.7D} + \frac{5.74}{Re^{0.9}} \right) \right]^2} \quad (2.1a)$$

$$Re = \frac{VD\rho}{\mu} \quad (2.1b)$$

$$h_f = \frac{10.67Q^{1.852}L}{C^{1.852}D^{4.8704}} \quad (2.2)$$

$$h_f = \frac{5.29L(nQ)^2}{D^{5.33}} \quad (2.3)$$

Where:

- h_f = head loss due to friction (m),
- C = Hazen-Williams roughness coefficient (dimensionless),
- D = diameter of pipe (m),
- e = pipe friction value (m),
- f = Darcy-Weisbach friction coefficient (dimensionless),
- g = gravitational constant (9.81 m/s²),
- L = length of pipe (m),
- n = Manning's n -value (s/m^{1/3}),
- Q = discharge (m³/s),

- ρ = density of the fluid (kg/m³),
 R_e = Reynolds number (dimensionless),
 μ = kinematic viscosity (m²/s), and
 V = mean velocity of the fluid (m/s).

The Hazen-Williams equation [Eq. (2.2)] is used internationally to calculate friction losses in pipe systems (Walski, 2006; Sun *et al.*, 2015) and is also the most widely used equation in the United States of America (USA) (Panguluri *et al.*, 2005). Equation (2.2) is also considered to be more user-friendly than Eq. (2.1), while the C-factor tends to be more constant compared to those used by Darcy-Weisbach (Walski, 2006). Notwithstanding the fact that the Hazen-Williams equation [Eq. (2.2)] is used globally, many academics dislike this method and it is considered less accurate than the Darcy-Weisbach equation due to its more conservative results (Mays, 2000; Walski, 2006). Even though this view is held by some academics, results yielded by the respective equations compare to such an extent that differences are negligible under most circumstances (Panguluri *et al.*, 2005). The Manning equation is mostly considered for open channel flow conditions (Rossman, 2000); however, results using the latter equation proved to be comparable to the other equations' results. Both the Manning's n -value (Manning equation) and the C-value (Hazen-Williams equation) are empirically derived; however, the Darcy-Weisbach equation also relies on a dimensionless number based on the applicable flow regime, *i.e.*, the Reynolds number [Eq. (2.1b)], in order to determine the friction coefficient (Mays, 2000).

In order to understand the impact which individual variables or input parameters might have on the final modelling results using EPANET, it is necessary to quantify the impact of each variable. Mays (2000) regarded both friction coefficients and pipe diameters as the most sensitive parameters in Eqs. (2.1) to (2.3).

Practitioners from developed and/or developing countries have made extensive use of EPANET for WDN modelling (Ramana *et al.*, 2015). Since its release in 1993, EPANET has provided professionals and academics alike, with a free software tool that can comprehensively analyse complex or simple WDNs

(Ormsbee, 2006). Despite the theoretical approach used in the hydraulic modelling of a WDN in EPANET, it is surprisingly evident that the modelling results are generally quite accurate, even prior to the calibration of the model (Gama *et al.*, 2015). In the Namibian context, EPANET has been used as part of the analysis of a water network for the town of Keetmanshoop (Salmivirta, 2015); however, the use of EPANET was limited to a transitional role as part of the conversion of data files.

2.9 Acquisition of Data

Spatial data pertained to the location of pipelines, demand nodes and supply nodes (*e.g.*, reservoirs and tanks) form the backbone of any WDN model. During the compilation of a WDN model, comprehensive data collection of the above-listed components, is of high importance (Gama *et al.*, 2015). This information can be obtained from client records (*e.g.*, as-built drawings, GIS data sets, CAD drawings, hard copies of archived drawings), client representatives (*e.g.*, maintenance personnel, operators and managers), field surveys and in some instances, aerial imagery for above-ground pipes (Pickering *et al.*, 1993). With the advent of GIS, data are nowadays better structured, maintained and stored by municipalities and other water utilities. Unfortunately, large backlogs do still exist, especially in the larger cities which may have in excess of 50 years' data in various recorded forms.

2.9.1 Demand values

The allocation of demand values to respective nodes within a WDN model can be done by either making use of actual metered data, counting structures with associated water demands or by referencing property zonings in conjunction with water usage parameters (Panguluri *et al.*, 2005). The latter two processes yield less accurate values in comparison to actual metered data. However, it is essential that metered data are linked to geographical coordinates.

Consumer data should provide information pertained to water demands for the purpose of creating a WDN model. In considering sources of network demands

and consumer data, Van Zyl *et al.* (2007) provided possible arguments against using meter readings for studying water demand:

- (a) Water meters do not exactly record the quantity of water used by the end-user.
- (b) It is difficult to access and extract historical water consumption data from municipal treasury databases.
- (c) Meter readings are not always taken consistently every month and may be estimated in some cases.
- (d) Meter readings may lack accuracy seen in the light that meters clock over or meters are replaced.
- (e) End-user information such as residential address, income level or user type contained in treasury systems, is not always correct.

Although the above-mentioned arguments in (a) to (e) are worth noting, the practitioner remains largely dependent on data from consumer billing records, especially in circumstances where no alternative records or sources of data are available. It remains the responsibility of the practitioner to conduct adequate data verification and to conscientiously work with the data received from water utilities.

Errors in residential addresses and factors that impede the establishment of a link between the treasury database and the GIS database, can have adverse effects on the compilation of a WDN model. Depending on the software used by the Treasury Department of a water utility, the linking of data to geospatial locations can be done with relative ease. A study conducted by Jacobs *et al.* (2004) considered records from more than 600 000 single residential units located in six different municipalities to determine a new guideline for estimating water demand in Southern Africa. In the latter study, Swift software records were linked to property codes from the Surveyor General (SG), which were subsequently linked to and displayed in GIS. It remains a challenging task to allocate water demands to single nodes across a WDN model with a wide range of users and varying demand categories which are often encountered within close proximity of each other (Trifunovic, 2015).

Metered data will enable the water utility to compile a WDN model based on actual historical demand patterns; however, should the water utility not have these records at their disposal or if an extension of a WDN model is required for planning purposes, demands may be allocated based on acceptable demand norms as set out in ‘The Neighbourhood Planning and Design Guide’ (Department of Human Settlements, 2019) or similar accepted norms. Internationally, demand norms for various categories of users have been published and adapted by different water utilities and institutions (Kok and Harris, 1992; Jacobs *et al.*, 2004; CSIR Boutek, 2005; Trifunovic, 2015). Factors that may affect demands, based on local conditions, should be taken into consideration when water demand figures are adopted. These factors include amongst others: (i) income level, (ii) level of service, (iii) type of zoning of properties, (iv) climate, (v) water charges, (vi) seasonal variations, (vii) network pressure, and (viii) stand size (Jacobs *et al.*, 2004; CSIR Boutek, 2005; Couvelis and Van Zyl, 2012).

Comparative demand norms for residential properties according to varying erf size and area are listed in Table 2.1. The variation (and correlation) in demand norms between different guidelines are also highlighted in Table 2.1.

Table 2.1: Comparative demand norms for residential properties according to varying erf size and area

Residential Demand Norms	Kok and Harris (1992)	CSIR Boutek (2005)	Tshwane Water and Sanitation Division (2010)	Department of Human Settlements (2019)
Residential Low Income Area ($A < 500 \text{ m}^2$)	500 l/house/day	500 l/house/day	700 l/house/day	600 - 800 l/house/day [460 – 670 m ²]
Residential Medium Income $501 \text{ m}^2 < A < 1\,000 \text{ m}^2$	1 000 l/house/day	750 l/house/day	1 200 l/house/day	800 – 1 000 l/house/day [670 – 1 000 m ²]
Residential High Income $1\,001 \text{ m}^2 < A < 1\,500 \text{ m}^2$	2 000 l/house/day	1 000 l/house/day	2 000 l/house/day	1 000 – 1 300 l/house/day [1 000 – 1 300 m ²]

Demands allocated to nodes determine the flow rate present within a WDN model; subsequently, the associated head loss would also be impacted. Even though metered data may be available from a water utility, or if demand allocations should be done by means of assumptions based on zonings of the various erven, the process of assigning demand values to nodes within the WDN model, may prove

to be a tedious task. For this reason, Voronoi polygons (also known as Thiessen polygons or Dirichlet cells) are adapted to assign demand values to the individual nodes within the WDN model (Burrough, 1988). Voronoi polygons represent areas created from irregularly spaced points, *i.e.*, nodes of a WDN, in such a fashion that the area which surrounds the node point within that polygon are closer to the control point than any other points (Dale and McLaughlin, 1988).

Erven are typically represented by polygons within a GIS database. As part of the attributes associated with the polygons, derived water demand or historical water use patterns may be included in the attribute fields. In using the *Polygon Centroids* geospatial algorithm in QGIS, the geometrical centre of an element is returned as a single point, containing all the attribute fields of the polygons used to create the centroid (QGIS Development Team, 2016). By making use of the *Join Attributes by Location* function, all specified point data located within a specific polygon layer can be summarised and the results are allocated to the polygon attribute table. These summed values, especially the water demands, are then linked to the WDN nodes with corresponding attribute code values.

2.9.2 Demand patterns

The analysis of a WDN cannot only be based on average values, since consumption patterns vary throughout the day and multipliers are required to simulate these variabilities (Trifunovic, 2015). In general, a distinction can be made between instantaneous, daily (diurnal), weekly and annual (seasonal) peaks. The larger the area to which a certain peak factor applies, the greater the variation will be due to the variation in consumer types and the inclusion of leakages (Trifunovic, 2015).

An example of a typical 24-hour demand pattern at 1-minute intervals for a municipal area in Amsterdam, which serves about 130 000 consumers, is shown in Figure 2.5.

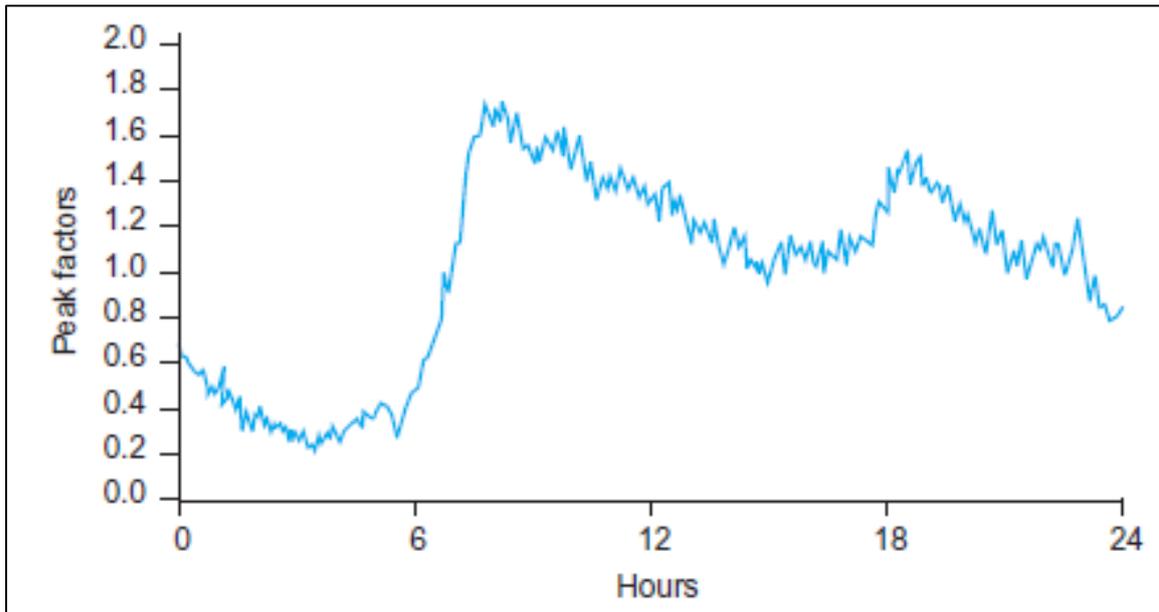


Figure 2.5: Typical 24-hour demand pattern in Amsterdam (Trifunovic, 2015)

In Figure 2.5, the morning and evening peaks are clearly distinguishable; however, the multipliers are much lower than those values specified by the CSIR Boutek (2005). This may be attributed to the likelihood that the metered values used to construct the demand pattern, included losses recorded downstream of the bulk meters.

Peak factors that influence the demand patterns will vary depending on the zoning and type of area served by the network. Detailed peak factors can only be derived based on actual metered data representative of (at least) hourly consumptions and a uniform consumer type served by the specific meter. In the event of clustered zonings, *i.e.*, industrial areas, different demand patterns based on site-specific conditions and peaks obtained from individual bulk consumers, should be applied (Trifunovic, 2015). Differences in hourly peak factors are clearly evident in the example shown in Figure 2.6, especially at 08:00.

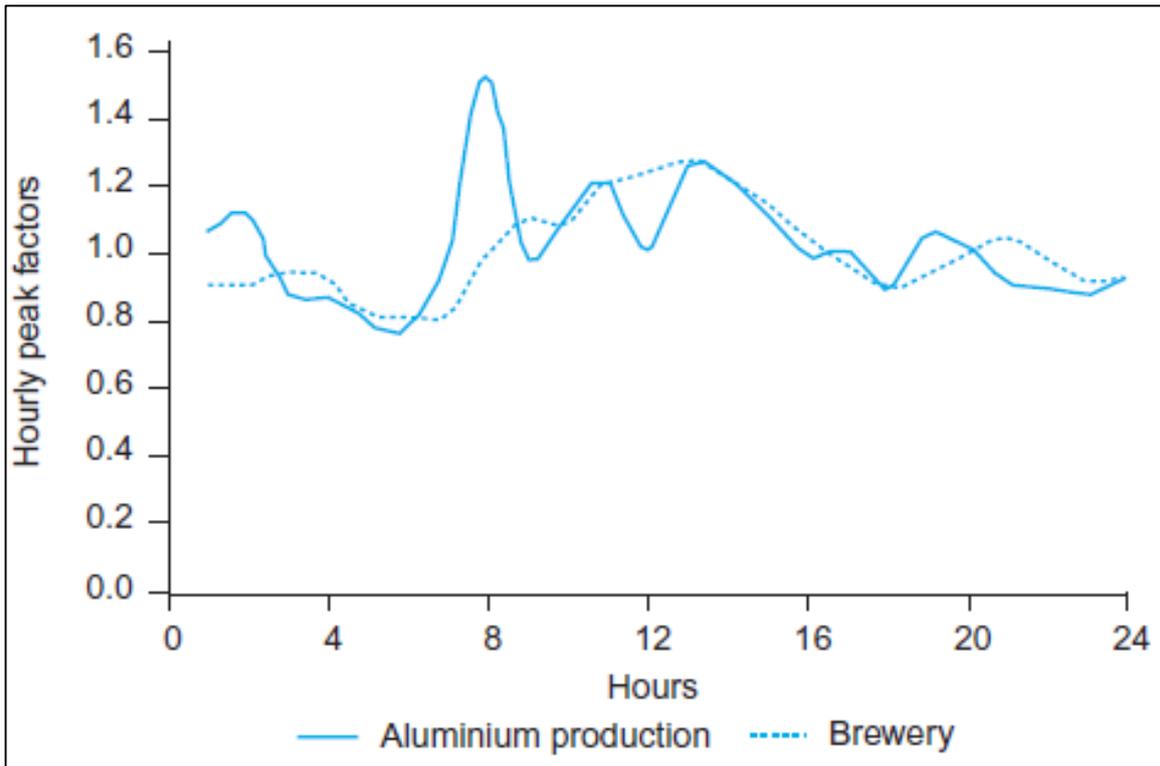


Figure 2.6: 24-hour Demand patterns for two industrial applications in Bosnia and Herzegovina (Trifunovic, 2015)

According to the CSIR Boutek (2005), the number of equivalent erven needs to be calculated in order to determine the instantaneous peak factor for developed areas. The number of equivalent erven is calculated by dividing the sum of the average annual daily demand (AADD; in litres/day) for all the erven by 1 000; based on the principle that one equivalent erf uses 1 000 litres of water per day.

Upon the calculation of the number of equivalent erven, Figure 2.7 can be used to determine the instantaneous peak factor pertained to any point in a WDN.

Guidelines for weekly, daily and hourly peak factors were developed and included in the recent ‘The Neighbourhood Planning and Design Guide’ (Department of Human Settlements, 2019). Table 2.2 shows hourly peak factors based on predominant land use which can be used for the compilation of a daily demand pattern.

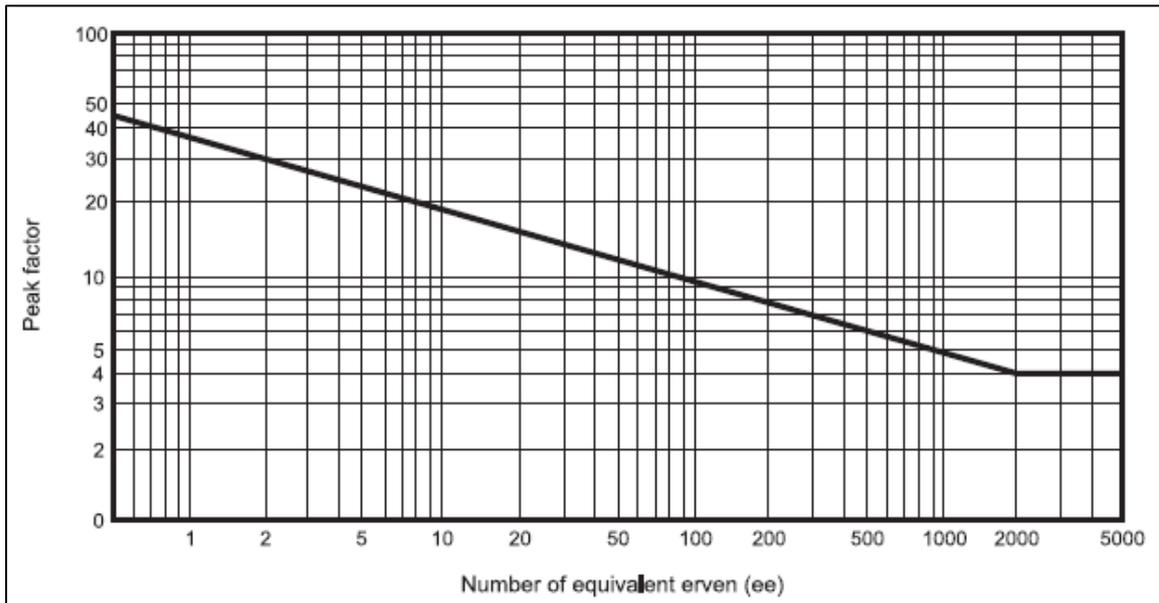


Figure 2.7: Instantaneous peak factor (CSIR Boutek, 2005)

Table 2.2: Recommended peak hour factors (Department of Human Settlements, 2019)

Recommended peak hour factor based on land use		
Predominant land use	Sum of all erven AADD (kℓ/d) in supply zone	Hourly Peak Factor
Low-income housing (LIH)	<1 000	3.6
	1 000 - 5 000	3.4
	5 000 - 10 000	3.3
	10 000 - 15 000	3.2
	15 000 - 20 000	3.1
	>20 000	3
Residential (RES)	<1 000	4.6
	1 000 - 5 000	4
	5 000 - 10 000	3.6
	10 000 - 15 000	3.5
	15 000 - 20 000	3.3
Business/commercial/industrial (BCI)	>20 000	3
	<5 000	3.3
	5 000 - 10 000	3.15
Large single users (LRG)	>10 000	3
	>500	2.5
Inner city (CBD)	<5 000	2

A typical example of metered data representative of a residential supply zone over a period of 24 hours, is illustrated in Figure 2.8. The variability in peak water consumption and the high leakage rate associated with night flows are evident.



Figure 2.8: Varying demand over a 24-hour period (McKenzie, 2014)

Again, as in the case of Figure 2.5, the morning and evening peaks are also easily distinguishable. A fairly large quantity, *i.e.*, 80 m³/h can be considered as losses associated with night flows. Although leakages could take place throughout a 24-hour period, night flows are still regarded as a good indicator of unaccounted for water, since the ‘non-leakage’ water consumption is normally limited to night time (Mays, 2000).

2.9.3 Elevation data

Besides the functionality offered by raster files in terms of as-built drawings, GeoTiff files can contain elevation data associated with each georeferenced cell. Pressures within a network are dependent on the available head differences between the supply points and the delivery points. Hence, an integral part of WDN modelling is to allocate elevations to nodes contained in the model. Topographical data for the area within which the WDN is situated, should generally be available from the municipality. While Shuttle Radar and Topography Mission (SRTM) data are analysed to derive morphometric parameters during the compilation of a hydrological model for flood estimation (Ezz, 2017), the same information can be

used during the compilation of a WDN model. The 1-arc second (± 30 m) SRTM data obtainable from the United States Geological Survey (USGS) may be utilised in order for elevations to be applied to the nodes (USGS, 2015). The latter SRTM data should however, only be used as a last resort due to the varying levels of accuracy and low resolution of recorded data (USGS, 2015).

2.9.4 Pipe network data

Sources of as-built data may vary from physical drawings, archived records, and local knowledge (Pickering *et al.*, 1993). Although as-built data form an essential part of records, it may be difficult to obtain and proved to be lacking in detail and relevance (Pickering *et al.*, 1993). Sources of as-built data could be scanned and converted into a raster format, *i.e.*, .tiff, .jpg, or .pdf file extensions. Raster data in GIS models consist of identically sized rectangular cells that are arranged in matrices which represent spatial features (QGIS Development Team, 2016). The *Georeferencer* plugin (*cf.* Figure 2.9) enables the referencing of raster files to a spatial coordinate system by creating a new GeoTiff file or by adding a world file to the existing image (QGIS Development Team, 2016).

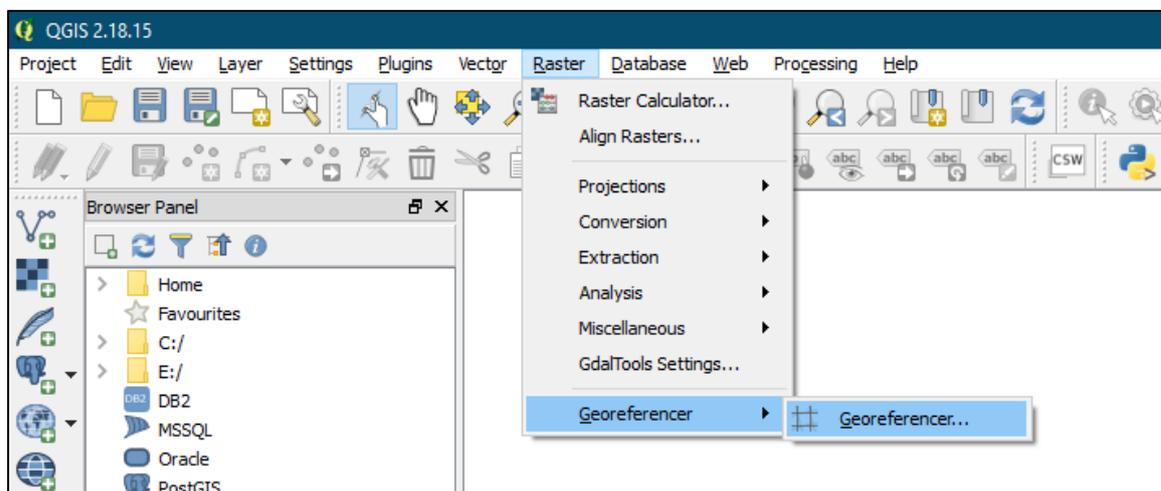


Figure 2.9: *Georeferencer* plugin (QGIS Development Team, 2016)

As explained before, the rudimentary methodology behind georeferencing a raster file is to locate points (at least three) on the raster for which coordinate values are known. Once a raster file has been georeferenced, positions of features may be digitised into vector shapefiles, typically consisting of points for valves, meters and

junctions, lines for pipes and polygons for erven and distribution zones (if available).

Furthermore, fieldwork including the digging of trial holes, underground surveys by means of electronic equipment and consultations with local artisans, may further supplement network data (Pickering *et al.*, 1993).

2.9.5 Reservoir and pump station data

As-built data may be sufficient for more 'sensitive' components of a WDN such as reservoirs and pump stations, provided that the data are reliable (Walski *et al.*, 2003). In cases where as-built data sources are not considered reliable, it may be more favourable to obtain reservoir data from field measurements (Gama *et al.*, 2015).

2.9.6 Assumptions of missing data

Machell *et al.* (2009) states that in situations where metered data are missing or corrupt (according to specific predetermined limits), three options may be applied automatically using operator-defined rules:

- (a) It can be replaced with a default value (which remains constant);
- (b) It can be replaced by an average of preceding values provided that those values do not include faulty or emulated data; or
- (c) A reference attribute value can be used in which measurements are substituted by the converted measurement from another attribute.

The above-listed operator-defined rules may apply to both missing meter readings and in principle, missing pipe data. With erroneous metered data, or data that deviate substantially from the average values over time, a longer consumption period for the data should be considered. Typically, all zero-consumption records should be omitted in order to establish a single database from multiple databases (Van Zyl *et al.*, 2007).

2.10 Calibration of Water Distribution Network Models

The extensive use of WDN models depends highly on the confidence that operators have in the results obtained from the analyses (Roma *et al.*, 2015).

A calibrated WDN model can be used for assistance in decision-making by a water utility (Alves *et al.*, 2014). Calibration data can also be incorporated into an EPANET model to make adjustments in order for the simulation model to reflect real-world observations (Rossman, 2000). A calibrated WDN model is regarded as efficient when both the simulated flow and pressures are representative of the real-world conditions (Savic *et al.*, 2009; Alves *et al.*, 2014).

Confidence in WDN models are highly dependent on the quality of the calibration of the model. Often it is encountered that calibration results are not available and the application of academic research and methods are not possible in practice (Sanz and Pérez, 2014). However, Savic (2009) suggested that the calibration of a model should be linked to the required level of accuracy and a sensitivity analysis should be conducted. This implies that the 'level of calibration' of a final WDN model should be established during the parsing phase (*cf.* Section 2.5); thereby, reducing the risk of project scope creep.

As with the case study of Swakopmund, Namibia, the results (*cf.* Chapter 5) obtained from the WDN model and actual measured pressures and flows were not compared, as the latter were not available from the Swakopmund Municipality. In cases where simulated results and measured data do not correspond satisfactorily, McKenzie (2014) indicated that the problem is most likely attributed to operational settings between the model and the actual WDN model that do not converge, and not necessarily due to any irregularities in the modelling software or approaches followed during the compilation of the model. An example may include valves which are closed while they should be open or crossing pipes that are linked in the model, while they are not linked in reality or *vice versa* (McKenzie, 2014).

The calibration of a WDN model is largely dependent on the adaptation of friction coefficient values, albeit the Darcy-Weisbach, Hazen-Williams or Manning roughness coefficients. Roughness values in older pipes may be significantly higher than those of new pipe networks due to incrustation (Krocová, 2016). Roughness values have a substantial impact on the analyses; hence, these values are important variables that need to be kept in mind during the calibration of WDN models (Krocová, 2016). It may be necessary to expose and sample existing pipelines in critical areas to conduct visual inspections on possible bio-film or tuberculation formations within the pipelines, as well as to establish the deterioration of pipeline material where applicable.

Two possible methods for the determination of head loss in a pipe section are documented in the EPA Calibration Manual (Panguluri *et al.*, 2005): (i) the two-gauge method, and (ii) the parallel hose method. These methods assist with the determination of actual friction coefficients within a WDN and should be conducted on representative sections of the WDN.

The standard two-gauge and parallel hose C-factor tests are shown in Figures 2.10 and 2.11, respectively.

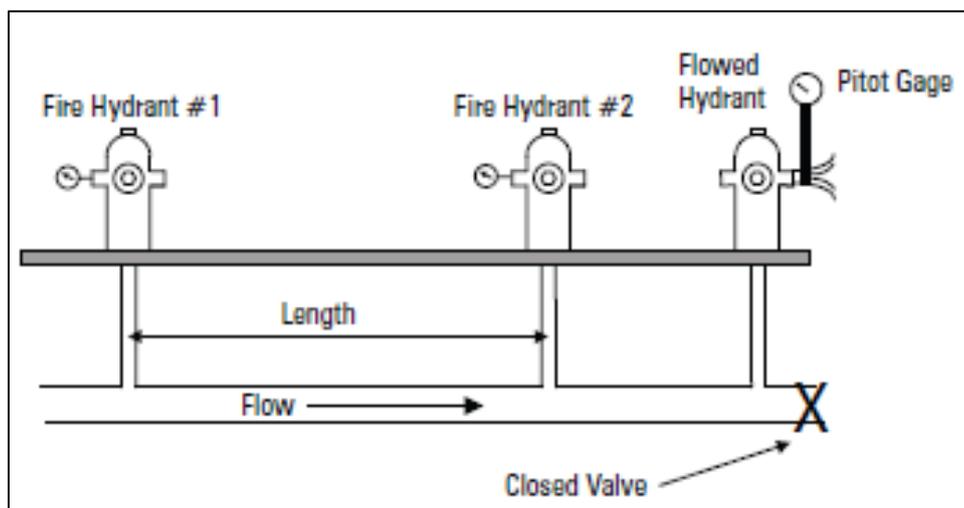


Figure 2.10: Standard two-gauge C-factor test (Panguluri *et al.*, 2005)

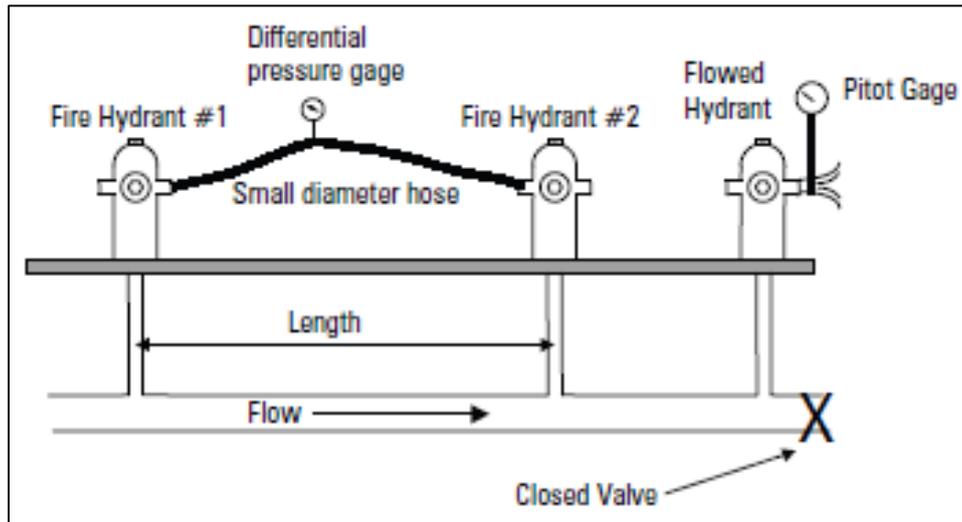


Figure 2.11: Standard parallel hose C-factor test (Panguluri *et al.*, 2005)

The following steps are applicable to the two-gauge test and are specifically aimed at determining the C-factor for the Hazen-Williams equation [Eq. (2.2)] to be used in a WDN model (McEnroe *et al.*, 1989; Panguluri *et al.*, 2005):

- (a) Identify a set of pipes in a WDN that are representative of the range of pipe materials, pipe age and pipe diameters in the system or section of the system.
- (b) Select a homogenous section of pipe between 130 m and 400 m in length which can be suitably isolated with hydrants and shut-off valves at the ends.
- (c) Record pressure at upstream and downstream hydrants.
- (d) Open downstream hydrant and measure the flow rate and dynamic pressure.
- (e) Calculate the head loss across the section of pipe considering the measured pressures and static head differences. A minimum head difference of 103 kPa is required.
- (f) Substitute all measured variables into Eq. (2.2) and calculate the resulting C-value.

The second method described by Panguluri *et.al.* (2005); the parallel hose method, is based on the same procedure as for the two-gauge method, except that a small hose is used to connect the two hydrants to a differential pressure gauge.

However, a minimum differential pressure of 20.68 kPa is required for the parallel hose method (McEnroe *et al.*, 1989). The two-gauge method is preferred and more commonly used due to the more specialised equipment required for the parallel hose method (Panguluri *et al.*, 2005). In both cases, a hydrant downstream of the test section is opened to induce flow and a sufficient pressure drop. Multiple downstream hydrants may also be employed to induce a higher flow rate and larger pressure drop (Panguluri *et al.*, 2005). In the event of calculated *C*-factors being very low, detailed measurements of the pipe diameters should be done to determine the actual diameter of the pipes (Panguluri *et al.*, 2005). The Hazen-Williams equation, Eq. (2.2), is the only equation for which the friction coefficient decreases with an increase in surface roughness (Mays, 2000). *C*-factors of less than 100 may be indicative of reduced effective diameter; however, the cause of the lower calculated *C*-factor should be investigated as part of the calibration process or maintenance procedures (Mays, 2000).

As part of flow rates and pressures to be modelled, the associated demand patterns, which comprise hourly fluctuations within a predefined time period, play an important role in the calibration of a WDN model. As a result of Supervisory Control and Data Acquisition (SCADA) systems becoming more readily available and affordable for water utilities, time series data for point-specific or region-wide applications have become available at increments with a typical resolution of 10 to 15 minutes (Candelieri *et al.*, 2015). With the availability of the latter systems, the control and calibration of WDN models are substantially simplified. Smaller water utilities need not be dismayed if a SCADA system is not deemed financially viable in their context. Field measurements by means of pressure meters (connecting to standpipes or hydrants) can be taken at key points in the network and can be processed to enable the comparisons between observed measurements and modelling results.

An overview of the location and characteristics of the study area is provided in the next chapter.

CHAPTER 3 : STUDY AREA

This chapter provides a broad overview of the study area pertained to climate, geographical, social and economic conditions.

3.1 Location and General Characteristics

Swakopmund is located on the western coastline of Namibia in the Namib Desert in Southwestern Africa.

3.2 Topography

The topography of the study area is generally uniform and relatively flat with no sudden changes in elevation encountered over short distances within the townlands (*cf.* Figure 3.1). The elevations within the town range from 45 m above mean sea level to 0 m at the shore (USGS, 2015). The topography enables the distribution of water by means of gravity feed for the greater part of the town, with a pressure tower and booster pumps for the higher lying areas in the network.

3.3 Geology

Swakopmund is located next to the dune belt of the Namib Desert. Petric gypsisols and rock outcrops are the dominant soils which are encountered in the area (Mendelsohn *et al.*, 2003).

3.4 Population Served

A vast array of consumers is served by the WDN of Swakopmund. The data used for the WDN analysis are based on information from $\pm 8\,920$ water meters which monitor water consumption on a continuous basis.

The end-user categories in Swakopmund, based on zoning data, are listed in Table 3.1.

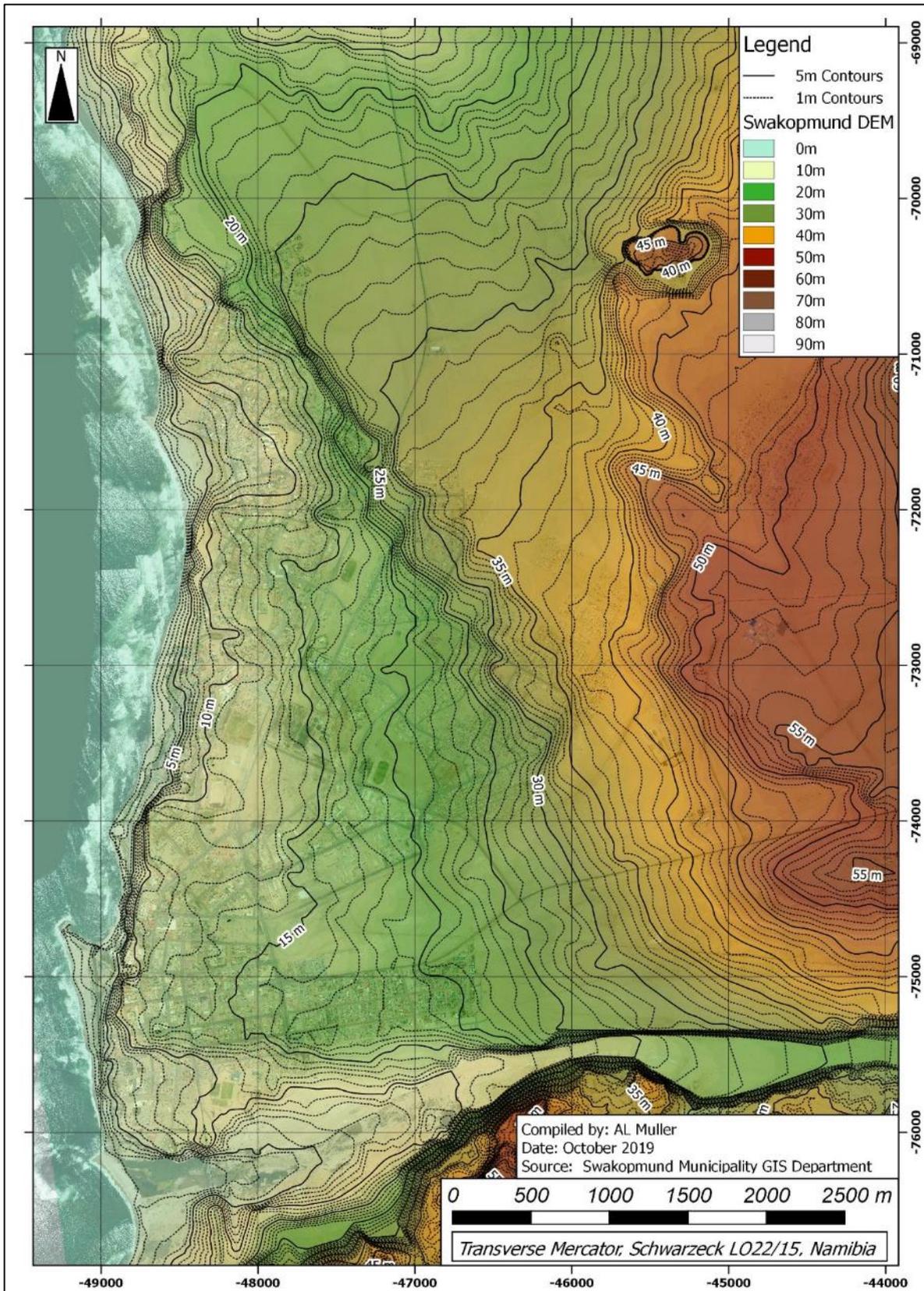


Figure 3.1: Topography of Swakopmund

Table 3.1: End-user categories and zonings for Swakopmund

Zoning	Typical land-use
Single Residential	Dwelling House
General Residential 1	Dwelling Houses Flats Residential Buildings Townhouses
General Residential 2	Dwelling Houses Residential Buildings Townhouses
General Business	Shops Office Buildings Parking Garages Licensed Hotels Blocks of Flats Residential Buildings
Local Business	Shop Office Building Dwelling House
Light Industrial	Light Industrial Warehouse Dry Cleanette Laundrette Service Industry
General Industrial	Industrial Building Scrap Yard Warehouse Building Yard Dry Cleanette Launderette Light Industry
Institutional	Institutional Building
Undetermined	Not yet defined in town planning scheme
Special	Special use according to the town planning scheme
Agriculture	Agricultural Building
Office	Office Buildings
Private Open Space	Private Open Space

Exceptions to the allowable land-use as listed in Table 3.1, are permitted with the consent of Council and according to the Swakopmund Town Planning Scheme.

Figure 3.2 is illustrative of the future erven; hence the significant differences between the number of residential erven and the number of water meters used in this study. Notwithstanding the 15 118 single residential properties (including proclaimed, but not yet serviced properties), the remainder of the number of erven per zoning are quantified in Figure 3.2.

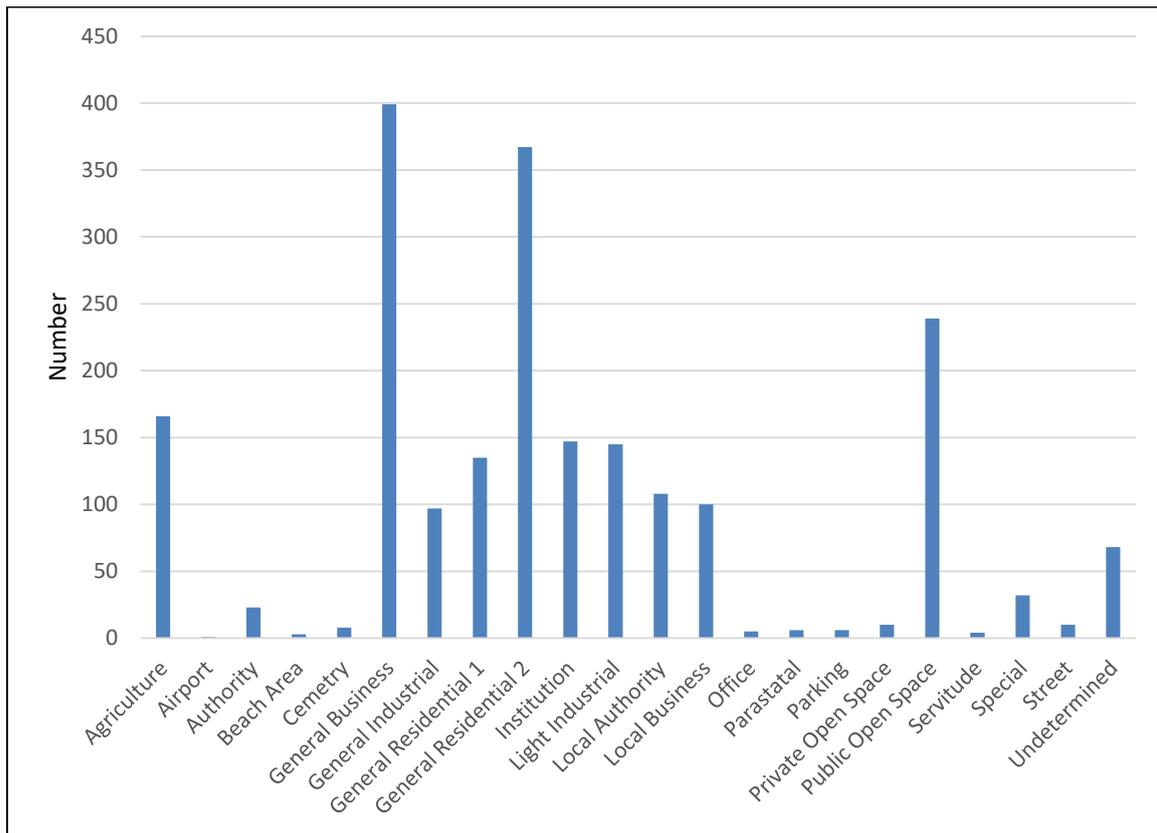


Figure 3.2: Zoning distribution in Swakopmund excluding single residential properties

3.5 Climate

Namibia is an arid country (*cf.* Figure 3.3), with the mean annual rainfall of Swakopmund less than 50 mm (Mendelsohn *et al.*, 2003). With no nearby fresh surface water sources, the low average rainfall further exacerbates the challenges faced by the Swakopmund Municipality in terms of assurance of water supply.

3.6 Water Supply to Swakopmund

The coastal towns of Walvis Bay, Swakopmund and Henties Bay are supplied with fresh water from the Central Namib Water Supply Scheme based at Swakopmund. The scheme is managed by NamWater and abstracts groundwater from wellfields in the Omaruru and Kuiseb Rivers, which are the nearest sources of potable water to the towns (Christelis and Struckmeier, 2011). Details pertained to the water supply schemes to Swakopmund, are shown in Figure 3.4. Shortfalls in the water supply capabilities at the NamWater well-fields are supplemented by desalinated water from the Orano (previously known as Areva) desalination plant.

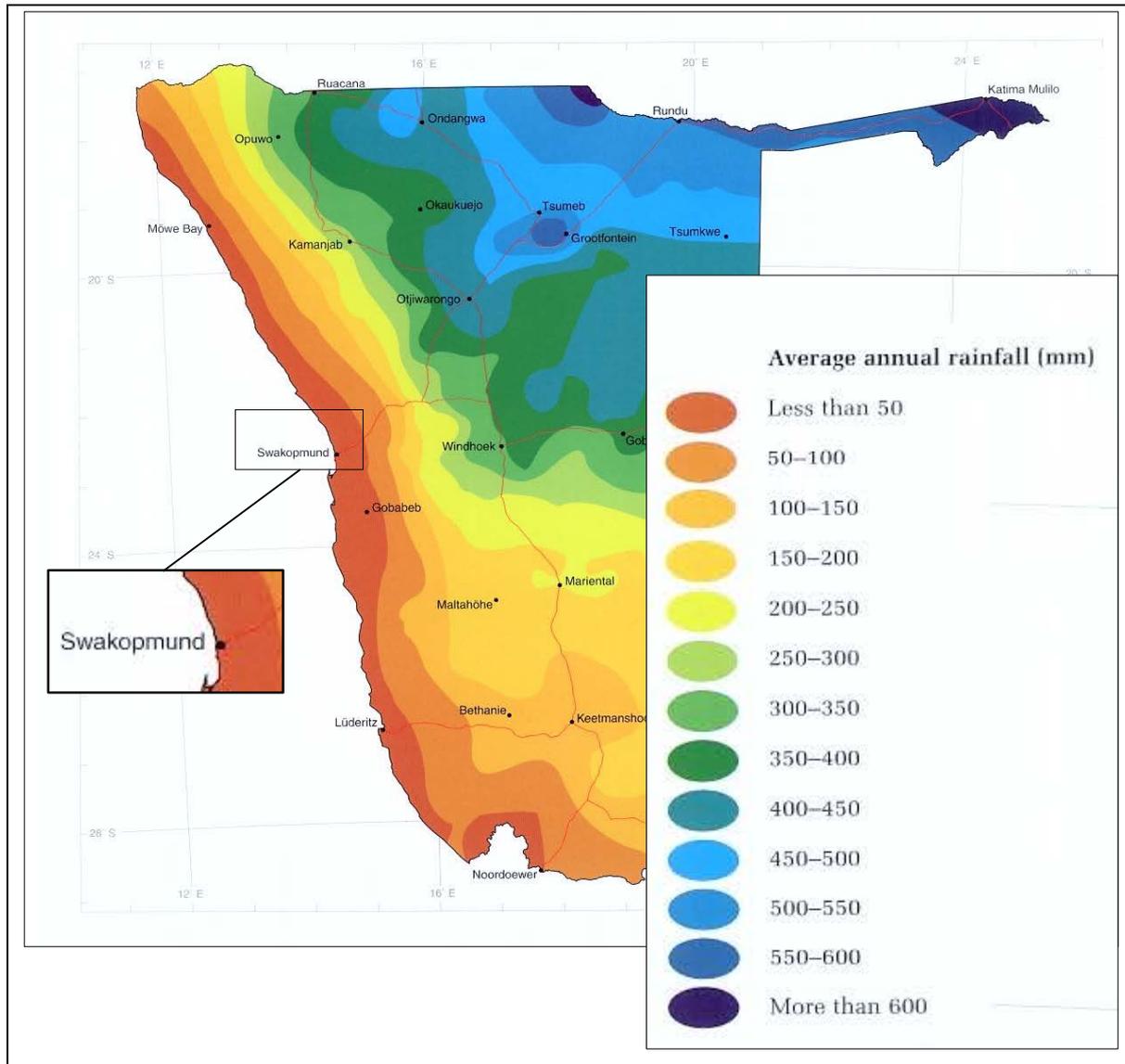


Figure 3.3: Mean annual rainfall distribution in Namibia (Mendelsohn *et al.*, 2003)

3.7 Supply Philosophy

Water from the Omdel and Kuiseb Schemes is supplied by the main water supply utility in Namibia, NamWater, to the main NamWater reservoir near the airport in Swakopmund. From here, the water is distributed by means of a gravity feed system to distribution reservoirs owned and managed by the Swakopmund Municipality.

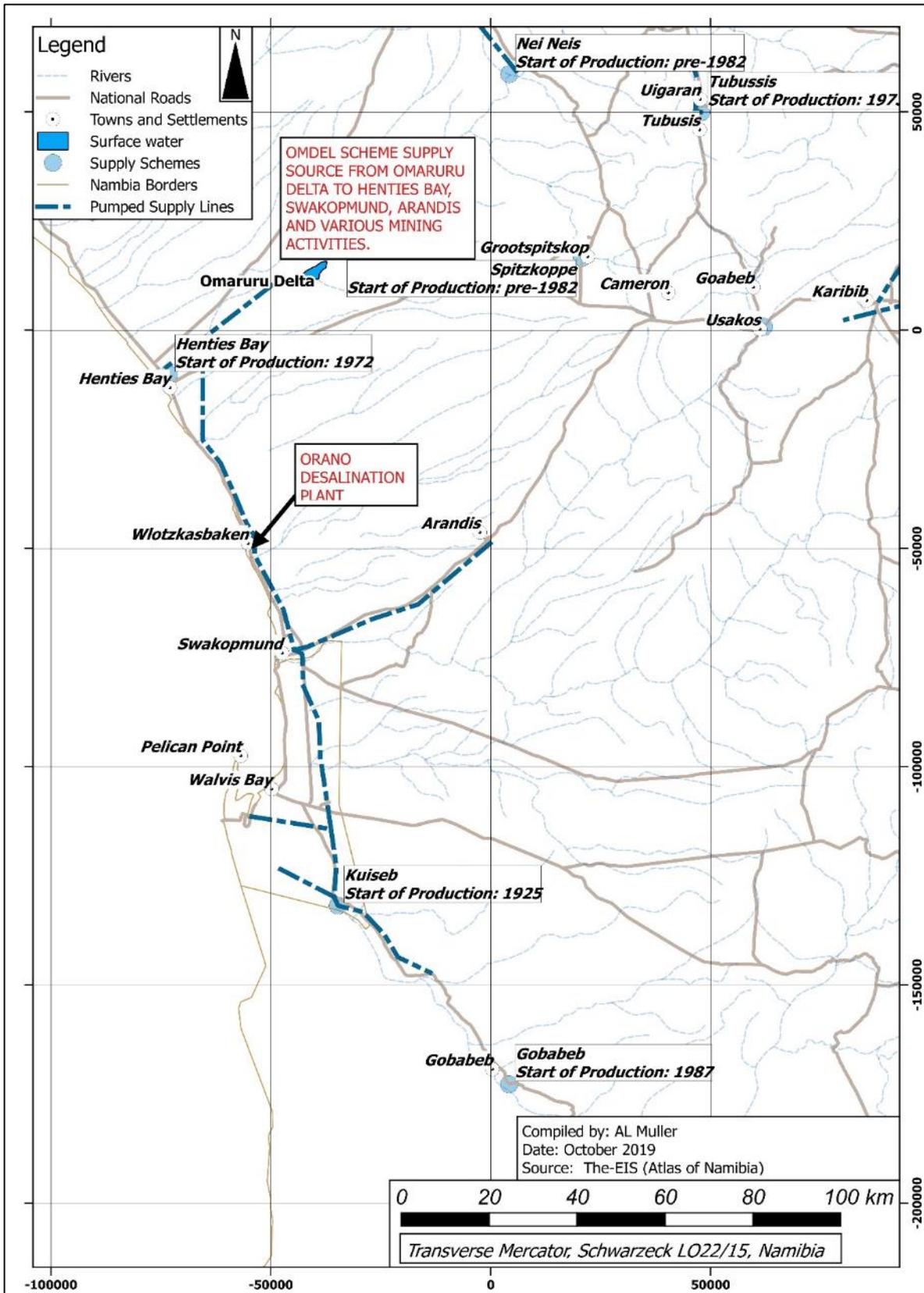


Figure 3.4: Water supply schemes to Swakopmund

Only one pressure tower is present in the supply area and, based on information provided by the Swakopmund Municipality, it supplies the airport and some higher lying residential areas. A booster pump station supplies some of the new residential areas which were more recently developed. Relatively low pressures associated with the WDN are attributed to the even topography of the area. A number of agricultural and residential properties located on the eastern outskirts of Swakopmund are supplied by a separate system from the main supply network. The latter end-users were not incorporated into the Swakopmund WDN model.

3.8 Municipal Record Keeping

Record drawings of the majority of the water supply infrastructure in Swakopmund have been digitised by scanning the original hard copy drawings. A CAD layout of the WDN was compiled in the past; however, the drawing was not compiled sensibly in terms of layers and has not been updated to reflect all the pipelines that have been decommissioned or constructed for a number of years. The compilation of a GIS database of the WDN formed part of the assignment by the Swakopmund Municipality to mitigate the risks associated with outdated databases.

Water consumption figures are recorded on a monthly basis for all consumers. The vast majority of the meters are captured by hand-held electronic devices by trained water meter readers. The hand-held devices have GPS software which provides the location of the meter as well as the associated captured reading. The consumption figures are uploaded to the financial database and accounts are calculated accordingly. Bulk meter readings are monitored by the Technical Department. Monthly reports are compiled and discussed during Council meetings. In general, Swakopmund Municipality has a good record-keeping system throughout the various departments with the bulk of information readily available for analysis purposes.

3.9 Age of Infrastructure

A notable number of pipelines were installed prior to 1960. The latter are located in the older part of the town and consist of Asbestos Cement (AC). Figure 3.5 is

illustrative of the construction dates associated with the WDN infrastructure for which as-built attribute data are available.

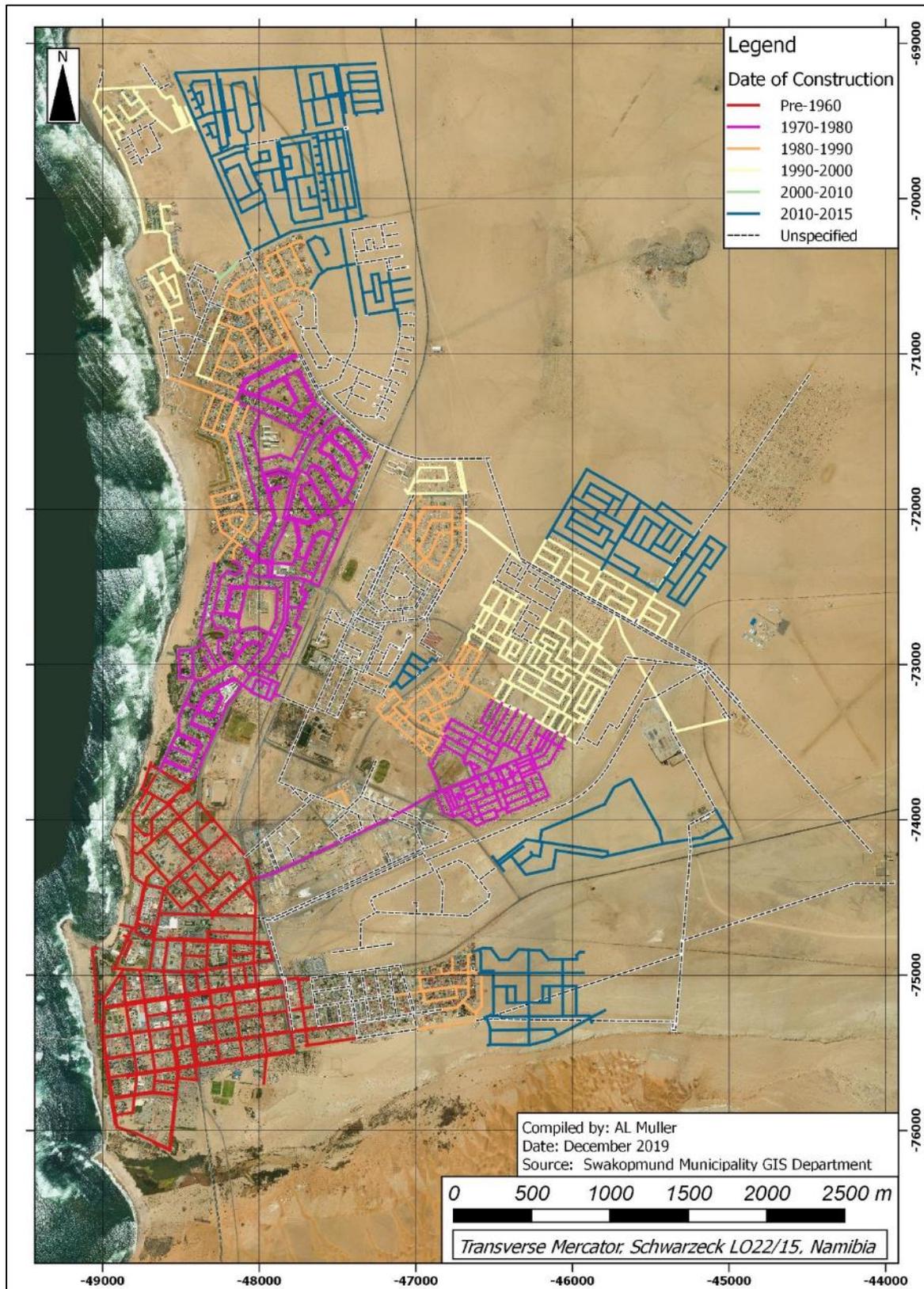


Figure 3.5: Construction dates of Swakopmund WDN infrastructure

3.10 Municipal Finances

All water meters are billed by the Financial Department according to the monthly consumption. The meter readings are recorded on a monthly basis and accounts are distributed to the end-users based on the number of units (cubic metres) of water consumed. Water is purchased from the bulk water provider, NamWater, and is sold at a certain mark-up to recover operational and maintenance costs pertained to the potable water network.

3.11 Types and Diameters of Pipe Material

According to the records, the Swakopmund WDN comprises of AC and Unplasticised Polyvinyl Chloride (uPVC) pipelines. The diameters of the various pipe materials range from 50 mm Ø up to a maximum of 500 mm Ø.

3.12 Maintenance Procedures

Leakages, bursts and pipe replacements are treated 'symptomatically' as problems occur. The majority of the AC pipes' service life has surpassed or is nearing the 'short service life' period of 65 years; however, a theoretical remainder of service life up to the 'long service life' of up to 105 years may still be applicable depending on various site conditions (Stenstedt, 2019). As for the remainder of the WDN consisting of uPVC pipes, a service life in excess of 50 years is expected (Lancashire, 1985). In cases where a section of pipeline has a recurring record of failures, the section is replaced; this is especially common for older sections that are long past the expected design life. Where possible, systematic replacements are done, especially in areas where older infrastructure is present and where new developments may be dependent on the old supply network. Whenever AC pipes are replaced, uPVC is used as pipe replacement material.

3.13 Future Developments

Future expansions are planned for Swakopmund with an additional 8 000 erven envisaged to be developed by 2026. The feasibility of these developments is dependent on the availability of potable water supply in the area. High level discussions are in progress between community stakeholders, the Swakopmund

Municipality and Local Government to investigate the various options available to safeguard and ensure a sustainable water supply. One such option comprises of the acquisition of the Orano desalination plant, situated north of Swakopmund.

The methodology adopted in meeting the specific objectives of this study is discussed in the next chapter.

CHAPTER 4 : METHODOLOGY

The methodology described in this chapter provides an overview of the processes that were followed during the execution of the case study project. Alternative sources of information and approaches are also discussed with the aim to provide a comprehensive, although not yet exhaustive, guideline for practitioners to create a WDN model of an existing network.

Although there are various software packages capable of creating Digital Elevation Models (DEMs) or processing data sets into different formats, as highlighted in Chapter 1, the aim of this study is to make use of open-source software. Hence, the procedures as outlined below mainly focus on the use of the QGIS software package and associated plugin repositories.

4.1 Methodological Framework and Summative Guidelines

The methodological framework developed in this study is presented in this section. The framework consists of a series of process flow diagrams which could serve as a practitioners' guideline to integrate open-source software and engineering practices when WDN models of an existing network need to be compiled in future. Typically, the framework is inclusive of the following key processes, as shown in Figures 4.1 to 4.9:

- (a) **Data acquisition:** Inclusive of consumption figures, network connectivity, elevation, cadastral, pipe network, reservoir and pump data sets (Figure 4.1).

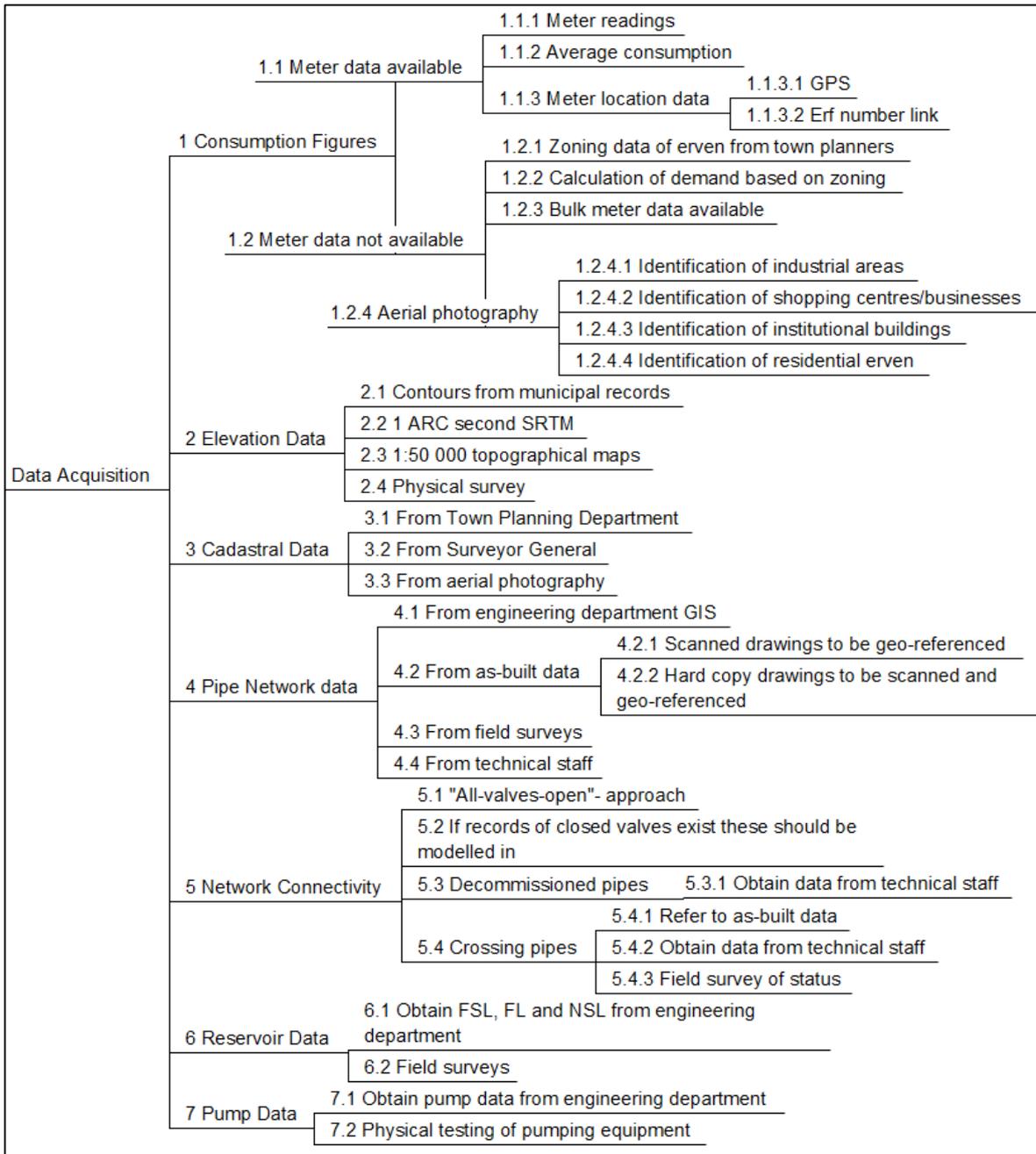


Figure 4.1: Data acquisition process flow diagram

- (b) **Data processing:** Inclusive of data verification, filling of gaps, topology checking, data validation, allocation of friction coefficients, the compilation of EPANET data sets and the subsequent processing and analysis thereof (Figure 4.2).
- (c) **Data verification:** Inclusive of the consideration of multiple input sources, setup and verification of attribute table contents and the detailed verification of pipeline data within a WDN (Figure 4.3).

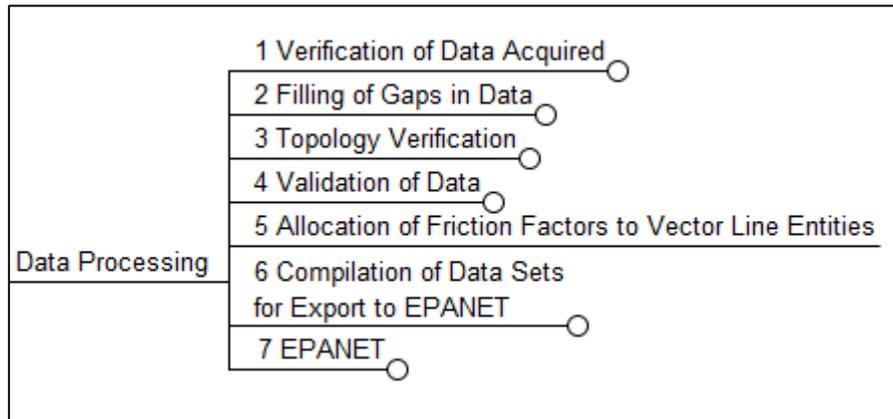


Figure 4.2: Data processing flow diagram

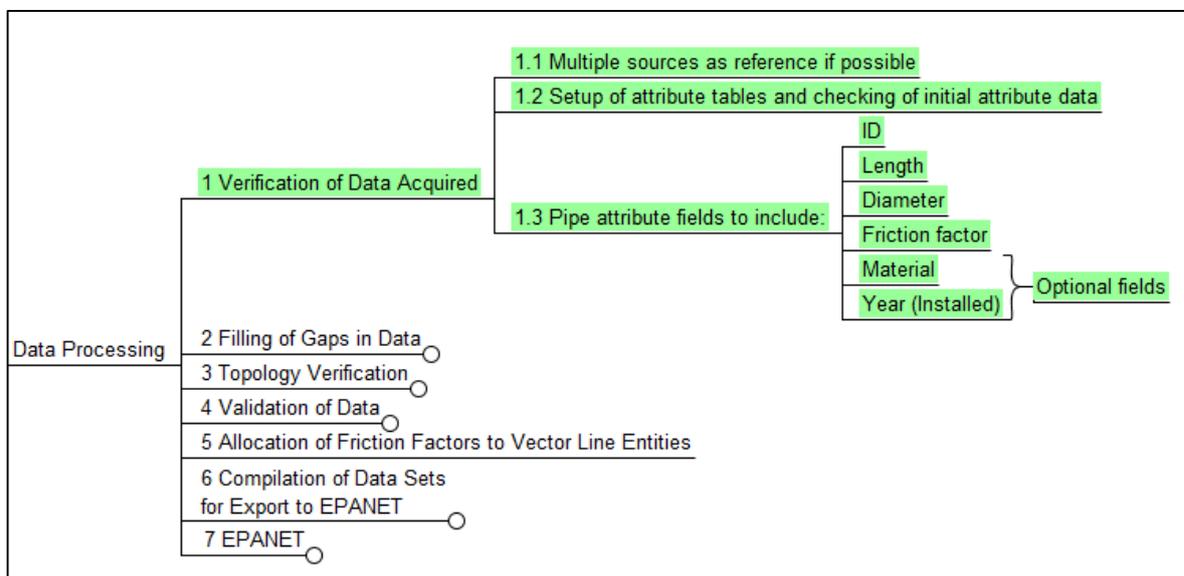


Figure 4.3: Data verification flow diagram

- (d) **Data repair and filling of gaps:** Inclusive of the identification of missing records and/or gaps and the application of standard engineering principles (Figure 4.4).
- (e) **Topology verification and data validation:** Inclusive of the removal of duplicate data, filtering and joining of data and the overall validation process to ensure proper quality control (Figure 4.5).

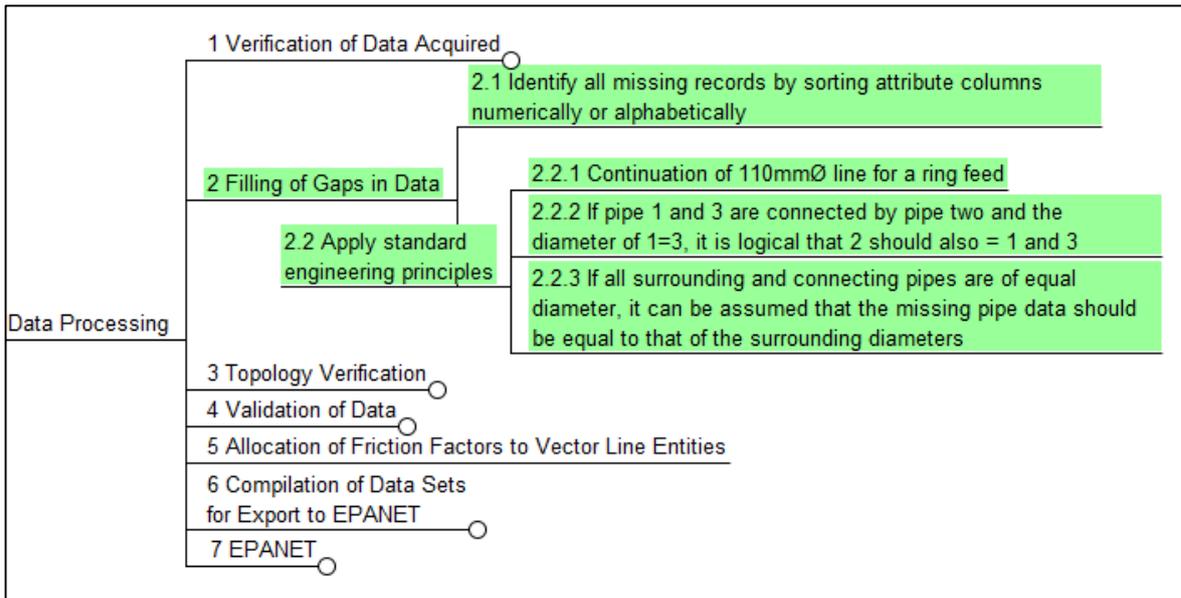


Figure 4.4: Data repair and filling of gaps flow diagram

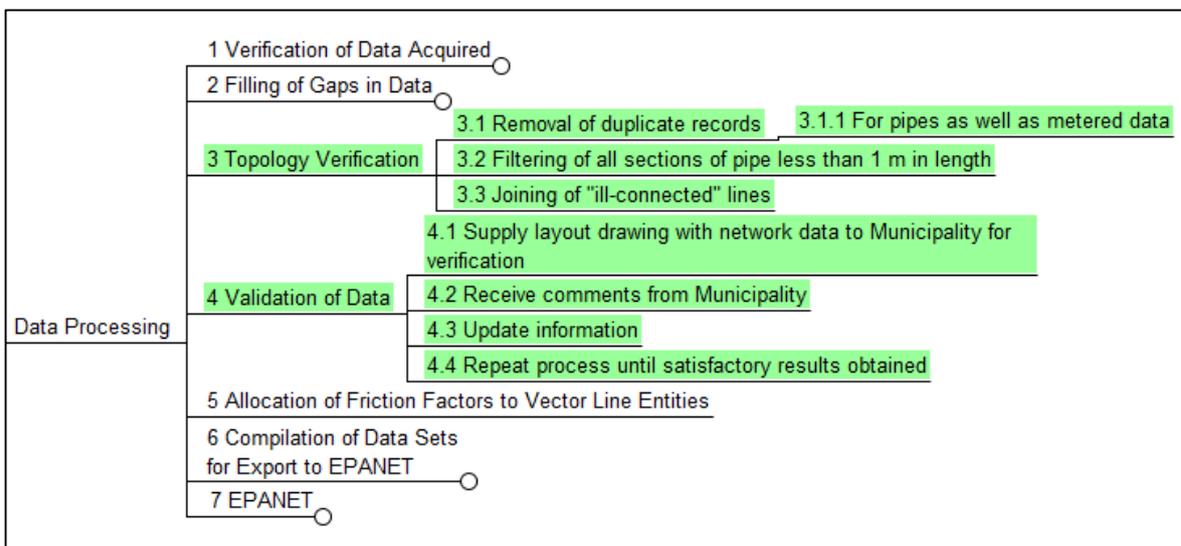


Figure 4.5: Topology verification and data validation flow diagram

(f) **Compilation of EPANET data sets:** Inclusive of the setup of attribute fields for pipelines, creation of nodes, linking of pipelines and nodes, creation of Voronoi polygons and the allocation of water demands to these polygon areas (Figure 4.6).

(g) **Pipeline attribute fields:** A checklist to ensure that all the major pipeline features are correctly captured and included (Figure 4.7).

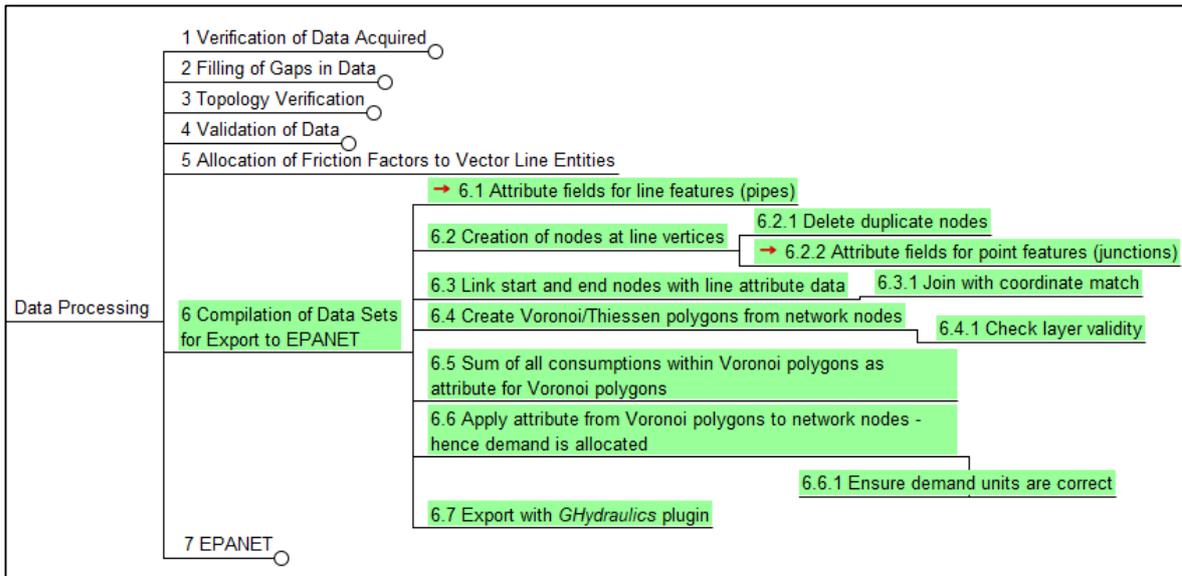


Figure 4.6: EPANET data set flow diagram

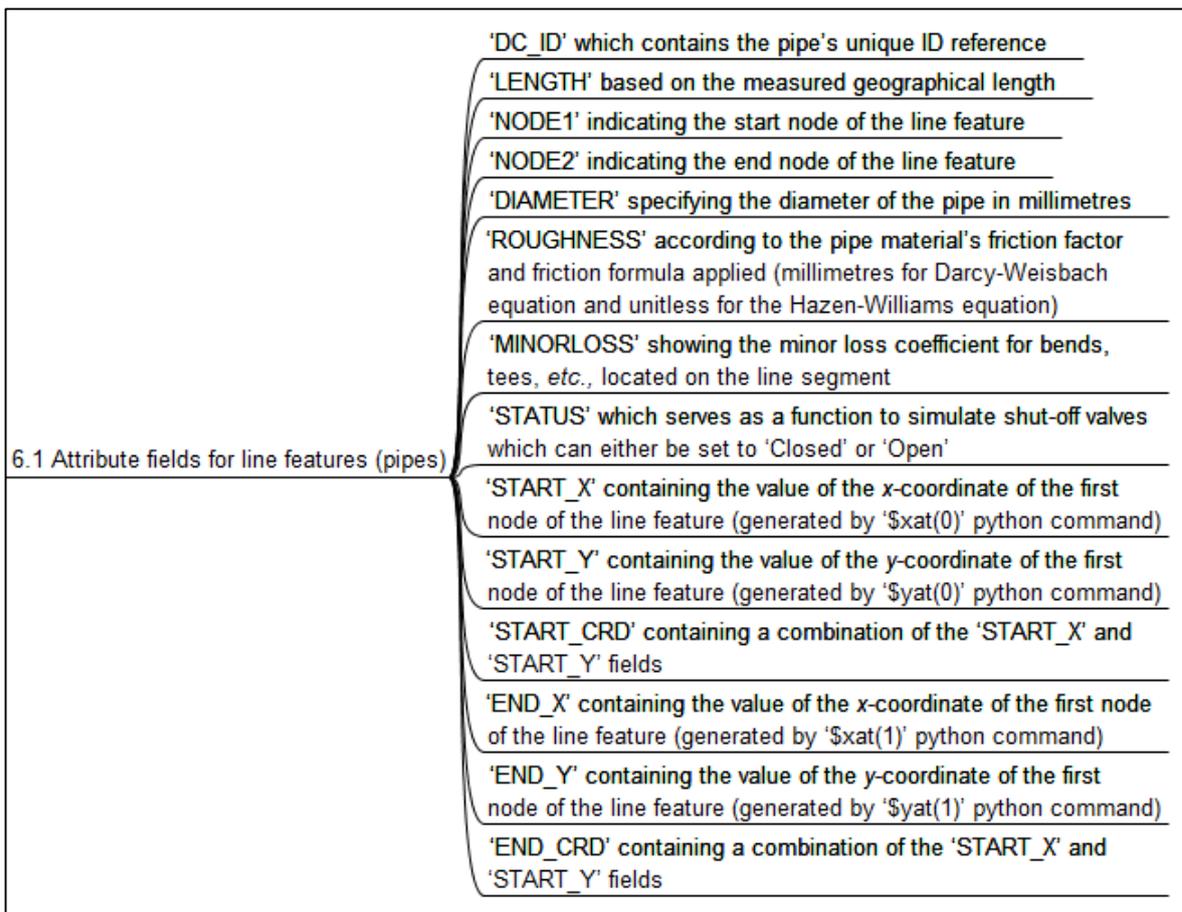


Figure 4.7: Pipeline attribute field checklist

(h) **Node attribute fields:** A checklist to ensure that all the major node features are correctly captured and included (Figure 4.8).

- (i) **EPANET data processing:** Inclusive of the setup of project parameters, data analysis and data verification (Figure 4.9).

6.2.2 Attribute fields for point features (junctions)	'DC_ID' which contains the junction's unique ID reference
	'NODE' indicating the node name, typically consisting of the prefix (i.e., 'J') and the unique ID reference (i.e., 'J1987')
	'ELEVATION' specifying the elevation of the node
	'X' containing the value of the x-coordinate of the point feature (generated by '\$x' python command)
	'Y' containing the value of the y-coordinate of the point feature (generated by '\$y' python command)
	'CRD' containing a combination of the 'X' and 'Y' fields.

Figure 4.8: Node attribute field checklist

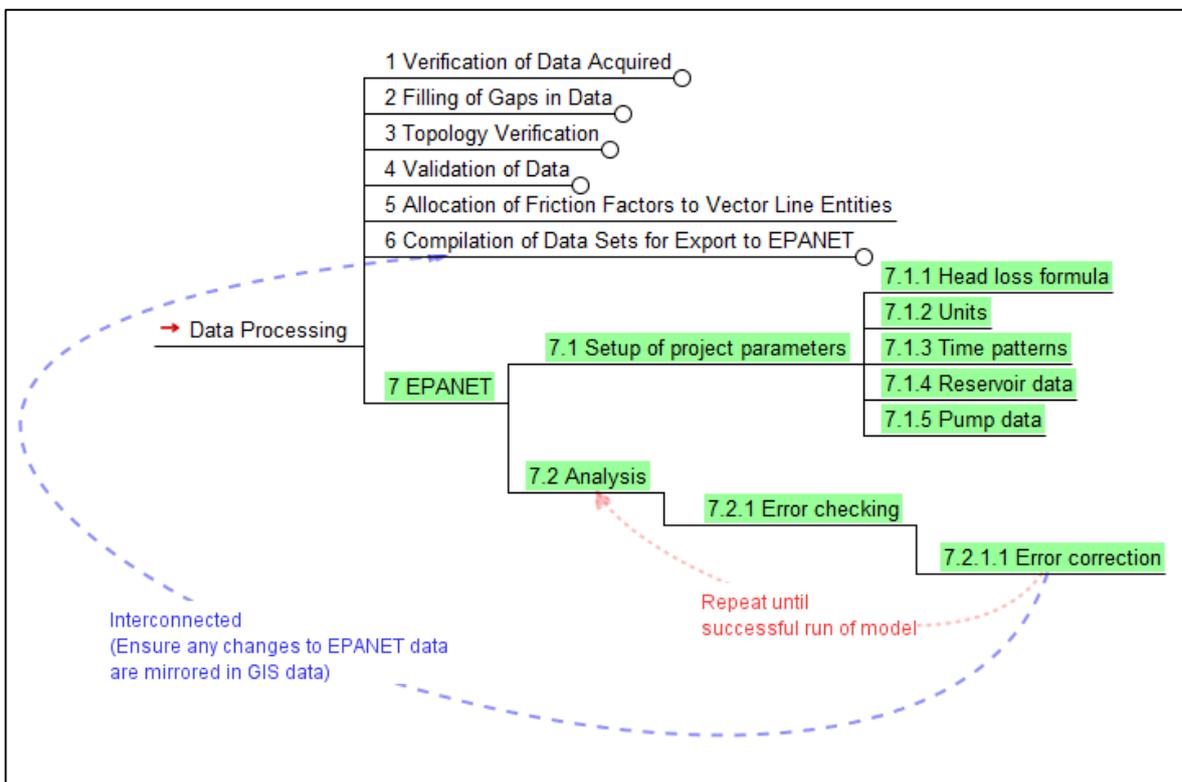


Figure 4.9: EPANET data processing flow diagram

4.2 Acquisition of Data

The acquisition of the required data sets may in some cases even prove to be a more tedious exercise than the compilation of the WDN model itself. Unavailability of data and the lack of record keeping by water utilities are some of the contributing factors that further complicate the practitioner's task of compiling a

WDN model. Establishing sound communication with municipal representatives bridges a large gap in the data acquisition process. Sufficient time should be allotted during the planning phases of a project for meetings and site visits with representatives of the municipality. Practitioners should familiarise themselves with the project area to enable the identification of possible discrepancies, suspicious results and/or data inputs.

It is important to note that special care should be taken to ensure that all data sets and information received from the municipality are stored separately from all working files. This measure serves to prevent corruption or alteration of source data during processing. The use of revision numbers and unambiguous file name allocations for new or processed files will prevent the loss of data and the use of incorrect or outdated data for processing and analysis.

4.2.1 Consumption figures

Consumption figures form an integral part of the WDN model due to the fact that the friction losses in the network are linked to the flow rates (*cf.* Chapter 5). Where meter reading data are available, it is required that this data should be used for the allocation of demands to nodes. The mere availability of meter reading data however, is insufficient, since in GIS, two data fields that allow plotting on the *x* and *y*-axis, are respectively required. In other words, the metered data should have coordinate data assigned to it in order to be usable in the WDN model.

The water meter readings of Swakopmund Municipality are linked to GPS coordinates. The latter resulted in less time spent on linking consumption figures to even on the GIS database. Average consumption figures derived from a period of 12 months along with GPS coordinates for each reading point were supplied by Swakopmund Municipality. The data were validated visually against the available cadastral layout and proved to be at an acceptable level of accuracy based on the location of the meters in relation to the aerial photographs available for Swakopmund.

The consumption figures which were obtained from the metered data, need to be converted to the AADD. The data cover different time bases; hence, an accumulated usage provides an indication of trends in the water consumption. Consumption data may not necessarily be available in such detail as in the case of Swakopmund. Therefore, alternative measures are required to link water consumption data with geospatial attributes.

Account data comprising historical water consumption should be available from the Financial Department of a water utility if there are no data available from the Engineering Department. Processing of this data can be done by means of software such as Microsoft Excel. Redundant columns containing administrative data (*i.e.*, name of owner, street address and property valuation) can be discarded if the practitioner is convinced that the data will not be required at a later stage. In GIS, a powerful function such as ‘tabular joins’ can be used to facilitate the linking of spatial data sets (*i.e.*, shapefiles) and non-spatial table formats (*i.e.*, Comma Separated Values (CSV) and Database File (DBF)), provided that there is at least one unique corresponding column in each of the data sets. For example, if meter readings are available without GPS coordinates, the probability is high that erf numbers are present in one of the reference columns of the data received from the water utility. Erf numbers, in turn are normally associated with projected and coordinated cadastral shapefiles. By making use of the erf numbers as common join field, the consumption figures can be linked to the shapefile.

Bearing in mind that in some cases adequate billing and consumer records may not be available to the extent required for use in a WDN model, water consumption figures from sources as described in Chapter 2 (*cf.* Section 2.9.1 and/or Table 2.1) or Chapter 5 (*cf.* Section 5.2) can be consulted. However, in using the latter sources, the accuracy of the WDN model may be compromised.

Information pertained to the applicable zoning of erven can be obtained from the Town Planning Department or even from aerial photography. By using visual interpretation of features such as buildings, roof coverage, trees, lawns, *etc.* from

aerial photographs, land-use can be deduced within reasonable certainty after which demands can be allocated accordingly.

If bulk water meter readings are available from the bulk water supplier or from zonal meters, these can be averaged and used for the 'calibration' of the demands which were allocated based on zoning. The principle relies on the fact that the average daily volume of water sold to the municipality or water utility from the bulk supplier, should under ideal circumstances, be equal to the sum of the daily consumption volume of all the erven. This comparison should give a general indication of the accuracy of the allocated demands. Where metered data of individual consumers are available, it is recommended that a water balance should be conducted as part of the analyses.

4.2.2 Elevation data

The elevation data were ultimately transferred to the node data shapefile as attributes, which in turn, were used for the hydraulic analysis of the WDN. The detailed procedures followed to generate the elevation data are discussed in the subsequent paragraphs.

The contour data were supplied by the Swakopmund Municipality from their GIS database. The contours were generated from a grid aerial survey which was conducted simultaneously with the aerial photography of the town. A DEM was created from the contour shapefile. The following steps were followed for the creation of the DEM:

- (a) Split contour lines into separate line segments by distance. The *V.split.length* geotool (Figure 4.10) splits vector lines into shorter segments using a maximum specified distance between nodes. The resulting length of all segments is expected to be equal and not longer than the given length parameter. Normally, 10 m lengths should suffice depending on the processing capabilities of the computer being used.

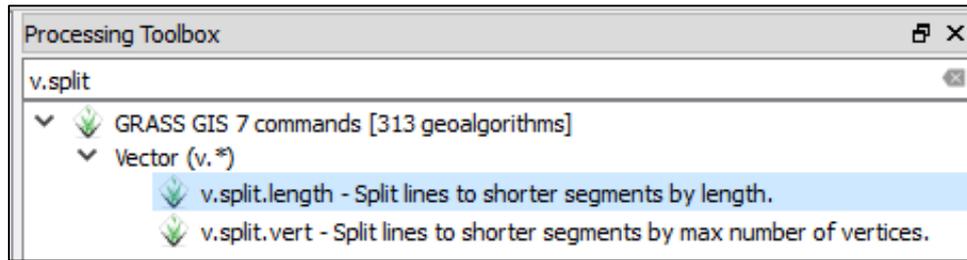


Figure 4.10: *V.split.length* geoalgorithm

(b) Nodes were extracted from lines using the *Extract Nodes* geoalgorithm (Figure 4.11), which uses a line or polygon layer to generate a point layer with points representing the nodes in the input lines or polygons.

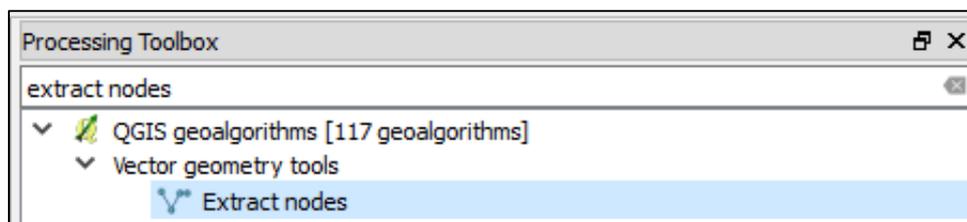


Figure 4.11: *Extract Nodes* geoalgorithm

(c) Bearing in mind that the start and end vertices of the different line segments overlap, duplicate points were created by the *Extract Nodes* geoalgorithm. The latter duplication was removed by using the *Delete Duplicate Geometries* geoalgorithm. In other words, duplicated geometries are identified and removed without considering attributes. Hence, in cases where two features have identical geometries, but different attributes, only one of them will be added to the resultant layer.

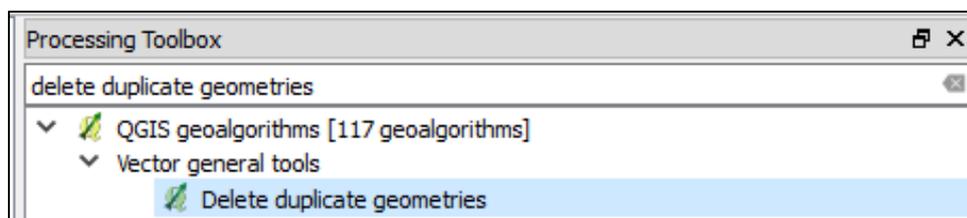


Figure 4.12: *Delete Duplicate Geometries* geoalgorithm

(d) The final step in the compilation of the DEM was completed by using the *SAGA Nearest Neighbour* geoalgorithm.

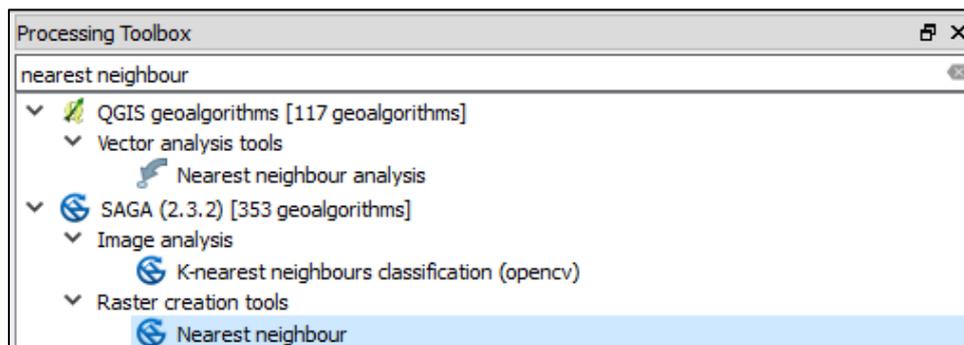


Figure 4.13: *Nearest Neighbour* geoalgorithm

The *Add Raster Values to Points* geoalgorithm (cf. Figure 4.14) was used to extract data from the specified raster (DEM) layer and to add attributes from the DEM to a selected point file (junctions). Subsequently, the geoalgorithm creates a new point layer based on the sampling points' positions and the attributes extrapolated from the underlying raster cells. In using such an approach, elevation values from GeoTiff or other elevation data files in raster format can typically be applied to the nodes within a WDN model.

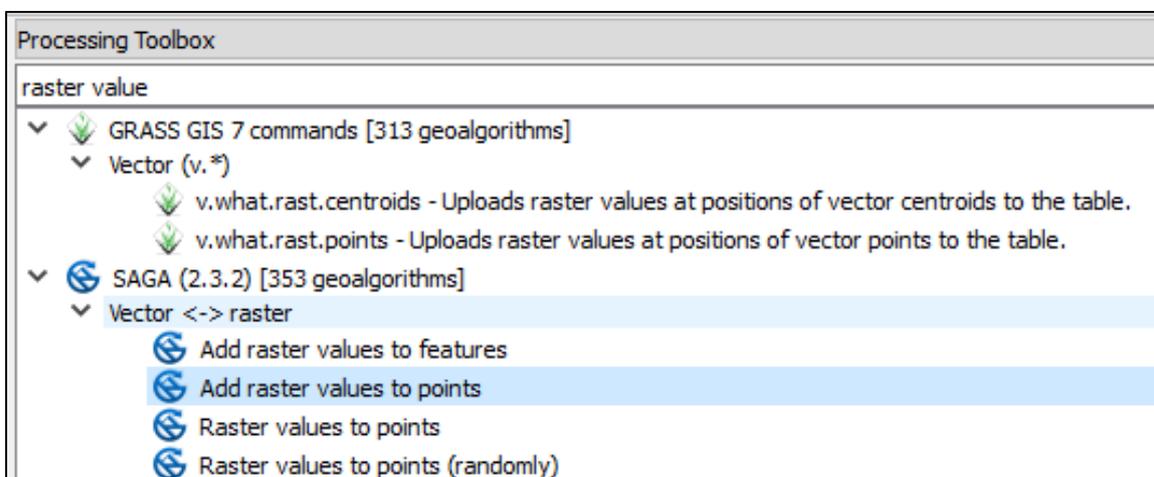


Figure 4.14: *Add Raster Values to Points* geoalgorithm

In some cases, elevation data may not be as readily available from the water utility as expected. Suitable alternative sources for elevation data include 1-arc second SRTM data from the USGS, 1:50 000 topographical maps or topographical surveys by means of conventional methods. SRTM data can be downloaded in a GeoTiff-format which consists of a raster with associated numerical values, *i.e.*, elevations, allocated to all cells. The 1:50 000 topographical maps can be obtained

from the SG in raster format. The raster format of the latter does not contain elevation data as part of the embedded values as is the case with SRTM, but each pixel (cell) has red, green and blue (RGB) numerical values to represent a colour on the RGB-scale. Contour data from the 1:50 000 topographical maps should be converted to vectors by tracing the contour lines with polylines, after which, the process as described for the creation of a DEM from contour lines can be followed.

4.2.3 Cadastral data

Cadastral data were made available by the Technical Department of Swakopmund Municipality. The Town Planning Department will normally assume responsibility for the updating and maintenance of the cadastral information of a town or city. At this stage, it is important to note that cadastral data can include erven information that form part of the initial town planning framework, which is not necessarily required or applicable to the analysis. During the Lexical Phase (*cf.* Chapter 2, Section 2.5) of a project, the geographical area to be included should be specified and as part thereof, the erven to be included in the study as well. Cadastral data may further be obtained from the SG if data are not available at the municipality. As a last resort, aerial imagery can be used for the demarcation of erven and properties.

For the Swakopmund WDN model, cadastral data did not play a crucial role in the compilation of the model, largely due to the fact that all metering/consumption data were linked to geographical coordinates; hence, an intermediate link, *i.e.*, cadastral data, was not required between the billing records and the geographical data.

4.2.4 Pipe network data

Skeletonisation refers to the process of identifying selected pipes within a WDN that are deemed to have the greatest importance and including them in a WDN model (Mays, 2000). The latter process largely involves a significant amount of engineering judgement in order to determine which pipes and/or components are to be included and excluded from a WDN model.

The initial skeletonisation for the Swakopmund WDN was based on all pipelines with a diameter of 100 mm and larger. After considering the size of the WDN and the actual number of pipes exceeding the initially specified diameter, it was decided that an all-mains approach would be followed. All-mains approaches yield more detailed results and have become easier to develop due to the digitisation of data by water utilities and the integration of GIS and analysis software (Mays, 2000).

During the gathering of the data for the Swakopmund WDN, the pipe network data included GIS shapefiles, CAD drawings, scanned as-built drawings (raster images) and sketches from the technicians/artisans. Where uncertainty existed regarding the accuracy of the data, or where there were contradictory data for certain pipe sections, field surveys were conducted by the artisans who reported back on these matters.

Network connectivity remained a challenge for the case study area. Due to the age of the infrastructure in the town, the data from the layouts resulted in numerous crossing lines representing pipelines. Where crossing lines were encountered, uncertainty arose as to whether these pipelines are connected by a crossing piece, two tee pieces or whether there is any connection at all. In cases where as-built drawings did not clearly indicate connections between crossing pipes of varying diameter, the pipelines were regarded as separate lines. The probability that crossing pipes of equal diameter are connected, was considered high and these pipes were modelled accordingly, while the contrary was true for pipes varying significantly in diameter.

The status of isolation valves in the network, albeit open or closed, may have a notable impact on pressures and flow velocities in the WDN. Isolating valves can either be closing clockwise or counter-clockwise. Especially, older isolating valves do not have appropriate markings that indicate the closing direction of the spindle, making the determination of their 'open' or 'closed' status a tedious exercise. As an initial approach for the compilation of the WDN model, an 'all-valves-open' approach was followed for all pipes, except where artisans could confirm the

status of the valves in the network. It is proposed that a similar approach should be followed when new WDN models need to be setup initially, after which, the identification of closed valves and pressure zones may be done as part of the calibration of the WDN model.

All line features representing pipelines were defined by only two vertices as the linking of nodes and line features during later stages of the model development depended on lines being defined as such. The *Explode Lines* geotool was used for the creation of single line features from polylines that originated from the data as received from the municipality. The afore-mentioned algorithm creates a new vector layer in which each line of the original layer is replaced by a set of lines, representing the segments in the original line.

A preliminary check for duplicate entities (*cf.* Figure 4.12) was done prior to commencing with data processing.

4.2.5 Reservoir data

The Engineering Department of any municipality or water utility should be able to furnish the practitioner with the relevant information pertained to the elevation data and storage capacities of reservoirs. Typical elevation levels that are of significance include the: (i) Full Storage Level (FSL), (ii) Floor Level (FL), and (iii) Normal Service Level (NSL). In the event that the aforementioned data are not available, physical measurements can be conducted on-site by using conventional methods. The storage capacity of the reservoirs was not subjected to scrutiny during the compilation of the Swakopmund WDN. The latter was largely due to the fact that a NamWater terminal reservoir is located upstream of the Swakopmund Municipality's storage reservoirs and that the modelling of the reservoir levels along with pump stations and pressure zones, were planned as part of future phases for the WDN model refinement. Subsequently, the reservoir levels were not analysed as part of the WDN model.

The relevant reservoir data used during the analysis of the Swakopmund WDN are listed in Table 4.1.

Table 4.1: Reservoir data used for Swakopmund WDN

Reservoir	Floor Level (m)	NSL (m)	Diameter (m)
8 Ml Reservoir North	53.15	60.36	37.20
8 Ml Reservoir South	53.15	60.36	37.20
12 Ml Reservoir North	54.40	60.36	50.00
12 Ml Reservoir South	54.40	60.36	50.00

4.2.6 Pump station data

Pump stations in a WDN are largely limited to booster pump stations and lifting pump stations that supply elevated tanks. Pump data can be obtained from site visits to the pump stations or from the Engineering Department. In the case of Swakopmund, no pump data were available from the Engineering Department and the configuration of the pumps supplying the elevated tank was unknown; subsequently, a ‘dummy tank’ could be provided at the pump station node to simulate the pressure supplied by the elevated tower and the booster pump station, respectively. The booster pump station supplies higher lying areas not served by the pressure tower.

The modelling and analysis of reservoir levels, in conjunction with the determination of pressure zones (including the lift pump station, the pressure tower and the booster pump station) form part of the future planning by Swakopmund Municipality and were not included as part of the initial scope of works for the compilation of a WDN.

4.3 Data Processing

Upon receiving all the data from the various sources as discussed above, the next step was to verify the data and conduct cross-checks between data sources where possible. Under ideal circumstances, it would be preferable to have multiple data sources which support each other; however, this is seldom encountered. For Swakopmund an as-built database of about 28 drawings was georeferenced. The georeferenced drawings enabled cross-checking of the CAD and GIS data, which proved to be quite accurate for the majority of the WDN.

4.3.1 Setup of attribute tables and checking of attribute data

The digitising and conversion of the data into shapefiles commenced after all the data were acquired and verified. During this step, special care was taken to ensure that the spatial projection of the data is correct as all further measurements would be dependent thereon. Depending on the requirements of the water utility and the extent of the project, the number of and details contained in the initial attribute fields may vary. Attribute fields should at least contain the length and diameter of the pipes along with a unique identification number (ID). Usually, an ID-field is automatically created by the software; however, a number string field was created and the unique ID values were assigned by using the '\$id'-command in the Python dialogue screen for the calculation of actual values.

In a similar fashion the 'length'-field was calculated by using the '\$length'-command. Although EPANET is capable of calculating pipeline lengths based on coordinate distances between nodes, the 'length'-field will be used during the topology verification process. Refer to Section 4.4 for the details pertained to the complete attribute field requirements for exportation to EPANET.

4.3.2 Filling of gaps in data

Further checking of attribute data entailed the verification between data sources and the contents of the attribute data table's contents, *i.e.*, pipe diameters in the line vector shapefile representing the pipelines should be the same as the diameter and material as indicated on georeferenced drawings. With the attribute tables compiled for the various vector features, columns were sorted, and empty data fields were identified. Attribute data fields were sorted numerically to identify which features lacked attribute data. Alternatively, 'Styles' were applied according to attribute data and the missing data were represented visually within QGIS. Figure 4.15 illustrates how alternative visual styles for pipelines were used during the initial processing phases to visually identify pipelines with unknown diameters. The diameter of the main supply pipeline, which would have a significant impact on the analysis results, was not digitised since the attributes thereof were confirmed by officials of the Swakopmund Municipality.



Figure 4.15: Example of pipelines with initially unspecified diameter data

Standard engineering principles were applied in other cases where data were missing (*cf.* Figure 4.15), for example:

- (a) If a pipeline with an unpopulated 'diameter' field formed part of a ring main, it was assumed that the line would have the same diameter as the ring main;
- (b) If Pipes 1 and 3 were connected by Pipe 2 and the diameter of Pipe 1 was equal to that of Pipe 3, it was assumed that the diameter of Pipe 2 should also be equal to those of Pipes 1 and 3, respectively; and
- (c) If all surrounding and connecting pipes were of equal diameter, it was assumed that the missing pipe information should be equal to that of the surrounding diameters.

4.3.3 Topology verification

Topology describes the relationships between points, lines and polygons that represent the features of a geographical region (QGIS Development Team, 2016). The relationship between features proved to be a crucial point of interest which required significant attention. Problems with regard to topology included duplicate records (both for pipes and metered data), short pipe sections, *i.e.*, less than 1 m in length, and ill-connected lines. The majority of the latter instances could directly be ascribed to poor draughting practice during the initial digitising of the water network drawings.

In cases where different pipelines did not converge at their start or end vertices, the connectivity between the pipes could not be accurately exported to EPANET, which ultimately resulted in errors during the initial analysis. In addition, duplicate features would also result in erroneous connectivity between entities exported to EPANET. On the other hand, if duplicate pipes were exported and did not register as errors during the analysis, inaccurate flow and pressure results would be obtained due to the flow being split between two pipes of the same diameter and length.

As for metered data represented by a point shapefile, duplicate features would cause an increase in point demand values; hence, an increase in flow rate and

head loss. A challenge posed by duplicate records/features is that it cannot easily be identified visually. The ‘must not have dangles’, ‘must not have duplicates’ and ‘must not have invalid geometries’ rules were applied in the *Topology Checker* plugin (Figure 4.16) to conduct the topology checking for the line vector layers. In the event that the *Topology Checker* plugin is not available under the Vector-menu, the plugin may have to be uploaded from the QGIS *Plugin Repository* under the Plugin menu.

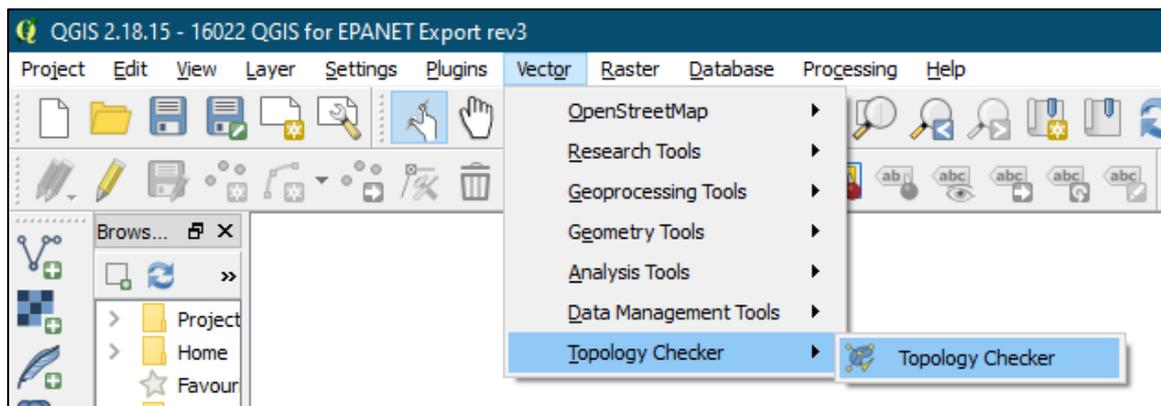


Figure 4.16: *Topology Checker* plugin

All duplicate line features were listed in the ‘Error’ field after the application of the ‘must not have duplicates’ rule. Duplicate entities were deleted with care taken to ensure that no data were lost in the process. Ill-connected features were identified during the application of the ‘must not have dangles’ rule. The latter identified pipelines were typically not connected to other pipelines at both ends. In some instances, the dangles were representative of the on-site pipeline network where single pipelines terminate at end-users’ properties, thus not forming part of a ring feed supply. For the remaining dangles which were identified, the pipelines were edited graphically and connected as required.

The topology of the point file containing the average consumption figures was evaluated by applying the same rules as used for the line features, except for the ‘must not have dangles’ rule. Pipelines with a length attribute of less than 1 m were regarded as redundant features. The practical analysis of such short sections of pipe was not deemed viable; hence, the data were filtered to indicate these

sections and rather combine them into single segments with the longer adjoining features.

All the attributes of the line features layer were updated due to the editing of these features during the topology checking stage.

4.3.4 Validation of processed data

Subject to the compilation of the data sets containing the WDN data, the data were circulated to the municipal representatives in the Technical and GIS departments for comment. The data were presented in drawing format and comments thereon were received. Typical inputs from the Swakopmund Municipality included pipelines that were decommissioned, new pipelines that were installed, but not being digitised and included in the GIS database, and clarification on connectivity where bulk mains and reticulation mains crossed. The circulation and updating of data were done over a period of approximately one month with updated information being recirculated to representatives for further comments.

The meetings held with the team of representatives from the municipality and the use of visual aids, e.g., a digital data projector to highlight the WDN features, yielded much quicker results, since the updating of attribute fields and connectivity of the features could be resolved immediately.

4.3.5 Allocation of friction coefficients to vector line entities

Friction coefficients were allocated based on a representative value of 140 for the Hazen-Williams *C*-factor which is a common value between AC and uPVC pipes (Mays, 2000). Further calibration, specifically in terms of friction coefficients could be done at a later stage (*cf.* Chapter 2, Section 2.10).

4.4 Compilation of Export Data Sets for EPANET

The correct setup of attribute fields was fundamental in the compilation of the data sets prior to export to EPANET using the *GHydraulics* plugin. Despite some of the automated features built into the *GHydraulics* plugin, a manual approach was

followed to ensure that the topology of the exported data was correct, especially with regards to junctions, their creation and connectivity with line features.

Attribute fields for line features consisted of:

- (a) 'DC_ID' which contains the pipe's unique ID reference;
- (b) 'LENGTH' based on the measured geographical length;
- (c) 'NODE1' indicating the start node of the line feature;
- (d) 'NODE2' indicating the end node of the line feature;
- (e) 'DIAMETER' specifying the diameter of the pipe in millimetres;
- (f) 'ROUGHNESS' according to the pipe material's friction coefficient and friction equation applied;
- (g) 'MINORLOSS' showing the minor loss coefficient for bends, tees, *etc.* located on the line segment;
- (h) 'STATUS' which serves as a function to simulate shut-off valves which can either be set to 'closed' or 'open';
- (i) 'START_X' containing the value of the x-coordinate of the first node of the line feature (generated by '\$xat(0)' python command);
- (j) 'START_Y' containing the value of the y-coordinate of the first node of the line feature (generated by '\$yat(0)' python command);
- (k) 'START_CRD' containing a combination of the 'START_X' and 'START_Y' fields;
- (l) 'END_X' containing the value of the x-coordinate of the first node of the line feature (generated by '\$xat(1)' python command);
- (m) 'END_Y' containing the value of the y-coordinate of the first node of the line feature (generated by '\$yat(1)' python command); and
- (n) 'END_CRD' containing a combination of the 'START_X' and 'START_Y' fields.

4.4.1 Creation of junctions

Junctions were created by converting polyline vertices to points using the *Convert Polygon/line Vertices to Points* geospatial algorithm (Figure 4.17). The resultant

point shapefile consisted of numerous duplicate features which were deleted (cf. Figure 4.12).

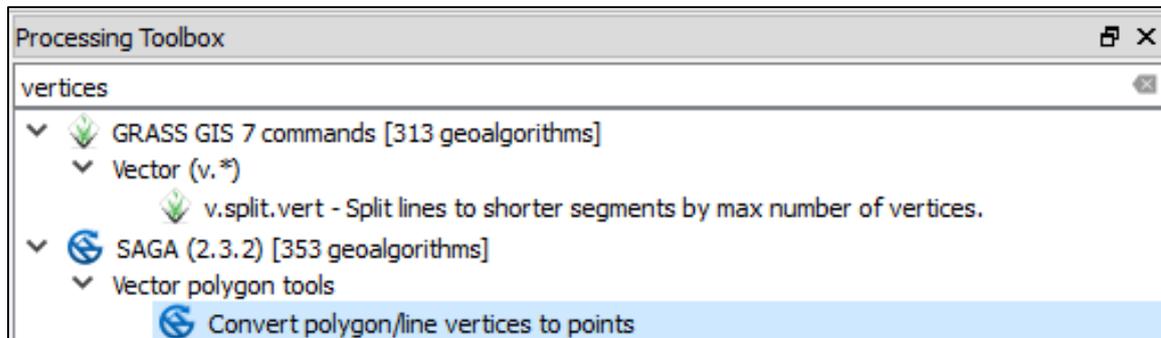


Figure 4.17: *Convert Polygon/line Vertices to Points* geoalgorithm

Attribute fields for point features representing junctions consisted of:

- (a) 'DC_ID' which contains the junction's unique ID reference;
- (b) 'NODE' indicating the node name, typically consisting of the prefix (*i.e.*, 'J') and the unique ID reference (*i.e.*, 'J1987');
- (c) 'ELEVATION' specifying the elevation of the node;
- (d) 'X' containing the value of the x-coordinate of the point feature (generated by '\$x' python command);
- (e) 'Y' containing the value of the y-coordinate of the point feature (generated by '\$y' python command); and
- (f) 'CRD' containing a combination of the 'X' and 'Y' fields.

All the attribute fields containing numerical values were entered as such to have a 'precision' up to 4 decimal places as illustrated in Figure 4.18.

4.4.2 Linking start and end nodes with line attribute data

The 'START_CRD', 'END_CRD' and 'CRD' fields provided common join fields between the line features layer and the junctions layer. After selecting the properties menu of the line features shapefile, a join was created between the 'START_CRD' attribute field of the line features shapefile and the 'CRD' attribute field of the point shapefile. The corresponding 'DC_ID' attributes were then copied

to the 'NODE1' attribute field. The process was repeated for the 'END_CRD' attribute join as well, and the corresponding 'DC_ID' attributes were populated to the 'NODE2' attribute field.

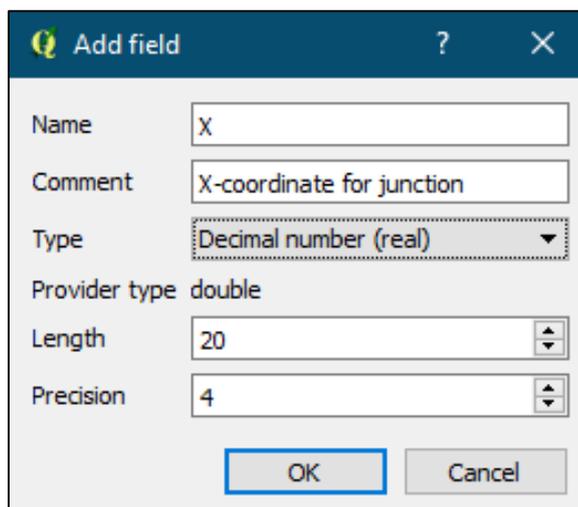


Figure 4.18: Typical attribute field setup for attribute values

4.4.3 Creation of Voronoi polygons from network nodes

Voronoi polygons were created by using the junctions' layer as input layer in order to obtain areas of representative demands which could be allocated to the junctions. The *Voronoi Polygons* geospatial algorithm is shown in Figure 4.19, while Figure 4.20 represents the various input fields to be completed.

The created layer was renamed and saved as a separate polygon shapefile. The validity of the created layer, *i.e.*, intersections, closed holes, and fix node ordering, was checked with the *Check Geometry Validity* geospatial algorithm. No errors or validity issues were identified during the analysis. However, should such errors occur, a systematic approach needs to be followed by visually inspecting each geometry issue and correcting the geometries accordingly.

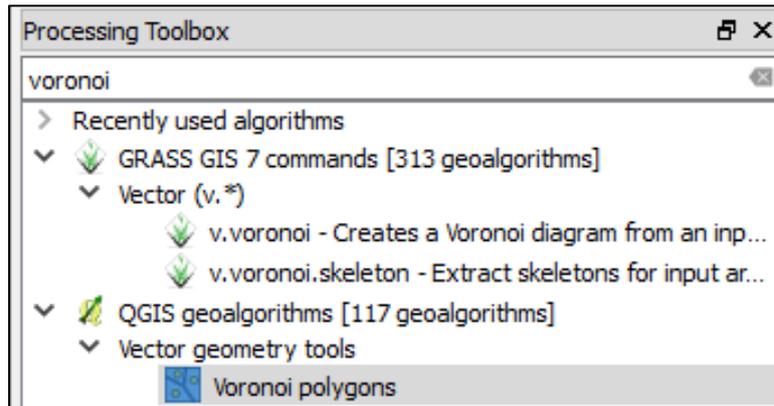


Figure 4.19: *Voronoi Polygons* geosalgorithm

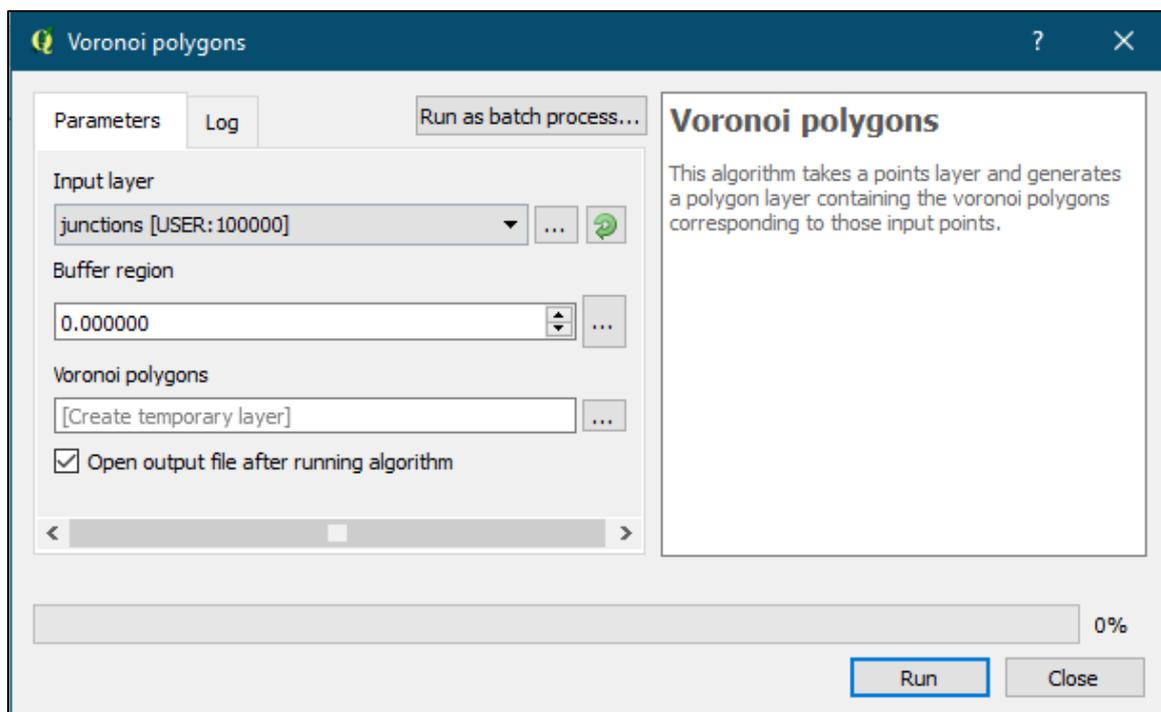


Figure 4.20: Input fields applicable to the *Voronoi Polygons* geosalgorithm

The Voronoi polygons applicable to the study area are shown in Figure 4.21.

4.4.4 Adding consumption figures to Voronoi polygons

The point file associated with the consumption figures of each water meter was overlaid in the GIS model. All these points are located within the Voronoi polygons. An average consumption figure associated with each water meter (represented by a point in the vector point file) was assigned to a specific attribute field in the ‘Swakopmund Water Meter Points’ point shapefile (Figure 4.22).



Figure 4.21: Voronoi polygons from junctions



Figure 4.22: Metering points contained within each Voronoi polygon

The *Join Attributes by Location* geospatial algorithm (Figure 4.23) was selected to combine the consumption figures from the individual points into a single value for each Voronoi polygon.

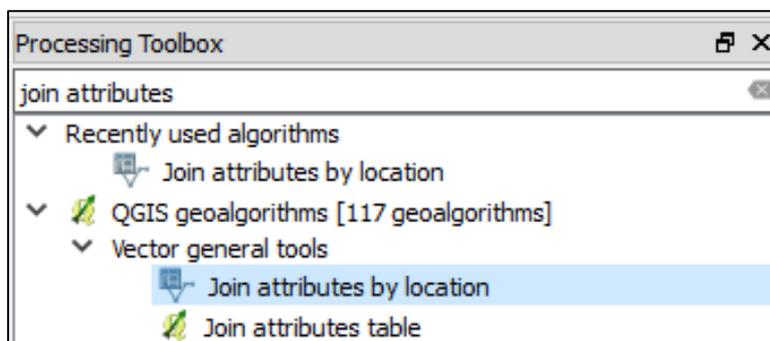


Figure 4.23: *Join Attributes by Location* geospatial algorithm

The Voronoi polygon shapefile created from the junctions and the point file containing the consumption data was set as the target vector layer and the join vector layer, respectively. The geometric predicate was set to include all points that are contained within the polygon areas of the Voronoi layer. It is important to note that the attribute summary setting (*cf.* Figure 4.24) must be set to: '*Take summary of intersecting features*'; otherwise, the consumption figures would not be inclusive of all records contained within a Voronoi cell, but would only include the value of the first data record encountered. Statistics for summary settings were required to be set to *Sum*, since the total sum of the consumption figures per Voronoi cell is required. Additional statistics (*e.g., mean, min, and max*) were included to enable further statistical analyses. The *Joined Table* setting was also selected to keep all records, including non-matching target records, especially in cases where a Voronoi cell may not contain any consumption figures covered by its area.

A resultant layer was also created which contains the original Voronoi polygon features and the sum of the consumption figures located within each polygon area.

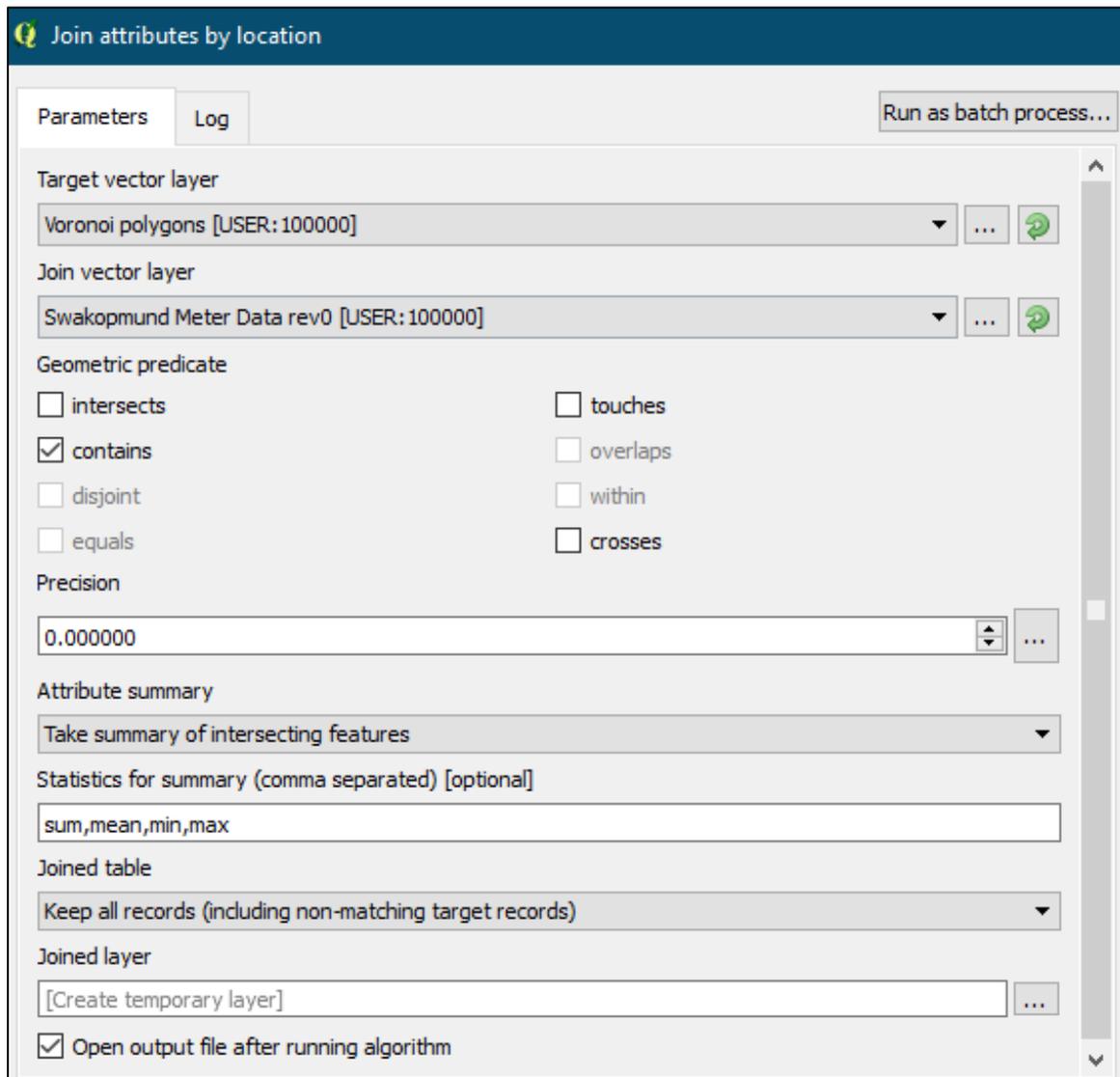


Figure 4.24: Settings for the *Join Attributes by Location* geospatial algorithm

4.4.5 Addition of consumption figures to nodes

A common attribute field (*i.e.*, DC_ID) exists between the Voronoi polygon and point shapefiles. Hence, the common attribute field was also transferred to the joined layer to link the consumption figures to the junctions (*cf.* Figure 4.25). At this point, special care was taken to ensure that the consumption figures' units were correct, *i.e.*, litres per second. Other units may be used; however, it should be noted that the setup of the project defaults within EPANET have to be aligned with the unit values allocated during this step.

For the Swakopmund data set, the average annual daily demand (AADD; (kl/day) figures were multiplied by a 1 000 and divided by 86 400, resulting in demand values in litres per second. The calculation was done during the transfer of the attribute values from the joined attribute column to the junctions' demand attribute column. As shown in Figure 4.26, an expression was compiled using the expression dialog for the calculation of attribute values.

With reference to Figures 4.25 and 4.26, it is important to note that the 'sumCONSUMPTIO'-field refers to the sum of all the AADDs of the demand nodes located within a Voronoi polygon cell.

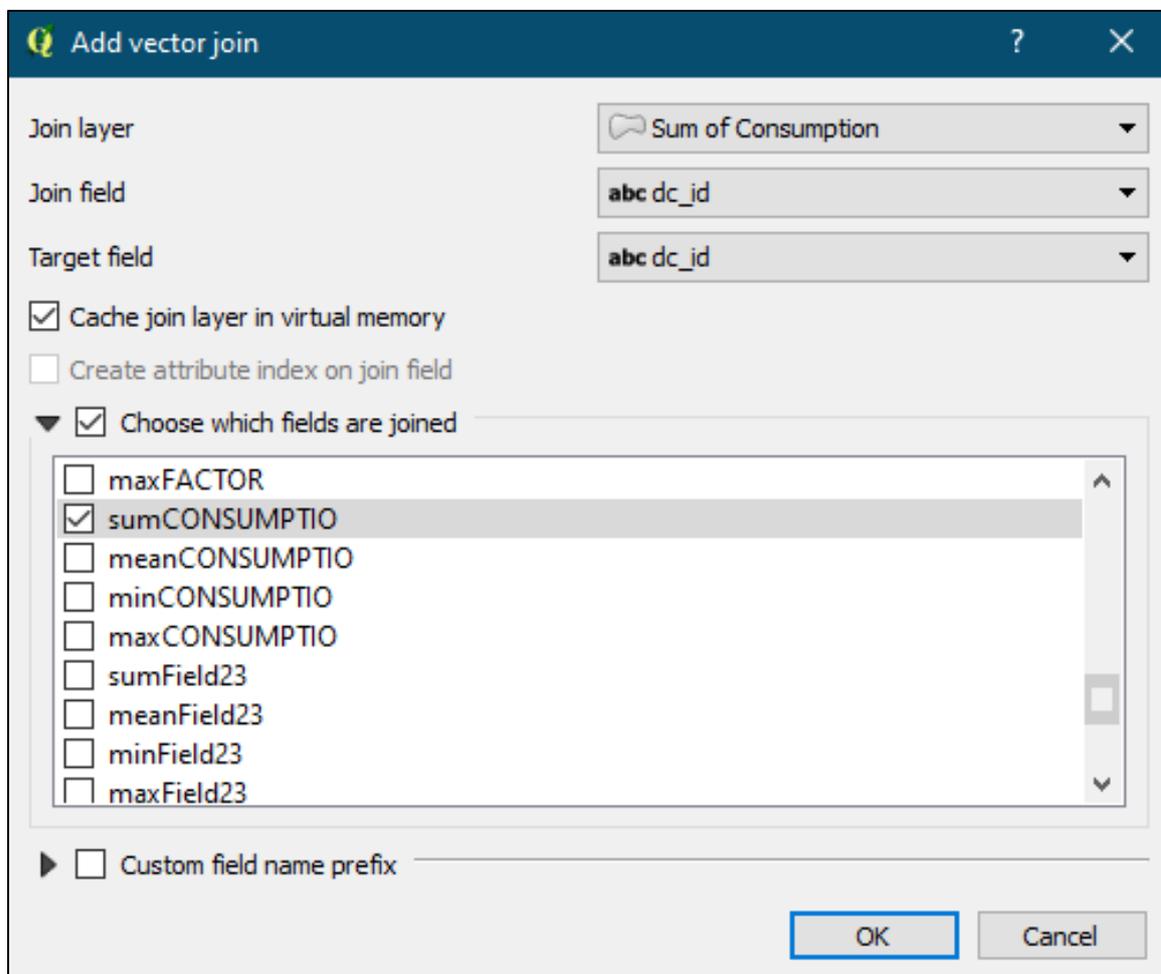


Figure 4.25: Vector join settings for linking consumption figures to junctions

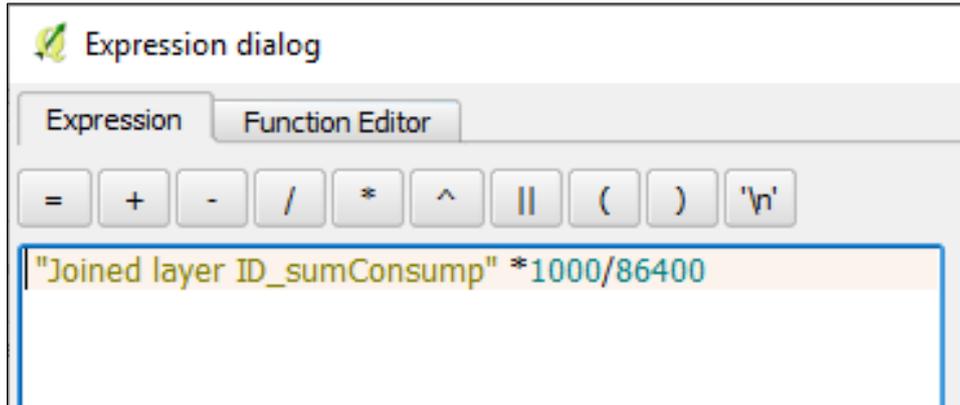


Figure 4.26: Expression for the conversion of average daily consumption figures

4.5 Exportation of Data with *GHydraulics* Plugin

All the fields that were required for the successful exportation of information were populated and the associated shapefiles were created. By selecting the settings tab under the *GHydraulics* menu as shown in Figure 4.27, the relevant shapefile layers were allocated to the correct predetermined fields within the *GHydraulics* plugin.

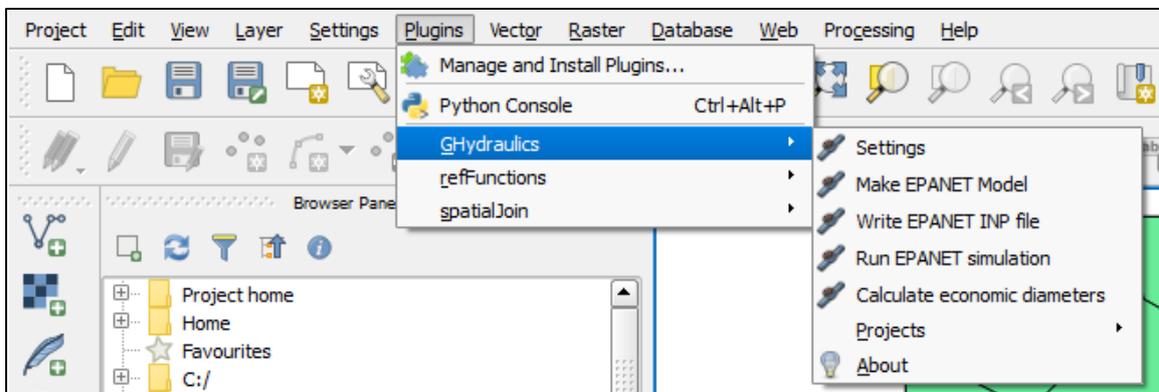


Figure 4.27: Menu for the associated *GHydraulics* commands

No reservoirs, pumps, tanks, *etc.* were added to the GIS database for exportation to EPANET. Furthermore, the purpose of the study is to use EPANET as analysis tool; hence, the 'Write EPANET INP file' command was selected. For smaller networks, inputs can be made as to run the analysis within QGIS using EPANET as analysis engine. However, troubleshooting using the latter method becomes

difficult when compared to the clear error messages generated within the EPANET interface.

The Darcy-Weisbach (litres per second template) was used for exportation due to the fact that the demands allocated to the individual junctions were in litres per second. Although the friction coefficient was specified for the Hazen-Williams equation, the Darcy-Weisbach template was used for exportation purposes. The required adjustments to the head loss equation were implemented in EPANET (*cf.* Section 4.6.1).

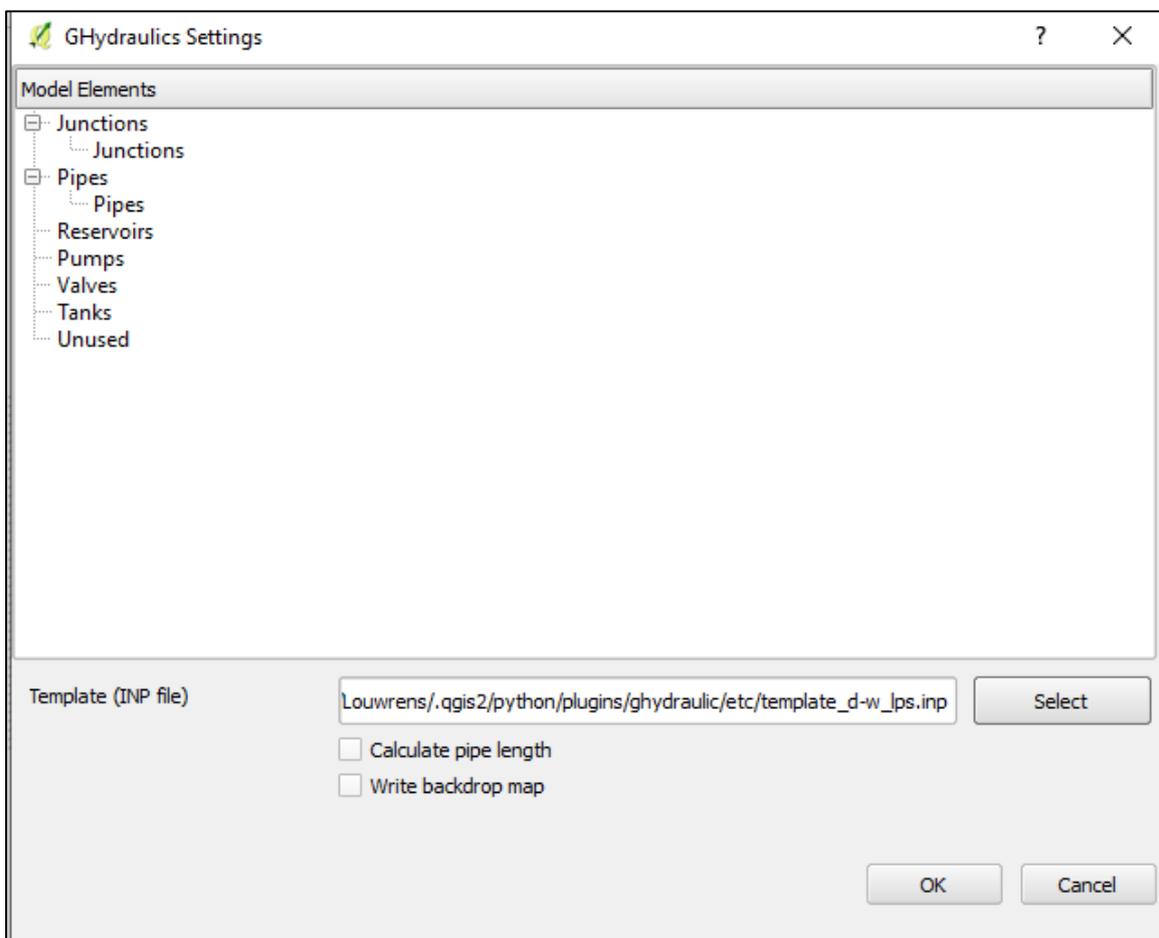


Figure 4.28: Export settings for an EPANET INP file for the *GHydraulics* plugin

4.6 WDN Model Setup in EPANET

The hydraulic analysis of the WDN was done by means of EPANET. It was preferred to conduct all hydraulic analyses using only the EPANET software to minimise risks of instability where EPANET is used in the background as analysis

engine. As the software develops in the future, it may be more feasible to conduct analyses within the QGIS software package; however, for the purposes of this study, the functionality of the *GHydraulics* plugin did not lend it towards the full scope of the analysis, especially in terms of demand patterns and peak factors.

4.6.1 Hydraulic project parameters

By making use of the ‘Darcy-Weisbach litres per second’ template within the *GHydraulics* plugin, the hydraulic project parameters were set up automatically. At this point, it is important to note that the friction coefficient (in the attribute table of the shapefile) was specified according to the Hazen-Williams C-factor of 140. The *GHydraulics* plugin does not have a standard template for the Hazen-Williams equation in combination with the ‘litres per second’ flow units. Therefore, the above template was initially used, bearing in mind that the settings as per the template applied to the exported INP-file, can be altered in the *Hydraulic Options* menu within EPANET. Upon accessing the INP-file in EPANET, the flow units were confirmed to be in ‘litres per second’ and the head loss formula was set to ‘H-W’ for the Hazen-Williams equation. The parameters used for the setup in EPANET are shown in Figure 4.29.

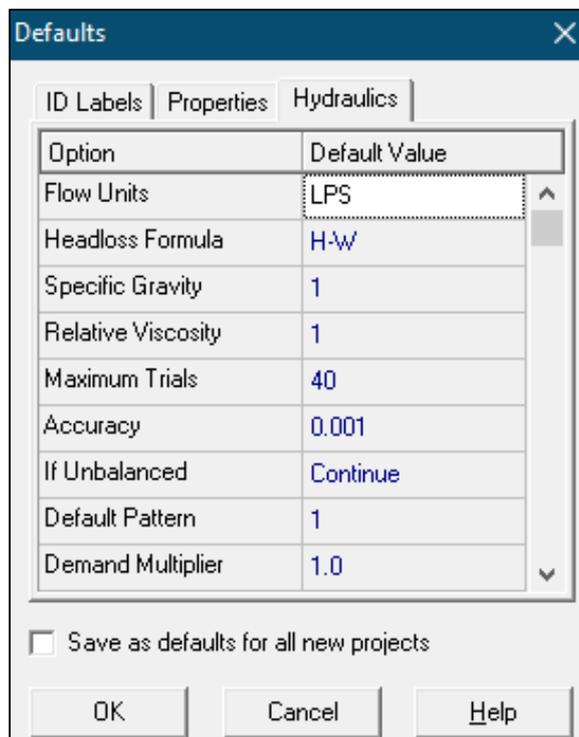


Figure 4.29: Parameter settings in EPANET using the hydraulic model options

4.6.2 Time patterns

According to Rossman (2000), a time pattern causes demands at nodes to vary in a periodic way over the course of the day. Hence, based on the latter, the time pattern as referred to in EPANET is similar to the demand pattern as described in Chapter 2, Section 2.9.2. A time pattern was used to apply multipliers to junctions throughout a 24-hour analysis cycle. The maximum instantaneous peak factor for the WDN was based on Eq. (5.3); please refer to Chapter 5, Section 5.3.

During this study, no bulk water meter consumptions linked to diurnal data were available. In liason with the representatives of the Swakopmund Municipality, a theoretical demand pattern was plotted. The theoretical demand pattern was based on engineering judgement and did not take into account any non-revenue water or losses which normally form part of the bulk meter data.

4.6.3 Reservoir settings in EPANET

In EPANET, tanks and reservoirs define the two types of storage that are available for modelling purposes. A reservoir is defined in EPANET as a theoretical endless water supply or body of water such as an aquifer, lake, river or tie-in with another system which serves as supply to the network (Rossman, 2000). The latter is similar to what is referred to in the Southern African context as surface water resources, *i.e.*, dams, rivers, streams or lakes (CSIR Boutek, 2005). Storage tanks in EPANET refer to structures with a defined volume and minimum and maximum operating levels which vary according to the outflow volumes attributed to the demand pattern in the WDN (Rossman, 2000). Storage tanks are colloquially referred to as reservoirs or storage reservoirs in the Southern African context.

Based on the fact that the storage capacity and levels of the reservoirs of the Swakopmund Municipality were not included as part of the analysis, a reservoir node was inserted in the WDN within EPANET at the position of the supply reservoir. The supply reservoir 'Total Head'-value was set in the properties menu, since this is the only required variable to be entered for the reservoir along with the 'Reservoir ID' or description.

4.7 Data Analysis in EPANET

The preceding steps as discussed in Sections 4.1 to 4.6, *i.e.*, the compilation of data sets and setup of the EPANET WDN model, enabled and culminated into the actual hydraulic analysis being run in EPANET. According to Rossman (2000), EPANET will issue specific *Error and Warning* messages when problems are encountered in running a hydraulic analysis. The five most common errors listed by Rossman (2000) are:

- (a) Pumps cannot deliver the flow or head;
- (b) Network is disconnected;
- (c) Negative pressures exist;
- (d) System unbalanced; and
- (e) Hydraulic equations unsolvable, *i.e.*, numerically unstable.

The most common error encountered during the setup of the EPANET WDN model was associated with network connectivity. From these errors that were identified, corrective measures were proposed as set out in the preparatory steps described in the methodology. In essence, error checking became an iterative process of correcting identified errors in the GIS shapefile, exporting data to EPANET, setting up of EPANET parameters and analysis of the data. The error checking process continued until a successful run of the WDN model was achieved.

After a number of rounds of editing of the GIS data, exporting to EPANET and running of the model, errors regarding connectivity became less abundant, and corrections were made within EPANET. However, it is important to emphasise that all changes to the EPANET model were duplicated in the GIS shapefile in order to maintain identical representations of the WDN in both the GIS and EPANET models. The aforementioned process would only apply to minor changes to individual pipes or nodes and in cases where the user does not wish to perform the setup of hydraulic parameters and other EPANET functions after every minor change in the GIS model. In cases where concerns regarding the practicality and the danger of 'duplication' using the afore-mentioned procedure exist, it is

recommended that all changes be limited to the GIS model and the resulting data should subsequently be exported to EPANET. Such an approach will ensure identical data sets being used in both the GIS and EPANET environment. In terms of updating the GIS and hydraulic models upon the installation of new pipes or decommissioning or upgrading of existing pipelines; similar procedures are recommended to ensure that both the GIS and EPANET models are representative of the actual on-site conditions.

In considering the continuous enhancement and development of plugins in the QGIS environment, a new QEPANET-plugin has been developed and released in November 2019. The latter plugin enables the analysis of WDNs within the QGIS environment by using EPANET as the processing engine. Hence, the need to update different data sets in different models or environments as discussed above, is mitigated.

The next chapter provides the results based on the methodology adopted in this chapter.

CHAPTER 5 : RESULTS AND DISCUSSION

In this chapter, the research results are presented and discussed with reference to the main purpose of this study, *i.e.*, to develop a methodological approach based on the integration of open-source software and engineering practices associated with WDN design.

As a prelude, comparative sensitivity analyses between the different head (friction) loss equations are conducted and the associated impact thereof on the hydraulic analysis of a WDN model is discussed in general. Thereafter, the equations derived to determine water demand based on erven size and peak factors based on equivalent erven, are presented. This is followed by a short discussion of the hydraulic analysis results of the Swakopmund WDN.

In conclusion, with reference to the main purpose of the study, a methodological framework is presented by two summative flow diagrams representative of the series of flow diagrams presented in Chapter 4 (*cf.* Figures 4.1 to 4.9) to ultimately serve as a practitioners' guideline to integrate open-source software and engineering practices when WDN models of an existing network need to be compiled.

5.1 Sensitivity Analysis of Head Loss Equations

A comparison between Eqs. (2.1) to (2.3), was performed in terms of the different parameters as discussed in Chapter 2. In other words, the impact of variables such as: (i) discharge (Figure 5.1), (ii) diameter (Figure 5.2), (iii) pipe length (Figure 5.3), and (iv) friction coefficient values (Figure 5.4) on the variation in head loss was investigated using Eqs. (2.1) to (2.3). As a result, Figures 5.1 to 5.4 depict the percentage variation in head loss as a function of the percentage change in each variable as listed in (i) to (iv), while all the other parameters not being considered, remained constant. The comparison is made in order to illustrate the possible effect that errors in 'real-world' data could have on the underlying calculations in a WDN model. Tables B.1 to B.3 inclusive of the results associated with these comparisons are included in Appendix B.

In Figures 5.1 to 5.4, Eqs. (2.1) to (2.3) were used where: C = Hazen-Williams roughness coefficient (dimensionless), D = diameter (m), f = Darcy-Weisbach friction coefficient (dimensionless), g = gravitational constant (9.81 m/s^2), h_f = head loss due to friction (m), L = length of pipe (m), n = Manning's n -value ($\text{s/m}^{1/3}$), and Q = discharge (m^3/s).

It is evident from Figure 5.1 that the differences between Eqs. (2.2) and (2.3) are negligible when variations in discharge are considered and that an increase in head loss is associated with an increase in discharge. This non-linear relationship is less sensitive at a higher discharge rate; however, a significant change in head loss is associated with a negative variation in discharge, *i.e.*, a 250% variation could be ascribed to a 50% reduction in discharge. Hence, where uncertainty exists regarding demands (discharge), it may be considered better practice to apply a higher limit value of the demands due to the non-linear relationship between the discharge and the associated head loss.

Figure 5.2 is illustrative of the relationship between Eqs. (2.1) to (2.3) in terms of the variation in head loss in relation to the variation in pipe diameter. An inverse relationship exists between these two variables, *i.e.*, with a reduction in pipe diameter, a significant increase in head loss is evident. Typically, a 10% decrease in the pipe diameter would result in a 50% increase in head loss. The latter relationship was also highlighted by Mays (2000), Neelakantan *et al.* (2008) and Gomes *et al.* (2009), and it was emphasised that the use of minimum pipe diameters for water distribution network designs is not a feasible option in most circumstances, as this may lead to excessive or unnecessary high pumping and associated energy costs over the whole network life-cycle. In agreement with Babić *et al.* (2014), it was also evident that smaller pipe diameters or partially open valves would also lead to a drop in pressure.

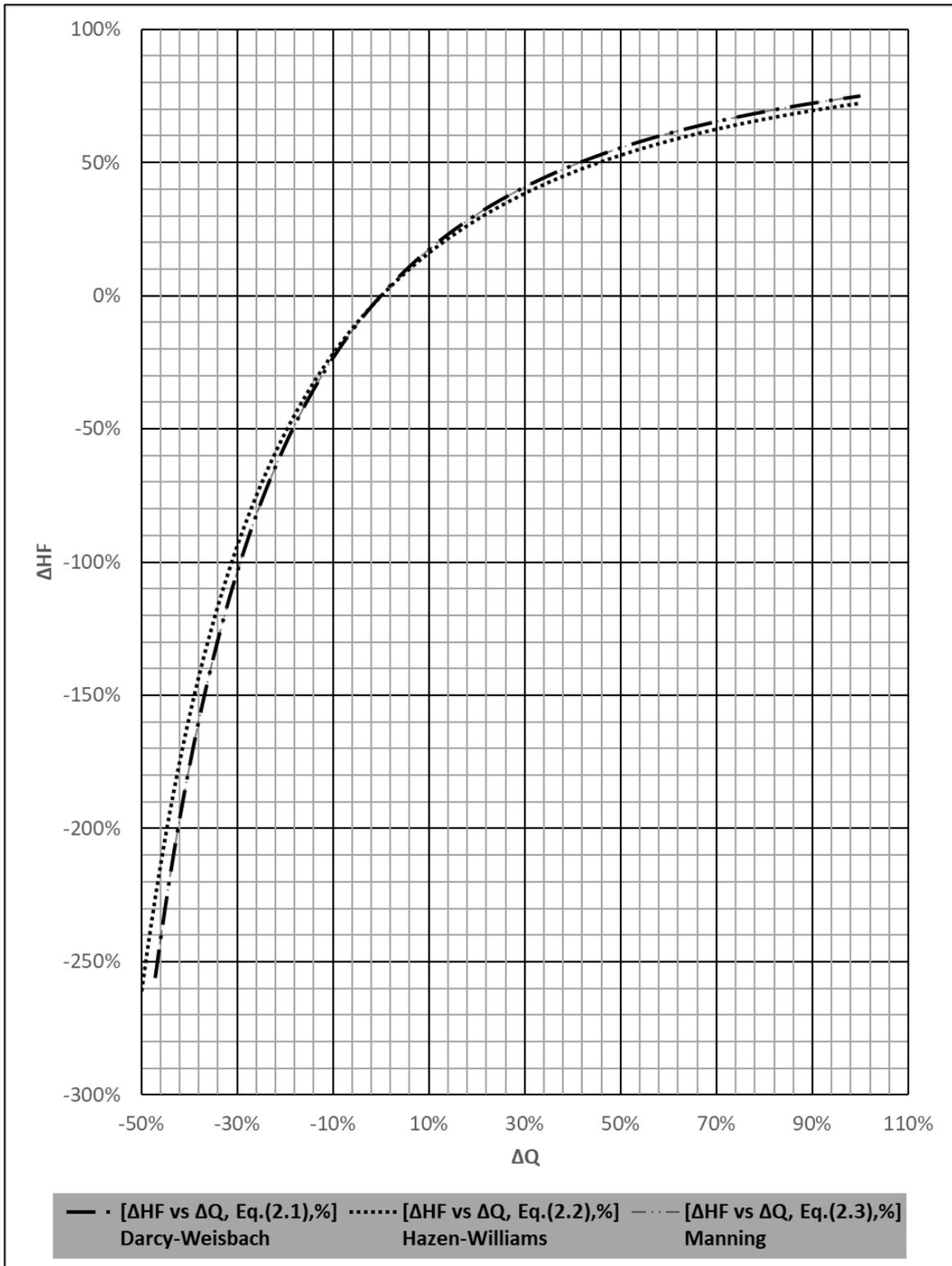


Figure 5.1: Percentage variation in head loss associated with the percentage variations in discharge

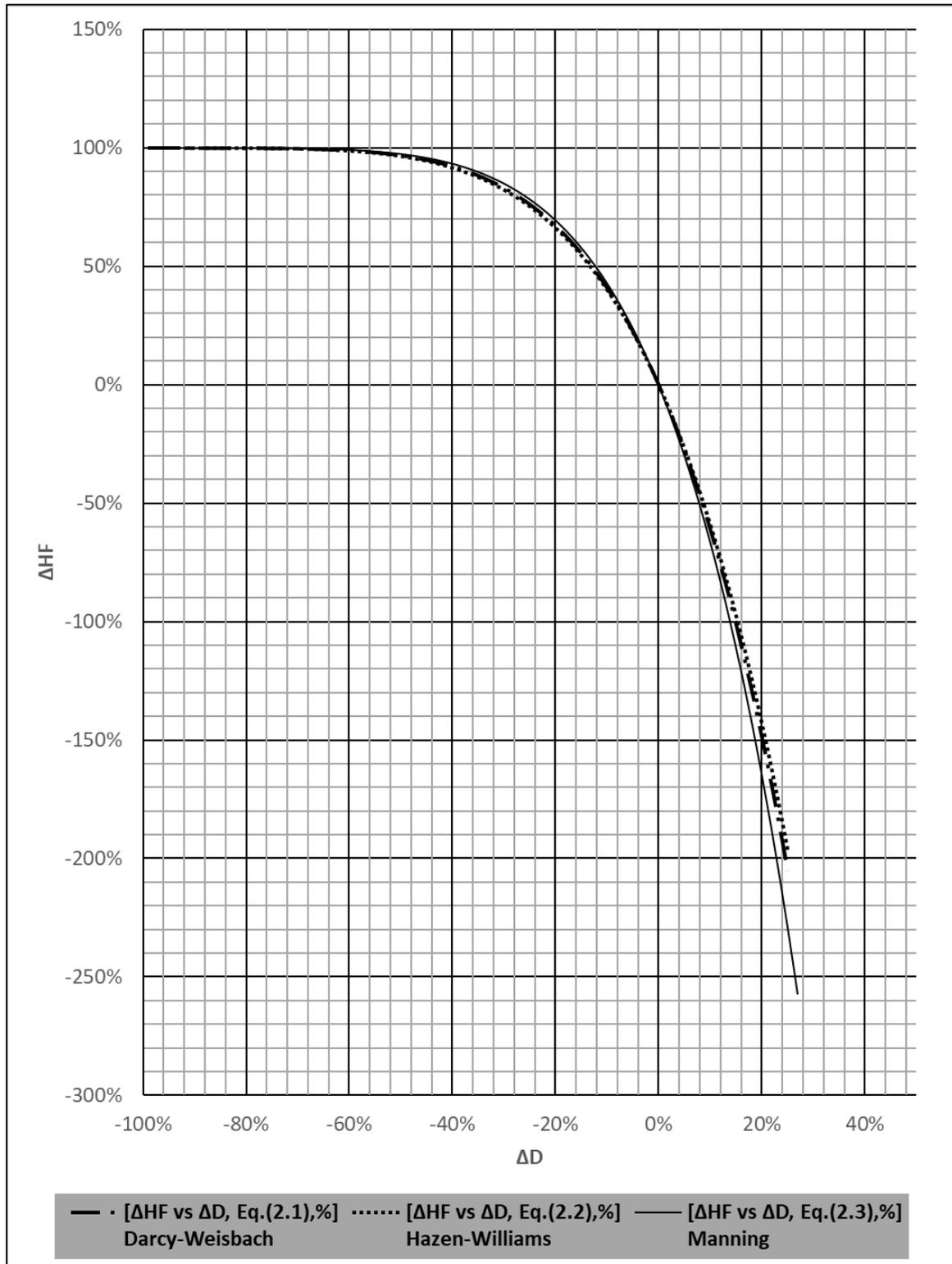


Figure 5.2: Percentage variation in head loss associated with the percentage variations in diameter

In terms of pipe diameters, the process may not be as rudimentary as it seems. The nominal diameter refers to the typical diameter of the pipe commercially available and referred to by industry, e.g., 200 mm Ø; however, the internal

diameter is the actual measurement from the inside of the pipe wall to the opposite side and this measurement varies based on the pressure class and material (Walski *et al.*, 2003).

The nominal diameter and the internal pipe diameter differ in most cases, while the former is commonly used in WDNs. Nevertheless, the use of nominal diameters for modelling purposes is not necessarily correct due to the decrease in diameter with time that can be attributed to accumulation of chemical deposits, corrosion products, tuberculation or scaling (Mays, 2000; Walski *et al.*, 2003; Ebacher *et al.*, 2011).

Another factor to bear in mind while assigning diameters to a WDN model, is that effective pipe diameters change with time depending on water quality and pipe material amongst others (Mays, 2000). The latter specifically applies to older, unlined iron pipelines which are prone to corrosion and to areas where water quality tends to be an issue in terms of chemical and/or mineral deposits. In general, minor changes to diameter are accounted for in the determination of the friction coefficient, although best practice is to confirm and utilise the correct diameter information as far as possible.

Figure 5.3 is illustrative of the relationship between Eqs. (2.1) to (2.3) in terms of the variation in head loss in relation to the variation in pipe length. Seen in the light that pipe length is not subjected to an exponent in Eqs. (2.1) to (2.3), it is regarded as being neutral having no influence on any variation.

Normally, pipe length between nodes is determined by rudimentary commands within GIS software. Within QGIS, length attributes can be updated by means of the geometry function command for length in the field calculator. Lengths measured in GIS are dependent on the correct projection system being applied to the shapefiles and information data sets. Should the projection parameters not be correct, inaccurate lengths will be calculated and in turn have a negative impact on any subsequent hydraulic analyses. The risk of having an error due to variation in length in the hydraulic model is highly unlikely if standard protocol is adhered to for the compilation of GIS shapefiles.

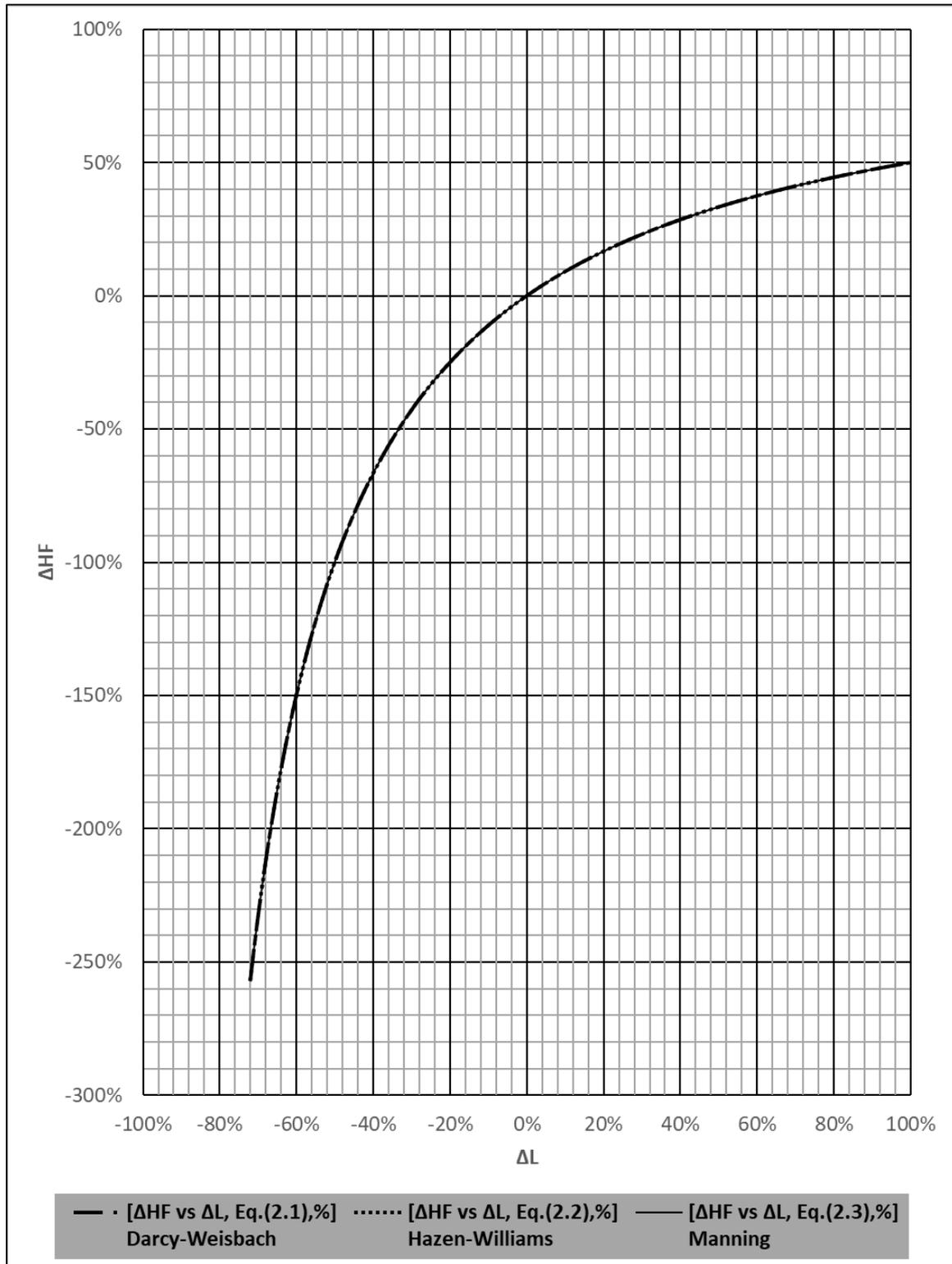


Figure 5.3: Percentage variation in head loss associated with the percentage variations in pipe length

In Figure 5.4 it is evident that the variation in friction coefficient values yields significant variations in head loss. This emphasises that special care should be taken when friction coefficient values are selected and applied to data sets.

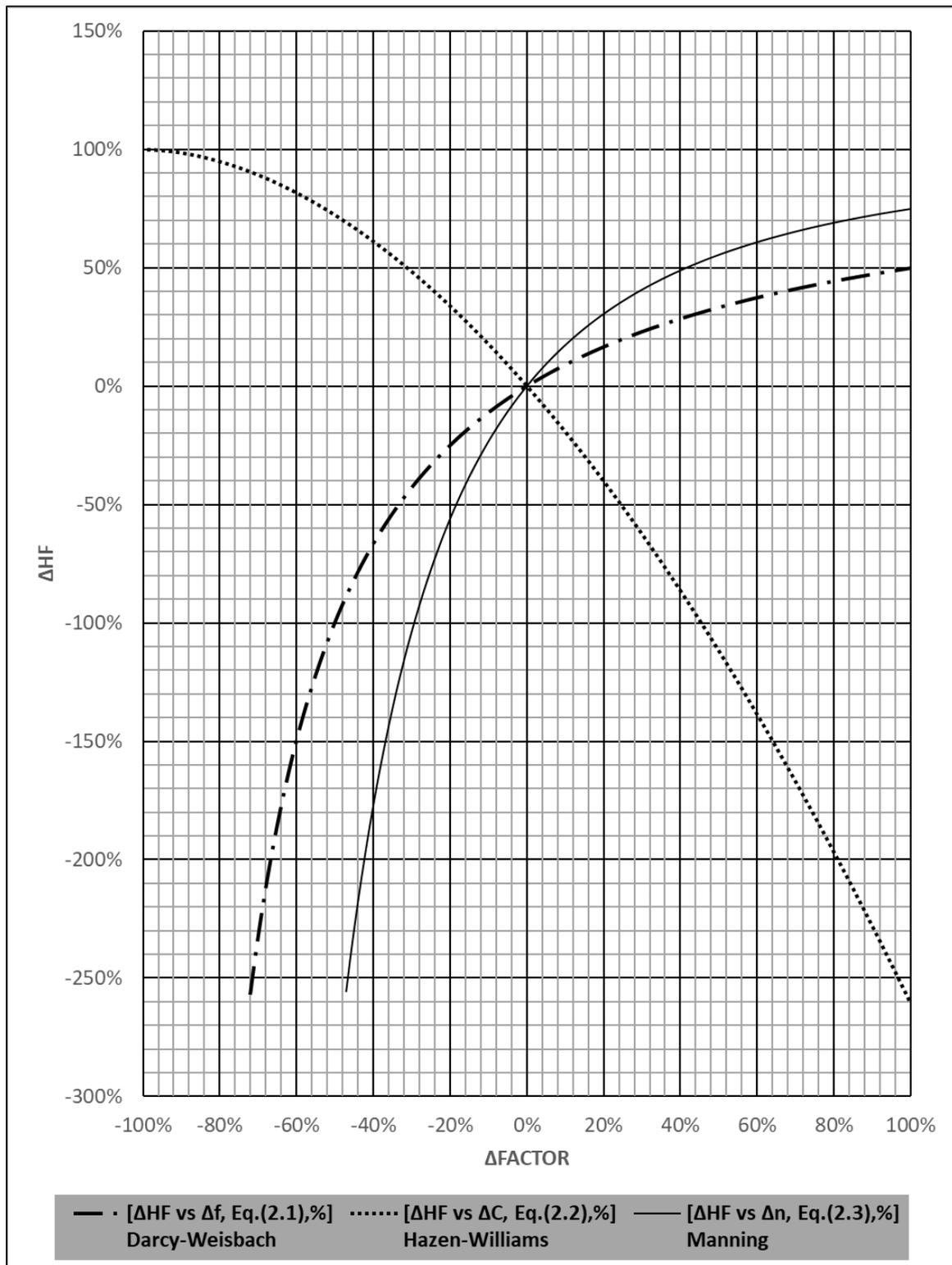


Figure 5.4: Percentage variation in head loss associated with the percentage variations in friction coefficient values

In general, upon completion of the WDN model's infrastructure data assembly, calibration is done by altering pipe diameter values or by adapting friction coefficient values, of which the latter is usually done (Ebacher *et al.*, 2011).

In cases where the friction coefficient is unknown for a certain pipe section in the WDN, a smaller Hazen-Williams coefficient may be adapted, especially with older pipes (Neelakantan *et al.*, 2008). The results shown in Figure 5.4 are in agreement with the latter statement, since lower *C*-values for the Hazen-Williams friction coefficient will yield higher head losses. However, the opposite would be applicable to the Darcy-Weisbach and Manning friction coefficients, respectively.

The typical percentage variations associated with different Hazen-Williams *C*-values and the resulting percentage differences in head loss are listed in Table 5.1.

Table 5.1: Typical head loss percentage variations associated with different Hazen-Williams *C*-values

Pipe material	Hazen-Williams <i>C</i> -values		% Difference from lower value	% Variation in head loss from Figure 5.4
	Lower	Upper		
Asbestos Cement	120	140	17%	-34%
Cast Iron	130	140	8%	-15%
Concrete/Lined Concrete	120	140	17%	-34%
Galvanised Iron	120	120	0%	0%
Plastic	140	150	7%	-13%
Steel	140	150	7%	-13%

Table 5.2 is illustrative of a hypothetical scenario considering a 250 mm Ø plastic pipe with a flow velocity of 2.5 m/s and a variation in the pipe roughness from 0.003 mm to 0.03 mm, while considering the average roughness (0.015 mm) as the ‘correct value’. Hence, the subsequent percentage variations in the Darcy-Weisbach *f*-values from the average and the corresponding variation in head loss from Figure 5.4, are listed in Table 5.2 for illustration purposes. However, it is important to note that these Darcy-Weisbach *f*-values are subjected to changes in the Reynold’s number, which in turn, is also dependent on the velocity and viscosity values. Hence, the percentage deviations listed in Table 5.2 will be different for different scenarios and/or conditions.

Table 5.2: Head loss percentage variations associated with different Darcy-Weisbach f -values

f Eq. (2.1a)	e (m)	D (m)	Re Eq. (2.1b)	V (m/s)	μ (m ² /s)	% Variation in f -values from average	% Variation in head loss from Figure 5.4
0.01306	0.000003	0.25	553097	2.5	1.13x10 ⁶	5.25%	5.30%
0.01313	0.000004	0.25	553097	2.5	1.13x10 ⁶	4.78%	4.80%
0.01319	0.000005	0.25	553097	2.5	1.13x10 ⁶	4.31%	4.40%
0.01326	0.000006	0.25	553097	2.5	1.13x10 ⁶	3.84%	3.90%
0.01332	0.000007	0.25	553097	2.5	1.13x10 ⁶	3.39%	3.40%
0.01338	0.000008	0.25	553097	2.5	1.13x10 ⁶	2.94%	3.00%
0.01344	0.000009	0.25	553097	2.5	1.13x10 ⁶	2.50%	2.60%
0.01350	0.00001	0.25	553097	2.5	1.13x10 ⁶	2.07%	2.10%
0.01356	0.000011	0.25	553097	2.5	1.13x10 ⁶	1.64%	1.70%
0.01362	0.000012	0.25	553097	2.5	1.13x10 ⁶	1.22%	1.30%
0.01367	0.000013	0.25	553097	2.5	1.13x10 ⁶	0.81%	0.90%
0.01373	0.000014	0.25	553097	2.5	1.13x10 ⁶	0.40%	0.50%
0.01379	0.000015	0.25	553097	2.5	1.13x10 ⁶	0.00%	0.00%
0.01384	0.000016	0.25	553097	2.5	1.13x10 ⁶	-0.40%	-0.40%
0.01389	0.000017	0.25	553097	2.5	1.13x10 ⁶	-0.79%	-0.80%
0.01395	0.000018	0.25	553097	2.5	1.13x10 ⁶	-1.18%	-1.20%
0.01400	0.000019	0.25	553097	2.5	1.13x10 ⁶	-1.56%	-1.60%
0.01405	0.00002	0.25	553097	2.5	1.13x10 ⁶	-1.93%	-2.00%
0.01410	0.000021	0.25	553097	2.5	1.13x10 ⁶	-2.31%	-2.40%
0.01415	0.000022	0.25	553097	2.5	1.13x10 ⁶	-2.67%	-2.70%
0.01421	0.000023	0.25	553097	2.5	1.13x10 ⁶	-3.04%	-3.10%
0.01425	0.000024	0.25	553097	2.5	1.13x10 ⁶	-3.40%	-3.40%
0.01430	0.000025	0.25	553097	2.5	1.13x10 ⁶	-3.75%	-3.80%
0.01435	0.000026	0.25	553097	2.5	1.13x10 ⁶	-4.10%	-4.20%
0.01440	0.000027	0.25	553097	2.5	1.13x10 ⁶	-4.45%	-4.50%
0.01445	0.000028	0.25	553097	2.5	1.13x10 ⁶	-4.80%	-4.80%
0.01449	0.000029	0.25	553097	2.5	1.13x10 ⁶	-5.14%	-5.20%
0.01454	0.00003	0.25	553097	2.5	1.13x10 ⁶	-5.47%	-5.50%

It should be noted that the head loss percentage variations in Tables 5.1 and 5.2 are not directly applicable to the nodal pressures in a WDN, but applicable to the head loss (friction) in the pipelines being analysed. Hence, for example, a -34% variation associated with AC pipes based on the upper and lower limits (cf. Table 5.1), will not be applied directly to a nodal pressure (e.g., 60 m) as such, but it will be applicable and illustrative of the possible variations in head loss in the pipeline.

5.2 Derivation of Water Demand Equation based on Erf Size

CSIR Boutek (2005) has documented upper and lower water demand limits (Figure 5.5) for residential erven in the medium-to-high income groups based on erf size. In this study, for the high-level analyses and processing of large data sets, equations were derived to represent the upper and lower demand limits, respectively.

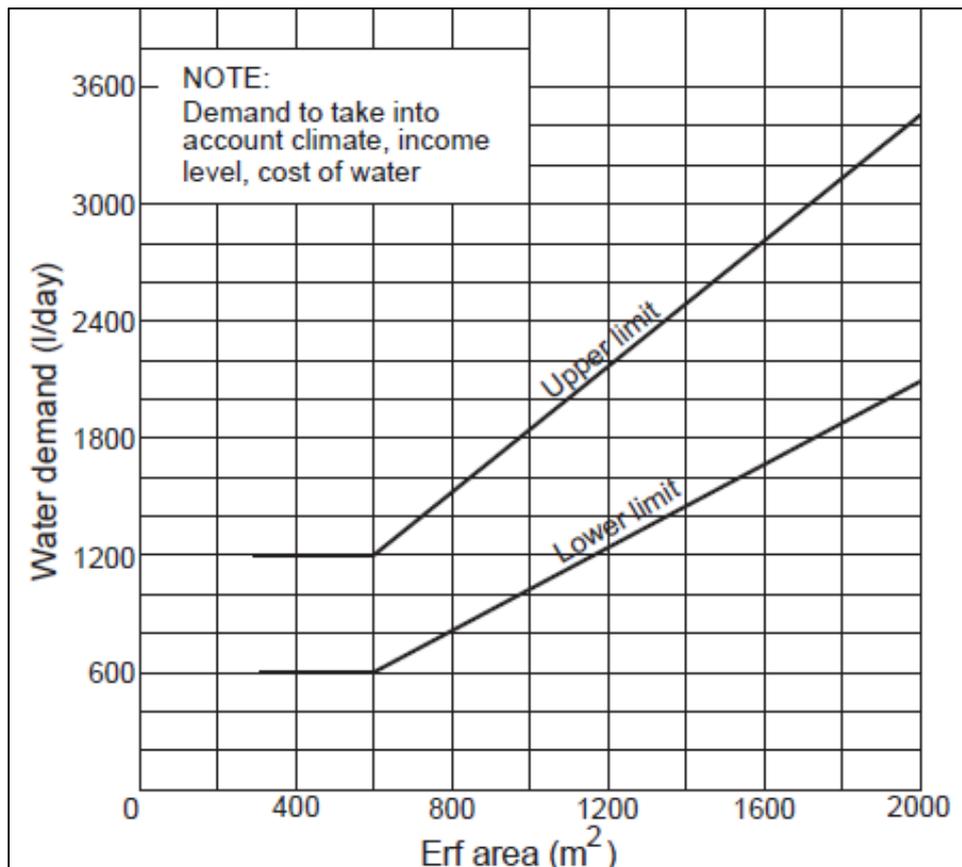


Figure 5.5: Upper and lower demand limits based on erf size (CSIR Boutek, 2005)

Emphasis is placed on the fact that these equations will only be applicable to erven in developed areas and also only for high-level analysis purposes. In other words, the linear regressions shown in Eqs. (5.1) and (5.2), would only apply to erf sizes exceeding 600 m². In cases where the erf sizes are less than 600 m², a constant lower and upper demand of 600 l/day and 1 200 l/day, would respectively apply.

$$D_L = 1.07A - 42.857 \tag{5.1}$$

$$D_U = 1.6A + 240 \tag{5.2}$$

Where:

D_L = lower limit demand (ℓ/day) with a minimum value of 600,

D_U = upper limit demand (ℓ/day) with a minimum value of 1 200, and

A = erf area (m²).

Equations (5.1) and (5.2) could be included in data processing software (e.g., Microsoft Excel) or directly into attribute tables of GIS shapefiles to determine theoretical demands in the absence of metered consumption figures.

5.3 Derivation of Peak Factor Equation based on Equivalent Erven

The peak factor values associated with equivalent erven as published by the CSIR Boutek (2005) are listed in Table 5.3. In using these tabulated values, a line graph and derived power-law trend line were respectively plotted in Figure 5.6. The degree of association (R^2 -value) between the original tabulated values and the derived trend line is regarded as acceptable, with $R^2 = 0.992$. As a result, the peak factor values estimated using the derived equation, Eq. (5.3), are also listed in Table 5.3.

Table 5.3: Original peak factor (CSIR Boutek, 2005) and derived peak factor [Eq. (5.3)] values

CSIR Boutek (2005)		Eq. (5.3)	% Difference
Equivalent erven	Peak factor	Peak factor	
1	37	37.2	0%
2	30	30.4	1%
5	23	23.2	1%
10	19	18.9	0%
20	16	15.5	-3%
50	12	11.8	-2%
100	10	9.7	2%
200	8	7.9	2%
500	6	6.0	0%
1 000	5	4.9	0%
2 000	4	4.0	0%
5 000	4	Minimum value of 4 to be used for more than 2 000 erven	

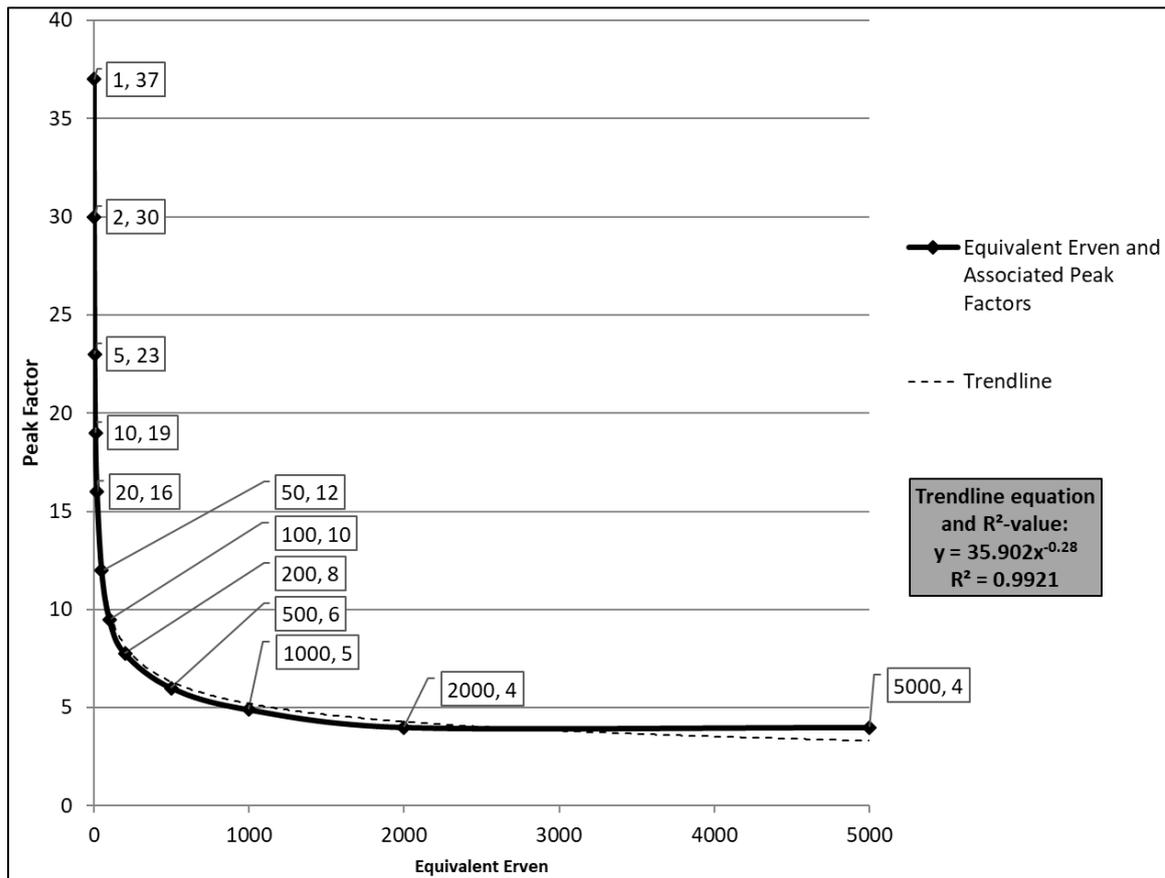


Figure 5.6: Equivalent erven and associated peak factors

$$P_f = 35.902E_e^{-0.28} \tag{5.3}$$

Where:

P_f = peak factor, and

E_e = number of equivalent erven.

In using Eq. (5.3), a resulting multiplier of 4 was obtained when E_e exceeded 2 000. In considering morning, afternoon and evening peaks, a maximum instantaneous peak of 4 was applied to the morning peak between 06:00 and 07:00. As shown in Figure 5.7, the remainder of the multipliers were based on engineering judgement and applied in such a way that the averages equal unity, since the average daily demand was used as the base demand for the nodes of the WDN. The afternoon peak could be distributed to 07:00, 08:00 and 19:00 to better conform to general South African diurnal demand patterns. However, the Swakopmund Municipality requested that an afternoon peak should be considered

during the initial analyses to provide them with a general idea of the impact of having peaks during the afternoon. The latter afternoon peaks were introduced, but in conclusion, it was recommended that diurnal demand patterns should be verified with actual monitoring to enable the future enhancements of the WDN model.

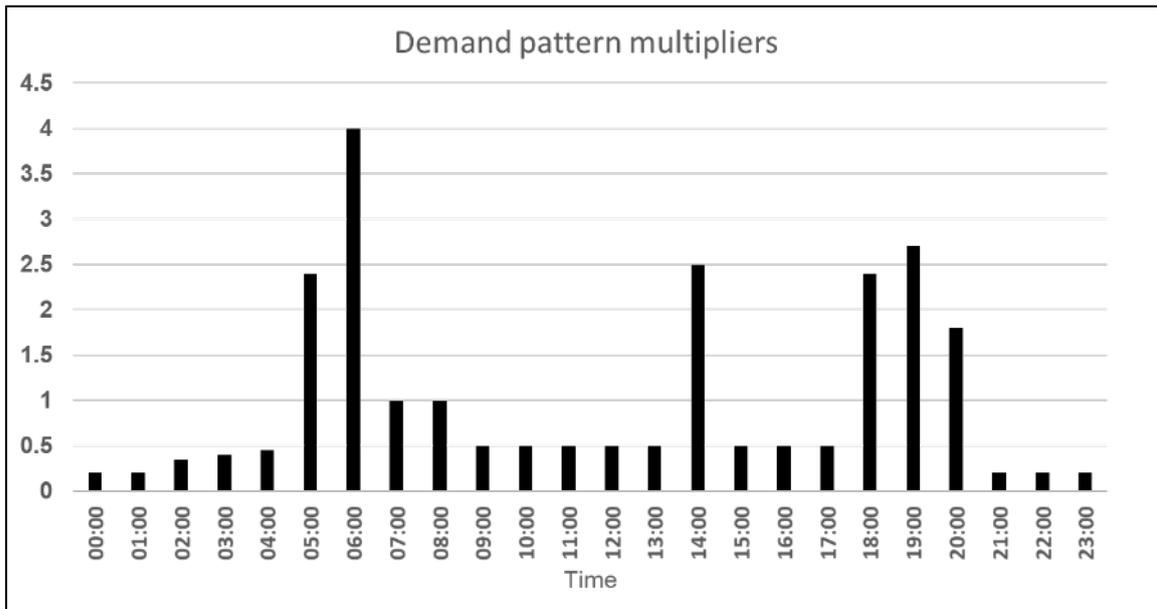


Figure 5.7: Demand-time pattern applicable to the Swakopmund WDN

5.4 Hydraulic Analysis of Swakopmund WDN Model

This section provides a short discussion of the hydraulic analysis results of the Swakopmund WDN, since the hydraulic analysis as such was not the main purpose of this study. The nodal pressures throughout the WDN model of Swakopmund are shown in Figure 5.8, while Figure 5.9 represents the flow velocities throughout the WDN model. The nodal pressures and flow velocities are based on the peak demand at 05:00 and 06:00, respectively, and according to the demand-time pattern adopted for the study (*cf.* Figure 5.7). The red dots (pressures lower than 15 m) are possibly indicative of: (i) higher lying areas which have a lower pressure due to the smaller elevation difference between the supply level of the reservoirs and the supply point (this is in contrast with the supply points located closer to sea level), (ii) areas which are prone to lower pressure due to supply pipelines being inadequately sized, or (iii) areas where the applied peak

factor is not representative of the actual peaks that are encountered in the network and therefore resulting in unrealistically high demands.

Points (i) to (iii) should specifically be focused on during the calibration of the WDN model and the identification of pressure management zones, since this will assist the practitioner to isolate pressure management zones and supply areas.

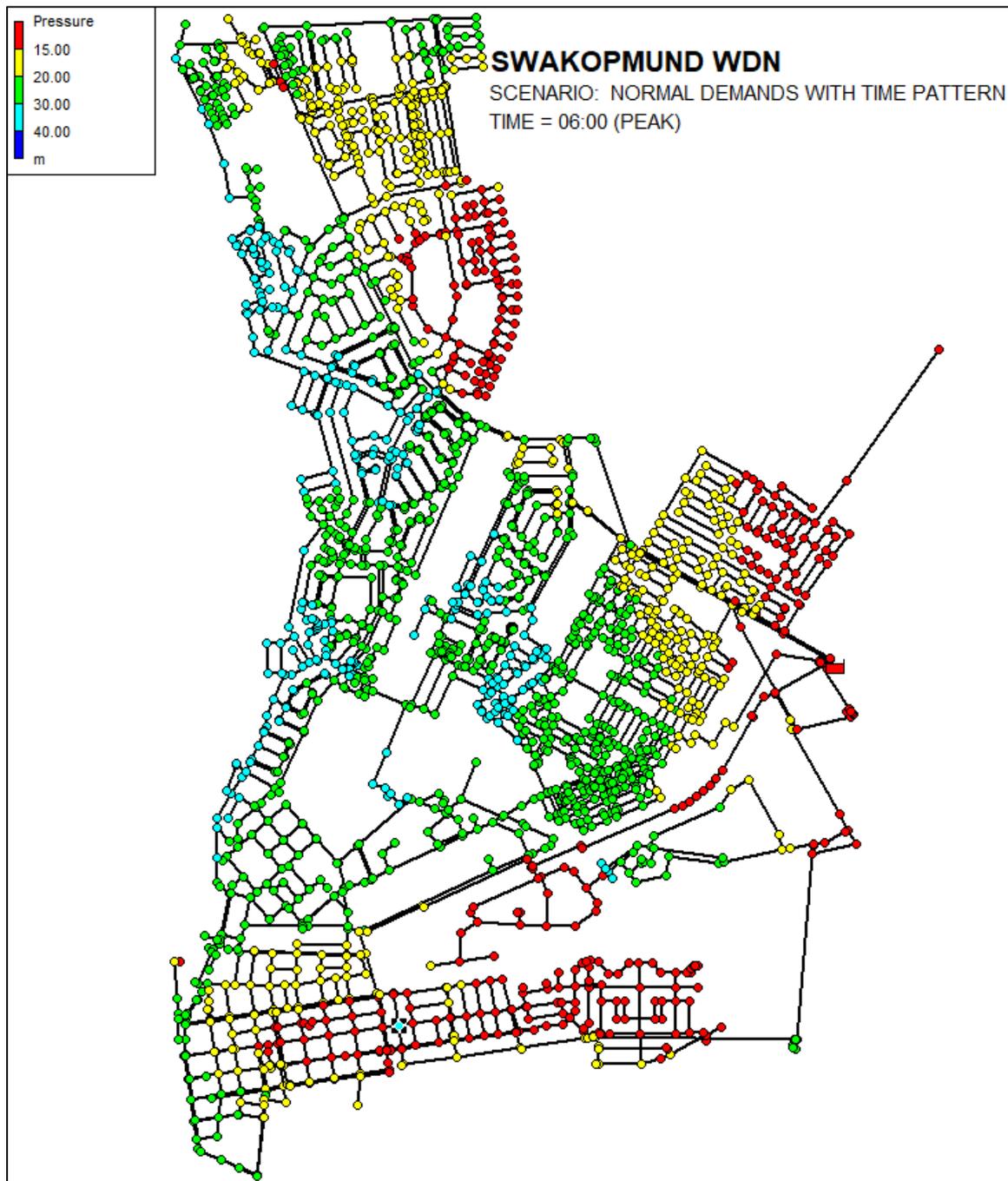


Figure 5.8: Swakopmund WDN model nodal pressures during peak demand

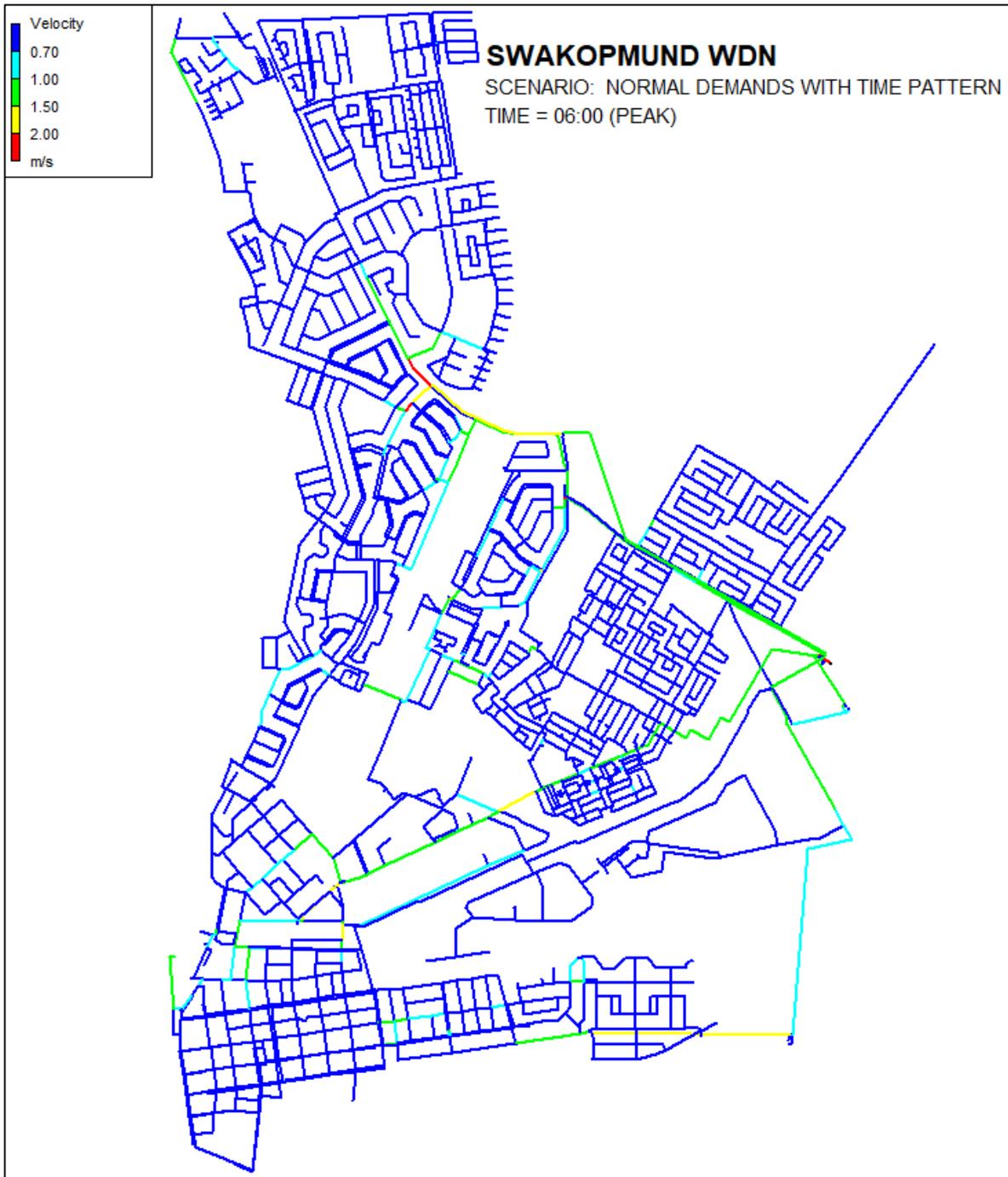


Figure 5.9: Swakopmund WDN model flow velocities during peak demand

In Figure 5.9, it is notable that for the majority of the WDN model, velocities lower than the minimum recommended velocity of 0.7 m/s in a WDN (CSIR Boutek, 2005), are evident. The latter low velocities could most likely be ascribed to the fact that, in addition to the consumer demands being supplied from the WDN, water for firefighting purposes is also supplied by the same network, in which case, the network has to cater for the higher water demands associated with firefighting

requirements as well. Furthermore, in justification of the latter low velocities, both Mays (2000) and McKenzie (2014) highlighted that for small pipes in a WDN, the sizing of the pipes is generally controlled by fire-flow requirements and consequently, the pipes are significantly oversized for average daily demands, *i.e.*, velocities are low.

In order to confirm the above statement on fire-flow requirements, a comparative analysis was done for a typical fire-flow scenario. Two hydrants each operating at 15 litres per second during a peak residential demand were considered at the northern section of the WDN. In Figure 5.10, the increase in velocity to the points earmarked for the comparative fire-flow analysis is clearly evident; hence, confirming the findings of Mays (2000) and McKenzie (2014), respectively.

5.5 Summative Methodological Framework

The summative methodological framework developed in this study is shown in Figures 5.11 to 5.13, respectively. Notably, the two key processes of the methodological framework are evident in the latter figures, *i.e.*, ‘Data Acquisition’ and ‘Data Processing’, all of which, are based on the methodology as discussed in Chapter 4.

It is envisaged that the developed methodological framework, as well as the extensive series of flow diagrams listed in Chapter 4 (*cf.* Figures 4.1 to 4.9), will not only provide guidance to the practitioner on how to integrate open-source software and engineering practices for the compilation of WDN models of an existing network, but ultimately, it possibly could also enable and assist municipalities in Southern Africa facing challenges in terms of technical skills shortages, service delivery backlogs, outdated records and budget constraints.

The final conclusions and recommendations for future research are discussed in the next chapter.

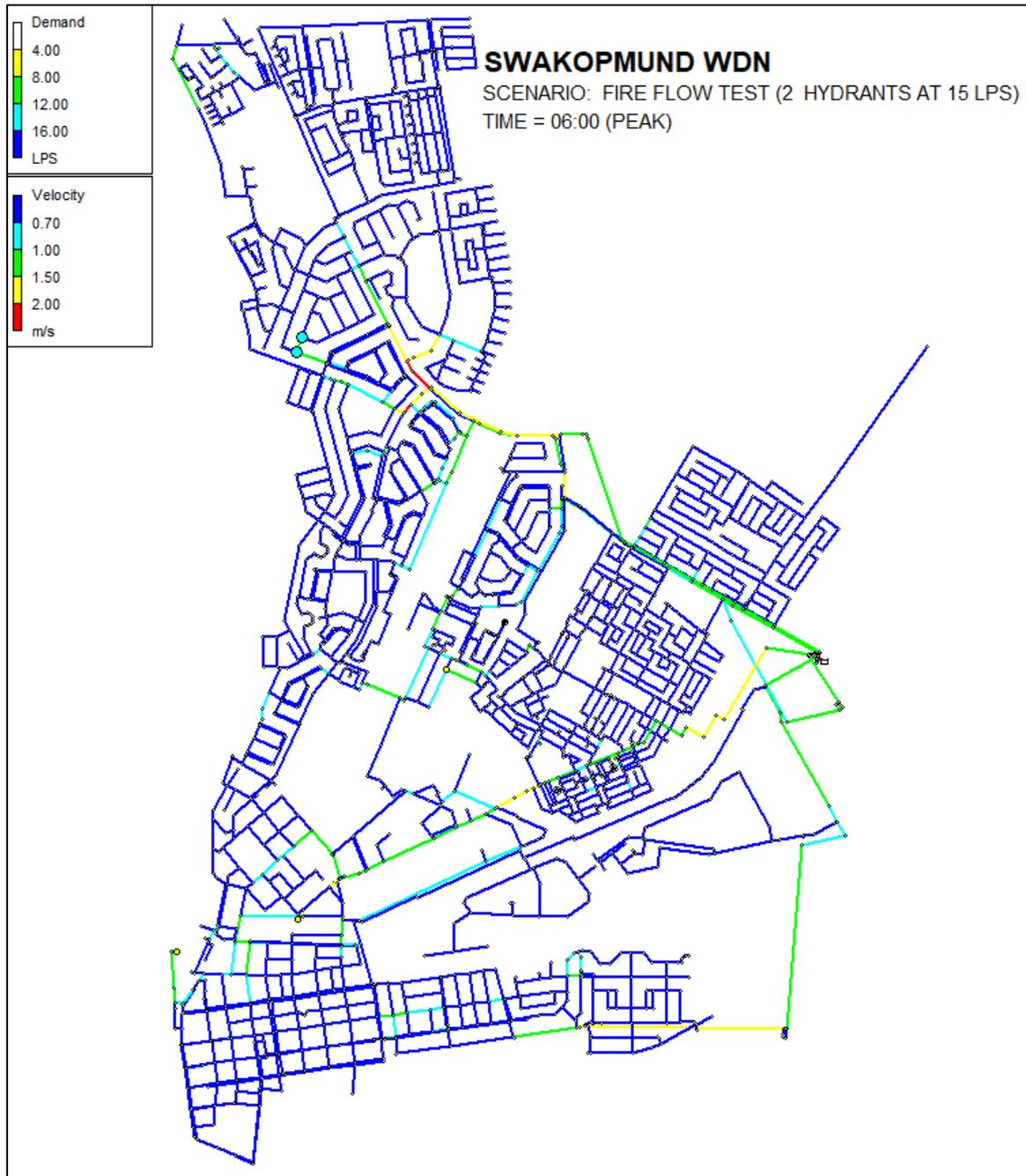


Figure 5.10: Swakopmund WDN model flow velocities inclusive of fire-flow during peak demand

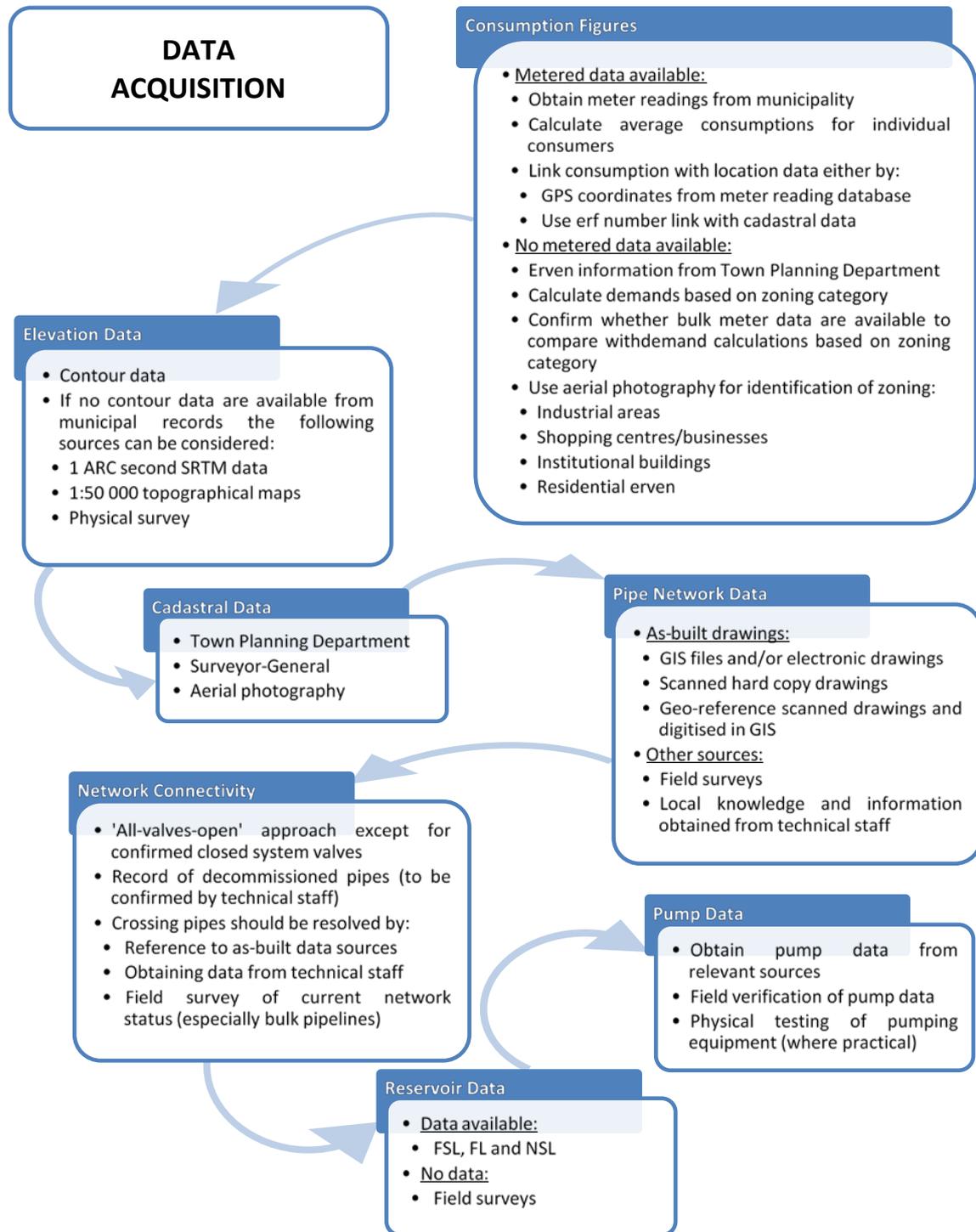


Figure 5.11: Summative flow diagram for WDN data acquisition

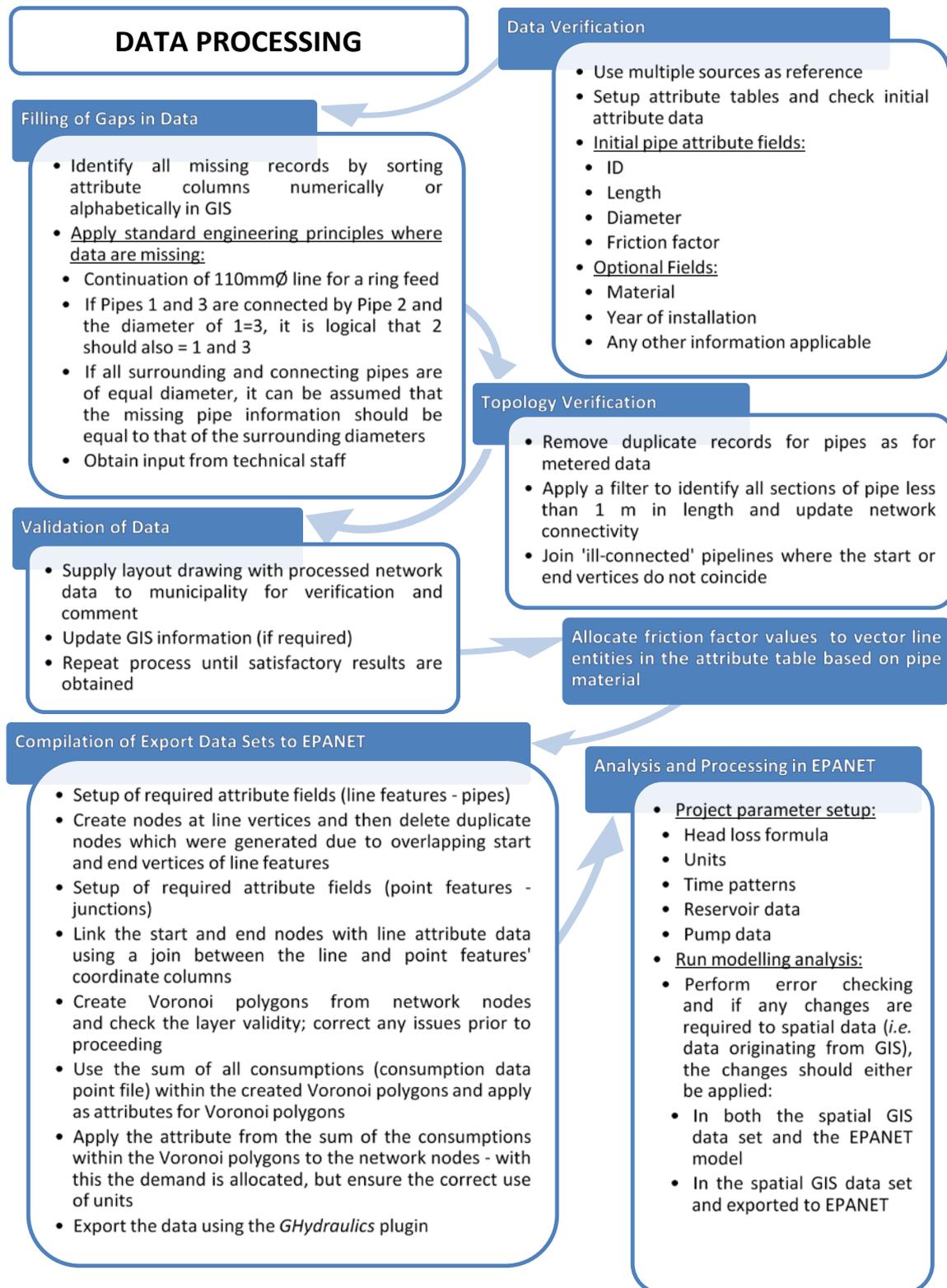


Figure 5.12: Summative flow diagram for WDN data processing

ATTRIBUTE FIELDS FOR EXPORTATION TO EPANET

Line Features (Pipes)

- 'DC_ID' which contains the pipe's unique ID reference
- 'LENGTH' based on the measured geographical length
- 'NODE1' indicating the start node of the line feature
- 'NODE2' indicating the end node of the line feature
- 'DIAMETER' specifying the diameter of the pipe in millimetres
- 'ROUGHNESS' according to the pipe material's friction factor and friction formula applied (millimetres for Darcy-Weisbach equation and unitless for the Hazen-Williams equation)
- 'MINORLOSS' showing the minor loss coefficient for bends, tees, etc. located on the line segment
- 'STATUS' which serves as a function to simulate shut-off valves which can either be set to 'Closed' or 'Open'
- 'START_X' containing the value of the x-coordinate of the first node of the line feature (generated by '\$xat(0)' python command)
- 'START_Y' containing the value of the y-coordinate of the first node of the line feature (generated by '\$yat(0)' python command)
- 'START_CRD' containing a combination of the 'START_X' and 'START_Y' fields
- 'END_X' containing the value of the x-coordinate of the first node of the line feature (generated by '\$xat(1)' python command)
- 'END_Y' containing the value of the y-coordinate of the first node of the line feature (generated by '\$yat(1)' python command)
- 'END_CRD' containing a combination of the 'START_X' and 'START_Y' fields

Point Features (Junctions)

- 'DC_ID' which contains the junction's unique ID reference
- 'NODE' indicating the node name, typically consisting of the prefix (*i.e.* 'J') and the unique ID reference (*i.e.* 'J1987')
- 'ELEVATION' specifying the elevation of the node
- 'X' containing the value of the x-coordinate of the point feature (generated by '\$x' python command)
- 'Y' containing the value of the y-coordinate of the point feature (generated by '\$y' python command)
- 'CRD' containing a combination of the 'X' and 'Y' fields

Figure 5.13: Attribute fields for exportation to EPANET

CHAPTER 6 : CONCLUSIONS AND RECOMMENDATIONS

This chapter contains a synthesised discussion of the research results presented in Chapter 5. The final conclusions and some recommendations for future research are included at the end of this chapter.

6.1 Study Objectives

The main purpose of this study was to develop a methodological approach based on the integration of open-source software, *e.g.*, QGIS and EPANET, and engineering practices associated with water distribution network design to enable the practitioner to create a WDN model of an existing network.

6.2 Specific Objectives

The specific objectives identified to achieve the overall objective of this study are discussed in the subsequent sections.

6.2.1 Acquisition of data for the compilation of a WDN model

The largest part of the data for the case study area (Swakopmund Municipality) was available from municipal records. The acquisition of data was discussed in detail in Chapter 4 and the preferred (primary) and secondary data sources are summarised in Table 6.1.

6.2.2 Processing of data for the compilation of a WDN model

The process and methodology followed to compile a WDN model were discussed in detail in Chapter 4. Consideration was given to the fact that the context and specific needs of water utilities may differ and that all the data sets might not be as readily available as in the case of this study. The different processing steps to be followed and applicable to different data sources were discussed as part of the literature review (Chapter 2) and detailed in the methodology section (Chapter 4).

Table 6.1: Summary of the preferred (primary) and secondary data sources for the compilation of a WDN

Data set	Primary source(s)	Secondary source(s)	Other
Consumption figures	Consumer billing records	Derived consumptions based on zoning and design norms Municipal bulk meter readings in conjunction with design norms	Aerial photography to determine the zoning category
Demand-time patterns	Recorded flow metering at strategic points in the network, <i>i.e.</i> , flow logging	Peak demand factors as per design norms	Combination of peak demand factors from design norms and engineering judgement
Elevation data	Detailed survey data Aerial survey data	SRTM Data (USGS)	1:50 000 Topographical maps
Cadastral data	Municipal records SG records	Aerial photography	
Pipe network data	Municipal records As-built data Local knowledge	Site survey data, <i>e.g.</i> , position of valves, hydrants, etc. Underground surveys	
Reservoir data	Detailed survey data Municipal records As-built data	Field measurements Aerial photography	
Pump data	Municipal records Site observations Data collection		

6.2.3 Identification of various data sources

Various data sources were identified for the respective data sets required for the compilation of the Swakopmund WDN model. This specific objective was largely combined with the acquisition of data; however, it was deemed necessary to make specific mention of alternative data sources. Innovative thinking and sound engineering judgement need to be demonstrated by any practitioner in cases where data sources are either incomplete or not suitable. For example, aerial photography can be used to determine a bulk pipeline’s route (in the absence of as-built information), since the historical aerial imagery may indicate the pipeline trench during construction. In addition, hand-held GPS devices can be used to pin

site features such as valves, hydrants, pipeline routes, *etc.*, after which, the data can be digitised and processed for further use.

The determination of demand-time patterns may prove one of the most intriguing challenges in the absence of electronically logged data or data from SCADA systems. A work-around can be established by manually recording bulk meter readings at a single or at multiple reservoirs in the network. If it is not practical to take recordings over a 24-hour period, peak consumption hours may be estimated and readings only be recorded for the concerned time periods. Sensitivity with regard to the context of the readings being taken should always be maintained, and questions should be asked in that regard. For example: 'Were the readings taken during a holiday period? Were there any power outages affecting pump stations while the readings were monitored? Were there any pipe bursts recorded during the period of record?' Ultimately, all these factors could have a significant influence on the derived data.

6.2.4 Utilisation of open-source software

In this study, the processing and analysis of all the data were accomplished by using open-source software, *e.g.*, QGIS and EPANET. In Chapter 4, methodological procedures were described to enable the effective utilisation of QGIS and EPANET. In using such a methodological approach based on open-source software applications, not only guidance is provided to the practitioner, but ultimately, this possibly could also enable and assist municipalities in Southern Africa facing challenges in terms of technical skills shortages, service delivery backlogs, outdated records and budget constraints. Students and practitioners can easily obtain the relevant software and familiarise themselves with the functionality and capabilities of the software without incurring excessive costs typically associated with the purchasing and/or annual license fees applicable to commercial software products.

6.2.5 Sensitivity analysis of head loss equations

Three different head (friction) loss equations are available for the hydraulic analysis of a WDN model in EPANET and were accordingly evaluated. In contrary

to situations where new WDNs are designed and the friction coefficients of pipes and their associated materials are known, existing networks have in most cases been subjected to various elements over time. Hence, besides the changes in physical characteristics of infrastructure with time, other unknowns may affect the variables used for the hydraulic analysis of a WDN model.

When designing a new WDN, optimal design assumptions, peak factors and different fire-flow scenarios should be considered. For the initial compilation of a WDN model, prior to any calibration, a conservative approach should also be followed. Hence, the former aspects also emphasise the need for the continuous calibration and updating of WDN models to ultimately ensure a realistic representation of the actual conditions. The following 'rules-of-thumb' as applied in this study, are recommended where uncertainty regarding variables exist to achieve a more conservative, *i.e.*, an increased head loss, approach during WDN modelling:

- (a) Use a higher or more conservative discharge (Q) value, *i.e.*, higher water demand value where uncertainty exists regarding demands (*cf.* Figure 5.1).
- (b) Use a lower diameter (D) value, *i.e.*, internal diameter where doubt *vis-à-vis* pipe size exists (*cf.* Figure 5.2).
- (c) If any inaccuracies are encountered in terms of pipe lengths (L) in a WDN, a longer length will result in an increased head loss value (*cf.* Figure 5.3).
- (d) In older WDNs, where the condition of pipes, as well as pipe materials are unknown, the Hazen-Williams equation [Eq. (2.2)] should be used with a C value ranging between 120 and 150, respectively.

It was evident that for the Darcy-Weisbach equation [Eq. (2.1)], a higher friction coefficient will result in an increased head loss. Despite the fact that Eq. (2.1) proved to be the less sensitive of the three equations evaluated, the small numbers (*e.g.*, 0.015) associated with the friction coefficient used in Eq. (2.1), subject the equation to be more likely prone to higher percentage errors. Hence, the preferential use of the Hazen-Williams equation [Eq. (2.2)] in practice.

6.2.6 Derivation of water demand and peak factor equations

Water demand [Eqs. (5.1) and (5.2)] and peak factor [Eq. (5.3)] equations were derived using linear and logarithmic regression analyses, respectively. All these equations could be included in data processing software (e.g., Microsoft Excel) or directly into attribute tables of GIS shapefiles to determine theoretical demands in the absence of metered consumption figures. However, these equations should only be applied to erven in developed areas.

6.3 Achievement of Objectives and Major Findings

Based on the preceding paragraphs and discussions in Sections 6.2.1 to 6.2.6, it is evident that the specific objectives were reached to achieve the overall study objective.

The use of open-source software (e.g., QGIS and EPANET) and the integration thereof with sound engineering practices and judgement, proved to be both viable and practical. The developed methodological approach and associated flow diagrams will not only provide guidance to the practitioner, but ultimately, it possibly could also enable and assist municipalities in Southern Africa facing challenges in terms of technical skills shortages, service delivery backlogs, outdated records and budget constraints. Hence, the motivation or need for such a methodology based on software that does not require annual licensing or additional capital expenditure.

6.4 Recommendations for Future Research

QGIS software containing improved features and processing capabilities is released on a frequent basis, e.g., QGIS Version 3.8.2 'Zanzibar' (August 2019) and QGIS Version 3.10 'A Coruña' (November 2019). Amongst the plugins that were developed for the latter versions, the *QEPANET* plugin exhibits similar functionalities as the *GHydraulics* plugin used in this study. Hence, it is recommended that the improved functionalities of the *QEPANET* plugin and further integration of analyses within the QGIS package, should be investigated and the required processes should be documented in a methodological manner.

Future studies should be directed towards the quantification of recommended man-hours and determination of associated consultation fees for the development of a WDN model based on various parameters such as: (i) WDN size and area, (ii) data availability, (iii) length of pipes in the network, (iv) number of nodes to be analysed, and (v) level of modelling detail required. The latter parameters would provide municipalities with guidelines to be used for budgeting purposes. Practitioners would also then have a baseline guide to determine the required resources and time to be allocated for the compilation of a WDN model.

In addition, the development of standard guidelines stipulating the minimum requirements (level of detail and methodology) of a completed WDN model for municipalities in Southern Africa may prove valuable to provide municipalities with a benchmark when the terms of reference need to be compiled for the appointment and evaluation of consultants. Such a set of standards may also provide common ground for different analysis software packages, *i.e.*, commercial and open-source software packages.

The Department of Human Settlements has published an updated version of the '*Guidelines for Human Settlement Planning and Design*' in August 2019, now titled '*The Neighbourhood Planning and Design Guide*'. Future research may be directed towards the comparisons between recorded consumption figures associated with various types of zoning and the values as recommended in the updated guidelines. In addition, comparisons between the zoning consumptions of coastal cities vs. coastal towns and inland cities vs. inland towns could be considered to establish whether there are noteworthy differences between the different locations and level of development.

The functionality of the water quality analysis component of EPANET was not considered during this study. Hence, the development of a methodology for water quality analysis using EPANET and QGIS and the subsequent calibrations thereof in the Southern African context should also be established.

6.5 Conclusions

The objective of this research was to develop a methodological framework as a practitioners' guideline to integrate open-source software and engineering practices when WDN models of an existing network need to be compiled. The achievement of this objective and the associated specific objectives, based on the research results, as well as the recommendations stemming from this have been discussed in this chapter. It is envisaged that the implementation of the methodology and the recommendations for future research will contribute to improved water infrastructure planning, design and management, especially in the Southern African context where water resources are scarce and service delivery remains a pressing issue.

CHAPTER 7 : REFERENCES

- Alves, Z, Muranho, J, Albuquerque, T and Ferreira, A. 2014. Water distribution network's modelling and calibration. A case study based on scarce inventory data. *Procedia Engineering* 70: 31–40.
- Babić, B, Đukić, A and Stanić, M. 2014. Managing water pressure for water savings in developing countries. *Water SA* 40 (2): 221–232.
- Berardi, L, Liu, S, Laucelli, D, Xu, S, Zeng, W and Giustolisi, O. 2014. Energy saving and leakage control in water distribution networks: a joint research project between Italy and China. *Procedia Engineering* 70:152–161.
- Blom, P, Bonthuys, G and Van Dijk, M. 2018. Water infrastructure asset management addressing the SDGs through energy recovery. *Civil Engineering* 26 (5): 10–14.
- Boano, F, Scibetta, M, Ridolfi, L and Giustolisi, O. 2015. Water distribution system modelling and optimisation: a case study. *Procedia Engineering* 119: 719–724.
- Burrough, PA. 1988. Principles of geographical information systems for land resources assessment. *Monographs on Soil and Resources Survey No 12*, Oxford Science Publications. Oxford, UK.
- Candelieri, A, Soldi, D and Archetti, F. 2015. Short-term forecasting of hourly water consumption by using automatic metering readers data. *Procedia Engineering* 119: 844–853.
- Cheng, WP, Yu, TC and Xu, G. 2014. Real-time model of a large-scale water distribution system. *Procedia Engineering* 89: 457–466.
- Christelis, G and Struckmeier, W. 2011. *Groundwater in Namibia: An Explanation to the Hydrogeological Map*. Ministry of Agriculture, Water and Rural Development. Windhoek, Namibia.
- Constitution of the Republic of South Africa. 1996. *Bill of Rights*. Government Gazette No. 17678.
- Cooper, AK. 1993. *The South African Standard for the Exchange of Digital Georeferenced Information*. Unpublished MSc dissertation. University of Pretoria, Pretoria, RSA.

- Couvelis, FA and Van Zyl, JE. 2012. *Apparent Water Losses related to Municipal Metering in South Africa*. WRC Report No. 1998/1/12. Water Research Commission, Pretoria, RSA.
- Cross, H. 1936. Analysis of flow in networks of conduits or conductors. *University of Illinois Bulletin* 34 (22): 1–34.
- CSIR Boutek. 2005. *Guidelines for Human Settlement Planning and Design*. Report No. BOU/E2001. CSIR Building and Construction Technology, Pretoria, RSA.
- Dale, PF and McLaughlin, JD. 1988. *Land Information Management*. Clarendon Press. Oxford, UK.
- Department of Human Settlements. 2019. *The Neighbourhood Planning and Design Guide*. The South African Government. Pretoria, RSA.
- Ebacher, G, Besner, MC, Lavoie, J, Jung, BS, Karney, BW and Prévost, M. 2011. Transient modelling of a full-scale distribution system: comparison with field data. *Journal of Water Resources Planning and Management* 2011: 173–182.
- Ezz, H. 2017. Integrating GIS and HEC-RAS to model Assiut plateau runoff. *The Egyptian Journal of Remote Sensing and Space Science* 21 (3): 219–227. DOI: [10.1016/j.ejrs.2017.11.002](https://doi.org/10.1016/j.ejrs.2017.11.002).
- Gama, MC, Lanfranchi, EA, Pan, Q and Jonoski, A. 2015. Water distribution network model building, case study: Milano, Italy. *Procedia Engineering* 119: 573–582.
- Georgescu, SC, Georgescu, AM, Madularea, RA, Piraianu, VF, Anton, A and Dunca, G. 2016. Numerical model of a medium-sized municipal water distribution system located in Romania. *Procedia Engineering* 119: 660–668.
- Gomes, HP, Bezerra, SDTM, De Carvalho, PSO and Salvino, MM. 2009. Optimal dimensioning model of water distribution systems. *Water SA* 35 (4): 421–431.
- Government of the Republic of Namibia. 2016. *Harambee Prosperity Plan 2016/17-2019/20*. GRN Printers, Windhoek, Namibia.
- Heywood, I, Cornelius, S and Carver, S. 1998. *Geographical Information Systems*. Addison Wesley Longman Limited, New York, USA.

- Iglesias-Rey, PL, Martinez-Solano, FJ and Rebelles-Aquilar, JV. 2017. Extending EPANET capabilities with Add-In Tools. *Procedia Engineering* 186: 626–634.
- Jacobs, HE, Geustyn, LC, Loubser, BF and Van der Merwe, B. 2004. Estimating residential water demand in southern Africa. *Journal of the South African Institution of Civil Engineering* 46 (4): 2-13.
- Jamer, M. 2015. Infrastructure asset management: Can the Canadian municipal experience help inform better practices in Southeast Asia? *The Governance Brief* (21): 1–7.
- Kara, S, Karadirek, IE, Muhammetoglu, A and Muhammetoglu, H. 2016. Hydraulic modelling of a water distribution network in a tourism area with highly varying characteristics. *Procedia Engineering* 162: 521–529.
- Kok, A and Harris, MS. 1992. *Re-evaluation of Water Demand Norms for Planning Purposes*. DWA Report No. 13/1/B. Department of Water Affairs, Republic of Namibia Planning Division. Windhoek, Namibia.
- Krocová, S. 2016. Water supply systems and their influence on increasing operational safety in industry. *Perspectives in Science* 7: 236–239.
- Lancashire, SJ. 1985. In-Service Durability of uPVC Water Mains. In: *Plastics Pipes VI Conference, (March 1985)*. York, England.
- Machell, J, Mounce, SR and Boxall, JB. 2009. Online modelling of water distribution systems: a UK case study. *Drinking Water Engineering and Science* 3 (21): 21–27.
- Macke, S. 2014. *EPANET Hydraulic Network Analysis*. [Internet]. Available from: <http://epanet.de/ghydraulics/index.html> [Accessed 20 June 2018].
- Mays, LW. 2000. *Water Distribution Systems Handbook*. McGraw-Hill, New York, USA.
- McEnroe, BM, Chase, DV and Sharp, WW. 1989. *Field Testing Water Mains to Determine Carrying Capacity*. Miscellaneous Paper EL-89. US Army Engineer Waterways Experiment Station, Vicksburg, USA.
- McKenzie, RS, Buckle, H, Wegelin, WA and Meyer, N. 2003. *Water Demand Management Cookbook*. United Nations Human Settlements Programme, Water Resource Planning and Conservation. Rand Water, RSA.

- McKenzie, RS. 2014. *Guidelines for Reducing Water Losses in South African Municipalities*. WRC Report No. TT 595/14. Water Research Commission, Pretoria, RSA.
- Mendelsohn, J, Jarvis, A, Roberts, C and Robertson, T. 2003. *Atlas of Namibia*. David Philip Publishers, Cape Town, RSA.
- Neelakantan, TR, Suribabu, CR and Lingireddy, S. 2008. Optimisation procedure for pipe-sizing with break-repair and replacement economics. *Water SA* 34 (2): 217–224.
- Ormsbee, LE. 2006. The history of water distribution network analysis: the computer age. In: *Proceedings of WDSA 2006: Water Distribution System Analysis Symposium, 27–30 August 2006*. University of Cincinnati, Cincinnati, USA.
- Panguluri, S, Grayman, WM and Clark, RM. 2005. *Water Distribution System Analysis: Field Studies, Modelling, and Management, a Reference Guide for Utilities*. EPA Report No. EPA/600/R-06/028. United States Environmental Protection Agency, Cincinnati, USA.
- Paulsson, B. 1992. *Urban Applications of Satellite Remote Sensing and GIS Analysis*. Report No. 11364. The World Bank, Washington, USA.
- Pickering, D, Park, JM and Bannister, DH. 1993. *Utility Mapping and Record Keeping for Infrastructure*. The International Bank for Reconstruction and Development. Washington D.C., USA.
- QGIS Development Team. 2016. *QGIS User Guide*. [Online] Open Source Geospatial Foundation Available from: <http://qgis.osgeo.org> [Accessed 26 July 2017].
- Ramana, GV, Sudheer, VSS and Rajasekhar, B. 2015. Network analysis of water distribution system in rural areas using EPANET. *Procedia Engineering* 119: 496–505.
- Ramesh, H, Santhosh, L and Jagadeesh, CJ. 2012. Simulation of hydraulic parameters in a water distribution network using EPANET and GIS. In: *Proceedings of the International Conference on Ecological, Environmental and Biological Sciences (ICEEBS'2012)*. Dubai.

- Roma, J, Pérez, R, Sanz, G and Grau, S. 2015. Model calibration and leakage assessment applied to a real water distribution network. *Procedia Engineering* 119: 603–612.
- Rossman, LA. 2000. *EPANET 2 User's Manual*. United States Environmental Protection Agency. Cincinnati, Ohio, USA.
- Salmivirta, T. 2015. *Water Distribution Development and Modelling in Keetmanshoop, Namibia*. Unpublished MSc. dissertation. Tampere University of Technology, Tampere.
- Saminu, A, Abubakar, N and Sagir, L. 2013. Design of NDA water distribution network using EPANET. *International Journal of Emerging Science and Engineering* 1 (9): 5–9.
- Sanz, G and Pérez, R. 2015. Comparison of demand calibration in water distribution networks using pressure and flow sensors. *Procedia Engineering* 119: 771–780.
- Savic, DA, Kapelan, ZS and Jonkergouw, PMR. 2009. *Quo Vadis* water distribution model calibration. *Urban Water Journal* 6 (1):3–22.
- Shital, K, Krunali, M, Mehta, D and Yadav, V. 2016. Simulation of existing water distribution network by using EPANET: a case study of Surat City. *Global Research and Development Journal for Engineering* 2016:184–192.
- Stenstedt, L. 2019. The asbestos beneath our streets. *WaterWorld* 35 (2):28-30.
- Sun, C, Morley, M, Savic, D, Puig, V, Cembrano, G and Zhang, Z. 2015. Combining model predictive control with constraint-satisfaction formulation for the operative pumping control in water networks. *Procedia Engineering* 119 (2015): 963–972.
- Tjiramba, S. 2009. *The Namibian*. [Internet]. Available from: <https://www.namibian.com.na/index.php?id=56408&page=archive-read> [Accessed 28 Augustus 2019].
- Trifunovic, N. 2015. *Introduction to Urban Water Distribution*. Taylor and Francis/Balkema. Leiden, Netherlands.
- Tshwane Water and Sanitation Division. 2010. *Guidelines for the Design and Construction of Water and Sanitation Systems*. City of Tshwane, Pretoria, RSA.

- United States Geological Survey. 2015. *SRTM Mission Summary*. [Internet]. Available from: https://lta.cr.usgs.gov/srtm/mission_summary [Accessed 16 July 2018].
- Van Riet, WF and Vosloo, PT. 1990. *The Role of Geographic Information Systems and Integrated Environmental Management in the Landscape Planning of Catchments*. WRC Report No. KV24/90. Water Research Commission, Pretoria, RSA.
- Van Zyl, HJ, Van Zyl, JE, Geustyn, L, Ilemobade, A and Buckle, JS. 2007. *Water Consumption Levels in selected South African Cities*. WRC Report No. 1536/1/06. Water Research Commission, Pretoria, RSA.
- Walski, TM. 2006. Water distribution system analysis before digital computers. In: *Proceedings of the 8th Annual Water Distribution Systems Analysis Symposium*. Cincinnati, USA.
- Walski, TM, Chase, DV, Savic, DA, Grayman, W, Beckwith, S and Koelle, E. 2003. *Advanced Water Distribution Modelling and Management*. 1st Ed. Haested Press. Waterbury, CT, USA.

APPENDIX A: ETHICAL APPROVAL

Letter from Swakopmund Municipality for use of municipal water data

	<h1>MUNICIPALITY OF SWAKOPMUND</h1>	<p> (064) 4104400  (064) 4104125  Fax2email: 0886519137  53 Swakopmund NAMIBIA  www.swkmun.com.na  townengineer@swkmun.com.na</p>
<p>Ref No: B 1/4/8/1; N 10/1</p>		
<p>Enquiries: AD Duvenhage</p>		
<p>Mr A.L. Muller P O Box 91263 KLEIN WINDHOEK Namibia</p>		<p>08 February 2017  muller@element.com.na</p>
<p>Dear Sir,</p>		
	<p>MASTER STUDIES IN ENGINEERING – USE OF MUNICIPAL WATER DATA</p>	
	<p>Permission is herewith granted for you to use the water and network related data which was made available to your employer for the compilation of a network model as per specifications of Quotation 12/2017/ES-HO.</p>	
	<p>Please, note that:</p> <ul style="list-style-type: none">• You may not disseminate this data to any third party but the academic institution where you enrolled for the master course in engineering studies.• The data may not be used for commercial purposes.• All data of specific users and their identities shall be maintained confidential, with all references to possible specific cases presented such to protect the identity and confidentiality of the water user.	
	<p>If you or the university see the need to disseminate the data to any third party, or use the data commercially, you need to obtain permission in writing from the Chief Executive Officer of the Municipality of Swakopmund prior to doing so.</p>	
	<p>You are further expected to provide the municipality with a copy of your master theses once it has been evaluated and approved.</p>	
	<p>Yours faithfully,</p> 	
	<p>AD Duvenhage GENERAL MANAGER: ENGINEERING SERVICES</p>	
	<p>ADD/vrb</p>	
	<p>All correspondence must be addressed to Chief Executive Officer</p>	

APPENDIX B: TABULATED INFORMATION AND RESULTS

Table B.1: Darcy-Weisbach equation variation in head loss results supplementary to Figures 5.1 to 5.4

D-W ΔQ							D-W ΔD							D-W ΔL							D-W Δf						
Hf	Q	D	L	f	ΔQ%	ΔHf%	Hf	Q	D	L	f	ΔD%	ΔHf%	Hf	Q	D	L	f	ΔL%	ΔHf%	Hf	Q	D	L	f	Δf%	ΔHf%
0.000	0.01	1	1	1	-99%	-999900%	826000000.000	1	0.01	1	1	-99%	100%	0.001	1	1	0.01	1	-99%	-9900%	0.001	1	1	1	0.01	-99%	-9900%
0.001	0.10	1	1	1	-90%	-9900%	8260.000	1	0.10	1	1	-90%	100%	0.008	1	1	0.10	1	-90%	-900%	0.008	1	1	1	0.10	-90%	-900%
0.003	0.20	1	1	1	-80%	-2400%	258.125	1	0.20	1	1	-80%	100%	0.017	1	1	0.20	1	-80%	-400%	0.017	1	1	1	0.20	-80%	-400%
0.007	0.30	1	1	1	-70%	-1011%	33.992	1	0.30	1	1	-70%	100%	0.025	1	1	0.30	1	-70%	-233%	0.025	1	1	1	0.30	-70%	-233%
0.013	0.40	1	1	1	-60%	-525%	8.066	1	0.40	1	1	-60%	99%	0.033	1	1	0.40	1	-60%	-150%	0.033	1	1	1	0.40	-60%	-150%
0.021	0.50	1	1	1	-50%	-300%	2.643	1	0.50	1	1	-50%	97%	0.041	1	1	0.50	1	-50%	-100%	0.041	1	1	1	0.50	-50%	-100%
0.030	0.60	1	1	1	-40%	-178%	1.062	1	0.60	1	1	-40%	92%	0.050	1	1	0.60	1	-40%	-67%	0.050	1	1	1	0.60	-40%	-67%
0.040	0.70	1	1	1	-30%	-104%	0.491	1	0.70	1	1	-30%	83%	0.058	1	1	0.70	1	-30%	-43%	0.058	1	1	1	0.70	-30%	-43%
0.053	0.80	1	1	1	-20%	-56%	0.252	1	0.80	1	1	-20%	67%	0.066	1	1	0.80	1	-20%	-25%	0.066	1	1	1	0.80	-20%	-25%
0.067	0.90	1	1	1	-10%	-23%	0.140	1	0.90	1	1	-10%	41%	0.074	1	1	0.90	1	-10%	-11%	0.074	1	1	1	0.90	-10%	-11%
0.083	1.00	1	1	1	0%	0%	0.083	1	1.00	1	1	0%	0%	0.083	1	1	1.00	1	0%	0%	0.083	1	1	1	1.00	0%	0%
0.100	1.10	1	1	1	10%	17%	0.051	1	1.10	1	1	10%	-61%	0.091	1	1	1.10	1	10%	9%	0.091	1	1	1	1.10	10%	9%
0.119	1.20	1	1	1	20%	31%	0.033	1	1.20	1	1	20%	-149%	0.099	1	1	1.20	1	20%	17%	0.099	1	1	1	1.20	20%	17%
0.140	1.30	1	1	1	30%	41%	0.022	1	1.30	1	1	30%	-271%	0.107	1	1	1.30	1	30%	23%	0.107	1	1	1	1.30	30%	23%
0.162	1.40	1	1	1	40%	49%	0.015	1	1.40	1	1	40%	-438%	0.116	1	1	1.40	1	40%	29%	0.116	1	1	1	1.40	40%	29%
0.186	1.50	1	1	1	50%	56%	0.011	1	1.50	1	1	50%	-659%	0.124	1	1	1.50	1	50%	33%	0.124	1	1	1	1.50	50%	33%
0.211	1.60	1	1	1	60%	61%	0.008	1	1.60	1	1	60%	-949%	0.132	1	1	1.60	1	60%	38%	0.132	1	1	1	1.60	60%	38%
0.239	1.70	1	1	1	70%	65%	0.006	1	1.70	1	1	70%	-1320%	0.140	1	1	1.70	1	70%	41%	0.140	1	1	1	1.70	70%	41%
0.268	1.80	1	1	1	80%	69%	0.004	1	1.80	1	1	80%	-1790%	0.149	1	1	1.80	1	80%	44%	0.149	1	1	1	1.80	80%	44%
0.298	1.90	1	1	1	90%	72%	0.003	1	1.90	1	1	90%	-2376%	0.157	1	1	1.90	1	90%	47%	0.157	1	1	1	1.90	90%	47%
0.327	1.99	1	1	1	99%	75%	0.003	1	1.99	1	1	99%	-3021%	0.164	1	1	1.99	1	99%	50%	0.164	1	1	1	1.99	99%	50%

Table B.2: Hazen-Williams equation variation in head loss results supplementary to Figures 5.1 to 5.4

H-W ΔQ							H-W ΔD							H-W ΔL							H-W ΔC						
Hf	Q	D	L	C	ΔQ%	ΔHF%	Hf	Q	D	L	C	ΔD%	ΔHF%	Hf	Q	D	L	C	ΔL%	ΔHF%	Hf	Q	D	L	C	ΔC%	ΔHF%
0.002	0.01	1	1	1	-99%	-505725%	58744122308.327	1	0.01	1	1	-99%	100%	0.107	1	1	0.01	1	-99%	-9900%	53971.491	1	1	1	0.01	-99%	100%
0.150	0.10	1	1	1	-90%	-7012%	791706.881	1	0.10	1	1	-90%	100%	1.067	1	1	0.10	1	-90%	-900%	758.865	1	1	1	0.10	-90%	99%
0.542	0.20	1	1	1	-80%	-1870%	27066.240	1	0.20	1	1	-80%	100%	2.134	1	1	0.20	1	-80%	-400%	210.212	1	1	1	0.20	-80%	95%
1.148	0.30	1	1	1	-70%	-830%	3756.584	1	0.30	1	1	-70%	100%	3.201	1	1	0.30	1	-70%	-233%	99.206	1	1	1	0.30	-70%	89%
1.955	0.40	1	1	1	-60%	-446%	925.319	1	0.40	1	1	-60%	99%	4.268	1	1	0.40	1	-60%	-150%	58.230	1	1	1	0.40	-60%	82%
2.956	0.50	1	1	1	-50%	-261%	312.105	1	0.50	1	1	-50%	97%	5.335	1	1	0.50	1	-50%	-100%	38.519	1	1	1	0.50	-50%	72%
4.143	0.60	1	1	1	-40%	-158%	128.427	1	0.60	1	1	-40%	92%	6.402	1	1	0.60	1	-40%	-67%	27.481	1	1	1	0.60	-40%	61%
5.512	0.70	1	1	1	-30%	-94%	60.618	1	0.70	1	1	-30%	82%	7.469	1	1	0.70	1	-30%	-43%	20.656	1	1	1	0.70	-30%	48%
7.058	0.80	1	1	1	-20%	-51%	31.634	1	0.80	1	1	-20%	66%	8.536	1	1	0.80	1	-20%	-25%	16.130	1	1	1	0.80	-20%	34%
8.779	0.90	1	1	1	-10%	-22%	17.825	1	0.90	1	1	-10%	40%	9.603	1	1	0.90	1	-10%	-11%	12.969	1	1	1	0.90	-10%	18%
10.670	1.00	1	1	1	0%	0%	10.670	1	1.00	1	1	0%	0%	10.670	1	1	1.00	1	0%	0%	10.670	1	1	1	1.00	0%	0%
12.730	1.10	1	1	1	10%	16%	6.708	1	1.10	1	1	10%	-59%	11.737	1	1	1.10	1	10%	9%	8.943	1	1	1	1.10	10%	-19%
14.956	1.20	1	1	1	20%	29%	4.391	1	1.20	1	1	20%	-143%	12.804	1	1	1.20	1	20%	17%	7.612	1	1	1	1.20	20%	-40%
17.346	1.30	1	1	1	30%	38%	2.973	1	1.30	1	1	30%	-259%	13.871	1	1	1.30	1	30%	23%	6.564	1	1	1	1.30	30%	-63%
19.897	1.40	1	1	1	40%	46%	2.072	1	1.40	1	1	40%	-415%	14.938	1	1	1.40	1	40%	29%	5.722	1	1	1	1.40	40%	-86%
22.609	1.50	1	1	1	50%	53%	1.481	1	1.50	1	1	50%	-621%	16.005	1	1	1.50	1	50%	33%	5.036	1	1	1	1.50	50%	-112%
25.480	1.60	1	1	1	60%	58%	1.081	1	1.60	1	1	60%	-887%	17.072	1	1	1.60	1	60%	38%	4.468	1	1	1	1.60	60%	-139%
28.507	1.70	1	1	1	70%	63%	0.805	1	1.70	1	1	70%	-1225%	18.139	1	1	1.70	1	70%	41%	3.994	1	1	1	1.70	70%	-167%
31.691	1.80	1	1	1	80%	66%	0.609	1	1.80	1	1	80%	-1651%	19.206	1	1	1.80	1	80%	44%	3.593	1	1	1	1.80	80%	-197%
35.028	1.90	1	1	1	90%	70%	0.468	1	1.90	1	1	90%	-2178%	20.273	1	1	1.90	1	90%	47%	3.250	1	1	1	1.90	90%	-228%
38.163	1.99	1	1	1	99%	72%	0.374	1	1.99	1	1	99%	-2755%	21.233	1	1	1.99	1	99%	50%	2.983	1	1	1	1.99	99%	-258%

Table B.3: Manning's equation variation in head loss results supplementary to Figures 5.1 to 5.4

Manning ΔQ							Manning ΔD						Manning ΔL						Manning Δn								
Hf	Q	D	L	n	$\Delta Q\%$	$\Delta Hf\%$	Hf	Q	D	L	n	$\Delta D\%$	$\Delta Hf\%$	Hf	Q	D	L	n	$\Delta L\%$	$\Delta Hf\%$	Hf	Q	D	L	n	$\Delta n\%$	$\Delta Hf\%$
0.000	0.01	1	1	1	-99%	-999900%	213003096360.531	1	0.01	1	1	-99%	100%	0.103	1	1	0.01	1	-99%	-9900%	0.001	1	1	1	0.01	-99%	-999900%
0.047	0.10	1	1	1	-90%	-9900%	996290.334	1	0.10	1	1	-90%	100%	1.033	1	1	0.10	1	-90%	-900%	0.103	1	1	1	0.10	-90%	-9900%
0.186	0.20	1	1	1	-80%	-2400%	24768.291	1	0.20	1	1	-80%	100%	2.066	1	1	0.20	1	-80%	-400%	0.413	1	1	1	0.20	-80%	-2400%
0.419	0.30	1	1	1	-70%	-1011%	2853.183	1	0.30	1	1	-70%	100%	3.099	1	1	0.30	1	-70%	-233%	0.930	1	1	1	0.30	-70%	-1011%
0.746	0.40	1	1	1	-60%	-525%	615.752	1	0.40	1	1	-60%	99%	4.132	1	1	0.40	1	-60%	-150%	1.653	1	1	1	0.40	-60%	-525%
1.165	0.50	1	1	1	-50%	-300%	187.446	1	0.50	1	1	-50%	98%	5.165	1	1	0.50	1	-50%	-100%	2.583	1	1	1	0.50	-50%	-300%
1.678	0.60	1	1	1	-40%	-178%	70.932	1	0.60	1	1	-40%	93%	6.198	1	1	0.60	1	-40%	-67%	3.719	1	1	1	0.60	-40%	-178%
2.283	0.70	1	1	1	-30%	-104%	31.190	1	0.70	1	1	-30%	85%	7.231	1	1	0.70	1	-30%	-43%	5.062	1	1	1	0.70	-30%	-104%
2.982	0.80	1	1	1	-20%	-56%	15.308	1	0.80	1	1	-20%	70%	8.264	1	1	0.80	1	-20%	-25%	6.611	1	1	1	0.80	-20%	-56%
3.775	0.90	1	1	1	-10%	-23%	8.171	1	0.90	1	1	-10%	43%	9.297	1	1	0.90	1	-10%	-11%	8.367	1	1	1	0.90	-10%	-23%
4.660	1.00	1	1	1	0%	0%	4.660	1	1.00	1	1	0%	0%	10.330	1	1	1.00	1	0%	55%	10.330	1	1	1	1.00	0%	55%
5.639	1.10	1	1	1	10%	17%	2.804	1	1.10	1	1	10%	-66%	11.363	1	1	1.10	1	10%	9%	12.499	1	1	1	1.10	10%	17%
6.710	1.20	1	1	1	20%	31%	1.763	1	1.20	1	1	20%	-164%	12.396	1	1	1.20	1	20%	17%	14.875	1	1	1	1.20	20%	31%
7.875	1.30	1	1	1	30%	41%	1.151	1	1.30	1	1	30%	-305%	13.429	1	1	1.30	1	30%	23%	17.458	1	1	1	1.30	30%	41%
9.134	1.40	1	1	1	40%	49%	0.775	1	1.40	1	1	40%	-501%	14.462	1	1	1.40	1	40%	29%	20.247	1	1	1	1.40	40%	49%
10.485	1.50	1	1	1	50%	56%	0.537	1	1.50	1	1	50%	-768%	15.495	1	1	1.50	1	50%	33%	23.243	1	1	1	1.50	50%	56%
11.930	1.60	1	1	1	60%	61%	0.381	1	1.60	1	1	60%	-1125%	16.528	1	1	1.60	1	60%	38%	26.445	1	1	1	1.60	60%	61%
13.467	1.70	1	1	1	70%	65%	0.275	1	1.70	1	1	70%	-1592%	17.561	1	1	1.70	1	70%	41%	29.854	1	1	1	1.70	70%	65%
15.098	1.80	1	1	1	80%	69%	0.203	1	1.80	1	1	80%	-2194%	18.594	1	1	1.80	1	80%	44%	33.469	1	1	1	1.80	80%	69%
16.823	1.90	1	1	1	90%	72%	0.152	1	1.90	1	1	90%	-2960%	19.627	1	1	1.90	1	90%	47%	37.291	1	1	1	1.90	90%	72%
18.454	1.99	1	1	1	99%	75%	0.119	1	1.99	1	1	99%	-3816%	20.557	1	1	1.99	1	99%	50%	40.908	1	1	1	1.99	99%	75%