TAMPEREEN TEKNILLINEN YLIOPISTO
TAMPERE UNIVERSITY OF TECHNOLOGY

TATU SALMIVIRTA

WATER DISTRIBUTION DEVELOPMENT AND MODELING IN KEETMANSHOOP, NAMIBIA

Master of Science Thesis

Examiners: Prof. Jukka Rintala, Development Manager Saila Kallioinen
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ABSTRACT

TATU SALMIVIRTA: Water Distribution Development and Modeling in Keetmanshoop, Namibia
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The objective of this study is to update earlier studies of Keetmanshoop's water distribution and to evaluate the performance of Keetmanshoop's water distribution network and water sector. This includes data such as the level of non-revenue water, pressures in the pipelines and network's appurtenances. The aim is also to model Keetmanshoop's water distribution network, and to simulate its performance for different outputs, such as maximum and minimum pressure and net flow. The model would then be used and updated by Keetmanshoop Municipality. Finally, the last objective is to present improvements to Keetmanshoop's water distribution.

The materials and methods used in this study were a literature survey, primarily of the previous studies, semi-structured interviews, observations and conversations, field measurements and Keetmanshoop Municipality's accounting. The research was mainly conducted in Keetmanshoop in June and July 2015. Moreover, the water distribution network model was constructed with WaterCAD, QGIS and FCGnet software.

The minimum pressure of 200 kPa in the whole network isn't achieved, thus the southern part of the network suffers from excessive pressure. The frequency of pipe breakages is high and indicates the network to be in a poor condition. Pipe breakages mainly occur from the old age of the pipe network, high pressure and lack of air release valves. The level of non-revenue water is 18.0 %, which is below the 20 %, the main target of this EU funded project. However, it should be higher due to the factors such as the overflowing of the municipality's reservoirs and high frequency of pipe breakages. These factors also indicate that the municipality's accounting of billed water is faulty.

The water distribution network model doesn't reliably replicate the actual network. There are many reasons for this, such as the complexity of water demand in the actual network and faulty map elements that the model was based on. The model is still approximate and indicates the performance of the actual network and some problematic areas, e.g. the southern part of the town where the pressure is excessive.

Keetmanshoop's water distribution can be improved in many ways. For instance, the 350 inactive water meters should be replaced and new gate valves, pressure reducing valves and air release valves are needed. The old asbestos cement pipes should be replaced with new plastic (uPVC) pipes as soon as possible. The municipality's reservoirs need to be protected, isolated and improved, and the use of Oxpass Hill reservoir is questionable. It dampens the pressure in the southwestern part of the network, overflows frequently and doesn't provide notable extra storage.

TIIVISTELMÄ

TATU SALMIVIRTA: Vedenjakelun kehittäminen ja mallintaminen Keetmanshoopissa, Namibiassa
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Avainsanat: vesi, jakelu, verkosto, malli, Namibia, Keetmanshoop

Tämän tutkimuksen tavoitteena on päivittää aikaisempia tutkimuksia Keetmanshoopin vedenjakelusta ja arvioida Keetmanshoopin vesijohtoverkoston ja vesisektorin toimintakykyä. Tämä sisältää tietoa esimerkiksi laskuttamattoman veden määrästä, putkijohtojen paineista ja vesijohtoverkoston varusteista. Tavoite on myös mallintaa Keetmanshoopin vesijohtoverkosto ja simuloida sen toimintaa erilaisille datatulosteille, kuten maksimi- ja minimipaineelle sekä nettovirtaamalle. Keetmanshoopin kunta voi siten jatkossa käyttää ja päivittää kyseistä mallia. Lopuksi, viimeinen tavoite on esittää parannuksia Keetmanshoopin vedenjakeluun.

Tutkimuksessa käytetyt materiaalit ja menetelmät olivat kirjallisuusselvitys, etupäässä aikaisemmista tutkimuksista, haastattelut, havainnot ja keskustelut, kenttämittaukset sekä Keetmanshoopin kunnan kirjanpito. Tutkimus toteutettiin pääasiassa Keetmanshoopissa kesä- ja heinäkuussa 2015. Lisäksi vesijohtoverkosto mallinnettiin WaterCAD, QGIS ja FCGnet -ohjelmistoilla.

200 kPa minimipainetta koko verkostossa ei saavuteta, ja lisäksi eteläinen osa verkostosta kärsii liiallisesta paineesta. Putkirikkojen toistumistiheys on korkea, mikä viittaa verkoston heikkoon kuntoon. Putkirikot pääosin johtuvat putkiverkoston vanhasta iästä, korkeasta paineesta sekä ilmanpoistoventtiilien puutteesta. Laskuttamattoman veden määrä on 18,0 %, mikä on tämän EU-rahoitteisen projektin päätavoitteen, 20 %:n, alapuolella. Se pitäisi olla kuitenkin suurempi johtuen esimerkiksi kunnan vesisäiliöiden ylivuodoista sekä putkirikkojen esiintymistiheydestä. Tämä myös viittaa kunnan kirjanpidon puutteellisuuteen.

Vesijohtoverkostomalli ei luotettavasti jäljennä todellista verkostoa. Tähän on monia syitä, kuten oikean verkoston monimutkainen vedentarve ja vialliset karttaelementit, joihin malli perustui. Malli on silti suuntaa antava ja antaa viitteitä todellisen verkoston toiminnasta ja ongelma-alueista, kuten eteläisen kaupungin alueen liiallisesta paineesta.

Keetmanshoopin vedenjakelua voidaan kehittää monin tavoin. Esimerkiksi toimimattomat 350 vesimittaria tulisi vaihtaa, ja uusia luisti-, paineenalennus- ja ilmanpoistoventtiileitä tarvitaan. Vanhat asbestisementtiputket tulisi vaihtaa uusiin muoviputkiin (uPVC). Kunnan vesisäiliöt tulisi turvata, eristää ja kohentaa, ja lisäksi Oxpass Hill -vesisäiliön käyttö on kyseenalaista. Se liiaksi vaimentaa painetta lounaisella kaupungin alueella, ylivuotaa usein eikä tarjoa huomattavaa lisävarastoa.

PREFACE AND ACKNOWLEDGEMENTS

This work is part of an EU funded project called Development of water services in Keetmanshoop, Namibia, with Kangasala Municipality being the coordinator of this project. The target of this project is to reduce the level of non-revenue water in Keetmanshoop to lower than 20 %; this hasn't been achieved yet.

D.Sc. Pekka Pietilä, the supervisor of my work, introduced me to this subject. I would like to thank him for this adventurous opportunity and for his help during this project. In addition, I would like to thank Prof. Jukka Rintala and Development Manager Saila Kallioinen for being the examiners of this thesis.

Arto Hietanen, a water engineer from Kangasala Water and my overseer during the field work, and Hosea Hashili, a student from Polytechnic of Namibia and a workmate of mine, enlightened me about the culture and lifestyle in Namibia and helped my stay in Keetmanshoop in terms of research and settling in.

Special thanks go to Markus Sunela and Kalervo Aho from Finnish Consulting Group for their assistance in modeling. Numerous personnel in Keetmanshoop Municipality were really helpful and friendly to me. To them I want to say: thank you. Moreover, I would like to express my gratitude to Calla Bonthuys from NamWater, Prof. Damas Mashauri from Polytechnic of Namibia, Arto Löppönen from Lempäälä Waterworks and Sami Virkajärvi, Kimmo Suonperä and Antti Kytövaara from Kangasala Water.

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Tampere December 19, 2015

Tatu Salmivirta

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ABBREVIATIONS AND NOTATIONS

AC	Asbestos cement
CAD	Computer-aided design
EPS	Extended-period simulation
FCG	Finnish Consulting Group Oy
GIS	Geographic information system
NamWater	The Namibia Water Corporation Limited
NRW	Non-revenue water
PRV	Pressure reducing valve
PVC	Polyvinyl chloride
QGIS	Quantum geographic information system
SCADA	Supervisory control and data acquisition
uPVC	Unplasticized polyvinyl chloride
WDN	Water distribution network

1 INTRODUCTION

Water is a basic human need. In municipalities, water is commonly supplied through distribution systems. (Al-Zahrani 2013, Walski et al. 2003) In Keetmanshoop, Namibia, there have been difficulties in the municipality's water supply from water losses and excessive pressure in the pipelines to the water sector's know-how. The water supply in Keetmanshoop has been previously studied by Seppänen (2009), Löppönen (2011), Aalto (2014), Tuovinen (2014) and Hambabi (2015).

The objective of this study is to update important data of the earlier studies of Keetmanshoop's water distribution, and to evaluate the performance of the water distribution network (WDN) and the water sector. This includes objectives such as the water sector's organization and work habits, network's appurtenances, water pressures in the network, water usage, non-revenue water (NRW) and financial data of the water sector.

Other aim of this thesis is to construct a workable model of Keetmanshoop's WDN that can be used and updated in the future by Keetmanshoop Municipality. The whole distribution network has never been modeled before, so this would mark the first time Keetmanshoop Municipality had a serviceable WDN model. In this thesis, the model is also used to pinpoint problematic areas of the network and to analyze its performance. The last objective is to present improvement proposals considering the water distribution, such as the organization of the water sector, network's appurtenances and WDN model.

This research is a case study of Keetmanshoop's water distribution. Field work was conducted in Keetmanshoop June 1st – July 31st, 2015. Research data was collected from Keetmanshoop Municipality's accounting, literature and previous theses of Keetmanshoop's water supply, by conducting semi-structured interviews, by having conversations and by doing observations and measurements in the field. Measuring instruments included a pressure gauge and a measuring tape. The WDN model was constructed before and after the field work. The software used in the modeling was Bentley WaterCAD (computer-aided design) V8i, QGIS (Quantum geographic information system) and FCG's (Finnish Consulting Group Oy) software FCGnet.

In Chapter 2, the universal basics of municipal water supply system, water demand and pressure management in WDN are covered. The basic theories behind modeling and the applications of a WDN model are also described in this chapter. In Chapter 3, background of Namibia and Keetmanshoop relevant to this thesis is covered. Additionally, the water services in Finland are shortly described for a comparison between a developing country and a first world country.

The study area is described in Chapter 4. A summary of semi-structured interviews and other research methods are also in this chapter. Furthermore, the construction of the WDN model is depicted in the last section of Chapter 4. The results of the research and the model's simulation results are presented in Chapter 5.

Improvement proposals considering Keetmanshoop's water distribution and the WDN modeling are presented in Chapter 6. The proposals are based on the results of this study and the discussions with Kangasala Water's Water Engineer & Project Manager Arto Hietanen and Keetmanshoop Municipality's Senior Manager of Department of Infrastructure and Technical Services Samuel Nashima. As development plans have also been presented in the previous studies, new proposals are introduced along with some old ones from the previous theses. Chapter 7 is where the discussion of the results takes place and Chapter 8 is for the conclusions.

2 WATER DISTRIBUTION NETWORK

Water and its supply are basic needs for communities. WDN is a complex infrastructural system consisting of many elements, such as pipes, reservoirs and valves. This distribution network acts as a water source for domestic, industrial, commercial and fire-fighting purposes (Al-Zahrani 2013). In Finland, 90 % of the population has access to municipal water supply, while in Namibia the proportion of total population served with piped water is 51 % (Knoema 2015, Vesilaitosyhdistys 2015).

The WDN should meet the demands and pressures at each node at all times. Analyzing the WDN is essential towards understanding the behavior of the supply and distribution. The information gained from hydraulic model simulations gives results such as characteristics of the WDN, e.g. flows, pressures and head losses, and the simulation can also assist in management planning for maintenance and equipment replacements as well as assessing the performance of water tanks and reservoirs. In addition, simulations can be used for water quality evaluation. (Al-Zahrani 2013) Nowadays when the calculation capacity has increased, simulations are conducted by modeling software such as WaterCAD and EPANET. A simplified simulation of the WDN or parts of it can also be manually calculated. In 1950, an electrical analog computer called The McIlroy network analyzer was developed to simulate the behavior of WDNs. After since, digitalization and hardware capacity has developed rapidly. (Walski et al. 2003)

First in this chapter, the universal basics of municipal water supply system and water demand are covered. In Section 2.3, the importance of pressure management in WDNs is explained. The basic theories of modeling of WDN are described in Sections 2.4–2.5. Applications and the purpose of the WDN model are described in Section 2.6.

2.1 Basics of municipal drinking water supply system

All WDNs serve the same purpose – to deliver water from the source or treatment facility to the customer (Walski et al. 2003). A municipal drinking water supply system consists of 3 major elements, which are source, treatment and distribution to the consumers (Mays 2000). The main principle of the supply system is presented in figure 2-1.

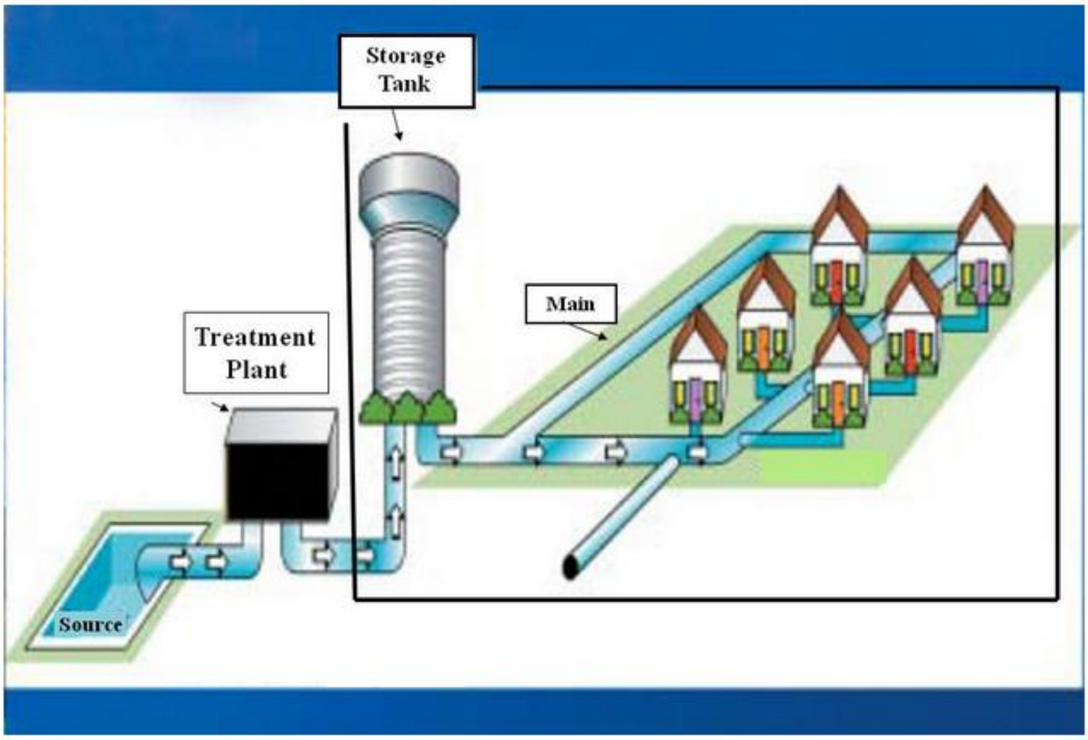


Fig. 2-1. *Principle of municipal water supply system (United States Environmental Protection Agency 2014a).*

2.1.1 Drinking water sources

There are 2 basic water sources for human consumption (Hickey 2008):

1. Well water to supply individual residents, farmstead properties and small public sector properties such as schools and public buildings.
2. Municipal water systems that provide potable water to a wide array of commercial property and domestic use buildings, fire protection and special services such as street cleaning and park irrigation.

The source for municipal water distribution needs to be adequate and reliable, and many municipalities use more than one type of water source (Hickey 2008). Sources of supply include surface water; streams, rivers, impounding reservoirs and lakes (Mays 2000, Walski et al. 2003), ground water and in some instances, brackish or seawater (Mays 2000). Reclaimed water is also used in some municipalities (Corbitt 1999). Table 2-1 presents the abstraction for water supply according to source in 2010 and 2012, and the sources' proportions are in figure 2-2.

Table 2-1. *Water sources in different countries [1.0 * 10^6 m^3/year] (The International Water Association 2014).*

COUNTRY/REGION	TRUE GROUND WATER		SPRING WATER		SURFACE WATER		OTHER SOURCES		IMPORT WATER		TOTAL WATER ABSTRACTION	
	2010	2012	2010	2012	2010	2012	2010	2012	2010	2012	2010	2012
AUSTRIA	355	325	446	472	0	0	0	0	0	0	801	797
BELGIUM	473	453	0	0	253	258	0	0	0	3	726	714
BRAZIL	n/a	n/a	n/a	n/a	n/a	n/a	n/a	n/a	0	0	14,518	15,424
BULGARIA	n/a	446	n/a	0	n/a	488	n/a	0	n/a	0	800	934
CANADA	46	79	0	0	1,790	1,261	0	0	0	0	1,836	1,340
HONG KONG, CHINA	0	0	0	0	228	217	0	0	681	709	909	926
CHINESE TAIWAN	465	448	6	6	3,955	3,508	0	0	0	0	4,425	3,962
CYPRUS	18	150	0	0	21	95	53	25	0	0	92	269
DENMARK	398	381	0	0	0	0	0	0	0	0	398	381
ENGLAND & WALES	1,787	1,684	0	0	4,072	4,144	0	16	0	0	5,858	5,844
FINLAND	261	261	0	0	184	184	0	0	0	0	445	445
FRANCE	6,000	3,630	0	0	2,750	1,870	0	0	0	0	8,750	5,500
GERMANY	3,079	3,081	422	419	1,537	1,569	0	0	0	0	5,038	5,069
HUNGARY	269	243	60	54	33	28	236	212	0	0	597	537
INDONESIA	n/a	0	n/a	10	n/a	281	n/a	0	n/a	0	n/a	292
IRAN	4,460	5,003	0	0	2,430	3,962	0	0	0	300	6,890	9,264
ISRAEL	834	864	0	0	479	354	388	357	148	0	1,849	1,576
ITALY	4,427	4,540	3,452	3,254	1,211	1,301	27	14	0	0	9,117	9,108
JAPAN	3,707	3,661	292	276	11,790	11,753	34	29	0	0	15,823	15,719
LIBYA	n/a	4,912	n/a	80	n/a	60	n/a	76	n/a	0	n/a	5,128
MACAO, CHINA	0	0	0	0	79	90	0	0	0	0	79	90
MAURITIUS	114	110	0	0	109	106	0	0	0	0	223	217
MEXICO	n/a	30,011	n/a	0	n/a	50,048	n/a	0	n/a	0	n/a	80,059
NETHERLANDS	761	754	0	0	395	394	0	0	10	0	1,166	1,148
NORWAY	12	82	0	1	710	717	0	0	0	0	722	800
POLAND	1,625	1,630	0	0	9,173	9,143	68	58	0	0	10,866	10,830
PORTUGAL	378	313	0	0	845	625	0	0	0	0	1,223	938
ROMANIA	354	469	0	0	825	693	0	0	0	0	1,179	1,162
SOUTH KOREA	556	98	0	0	6,606	7,078	0	0	0	0	7,162	7,175
SPAIN	615	723	135	0	4,156	3,061	152	205	0	0	5,058	3,989
SWEDEN	197	174	0	0	490	521	222	175	0	0	909	870
SWITZERLAND	382	397	369	364	189	174	0	0	0	0	940	935
UNITED STATES OF AMERICA	15,546	11,132	0	0	26,293	36,814	10,329	0	0	6,776	52,169	54,723

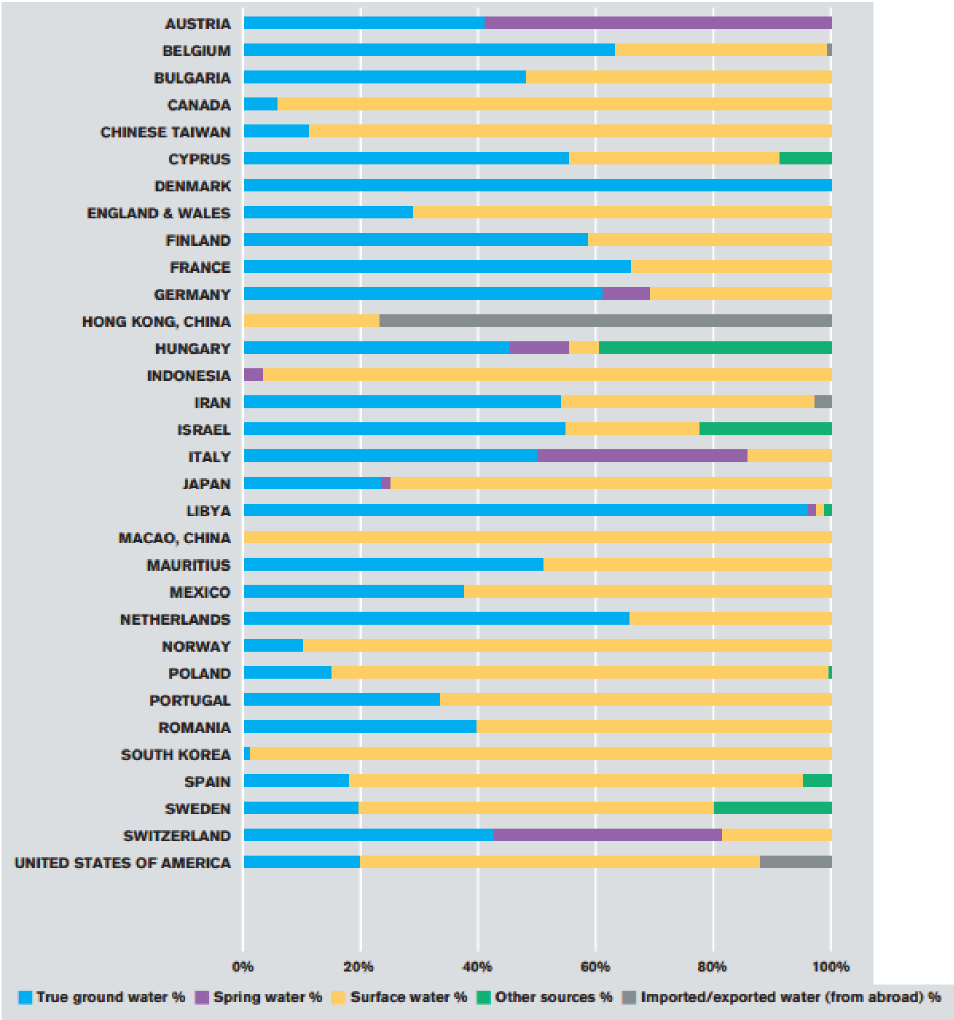


Fig. 2-2. *Abstraction source for drinking water supply in % in 2012 (The International Water Association 2014).*

In many countries, surface water and true ground water are the most common drinking water sources. In figure 2-2, spring water which is ground water, is separated from true ground water, hence the name true ground water.

Ground water is the water found beneath the earth's surface, being an important water source for many people **(**United States Environmental Protection Agency 2014b). Approximately 10.5 * 10^6 km^3 of ground water, 30.1 % of the world's freshwater, are estimated for the entire planet of Earth (National Ground Water Association 2010). Ground water collects naturally through the water cycle forming large pools under surface called aquifers. They can be as deep as several dozens of meters, which diminishes the exposure to pollutants. (New Jersey American Water 2013) From the soil and bedrock, water is produced by a well (National Ground Water Association 2010).

Lakes are supplied by streams and rivers, which in turn are supplied from water runoff from hills. This is usually considered an inexhaustible supply of water. Intake lines should be below the ice formation level to prevent jamming at the inlet ports. (Hickey 2008) Of all freshwater, approximately 0.26 % is found in lakes and only 0.006 % in rivers, the total volumes being 91 000 km^3 and 2 120 km^3, respectively (United States Geological Survey 2015).

Reservoirs created by placing dams across streams, rivers or at the neck of a valley to capture runoff are an old invention. In properly designed reservoirs, the head of a water source creates a pressure to move the water through the treatment plant and the distribution system. Impounding reservoir systems are considered the most reliable and economical source of municipal water supply; they are not dependent on mechanical interface. (Hickey 2008)

2.1.2 Water treatment

Raw water is either pumped to the treatment plant or it flows there by gravity. In treatment plant, raw water is processed to achieve potability. The degree to which it has to be treated depends on the characteristics of the raw water and the distribution system, relevant drinking water standards and treatment processes used. (Walski et al. 2003) As the result from this treatment, raw water achieves potability and can be distributed to the customers. For most communities, raw water is treated within public utilities to ensure acceptable water quality. Despite the treatment however, potable waters are not sterile and contain microorganisms: microbial growth can occur within distribution systems and in addition, microorganisms can be introduced into the treated water via intrusion. (Gerba & Pepper 2014)

A typical surface water treatment plant consists of the following treatment processes, respectively (Centers for Disease Control and Prevention 2015):

1. Coagulation and flocculation. Chemicals with a positive charge (coagulants) are added to the raw water. The positive charge neutralizes the negative charge of dirt and other dissolved particles. This occurring, the particles bind with the chemicals and form larger particles called flocs.
2. Sedimentation. Due to flocs weights, they settle to the bottom of the tank.
3. Filtration. After sedimentation, the clear water on top will pass through filters of varying compositions (sand, gravel and charcoal) and pore sizes in order to remove dissolved particles such as dust, parasites, chemicals, bacteria and viruses.
4. Disinfection. A disinfectant, such as chlorine or chloramine, is added to the water in order to kill any remaining parasites, bacteria and viruses and to protect the water from germs when it’s piped.

Surface water typically requires more treatment and filtration than ground water since it contains more sediment and pollutants (Centers for Disease Control and Prevention 2015). This makes treating surface water more complex and expensive than treating ground water (New Jersey American Water 2013).

Treated surface water usually enters a unit called a clearwell before leaving the treatment plant. According to Walski et al. (2003), this clearwell serves 3 main purposes:

1. It provides adequate contact time for disinfects at the end of the treatment process.
2. It provides extra storage that acts as a buffer between the treatment plant and distribution system.
3. It serves as a source for backwash water for cleaning plant filters.

2.1.3 Water distribution network

A WDN consists of 3 major components: distribution piping, distribution storage and pumping stations (Al-Zahrani 2013). These components convey drinking water and meet fire protection needs for homes, schools, hospitals, businesses, industries and other facilities. Public water systems are dependent on distribution systems to provide constant supply of pressurized safe drinking water to all consumers. (United States Environmental Protection Agency 2014a) Pipes are the most abundant elements in the network (Mays 2000).

Transmission mains consist of components that are designed to convey large amounts of drinking water over great distances, such as from a treatment facility to storage tanks throughout several cities within the water supply system. Individual customers are not usually served from this system. (Walski et al. 2003) Distribution mains carry water from the treatment plant to service lines (United States Environmental Protection Agency 2014a). Distribution mains are smaller in diameter than transmission mains and usually follow the typology and alignment of city streets. Both transmission and distribution systems can be either looped or branched (figure 2-3). Service lines transmit the water from distribution mains to the consumers. (Walski et al. 2003)

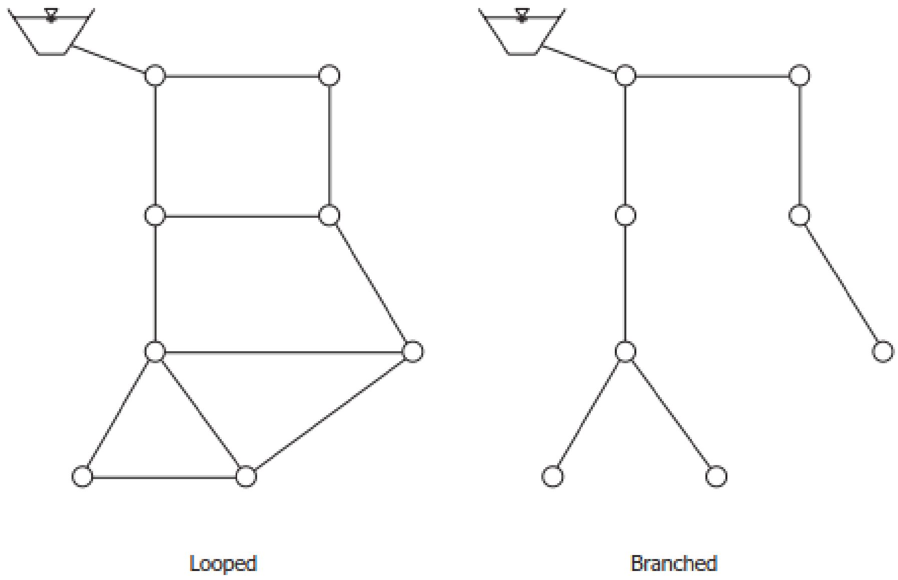


Fig. 2-3. *Looped and branched distribution networks (Walski et al. 2003).*

In looped systems, there may be several paths the water can follow from source to customer, whereas in branched systems the water has only one possible path. Looped systems provide an additional level of reliability. Most WDNs are a combination of both, with a trade-off between loops for reliability and branches for infrastructure cost savings. (Walski et al. 2003)

Pipes are manufactured in varying sizes and are composed of different materials such as steel, ductile iron, asbestos cement (AC) and polyvinyl chloride (PVC). As pipes age, deterioration can occur due to temperature changes, material erosion, corrosion and external pressure. This can lead to pipe breaks, water pressure fluctuations and other situations that pose health risks. (New Jersey American Water 2013, United States Environmental Protection Agency 2014a)

Distribution storage, which includes elements such as reservoirs and water tanks, is included in the network for firefighting and for balancing storage in events of varying demands to equalize pump discharge near an efficient operating point (Al-Zahrani 2013, Mays 2000). It also supplies the network during outages of individual components and dampens out hydraulic transients (Mays 2000). As with pipes, storage tanks also suffer from deterioration resulting in openings of the tanks (United States Environmental Protection Agency 2014a).

Storage is provided as elevated storage or ground-level storage with high service pumping. Elevated storage provides the most reliable and useful form of water storage maintaining constant water pressure. Water kept in ground storage must be delivered to the water demand points by pumping equipment. This arrangement limits water system effectiveness: there must be excessive pumping capacity for peak demands as well as firefighting demand, pumps must be maintained and functioning at all times, and the distribution lines must be oversized to handle peak demand use plus required fire flow. (Hickey 2008) A typical elevated storage tank installation is illustrated in figure 2-4.

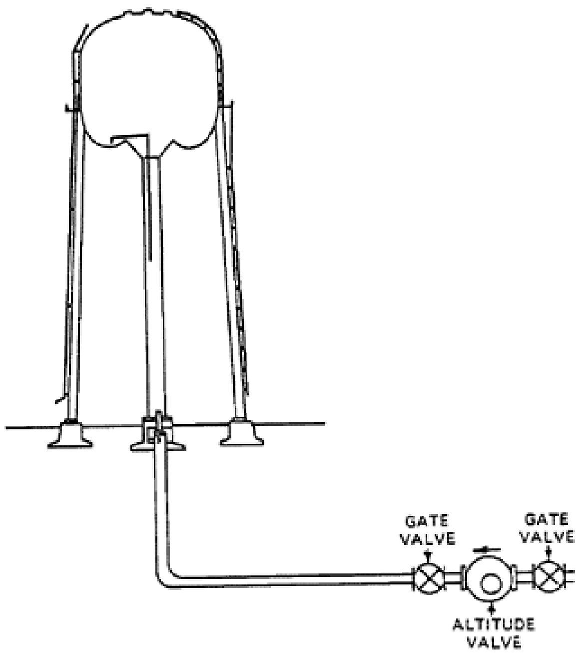


Fig. 2-4. *Typical elevated water storage (Mays 2000).*

Most WDNs require pumps to supply extra head to overcome the head losses due to friction or to increase hydraulic head (Al-Zahrani 2013, Hickey 2008). Some WDNs are solely based on gravity, with the network achieving pressure on each water demand point due to pressure head. If the whole water supply system isn't a gravity feed system, pumping pressures systems are needed for taking water from a supply source, pass the water through a treatment plant and transport the water into the distribution system (Hickey 2008).

There are different types of pumps, such as positive-displacement pumps, kinetic pumps, turbine pumps, horizontal pumps, vertical pumps and horizontal centrifugal pumps. In WDNs, the most commonly used is the centrifugal pump. Pump stations house the auxiliary equipment, motors and pumps. (Mays 2000)

Other common appurtenances of the WDN include valves, fire hydrants, wells and monitoring systems (Mays 2000, Walski et al. 2003). They are frequently connected to the distribution mains (Walski et al. 2003). Valves come in different materials and functions such as (Hwandon 2005, Mays 2000):

- Gate valves. The dominant valve type. Isolates the subsystem by preventing any flow through it. Usually installed to on-off.
- Butterfly valves. Also an isolation valve. Used for throttling (flow control).

- Ball valves. An isolation valve as well. Utilizes a ball with a hole drilled through it as the opening/closing device. Usually installed to on-off.
- One-way valves. Allows the water flow through it in one direction.
- Pressure reducing valves (PRVs). Installed to maintain a constant and specific pressure at the downstream side of the valve for all flows with a pressure lower than the upstream. Permits flow from the high pressure system if the pressure on the lower is acceptable.
- Air release valves. Release unwanted air pockets from the mains to control flow. Air pockets reduce pipeline capacity and can also cause damage to the pipelines and fittings through water hammers: when air pockets in the mains dislodge, e.g. due to rapid closing of valves, pressure spikes are generated and may result in water hammers. Water hammer is a banging sound or vibration of the piping system that damages the network. (Merritt 2001)
- Blow off valves. Drain water to lower excessive pressure.

Fire hydrants in WDNs are outlets from water mains to which fire hoses can be connected (AccessEngineering a). Fire hydrants are used for flushing of mains and for firefighting (Corbitt 1999). There are 2 types of fire hydrants: dry barrels and wet barrels. The dry barrel prevents freezing and is recommended for areas that have any remote possibility of freezing. It has a device that allows the water in the barrel to drain down to the frost line after the valve controlling water to the outlets has been closed. (Frankel 1996) Wet barrel hydrants can only be used in areas that are free from freezing (Corbitt 1999).

In WDNs, water meters are used by water utilities for monitoring water consumption and flow in different parts of the network, especially the water use of consumers. Water meters help utilities to collect revenue they are due, enable accounting and financial plans, pinpoint leaks, locate pressure problems along their waterways and identify periods of peak and off-peak use among residential and business consumers. Metering must be provided at all important water delivery locations including the supply source. (Environmental Protection Division 2007) Water meters that meter the water usage of consumers are installed at service connections and are under the responsibility of water utility (Environmental Protection Division 2007, Tukes 2013).

Wells can also be used in WDNs, mainly for emptying and flushing the water pipe (RIL 124-2-2004 Vesihuolto 2 2004). In addition, sensor networks can be used in WDNs for many purposes, such as water quality and flow monitoring and leakage and burst detection. They can be highly helpful for identifying potential issues early on and for proactive maintenance. (Ediriweera & Marshall 2010) A monitoring system used by Lempäälä Waterworks is presented in figure 2-5. This online monitoring system lies inside a well and is used for monitoring water characteristics such as flow rate, pressure, temperature and turbidity (Arto Löppönen, Lempäälä Waterworks' Water Network Engineer 2015).

Fig. 2-5. *Monitoring system used in a WDN in Lempäälä (Salmivirta 2015).*

2.2 Water demand

The determination of the quantity of the water required presently and in future is the first consideration of designing a WDN. For domestic consumption, the water demand in a community is estimated on the basis of per capita consumption. (Hickey 2008) Per capita consumption is the volume of water distributed to the network divided by the number of residents connected to the network in a 24-hour period [l/capita/d]. Per capita consumption also includes non-residential use, which makes it higher than the actual residential consumption. (RIL 237-2-2010 Vesihuoltoverkkojen suunnittelu 2010) Since water consumption varies in time and between communities, water demand must be studied in each community separately.

These studies include (RIL 124-2-2004 Vesihuolto 2 2004):

- population and employment records and trends
- measured data
- town planning, land use planning and building effectiveness
- companies' action and development plans
- period of design (approximately 30 years in Finland)
- rate of consumers connected to the network
- area of distribution.

Land use is a way to determine water demand in a metered community (Mays 2000). Table 2-2 presents water use for different establishments.

Table 2-2. *Typical rates of water use for different establishments (Mays 2000).*

User	Range of flow [l/person/d]
Airport, per passenger	10–20
Hotel	200–400
Apartment house on public water supply, unmetered	300–500
Private dwelling on individual well or metered supply	200–600
Private dwelling on public water supply, unmetered	400–800
Hospital	700–1 200
Office space	40–60
Restaurant, average including toilets	25–40
School, with cafeteria or lunchroom	40–60
School, with cafeteria and showers	60–80

In table 2-3, water demand is examined by developing water duties (water usages) for the various types of land use that can be used for future planning. The values are for water duties in the western USA and the definitions of land use terms vary by community. (Mays 2000)

Table 2-3. *Typical water duties in the western USA (Hickey 2008, Mays 2000).*

	Water duty [m^3/day/m^2]		
Land use	**Low**	**High**	**Average**
Low-density residential	0.37	3.1	1.6
Medium-density residential	0.84	3.6	2.4
High-density residential	2.2	11.2	3.9
Single-family residential	1.2	2.7	2.2
Multifamily residential	2.4	6.2	3.9
Office commercial	1.0	4.8	1.9
Retail commercial	1.0	4.8	1.9
Light industrial	0.19	4.4	1.5
Heavy industrial	0.19	4.5	2.1
Parks	0.37	2.9	1.9
Schools	0.37	2.3	1.6

Variations in demand depend on numerous factors. Firstly, water use fluctuates hourly, daily, monthly and yearly. Moreover, geographic locations, community size, climate, industrialization, infrastructure, water fittings and water use habits unique to every community result in varying and fluctuating uses. (Hickey 2008, Mays 2000)

Average day demand is the total annual quantity of water produced by an agency or municipality divided by 365 [m^3/d]. Fluctuations in demand are greater in small communities and during short rather than during long periods of time (Mays 2000).

In Finland, peak hour demand ranges from 1.9 to 4.3 times greater than average day demand depending on the amount of water users: for less than 10 000 users it's at its highest and for more than 100 000 users it's at its lowest (RIL 124-2-2004 Vesihuolto 2 2004). Some U.S. municipalities use peak demand factor up to 7 times greater than average day demand, globally it's usually 2.5…4.0 times greater. Peak hour demand is the highest demand of the year during any one hour period. (Dewberry 2008, Mays 2000)

Maximum day demand in Finland ranges from 1.3 to 1.8 times greater than average day demand similarly to peak hour demand, and common range globally is 1.8…2.8 times more. Maximum day demand is the highest demand of the year during any 24-hour period. (Mays 2000, RIL 124-2-2004 Vesihuolto 2 2004).

The source of supply to the WDN should meet maximum day demand at minimum. It is common for communities that the supply meets maximum day demand with the additional supply to meet peak hour demand coming from water storage. (Mays 2000) Another typical situation is where distribution lines are sized for maximum day demand plus fire flow (Dewberry 2008). Some communities find it more economical to develop a source of supply that meets maximum day and peak hour demand (Mays 2000). In Finland, maximum day demand is used for storage, and the network should meet peak hour demand and average day demand/16 h plus fire flow (RIL 124-2-2004 Vesihuolto 2 2004).

The demand analysis also includes fire flow requirements. The amount of water required for this puts a heavy strain on the system, although the overall volume of water used for firefighting is quite low relative to most other uses. (Dewberry 2008) Fire demands are dependent on land use and local conditions, therefore each municipality establishes its own parameters (Dewberry 2008, Mays 2000). Some typical fire flow requirements are presented in table 2-4.

Table 2-4. *Typical fire flow requirements according to land use (Mays 2000).*

Land use	Fire flow requirements [l/s]
Single-family residential	2–9
Multifamily residential	7–13
Commercial	11–22
Industrial	15–44
Central business district	11–66

There are also differences between communities' service pressure requirements in WDNs. Pressure requirements are a combination of normal service requirement and fire flow demand. (Dewberry 2008) Pressure must be high enough to overcome all energy losses within the mains and service lines and not to cause annoying flow reductions. Pressure should also cover the losses incurred from hydrants, hoses, nozzles and other firefighting equipment. (Dewberry 2008, Mays 2000)

Too high pressure adds to system costs by increasing leakage and burst volumes and potential damage to fixtures (Dewberry 2008, Mays 2000). Normal pressure in a junction between mains and service lines should be 2.5…7.0 bars in Finland (RIL 124-2-2004 Vesihuolto 2 2004). The pressures maintained in the distribution systems also vary in various communities. For instance, in Namibia, the minimum pressure in households should always be above 200 kPa (Nashima 2015). Table 2-5 presents typical pressure criteria under varying conditions.

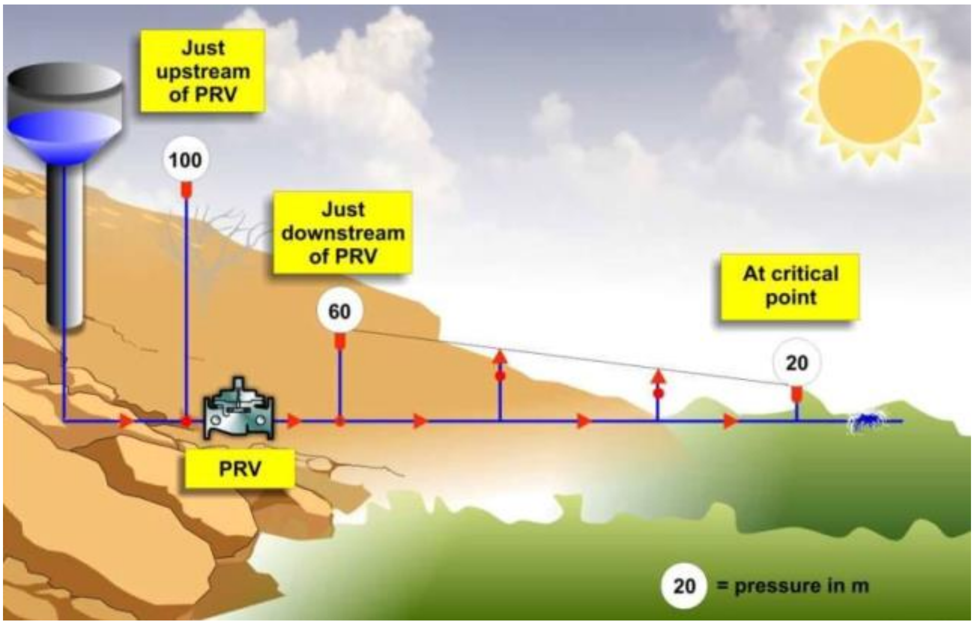
Table 2-5. *Typical service pressure criteria (Mays 2000).*

Condition	Service pressure criteria [bar]
Maximum pressure	4.5–5.2
Minimum pressure during maximum day	2.1–2.8
Minimum pressure during peak hour	1.7–2.4
Minimum pressure during fires	1.4

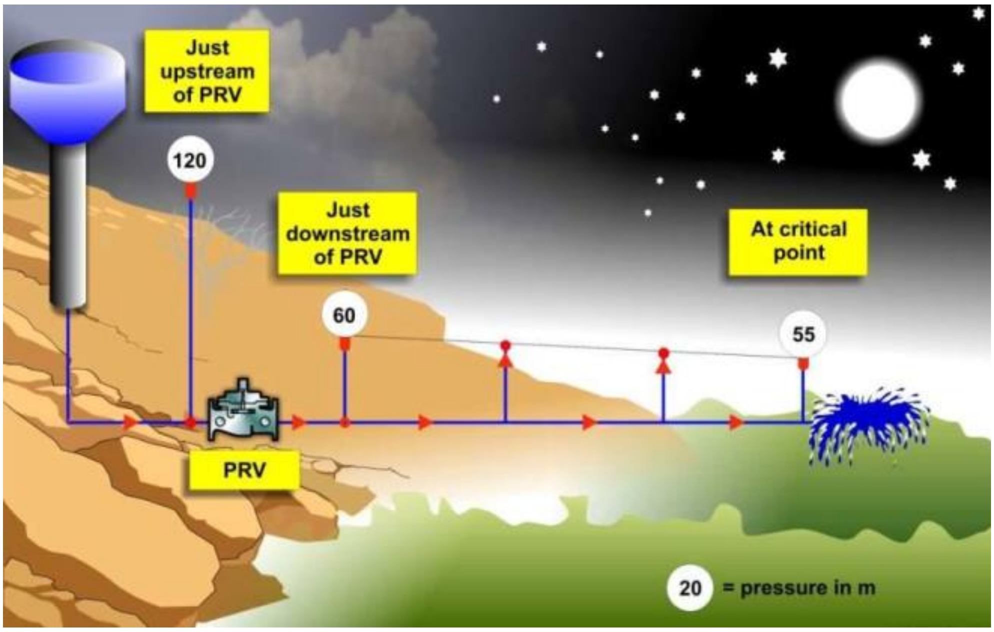
A pressure of 5.5 bar is recommended as the upper limit in most cases. Booster pumps for tall buildings can be used to eliminate the need for excessive pressure in mains. (Dewberry 2008)

2.3 Pressure management in water distribution network

Whole water supply systems including WDNs are generally designed to provide water to consumers at a minimum level of pressure at the point of the lowest pressure in the network. In addition, there are also fire flow requirements which can override the normal consumer requirements. Distribution networks are designed to accommodate pressure and flow requirements during the peak of demand. This means that WDNs tend to operate at pressures significantly higher than required most of the time. (McKenzie & Wegelin 2010, Thornton et al. 2008) In figure 2-6, a PRV is used for lowering the pressure to acceptable level during a) peak period (day time). However, during the b) period of low consumption (night time) the pressure at critical point is excessive, even though the PRV is operated in the same way. (McKenzie & Wegelin 2010)



a)



b)

Fig. 2-6. *Typical pressures during a) peak and b) off-peak periods (McKenzie & Wegelin 2010).*

2.3.1 Implementation

Pressure management includes pressure reduction, increase and sustaining, surge control and the management of reservoir and tank levels (Thornton et al. 2008). It can serve many purposes. Most importantly, it's an immediate way of reducing leakage volumes and leakage frequencies by lowering and stabilizing water pressure (Shammas & Ai-Dhowalia 1992, Thornton et al. 2008). This is an effective way of water conservation, as pressure is a significant factor in the influence of new leakages (Thornton et al. 2008).

Pressure reduction can be implemented in many ways. The level of pumping can be increased during high demand and decreased during low demand. This also conserves energy use of water utility. Also, partially closed gate or butterfly valves can be used to create a head loss and reduce pressure. However, this is the least effective method of pressure reducing since head losses created will change as water demands change. This creates a case of an upside down zone. (Thornton et al. 2008)

Moreover, fixed outlet technique can be used to reduce and control pressure. It involves a use of a device, usually a PRV, to control the maximum pressure entering a zone. Fixed outlet is low-cost and simple to implement, maintain and operate since it only requires one valve with no additional equipment. However, it doesn't have the flexibility to adjust water pressures during different times of the day (demand). Figure 2-6 demonstrates this event. (McKenzie & Wegelin 2010, Thornton et al. 2008)

Time-modulated pressure system operates like fixed outlet system with an additional device which can provide further reduction in pressure during off-peak periods. The system has an electronic controller connected to a PRV, which allows pressures to be reduced at specific times of the day resulting in greater savings. This system is relatively low-cost and easy to set up and operate. The main disadvantage is that it doesn't react to demand fluctuations, which can cause water shortage during high demand especially if full pressure fire flow is required in case of a fire. (McKenzie & Wegelin 2010, Nicolini & Zovatto 2009)

Flow modulated pressure control provides even greater control and flexibility than the time-modulated technique. The system has a flow modulated controller device connected to a PRV, which makes it able to control the downstream pressure according to demand conditions by controlling flow rates. During peak demand periods, the system dictates the maximum pressure, whereas for low demand periods the downstream pressure is adjusted to minimum. (McKenzie & Wegelin 2010, Nicolini & Zovatto 2009) Flow modulated pressure control provides savings and doesn't hamper the water supply, but the electronic controller is expensive and it requires a properly sized water meter in addition to the PRV. Furthermore, the use of flow modulated control is sophisticated and requires human resource skills. Lastly, PRVs with flow modulated capabilities are practical in firefighting, since the system has sufficient hydraulic capability to maintain pressures and flows. (Thornton et al. 2008)

Sectorization into pressure zones is a basic and an effective form of pressure management (Nicolini & Zovatto 2009). WDN is divided into subsectors either naturally or by physical valving. Usually, the sectors are quite large with multiple feeds, so they don't develop that many hydraulic problems (e.g. feeding pressure) caused by valve closures. Gravity feed systems are usually sectorized by ground level, whereas pump feed systems are sectorized depending on the level of elevated tanks and reservoirs. In its simplest form, sectorization doesn't require automatic control valves and controllers but just gate valves. However, it's not completely efficient then. (Thornton et al. 2008)

Pressure management also includes water level control. Overflows can often be sources of considerable annual water losses. Due to the lack of demand and head loss in the system, overflows usually occur at night. Level control can be performed by hydraulic control using altitude valves or ball valves. (Thornton et al. 2008) Even gate valves can be used to shut down the inflow to the storage system (Nashima 2015). Level control can be also be managed by SCADA (supervisory control and data acquisition) which involves automatic control by computer-linked software, or manually by pump control (Thornton et al. 2008). SCADA systems enable an operator to remotely view real-time measurements, such as the water level in a tank, and remotely operate network elements, e.g. pumps and valves (Walski et al. 2003).

Using break pressure tanks is one method of pressure reducing (Shammas & Ai-Dhowalia 1992). The principle of this system is that the outlet pressure from the break pressure tank is caused by the pressure head inside the break pressure tank, which is lower than the inlet pressure to the tank. This system can be implemented if there is, for instance, excessive pressure in the pipelines caused by high elevation differences. (Hietanen 2015, Damas Mashauri, Polytechnic of Namibia's Professor 2015)

2.3.2 Benefits of pressure management

Pressure management leads to reduction of leakage volumes and frequencies, which then leads to secondary benefits. These are (Fantozzi 2015, Nicolini & Zovatto 2009):

- reduced water treatment costs (or purchase costs from bulk water supplier) of utilities
- reduced investment and repair costs of utilities
- conservation of water and environment
- energy savings of utilities
- improvements to customer service.

Other benefits also exist which are not straight related to leakage control. Utilities can conserve water by using pressure reduction as a way of controlling unwanted demand: less water will be consumed at a lower pressure. Moreover, some utilities have to face nonpayment situations. Due to social or political reasons, utilities have to supply water even when the customers aren't paying. In these situations, pressure management is a way to reduce consumption while maintaining a minimum level of supply. Even when drought takes place, water demand can be cut by pressure reduction. (Thornton et al. 2008)

Using pressure management, pressure can be sustained and boosted to enable efficient distribution of water. The reason for inefficient distribution may be due to geographic constraints or poor network designs, for example. Pressure management ensures required volumes for a majority of the customers. Also, pressure management and water level control guarantees that there is enough water in storage without any overflows. (Thornton et al. 2008)

Most WDNs have situations where an operator closes or opens a valve too fast or a large consumer suddenly stops drawing water. This creates transient waves to travel backward and forward within the pipelines, causing damage at weak points if there is no valve control. Simply limiting pressure to required levels is a way of reducing the negative impact of transient waves. (Thornton et al. 2008)

Due to constant supply of water pressure and volume, there are fewer water supply failures in customer service connections and plumbing systems. This results in increased customer satisfaction. (Fantozzi 2015, Thornton et al. 2008)

Pressure management also plays a vital role in firefighting: WDN has to have sufficient hydraulic capacity to maintain pressures and flows. PRVs with flow modulated capabilities are usable for this. Other possibility is to use so called sleeper valves which will open only when the system pressure drops to a certain pressure due to head losses created by the fire flow. (Thornton et al. 2008)

To summarize, pressure management often provides financial savings in a short period of time. It also extends the useful life of the WDN and is a suitable means of controlling water losses. (McKenzie & Wegelin 2010)

2.4 Modeling process of water distribution network

In this context, simulation refers to the process of using a mathematical representation of a real WDN, called a model. Assembling, calibrating and using a WDN model can seem like an oppressive assignment with stacks of data and maps and hundreds of elements. The way to complete it is to break it down into components. Some tasks must be done in series, but some of them can be done in parallel. (Walski et al. 2003) Figure 2-7 illustrates the tasks of the modeling process.

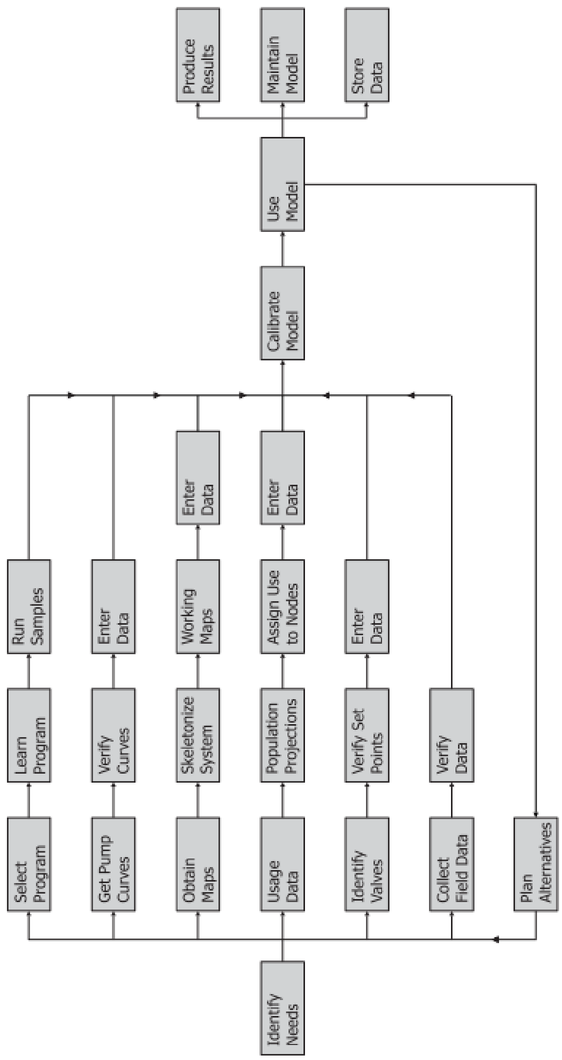


Fig. 2-7. *WDN modeling process (Walski et al. 2003).*

Modeling should be viewed as a utility-wide effort with the modeler as the key worker. Modeling is a project to develop consensus within the water utility regarding the need and the purposes for the model. (Walski et al. 2003)

Before the WDN model can be used to compute real problems (e.g. minimum and maximum pressures), most of the modeling work must be done. First task is to represent pipes, pumps and other elements in maps and drawings. Then the maps are converted to a model; WDN is represented by a series of links and nodes. A link, which represents a pipe, has a node at each end. Nodes represent junctions, reservoirs and tanks. Pumps and valves can be classified either way. The behavior of the links and nodes are described mathematically as another layer of abstraction. Next, the model equations are solved and the solutions are displayed as numerical or graphical data. (Al-Zahrani 2013, Walski et al. 2003)

In practice, the network data is often quite uncertain. As the exact pipe properties may not be completely known, the use of estimates like pipe roughness is required. Because of these estimations, the output of the WDN model might not correspond reliably with the reality and calibration of the model is needed. Also, sometimes modeling requires the use of other software, such as GIS (geographic information system), and not all modeling software support them. Furthermore, there usually isn't adequate or reliable data of other important elements such as pump curves and tank properties and even pipe sizes, locations and connections. (Mays 2000, Savic 2006) Moreover, pipe roughness coefficients change during the time, and big water consumers produce problems considering node demands (Walski et al. 2003).

After the WDN model is built, it needs to be calibrated. Basically, the output of the model is compared with the measured output data of the real system, and the model is changed until it matches the real system better. There are always inaccuracies, errors and uncertainties in the input data. (Mays 2000, Walski et al. 2003) Walski et al. (2003) divide errors in input data into 2 categories, typographical and measurement errors. Typographical error is a mistake made in the typing process of input values. Measurement errors are due to scaled and obscured maps. For example, if a length of a pipe is measured from a map that has a scale of 2.54 cm (1 in) = 304.8 m (1 000 ft), the measured length may be within ± 15.24 m (50 ft) of its real length.

The first step of calibration is known as rough-tuning or macrocalibration. This is where large discrepancies are fixed. After this, the WDN model can be fine-tuned or microcalibrated. This is the final step of calibration and involves adjusting the pipe roughness values and nodal demand estimates. Validating the model takes place after calibration. By validating the model with test data obtained under different conditions, shortcomings can be identified. For example, a model that was calibrated for a peak day may be validated for average day conditions. If the validation fails, the model is defected and needs to be reconsidered and fixed. (Walski et al. 2003)

The final step of modeling process is simulation. It can be used for predicting WDN responses to events under a wide range of conditions without disrupting the actual network. Steady-state simulations are used to determine network's behavior (flows, pressures, valve positions etc.) under static conditions at a specific point of time assuming that hydraulic demands and boundary conditions don't change with respect to time. In this simulation, tank and reservoir levels and pump operations remain constant. Steady-state simulation provides information regarding equilibrium variables. This type of simulation can be useful in determining the short-term effects of events such as average demand conditions, network component failures or fire flows. However, as WDNs rarely are in a true steady state, the notion of a steady state is a mathematical construct. Demands, water levels and pump operations are always changing. Still, steady-state simulation is the basis for extended-period simulation (EPS): steady-state simulations are actually hydraulic time steps that are strung together in sequence to form EPS. (Mays 2000, Walski et al. 2003)

EPS allows to simulate the system response over time through hydraulic time steps, and it's used to evaluate WDN performance over time. This allows the modeler to model tanks' water level fluctuations, valves opening and closing, and changes of pressures and flow rates throughout the network in response to varying demand conditions and automatic control strategies. EPS needs more input data than steady-state simulation, therefore the modeler needs to be sure that the steady-state simulation works properly before the EPS can be attempted. In EPS calculation process, after each hydraulic time step, the system boundary conditions are re-evaluated and updated to reflect changes in junction demands, tank levels and so on. Then another time step is taken, and this process continues until the end of the simulation. (Mays 2000, Walski et al. 2003)

Hydraulic time step is the length of time for one steady-state portion of EPS, usually being one hour. The shorter the time step, the more accurate the simulation is. Time step should be selected such way that changes in network hydraulics from one increment to the next are gradual. The most common EPS duration is usually a multiple of 24 hours, as the most recognizable patter for demands and operations is a daily one. The simulation can also be run for a duration of weeks or for a few hours when modeling short-term emergencies or disruptions. (Walski et al. 2003)

Other types of advanced simulations, which are derived from steady-state simulations and EPSs, can also be built. Advanced simulations can be performed to water quality, automated fire flow, costs and transient events. (Walski et al. 2003)

Skeletonization is the process of representing only selected pipes and other elements in the WDN model. At one end of the spectrum, a model might include only the major pipelines connecting points (water entry, pump stations, storage, control valves and major consumers). The other end of the spectrum includes every pipe of the network. Usually network is skeletonized by first deciding on the smallest diameter of pipe to include in the model. Skeletonization reduces data-handling requirements and eases comprehension of model output. Disadvantages include the need to use engineering judgement of which pipes and elements to omit and difficulties in aggregating demands from individual users. All-mains models depict true system behavior more accurately, and as the user interfaces and data-handling capabilities of network modeling software have become more sophisticated, it has become increasingly easier for utilities to develop and use all-mains models. (Mays 2000)

2.5 Basics of mathematical representation of water distribution network

Simulation of WDN is based on mathematical representation of the actual network. In this section, the most well-known and vital equations concerning the flow in a pipe system are presented. The following equations are only for pressurized pipelines, meaning that pipelines are applied to water supply and a free water surface is almost never found within the conduit itself.

Flow can be characterized as laminar or turbulent. Laminar flow is streamline flow of an incompressible, viscous Newtonian fluid. In laminar flow, all particles of the fluid move in distinct, separate lines. The velocity adjacent to the pipe wall is zero and increases to a maximum in the center of the pipe. (AccessEngineering b) Turbulent flow on the other hand means the motion of fluids in which local velocities and pressures fluctuate irregularly (AccessEngineering c). The structure of the fluid breaks up and there is rapid mixing among the fluid particles. The flow in pipes is typically turbulent. (Mays 2000) Sometimes a third flow classification, transitional, is used. In that state, flow is transitional between laminar and turbulent flow. Using the Reynold’s number, the state of the flow can be determined:

$$R_e = \frac{VD\rho}{\mu} \tag{2.1}$$

where R_e is the Reynold’s number *[dimensionless]*, V is the mean velocity of the fluid *[m/s]*, D is the pipe diameter *[m]*, ρ is the fluid density *[kg/m^3]* and μ is the dynamic viscosity *[kg/m*s]*. (Giles et al. 2014, Mays 2000) Given the result, flow regime can be determined as following (table 2-6).

Table 2-6. *Reynold’s number for flow regimes (Mays 2000, Walski et al. 2003).*

Flow regime	Reynold's number
Laminar	< 2 000
Transitional	2 000–4 000
Turbulent	> 4 000

Equation 2.2 depicts the volume of fluid in a pipe passing per unit time, in other words the pipeline flow rate. Equation 2.3 depicts the equation of continuity. In equation 2.4, conservation of energy is described.

$$Q = v * A \tag{2.2}$$

$$Q_1 = Q_2 = v_1 * A_1 = v_2 * A_2 \tag{2.3}$$

$$\frac{p_1}{\gamma} + \frac{v_1^2}{2g} + z_1 + \sum h_p = \frac{p_2}{\gamma} + \frac{v_2^2}{2g} + z_2 + \sum h_L + \sum h_m \tag{2.4}$$

where Q is the pipeline flow rate *[m³/s]*, A is the cross-sectional area of the pipe *[m²]*, v is the average fluid velocity *[m/s]*, p is the pressure *[Pa]*, γ is the specific weight of water *[N/m³]*, g is the gravitational acceleration constant *[m/s²]*, z is the elevation above datum *[m]*, h_p is the head added at pumps*[m]*, h_L is the head loss in pipes due to friction *[m]* and h_m is the head loss due to minor losses in bends, fittings etc. *[m]*. (Giles et al. 2014, Mays 2000, Walski et al. 2003)

Head loss due to friction, h_L *[m]*, can be calculated using several methods. Hazen-Williams formula is frequently used:

$$h_L = \frac{C_f L}{C^{1.852} D^{4.87}} Q^{1.852} \tag{2.5}$$

where C_f is the unit conversion factor *[10.7* in *SI*-system*]*, L is the distance between sections 1 and 2 of the pipe *[m]*, C is the Hazen-Williams C-factor and D is the pipe diameter *[m]*. C-factor in this equation describes pipe carrying capacity; higher C-factor represents smoother pipes with higher carrying capacities. (Walski et al. 2003) For instance, in Bentley WaterCAD v8i, PVC pipes have a C-factor of 150 and AC pipes 140.

Minor losses are due to turbulence within the bulk flow as it moves through fittings, bends, valves and other appurtenances. Minor losses can be calculated as following:

$$h_m = k \frac{V^2}{2g} \tag{2.6}$$

where h_m is the head loss due to minor losses *[m]* and k is a minor loss coefficient that has no dimension and is derived empirically from testing the head loss of the appurtenance in question. Some empirically determined values of k are presented in table 2-7 (Giles et al. 2014). In addition, the total sum of head losses around each loop should be zero; the changes in energy must sum to that value (figure 2-8). (Walski et al. 2003)

Table 2-7. *Values of k for contractions and enlargements (Giles et al. 2014).*

Sudden contraction		**Gradual enlargement for total angles of cone**			
D_1/D_2	k	4°	10°	20°	50°
1.2	0.08	0.02	0.04	0.16	0.35
1.6	0.26	0.03	0.07	0.26	0.57
2.0	0.37	0.04	0.07	0.29	0.63
2.5	0.41	0.04	0.08	0.30	0.65
3.0	0.43	0.04	0.08	0.31	0.66
5.0	0.46	0.04	0.08	0.31	0.67

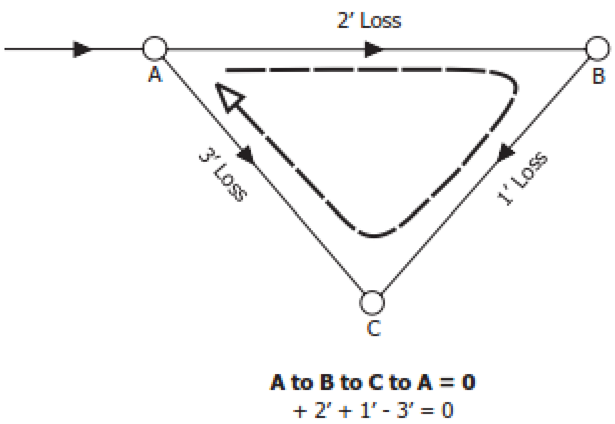


Fig. 2-8. *Sum of head losses around a loop (Walski et al. 2003).*

Conservation of mass is a key expression in pipeline engineering. It dictates that the fluid mass entering the pipe will be equal to the fluid mass exiting the pipe. In WDN modeling, all outflows are connected to the nodes or junctions. Conservation of mass can be presented as following:

$$\sum_{pipes} Q_i - U = 0 \quad (2.7)$$

where Q_i is the inflow to node in *i*-th pipe *[m^3/s]* and U is the water used at node *[m^3/s]*. To involve water storage and withdrawal from tanks, formula 2.7 can be expanded to following:

$$\sum_{pipes} Q_i - U - \frac{dS}{dt} = 0 \quad (2.8)$$

where $\frac{dS}{dT}$ is the change in storage *[m^3/s]*. (Mays 2000, Walski et al. 2003)

Continuity and energy equations must be developed for each node and each pipe in the system, respectively. For real WDNs, the equations can number in thousands. Hardy Cross method (1936) was the first systematic approach for solving these equations, however the invention of digital computers allowed more powerful techniques to be developed. These techniques set up and solve the system of equations in matrix form. (Walski et al. 2003)

2.6 Applications of water distribution modeling

Models of WDNs can be applied for various purposes depending on their features. According to Mays (2000), after the model has been verified, the general process of using it to answer to an operational or design question is following:

1. Problem definition: Identification of the operation or design problem and inclusion of the situation in the model (e.g. demands or operation decisions).
2. Model application: The problem conditions are simulated (see Section 2.4).
3. Display of results: Simulation results are presented in graphic or tabular form. If the results aren't reasonable, the process has to be restarted from problem definition.

Most water distribution models can also be used to analyze pressure piping systems carrying an incompressible, single-phase Newtonian fluid, such as industrial cooling systems or oil pipelines. However, municipal water utilities are the most common ones. Simulations of WDNs are used for a variety of purposes (Mays 2000, Walski et al. 2003):

- Long-range master planning is used to project system growth and water usage for the next 5…20 years. The capability of the WDN capacity to serve its customers must be evaluated. Long-range master planning includes rehabilitation plans, identifying problematic areas and sizing new hydraulic elements.
- Fire protection requirements can be tested by the model to see if the network can meet them. If the system doesn't provide certain pressures and flows, hydraulic elements can be sized with the model to correct the problem.
- Water quality can be simulated in addition to hydraulic simulation. Water age, source tracing, constituent concentration and disinfection by-product formation in the network can be studied in addition to studying the modifications of hydraulic operations improving water quality.
- Energy and pumping management. Pumping is one of the largest operating expenses of many water utilities next to infrastructure maintenance and repair costs. Operating characteristics and energy usage of pumps can be studied through hydraulic simulations and then optimized.
- System design. Computerized calculations enable engineers to focus on design decisions without them having to perform oppressive iterations. Models also give the designers increased confidence that the design will work compared to manual calculations. Finally, as models can be used with ease and speed, more alternatives under a wide range of conditions can be explored.

- Vulnerability studies are used to test a WDN's susceptibility to unforeseen events, such as extended drought periods or loss of power.
- Daily operational uses. Simulations can be run to determine the impact of various possible actions the operator might make. This provides the operator with better information of ones actions. Operators can also be trained to be better workers by simulating different events, which teaches them system behavior. Furthermore, emergency response planning by using a model prepares operators to real life emergencies and therefore might prevent water distribution service from being compromised. Finally, system troubleshooting can be located by a series of simulations, and then a field crew can be dispatched to the area in question.

Operations and maintenance departments will use the WDN model to make adjustments to their pump schedules, chemical feeds, filter runs, and to plan shutdowns for scheduled maintenance. Failures occurring at critical areas of the water system can be investigated with a model so that proper planning can take place before actual field work. Model studies can pinpoint locations of large water losses, water contamination and uncharted closed valves, and it helps to determine the need for surveys for issues such as leakages and water quality. (Walski et al. 2003)

The condition of the WDN's infrastructure, that is pipe condition, flow conditions and the surrounding environment, are key contributors to the quality and state of the transmitted water. WDN has a significant impact on human health and it's still concerning the worldwide water community. External contaminants can find their way into the supply system magnifying the health risks of the consumers at the receiving end. Instead of using the traditional end-product testing, a proper monitoring of water from source to consumers can be ensured through adoption of quality assurance schemes, one of which is water quality modeling. (Vairavamoorthy & Sempewo 2011)

3 BACKGROUND OF NAMIBIA AND KEETMANSHOOP

The case study location is Keetmanshoop. Keetmanshoop is a rather small town situated in Namibia, Africa. In Chapter 3, basic knowledge of Namibia and its water services important to this thesis is covered; also relevant facts of Keetmanshoop considering Keetmanshoop's water distribution are presented. Also the water services in Finland are shortly described in Section 3.2 for a comparison between a developing country and a first world country.

Namibia is located in Southern Africa along the coast line of the Atlantic Ocean. Namibia's neighbor countries are South Africa in the south, Angola and Zambia in the north, and Botswana and Zimbabwe in the east. The location of Namibia is pointed out in figure 3-1.

a)

b)

Fig. 3-1. *Location of Namibia in a a) global scale and in b) southern Africa (Geology 2007).*

The Namibian population growth rate in urban and rural areas is shown in figure 3-2. In 10 years, the total population in Namibia has increased from 2 027 026 (situation in 2005) to 2 392 370 (September 2015). As of September 2015, 1 276 090 people live in rural areas and 1 116 280 in urban areas. (Knoema 2015)

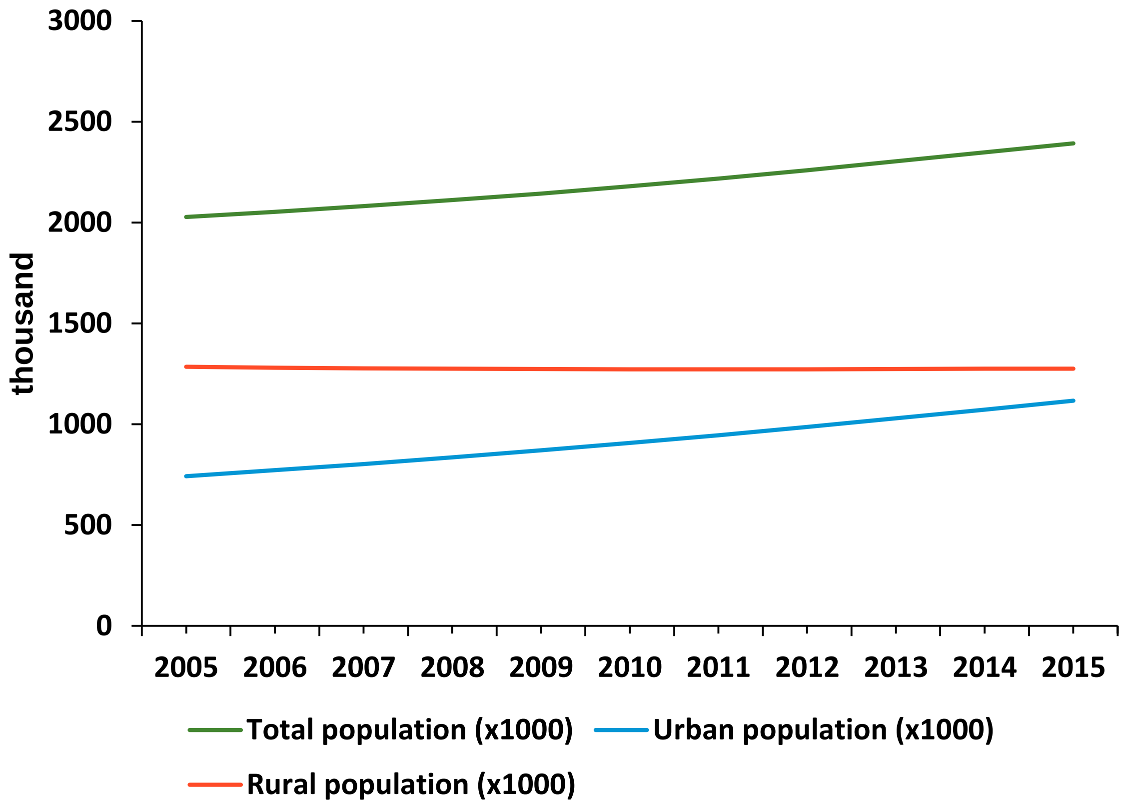


Fig. 3-2. *Urban, rural and total population growth rate in Namibia (Knoema 2015).*

Total and urban population is growing and rural population is slightly declining. Projected growth rate in Namibia is 0.87 %/year (countrymeters 2015). Namibia's total surface area is 824 268 km^2 and population density 2.90 people/km^2, which is low (United Nations Statistic Division 2012).

Namibia suffers from drought being the driest country south of the Sahara (Uhlendahl et al. 2011). 2 deserts flank Namibia; the Namib desert in the west and Kalahari desert in the east. More than 80 % of whole Namibia consists of desert or semi-desert. (Lahnsteiner & Lempert 2007) In many regions of the country, arid climate and annual fluctuating rainfall regimes form the basis of the problem of ensuring safe and continuous water supply for the Namibian people (Uhlendahl et al. 2011). Average annual rainfall and evaporation in Namibia are presented in figures 3-3 and 3-4, respectively. It is estimated that only 2 % of the rainfall ends up as surface runoff and 1 % becomes available to ground water recharge (Government of the Republic of Namibia 2010).

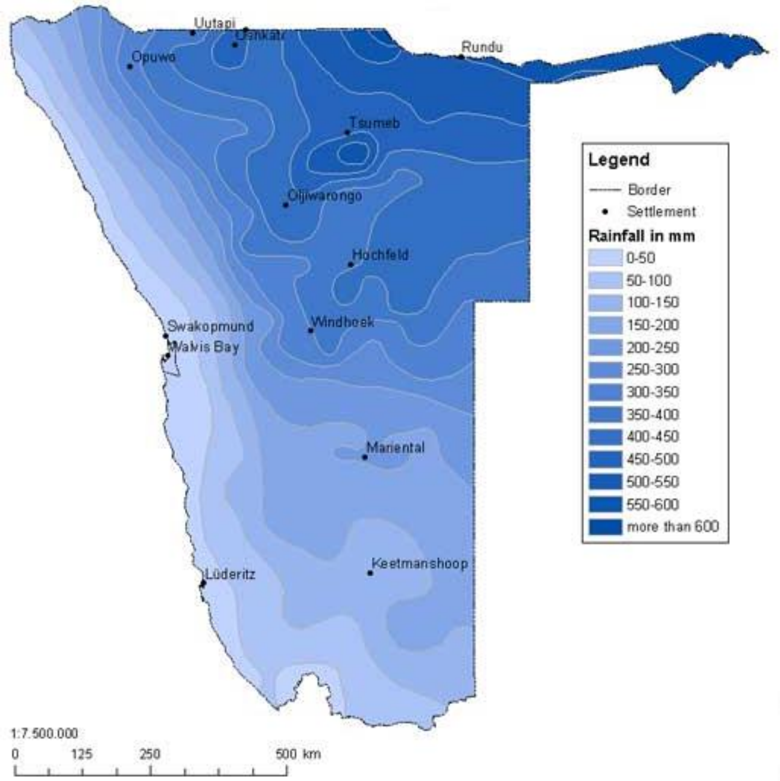


Fig. 3-3. *Average annual rainfall in Namibia (Digital Atlas of Namibia 2002).*

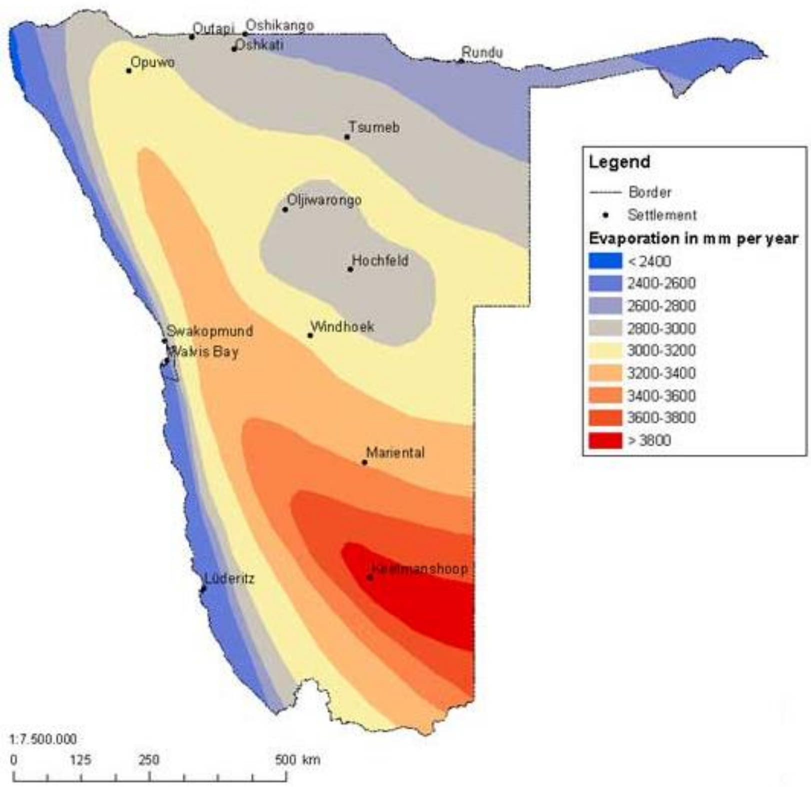


Fig. 3-4. *Average annual evaporation in Namibia (Digital Atlas of Namibia 2002).*

3.1 Water services in Namibia

According to 2015 WHO/UNICEF Water Supply Statistics offered by Knoema, total Namibian population served with improved water is currently 2 176 100, which counts to 90.96 % of total population. In this context, improved water source means it has been adequately protected from outside contamination. Improved sources of drinking water are (WHO/UNICEF Joint Monitoring Programme (JMP) for Water Supply and Sanitation 2015):

- piped water into dwelling
- piped water to yard/plot
- public tap or standpipe
- borehole or tube well
- protected dug well
- protected spring
- harvested rainwater.

However, total population served with piped water is 1 212 200, which is 50.67 % of total population. Piped water means the water is distributed to customers by service mains. (Knoema 2015) Proportions of total Namibian population served with improved water and piped water from 2005 to 2015 are presented in figure 3-5.

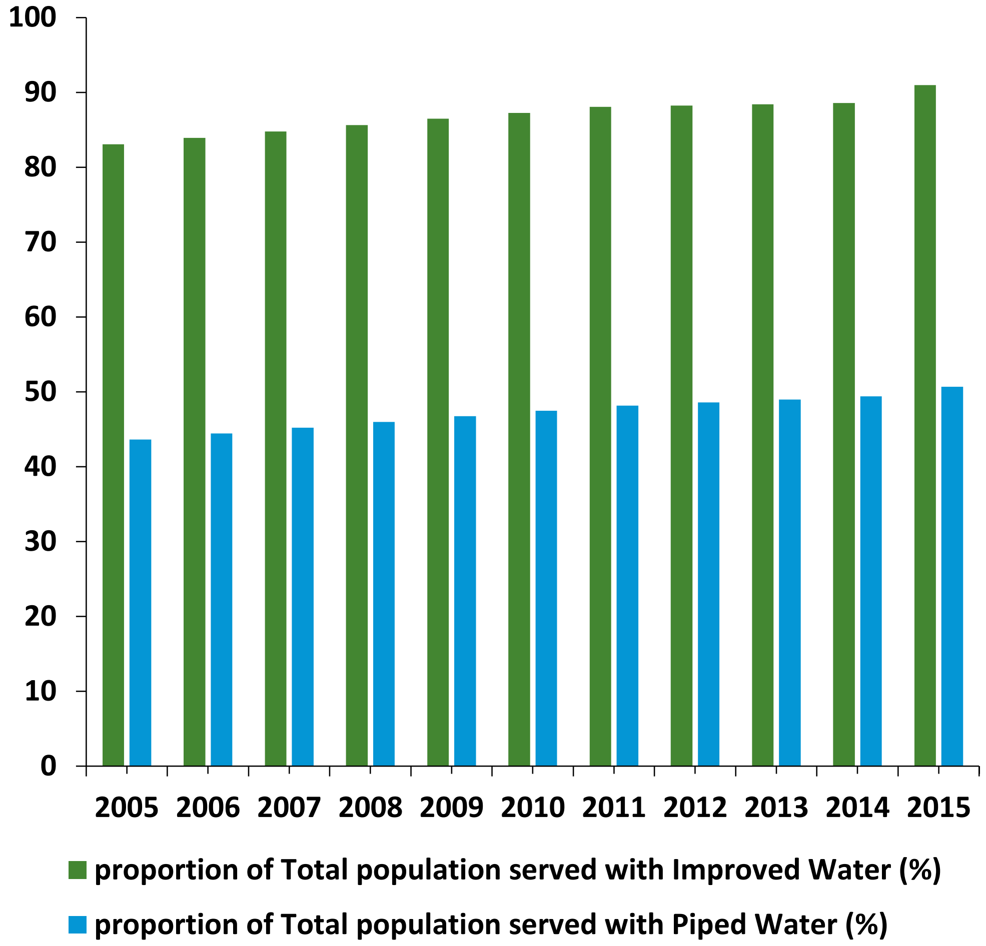


Fig. 3-5. *Proportions of total Namibian population served with improved and piped water (Knoema 2015).*

The water supply conditions differ between urban and rural areas, even though supply conditions in rural areas have improved in the past 10 years. The difference between water supply in urban and rural areas in 2015 can be seen in table 3-1.

Table 3-1. *Proportion of urban and rural population served with piped and improved water in Namibia in 2015 [%] (Knoema 2015).*

Urban, piped water	Urban, improved water	Rural, piped water	Rural, improved water
69.25	98.17	34.41	84.65

The Namibia Water Corporation Limited (NamWater) is a state owned enterprise that was established as the major bulk water supplier in 1997. It supplies bulk water to industries and municipalities and to the Directorate of Rural Water Supply in the Ministry of Agriculture, Water and Forestry. NamWater remains under the auspices of the government as the water resource manager. (Ministry of Agriculture, Water and Forestry 2008, NamWater 2015a) As is the case with other water utilities, NamWater is committed to providing its customers a reliable source of quality water at the lowest possible rates (NamWater 2015a).

In 2008, NamWater supplied approximately 66.9 Mm^3 surface and ground water to the urban water sector, mining sector, tourism facilities, rural water supply schemes and other minor consumers (Government of the Republic of Namibia 2010). NamWater derives raw water from 2 sources, 69 % from surface water and 31 % from ground water supply. The main sources of surface water supply are the Okavango River and the Orange River, the Kunene River in Angola and the dams across Namibia. NamWater has 574 boreholes in production, 19 dams, 17 treatment plants and 377 reservoirs. Together they provide the water supply source for Namibia. (Fitch Ratings 2015)

In addition to bulk water supplier company NamWater, there are other institutions responsible for water infrastructure planning and development. Department of Water Affairs and Forestry is responsible for resource management and rural water supply. Regional Authorities is responsible for small scale water supply to small communities. Some Local Authorities are responsible for water supply and reticulation. Finally, Private Sector is responsible for water supply in agriculture, mining and tourism. (Government of the Republic of Namibia 2010)

Projected water demand in Namibia is presented in table 3-2. Over the next 15 years, water demand will increase rapidly, particularly in expanding urban areas and also due to increased irrigation.

Table 3-2. *Projected water demand per sector in Namibia in Mm^3/a (Government of the Republic of Namibia 2010).*

Consumer group	2008	2015	2020	2025	2030
Urban	66.0	80.0	91.1	103.5	117.2
Rural domestic	10.3	10.6	10.9	11.1	11.4
Livestock	86.8	86.8	86.8	86.8	86.8
Irrigation	135.3	204.6	344.6	379.8	497.2
Mining	16.1	17.2	18.1	19.1	20.3
Tourism	19.6	27.5	31.9	35.2	38.9
TOTAL	**334.1**	**426.7**	**583.4**	**635.6**	**771.7**

Per capita consumption in Windhoek, the capital of Namibia with 320 000 people, was calculated as 200.9 l/capita/d (including non-residential use) in 2010 (Uhlendahl et al. 2011).

3.2 Water services in Finland

Raw water sources are highly abundant in Finland. About 10 % of the total surface area is brooks, rivers, ponds and lakes with great regional variety. Water services in Finland are one of the best in the world. (Ministry of Agriculture and Forestry 2009)

On average, annual precipitation in Finland fluctuates between 500…650 mm. In Northern Finland, precipitation is lower and in southern and central parts it is higher. (Ilmatieteen laitos 2015) Unlike in Namibia, precipitation is higher than evaporation in Finland. Average annual evaporation is 60 % in Southern Finland and usually less than 50 % in Northern Finland (Kotola & Nurminen 2003).

In Finland, municipalities are responsible for water services within their areas of jurisdiction. However, there are many households outside these territories. Basically, water services are arranged via 3 different arrangements based on population density (Pietilä et al. 2006):

- Population centres: Municipal water utilities take care of water supply (and wastewater) services.
- Rural areas: User cooperatives or partnerships take care of water supply services.
- Sparsely populated areas: People have their own wells or boreholes.

90% of Finnish population (4.8 million residents) has access to municipal water supply (Vesilaitosyhdistys 2015). Since water is supplied to almost every inhabitant in Finland, the rest 10 % covers non-municipal water supply; in addition to municipal water undertakings, there is a large number of other forms of, mainly small, organizations in operating water services. Finnish water (and wastewater) undertakings can be classified into 4 main categories based on the organizational form (Pietilä et al. 2006, Vesilaitosyhdistys 2015):

1. Partnerships. A total of 420 (in 2003), they are mainly small and serve sparsely populated areas within municipalities.
2. Cooperatives. The largest group, approximately a total of 1500 (Suomen Vesiosuuskuntien Liitto ry 2015). Provide water supply services to sparsely populated areas and countryside communities.
3. Municipal undertakings. By volume, they provide the bulk of water services. There are more than 400 municipal undertakings in Finland. In practice, there is one in almost every municipality.
4. Sharehold companies. A total of 160 (in 2003), they are mainly small and owned by the consumers. There are also some large joint-stock companies owned by neighboring municipalities.

In Finland, ground water and artificial ground water accounts for 61 % of the water distributed by waterworks. The remaining part, 39 %, is surface water.

(Vesilaitosyhdistys 2015) A typical residential per capita consumption in Finland is 90–270 l/capita/d while the average is 155 l/capita/d (Motiva 2015). Per capita consumption including non-residential use is approximately 230 l/capita/d (Vesilaitosyhdistys 2012).

According to water utilities in Finland, the portion of pipes which are in bad or very bad condition was 6 % in 2008 (Ministry of Agriculture and Forestry 2008). The main reason for the deterioration of pipe network is the poor quality of cast steel from the beginning of "crazy growth" since the 1960s. From the early 1960s to 1980, the total length of Finnish water network expanded to about fourfold. (Hellsten & Korhonen 2010) In 2008, approximately 46 % of water pipes were less than 20 years old and about 30 % were more than 30 years old (Ministry of Agriculture and Forestry 2008).

3.3 Keetmanshoop

Keetmanshoop is a town named after Johann Keetmann, a German Industrialist, in 1886 (Keetmanshoop Municipality 2015a). The location of Keetmanshoop is in figure 3-1. A typical view of Keetmanshoop's urban area is shown in figure 3-6.

Fig. 3-6. *Keetmanshoop's urban area (Salmivirta 2015).*

Keetmanshoop is the biggest urban center in the Karas Region, southern Namibia and widely acknowledged as the administrative capital of southern Namibia. Nowadays, it's attracting significant interest from the public and private investors, e.g. the University of Namibia Campus and the mass housing programme. (Keetmanshoop Municipality 2015a)

3.3.1 Population

According to Namibian National Planning Commission (2012), the population of Keetmanshoop was 26 000 in 2011, comprising of 7 100 residents in rural and 18 900 in urban areas. However, Keetmanshoop Municipality (2015a) says the population in Keetmanshoop is roughly 30 000 inhabitants.

Some sources claim the population to be somewhere between 15 000 to 20 000 inhabitants. Due to harsh living conditions in Keetmanshoop's rural areas and slums, collecting a census can be an oppressive task. Additionally, some sources might not include rural inhabitants to their census.

3.3.2 Hydrology and water supply to the town

Keetmanshoop is a dry town. According to Digital Atlas of Namibia (2002), the annual precipitation is 100–150 mm while the annual evaporation is higher than 3800 mm. However, the town still has access to abundant water supply, as the municipality's raw water source is Lowen River with a large drainage basin gathering sufficient volume of water (Aalto 2014, NamWater 2015b).

NamWater is also the bulk water supplier in Keetmanshoop. NamWater abstracts water from Lowen River by using a surface water dam called Naute Dam. From Naute Dam, raw water is pumped a distance of 1800 m to NamWater's purification plant. The purified water is then pumped a total distance of 43 km to NamWater's 3 clear water concrete reservoirs in Keetmanshoop. The purified water is still further chlorinated in these reservoirs. From NamWater reservoirs, water flows by gravity through 2 sales meters, one of which is NamWater's and the other one is Keetmanshoop Municipality's. (NamWater 2015c) After the water meters, Keetmanshoop Municipality is responsible for the water and its distribution to the town. The distribution doesn't concern inhabitants living in rural areas of Keetmanshoop.

4 MATERIALS AND METHODS

The field work period in Keetmanshoop was June 1st – July 31st, 2015. In Chapter 4, the study area, Keetmanshoop and its WDN, is first described. In Section 4.2, a list of interviewees for this thesis is presented. Section 4.3 is where other research methods for each subject are described.

The modeling of Keetmanshoop's WDN is also an objective in this thesis. The modeling process is depicted in Section 4.4.

4.1 Description of the study area

Keetmanshoop's WDN prior to any changes to its operation during the field work period is described in Section 4.1.2. Keetmanshoop's WDN is presented in Appendix 1.

Keetmanshoop's urban area is divided into 5 suburbs (see Appendix 2 for the layout): Westdene, Noordhoek, Kronlein, Tseiblaagte and Town Area. Westdene is located in the west and is an area of middle and high income people and also where most of the white inhabitants live. Noordhoek is located in the north and is an area of middle income people. Kronlein is in the northeast having low to middle income residents. Tseiblaagte is an area of black and low income people and it's located in the southeast. Town Area mostly comprises of buildings related to service sector. (Löppönen 2011) The industrial area is also part of Town Area; it's located in the southernmost and the lowest area of Keetmanshoop.

4.1.1 Organization of Keetmanshoop Municipality

Keetmanshoop Municipality is governed by a Municipal Council, which consists of 7 council members who are elected by the residents of Keetmanshoop for a period of 5 years on the basis of proposed political party list. Current members were elected in December 2010 and shall govern until December 2015. (Keetmanshoop Municipality 2015b)

The municipality's organization has 4 departments, which are (Keetmanshoop Municipality 2015a):

- Corporate Affairs and Human Resources
- Infrastructure and Technical Services
- Local Economic Development
- Financial Services.

Chief Executive Officer (CEO) office is the highest position in the municipality and his or her office is above these departments. The organizational chart of Keetmanshoop Municipality's Department of Infrastructure and Technical Services is presented in figure 4-1.

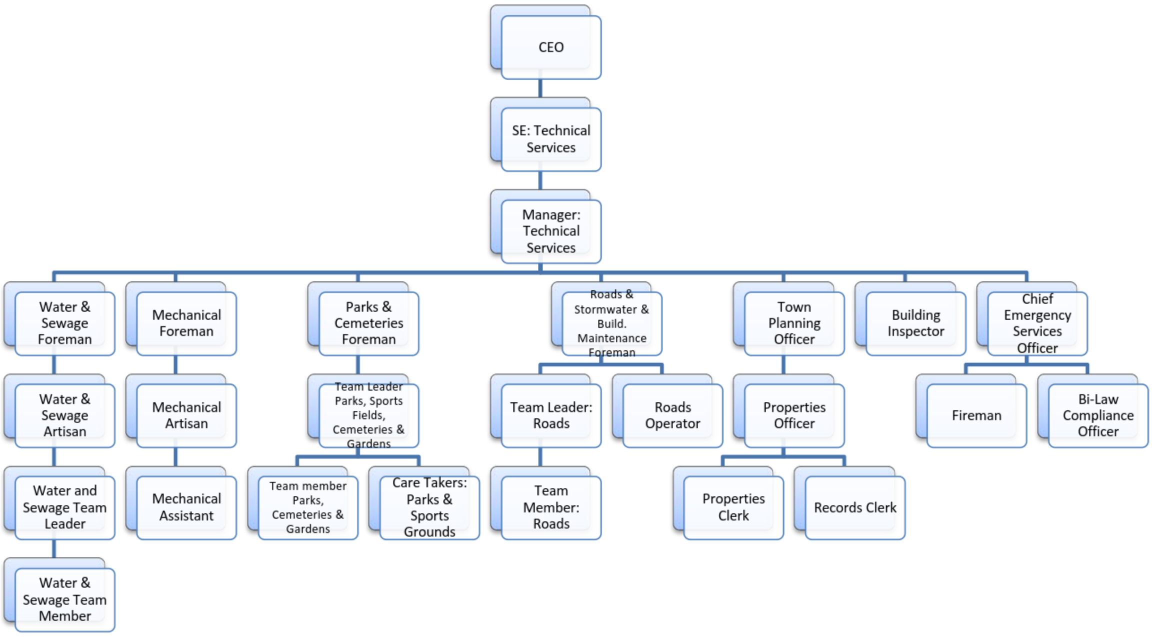


Fig. 4-1. *Department of Infrastructure and Technical Services, organizational chart (Easy HR Consultancy 2014).*

Water and sewerage sectors are not separated as they operate under the sector of Water & Sewage. The organization of Water & Sewage is described in Section 5.1.

4.1.2 Water distribution network in Keetmanshoop

NamWater supplies water to Keetmanshoop's WDN. NamWater has 3 clear water concrete reservoirs in Keetmanshoop connected in series, from which the water flows by gravity to the network. NamWater operates the reservoirs in the following way: The incoming water to NamWater reservoirs flows through all of them for proper mixing. Only one reservoir has discharge at a time, and under normal operation the water level fluctuates between 80 and 99 %. There is always at least one and a half reservoirs of extra storage for events of varying demands. (Calla Bonthuys, NamWater's Super Intendent 2015) NamWater reservoirs are located in the west of Keetmanshoop at the highest ground of the WDN.

From NamWater reservoirs, the water flows by gravity through 2 sales meters, one of which is NamWater's and the other one is Keetmanshoop Municipality's. Both meters give the same readings, and the meter readings are reliable (Bonthuys 2015). Keetmanshoop Municipality is responsible for the distribution once the water has passed

NamWater's meter. After the meters, the water flows to Donkie Drai reservoir in the north, Oxpass Hill reservoir in the southwest and to northern Westdene. The municipality's reservoirs are mainly used for balancing storage. From these reservoirs, the water flows to the consumers by gravity.

The length of the distribution mains is approximately 160 km (Aalto 2014). The construction of Keetmanshoop's distribution mains begun in the early 1940s. By the 1960s, all the suburbs had their water infrastructure. AC was the only pipe material that was used from the 1940s to 2004. Since 2004, only uPVC (unplasticized polyvinyl chloride) pipes have been installed to the network. (Löppönen 2011) As of July 2015, approximately 80 % of the mains are still AC (Nashima 2015). The pipe sizes range from 40 to 250 mm and are mainly between 50 and 200 mm (Tuovinen 2014).

Boxall et al. (2007) report that pipe age indicates the time the pipe has been in operation and exposed to loading and the surrounding environment, which then increases the probability of bursting. In addition, as there is generally no pressure management in the network of Keetmanshoop, there are areas where service pressure criteria are neglected. These are significant factors in terms of Keetmanshoop's WDN's frequency of pipe bursts and the level of NRW.

The number of gate valves in the network is insufficient. Also, the condition of the existing gate valves is unclear. Furthermore, there aren't any other appurtenances for flow control, such as air release valves, in the network.

There are no pumps in the municipality's network since all the water flows by gravity. Only Keetmanshoop Airport operates their own pumping system and they have their own supply pipeline to get water from Keetmanshoop Municipality.

4.2 Semi-structured interviews

Semi-structured interviews were conducted as a research method in Keetmanshoop and at Lempäälä Waterworks. A semi-structured interview is an open conversation between the interviewer and the interviewee with a framework of themes to be explored (Fylan 2005).

The interviews are summarized in table 4-1. An interview with Löppönen took place before the field work; all the other interviews were had during the field work.

Table 4-1. *Summary of semi-structured interviews.*

Date	Subject	Person and title	Company
23.4.2015	Water distribution network	Arto Löppönen, Water Network Engineer	Lempäälä Waterworks
13.6.2015 + various other	Pressure management + various other	Arto Hietanen, Water Engineer & Project Manager	Kangasala Water
13.6.2015	Pressure management	Damas Mashauri, Professor	Polytechnic of Namibia
22.6.2015	Night flow + various other	Calla Bonthuys, Super Intendent	NamWater
14.7.2015	Organization of the water sector	Lasarus Neshuku, Water & Sewage Team Leader	Keetmanshoop Municipality
21.7.2015	Fire hydrants	George Joseph, Fire Brigade Chief	Keetmanshoop Municipality
22.7.2015	Mass housing	Jegg Christiaan, Strategic Executive	Keetmanshoop Municipality
23.7.2015	Water meters, water tariffs	Sandra Poulton, Finance Officer	Keetmanshoop Municipality
23.7.2015	Water meters	Lettie Swarz, Finance Officer	Keetmanshoop Municipality
23.7.2015 + various other	Pressure measurements + various other	Samuel Nashima, Senior Manager	Keetmanshoop Municipality

4.3 Other research and work methods

Besides semi-structured interviews, other research methods were performed in Keetmanshoop. They are described in Sections 4.3.1–4.3.10.

Keetmanshoop's WDN was modeled before and after the field work. Data gathered during the field work was utilized in the modeling process. The work methods of the model's construction are described in Section 4.4.

4.3.1 Organization and work habits of the water sector

The organization and the work habits of Keetmanshoop Municipality's water sector were studied during the field work. The results are based on observations and conversations with Hietanen and Keetmanshoop Municipality's staff.

Furthermore, an interview with Water & Sewage Team Leader Lasarus Neshuku took place on July 14th, 2015. The interview only concerned the organizational structure of the water sector.

4.3.2 Fire hydrants, gate valves and one-way valves

Approximately 10 gate valves were searched and tested in Westdene during the field work. Visual perceptions were made from stolen valve and fire hydrant lids all over Keetmanshoop.

Further information about fire hydrants was acquired by interviewing Keetmanshoop Municipality's Fire Brigade Chief George Joseph on July 21st, 2015. Finally, conversations of a one-way valve that was being installed to the WDN were had with the municipality's staff.

4.3.3 Pressure measurements

Keetmanshoop's WDN suffers from low and high pressures. For example, this can be seen in Löppönen's (2011) and Tuovinen's (2014) theses. The water pressure in households in Namibia should always be above 200 kPa (Nashima 2015).

Pressure readings were taken with a pressure gauge in Westdene area on July17th, 2015 during the morning and day (figure 4-2). The readings were measured inside the residents' properties from water taps. To see how the pressure accumulates according to the elevation difference from Oxpass Hill reservoir, the elevations of the measurement points were estimated from the FCGnet file of Keetmanshoop's WDN (see Section 4.4).

Fig. 4-2. *Taking pressure readings in Westdene (Salmivirta 2015).*

While the measuring took place, it was made sure that Oxpass Hill reservoir had approximately maximum water level. This action made sure there were no differences in the static head caused by Oxpass Hill reservoir water level. Also, the inlet from NamWater reservoirs to Oxpass Hill reservoir was closed. The pressure gauge was left to the tap for a while to make sure of the valid result (e.g. unknown water usage from the same service line).

There were customer complaints considering abnormally low pressures from erf 1000 during the field work. For this, the pressures in this erf were read 9 times at different days and day times. During this measuring, the inlet to Oxpass Hill reservoir was closed and opened as the inlet's influence to the pressure was studied.

4.3.4 Pipe bursts and leakages

In this context, pipe bursts mean significant and visible water leaks occurring from pipes breaking or joint failures. Pipe leakages on the other hand mean any slight detectable leakages in the pipe network or fire hydrant leakages.

The amount of pipe bursts and leakages from January 2015 to June 2015 were manually counted from Keetmanshoop Municipality's bookkeeping. Due to the bookkeeping's practice, pipe bursts and pipe leakages were counted separately.

4.3.5 Water consumption and night flow

The volume of water Keetmanshoop Municipality distributed to customers in the financial year of 2014/2015 (1.7.2014–30.6.2015) was acquired from the municipality's accounting. Also, a daily variation graph of the municipality's water use from one-week period in July 2015 was acquired from NamWater.

Night flow was shortly investigated based on conversations with the municipality's staff and Hietanen and an interview with Bonthuys on June 22nd, 2015. In addition, a visit to Keetmanshoop Hospital was made on June 5th, 2015.

4.3.6 Biggest water consumers

The biggest water consumers in Keetmanshoop have been previously studied by Tuovinen (2014) and Aalto (2014). In their studies, private apartments have been left out as it's unnecessary to point out personal uses. This is also the practice in this thesis.

A list of Keetmanshoop's biggest water consumers was received from the municipality's accounting on October 28th, 2015. The recording period of the list was the financial year of 2014/2015 and the 3 previous financial years.

4.3.7 Water meters and prepaid water meters

Keetmanshoop Municipality’s Finance Officers Sandra Poulton and Lettie Swarz were interviewed about the water meters and their reading policies in Keetmanshoop on July 23rd, 2015. Prepaid water meters were also shortly covered.

Municipality’s debtor’s statistics from 2015 were used for the amount of water meter readings taken and their readings. The recording period was from the beginning of January to the end of May.

4.3.8 Non-revenue water

An annual water balance is generally used to evaluate NRW and its components. International Water Association Best Practice Standard for determining water balance is shown in figure 4-3. (Lambert 2003)

<table>
<tr><td rowspan="9">System Input Volume (corrected for known errors)</td><td rowspan="4">Authorised Consumption</td><td rowspan="2">Billed Authorised Consumption</td><td>Billed Metered Consumption (including water exported)</td><td rowspan="2">Revenue Water</td></tr>
<tr><td>Billed Unmetered Consumption</td></tr>
<tr><td rowspan="2">Unbilled Authorised Consumption</td><td>Unbilled Metered Consumption</td><td rowspan="7">Non-Revenue Water (NRW)</td></tr>
<tr><td>Unbilled Unmetered Consumption</td></tr>
<tr><td rowspan="5">Water Losses</td><td rowspan="2">Apparent Losses</td><td>Unauthorised Consumption</td></tr>
<tr><td>Customer Metering Inaccuracies</td></tr>
<tr><td rowspan="3">Real Losses</td><td>Leakage on Transmission and/or Distribution Mains</td></tr>
<tr><td>Leakage and Overflows at Utility’s Storage Tanks</td></tr>
<tr><td>Leakage on Service Connections up to point of Customer metering</td></tr>
</table>

Fig. 4-3. *Water balance of WDN (Lambert 2003).*

System input volume is the volume of water input to a distribution or transmission system. It comprises of authorized consumption and water losses. Authorized consumption is the volume of metered and unmetered consumption. Water losses consist of apparent and real losses: apparent losses are due to unauthorized consumption and customer metering inaccuracies, while real losses are caused by leakages on transmission and distribution mains and service connections up to the point of customer metering, and leakages and overflows at storage tanks. (Lambert 2003)

NRW is the difference between system input volume and billed authorized consumption and it’s counted from a period of 12 months. NRW consists of water losses and unbilled authorized consumption. NRW is a good indicator of how efficient the WDN is and how much water gets lost on the way to customer. (Lambert 2003) Usually, the level of NRW is presented in percentages.

The NRW in Keetmanshoop has been studied in previous theses, such as Aalto's (2014) and Tuovinen's (2014). In these theses, the level of NRW has been found high.

Pipe leakages can be a major contributor to real losses and therefore to NRW. Pressure-leakage relationship can be mathematically represented in the following way:

$$Q = CH^{\alpha} \quad (4.1)$$

where Q is the leak flow rate *[m³/s]*, H is the pressure head in the pipe *[m]*, C is a constant leakage coefficient and α is a constant leakage exponent. The leakage coefficient provides a measure of the factors, such as the hole's diameter and the secondary loss coefficient, to influence the leak in way that is not affected by the flow rate. The leakage exponent provides a measure of the sensitiveness of the leakage rate to the pressure in the pipe. (van Zyl 2004) Ultimately, this equation indicates that the higher the pressure, the bigger the real losses and the level of NRW.

The following equation is known as the orifice equation:

$$Q = CH^{0.5} \quad (4.2)$$

where α = 0.5. Equation 4.2 represents conventional theory, which says that the rate of leakage through a fixed diameter hole in a pipe is proportional to the square root of the pressure. It indicates that the leakage rate is relatively insensitive to pressure; doubling the pressure increases the leakage rate by 40 %. However, measurements in real WDNs have showed that leakage is actually more sensitive to pressure than the values given by equation 4.2. It has been empirically proved that the leakage exponent can be as high as 2.5, which will then increase the leakage rate substantially. Doubling the pressure in a pipe will increase the leakage rate by 570 % for a leakage exponent of 2.5. (van Zyl 2004)

According to Hietanen (2015), Kangasala Water uses a simplified method for estimating leak volumes occurring from pipe bursts. The estimation can be manually counted in the following way: First, it's assumed that the leakage area equals the inner diameter of the leaking pipe. It's also assumed that the velocity of the water is 1.0 m/s in 1.0 bar pressure. Then, the velocity of the leaking water can be calculated.

$$v_2 = v_1 * p_2/p_1 \quad (4.3)$$

where v_1 is 1.0 m/s and p_1 is 1.0 bar, v_2 is the velocity of the leaking water *[m/s]* and p_2 is the pressure in the leaking pipe *[bar]*. By using equation 2.2, the leak flow rate can be determined.

Next, it is assumed that the pressure in the leaking pipe drops by 50 % after 15 minutes (900 s), but after that the pressure is constant. Then the leak volume can be calculated:

$$V = Qt_1 + 0.5Qt_2 \tag{4.4}$$

where V is the leak volume *[m³]*, Q is the leak flow rate, t_1 is the leak time in the first 15 minutes *[s]* and t_2 is the leak time after the first 15 minutes *[s]*. Equations 4.3 and 4.4 are only used for estimations.

The level of NRW in Keetmanshoop in the invoicing period of 2014/2015 was manually counted from the municipality's accounting. The formula used was (Hambabi 2015):

$$NRW[\%] = \frac{Q_{in} - Q_{revenue}}{Q_{in}} * 100\% \tag{4.5}$$

where Q_{in} is the annual system input volume $[m^3]$ and $Q_{revenue}$ is the annual billed volume $[m^3]$.

4.3.9 Municipality's reservoirs

During the field work Oxpass Hill reservoir was visited regularly. Also, conversations were had with the municipality's workers and Hietanen. Oxpass Hill reservoir water level and overflows were checked almost daily during the field work, and July 9–12, 2015 they were monitored 2–4 times a day. The reservoir volume was estimated by measuring its external dimensions and the roof and wall thicknesses with a measuring tape on July 1st, 2015. The volume of a cylinder shaped reservoir was estimated with the equation (Tekniikan Kaavasto 2010):

$$V = \pi r^2 h \tag{4.6}$$

where V is the estimated volume of the reservoir $[m^3]$, r is the inner radius of the reservoir *[m]* and h is the maximum water level in the reservoir *[m]*.

The overall condition of the reservoir and the surrounding environment were visually estimated during the field work. Finally, due to insufficient pressure levels in western Westdene, the nearby pipes of the reservoir were dug out with a shovel and an excavator July 24–28, 2015 to see how the pipes were connected.

Donkie Drai reservoir was visited a few times during the field work. In addition, the municipality's staff was talked to as was Hietanen. On July 1st, 2015, Donkie Drai reservoir volume was estimated the same way as Oxpass Hill's (equation 4.6). Donkie Drai reservoir's overall condition and the surrounding environment were visually estimated. Lastly, during 8 days in March 2015, Hietanen measured Donkie Drai reservoir water levels with a yardstick.

4.3.10 Finance of the water sector

In theory, the municipality's water sector is financially independent and not dependent on the other income of the municipality (Jegg Christiaan, Keetmanshoop Municipality's Strategic Executive, Department of Local Economic Development 2015). Keetmanshoop Municipality's water sector's budget in 2014/2015 and 2015/2016 and the actual income and expenditure in 2014/2015 were collected from the municipality's accounting. The municipality's financial year is 1.7–30.6.

Keetmanshoop Municipality's water sector has had financial struggles over the years to cover the expenses (Tuovinen 2014). At the same time, their tariffs have increased. However, NamWater raises their tariffs first and the municipality follows later (Christiaan 2015). Poulton was interviewed on July 23rd, 2015 about the rate of water tariffs in the past 10 years.

Mass housing (National Mass Housing Project) is a Namibian governmental initiative to deliver 185 000 low-income houses to Namibia by 2030. The programme aims to create approximately 2.5 jobs for every house constructed. An estimation, 10 278 houses on average will be constructed on an annual basis which creates close to 25 700 jobs. (New Era 2014a) Mass housing is also a big project in Keetmanshoop that affects the pipe network and the finance of the municipality. An interview with Christiaan about the project in Keetmanshoop took place on July 22nd, 2015. Moreover, a short literature review was conducted.

4.4 Construction of the water distribution network model

The WDN model was constructed with Bentley WaterCAD V8i, QGIS and FCG's software FCGnet with the assistance from FCG Team Leader Markus Sunela and Design Engineer Kalervo Aho in April, May and September 2015. First, all necessary elements (pipes, junctions, reservoirs and tanks) were drawn in WaterCAD using an AutoCAD DWG file as a background layer. Hazen-Williams formula for the calculation of unit head losses was adopted to the pipes with the roughness coefficient *k* (C-factor, see Section 2.5) being automatically selected by the software. The DWG file was a Keetmanshoop water distribution map from 2012 containing all the needed elements. Later on, a newer DWG file from November 2014 was used for updating pipe connections and properties in WaterCAD.

Some skeletonization was used in the modeling process. Pipes with diameters less than 75 mm were not drawn if they didn't form complete loops.

After the elements were drawn, junction demands and elevations were produced in the following way. All the erf numbers from the 2012 DWG file were exported to a GIS application called QGIS: the center coordinates of the erf numbers were exported to QGIS as geocoded demands. Using QGIS, all street and road numbers that were also part of the exportation were manually deleted. Now only the erven's center points existed in their right locations in Keetmanshoop's 5 suburbs. The water consumptions of the suburbs for the model were based on Löppönen's (2011) master's thesis (table 4-2).

Table 4-2. *Annual consumptions in each suburb in 2011 (Löppönen 2011).*

Suburb	Annual consumption [m^3]
Town Area	270 000
Westdene	220 000
Noordhoek	110 000
Kronlein	190 000
Tseiblaagte	190 000

Using QGIS, annual consumptions were divided equally between the erven on each suburb distinctively. Now each erf had its demand in QGIS.

The 2012 DWG file was imported to QGIS as a DXF file and was set to UTM zone 34N coordinate system, since this coordinate system was geometrically the most accurate one for the DXF file. Now the coordinates of the DXF file were known. The WaterCAD file was exported as an EPANET file to QGIS to its correct location with the right coordinates. The elevation data of Keetmanshoop was acquired from a QGIS elevation plugin. Still using QGIS, discrete elevation points were pointed in whole Keetmanshoop area and then interpolated into one continuous elevation distribution area comprising the whole area of Keetmanshoop. This continuous elevation data was linked to the EPANET file's junctions.

Next, previously calculated erf demands were linked to the nearest junctions originating from the WaterCAD file. Now every junction had the exact and individual elevation and demand in QGIS software.

The QGIS file was exported to FCGnet. In FCGnet, NRW was taken into consideration for acquiring junction emitter coefficients to each junction. The NRW was acquired from Hambabi's (2015) thesis. The level of NRW was calculated with equation 4.5 where the annual billed volume = 1 314 490 m^3 and the annual system input volume = 2 073 903 m^3. Therefore NRW = 36.6 %.

The measurements for NamWater reservoirs, which are technically tanks, were acquired from Bonthuys (2015). However, the 3 NamWater tanks were simplified to operate as an infinite water source; hence they were modeled as one reservoir (table 4-3). Municipality's own reservoirs, Oxpass Hill and Donkie Drai reservoirs, were physically measured with a measuring tape for acquiring their physical sizes and estimated water levels (operating ranges). Hietanen's research (see Section. 5.9.2) in Donkie Drai water levels was also used in water level estimations. Finally, the municipality's reservoirs were modeled as tanks since they operate as ones (table 4-4). However, the minimum elevations for the municipality's reservoirs were modeled as the same value as the base, since in theory the reservoirs could get empty and also because the simulation would run more smoothly.

Table 4-3. *NamWater tank sizes and the simplified reservoir elevation for the model (Bonthuys 2015).*

Diameter (inner) [m]	Height (inner) [m]	Tank volume [m^3]	Reservoir elevation [m]
24.81	6.82	3 296	1 052.00

Table 4-4. *Municipality's tanks' (reservoirs') operating ranges for the model [m above sea level].*

Reservoir	Base	Minimum	Initial	Maximum
Oxpass Hill	1 035.00	1 035.00 (1 038.00)	1 038.35	1 038.70
Donkie Drai	1 034.00	1 034.00 (1 036.70)	1 037.00	1 037.50

Next, an hourly hydraulic pattern for the whole network was produced. The pattern was an average hourly pattern of a Finnish municipality resembling the amount of water consumers and water consumption of Keetmanshoop. Even though the water use fluctuates differently in Keetmanshoop and Finland, FCGnet can simulate maximum and minimum events (e.g. pressures) regardless of day time.

Emitter coefficients in FCGnet were modeled next. By knowing the level of NRW of the whole network (which is the same 36.6 % in every suburb) and pipe sizes, lengths and pressures in each area (suburb in this case), FCGnet can calculate emitter coefficients and leaks in each junction in the following way:

$$C_L = \frac{NRW}{\sum_{i=1}^{n_{pipe}}(L_{tot} * D_{avg}) * \frac{\sum_{i=1}^{n_{node}} p_{avg}}{n_{nodes}}} \quad (4.2)$$

$$E = \frac{\sum_{i=1}^{n_{pipe}}(L_i * D_i)}{2} * C_L \quad (4.3)$$

$$Q_{leak} = E * p \quad (4.4)$$

where C_L is the discharge coefficient of the area *[l/(s*km*cm*m)]*, NRW is the annual non-revenue water in the area *[l/s]*, L_{tot} is the total pipe length of the area *[km]*, D_{avg} is the average pipe diameter of the area *[cm]*, p_{avg} is the average pressure head in the area *[m]*, E is the emitter coefficient distinct for each junction *[l/(s*m)]*, L_i and D_i are the pipe length and diameter, Q_{leak} is the junction leak *[l/s]* and p is the pressure head of the junction *[m]*. Now all the necessary data for all the elements considering the simulation was input to FCGnet. The simulations were computed with FCGnet.

5 RESULTS

The results of the field work in Keetmanshoop during June and July 2015 are presented in this chapter. The results are mainly based on the investigations during the field work (see Section 4.3). However the list of the biggest water consumers was received on October 2015 and Hietanen's previous research results of Donkie Drai reservoir were used.

The WDN model was mostly constructed before the field work. The simulations were run in September and October 2015.

5.1 Organization and work habits of the water sector

The organization of Water & Sewage is shown in figure 4-1. Water and sewerage sectors in Keetmanshoop Municipality are joint under one sector, Water & Sewage. In practice it means that the same Team Members and Team Leaders work simultaneously with water distribution and sewerage, and the Water & Sewage Foreman manages both services. During working hours, there are 3 teams working. The 4^{th} team is a Standby Team working after hours. One Water & Sewage Team consists of 5 or 6 Team Members, of which 1 or 2 are Team Leaders. (Neshuku 2015) The Water & Sewage Foreman is the head of Water & Sewage Teams and the Senior Manager of Department of Infrastructure and Technical Services is above the Foreman's post.

According to the observations and conversations with Hietanen (2015) and Keetmanshoop Municipality's staff, the organization type for Water & Sewage causes difficulties along with under manning and poor work habits. Daily pipe bursts and sewerage overflows keep the teams over occupied. During many occasions, all the teams were working with pipe bursts and pipe installations, but at the same time there were serious sewerage overflows taking place. There is no special know-how for water nor sewage as the teams are not separated and properly educated. The plumbers' ability to read water distribution maps is inadequate. Teams might even show up to work late and sometimes their response time to pipe bursts is slow. This is also due to poor communication between the workers and the foremen. Also, sometimes old broken pipes are left at site after new pipes have been installed (figure 5-1). Finally, uPVC pipes are stored outdoors exposed to sunlight. This can cause deterioration of the pipes; discoloration and decreased impact resistance (JM Eagle 2009).

Fig. 5-1. *Old AC pipes left at site after disposal (Salmivirta 2015).*

Standby Teams work at least15-hour shifts outside working hours as well as weekends for a month at a time. Sometimes they have to additionally do standard working hours. They are on call and appear at site when necessary. Standby Team is alerted when they get a call from Standby Office. Standby Teams are usually heavily over occupied; overtime hours can be exceeded in less than 2 weeks.

The organizational chart of Keetmanshoop Municipality's Department of Infrastructure and Technical Services is less than 12 months old (by September 2015), so the Water & Sewage Artisan is still vacant and so is the Manager of Technical Services. These posts are filled in the future.

5.2 Fire hydrants, gate valves and one-way valves

The gate valves that were investigated in Westdene are in bad condition. Approximately 50 % (5 out of the total 10) of them can't be operated since they are stuck and fragile; this is because they are as old as the network (> 50 years) and haven't been turned on-off enough times. Moreover, their handednesses are regularly unsure and about 50 % of them are wrongly marked to the distribution maps, so they are difficult to find, and even some valves marked on the maps don't exist on the field. Finally, the investigated valves have no visible shafts so no visual conclusion about the valve position can be made: this combined with the valves being non-operational makes it hard for the plumbers to know if there is water flowing through them and which the flow direction is. According to the municipality's staff, same kind of non-operational and problematic valves also exist in other Keetmanshoop's suburbs.

There were numerous incidents all over Keetmanshoop where fire hydrant and valve lids were stolen. Since the lid material is typically cast iron, they can be sold.

Fire hydrants in Keetmanshoop are the responsibility of the Fire Brigade. The Fire Brigade isn't sure about the number of fire hydrants in Keetmanshoop. Pipe bursts and installations take place often, and fire hydrants are removed if they are on the way of new pipes during the installation work. Water & Sewage Teams do install new fire hydrants when this happens, but the flow of information to the Fire Brigade is lousy. (Joseph 2015)

The majority of the fire hydrants work properly, and the ones that don't are replaced: usually 4 or 5 fire hydrants break in a year. The condition of the fire hydrants is checked monthly by the Fire Brigade staff. Fire hydrants are installed below and above ground, but some of the above ground fire hydrants stand inside the consumers' premises due to illegal fencing. Moreover, there are approximately 5–10 fire hydrants in Keetmanshoop that can't be properly accessed because they are covered with fences or garbage. Also in several cases valuable parts of the fire hydrants were stolen, e.g. metal parts. (Joseph 2015)

According to the municipality's staff, a non-return valve was installed on July 6th, 2015 to the pipe connecting Westdene and Town Area. The valve doesn't let any water flow backwards from Town Area to Westdene as the back flow can increase the risk of Oxpass Hill reservoir to overflow (see Section 5.9.1).

5.3 Pressure readings

Pressure readings were taken with a pressure gauge in Westdene on July17th, 2015 (table 5-1). The readings were measured inside the residents' properties from water taps. The elevation differences between Oxpass Hill reservoir (maximum water level) and the measurement points were estimated from Keetmanshoop's WDN model to see how the pressure accumulates. The measured pressures are marked on the map in figure 5-2.

Table 5-1. *Pressure readings from the residents' water taps in Westdene.*

Erf	Time	P [kPa]	Elevation [m]	$\Delta_{\text{elevation, Oxpass Hill}}$ [m]
1000	10:25	50	1020	18.7
1016	10:17	70	1012	26.7
1166	9:40	150	1010	28.7
1115	9:45	150	1010	28.7
1132	10:05	150	1010	28.7
801	11:34	150	1010	28.7
1197	9:50	170	1009	29.7
492	12:05	200	1010	28.7
580	11:57	255	1002	36.7
673	12:10	260	1012	26.7
600	11:45	260	1000	38.7
430	14:25	260	1000	38.7
529	14:16	300	995	43.7
461	14:31	340	991	47.7

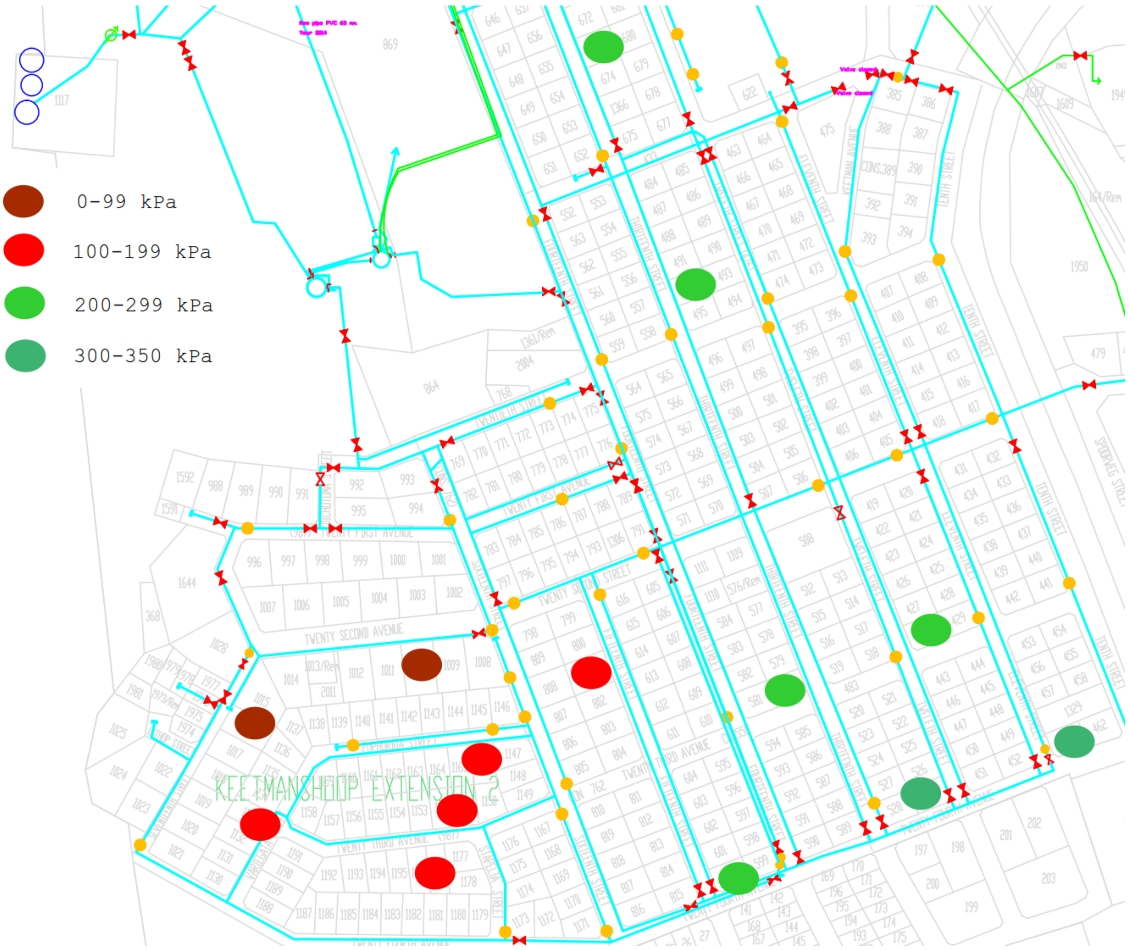


Fig. 5-2. *Pressure readings from the households in Westdene area. Colored spheres represent the pressure.*

The readings are low in western Westdene. The elevation differences between Oxpass Hill reservoir and the measurement points are 18.7 meters at minimum (approximation), yet the lowest readings are 50 and 70 kPa. The pressure still accumulates according to the elevation differences, though it’s inconsistent: the pressure in erf 673 is 260 kPa which is high when comparing it’s elevation to the other measurement points. The relatively high pressure in erf 673, which is the northernmost measurement point, can also be due to the static head from NamWater reservoirs (see figure 5-13).

During the measuring the inlet to Oxpass Hill reservoir was closed as it’s normally operated. The majority of inhabitants was at work or at school, meaning the pressure was high in the network. There has to be something interfering with the gravitational flow that diminishes the pressure in western Westdene. One chance is that in those particular erven that suffer from the low pressure, the distribution mains could only be 40 mm uPVC pipes for a long distance. This would mean the distribution mains there wouldn’t have enough water carrying capacity. However, these hearsays were not confirmed.

The pressure in eastern and in the southernmost Westdene are decent or better with 200 kPa being the lowest pressure and all the other erven achieving 250 kPa at minimum. In these measurement points, the pressure increases according to the elevation difference from Oxpass Hill reservoir.

There were customer complaints considering unusually low pressures from erf 1000 (figure 5-3) during the field work. Therefore, the pressures in this erf were read 9 times from the resident’s water tap at different days and day times (table 5-2). During the measuring the inlet to Oxpass Hill reservoir was closed and opened, as the inlet’s influence to the pressure in erf 1000 was investigated.

Table 5-2. *Pressure readings in erf 1000.*

Date and time	P[kPa]
17.7.2015 10:25	50
20.7.2015 16:30	290
21.7.2015 10:20	60
21.7.2015 16:30	330
22.7.2015 12:30	70
22.7.2015 14:00	70
22.7.2015 16:30	270
24.7.2015 11:45	150
24.7.2015 12:00	140

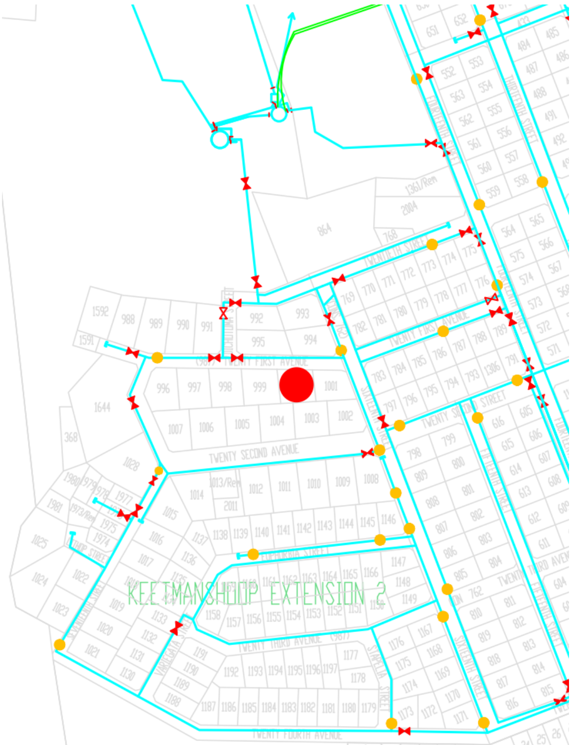


Fig. 5-3. *Location of erf 1000 marked with a red sphere.*

Pressure in erf 1000 fluctuates from low to sufficient. The minimum pressure of 200 kPa at all times is not achieved. The lowest readings, 50–70 kPa, were measured when the inlet from NamWater reservoirs to Oxpass Hill reservoir was closed. The highest readings, 290, 330 and 270 kPa were measured in the afternoon when the inlet was opened. This implicates that the water from NamWater reservoirs partially bypasses Oxpass Hill reservoir and flows to western Westdene giving higher static head. The newly found pipelines in and out from Oxpass Hill reservoir are covered in Section 5.9.1. Moreover, pressure readings 150 and 140 kPa were measured in the daytime after the inlet valve was left opened. Pressure increased from 50…70 to 140…150 kPa. This further suggests that the water from NamWater partially bypasses Oxpass Hill reservoir. Finally, the pressure increases approximately 100 % from around 12 o'clock to 16:30 when the inlet to Oxpass Hill reservoir is opened during both times. However, the reason for this increase was not studied. It's possible that the demand is high after all due to the irrigation of erven in Westdene before afternoon. As there aren't any water meters for flow readings in the area, this can't be confirmed.

5.4 Pipe bursts and leakages

The amount of pipe bursts and leakages from January 2015 to June 2015 were manually counted from the municipality's bookkeeping. The amount of pipe bursts in Keetmanshoop is presented in table 5-3.

Table 5-3. *Number of pipe bursts in Keetmanshoop in 2015.*

Suburb	January	February	March	April	May	June
Westdene	6	4	3	1	5	3
Noordhoek	3	2	2	0	4	1
Kronlein	2	6	4	2	0	2
Tseiblaagte	29	19	25	12	24	12
Town Area	2	6	5	6	3	5
Total	42	37	39	21	36	23
6 months total	**198**					

There were a total of 198 pipe bursts in 6 months. It means there were 33 pipe bursts a month on average, which makes it more than one pipe burst per day. Although the temperature in Keetmanshoop is always above 0 °C from January to March and the pipes aren't under a stress caused by freezing, these months were the worst in terms of pipe burst frequencies. This was due to the earthworks in construction sites at the time, mostly in Tseiblaagte (see Section 5.10.2).

It's evident that Tseiblaagte suffers the most from pipe bursts. Out of 198, 121 pipe bursts took place there. According to Tuovinen (2014), the daytime pressures there are 300…600 kPa being approximately 100…200 kPa higher than in the rest of the town expect the industrial area in the south. The pipe material in Tseiblaagte is mainly AC which is also the dominant pipe material in other parts of the town. This indicates that the higher the water pressure, the higher is the frequency of pipe bursts. Also, as mentioned in the previous paragraph, there were many construction sites in Tseiblaagte from January to March that resulted in the increased frequency of pipe bursts.

The number of pipe leakages is presented in table 5-4. The amount of pipe leakages is lower than pipe bursts.

Table 5-4. *Number of pipe leakages in Keetmanshoop in 2015.*

Suburb	January	February	March	April	May	June
Westdene	3	1	1	0	0	0
Noordhoek	1	0	0	1	1	0
Kronlein	1	4	1	0	0	1
Tseiblaagte	5	5	6	1	1	2
Town Area	2	1	2	2	2	1
Total	12	11	10	4	4	4
6 months total	**42**					

There were 42 leakages in 6 months. The leakage rate is the highest in Tseiblaagte due to the high pressure, and the earthworks caused many pipe leakages during the first 3 months, especially in Tseiblaagte.

In practice, there are 3 main reasons for pipe bursts and leakages in Keetmanshoop (Bonthuys 2015):

1. Old age of the pipe network. Approximately 50 % of the pipes are more than 50 years old (Löppönen 2011).
2. High pressure of the network (see Section 2.3).
3. Lack of air release valves in the long pipelines. This causes air pockets, and consequently, water hammers (see Section 2.1.3).

Pipe bursts and leakages also means that repair works are often needed. This impairs service ability level as some customers have to be without water for hours.

5.5 Water consumption and night flow

According to the municipality's accounting, Keetmanshoop Municipality billed 1 627 663 m^3 of distributed water and purchased 1 985 943 m^3 of water from NamWater in the financial year of 2014/2015. The average daily flow is therefore 227 m^3/h. According to Aalto (2014), there has been no trend in the varying consumption in 1999–2011.

In figure 5-4, a water consumption graph of Keetmanshoop from a week's period in June 2015 is presented. The graph shows the peak consumptions and the lowest consumptions (night flow).

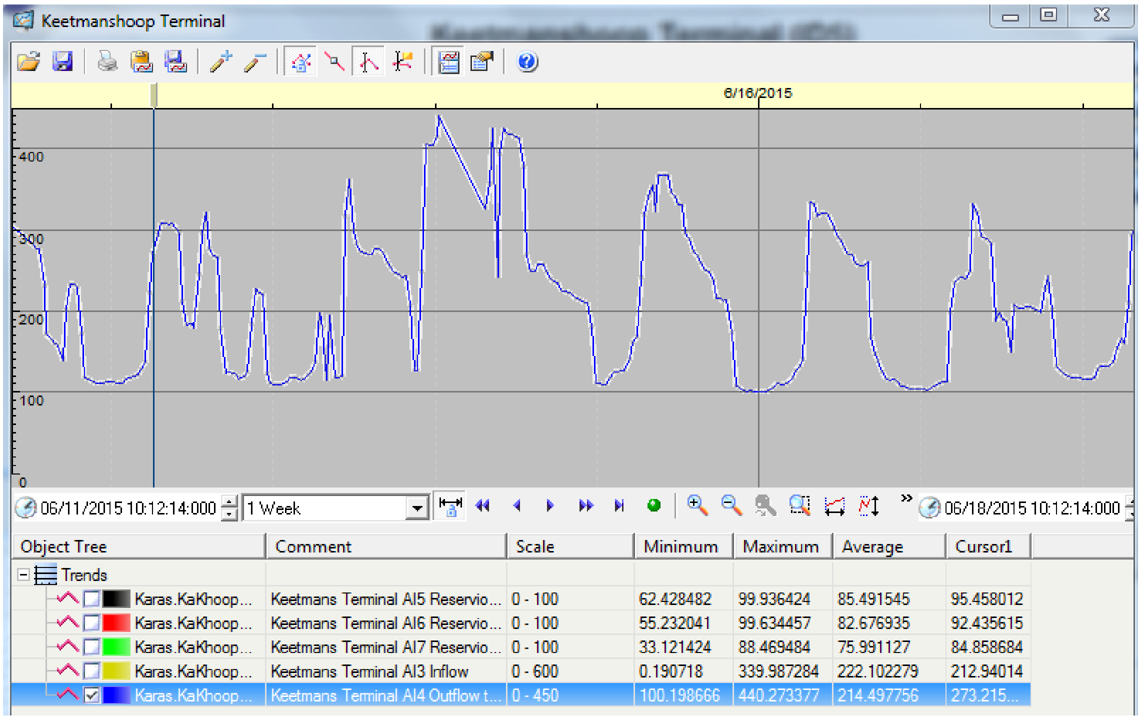


Fig. 5-4. *NamWater's daily variation graph of Keetmanshoop's water use from one-week period in June 2015. Time is on the x-scale and the consumption in m^3/h is on the y-scale.*

The night flow in Keetmanshoop can be seen in figure 5-4. During the week's monitoring period it was always higher than 100 m^3/h. According to several conversations and to Löppönen (2011), Tuovinen (2014) and Aalto (2014), night flow of 100 m^3/h is regular in Keetmanshoop.

No metering for night consumption survey was done during the field work. However, there are many possible reasons for the night flow rate in addition to the regular night time use of the consumers. According to Keetmanshoop Hospital maintenance, they pump water during the night time. Additionally, Oxpass Hill reservoir overflows partly explain the night flow rate in Keetmanshoop. Also, continuous background leakage of the pipe network and the leakages of Donkie Drai reservoir can be high in the night time. (Bonthuys 2015) Löppönen (2011) concluded in his thesis that the night flow rate is due to real losses and apparent losses.

5.6 Biggest water consumers

The biggest water consumers in Keetmanshoop in the financial year of 2014/2015 were acquired from the municipality's accounting. The list is comprised of the 31 biggest water users excluding private apartments. The list as a whole is presented in Appendix 3; the 10 largest users in 2014/2015 are shown in table 5-5.

Table 5-5. *The 10 largest water users in Keetmanshoop in 2014/2015 [m^3].*

Place	2011/2012	2012/2013	2013/2014	2014/2015	Inactive meter 2014/2015 [months]
Hospital	7 747	76 002	67 808	88 472	0
P.K Boys Hostel	13 657	17 915	14 219	15 075	0
Transnamib werk			1 190	12 005	0
NIMT	8 815	11 936	13 007	11 728	0
Canyon Hotel	6 150		2 892	9 947	0
Multi-purpose	5 208	2 009	4 094	8 454	0
Karma Property	1 411	3 466	3 683	8 443	0
Dan Viljoen Clinic	9 459	9 335	7 528	7 990	1
P.S.K School				7 861	0
Maritz Country Lodge	3 862	4 708	8 592	7 746	0

In 2014/2015, there were 3 distinct, anomalous water meter readings in the list of the biggest water consumers (see Appendix 3): Suiderlig Hostel and Suiderlig School had inactive meters for 9 and 12 months, respectively. The reading in Suiderlig Hostel was more than 2 times lower than in the 3 previous financial years, and the reading in Suiderlig School was 0. In addition, the reading in P.K Girls Hostel was more than 37 times lower than previously, yet the water meter was active the whole financial year.

5.7 Water meters and prepaid water meters

Keetmanshoop Municipality has 4 water meter readers. Their goal is to read each water meter reading once a month. An estimated bill is used in December since the municipality’s staff is on leave that month. 50 286 readings were taken in the 12-month accounting period of 2014–2015, which makes it 4 191 readings per month on average. (Poulton 2015, Swarz 2015) However, the municipality’s debtor’s statistics show that the amount of readings taken vary. The amount of water meter readings taken and their results from January 2015 to May 2015 is presented in table 5-6.

Table 5-6. *Amount of water meter readings taken and their readings in 2015 (Poulton 2015).*

Month	Nr. of consumers	m^3
January	6 598	207 316
February	3 963	606 222
March	3 981	153 859
April	4 018	128 218
May	4 030	111 698

The amount of readings varies every month, although it's close to the average. January's high number of consumers can be explained by December's holidays, as some of December's readings are taken in January. According to Poulton (2015) and Swarz (2015), the readings are reliable. As of July 19th, 2015, there were 350 inactive water meters still in use. Inactive water meters are supposed to be replaced monthly once the water meter reader has discovered them inoperative. However, the replacement is not usually done in a month. If a meter gives a false reading, an estimated bill is used (see Appendix 3). (Poulton 2015, Swarz 2015) A working water meter is presented in figure 5-5. Water meters are located inside the consumers' premises and are usually installed on the yard above ground.

***Fig. 5-5**. Working water meter (Salmivirta 2015).*

During June and July 2015 5 F-Tech branded prepaid water meters were installed into new public water taps in Tseiblaagte extensions 4 and 6. This is an effort to give poorer citizens an access to potable water; though they have to pay beforehand to avoid water thefts.

5.8 Non-revenue water

The NRW in Keetmanshoop in the invoicing period of 2014/2015 was calculated from the municipality's accounting. The annual system input volume (purchased water from NamWater) is 1 985 943 m^3 and the annual billed volume is 1 627 663 m^3. Therefore (equation 4.5) NRW = 18.0 %. This means the level of NRW is lower than 20 %, the main objective of the EU funded project. However, the level of NRW has decreased 15–

20 % percentage points compared to the previous studies, which is unlikely (see Chapter 7).

Keetmanshoop Municipality's own use, which was 53 160 m^3 in 2013 (Tuovinen 2014), is non-revenue water to the municipality and therefore it increases the NRW-%. However, 53 160 m^3 is less than 3 % of the purchased water.

5.9 Municipality's reservoirs

Keetmanshoop's WDN has 2 municipal reservoirs, Oxpass Hill reservoir in Westdene and Donkie Drai reservoir in Noordhoek. Different studies were conducted (see Section 4.3.9) for both reservoirs.

The emphasis of the investigations was on Oxpass Hill reservoir. Also, Hietanen's research on Donkie Drai reservoir water level was utilized.

5.9.1 Oxpass Hill reservoir

Oxpass Hill reservoir is shown in figure 5-6. It has numerous cracks in the concrete in the roof and exterior walls. There are no fences that would protect and isolate the reservoir. The hatch on the top of the roof can be easily opened and so the water can be accessed.

Fig. 5-6. *Oxpass Hill reservoir (Salmivirta 2015).*

The inner radius of the reservoir is 7.6 m and the maximum water level is 3.7 m. The estimated volume (equation 4.6) of Oxpass Hill reservoir is 670 m^3. All the inlet valves in the close proximity and the bypass valve of the reservoir are stuck. They can't be operated and their condition is poor and fragile. The bypass line can't be used, since there is always water flowing through the inlet of the reservoir as it can’t be closed. The reservoir had a deteriorated and dysfunctional float valve (figure 5-7) that didn’t prevent the reservoir from overflowing. A new float valve was installed to Oxpass Hill reservoir on July 18th, 2015. It was supposed to block the excessive inflow from NamWater reservoirs and stop the reservoir from overflowing. Despite of the new float valve, the overflows didn’t stop.

Fig. 5-7. *Old float valve in Oxpass Hill reservoir (Salmivirta 2015).*

There is an inlet valve before Oxpass Hill reservoir which function was misunderstood. It was wrongly used for operating the water flow from NamWater reservoirs to Oxpass Hill reservoir (see Appendix 1). It was operated in the following way: The Standby Team should close the valve in the afternoon and open it in the morning. This would prevent the reservoir from overflowing, since when the inlet is closed during the evening and night there is no risk of overflowing and there would still be enough water in the reservoir for consumption. During the monitoring period, July 9–12, 2015, the water level in the reservoir was high at all times, yet there was no overflowing of the reservoir. This was due to proper valve operating at the time: the inlet valve was closed at 16:00 and opened at 8:00.

However, there were generally no specific times or any specific Water & Sewage Teams responsible for this valve operating. The communication was lacking between the water teams, Standby Team and foremen. This resulted in numerous overflows during the field work. In general, Oxpass Hill reservoir overflows partly explain the night flow rate in Keetmanshoop.

In reality, the inlet to Oxpass Hill reservoir works differently. This was confirmed when the pipes were dug out July 24–28, 2015. The inlet valve does let the water from NamWater flow through it, but the pipeline after the inlet valve splits and the other one bypasses Oxpass Hill reservoir and leads to western Westdene. When the inlet valve is closed, the pressure in western Westdene decreases. When the inlet valve is opened, the pressure then increases (see Section 5.3). If the inlet valve is opened during the night when there is low consumption, the pressure in Westdene increases high. This generates a back flow to Oxpass Hill reservoir outlet that leads to the overflows. This is why the newly installed float valve doesn’t function as it was meant to: the float valve doesn’t prevent the back flow to the reservoir. The newly found pipelines in the proximity of Oxpass Hill reservoir were drawn with FCGnet (figure 5-8). The newly found pipelines also mean that the current water distribution maps show the pipelines incorrectly, at least in the proximity of Oxpass Hill reservoir.

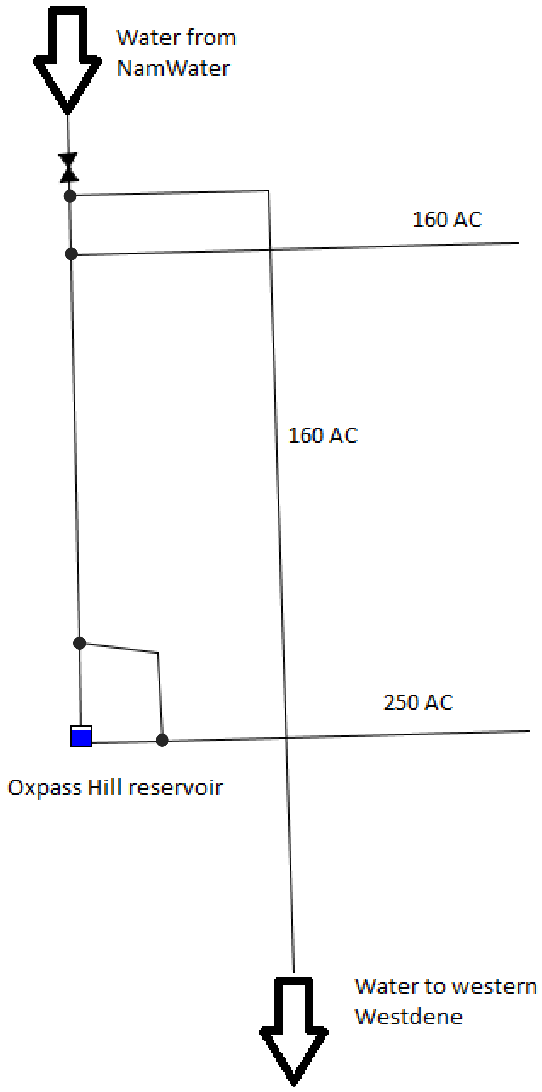


Fig. 5-8. *Found pipelines in Oxpass Hill reservoir area.*

As no further digging was carried out, the tail ends of the pipelines in question remained unknown during the field work. Due to the back flow, Nashima ordered a one-way valve to be installed to the 250 AC outlet of Oxpass Hill reservoir.

5.9.2 Donkie Drai reservoir

The inner radius of the reservoir is 14 m and the maximum water level is 3.5 m. Donkie Drai reservoir's estimated volume (equation 4.6) is 2160 m^3. The reservoir has numerous cracks all over the concrete surface. There is no fence around the reservoir and the hatch can be easily opened. Even some small animals have been seen swimming in the reservoir. According to the municipality's staff, the bypass valve in Donkie Drai reservoir is not functioning.

The reservoir is occasionally overflowing and it could be leaking through the walls. This can be concluded from the night flow rate (Bonthuys 2015). Nevertheless, the leaks or the overflows were not further studied during the field work.

Donkie Drai reservoir water levels (Hietanen 2015) are presented in table 5-7 and in figure 5-9. The reservoir water level was always above 2.70 m during the monitoring period. The water level was the lowest at 12:00 and the highest at 3:00 and 6:00.

Table 5-7. *Donkie Drai reservoir water levels [m] (Hietanen 2015).*

Date/Time	9:00	12:00	15:00	18:00	21:00	24:00	03:00 = max.	06:00 = max
2.3.2015	3.15	2.90	3.08	3.22	3.10	3.15	3.50	3.50
3.3.2015	2.70	2.80	2.95	3.30	3.00	3.10	3.50	3.50
4.3.2015	2.90	2.85	3.00	3.20	3.10	3.20	3.50	3.50
5.3.2015	3.10	2.85	3.10	3.35	3.15	3.00	3.50	3.50
6.3.2015	3.20	2.90	3.50	3.30	3.00	3.50	3.50	3.50
7.3.2015	2.95	2.80	3.00	3.20	3.10	3.35	3.50	3.50
8.3.2015	2.85	2.90	3.35	3.30	3.15	3.40	3.50	3.50
9.3.2015	2.90	2.85	3.40	3.35	3.20	3.35	3.50	3.50

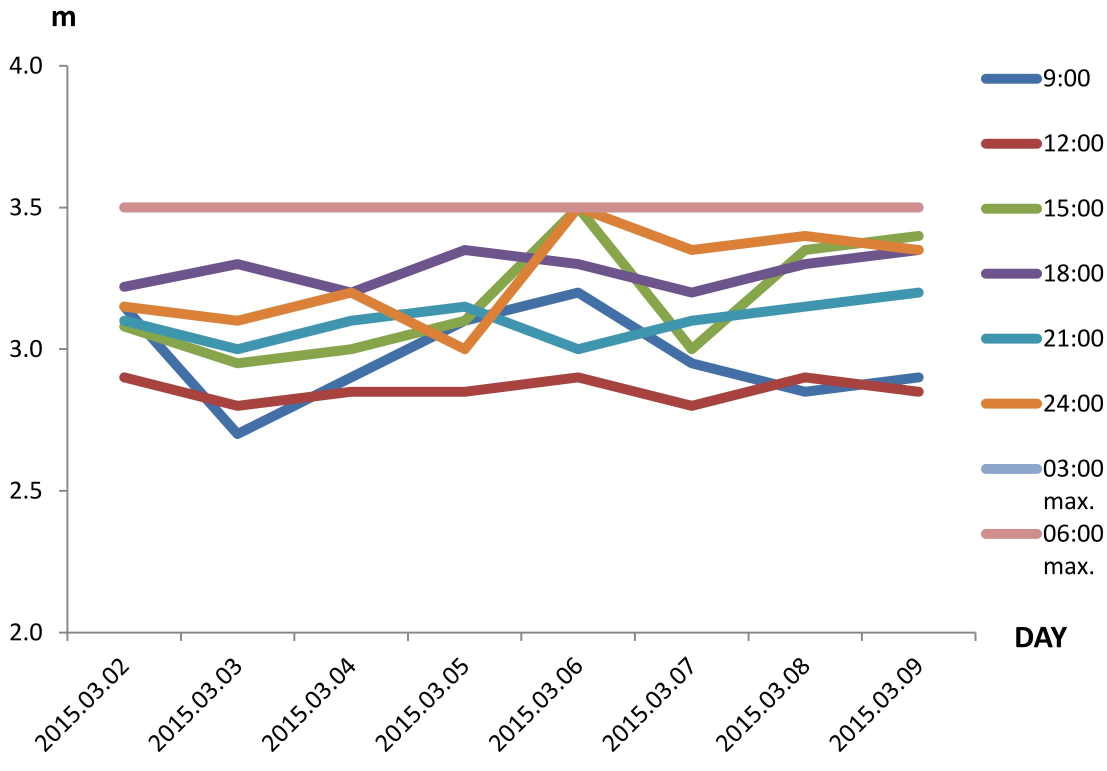


Fig. 5-9*. Donkie Drai reservoir water levels (Hietanen 2015).*

The water levels implicate that the demand in Kronlein and Tseiblaagte is the highest in the mornings. As the water level is constantly high, it can be questioned if the turnover rate of the water is enough in terms of drinking water quality.

5.10 Finance of the water sector

Keetmanshoop Municipality's water sector's budget in total in 2014/2015 and 2015/2016 and the actual income and expenditure in 2014/2015 are presented in Appendix 4. In table 5-8, they are presented as a summary. The water sector's and the sewerage sector's budgets are separated.

Table 5-8. *Summary of Keetmanshoop Municipality's water sector's finance [N$].*

	Budget 2014/2015	Actual 2014/2015	Budget 2015/2016
TOTAL INCOME	**17 045 000**	**22 291 229**	**26 402 000**
EXPENDITURE			
Total staff expenses	2 710 000	3 821 373	4 462 933
Total general expenses	18 224 000	17 234 937	21 971 000
Total R & M	533 000	934 065	638 000
Total redemption & interest	183 000	108 143	190 000
Total capital outlay	1 515 000	0	1 757 000
TOTAL EXPENDITURE	**23 165 000**	**22 198 518**	**28 838 933**
SURPLUS/DEFICIT	**-6 120 000**	**192 711**	**-2 436 933**

The water sector's business was profitable in 2014/2015 unlike budgeted. The 2015/2016 budget is deficient again. In practice, all the water sector's income comes from the water sales (99 % in 2014/2015). The majority of the expenses comes from the water purchase from NamWater (77 % in 2014/2015).

5.10.1 Water tariffs

NamWater's and Keetmanshoop Municipality's water tariffs in the past 10 years are presented in table 5-9 and figure 5-10. Water tariffs have been increasing in the past 10 years. In that time, the prices have almost doubled for both NamWater and Keetmanshoop Municipality.

Table 5-9. *Water tariffs in Keetmanshoop in 2005–2015 in N$/m³ (Poulton 2015).*

	NamWater	Municipality
2014–2015	9.90	12.00
2013–2014	8.60	11.60
2012–2013	8.60	10.94
2011–2012	7.45	10.32
2010–2011	6.45	10.22
2009–2010	6.45	10.22
2008–2009	5.93	9.46
2007–2008	5.93	8.60
2006–2007	5.32	7.70
2005–2006	5.00	7.00

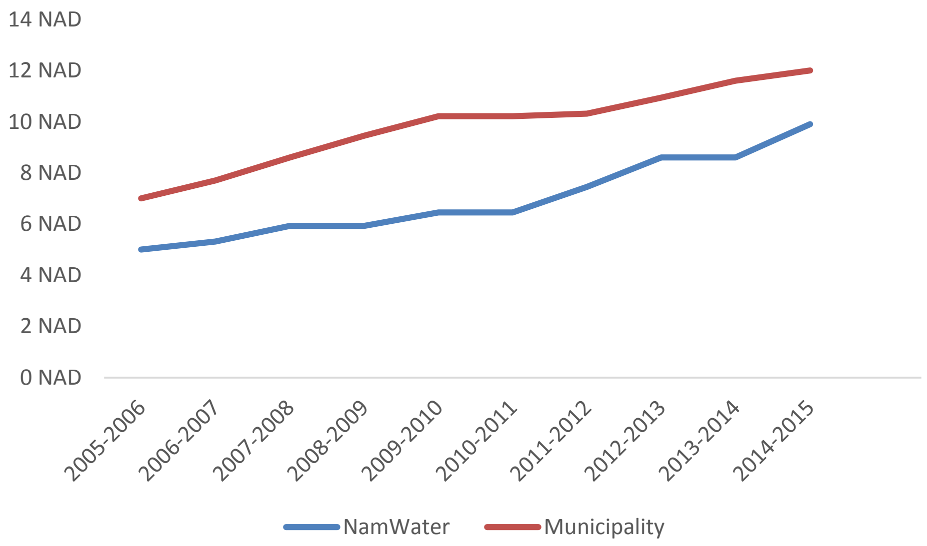


Fig. 5-10. *Water tariffs in Keetmanshoop in 2005–2015 (Poulton 2015).*

Water billing in Keetmanshoop is based on consumption and basic charges. The consumption charge is the same with all the consumers, but the basic charges vary from 0 to 206 N$/year. (Poulton 2015)

5.10.2 Mass housing

Namibian Homes is the main contractor at the site of the mass housing project in Keetmanshoop. In figure 5-11, the sign of National Mass Housing Project and typical mass housing end products are shown.

Fig. 5-11. *National Mass Housing Project in Keetmanshoop (Salmivirta 2015).*

In mass housing project phase one in Keetmanshoop, the aim is that 320 houses are built by the end of 2015. Of these 320 houses, 200 have already been finished as of July 2015. They are all social houses and subsidized 50–60 % by the government depending on the house type. The cost for the most expensive one is 130 000 N$ after the subsidy. The houses are only for sale and not for rent, and the period of payment is 20 years. The finished houses are not occupied yet, but there are more than sufficiently people on the waiting list for the 320 houses. The programme employs numerous low-income inhabitants of Keetmanshoop which improves their wealth. (Christiaan 2015, Lela Mobile Online 2014)

New houses mean new water consumers and more income to the water sector of Keetmanshoop Municipality. All the 320 houses are or are going to be connected to the distribution mains. The increased frequency of pipe bursts and leakages during the first 3 months of 2015 were due to the installation works of new pipes for the mass housing project houses. (Christiaan 2015)

On Namibian level, mass housing project hasn’t been successful. The project is plagued by deficiencies such as improper planning, questionable construction quality, exorbitant costs and slow pace of delivery: in 2014, only 1200 of envisaged 5000 houses were completed. (Informante 2015, New Era 2014b)

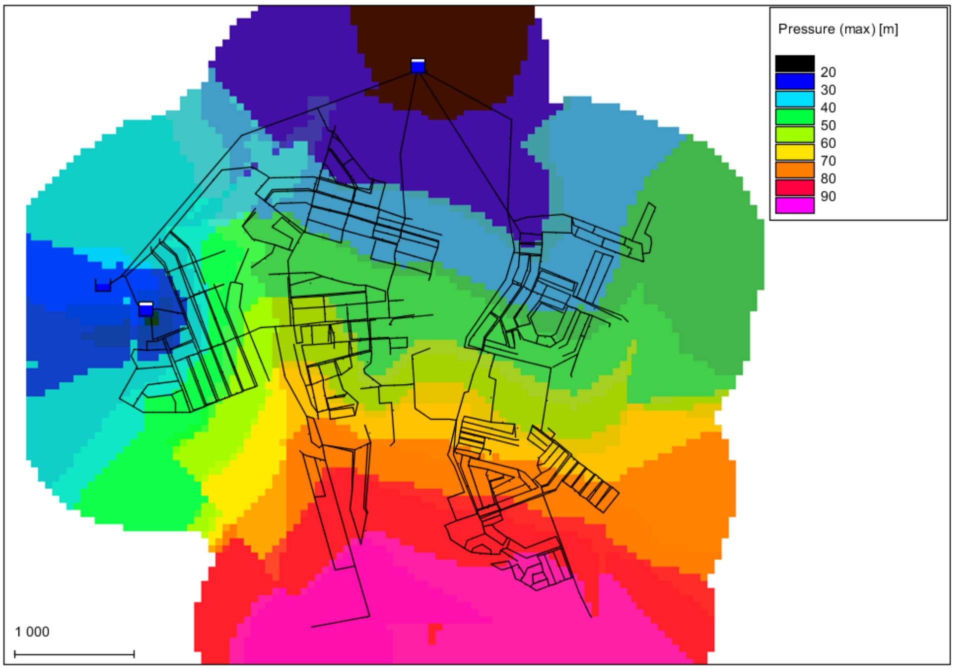
5.11 Model of the water distribution network

The simulations of Keetmanshoop’s WDN were computed with FCGnet in September and October 2015. EPS was used; the time period was 24 hours. Hydraulic time step was

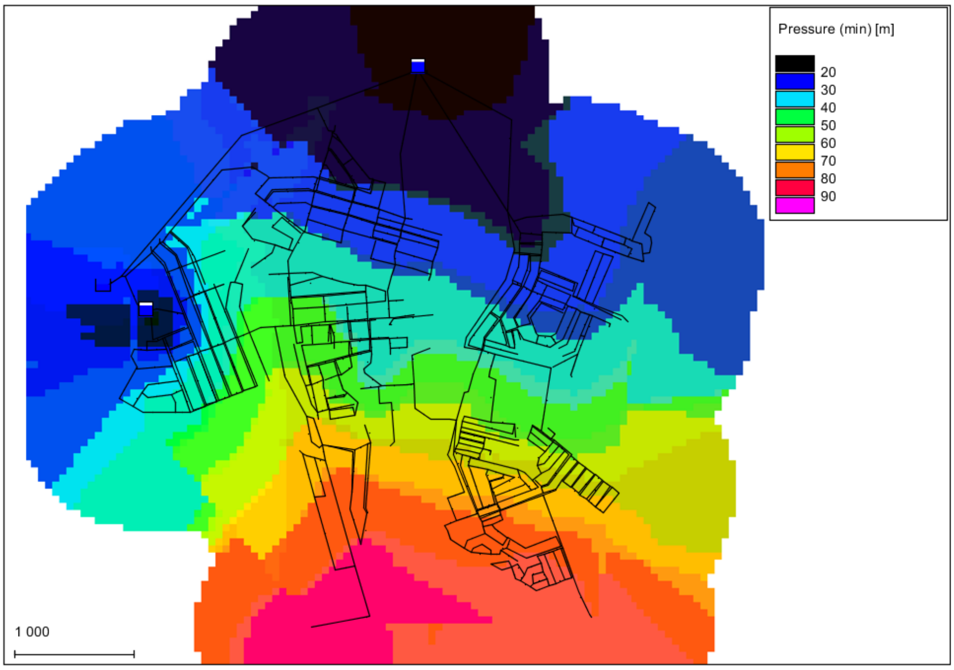
10 minutes. The following results are values regardless of day time, e.g. the maximum pressure in one node compared to another node may occur at different day time.

All the results are from a standard scenario except figure 5-17. Water usage is yearly average and there aren't any new instruments in operation in the network. One of the applications of a WDN model is that it can be simulated under new conditions. In figure 5-17, a fictitious PRV is installed to the model of Keetmanshoop's WDN.

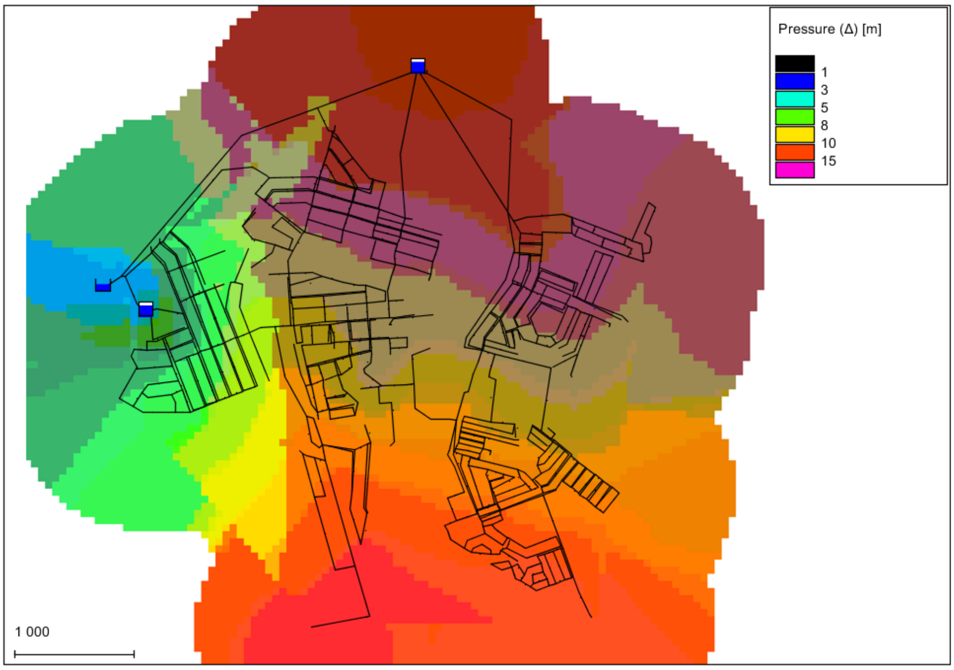
In figure 5-12, a) maximum and b) minimum pressure of the WDN and c) their difference are presented, respectively. The maximum pressure is the lowest near the municipality's and NamWater reservoirs in the north and in the west of Keetmanshoop. The pressure increases gradually according to the topology when going south. Tseiblaagte and the industrial area are the areas suffering most from the excessive pressure. There, the pressure goes to higher than 80 m. Maximum pressure shouldn't exceed 70 m. Tseiblaagte is the suburb where the majority of pipe bursts takes place.



a)



b)



c)

Fig. 5-12. *A) maximum and b) minimum pressure of Keetmanshoop's WDN and c) their difference [m].*

The minimum pressure accumulates almost the same way as the maximum pressure: the pressure is just lower this time. According to the model, the minimum pressure of 200 kPa (≈ 20 m) is achieved with every consumer. However, this isn't the real case (see

Section 5.3). 70 m of pressure is exceeded in Tseiblaagte and the industrial area. In these areas, pressure should be lowered.

Consumers can notice a pressure difference of 1 bar (≈ 10 m). The pressure difference is the lowest in Westdene where the difference is less than 8 m. Town Area is also decent in terms of pressure difference, but there are sections in the industrial area, Tseiblaagte, Noordhoek and Kronlein where the difference of 10 m is exceeded.

Net flow is presented in figure 5-13. It exposes the major distribution lines in the WDN.

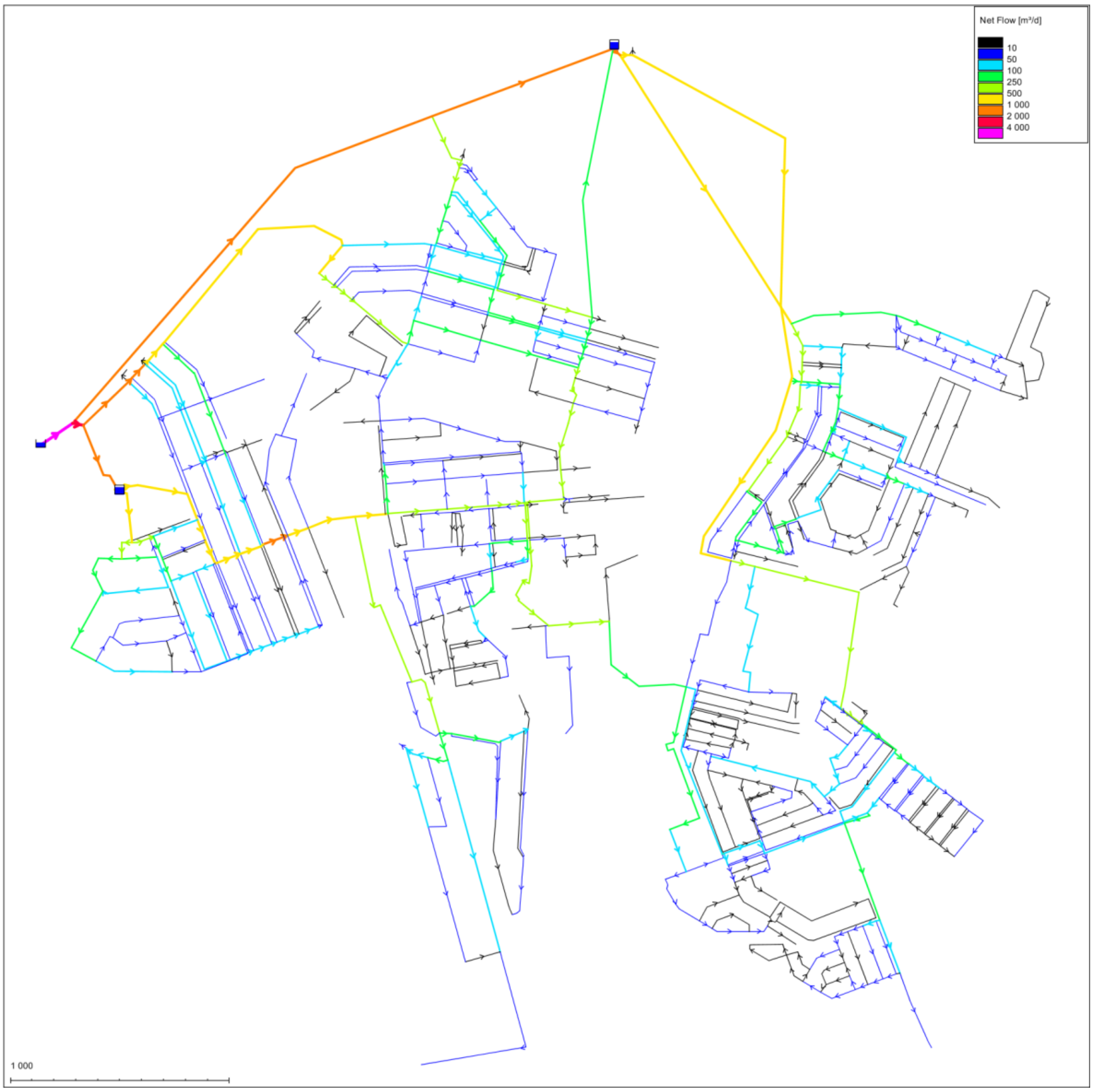


Fig. 5-13. *Net flow in Keetmanshoop's WDN [m^3/d].*

The major lines are the pipeline from NamWater reservoirs, the pipelines to northern Westdene after the first junction near NamWater reservoirs and the inlets and outlets of the municipality's reservoirs. Also the pipeline that connects Westdene to Noordhoek and the pipeline from Westdene to Town Area and a pipeline before that are major distribution lines.

Maximum flow is shown in figure 5-14. Maximum flows occur during maximum demand in the network.

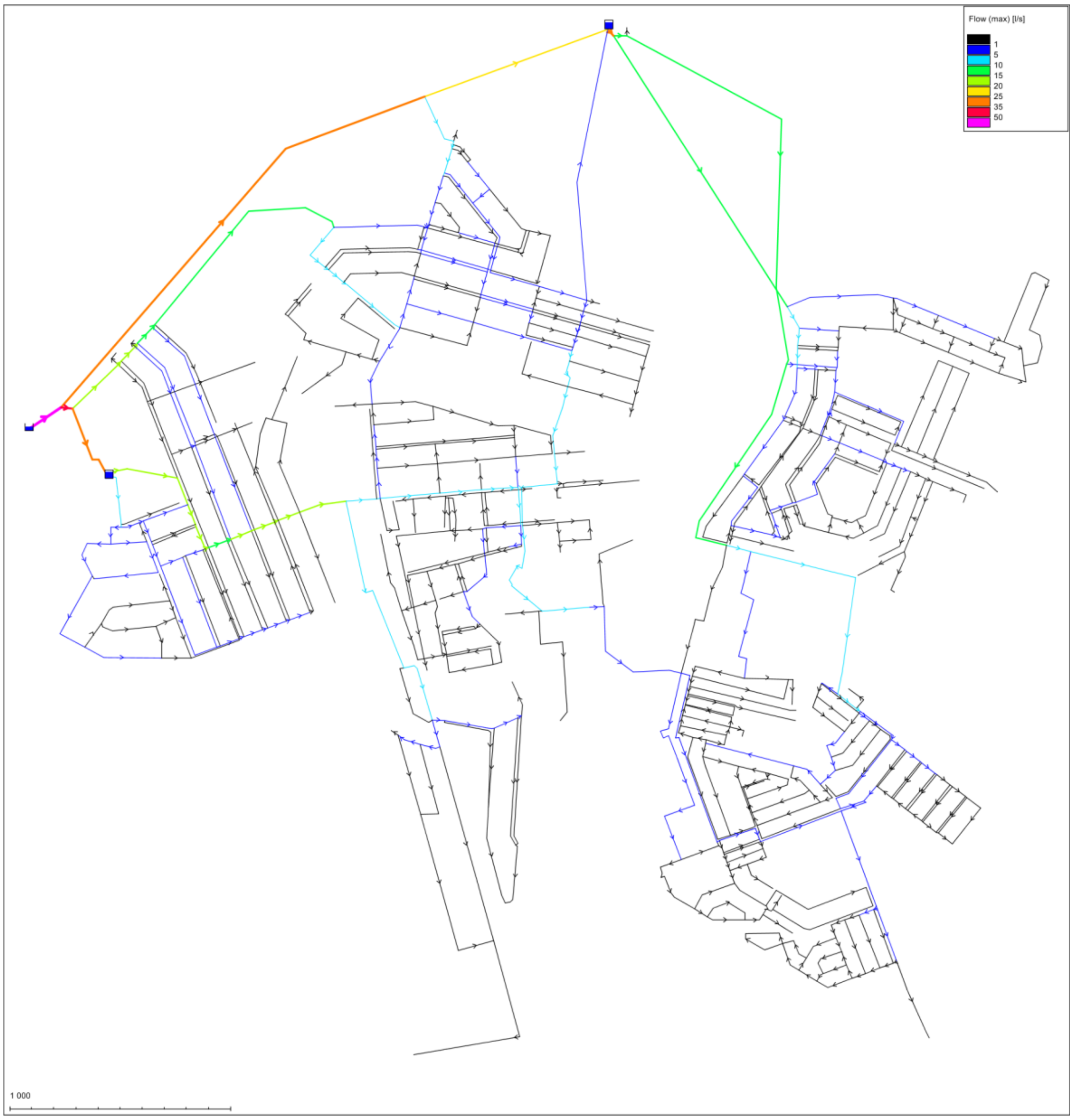


Fig. 5-14. *Maximum flow in Keetmanshoop's WDN [l/s].*

Maximum flow demonstrates which the flow directions and magnitudes are during maximum demand. The highest maximum flows occur in the same pipelines as the highest net flows.

Maximum unit head loss and average flow velocity are presented in figures 5-15 and 5-16, respectively. Unit head losses are small for the majority of the network.

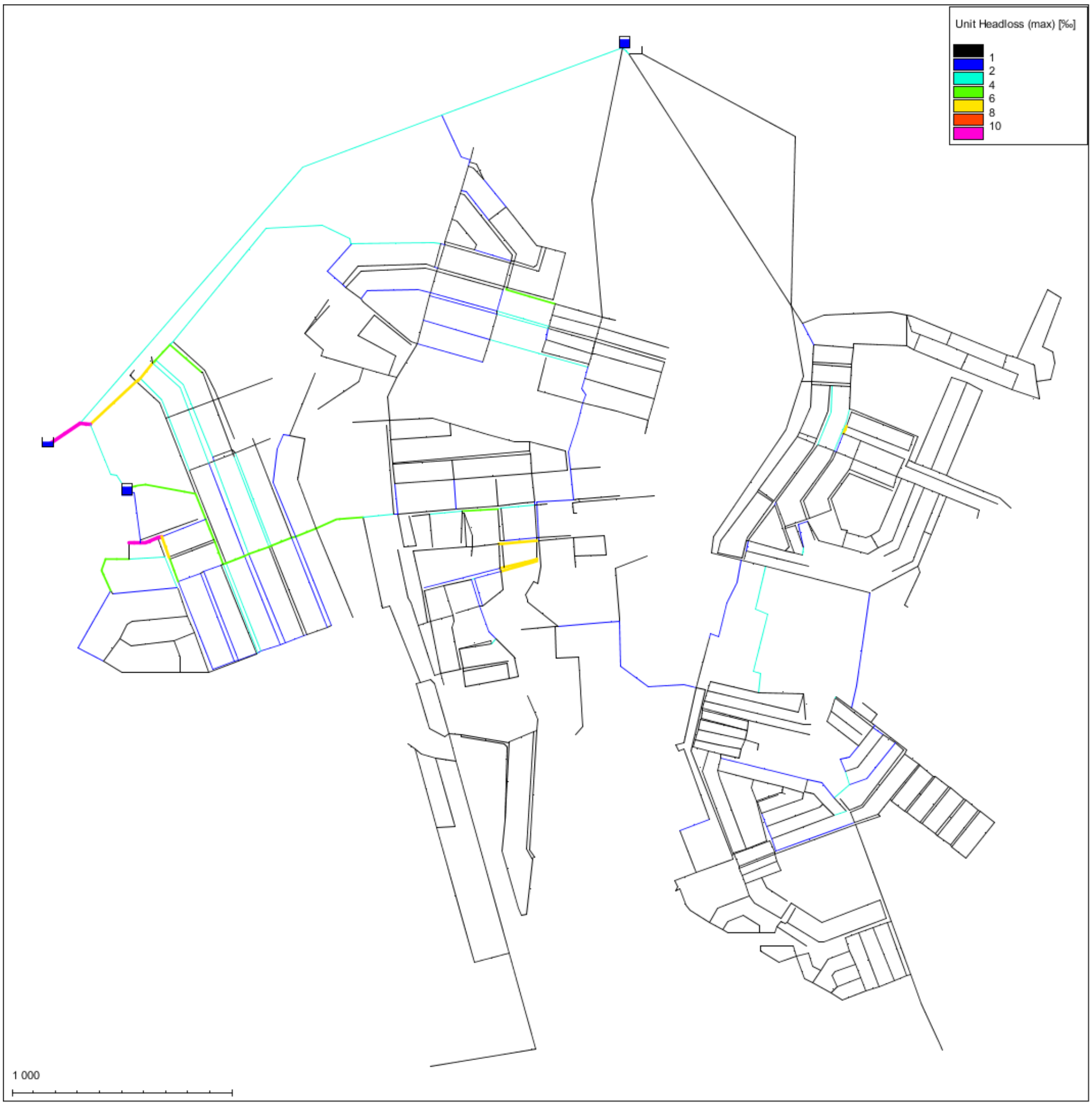


Fig. 5-15. *Maximum unit head loss in Keetmanshoop's WDN [‰].*

Fig. 5-16. *Average flow velocity in Keetmanshoop's WDN [m/s].*

According to equations 2.5 and 2.6, higher flow velocity causes higher head losses in the pipelines. This can be seen by comparing figures 5-15 and 5-16. Unit head losses greater than 6 ‰ are rare; the highest losses are in the pipeline from NamWater reservoirs and in the pipeline in Westdene close to Oxpass Hill. The pipeline in Westdene is only a 3″ AC pipe, meaning it could be replaced with a bigger one.

Flow velocities higher than 1 m/s are excessive, e.g. it causes high friction and head losses. The only area in terms of high velocity is the pipeline from NamWater reservoirs – there the average flow velocity exceeds 1.5 m/s.

In figure 5-17, an installation of a PRV and its effect on maximum pressure is simulated. The PRV is set to release 20 m of pressure.

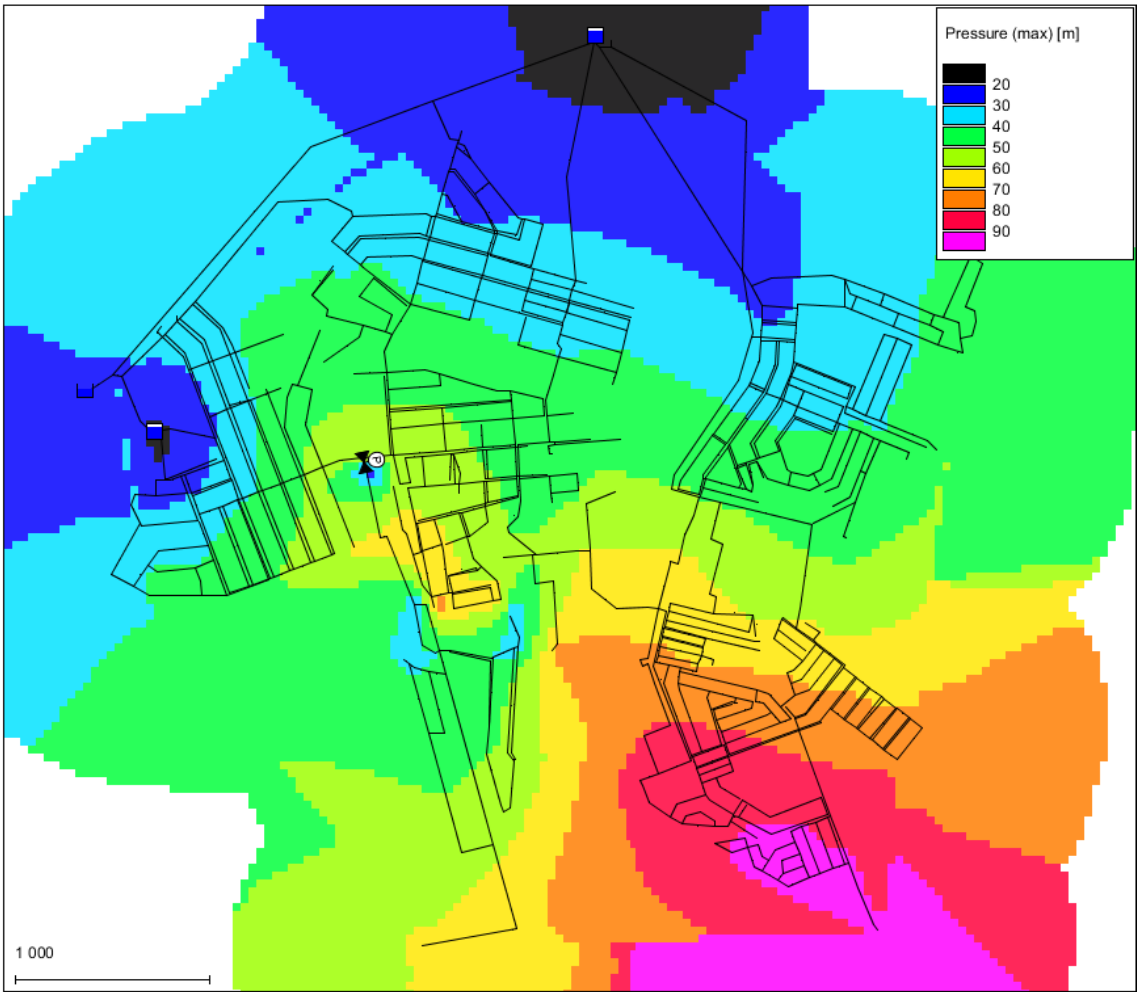


Fig. 5-17. *Maximum pressure with the new fictitious PRV (marked with P in the pipe near the connection between Westdene and Town Area) in Keetmanshoop's WDN.*

With one PRV, the maximum pressure in the industrial area can be lowered to acceptable level while the rest of the network still achieves the minimum pressure of 200 kPa. However, the PRV doesn't improve the issue of excessive pressure in Tseiblaagte. The overall maximum pressure in the network is slightly lower with the PRV.

6 IMPROVEMENTS TO KEETMANSHOOP'S WATER DISTRIBUTION

Improvements to the network and to the work habits have been previously presented in Seppänen's (2009), Löppönen's (2011), Aalto's (2014) and Tuovinen's (2014) theses. In this chapter, new proposals are presented along with some old ones.

Improvement proposals presented in this chapter, apart from Section 6.8, have been discussed with Hietanen and Nashima during the field work. The proposals are based on the results of this study (Chapter 5) and the discussions with Hietanen and Nashima.

6.1 Organization and work habits of the water sector

Water & Sewage Teams should be separated into individual teams, e.g. Water Teams and Sewerage Teams. There should be 2 Water Teams and 2 Sewerage Teams working simultaneously with the same amount of crew members as before: 5 or 6 Team Members, of which 1 or 2 are Team Leaders. This means the municipality should hire new employees. Both Water and Sewerage Teams should only focus on and specialize in their own works. Moreover, NamWater could train the municipality's plumbers at NamWater Training Centres located around Namibia; one Training Centre is even found in Keetmanshoop. Especially for the plumbers, the ability to read water distribution maps is necessary and they should be educated for that.

Standby Teams should work only a week at a time instead of a month. Thus, they shouldn't do standard working hours. This would result in more motivated and less exhausted Standby Team workers. Standby Team should consist of both Water and Sewerage Team Members, for instance 4 plumbers and a Team Leader for both water and sewerage would be sufficient. This would also decrease Team Members' workload. Finally, Standby Teams should have their own call center: now the calls go through Standby Office which takes excessive time.

The communication between the foremen and the workers shouldn't be lacking as much as it does now. Additionally, the work attitudes of the workers and the foremen could be better. The whole teams should always show up at the work site on time. Moreover, after new pipes have been installed, the old ones should be removed from the worksite (see figure 5-1).

It's a common practice in Keetmanshoop to store uPVC pipes outdoors and expose them to solar radiation (figure 6-1). UPVC pipes should be kept indoors or properly covered to avoid the damage resulting from the ultraviolet radiation. The exposure results in visible discoloration and decreased impact resistance. (JM Eagle 2009)

Fig. 6-1. *Storage of uPVC pipes and gate valves at the municipality's warehouse (Salmivirta 2015).*

6.2 Fire hydrants, gate valves and water meters

The fire hydrants that are covered with fences or garbage should be uncovered by the Fire Brigade. This would increase the fire safety in Keetmanshoop.

More gate valves are needed for better isolation of the network. When new pipe bursts and leakages occur, the broken pipeline could be isolated from a shorter range. This would result in better user service during repair works. (Cattafi et al. 2011) The numerous non-functioning gate valves should be located in the whole network and replaced with functioning ones. Aalto (2014) pointed out that in some cases, the flow in the leaking pipes was not possible to stop with gate valves, and so the pressures in these pipes were reduced by opening fire hydrants and closing gate valves further away. This kind of action results in great water losses.

The inactive water meters, 350 in total, should be replaced immediately. An exact water meter reading would be fairer for both the user and the municipality. Also, the use of estimated bills with inactive meters may result in water and financial losses (see Appendix

3). According to Aalto (2014), replacing an inactive meter would be profitable for the municipality. Additionally, NamWater has a calibration bench for water meters in Keetmanshoop. The municipality could use this opportunity and start a calibration programme.

6.3 Growing share of uPVC pipes

According to Nashima (2015), the share of uPVC pipes in the network is approximately 20 % as of July 2015. As reported by Löppönen (2011), the rate of replacement is approximately once in every 400 years: it is obvious that pipes don't last that long. So the rate of replacement should be increased to as frequent as possible, as the majority of the pipe network is still the old AC. This would result in reduced frequency of pipe bursts and leakages.

Furthermore, studies have found uPVC pipes to have high flow rates, resistance to corrosion and a low leakage record (Visser 2009). The standard design life of AC pipes is 50 years (Williams & Von Aspern 2013). This means that the new uPVC pipes wouldn't burst that often.

6.4 Pressure management

Pressure management is a major issue in Keetmanshoop's WDN along with the old pipe infrastructure. The excessive pressure could be suppressed with PRVs and the unwanted air in the big pipelines could be released with air release valves.

According to Hietanen (2015) and Nashima (2015), a new pipeline could be constructed in the future from NamWater reservoirs to the new planning area in southern Westdene. The pipeline would go around the western Westdene. From this pipeline, 2 smaller pipelines could be constructed in western Westdene: one could go to 21st Avenue and the second to Lithop Street. This could greatly increase the low pressure in western Westdene. At least Oxpass Hill reservoir would be bypassed and the static head increased.

6.4.1 Installation of pressure reducing valves

There are no PRVs in the network. The effect of PRVs can be easily seen by using the model of Keetmanshoop's WDN (e.g. see figure 5-17). Fixed outlet system would greatly decrease excessive pressure and consequently decrease the frequency of pipe bursts.

According to Hietanen (2015), NamWater might buy 6 water distribution monitoring systems and the systems would have built-in PRVs. NamWater would also be responsible for the operation and maintenance of the monitoring systems. The locations of them should be carefully considered. In terms of pressure reduction, the pipelines in question could be the pipeline to the industrial area, the pipeline connecting Westdene and Tseiblaagte and the pipelines (1–3) connecting Kronlein and Tseiblaagte.

6.4.2 Installation of air release valves and one-way valves

At the moment, there aren’t any air release valves in the network. Air release valves should be installed to big pipelines to release unwanted air, as air pockets can cause damage to the pipelines and fittings through water hammers. Air pockets also reduce pipeline capacity. (Mays 2000, Merritt 2001)

2 pieces of one-way valves were installed to the network during and after the field work period. One was installed to the 250 AC outlet of Oxpass Hill reservoir and the other one to the pipe connecting Westdene and Town Area. Both of them are supposed to prevent Oxpass Hill reservoir from overflowing. For Keetmanshoop Municipality, it remains to be seen how they function.

6.5 Monitoring systems

As mentioned in Section 6.4.1, NamWater might buy and install 6 water distribution monitoring systems to the network and be in charge of them. The monitoring systems would have built-in PRVs, pressure and flow rate measuring and water quality measuring. The locations need to be seriously considered as there are variables (water and pressure characteristics) that are neglected all over the network and need extra planning. One chance is that Keetmanshoop’s WDN is divided into 5 monitoring zones according to the suburbs.

The monitoring systems can be simulated with a WDN modeling software. In order to do this, the WDN model really needs to replicate the actual network to give realistic and useful results.

6.6 Municipality’s reservoirs

Both the municipality’s reservoirs are in a bad shape. Both reservoirs should be protected and isolated: fencing and proper sealing of the hatches would do this. Also, the background leakage of the reservoirs should be measured at night when the inlets and outlets are closed. The amount of leaking water would tell how urgent the repairs are. In any case, the reservoirs need to be repaired if they are used in the future; also the non-functioning inlet, outlet and bypass valves have to be replaced.

It can be questioned if Oxpass Hill reservoir is needed at all. NamWater reservoirs hold enough extra storage: Oxpass Hill reservoir volume is approximately 670 m^3 while NamWater reservoirs have a volume of 3 * 3 300 m^3. In addition, when the inlet from NamWater reservoirs is opened, Oxpass Hill reservoir dampens the pressure in western

Westdene to unacceptable level (see Section 5.3). Moreover, the overflowing of the reservoir becomes expensive due to serious water losses. If the reservoir was bypassed, pressure levels in western Westdene would improve and water losses would be reduced.

6.7 Budgeting and investments

The water sector's budgeting should be done more accurately (see Appendix 4). Also, there were inconsistencies when the budgets from 2 different printouts were compared; close attention is also needed in this.

The municipality should invest in upgrading the WDN. This can be seen in Appendix 4; the water sector's capital outlay is 0 unlike budgeted. Good quality valves, water meters and other equipment (e.g. fittings) should always be found in store. More importantly, the non-functioning appurtenances in the network should be replaced.

6.8 Future for the modeling of Keetmanshoop's water distribution network

In the future, it's important for Keetmanshoop Municipality to constantly update and improve the WDN model. To make the model work better, specific and reliable data of pipe materials, sizes, locations, and especially connections, is needed. Separate junctions for big water consumers should be added to the model. Also, the junction demands that were based on suburb consumptions in 2011 should be re-checked and updated. The hydraulic pattern could also be redefined so that it would vary in daily, monthly and yearly cycles. Finally, closed gate valves and other operational valves such as one-way valves should be added to the model.

For simulation computation, the WDN's infrastructure registration and the flows and pressures are necessary to be known. The model needs to represent the real WDN for decision-making in water supply management. For this, the model must be calibrated. According to Walski et al. (2003), calibration is a "process of comparing the results of a model with field observations to… adjust the data describing the system until the predicted behavior agrees reasonably with the behavior observed in reality…" In Keetmanshoop's case, flow and pressure values should resemble those in the field. Calibration data analysis should be performed and the correlation between observed means and computed means should be around 1. (Alves et al. 2014)

In order to make all this feasible, some of the municipality's staff needs to be trained to use WaterCAD or other modeling software and to read and produce water distribution maps. At least the Senior Manager of Department of Infrastructure and Technical Services should know the software for modeling and drawing maps inside out, and the plumbers should know how to read water distribution maps. For this, the flow of information should be immaculate and a data processing system is needed.

6.9 Updating the maps and IT systems

The water distribution maps need to be rechecked, corrected and updated. At the moment, there are defects in the maps (see Sections 5.2 & 5.9.1). These include the lay out of the pipelines, pipe materials and sizes and the locations of gate valves and fire hydrants. Also, the handednesses of the gate valves should be added to the maps. All this would be a burdensome task, but the functioning of the network is unclear at the moment.

Hietanen (2015) suggested that gate valves could be marked with a number and the suburb location, e.g. WD 1 would present Westdene 1. Then the information would be shifted to the WDN model or to another map. This way all gate valves in Keetmanshoop's WDN would be counted and their position would be easier to determine in case of valve leakages.

A data processing system for the water sector is highly recommended. At the moment, the municipality's staff's knowledge of the WDN's pipelines, reservoirs and gate valves, not to mention the flow of information, is basically based on misleading maps, conversations and hearsays. This kind of crucial data should be documented and easily accessed.

As for mapping software, AutoCAD or similar software should be utilized. Changes to the distribution network would be easy to draw with AutoCAD. At least the Senior Manager of Department of Infrastructure and Technical Services should know how to use the design programme. Also a modeling software should be used, and the user(s) should go through thorough training on it. The Senior Manager should be the first to get this training.

7 DISCUSSION

When comparing the work habits of the water sector's workers and foremen with previous studies (Löppönen 2011, Aalto 2014, Tuovinen 2014), it's clear that they are still poor and there haven't been any improvements. The plumbers have not yet been educated according to these studies, new plumbers are further needed and there are still 2 water sector foremen positions vacant.

Compared to Tseiblaagte, there are not that many pipe bursts in the industrial area, yet it's the area of the highest water pressure. There can be an additional explanation for this in addition to the previously mentioned earthworks (see Section 5.4). More pipe bursts and new pipe installations in Tseiblaagte mean there is more and more unwanted air entering the pipe network, which can eventually result in water hammers. Furthermore, if and when gate valves are closed rapidly during the installation of pipes in Tseiblaagte, sudden changes in water velocity or sudden stoppage of flow develop pressure and can result in even more water hammers (Mays 2000, Merritt 2001).

The frequency of pipe bursts and leakages is very high. In 6 months, there were a total of 240 pipe bursts and leakages (including fire hydrant leakages) in the pipe network (total length = 160 km), which makes it 300 pipe breakages per 100 km per year. According to Pelletier et al. (2003), a ratio of 40 or more breaks/100 km/year is considered to be high and indicates a network in poor condition. Moreover, in Finland, the pipe break rate was approximately 4.5/100 km/year in 2010 (Vesilaitosyhdistys 2012). If Tseiblaagte was excluded from this calculation, the ratio would be 99 breaks/160 km/6 months = 124 breaks/100 km/year, which is still very high. The pipe breakage rate in Keetmanshoop is reliable, as all the breakages were separately recorded by the municipality.

Pressure is still an issue in Keetmanshoop's WDN. The minimum pressure of 200 kPa in the whole network is not achieved: at least in the studied suburb, Westdene, there were erven where the pressure was lower than 200 kPa. In Löppönen's (2011) and Tuovinen's (2014) theses, the pressure was always found higher than 200 kPa, but they also found the pressure to be excessive (> 700 kPa) in Tseiblaagte and the industrial area. This can also be seen from the simulation results in this thesis (see Section 5.11). However, the simulation results can't be considered reliable: e.g. pipe materials, sizes, locations and connections are uncertain in the WDN model. For example, the pressure in Westdene in erf 1000 was measured 50 kPa while the model claims the minimum pressure there is always above 200 kPa.

According to the municipality's accounting, the billed water use in Keetmanshoop, which was 1 627 663 m^3 in 2014/2015, has increased. It was 1 284 317 m^3 in 2012/2013 (Tuovinen 2014) and 1 115 000 m^3 in 2011/2012 (Aalto 2014). This much of increase, 26.7 % compared to 2012/2013, can't be considered reliable, as the volume of purchased water from NamWater has remained more of the same: it was 1 985 943 m^3 in 2014/2015, 1 909 215 m^3 in 2012/2013 (Tuovinen 2014) and 1 722 000 m^3 in 2011/2012 (Aalto 2014). Compared to 2012/2013, the increase of purchased water is only 4.0 %. Moreover, the level of NRW has dropped to 18.0 % compared to 33–38 % in the recent years (Löppönen 2011, Aalto 2014, Tuovinen 2014, Hambabi 2015). This further indicates that the municipality's accounting of distributed water is faulty.

Per capita consumption in Keetmanshoop can't be reliably determined, as there is no data of how many individual residents there are connected to the municipal water network. It is somewhere between 181 l/capita/d (30 000 inhabitants) to 288 l/capita/d (18 900 inhabitants) including non-residential use. Per capita consumption is 201 l/capita/d in Windhoek (the capital of Namibia with 320 000 people) and 226 l/capita/d in Finland including non-residential use (Uhlendahl et al. 2011, Vesilaitosyhdistys 2012). The optimal access – water service level is 100 l/capita/d, which is achieved in Keetmanshoop (World Health Organization 2003).

The usual night flow of 100 m^3/h in Keetmanshoop is high: it's 44 % of the average daily flow of 227 m^3/h. Normally, night flow should be less than 10–15 % of the average daily flow (Desert Research Foundation of Namibia 2010). Moreover, the night flow ratio of 44% indicates a possible leakage problem (WRP Consulting (Pty) Ltd 2013).

If the use of estimated bills with inactive water meters is inaccurate, it can be unprofitable for the municipality through water losses, at least when it comes to the biggest water users (see Appendix 3). Moreover, even though the replacement of inactive water meters should be done once a month, it may take more than 12 months. The replacement of inactive water meters has been an issue in Keetmanshoop at least since 2010 (Löppönen 2011, Aalto 2014, Tuovinen 2014).

The level of NRW in Keetmanshoop has been high during recent years, approximately 33–38 % according to the latest studies (Löppönen 2011, Aalto 2014, Tuovinen 2014, Hambabi 2015). The high NRW level is due to the overflowing of the municipality's reservoirs (mainly in Oxpass Hill) and the high frequency of pipe bursts and leakages (see Section 5.4). Background leakage is also suspected as a factor for the NRW in Keetmanshoop but it's difficult to locate and confirm. Metering inaccuracies (see Appendix 3, P.K Girls Hostel) and poor estimation of inactive water meter readings can also affect the level of NRW in Keetmanshoop. During the financial year of 2014/2015, the actual NRW level in Keetmanshoop was expected to remain at the same level as previously, yet according to the municipality's accounting it was only 18.0 %. Most likely some of the figures in the municipality's water use accountancy are incorrect. In Namibia, the level of NRW was 15 %, so the NRW level in Keetmanshoop is higher than the national average (van den Berg & Danilenko 2011) In addition, the NRW level is 14.0 % in Windhoek and 17.0 % in Helsinki (The Smart Water Networks Forum 2011).

During the field work, new pipelines were found in the proximity of Oxpass Hill reservoir (see figure 5-8). These pipelines and their connections change how Oxpass Hill reservoir functions: The inlet valve lets the water flow through it from NamWater to the reservoir, but the pipeline after the inlet valve splits and the other one bypasses the reservoir and leads to western Westdene. When the inlet valve is closed, the pressure in western Westdene decreases and when it's opened, the pressure then increases. This is due to the static head from NamWater reservoirs. If the inlet valve is opened during the night, the pressure in Westdene increases high which generates a back flow to Oxpass Hill reservoir outlet. This leads to the overflows and results in significant water losses and higher night flow rate. The need of Oxpass Hill reservoir is questionable: NamWater reservoirs hold enough extra storage, Oxpass Hill reservoir dampens the pressure in western Westdene and the overflowing of the reservoir becomes expensive due to serious water losses. Finally, the estimated volumes of the municipality's reservoirs were rough approximations as there weren't any dimensional drawings of them.

The water tariff in Keetmanshoop, 12.00 N$/m^3, is average priced. The water tariffs in Keetmanshoop-sized Namibian towns, Ondangwa and Mariental, were around 11 N$/m^3 in 2013 (Tuovinen 2014). In Windhoek, the minimum tariff is 13.86 N$/m^3 (domestic water consumption 0–0.200 m^3 per day) and goes up to 90 N$/m^3 (domestic water consumption > 1.70 m^3 per day) (City of Windhoek 2015). The water tariff in Kangasala, a Keetmanshoop-sized town also, is 21.15 N$/m^3 (value-added taxes excluded) (Kangasalan kunta 2015).

NamWater's water tariff is 9.90 N$/m^3. Both NamWater's and the municipality's tariffs have almost doubled in the past 10 years. Practically all Keetmanshoop Municipality's water sector's income comes from the water sales. As the water sector's budgets in 2014/2015 and 2015/2016 are deficient, the municipality should raise the tariff or find alternative ways to increase the water sector's income and decrease the expenditure. For instance, if the level of NRW was 35 %, Keetmanshoop Municipality would then earn 0.65 * 12.00 N$/m^3 = 7.8 N$/m^3 of billed water. This is absolutely too little, as it's less than NamWater's water tariff. The actual business of the water sector was profitable in 2014/2015, but that was due to municipality's incorrect accounting of distributed water.

In the mass housing project, there can be an issue when it comes to the house payments. The new residents may not have enough money to deal with the mortgage and they are not going to be evicted in many months (Hietanen 2015). This possible loss of the municipality's income doesn't directly hamper the income of the water sector as the water sector is independent, but it can have indirect effects, e.g. less money for new investments for the water sector.

Keetmanshoop's WDN model doesn't reliably replicate the actual network. This can be seen from the pressure readings (see Section 5.3) and the simulation results (see figure 5-12). However, the maximum pressure in the actual network resembles the simulation results of maximum pressure (Löppönen 2011, Tuovinen 2014). There are many reasons why the model is imperfect: The model was based on existing AutoCAD network map data, which was not completely correct in several aspects. Defects in the maps considering the modeling process include the lay out of the pipelines, faulty information of pipe materials and sizes and the locations of possibly closed gate valves (thus closed gate valves could not be integrated into the model). Moreover, separate junctions for big water consumers were not used in the model, and the junction demands were based on suburb consumptions in 2011. For more precise simulation, the hydraulic pattern used in the modeling should have been based on more recent information on Keetmanshoop's water consumption and its daily, monthly and yearly variation.

8 CONCLUSIONS

Pressure is still a significant issue in Keetmanshoop's WDN. The minimum pressure of 200 kPa in the whole network wasn't achieved in the studied suburb, Westdene. In the previous studies, the pressure has been found excessive in Tseiblaagte and the industrial area, and at present, there is no reason why the pressure there would have reduced as no means for pressure reduction have been carried out.

There were a total of 240 pipe bursts and leakages in Keetmanshoop's WDN, the majority of them in Tseiblaagte, in a 6-month period. This is a very high frequency considering the length of the network and indicates the network to be in a poor condition. Pipe breakages in the network mainly occur from the old age of the pipe network, high pressure and lack of air release valves in the long pipelines.

The volume of water Keetmanshoop Municipality purchased from NamWater during the financial year of 2014/2015 increased only slightly compared to the previous studies, yet according to the municipality's records, the billed water use increased substantially. This results in lower level of NRW. The level of NRW is 18.0 % in Keetmanshoop, which is below the 20 %, the main target of this EU funded project. However, this can't be considered reliable. The level of NRW should be higher, above 30 %, due to the overflowing of the municipality's reservoirs (mainly in Oxpass Hill) and the high frequency of pipe bursts and leakages. Background leakage is also suspected as a factor for the NRW in Keetmanshoop. Metering inaccuracies and poor estimation of inactive water meter readings can also affect the level of NRW, whether decreasing or increasing it. The calculated level of NRW in this study indicates that the municipality's accounting of billed water is faulty.

New pipelines were found in the proximity of Oxpass Hill reservoir. The pipeline after the inlet valve that lets the water flow from NamWater to the reservoir, splits, and the other one bypasses the reservoir and leads to western Westdene. This increases the static head in western Westdene when the inlet valve is opened. If the inlet valve is opened during the night (low consumption), the excessive pressure in Westdene generates a back flow to Oxpass Hill reservoir outlet. This causes the overflows and results in significant water losses and higher night flow rate.

The constructed model of Keetmanshoop's WDN doesn't replicate the actual network in a reliable way. There are many reasons for this, such as faulty map elements in the background layer of the model and the complexity of water demand in the actual network. However, the model is approximate and indicates the performance of the actual network and some problematic areas. According to the model, the maximum pressure is the lowest near the municipality's and NamWater reservoirs, and the pressure increases gradually according to the topology when going south. Tseiblaagte and the industrial area are the areas of the highest pressure. This is also the situation in the real network. Moreover, the model illustrates that there are many sections where the pressure difference can be noticed by the consumers, the unit head losses are small for the majority of the network and the only area in terms of high flow velocity is the pipeline from NamWater reservoirs. The model was also used to simulate maximum pressure with a fictitious PRV installed to the network. The use of this PRV resulted in great reduction of pressure in the network, which shows that PRVs should be considered to be installed in Keetmanshoop's WDN.

Water distribution in Keetmanshoop can be improved in many ways. More gate valves are needed for better isolation of the network, the numerous non-functioning gate valves should be located in the whole network and replaced with functioning ones, and the 350 inactive water meters should be replaced. Moreover, the rate of replacement of the old AC pipes with uPVC pipes should be increased to as frequent as possible. The excessive pressure could be suppressed with PRVs and the unwanted air in the big pipelines could be released with air release valves, but at the moment there is neither in the network. Both the municipality's reservoirs should be protected and isolated, and also repaired if they are used in the future; also the non-functioning inlet, outlet and bypass valves should be replaced. Moreover, the use of Oxpass Hill reservoir can be questioned, as it dampens the pressure in western Westdene and the overflowing results in water losses, thus NamWater reservoirs provide a sufficient extra storage.

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Löppönen, A. Water Network Engineer. Lempäälä Waterworks. April 23rd, 2015.

Mashauri, D. Professor. Polytechnic of Namibia. June 13th, 2015.

Nashima, S. Senior Manager: Department of Infrastructure and Technical Services. Keetmanshoop Municipality. July 23rd, 2015 + various other dates.

Neshuku, Lasarus. Water & Sewage Team Leader. Keetmanshoop Municipality. July 14th, 2015.

Poulton, S. Finance Officer. Keetmanshoop Municipality. July 23rd, 2015.

Swarz, L. Finance Officer. Keetmanshoop Municipality. July 23rd, 2015.

Personal photos

Fig. 2-5. Monitoring system used in a WDN in Lempäälä. Lempäälä, Finland. April 23rd, 2015.

Fig. 3-6. Keetmanshoop’s urban area. Keetmanshoop, Namibia. June 13th, 2015.

Fig. 4-2. Taking pressure readings in Westdene. Keetmanshoop, Namibia. July 17th, 2015.

Fig. 5-1. Old AC pipes left at site after disposal. Keetmanshoop, Namibia. June 4th, 2015.

Fig. 5-5. Working water meter. Keetmanshoop, Namibia. June 13th, 2015.

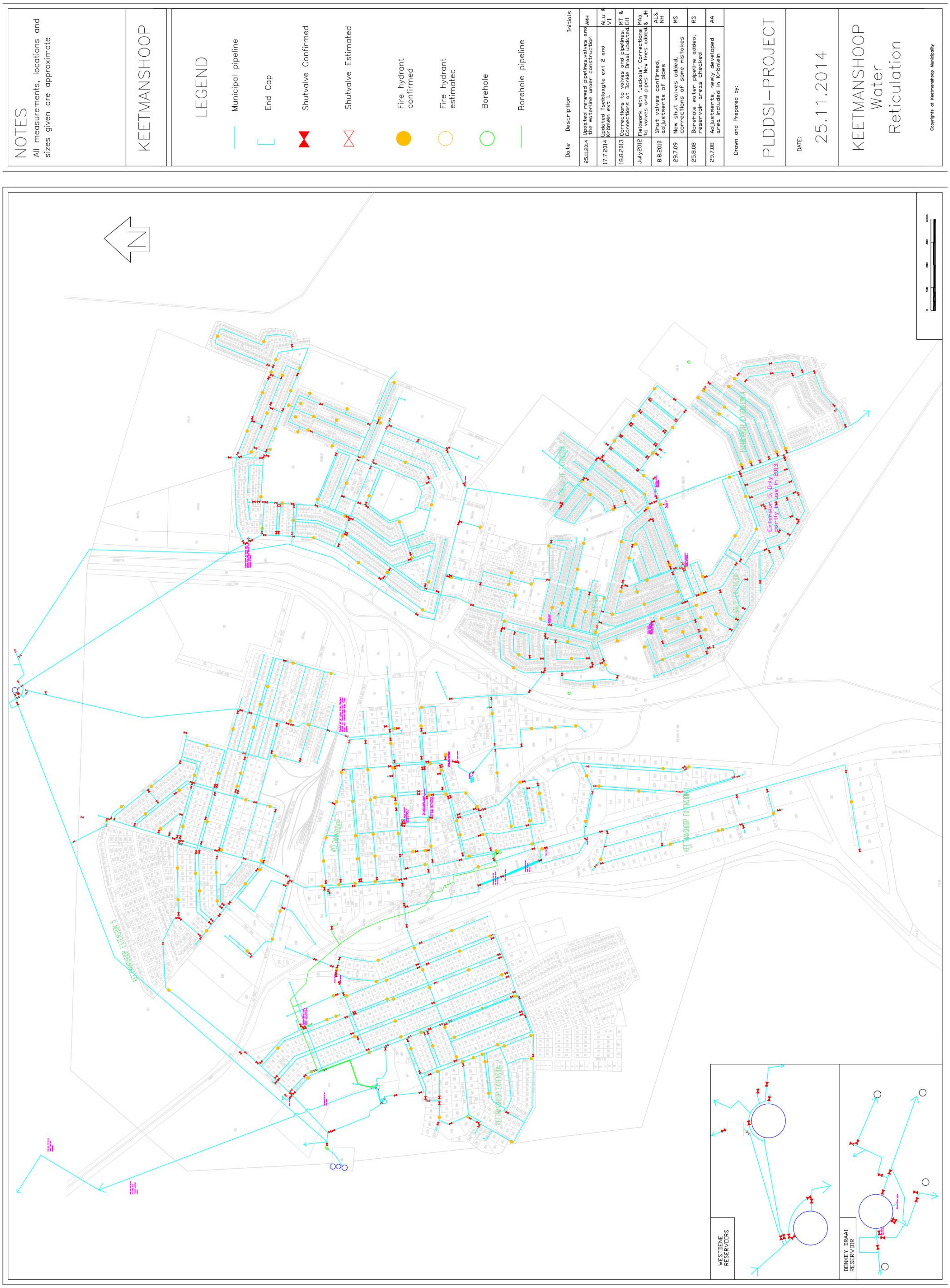
Fig. 5-6. Oxpass Hill reservoir. Keetmanshoop, Namibia. July 20th, 2015.

Fig. 5-7. Old float valve in Oxpass Hill reservoir. Keetmanshoop, Namibia. July 9th, 2015.

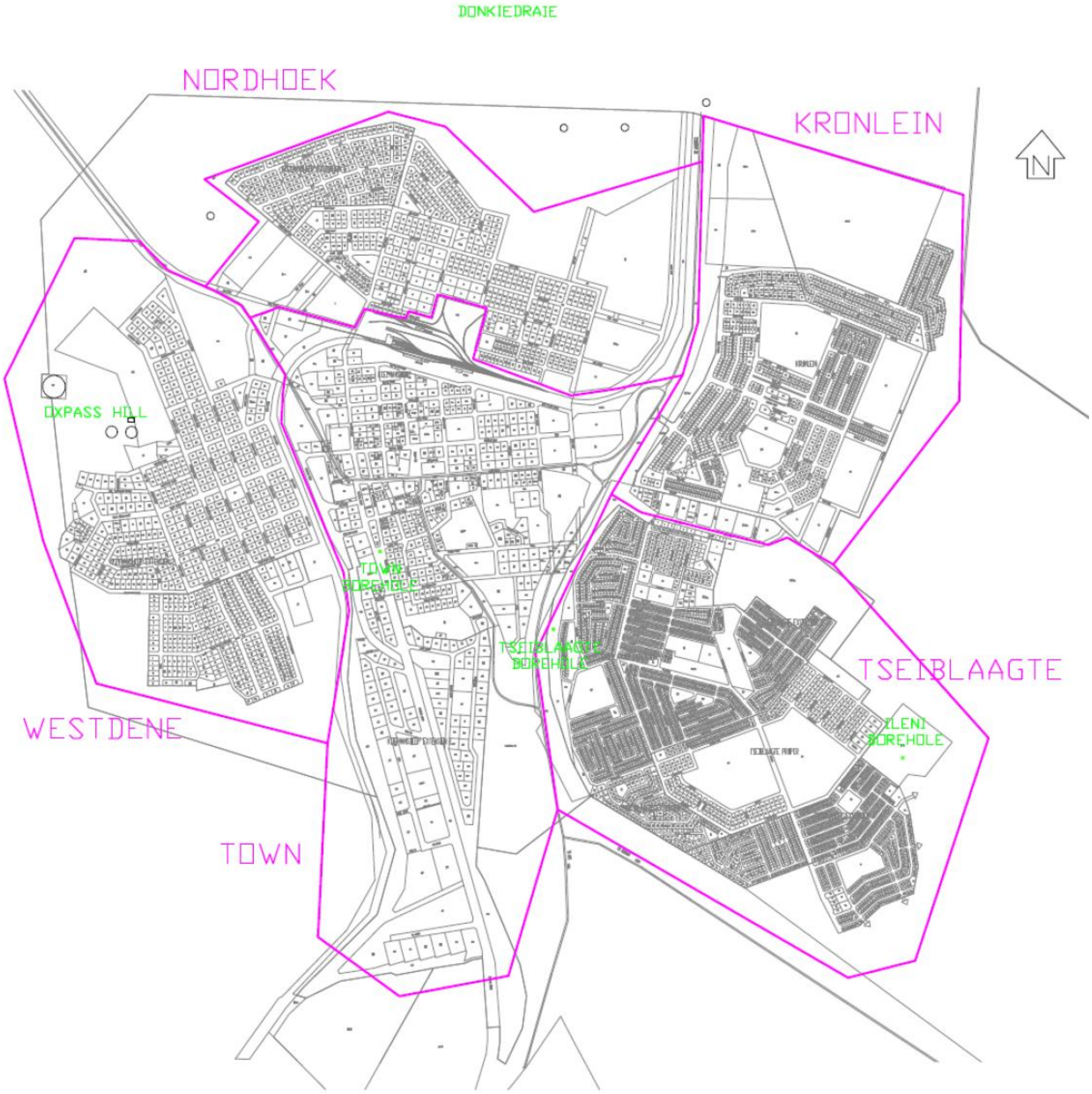
Fig. 5-11. National Mass Housing Project in Keetmanshoop. Keetmanshoop, Namibia. June 23rd, 2015.

Fig. 6-1. Storage of uPVC pipes and gate valves at the municipality's warehouse. Keetmanshoop, Namibia. June 4th, 2015.

APPENDIX 1: KEETMANSHOOP WATER DISTRIBUTION MAP, NOVEMBER 2014



APPENDIX 2: KEETMANSHOOP TOWN LAYOUT (AALTO 2014)



APPENDIX 3: LIST OF THE BIGGEST WATER USERS [M^3]

Place	2011/2012	2012/2013	2013/2014	2014/2015	Inactive meter 2014/2015 [months]
Hospital	7 747	76 002	67 808	88 472	0
P.K Boys Hostel	13 657	17 915	14 219	15 075	0
Transnamib werk			1 190	12 005	0
NIMT	8 815	11 936	13 007	11 728	0
Canyon Hotel	6 150		2 892	9 947	0
Multi-purpose	5 208	2 009	4 094	8 454	0
Karma Property	1 411	3 466	3 683	8 443	0
Dan Viljoen Clinic	9 459	9 335	7 528	7 990	1
P.S.K School				7 861	0
Maritz Country Lodge	3 862	4 708	8 592	7 746	0
St. Mathias Primary	7 789	7 460	5 797	6 844	1
KRC Teachers			4 400	6 709	0
Jazelra Badmintons	4 145	4 215	5 617	5 001	0
Ons Tuiste Old age	5 966	4 567	4 658	4 665	2
Min of fin. Inland rev	3 901	2 773	3 653	4 500	0
P.K School				3 447	0
Bonsec Investment			3 638	3 189	6
Southern Abattoir			2 256	3 115	0
Military base		1 694	4 724	3 007	6
Pick & Pay			3 267	2 758	0
Don Bosco Primary			2 455	2 231	
Suiderlig Hostel	5 929	4 958	4 000	1 912	9
La Rochelle				1 839	0
Judith Maria	1 775	2 660	1 471	1 352	0
Keetmans Private School	1 363	2 151	1 633	1 140	1
Police Barracks	1 848	1 645	1 635	995	2
P.K Girls Hostel	9 332	10 287	11 287	247	0
Suiderlig School	11 646	28 977	28 939	0	12
Prison	11 664	11 325	8 729		
Transnamib station	8 655	5 635	2 059		
Police station	10 145	9 518	1 033		

APPENDIX 4: BUDGETS AND THE ACTUAL INCOME & EXPENDITURE OF THE WATER SECTOR [N$]

INCOME	**Budget 2014/2015**	**Actual 2014/2015**	**Budget 2015/2016**
Sundries	45 000	5 023	45 000
Public sales	17 000 000	21 965 641	26 000 000
Department sales	0	320 565	357 000
TOTAL INCOME	**17 045 000**	**22 291 229**	**26 402 000**

EXPENDITURE	**Budget 2014/2015**	**Actual 2014/2015**	**Budget 2015/2016**
Staff expenses			
Salaries & allowances	1 480 000	1 885 825	2 386 327
Annual bonus	123 000	144 385	198 861
Fuel & vehicle all	180 000	181 200	201 600
Social security	13 000	16 339	20 436
Medical aid	73 000	134 498	139 862
Pension fund contribution	321 000	409 043	517 833
Housing allowance	298 000	446 471	555 014
Standby allowance	0	33 100	243 000
Cell Phone allowance	0	0	0
Overtime	222 000	570 512	200 000
Total staff expenses	**2 710 000**	**3 821 373**	**4 462 933**
General expenses			
Fuel	90 000	131 838	120 000
Protective clothing	20 000	36 718	50 000
Cleaning materials	15 000	28 271	30 000
Vehicle registration	4 000	3 484	5 000
Water purchase - NamWater	17 764 000	17 000 644	20 000 000
Treatment - water	30 000	0	34 000
Printing & stationery	1 000	2 015	2 000
EU funded water project	300 000	3 801	300 000
Casuals	0	25 426	50 000
Prepaid yard meters	0	0	1 200 000
Sundry payments	0	2 740	0
Total general expenses	**18 224 000**	**17 234 937**	**21 971 000**
Repairs & maintenance			
R & M - tools & equipment	10 000	8 250	11 000
Machinery & equipment	3 000	5 719	4 000
Water - network	500 000	890 211	600 000
Vehicles	20 000	29 885	23 000
Total R & M	**533 000**	**934 065**	**638 000**

<u>Redemption & interest</u>			
Interest on external loan	39 000	15 392	45 000
External loan redemption	144 000	92 751	145 000
Total redemption & interest	**183 000**	**108 143**	**190 000**
<u>Capital Outlay</u>			
Hydro blaster	0	0	0
Water pressure meter	75 000	0	86 000
Water network upgrade	1 000 000	0	1 200 000
Camping of reservoirs	400 000	0	460 000
1 * centrifugal pump	10 000	0	11 000
3 * pick ups	30 000	0	0
Total capital outlay	**1 515 000**	**0**	**1 757 000**
TOTAL EXPENDITURE	**23 165 000**	**22 098 518**	**28 838 933**

SURPLUS/DEFICIT	**Budget 2014/2015**	**Actual 2014/2015**	**Budget 2015/2016**
	-6 120 000	**192 711**	**-2 436 933**