IMPACTS OF PROCESS SELECTION AND PROCESS TRAIN IN THE DESIGN OF WATER TREATMENT PLANTS IN SOUTH AFRICA

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DECLARATION

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ABSTRACT

Background

The South African Government is spending huge amounts of money (about R64 billion for the fiscal years 2015/16 to 2018/19 as per the revised strategic plan – vote 36) and more than R30 billion was targeted water infrastructure investments by the end of 2014/15 financial year (Ruiters & Matji, 2015) striving to provide all its citizens with access to good quality water services through its organs of state such as municipalities and water boards. A study of more than one hundred (100) waterworks carried out throughout the country in the early years of the 21st century by Chilton and Polasek (2013) revealed that none of them was found to be appropriately designed in terms of the processes installed.

The author of this dissertation aims to bridge the gap in Chilton and Polasek's (2013) study of more than a hundred (100) waterworks, in which they did not quantify the actual impacts caused by the design flaws they identified. The present study was carried out on four waterworks situated in two provinces, namely Gauteng and North West. The aim was to quantify the financial implications, operations and maintenance complications/difficulties.

The four methods used to carry out this study are: *Initial design catering for the ultimate plant capacity; design conforming to surface water treatment regime; actual operational performances of plants; and financial implications for the clients of the case study plants selected.*

The approach was to compare the design against the recommended and well documented treatment regime applicable to surface water, selection of processes, design and installation using the four methods mentioned above. All the water treatment plants selected as case studies for this report have several inherent design deficiencies which negatively affect the ability to produce good quality potable water, and to promote and facilitate water services delivery most cost effectively in a sustainable manner. Inherent design deficiencies confirmed in the four case study plants include *failure to cater for the ultimate design capacities; failure to utilise the value engineering tools in* *the design; poor selection of processes and their arrangement and negative financial Implications for the owners of these waterworks.*

The report recommends formation of a Water Treatment Design Committee (WDC) or a National Water Agency of South Africa (NWASA) responsible for water capital projects implementation.

Keywords: conventional, water treatment, process, South African Government, waterworks, guidelines, capital project, consultant, engineer, and services delivery.

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Contents

LIST OF ABBREVIATIONS

ACESL	Association of Consulting Engineers, Sri Lanka
AWWA	American Water Works Association
APEGBC	Association of Professional Engineers and Geoscientists of British Columbia
CAPEX	Capital Expenditure
C&AT	Commissioning and Acceptance Testing
CEBC	Consortium of European Building Control
CMA	Catchment Management Agency
COCODAFF	Counter Current Dissolved Air Flotation and Filtration
CPE	Cationic Polyelectrolytes (organic coagulant)
DAF	Dissolved Air Flotation
DBPs	Disinfection By-Products
DBSA	Development Bank of Southern Africa
DDR	Design Development Report
DHS	Department of Human Settlement
DWA	Department of Water Affairs
DWAF	Department of Water Affairs and Forestry
DWS	Department of Water and Sanitation
EBK	Engineers Board of Kenya
ECSA	Engineers Council of South Africa
GAC	Granular Activated Carbon
IESL	Institution of Engineers, Sri Lanka
IRR	Internal Rate of Return
kl/a	kilolitres per annum
kl/d	kilolitres per day
LOX	Liquid oxygen
ML/d	Million litres per day
mg/L	milligrams per litre
NGO	Non-Governmental Organisation
NOM	Natural Organic Matter (dissolved organic pollution)
NTU	Nephelometric Turbidity Unit
NWASA	National Water Agency of South Africa

NW&SMP	National Water and Sanitation Master Plan
O&M	Operations & Maintenance
PAC	Powdered Activated Carbon
PFMA	Public Finance Management Act
PSP	Professional Service Provider
RBIG	Regional Bulk Infrastructure Grant
RDP	Reconstruction and Development Plan
RFP	Request for Proposal
RFQ	Request for Quotation
RK	Republic of Kenya
RSA	Republic of South Africa
SCM	Supply Chain Management
TCTA	Trans-Caledon Tunnel Authority
THM	Trihalomethane
TOC	Total Organic Carbon
ToR	Terms of Reference
VE	Value Engineering
VEA	Value Engineering Assessment
WDC	Water Design Committee
WISA	Water Institute of Southern Africa
WRC	Water Research Commission
WSA	Water Services Authority
WSP	Water Services Provider
WTP	Water Treatment Plant

LIST OF FIGURES

LIST OF TABLES

LIST OF TEMPLATES

CHAPTER 1: ORIENTATION

1.1 Introduction

The South African Government is spending huge amounts of money (about R64 billion for the fiscal years 2015/16 to 2018/19 as per the revised strategic plan – vote 36) and more than R30 billion targeted water infrastructure investments by the end of 2014/15 financial year (Ruiters & Matji, 2015) striving to provide all its citizens with access to good quality water services through its organs of state such as municipalities and water boards. The study of more than one hundred (100) waterworks throughout the country carried out in the early years of the 21st century by Chilton & Polasek (2013) revealed that none of them was found to be appropriately designed in terms of the processes installed. South Africa is facing a water crisis driven by a massive backlog in water infrastructure maintenance and investment, recurrent droughts driven by climatic variation, glaring inequalities in access to water, and deteriorating water quality. This crisis is already having significant impacts on economic growth and the wellbeing of most South Africans. The situation will deteriorate even further if it is not addressed (RSA, Department of Water & Sanitation, 2018).

Rural areas and decentralized municipalities are severely affected in terms of water services delivery, and the challenges regarding access to safe and usable water are most predominant in these regions (Khuzwayo & Chirwa, 2020).

Chilton & Polasek (2013) found that generally the purification processes do not take into consideration the raw water quality. As a result, waterworks are not capable of purifying water to the best attainable quality in the most efficient and economical way in a user-friendly arrangement. The study did not quantify the financial impacts as well as other complications resulting from these design shortcomings. No recommendations were made to correct the situation nor action to be taken to minimise the impacts.

The author of this dissertation aims to bridge the gap in Chilton & Polasek's (2013) study of more than a hundred (100) waterworks, in which they did not quantify the actual impacts caused by the design flaws they identified. Neither did they quantify the actual impacts such as financial losses by clients nor operations and maintenance complications/difficulties due to the design flaws they identified. They did not propose corrective actions to prevent further damage due to design flaws.

The author chose four waterworks/plants, two in Gauteng province and two in North West province as case study plants to address the shortcomings of the study carried out by Chilton & Polasek (2013) The four case study plants were the Bospoort and Vaalkop water treatment plants (WTPs) in North West province and Klipdrift and Roodeplaat WTPs in Gauteng Province. The reason for selecting these plants is that the author of this report operated the Bospoort and Roodeplaat WTPs for a number of years, and that the other two, namely Klipdrift and Vaalkop WTPs are owned by the employer of the author. This would have allowed the author access to financial and other pertinent data that Chilton & Polasek (2013) may have lacked to quantify the impacts of the design flaws they observed.

Due to the complexity of water treatment processes and continuous deterioration of water resources, it is imperative that knowledgeable and experienced water process scientists or at least those that have undergone specialised training in water treatment, together with experienced process controllers, should be guided by scientifically developed process train design guidelines to design process trains or physical arrangements of water treatment works. Process trains should be specific to raw water type, that is, the process arrangement must be based on the raw water quality and the results of field testing (Chilton & Polasek). This is commonly not done in the design of some of our water treatment works.

1.2 Problem statement

Chilton & Polasek (2013) and Schutte (2006) argue that the prime objective of water purification is to produce wholesome drinking water, which is free from health risks for lifelong consumption. This means that all undesirable pollutants that are a health risk must be removed from the water with maximum attainable efficiency and economy. To achieve this, waterworks must be flawless in all design aspects, starting with the raw water quality and its current and projected fluctuations. The waterworks must be designed according to the results of field testing so that they are capable of treating water to its best attainable quality. This is the test that all waterworks must pass.

Acceptable waterworks processes that have been selected, designed and installed should be derived from the quality of the raw water to be purified such that it meets the prescribed quality and avoid potential malfunctioning of both existing and newly installed waterworks (Chilton & Polasek, 2013).

Design flaws in water treatment works result in huge initial capital expenditure and high operating and maintenance costs. These flaws directly and indirectly rob millions of South Africans of access to potable water of the right quality and quantity (Chilton & Polasek, 2013). Master Plan 8.3 (RSA, Department of Water & Sanitation, 2018) states that in April 2017, 5.3 million households in South Africa did not have access to reliable water services, and that 14.1 million people still used sanitation facilities below RDP standards. It goes on to say that only 10.3 million households (63%) have access to a reliable water supply (RSA, Department of Water and Sanitation, 2018).

The aim of this research dissertation is to assist in the development of guidelines to aid in the effective management of water treatment plant design, and in the process to ensure that appropriate treatment processes are installed for the optimal production of good quality potable water. The aim is also to eliminate wastage in the initial capital expenditure, operations and maintenance costs while enhancing delivery of water services. This dissertation covers the following:

1. The consideration of raw water quality.
2. Current design considerations by consulting engineers, comparing and/or appraising them with existing design guidelines.
3. Adequacy or lack thereof of the current design guidelines.

4. Possible causes for the poor designs and flaws in the process selection for specific waters. Specific examples of such water treatment works are critically examined, with the mentioned flaws in the selection of the sequence or process train in the two provinces, namely North West and Gauteng, being highlighted.
5. Most important, the proposed development of guidelines on how to effectively manage water treatment plant designs.

1.3 Outline of the dissertation

A brief summary is given of the content of this dissertation.

Chapter 2 (Literature Review): the chapter introduces the reader to the institutional arrangement of water services provision in South Africa and the implications associated with these institutions. It further discusses other countries and water treatment plant design practitioners, comparing guidelines used in different countries and good practices.

Chapter 3 (Project Scope and Study Methodology): the project scope is defined followed by a description of the research methodology used to gather data and other relevant information in evaluating the four case study plants.

Chapter 4 (Design Deficiencies – Case Studies): specific cases of flaws in the design of specific water treatment plants (four case study plants) in South Africa are discussed, with specific reference to the complications/difficulties, and the costs directly or indirectly associated with these flaws are quantified.

Chapter 5 (Proposed Guidelines for South Africa): guidelines are proposed for optimally managing the implementation of capital projects of water treatment plants.

CHAPTER 2: LITERATURE REVIEW

2.1 Introduction

There are no guidelines and/or systems in place for the effective management of consulting engineers in the design and installation of water and wastewater treatment plants in South Africa as is the case with other regions such as the Great Lakes-Upper Mississippi River Board of State and Provincial Public Health and Environmental Managers (GLUMRB) in the USA. GLUMRB first developed the Water Supply Committee in 1950, which was tasked with the development of water and wastewater designs, the selection of processes and their trains in the quest to ensure optimal quality of the products produced. The 10 US states of the GLUMRB are Illinois, Indiana, Iowa, Michigan, Missouri, New York, Ohio, Ontario, Pennsylvania and Wisconsin province (USA, member States and Province, 2012).

The selection of appropriate and effective treatment processes and proper design and combination of the individual process units are essential for the successful performance of a treatment plant (van der Merwe-Botha, Ceronio, & Borland, 2013). Chilton & Polasek (2013) went further to recommend that the design of water and wastewater treatment plants be classified as a specialised skilled function for South Africa like those of designing and constructing dams, because the cost of not doing so is insurmountable and has profound impacts on the service delivery in the country in general.

The Michigan Safe Drinking Water Act 1976 PA 399, as amended, and the Administrative Rules suggested practice for water works design, construction and operation for type one public water supplies.

The Great Lake-Upper Mississippi River Board of State and Province (aka Ten States Standards) developed comprehensive design standards to be adhered to when designing and selecting processes and their trains for new water and wastewater treatment plants with the last and latest standards developed in 2012 (USA, member States and Province, 2012).

The Master Plan 8.3 (RSA 2018) classed South Africa as a country that is facing a water crisis driven by a backlog in water infrastructure maintenance and investment, recurrent droughts driven by climatic variation, glaring inequalities in access to water, and deteriorating water quality. The Plan further states that this crisis is already having significant impacts on economic growth and the wellbeing of everyone in South Africa, which will be exacerbated if not addressed.

2.2 Water as a world resource

Water is life, in all forms and shapes. Every human being, now and in the future, should have enough clean water for drinking and sanitation, and enough food and energy at reasonable cost. Providing adequate water to meet these basic needs must be done in an equitable manner that works in harmony with nature. Water is the basis for all living ecosystems and habitats and part of the immutable hydrological cycle that must be respected if the development of human activity and wellbeing is to be sustainable (Donkor, Yahaya, Woudeneh & Wright, 2014).

Not so long ago, the world realized that there is a chronic, pernicious crisis in the world's water resources and called for a World Water Vision to increase awareness of the water crisis and develop a widely shared view of how to bring about sustainable use and development of water resources. The World Water Council responded and developed **World Water Vision 2025** as its main programme (Cosgrove & Rijsberman, 2000).

World Water Vision 2025 concludes that there is a water crisis, but it is a crisis of management. We have threatened our water resources with bad institutions, bad incentives and bad allocations of resources. In this we have a choice. We can continue with business as usual and widen and deepen the crisis tomorrow. Alternatively, we can launch a movement to move from vision to action – by making water everybody's business.

2.2.1 World Water Vision 2025

World Water Vision 2025 sees a world where almost every woman and man, girl and boy in the world's cities, towns and villages will know the importance of hygiene and enjoy safe and adequate water and sanitation. People at the local level will work closely with governments and nongovernmental organisations, managing water and sanitation systems that meet everybody's basic needs without degrading the environment. People will contribute to these services according to the level of service they want and are willing to pay for. With people everywhere living in clean and healthy environments, communities and governments will benefit from stronger economic development and better public health (Cosgrove & Rijsberman, 2000).

Making Water Everybody's Business (Cosgrove & Rijsberman, 2000).

World Water Vision 2025 has three primary objectives of integrated water resource management:

1. Empowering women, men and communities to decide on the level of access to safe water and hygienic living conditions and on the types of water-using economic activities that they desire, and to organise to obtain it.
2. Producing more food and creating a more sustainable livelihood per unit of water applied (more crops and jobs per drop), and ensuring access for all to food required for healthy and productive lives.
3. Managing water use to conserve the quantity and quality of fresh water and terrestrial ecosystems that provide services to humans and all living things.

The five key actions of World Water Vision 2025 to achieve these objectives are:

1. Involve all stakeholders in integrated management
2. Move towards full-cost pricing of all water services
3. Increase public funding for research and innovation in the public interest
4. Increase cooperation in international water basins
5. Massively increase investments in water.

World Water Vision 2025 envisages a world in which:

- ***Women and men are empowered***. Water Services will be planned for sustainability, and good management, transparency and accountability will be standard. Inexpensive water-efficient equipment will be widely available. Rainwater harvesting will be broadly applied. Municipal water supplies will be supplemented by extensive use of reclaimed urban wastewater for non-potable uses (even for potable uses in seriously water-short urban areas). At local levels, the empowerment of women, traditional ethnic groups, and the poor and marginalised people will make local communities and weak nations stronger, more peaceful and more capable of responding to social and environmental needs.

- ***More food is produced and water is used more productively***. Extensive field research on water management policies and institutions in developing countries early in the 21st century will have focused on bringing average yields closer to the yields achieved by the best farmers. Closing the yield gap will make the rural livelihoods of poor women and men much more sustainable. Investments in cleaner technologies and reduced water and wastewater use will continue to help many industries lower their production costs while reducing their effluent taxes.

- ***Ecosystems are conserved***. Concerns about polluting groundwater through leaching nitrates and other chemicals will be addressed. Restrictions will be placed on fertilisers, pesticides and other chemicals in recharge areas after research, which will maximise the rate of recharge and control pollution. Water management in 2025 will be based on recognising the environmental goods and services that healthy catchments provide. Innovation in most areas of water resource management – supported by the best of science and traditional knowledge – will accelerate. Governance systems in 2025 will facilitate trans-boundary collaborative agreements that conserve freshwater and related ecosystems and maintain local livelihoods.

2.2.2 Africa Water Vision 2025

Africa as a continent faces a number of serious socio-economic problems that call for urgent remedial action if current trends towards endemic poverty and pervasive underdevelopment are to be turned around. Water resources availability is crucial in changing and accomplishing the needed socio-economic development goals, some of which are embedded in the millennium goals for all African countries. On the face of it, water resources should not be a constraint to such development as there is generally an abundance of it in Africa, particularly surface water (Donkor et al., 2014).

The sustainability of these precious water resources cannot be taken for granted as they are threatened by certain natural phenomena, namely the multiplicity of trans-boundary water basins; extreme spatial and temporary variability of climate and rainfall coupled with climate change; growing water scarcity caused by shrinking of some water bodies and desertification as well as humans factors such as inappropriate governance and institutional arrangements in managing national and transactional water basins; depletion of water resources through pollution, environmental degradation and deforestation; failure to invest adequately in resource assessment, protection and development; and unsustainable financing of investments in water supply and sanitation (Donkor et al., 2014).

Africa Water Vision 2025 further states that these threats brought about by either nature or humans pose challenges to the management of water resources on the continent and to the satisfaction of competing demands for basic water supply and sanitation, food security, economic development and the environment. The document defines milestones and potential packages of actions, such as investments and specific tools needed to achieve the desired end state or vision by 2025, and it complements them with a set of mechanisms for translating inherent commitments in the Vision into actions.

An Africa where there is an equitable and sustainable use and management of water resources for poverty alleviation, socio-economic

development, regional cooperation, and the environment (Donkor et al., 2014).

1. There is sustainable access to safe and adequate water supply and sanitation to meet the basic needs of all.
2. There is sufficient water for food and energy security.
3. Water for sustaining ecosystems and biodiversity is adequate in quantity and quality.
4. Institutions that deal with water resources have been reformed to create an enabling environment for effective and integrated management of water in national and trans-boundary water basins, including management at the lowest appropriate level.
5. Water basins serve as a basis for regional cooperation and development, and are treated as natural assets for all within such basins.
6. There is an adequate number of motivated and highly skilled water professionals.
7. There is an effective and financially sustainable system for data collection, assessment and dissemination for national and trans-boundary water basins.
8. There are effective and sustainable strategies for addressing natural and man-made water resources problems, including climate variability and change.
9. Water is financed and priced to promote equity, efficiency and sustainability.
10. There is political will, public awareness and commitment among all for sustainable water resources management, including the mainstreaming of gender issues and youth concerns and the use of participatory approaches.

The framework for achieving this vision calls for:

- Strengthening the governance of water resources
- Improving water wisdom
- Meeting urgent water needs
- Strengthening the financial base for the desired water future

Africa Water Vision 2025 brings us closer home with the South African water institutional arrangements for the provision of water services to the country's people.

2.3 Institutional arrangements for water services provision in South Africa

The Department of Water and Sanitation (DWS) is responsible for the regulation of the use of water across the country, including the issuing of licenses for water abstraction, waste discharge, dam safety and setting the charges for the use of raw water and discharge effluent. Strong regulation is a critical tool in achieving water security in South Africa in terms of water quality in rivers and taps, balancing supply and demand, ensuring the safety of dams, and meeting the challenges of climate change (RSA, Department of Water & Sanitation, 2018).

Although South Africa has a strong water industry with a record of accomplishment in innovation in wastewater treatment, significant problems remain concerning the functionality of waterworks operation. Another area of concern is the financial sustainability of service providers, without which there will be a lack of attention to maintenance. Uncertainty about the government's ability to sustain funding levels in the sector is also a concern, along with institutional arrangements that are overly complex, resulting in inefficiencies (RSA, Department of Water & Sanitation, 2018). This then leads to discussion by stakeholders on their arrangements and proposed changes in the water sector in South Africa.

Institutional arrangements of water services provision in South Africa are characterised by a series of multifaceted government institutions and departments, which are organised in the following three tiers as per the legislative framework of the water sector of the country:

- The National Government represented by the Department of Water and Sanitation (DWS) as a policy setter.

- Water boards, which provide primarily bulk water, but also some retail services, and operate some wastewater treatment plants, in addition to playing a role in water resources management.
- Municipalities, which provide most retail services and own some of the bulk supply infrastructure.

Banks, the Water Institute of South Africa (WISA), the Water Research Commission (WRC) and civil society are important stakeholders in the sector.

2.3.1 The National Government

The Department of Water and Sanitation (DWS) is primarily responsible for the formulation and implementation of policy governing water resources management as well as bulk supply of raw water, drinking water supply and sanitation. This department was formed to address the concerns regarding sanitation, which was not part of the responsibilities of its predecessor, the Department of Water Affairs (DWA) under the Ministry of Water and Environmental Affairs, function/responsibility of which had been moved in 2010 to the Department of Human Settlement (DHS), but with some regulatory functions remaining with the DWA.

The institutional realignment as revived in 2008 is reported in the Institutional Re-alignment Project report: Emerging Institutional Models for the Water Sector in South Africa (Kubheka, 2008) made some inroads into the implementation of the water sector optimisation project in the country as follows:

2.3.2 The municipalities – Water Services Authorities (WSAs)

The country has 231 **municipalities** (which in 2015 started to be reviewed) which are in charge of water distribution and sanitation directly or indirectly either through municipally owned enterprises or private companies. Government-owned water boards are in charge of operating bulk water supply infrastructure and some wastewater systems; the Trans-Caledon Tunnel Authority (TCTA) finances and develops dams and bulk water supply infrastructure.

According to the Municipal Structures Act 117 of 1998 and the Water Services Act 108 of 1997, responsibility for water and sanitation lies with the Water Services Authorities (WSA), which the Water Services Act defines as the municipalities. There are 52 district municipalities and 231 local municipalities in South Africa. In many cases, the district municipalities are WSAs. However, the National Government can assign responsibility for service provision to local municipalities. Overall, there are 169 water services authorities in South Africa, including water boards, district municipalities, local municipalities and municipal companies.

Since 1994, some municipalities have involved the **private sector** in the service provision in various forms, including contracts for specific services such as wastewater treatment, short-term management contracts and long-term concessions.

The Department of Water Affairs and Forestry's (DWAF) 2013 strategic overview document of the water sector in South Africa reported that in its 2011 Demarcation study, only 72% of the municipal posts were filled and only 76% of municipal organogram posts were budgeted for. Of the funded posts, 33% were vacant and that the average municipal manager remained in his post for three years and possessed only nine years' relevant work experience, while the technical manager had 11 years' experience. The report also stated that half of the technical managers were under-qualified, and were unable to adequately manage their infrastructure, and that there was an ongoing chronic shortage of municipal engineers and a high management turnover, with 25% of management posts being vacant for more than three months. One in six managers left the municipality in the course of the year (RSA, Department of Water Affairs & Forestry, 2013).

2.3.3 The water boards – Water Services Providers (WSPs)

The Water Services Act 108 of 1997 as amended recognises and defines the role of local government as a water services authority and provides powers and functions as defined in the Constitution. The Act further defines the relationship of local government and water boards in a contractual manner, which becomes a prerogative of local government to either benefit from their existence or not after following a particular process. This negates the value that water boards would add to the capacity of local government to deliver services effectively (Kubheka, 2008).

The Municipal Structures Act 117 of 1998 as amended gives powers to local government to provide potable water to its citizens. This Act defines the mechanisms to deliver the functions for which a particular municipality has powers, and does not recognise the existence of water boards, but rather establishes its own entities (not always manned by appropriately knowledgeable and experienced staff). This poses a very serious threat to existing water boards. Kubheka (2008) further mentions that not all water boards have sufficient capacity to deal with all the capacity problems of a particular municipality, even when municipalities have consented in some instances and contracted water boards. This is particularly the case in poor rural municipalities across the country. The unequal capacity landscape in the water sector raises the question as to whether water boards or municipal entities are the way to go. The answer seems to be that the sector, particularly the minister, must look at restructuring and re-aligning the institutional arrangements of the water services as well.

All of the issues raised which have been identified as making the institutional arrangements of the water sector even more complex and in the process, hindering water services delivery is beyond the scope of this report, but is fully discussed in the Institutional Re-Alignment Project (2008) draft 001 by Vusi Kubheka.

This report is limited to water boards as Water Services Providers (WSPs), municipalities as Water Services Authorities (WSAs) and other stakeholders as mentioned below.

Because of the water sector's institutional arrangements, there are now nine (9) government-owned **water boards**, which play a key role in the South African water sector. They operate dams, bulk water supply infrastructure, some retail infrastructure and some wastewater systems. Some also provide technical assistance to municipalities. Water boards derive their mandate from the Water Services Act 108 of 1997 and are categorised as national government business enterprises in terms of schedule 3B of the Public Finance Management Act 29 of 1999. Water boards are separate legal entities that have their own governance structures and assets and are required to be self-funding. The minister appoints board members and chairpersons.

The nine (9) water boards provide bulk potable water services to the municipalities in which they operate, and to other water service institutions and major customers within designated service areas. Water boards vary considerably in size, activities, customer mix, revenue base and capacity. The Botshelo Water, Pelladrift Water and Bushbuckridge water boards were disestablished during the 2013/14 and 2014/15 financial years as part of the institutional re-alignment and reform process. Both Botshelo Water and Pelladrift Water have been incorporated in Sedibeng Water, while Bushbuckridge Water has been incorporated in Rand Water. Most of the older and more established water boards are located in areas where there are significant urban development nodes (such as Rand Water, Umgeni Water and Magalies Water), while other boards operate in more demographically diversified areas (Revised Strategic Plan (Vote 36) for the fiscal years 2015/16 to 2019/20)

In support of the Department's strategic objective of ensuring the effective performance of water management and services institutions, the water board will focus on:

- Quality potable bulk water supplied to municipalities, industries and mines
- Infrastructure development and job creation

Table 1 below shows the infrastructure development expenditure budgets per water board including TCTA from 2014/15 projected to 2018/19, which totals up to about R64 billion excluding more than R30 billion which was targeted water infrastructure investments by the end of 2014/15 financial year (Ruiters & Matji, 2025). This is a huge amount of money requiring close monitoring and optimisation of the funds to facilitate the much-needed water services to all South Africans.

Table1: Entities consolidated capital expenditure

Name of entity	Budget in R'000					Totals
	2014/15	2015/16	2016/17	2017/18	2018/19	
Amotola Water	R 83,756	R 189,903	R 92,353	R 88,807	R 31,600	R 486,419
Bloem Water	R 88,100	R 87,100	R 135,000	R 138,501	R 289,000	R 737,701
Lepelle Water	R 92,236	R 84,170	R 173,160	R 212,994	R 157,740	R 720,300
Magalies Water	R 550,000	R 929,331	R 1,653,841	R 1,112,234	R 1,257,543	R 5,502,949
Mhlatuze Water	R 137,682	R 122,826	R 49,150	R 186,640	R 191,250	R 687,548
Overberg Water	R 2,300	R 8,699	R 66,547	R 30,868	R 6,457	R 114,871
Rand Water	R 2,580,000	R 3,091,500	R 4,113,000	R 4,019,000	R 3,413,000	R 17,216,500
Sedibeng Water	R 262,000	R 63,064	R 74,500	R 30,868	R 6,457	R 436,889
Trans Caledon Tunnel Authority	R 1,693,000	R 1,712,225	R 1,878,250	R 631,235	R 959,822	R 6,874,532
Umgeni Water	R 3,985,000	R 4,370,000	R 5,836,000	R 8,233,000	R 8,651,000	R 31,075,000
Total	**R 9,474,074**	**R 10,658,818**	**R 14,071,801**	**R 14,684,147**	**R 14,963,869**	**R 63,852,709**

Table 1: Entities' consolidated capital expenditure, courtesy of DWS Strategy Plan (vote 36)

2.3.4 Trans-Caledon Tunnel Authority (TCTA)

The TCTA was established in 1986 as a state-owned entity specializing in project financing, implementation and liability management. It is responsible for the development of bulk raw water infrastructure and provides an integrated treasury management and financial advisory service to the Department, water boards, municipalities and other entities linked to bulk raw water infrastructure. It is listed as a schedule 2 major public entity in the PFMA. In contributing to the Department's strategic objective of ensuring the availability of and access to water supply for environmental and socio-economic use, the TCTA will focus on:

- Facilitating water security through planning
- Financing and implementing bulk raw water infrastructure and developing new capability around bulk sanitation provision.

2.3.5 The Water Research Commission (WRC)

The WRC was established in 1971 to generate new knowledge and to promote the country's water research. Its mandate includes promoting co-ordination, cooperation and communication in the area of water research and development; establishing water research needs and priorities; stimulating and funding water research according to priority; promoting effective transfer of information and technology; and enhancing knowledge and capacity building within the water sector. The WRC is listed as a schedule 3A entity in the PFMA.

In contributing to the Department's strategic objective of improving and increasing the skills pool and building competencies within the sector, the WRC focuses on:

- Promoting co-ordination, co-operation and communication in the area of water research and development
- Establishing water research needs and priorities
- Stimulating and funding water research according to priority
- Promoting effective transfer of information and technology
- Enhancing knowledge and capacity building in the water sector
- Developing a strategic framework for water research in South Africa.

2.3.6 Catchment Management Agencies (CMAs)

Meissner, Stuart-Hill & Nakhooda (2017) state that in October 1999, the government established 19 water management areas (WMAs) and at that time, the government contemplated the establishment of 19 CMAs, one in each WMA. The boundaries of these areas are along catchment divides and do not coincide with the administrative boundaries of local and provincial government spheres. These 19 CMAs were later, in 2012, reduced to 9 after the DWS reconsidered management model and viability assessments related to water resource management, funding, capacity, skills and expertise in regulation and oversight. The decision was also an attempt to improve integrated water resource management. The nine CMAs are Limpopo, Olifants (Mpumalanga Province), Inkomati-Usutu, Pongola-UMzimkhulu, Vaal, Orange, Mzimvubu-Tsitikama, Breede-Gouritz and Berg-Olifants (Western Cape). Only two CMAs have been established to date, namely, the Breede-Gouritz and Inkomati-Usutu CMAs.

In contributing to the Department's strategic objective of improving the protection of water resources and ensuring their sustainability, the CMAs will focus on (Revised Strategic Plan (Vote 36) for the fiscal years 2015 to 2018/19):

- Finalisation of catchment management strategies
- Registering water use
- Building catchment management forums
- Facilitating the transformation of irrigation water boards
- Supporting verification and validation (V & V) processes
- Dealing with pollution incidents.

2.3.7 Other stakeholders

Ruiters & Matji (2015) list amongst others the following as other important water resource management stakeholders:

The **Komati Basin Water Authority** (KOBWA) established in 1992, as a bi-national (RSA and Swaziland) implementing agent, for shared water resources between the two countries.

The **Limpopo Watercourse Commission** (LIMCOM) acts as a technical advisor to the contracting parties (Botswana, Mozambique, RSA, and Zimbabwe) on matters relating to the development, utilization and conservation of the water resources of the Limpopo.

The **Council for Geoscience** (CGS) was formed to develop and publish world-class geoscience knowledge products and to render geoscience-related services to the South African public and industry.

The **Agricultural Research Council** (ARG) fund and undertake water research relating to agricultural sector.

South Africa experienced a huge brain drain immediately before and immediately after the 1994 democratic elections, especially the sought-after skills such as experienced engineers and scientists. One reason was the official policy of cadre deployment, whereby persons loyal to the ruling party, the

African National Congress, are given jobs in the different government departments and organs of state. The process puts party loyalty ahead of competence and demoralises public service employees, according to a Twenty Year Review of South Africa background paper from 1994 to 2014: "The new democratic, wall-to-wall local government system inherited many gaps in terms of skills and capacity. The gaps in capacity have been exacerbated by the amalgamation and restructuring of local government, which, together with the countrywide 'brain drain', has left significant areas of local government understaffed and unskilled, rendering them unable, in many instances, to deliver on their mandates and meet their public obligations" (RSA, Department of Local Government, Background Paper, 2014: 11). This paper continues by stating that staffing levels have a direct impact on institutional functionality and the delivery of services – in 2011, the average percentage of total posts filled was 72 per cent, suggesting that, on average, vacancies in municipalities are in the region of 28 per cent.

2.4 Water infrastructure provisions

South Africa is facing a water crisis driven by a massive backlog in water infrastructure maintenance and investment, recurrent droughts driven by climatic variation, glaring inequities in access to water, and deteriorating water quality (RSA, Department of Water & Sanitation, 2018). The document further states that the crisis is having significant impacts on economic growth and on the wellbeing of everyone in the country, which will be exacerbated if not addressed. The following facts highlight the extent of the problem in the country:

- Just over five million households do not have access to safe drinking water
- About fourteen million people do not have access to reliable sanitation
- Around sixty per cent of households have access to a reliable water supply service
- About forty per cent of municipality water does not generate revenue, with thirty-five per cent of this forty per cent is lost through leakage due to infrastructure failures.

- Fifty-six per cent of wastewater treatment works and forty-four per cent of water treatment works are in poor or critical condition and eleven per cent are dysfunctional
- About five per cent of agricultural water is used by black farmers
- Forty-eight per cent of remaining wetlands are critically endangered.

Schutte (2006) states that the main factors that must be taken into account when developing a treatment process train include:

- The source water quality (normally referred to as the raw water quality)
- Seasonal (and other) variations in the raw water quality
- The required treated water quality
- Regulatory requirements.
- Factors such as plant size (capacity), site conditions, availability of skilled labour, degree of automation required, economics and many others.

He continues by stating that process selection for the treatment of water is based on an overall assessment of the quality of the raw water, and this in practice means that water quality and treatment are evaluated in terms of general quality parameters such as turbidity on the one hand, and specific quality parameters on the other, such as the presence of high iron in the raw water. The turbidity of the raw water determines which clarification processes (coagulation-flocculation, sedimentation and filtration) could be used, while the presence of specific substances of concern determines the inclusion of specific processes in the treatment train.

Concise operations and maintenance instructions must be developed specific to the end water treatment facility constructed to enable servicing of all unit operations, plant and equipment in order to keep them sustainably and reliably operating. Also plant commissioning and acceptance testing – the purpose of plant commissioning and acceptance tests is to verify that all project objectives are met and the works are capable of purifying water to its best attainable quality and to do so most effectively and economically. Alternatively, to identify bottlenecks preventing the plant from meeting the project objectives, that is, shortcomings inherent in the plant design (AWW, 1999).

These factors are aimed at assisting the water treatment plant operator to achieve his main goals, namely ***protecting the public's health*, *protecting the environment and protecting the public's purse*** as stated in the Class 1 Water Treatment Plant Operator Program Manual prepared by FSC Architects & Engineers – FSC Project number 2003-0070 (Northwest Territories, 2003).

The Program Manual (Northwest Territories, 2003) and many others assume that water treatment plants that get handed over to operators are designed according to the individual needs of the specific source raw water and associated field tests results, enabling the plant operator to achieve all three goals mentioned above. This operator program manual calls for the multi-barrier approach, which is an integrated system of procedures, processes and tools that collectively prevents or reduces the contamination of drinking water from source to tap in order to reduce risks to public health. The multi-barrier approach is effective where process designs are in accordance with the requirements of the source water to be treated with process train aligned such that systematic, effective and efficient removal of harmful, unsightly substances is achieved with the possible minimal efforts and costs.

The following considerations all influence the selection of the treatment process scheme and facility designs, including the experience acquired through treatment of the same or similar source waters (Crittenden, et al, 2012; Schutte, 2006; Montgomery, 1985):

1. Cost-effectiveness of the system both in terms of capital and O&M costs, including off-site requirements (i.e. pipeline and storage facilities)
2. Overall system reliability
3. Flexibility and simplicity of operation
4. Ability to meet water quality objectives
5. Adaptability of process to both seasonal and long-term changes in raw water quality

6. Capability of process to be upgraded in cases where water quality and/or drinking water regulations are changed (e.g. if a direct filtration plant is designed, provisions for the addition of future sedimentation basins should be included
7. Capability of process to meet both hydraulic peaks and quality excursions (reserve capacity)
8. Availability of skilled operational and maintenance personnel
9. Availability of major equipment items
10. Post-installation service and chemical delivery
11. State and federal requirements
12. Ease of construction of facilities.

The above list of things that need to be considered when designing and/or selecting treatment alternatives is comprehensive and should assist the consulting engineers designing and selecting effective water treatment plants. If followed, the overall plant design should be as simple as possible. This includes a simple, logical arrangement of process units and a minimal amount of equipment sufficient to provide adequate standby capability.

Montgomery (1985) emphasises the importance of simple and basic designs which are frequently the most economic and reliable solutions to water treatment problems. Designers have a tendency to include as much recent technology as possible into the design. There is merit in this progressive concept, but design is a process of compromises, and conscious efforts should be made to test innovations on a “one-at-a-time” basis. Further efforts should be made to minimise the number of units, systems and components to have a cascading cost-saving effect on both capital and operation and maintenance costs.

The solution to a water treatment problem generally depends on five major steps (Crittenden et al, 2012; Montgomery, 1985):

1. Characterisation of the source and definition of the treated water quality goals or standards

2. Pre-design, including process selection
3. Detailed design of the selected alternative
4. Construction
5. Operation and maintenance of the completed facility

These five steps require input from a wide range of disciplines, including engineering, chemistry, microbiology, geology, architecture and financial analysis. Each plays an important role at various stages in the process.

Lifecycle cost must be understood within a meaningful context (CEBC, 2009). The long-term operation and maintenance of infrastructure or building asset costs are in the order of 80 to 93 per cent of the asset's lifetime costs. At one to two per cent, the cost of engineering is a relatively small percentage (see Figure 1 below).

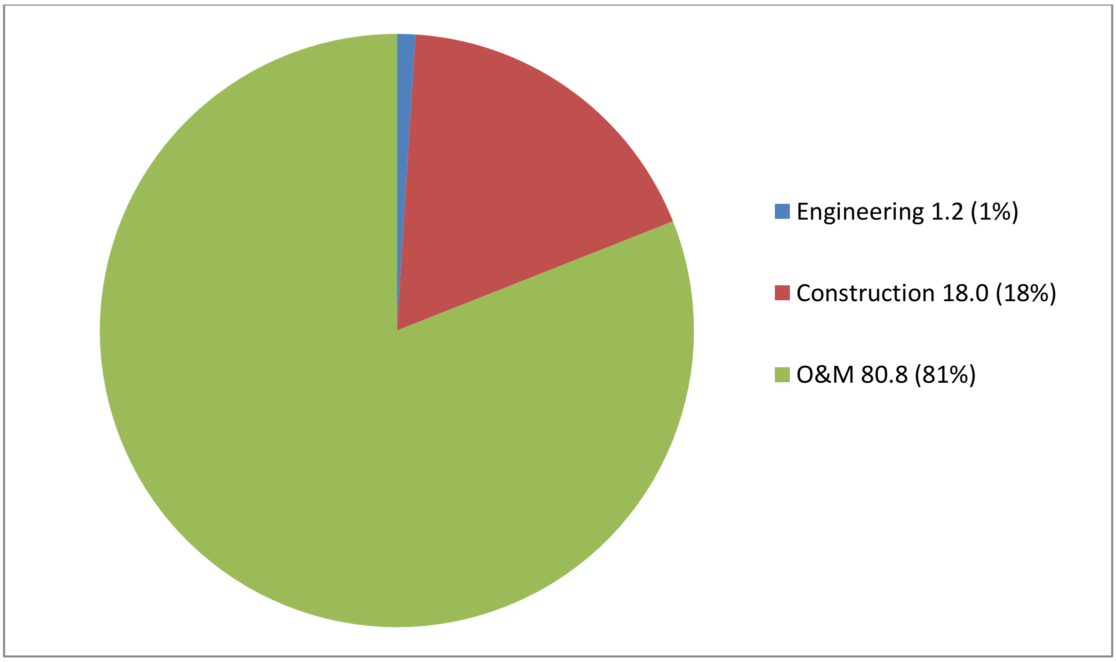


Figure 1: Engineering, Construction and O&M costs split (Courtesy of CEBC, 2009)

However, the role of the engineer is pivotal in meeting the client's objectives because it is during the design process that construction, operations and maintenance cost savings are most easily achieved. Selecting engineering services for apparent least cost is false economy, and can be a disservice to the project and the client. With this, CEBC, (2009) and APEGBC, (2009) recommend a qualifications-based selection (QBS) process for selecting the most competent and qualified consultant for a specific project who shares the client's priorities and interest in achieving the best outcomes for the project and the client. John Ruskin (1819-1900), author and scientist at Oxford University once said, "It is unwise to pay too much, but it is worse to pay too little. When you pay too little, you sometimes lose everything because the thing you bought was incapable of doing the thing you bought it to do" (Consulting Engineers of South Africa, (1998): 85).

Anderson et al. (2009) assert that engineers are generally trained in the hardcore technical expertise in educational institutions that equip them very well for technical and economic analysis skills. These writers further say that while these skills are important for the technical design part of a water treatment plant, if used alone without non-technical factors, such as critical and important parameters that will greatly affect the lifecycle, operations, maintenance and residual disposal costs, the final product will not be the best possible one. To achieve the best possible final product for the client, Hidalgo et al. (2007) in Anderson et al. (2009) propose a system analysis that will not only consider technical design analysis but include social, political, economic, legislative and even climatological features of the area it is intended to serve.

Balkema, Lambert, Otterpohl, Preisig & Weijers (2001) say that technical analysis provides specific insights into performance efficiency and effectiveness. Economic analysis focuses on real costs, and system analysis focuses on the bigger picture, which includes the aspects of cost, technical performance and social, legal and environmental interactions. They further say that in case of conflicting design objectives (optimum design solutions), the search can be for Pareto-optimal solutions where at least most objectives are satisfied without violating the others. This exercise calls for the specialised expertise of different experts in different fields of water treatment practice to be involved in critical evaluation of the process train selection and design.

Chilton & Polasek (2013) argue that the national government is spending huge amounts of money on water services infrastructure. It is questioned whether the government and people of South Africa receive appropriate value for the expenditure due to improper design of purification works with respect to the processes selected and the plants selected for individual units operation. During the past few years the authors inspected more than one hundred (100) waterworks throughout South Africa and not one was found to be appropriately designed in terms of processes and their selection. Generally it was found that the purification processes did not take into consideration the raw water quality. As a result, the waterworks are not capable of purifying water to its best attainable quality most efficiently and economically in a user-friendly arrangement. Furthermore, it was established that inappropriate operation and the lack of maintenance were largely due to the lack of comprehensive O&M manuals.

2.5 Fee structure guidelines for consulting engineers

In principle, a consultant does not competitively bid his or her engineering services. If consultants are selected based on price, the client or owner risks hiring incompetent and inadequate services. It is therefore important for the client or owner to hire the best professional services available by paying an appropriate fee (Kawamura, 2000).

Kawamura (2000) proposed what he calls normal procedure for selecting a professional consultant in the following order:

1. Issue a Request for Qualification (RFQ) for the project and review the qualifications submitted by each firm.
2. Compile a short list of three to five firms based on their experience, knowledge and ability to undertake the project.
3. Issue a Request for Proposal (RFP) to each firm selected. Each company should be asked to submit a detailed presentation of its qualifications and ability to undertake the project. The proposal must include information on the size of the firm, number of staff members, availability of qualified personnel to be assigned to the project, and experience of these engineers in similar lines of work.

4. Select the most qualified and a back-up in case the contract negotiations with the first choice fail.
5. Notify the firm of its selection and begin negotiating the fee for the project and a detailed scope of the professional services that are to be rendered.

Kawamura (2000) further states that several important points should be evaluated during selection of the consultant: technical qualifications of the firm; personality and administrative skills of the key engineers such as the project manager and project engineer; existing workload of the firm (i.e. the ability of the firm to absorb the additional workload in relation to its capacity); the financial stability of the firm; and experience, reputation and past accomplishments of the firm in similar lines of work.

2.5.1. Fee structures

The above discussion brings us to the different fee structures used and/or recommended by different engineering associations. In general, the engineering fee budgets for a project can be based on one or more of the following three methods of calculation (CEBC, 2009):

- **Method 1:** Time Basis
- **Method 2:** Percentage of Cost of Construction
- **Method 3:** Fixed Fee or Lump Sum

The method selected depends largely on the stage of the project, its complexity, and how well it is defined. The three methods mentioned above are briefly defined below as well as their recommended applications depending on the project type, stage and complexity.

Method 1 – Time Basis

This method is generally recommended when the scope of engineering services is difficult to determine, cannot be determined, is not well defined, or when the consultant is not in total control of the required time and disbursements at any stage of the project. All time expended is billable, including travel, time in the consulting engineer’s office and time on the client’s premises or elsewhere.

Method 2 – Percentage of Cost of Construction

This method is suitable for engineering services where the cost of the consulting engineering service is a function of the construction or installation costs, and where the project scope and construction or installation budgets are well defined. Client agreements should clearly define whether the cost of construction is based on an estimate at commencement of a project or on the completed actual construction cost.

Fees for full-time resident engineers are in addition to fees determined under method 2. For full-time resident engineers, Method 1 – Time Basis is recommended.

This method of determining fees for professional consultants (Kawamura, 2005) is not recommended for expansion projects because the design work is generally more detailed and therefore costlier than for new plants.

Method 3 – Fixed Fee or Lump Sum

A Fixed Fee or Lump Sum Contract is suitable if the scope and schedule of the project are sufficiently defined to allow the engineer to accurately estimate the effort required. This method has the merit of simplicity and is satisfactory when the nature, scope and duration of the assignment is known and fixed.

All engineering guidelines by different engineering associations perused (ACESL, 2005; APEGBC, 2009; IESL, 2018; RK & EBK, 2013; RSA DWS, 2016 and RSA ECSA 2014) emphasise the Terms of Reference (ToR) for the consultancy assignment in the preparation of a fee proposal by a consulting engineer. The more comprehensive and precise the ToR are, the more consistent will be the fee proposal. Therefore the ToR for any consultancy assignment should be carefully drawn up, avoiding ambiguities. Furthermore, it is recommended that Value Engineering (VE) be conducted by a team of experts in different fields of the water treatment plant, such as design, environment, operations and maintenance experts (Kawamura, 2005).

Chilton & Polasek (2013) had a very strong view against the normally applied and accepted Professional Service Providers' (PSPs) fee structure, which is based on a percentage of the total project cost. His argument is that this tends to result in maximising Capital Expenditure (CAPEX) cost, whereas the quality of the purified water, life cycle costs, clients' operation skills set, maintenance capability, affordability, sustainability requirements and costs tend to be of secondary concern. This argument is supported by National Treasury Supply Chain Management Note 3 of 2003, which discourages the use of this method. Evidence of CAPEX maximisation can be seen in the excessively large footprints of some of the waterworks versus their outputs, particularly those in northern Gauteng and North West provinces where PSPs have used and are still using this fee structure method to charge clients for their services (RSA ECSA, 2013).

Apart from choosing between the three methods that can be used to determine the fees of consulting engineers for rendering professional services, it is also imperative that clients also get value for their money. The value for money determination is a complete exercise on its own called Value Engineering (VE), which needs to be carried out by a team of experts in different fields contained within the project (Miladi Rad & Yamini, 2016). The following is a brief description of VE.

2.5.2. Value Engineering

Mandelbaum & Reed (2006) define VE as an organized way of thinking or looking at an item or a process through a functional approach. It involves an objective appraisal of functions performed by parts, components, products, equipment, procedures, services, and anything that costs money. VE is performed to eliminate or modify any element that significantly contributes to the overall cost without adding commensurate value to the overall function.

Kawamura (2000) define VE as both systematic and creative method of identifying unnecessary cost within the project with the aim to achieve savings in construction costs, as well as operation and maintenance costs, without sacrificing the effectiveness and reliability of the project. The parameters are tight: maintain the necessary functional rationale for the design project constraints, design considerations, raw water quality and treatment goals, treatability studies, factors affecting decision-making, the rationale for selection of the processes, hydraulic profile, and layout of the treatment plant. He (Kawamura, 2000) continued to say that VE exercise is all about value for money for the client or owner, hence team members conducting this exercise should be most knowledgeable about the design and be able to critically analyse the various costs and effectiveness of the proposed treatment process train, each process unit, equipment and control systems. It is important for the team to review and asses the preliminary cost estimates of the project, as this figure serves as a benchmark for various alternatives generated during the review workshop.

Kawamura (2000) further advises as follow on the formation of the VE review team composition: that it should consist of a minimum of one or two licensed VE engineers and several experts in all the technical disciplines being analysed. He further recommends that the inclusion of academics and scientists be limited only to those who possess both theoretical and actual experience in the design and construction of waterworks or wastewater works.

Gurav & Dolas (2012) defined VE as the systematic effort directed at analyzing the functional requirements of systems, equipment, facilities, procedures and supplies for the purpose of achieving the essential function at lowest total (life-cycle) cost, consistent with meeting needed performance, reliability, quality, maintainability, aesthetics, safety and fire resistance. This means working on the big picture and not individual processes, thus facilitating more thoughtful decisions to improve the value streams of projects.

2.6 Water treatment processes

Van Duuren (1997), and Momba & Swartz (2009) state that designing a water treatment plant not only involves the hardcore technical engineering aspects, but should include the human component. All equipment requiring manual operation should be designed for spatial comfort and ease of effort with the safety of operating staff being of paramount importance.

The other important point made by Van Duuren (1997) is the selection of process train of a water treatment plant, particularly in the phase separation processes. He lists a sequence of processes exactly as in a conventional water treatment plant: intake of raw water; coagulation, flocculation, sedimentation, dissolved air flotation (optional, based on the extent of eutrophication of raw water source) and filtration. This is an important point in regard to where the dissolved air flotation should be placed in the process train of a water treatment plant.

Selection of appropriate and effective treatment processes and proper design of the individual processes and process combinations are essential for the successful performance of a water treatment plant. Selection and design decisions made during the design stage can have major impacts on processes and plant performance and the total project cost. Errors in process selection and design may also have a profound impact on the quality of the final water that can be produced, and may require extensive changes during operation to satisfy treated water quality standards (Schutte, 2006).

Donnenfeld, Crookes & Hedden (2018) classed South Africa as a water-scarce country which has to contend with surface waters concentrated in both natural and anthropogenic organic material in a report written for the Institute for Security Studies titled “A delicate balance, water scarcity in South Africa.” This is exacerbated by the fact that heavy thunder showers and floods cause severe erosion, which results in high surface water turbidity during the summer months. In many catchment areas, low-turbidity eutrophic water is supplanted by high-turbidity floods within minutes. Eutrophic conditions are frequently accompanied by high turbidities. This rather unwelcome plethora of possible feed water quality that must be taken into account in the design of water purification works has caused many a designer inconvenience in time, money and efficiency.

When water treatment engineering first evolved in the early part of the twentieth century, its main goal was to ensure that infectious organisms in drinking water supplies were removed or inactivated (Baruth, 2005), chlorination and filtration practices were applied with tremendous success to the point that major death-causing waterborne disease outbreaks in the United States were virtually eliminated by the 1930s. As a result, for engineers trained in the 1960s, 1970s, and 1980s, both education and industry belief was that all concerns of microbiological contamination in surface waters could be eliminated by providing filtration (with suitable pretreatment) to produce water of sufficient clarity (turbidity less than 1.0 or 0.5 NTU) and then chlorinating. Groundwater was thought to be already filtered, requiring only chlorination to maintain a distribution system residual. Any additional treatment was generally considered necessary only to address non-health-related parameters, such as excessive hardness or water discoloration caused by iron and manganese. Baruth (2005) further state that the principal challenge to water treatment engineers in the 1960s and 1970s was engineering cost-effectiveness: how to accomplish these simple treatment goals at the lowest total cost to the water utility. Thus, in these decades many new techniques and processes were developed to clarify surface water economically. These developments included improvements to sedimentation basin designs; high-rate clarification processes such as tube settlers, plate settlers, and dissolved air flotation; high-rate filtration processes; and proprietary pre-engineered or package equipment integrating flocculation, settling, and filtration processes. In the 1970s and 1980s a new drinking water concern arose: the potential long-term health risks posed by trace amounts of organic compounds present in drinking water.

A wave of regulations ensued with new maximum contaminant levels (MCLs) established for total Trihalomethane (TTHMs), pesticides, and volatile organic chemicals (VOCs). This trend continues today. In response to this concern and resulting treatment needs, water treatment engineers have successfully devised new methods of water treatment to remove organic compounds. These methods, such as air stripping, activated carbon adsorption, and enhanced coagulation, have been a primary focus of water treatment engineering over the last 20 years (Baruth, 2005). He further state that engineers who design water treatment systems today face many challenges, the most important of these are described as follows:

1. **Integrated Treatment Systems.** Traditional treatment engineering has focused on the treatment plant as the sole vehicle for controlling drinking water quality. The engineer's role was to characterise the quality of the source water entering the plant and devise treatment facilities to produce water meeting drinking water standards. The point of measurement for drinking water standards was the finished water exiting the plant. Today's engineer must view the water treatment plant as only a major component in a multistep treatment process. This process includes consideration of the path that the water travels upstream of the plant in the watershed and the elements of the water trans- mission and distribution system downstream of the plant. Changing water quality must be managed in each of these steps, and new regulations require that drinking water standards be met at the customer's tap.

2. **Regulatory Uncertainties.** The definition of "safe" drinking water, which remained relatively fixed in the 1950s, 1960s, and 1970s, now seems to be constantly changing or under review as the water utility industry grapples to understand the potential health effects of trace amounts of an increasing variety of chemical compounds and infectious organisms. Today's treatment system engineer, in addition to addressing current drinking water standards, must anticipate potential future requirements. A water system designed today must be designed with sufficient flexibility to be modified to meet these potential requirements. Regulatory uncertainties extend to other environmental concerns important to water treatment plant design, including waste management practices, chemical storage and feed operations, and workplace safety.

3. **New Technologies.** The state of the art of water treatment plant design is continually changing as new technologies emerge, offering new unit processes for water treatment or making currently used processes more efficient or economical. In addition, advances in computer technology

and building materials are rapidly changing and improving the support systems associated with water treatment plants.

4. **Multidiscipline Teams.** A water treatment plant engineering design team traditionally consisted simply of a small group of civil engineers. This single-discipline team performed the majority of design work for virtually all plant components. Support disciplines of architects and structural, electrical, and mechanical engineers were used to execute the basic decisions made by the design team. Today, the complexity of project and regulatory requirements dictates that a far more multidiscipline approach be used. Typically, a small group of civil engineers remains as the "project" engineers, but this group uses the expertise and resources of many different specialists to execute the design. In addition to traditional design support disciplines, these may include:

    - Process engineers
    - Plant operations specialists
    - Instrumentation and control engineers
    - Health and safety specialists
    - Environmental scientists
    - Specialists in environmental permitting and public participation

Major design decisions today are no longer made unilaterally by the project team. Instead, a consensus is reached after participation by members of the design team and by individuals outside the team, including owners, operators, regulatory agencies, and the general public.

Baruth (2005) further define water treatment design project development as a project that passes through many steps between the time when the need for a project is identified and the time that the completed project is placed into service. The period before construction commences can generally be divided into the following phases:

1. **Master planning**. Treatment needs and feasible options for attaining those needs are established in a report. In subsequent phases, this report may be periodically updated to adjust to both system and regulatory changes.

2. **Process train selection.** Viable treatment options are subjected to bench, pilot, and full-scale treatment investigations. This testing program provides background data sufficient in detail to enable decisions on selecting the more advantageous options for potential implementation. These tests provide design criteria for major plant process units.

3. **Preliminary design.** In this "fine-tuning" procedure, feasible alternatives for principal features of design, such as location, treatment process arrangement, type of equipment, and type and size of building enclosures, are evaluated. In this phase, preliminary designs are prepared in sufficient detail to permit development of meaningful project cost estimates. These estimates help in evaluating and selecting options to be incorporated into the final design and allow the owner to prepare the required project financial planning

4. **Final design.** Contract documents (drawings and specifications) are prepared that present the project design in sufficient detail to allow for gaining final regulatory approvals, obtaining competitive bids from construction contractors, and actual facility construction. Many technical and nontechnical individuals must be involved, not only during the four phases of project development, but also between these phases to ensure that a project proceeds without undue delay. In addition to the engineer's design staff and the owner, these may include public health and regulatory officials, environmental scientists, and the public affected by both the proposed construction and the future water supply services to be provided. The process train selection phase is only briefly covered in this book. Theory and procedures needed for this phase are the focus of Water Quality and Treatment. It is important that the interface between phase 1 and phase 2 and between phase 2 and phase 3 be carefully

coordinated to allow uninterrupted continuity of design. In other words, viable options developed for consideration in phase 1, master planning, should provide a base for developing unit process test studies in phase 2. The process train selected in phase 2 provides the basis for phase 3, preliminary design, in which other factors influencing design are included in the evaluations before criteria for final design are developed and finalized. Careful coordination of the various phases and entities involved provides the owner and the engineer with the opportunity to develop the most advantageous treatment solutions and designs, and helps avoid pitfalls in the schedule and decisions that might add to the cost of the project.

Baruth (2005) states that while the treatment rules become more demanding, the list of available "tools of the trade" is also expanding. It is up to the designer to take advantage of the many treatment options text that are best adapted to the particular plant application influenced by the quality of raw water. He further says that it is essential that issues other than treatment capability be investigated for each option and each treatment train. These other issues may include the following (not necessarily in order of importance):

- Construction cost
- Annual operation costs • Site area required
- Complexity of operation (required capability of operating staff and laboratory monitoring)
- Operation risk (most common causes, if any, of treatment failure)
- Flexibility of plant arrangement for future changes
- Waste disposal options

Consideration of viable options would also be critical to provide a flexible facility arrangement in which additions and modifications may be made for future treatment requirements. Drinking water treatment design is not static; it is a dynamic, ever-changing process.

The Master Plan 8.3 (RSA, 2018) also classed South Africa as an arid to semi-arid country, with an average annual rainfall of 465 mm (half the world average), producing a total annual runoff of approximately 49 000 million cubic metres of water, of which 70% of the current reliable yield of approximately 10 200 million cubic metres per annum of surface water at an acceptable assurance is stored in the country's 252 largest dams versus the total realistic accessible groundwater potential of some 4 500 million cubic metres per annum, of which only between 44% to 67% is currently being used. These facts lead to the conclusion that South Africa's available and used water resources are mainly surface water (RSA, 2018).

Momba & Swartz (2009) list the following as the critical design aspects that design engineers should not miss out when designing a water treatment plant:

- The design of a specific treatment plant should be based on the results of a detailed characterisation of the raw water sources. Design of unit treatment allowing for ease of cleaning and maintenance of the structures and equipment.
- Technology applied should be appropriate to the capacity of the community/owner to manage and operate the plant. Complicated treatment schemes should be avoided where ever possible. Every drinking water treatment plant should be provided with a comprehensive set of operating instructions contained in a well written operational manual, which should be supplied by the design engineers during commissioning of the plants.
- Design engineers should provide (or facilitate) full training for the designated process controllers. If it is found that non-compliance is ascribed to design shortcomings or inadequacies, this should be pointed out to the municipal engineer so that it can lead to the upgrading or extension of the treatment plants.

Chilton & Polasek (2013) claim that most problems experienced by operators and maintenance personnel of the hundred waterworks that they visited in South Africa are directly and indirectly due to design deficiencies which adversely affect the performance efficiencies of these waterworks, resulting in inferior quality of water produced. Appropriately designed processes and the operation of correctly selected units minimise the operational and maintenance requirements and costs which are incurred by the operating entity. They further claimed that some processes and plant designs replicate the mistakes of the past. These have become common practice in South Africa. It should be highlighted that a small overall footprint of waterworks and indoor installation improve operation and performance efficiency, minimise maintenance and assist in creating a pleasant working environment.

The design of the water treatment plant should be based on internationally accepted design guidelines as contained in various local and international hand books and guideline documents (Momba & Swartz, 2009)

The above facts then lead us to the following discussion on conventional water treatment processes for surface waters and some advanced treatment technologies that are sometimes added to treat specific types of raw water pertinent to certain areas of the country. All surface waters require disinfection regardless of the treatment process train chosen (AWWA, 1999).

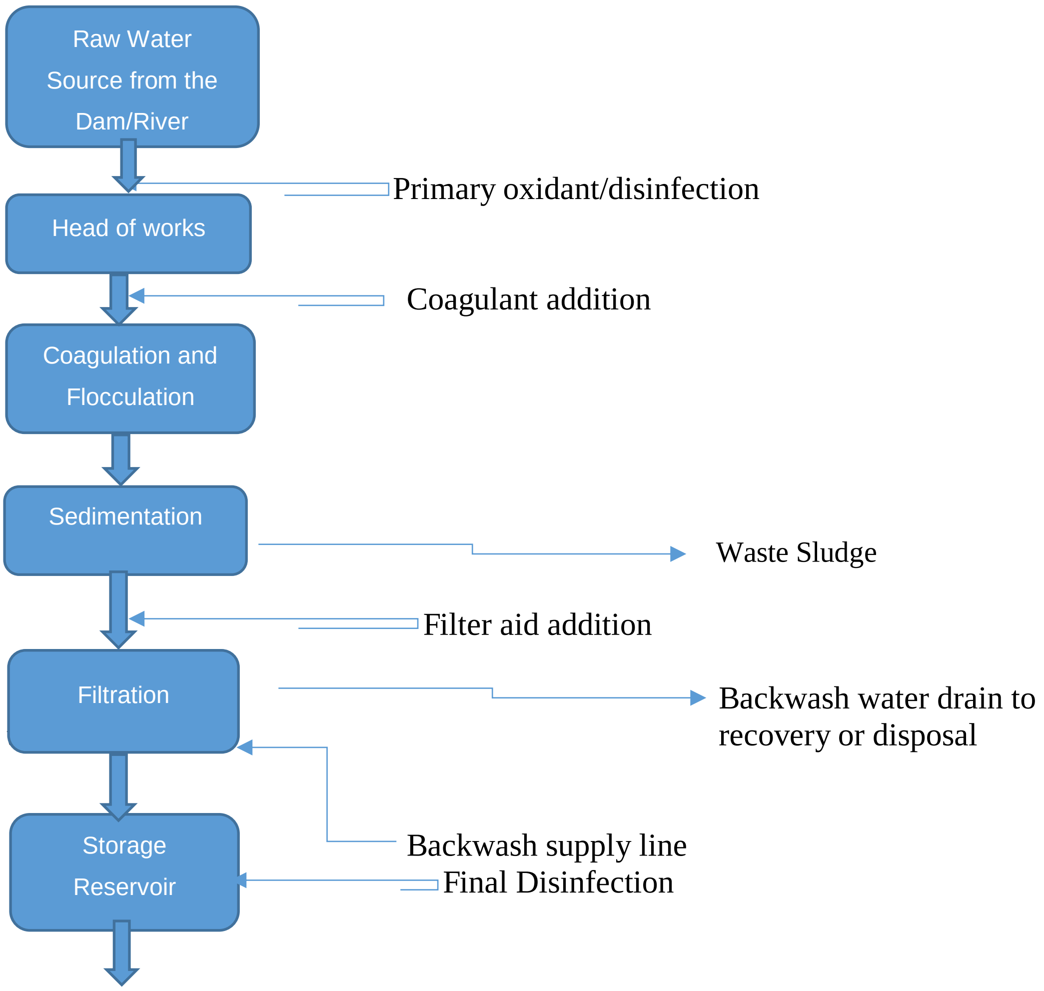
2.6.1 Conventional water treatment processes

AWWA (1999) concluded in the water treatment studies done at Louisville in the 1990s that effective pretreatment, including clarification, was necessary for effective filtration of turbid or muddy surface waters, and in the decades following this work, a treatment train consisting of chemical feed, rapid mixing, flocculation, sedimentation and filtration came to be considered conventional treatment. The Handbook further states that conventional treatment (see Figure 2) has been the norm for water treatment process requirements in the Ten State Standards for surface waters with turbidities ranging from 10 NTU to a high of over 1 000 NTU during floods since 1997.

Schutte (2006), AWWA (1999) and USA, Member States & Province (2012) define conventional water treatment as the treatment suitable for surface water in a series of processes aimed at removing suspended and colloidal material from the water, disinfecting the water and stabilising the water chemically. For the purposes of this dissertation report, the following conventional water treatment processes as shown in Figure 2 below are discussed:

- Surface water as raw water source – dams
- Disinfection, specifically chlorine, chlorine dioxide and ozone
- Sedimentation, specifically rectangular tank design
- Filtration
- Other treatment processes for the purposes of this report.

Conventional treatment, surface water Flow Diagram



Potable Water to distribution system

Figure 2: Conventional Treatment, surface water (Adopted from AWWA handbook, 5th edition)

NOTE: Chemical application points may be different than shown above. This is one potential alternative

2.6.1.1 Dams

South Africa, as mentioned above, has an arid to semi-arid climate and uses surface waters stored in approximately 5 000 registered dams, of which 3 832 are small (RSA, 2018) and serve farms and municipalities. Agriculture is the largest water user at 61% of total water use, followed by municipal use at 27% (including industrial and commercial users provided from municipal systems), with power generation, mining and bulk industrial use, livestock and conservation and afforestation jointly making up the remaining 12% (see Figure 3 below). If demand continues to grow at current levels, the deficit between water supply and demand could be between 2.7 and 3.8 billion m^3/a by 2030, a gap of about 17% of available surface and ground water.

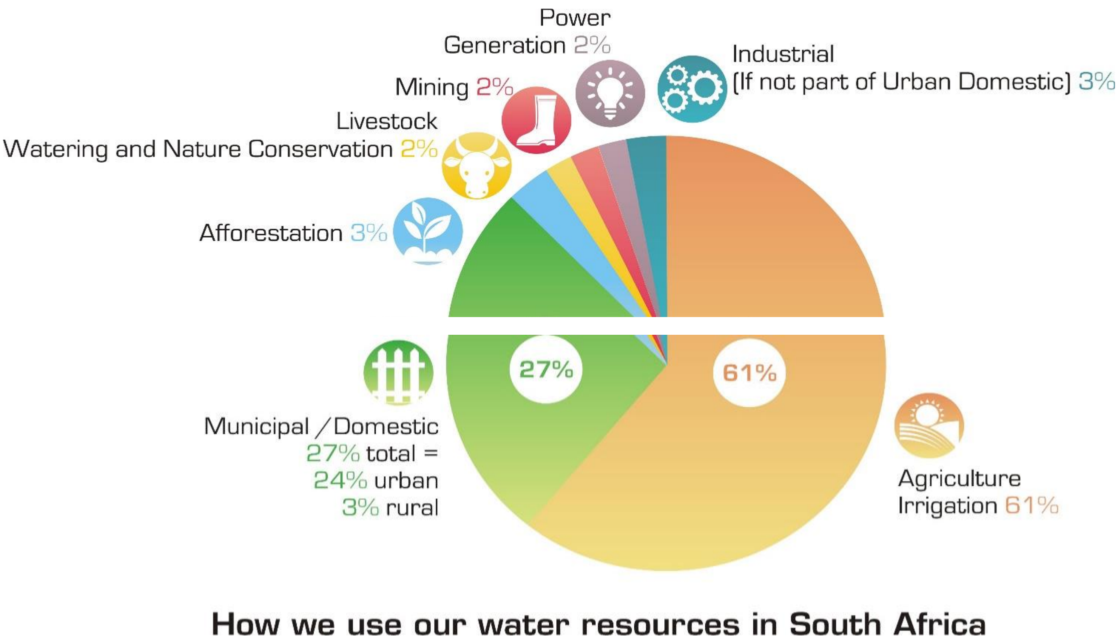


Figure 3: Current water use by water sector (Courtesy of the Master Plan 8.3, RSA, 2018)

According to the recent study as per the Master Plan 8.3 (RSA, 2018); the average domestic water use in South Africa is around 237 litres per person per day, 64 litres per person per day more than the world average of 173 litres per person per day. The high water use is partly due to municipal non-revenue water, which is currently about 41%. The Master Plan further states that since large numbers of South Africans use very small amounts of water per day, this average masks the high water use by privileged sectors of the population. These figures vary greatly between municipalities and service providers, average physical losses in municipal systems are estimated to be around 35%, against the global best practice of approximately 15%. South African municipalities are losing about 1660 million m^3 per year through Non-Revenue Water (NRW). At a cost of R6/m^3 this amounts to R9.9 billion each year.

Most water treated for domestic use is abstracted from these dams and should be subjected mainly to the treatment train as depicted in Figure 2 above, with some variations where deemed necessary. For the purposes of this report, the treatment processes, namely disinfection, coagulation, flocculation, sedimentation and filtration will be discussed, touching mainly on design adequacies, selections and physical arrangements or trains.

2.6.1.2 Disinfection

Disinfection is a process designed to deliberately reduce the number of pathogenic microorganisms during water treatment processes such as filtration or coagulation-flocculation-sedimentation as well as the use of a variety chemicals and/or physical agents (AWWA, 1999). AWWA (1999) further lists the chemicals used in water treatment as chlorine, chlorine dioxide, ozone, UV radiation, heat, extremes in pH, metals (silver, copper), surfactants, permanganate, UV radiation, and electron beam irradiation. However, their feasibility in drinking water is uncertain.

Chlorine

Chlorine remains the most widely used disinfectant and oxidant in water treatment, and the following are important to note (courtesy of FSC Architects & Engineers, 2003):

- For water supplies where it is uncertain whether pathogenic organisms are present or not, the minimum available free chlorine residual should be 0.5 mg/L following a contact time of at least 20 minutes (a minimum of 2 hours is recommended as best practice).
- Chlorine residuals in excess of 0.5 mg/L do little to improve disinfection and can give an unpleasant taste and odour to the water.

The above requirement of using chlorine should be included in the design and installation of the water treatment works for optimum disinfection of potable water, such that breakpoint chlorination and a residual of up 0.5 mg/L are achieved (see Figure 4 below).

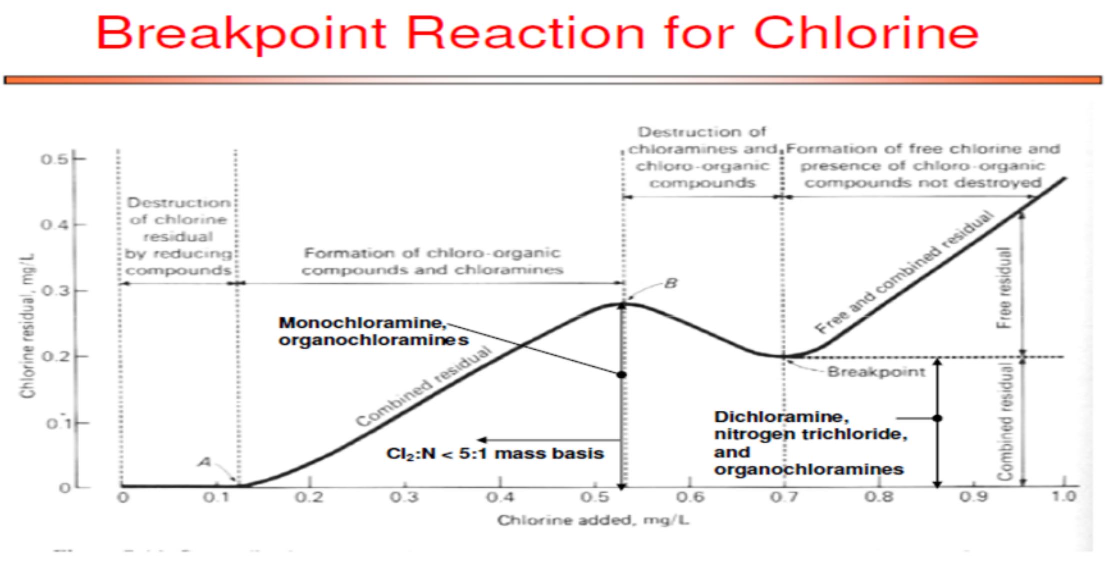
Schutte (2006) describes how breakpoint chlorination is achieved in the presence of dissolved constituents in water as follows: chlorine reaction with any compound containing a nitrogen atom with one or more attached hydrogen atoms will form compounds classified as N-chloro compounds or chloramines that are distinctly organic or inorganic. In the presence of an ammonia ion, free available chlorine reacts in a stepwise manner to form chloramines depicted in the simplified reactions given below:

$NH_4OH + HOCl \leftrightarrows NH_2Cl$ (Monochloramines) $+ 2H_20$

$NH_2Cl + HOCl \leftrightarrows NHCl_2$ (Dichloramine) $+ H_20$

$NHCl_2 + HOCl \leftrightarrows NCl_3$ (Trichloramine) $+ H_20$

The compounds monochloramine, dichloramine and trichloramine together form the total **chlorine residual** in the water, and the term **total available chlorine** is used to indicate the sum of the **free available chlorine** and the **reactive chloramines**.



Ref: Metcalf & Eddy, Inc., Wastewater Engineering, Treatment and Disposal. McGraw-Hill, New York.

Figure 4: Breakpoint chlorination curve

Breakpoint chlorination is achieved in the presence of ammonia-based compounds (chlorine-demanding compounds) when chlorine is added to water until the chlorine demand has been satisfied (FSC Architects & Engineers, 2003; Schutte, 2006). At this point, further addition of chlorine will result in a free residual chlorine that is directly proportional to the amount of chlorine added beyond the breakpoint. Thus, the process through which the ammonium compounds are oxidised is called breakpoint chlorination as illustrated in Figure 4 above. When more chlorine is progressively added and reacts with the available compounds, the measurable chlorine concentration (only detectable as combined chlorine) will increase up to a maximum point indicated by a "hump" in Figure 4 above. If more chlorine is added, the detectable chlorine concentration will decrease until a minimum is reached at the "dip" in the curve, as oxidative destruction of combined residual chlorine/ammonium compounds accompanied by loss of nitrogen occurs. Finally, after the ammonia nitrogen has been oxidised completely, the chlorine residual will consist almost exclusively of free available chlorine. Therefore, if more chlorine is added beyond this point, the free available chlorine concentration will increase in proportion to the amount added.

Schutte (2006) continues by saying that it is crucially important to maintain a chlorine-to-ammonia (as nitrogen) ratio, as measured on a mass basis, of 5:1 (CL_2:NH_3 as N) or a 1:1 molar ratio, so that all the ammonia is converted to monochloramine and not oxidised further to di- and trichloramine, as these compounds could impart tastes and odours to the water. If the ratio is below 5:1, not all the ammonia will react to form monochloramine, and if the ratio exceeds 5:1 and approaches 7.6:1, di- and trichloramine will be formed and eventually breakpoint will take place.

The American Water Works Association's (AWWA) Manual M56 (AWWA M56, 2006) states that free chlorine and chloramine, which are two disinfectants used in the distribution system, each has advantages and disadvantages – free chlorine provides a strong disinfectant residual but reacts with organic matter to form disinfection by-products, while chloramine has a lower disinfection strength than free chlorine, but provides a more stable residual and halts the formation of Trihalomethane (THM) and haloacetic acids. The Manual states that monochloramine is the desired inorganic species to form in drinking water treatment and should be maintained in the distribution system. Monochloramine is preferred because it does not normally cause significant taste and odour problems, while dichloramine and trichloramine are known to produce detectable chlorinous tastes and odours at relatively low concentrations. The AWWA M56 (2006) claims that the microbiological water quality of a distribution system deteriorates within four to five days due to an increase in microorganisms (microbiological after growth) if free chlorine alone is present. The microbiological quality in the distribution system can be maintained for up to ten or eleven days if a more persistent, longer-lasting disinfectant such as monochloramine is used. Chloramination as a secondary disinfection step is ideal following primary breakpoint chlorination. Primary breakpoint chlorination happens when all the naturally occurring ammonia has been converted to nitrogen as well as all associated oxidisable compounds have been destroyed/consumed, resulting in any additional added chlorine remaining as free residual chlorine in the water.

Chlorine dioxide

Chlorine dioxide (CLO_2) has been used as a potable water disinfectant and oxidant since the early 1940s in Europe, the USA and Canada (Schutte, 2006). Initially it was used purely for disinfection and removal of iron, manganese and taste and odour-causing compounds. More importantly, it was later discovered that it does not react with organic compounds to form Trihalomethane (THM) or organo-halogen compounds (TOX) when used in water for human consumption (Rovel, Mouchet and Andriamirado, 2004). Chlorine dioxide is a more effective biocide than chlorine. It destroys viruses, bacteria and spores as well as biofilms in pipelines and channels. Its effectiveness is not affected by ammonia as it does not react with ammonium compounds and therefore none of its oxidative power is lost in a “breakpoint” reaction as is the case with free available chlorine (Schutte, 2006).

Chlorine dioxide (ClO_2) is produced by the acidification of either sodium chlorate ($NaClO_3$) or sodium chlorite ($NaClO_2$) with a suitable acid such as hydrochloric or sulphuric acid, or by a direct reaction between chlorine and sodium chlorate. The conversion rates and efficiency of these reactions are dependent on mixing in the correct proportions and a reduced pH.

The following reactions illustrate the possible methods used to produce chlorine dioxide for application in water:

$2NaClO_2 + Cl_2 \leftrightarrows 2ClO_2 + 2NaCl$

$2NaClO_3 + 4HCl \leftrightarrows 2ClO_2 + Cl_2 + 2NaCl + 2H_2O$

$2NaClO_3 + H_2SO_4 + SO_2 \leftrightarrows 2ClO2 + H_2SO_4 + Na_2SO_4$

The residual products of chlorine dioxide in water are always a chlorite or chlorate ion compared to chlorine, which is reduced to a chloride ion. The allowable concentration of these compounds in drinking water limits the concentration at which chlorine dioxide may be applied as both chlorite and chlorate may pose a health risk as they are linked to methaemoglobinaemia and haemolytical anaemia (Schutte, 2006).

Ozone

Ozone (O_3) was first discovered due to its very characteristic odour. The term ozone originates from the Greek "*ozein*", to smell, and was coined by C. F. Schonbein in 1840 (Eriksson, 2005). Today ozone is a known powerful oxidant and is used in many industrial applications, namely for water treatment and odour control, as a chemical oxidant, and for bleaching and cleansing of semiconductors (Rajagopaul et al., 2008; Mandavgane & Yenkie, 2011). However, the performance of this oxidising agent is affected by its rather quick decomposition in aqueous solutions, which is complex, and is affected by many properties such as pH, temperature and substances present in the water (Rajagopaul et al., 2008; Eriksson, 2005) They further argue that additives can either accelerate the decomposition rate of ozone or have a stabilising effect on the ozone decay. This implies that by controlling the decomposition of ozone, it is possible to increase the oxidative capacity of ozone. Table 2 below summarises the different features of ozone as a disinfectant.

Table 2: Summary of the different features of ozone as a disinfectant.

Method of application	Equipment required	Mechanism of action against microorganisms	Advantages/positive aspects	Limitations/Negative aspects
Ozone is poorly water-soluble and equipment is specially designed to effectively transfer the ozone gas to the water phase. Counter current bubbling, turbine mixers or under induced pressure in a U-tube arrangement are used. Ozone is toxic and the gas that is not absorbed must be removed from the off gas before release to the atmosphere. This is mostly done by catalytic destruction.	Ozone must be produced on site. The most practical method of generation remains the corona-type discharge at high voltage (4000 to 30 000 volts), from dry air or oxygen.	Ozone forms free radicals HO_2 *, and HO*, which are the reacting species. It kills bacteria through the disintegration of the cell walls	Enhances coagulation and micro flocculation. Limits THM formation (only if chlorine is not used for final disinfection). Is a strong oxidant, destroys taste and odour causing organic compounds, and inorganic compounds such as iron and manganese. Biological activated carbon (BAC) is formed on activated carbon. The action of ozone is not affected by pH.	Ozone is instable and no persistent residual is formed in water. Higher levels of assimilable organic carbon (AOC) compounds are formed that lead to after growth of microorganisms. End products of ozone with chlorine and bromine is chlorate and bromate. Need special equipment to determine ozone concentration.
Table 2: Summary of the main features of ozone disinfection (Schutte, 2006)				

The use of ozone, particularly as a pre-oxidant for water, is gaining momentum in South Africa for oxidation in the treatment of raw water with high levels of iron and manganese, colour through the presence of humic acids, tastes and odour, and chlorophyll 'a' (Rajagopaul et al., 2008). These authors caution against the use of this powerful oxidant without a systematic and rigorous analysis due to the fact that it is very energy intensive and costly – ozone production and operation account for 35% of the total energy consumption of water treatment works (Friedrich & Buckley, (2002) in Rajagopaul et al., 2008) and has the potential to form harmful disinfection by-products (DBP).

Mandavgane & Yenkie (2011) assert that pH (see Table 3 below) is one of the most important parameters related to the degree of ozone dissociation. In their study, they categorically conclude that the rate of degradation of ozone increases in the neutral to alkaline range compared to acidic pH conditions (contrary to Schutte's claim in Table 2 above), and this follows the second-order reaction depicted as

$$dC/dt = kC^2$$

Where k = second-order reaction rate

C = concentration of substrate

Ozone decomposition in aqueous solution is a complex, radical type of chain reaction which is very sensitive to conditions applied (Taube & Bray, 1940). The presence of trace amounts of impurities acting as scavengers or promoters, irradiation by light, a change in the ionic media, pH, etc. may each significantly affect the lifetime of ozone in aqueous solution. Decompositions follow a second-order kinetics and the mechanism can be proposed as:

$$0_3 \leftrightarrows O_2 + O \qquad (1)$$

$$O_3 + O \rightarrow 2O_2 \qquad (2)$$

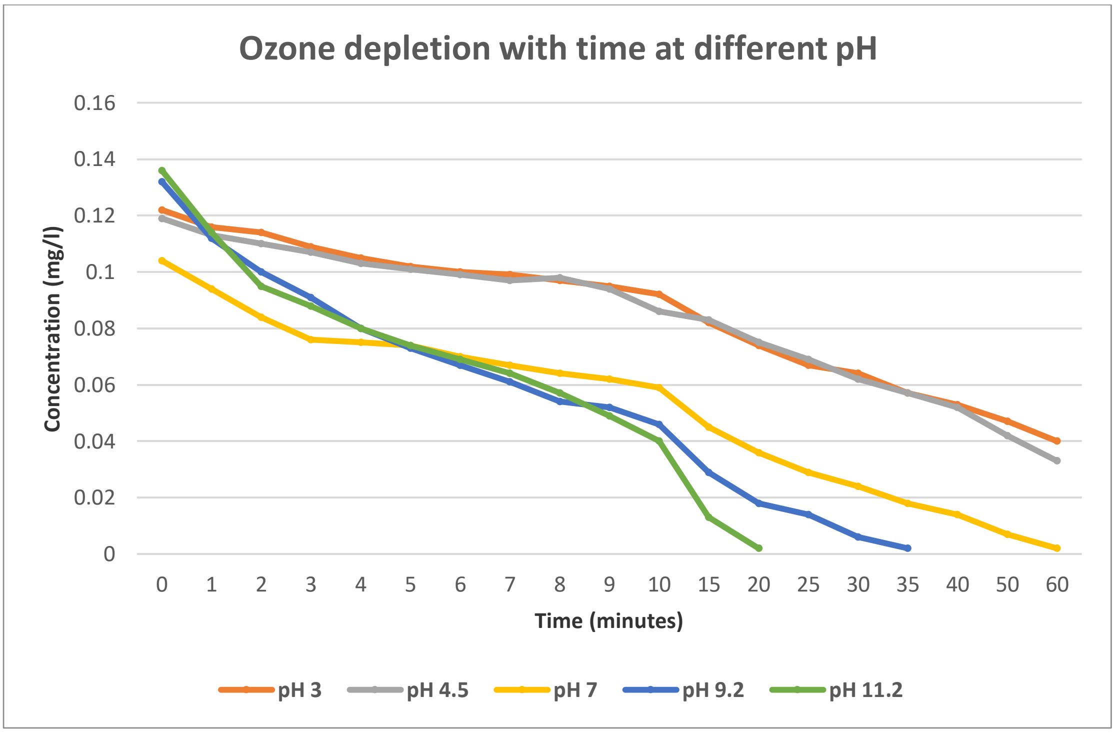


Figure 5: Ozone concentrations decrease with time and increasing pH (Mandavgane & Yenkie, 2011)

Mandavgane & Yenkie (2011) concluded the following:

1. The self-decay of ozone in aqueous medium investigated could be well simulated with a second-order reaction rate form.
2. The effect of pH of the medium on self-decay of ozone was studied, and it was found that the rate of degradation increases with the increase in pH. The rate of degradation increases from acidic to neutral to alkaline pH, which is attributed to the increase in the number of hydroxyl radicals.

Ozonation should be carried out after sand filtration and final pH adjustment together with stabilisation of the purified water thereafter (Rajagopaul et al., 2008; Mandavgane & Yenkie, 2011). By so doing the optimisation of the reaction conditions of the treatment process will result in considerable reduction in operating cost and improved quality of water produced. Consideration of material selection of ozone equipment is very important as it has a great influence on the operation and maintenance of the ozone plant in terms of lifecycle cost. Major components are air/oxygen feed, ozone generation, ozone dosing and contacting, ozone dose control and ozone destruction (Rajagopaul et al., 2008; Schutte, 2006; Eriksson, 2005). Table 3 below lists the advantages and disadvantages of different feed gases for ozone production.

Table 3: Advantages and disadvantages of various feed gas supply for ozone generation (Rajagopaul et al., 2008)

Feed Gas	Advantages	Disadvantages
Air	More common More prevalent for small ozone systems	Problematic in dusty, high humidity conditions. Higher specific energy (kWh/kg O3). Largest gas handling requirement. Maximum O3 concentration, 2.5% by weight.
High Purity Oxygen (LOX)	Simplest system. Lowest specific energy (kWh/kg O3). Least capital cost. Most cost competitive with high efficiency generators.	Variable operational cost due to LOX purchase. Depending on location, transport costs may be prohibitive.
High Purity Oxygen (Cryogenic generation)	Suitable for large ozone applications.	Capital intensive. Complex to operate and maintain.
High Purity Oxygen (Pressure swing adsorption, PSA, air separation)	Alternate to LOX in very small, small and medium ozone systems. Simple system.	Energy costs and operating costs higher than for VPSA system.
High Purity Oxygen (Vacuum swing adsorption, VSA, air separation)	Preferred over PSAs for larger ozone systems. Lower operating and energy costs relative to PSA.	High level of maintenance required.
Table 3: Advantages and disadvantages of various feed gas supply in Ozone generation (Rajagopaul et al, 2008)		

Most authors (USA, Member States and Province, 2012; Polasek, 2013; Rajagopaul et al., 2008; AWWA, 1999; Montgomery, 1985; Kawamura, 2000) of water and wastewater designs and process selection criteria list skills set of both maintenance and operating personnel as one of the prerequisites to be considered when designing water or wastewater treatment plants. Should the skills set be a deficiency in one or more of the selected processes, the consulting engineers must advise the client of this and arrange to have them acquired.

Rajagopaul et al. (2008) state that several feed gas options must be considered and evaluated, particularly against the available skills set of both operation and maintenance personnel, amongst others. In situations where a strong operational and maintenance team is lacking, less maintenance-intensive processes would be the preferred option. Liquid oxygen (LOX) systems are less complex than air-fed systems, which use additional unit processes to sufficiently dry the feed gas. They further claim that high humidity levels and high dust content in the air, as well as higher specific energy (kWh/kgO_3), large gas-handling equipment, and maximum O_3 concentration production of some 2.5% by weight further increase the costs of extracting oxygen from the air to generate ozone.

2.6.1.3 Coagulation

AWWA (1999) defines coagulation as a complex process involving many reactions and mass transfer steps. As practiced in water treatment, the process essentially consists of three separate and sequential steps: coagulant dosing, particle destabilisation and interparticle collisions. Coagulant formation, particle destabilisation and coagulant-NOM interaction typically occur during and immediately after chemical dispersal in rapid mixing; interparticle collisions that cause aggregate (floc) formation begin during rapid mixing but usually occur predominantly in the flocculation process. He continues by stating that the water treatment literature sometimes makes a distinction between the terms “coagulant” and “flocculant.” When this distinction is made, a coagulant is a chemical used to initially destabilise the suspension and is typically added in the rapid-mix process. In most cases, a flocculant is used after the addition of a coagulant; its purpose is to enhance floc formation and so increase the strength of the floc structure.

Tzoupanos & Zouboulis (2008) define coagulation as a process that accelerates the settling time of very small particles in water by first destabilising their charges so that they can come together and form bigger particles that can settle in a short a time as possible. The destabilisation can be achieved with one or a combination of two or more of the following mechanisms after the addition of a coagulant agent:

1. Compression of electrical double layer
2. Adsorption and charge neutralisation
3. Adsorption and interparticle bridging
4. Enmeshment in precipitate (by use of an excess coagulant dose, “sweep flocculation”)

The two definitions provided above are in agreement with the general definitions available in the literature and the writer will not repeat them here as the difference will be semantic only. The emphasis here will be on the design requirements that specifically enhance the coagulation process in water treatment.

Design requirements

The purpose of adding a coagulant is to neutralise the charge, and since most particles in water are negatively charged, any positive ion can be used as a coagulant, for example a sodium compound (such as sodium hydroxide) which contributes a monovalent ion, Na^+, a calcium compound (such as calcium hydroxide) which contributes a divalent ion, Ca^{2+}, and aluminium and iron coagulants which contribute trivalent aluminium ions, Al^{3+} and trivalent iron ions, Fe^{3+} respectively (Engelhardt, 2010). He continues by mentioning that two chemists, Schultz in 1882 and Hardy in 1900, demonstrated the greater the charge of the cation, the greater the effectiveness of charge neutralisation in what they termed the **“Schultz-Hardy Rule” –** this rule indicates that the relative effectiveness of mono- vs. di- vs. trivalent ions is in the ratio of 1:100:1000 respectively. For a variety of reasons, for drinking water applications the relative effectiveness of monovalent (Na^+) vs. divalent (Ca^{2+}) vs. trivalent (A^{3+}) ions is 1:60:700 respectively. That is, a trivalent aluminium ion will be 700 times more effective in charge neutralisation than the monovalent sodium ion. Thus aluminium and iron compounds are most often used as coagulants. Sodium or calcium salts added for pH adjustment may contribute to the coagulation process.

The literature in general (AWWA, 1999; Schutte, 2006; Engelhardt, 2010) corroborates that neutralisation occurs very rapidly, thus the rapid mix system step or process unit should be designed so that dispersal of a coagulant in a water treatment plant is as rapid as possible. Where the coagulation process and subsequent flocculation appear to be inefficient or ineffective, it is reasonable to suspect inadequate mixing as at least part of the cause.

Table 4 below illustrates the resulting change in particle size, total surface area, number of particles and settling as the initial particle is ground up to make smaller particles. One particle 10 mm in diameter becomes 10^{12} particles by the time it is ground to a size of 0.001 mm (1 µm). Notice also while the mass per unit particle decreases, the total mass in the system remains unchanged. Clearly, there is not necessarily any correlation between particle counts and mass, turbidity and mass or between particle counts and turbidity.

Table 4: Different particle sizes and their settling rate (Engelhardt, 2010)

Particle Size Vs. Settling Rate Table (Assuming specific gravity of 2.65)							
Particle Diameter, mm	Example	Total Surface Area		Mass , mg per particle	Total number of Particles	Time to Settle One Ft.	Time to Settle One Meter.
		Metric	English				
10	Gravel	3.419cm^2	0.487in.^{2}	1.39E+03	1.00E+00	0.3 sec.	0.98 sec
1	Coarse Sand	31.4193 cm^2	4.87 in. ^{2}	1.39E+00	1.00E+03	3.0 sec.	9.84 sec
0.1	Fine Sand	314.1929cm^2	48.7 in.^{2}	1.39E-03	1.00E+06	38 sec.	2.08 min
0.01	Silt	0.3140 m^2	3.38. ft. ^{2}	1.39E-06	1.00E+09	33 min	1.80 hrs
0.001	Bacteria	3.1340 m^2	33.7 ft.^{2}	1.39E-09	1.00E+12	55 hrs	7.52 days
0.0001	Colloidal	31.7728 m^2	38 yd^2	1.39E-12	1.00E+15	230 days	2.07 yrs
0.00001	Colloidal	2832. 7995 m^2	0. 7acres	1.39E-15	1.00E+18	6.3 yrs	20.66 yrs
0.000001	Colloidal	28327.99 m^2	7.0 acres	1.39E-18	1.00E+21	63 yrs	206.64 yrs
Table 4: Different Particle sizes and their settling rate (Engelhardt, 2010)							

Coagulation with salts of aluminium and iron

Powdered, granular or crystalline salts and solutions of iron and aluminium such as hydrated aluminium sulphate, liquid alum, ferric sulphate and ferric chloride are still widely used. Measurement and management of pH and alkalinity are critical when these salts are used because alkalinity is consumed when these compounds are used (see Table 5 below). There is an ideal range of pH for each of the compounds (Engelhardt, 2010).

Table 5: pH and alkalinity consumed for aluminium and Iron coagulants (Engelhardt, 2010)

Coagulant	Empirical Formula	pH Range(s)	Alkalinity Consumed
Aluminum Sulfate	$Al_2(SO4)_3$ .14 H2O	Theory 5.5 to 7.8. Typical 6.0 to 7.4	0.49m/L for each mg/L of alum
Ferric Sulfate	$Fe_2SO_4 .9H_2O$	4.0 to 11.0	0.53mg/L for each mg/L of ferric sulfate
Ferric Chloride	$FeCl_3$	4.0 to 11.0	0.92mg/L for each mg/L ferric chloride
Table 5: pH ranges and alkalinity consumed for aluminum and iron coagulants (Engelhardt, 2010)			

Other coagulants and coagulant aids

A number of other compounds are being used today either to replace the metal salts or to complement them (Engelhardt, 2010). When used to complement the metal salts they are referred to generally as coagulant aids or perhaps as flocculant aids. The interest in the use of other compounds is generally driven by one or a combination of three factors: reduced cost, reduced solids or less dependence on conditions of alkalinity and pH. The most commonly used of these other coagulants is polyaluminium chloride (PACl).

Polyelectrolytes used in water treatment are generally low molecular weight and may be used as primary coagulants, coagulant aids, Flocculant aids or as filter aids (Engelhardt, 2010). Cationic, anionic and nonionic compounds are available. Polymers used for primary coagulants and coagulant aids are generally cationic compounds. Flocculant aids will typically be anionic or nonionic and have a slightly higher molecular weight. Those used as filter aids may be slightly cationic or nonionic.

Engelhardt (2010) further states that cationic polymers most often encountered are one of two quaternary amines: polydiallyldimethyl ammonium chloride (polyDADMAC) or epichlorohydrin dimethylamine (epiDMA). There are a large number of chemical suppliers compounding a large variety of polymers. Each product, of course, claims to be superior to anything else. The fact is most of them will work well – somewhere! The only way to be certain a particular polymer will work in a particular treatment system is to do a jar test and pilot test the use of the compound.

Health effect concerns for use of polymers

As one might suspect, addition of these compounds to water is not without some concern. PolyDADMAC and epiDMA have been associated with the formation of nitrosamines. There are about nine compounds in this general group that can be produced as disinfection by-product (DBP) from chlorination and chloramination practices (Engelhardt, 2010). These compounds are toxic and may be carcinogenic. N-nitrosodimethylamine (NDMA) specifically is of concern and is on the US Environmental Protection Agency's (US EPA) Priority Pollutant and Contaminate Candidate List 3. The Environmental Protection Agency currently has no MCLs set for any of the nitrosamines, but some US states and the World Health Organization (WHO) have set guidelines (WHO, 2017). These Guidelines for NDMA call for less than 100 ng/l. The State of California has set an action level and public health goal of 10 ng/l and 3 ng/l of NDMA respectively.

Caution must also be exercised when selecting the type of chemical that is to be fed upstream of the GAC filter bed (Kawamura, 2000). Polymer feeding as a filter aid is common practice for high-rate filters, but this practice potentially decreases the adsorption capacity of the GAC and may also release monomerous acrylamide, a known carcinogen.

Enhanced coagulation

Natural Organic Matter (NOM) is now the key parameter with respect to the design and operation of water treatment processes (Eikebrokk, Vogt & Liltveld, 2004; Polasek, 2015) because of its impacts on the quality of water produced in terms of colour, taste, odour and production of disinfection by-products (DBP) when a disinfectant such as chlorine is used. "Enhanced coagulation" is the term used to define the process of obtaining improved removal of DBP precursors by conventional treatment (Engelhardt, 2010). Concern for the formation of DBP resulting from reactions of chlorine with NOM led to the Disinfection and Disinfection By-products Rule (DDBPR). The DDBR requires the use of a NOM removal strategy called “enhanced coagulation”. Specific goals are spelled out for managing the water treatment process in order to optimise the removal of NOM. Because total organic carbon (TOC) is easily measured and monitored, the treatment technique uses a TOC removal requirement. However, basing a performance standard on a uniform TOC removal requirement is inappropriate because some waters are especially difficult to treat. If the TOC removal requirements were based solely upon the treatability of "difficult-to-treat" waters, many systems with "easier-to-treat" waters would not be required to achieve significant TOC removal. Alternatively, a standard based upon what many systems could not readily achieve would introduce large transactional costs to utilities.

The concentration of NOM in water is typically expressed using the amount of organic carbon (AWWA, 1999). Organic carbon that passes a 0.45 µm pore-sized membrane filter is defined as Dissolved Organic Carbon (DOC), and the amount that does not is known as Particulate Organic Carbon (POC). TOC is the sum of DOC and POC. The amount of by-products formed by disinfectant chemicals such as chlorine is proportional organic carbon in the water. A number of relationships between organic carbon and disinfection by-product concentration have been presented in the literature. For example, Chapra, Canale & Amy, (1997) used data from groundwater, agricultural drains and surface water (rivers, lakes and reservoirs) to show a highly anti-correlation ($r^2 = 0.936$, $n = 133$) between the TOC and the THM formation potential (THMFP). The relationship is given by:

$$\mathbf{THMFP = 43.78TOC^{1.24}}$$

Where THMFP is in µg and TOC is in mg/L. The data gathered by Chapra et al. (1997) are consistent with the frequent observation that high-TOC waters (with a higher fraction of humic acids) yield a greater amount of THMs per amount of TOC than do low-TOC waters.

To address these concerns, a two-step standard for enhanced coagulation and enhanced precipitative softening was developed. Step 1 includes TOC removal performance criteria which, if achieved, define compliance. The Step 1 TOC removal percentages are dependent on alkalinity, as TOC removal is generally more difficult in higher-alkalinity waters and source water with low TOC levels. Step 2 allows systems with difficult-to-treat waters to demonstrate to the State, through a specific protocol, an alternative TOC removal level for defining compliance. The final rule also contains certain alternative compliance criteria that allow a system to demonstrate compliance. Achieving NOM reduction may also involve the use of a pre-oxidant such as ozone, chlorine dioxide or permanganate (sodium or potassium permanganate). Some utilities will find the measurement of TOC and/or UV absorbance (UV254) to be useful for optimising coagulation (AWWA, 1999).

Enhanced coagulation refers to optimising coagulation, flocculation, clarification and filtration to remove organic matter from water that may contribute to the formation of disinfection by-products. The organic matter may be from synthetic sources such as industrial discharges – anthropogenic origin as well as from nature (Eikebrokk, Vogt & Liltveld, (2004); Engelhardt, (2010); Polasek & Associates, 2015). Decaying vegetable matter in a high mountain meadow (see Figure 6 below) and decaying matter from mangroves such as the Florida Everglades (see Figure 7 above) can contribute significant organic matter. The brown colour of the water around the mangroves is due to the tannins and humic substances from the decomposing plant material. The contribution from a high mountain meadow may be seasonal or occur after a storm, while levels of organic matter in warmer climates will be more constant. The treatment process, once established, needs only to be monitored and maintained. Seasonal or intermittent start/stop treatments may be more difficult to control. In either case, the key to successful enhanced coagulation is measurement.

Figure 6: NOM from decaying vegetation in a high mountain meadow

Figure 7: NOM from decaying mangroves, e.g. the Florida Everglades

2.6.1.5 Flocculation

Flocculation is achieved by gentle stirring or agitation to encourage the particles formed during coagulation to agglomerate into masses large enough to settle or be filtered from solution (Engelhardt, 2010). Particles in water smaller than about 10 microns are difficult to remove by simple settling or filtration. This is especially true for particles smaller than 1 micron – colloids.

2.6.1.6 Sedimentation and flotation

In the literature the accepted definition of sedimentation is the process in which the aggregates that formed during coagulation and flocculation are allowed to settle from water (Schutte, 2006). A variety of designs for sedimentation tanks are available, which include a large variety of rectangular and circular tanks. These types of tanks use gravity to settle flocs formed during coagulation and flocculation, as well particles that settle readily. Schutte (2006) continues by stating that certain flocs are relatively light and do not settle readily, and a process such as flotation is used for their removal, particularly flocs formed from algae.

Sedimentation and flotation are solid-liquid separation processes used in water treatment mostly to lower the solids concentration, or load, on granular filters. As a result, filters can be operated more easily and cost effectively to produce acceptable quality filtered water. Many sedimentation and flotation processes and variants that exist for a particular application will depend on the water to be treated as well as local circumstances and requirements (AWWA, 1999). The Handbook also states that with rectangular horizontal flow tanks, the water to be settled flows in one end and exits at the other end. The inlet arrangement must provide a flow distribution that maximises the opportunity for particles to settle – length and cross-sectional shape of the tank must not encourage the development of a counter-productive circulatory flow pattern and scour. Outlet flow arrangements also must ensure appropriate flow patterns – the principal differences between tanks relate to inlet and outlet arrangements: length, width and depth ratios, and the method of sludge removal. Mechanically aided sludge removal methods are installed to avoid interruption in operation and reduce the work force where sludge is scraped and pushed towards the hoppers at the inlet end of the tank.

Most authors (AWWA, 1999; Schutte, 2006; US Member States & Province, 2012) argue strongly that the bottom floor should slope slightly towards the sludge collection hoppers to facilitate sludge removal. Slopes that are regularly mentioned in the literature include the ranges from 6% (3.43º) to 16% (9.10º). 8% (4.57º) to 12% (6.84º) typical for circular tanks and 2% (1.15º) to 8% (4.57º) for rectangular tanks. All the authors emphasise the importance of installing scrapers even with these slopes to prevent the frequent manual removal of sludge. US Member States & Province (2012) regulate that where mechanical scrapers are not installed, the bottom floors of all rectangular sedimentation tanks should have a slope of a one-foot drop for every twelve feet (4.8º slope), which is approximately a 0.35 m drop for every 4.2 m across the length of the sedimentation.

Rovel, Mouchet & Andriamirado (2004) argue that in the case of water or liquor that is heavily loaded with suspended solids, the “density currents” can produce velocity distributions that tend to cause suspended solids that have accumulated on the tank floor to rise in the direction of the recovery channel. This is usually the case in conventional rectangular settling tanks used to clarify activated sludge that are too long. Temperature fluctuations and water exposed to direct sunlight have huge impacts on creating conventional currents.

Because the settling properties of flocculent suspensions cannot be formulated, a sedimentation tank’s performance cannot be accurately predicted. However, for new plants, settling rates can be estimated from batch settling data developed from laboratory jar tests, and for expanding existing plants, settling rates can be derived from evaluating the performance of existing sedimentation tanks during various influent water quality conditions. These evaluations often allow increasing rates for existing basins and the establishment of higher rates compared to published guidelines for new basins (Baruth, 2005; Kawamura, 2000; Montgomery, 1985).

Water treatment plants located in warm or hot weather regions usually have problems with algal blooms and heavy growth of vegetation in the watershed. This can be prevented by either covering the basin or providing sufficient chlorine residual to the influent water. However, chlorine should be added with extreme caution because of the likelihood of DBP formations that may exceed tolerable limits (Kawamura, 2000).

Flotation involves the formation of small air bubbles in water that has been coagulated. The bubbles attach to the flocs, causing them to rise to the surface where they are collected as a froth which is removed from the top of the flotation unit (Schutte, 2006). The clean water is then withdrawn from the bottom. The mechanism of flotation is exactly the opposite of that of sedimentation where heavier particles are allowed to settle to the bottom under gravitational influence – in flotation light particles are floated to the surface of water and removed as scum.

The requirement for installing either flotation/DAF units or sedimentation tanks depends on the suspended particles of up to 50NTUs, DAF units are normally effective sometimes even to 100 NTUs for a short duration, and for anything above these sizes, sedimentation would be appropriate to install, but not both (Montgomery, 1985; Kawamura, 2000). These authors and others emphasise that excessive and unnecessary capital expenditure would result if both DAF and sedimentation were installed. They further emphasised the following critical design considerations when installing flotation/DAF units: (1) high mixing energy, (2) shorter mixing period, and (3) installation of an inclined baffle at the effluent of the tank so that the bubble-floc agglomerates are directed towards the surface of the flotation tank. The time required for flocculation is shorter than for conventional settling processes, and the hydraulic surface loading rate is 10 times or more than for conventional sedimentation tanks. Furthermore, the DAF process is most suited for treatment of algae-laden coloured water, which has relatively low turbidity. However, the TOC level of raw water is an important consideration and should not be above the value of 8 to 10 mg/L as the DAF process does not operate effectively in enhanced coagulation mode because many flocs become too heavy and will not float due to the high dosage of inorganic coagulant and polymer.

In considering conventional clarification/filtration (Baruth, 2005), the type of clarification selected would also be influenced by the type of source water solids. Where suspended and/or dissolved organic matter predominates, high-rate clarifiers including dissolved air flotation could be the more effective and more economic application. Where seasonal algal blooms occur, dissolved air flotation may be the preferred alternative. Plain settling with greater detention would be desired where more profuse, denser solids predominate, such as in many river sources.

Effluent collection troughs or launderers are an important design consideration for this purpose. Rectangular sedimentation tanks may have a single trough at the end of the basin or multiple parallel troughs. Regardless of configuration, the collection trough (launderer) has one purpose – to collect the effluent water uniformly and discourage short-circuiting (Engelhardt, 2010). The author further argues that the wall of the effluent trough should be fitted with a weir plate, which may be a simple flat plate, a v-notch plate or a plate with uniformly spaced circular orifices to minimise collection of solids, oils or chemical films that would inhibit the free escape of the water past the weir. However, he warns that the orifices or perforations are prone to plugging that result in non-uniform collection and short-circuiting.

2.6.1.7 Filtration

Schutte (2006) describes filtration as the removal of flocculated and particulate matter by passing fluid containing these through granular media, usually sand. The most common system is filtration through a layered bed of granular media, usually a coarse anthracite coal underlain by a finer sand. He further says that filters may be classified according to the types of media used as follows:

Single-media filters: These filters have one type of medium, usually sand or crushed anthracite coal.

Dual-media filters: These filters have two types of media, usually crushed anthracite coal and sand.

Multi-media filters: These filters have three types of media, usually crushed anthracite coal, sand and garnet. Figure 8 below shows different media used in water purification filters.

Figure 8: **Different media used in the filtration process (Courtesy, Anonymous [n.d.])**

In water treatment, all three types are used; however, the dual- and multi-media filters are becoming increasingly popular. Particle removal is accomplished only when the particles make physical contact with the surface of the filter medium. Louis Pasteur in France developed filtration prior to the discovery of the germ theory. Pasteur (1822–1895) was a French chemist and microbiologist. He is remembered for his remarkable breakthroughs in the causes and prevention of diseases. In the 1700s, the first water filters for domestic application were applied. These were made of wool, sponge and charcoal. In 1804 the first actual municipal water treatment plant designed by Robert Thom was built in Paisley, Scotland. The water treatment was based on slow sand filtration, and the water was distributed by horse and cart. Some three years later, the first water pipes were installed. In 1854, it was discovered that a cholera epidemic had spread through water. The outbreak seemed less severe in areas where sand filters were installed. British scientist John Snow found that the direct cause of the outbreak was water pump contamination by sewage water. He applied chlorine to purify the water, and this paved the way for water disinfection. John Snow (1813–1858) was an English physician and a leader in the adoption of anaesthesia and medical hygiene. He is considered to be one of the fathers of epidemiology, because of his work in tracing the source of a cholera outbreak in Soho, England, in 1854 (Cameron & Jones, 1983).

Filtration efficiency is greatly increased by destabilisation or coagulation of the particles prior to filtration. Filtration normally follows sedimentation or flotation as a final polishing step in conventional water treatment (Schutte, 2006). The author further states that there are two types of sand filtration, namely rapid gravity sand filtration and slow sand filtration as described below.

Rapid sand filtration is used in conventional water treatment following sedimentation or flotation (Schutte, 2006). The filters are open to the atmosphere and flow through the filter is achieved by gravity. Flow is normally downward at a rate of about 5 m/h and the filters are cleaned by backwashing at intervals that vary from 12 to 72 hours and sometimes 96 hours. Some filters are not open to the atmosphere, but operate under pressure. These types of filters are often used in package treatment plants.

During the initial step of filtration, surface straining and interstitial removal results in the accumulation of deposits in the upper portion of the filter media. Because of the reduction in pore area, the velocity of the water through the remaining voids increases, shearing off pieces of captured floc and carrying impurities deeper into the filter bed. The effective zone of removal passes deeper and deeper into the filter. Eventually clean bed depth is no longer available and breakthrough occurs, carrying solids out in the underflow and causing termination of the filter run.

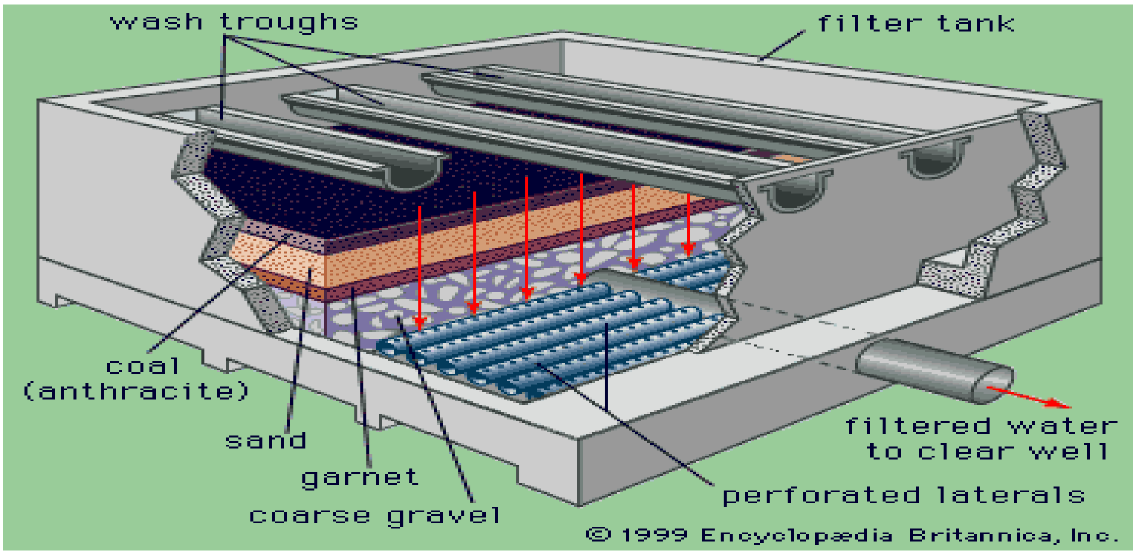


Figure 9: Interior/cross section view of gravity sand filter. (Courtesy, Anonymous [n.d.])

In rapid sand filters, filtration with much higher application velocities are used. Filtration occurs through the depth of the filter (see Figures 9 and 10). Most modern filters employ two separate filter media in layers:

- The lower layer is composed of a dense, fine medium, often sand.
- The upper layer is composed of a less dense, coarse medium, often anthracite coal.
- The coarse upper layer removes larger particles before they reach the fine layer, allowing the filter to operate for a longer period before clogging. As the filter begins to clog from accumulated solids, less water will pass through it. At some point cleaning is required.
- Usual filter operation before cleaning is from a few hours to 2 days.
- Cleaning is accomplished by reversing the flow of water to the filter or backwashing (see Figure 9 below).

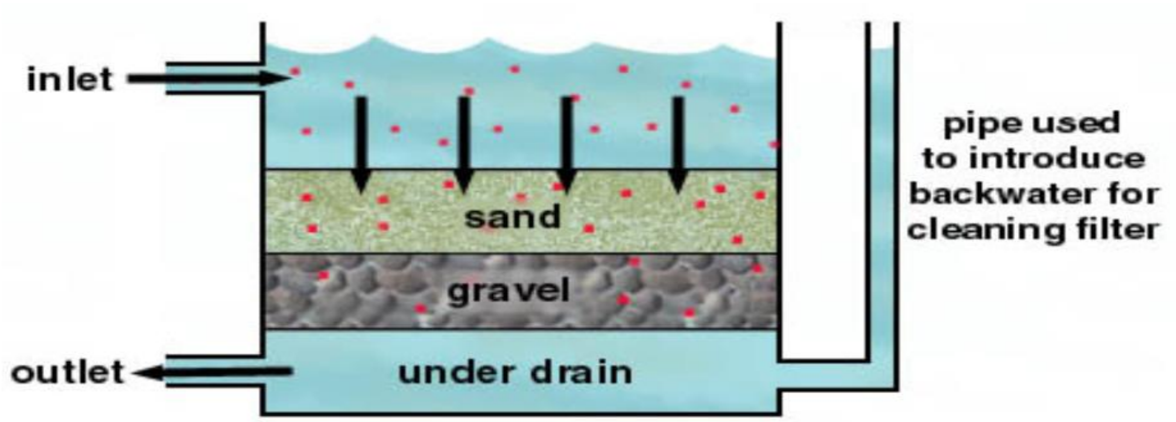


Figure 10: Side view of gravity sand filter. (Courtesy, Anonymous [n.d.]).

Slow Sand Filtration (SSF) on the other hand, has a very slow rate of filtration (compared to rapid sand filtration) and is a process that can be employed as a standalone treatment process. The filter media in SSF are not back-washed at all, but are cleaned by removal of the top layer of sand at long intervals of weeks.

Other types of filters

Different hybrids and combinations of filters are used in the water treatment industry worldwide and the principle is the same as described above. The most important thing is the maximisation of the effectiveness and efficiency during operations, usually measured by product water produced, backwashing efficiencies and filter run times (Schutte, 2006).

CONCLUSION

South Africa is classed as a water-scarce country and it also has to contend with surface waters concentrated in both natural and anthropogenic organic material. This is exacerbated by the fact that heavy thundershowers and floods result in high surface water turbidities during the summer months. In many catchment areas, low-turbidity eutrophic water is supplanted by high-turbidity floods within minutes. Eutrophic conditions are frequently accompanied by turbidity. This rather unwelcome plethora of possible feed water qualities to be taken into account in the design of water purification works has caused many a designer inconvenience in terms of time, money and efficiency.

The literature reviewed showed that a number of things have to be adhered to in order to achieve good quality potable water economically. However, sometimes these design considerations become elusive due to many factors, which amongst others include the country's previous, current and future politics, availability of water resources, plant designers' attitudes and failure to recognise that water as a natural resource is limited.

The literature further revealed that some countries have implemented systems that minimise and/or eliminate these hindrances when implementing capital water projects for the benefit of all – the Ten States created a Water Supply Committee in 1950 to develop design guideline standards for water and wastewater treatment plants. These states are Illinois, Indiana, Iowa, Michigan, Minnesota, Missouri, New York, Ohio, Pennsylvania and Wisconsin, and later in 1978 included the Province of Ontario in Canada**.** These Guidelines Standards are periodically reviewed to keep up with the latest developments in the water industry.

The Master Plan 8.3 (2018) is based on five objectives, which all relate directly to the reliable, resilient, universal and equitable water and sanitation infrastructure that will reduce future water demand. The design and ultimate installation of the entire water infrastructure in a country need to be as effective and efficient as possible for these objectives to be achieved. The Ten States developed their own design standards after realising that the products from their design engineers were not optimum in maximising the return on investment. Similarly, South Africa developed five objectives to enable the achievement of the National Development Plan's vision 2030 of affordable and reliable access to sufficient and safe water and hygienic sanitation for socio-economic growth and well-being, with due regard to the environment. The key is the effective implementation of these objectives. Optimising the design and installation of water infrastructure by optimising adherence to design guidelines will go a long way towards effective implementation of the objectives of the Master Plan 8.3 (2018).

The other aggravating circumstances in the inefficient water services delivery in South Africa is not limited to wastage through design flaws as highlighted in this report, where four water treatment plants were subjected to design scrutiny, but to institutional alignments within the water sector in the country. There are many levels and parties that exist that do not necessarily add justifiable value in the value chain, for example, there is the DWS with its own ministry as the regulator and custodian of bulk water supply through water boards which are WSPs (Water Services Providers) on the one hand, and on the other, there are municipalities that are both WSPs and WSAs (Water Services Authorities) with their own ministry. This institutional arrangement confuses the issue as to who is actually responsible for the ultimate water services delivery to the end consumers.

CHAPTER 3: PROJECT SCOPE AND STUDY METHODOLOGY

The project scope and the four study methods will be individually discussed in this chapter in the following sections. The reasons for limiting the scope to the selected four water treatment plants as case studies will be provided, as well as the four selected methods.

3.1 Project scope

The project carried out by Khuzwayo & Chirwa (2020) revealed that most locations they studied in South Africa showed various limitations which include the lack of adequate water treatment infrastructure, insufficient operation and maintenance schedules, limited technical skills and training, and poor management capacities. They further state that sustainable water management systems have thus become an important goal of sustainable development plans of many countries including South Africa.

Government authorities and the land development industry are increasingly seeking to use alternative sources to conserve drinking water supplies and minimise the stresses of high levels of water consumption It should be recognised that there is generally a correlation between locations of water services delivery underperformance and socioeconomic hardships (Khuzwayo & Chirwa, 2020).

The sustainability of water resources is key as they are being threatened by natural phenomena, which include the multiplicity of trans-boundary water basins, the extreme spatial and temporary variability of climate and rainfall, and desertification as well as the threat brought about by human factors such as inappropriate governance and institutional arrangements in managing national and transnational water basins. Depletion of the country’s water resources is being exacerbated by pollution, environmental degradation and deforestation; failure to invest adequately in resource assessment, protection and development; and unsustainable financing of investments in water supply and sanitation (Donkor et al. 2014).

Donnenfeld et al. (2018) state that South Africa is classed as a water-scarce country, which also has to contend with surface waters concentrated in both natural and anthropogenic organic material. This is made worse by the fact that heavy thundershowers and floods result in high surface water turbidities during summer months. In many catchment areas, low-turbidity eutrophic water is supplanted by high-turbidity flood water within minutes. Eutrophic conditions are frequently accompanied by turbidity. This rather unwelcome plethora of possible feed water qualities which has to be taken into account in the design of water purification works has caused many a designer inconvenience in terms of time, money and efficiency.

Chilton & Polasek (2013) questioned whether the government and people of South Africa receive appropriate value for the expenditure due to improper design of purification works with respect to the process selected and the plants selected for operation as individual units. This was said after the authors had inspected more than 100 waterworks throughout South Africa, all of which suffered from inappropriate design of the processes installed. Generally it was found that the purification processes did not take into consideration the raw water quality. As a result, the waterworks are not capable of purifying water to its best attainable quality most efficiently and economically in a user-friendly arrangement. Furthermore, it was established that inappropriate operation and the lack of maintenance are to a large extent due to the lack of comprehensive O&M manuals generated by the consulting engineers.

Chilton & Polasek's (2013) extensive study/investigation mentioned above did not indicate or quantify cost implications for the client resulting from the design shortcomings. The author of this report aims to quantify these costs and other complications wherever possible due to these design shortcomings. However, due to difficulties in accessing the necessary data to conduct the exhaustive exercise covering all or most of the waterworks mentioned in Polasek's study, only four water treatment plants were selected, and an in-depth analysis was done on only two. This then brings us to the project scope of this study, namely Bospoort WTP (North West), Klipdrift WTP (Gauteng), Roodeplaat WTP (Gauteng) and Vaalkop WTP (North West). Two of the four waterworks were the subject of Polasek's study, namely Bospoort and Roodeplaat WTPs. The other two (Klipdrift and Vaalkop WTPs) belong to the employer of the author of this report.

The impact of water-borne disease in South Africa is significant with an estimated 43 000 deaths per annum coupled with 3 million incidences of illnesses with an associated treatment cost of some R3.4bn in 2000, 20% of deaths in the 1-5 years age group, are attributable to diarrheal diseases (Mackintosh & Colvin, 2003). Statistic South Africa of June 2018 where in it is reported that 3 million people still do not have access to basic water supply services and 14.1 million people do not have safe sanitation

This discussion then leads us to the discussion of the case study plants selected for this report, followed by the four methods that are used to critique and quantify financial losses as well as other losses and/or operational and maintenance complications incurred because of design shortcomings of these waterworks. The four case study plants are used as confirmatory sample to what Chilton & Polasek's (2013) extensive study of 100 plants which study found that none were without design flaws with the aim to quantify negative impacts as a result of these design inefficiencies.

3.2 Four case study plants

The purpose of this section and its subsections is to orientate readers on the individual waterworks selected. Each water treatment plant is discussed, including the actual installed processes and their sequences listed, supplemented by Googles Maps images and flow diagrams. The author of this report operated and maintained all these waterworks, thus the information presented here is first hand. Also two of the four plants were the subject of Chilton & Polasek's (2013) study.

3.2.1 Bospoort Water Treatment Plant

The raw water source is the Bospoort dam. The water is characterised by high algal growth as the dam is situated below two wastewater works and receives effluent discharges and runoffs from Rustenburg town and surrounding townships. This plant supplies water to part of the Rustenburg Local Municipality.

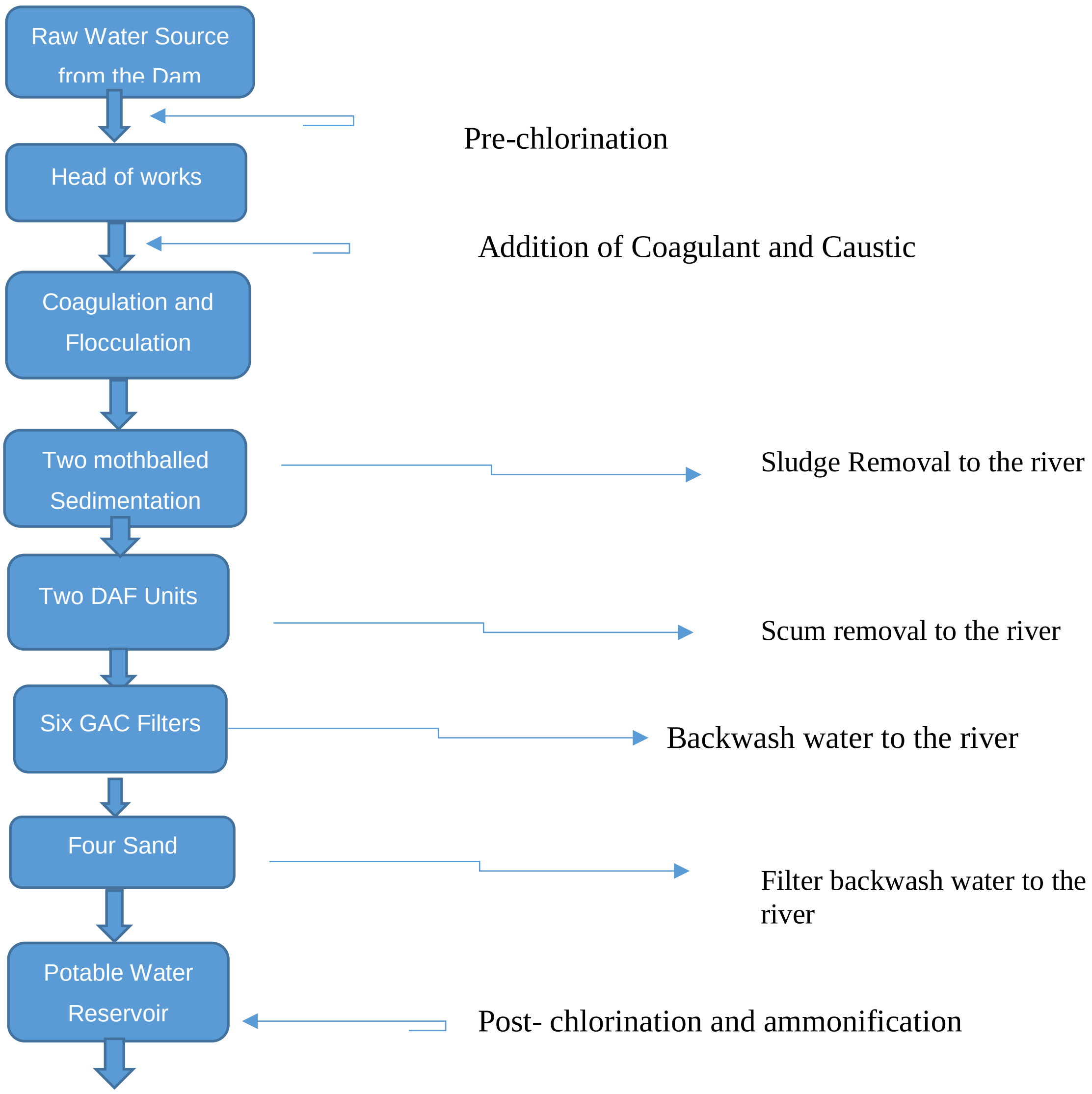
Figure 11: Bospoort WTP, Google Maps, 11 August 2018

Bospoort waterworks has a design capacity of 12 ML/d and has the following processes in order from head of works through to when the final treated water is pumped out to consumers as shown in Figures11 and 12:

- Pre-chlorination
- Coagulation
- Flocculation
- Sedimentation (two) mothballed
- DAF units (two)
- GAC filtration (six)
- Sand filtration (four)

- Post-chlorination and ammonification

Bospoort Waterworks Flow Diagram



Potable Water is pumped to the distribution system

Figure 12: Bospoort WTP Flow Diagram

3.2.2 Klipdrift Water Treatment Plant

Klipdrift waterworks is situated on the eastern side of Hammanskraal north of Pretoria and has a design capacity of 18 ML/d. The raw water source is Roodeplaat Dam (heavily eutrophic water) via the irrigation canal and Pienaars River. It is currently being upgraded to 42 ML/d. The plant supplies water to the City of Tshwane, Bela Bela Local Municipality, Modimolle Local Municipality and Moretele Local Municipality.

Figure 13: Klipdrift plant after upgrade to 42 ML/d – Google Maps, 11 August 2018

This waterworks consists of the following unit processes in order from the head of works until the potable water is pumped out to consumers (Figures 13 and 14):

- pH correction by lime slurry
- Coagulation (four dose points)
- Pre-chlorination
- Flocculation
- Concurrent dissolved air flotation and filtration
- Storage
- Post-chlorination
- Distribution

Klipdrift Waterworks flow diagram (before upgrade to 42 ML/d)

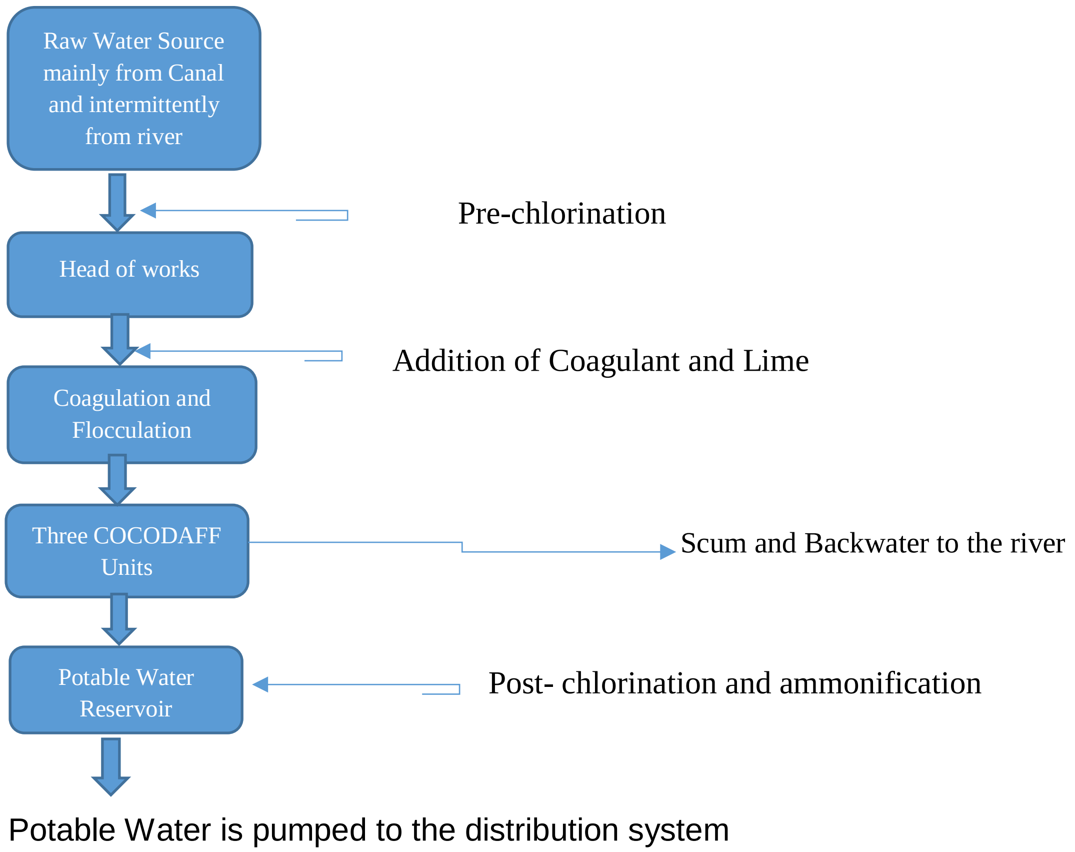


Figure 14: Klipdrift WTP Flow Diagram

The upgrade of this plant to 42 ML/d will retain the above processes with the addition of an ozone plant and a combined raw water inlet structure (correcting the errors of the past) for both the canal and river water. Filter backwash water will be recycled to the head of works for treatment.

3.2.3 Roodeplaat Water Treatment Plant

The 60 ML/d Roodeplaat Water Treatment Works was commissioned in 2005 as part of the larger Roodeplaat Bulk Water Supply project to augment the water supply to the rapidly expanding northern areas of the City of Tshwane from feasible local sources in lieu of extending its existing supply scheme from Rand Water (and thus effectively from the Lesotho Highlands scheme). The waterworks extracts raw water from Roodeplaat Dam, which is approximately 20 km north-east of Pretoria. The dam is currently classified as highly eutrophic because of the influx of mainly effluent from two wastewater treatment facilities situated in the dam's catchment area.

Roodeplaat Waterworks has the following process units in the order from the head of works (Figures 15 and 16 below):

- Pre-chlorination/ozonation
- Coagulation
- pH correction
- Flocculation
- Dissolved air flotation (six)
- Sedimentation (twelve)
- Sand filtration (eight)
- Post-ozonation
- GAC filters (twenty)
- Storage
- Post-chlorination

Figure 15: Roodeplaat WTP Google Maps, 11 August 2018

Roodeplaat Waterworks Flow Diagram

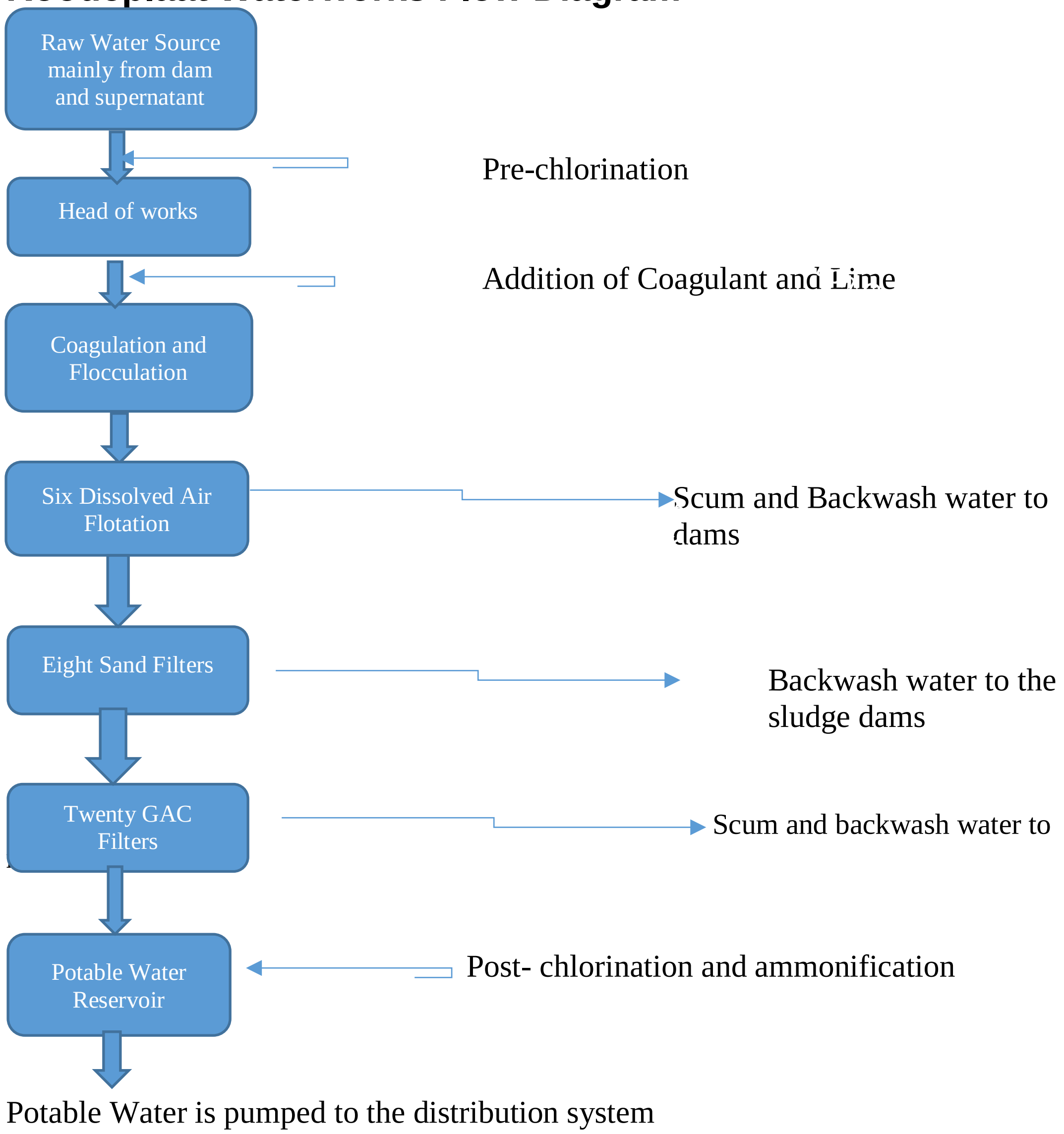


Figure 16: Roodeplaat WTP Flow Diagram

3.2.4 Vaalkop Water Treatment Plant

Vaalkop Water Treatment Plant has a history spanning some forty-six (46) years, during which time it had to be upgraded several times in order to cope with the ever-increasing demand for water on the one hand while dealing with the deteriorating raw water quality on the other. As new technologies became available and affordable, they were installed at this waterworks. Dissolved air flotation (DAF) was introduced in 1991 to deal with algae; powdered activated carbon (PAC) filters were added in 1999 to deal with sporadic occurrences of undesirable tastes and odours; ozone was added in 2006 (Plant 1 only) to deal with pathogens, manganese and iron, and to aid in the removal of organic material; and granular activated carbon (GAC) was added the same year (Plant 1 only) to remove organics that had been oxidised by ozone.

The Vaalkop Plant has grown from 18ML/d 270ML/d in late 2016. Figure 16 below shows all four plants.

Figure 17: Vaalkop WTP Google Maps, 11 August 2018.

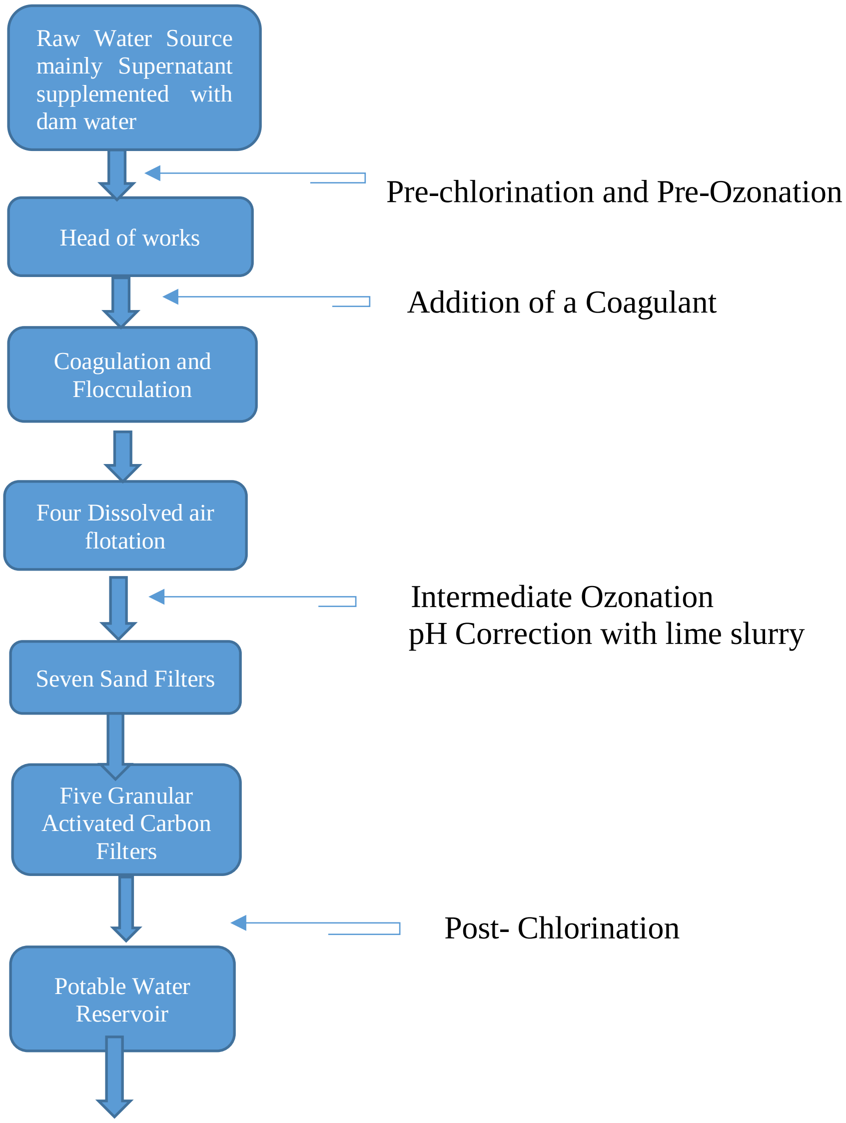
Vaalkop Plant has of late (2016) four independent plants, each with differing processes, capacities, intakes (except Plants 3 and 4) and process arrangements/train. Each plant will therefore be discussed individually below.

3.2.4.1. Plant 1

This is the oldest plant which was first commissioned in 1971 (phase I, 1971) with a capacity of 18 ML/d and was upgraded to 30 ML/d (phase 2, 1979). The role of this plant has changed over time from being the plant that treated raw water from Vaalkop Dam to a plant that is dedicated to treating supernatant water from three sludge dams. The plant is equipped with advanced treatment technologies, namely Ozone, DAF and GAC filters. The process train is depicted in Figure 17 below.

- Pre-chlorination
- Pre-ozonation
- Coagulation
- Flocculation
- Dissolved air flotation (four)
- Intermediate ozonation
- pH correction by adding lime slurry
- Sand filters (seven)
- Granular activated carbon filters (five)
- Post-chlorination

Plant 1 Flow Diagram



Potable Water is mixed with water from Plant 2 and 3 before distribution to consumers

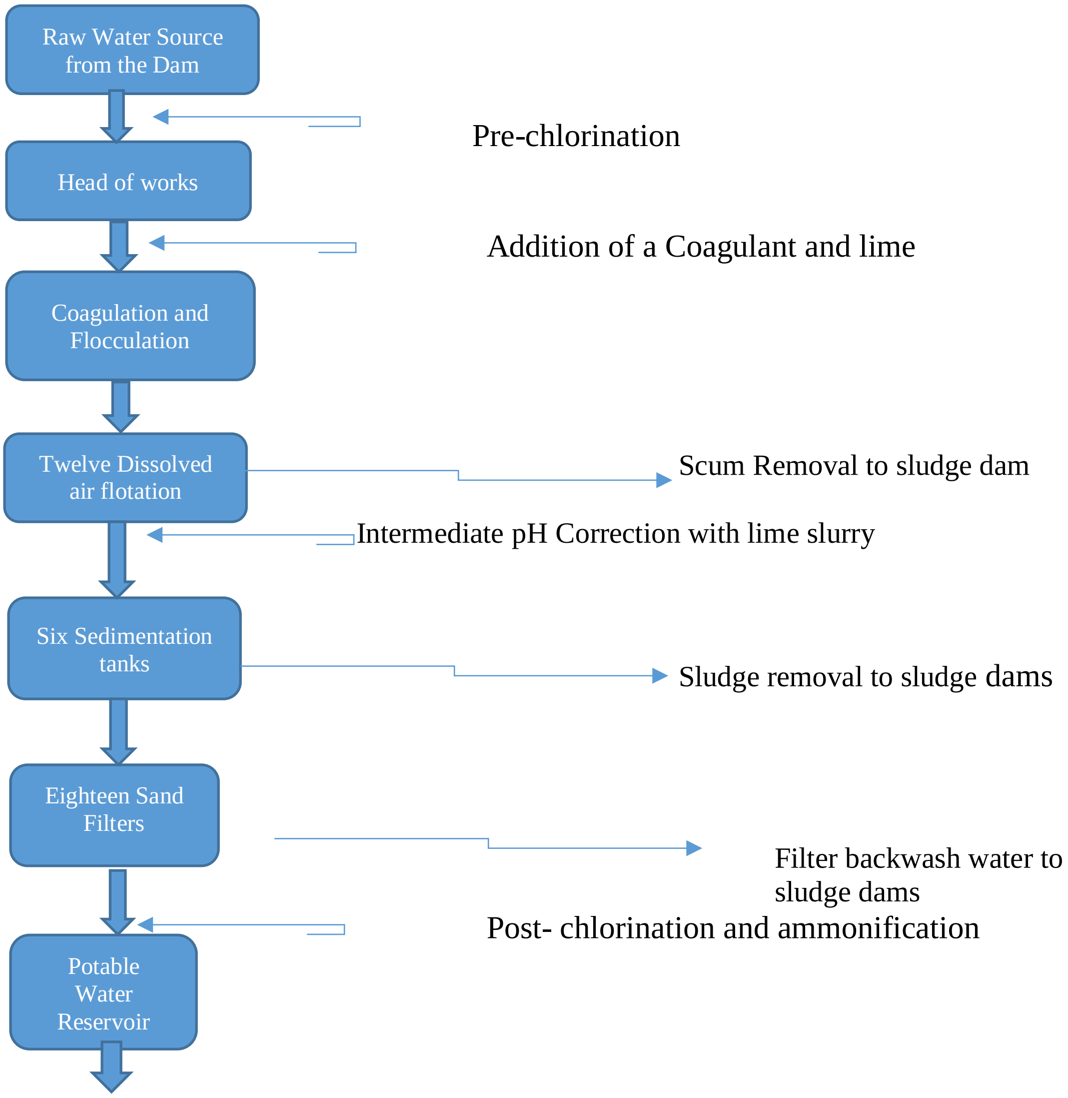
Figure 18: Plant 1 Flow Diagram

3.2.4.2. Plant 2

This is the second-oldest plant after Plant 1, which was commissioned after the phase 2 plant upgrade completion in 1983 with a capacity of 90 ML/d. This phase 2 plant is referred to as Plant 2. The configurations (process train, shown in Figure 18 below) of the processes of this plant are as follows:

- Pre-chlorination
- Coagulation
- Flocculation
- Dissolved air flotation (twelve)
- Sedimentation (six)
- Sand filters (eighteen)
- Post-chlorination

Plant 2 Flow Diagram



Potable Water is pumped to the distribution system

Figure 19: Plant 2 Flow Diagram

3.2.4.3. Plants 3 and 4

The designs of these two plants followed the same process selection and configuration as depicted in the flow diagram (see Figure 20) below:

- Pre-chlorination
- Coagulation
- Flocculation
- Sedimentation (four for Plant 3 and one for Plant 4)
- Concurrent dissolved air flotation and filtration (eight for Plant 3 and two for Plant 4 COCODAFF)
- Post-chlorination

Plants 3 and 4 Flow Diagram

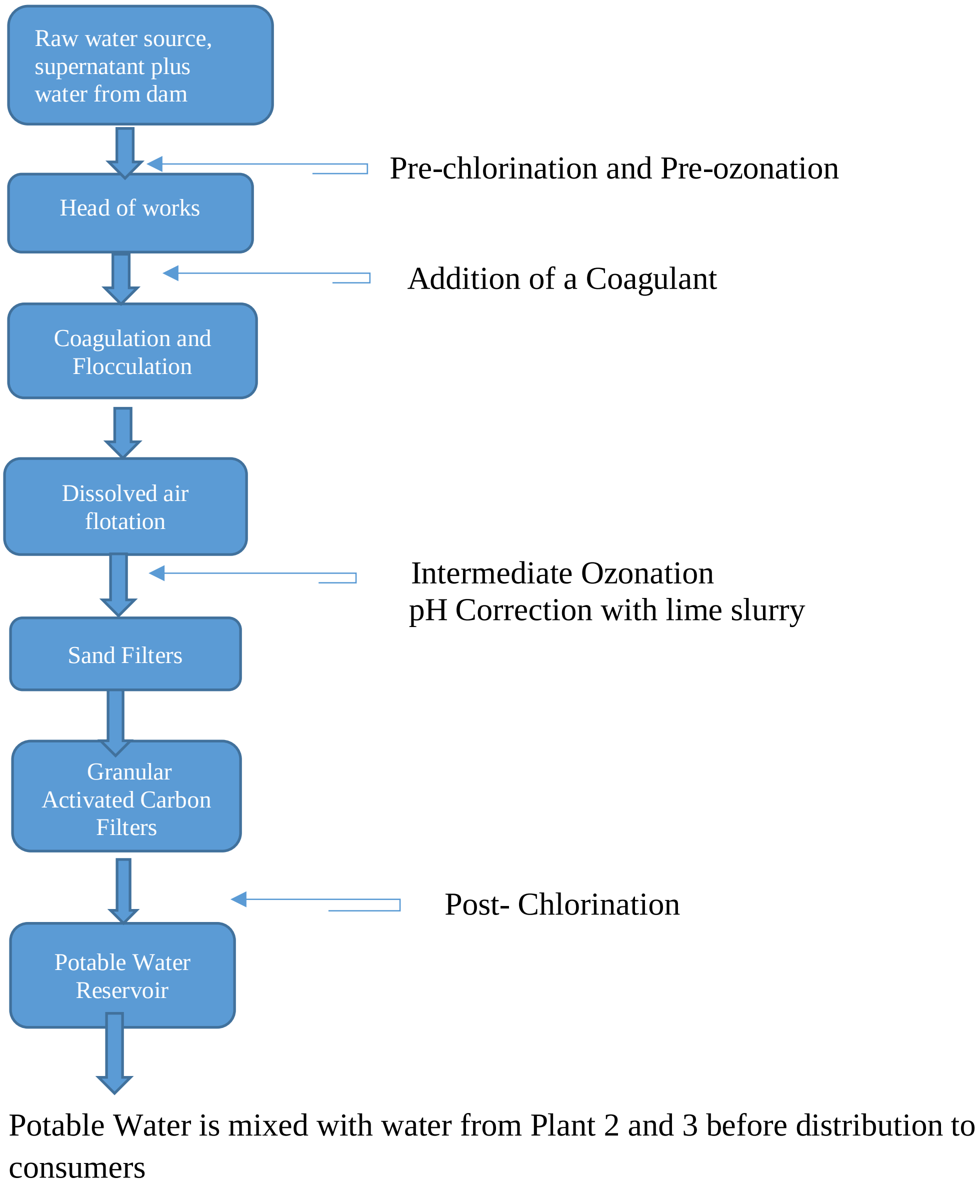


Figure 20: Plant 3&4 Flow Diagram

3.3 Study methods

The four methods used to carry out this study are discussed in this section. They are:

3.3.1 Initial design catering for the ultimate plant capacity

3.3.2 Design conforming to surface water treatment regime

3.3.3 Actual operational performances of plants

3.3.4 Financial implications for the clients of the case study plants selected.

The methods used to conduct this study were both qualitative and quantitative (Mixed Method) with more emphasis on the latter as it directly addresses the research problem. These methods gave the researcher an opportunity to visit the identified water treatment works in order to guide the data collection while interacting with staff (Mackintosh & Colvin, 2003). The rationale for selecting these methods is the type of water being treated in South Africa, which is mainly surface water in all instances collected from rivers and stored in dams. The approach will be to compare and contrast the design against the recommended and well-documented treatment regime applicable to surface water, namely conventional water treatment, selection of processes, design and installation using the abovementioned methods. It is therefore logical to discuss these methods individually in the following sections and the results/findings in Chapter 4.

3.3.1 Initial design catering for the ultimate plant capacity

The literature is unanimous (Baruth, 2005; Schutte, 2006; Kawamura, 2000; Van Duuren, 1997; Montgomery, 1985) about the initial design of certain structures that should cater for the ultimate treatment capacity of the water treatment plant, irrespective of the source of the raw water to be treated. These structures cannot generally be conveniently or economically retrofitted in subsequent or future upgrades. Structures include intake, pump stations, control building which should ideally be placed where operators have a good view of all process units installed and the main entrance gate, clear wells, chemical feed mechanisms and storage facilities. The key test will be whether the four selected plant designers adhered to this requirement on the already installed structures and/or whether any attempts were ever made to correct it during subsequent upgrades of these plants. Template 1 below will be used to indicate if this requirement was met when these plants were initially installed.

Template 1: Matrix evaluating WTP against initially designing for the ultimate design capacity

Design consideration	Intake Structure	Chemical feed mechanisms storage	Control Building	Pump stations	Clear wells	Implications
Designing for ultimate capacity						
Template 1: Matrix evaluating WTP against designing for the ultimate design capacity						

The impacts as a result of not adhering to design requirement (catering for the ultimate plant capacity in the initial design) will be highlighted in the following two sections with the emphasis on Vaalkop WTP, which has had a number of upgrades from 18 ML/d to the present 270 ML/d. The results will be discussed in Chapter 4.

The Vaalkop Plant was also assessed on how soon it takes for all three independently built plants (Plants 1, 2 and 3) to get back to full production capacity against the industry accepted time of between ten and twenty minutes. Records from the shift logbook were used to find the time it takes the shift operators to get all three plants back to full production rate. The reason for assessing the Vaalkop Plant is because it is the only one that has evolved from Plant 1 to 4 in subsequent upgrades, while the others have not.

In addition, similar sized water treatment plants to Vaalkop WTP were identified for comparison where the design criterion of catering for the ultimate capacity was adhered to. The data were captured using Template 2 below, and the findings are discussed in the results section of Chapter 4 of this report.

Template 2: Matrix comparing similar sized waterworks to Vaalkop WTP only

Items	Name of Case study WTP	Similar sized WTP 1	Similar Sized WTP 2	Name of case study WTP Capacity per equipment	Similar sized WTP 1 Capacity per equipment	Similar sized WTP 2 Capacity per equipment
Design Capacity (ML/d)						
Number of raw water pumps installed						
Raw water Pipelines						
Raw water inlets						
Chemical inlet coagulant dosing system						
Number of Clarifiers						
Number of filters						
Infrastructure Footprint in m^2 (Google Maps)						
Template 2: Matrix Comparing similar sized waterworks to Vaalkop WTP (Momba & Swartz, 2009).						

3.3.2 Design conforming to surface water treatment regime

The case study WTPs were subjected to the criterion recommended for the raw water type, namely the surface water that these WTPs treat and purify for human consumption. This was done by comparing the actual installed processes and sequences with the conventional water treatment processes as discussed under section 2.6 subsection 2.6.1 of the literature review. Template 3 below was used to summarise each case study WTP against this design criterion. The findings are highlighted and discussed in Chapter 4 of this report.

Template 3: Matrix evaluating four case study plants against conventional processes

Design consideration	Water Treatment Plant	Implications
Conformity to surface water treatment regime (Conventional treatment)		
Relevant Processes selected		
Correct Process train/sequence		
Advanced Technologies		
Redundancy and/or over design		
Operational problems		
Performance efficiencies		
Template 3 Matrix evaluating four case study plants against conventional processes		

3.3.3 Actual operational performances of plants

Selected case study plants were assessed for any operational performances related to design inadequacies listed and discussed in the literature review in Chapter 2 developed by Swartz et al (2009), specifically processes selected and installed, the order in which they were installed, their configuration and any other technologies that may have been installed with the intention of mitigating against poor qualities of source raw waters (Momba & Swartz, 2009). Template 4 below was useful for testing operational performance.

Template 4: Matrix capturing potable operational water quality compliance (DWS' IRIS data, 2015 to 2018)

Yearly average % Compliance to Operational Quality Parameters				
Year	Bospoort WTP	Klipdrift WTP	Roodeplaat WTP	Vaalkop WTP
2015				
2016				
2017				
2018				
Template 4: Matrix capturing potable operational water quality compliance (DWS' IRIS data, 2015 to 2018)				

Compliances of individual plants with specific operational quality parameters such as somatic Coliphages, turbidity, total coliform, total plate count and pH are discussed below against South Africa's regulatory requirement of the quality of water produced using Blue Drop data from 2015 to 2018 in conformance with SANS 241 standards (RSA, Department of Water & Sanitation, 2015/18). Findings are discussed later in Chapter 4 per plant, except Bospoort WTP due to unavailability of data. The operational quality parameters are a direct reflection of the operational performance of a water treatment plant. The requirement is that these parameters should comply by more than 99.9%, with anything less than 95% unacceptable. These parameters are briefly described below, courtesy of AWWA, (1999).

Somatic Coliphages

Coliphages are viruses that infect the bacterium *E. coli*. They are common in sewage and wastewater. Coliphages are often divided into two major categories: 1) somatic phages, which gain entry into *E. coli* cells through the cell wall, and 2) male-specific (or F-specific) phages, which gain entry only through short structures (pili) of those *E. coli* cells that have them (males). They are far easier to analyse than human or animal viruses, making them a promising indicator of faecal contamination. There is a reasonable correlation between enteroviruses and both somatic and male-specific Coliphages in filtered water, but not in river water.

Turbidity

Turbidity is a non-specific measure of the amount of particulate material in water (e.g. clay, silt, finely divided organic and inorganic matter, as well as microorganisms), and is measured by detecting the amount of light scattered by particles in a sample relative to the amount scattered by a reference suspension. Turbidity has been used for many decades as an indicator of drinking water quality and as an indicator of the efficiency of drinking water coagulation and filtration processes. Achieving adequate removal should at least partially remove pathogens in the source water, especially those pathogens that aggregate with particles.

Turbidity is a relatively crude measurement, which detects a wide variety of particles from a wide assortment of sources; it provides no information on disinfection nor about the nature of the particles. High turbidity levels can reduce efficiency of disinfection by creating a disinfection demand. The particles may also provide adsorption sites for toxic substances in the water, and may protect pathogens (and coliforms) from disinfection by adsorbing or encasing them. They may therefore interfere with the total coliform analysis.

Total coliforms

Total coliforms are a group closely related bacteria (family *Enterobacteriaceae*) that have been used for many decades as an indicator of choice for drinking water. The group is defined as aerobic and facultatively anaerobic, Gram-negative, non-spore-forming, rod-shaped bacteria that ferment lactose to produce acid and gas within 48 hours at $35^{o}C$. Few bacteria other than coliforms can metabolise lactose; for this reason, lactose is used as the basis for identification. The hydrolysis of *o-nitrophenyl-β-d-galactopyroranoside*, or ONPG, is also used for identification in some coliform tests. The total coliform group includes most species of the genera *Citrobacter, Enterobacter, Klebsiella* and *Escherichia coli*. It also includes some species of *Serratia* and other genera. Although all coliform genera can be found in the gut of animals, most of these bacteria are widely distributed in the environment, including water and wastewater. A major exception is E. coli, which usually does not survive long outside the gut, except in warm water associated with tropical climates.

Total coliforms are used to assess water treatment effectiveness and the integrity of the distribution system. They are also used as a screening test for recent faecal contamination. Treatment that provides coliform-free water should also reduce pathogens to minimal levels.

Total plate count

Heterotrophic bacteria are members of a large group of bacteria that use organic carbon for energy and growth. These bacteria are often quantified by the Heterotrophic Plate Count (HPC) method. Because of its lack of specificity, the HPC has not been used to assess the likelihood of waterborne disease; a specific HPC level might contain many, few, or no pathogens. A sudden significant increase in the HPC may suggest a problem with treatment, including poor disinfection practice of drinking water.

pH

The adjustment of pH is the most common method of reducing corrosion in water distribution systems. The pH of the water plays a critical role in corrosion control for several reasons.

The operational quality parameters are the indicators of the performance of the water treatment in terms of its operations, and process controllers or operators have direct control over them using treatment processes already installed. The effectiveness of the process controller/operator is directly linked to how well designed are the selected and installed processes of the water treatment plant for the particular typed quality raw water to be treated to produce potable water meeting the set quality standards effectively, economically and in a user-friendly manner.

3.3.4 Financial implications for clients of the case study plants

Individual negative financial impacts were quantified wherever possible due either to the selection or installation of inappropriate treatment processes and technologies (e.g. ozone plant installed, DAF units versus sedimentation tanks, etc.), duplication of processes (e.g. installation of both phase-separation processes such as DAF and sedimentation in one process train), etc. In some cases, financial instruments such as Payback Period, Net Present Value (NPV), and Internal Rate of Return (IRR) were used wherever financial records were made available. These financial instruments are briefly defined below.

Payback period

Payback period is defined as the expected number of years required to recover the original investment. All factors being constant, a project with a shorter payback period is considered as better because investors can recover the capital invested in a shorter period, and besides, a shorter payback period means greater project liquidity (Thum & Ong, 2013). Payback period is used to indicate the riskiness of the project since cash flows expected in the distant future are generally riskier than near-term cash flows. The formula used to calculate Payback Period is as follows:

$$\textbf{Payback Period} = \textbf{Year before full recovery} + \frac{\textbf{Unrecovered cost at the start of the year}}{\textbf{Cash flow during the year}}$$

Net Present Value (NPV)

NPV is the sum of the present values of all positive and negative cash flows associated with a project. A positive NPV implies that the project is making a return in excess of the discount rate used (Bender and Ward, 2005). The formula for calculating Net Present Value is as follows:

$$NPV = CF_0 + \frac{CF_1}{(1+r)^1} + \frac{CF_2}{(1+r)^2} + \ldots + \frac{CF_n}{(1+r)^n}$$

Where: CF_0 **= Cash flow in year zero**

CF_1 **= Cash flow in year one, etc.**

r **= Project's Cost of Capital**

n **= Project life**

Internal Rate of Return (IRR)

IRR is the discount rate which, when applied to all the cash flows to be generated by a project, results in a NPV of zero. If the IRR exceeds the company's criterion discount rate, this is an indication that the company is returning more than its target rate (Bender and Ward, 2005).The formula for calculating Internal Rate of Return is as follows:

IRR = NPV = 0; that is

$$CF_0 + \frac{CF_1}{(1+IRR)^1} + \frac{CF_2}{(1+IRR)^2} + \ldots + \frac{CF_n}{(1+IRR)^n} = 0$$

Where: CF_0 **= Cash flow in year zero**

CF_1 **= Cash flow in year one**

IRR **= Internal Rate of Return**

n **= Number of period, usually in years**

CHAPTER 4: DESIGN DEFICIENCIES – CASE STUDIES

The four water treatment plants selected for this project were evaluated using the methods described in Chapter 3. The findings/results are discussed in this chapter, 4 with detailed analysis confined to the Klipdrift and Vaalkop Waterworks for the following reasons:

- Vaalkop is the biggest WTP in terms of design capacity at 210 ML/d at the beginning of this study and 270 ML/d on completion of the report.
- Accessing information was envisaged to be relatively easier compared to others as the author works for the owner of both waterworks (Vaalkop and Klipdrift) at the time of compiling this report.
- It is relatively easier to obtain design specifications of similar sized waterworks for comparison purposes.

Analysis of the other waterworks (Bospoort and Roodeplaat WTPs) will be kept to a minimum (limited by the accessibility of data), and serves to confirm that identified design deficiencies are not only confined to a particular plant or province, but are found across South Africa (Chilton & Polasek, 2013).

4.1 Findings/Results

The four case study plants' currently installed treatment processes, train and their relevance were tested using the methodologies described in Chapter 3 wherever data and information were available. Findings per plant are discussed below, including others that are outside the scope of these methods but relevant to the objectives of this report. The approach will be to compare and contrast the design against the recommended and well-documented treatment regime applicable to surface water, namely conventional water treatment, selection of processes, design and their installation using these methods. It is therefore logical to discuss the results/findings per plant below.

4.1.1 Vaalkop WTP

Initial design requirement of catering for the ultimate plant capacity.

The Vaalkop WTP was subjected to Template 1 matrix described under research methodology above by observing the already installed treatment process. Table 6 below highlights the failure to design certain structures for ultimate design capacity.

Table 6: Matrix evaluating the Vaalkop WTP against initially designing for the ultimate design capacity

Design consideration	Intake Structure	Chemical feed mechanisms storage	Position of Control Building	Pump stations	Clear wells	Implications
Designing for ultimate capacity	No	No	No	No	No	Direct or indirect future costs
Table 6: Matrix evaluating Vaalkop WTP against designing for the ultimate design capacity initially						

The plant has been upgraded many times since its first installation of 18 ML/d to 270 ML/d current capacity. These upgrades were not done in strict practical adherence to good design principles, and the subsequent upgrades never addressed the errors of the past. That is, the first module, Plant 1 and the subsequent Plants 2 and 3 did not cater for the ultimate plant capacity by initially providing for those structures that were not convenient or economically constructed in stages to provide additional facilities in phases as the needs arose.

Extra equipment and process units have a negative multiplying effect on operation and maintenance costs as well as on initial capital outlay. However, the latest upgrade commissioned in 2016 brought about change in the composition of the project implementation team initiated by the author of this report, which resulted in some improvements such as a common intake, and a 1 500 mm raw water pipeline and chemical storage facility were installed to cater for the ultimate design capacity of the plant. Due to lack of funds, only Plants 3 and 4 were linked to these structures, reducing the amount of equipment installed, the operational complexity and the associated maintenance costs.

The other requirement was the positioning of the control building, which should be such that the operators have a good view of the plant processes. This building had a partial view of Plant 3 processes until the last capacity upgrade of the Vaalkop WTP from 210 ML/d to 270 ML/d, when a new intake was built to cater for the ultimate plant capacity and in the process improve the view the operators have of the plant processes. The main entrance gate is still out of sight of the control room even after this last upgrade.

The Vaalkop WTP was subjected to Template 2, a matrix that compares similar sized waterworks as this plant has been upgraded several times since its first installation as an 18 ML/d capacity plant early in the 1970s and the following upgrades that increased its capacity to 210 ML/d. The impact of not initially catering for ultimate plant capacity in certain structures is the multiplicity of such structures and equipment as shown in Table 7 below (Umgeni Water, Water Supply Infrastructure Masterplans, 2016). These negative impacts, include the following:

1. Amount of equipment, for example number of raw water pumps installed (13 versus 3), raw water pipelines installed (5 versus 1), raw water intakes installed (5 versus 1), etc. for the Vaalkop WTP 270 ML/d plant and the Midmar WTP 250 ML/d plant respectively

2. The footprint of the plants, for example 41 499 m^2 and 4 584 m^2 for the Vaalkop 270 ML/d plant and the Midmar 250ML/d plant respectively.

Table 7: Matrix comparing Vaalkop WTP to similar sized waterworks

Items	Vaalkop WTP	Wiggins WTP	Midmar WTP	Vaalkop Capacity per equipment	Wiggins Capacity per equipment	Midmar Capacity per equipment
Design Capacity (ML/d)	**270**	**350**	**250**			
Number of raw water pumps installed	13	By Gravity	3	21ML/Pump	Gravity fed	83ML/Pump
Raw water Pipelines	5	1 aqueduct	1	54ML/Pipeline	350ML/Pipeline	250ML/Pipeline
Raw water intakes	5	1	1	54ML/Inlet	350ML/Inlet	250ML/Inlet
Chemical inlet coagulant dosing system	4	1	1	68ML/system	350ML/system	250ML/system
Number of Clarifiers	11 Sed. Tanks and 16 DAFF units	4 Pulsators	4 Pulsators	10ML/DAF or sed. tank	88ML/Pulsator	63ML/Pulsator
Number of filters	25 RGF, 10 cocodaff and 5 GAC (=40filters)	24 RGF	12 RGF	7ML/Filter	15ML/Filter	21ML/Filter
Infrastructure Footprint in m^2 (Google Maps)	41 499	24 221	4 584	$0.00651ML/m^2$	$0.0145ML/m^2$	$0.055ML/m^2$
Table 7: Comparing Vaalkop WTP to similar sized Waterworks						

The multiplicity of equipment and the huge land requirement, coupled with the huge footprint of this plant had immense negative financial implications, both in the capital spent and the associated operations and maintenance costs (lifecycle costs) for the client/owner.

Correcting errors of the past led to the high capital costs of about R58m (CSV Water Consulting Engineers (Pty) Ltd, 2014) incurred by the client and owner of the Vaalkop WTP. This money was spent on the construction of a common intake structure that caters for the ultimate plant design capacity. The control building, a 1 500 mm diameter pipeline and chemical feed were installed and commissioned at the end of 2016.

Design conforming to surface water treatment regime.

The Vaalkop WTP installed processes were subjected to Template 3, and the results are summarised in Table 8 along with the implications.

Table 8: Matrix evaluating the Vaalkop WTP against conventional water treatment processes

Design consideration	Vaalkop WTP	Implications
Conformity to surface water treatment regime (Conventional treatment)	**Yes.**	Effective treatment of this source water.
Relevant Processes selected	**Yes**. However, sequence in which DAF and sedimentation tanks installed are in reverse.	Complications in operation, additional & unnecessary initial costs incurred and huge operating and maintenance costs.
Correct Process train/sequence	**No.** DAF before sedimentation. Duplication.	Duplication (Costly) of processes resulting in frequent quality failures being experienced.
Advanced Technologies	**Yes.** Atmospheric oxygen Ozone generating Plant installed in Plant 1 (30ML/d) only but with capacity to treat 210ML/d, the total capacity of all three Plants at the time.	Atmospheric oxygen is problematic in dusty and high humidity conditions, higher specific energy (kWh/kgO_3), largest handling requirement, & limited to a maximum O_3 concentration of 2.5% by weight. This type Ozone generation system requires advanced skills of both operating and maintenance personnel.
Redundancy and/or over design	**Yes.** Complicated by independently built Plants 1, 2, 3&4. Expensive Ozone technology (R78m in 2008 only for 30ML/d) whereas alternative technology, Chlorine Dioxide. The latter technology installed for 270ML/d in 2018 was about fifteen times cheaper.	Complicated operations due multiplicity of buildings infrastructure and ancillary equipment (Multiple inlets etc. – see table 14 above).
Operational problems	Troubleshooting quality related problems difficult as well as operating many independent Plants.	Costly initial capital outlay and huge maintenance costs requirements.
Performance efficiencies	Not optimal	Not optimal

Table 8: Matrix evaluating the Vaalkop WTP against Conventional Water Treatment Processes

Actual operational performance of the plant

The Vaalkop WTP's actual performance in terms of quality of potable water produced was assessed against South Africa's regulatory requirement for the quality of water produced using Blue Drop data from 2015 to 2018 to conform to SANS 241:2015 Standards – downloaded from the DWS Integrated Regulatory Information System (IRIS) using Template 4. The results are summarised in Table 9 below against the minimum required compliances for the period. This plant failed to comply in all four years with the target of more than 99.9% (>99.9%) IRIS requirement, and only complied 75% with an acceptable requirement of 95.0% during the same period.

Table 9: Vaalkop WTP potable operational water quality compliance (DWS IRIS data, 2015 to 2018)

Yearly average % Compliance to Operational Quality Parameters			
Year	Vaalkop WTP actual percentage performance	Required % IRIS Compliance to Operational Parameters	Acceptable % IRIS Compliance to Operational Parameters
2015	85.7	>99.9	95.0
2016	96.8	>99.9	95.0
2017	96.6	>99.9	95.0
2018	95.3	>99.9	95.0

Table 9: Vaalkop WTP Potable Operational Water Quality Compliance (DWS' IRIS data, 2015 to 2018)

Another observation concerning Vaalkop WTP's Plant 2 is that both DAF and sedimentation are installed as particle-separation processes. This is overdesign, and unnecessarily increases the capital costs for the client. Table 10 below shows that there is little or no improvement in the turbidities of overflows from these units, which are installed in series. The average NTUs for DAF South and Sedimentation South (Vaalkop Plant 2) are 1.43 and 1.26 respectively for this period:

- December 2005: 0.74 and 0.89 for DAF and Sedimentation respectively
- January 2006: 1.19 and 1.25 for DAF and Sedimentation respectively
- February 2006: 0.89 and 1.15 for DAF and Sedimentation respectively

Data as per Table 10 below clearly indicate that either one particle-separation process should be installed, as having both does not improve the quality of water produced by these processes.

Table 10: Plant 2 DAF and Sed. tanks – NTUs from August to November 2017* (Shift logbook)

VAALKOP WTP DAF AND SEDIMENTATION TANKS NTUs			
Date	**Dissolved Air Flotation South**	**Sedimentation North**	**Sedimentation South**
20/08/2017	3.31	1.26	1.44
27/08/2017	1.23	1.10	1.13
03/09/2017	0.93	1.10	0.86
10/09/2017	1.14	1.09	0.95
17/09/2017	1.87	1.63	1.53
24/09/2017	1.86	1.79	1.55
01/10/2017	1.25	1.57	1.69
08/10/2017	1.06	1.38	1.17
04/11/2017	1.99	2.04	1.54
04/11/2017	3.17	3.52	2.75
04/11/2017	4.04	4.51	3.67
Maximum	**4.04**	**4.51**	**3.67**
Minimum	**0.93**	**0.93**	**0.86**
Average	**1.43**	**1.33**	**1.26**
Table 10: Plant 2 DAF and Sed. Tanks NTUs from August to November 2017*(Shift logbook)			

* DAF North feeding sedimentation north was offline during this period

In addition, the ozone plant installed does not meet the basic process design for NOM removal by coagulation, as the reaction process conditions are not optimised. Ozonation (intermediate) and pH correction are carried out in an inappropriate order in the process train, thus jeopardising the optimised reaction conditions of the treatment process.

Financial implications for the client of the water treatment plant selected as case study.

The financial instruments, namely payback period, internal rate of return and net present value, were used to quantify the financial implications for the client/owner of the Vaalkop WTP. The results and analysis are discussed below.

Payback Period

Tables 11 and 12 below show the calculation of the payback period for installing the ozone plant only to treat 30 ML/d with the capacity to treat the whole 210 ML/d at the time.

Table 11: Ozone plant payback period for 30 ML/d

Payback Period for only 30ML/d			
Volume in 2008 first year of Ozone generation	**30,000**	**kl/d**	
Production volume actually treated	**10,950,000**	**kl/a**	
Opportunity Cost (R0.71-R0.10)	**0.61**	**R**	
RSA Targeted Inflation Rate of 6%	**0.06**		
Period in years	**Net Cash Flow**	**Cumulative NCF**	**Payback period in years**
0	-R78,000,000.00		
1	R1,095,000.00	-R76,905,000.00	
2	R1,182,600.00	-R56,817,400.00	
3	R1,277,208.00	-R56,722,792.00	
4	R1,379,384.64	-R56,620,615.36	
5	R1,489,735.41	-R56,510,264.59	
6	R1,608,914.24	-R56,391,085.76	
52	R55,465,846.96	-R2,534,153.04	
53	R59,903,114.71	R1,903,114.71	**52.0**
Table 11: Ozone Plant Payback period for 30ML/d			

Fifty-two (52) years (see Table 11 above) is too long a period for an ozone facility installation of this type, which has a maximum lifespan expectancy of about twenty (20) years. However, if this facility were installed to dose the total plant capacity of 210 ML/d at the time, the payback period would only be five (5) years (see Table 12 below), a mere 25% of the maximum lifespan expectancy for this installation. This has huge financial implications for the client – no returns on investment coupled with huge wastage of scarce resources.

Table 12: Ozone plant payback period for 180 ML/d

Payback Period for 210ML/d			
Volume in 2008 first year of Ozone generation (210-30)	180,000	kl/d	
Potential Production volume that would have been treated	65,700,000	kl/a	
Opportunity Cost (R0.71-R0.10)	0.10	R	
RSA Targeted Inflation Rate	0.06		
Period in years	Net Cash Flow	Cumulative NCF	Payback period in years
0	-R78,000,000.00		
1	R40,077,000.00	-R37,923,000.00	
2	R43,283,160.00	-R14,716,840.00	
3	R46,745,812.80	-R11,254,187.20	
4	R50,485,477.82	-R7,514,522.18	
5	R54,524,316.05	-R3,475,683.95	
6	R58,886,261.33	R886,261.33	5.1
Table 12: Ozone Plant Payback period for 180ML/d			

Net Present Value (NPV)

Table 13 below shows a positive NPV of close to sixty million rand of R57.7m for the five-year payback period (opportunity cost), which is a huge financial loss for the client.

Table 13: IRR and NPV for inlet structure over the 5-year payback period

Internal Rate of Return (IRR) for inlet Structure (For 5year Payback Period)					
Inlet structure					
r	12%				
Time	1	2	3	4	5
Cash flow=	-R78,000,000.00	R40,077,000.00	R43,283,160.00	R46,745,812.80	R50,485,477.82
NPV=	R57,645,285.94				
IRR=	42%				
Table 13: IRR and NPV for inlet structure over the 5 year payback period					

The other observation is that this ozone plant has only been in production for 23 475 hours versus the possible production hours of 35 424, a mere 66.3% for the period from 2008 to 28 February 2017. In addition, the decision was made to install an oversized ozone production facility for only 30 ML/d, capable of treating the combined volume of water (210 ML/d) of all three plants at the time.

Moreover, the technology chosen to generate ozone was to extract oxygen from the atmosphere instead of using LOX technology. These decisions had a huge impact on the treatment cost, quantified to be R0.71 per kilolitre for 30 ML/d versus the possible R0.10 per kilolitre for 210 ML/d of raw water treated.

Internal Rate of Return (IRR)

The loss of IRR (opportunity cost) for the client had the ozone technology been installed to treat 210 ML/d, was 42% (see Table 13 above).

Other financial implications

The literature in general requires that provision for effective removal of sludge be made by installing scraping mechanisms in sedimentation tanks with near flat surface bottom floors. All four plants of the Vaalkop WTP have near flat bottom floors with no mechanical scraping mechanism to remove sludge, and not covered to mitigate extreme subtropical temperatures, resulting in additional operating costs for the client. This cost was quantified for Vaalkop's Plant 3 at about R1.50 m every five months due to cleaning and removal of accumulated sludge, an equivalent of R3.6 m over twelve months in 2015 (see Table 14 below).

Table 14: Financial implications of manually cleaning sedimentation tanks (Vaalkop WTP shift logbook)

Capacity of each sedimentation tank (ML)	5,46			
Plant 3 Sedimentation Tanks	Number of times tank cleaned	Semi-treated volume wasted	Time taken to clean (hours)	Production volume loss (ML)
1	2	10,92	48	60
2	2	10,92	48	60
3	2	10,92	48	60
4	2	10,92	48	60
Total	8	43,68	192	240
Total Volume inclusive of Production loss (ML)	283,68			
Total Revenue loss	R 1 497 830*			
Table 14: Financial implications of manually cleaning sedimentation tanks (Vaalkop WTP shift logbook				

*R5.28/m^3 tariff in the financial year 2016/17 Magalies Water Corporate Plan

Vaalkop WTP's Plant 4 was commissioned on 11 October 2016 with the same hydraulic loading as for each of the four sedimentation tanks of Plant 3. But

Plant 4's sedimentation tank has never been taken offline for cleaning since then, whereas Plant 3's sedimentation tanks have each been taken offline twice, which wasted semi-treated water and led to a combined production time loss of a hundred and ninety-two hours **(192 hours)** as of 28 February 2017. This can be linked to the v-notch type overflow weirs in Plant 4's sedimentation tank as the bottom floor slopes remained the same. The sedimentation tanks without v-notches waste about **5.46 ML per tank of semi-treated water** and reduce plant output by decommissioning each tank for two days in five months, an equivalent of **283 680 m^3** of production loss volume, inclusive of discarded volume (see Table 15 above and Figures 21 and 22 below).

Figure 21: Plant 3 sedimentation tank overflow weir, bulking sludge and accumulated suspended solids on the surface.

Figure 22: Plant 4 sedimentation tank overflow weirs – v-notch type

Vaalkop Waterworks has four stand-alone plants except for Plants 3 and 4 in terms of raw water intakes, that is, each plant has its own independent inlet structure. One of the main reasons given for this is that the plants were built at different times. This is not a good reason but a deficiency on the part of the engineers who designed the first plant module. The engineer should design the first plant module with future plant upgrades in mind, particularly the centralisation of infrastructure for chemicals, operation building and inlet structure to avoid preventable future capital and associated operating and maintenance costs. The installation of many raw water pipelines and inlets results in the duplication of equipment at huge capital costs, which doubles or triples maintenance costs depending on how many times the equipment has been duplicated.

Vaalkop Waterworks is supplied by a rural Eskom power supply system and as such is prone to power failures and power dips. The cost implications are huge when a power failure or dip is experienced, particularly after hours and on public holidays when there are only shift workers on duty. Approximately four hours of production time (shift logbook and monthly reports) are lost each time there is a power failure or dip, irrespective of how short a time they may last. This is because the operator(s) has to start one plant at a time and optimise the chemicals being dosed before moving on to the next one. The accumulated recorded production loss time for the financial year ended June 2015 was 104 hours (Quarterly report, June 2015), an equivalent production loss volume of **838 305 m^3**. Using basic household free water allocation of 6 m^3 per month, about hundred and forty thousand (**140 000)** more households could have been supplied in that year from the Vaalkop Waterworks, and of course a revenue loss of **R4.418 m** was experienced by Magalies Water and will continue to be lost in future.

The choice of using atmospheric oxygen as a source of generating ozone instead of LOX increased the operating cost exponentially. The choice of oxygen source not only increased the amount of bulk equipment needed to dry and clean the air before use, but automatically increased the amount of equipment to be maintained and operated, which consequently increased the costs and skills needs of both operating and maintenance staff.

Other flaws in this ozone plant included the failure to identify the correct/suitable pH correcting chemicals as a lime slurry dosing system was installed just before the GAC filters. While the design engineers took cognisance of the fact that raising the pH before dosing with ozone may reduce its efficacy, they failed to note the more severe and costly impact of calcification of the GAC media of the filters and the subsequent reduction run times of the filters as well as shortening the lifespan and effectiveness of these media. An alternative pH-correcting chemical with high solubility in water such as caustic soda (sodium hydroxide) would have been a better option for raising the pH at this point.

4.1.2 Klipdrift WTP

Initial design catering for the ultimate plant capacity.

The Klipdrift WTP was subjected to Template 1 matrix described under research methodology above by observing the already installed treatment process. It was also observed that the plant was operating at its maximum and sometimes above its design capacity of 18M L/d and that it was being upgraded to 42 ML/d. The plant has structures such as intake, control building, chemical feed mechanisms and storage capacity, high lift pump station and clear wells), all of which were not initially designed for ultimate capacity of this plant as indicated in Table 15 below.

Table 15: Matrix evaluating Klipdrift WTP against designing initially for the ultimate design capacity

Design consideration	Intake Structure	Chemical feed mechanisms storage	Position of Control Building	Pump stations	Clear wells	Implications
Designing for ultimate capacity	No	No	No	No	No	Direct or indirect current and future costs

Table 15: Matrix evaluating Klipdrift WTP against designing initially for the ultimate design capacity

The other requirement is the positioning of the control building which should be such that the operators have a good view of the plant facilities or processes installed. The existing control building is situated one floor above all the processes with no view of them at all, including the main entrance gate.

The Klipdrift WTP is being upgraded (2018) to 24 ML/d and the cost of not catering for the ultimate capacity in the design of intake structure, chemical feed mechanisms and storage, high lift pump stations, clear wells and position of control building will be directly or indirectly incurred. The cost will be direct if the designers address this flaw in the current upgrade design which will modify these structures to cater for the ultimate plant capacity, or indirect if the designers opt to design new independent plants, which will result in a multiplicity of equipment and processes, complicated operations and high maintenance costs.

Design conforming to surface water treatment regime.

The Klipdrift WTP installed processes were subjected to Template 3 matrix discussed under methodology and the results are given in Table 16 below, which includes the implications.

Table 16: Matrix evaluating the Klipdrift WTP against conventional treatment processes

Design consideration	Klipdrift WTP	Implications
Conformity to surface water treatment regime (Conventional treatment)	**Yes**.	Effective treatment of this source water.
Relevant Processes selected	**No**. Omission of key sedimentation process in favour of DAF. Raw water NTUs from river source that frequently go up to more than 100NTU during rainy seasons coupled with high TOCs (greater than 10mg/L).	Complications in Plant operation resulting in frequent water quality failures, additional & unnecessary initial capital costs coupled with huge current operating and maintenance costs associated with DAF units operation.
Correct Process train/sequence	**Yes**.	However not optimal for the type of raw water source being treated now and into the future
Advanced Technologies	**Yes**. LOX Ozone being installed**.**	Efficient ozone generating Plant installed. Instead of the less capital-intensive Chlorine dioxide technology.
Redundancy and/or over design	None. Except expensive Ozone technology where alternative technology, Chlorine Dioxide is about twenty times cheaper.	High initial capital outlay and operating costs of ozone Plant due to high-energy consumption.
Operational problems	Choice of DAF as opposed to sedimentation unable to treat high NTUs in rainy seasons coupled with high TOCs (greater than 10mg/L).	Frequent poor water quality being produced endangering the lives of the consumers.
Performance efficiencies	Not optimal	Not optimal
Table 16: Matrix evaluating the Klipdrift WTP against Conventional Treatment Processes		

Actual operational performance of the plant

Klipdrift WTP's actual performance in terms of the quality of potable water produced was assessed against South Africa's regulatory requirement for the quality of water produced using Blue Drop data for the period 2015 to 2018 in conformance with SANS 241 Standard's Integrated Regulatory Information System (IRIS) using Template 4. The results are summarised in Table 17 below against the minimum required compliances for the period. This plant failed to comply in any of the four years studied with the target of more than 99.9% (>99.9%) IRIS requirements and only complied by 25% with the acceptable requirement of 95% during the same period.

Table 17: Klipdrift WTP Potable Operational Water Quality Compliance (DWS IRIS data, 2015 to 2018)

Yearly average % Compliance to Operational Quality Parameters			
Year	Klipdrift WTP actual percentage performance	Required % IRIS Compliance to Operational Parameters	Acceptable % IRIS Compliance to Operational Parameters
2015	95.0	>99.9	95
2016	88.7	>99.9	95
2017	89.9	>99.9	95
2018	98.1	>99.9	95

Table 17: Klipdrift WTP Potable Operational Water Quality Compliance (DWS' IRIS data, 2015 to 2018)

In addition, the design engineer was adamant that there was no need for a settling/sedimentation step as stated in his conceptual design report, where he said, "The introduction of an additional settling/sedimentation step is not recommended based on the historic and current raw water data which data does not support the need therefor. Being situated directly downstream of a large "settling" storage (Roodeplaat Dam) and the ability to utilise the irrigation canal during periods of excessive high river water turbidity (over 100 TDU) eliminates the need for this additional step - although previously preferred by operations" (Royal HaskoningDHV, 2013).

This may be true when only considering raw water from the river and the canal water average NTU, but does not hold when the total organic carbon averages above 10 mg/LC (see Table 18 below). Managers of both the Klipdrift Plant and Wallmannsthal WTP justify the need for sedimentation, citing the difficulty of operating Klipdrift WTP coupled with the frequent production of inferior quality water compared to Wallmannsthal WTP. The latter WTP receives and treats the same raw water from the canal and river, but their processes include settling/sedimentation.

Table 18: River and canal water average TOC in mg/L (Magalies Water Scientific Services Archives)

TOC Klipdrift Raw Water 2012 to 2016				
Year	Sample Source		No of Samples	
	River Water Average yearly TOC (mg/L C)	Canal Water Average yearly TOC (mg/L C)	River Water	Canal Water
2012	11,9	11,5	22	25
2013	12,4	10,4	4	48
2014	No data	10,2	No data	10
2015	No data	21,7	No data	1
2016	10,8	11,1	8	20
Table 18: River and Canal Water Average TOC in mg/L C (Magalies Water Scientific Services Archives)				

The argument in the same report (Royal HaskoningDHV, 2013), that the poor performance of the Klipdrift plant can be attributed to operating it above its design capacity, is to some extent true, but Wallmannsthal has been operated above its design capacity. The only difference between the Klipdrift and Wallmannsthal WTPs is the inclusion of settling/sedimentation step in the latter (Table 12 below).

Financial implications for the client of the water treatment plant selected as case study

The financial instruments, namely Payback Period, Internal Rate of Return and Net Present Value could not be determined, as the necessary financial data could not be obtained. However, the following paragraph provides the direct cost implications due to the processes selected and installed.

Klipdrift has DAF units as the particle-separation process selected over the sedimentation or settling processes and this plant was constructed and commissioned after the Wallmannsthal WTP, which included the sedimentation/settling step in their process trains. All three plants treat the same raw water source from Roodeplaat Dam. Table 19 below shows the plant losses for two of these plants, namely Klipdrift and Wallmannsthal WTP as 13.7% and 4.8% (financial years F1314 to F1617) respectively. The Klipdrift WTP losses translate to about **R3m** in costs incurred in raw water purchases only during this period, and excludes the costs of labour, energy and treatment chemicals.

Table 19: Klipdrift plant losses compared to those of Wallmannsthal and the financial implications thereof

Plant Loss Klipdrift and Wallmannsthal					
Financial Year	**Klipdrift loss (%)**	**Wallmannsthal loss (%)**	**Volume (m3)***	**Raw Water Tariff (R)**	**Amount loss (R)**
F1314	12,6	7	904 638	R 0.66	R 597 061.08
F1415	16,4	4,8	1 049 795	R 0.71	R 745 354.45
F1516	16,6	2,7	1 014 532	R 0.76	R 771 044.32
F1617**	9,3	4,7	669 765	R 0.85	R 569 300.25
Total	**13,7**	**4,8**	**3 638 730**		**R 2 682 760.10**
Table 19: Klipdrift Plant loss Compared to Wallmannsthal and Financial implications thereof					

* =Klipdrift WTP loss volumes due to yearly Plant losses;

**= F1617 only 8 months considered due to data availability.

4.1.3 Bospoort WTP

Initial design requirement of catering for the ultimate plant capacity.

Bospoort WTP was subjected to Template 1 matrix described in the research methodology above by observing the already installed treatment process. It was also observed that the plant was operating at its maximum design capacity of 12 ML/d and that it was being upgraded to 24 ML/d.

This plant is currently operating at its maximum design capacity of 12 ML/d structures (intake, control building, chemical feed mechanisms and storage capacity, high lift pump station and clear wells), some of which were not initially designed for ultimate capacity of this plant as indicated by either “yes” or “no” in Table 20.

Table 20: Matrix evaluating Bospoort WTP against initially deigning for the ultimate design capacity

Design consideration	**Intake Structure**	**Chemical feed mechanisms storage**	**Position of Control Building**	**Pump stations**	**Clear wells**	**Implications**
Designing for ultimate capacity	Yes	Yes	No	No	No	Direct or indirect present and future costs
Table 20: Matrix evaluating Bospoort WTP against initially designing for the ultimate design capacity						

The Bospoort WTP is being upgraded (2018) to 24 ML/d and the cost of not catering for the ultimate capacity in the design of the position of the control building, chemical feed and storage capacity, high lift pump station and clear wells will be exorbitantly high directly or indirectly. It will be directly high if the designers address this flaw in the upgrade design by modifying these structures to cater for the ultimate plant capacity, and indirectly high if the designers opt to design new independent plants, which will result in the multiplicity of equipment and processes, complicated operations and high maintenance costs. The Bospoort WTP was not subjected to Template 2, a matrix that compares similar sized waterworks, as this plant is currently undergoing an upgrade for the first time.

Design conforming to surface water treatment regime.

The processes installed at the Bospoort WTP were subjected to Template 3 discussed under methodology and the results are summarised in Table 21 showing the implications.

Table 21: Matrix evaluating the Bospoort WTP conventional treatment processes

Design consideration	Bospoort WTP	Implications
Conformity to surface water treatment regime (Conventional treatment)	**Yes.**	Effective treatment of this source water.
Relevant Processes selected	**Yes.** However, GAC filters installed before sand filters. This arrangement defeats the purpose of GAC filters.	Complications in operation, additional & unnecessary initial costs incurred and huge operating and maintenance costs without realizing the intended quality of potable water.
Correct Process train/sequence	**No.** GAC installed before sand filters.	Duplication and wrong positioning of processes without realizing the intended results.
Advanced Technologies	**Yes.**	GAC filters installed in the wrong place in the process train of the Plant. The purpose of GAC is not realised. This arrangement drastically increase the operating costs as the GAC media get blinded in a very short space of time necessitating frequent costly replacement of the media.
Redundancy and/or over design	**None.** But possible with current upgrade underway	Complicated operations due to multiplicity of buildings infrastructure and ancillary equipment (Multiple high lift pump stations, etc.)
Operational problems	Operators' room out of view of the whole Plant making it impossible to see processes in operation.	Operators are unable to have a constant view of the processes and likely to miss undesired physical appearances of the water in the process stream.
Performance efficiencies	Not optimal	Not optimal
Table 21: Matrix evaluating the Bospoort WTP Conventional Treatment Processes		

With the current upgrade of this plant from 12 ML/d to 24 ML/d, the design engineers have addressed the installation of GAC before sand filters by first directing the overflow water from the DAF units to the sand filters and then pumping it to the GAC filters (Personal communication with Plant Manager, 2019). The client will incur this cost for the rest of the lifecycle of this plant.

Actual operational performances of the plant

The Bospoort WTP could not be tested and/or assessed using Template 4, as quality data from IRIS system could not be accessed. However, the reversed installation of GAC and sand filters, would negatively affect this plant's operational performances (figure 11 above).

Financial implications for the client of the water treatment plant selected as case study.

The financial instruments, namely payback period, internal rate of return and net present value could not be determined as the necessary financial data could not be obtained. However, the following paragraph explains the direct cost implication due to the processes selected and installed.

The Bospoort WTP was originally designed with gravity sand and GAC filters installed in reverse order, that is, GAC filters were installed before rapid gravity filters (see Google Maps, Figure 10 and process flow diagram, Figure 11) which remove suspended particles that have escaped the prior particle removing processes. This plant is currently (2018) undergoing an upgrade to address this costly flaw resulting from the fact that the surface media pore of the GAC medium responsible for removal of dissolved organics is quickly blinded by suspended particles that may have been removed by filtration process, thus drastically reducing the lifespan of this very costly medium.

4.1.4 Roodeplaat WTP

Initial design catering for the ultimate plant capacity.

The Roodeplaat WTP was subjected to Template 1 matrix described under research methodology by observing the already installed treatment process and the results are shown in table 22 below. It was also observed that the plant was operating just below its maximum design capacity of 60 ML/d.

Table 22: Matrix evaluating the Roodeplaat WTP against initially designing for the ultimate design capacity

Design consideration	Intake Structure	Chemical feed mechanisms storage	Position of Control Building	Pump stations	Clear wells	Implications
Designing for ultimate capacity	No	No	No	No	No	Direct or indirect future costs
Table 22: Matrix evaluating Roodeplaat WTP against designing for the ultimate design capacity initially						

This plant is operating at about 55 ML/d on average, utilising structures (intake, control building, chemical feed mechanisms, storage capacity, high lift pump station and clear wells) that were not designed to accommodate the ultimate capacity of this plant as indicated in Table 22 above.

The other requirement is the positioning of the control building which should be such that the operators have a good view of the plant facilities or processes installed. This building has no view of all the processes installed nor of the main entrance gate (see Figure 14 above).

This plant is due for an upgrade as it operates at close to its design capacity and above the industry norm of 85% at about 92% on average. The opportunity cost of not catering for the ultimate capacity in the design of certain structures will be directly or indirectly incurred.

Design conforming to the surface water treatment regime.

The Roodeplaat WTP installed processes were subjected to Template 3 discussed under methodology and the results are summarised in Table 23 below, and include the implications.

Table 23: Matrix evaluating the Roodeplaat WTP conventional water treatment processes

Design consideration	Roodeplaat WTP	Implications
Conformity to surface water treatment regime (Conventional treatment)	**Yes.**	Effective treatment of this source water.
Relevant Processes selected	**Yes.** However, installation of two particle-separation processes.	Complications in operation, additional and unnecessary initial costs incurred coupled with huge operating and maintenance costs without realizing the intended quality of potable water.
Correct Process train/sequence	**No.** DAF before sedimentation.	Duplication (Costly) of processes resulting in quality problems being experienced.
Advanced Technologies	**Yes**. LOX ozone, and UV installed	Efficient ozone generating Plant installed. However, having both UV and Ozone installed is a costly duplication.
Redundancy and/or over design	**Yes.** Disinfection (CL_2, O_3, and UV), DAF and sedimentation (either or is sufficient and worse DAF before sedimentation). But possible with current upgrade underway	Complicated operations due to multiplicity of processes and ancillary equipment. This increased initial capital outlay for the client unnecessarily in the process increasing operations and maintenance costs.
Operational problems	DAF good removal of suspended particles reversed in sedimentation	Costly initial capital outlay and huge operations and maintenance costs.
Performance efficiencies	Not optimal	Not optimal and costly
Table 23: Matrix evaluating Roodeplaat WTP Conventional Water Treatment Processes		

Actual operational performance of the plant

Roodeplaat WTP's actual operational performance in terms of the quality of potable water produced was assessed against South Africa's regulatory requirement for the quality of water produced using Blue Drop data for the period 2015 to 2018 in conformance with SANS 241:2015 Standards – downloaded from the DWS Integrated Regulatory Information System (IRIS) using Template 4. The results are compared in Table 24 below against the minimum required compliances for the period. This plant failed to comply in all four years with the target of more than 99.9% (>99.9%) IRIS requirement and only complied 25% to an acceptable requirement of 95% during the same period.

Table 24: Roodeplaat WTP potable operational water quality compliance (DWS IRIS data, 2015 to 2018)

Yearly average % Compliance to Operational Quality Parameters			
Year	Roodeplaat WTP actual percentage performance	Required % IRIS Compliance to Operational Parameters	Acceptable % IRIS Compliance to Operational Parameters
2015	83.3	>99.9	95.0
2016	91.0	>99.9	95.0
2017	95.0	>99.9	95.0
2018	90.4	>99.9	95.0
Table 24: Roodeplaat WTP Potable Operational Water Quality Compliance (DWS' IRIS data, 2015 to 2018)			

Another observation of the Roodeplaat WTP is that both DAF and sedimentation have been installed as particle separation processes. The plant has both DAF and sedimentation process units installed in reverse order with long flocculation channels that promote the formation of bigger and denser flocs that are more likely to be heavier and difficult to float in the DAF unit. This is overdesign, and unnecessarily increases the capital costs for the client. Table 16 below shows that there is little or no improvement in the turbidities of the overflows from these units which have been installed in series. The average NTUs for DAF and SED were as follows for the indicated period (data abstracted from shift quality reports):

- December 2005 as 0.74 and 0.89 for DAF and sedimentation respectively
- January 2006 as 1.19 and 1.25 for DAF and sedimentation respectively
- February 2006 as 0.89 and 1.15 for DAF and SED respectively

The data in Table 25 below clearly indicates that either a one-particle separation process should be installed, but not both.

Table 25: Roodeplaat WTP DAF and sedimentation NTUs, December 2005 to March 2006* (shift logbook)

ROODEPLAAT WTP DAF AND SEDIMENTATION TANKS NTUs											
Dec-05	**DAF**	**SED**	**Jan-06**	**DAF**	**SED**	**Feb-06**	**DAF**	**SED**	**Mar-06**	**DAF**	**SED**
1	0.58	0.62	**1**	0.86	0.56	**1**	0.70	0.83	**1**	1.18	1.61
2	0.56	0.56	**2**	0.75	0.61	**2**	1.10	2.16	**2**	0.79	1.31
3	0.58	0.57	**3**	0.55	0.57	**3**	0.66	0.82	**3**	1.28	0.83
4	0.55	0.70	**4**	0.80	0.52	**4**	1.69	1.61	**4**	1.29	1.08
5	0.65	0.48	**5**	0.96	1.16	**5**	3.35	3.27	**5**	0.96	0.96
6	0.56	0.54	**6**	1.23	0.90	**6**	4.35	5.17	**6**	1.05	1.05
7	0.70	0.84	**7**	0.81	1.04	**7**	2.95	2.38	**7**	1.17	1.05
24	0.69	0.94	**24**	0.99	1.61	**24**	0.45	0.53	**24**	0.58	1.05
25	0.71	1.10	**25**	0.74	0.74	**25**	0.95	1.01	**25**	0.58	0.76
26	0.91	0.69	**26**	1.23	0.94	**26**	0.42	1.15	**26**	0.63	1.79
27	0.58	1.62	**27**	1.37	1.07	**27**	0.65	1.55	**27**	0.66	0.95
28	0.69	1.47	**28**	2.90	1.64	**28**	0.64	1.86	**28**	0.64	0.73
29	0.46	1.33	**29**	2.50	2.39				**29**	0.53	0.92
30	1.38	0.86	**30**	1.51	2.03				**30**	0.57	0.99
31	1.25	2.12	**31**	1.34	2.00				**31**	0.86	1.15
Maximum	**1.38**	**2.12**		**2.90**	**2.46**		**4.35**	**5.17**		**1.70**	**1.79**
Minimum	**0.46**	**0.48**		**0.55**	**0.52**		**0.42**	**0.53**		**0.53**	**0.73**
Average	**0.74**	**0.89**	**Total**	**1.19**	**1.26**		**1.72**	**1.80**	**Total**	**0.89**	**1.15**
Table 25: Roodeplaat WTP DAF and Sedimentation NTUs, December 2005 to March 2006* (Shift logbook)											

* Some data hidden to reduce the size of this table

Financial implications for the client of the water treatment plant selected

The financial instruments, namely Payback Period, Internal Rate of Return and Net Present Value could not be determined as the necessary financial data could not be obtained. However, the following paragraph provides the direct cost implication due to the processes selected and installed.

Some processes installed in the Roodeplaat WTP are redundant, unnecessary and are a depletion of scarce capital and unnecessarily increase operating and maintenance costs. Examples are the installation of both particle-separating processes, namely DAF and sedimentation, and multilayer disinfection processes, namely chlorine, UV, chloramination and of late, ozone.

CONCLUSION

All the water treatment plants selected as case studies for this report have a number of inherent design deficiencies which negatively affect the ability to produce good quality potable water, and to promote and facilitate water services delivery most cost effectively in a sustainable manner. The most common design flaw observed in this study is when designing the rectangular sloping bottom floor of the sedimentation tank. The slopes are not steep enough to allow the settled sludge to flow towards the sludge hoppers without mechanical scrapers installed. It is recommended that there should be a one-foot drop for twelve feet where mechanical scrapers are not installed (USA, Member States & Province, 2012). The negative impacts are consolidated and highlighted below in no particular order:

1. Failure to cater for the ultimate design capacities

All four plants showed design shortcomings and/or deficiencies in the ultimate design capacities of structures such as intakes, pump stations, centrally positioned control room buildings, clear wells, chemical feeds and storage facilities. These shortcomings have huge financial impacts on operations and maintenance as well as future upgrades of water treatment plants.

2. Failure to utilise the value engineering tools in the design

Limited application of VE tools resulted in the duplication of processes such as having two particle-separating processes, namely DAF and sedimentation, installed in three of the case study plants. This evaluation tool is used to justify all the processes before they are installed, and in the process prevent unnecessary initial capital costs.

3. Poor selection of processes and their arrangement

Conventional water treatment processes have been proven to be cost effective and efficient in the treatment of surface waters, according to researchers in water treatment practice (see literature review, Chapter 2 above). Failure to adhere to these processes often results in duplication and redundancies as discussed under Chapter 4 above. An example is the observed popular installation of capital-intensive ozone generation facilities instead of the much cheaper and less complex chlorine dioxide generation facilities that do more or less the same as ozone as a disinfectant.

4. Financial Implications

The Vaalkop waterworks is one of the biggest water treatment works in North West province, and through direct Government funding it was possible to implement a regional water scheme to the tune of a billion rand, which was commissioned in October 2016. Some of this money went into correcting the design deficiencies of the past, for example, the construction of a common intake structure that caters for the ultimate design capacity of the plant at a cost of R58 m, redesigning the sedimentation tank overflow weirs at no additional cost, and installation of a common 1 500 mm diameter pipeline and chemical storage facility to cater for the ultimate design capacity of the plant. These extra costs negatively affected the initial plan of the plant upgrade to increase the water services delivery to the surrounding communities.

The following are the financial implications incurred or being incurred by the owner of the Vaalkop WTP:

1. The volume loss due to full production start-up of 4 hours required every time there is a power dip is 838 305 m^3 per year. This was calculated as equivalent to **R4.418 m** per year in lost revenue in 2015.

2. The construction of a 1 500 mm diameter raw water pipeline, common intake structure, as well as chemical feed mechanisms and storage catering for the ultimate plant output was done at a cost of **R58 m** (see Appendices 1 to 3 below). The first sedimentation tank of Plant 4 with v-notch overflow weir is shown in Appendix 4. Appendix 5 shows the Plant 3 orifice overflow weirs for comparison and completeness.

3. About **R1.50 m** is lost every five months due to cleaning and removal of accumulated sludge in Plant 3. This is the equivalent of **R3.6 m** over the twelve-month period in 2015.

4. The installation of an ozone generation facility only in Plant 1 but with the capacity to dose 210 ML/d at the time cost **R0.71/kl** instead of **R0.10/kl**. The opportunity cost for this decision over the 20-year lifespan of an ozone generation facility is in the region of **R1.94 bn.** This excludes the cost of maintaining an outdated ozone generation technology. The cost reduction of installing an ozone generation facility for only 30 ML/d instead of 210 ML/d could have gone towards redeeming the capital outlay (R78 m) and shortening the payback period drastically from **52 years** to just over **5 years**, thus increasing the liquidity of the ozone installation project by over ten times.

5. The internal rate of return after the common intake structure installation is **42%.** The internal rate of return (IRR) of a project is the expected rate of return for the investor. If this IRR exceeds the cost of the funds used to finance the project, a surplus will remain after paying for the capital, and this surplus will accrue to the company's shareholders.

6. The NPV for installing the ozone generation facility with a capacity volume of 210 ML/d at the time of installation would have been about **R57.6 m** versus **R30.8 m** for treating only the 30 ML/d in Plant 1.

The identification of design flaws as highlighted in the four case studies of water treatment plants bring us to the next chapter, in which the author proposes guidelines to be followed in the implementation of capital projects of bulk water treatment plants. The recommended guidelines if implemented, can most likely yield positive returns on capital investment and savings of precious natural resources now and into the future, in the process increase the number of people that will have access to drinking water going forward.

CHAPTER 5: PROPOSED GUIDELINES ON HOW TO EFFECTIVELY MANAGE WATER TREATMENT PROJECTS

Water treatment plant facility design is the combination and assemblage of all expertise relating to the water treatment field. For the water treatment plant facility to be acceptable, workable, and built within the predetermined budget and on a selected site, it is necessary to utilise and combine the expertise of many separate disciplines.

The successful project should incorporate the skills of chemists, biologists, microbiologists, laboratory technicians, process engineers, civil engineers, architects, designers, management and administrative personnel, financial analysts, surveyors, construction specialists, operations and maintenance specialists and many others (Montgomery, 1985; Baruth, 2005). It is further stipulated that water treatment design requires a highly skilled, interdisciplinary team and is not simply the province of civil, mechanical or electrical engineers. Successful design requires the input and expertise of personnel in fields as vastly different as microbiology and structural design.

A water treatment design project passes through many steps between the time that the need for a project has been identified and the time that the completed project is put into service. The period before construction commences can generally be divided into ***master planning,*** which entail treatment needs and feasible options for meeting those needs, followed by ***process train selection***, where viable treatment options are subjected to bench, pilot and full-scale treatment investigations, followed by ***preliminary design***, which is the fine-tuning procedure where feasible alternatives for principal design features, such as location, treatment process arrangement, type of equipment and type and size of building enclosures, are evaluated, and ***final design***, where contract documents (drawings and specifications) are prepared that present the project design in sufficient detail to allow final regulatory approvals to be obtained, obtaining competitive bids from construction contractors, and actual facility construction (RSA ECSA, 2013).

For the above steps to be effectively and efficiently implemented without undue delay and omitting crucial steps, many technical and non-technical individuals must be involved, not only during the four phases of project development, but between these phases as well.

In the context of South Africa as shown in the literature review and the case studies discussed in this dissertation, mistakes can be avoided by following all the phases required when designing water treatment plants. To arrest costly gaps from occurring, the author of this report recommends the following for improving and optimising the designing and installation of effective and efficient water treatment facilities:

1. A **Water Treatment Design Committee (WDC)** or a **National Water Agency of South Africa (NWASA)** should be formed that would be responsible for handling projects throughout South Africa, comprising people possessing these skills coupled with at least five years of water treatment-related experience like it is the case with the Ten states and province and Umgeni Water:

    a) **Water treatment specialist** with specific training and hands-on experience in the water treatment practice
    b) **Water treatment chemist** with specific training and hands-on experience in the water treatment practice
    c) **Water treatment plant design engineer** with specific training and hands-on experience in the water treatment practice;
    d) **Water treatment microbiologist** with specific training and hands on experience in the water treatment practice
    e) **Maintenance engineer** with hands-on experience in the maintenance of water treatment plants
    f) **Instrumentation engineer** with hands-on experience in the installation and maintenance of water-related instruments and software applicable to the water industry
    g) **Human resources practitioner** with sound knowledge of the water treatment industry

h) **Finance practitioner** with sound knowledge of project funding and regulatory requirement
i) **Specific project sponsor representative**, e.g. DWS or utility representative
j) **Value engineer** with relevant training and experience in the design, operation and maintenance of water treatment plants.

2. The design should follow the conventional water treatment plant design, and where this deviates, a plausible explanation/motivation must be provided by the consultant responsible for the change for approval by the WDC or NWASA.

3. Advanced treatment technologies should be incorporated once they have been approved by the WDC or NWASA after considering the following:

   a) The relevance and the need of the treatment technology proposed to the water to be treated
   b) Value (financial versus expected benefits) of the proposed technology
   c) Levels of skills of both operating and maintenance personnel
   d) Alternative treatment technologies
   e) Value engineering
   f) Availability of parts and spares for the proposed technology
   g) Lifespan costing
   h) Required training for operating and maintenance personnel
   i) Experienced people using or who have used the same technology should be sought

4. New process technology to be limited to cases where there are no proven alternate technologies.

5. Preliminary and final designs to be approved by the design committee after considering the following:

a) General design considerations, namely the investigation phase, design specifications, the tender procedure to be followed, detailed design, construction and commissioning, operation control and management, and monitoring
b) Raw water quality
c) Regulatory requirements in terms of final water quality to be produced
d) Process selection, configuration and justification thereof
e) Provision of future expansion in the design in relation to the ultimate plant design capacity
f) Elimination of unnecessary multiplicity of processes and installed equipment by catering for the ultimate plant capacity in the initial design of inlet, chemical storage facility, control centre and administration building location as a minimum.

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APPENDICES

The following are changes and/improvements already implemented in the Vaalkop WTP because of this dissertation.

Appendix 1: 1 500 mm diameter raw water pipeline line (green/lime coloured line)

Appendix 2: Common raw water intake catering for ultimate plant design capacity

Appendix 3: Chemical building catering for the ultimate plant design capacity

Appendix 4: Plant 4 sedimentation tank with v-notch overflow weir

Appendix 5: Plant 3 sedimentation tank with orifice overflow weirs