PLANNING FOR SEAWATER DESALINATION IN THE CONTEXT OF THE WESTERN CAPE WATER SUPPLY SYSTEM

by

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DECLARATION

By submitting this thesis electronically, I declare that the entirety of the work contained therein is my own, original work, that I am the authorship owner thereof (unless to the extent explicitly otherwise stated) and that I have not previously in its entirety or in part submitted it for obtaining any qualification.

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SINOPSIS

Suid-Afrika maak histories staat op goedkoop oppervlak- en grondwaterhulpbronne. Namate druk op hierdie hulpbronne aanhou toeneem, begin seewaterontsouting egter as ’n moontlike toekomstige waterbron na vore tree. Een van die stede wat as ’n kandidaat vir grootskaalse ontsouting geïdentifiseer is, is Kaapstad. Vir die meeste moontlike voordele teen die laagste moontlike koste, moet so ’n ontsoutingsaanleg as ’n integrale deel van die huidige stelsel beskou word. Geïntegreerde beplanning het tot dusver tekortgeskiet by bestaande ontsoutingsaanlegte in Suid-Afrika, wat merendeels as noodskemas opgerig is en waarvan die bedryf en bestuur ’n finansiële las op munisipaliteite plaas.

Onlangse navorsing oor skemas vir tussenbekkenwateroordrag toon dat ’n omvattende benadering vereis word om watervoorsiening uit ’n nuwe skema te beoordeel. Volgens so ’n benadering word die stelsel in die geheel stogasties gemodelleer en die geraamde wateroordrag onttrek. Dié omvattende benadering was dan ook die grondslag vir die modellering wat in hierdie navorsing onderneem is. Bestaande modelle van die Wes-Kaapse stelsel is aangepas om ’n ontsoutingsaanleg in te sluit, en kort- en langtermynontledings is vir verskeie moontlike ontsoutingsaanlegvermoëns en -bedryfscenario’s voltooi. Die toename in stelselopbrengs en die jaarlikse watervoorsiening uit die ontsoutingsaanleg is bepaal. Kapitaal- en bedryfskoste van die eerste orde is geraam, welke koste toe met die jaarlikse voorsieningswaardes gekombineer is om eenheidsverwysingswaardes te bereken en te vergelyk.

Die maksimum toename in opbrengs blyk te wees wanneer die ontsoutingsaanleg as ’n basisbron dien wat te alle tye in werking is. Wat stelselopbrengs betref, was daar weinig voordeel in die gebruik van die aanleg as ’n noodwaterbron. Eenheidsverwysingswaardes vir die ontsoutingsaanleg neem af namate die persentasie voorsiening uit die aanleg toeneem, wat beteken dat die laagste moontlike koste per kubieke meter water verkry word wanneer die ontsoutingsaanleg as ’n deurlopende basisbron dien. Dit was ook duidelik dat die eenheidsverwysingswaardes afneem met ’n toename in aanlegvermoë, wat te kenne gee dat die optimale oplossing uit ’n ekonomiese oogpunt sou wees om onmiddellik een groot ontsoutingsaanleg in bedryf te stel. Hoe laer die opgaardamvlak waarop die ontsoutingsaanleg in werking tree, hoe groter die stogastiese variasie in watervoorsiening uit die aanleg, en hoe groter die variasie in koste. Daarom is die gebruik van stogastiese modellering om eenheidsverwysingswaardes te bereken veral belangrik vir ’n ontsoutingsaanleg wat as ’n spits- of noodwaterbron dien.

SYNOPSIS

South Africa has historically been reliant on inexpensive surface and groundwater resources; however, as pressure on these resources continues to grow, seawater desalination has begun to emerge as a potential future supply source. One of the towns earmarked as a candidate for large-scale seawater desalination is Cape Town. In order to maximise the benefits and minimise the costs of such a scheme, the desalination plant needs to be considered as an integral part of the current system. Integrated planning has been lacking at the existing seawater desalination plants in South Africa, most of which were constructed as emergency schemes and are financially cumbersome for the municipalities to operate and manage.

Recent research related to inter-basin water transfer schemes has shown that a comprehensive approach is required in assessing water supply from a new scheme in which the system as a whole is modelled stochastically and the estimated water transfer extracted. This comprehensive approach was the foundation of the modelling undertaken in this research. Existing models of the Western Cape system were adapted to include a seawater desalination plant, and short-term and long-term analyses were completed for a variety of possible desalination plant operating scenarios and capacities. The increase in system yield and the annual supply from the desalination plant were determined. First-order capital and operating costs were estimated, and these costs were combined with the annual supply values to calculate and compare unit reference values.

The maximum increase in yield was found to occur when the seawater desalination plant is used as a base supply, operational all the time. There was little benefit, in terms of system yield, in using the desalination plant as an emergency supply source only. Unit reference values for the desalination plant decrease as the percentage supply from the desalination plant increases, meaning that the lowest possible cost per cubic metre of water supplied is when the desalination plant is used as a base supply. It was also apparent that the unit reference values decrease with an increase in desalination plant capacity, suggesting that, from an economic perspective, the optimal solution would be to have one large desalination plant operational immediately. The lower the reservoir trigger level at which the desalination plant becomes active, the larger the stochastic variation in the supply from the desalination plant and hence the larger the variation in the costs. Hence, using stochastic modelling to calculate unit reference values is particularly important for integrating a desalination plant

into an existing conventional supply system when used as a peak or emergency supply source.

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LIST OF ABBREVIATIONS AND SYMBOLS

A_{mem}	Total required membrane area
ARMA	Auto-regressive moving average
A_t	Sequence of independent random variables with normal distribution having zero mean and constant variance
AUS$	Australian dollar
b	Power law factor
Cap	Total estimated capital cost
C_n	Costs incurred in year n
$Cost_m$	Membrane cost
D	Electricity tariff
DWS	Department of Water and Sanitation
E_{desal}	Energy consumption of desalination plant per pass
E_{other}	Energy consumption of pre-treatment, post-treatment and backwash systems
EWRs	Environmental water requirements
€	Euro
F_n	Long-term risk of failure
F	Annual risk of failure
FSL	Full supply level
GIS	Geographical information system
ha	Hectares
HFY	Historic firm yield
i	Interest rate
IB	Irrigation board
km	Kilometre
kW	Kilowatt
ℓ	Litre
LN2 (or 3)	Log normal type 2 or 3 distribution
m	Metres
m^2	Square metres

m^3	Cubic metres
m^3/a	Cubic metres per annum
m^3/d	Cubic metres per day
m^3/h	Cubic metres per hour
m^3/s	Cubic metres per second
M_{cap}	Specific cost of capital redemption of desalination plant
M_{energy}	Specific energy cost of desalination plant
M_{maint}	Specific cost of maintenance of desalination plant
M_{mem}	Specific cost of membrane replacement
M_{tot}	Total specific cost of desalination plant
mg	Milligram
Mℓ	Megalitre
MOL	Minimum operating level
MW	Megawatt
NMBMM	Nelson Mandela Bay Metropolitan Municipality
n	Planning period / period of loan
$P_{F(T)}$	Feed water pressure at temperature T
P_{in}	Sum of feed pressures for intake and pre-treatment systems
PV_c	Present value of costs
PV_w	Present value of the water supplied
Q_d	Daily desalination plant production rate
Q_{eff}	Brine discharge rate
Q_f	Hourly desalination plant feed flow rate
Q_h	Hourly desalination plant production rate
$Q_{h(in)}$	Hourly feed capacity of pre-treatment plant
R	Rand
r	Discount rate
R_f	Water recovery factor
RO	Reverse osmosis
RSM	Rapid Simulation Model
SB3 or 4	3 or 4 parameter bounded distribution

SSC	Strategy Steering Committee
STOMSA	Stochastic Streamflow Model
T	Feed water temperature
t	Recurrence interval of failure
TDS	Total dissolved salts
TMG	Table Mountain Group
u	Percentage allowance for maintenance
URV	Unit reference value
US$	United States Dollar
WAAS	Water Availability Assessment Study
WC/WDM	Water conservation and water demand management
WCRWS	Western Corridor Recycled Water Scheme
WCSA	Western Cape System Analysis
WCWSS	Western Cape Water Supply System
W_n	Water supplied in year n
WReMP	Water Resources Modelling Platform
WRPM	Water Resources Planning Model
WRYM	Water Resources Yield Model
WTW	Water treatment works
WUA	Water user association
x	Annual streamflow variate
Xt	Stationary sequence of centred normal variates
Y	Frequency of membrane replacement
y	Transformed variate
z	Factor for number of desalination passes

α	Fraction of day for which desalination plant is operational
β	Fraction of feed water lost in pre-treatment
ε	Flux per driving pressure
φ	Average flux per pass
η_p	Efficiency of pump

η_r	Efficiency of energy recovery unit
θ_1, θ_2	Moving average model parameters
ϕ_1, ϕ_2	Auto-regressive model parameters

1 INTRODUCTION

1.1 Background

Desalination can take the form of seawater desalination, desalination of brackish or polluted groundwater and the use of membrane processes in advanced water treatment for water reuse. All of these forms of desalination are relevant in South Africa and are currently being considered as possible supply sources for many major cities, including Cape Town. However, the focus of this research is seawater desalination only, firstly because it is likely to be a large-scale supply source (as opposed to brackish groundwater desalination which will be on a much smaller scale) and secondly because the practicality and acceptance of reuse of treated effluent has not yet been fully researched.

Due to the availability of less costly surface and groundwater resources to meet water demands, desalination of seawater has historically not been considered an economically viable water source in South Africa. There are currently only six small seawater desalination plants in operation in the country, with the largest being the 15 Mℓ/day Mossel Bay desalination plant commissioned in September 2011. Half of the existing seawater desalination plants were constructed as emergency supply schemes in response to the severe drought experienced in the Southern Cape in 2009/2010.

As pressure on surface water resources in South Africa continues to grow, as a result of population growth, increased levels of service, economic growth, climate change and declining water quality, desalination has started to gain traction as a potential source for meeting future water demands. It is predicted that the South African desalination market will grow by 28 percent over the next five years (Slabbert, 2012), and seawater desalination is currently being investigated at a feasibility level in Saldanha Bay, Cape Town, Port Elizabeth and Durban. In addition, all of the national water planning documents highlight seawater desalination as a possible future supply source.

South Africa is inevitably moving towards the implementation of large-scale seawater desalination plants at the major coastal cities; hence the focus should now be on developing the necessary skills and planning mechanisms to implement these projects. However, considering the existing seawater desalination plants, there have been some obvious limitations in terms of holistic planning for seawater desalination. In Mossel Bay, for example, heavy rains have fallen since the plant was commissioned in 2011, and it has therefore remained almost entirely untapped for the past three years, with the Municipality preferring to use its significantly less costly surface water resources. Recent research has shown that the Municipality would prefer to reduce the assurance of supply to its users rather than make use of the costly seawater desalination plant, raising questions about the long-term sustainability of the R250 million project.

South Africa is not the only country where seawater desalination plants have been hastily built as emergency schemes with major long-term financial implications. Australia, for example, suffered a decade-long drought between 2000 and 2009 which prompted emergency construction of large-scale seawater desalination plants in all of the major cities at huge cost.

The total design capacity of seawater desalination plants in Australia for potable supply jumped from 153 Mℓ/day in 2008 to 1 700 Mℓ/day in 2013 (Hoang *et al*., 2009). However, just when the bulk of the desalination plants were completed and commissioned, the drought ended rendering them unnecessary and hence a huge financial burden. In some cases, water authorities were locked into supply agreements with desalination plant contractors, meaning that the plants had to remain operational regardless of the availability of water from other sources. In response to some of these issues, water authorities have developed integrated operating rules with specific trigger levels in the overall system storage at which the seawater desalination plants becomes operational.

The South African and Australian examples clearly illustrate the importance of integrated system analysis when planning for seawater desalination in the context of an existing supply system which is predominantly reliant on surface water.

1.2 Seawater Desalination for Cape Town

Recent strategic water resource planning undertaken by the Department of Water and Sanitation (DWS)[1] and major stakeholders confirmed that seawater desalination is a potential water supply source for Cape Town, prompting the commissioning of a Feasibility Study in late 2011. The findings of the study have not yet been released; however in reviewing the terms of reference it appeared that insufficient attention had been paid to how the seawater desalination plant would be integrated into the current Western Cape Water Supply System (WCWSS) which currently supplies the bulk of Cape Town’s water demands.

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[1] DWS was previously the Department of Water Affairs and Forestry (prior to 2009) and subsequently the Department of Water Affairs (from 2009 to 2014). The department is hereafter referred to by its current designation (DWS) apart from in the references, where the name at the time of publication of the referenced document is used.

The decision regarding when to implement future water supply schemes, including seawater desalination, is currently being driven by the Western Cape Reconciliation Strategy Steering Committee (SSC) and the WCWSS Reconciliation Strategy. However, seawater desalination is very different to any of the surface water interventions being considered for the WCWSS. Firstly, seawater desalination is likely to have much higher capital and operating costs. Secondly, seawater desalination has an assurance of supply of essentially 100 percent as it is not affected by rainfall variability in the way that surface water resources are. In other words, seawater desalination is climate resilient rather than climate reliant. The decision of when to implement a seawater desalination plant may, therefore, be dictated by integration into the WCWSS, associated supply risks and cost implications, rather than by a 98 percent assurance of supply water balance, as is currently used for water resource planning of the WCWSS. Furthermore, the higher costs and assurance of supply will impact on the operational philosophy of the plant.

Despite advances in seawater desalination technology over the past few decades, which have markedly reduced costs, large-scale seawater desalination for Cape Town will undoubtedly be an expensive option. In South Africa, the costs for potential water supply schemes are usually compared based on the unit reference value (URV) approach, in which the present value of the capital and operating costs for a scheme is divided by the present value of water supplied over a selected planning horizon, giving a cost per unit of water supplied. In integrating a seawater desalination plant into the WCWSS, the choice of operating rules would directly influence the actual supply from the desalination plant and hence impact on the URV. It is therefore essential that the timing, capacity and operating philosophy of the plant be thoroughly analysed to ensure that an optimal solution is found in terms of cost and risk.

1.3 Objectives

The objective of this research was to determine what operating rules and modified planning criteria are required to optimise the implementation of large-scale seawater desalination as part of the WCWSS, in order to ensure cost-effective provision of water at an appropriate assurance of supply. Achieving this objective required a thorough review of previous research into integrated seawater desalination planning, an understanding of the complex operation of the WCWSS, detailed long-term and short-term system modelling and a comparison of the costs (URVs) of the modelled scenarios.

1.4 Outline of Thesis

This thesis is divided into eight sections as follows:

- Section 1 (this section) presents an introduction, outlining the background and objectives of the research.
- Section 2 starts with an overview of the basic concepts of water supply system analysis, followed by a description of the WCWSS, its main system components and their operation. Historic and current WCWSS planning documents are also reviewed.
- The focus of Section 3 is on planning for seawater desalination, starting with an overview of the existing and planned seawater desalination plants in South Africa, and references to seawater desalination in national and regional planning documents. Local and international methodologies for analysing the conjunctive use of seawater desalination plants with surface water supply sources are discussed along with methodologies for seawater desalination plant cost estimation. Section 3 ends with a case study of seawater desalination planning in Australia.
- In Section 4, the selection of the most appropriate tools for carrying out the modelling required as part of this research is discussed, and the theoretical background of the chosen models, the Water Resources Yield Model (WRYM) and Water Resources Planning Model (WRPM), is presented.
- Section 5 is focussed on the methodology developed as part of this research for integrating seawater desalination into the existing WCWSS setup and for estimating first-order seawater desalination plant costs and URVs.
- The system modelling and cost analysis results are presented and discussed in Section 6.
- Section 7 provides final conclusions as well as recommendations for future research.
- Lastly, a list of referenced documents is provided in Section 8.

2 WESTERN CAPE WATER SUPPLY SYSTEM

The focus of this research, as discussed in Section 1, is seawater desalination in the context of the WCWSS. Before delving into the theory and practicality of seawater desalination planning in South Africa and around the world, which is discussed in Section 3, it was considered worthwhile to provide some background on the WCWSS.

This section presents an overview of the main components of the WCWSS, the fundamentals of its operation as well as the key planning and management mechanisms which are in place in order to ensure sufficient water is available to meet the demands of all users at an acceptable risk of failure. The section starts with a brief discussion of some of the key concepts and terminology which formed the building blocks of this research.

2.1 Terminology and Basic Concepts

Throughout Section 2, 3 and 4 reference is made to a number of commonly used terms and principals related to water supply modelling, system analysis and planning. A brief discussion of these concepts including flow sequences, yield, assurance of supply and URVs is provided in Sections 2.1.1 to 2.1.4.

2.1.1 Flow sequences and hydrology

The fundamental building block of all hydrological analysis is a time series or data sequence, which is defined as a list of data in chronological order (Basson, Allen, Pegram & Van Rooyen, 1994:12). Sequences can be at a daily or monthly time-step, can comprise rainfall, evaporation or flow data, and can be classified as one of the following (Basson *et al.,* 1994:12-15):

- An <u>observed sequence</u> is generally regarded as a series of raw data recorded at regular time intervals using whatever method is applicable to the type of data that is being collected. An observed sequence will often have errors and missing values resulting from human error or malfunctioning of the recording equipment.
- A <u>patched sequence</u> is an observed sequence where any missing or erroneous values have been corrected using regression, deterministic or stochastic techniques.
- A <u>naturalised sequence</u> is a patched sequence that is artificially corrected to be representative of the river flow without human impacts such as domestic water abstractions, irrigation, afforestation or increased runoff from paved areas.

- A historical sequence is a naturalised, patched, observed flow sequence which has been extended using a deterministic or stochastic technique, to suit a required analysis period.
- A simulated sequence is one which has been artificially created using a computer model. Simulated sequences can be either deterministic or stochastic.
- A deterministic sequence is generally created using a rainfall-runoff model which uses actual catchment parameters and observed rainfall to generate daily or monthly flows. Often, the simulated flows are calibrated based on an observed sequence. Typically rainfall data in South Africa dates back to the early 1900s and is quite reliable whereas flow records only start in the 1950s, are less consistent and are only available at a limited number of locations. Rainfall-runoff models make it possible to generate long-term flow sequences at any location provided there is sufficient rainfall data available.
- Stochastic sequences are generated using the statistical characteristics of a historic sequence based on carefully devised mathematical models and are representative of possible variations of a historic sequence which might have occurred.

2.1.2 Basic concepts of yield

Consider a simple water supply system with only one reservoir. The reservoir will have a simulated inflow sequence created using a calibrated rainfall-runoff model. It will have physical characteristics including a level-volume-area curve, which defines the surface area and volume of water in the dam at different elevations, and a spillway with a specific shape and discharge equation. Also important are the full supply level (FSL), which is the level at which the dam will spill, and the minimum operating level (MOL), which is the level at which the dam can no longer supply water.

The volume of water which one aims to obtain from the reservoir is referred to as the target draft (Basson *et al.*, 1994:16). In order to analyse how a dam will perform based on a given set of physical characteristics, an inflow sequence and a specific target draft, a time-step-based analysis would be performed. The dam would start at an assumed storage level, often taken as the FSL. In one time step, there will be inflow into the dam, rainfall on the surface of the dam, evaporation from the surface of the dam, infiltration losses, spillage if the dam is full, and abstractions to meet the target draft. The water level in the dam plotted as a sequence over the analysis period, taking these inflows and outflows into account, is referred to as its trajectory.

The yield of a dam or system is the volume of water which is abstracted over a specific time period (Basson *et al.*, 1994:16). Ideally the yield would be equal to the target draft, meaning that the demand is always supplied. However, the ability to supply the target draft at a given time is reliant on the volume of water left in the reservoir for that time step as well as any infrastructure constraints which limit the volume of water that can be abstracted. Yield values are typically quoted in million cubic metres per annum regardless of whether the analysis is carried out on a daily or monthly time step.

The lowest annual yield recorded over the analysis period is referred to as the base yield. If the target draft is increased from zero in small enough increments, the base yield would initially be equal to the target draft, meaning that the demand can be met for the entire analysis period. However, as the target draft increases, it will eventually reach a point where it cannot be met for at least one year and the base yield will therefore be less than the target draft. The point where this change occurs will be the maximum possible base yield of the system, referred to as the firm yield, or historic firm yield if it has been calculated based on a historic flow sequence. These principals are illustrated on **Figure 2.1.**

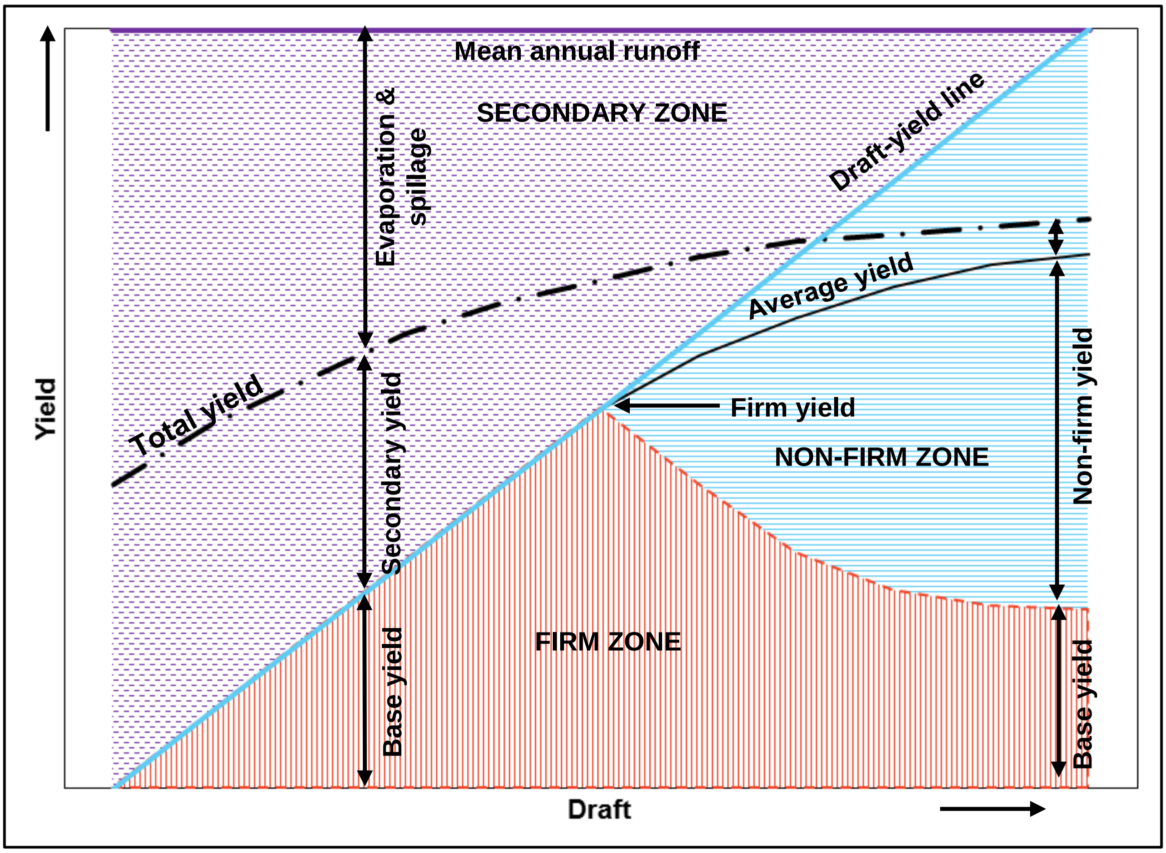

Figure 2.1: Typical plot of yield against target draft derived from Basson *et al.* (1994:18)

For base yields below the firm yield the reservoir trajectory will never reach the MOL, however at the firm yield point, the reservoir trajectory will just reach the MOL. The period in which this occurs is referred to as the critical period, defined as the "*time taken for a reservoir to be drawn down from full to empty and be just about to start to fill again*" (Basson *et al.*, 1994:29).

Any yield that can be abstracted in addition to the base yield to meet the target draft is referred to as non-firm yield. The non-firm yield is essentially water that can be abstracted some of the time in order to meet the target draft, but is not a yield that is always available (Basson *et al.,* 1994:22). The sum of the average value of the non-firm yield and the base yield gives the average yield, which is essentially the average volume of water supplied to meet the target draft over the analysis period. For target drafts up to the firm yield point, the non-firm yield will be zero and the average yield will, therefore, equal the base yield. For target drafts higher than the firm yield, the average yield line will diverge from the draft-yield line. This is illustrated on **Figure 2.1**

Secondary yield is the yield that could theoretically be abstracted in excess of the target draft during times when the reservoir is at FSL, in other words when the dam is spilling. At any point in time it will be equal to the spillage from the dam, provided that this value is less than or equal to the installed abstraction capacity. Secondary yield can only be utilised if there is downstream storage available after abstraction (Basson *et al.,* 1994:21).

Lastly, as shown on **Figure 2.1,** the sum of the firm (base yield), non-firm yield and the secondary yield is referred to as the total yield which is the average volume of water that could theoretically be abstracted over the analysis period restricted only by the installed abstraction capacity.

2.1.3 Assurance of supply

The principles discussed in Section 2.1.2 are useful for understanding the characteristics of a reservoir or system based on a historic flow sequence. However, in analysing only one sequence, it is not possible to derive any information concerning the assurance of supply associated with the calculated yield.

The risk of failure is defined as the "*probability of not being able to supply the base yield associated with a specific target draft at least once over a specified time horizon*" (Basson *et al.*, 1994:30). If a series of stochastic sequences is generated and the yield determined based on each sequence with all other factors the same, the risk of failure can be calculated.

It is recommended that at least 40 stochastic sequences be analysed for a long-term yield analysis (Basson *et al.*, 1994:34) and at least five times this number, hence 200 sequences, be analysed for a short-term yield analysis (Basson *et al.*, 1994:46) in order to ensure a reasonable degree of accuracy without excessive computational requirements.

The most commonly used concept in yield analysis is the recurrence interval of failure, which is essentially the reciprocal of the annual risk of failure (the probability of failing in a particular year). For example, a recurrence interval of failure of 100 years means that, on average, there will be a failure for 1 in every 100 years. The corresponding annual risk of failure will be equal to 1 divided by 100 or 1 percent. The annual assurance of supply is therefore 99 percent (100 percent minus 1 percent). The long-term risk of failure is a function of the annual risk of failure and the planning or analysis period, as per the Bernoulli probability equation provided in Equation 2.1 (Basson *et al.,* 1994:31).

$$F_n = 1 - (1 - F)^n = 1 - (1 - 1/t)^n \tag{2.1}$$

Where:

F_n	Long-term risk of failure	[-]
F	Annual risk of failure	[-]
t	Recurrence interval of failure	[years]
n	Planning period	[years]

In a long-term yield analysis, stochastic sequences are generated based on the historic flow sequence, and each is analysed for a range of target drafts. For some of the sequences, the base yield will be equal to the target draft and for others it will be less than the target draft. The results are then ranked and the percentage of the observed sequences which exceed each yield value is calculated and plotted, creating a family of curves as shown in **Figure 2.2**. The break points of each curve give an indication of the long-term assurance of supply of that particular target draft. A line drawn through the break points of each curve is referred to as the firm yield line, and using this line, yield values with a given recurrence interval can be read off as shown in **Figure 2.2.**

A family of yield-reliability curves can also be used to determine whether a system can supply a variety of users at different assurances of supply. An example illustrating this principal is provided in **Figure 2.3**. Three users have demands of 10 million m^3/a, 23 million m^3/a and 14.5 million m^3/a at assurances of supply of 99.5 percent, 99 percent and 98 percent respectively. None of the demands exceed the yield-reliability curve for a target draft of 47.5 million m^3/a meaning that all three users will be supplied as required.

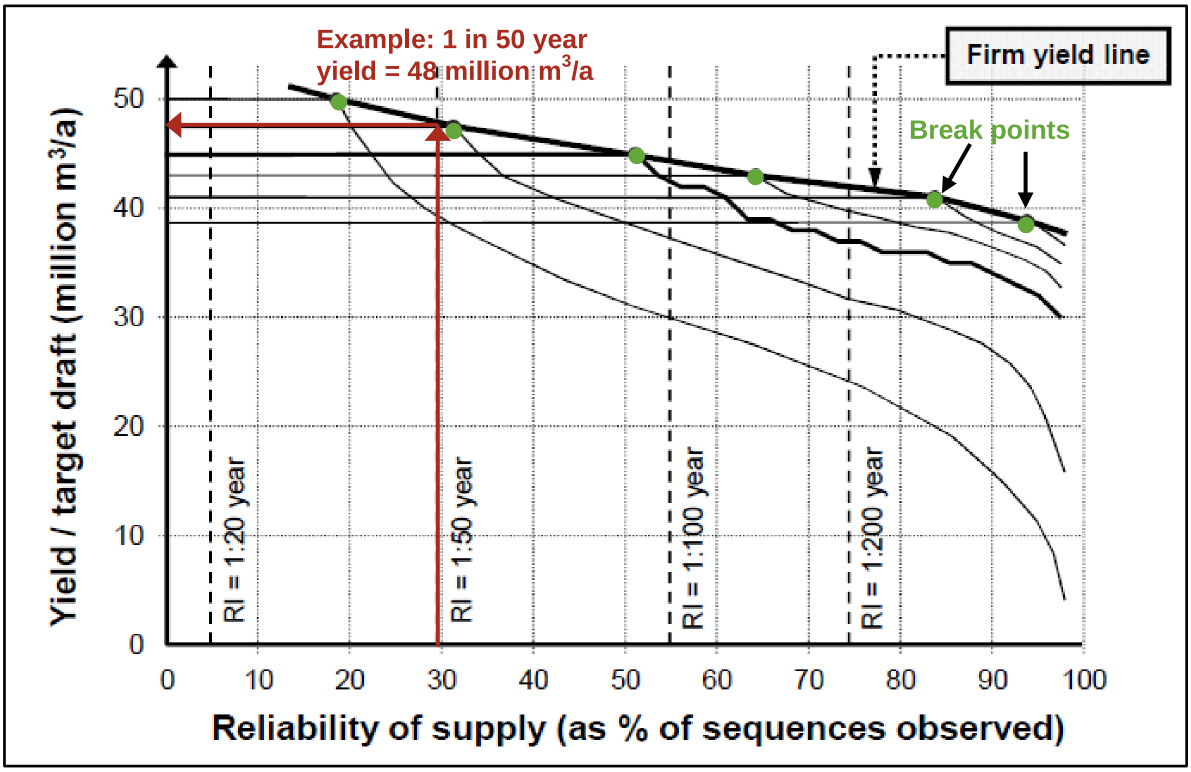

Figure 2.2: Example of a typical family of yield-reliability curves (De Jager, 2011)

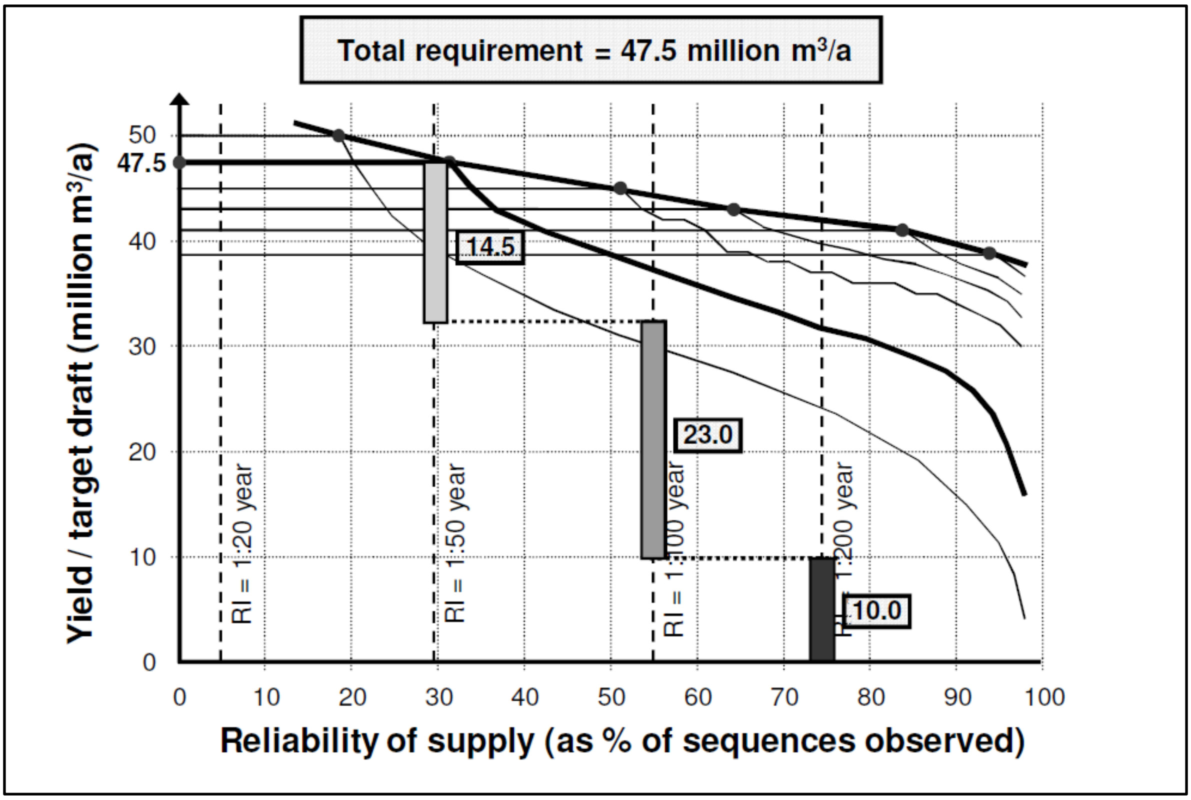

Figure 2.3: Example illustrating the use of yield-reliability curves to assess supply of demands at different assurances of supply (De Jager, 2011)

2.1.4 Unit reference value

The last of the important parameters to be aware of relates to the costing and comparison of different water supply interventions. The URV method was developed specifically for the evaluation and comparison of projects in the water sector which have different costs and yields and is widely used in water resource planning in South Africa. The URV of a scheme or intervention is calculated by dividing the present value of the scheme costs by the present value of the volume of water supplied over the project lifecycle, as per Equation 2.2 (Hoffman & Du Plessis, 2008).

$$URV = PV_c / PV_w \tag{2.2}$$

Where:

URV	Unit reference value	[R/m^3]
PV_C	Present value of costs	[R]
PV_w	Present value of the volume of water supplied	[m^3]

The present value of the scheme costs is calculated using Equation 2.3 and the present value of the water supplied is calculated using Equation 2.4 (Hoffman & Du Plessis, 2008).

$$PV_c = \sum (C_n)/(1+r)^n \tag{2.3}$$

Where:

C_n	Costs incurred in year n	[R]
r	Chosen discount rate	[-]

$$PV_w = \sum (W_n)/(1+r)^n \tag{2.4}$$

Where:

W_n	Volume of water supplied in year n	[m^3]

The costs included in the calculation would include the full lifecycle costs associated with the scheme (capital investment, operating costs and maintenance costs) and the volume of water supplied would be equivalent to the actual yield supplied in each year. Typically, URVs would be calculated for a range of at least three discount rates (typically 6, 8 and 10 percent) for an analysis period of around 20 years, but this obviously varies depending on the project specifics (Van Niekerk & Du Plessis, 2013:551).

2.2 Description of the Existing WCWSS

2.2.1 Overview of the WCWSS

The WCWSS, presented schematically in **Figure 2.4**, provides water to various towns and irrigators in the Western Cape including:

- The City of Cape Town;
- Certain towns in the Overberg, Boland, West Coast and Swartland areas;
- Irrigators along the Berg River, Eerste River and Riviersonderend River; and
- Rural and stock watering schemes in the West Coast, Swartland and Overberg areas.

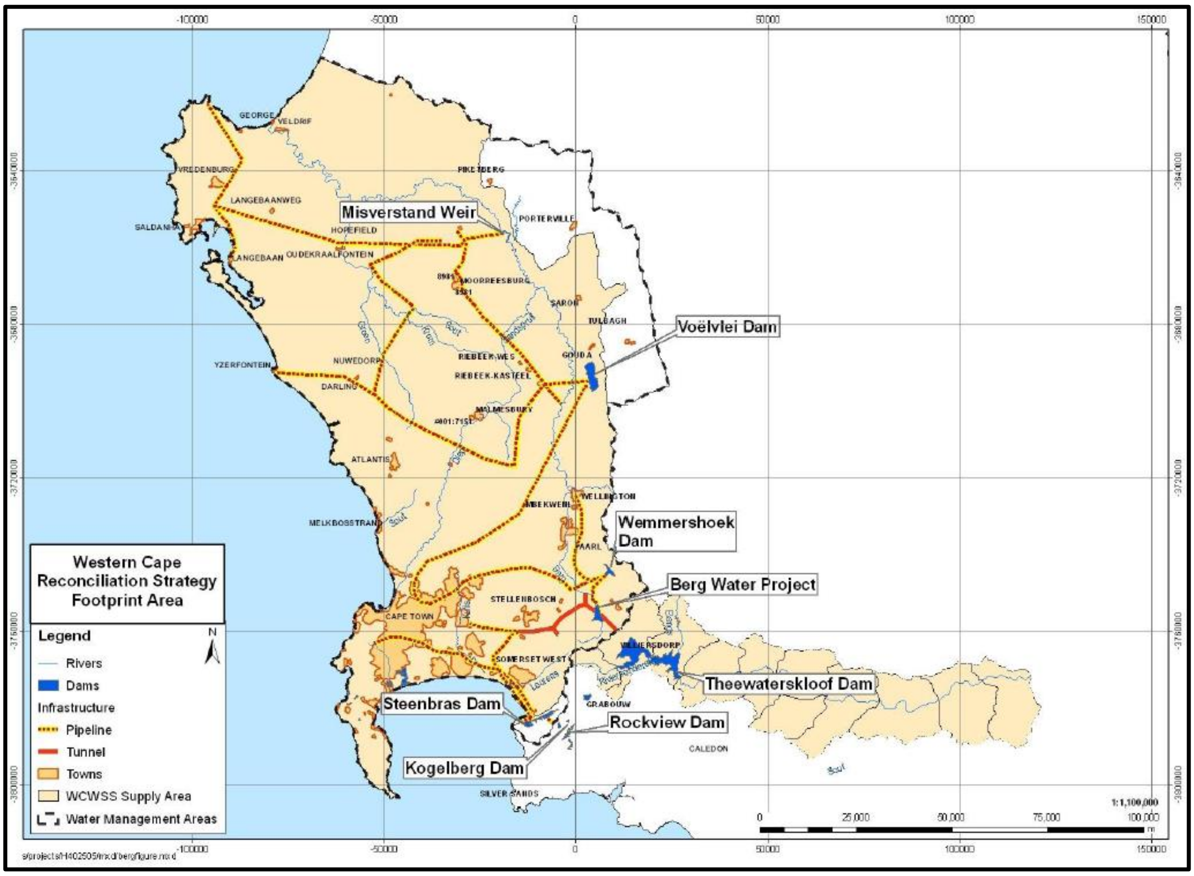

Figure 2.4: Western Cape Water Supply System (Department of Water Affairs, 2009a:Appendix A)

The main dams, as indicated on **Figure 2.4**, along with their storage capacities, 1 in 50 year yields, owners and operators are listed in **Table 2.1**. **Table 2.2** presents a summary of the key water treatment works (WTW) which form part of the water supply system along with their capacities and supply sources. More details on the dams and individual supply schemes are provided in Section 2.2.4 to 2.2.10

Table 2.1: Summary of main dams in the WCWSS (Shand & Sparks, 2004:1)

Dam	Mean annual runoff (million m^3/a)	Storage capacity (million m^3)	1 in 50 year system yield (million m^3/a)	Dam owner	Dam operator
Theewaterskloof (includes Banhoek and Wolwekloof)	267	480	219	DWS	DWS
Voëlvlei	126	172	105	DWS	DWS
Berg River	116	127	56	TCTA	DWS
Wemmershoek	68	59	54	City of Cape Town	City of Cape Town
Upper Steenbras	17	32	40 (62.5)(1)	City of Cape Town	City of Cape Town
Lower Steenbras	26	34		City of Cape Town	City of Cape Town

(1) Including Palmiet transfer

Table 2.2: Summary of WCWSS water treatment works

Water treatment works	Treatment capacity (Mℓ/day)	Sources
Faure WTW	500	Riviersonderend-Berg River Government Water Scheme and Upper Steenbras Dam
Blackheath WTW	400	Riviersonderend-Berg River Government Water Scheme
Wemmershoek WTW	270	Wemmershoek Dam and Theewaterskloof Tunnel
Voëlvlei WTW	270	Voëlvlei Dam
Steenbras WTW	150	Lower Steenbras Dam
Withoogte WTW	72	Misverstand Weir
Swartland WTW	30	Voëlvlei Dam
Paradyskloof WTW	10	Riviersonderend-Berg River Government Water Scheme

The Western Cape area is characterised by a Mediterranean climate with rainfall occurring in the winter months from May to October and little to no rainfall occurring in the hot, dry summer months. Approximately 90 percent of the inflows into the dams occur during the winter and only 10 percent in the summer. However, only 30 percent of the demand occurs in winter and 70 percent in summer. The dams are, therefore, filled during the winter months and drawn down during the dry summer months. Approximately 50 percent of the total system storage is available for storing winter flows to meet summer demand. The remaining 50 percent is required for long-term storage to meet demands during periods of reduced rainfall (Shand & Sparks, 2004:3).

2.2.2 System flexibility

The main dams including Theewaterskloof, Voëlvlei, Steenbras and Wemmershoek Dams are operated as an integral system in order to reduce the probability of spillage, hence increasing the overall system yield. In other words, the overall yield of the system is greater than the sum of the yields of the individual components. This requires close cooperation between DWS and the City of Cape Town, ensuring that demands on each dam are adjusted in order to maintain similar ratios between the projected median winter inflow and the storage available at each dam (Department of Water Affairs and Forestry, 2007:31).

In order to make such adjustments possible, the City of Cape Town has created additional capacity in their WTW and bulk water pipelines thus enabling flexibility in the system for allocating demands. In addition, the City can reduce demands on Wemmershoek and Steenbras (their cheaper sources) in order to reduce the probability of other dams spilling. If the dams are likely to spill even without inflow from the Palmiet Scheme (discussed in Section 2.2.4) or Berg River Supplement Scheme (discussed in Section 2.2.9) then supply from these schemes can be curtailed, hence limiting unnecessary pumping costs (Jonker, Dobinson, Sparks, Hallowes, Leirião & Shand, 2012:19).

2.2.3 Drought management

Yield and planning models of the WCWSS have been set up in the WRYM and WRPM respectively in order to simulate all inflows and demands on the system and plot storage trajectories of the coming year. The models (WRYM and WRPM) are discussed in more detail in Section 4.3 and 4.4 respectively.

In terms of drought management, the WRYM and WRPM are run annually at the end of the rainy season (November) in order to determine whether the dams are full enough to meet the summer demands at the required assurance of supply or whether water restrictions should be implemented. For domestic and industrial users, a 97 percent overall assurance of supply is required (including a sub-requirement of a 100 percent assurance for 70 percent of the time and 90 percent assurance for 30 percent of the time) and, for agricultural users, a 91 percent assurance of supply is maintained (including a sub-requirement of 100 percent assurance for 70 percent of the time and 70 percent assurance for 30 percent of the time) (Jonker *et al.*, 2012:20).

The principal applied is that less severe restrictions can be applied more frequently with more severe restrictions applied less frequently. Higher priority users are also restricted to a lesser extent than lower priority users, as shown in **Table 2.3** (Shand & Sparks, 2004:6).

Table 2.3: Frequency of water restrictions and restricted demands per user sector in the WCWSS (Shand & Sparks, 2004:6)

Level of restrictions	Acceptable frequency of restrictions	Restricted water demand as a percentage of normal demand		
		Domestic	Industrial	Agricultural
0	-	100%	100%	100%
1	1 in 20 years	90%	100%	90%
2	1 in 100 years	70%	90%	70%
3	1 in 200 years	50%	60%	50%

2.2.4 Palmiet scheme

The Palmiet River Government Water Scheme, presented schematically in **Figure 2.5**, comprises the Kogelberg and Rockview Dams which are the two major components of the Palmiet Pumped Storage Scheme, owned and operated by Eskom. During off peak periods (weekends and at night) water is pumped from Kogelberg Dam to Rockview Dam and, during peak periods, released back to Kogelberg Dam to generate up to 400 MW of power (Department of Water Affairs and Forestry & City of Cape Town, 2003:H-5). At the end of the weekend, approximately 17.5 million m^3 of capacity is available in the Kogelberg Dam to be filled by the daily releases from Rockview (Jonker *et al.*, 2012:23).

When the flow in the Palmiet River is sufficient to meet the wet season environmental requirements and transfer the weekly volumes for electricity generation, Eskom may be requested to extend their pumping hours and transfer additional water from Kogelberg Dam to Rockview Dam from where it can be released into the Upper Steenbras Dam, part of the Steenbras Pumped Storage Scheme (Department of Water Affairs and Forestry, 2004:26).

The operation of this diversion requires close cooperation between DWS, the City of Cape Town and Eskom, firstly to ensure that the water pumped to Rockview Dam is released as soon as possible to the Upper Steenbras Dam, and secondly to ensure that water is conveyed timeously from the Upper Steenbras Dam to Faure WTW to prevent this water being ‘lost’ due to the dam spilling (Shand & Sparks, 2004:4).

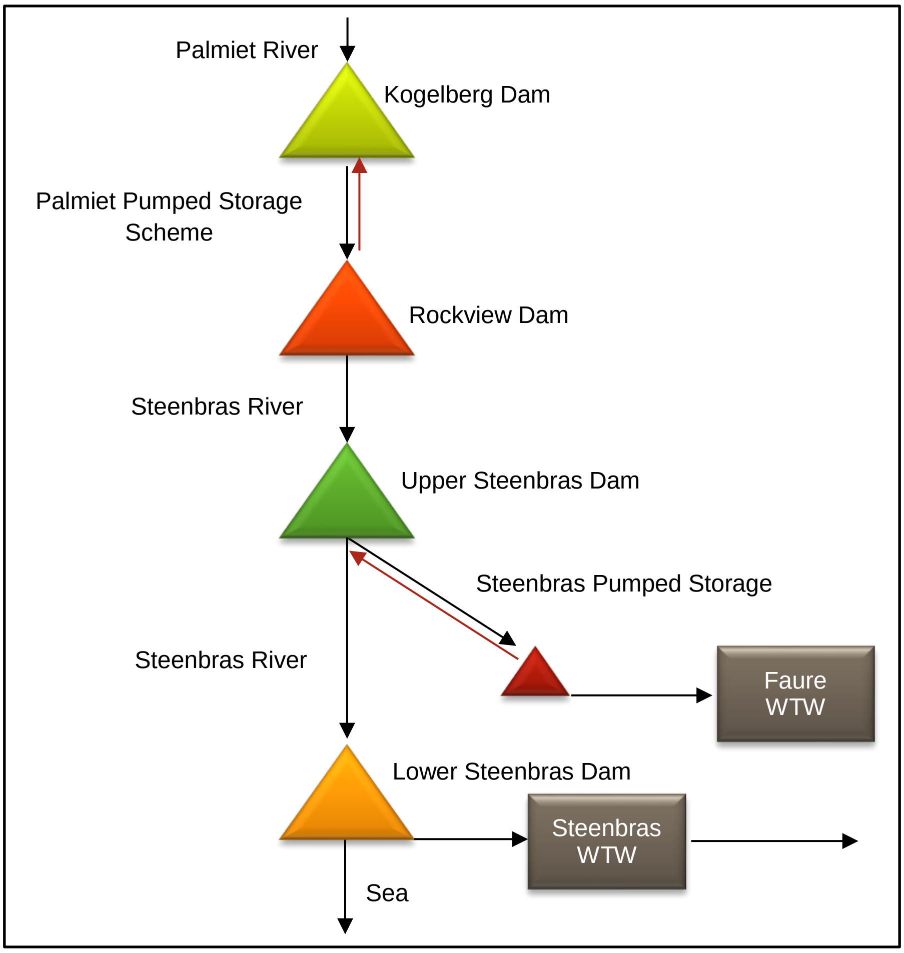

Figure 2.5: Palmiet River Government Water Scheme and Steenbras Scheme adapted from Department of Water Affairs and Forestry & City of Cape Town (2003:A8)

2.2.5 Steenbras scheme

The Steenbras Scheme, also shown on **Figure 2.5**, comprises the Upper and Lower Steenbras Dams supplemented by diversions from the Palmiet Scheme. The Upper Steenbras Dam is fed by its own catchment as well as water transferred from Rockview Dam as discussed in Section 2.2.4 (Department of Water Affairs and Forestry & City of Cape Town, 2003:H6). From the Upper Steenbras Dam, water can either be released for capture by the Lower Steenbras Dam or conveyed to the Faure WTW (32.5 million m^3/a) via the Steenbras Pumped Storage Scheme and the 300 Mℓ/day Firlands pump station, owned by the City of Cape Town (Jonker *et al.*, 2012:16). Water from the Lower Steenbras Dam is treated at the Steenbras WTW (30 million m^3/a) before conveyance to Cape Town.

2.2.6 Wemmershoek scheme

The Wemmershoek Dam receives inflow from its own catchment on the Wemmershoek River. The dam supplies the Wemmershoek WTW which in turn supplies Cape Town via the Wemmershoek pipeline, and Paarl and Wellington via the offtake 5 km downstream of the Wemmershoek WTW (Department of Water Affairs and Forestry, 2004:26). Approximately 38 million m^3/a is supplied to Cape Town, 16 million m^3/a to Paarl and Wellington and 3 million m^3/a is released from the dam for downstream irrigators (Jonker *et al.*, 2012:25). The WTW can also receive water via a pipeline connection from the Theewaterskloof Tunnel at the Berg River Siphon (Department of Water Affairs and Forestry & City of Cape Town, 2003:H8). The scheme is presented schematically in **Figure 2.6**.

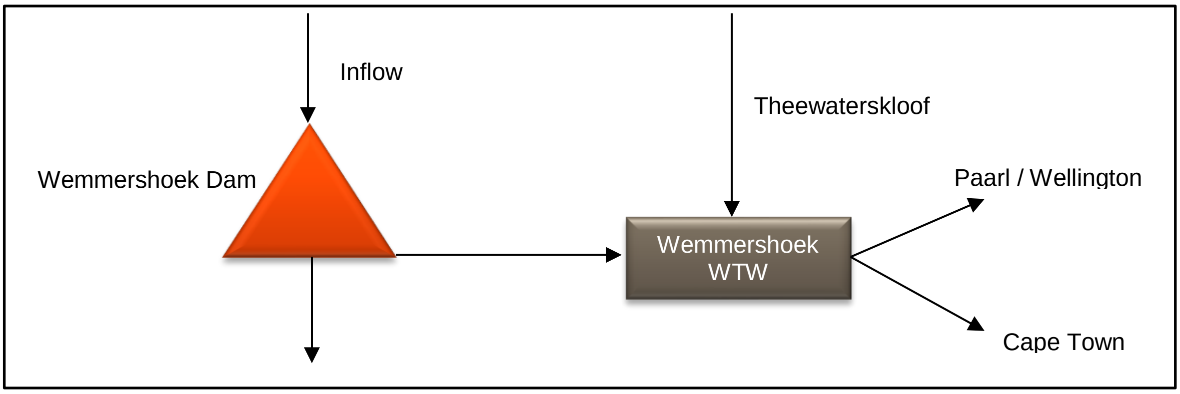

Figure 2.6: Wemmershoek Scheme adapted from Department of Water Affairs and Forestry and City of Cape Town (2003:A9)

2.2.7 Voëlvlei scheme

The Voëlvlei Dam is an off-channel dam located adjacent to the Berg River. At present it is supplied from two sources, namely the Klein Berg River and the Twenty-Four Rivers, both via gravity run-of-river diversions and canals (Department of Water Affairs and Forestry, 2004:26). The Twenty-Four Rivers canal feeds the dam in the winter months and local irrigators in the summer months (Jonker *et al.*, 2012:25). The dam supplies the Voëlvlei WTW (which in turn supplies Cape Town) via a high lift pump station, as well as supplying the Swartland Regional System via the Swartland WTW, Saldanha Regional System via the Misverstand Weir and irrigators in the Lower Berg River (Department of Water Affairs and Forestry, 2004:27).

Water is released from the Voëlvlei Dam via a canal into the Berg River for irrigation needs along with whole length of the Lower Berg River of approximately 18 million m^3/a over 3 644 ha (Department of Water Affairs and Forestry & City of Cape Town, 2003:H7). The Swartland Regional Scheme is supplied with 4.2 million m^3/a directly from the Voëlvlei Dam, whilst the Saldanha Regional System is supplied with 17.4 million m^3/a from the Misverstand Weir via the Withoogte WTW (Department of Water Affairs and Forestry, 2004:27). **Figure 2.7** shows a schematic layout of the scheme.

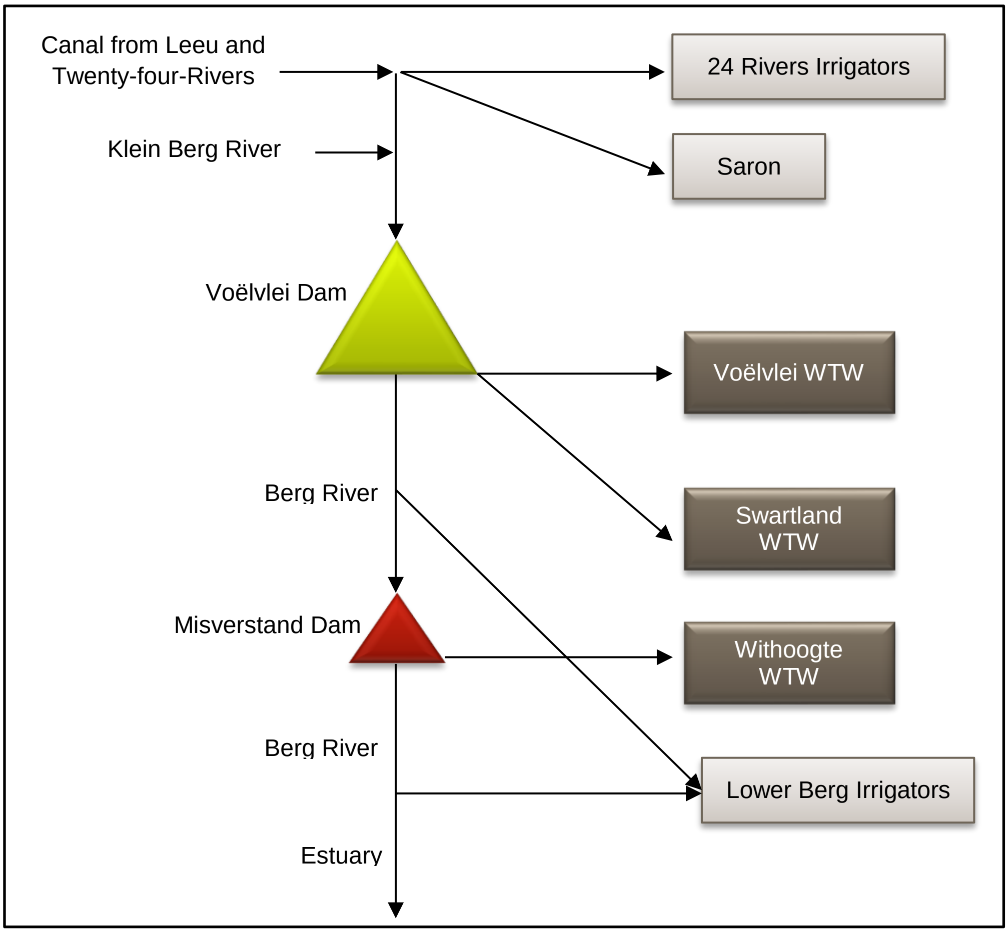

Figure 2.7: Voëlvlei Government Water Scheme adapted from Department of Water Affairs and Forestry and City of Cape Town (2003:A10)

2.2.8 Riviersonderend-Berg government water scheme

The Riviersonderend-Berg Government Water Scheme, presented schematically in **Figure 2.8**, comprises the Theewaterskloof Dam, Riviersonderend-Berg River Tunnel System and the Kleinplaas Dam.

The Theewaterskloof Dam is the main storage unit of the system, supplied from its own catchment, winter diversions from the Wolwekloof and Banhoek Rivers via the Theewaterskloof Tunnel System and winter diversions from the Berg River Dam (Department of Water Affairs and Forestry, 2004:27). The dam supplies the Riviersonderend Irrigation Board and Overberg's Ruênsveld schemes via releases into the Riviersonderend River, as well as the Vyeboom Irrigation Board directly from the dam. Releases from the dam into the Wemmershoek River tributary via the Dasbos Tunnel can supply Berg River irrigators and water can also be released to the Wemmershoek WTW (Jonker *et al.*, 2012:26).

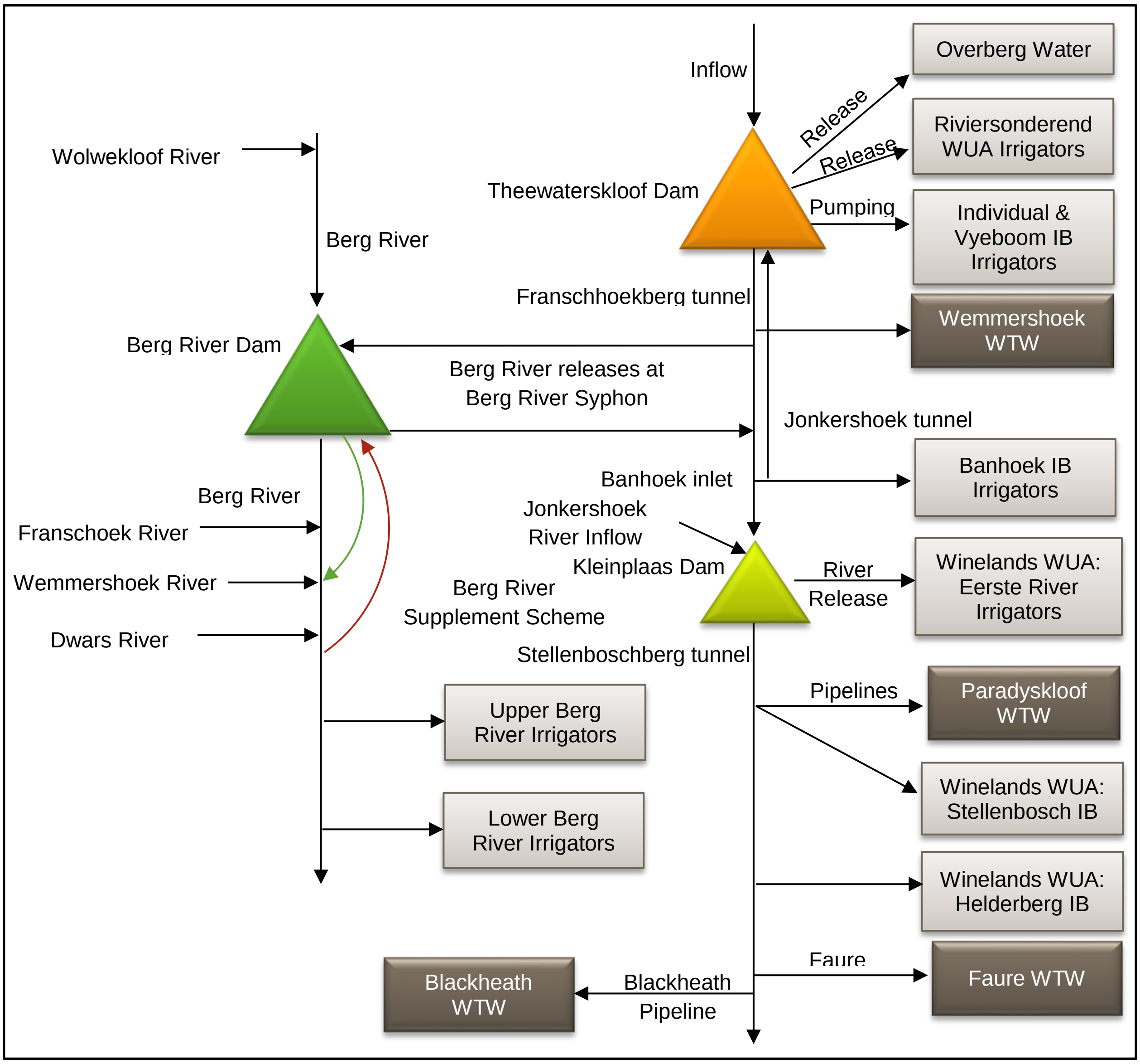

Figure 2.8: Riviersonderend-Berg River Government Water Scheme adapted from Department of Water Affairs and Forestry and City of Cape Town (2003:A12)

The Franschhoek/Jonkershoek tunnel system conveys water from the Theewaterskloof Dam to the Kleinplaas Dam, located on the Eerste River at Jonkershoek. During summer, most of the supply to Cape Town and irrigators passes through Kleinplaas Dam (Department of Water Affairs and Forestry & City of Cape Town, 2003:H9). Kleinplaas Dam supplies Stellenbosch Municipality via the Paradyskloof WTW, as well as the Winelands Water User Association comprising the Helderberg, Stellenbosch and Lower Eerste River Irrigation Boards. In addition, the dam supplies the Faure WTW via the Faure pipeline and the Blackheath WTW via the Blackheath pipeline, both of which supply Cape Town (Jonker *et al.*, 2012:26).

Jonker *et al.* (2012:21) note that the WCWSS is heavily reliant on the Riviersonderend-Berg-River Tunnel System from Theewaterskloof Dam, requiring careful planning of routine closures for maintenance. In order to meet the needs of the Eerste River, Stellenbosch and Helderberg irrigators as well as the Stellenbosch Municipality, no portion of the tunnel can be closed in summer. The City of Cape Town requires advance notice of summer tunnel closures to ensure that their demands are allocated to dams with adequate storage and to assist them in scheduling WTW maintenance. Berg River irrigators require that the Franschhoekberg portion of the tunnel be kept open throughout summer; however the construction of the Berg River Dam, discussed in Section 2.2.9, has assisted in providing some flexibility for tunnel maintenance.

2.2.9 Berg River scheme

As shown in **Figure 2.8**, the Berg River Dam, completed in 2006, receives water from its own catchment (Berg River) as well as supplemented water via the supplement scheme located 12 km downstream of the dam at Bien Donné. Upstream of the dam at Wolwekloof, winter flows are normally diverted into the Theewaterskloof Tunnel before reaching the dam (Department of Water Affairs and Forestry, 2004:28). Releases are made into the lower Wemmershoek River before the confluence with the Berg River for upper Berg River irrigators via the supplement scheme pipeline in order to minimise the impacts of these unseasonal flows on the river directly downstream of the dam. In order to reduce spillage, water can be pumped from the dam into the Theewaterskloof Tunnel via the Dasbos Tunnel and thus made available to all Theewaterskloof users as discussed in Section 2.2.8 (Jonker *et al.*, 2012:27).

Low flow environmental releases are made from the dam weekly in arrears based on monthly duration curves required as part of the Ecological Reserve for the dam. In addition, the dam is required to make high flow environmental releases which coincide with natural flood events (Jonker *et al.*, 2012:27).

2.2.10 Local schemes

Individual local authorities and limited areas of Cape Town are also supplied by various minor surface and groundwater schemes. The minor schemes supply a total of 13.8 million m^3/a to Cape Town and 12 million m^3/a to other areas including Paarl, Wellington, Stellenbosch, Piketberg, Saron, Porterville, Tulbagh, Franschhoek and Pniel.

2.3 Future Planning: Western Cape Reconciliation Strategy Study

Demands on the WCWSS are continually increasing as a result of population and economic growth, which prompted DWS in partnership with the City of Cape Town to commission the Western Cape Reconciliation Strategy Study in 2005 in order to reconcile the projected demands on and yield of the system and to propose interventions to meet the future demands within the planning horizon (up to 2030). The Study was completed in June 2007. A brief overview of the Study and the key findings is presented in Sections 2.3.1 to 2.3.5.

2.3.1 Water requirements

High and low future water requirement scenarios, as presented in **Figure 2.9**, were developed using a model created by the City of Cape Town. The high water requirement scenario is based on high economic and population growth resulting in an average growth in water demand of 3 percent per annum, and the low water requirement scenario is based on low economic and population growth resulting in a growth in water demands of 1.4 percent per annum (Department of Water Affairs, 2009a:3).

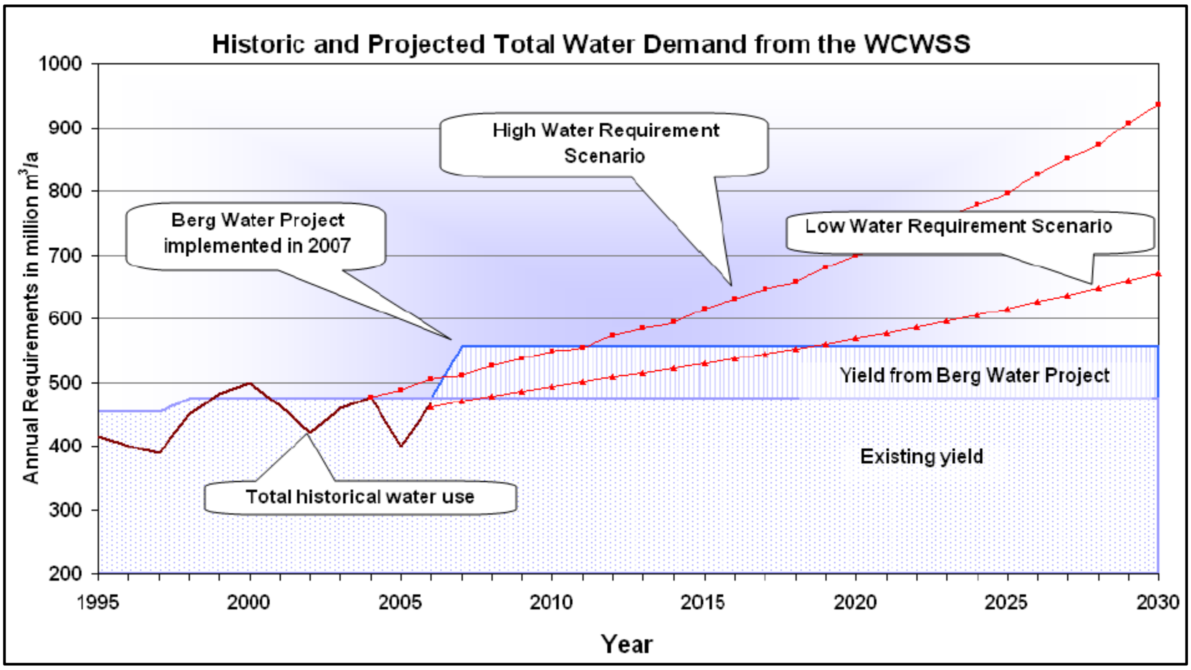

Figure 2.9: Water requirement scenarios for the WCWSS as taken from the Reconciliation Strategy Study (Department of Water Affairs, 2009a:3)

2.3.2 Water conservation and water demand management

The City of Cape Town developed a Water Conservation and Water Demand Management (WC/WDM) Policy and Implementation Strategy in 2001 aimed at reducing water demands and losses. Implementation of the strategy was initially slow, however in 2004 a ten point strategy was developed in partnership with DWS, outlining input and output goals along with an eight year budget. The drought of 2004/05 delayed the implementation of the strategy as focus was shifted to drought measures, however in 2006 the strategy was updated, and this strategy included in the reconciliation study. The eight year strategy targeted a cumulative saving of 44 million m^3 by 2014/15 (Department of Water Affairs, 2009a:8).

2.3.3 Supply side interventions

Various supply side interventions were proposed in order to meet the projected future demands including surface water and groundwater schemes, reuse of water, seawater desalination and the removal of alien vegetation. All interventions were investigated based on financial, socio-economic and environmental considerations and implementation programmes were developed for each (Department of Water Affairs, 2009a:4). A total of 66 interventions were discussed at a multi-stakeholder workshop in August 2005 and of these, 19 were screened out based on failings in terms of potential scheme yields, costs, negative socio-economic impacts and ecological impacts (Department of Water Affairs and Forestry, 2007:C-1).

2.3.4 Reconciliation scenarios

The proposed interventions were grouped into implementation options which allowed for some flexibility to ensure that interventions can be brought forward or pushed out as water requirements change (Kleynhans, English, Botha & Mugumo, 2011:1). The objective of the scenario planning process was not to select a preferred scenario, but rather to identify which interventions should be studied further at a pre-feasibility or feasibility level. A list of 13 interventions was proposed with timelines and responsible organisations assigned to each. The benefits of implementing WC/WDM, the impact of implementation of environmental water requirements (EWRs) and effects of climate change were also considered (Department of Water Affairs, 2009a:5).

2.3.5 Recommendations

The recommendations of the study ranged from implementing the eight year WC/WDM strategy to conducting pre-feasibility and feasibility studies for various shortlisted surface water and groundwater interventions. Included in the list is the implementation of a pilot seawater desalination plant for Cape Town.

2.3.6 Strategy steering committee

One of the recommendations of the Reconciliation Strategy Study was that a Strategy Steering Committee be formed. The objectives of the SSC are to (Department of Water Affairs, 2011a:2):

- Monitor the implementation of the recommendations of the Reconciliation Study;
- Update the Study to ensure that it remains relevant; and
- Ensure that the Study, recommendations and implementation progress are appropriately communicated to all stakeholders.

The committee comprises representatives from the main users and institutions with a stake in the WCWSS. Meetings have been held bi-annually since the formation of the committee in September 2007, with a total of 10 meetings held to date. An Administrative and Technical Support Group was formed to support the SSC and ensure that the recommendations of the SSC are implemented (Department of Water Affairs, 2013a:6).

2.4 Updates of the Reconciliation Strategy

The Reconciliation Strategy was reviewed by the SSC in 2009, 2010, 2011 and 2013 and updated in the form of a status quo / progress report. The findings of the latest progress reports dated November 2011 and October 2013 are discussed in Sections 2.4.1 to 2.4.3.

2.4.1 Water conservation and demand management

The revised 10-year City of Cape Town WC/WDM Strategy and Programme was approved in May 2007, targeting a total saving of 90 million m^3 by 2016/2017. A number of successful measures have been implemented, and it is estimated that 40 million m^3 has been saved from 2007/08 to 2011/2012 as a result of WC/WDM interventions. This is approximately two thirds of the *pro rata* targeted savings in the 2007 Strategy, suggesting that either the growth in water demands is exceeding the high demand scenario developed as part of the 2007 Reconciliation Strategy, or that the targeted water savings are not being met (Department of Water Affairs, 2013a:8).

The WC/WDM strategy was updated during June 2013 and the original strategic goals revised based on updated City of Cape Town growth forecasts. In the 2012/2013 financial year, the City managed to achieve their targets, and other municipalities who are supplied by the WCWSS have WC/WDM measures in place (Department of Water Affairs, 2013a:10).

2.4.2 Supply side interventions

Regarding surface water interventions, feasibility studies for the first phase augmentation of Voëlvlei Dam (Voëlvlei Phase 1) and the Breede-Berg (Michell's Pass) Diversion Scheme have been completed by DWS. Voëlvlei Phase 1 proved to be the most cost-effective option with a URV of R1.52/m^3 and an incremental increase in yield of 23 million m^3/a. The environmental impact assessment for this scheme is currently underway. In addition, the Lourens River Augmentation Scheme Feasibility Study was set for advertisement at the end of 2013, despite concerns surrounding development in the area of the proposed diversion site as well as water quality issues (Department of Water Affairs, 2013a:12).

In terms of other sources, the City of Cape Town has completed the exploratory phase of the feasibility study and pilot project into the development of the Table Mountain Group (TMG) groundwater aquifer and is committed to continuing with the pilot project despite some delays in procurement. A feasibility study into the possibility of seawater desalination as a water source (discussed in more detail in Section 3.1.3.4) was set for completion in December 2013. A similar study concerning treated effluent reuse was advertised in early 2013 and a consultant has been appointed (Department of Water Affairs, 2013a:13).

In addition, various other municipalities who benefit from the WCWSS have proceeded with supply side interventions including the Saldanha Bay seawater desalination project (discussed in more detail under Section 3.1.3.1), construction of the new 8 Mℓ/day Meulwater WTW in Paarl, as well as a full Bulk Water Supply Improvement Study carried out by the Stellenbosch Municipality (Department of Water Affairs, 2013a:14).

2.4.3 Water balance

Figure 2.10 shows the actual and projected water requirements from the WCWSS as presented in the 2013 WCWSS SSC Progress Report. The adjusted total water usage from the WCWSS was 503 million m^3/a in 2013 (Department of Water Affairs, 2013a:21). The updated total integrated system yield at a 98 percent level of supply assurance was estimated at 596 million m^3/a (Department of Water Affairs, 2011a:7).

In order to update the range of possible implementation dates for the proposed interventions, three water requirement scenarios were developed including the base scenario, planning scenario and worst case scenario. The planning scenario was selected as the most likely and comprises the revised integrated system yield with high water requirements assuming that the City of Cape Town WC/WDM strategy is 50 percent successful from 2014 to 2020 with potential effects of climate change ignored.

Under this scenario, the requirement for water would exceed the available supply in 2022, meaning that the Voëlvlei Phase 1, TMG Aquifer, water reuse or seawater desalination options could all be ready, provided that an option is selected by October 2015 at the latest (Department of Water Affairs, 2013a:22). However, if the growth in water requirements is lower than the high water requirement scenarios, then it may be possible to implement one of the options with a longer lead time.

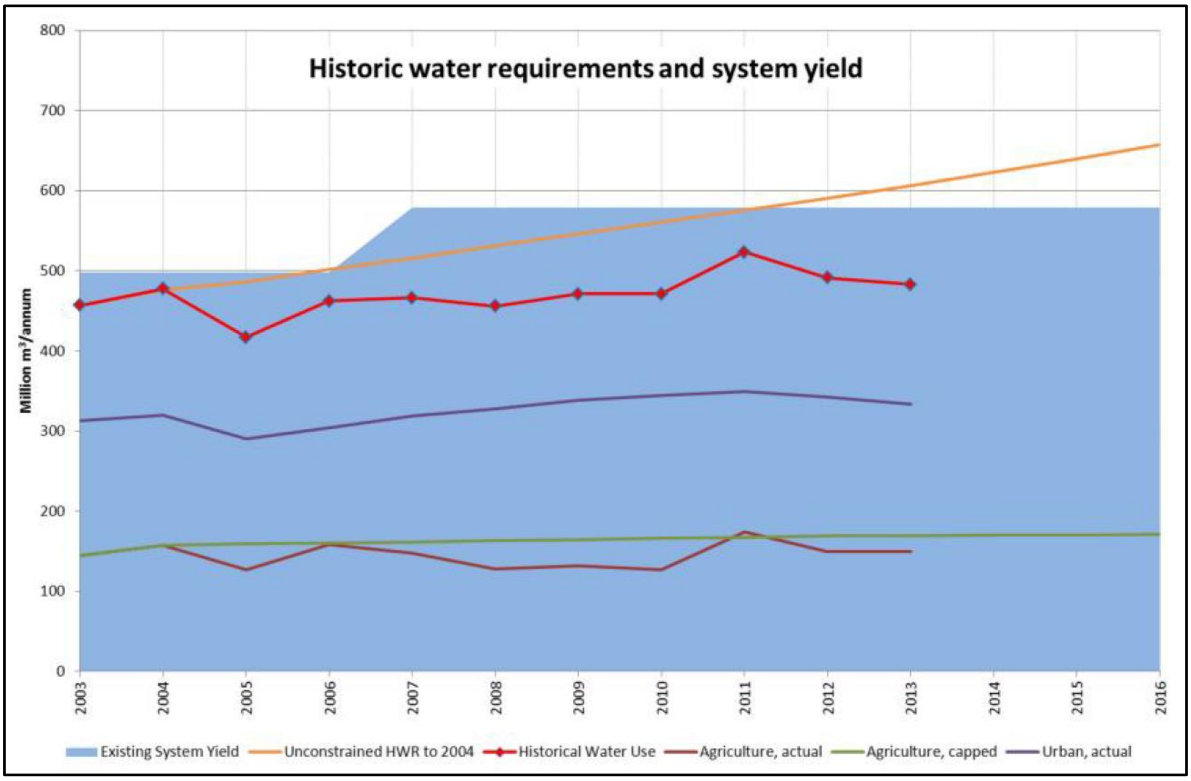

Figure 2.10: Latest total historical and projected water requirements for the WCWSS (Department of Water Affairs, 2013a:21)

2.5 Conclusion

The WCWSS supplies over 500 million m^3/a of water to users throughout the Western Cape. It is a system reliant almost entirely on surface water supply, with a network of dams, pump stations, pipelines and canals that must be operated as an integral unit in order to maximise the available yield. Planning surrounding the WCWSS is done collaboratively, consistently and thoroughly through the various tools and mechanisms discussed throughout Section 2. Consideration of seawater desalination as a potential future additional supply source for the area has been earmarked through these planning processes. However, any seawater desalination feasibility study would have to consider a desalination plant as an integral component of the WCWSS, making a thorough understanding of the operation of the system critical.

3 PLANNING FOR SEAWATER DESALINATION

The focus of Section 3 is on planning for desalination in a South African context, however international literature is also reviewed and some international case studies presented.

3.1 Seawater Desalination in South Africa

In order to understand the context of seawater desalination planning within the WCWSS it is important to understand the status quo of seawater desalination in South Africa and the history of seawater desalination planning at a national and regional level. This section presents an overview of the current operational seawater desalination plants, a summary of desalination references in national and regional planning documents and a discussion of possible future large-scale seawater desalination schemes which are currently being investigated for the major coastal cities.

3.1.1 Operational seawater desalination plants in South Africa

Due to the availability of less costly surface and groundwater resources to meet water demands, seawater desalination has historically not been considered a viable water source in South Africa and there are therefore currently no major seawater desalination plants in the country. **Table 3.1** presents a summary of the existing seawater desalination plants in South Africa which are discussed in more detail in Sections 0 to 3.1.1.6.

Table 3.1: Summary of existing seawater desalination plants in South Africa

	Mossel Bay	Sedgefield	Plettenberg Bay	Bushman's Mouth	Transnet – Saldanha Bay	Lambert's Bay
Capacity	15 Mℓ/day	1.5 Mℓ/day	2 Mℓ/day	1.8 Mℓ/day	3.6 Mℓ/day	1.7 Mℓ/day
Current output	Very low	1.5 Mℓ/day	0 Mℓ/day	1.8 Mℓ/day	2.4 Mℓ/day	1.7 Mℓ/day
Capital Cost	R210 million	R 16 million	R 32 million	Not available	Not available	R 60 million
Funding	PetroSA 32% Treasury 37% Municipality 31%	Not available	Not available	Not available	Transnet	Cederberg Municipality
Commissioned	September 2011	December 2009	December 2010	1982	August 2012	Not available

3.1.1.1 Mossel Bay

The biggest seawater desalination plant in South Africa, shown in **Figure 3.1**, is situated in Mossel Bay, and was commissioned in September 2011. The Southern Cape region was hit by a severe drought, with an estimated return period of 1 in 130 years, resulting in dams reaching critically low levels and severe water restrictions being imposed across the region. Wolwedans Dam, for example, reached a minimum level of 14.5 percent (Mossel Bay Municipality, 2011).

A seawater desalination plant was always planned for Mossel Bay for 2014 in order to meet the future water demands of the Municipality and PetroSA; however this project was brought forward in response to the drought and implemented as an emergency scheme. One advantage for the Municipality of emergency implementation is that part of the project was funded by National Government due to the dire water situation in the area at the time.

Figure 3.1: Mossel Bay desalination plant (Mossel Bay Municipality, 2011)

The plant has a total capacity of 15 Mℓ/day shared between PetroSA and Mossel Bay Municipality; however it has been largely untapped since the end of the drought in late 2011 as it is more expensive to operate than the surface water supply sources. This is discussed in more detail in Section 3.2.1. Despite R210 million in capital expenditure and the high costs associated with maintaining a virtually unused desalination plant, some stakeholders view it as an asset for ensuring drought resilience, system flexibility and capacity to deal with future population and economic growth (Williams, 2013).

3.1.1.2 Sedgefield

The Sedgefield desalination plant is the third largest in South Africa with a capacity of 1.5 Mℓ/day. It was also constructed in response to the severe Southern Cape drought, with water having to be trucked into Sedgefield from George at great cost during the summer months of 2008/09. The plant took only three months to design and construct and was therefore a good emergency solution, providing relief during the peak summer demand period of 2009/2010. There is still discussion as to the best operational philosophy for the plant, in other words whether to use it to supply the base demands all year round or to supply peak demands in the summer months only (Perring, Turner & Erwee, 2010).

3.1.1.3 Plettenberg Bay

Another seawater desalination plant constructed during the Southern Cape drought was a 2 Mℓ/day plant in Plettenberg Bay. Initial investigations found that it would not be possible to abstract seawater via beach wells on Robberg Beach as was originally planned and the design was therefore changed to abstract from the Piesang River estuary via a series of boreholes. Various uncertainties in the environmental process were noted, and as a result it was decided to operate the plant on a pilot basis, monitoring the quality of the estuary, the estuary level and the status of the river mouth (Killick, 2010). Following three months of pilot operation, the plant was shut down in April 2011 after the estuary level was drawn down causing the Piesang River mouth to close. The plant is currently operated intermittently.

3.1.1.4 Ndlambe Municipality

The Bushman’s Mouth desalination plant situated near Kenton-on-Sea was constructed in 1982, extended in 2000 and refurbished in 2010. The plant is owned and operated by the Albany Coast Water Board that has now been merged with Amatola Water Board. It has a capacity of 1.8 Mℓ/day (Creamer Media Reporter, 2010).

A centralised groundwater desalination plant supplies 0.75 Mℓ/day to Cannon Rocks and Boknes Strand. The reverse osmosis (RO) plant removes dissolved salts which are present as a result of seawater intrusion into the groundwater. It was commissioned in May 2009 (Veolia Water Solutions, 2011).

3.1.1.5 Port of Saldanha

In August 2012, Veolia Water Solutions commissioned an RO seawater desalination plant at Transnet's iron ore terminal in Saldanha Bay. The plant has a capacity of 2.4 Mℓ/day, upgradeable to 3.6 Mℓ/day. The water is desalinated and treated to potable standards in order to be utilised for iron ore dust suppression. The plant is now operated by Transnet, after Veolia were contracted to operate and maintain the plant for a period of 12 months (Veolia Water Solutions, 2013a). This is a practice that will likely become commonplace for seawater desalination plants in South Africa as it facilitates a transfer of skills from the desalination plant supplier to the eventual owner/operator.

3.1.1.6 Lambert’s Bay

Growing municipal and agricultural water demands have resulted in a strain on the existing groundwater resources in the Cederberg Municipality. As a result, the municipality turned to seawater desalination as a possible source, appointing Veolia Water Solutions in early 2013 to design and build a 1.7 Mℓ/day seawater desalination plant at Lambert’s Bay.

Raw water is withdrawn via beach wells close to the shore near Muisbosskerm and treated using dual-media pressure filters, RO membranes and chlorination. The plant was set for commissioning in October 2013 but is still not in operation due to funding constraints in completing the sea outfall works (Veolia Water Solutions, 2013b).

3.1.2 Desalination planning in South Africa

As mentioned in Section 3.1.1, the existing seawater desalination plants in South Africa were generally built in response to drought conditions and are fairly small-scale interventions. As water demands have grown in South Africa over the past decade, DWS and other water institutions have begun to consider the possibility of large-scale seawater desalination as a future source of water supply for the major coastal cities. A summary of the information available regarding seawater desalination in national and regional planning documents is discussed in Sections 3.1.2.1 to 3.1.2.6, discussed in chronological order.

3.1.2.1 Berg Water Management Area Internal Strategic Perspective

The Berg Water Management Area Internal Strategic Perspective (Department of Water Affairs and Forestry, 2004:51) lists seawater desalination as the last item in the list of reconciliation interventions which may be considered to meet the long-term water requirements of the area, noting that it is expensive but has “*unlimited potential*”.

The strategy recommends that the City of Cape Town commissions a pilot seawater desalination plant in order to allow development of the necessary skills to evaluate seawater desalination once it is considered on a larger scale. It is further noted that the cost of supplying water at a higher assurance of supply (climate resilient water sources) is higher than for a lower assurance of supply (climate reliant water sources), however a seawater desalination plant could be integrated into the system in such a way as to provide a higher assurance of supply to high priority users and therefore allow the other system components to supply more water to lower priority users.

3.1.2.2 Water for Growth and Development

Water for Growth and Development (Department of Water Affairs and Forestry, 2008:26) indicates that in 2008, desalination (of both seawater and brackish groundwater) contributed only 1 percent of the total water supply in South Africa. DWS anticipates that this will grow to 5 percent by 2025 and 7 percent by 2040, based on high demand scenarios. The report lists the benefits of desalination as proximity to demand, reduced infrastructure costs and water losses, and favourable extension and replacement costs. It is further noted that desalination plants have shorter lead times than large surface water projects (like dams) and implementation can be easily phased.

Department of Water Affairs and Forestry (2008:26) conclude that desalination is, therefore, an efficient solution for prolonged droughts, for coping with climate change and for areas of sudden rapid growth in demand. The report recommends that full feasibility studies be undertaken for the desalination of seawater at all the major coastal cities in South Africa.

3.1.2.3 Reconciliation strategies

The Algoa Reconciliation Strategy (Department of Water Affairs, 2011b:69) indicates that seawater desalination may be the only potential supply source available to meet the long-term needs of the Nelson Mandela Bay Metropolitan Municipality (NMBMM). The strategy recommends that NMBMM consider development of the Swartkops Desalination Plant as an emergency scheme, liaise with the Coega Development Council regarding a seawater desalination scheme at the Coega Industrial Development Zone, monitor seawater quality around potential sites and investigate the feasibility of integrating the proposed seawater desalination schemes into the Algoa System.

The Water Reconciliation Strategy for the KwaZulu-Natal Coastal Metropolitan Areas (Department of Water Affairs, 2009b:34) included seawater desalination as a potential water supply option. Initial investigations concluded that seawater desalination would be the most costly of all interventions for the area; however subsequent calculations showed that seawater desalination is actually a competitive option, thus warranting further investigation. The report indicates that seawater desalination would add flexibility to the system and could delay some of the suggested long-term solutions. Feasibility studies into seawater desalination to supply the Mgeni River System and the North Coast are recommended (Department of Water Affairs, 2009b:19). The pre-feasibility study has been completed and is mentioned in the Second Stage Strategy Report (Department of Water Affairs, 2010a:46) which indicates that seawater desalination is becoming economically competitive when compared to other options.

3.1.2.4 Integrated Water Resource Planning for South Africa Situation Analysis

The Integrated Water Resource Planning for South Africa Situation Analysis (Department of Water Affairs, 2010b:7) indicates that seawater desalination plants can provide fresh water at a "*high but affordable*" cost, noting that seawater desalination has the advantage of short lead times. The authors indicate that seawater desalination will be available to meet coastal demands but that the costs of pumping inland will likely be too high, even for industry. It is further noted that South Africa needs to develop skills in the field of seawater desalination.

3.1.2.5 National Water Resource Strategy

The latest version of the National Water Resource Strategy (Department of Water Affairs, 2013b:31) indicates that DWS will give particular attention to the possibility of seawater desalination as a water supply source for coastal towns. Cognisance is taken of the fact that energy requirements are a major constraint for seawater desalination and any developments will therefore take place in close liaison with the Department of Energy and Eskom, taking available energy sources into consideration. Reference is made to the existing small scale seawater desalination in South Africa which provides a cost-effective solution in remote coastal towns.

3.1.2.6 National Desalination Strategy

A National Desalination Strategy is included as Annexure C of the National Water Resource Strategy (Department of Water Affairs, 2013b). An overview of the relevant conclusions of the strategy is summarised as follows:

1. Desalination and water reuse have a critical role to play in the long-term security of South Africa’s future water supply due to declining water quality, increased rainfall variability and an international emphasis on reducing effluent discharge (Department of Water Affairs, 2013b:C3).
2. South Africa currently lacks expertise in the implementation of large-scale seawater desalination projects. Local research and development into desalination technologies should continue and desalination centres of excellence should be established, as in countries such as Australia (Department of Water Affairs, 2013b:C6).
3. Financing of large-scale desalination projects will require huge capital investment. As with other large water supply projects in South Africa, a design and build model could be followed. Alternatively, as with many Australian plants, a design, build and operate approach could be adopted (Department of Water Affairs, 2013b:C6). Some issues with this type of financing model are discussed in Section 3.4.4.
4. DWS will develop further guidelines on the implementation of desalination projects (Department of Water Affairs, 2013b:C9). In terms of such guidelines, DWS and the Water Research Commission funded the drafting of A Desalination Guide for South African Municipal Engineers (Du Plessis, Burger, Swartz & Musee, 2006). The guide provides an overview of desalination technologies along with guidelines for process selection, costing, environmental considerations and operation and maintenance. This guideline is referenced in the cost estimation discussions in Section 3.3.1

3.1.3 Planned seawater desalination plants in South Africa

Seawater desalination is currently being investigated at a feasibility level in a number of areas of South Africa including Saldanha Bay, Cape Town, Port Elizabeth and Durban as discussed in Sections 3.1.3.1 to 3.1.3.5.

3.1.3.1 Saldanha Bay

As part of a feasibility assessment process which commenced in 2007, various water supply options were investigated for the West Coast District Municipality. It was concluded that seawater desalination would be the most cost-effective option, and the decision was taken to proceed with a 25.5 Mℓ/day seawater desalination plant at Danger Bay. Environmental approval was obtained in August 2013, however implementation of the project is hinging on the availability of funding, given that the estimated capital cost of R 500 million is way beyond the reach of the municipality. The seawater desalination plant will be phased in three 8.5 Mℓ/day lanes, with the associated bulk infrastructure constructed for the ultimate capacity of 25.5 Mℓ/day (Department of Water Affairs, 2013a:13).

3.1.3.2 Port Elizabeth

All of the catchment areas of the dams which supply NMBMM were hit by severe drought from 2008 to early 2011. Various emergency drought relief options were identified for the area as part of an emergency drought action plan, with water restrictions being imposed in October 2009 and emergency restrictions following in February 2010. The construction of an emergency seawater desalination plant was approved as a suitable drought relief option due to its short lead time. A site was identified at Swartkops Estuary providing an ideal emergency solution as it would allow for the utilisation of existing seawater intake and outlet infrastructure. The preliminary design for the associated infrastructure was completed and tender documents were prepared for the design and build contract estimated at R 450 million. In February 2011 it was announced that NMBMM would only receive a portion of the funding required to implement all of the planned emergency drought relief schemes. As a result, they were forced to implement some of the less expensive options, resulting in the seawater desalination plant being put on hold (De Kock, 2011).

The drought ended in 2011, meaning that implementation of emergency schemes was no longer required. The construction of a seawater desalination plant for the city is still an option, however as part of the updated 2011 Algoa Reconciliation Strategy Study it has been proposed that a new feasibility investigation be done primarily to identify a site which would be more suitable from a long-term planning perspective (Department of Water Affairs, 2011b:69).

The latest SSC Progress Report (s.n., 2013:4) indicates that investigations into potential seawater desalination technologies are complete and potential sites have been identified. Feasibility investigations and conceptual design of the selected alternative will be completed by June 2015.

3.1.3.3 Durban

Umgeni Water issued a tender in 2011 for a detailed feasibility study of seawater desalination for the Durban area, and appointed a consultant to undertake the study which is set for completion in June 2014. There are currently two proposed sites, one near the Lovu River on the South Coast and one neat the Mdloti River on the North Coast. Either one or both of these will be developed with a proposed capacity of 150 Mℓ/day each (Singh, 2013). In terms of the long-term planning for the Durban area, seawater desalination is being considered along with various other surface water augmentation options which will be compared once all feasibility studies are complete (West, 2011).

3.1.3.4 Cape Town

The option of implementing a seawater desalination plant to supply water to Cape Town was first assessed at a pre-feasibility level in 2003 as part of the Bulk Water Supply Study. As part of the study, potential locations for a seawater desalination plant were identified and the process design, costs and integration into the existing water supply system were assessed.

On-going strategic water resource planning undertaken by DWS in conjunction with major stakeholders confirmed that seawater desalination is a potential water source for Cape Town. As a result, a Feasibility Study was commissioned in 2011 with a consultant appointed in early 2012. The study was set for completion at the end of 2012 but was postponed until December 2013 (Department of Water Affairs, 2013a:12). The findings of the study have not yet been released.

As part of the Feasibility Study, potential sites, pipeline routes, conceptual infrastructure designs, treatment processes and environmental considerations (at a screening level) would be recommended. This would bring the planning of a seawater desalination scheme for Cape Town to the point where an environmental impact assessment and preliminary design could commence. The anticipated capacity based on the feasibility study would be 150 Mℓ/day, upgradeable to 450 Mℓ/day, with a lead time of approximately eight years (Department of Water Affairs, 2013a:12).

3.1.3.5 Zandkopsdrift

In February 2013, a feasibility study concerning a seawater desalination plant at Abraham Villiers Bay in the Northern Cape was commissioned by Sedex Desalination, a subsidiary of Frontier Rare Earths. The proposed plant would have a capacity of 7 Mℓ/day and would supply water to the Zandkopsdrift mine which is currently under development. The study which was set for completion at the end of 2013, would address process configuration options, lifecycle costing and possible social and environmental impact (Esterhuizen, 2013).

3.1.4 Outlook for further seawater desalination development in South Africa

As pressure on surface water resources in South Africa continues to grow as a result of population growth, increased level of service, economic growth, climate change and declining water quality, seawater desalination will certainly gain traction as a potential source for meeting future demands. This is reiterated throughout the various planning documents discussed in Section 3.1.2.

A recent report published by TechSciResearch, a global research-based management consulting firm, predicts that the desalination market in South Africa will grow by, on average, 28 percent per annum for the next five years (2012 to 2017) and that the number of desalination plants in operation will triple over the same period (Slabbert, 2012). The challenge will be developing the local skills in planning, design, implementation and operation of desalination plants to meet this growth. It should be noted that the Lambert's Bay plant is the only new seawater desalination plant that has been constructed since 2012, hence the projected growth has not yet been realised.

3.2 Conjunctive Use of Desalination

Globally, seawater desalination has traditionally only been practiced in areas which have little or no surface water resources or in areas where the high costs of desalinated water for domestic use are considered acceptable in terms of the overall economy of the country, such as the oil-rich nations of the Middle East. This has begun to change as large coastal cities begin to embrace seawater desalination as a supply source in the face of droughts, climate change and limited surface water resources. This section provides a brief review of the available literature on the integrated use of seawater desalination with other water supply sources, in South Africa and internationally.

3.2.1 South African research

Despite the various seawater desalination feasibility studies which are currently underway, as discussed in Section 3.1.3.1 to 3.1.3.5, as well as the ambitious outlook for seawater desalination development in South Africa, as discussed in Section 3.1.4, there has been very little research in South Africa to date into the conjunctive use of seawater desalination with surface water supply sources and how to integrate seawater desalination plants into existing water supply systems.

The only South African authors who have presented any material on the topic to date are Mallory, Ballim, Pashkin and Ntuli (2013) who undertook a study into the optimal operation of the seawater desalination plant, and PetroSA's 5 Mℓ/day wastewater reclamation plant, in Mossel Bay. As mentioned in Section 0, the Mossel Bay desalination plant was built as an emergency project in response to a terrible drought in the area, but since the end of the drought it has remained largely untapped due to the high costs in operating the plant.

Mossel Bay has historically been reliant on surface water supply from a number of dams, and is required to supply water for domestic use, industrial use (PetroSA) and environmental use at assurances of supply of 98 percent, 99.5 percent and 100 percent respectively. The municipality is currently reluctant to accept the high costs associated with operating the desalination plant and, therefore, prefers to make maximum use of their cheaper surface water sources. This will ultimately mean reducing the assurance of supply to their users but keeping water tariffs low. PetroSA would also prefer to make use of the cheaper sources first, but would not be willing to accept a lower assurance of supply and hence would prefer to pay the costs associated with reclaimed and desalinated water. Mallory *et al.* (2013) attribute this to the "*monetary value that users put on water*", a concept which is critical to this research.

Mallory *et al.* (2013) made use of the Water Resources Modelling Platform (WReMP), a node- and channel-based model which uses a rule-based simulation approach, essentially mimicking the decision-making processes of the system operator. The results showed that if the desalination plant is never used, by 2030 the assurance of supply for domestic users would drop to 88 percent and for PetroSA would drop to 91 percent. It is unlikely that users would accept this risk of failure, and the desalination plant will therefore eventually have to be used. Mallory *et al.* (2013) further showed that if the optimum objective is to maintain the current assurance of supply for all users at the lowest possible cost (in other words, using the desalination plant as seldom as possible), the desalination plant would have to be operated 3 percent of the time in 2012, increasing to 22 percent by 2030.

The work of Mallory *et al.* (2013) is valuable in that it begins to tackle some of the key questions associated with the value of water to different users, and setting of operating rules for a seawater desalination plant. However, there are some shortcomings. Firstly, the authors assume that the capital costs of the desalination plant are 'sunk' costs and thus any loan repayments are ignored. As will be shown in Section 3.4.4, it is often impossible to separate the capital expenditure from the decision to operate a seawater desalination plant due to the type of project funding mechanisms used. Secondly, the authors assumed that the plant can be switched on or off at any stage which in reality could prove costly and technically challenging. Thirdly, the assumption that the seawater desalination plant is significantly more expensive to operate than the other supply sources is based only on the perception of the municipality and no actual cost figures are provided.

Despite the limitations in terms of research regarding conjunctive use of seawater desalination, some relevant local research has been carried out into conjunctive evaluation of the costs and benefits of inter-basin water transfer schemes. Van Niekerk and Du Plessis (2013b:543) evaluated four South African and two international inter-basin water transfer schemes and found that, in all cases, the so-called *incremental approach* was followed in evaluating the viability of the schemes. The incremental approach assumes that the inter-basin transfer would have to supply the difference between the projected future water demands and the current system yield. In comparing projected water transfers using the incremental approach with actual transfers for the Usutu-Vaal Government Water Scheme, it was found that the original projections vastly exceeded the actual transfers, mainly because the incremental approach ignores the stochastic nature of the conditions of the receiving system.

To address these concerns, Van Niekerk and Du Plessis (2013b:543) propose a *comprehensive approach* in which the receiving water basin and the transfer scheme are modelled as a whole in an integrated setup in the WRPM. This is discussed in more detail in Section 4.4. The approach involves a stochastic analysis based on real-time operating rules, giving the actual anticipated total water transfer over the project lifecycle.

Taking this concept further, Van Niekerk and Du Plessis (2013a:551) postulate that one of the shortcomings of the typical URV approach for evaluating water supply options, as discussed in Section 2.1.4, is that it does not account for the stochastic variability of the actual water transferred, which affects both the lifecycle costs (which are included in the numerator of the URV Equation 2.2) and the water supplied (which is the denominator of Equation 2.2). Van Niekerk and Du Plessis (2013a:552) suggest that the URV for an inter-basin transfer scheme should be calculated by using the present value of the quantity of water which is incrementally assured as the denominator of Equation 2.2, and calculating lifecycle costs using Equation 3.1, where the variable operating costs are based on a stochastic hydrological analysis. Further adaptation of the work of Van Niekerk and Du Plessis (2013a and 2013b) to suit seawater desalination projects is provided in Section 5.2.3.

$$\begin{aligned} PV_{lifecycle\ cost} = {} & PV_{capital\ cost} + PV_{maintenance\ cost} + PV_{fixed\ operating\ cost} \\ & + Expected\ PV_{variable\ operating\ cost} \end{aligned} \tag{3.1}$$

3.2.2 International perspective

Mawer and Burley (1968) were the first to recognise that as water demands increase and major cities begin to consider seawater desalination as a potential supply source, the assessment of how to best combine seawater desalination plants with existing conventional water sources will be of utmost importance. Mawer and Burley realised that rather than operating a seawater desalination plant continuously to supply a base load, it could be operated intermittently to supply only during dryer periods when there is a low risk of the dams filling or spilling, hence saving on operating costs. Initial application of this principal to two case studies showed that for the same incremental yield of desalination, application of defined desalination operating rules could reduce the overall costs by up to 50 percent. Their approach is quite complex, and essentially involves calculating costs per unit of incremental yield based on a variety of desalination plant capacities, plant factors and operating rules to find the optimum solution.

Mawer and Wyatt (1973) applied these principals of conjunctive seawater desalination use to the islands of Cyprus and Jersey, both of which have complex existing surface water supply systems. Results from the Cyprus case study revealed a reduction in the unit cost per incremental yield for seawater desalination of 10 percent when using the desalination plant conjunctively rather than as a stand-alone plant meeting base demands. In the Jersey case study, the authors developed a set of operating rules for integrating the existing seawater desalination plant into the water supply system by performing rainfall-runoff modelling and long-term stochastic analyses. Mawer and Wyatt concluded that it is important to consider a seawater desalination plant as an integral component of the overall water supply system rather than as a standalone supply source.

Buros (1976) indicates that the high costs and complex operations associated with seawater desalination necessitate careful water resource planning and management. Buros notes that "*although desalinized seawater is physically the same as water from other sources, its cost factor singles it out as warranting special water management techniques*". Some of the techniques described for reducing the overall water cost when using seawater desalination are:

- Implementing multiple water distribution systems where high quality potable water is used only for potable applications.
- Reclamation of wastewater for non-potable use and artificial groundwater recharge.
- Water conservation and demand management through installation of water-conserving hardware and price control.

- Using surplus energy to desalinate seawater and then storing the treated water in underground formations for later use.

Like many other authors, Qashu (1994) emphasises that decisions regarding water supply alternatives to meet future demands must consider wastewater management and desalination as integral parts of the water resource system. He proposes that non-conventional water resources require non-conventional planning processes. Qashu states that when comparing the economics of seawater desalination with other supply sources, the cost of uncertainties in freshwater supplies, including both random climatic events and cyclic patterns, must be taken into consideration. The aim should be to reduce the frequency of requiring emergency water supply interventions, as these are generally the most costly form of intervention to implement, and often the least sustainable in the long-term.

In a review of desalination in the 21st century and its role in achieving the United Nations Millennium Development Goals, Schiffler (2004) notes that making optimal use of seawater desalination is subject to much wider water sector related conditions. Weak water utilities and planning, unreasonably low water tariffs and high water losses which are prevalent in many countries, including certain regions of South Africa, can easily render seawater desalination unfeasible. He suggests focusing on reducing non-revenue water, integrated water resources management, and capacity building in seawater desalination before implementing seawater desalination projects. Schiffler further remarks that seawater desalination should be the "*last resort*" and should only be considered after careful analysis of less costly alternatives.

Liu, Gikas and Papageorgiou (2010) developed a mixed-integer linear programming model to determine the optimal placement and capacity of seawater desalination plants, water reclamation plants and wastewater treatment works for a water supply system in an arid area with limited freshwater supplies. Their objective function was to minimise the annualised total cost, including both capital and operating costs of the entire water supply system. Liu *et al.* (2010) successfully applied this methodology to the water supply of Syros, an island in the Aegean Sea in Greece. Water demands, potential plant locations, reservoir locations, pipe options and pump options were inputted and an optimal solution was found.

3.3 Seawater Desalination Cost Estimation

As is highlighted in the literature discussed in Section 3.2, a comprehensive evaluation of conjunctive seawater desalination requires an assessment of two main components, water supply and costs. For the water supply component, more general water resource modelling tools can be applied, as discussed in detail in Section 4. For the cost component, however, a more seawater desalination-specific approach is required.

This section presents a discussion of first-order seawater desalination costing techniques available in literature, both in South Africa and abroad. Note that information related to RO seawater desalination only is presented, firstly because RO is by far the most commonly used technology worldwide (currently 60 percent of the world's desalination capacity) and secondly because it is the most likely technology to be adopted for the Cape Town seawater desalination plant which is the focus of this research.

The following exchange rates were used in the costing analyses and comparison of known costs as presented in this section:

- 1 Australian Dollar (AUS$) is equivalent to R9.80
- 1 United States Dollar (US$) is equivalent to R10.70
- 1 Euro (€) is equivalent to R14.70

These rates are based on the average exchange rate from 1 January to 30 June 2014 (South African Reserve Bank, 2014).

3.3.1 South African approach to seawater desalination costing

Du Plessis *et al.* (2006) present a step by step guide to determining first-order capital and operating costs for desalination plants of different sizes in South Africa. The process is outlined as follows:

Step 1: Determine the desalination plant capacity requirements

The daily plant capacity would be based on an analysis of the receiving community's projected future water requirements taking into account population growth, planned developments and increased water requirements that result from an improved level of service. The hourly production rate is then calculated using Equation 3.2.

$$Q_h = \frac{Q_d}{24\alpha} \tag{3.2}$$

Where:

Q_h	Hourly production rate	[m^3/h]
Q_d	Daily production rate	[m^3/d]
α	Fraction of day for which plant operates at full capacity (typically 0.9 to 0.95)	[-]

Step 2: Identify the available saline water quality and recovery

The volumetric fraction of feed water which is converted to desalinated product water after passing through the membranes is called the water recovery, calculated as per Equation 3.3. The remaining fraction is discharged from the plant as concentrated brine. For seawater desalination, R_f is typically between 0.3 and 0.5, with a value of 0.4 being the norm.

$$R_f = \frac{Q_h}{Q_f} \tag{3.3}$$

Where:

R_f	Water recovery factor	[-]
Q_h	Hourly production rate	[m^3/h]
Q_f	Hourly feed flow rate	[m^3/h]

Step 3: Estimate total membrane area and required feed pressure

Du Plessis *et al.* (2006) postulate that for first-order costing, desalination capital costs can be estimated as a function of the installed membrane area, since the membrane area will most likely dictate the mechanical and civil footprint of the plant. The required membrane area can be estimated based on Equation 3.4. For seawater, the flux is typically between 9 and 18 ℓ/hm^2, with a value of 10 ℓ/hm^2 used for cold west coast water. For the second pass, a much higher flux can be used of up to 30 ℓ/hm^2.

$$A_{mem} = \frac{1000Q_h}{\varphi_1} + z\frac{1000Q_h}{\varphi_2} \tag{3.4}$$

Where:

A_{mem}	Total required membrane area	[m^2]
z	Factor of 1 or 0 for a two pass or one pass system respectively	[-]

φ_1	Average flux for first pass	[ℓ/hm^2]
φ_2	Average flux for second pass	[ℓ/hm^2]

The energy costs for a desalination plant are essentially based on the feed water pressure and the flow rate. The feed water pressure at a specific feed water temperature can be estimated using Equation 3.5 and should generally not exceed 80 bar.

$$P_{F(T)} \approx \frac{0.00076TDS_F}{1-R_f} + \left(\frac{\varphi}{\varepsilon} + 5\right) 1.034^{25-T} \tag{3.5}$$

Where:

$P_{F(T)}$	Feed water pressure at temperature T	[bar]
TDS_F	Total Dissolved Salts in feed water	[mg/ℓ]
ε	Flux per driving pressure	[ℓ/m^{2}hbar]
T	Feed water temperature	[°C]

The flux per driving pressure is membrane-specific but is typically between 1 and 5 ℓ/m^{2}hbar with a value of 1 ℓ/m^{2}hbar applicable for seawater. The TDS for seawater can be assumed to be 36 000 mg/ℓ with a temperature in the order of 10 to 11°C for west coast seawater.

If a two stage system is used, the TDS of the second stage can be assumed to be 1 percent of the first stage (360 mg/ℓ) with a recovery of 85 to 90 percent. The flux per driving pressure will be at the higher end of the range, typically 4.5 ℓ/m^{2}hbar.

Step 4: Determine the pre-treatment and post-treatment requirements

The pre-treatment costs can be estimated as a function of the incoming feed flow rate which can be calculated using Equation 3.6. β is typically between 3 and 15 percent depending on the pre-treatment process.

$$Q_{h(in)} = \frac{Q_h}{R_f(1-\beta)} \tag{3.6}$$

Where:

$Q_{h(in)}$	Feed capacity of pre-treatment plant	[m^3/h]
β	Fraction of feed water lost in pre-treatment	[mg/ℓ]

For post-treatment, the flow rate is approximately equal to the hourly production rate (Q_h).

Step 5: Determine the plant energy consumption

The energy consumption of a desalination plant can be estimated based on the feed pressure and the recovery using Equation 3.7. For seawater desalination plants which are typically energy intensive, energy recovery units are installed in order to recover energy from the high pressure concentrate stream. The pump efficiency will typically be about 75 percent and the energy recovery unit efficiency between 85 and 90 percent. The value for s is 1 if an energy recovery unit is used and 0 if no energy recovery unit is used.

$$E_{desal} \approx \frac{Q_h P_{F(T)}}{36R\eta_p} - s\frac{Q_h\left(P_{F(T)} - 5\right)\left(1 - R_f\right)}{36R\eta_r} \tag{3.7}$$

Where:

E_{desal}	Energy consumption of desalination plant (per pass)	[kW]
η_p	Efficiency of pump	[-]
η_r	Efficiency of energy recovery unit (if using)	[-]

In addition to the desalination energy requirements, energy is also required to feed the pre-treatment system and post-treatment system as well as for backwash filtration systems. These additional requirements can be estimated based on Equation 3.8. P_{in} is typically 2 to 6 bar depending on the pre-treatment and intake configuration.

$$E_{other} \approx \frac{Q_{h(in)} P_{in}}{36\eta} \tag{3.8}$$

Where:

E_{other}	Energy consumption of pre-treatment, post-treatment and backwash systems	[kW]
P_{in}	Sum of feed pressures for intake and pre-treatment systems	[bar]

Step 6: Discharge of residuals

For all desalination plants, the highly concentrated brine stream must be disposed of appropriately. For a seawater desalination plant this is normally done via a brine outfall pipeline into the sea. The total volume of effluent from the plant is calculated using Equation 3.9.

$$Q_{eff} \approx Q_{h(in)} - Q_h \tag{3.9}$$

Where:

Q_{eff}	Brine flow rate	[m^3/h]

Step 7: Estimate capital costs

The capital cost for the desalination portion of the plant can be estimated as a function of the membrane area based on the series of curves provided in Du Plessis *et al.* (2006). The total capital cost includes civil, electrical and mechanical costs. **Figure 3.2** presents the capital cost curve for larger desalination plants. For two pass systems, the total installed membrane area should be used for determining the capital costs.

For the pre-treatment costs, Du Plessis *et al.* (2006) provide capital costs for a typical single stage fine media sand filtration system as a function of the feed flow rate as presented in **Figure 3.3**. If a different type of pre-treatment system is used, factors must be applied to these costs of 0.7 for coarse media filtration at high down-flow rates, 1.5 for a membrane-based ultra-filtration system or 1.7 for a complete lime softening system.

In addition, for seawater desalination plants, Du Plessis *et al.* (2006) indicate that the combined cost of the intake, outlet and post-treatment system is typically 15 to 30 percent of the capital cost of the desalination plant and the pre-treatment system.

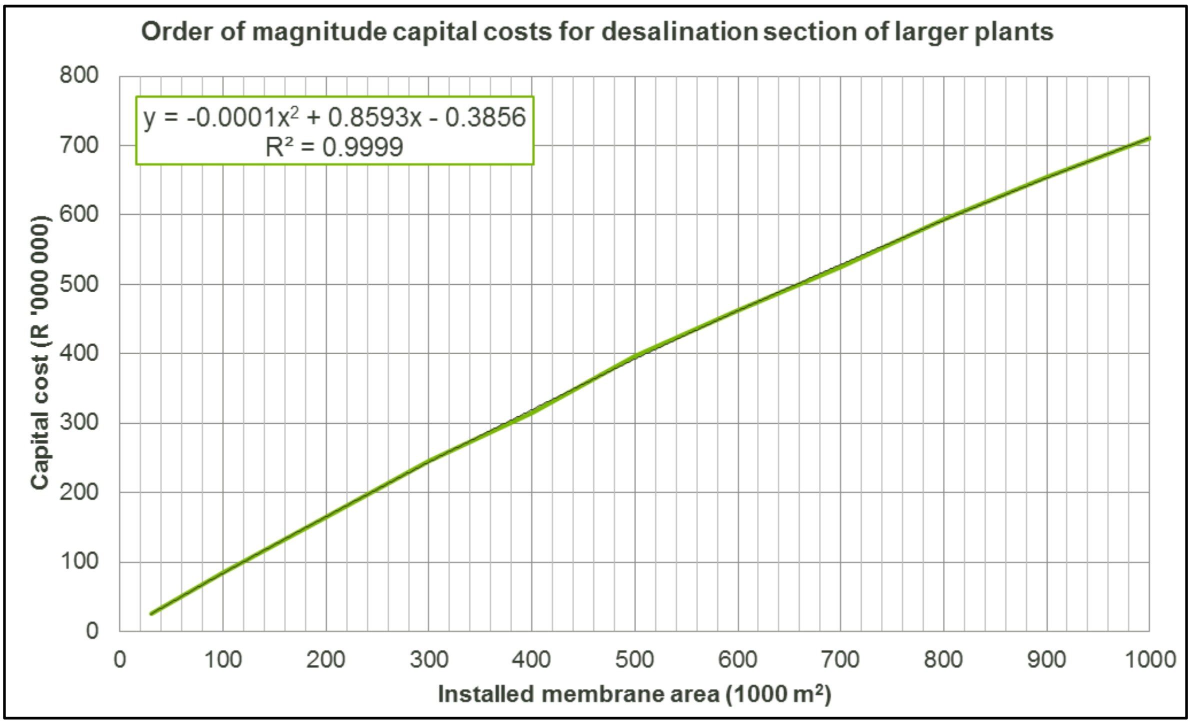

Figure 3.2: Order of magnitude capital costs for desalination section of larger plants based on Du Plessis *et al.* (2006)

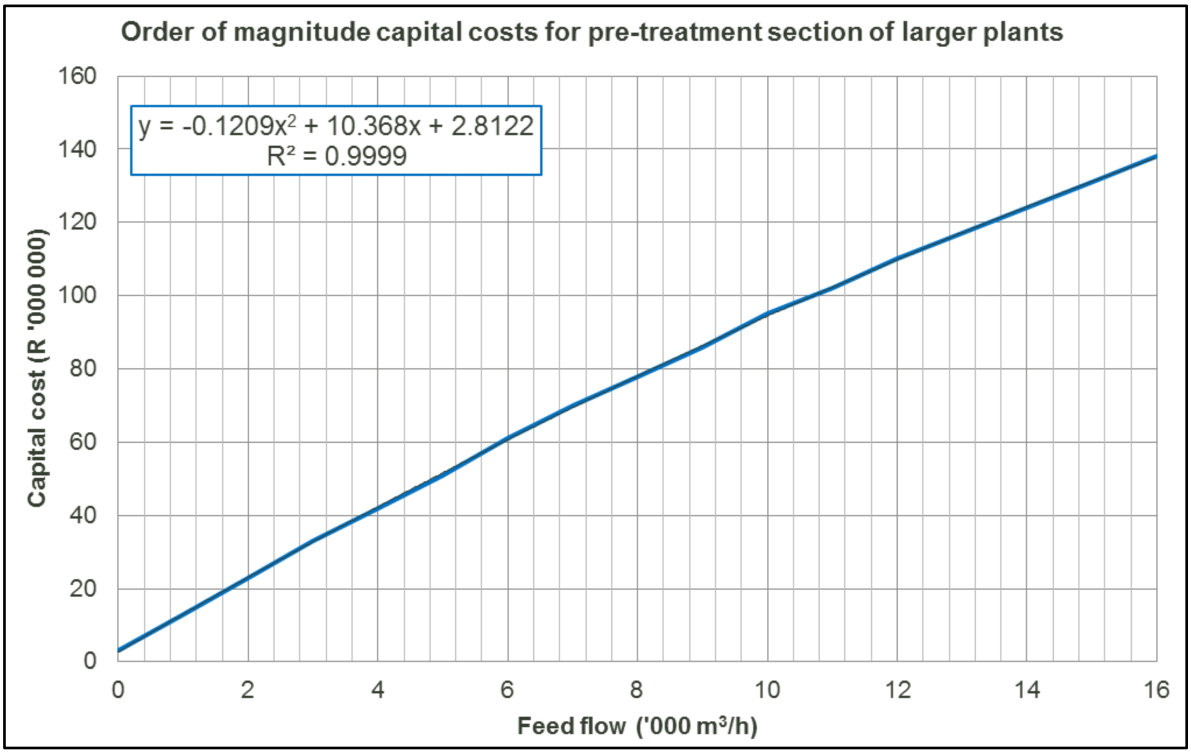

Figure 3.3: **Order of magnitude capital costs for pre-treatment section of larger plants based on Du Plessis *et al.* (2006)**

Step 8: Estimate unit costs

The specific cost of capital redemption can be calculated by dividing the monthly capital cost repayments by the monthly production rate using Equation 3.10.

$$M_{cap} = \frac{\dfrac{Cap\ i\ (1+i)^n}{(1+i)^n - 1}}{30Q_d} \tag{3.10}$$

Where:

M_{cap}	Specific cost of capital redemption	[R/m^3]
Cap	Total estimated capital cost of desalination plant	[R]
i	Interest rate	[-]
n	Period of loan	[months]

Specific energy costs can be calculated using Equation 3.11.

$$M_{energy} = D\frac{E_{desal} + E_{other}}{Q_h} \tag{3.11}$$

Where:

M_{energy}	Specific energy cost	[R/m^3]

D	Electricity tariff	[R/kWh]

According to Du Plessis *et al.* (2006) seawater RO membranes cost approximately 16.5 US$ (R 177) per square metre and must be replaced every three to six years. Based on this assumption the specific cost of membrane replacement can be estimated using Equation 3.12.

$$M_{mem} = \frac{Cost_m A_{mem}}{YQ_d 365} \tag{3.12}$$

Where:

M_{mem}	Specific cost of membrane replacement	[R/m^3]
$Cost_m$	Membrane cost	[R/m^2]
Y	Frequency of membrane replacement	[years]

Chemical costs depend on the level of pre-treatment required, with the requirements typically being more onerous for seawater desalination. Du Plessis *et al.* (2006) suggest a typical value for the specific chemical cost (M_{chem}) of seawater RO plants of R 0.50/m^3, which includes chlorination, acid dosing, coagulant dosing, dechlorination, anti-scalant dosing and post-treatment.

Lastly, labour and maintenance costs should be added which can be estimated as a percentage of the capital cost using Equation 3.13. The percentage allowance for maintenance is typically about 5 percent for large desalination plants.

$$M_{maint} = \frac{uCap}{365Q_d} \tag{3.13}$$

Where:

M_{maint}	Specific cost of maintenance	[R/m^3]
u	Percentage allowance for maintenance	[-]

The total specific unit cost for desalinated water is thus the sum of all of these costs, as given in Equation 3.14.

$$M_{tot} = M_{cap} + M_{energy} + M_{mem} + M_{chem} + M_{maint} \tag{3.14}$$

Where:

M_{tot}	Total specific present cost	[R/m^3]

Example

An example of the application of this methodology to the proposed Cape Town seawater desalination plant is provided in **Appendix A**, based on a capacity of 100 Mℓ/day and a two pass system, situated on the west coast. All costs were escalated from 2006 to 2014 at 6 percent per annum based on the current South African inflation rate. Values in green are inputs or assumptions and those in black are calculated.

Based on these calculations, a first-order estimate of the capital cost of a 100 Mℓ/day seawater desalination plant would be R 1 054 million. The total specific cost including capital redemption and all operational and maintenance costs would be R 16.44 / m^3.

3.3.2 International approach to seawater desalination costing

Karagiannis and Soldatos (2008) studied almost 100 different seawater desalination plants and present a summary of first-order seawater desalination costs derived from different sources in literature for different desalination technologies. **Table 3.2** presents a summary of the costs for RO seawater desalination plants of different capacities as quoted in €/m^3 and converted to R/m^3. Note that these are 2007 values. Escalated to 2014, for a 100 Mℓ/day seawater desalination plant, the specific cost would therefore be approximately R9.84 / m^3.

Table 3.2: Cost of desalinated water for different plant capacities (Karagiannis & Soldatos, 2008)

Capacity of plant (Mℓ/day)	Cost (€/m^3)	Cost (R/m^3)
<0.1	1.20 to 15.00	17.64 to 220.5
0.25 to 1	1.00 to 3.14	14.70 to 46.16
1 to 4.8	0.56 to 1.38	8.23 to 20.29
15 to 60	0.38 to 1.30	5.59 to 19.11
100 to 320	0.36 to 0.53	5.29 to 7.79

Park, Park, Mane, Hyung, Gandhi, Kim and Kim (2010) present a novel stochastic approach to desalination cost determination which takes into account the potential for variation in parameters which are typically assumed to be fixed in deterministic cost methods. The authors present the desalination costs as the sum of capital costs, pre-treatment costs, high pressure pump costs, pressure exchange costs, RO equipment costs, construction costs and other items. Formulae are provided for determining the costs of each of these components. Although the application of stochastic analysis is an important aspect of this research, it was felt that this method would be too complex for the purposes of calculating first-order costs and the detailed cost formulas are, therefore, not repeated here.

Frioui and Oumeddour (2008) present a semi-empirical method for calculating the capital and production costs for various desalination processes including capital, energy, operation, maintenance, and membrane replacement costs. The authors provide a typical product cost for RO seawater desalination plants of 1.81 US\$/m^3 or R 19.37/m^3. According to their analysis, 12 percent of this is the capital redemption cost, 3 percent is the energy cost and 34 percent is the chemical cost. It should be noted that their analysis appears to be based on a typical 1 Mℓ/day plant and therefore may not be applicable to large-scale seawater desalination.

Atikol and Aybar (2005) present a very similar methodology to Du Plessis *et al.* (2006) in which the total cost is calculated based on the capital, electricity, maintenance, chemical, membrane replacement, pre-treatment and manpower costs, however no method is provided for estimating the capital costs. Again, the application of the methodology was for a small 1 Mℓ/day plant in Cyprus, and may not be relevant for large-scale desalination. However, the authors suggest the following regarding some of the other cost components, providing a useful comparison with the values in Du Plessis *et al.* (2006):

- Maintenance costs are approximately 2 percent of the capital cost
- Chemical costs are between 0.025 and 0.035 US\$/m^3 (R0.27 and R0.38 /m^3)
- Annual membrane replacement costs are 5 to 20 percent of the membrane purchase costs

One of the larger desalination cost studies was undertaken by Wittholz, O'Neill, Colby and Lewis (2008) who collated a database of 331 plants, classified them according to their process capacity and size, and converted all costs to present day US\$. The well-known capacity-cost correlation equation, as presented in Equation 3.15, was then applied where the power factor *b* in the equation accounts for the economy of scale associated with any capital infrastructure project. The authors suggest that for desalination plants, a value of 0.8 is applicable.

$$\frac{Cap_1}{Cap_2} = \left(\frac{Q_{\mathrm{d1}}}{Q_{\mathrm{d2}}}\right)^b \tag{3.15}$$

Where:

b	Power law factor	[-]

Based on their analysis of the cost database, Wittholz *et al.* (2008) derived the equation given in Equation 3.16 for estimating capital costs for RO seawater desalination plants. The equation has an R^2 value of 0.907. Attempts by Wittholz *et al.* (2008) at deriving a similar relationship for estimating production costs were not successful.

$$\ln(Cap) = 0.81\ln(Q_d) + 4.07 \tag{3.16}$$

Table 3.3 presents a summary of the capital costs and unit production costs for RO seawater desalination plants derived by Wittholz *et al.* (2008). Based on the values in the table, a 100 Mℓ/day seawater desalination plant would cost R 1 313 million and would have a unit production cost of R 7.01/m^3 based on the 2008 values. Escalated to 2014, the capital cost would be R 1 862 million and the unit production cost would be R 9.95/m^3.

Table 3.3: Summary of seawater desalination capital and production costs (Wittholz *et al.*, 2008)

Capacity of plant (m^3/day)	Capital Cost (million US$)	Capital Cost (million R)	Unit Production Cost (US$/$m^3$)	Unit Production Cost (R/m^3)
10 000	20.1	215.1	0.95	10.17
50 000	74.0	791.8	0.70	7.49
275 000	293.0	3135.1	0.50	5.35
500 000	476.7	5100.7	0.45	4.82

Global Water Intelligence (2011) provide the chart shown in **Figure 3.4** which gives unit capital costs for seawater RO seawater desalination plants plotted against desalination plant capacity. Based on this curve, a 100 Mℓ/day plant would cost R 1 298 million.

In a more recent study, Ghaffour, Missimer and Amy (2013) provided a review of the economics of desalination worldwide. The authors note that unit water costs for seawater RO have decreased rapidly over the past decade and are now between 0.50 and 1.2 US$ per m^3 (R5.35 to R12.84 per m^3) for large plants. Investment costs are between 900 and 2500 US$ per m^3/day (R9 630 to R26 750 per m^3/day). This would mean a 100 Mℓ/day seawater RO plant would cost as little as R963 million. Some of the improvements over recent years include increased recovery factors of up to 45 percent for single stage systems or 60 percent for double stage systems, the development of energy recovery systems which reduce the overall energy input to about 4 kWh/m^3, as well as improved membrane performance which reduces chemical consumption. Ghaffour *et al.* (2013) also provide a list of capital costs for recently constructed seawater desalination plants. Some of these costs are included in the discussion in Section 5.2.1.

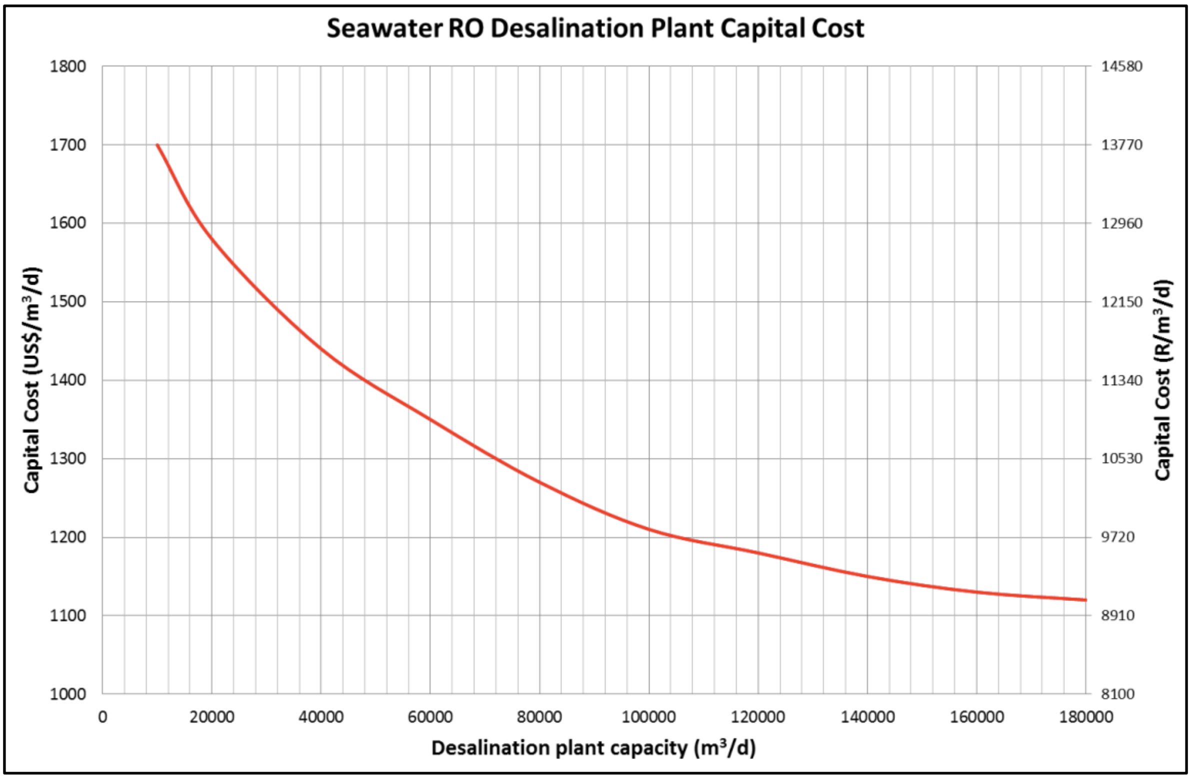

Figure 3.4: **Seawater RO desalination capital costs for increasing plant capacity adapted from Global Water Intelligence (2011)**

In a similar article on the economics of desalination, Al-Karaghouli and Kazmerski (2013) highlight that improvements in membrane technology have dramatically reduced the costs of seawater RO in recent years. For large plants of 100 to 320 Mℓ/day, the authors give an average unit production cost of 0.45 to 0.66 US$/m^3 (R4.82 to R7.06/m^3), and indicate that the typical energy consumption of a seawater RO plant with an energy recovery system is 4 to 6 kWh/m^3.

3.4 Seawater Desalination Case Study: Australia

As a final point in the study of literature related to seawater desalination planning, it was deemed worthwhile to consider the development of seawater desalination in Australia, a country which has seen a massive growth in seawater desalination plant construction over the past five years at great cost to the government and consumers. There are a number of lessons that can be learnt from Australia's approach in terms of seawater desalination development, operational rules and system planning.

3.4.1 Development of seawater desalination in Australia

Australia has a long history of desalination of seawater and groundwater; however its use was previously limited to mining and industrial applications and for supply to remote towns with no access to surface or groundwater resources (Hoang, Bolto, Haskard, Barron, Gray and Leslie, 2009). Like South Africa, Australia has been slow in embracing seawater desalination on a larger scale for municipal use due to the availability of fresh water resources which have, historically, been markedly cheaper to develop.

Between 2000 and 2009, Australia experienced a decade-long drought, reported as the worst in recorded history and widely believed to have been worsened by climate change. It took authorities until 2005 to realise the extent of the problem, however they acted quickly, planning and implementing various emergency schemes including seawater desalination plants in all the major cities in Australia (Onishi, 2010). In 2008, the total design capacity for desalination plants in Australia was 153 Mℓ/day for potable supply and 141 Mℓ/day for industrial purposes, which equates to 0.54 percent of the total water demand of the country. By 2013, the total capacity was expected to be in the order of 1 734 Mℓ/day for potable supply and 461 Mℓ/day for industrial supply, 4.3 percent of the total water demand (Hoang *et al*., 2009).

A summary of the major seawater desalination plants, their location and date of completion is presented in **Table 3.4**. Two of these plants, Kurnell and Tugun are discussed in more detail in Section 3.4.2 and 3.4.3 respectively.

Table 3.4: Seawater desalination plants in Australia (Onishi, 2010)

Desalination Plant	Location	Initial capacity (Mℓ/day)	Ultimate capacity (Mℓ/day)	Date completed	Estimated Cost
Kwinana	Western Australia (Supplying Perth)	130	N/Av.	2006	AUS$350 million (R3.4 billion)
Gold Coast	Tugun, SEQ (Supplying Brisbane)	125	167	February 2009	AUS$1.13 billion (R11.1 billion)
Kurnell	New South Wales (Supplying Sydney)	250	500	January 2010	AUS$1.84 billion (R18.0 billion)
Southern	Binningup, Western Australia (Supplying Perth)	100	550	2012	Not available
Wonthaggi	Victoria (Supplying Melbourne)	410	N/Av.	2012	AUS$3.5 billion (R34.3 billion)
Port Stanvac	South Australia (Supplying Adelaide)	270	270	December 2012	AUS$1.83 billion (R17.9 billion)

3.4.2 Kurnell desalination plant

During the drought years, the New South Wales Government carried out planning and design, purchased the land and completed all the necessary approvals so that a seawater desalination plant could be implemented for Sydney when required. The 2006 Metropolitan Water Plan indicated that the project would be initiated if total system storage volumes dropped below 30 percent. This occurred in February 2007, prompting the construction of the 250 Mℓ/day Kurnell Desalination Plant, shown in **Figure 3.5**, which was commissioned in January 2010 (Patty & Cubby, 2012).

Figure 3.5: Aerial view of Kurnell Desalination Plant (La Perouse, 2010)

The initial contract agreement was to allow the plant operators to run the plant at full capacity during the defects liability period of two years (ending in June 2012). However, the drought was followed by heavy rains forcing the government to negotiate a reduction in the output to 90 Mℓ/day in January 2012. This was further reduced to 45 Mℓ/day in March 2012, which is the lowest volume the plant can contractually produce without the defects liability being transferred to the government. This decision was sparked by the Warragamba Dam, one of the primary sources of supply to the area, reaching 87 percent of its capacity. Following further heavy rains the dam did spill (in early March 2012) prompting critics to question whether the decision to step down the desalination plant was taken too late. Operation of the plant will be ramped up again if the storage volumes drop below 70 percent (Patty & Cubby, 2012).

3.4.3 Tugun desalination plant

Queensland’s seawater desalination plant shown in **Figure 3.6** was commissioned in February 2009 and despite some early problems, provided relief to the strained water supplies of the region towards the end of the drought. The plant was extremely costly to construct (in excess of R 9 billion) and is also expensive to run, resulting in a marked increase in the water tariffs for Queensland residents. This cost was considered acceptable during the drought, however once the dams filled it became a cause for concern and began to impact on the public’s perception of seawater desalination (Australian Associated Press, 2010).

Figure 3.6: Tugun (Gold Coast) Desalination Plant (Water Technology, 2011)

In December 2010 it was announced that the plant would be mothballed, saving R 80 million per year and hence allowing a reduction in the water tariffs for the region (Australian Associated Press, 2010). Later reports in 2011 corrected this statement, indicating that the plant is in standby mode ready to return to full time operation within 72 hours if the dam levels drop below 60 percent (ABC Gold Coast, 2011). The South East Queensland Operational Plan states that the seawater desalination plant should be operated at full capacity when the total system storage volume drops below 40 percent (Queensland Water Commission, 2013:2). This is discussed in more detail in Section 3.4.5.

3.4.4 Concerns and criticisms

Critics in Australia believe that the implementation of seawater desalination was premature and that cheaper alternatives including improved catchment management, groundwater reserve management and water conservation measures would have been better suited for the application. The Institute for Sustainable Futures at the University of Technology in Sydney suggests that towns should be "*desalination ready*", ensuring that the planning and design for seawater desalination is complete but that implementation should only be initiated in the event of a severe drought (Onishi, 2010). This is the same approach that has been adopted by Umgeni Water in their seawater desalination planning as mentioned in in Section 3.1.3.3.

Knights, MacGill and Passey (2007) refer to the importance of integrated urban water management in planning for future water supply in Australia. The authors feel that seawater desalination does not fit the normal water management paradigm, primarily because it provides a specific volume of water with limited complexity and high assurance of supply. Proponents of seawater desalination will view this as one of its strengths; however opponents see this as a weakness as the result is a rigid and inflexible system which does not allow for optimum utilisation of the existing resources.

A final concern regarding the Australian cases is the issue of flexibility to reduce output once a seawater desalination plant has been constructed. As discussed, the plants at Kurnell and Tugun have been placed in standby mode as a result of the major storage dams reaching capacity. This obviously reduces the operating costs of the plants and the literature suggests that this can be translated to a reduction in water tariffs for consumers. However, what isn't accounted for or discussed in any of the literature reviewed is that a significant portion of the tariff charged for desalinated water will be for repayment of the capital cost of the infrastructure. Regardless of whether or not the plant is operating, it will be necessary to recoup the capital redemption portion of the tariff in order finance the debt.

It is therefore essential that system operating rules for the plant are clearly defined at the outset and that the likely total volume generated over the planning period is used for option comparison and tariff analysis rather than the total design capacity of the plant. This links back to the concerns raised by Van Niekerk and Du Plessis (2013a and 2013b) in their research on inter-basin transfer schemes as presented in Section 3.2.1

3.4.5 South East Queensland water planning

Following the many decisions and changes (both good and bad) made during the drought, water resource planning has been undergoing a major overhaul across Australia, with new planning and strategic documents being prepared across the states. The most relevant example is found in South East Queensland, a region which is supplied predominantly by surface water resources but now has a seawater desalination plant and large-scale water reuse scheme, both of which were constructed during the drought. South East Queensland is therefore a good example of how climate-resilient water resources can be integrated into the overall water supply system at a regional level.

3.4.5.1 Planning criteria

The South East Queensland strategic planning documents define that during normal conditions, an average urban water demand of 375 ℓ/capita/day must be available (gross demand including water use and water losses) of which 230 ℓ/capita/day is assigned for residential demand (Queensland Water Commission, 2012:2).

In terms of infrastructure planning, decisions regarding the implementation of future schemes are based on a set of criteria as follows (Queensland Water Commission, 2012:2):

- Medium level water restrictions are required on average once in every 25 years.
- Restrictions only require a 15 percent reduction in the total average consumption.
- The restrictions only last longer than six months once in every 50 years.
- Emergency drought response infrastructure is not required more than once in 100 years.
- Combined regional storage supplies do not drop below 10 percent more than once in 1 000 years and never drop below 5 percent.
- Primary dams do not reach MOL.

To some extent, the approach is similar to that of the WCWSS planning criteria in that less severe water restrictions are accepted more often than more severe restrictions. Generally, the criteria are more stringent than those applied in South Africa.

3.4.5.2 Operating rules and drought response

In accordance with the criteria defined in Section 3.4.5.1, the system storage is divided into key planning levels or trigger levels as shown in **Figure 3.7**.

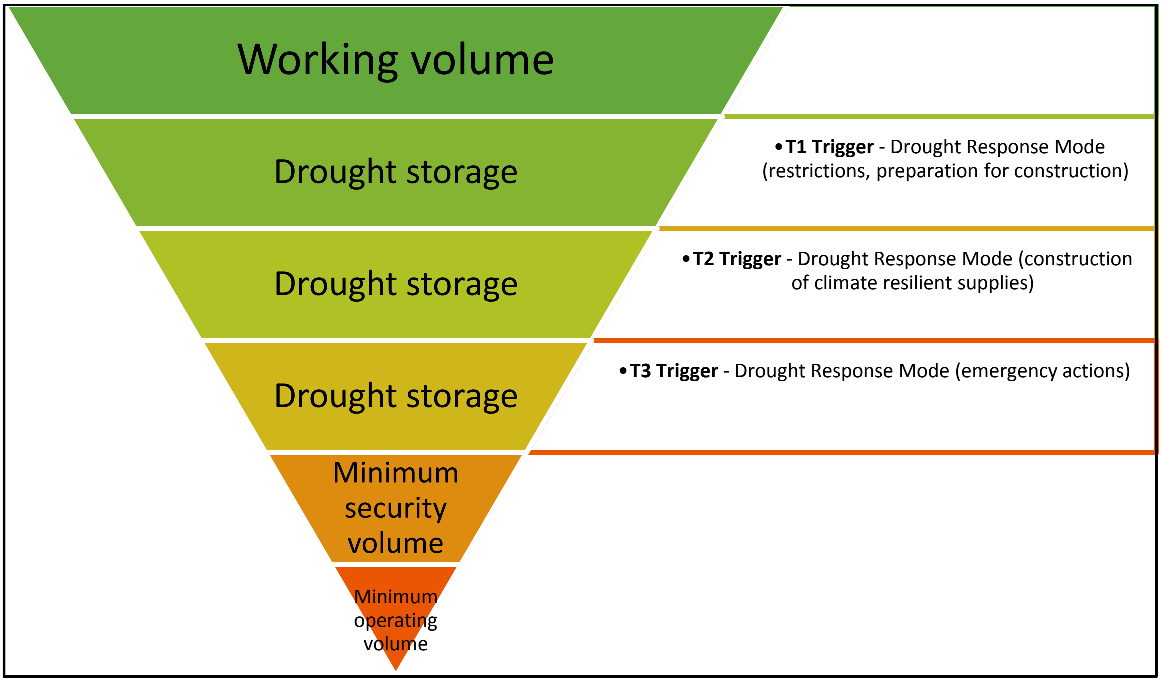

Figure 3.7: **System storage trigger levels in Queensland (Queensland Water Commission, 2010:40)**

Level T1 is set at 40 percent of the total system storage. At this point, medium level water restrictions are implemented and drought response measures are prepared for construction in accordance with a pre-defined drought response plan. In addition, treated effluent is introduced into the Wivenhoe Dam from the Western Corridor Recycled Water Scheme (WCRWS), one of the largest treated effluent water schemes worldwide. Completed in 2008, it can supply up to 232 Mℓ/day from three advanced WTW (Queensland Water Commission, 2010:41). In addition, the South East Queensland Operating Plan (Queensland Water Commission, 2011:2) indicates that at this level (T1), the supply of water from the Gold Coast desalination plant must be maximised, subject to operational constraints.

Level T2, set at 30 percent of the combined capacity of the key storage reservoirs, triggers the construction of the planned drought response measures which will typically entail climate-resilient supply sources such as seawater desalination. **Level T3** is set at 20 percent and triggers emergency actions. The trigger levels are reassessed in accordance with changing demands, new infrastructure and the type of drought measure to be implemented (Queensland Water Commission, 2010:41). In addition, a trigger level is set above T1 which defines the point at which restrictions can be lifted when coming out of a drought (Queensland Water Commission, 2010:42).

With reference to these limits, the system operating rules are based on a set of risk criteria and for the various trigger levels an assurance of supply is set, much in the same way as in South Africa. The risk criteria are presented in **Table 3.5**.

Table 3.5: Queensland planning risk criteria (Queensland Water Commission, 2011:2)

Total system storage level	Probability of reaching level		
	Within one year	Within three years	Within five years
40% (T1)	< 0.2%	Not specified	< 5%
30% (T2)	Not specified	< 0.5%	< 1%

3.4.5.3 Impacts of millennium drought response

Despite the issues with operating the Tugun desalination plant (discussed in Section 3.4.3) and the widespread criticism of Australia's drought response, there are three key aspects of the emergency drought relief measures that were implemented in Queensland which have had a positive impact on the water supply situation.

Firstly, as part of the emergency drought mitigation measures in the region, the eight separate water supply zones were linked into one water grid. This allows a great amount of flexibility in the system allowing transfer of water in times of drought and optimal use of water during times of plenty thus minimising losses through dams spilling. As a result, the overall system yield has increased by 14 percent (Queensland Water Commission, 2010:83). As indicated in Section 2.2.2 and 2.4.3, the importance of integrated system operation has been recognised as part of the WCWSS.

Secondly, the construction of the Tugun desalination plant and the WCRWS has meant diversification in the water supply system moving from 95 percent supply from dams and weirs in the past to 74 percent in 2012 (Queensland Water Commission, 2010:86). The option of supplying the WCRWS water into the Wivenhoe Dam during times of drought dramatically increases the yield of the system. The trigger level at which this supply happens is set at a point where supplying recycled water to the dam will have an optimum impact on the system yield (Queensland Water Commission, 2010:87). Determining this optimum point is the crux of integrating climate-resilient water resources into a water supply system.

Thirdly, the implementation of water restrictions and water conservation and demand measures had a significant and lasting impact on water demands, suggesting a permanent change in the attitude of the citizens to water use (Queensland Water Commission, 2010:54).

3.4.5.4 *Future planning*

In terms of future water supply, the South East Queensland Water Strategy adopted a very similar approach to the WCWSS Reconciliation Strategy. Two portfolios were presented, one based on a medium growth scenario and the second based on a high growth scenario. For both, implementation measures with and without climate change were considered. The area has few viable surface water or groundwater alternatives and seawater desalination is therefore recommended for implementation to meet the water demand for the next 30 years. The WCWSS fortunately still has a number of viable surface and groundwater options, however the outlook for Cape Town as well as other large coastal cities in South Africa is likely to look very similar to Queensland in the future (Queensland Water Commission, 2010:113).

3.5 Conclusion

Seawater desalination planning in South Africa, as in Australia, has historically happened on an *ad hoc* basis in response to severe droughts, requiring the rapid construction of seawater desalination plants as emergency supply schemes. The national and local planning documents in South Africa, however, all highlight the role that seawater desalination will play in future water supply and recent feasibility studies certainly show the characteristics of more holistic and integrated planning. The examples of the Mossel Bay desalination plant in South Africa and the Kurnell and Tugun plants in Australia illustrate just how important this integrated planning is.

4 THE WATER RESOURCES YIELD MODEL (WRYM) AND WATER RESOURCES PLANNING MODEL (WRPM) SOFTWARE TOOLS FOR WATER RESOURCES ANALYSIS

Section 2.1 provided an introduction to some of the key principals of water resource analysis and modelling. In order to apply these principals to the large and complex WCWSS it is necessary to make use of water simulation tools or software packages. Section 4.1 provides an overview of available tools for water resource system simulation which were considered as possibilities for such an analysis. The WRYM and the WRPM were selected as the most appropriate tools, as explained in Section 4.2, and an overview of each is presented in Section 4.3 and 4.4. Existing WRYM and WRPM model configurations were sourced for the WCWSS from previous studies and these formed a basis for the modelling. A brief discussion of the historical development of these models is provided in Section 4.5. Note that this section is intended to provide a theoretical overview of the WRYM and WRPM for a reader who does not have extensive exposure to working with the models.

4.1 Available Models

Görgens, Tanner, Viljoen, Sami, Dennis and Nel (2003) undertook a comprehensive evaluation for DWS of the available tools for water resource modelling in South Africa, in support of the implementation of licensing under the National Water Act. Through a series of workshops comprising modelling-related specialists, researchers and water resource managers, a categorised list of available models was prepared through a collaborative screening process. A summary of the shortlisted models for system modelling based on Görgens *et al.* (2003:36) and updated with more recent information is provided in Section 4.1.1 to 4.1.5.

4.1.1 Water resources yield model (WRYM)

The WRYM is a water resource system analysis model developed by DWS which operates on a monthly time step and allows a user to carry out scenario analyses for determining resource potential, evaluating intervention options, evaluating operating rules and assessing system behaviour. The model has both historic and stochastic functionality allowing for the calculation of long-term or short-term yields and assurance of supply. The WRYM is currently under the custodianship of DWS who is responsible for model maintenance and support. The latest available version is 4.0.2.1, released in November 2013, and it is available free of charge provided that users register.

4.1.2 Mike basin

Mike Basin is a quasi-steady-state mass balance network model owned and maintained by DHI Water and Environment. Mike Basin has a Geographical Information System (GIS) based interface in ArcGIS and allows for hydrologic modelling, water quality simulation, and groundwater modelling using a two layer linear reservoir equation including pumping. It is capable of modelling variable time scales (daily or monthly) and can be used for assessing water allocations, conjunctive use and reservoir operation. The latest available version is the 2012 release (with service pack 3), and it costs between R80 000 and R120 000 to license. As part of DHI's 2014 software release, Mike Hydro Basin has replaced Mike Basin. It has most of the same functionality, but is now independent of ArcGIS or any other third party software packages.

4.1.3 Rapid simulation model

The Rapid Simulation Model (RSM) is a water resources yield model comprising a range of data capture, water use analysis and scenario analysis tools. It was developed by Stephen Mallory for Tlou and Matji as a yield model for reconnaissance level studies providing quick results which are accurate and user-friendly enough to be used in a workshop environment. It caters for all types of water use with hydrology based on previous quaternary-scale assessments. RSM was regarded as an "*emerging model*" by Görgens *et al.* (2003) but it appears that it has not gained much traction over the past decade.

4.1.4 Water resources planning model (WRPM)

The WRPM, developed by DWS, is a network-based model operating on a monthly time-step used for allocation of water based on a set of penalty structures. As for the WRYM, it allows for stochastic modelling, and is therefore capable of assessing assurance of supply. The model is used mainly to assess different augmentation and water restriction strategies based on projected water demands over a planning horizon. It also has salinity modelling capabilities. It differs from the WRYM mainly in its ability to assess growing water demands and short-term operational decisions. WRPM is under the custodianship and ownership of DWS and is therefore available free of charge. The latest version (v4.3.4) was released in August 2012 although the user manual has not been updated since 2000.

4.1.5 Water resources modelling platform (WReMP)

WReMP was developed subsequent to the study by Görgens *et al.* (2003) by Stephen Mallory, and, as discussed in Section 3.2.1, is a network-based model with similar functionality to WRYM. It differs from WRYM in that it uses a rule-based simulation approach which uses a solver to imitate an operator's decision-making process through an iterative analysis. Water supply shortages are first quantified in the system, and then these shortages are supplied out of dams via release channels. The WReMP is owned by IWR Water Resources, and is not freely available for use by external parties.

4.2 Model Evaluation and Selection

Görgens *et al.* (2003) focused primarily on the general characteristics of consistent water resource modelling; however the authors did highlight four important points which influenced the selection of an appropriate model for this research. Firstly, large regulated catchments lean towards the use of monthly time-step models, which means any of the listed available models would be appropriate for the WCWSS. Secondly, it is emphasised that unless stochastic modelling is undertaken, it is not possible to realistically determine annual probability of exceedance or recurrence interval in years. This therefore favours the WRYM and WRPM. Thirdly, it is recommended that modellers make use of databases and configurations of earlier studies as starting points rather than 'reinventing the wheel'. Again, this favours the WRYM and WRPM as up-to-date models of the WCWSS have been set up in both these platforms which are freely available for research purposes.

Lastly, practical factors such as cost, availability of user support and ease of use were identified in the study workshops as important factors to consider in choosing an appropriate model. The WRYM and WRPM are both freely available, are combined in an easy-to-use interface, and support is available directly from DWS via email. However, these models have frequent programming bugs and instabilities which are often frustrating to a user. On the contrary, Mike Basin, being a commercial package, is stable and robust but is extremely costly for a private/commercial user.

Given the ease of use, the available functionality, the low costs and the fact that there are existing models of the WCWSS already available, the WRYM and WRPM were selected as the most appropriate models for analysing seawater desalination in the context of the WCWSS. As part of future research, it may be worth investigating the possibility of using Mike Basin.

4.3 Overview of the WRYM

The WRYM (De Jager, 2008) is a network-based model comprising basic building blocks including reservoirs, nodes, and other system features which are connected using channels. The model has the advantages of being flexible in terms of size and configuration, user-friendly in its graphical presentation of results, and robust in its ability to manage scenarios and carry out stochastic analyses. It does, however, have a few limitations which are important to remember. Firstly, hydrological (rainfall-runoff) modelling is not built in, so extensive pre-processing is required to generate inflow files and demands. Secondly, modelling is only possible at a monthly time step. Thirdly, it does not have hydraulic analysis and river routing capabilities.

4.3.1 Model components

A total of 21 system configuration data files (*F01.DAT to *F21.DAT) are used to store the input information for a study. Each study is assigned a run code of up to five characters which is used as a prefix for all of the system files. The run code as well as the directory path for the input and output files is specified in the directories file, WRYM.DAT.

Catchment hydrology components of the model comprise monthly time-series files of incremental natural runoff (*.INC), streamflow reductions (*.AFF) and diffuse irrigation demands (*.IRR) which are created outside of the model. The net runoff into a node or reservoir is calculated as the natural runoff minus the irrigation demands and the streamflow reductions. In addition a monthly point rainfall (*.RAN) file is required along with a series of 12 average monthly evaporation values. These are used for calculating the rainfall on and evaporation from the surface of the reservoirs in the network.

Nodes are simply junctions in the network where tributary catchments join, conveyance routes split, or where water is abstracted. ‘Nodes without inflow’ require no input information other than a name or identifier and ‘nodes with inflow’ have an associated suite of catchment hydrology files. The node inflow information is stored in the *F02.DAT file.

Operating rules within the WRYM are defined based on penalties. Penalties are dimensionless values assigned to channels and reservoirs, which are compared based on relative size. Penalties are used to set the priority of supplying a user, as well as defining which channels and reservoirs will be used to meet the demands and in what order. The general principle is that preference will be given to abstracting from the reservoir or channel system with the lowest penalty to supply the user with the highest penalty.

Reservoirs are modelled as nodes with storage capacity and their input parameters are stored in the *F02.DAT file. Reservoirs require input of the physical reservoir characteristics (level-area-volume relationship), associated incremental sub-catchment hydrology, a starting storage level and zone or rule-curve elevations. Reservoir penalty structures are defined in terms of reservoir zones with a penalty assigned to each zone. A rule-curve level is also required, which is usually set equal to the FSL of the reservoir. Above the rule-curve level, the penalty on each zone is equivalent to a detriment of having water in storage and below the rule curve level the penalty on each zone represents the benefit of having water in storage. The reservoir storage zone, rule-curve and penalty structures are defined in the *F05.DAT file and the starting storage levels are included in the *F06.DAT file.

Channels are water carrying conduits and are used to represent physical features such as rivers, canals or pipelines or model features such as target drafts, water demands or system losses. There are a variety of channel types available, all of which perform different functions as summarised in **Table 4.1**. Channels are also assigned penalty structures. The basic building block of a channel penalty structure is referred to as an arc, which allows particular flows under specific circumstances. Each arc will have a lower limit, upper limit and penalty. Most commonly used channels have one or two arcs. The channel penalty structures along with the channel numbers, types, upstream and downstream nodes and penalties are stored in the *F03.DAT file. Further input information for the various channel types are stored in the files as indicated in **Table 4.1.**

Table 4.1: Summary of types of channels used in the WRYM

Channel type	Use	No of arcs	Data File
Master control (yield)	Imposing target drafts to determine system yield	2	F13.DAT
Diversion	River diversion structure / abstraction works	2	F10.DAT
Minimum flow	Channels which must maintain a minimum flow (e.g. releases from reservoirs)	2	F11.DAT
Loss	Flow related losses such as seepage or evaporation	2	F11.DAT
Multi-purpose min-max	Channel with restricted capacity	1	F12.DAT
	Water requirements and return flows	2 or more	
Pumping	Hydraulics of pump stations and pipelines	2	F03.DAT only
Specified inflow	Monthly time series of inflows – typically from a separate system	2	F03.DAT only
Specified demand	Specified abstractions which vary on a monthly and annual basis (e.g. irrigation)	2	F03.DAT only

Channel type	Use	No of arcs	Data File
General	River reaches or flow routes with no capacity constraint	1	F03.DAT only
In-stream flow requirements	Environmental water requirements based on rule curves	2	F14.DAT
Flow constraint	Physical flow constraints in a channel / control structures	1 or 2	F04.DAT
Hydropower	Power generating capacity of a hydropower plant	2	F07.DAT F08.DAT

4.3.2 Historic analysis

In order to perform a historic yield analysis (as defined in Section 2.1.2), a range of target drafts are inputted, the network model is run separately for each, and the base yield is calculated. The run configuration is stored in the *F01.DAT file and outputs are saved in *.OUT suite of files. The output interface allows the user to view a draft-yield curve which is drawn based on the target drafts and base yields analysed. The historic firm yield can then be read off. There is also functionality to automatically determine the historic firm yield; however this often causes the program to crash so a manual approach is generally more effective. Any individual result files can be viewed as monthly time series or graphical plots including channel flows, reservoir water balances and reservoir trajectories. These plots can be used to assess the operational behaviour of the system and check the logic of the results.

4.3.3 Stochastic analysis

In order to perform a stochastic analysis and determine long-term assurance of supply, stochastic flow sequences are generated synthetically based on the historical flows using marginal distribution, serial correlation and cross-correlation in the Stochastic Streamflow Model (STOMSA) (Department of Water Affairs, 2013c), developed by DWS and integrated into the WRYM:

Marginal distribution refers to the relationship between the total annual flows when ranked, typically plotted as annual flows versus probability of exceedance. Four marginal distribution models are available in STOMSA, including the 2-parameter Log-normal (LN2) and 3-parameter Log-normal (LN3) as defined in Equation 4.1, and the 3-parameter Bounded (SB3) and 4-parameter Bounded (SB4) as defined in Equation 4.2. These distributions produce normalised flows. The selection of the model which best fits the historical data is based on statistical criteria described by the Hill Algorithm which is based on the Johnson Transform Suite (Department of Water Affairs, 2013c:2-2).

$$y = \gamma + \delta \ln(x - \xi) \tag{4.1}$$

$$y = \gamma + \delta \ln(x - \xi)/(\lambda + x - \xi) \tag{4.2}$$

Where:

y	Transformed variate	[million m^3/a]
x	Annual streamflow variate	[million m^3/a]
$\gamma, \delta, \xi, \lambda$	Distribution parameters	[-]

<u>Serial correlation</u> refers to the relationship between normalised annual flows and is represented by a correlogram. A correlogram is a plot of correlation coefficients against lag in years. The sequence of normalised annual flows is analysed based on an auto regressive moving average (ARMA) model, defined as shown in Equation 4.3. Nine possible ARMA(ϕ, θ) time-series models are available with phi and theta equal to 0, 1 or 2. Application of the models removes the serial correlation characteristics of the series, resulting in normalised residual annual flows (Department of Water Affairs, 2013c:2-3).

$$X_t - \phi_1 X_{t-1} - \phi_2 X_{t-2} = A_t - \theta_1 A_{t-1} - \theta_2 A_{t-2} \tag{4.3}$$

Where:

X_t	Stationary sequence of centred normal variates	[-]
A_t	Sequence of independent random variables with normal distribution having zero mean and constant variance	[-]
ϕ_1, ϕ_2	Auto-regressive model parameters	[-]
θ_1, θ_2	Moving average model parameters	[-]

<u>Cross correlation</u> refers to the relationship between flows from different catchments and is determined using singular value decomposition (Department of Water Affairs, 2013c:2-3). The resulting set of matrices is written to a parameter file (Param.DAT) together with the results of the marginal distribution and serial correlation.

STOMSA generates flow sequences in a three step process. Firstly, random numbers are generated; secondly, the selected statistical distribution models are applied (cross-correlation, serial correlation and marginal distribution in that order); and, thirdly, the annual flows are disaggregated into monthly values. The user is required to select ‘key gauges’ which are the most important or representative catchments. Using these gauges a least-square-fit approach is followed to select the single year for which the difference between the historical and generated annual flow values is smallest. The monthly distribution pattern for this key year is then applied to all catchments (Department of Water Affairs, 2013c:2-4).

Before a yield analysis is undertaken, it is critical that the results of the stochastic flow generation are verified and validated. Verification involves re-sampling of various statistics from the generated sequences and confirming that these fall within reasonable bounds from the historic values. Validation tests involve testing aspects of generated sequences which were not used directly during the generation process. The results are typically plotted on a box-and-whisker plot, an example of which is provided in **Figure 4.1**.

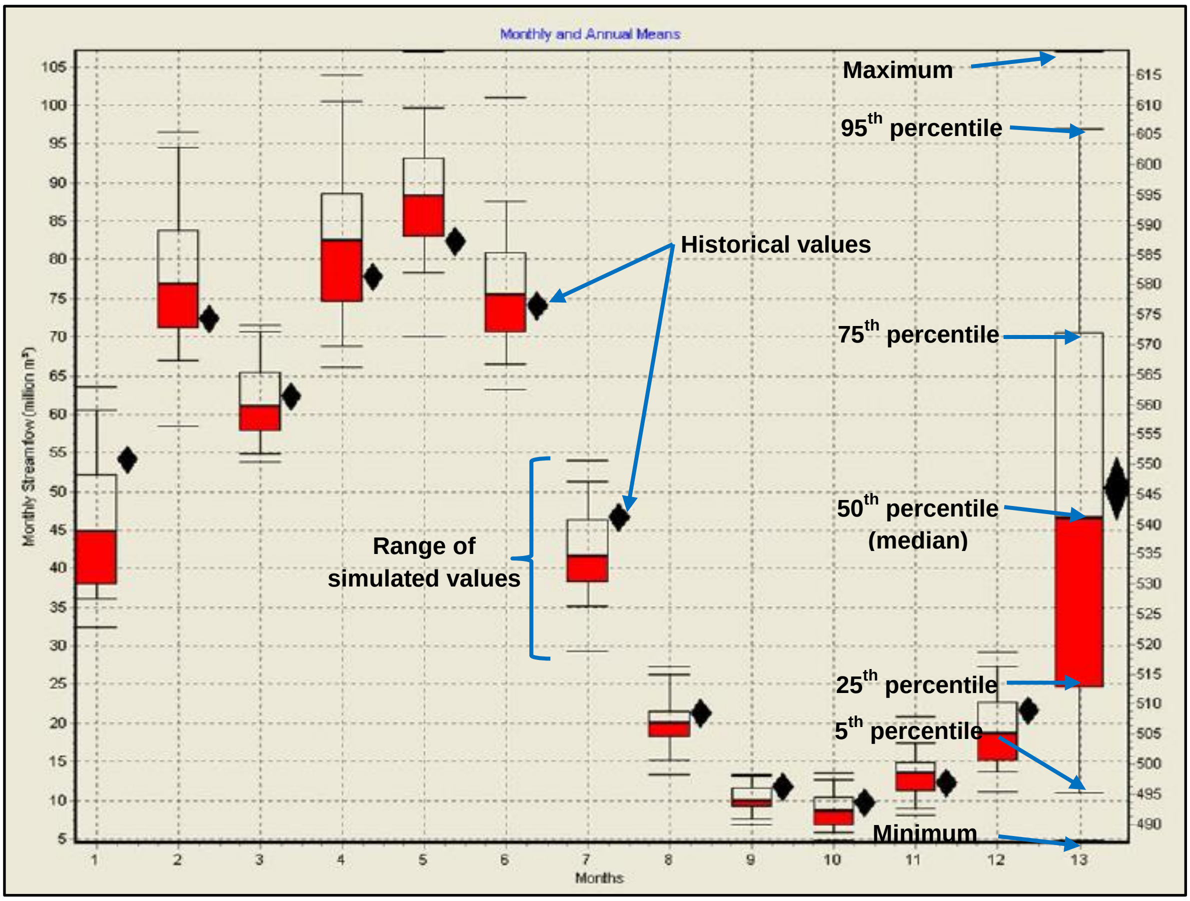

Figure 4.1: Typical example of a box-and-whisker plot

Once the verification and validation is complete, each of the generated sets of stochastic flow sequences is then run through the yield model and the base yield calculated for each. This can be done for a range of selected target drafts in one simulation, as specified in the *F01.DAT file. The family of yield-reliability curves can be viewed as an output and the targeted yields and reliabilities read off.

4.4 Overview of the WRPM

The WRPM (Van Rooyen, 2000) is a sister program of the WRYM used to assess water supply of a dynamically changing water resource system. It has two main functions:

1. Development Planning: To schedule implementation dates of possible future developments based on projected future water demands over a medium-term planning horizon. The model allows the user to determine the timing and delivery schedule of possible interventions, as well as filling requirements for future dams.
2. Operational Planning: To develop operating rules for reservoirs, inter-sub-system transfers, dual users and water quality aspects.

Like the WRYM, the WRPM comprises a network simulation algorithm along with a stochastic flow generator, but has additional functionalities including a water resource allocation procedure and salinity modelling procedure. The linkages between the components within the WRPM are summarised in **Figure 4.2**.

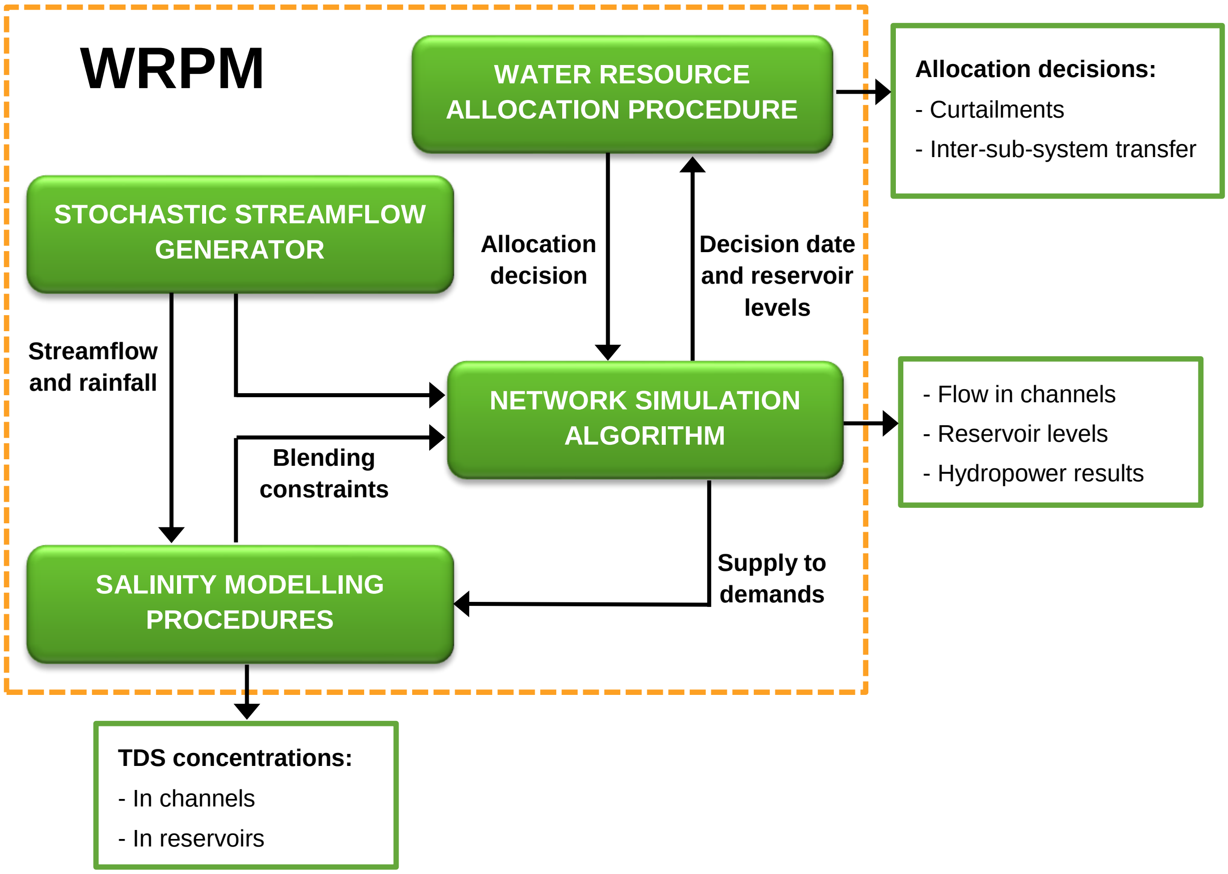

Figure 4.2: Linkages between components of the WRPM

4.4.1 Model components

The run code and locations of the input and output files are defined in the WRPM.DAT file and run configuration is included in the *F01.DAT file. The input file structure is similar to the WRYM but with 14 additional files to store planning-specific data.

Hydrological data required, as for the WRYM, includes monthly time-series data for incremental natural stream flows (*.INC), point rainfall (*.RAN), afforestation (*.AFF) and irrigation (*.IRR) per catchment along with average monthly evaporation values (stored in *F02.DAT) and stochastic flow parameters (PARAM.DAT).

Reservoir characteristics including evaporation, area, volume, catchment hydrology and penalties are defined in *F02.DAT, *F05.DAT and *F06.DAT as for the WRYM. In addition, reservoir implementation dates, capital and maintenance costs, economic life and short-term yield data are saved in *DAM.DAT. Additional reservoir controls can also be added on a time-based approach where reservoir characteristics are changed at particular points in the analysis period.

Channel information is saved in *F03.DAT with additional information included in *F04.DAT and *F07.DAT to *F14.DAT for the various channel types, exactly as per the WRYM. Additional control features can be added for each channel to enable it to open or close during the analysis. The control can either be time-based, i.e. open or close at a particular point in time, or switch-based, i.e. dependant on the level in the reservoir. This information is saved in *SW.DAT.

4.4.2 Allocation procedures

The fundamental allocation procedure used in the WRPM requires classification of user groups according to priority of supply. In order to avoid failure of a system, curtailments of supply to these user groups may be required over the analysis period. The model is assessed for a set month each year, and the average annual water requirements for the year in question are compared to the short-term yield-reliability curves in order to determine whether curtailments are required. The model implements curtailments based on the defined user priorities, with lower user classes curtailed first to protect the supply to high user classes. The level of curtailment required to prevent failure is determined on an iterative basis, with the aim of balancing curtailments for the different user groups.

It is important to note that unlike long-term yield curves, short-term yield curves are highly dependent on the assumed starting reservoir storage, and the difference between yields for different starting volumes is generally more pronounced for lower recurrence intervals. A range of starting reservoir storage levels can therefore be specified to be run as different scenarios within the same analysis. The short-term yield curves are typically derived using the WRYM, and the curve coefficients for a range of target drafts and starting storage levels is stored in the *FM*.DAT file.

Projected annual average water demands along with an average monthly distribution over the year, priority classification of the different user categories and reliability criteria for each user category are also defined in the allocation control file, *FM*.DAT file. *FM*.DAT also contains the sub-system definitions, inter-basin support control data, and the structure of which sub-systems can support a particular user.

Growth factors for demand channels, min-max channels and hydrology files are specified in *GTH.DAT. Lastly, for each master control channel, the detriment (from a financial perspective) of not supplying the target demand are quantified using a cubic equation with user-specified coefficients. This information is stored in *DBF.DAT. In addition, unit tariffs for water supply via the master control channel are inputted and written to *TAR.DAT.

4.4.3 Simulations and outputs

There are two types of solving structure available in the allocation algorithm, which are hierarchical or capability dependent. The hierarchical solving structure solves the system in the order in which they are specified in the data files based on the physical layout of the subsystems. The capability-based structure was developed specifically for the WCWSS and uses the yield capability of the sub-systems as a reference index.

Many of the outputs from WRPM are similar to WRYM, allowing for graphical or tabular views of average or monthly reservoir volumes and channel flows. The demands on and supply from the system, levels of curtailment implemented per sub-system, and the total system storage can also be viewed on a monthly or annual time-step. In addition, single sequences can be analysed to check on the correctness of the system configuration, impact of operating rules and system behaviours for different scenarios.

4.5 WCWSS Model Setup in WRYM and WRPM

Development of the WRYM and WRPM models of the WCWSS has evolved over a number of planning studies over the past two decades. Development of the models began in 1994, with the Western Cape System Analysis (WCSA) (De Smidt, 1994) study undertaken by DWS and the City of Cape Town. As part of the study, system models were configured for the various sub-systems, catchment hydrology was developed and demands were estimated.

Development of the upper portions of the WCWSS models including the Berg, Riviersonderend and Molenaars Rivers took place in 1999, during the Feasibility Study for the Berg River Dam which also included an extension and update of the catchment hydrology. In 2001, the hydrology of the incremental Berg River catchment was again updated and extended as part of the Feasibility Study into the Voëlvlei Augmentation Scheme.

The most recent detailed update of the models was carried out in 2010 as part of the Berg Water Availability Assessment Study (WAAS). This included modelling of the groundwater and surface water interaction, and adjusting the model configuration to be able to calculate the yield of the system as a whole (Department of Water Affairs, 2010c:i).

From 2012 to date, a DWS study into the development of integrated annual and real-time operating rules for the WCWSS has been underway, part of which involves an update of the water requirements, system losses and WRYM and WRPM models. The updates include integrating the Berg WAAS hydrology into the models, as well as updating the infrastructure and operating rules to ensure that they match the current system.

The existing WRYM and WRPM setups are integrated and comprehensive comprising all of large dams, lumped farm dams for each catchment, tunnels and conveyance infrastructure, environmental flow requirements for the Palmiet River and Berg Water Projects, evaporation and operational losses, and recently projected water demands. The models are tailored to match the existing conveyance and water treatment infrastructure capacities. For further details on the existing model setups, reference can be made to the various planning reports as referenced in this section.

5 ANALYSIS METHODOLOGY

In order to analyse the integration of desalination into the WCWSS for the purpose of this research, an approach was required in which the desalination plant could be modelled as an integral component of the existing system. As discussed in Section 4, the WRYM and WRPM were selected as the most appropriate tools to achieve this. The approach for incorporating seawater desalination into these models is discussed in Section 5.1, including the modelling methodology and the various scenarios that were considered. Section 5.2 provides a discussion of the selection of the most appropriate costing methodology out of those described in Section 3.3, and how the selected methodology was adapted to integrate with the water resource simulations.

5.1 Integration of Seawater Desalination into the WCWSS

As discussed in Section 3.2.1, there has been very little research to date on the integration of seawater desalination into predominantly surface water driven water supply systems. As was shown in Section 3.1.1, these planning aspects were not taken into consideration in the planning of the current operational seawater desalination plants in South Africa. As a result of the limited research, the system modelling approach adopted for this research comprised a trial-and-error type assessment using different types of system components, assessing a variety of operating rules, and testing different ways of integrating the plant into the system.

5.1.1 Base model setup

Given the long history involved in the development of the existing WCWSS models in WRYM and WRPM and described in Section 4.5, and the fact that these models have been tried and tested for a number of studies, it seemed logical to use them as the base for the modelling required in this research. The latest versions of the WRYM and WRPM models of the WCWSS were obtained from Aurecon with the permission of DWS. The WRYM setup spans 77 hydrological years from 1928 to 2004, and the WRPM setup contains 10 years of demands projected from 2013.

Before developing the modelling philosophy, each component of the existing model was studied in detail and the model inputs for the various network features were summarised and tabulated. A spreadsheet-based network diagram was also prepared which, along with the summary tables, provided an easy reference point for understanding the operation of the complex system.

5.1.2 Modelling approach

The following options were considered and tested for modelling a seawater desalination plant in the WRYM and WRPM:

- Option 1: An artificial natural runoff (*.INC) file would be created to represent the seawater desalination plant, with uniform flows in each month equal to the seawater desalination plant capacity. The seawater desalination plant inflow would connect to the system via a general flow channel. Separate inflow files would be required in order to test different seawater desalination plant capacities.
- Option 2: A specified inflow channel (mentioned in **Table 4.1**) would be used to represent the seawater desalination plant, with the specified inflows equivalent to the seawater desalination plant capacity.
- Option 3: Using a multi-purpose min-max channel with one or two arcs as an inflow channel from a zero node, with the flow limited to the seawater desalination plant capacity.

For Option 1, the artificial inflow files would have to be incorporated into the parameter file in order to be able to import them into the yield model and run stochastic analyses. Attempts to run the files through STOMSA failed as the statistical models (discussed in Section 4.3.3) do not work for a uniform flow series. It may be possible to create artificial statistical parameters manually; however, from experience, this can often be challenging.

Upon further investigation of Option 2 it was found that specified inflow channels are very rarely, if ever, used in the WRYM. In addition, a specified inflow channel must be created upstream of a node with inflow, with the inflow file included in the suite of hydrology files for that node. This again would require creating artificial hydrology files which, as discussed, can be problematic.

Considering Option 3, at first a single-arc min-max channel was used with the channel flow constraint equal to the seawater desalination plant capacity. Although the channel seemed to work as desired, it was found that with a single arc channel there were some limitations on the possible operating scenarios that could be modelled. As a result, a two-arc channel was tested, and it was found that with some minor changes to the penalties and constraints the full range of operating scenarios (discussed in Section 5.1.3) could be modelled. Option 3 was therefore selected as the best approach for modelling the seawater desalination plant.

5.1.3 Operating scenarios

The main operating philosophy of the WCWSS is to minimise spillage from the dams. This is achieved in the WRYM and WRPM by dividing the reservoirs into a large number of storage zones, with the upper zones having a lower penalty and the lower zones having a higher penalty. This forces the demands to withdraw from the upper zones first and reduce the possibility of water being 'lost' from the system due to spillage. The major dams in the system all have the same penalty which imposes a steadily increasing penalty as the dam levels drop, meaning that the main dams are drawn down uniformly.

Based on the current system operation, four possible scenarios were developed for the implementation and integration of a seawater desalination plant:

- <u>Scenario A:</u> Base scenario with current system and no seawater desalination plant, representing the status quo or base case for comparison purposes.
- <u>Scenario B:</u> Seawater desalination plant used as a base load supply, always operational regardless of the conditions in the rest of the supply system (i.e. even when the dams are spilling). Under this option, the seawater desalination plant min-max channel was allocated a penalty greater than the penalty of having water in storage, and the flow constraints of both arcs were set equal to the seawater desalination plant capacity. This forced the yield channel to draw from the seawater desalination plant regardless of what was happening in the rest of the system.
- <u>Scenario C</u>: Seawater desalination plant used as a base load supply, but only if the major dams are not spilling. Similar to Scenario B, a high penalty was assigned to the desalination min-max channel with the capacity constraint assigned to both arcs, but in this scenario the penalty assigned was less than the penalty associated with the main dams spilling.
- <u>Scenario D</u>: Seawater desalination plant used as an emergency supply, only operational when the dams reach a certain threshold. Considering the penalty structure zones used for the main reservoirs, four threshold or 'trigger' levels were selected to give a representative spread. Based on the level-capacity curves of the major reservoirs, the four trigger levels were estimated to be equivalent to 90 percent, 70 percent, 45 percent and 15 percent of the capacity of the main system reservoirs respectively. The penalty structure of the desalination channel was set just higher than the penalty associated with the reservoir trigger level in question, thus ensuring that the seawater desalination plant would only kick in when the reservoir storage reached that level.

For each of the desalination operational scenarios B to D, three different seawater desalination plant capacities were tested based on information extracted from the latest WCWSS SSC Progress Report (Department of Water Affairs, 2013a:13), i.e. 150 Mℓ/day (54.8 million m^3/a), 300 Mℓ/day (109.6 million m^3/a) and 450 Mℓ/day (164.4 million m^3/a). In addition, some of the scenarios were tested for an unlimited plant capacity in order to determine the maximum capacity required in order to meet the projected future demands of the system.

5.1.4 WRYM setup

Table 5.1 shows the six channel penalty structures which were developed to represent the different scenarios as discussed in Section 5.1.3.

Table 5.1: Channel penalty structures in WRYM for different scenarios

Channel Penalty Structures				
Penalty Structure Number	**Description**	**Penalty for Arc 1**	**Penalty for Arc 2**	**Trigger Level**
58	Scenario B	0	50000	Constant supply
59	Scenario C	0	1800	Constant supply if dams not spilling
60	Scenario D1	25	0	DRDWN4: System storage at 90%
61	Scenario D2	55	0	DRDWN7: System storage at 70%
62	Scenario D3	85	0	DRDWNA: System storage at 45%
63	Scenario D4	115	0	DRDWNE: System storage at 15%

For all scenarios, a multi-purpose min-max channel was used to represent the seawater desalination plant supply. The input parameters are summarised in **Table 5.2**. The desalination supply channel feeds into Node 38 in the WRYM which is the node upstream of the master control channel. This allows the urban demands only to be supplied from the seawater desalination plant. The flow constraint of 1.736 m^3/s, as shown in **Table 5.2**, is for a 150 Mℓ/day plant. For a 300 Mℓ/day plant the corresponding constraint is 3.472 m^3/s, and for a 450 Mℓ/day plant it is 5.208 m^3/s.

Table 5.2: Input parameters for desalination supply channel in WRYM

Desalination Supply Channel					
Channel Properties	Channel Number	**598**		**598**	
	Channel Name	Desalination Supply		Desalination Supply	
	Upstream Node	0		0	
	Downstream Node	38		38	
	Channel Penalty Structure	58/59		60/61/62/63	
	Arcs	2		2	
Min-max Flow Feature	Monthly maximum flow constraints	Flow Constr. (m^3/s)		Flow Constr. (m^3/s)	
		Arc 1	Arc 2	Arc 1	Arc 2
	OCT to SEP	1.736	1.736	1.736	0.000

Historic analyses were run for a total of 77 years (1928 to 2004) for a range of target drafts in order to determine the historic firm yield, and to calculate the long-term assurance of supply, long-term stochastic analyses were performed for the full 77 years.

As discussed in Section 2.1.3, Basson *et al.* (1994:34) suggest that reasonable results in a long-term analysis can be obtained with at least 40 stochastic sequences. Given the complexity of the WCWSS, it would not be preferable to analyse too many more than this, as the computational time can become a major constraint. As a result, 51 sequences were used for the long-term analyses, which ties up with other WCWSS planning studies. Note that the number of sequences is always selected as an odd number to facilitate statistical analysis of the results.

Short-term stochastic analyses were also performed in the WRYM in order to create short-term yield curves for use in the WRPM. For short-term analyses, Basson *et al.* (1994:34) suggests that at least five times the number of sequences used in a long-term analysis are required, hence at least 255 in this case. The most recent WRPM studies carried out by Aurecon use 401 sequences. Given that the computational time is much shorter for a short-term analysis, 401 sequences seemed reasonable. Three different assumed system starting storage levels were analysed (i.e. 100 percent, 60 percent and 20 percent). Based on previous studies, the most conservative analysis durations for the short-term curves at these starting levels are six, three and two years respectively.

5.1.5 WRPM setup

The WRPM setup was modified in the same way as for the WRYM, with a two-arc min-max channel representing the seawater desalination plant. The only differences were the channel number of the desalination supply (channel 401), the penalty structure numbers (53 to 58) and that the channel feeds into node 36, which is just upstream of the main demand channel. The original setup included six reservoir starting volumes; however these were reduced to three, as discussed in Section 5.1.4, in order to reduce the computational time for each scenario.

The base model setup had only 10 years of growth information, starting in 2013, for the 12 master control channels. As part of this research, the growth factors were extended to cover a period of 20 years, based on the growth calculation spreadsheets prepared by Aurecon as part of the DWS study into the development of integrated annual operating rules for the WCWSS (mentioned in Section 4.5). The irrigation demands were kept constant whilst the urban demands were projected at 3 percent per annum. The growth scenario further assumed that only 80 percent of the anticipated WC/WDM savings for the City of Cape Town would be achieved. The projected demands for each master control channel are shown in **Figure 5.1**.

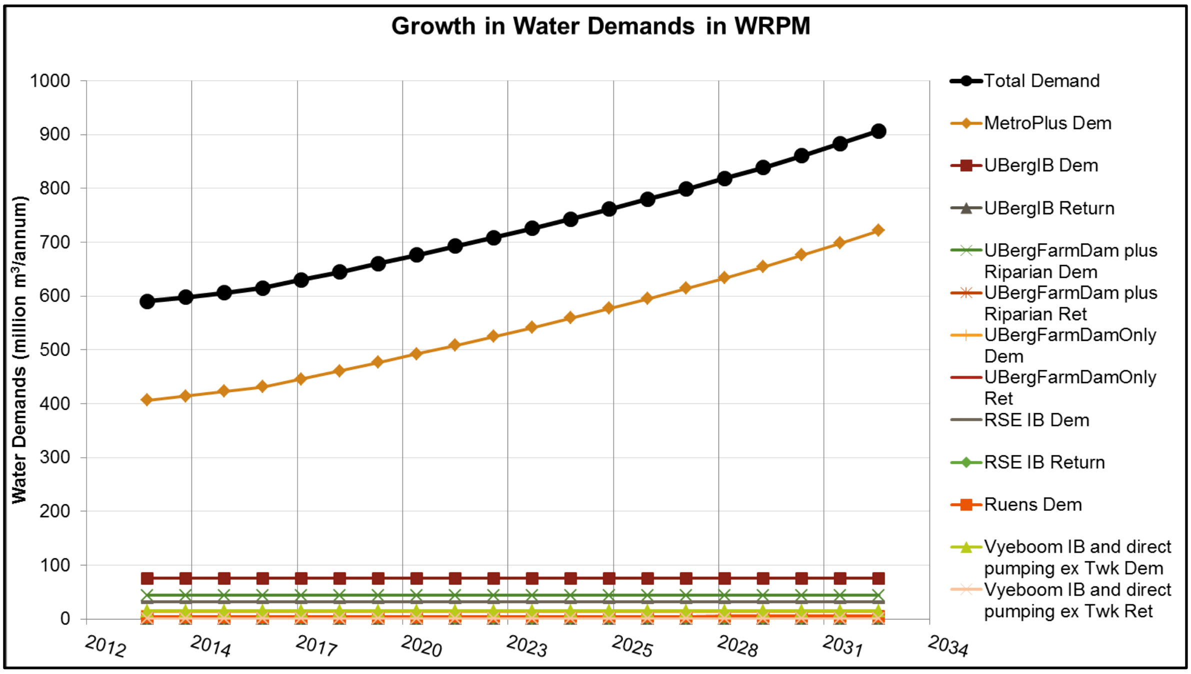

Figure 5.1: Projected growth in water demands for the WCWSS as used in the WRPM

For each operating scenario and seawater desalination plant capacity, the WRPM was run for a period of 20 years starting in 2013. A total of 401 stochastic sequences were analysed in order to match the number of sequences used in deriving the short term yield curves as discussed in Section 5.1.4.

5.1.6 Model outputs

For each of the operating scenarios and seawater desalination plant capacities discussed in Section 5.1.3, the following key outputs were extracted from the WRYM analyses:

- Historic firm yield of the system based on the historic analyses
- 1 in 50, 1 in 100 and 1 in 200 year yields of the system based on the long-term stochastic analyses
- Short-term characteristic curves with six target drafts each based on the short-term stochastic analyses

For the WRPM analyses, the following outputs were extracted:

- Box-and-whisker data of the annual water supply from the desalination plant
- Box-and-whisker plots of the level of curtailment imposed

- Box-and-whisker plots of the total system storage
- Box-and-whisker plots of the total supply versus demand

The results are presented and discussed in detail in Section 6.

5.2 Seawater Desalination Costing

In addition to comparing the yields and supply from the WCWSS with the introduction of seawater desalination, it was considered worthwhile to compare the costs associated with the operating strategies, and to assess the impact of stochastic variability in supply on the seawater desalination plant costs. This required a simple and robust methodology for estimating first-order capital and operating costs for a seawater desalination plant in South Africa.

5.2.1 Analysis of known costs

Table 3.4 in Section 3.4.1 gives the capital costs of recently constructed seawater desalination plants in Australia. These costs were converted to costs in R per m^3/day of desalination plant capacity and plotted against the plant capacities as shown in **Figure 5.2**. In addition, some seawater desalination plant capital costs for SWRO plants in Israel, Algeria and Spain were extracted from Ghaffour *et al.* (2013) (mentioned in Section 3.3.2) and included on the graph. The Mossel Bay desalination plant, being the only large seawater desalination plant in South Africa, was also plotted.

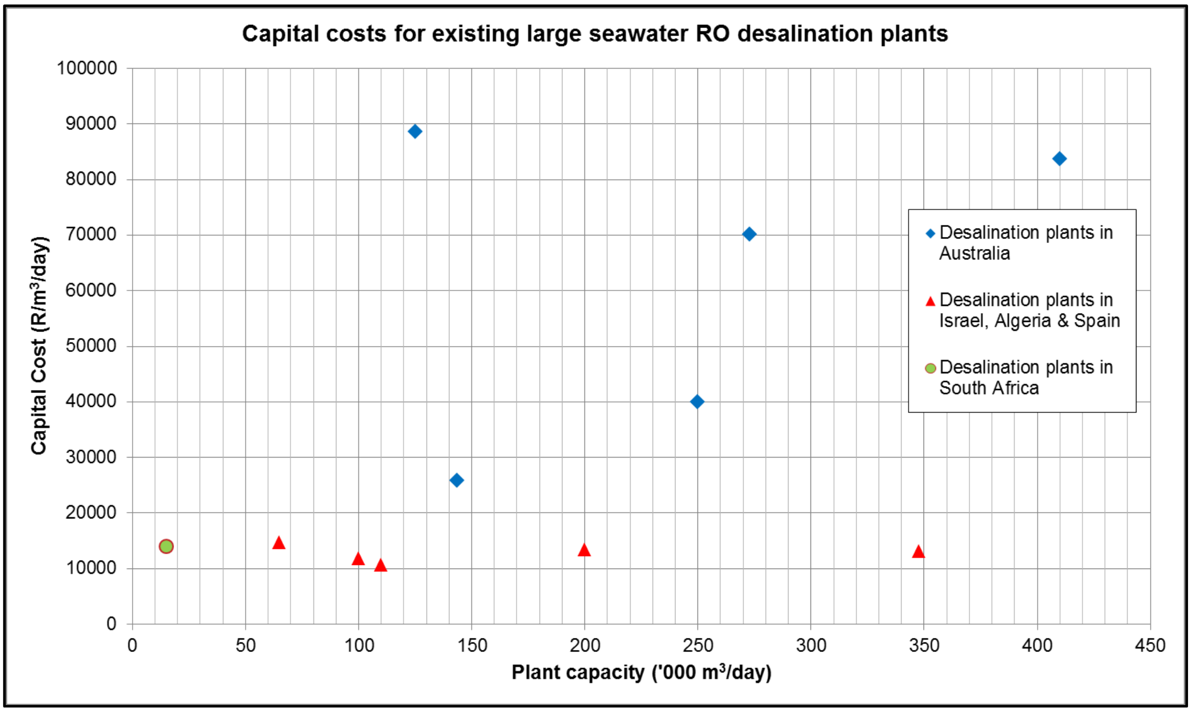

Figure 5.2: Seawater desalination plant capital costs in Australia, Israel, Algeria, Spain and South Africa

Attempts to derive any kind of relationship from the values presented in **Figure 5.2** were unsuccessful given the large scatter; however it was possible to draw a number of conclusions.

Firstly, it is clear that the Australian seawater desalination plants were significantly more expensive than recent plants in other parts of the world and the costs are highly variable, ranging from R25 800 to as high as R88 600 / m^{3}day. This is most likely a result of the emergency nature of the schemes, which invariably pushes up costs.

Secondly, when looking at plants from the rest of the world (Israel, Spain and Algeria), the costs are fairly similar across the range of capacities, varying from R10 700 to R14 600/m^{3}day. At these rates, a 100 Mℓ/day plant would cost in the order of R1 290 million which seems reasonable when comparing the values derived from the various methodologies in Section 3.3.

It should further be noted that one of the problems with seawater desalination plant costs that are quoted in articles and literature is that there is rarely any clarification of what the quoted costs include. For example, the inclusion or exclusion of marine works for sea intakes and outfalls, pre-treatment facilities or post-treatment facilities from a quoted cost figure would make a major difference, particularly for large-scale sweater desalination plants. As a result, any comparison of known costs should be treated with some caution. The analysis of existing seawater desalination plant costs therefore provided a useful sense-check for any calculated costs but was not useful as a method for providing realistic cost estimates.

5.2.2 Selected costing methodology

Based on a review of the available costing methodologies as presented in Section 3.3 it was decided that the South African approach followed by Du Plessis *et al.* (2006) would be the most appropriate based on the following reasons:

- It is a South African-based methodology and is therefore relevant to the context of this Study.
- The costs are related to membrane area which in turn is a function of the plant capacity, seawater characteristics and number of passes. This is considered a better base for cost estimation than using the plant capacity alone.

- The method is simple to apply and will therefore not complicate the overall modelling approach.
- For the test plant of 100 Mℓ/day, the costs compare well to those calculated using the other methods and as a first-order costing tool it therefore seems reasonable and realistic.

5.2.3 Costing approach

The selected costing methodology of Du Plessis *et al.* (2006), described in Section 3.3.1, can be used for estimating capital and operating costs, but the methodology as is only gives unit costs of water based on average plant capacity. In order to provide meaningful costs, which can be used to compare the operational scenarios, it was considered more appropriate to calculate URVs.

As discussed in Section 3.2.1, Van Niekerk and Du Plessis (2013a) adopted the approach of using the actual volume of water supplied based on a stochastic analysis in the WRPM to calculate the URVs of inter-basin transfers (based on Equation 3.1). This approach can be easily applied to a seawater desalination plant, using the annual volumes of water supplied from the seawater desalination plant, as extracted from the WRPM analyses, to calculate costs.

The following adaptations were made to the costing approach of Du Plessis *et al.* (2006) in order to provide URVs for the modelled scenarios:

- Capital costs were determined based on the desalination plant capacity and escalated to the start date of the analysis. A construction period of two years was assumed, and the costs were spread equally over the two years.
- The method proposed by Du Plessis *et al.* (2006) was used for calculating the total energy consumption of the plant at maximum plant capacity. The annual energy costs were then calculated based on the annual volume of water supplied, as derived from the WRPM analyses. An electricity cost of R1/kWh was used.
- It was assumed that the membranes would be replaced every six years, provided that the plant was operating at more than 75 percent of its capacity on average. For a plant operating at between 50 percent and 75 percent of its capacity, this was increased to eight years, and ten years was used for a plant operating at between 25 percent and 50 percent of its capacity. Operating at below 25 percent of its capacity, the membrane replacement frequency was assumed to be every 12 years.

- Using the methodology of Du Plessis *et al.* (2006) maintenance and labour costs were calculated as a function of the capital costs. Similar to the membrane replacement costs, these were then adjusted depending on the annual output of the desalination plant, on the assumption that if the plant is operating at a lower output, there will be a reduction in operating staff requirements, wear-and-tear on equipment and other maintenance requirements. With the plant operating below 25 percent, 50 percent or 75 percent of its capacity, the maintenance costs were reduced by 65 percent, 40 percent and 15 percent respectively.
- Chemical costs were calculated by multiplying Du Plessis *et al.* (2006)'s specific cost of chemicals of R0.75/m^3 by the actual desalination plant supply per annum.
- The capital, operating and maintenance costs were summed per annum, and the NPV was determined over the analysis period for discount rates of 6, 8 and 10 percent using Equation 2.3 (Section 2.1.4).
- The NPV of the water supplied from the desalination plant was calculated for the analysis period using Equation 2.4 (Section 2.1.4).
- The URV was calculated for each scenario using Equation 2.2 (Section 2.1.4).

As in the approach of Van Niekerk and Du Plessis (2013a), cognisance was taken of the stochastic variation in the supply and the resulting stochastic variation of the URVs. URVs were therefore calculated for the typical box-and-whisker plot boundary values (refer to **Figure 4.1** in Section 4.3). The components of the URV which vary stochastically include the energy consumption, chemical costs and total volume of water supplied.

Note that all URVs calculated as part of this research were for the desalination plant and its associated infrastructure only and not for the WCWSS as a whole.

The results of the cost analyses are presented and discussed in Section 6.4.

6 PRESENTATION AND DISCUSSION OF RESULTS

As discussed in Section 5.1, the selected seawater desalination plant operating scenarios were run in the WRYM and WRPM for the different plant capacities, and the yield and supply from the plant extracted for each run. The effects of the seawater desalination plant on the historic and stochastic system yields are presented in Section 6.1. The annual supply from the seawater desalination plant for each operating scenario is provided in Section 6.2. Section 6.3 discusses the impact of the seawater desalination plant on future curtailment requirements.

As was described in Section 5.2, the supply from the seawater desalination plant was used to calculate costs (URVs) for the different operating scenarios and capacities. The URVs are presented and compared in Section 6.4.

For ease of reference, the operating scenarios as presented in Section 5.1.3 can be summarised as follows:

- Scenario A: Base scenario with no seawater desalination plant
- Scenario B: Seawater desalination plant always operational
- Scenario C: Seawater desalination plant only operational if the reservoirs are not spilling
- Scenario D: Seawater desalination plant only operational when the total system storage reaches a trigger level of 15 percent, 45 percent, 70 percent or 90 percent

6.1 System Yield

6.1.1 Base scenario

For Scenario A, the base scenario with no seawater desalination plant, the historic firm yield of the system was calculated as 530 million m^3/a. Based on the stochastic analysis, which produced the yield-reliability curve provided in **Figure 6.1**, this corresponds to an assurance of supply of approximately 70 percent or 1 in 215 years. The 1 in 50, 1 in 100 and 1 in 200 year yields of the system were read off from the yield-reliability curve as 580 million m^3/a, 553 million m^3/a and 532 million m^3/a respectively. The results for Scenarios B to D were compared to the base scenario results in order to determine the increase in yield resulting from the addition of a desalination plant to the system.

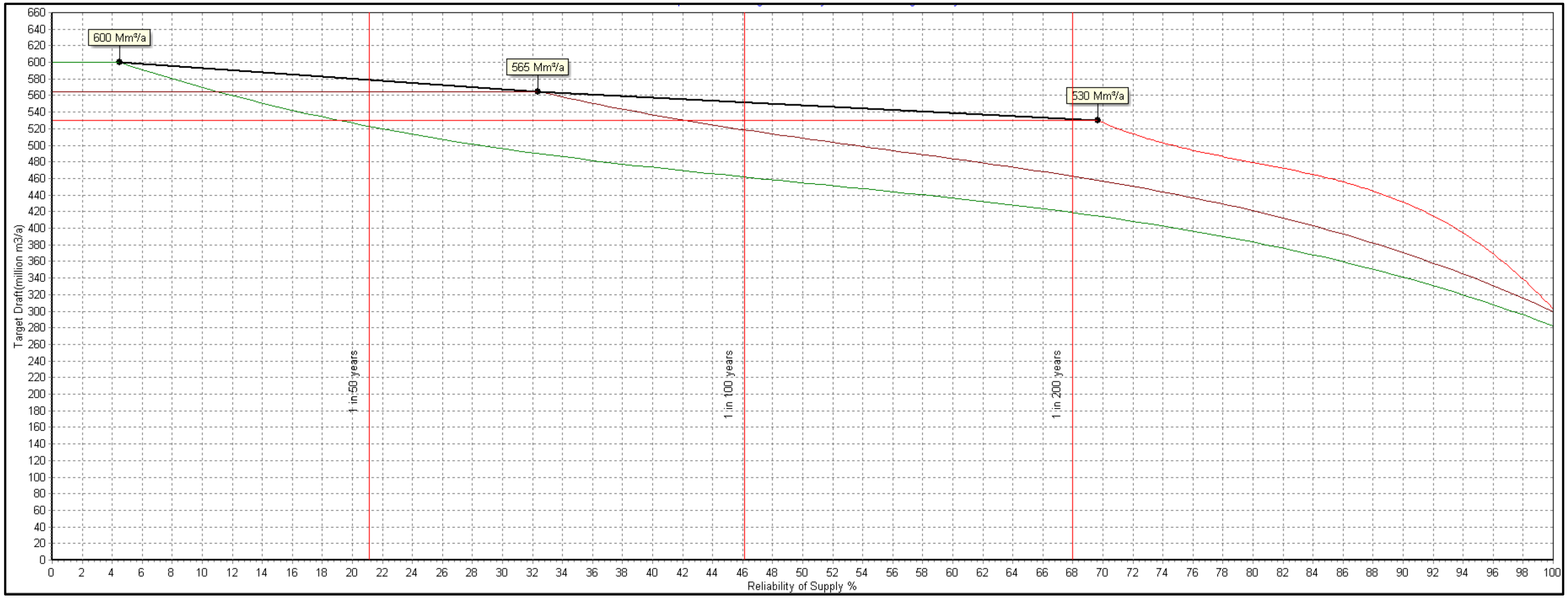

Figure 6.1: **Yield-reliability curve for Scenario A**

6.1.2 Historic firm yield

Figure 6.2 shows the increase in the historic firm yield compared to the base scenario for all the seawater desalination plant operating scenarios that were analysed. For Scenario B, with the desalination plant operational 100 percent of the time, the increase in historic firm yield is very close to the desalination plant capacity for all three capacities (i.e. 53 million m^3/a, 106 million m^3/a and 159 million m^3/a). The small difference (3 percent) between the increase in yield and the desalination plant capacity is possibly due to increased evaporation due to the dams being fuller on average over the analysis.

For Scenario C, with the desalination plant operational only when the dams are not spilling, the results are almost identical, which suggests that reducing the desalination plant output when the dams are spilling has little impact on the overall system yield. Note that Scenario B and C in **Figure 6.2** plot on almost exactly the same line.

Considering Scenario D, as the reservoir trigger level at which the desalination plant kicks in is lowered, the increase in historic firm yield decreases substantially, particularly when dropping below the 50 percent trigger level mark. This trend is more clearly viewed in a plot of the increase in historic firm yield of the system against the desalination plant trigger level as presented in **Figure 6.3**.

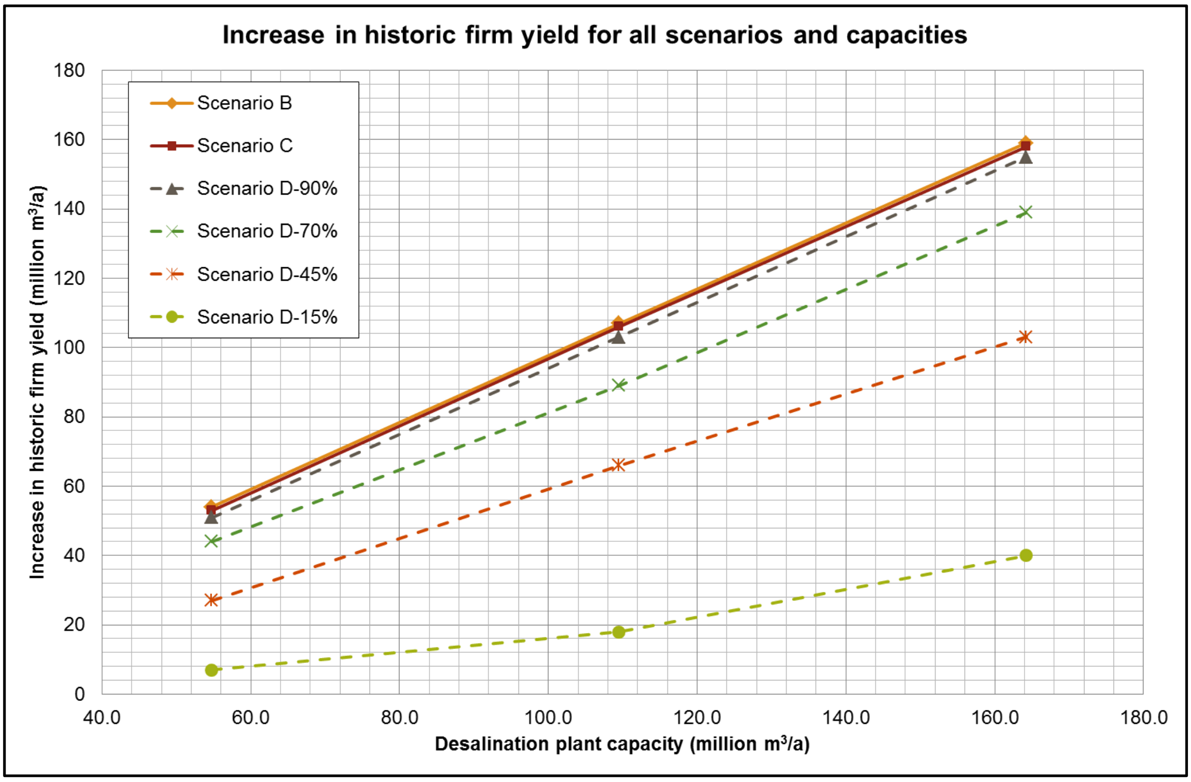

Figure 6.2: Increase in historic firm yield for increasing desalination plant capacities

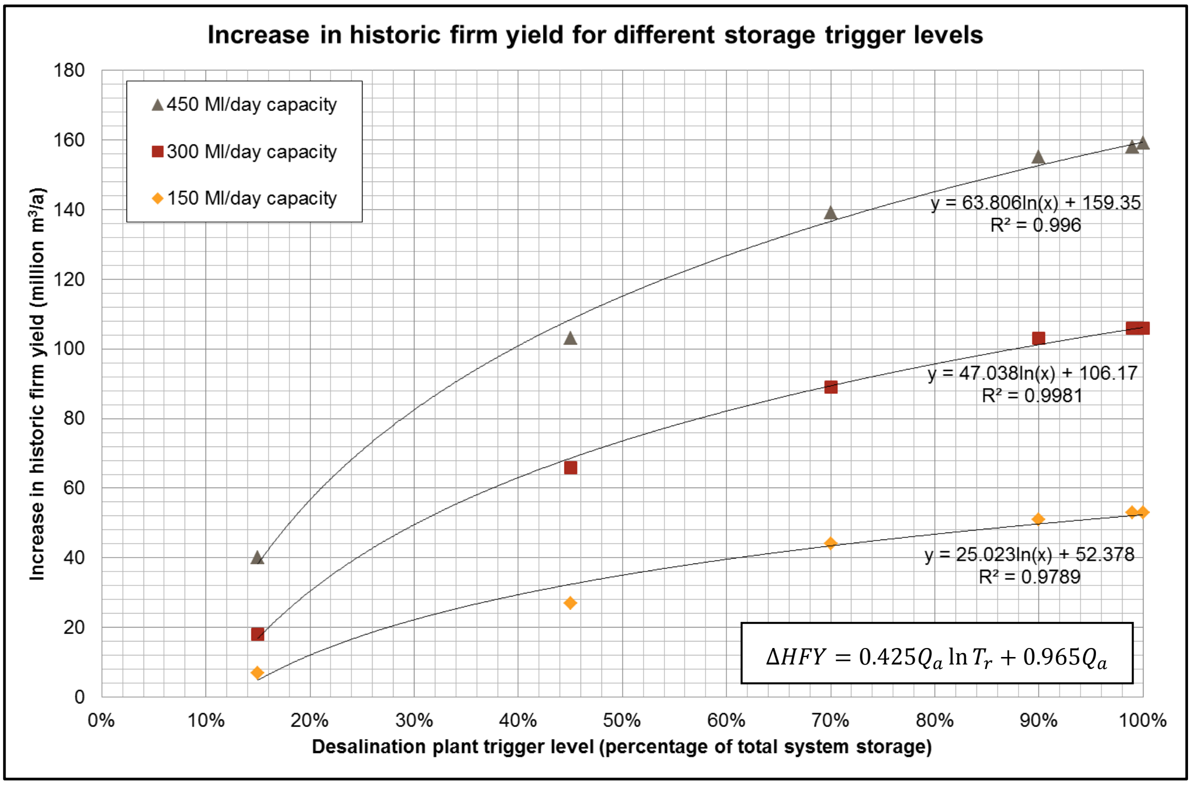

Figure 6.3: **Increase in historic firm yield versus reservoir trigger level**

The results show a clear logarithmic pattern. Logarithmic trend lines plotted for each desalination plant capacity, as shown on **Figure 6.3**, have a very good correlation with the modelled data particularly for the larger desalination plant capacities. The coefficients of the logarithmic equations for each curve appeared to be related to the desalination plant capacity. The coefficients were therefore normalised based on the plant capacity and averaged to provide the generic equation as shown in Equation 6.1. This equation could be used to estimate the increase in historic firm yield of the WCWSS for any seawater desalination plant capacity and reservoir trigger level.

$$\Delta HFY = 0.425Q_a \ln T_r + 0.965Q_a \qquad (6.1)$$

Where:

ΔHFY	Increase in historic firm yield of WCWSS	[million m^3/a]
Q_a	Average annual desalination plant capacity	[million m^3/a]
T_r	Desalination plant trigger level as a percentage of the system storage in the major reservoirs	[%]

The logarithmic shape suggests that increasing the dam trigger level from say 20 percent to 30 percent, will have a significantly greater impact on the historic firm yield than increasing the trigger level from say 80 percent to 90 percent. It also shows that there is no ‘turning point’ or optimal trigger level. In other words, the maximum increase in historic firm yield is achieved when the seawater desalination plant is always operational.

6.1.3 Stochastic results

Figure 6.4 shows the increase in 1 in 50 year yield from the base scenario based on the results from the stochastic analyses. For Scenario B and C, the increase in 1 in 50 year yield is similar to the increase in historic firm yield as presented in **Figure 6.3**. For Scenario D, particularly at the lower trigger levels, the increase in 1 in 50 year yield is generally larger than the increase in historic firm yield.

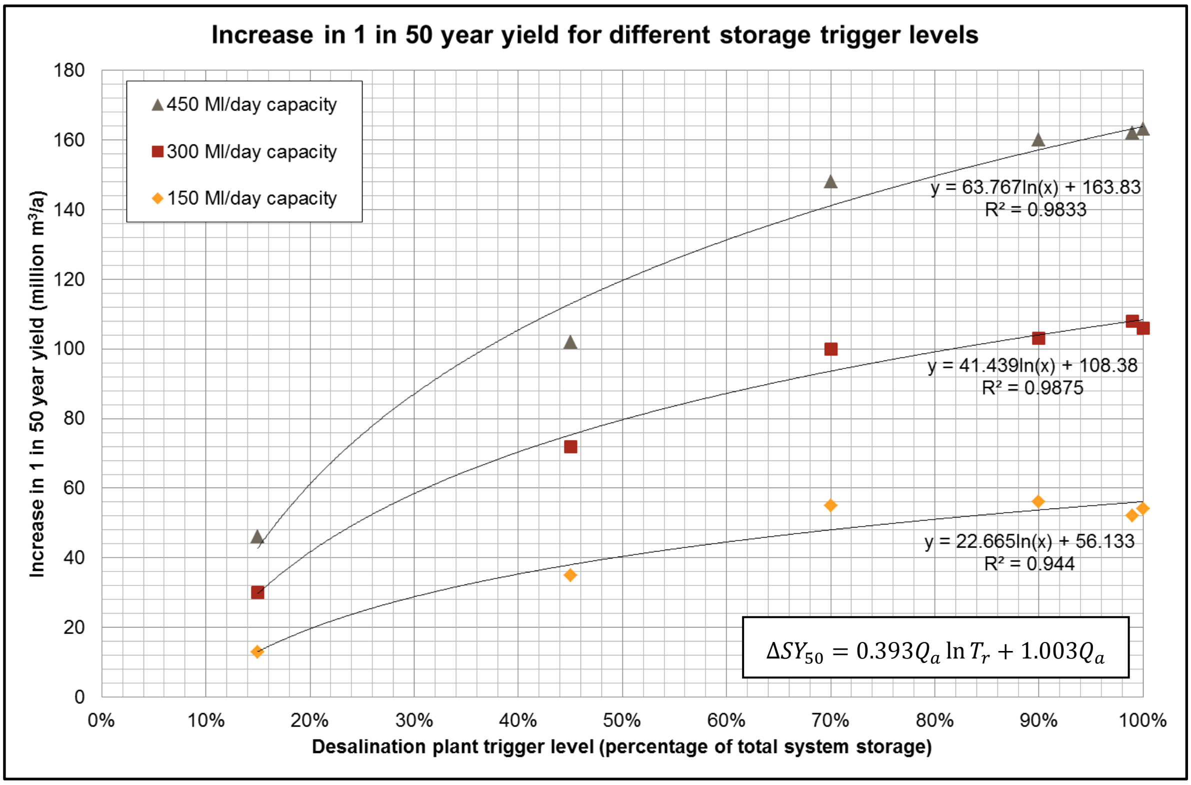

Figure 6.4: Increase in 1 in 50 year yield versus reservoir trigger level

The trends across the trigger levels and capacities are very similar to the historic firm yield results, showing a clear logarithmic pattern, and logarithmic trend lines fitted to the data show a good correlation. As for the historic firm yield results, the logarithmic equation coefficients were normalised based on the desalination plant capacity and averaged, giving the generic Equation 6.2 for calculating the 1 in 50 year yield for any desalination plant capacity and trigger level.

$$\Delta SY_{50} = 0.393Q_a \ln T_r + 1.003Q_a \tag{6.2}$$

Where:

ΔSY_{50}	Increase in 1 in 50 year yield of WCWSS	[million m^3/a]

Similar plots were prepared for the 1 in 100 year and 1 in 200 year yields and are presented in **Appendix B**. The results were very similar across all return periods, suggesting that the introduction of the desalination plant merely shifts the yield-reliability curve up without changing its shape. A summary of the equations derived for estimating the increase in yield for the three return periods is presented in **Table 6.1**.

Table 6.1: Equations derived for estimating the increase in yield of the WCWSS for any desalination plant capacity and reservoir trigger level

Return period of yield (years)	Equation for estimating increase in yield
1 in 50	$\Delta SY_{50} = 0.393Q_a \ln T_r + 1.003Q_a$
1 in 100	$\Delta SY_{100} = 0.384Q_a \ln T_r + 0.996Q_a$
1 in 200	$\Delta SY_{200} = 0.387Q_a \ln T_r + 0.948Q_a$

6.2 Seawater Desalination Plant Supply

6.2.1 Historic analysis

In order to determine how effectively the seawater desalination plant will be used for the different operating scenarios and reservoir trigger levels (Scenario D), the average annual supply from the desalination plant over the 77 year historical analysis (in WRYM) expressed as a percentage of its capacity was plotted against the reservoir trigger levels, as shown in **Figure 6.5**. The results for Scenario C are also shown, plotted against a 99 percent trigger level. For Scenario B, the penalty structure was specifically set so that the desalination plant will operate 100 percent of the time regardless of the system conditions, hence these results were not included on the curve.

The relationship between the supply from the desalination plant (as a percentage of its capacity) and the desalination plant trigger level best fits a power curve distribution. Considering Scenario D, for the lowest trigger level of 15 percent, the plant only operates at about 1 percent of its capacity on average, increasing to operating at 77 percent of its capacity on average for the 90 percent trigger level. In general, it would appear that the average supply from the desalination plant as a percentage of capacity increases with an increase in desalination plant capacity. However, considering Scenario C, this trend is reversed.

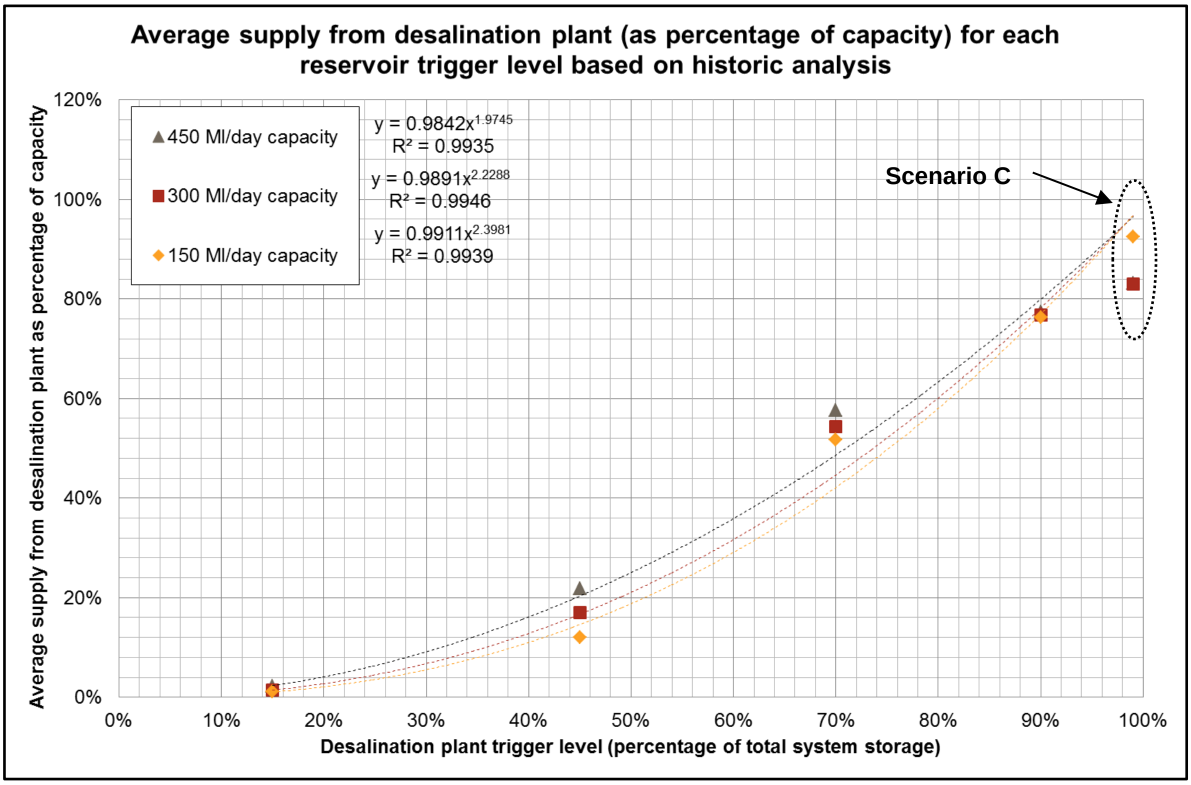

Figure 6.5: Average supply from desalination plant versus reservoir level trigger levels based on historic analysis

6.2.2 Future analysis

A much more meaningful analysis is to consider the percentage supply from the desalination plant for the WRPM analyses which are based on actual projected future demands. The supply from the desalination plant is important as it feeds directly into the calculation of URVs, as discussed in Section 6.4.

Figure 6.6 shows the supply from the desalination plant as a percentage of its capacity in each year (2013 to 2032), for all scenarios, based on a 150 Mℓ/day plant. The values shown are based on the median annual supply of the 401 stochastic sequences analysed.

For Scenario B (as expected) the desalination plant would always be 100 percent operational. For Scenario C the desalination plant would start out at 90 percent of its capacity, increasing to 100 percent by 2015. Lowering the trigger level to 90 percent (Scenario D), the desalination plant would start at 60 percent of its capacity, increasing to a maximum of 80 percent. For a 70 percent trigger level, the plant would be between 30 and 50 percent operational. For the lower trigger levels of 45 percent and 15 percent, the desalination plant would not be operational for the entire analysis period.

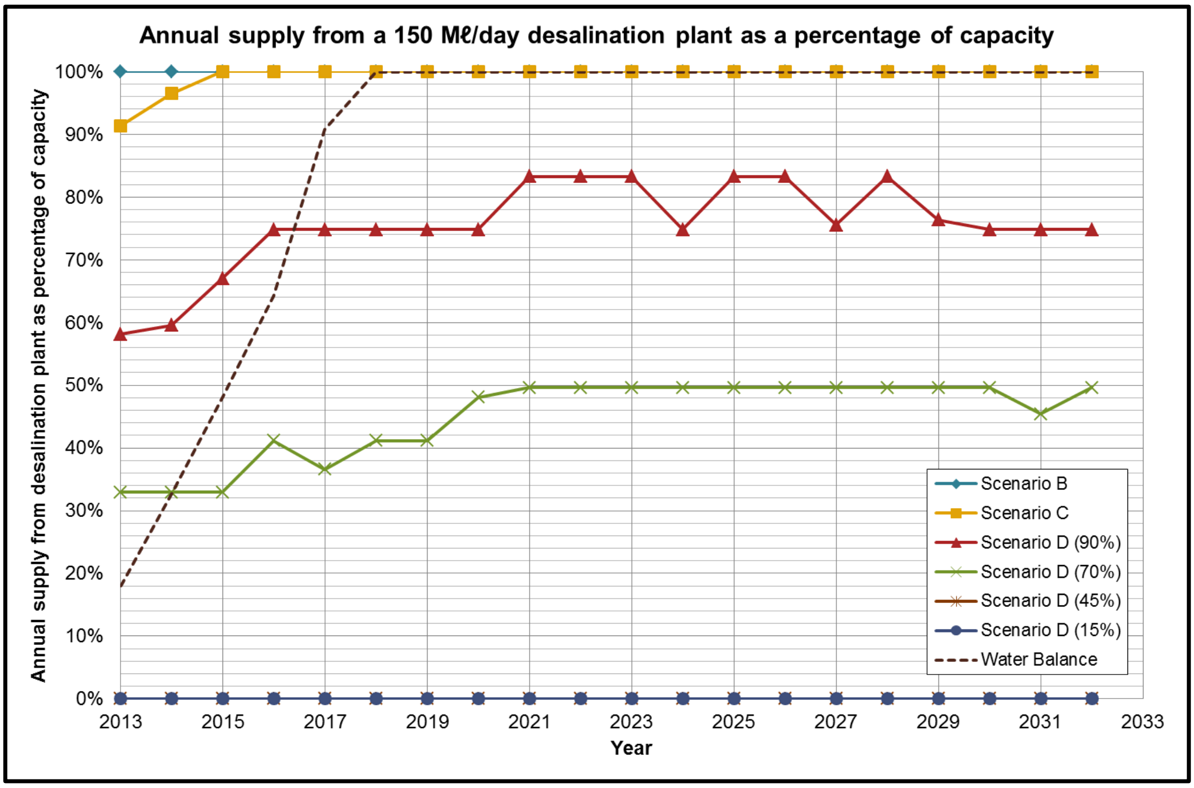

Figure 6.6: Annual supply from a 150 Mℓ/day desalination plant as a percentage of capacity

Similar graphs for the 300 Mℓ/day and 450 Mℓ/day plants are presented **in Appendix C**. The results show that, in general, the supply from the desalination as a percentage of capacity decreases with an increase in desalination plant capacity.

The results presented in **Figure 6.6** and in **Appendix C** are based on median values of the 401 stochastic sequences analysed, and were selected as representative based on the assumption that these results would be used in the calculation of URVs. In order to illustrate the possible stochastic variability in supply from the desalination plant, the 5th and 95th percentile annual values were plotted for Scenario C and for Scenario D with a 70 percent trigger level, as shown in **Figure 6.7**.

For Scenario C the maximum stochastic range varies between -28% and +9% from the median, decreasing to zero by the end of the analysis period (2028), when even the wetter sequences required the desalination plant to be fully operational. For Scenario D with a 70 percent trigger level, the stochastic range is much greater, varying between -25% and +50% from the median, and hence making the choice of what values to use as input into the URV calculations more critical. This is discussed further in Section 6.4.

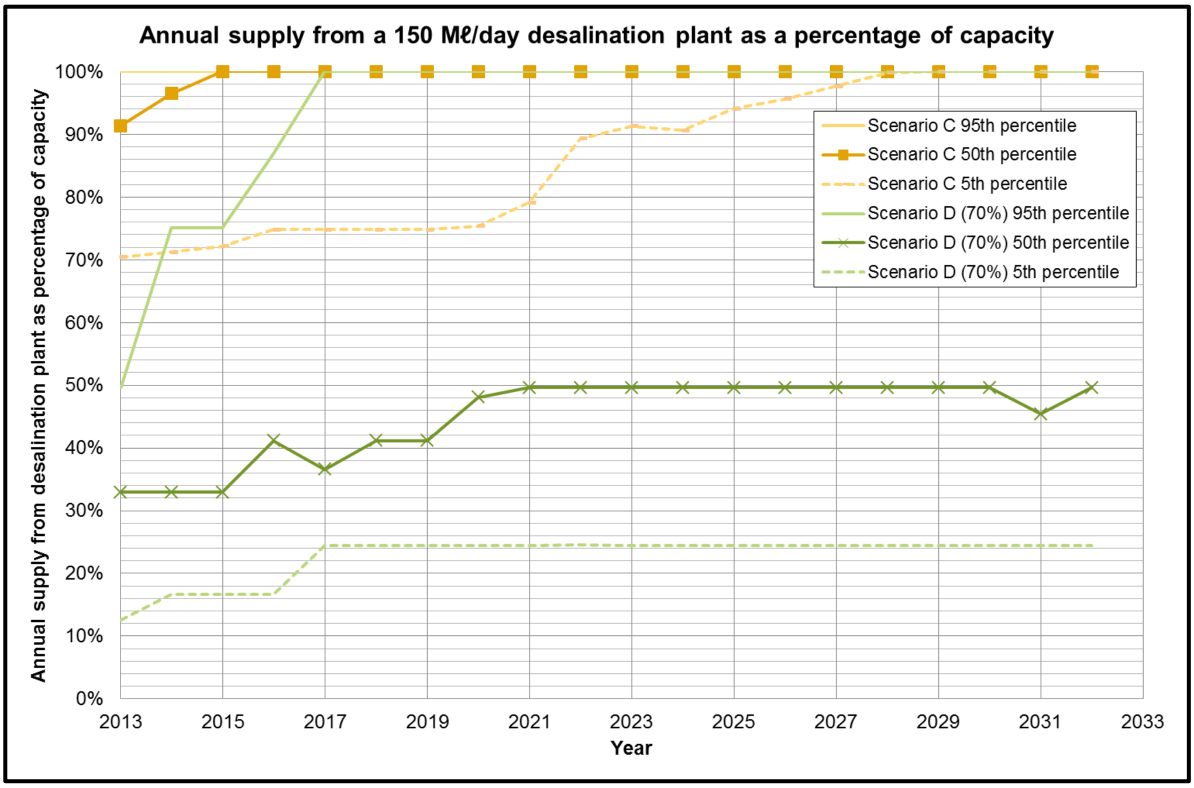

Figure 6.7: Stochastic variation in annual supply from a 150 Mℓ/day desalination plant

6.2.3 Comparison with traditional approach

As was discussed in Section 3.2.1, the research of Van Niekerk and Du Plessis (2013b:543) showed that the annual supply from a new water supply source (in their case an inter-basin transfer scheme) is generally estimated by calculating the deficit between the projected annual demands of the system and the existing system capacity, taking any limitations in the capacity of the new water supply source into consideration. Applying this principal, using the demand projections as presented in **Figure 5.1** and the 1 in 50 year yield of 580 million m^3/a as presented in Section 6.1.3, the annual system deficit was calculated. Assuming that a new desalination plant would have to meet this deficit, for the selected capacities of 150 Mℓ/day, 300 Mℓ/day or 450 Mℓ/day the annual supply as a percentage of the desalination plant capacity was calculated, as shown in **Figure 6.8.**

Based on the traditional approach, a 150 Mℓ/day desalination plant would operate at 18 percent of its capacity in 2013, increasing to 100 percent by 2018. Comparing these values to the WRPM results in **Figure 6.6**, the traditional approach and the WRPM approach provide vastly different values in terms of supply from the desalination plant, particularly for the lower reservoir trigger levels.

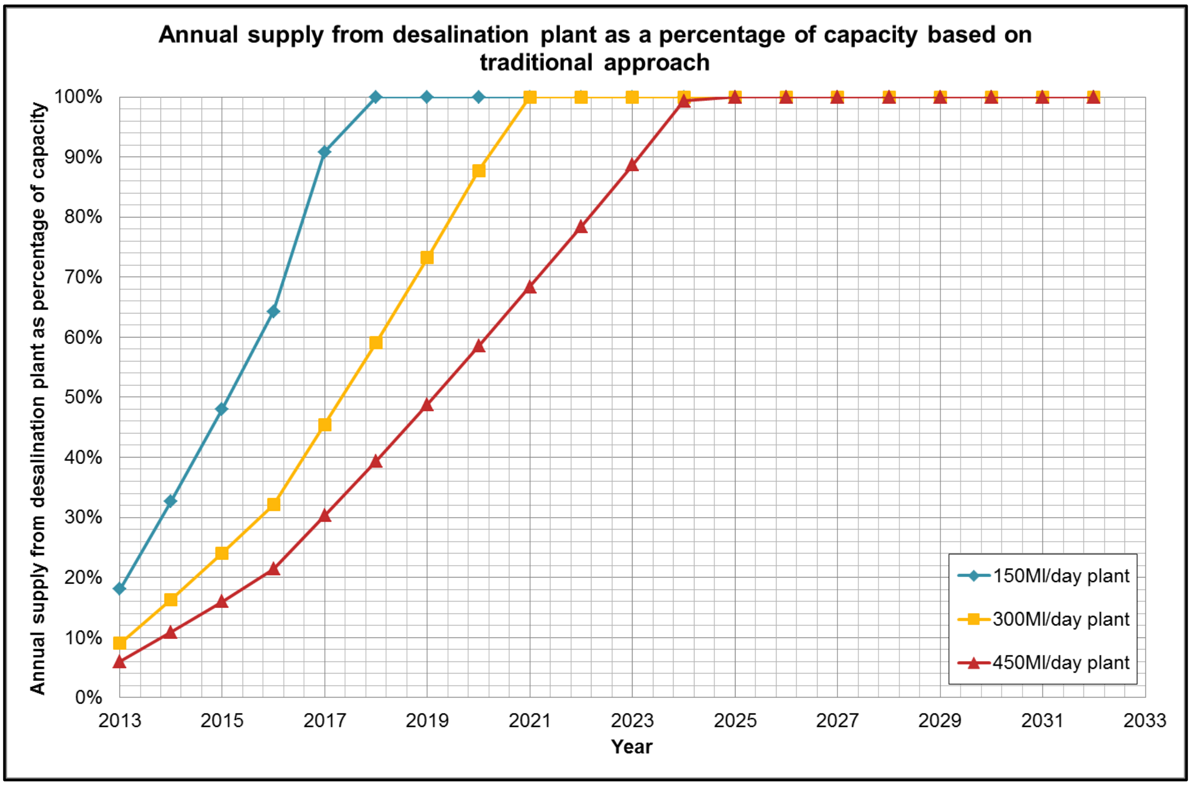

Figure 6.8: Annual supply from a desalination plant based on the traditional approach

Further implications of the differences between the traditional and WRPM methods for calculating the annual supply from the desalination plant are discussed in Section 6.4.3 which provides a comparison between the calculated URVs for the two approaches.

6.3 Planning for Future Interventions

6.3.1 Traditional approach

The typical approach that would be followed in determining when a future intervention is required in a water supply system would be a water balance of the demands on and the supply from the system. Examples of such water balances prepared as part of previous planning studies in the WCWSS are presented in **Figure 2.9** (Section 2.3.1) and **Figure 2.10** (Section 2.4.3). The latest SSC report indicated that a new intervention would be required by 2022 in order to meet the projected demands on the system.

The water demands that were projected as part of this study, based on the recent work by Aurecon (presented in **Figure 5.1** in Section 5.1.5), were plotted along with the calculated 1 in 50 year base yield of 580 million m^3/a (Scenario A) and the 1 in 50 year yields with a desalination plant as derived from the WRYM analysis (Scenario B). The demands and yields are shown on **Figure 6.9.**

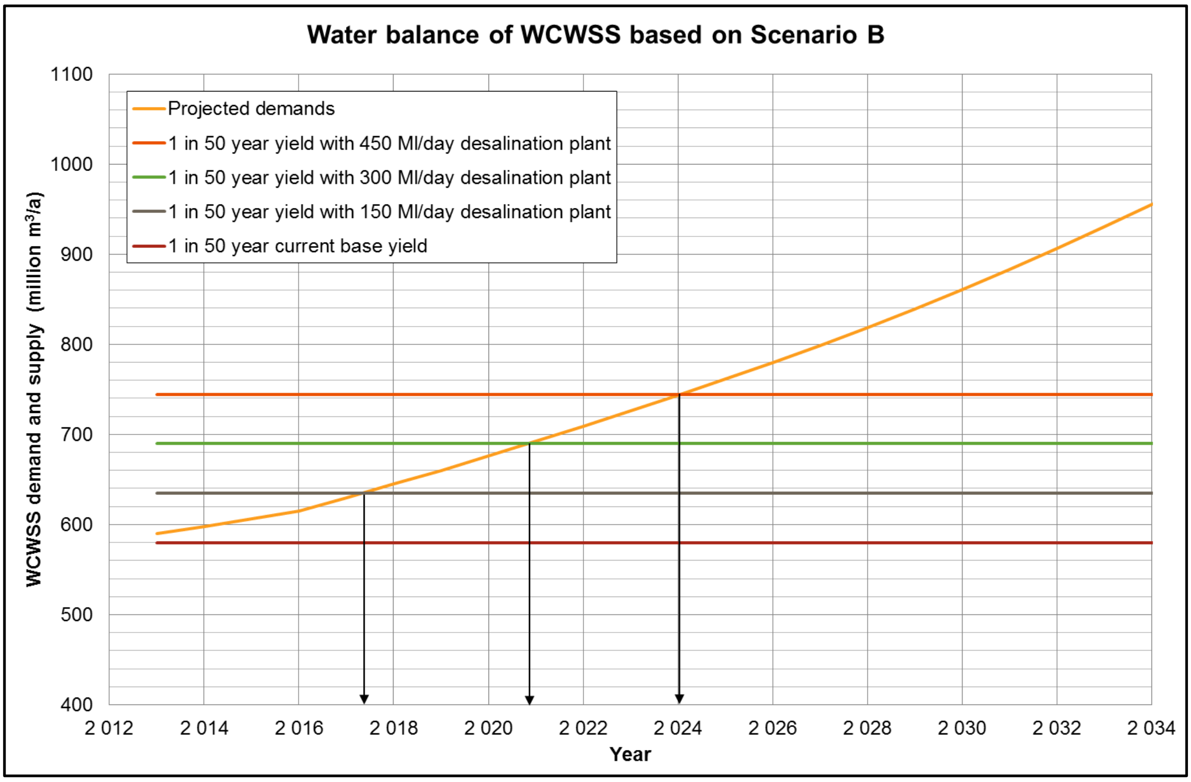

Figure 6.9: Projected demand and supply of WCWSS with and without a desalination plant

Based on **Figure 6.9**, it would appear that the current demands exceed the 1 in 50 year yield. This does not tie up with the water balance for the WCWSS reports that have been studied, most likely due to updated water demands. A 150 Mℓ/day desalination plant would meet the system demands until 2017, a 300 Mℓ/day plant would meet the demands until 2021 and a 450 Mℓ/day plant would meet the requirements until 2024.

6.3.2 Base scenario using the WRPM

In the WCWSS analysis in the WRPM and part of the recent study on the development of integrated annual and real time operating rules for the WCWSS, revised water restriction levels were developed. The scenario which Aurecon found to be the most realistic is provided in **Table 6.2**, and was used in the WRPM analyses. As shown, Level 1 curtailments cannot be applied more than once in every 20 years, Level 2 curtailments no more than once in every 100 years and Level 3 curtailments no more than once in every 200 years. Stricter curtailments are applied to agricultural demands than domestic demands.

Table 6.2: Level of restrictions used in WRPM analyses

Level of curtailment	Acceptable frequency of restrictions	Restricted water demand as a percentage of normal demand	
		Domestic	Agricultural
0	1 in 10 years	100%	100%
1	1 in 20 years	93%	75%
2	1 in 100 years	85%	50%
3	1 in 200 years	71%	20%

Figure 6.10 shows a box-and-whisker plot of the subsystem curtailment for the base scenario (Scenario A) from 2013 to 2032 as derived from the WRPM analysis. The critical lines or whiskers which correspond to 1 in 20 year, 1 in 100 year and 1 in 200 year curtailment levels are labelled. Reference can also be made to the typical box-and-whisker plot provided in **Figure 4.1** of Section 4.3.3.

Reading from **Figure 6.10**, curtailment Level 1 would be reached at a frequency of 1 in 20 years by 2018. Curtailment Level 2 would be reached at a frequency of 1 in 100 years by 2016, and curtailment Level 3 would be reached at an assurance of 1 in 200 years by 2016. These critical points are marked with circles. Considering these three dates, the earliest that a new water supply intervention would be required for the WCWSS in order to ensure that the frequency of curtailments stays within acceptable limits would be 2016.

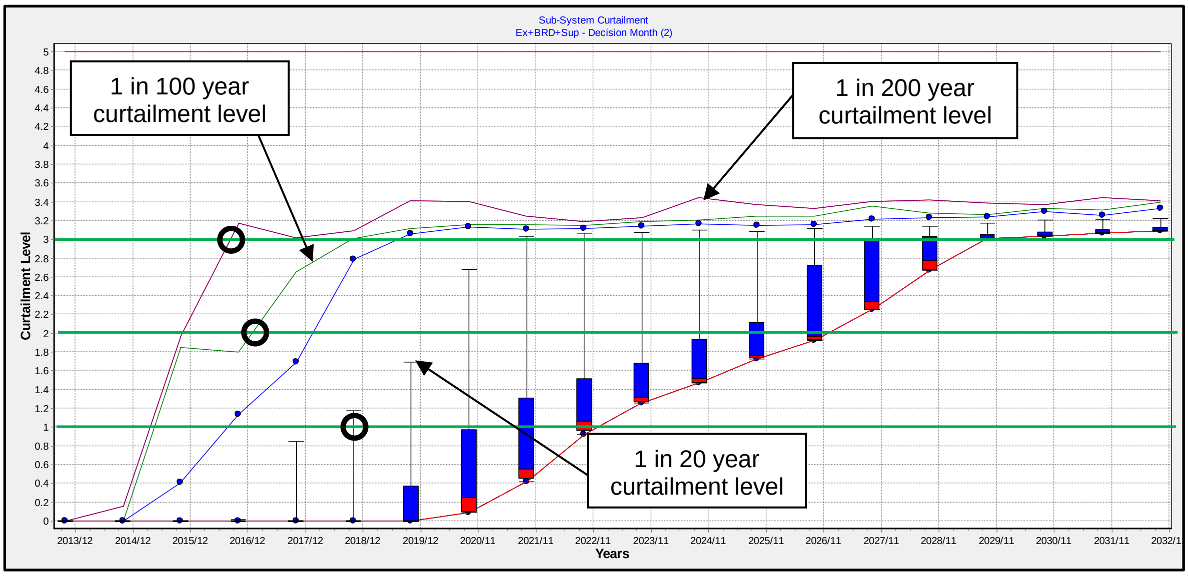

Figure 6.10: System curtailment for base scenario with no desalination plant (Scenario A)

6.3.3 Desalination plant scenarios in the WRPM

Box-and-whisker plots of the curtailment levels for each of the desalination plant scenarios and capacities, as derived from the WRPM analyses, are provided in **Appendix D**. On each plot, the critical points at which the acceptable frequencies are exceeded for the three curtailment levels are marked with circles. A summary of these dates is provided in **Table 6.3**. The earliest dates at which an intervention is required for each scenario and capacity are highlighted in bold.

Table 6.3: Year at which curtailment level would be reached for all scenarios

Scenario	1 in 20 year	1 in 100 year	1 in 200 year	1 in 20 year	1 in 100 year	1 in 200 year	1 in 20 year	1 in 100 year	1 in 200 year
	150 Mℓ/day plant			**300 Mℓ/day plant**			**450 Mℓ/day plant**		
B	2021	2019	**2019**	2024	2021	**2021**	2027	2025	**2023**
C	2021	2019	**2019**	2024	2021	**2021**	2027	2025	**2023**
D (90%)	2021	2019	**2019**	2027	**2021**	2022	2027	2024	**2023**
D (70%)	2021	**2018**	2019	2023	2021	**2019**	2027	2023	**2023**
D (45%)	2020	2017	**2016**	2020	2018	**2018**	2021	2018	**2018**
D (15%)	2019	2017	**2016**	2020	2018	**2018**	2020	2018	**2018**
	No desalination plant								
A	2018	2016	**2016**	2018	2016	**2016**	2018	2016	**2016**

Considering Scenario B, the addition of a 150 Mℓ/day to the system would delay the date at which a new intervention is required by three years from the base scenario to 2019. A plant capacity of 300 Mℓ/day would provide a further two years to 2021, and a capacity of 450 Mℓ/day, an additional two years to 2023. A possible phasing strategy for implementation of the desalination plant based on Scenario B would therefore be as follows:

- First 150 Mℓ/day lane in operation by 2016
- Second 150 Mℓ/day lane in operation by 2019
- Third 150 Mℓ/day lane operation by 2021
- Alternative supply source required by 2023

For Scenario C and for Scenario D with a 90 percent trigger level, the results are identical to those of Scenario B with no change in the implementation dates. This suggests that the decision regarding whether to operate the desalination plant 100 percent of the time or only when the dams are not spilling would not be dictated by system yield or water supply considerations but rather by other factors such as the added cost of operating the desalination plant when the dams are spilling. This is discussed in more detail in Section 6.4.

As the trigger levels decrease, however, the benefit of the desalination plant in terms of delaying the requirement for a new scheme is reduced. For a 70 percent reservoir trigger level, a 150 Mℓ/day plant would only give two additional years from the base scenario, however a 450 Mℓ/day would still give seven years. For the 45 percent and 15 percent trigger levels the desalination plant provides almost no benefit to the system in terms of reducing curtailments. This suggests that there is very little benefit in using the desalination plant as an emergency supply source rather than a base supply source.

6.3.4 Unlimited desalination plant capacity

The results presented in Section 6.3.3 are derived from assumed desalination plant capacities of 150, 300 and 450 Mℓ/day. The reverse of this approach would be to ask "What desalination plant capacity is required in order to meet future demands for the next 10 or 20 years?". Considering **Figure 6.9** in Section 6.3.1, based on the traditional approach, a 400 Mℓ/day desalination plant would meet the system requirements up to 2023 and a 1 000 Mℓ/day plant would meet the requirements up to 2033. The WRPM results, as presented in **Table 6.3** in Section 6.3.3, confirm that with a 450 Mℓ/day desalination plant, a new supply source would, at best, be required in 2023. However, from the WRPM results it is not possible to confirm what capacity is required to meet the demands beyond this point.

The WRPM analyses were therefore repeated for Scenario C and Scenario D (with a 70 percent trigger level only) with a desalination plant of unlimited capacity. The average monthly supply from the desalination plant over the analysis period for both scenarios is presented in **Figure 6.11**. The results shown are the median values for each month based on the 401 stochastic sequences that were analysed. Referring to **Figure 6.11**, the maximum monthly supply for Scenario C would be 2 100 Mℓ/day in 2023 increasing to 2 900 Mℓ/day in 2033. For Scenario D the plant would have to supply up to 1500 Mℓ/day in 2023, increasing to 2100 Mℓ/day by 2033. Reviewing the curtailment requirements for the unlimited capacity, it was confirmed that no curtailments would be required for the entire analysis period which is as expected. It is interesting to note that for the lower trigger level, the required total desalination plant capacity would be lower.

The problem with adopting the above-mentioned approach is that it does not allow for any failures over the analysis period. In other words, for Scenario D with a 70 percent trigger level, a 1500 Mℓ/day plant would meet the demands until 2023 with a 100 percent assurance of supply, and a 2100 Mℓ/day will meet the demands until 2033 at a 100 percent assurance of supply. This differs from the normal risk allowance that would be applied in planning for a water supply system in South Africa.

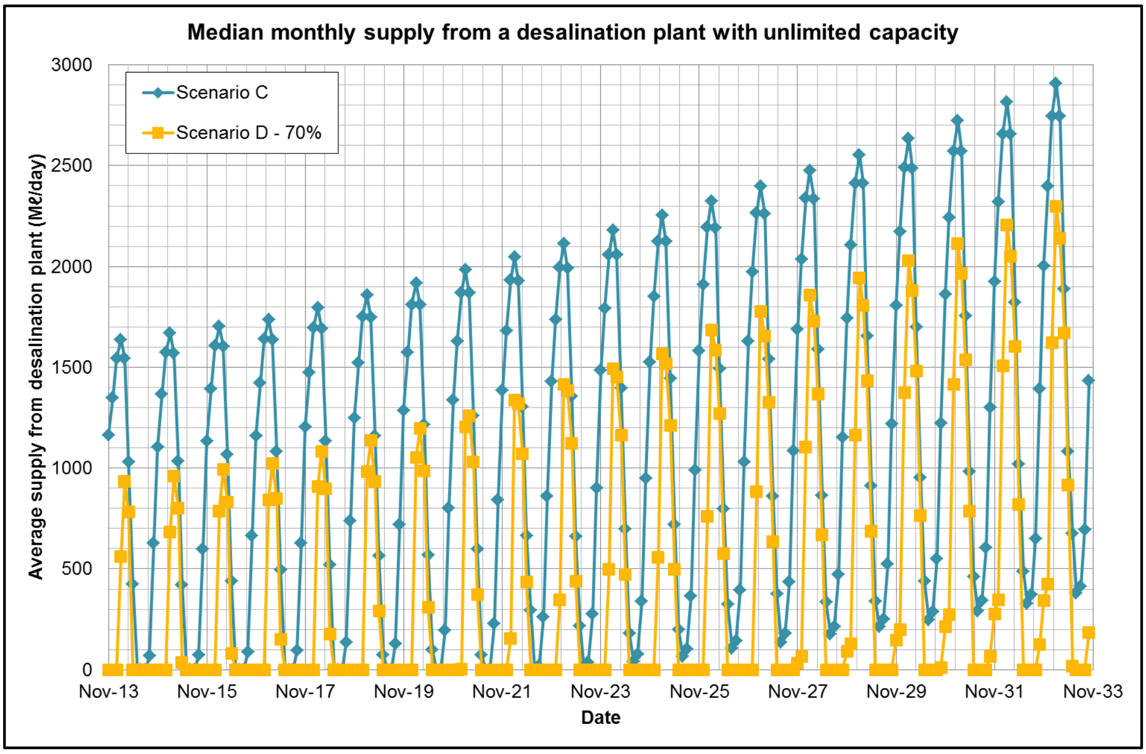

Figure 6.11: Median monthly supply from a desalination plant with unlimited capacity

Another interesting point to note is that the 95th percentile values in terms of monthly supply from the desalination plant, provided in **Figure 6.12**, are very similar to the median values. This suggests that in the case of a desalination plant with unlimited supply, the stochastic variability in the system has very small impact on the supply from the desalination plant.

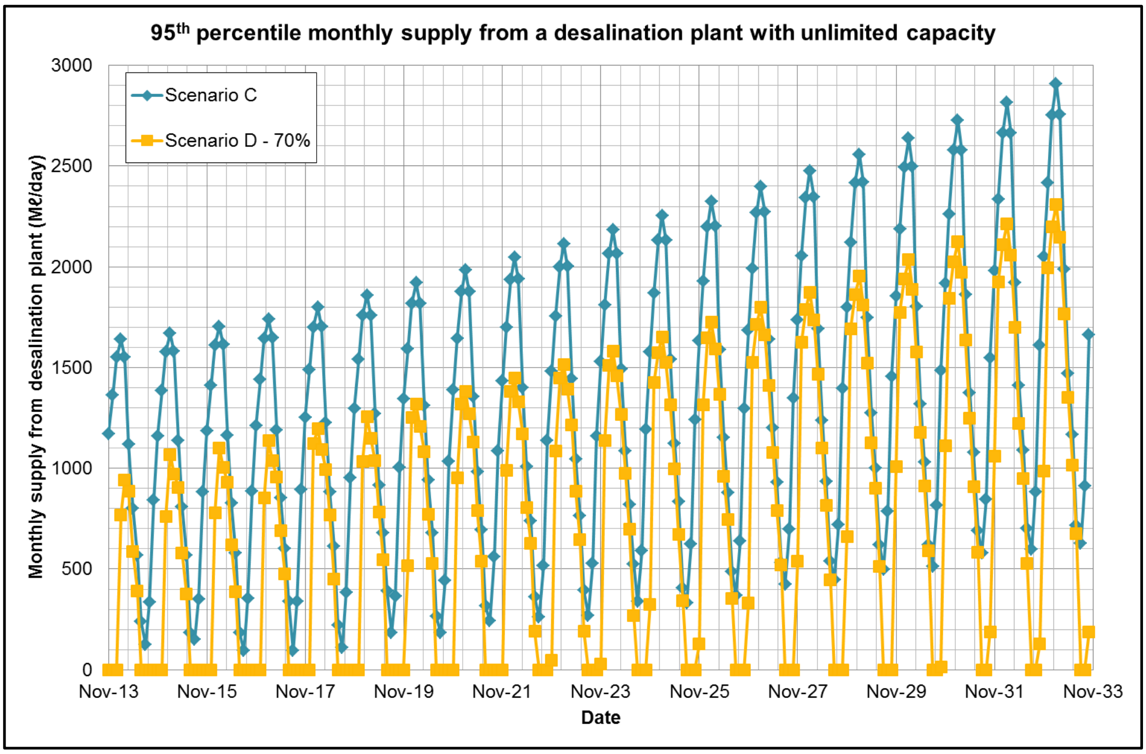

Figure 6.12: 95th percentile monthly supply from a desalination plant with unlimited capacity

6.4 Comparison of Costs

6.4.1 Traditional approach

As discussed in Section 3.2.1, the research of Van Niekerk and Du Plessis (2013a:551) showed that the traditional approach to calculating URVs uses the water supply from a scheme calculated based on a water balance between annual demands and a 1 in 50 year system yield. As a starting point, this traditional approach was applied for calculating the URV of the seawater desalination plant. Using the method discussed in Section 6.2.3, the annual supply from the seawater desalination plants of different capacities were calculated, and these values were fed into the seawater desalination URV model. The calculated URVs for the desalination plant for a discount rate of 8 percent are as follows:

- 150 Mℓ/day seawater desalination plant: R11.35/m^3
- 300 Mℓ/day seawater desalination plant: R11.46/m^3
- 450 Mℓ/day seawater desalination plant: R11.48/m^3

6.4.2 WRPM based approach

As discussed in Section 5.2.3, in the WRPM-based approach applied for this research, capital and operating costs were calculated for each scenario using the annual supply from the desalination plant as derived from the WRPM analyses. Using these costs and the associated desalination plant supply, URVs were calculated for each scenario and seawater desalination plant capacity as presented in **Table 6.4**. The values highlighted in bold are the median values which, as discussed in Section 6.2.2, were considered to be the most reasonable for the purposes of comparison between the scenarios and with other possible interventions. The 5th and 95th percentile URVs are also shown in **Table 6.4** in order to provide an indication of the possible stochastic range of the costs.

Table 6.4: URVs for all scenarios and desalination plant capacities

	150 Mℓ/day plant			300 Mℓ/day plant			450 Mℓ/day plant		
Scenario	5th %ile	50th %ile	95th %ile	5th %ile	50th %ile	95th %ile	5th %ile	50th %ile	95th %ile
B	R 10.96	**R 10.96**	R 10.96	R 10.38	**R 10.38**	R 10.38	R 9.81	**R 9.81**	R 9.81
C	R 10.96	**R 10.96**	R 11.75	R 10.38	**R 10.53**	R 11.38	R 9.81	**R 10.08**	R 10.92
D (90%)	R 10.96	**R 12.21**	R 14.56	R 10.38	**R 11.71**	R 13.48	R 9.83	**R 11.11**	R 12.47
D (70%)	R 11.16	**R 15.49**	R 22.77	R 10.69	**R 14.79**	R 19.90	R 10.33	**R 13.71**	R 17.71
D (45%)	R 37.06	**R 99.00***	R 99.00*	R 28.43	**R 99.00***	R 99.00*	R 24.22	**R 99.00***	R 99.00*
D (15%)	R 54.68	**R 99.00***	R 99.00*	R 36.71	**R 99.00***	R 99.00*	R 31.35	**R 99.00***	R 99.00*

* URVs in excess of R99/m^3 were capped at R99/m^3 as a representative upper limit.

Box-and-whisker plots of the calculated URVs are provided in **Appendix E**, showing the full stochastic range of values for all scenarios.

6.4.3 Discussion of costs

Considering the median (50th percentile) values in **Table 6.4**, it would appear that for all capacities, the URV increases as the trigger level decreases. For the lowest trigger levels of 15 percent and 45 percent, the URV exceeds R99 / m^3 for all capacities. It would therefore seem that operating the seawater desalination plant as an emergency type scheme would be an expensive option. Although the annual operating costs would be much lower for an emergency scheme, the initial capital cost would be the same regardless of the actual desalination plant output. The results suggest that this initial capital cost outweighs any reduction in operating costs when the desalination plant output is reduced.

For a trigger level of 70 percent, the URV for a 150 Mℓ/day plant is much more reasonable at R15.49 / m^3, decreasing to R12.21 / m^3 for a 90 percent trigger level. For Scenario B and Scenario C, the desalination plant would be fully operational once constructed, giving a URV of R10.96 / m^3. Based on these values, it would appear that the lowest possible URV occurs when the desalination plant is operated as base supply.

Comparing the three capacities that were analysed, the URVs decrease slightly with an increase in desalination plant capacity despite the fact that the desalination plant would take longer (in years) to reach full operating capacity. For each doubling in capacity, the URVs reduce by about 5 percent. This is most likely due to the economy of scale, in other words the reduction in unit capital costs with an increase in desalination plant capacity.

Comparing the results from the traditional approach to the WRPM-based approach, the results for a 150 Mℓ/day are actually quite similar with the traditional approach, about 4 percent higher than Scenario B/C. The traditional approach, however suggests an increase in the URV with an increase in capacity, and is 10 percent higher than the WRPM approach for a 300 Mℓ/day plant and 17 percent higher for a 450 Mℓ/day plant (comparing with Scenario B).

This suggests that although the annual supply from the desalination plant differs vastly between the traditional and WRPM approaches in the initial years, as discussed in Section 6.2.3, both approaches suggest that the desalination plant would be 100 percent operational in the later years meaning that the calculated URVs are similar. However, the lower trigger level scenarios have much higher URVs than the traditional approach.

Considering the 5th and 95th percentile values, it appears that the possible stochastic range increases with a decrease in the trigger level and with an increase in capacity. For Scenario B, the desalination plant is set to be 100 percent operational so there is no variability in the supply, but Scenario C already shows a deviation of up between -3% and +8% from the median for a 450 Mℓ/day plant. Therefore, for a fairly predictable scenario in terms of desalination plant supply (when it will be operational most of the time), the stochastic variability in the existing system would have little effect on the URVs and would not differ too greatly from URVs calculated using the traditional approach. For a 70 percent trigger level, the variation from the median is much more substantial, ranging from -28% to +40% for a 150 Mℓ/day plant. Hence, for a less predictable operating scenario, such as using the desalination plant as an emergency type scheme, the stochastic variability in the URV would be more noticeable. This would mean that the selection of an appropriate percentile for reporting on the costs would become more relevant.

7 CONCLUSIONS AND RECOMMENDATIONS

7.1 Conclusions

A detailed review was undertaken of literature related to basic concepts of water resource planning, planning and operation of the WCWSS, planning for seawater desalination in South Africa, the conjunctive use of seawater desalination with surface water resources, seawater desalination costing techniques, and seawater desalination planning in Australia. The following conclusions can be drawn from the literature review:

- The WCWSS supplies over 500 million m^3/a to users throughout the Western Cape. It is a system reliant almost entirely on surface water supply, with a network of infrastructure that requires integrated operation in order to maximise the available yield.
- Through the current WCWSS planning processes, seawater desalination has been earmarked as a potential future supply source for the area, and a feasibility study is currently underway. In order to maximise the benefits and minimise the costs of a seawater desalination plant, it needs to be considered as an integral part of the current system.
- Seawater desalination planning in South Africa has historically happened on an *ad hoc* basis in response to droughts, requiring the rapid construction of small-scale seawater desalination plants as emergency supply schemes. However, the national and local water-related planning documents all highlight the role that seawater desalination will play in future water supply. Seawater desalination feasibility studies are currently underway in Saldanha Bay, Port Elizabeth, Durban and Cape Town, suggesting a move towards more holistic and integrated planning.
- The examples of the Mossel Bay desalination plant in South Africa and the Kurnell and Tugun plants in Australia illustrate how implementing seawater desalination in an emergency, without careful planning, can leave water authorities with an expensive and under-utilised water supply source, placing a financial burden on the consumers.
- Internationally, the importance of considering a seawater desalination plant as an integral component of a water supply system has been recognised and studied for over five decades, however much of the international research has been related to smaller plants for remote locations. Locally, research surrounding the Mossel Bay desalination plant highlights the value of water to different users and the attitude of a typical municipality to the operation of an expensive supply source.

- Recent research related to inter-basin water transfer schemes has shown that the traditional approach in evaluating the volume of water transferred differs vastly from the actual volumes supplied, mainly due to a lack of consideration of the stochastic nature of the receiving environment. This would most certainly also apply to a seawater desalination plant. A comprehensive approach is therefore proposed in which the system as a whole is modelled stochastically in the WRPM and the estimated water transfer extracted. The comprehensive approach was the foundation of the modelling undertaken in this research.
- There are many examples in literature of techniques for estimating costs of seawater desalination plants, most of which are based on databases of completed plants. The problem in comparing known costs from literature is that there is often ambiguity surrounding what has been included and excluded. Often the quoted costs are for the RO portion only and exclude any pre-treatment, post-treatment and marine works.

In order to undertake integrated system analysis, the WRYM and WRPM were selected as the most appropriate tools given that these models are freely available, easy to use and verified system models of the WCWSS are available in both. Short-term and long-term analyses were completed for a variety of possible seawater desalination plant operating scenarios and capacities in order to determine the increase in system yield and the annual supply from the desalination plant. First-order capital and operating costs were estimated using the South African methodology of Du Plessis *et al.* (2006). Combined with the annual supply values from the WRPM, these costs were used to calculate and compare URVs of the desalinated water.

The following conclusions can be drawn from the modelling analysis:

- The WRYM and WRPM modelling of the WCWSS showed that it is possible to model a seawater desalination plant as an integral part of a surface water supply system using a multi-purpose min-max channel.
- The increase in system yield appears to be logarithmically related to the reservoir trigger level. Generic equations were developed which could be used to estimate the increase in historic firm yield and 1 in 50 year to 1 in 200 year yields of the WCWSS for any seawater desalination plant capacity and reservoir trigger level.
- The maximum increase in yield occurs when the seawater desalination plant is used as a base supply, operational all the time. The increase in yield decreases with a decrease in the reservoir trigger level, meaning that there is little benefit in terms of

increase in system yield in using the seawater desalination plant as an emergency supply source.

- With a 150 Mℓ/day seawater desalination plant, curtailment requirements would be kept within acceptable levels until 2019, a 300 Mℓ/day plant until 2021 and a 450 Mℓ/day plant until 2023, for a plant which is 100 percent operational. Using the plant as an emergency supply source would add little benefit in terms of limiting curtailment requirements.
- URVs for the seawater desalination plant decrease as the percentage of time for which the desalination plant is operational increases. This means that the lowest possible cost per cubic metre of desalinated water supplied is when the seawater desalination plant is operational 100 percent of the time (R10.96/m^3 for a 150 Mℓ/day plant).
- It would also appear that the URV's decrease with an increase in desalination plant capacity, which suggests that instead of phasing the desalination plant, it would be more beneficial from an economic perspective to have one large desalination plant operational immediately. However, it would have to be constructed in small enough lanes to enable restriction of the output when the total supply is not required, and cognisance would have to be taken of other relevant factors such as availability of other schemes already in place and growth in water demands.
- The lower the reservoir trigger level, the larger the stochastic variation in the supply from the desalination plant and hence the larger the variation in the URVs. Hence for a seawater desalination plant operational as a base supply source, undertaking stochastic modelling to calculate URVs is less important than for an emergency supply source.

7.2 Recommendations

The following recommendations are made in terms of further considerations and research:

- The WRYM and WRPM were selected as the most appropriate tools for modelling the seawater desalination plant as an integral component of the system. However, there are a number of other tools, such as Mike Basin, which would also be suitable. Repeating the analyses in a different model may provide a useful comparison.
- The calculated URV's presented here can be compared with URV's for other potential water supply sources for the WCWSS. However, a limitation is that only the cost of the desalination scheme was considered in the analysis. As part of future research, it may be worthwhile to consider developing a costing model for the

WCWSS as a whole and attempting to optimise the overall cost for the system with the implementation of a seawater desalination plant. Under such an approach, reducing the desalination plant output may prove to be more cost effective.

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APPENDIX A: SEAWATER DESALINATION COSTING EXAMPLE

Table A1: Example calculation using the methodology of Du Plessis *et al.* (2006)

STEP	DESCRIPTION	SYMBOL	VALUE	UNIT
1	Desalination Plant Capacity	-	100	Mℓ/d
	Desalination Plant Capacity	Q_D	100 000	m^3/d
	Operational factor	α	1	-
	Hourly product flow rate	Q_h	4 167	m^3/h
2	Recovery Factor	RF	0.4	-
	Feed water flow rate	Q_F	10 417	m^3/h
3	Number of passes	-	2	-
	Factor based on number of passes	z	1	-
	Flux through first pass	φ_1	10	ℓ/hm^2
	Flux through second pass	φ_2	28	ℓ/hm^2
	Total membrane area	A_{mem}	565 476	m^2
	Total Dissolved Salts of Feed Water	TDS_{F1}	36000	mg/ℓ
	Total Dissolved Salts after first pass	TDS_{F2}	360	mg/ℓ
	Flux per driving pressure for first pass	ε_1	1	ℓ/m^2hbar
	Flux per driving pressure for second pass	ε_2	4.5	ℓ/m^2hbar
	Recovery of second pass	R_2	0.9	-
	Feed water temperature	T	11	°C
	Pre-treatment head loss	-	5	bar
	Feed pressure for first pass	$PF(T)_1$	70	bar
	Feed pressure for second pass	$PF(T)_2$	21	bar
4	Pre-treatment loss factor	β	0.05	-
	Incoming flow	$Q_{h(in)}$	10 965	m^3/h
	Outgoing flow	Q_h	4 167	m^3/h
5	Pump efficiency	η_{p1}	0.7	-
	Recovery system	-	Y	-
	Factor depending on recovery system	s	1	-
	Recovery system efficiency	η_{r1}	0.9	-
	Energy consumption for desalination first pass	E_{desal1}	16 298	kW
	Energy consumption for desalination second pass	E_{desal2}	3 571	kW
	Feed pressure for intake and pre-treatment systems	P_{in}	3.0	bar
	Other energy requirements	E_{other}	1 305	kW
	Specific Energy	E_s	5.08	kWh/m^3

STEP	DESCRIPTION	SYMBOL	VALUE	UNIT
6	Brine discharge rate	Q_{eff}	6 798	m^3/h
	Total Dissolved Salts of brine	TDS_C	60000	mg/ℓ
7	Desalination capital cost	-	453.6	R millions
	Pre-treatment factor	κ	1.00	-
	Pre-treatment capital cost	-	110.2	R millions
	Post treatment factor	-	0.2	-
	Post-treatment capital cost	-	110.2	R millions
	Total capital cost (2006)	Cap	661.5	R millions
	Current year	-	2014	-
	Inflation rate	-	6.0%	p.a.
	Total capital cost (present day)	-	1054.3	R millions
8	Interest rate	i	8.5%	p.a.
	Payment period	n	25	years
	Payment period		300	months
	Monthly payments	B	8.49	R millions
	Specific cost of capital redemption	M_{cap}	2.83	R/m^3
	Unit cost of electricity	D	1.0	R/kWh
	Specific electricity cost	M_{energy}	5.08	R/m^3
	Membrane cost	$Cost_m$	262.98	R/m^2
	Membrane replacement frequency	Y	6	years
	Specific cost of membrane replacement	M_{mem}	0.68	R/m^3
	Specific cost of chemicals	M_{chem}	0.75	R/m^3
	Labour and maintenance factor	u	0.05	-
	Specific cost of Labour and Maintenance	M_{maint}	1.44	R/m^3
	Total specific cost	M_{tot}	16.44	R/m^3

APPENDIX B: STOCHASTIC YIELD RESULTS

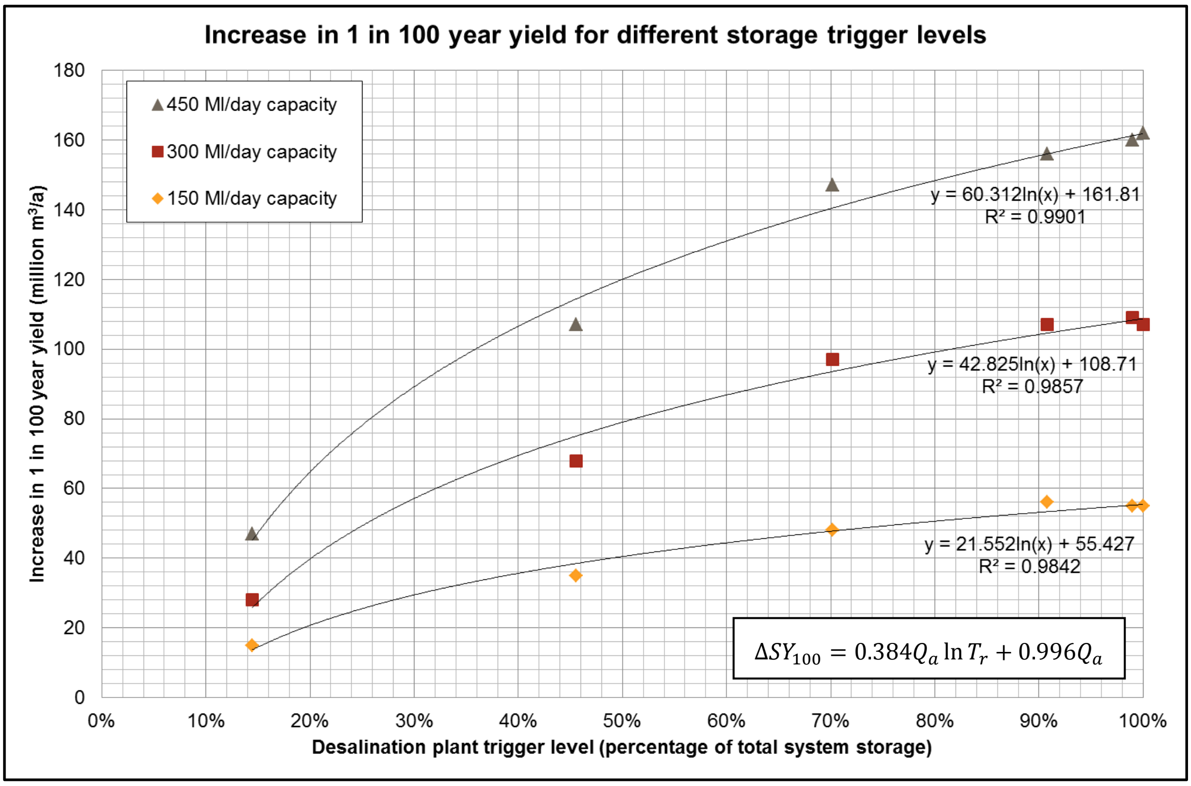

Figure B1: Increase in 1 in 100 year yield versus desalination plant trigger level for all capacities

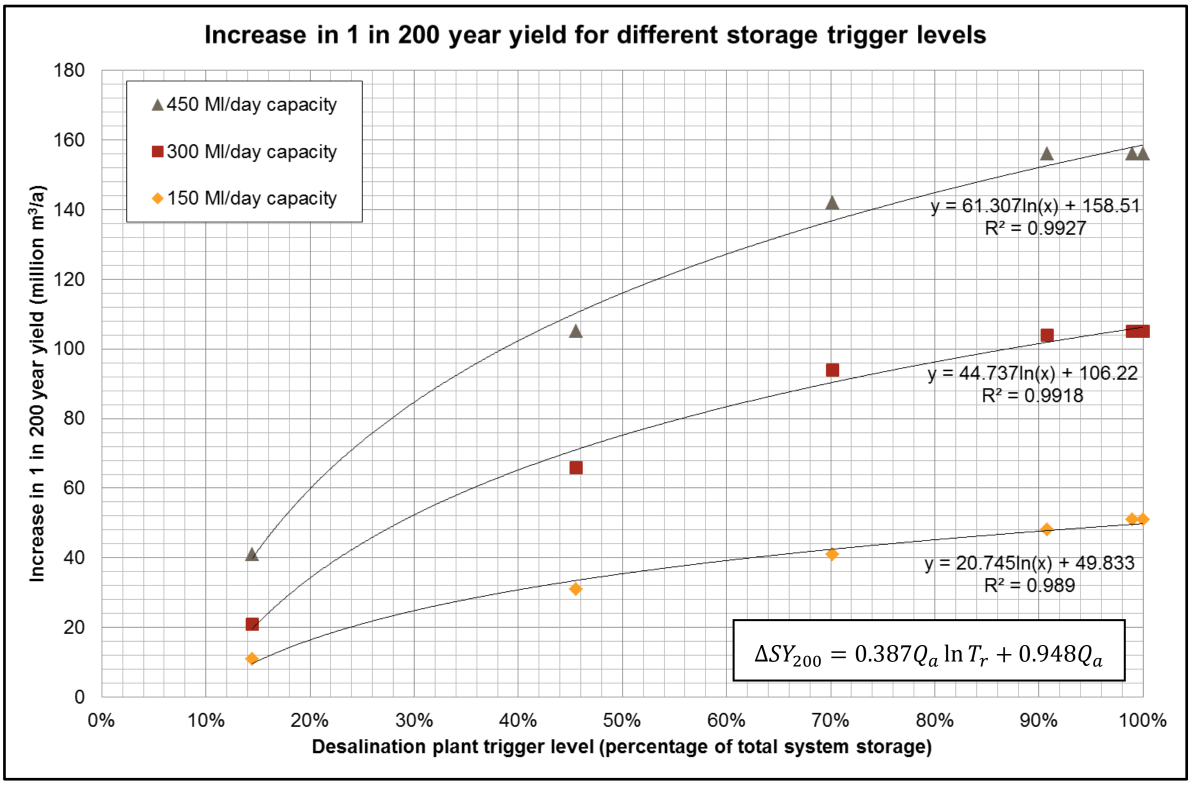

Figure B2: Increase in 1 in 200 year yield versus desalination plant trigger level for all capacities

APPENDIX C: SEAWATER DESALINATION PLANT SUPPLY

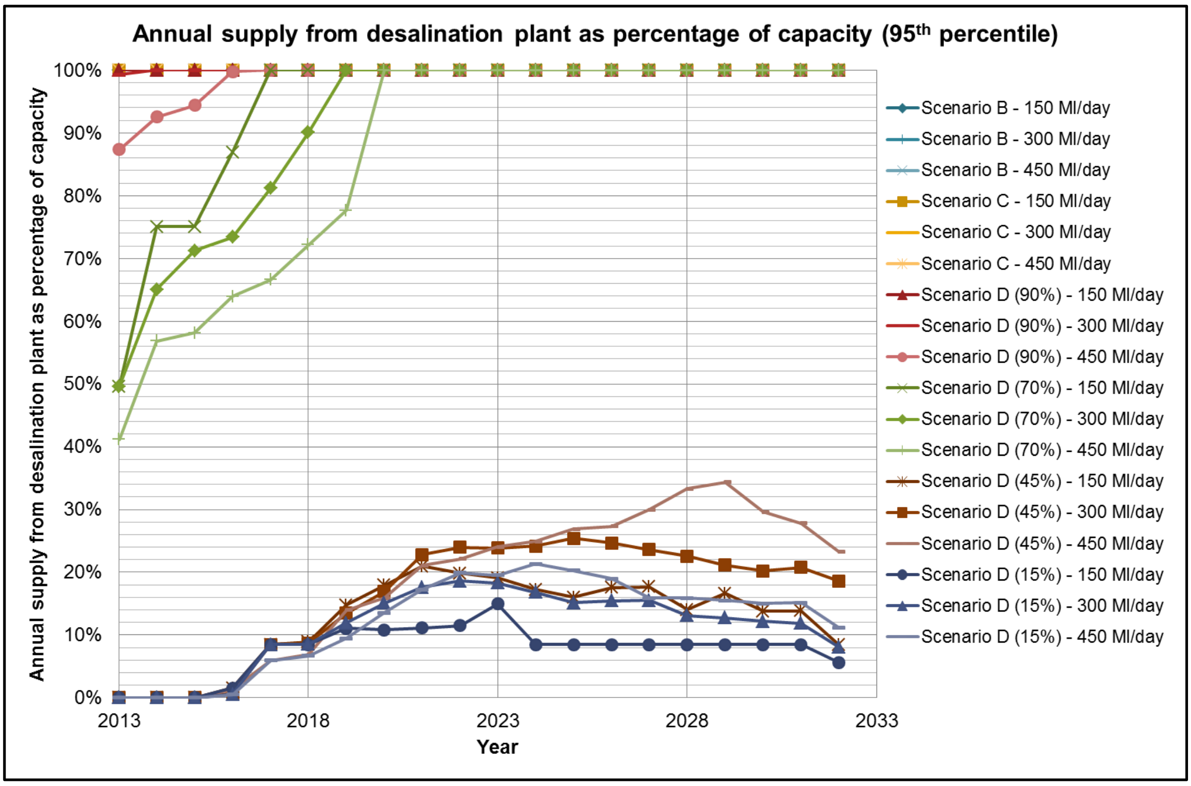

Figure C1: 95th percentile annual supply from desalination plant as a percentage of the desalination plant capacity

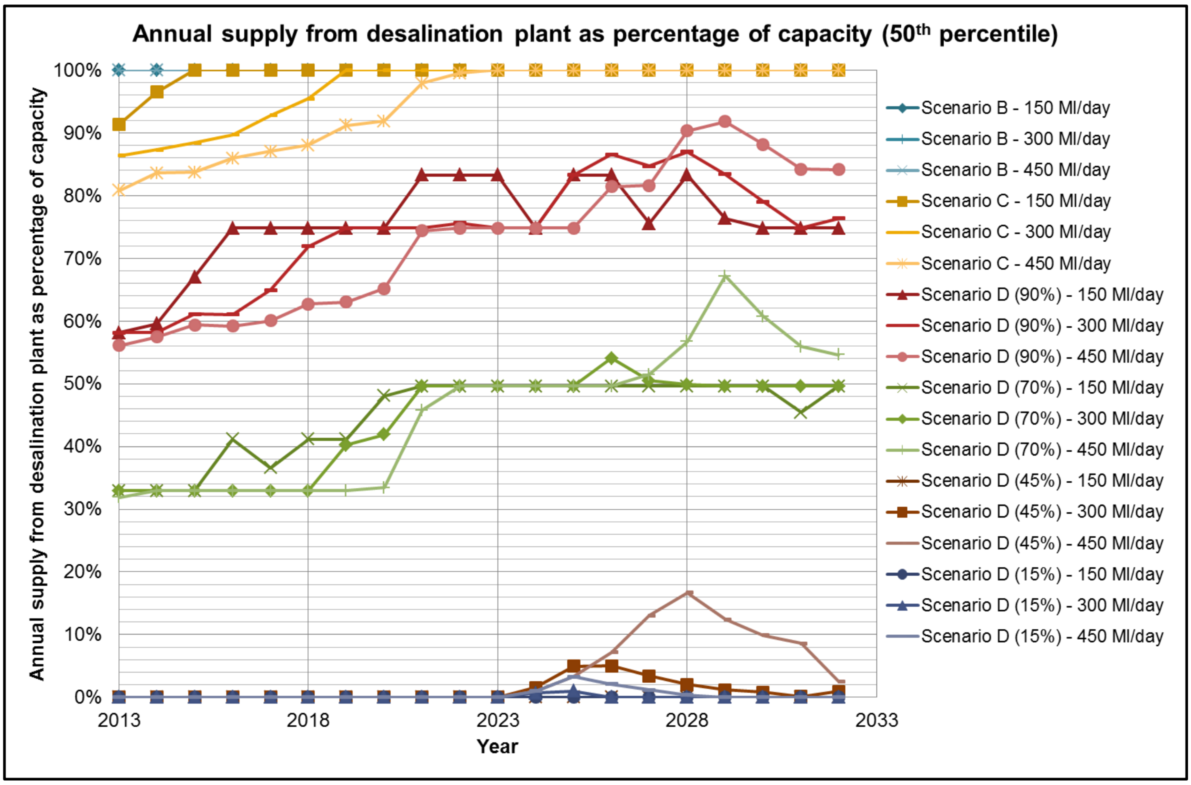

Figure C2: 50th percentile annual supply from desalination plant as a percentage of the desalination plant capacity

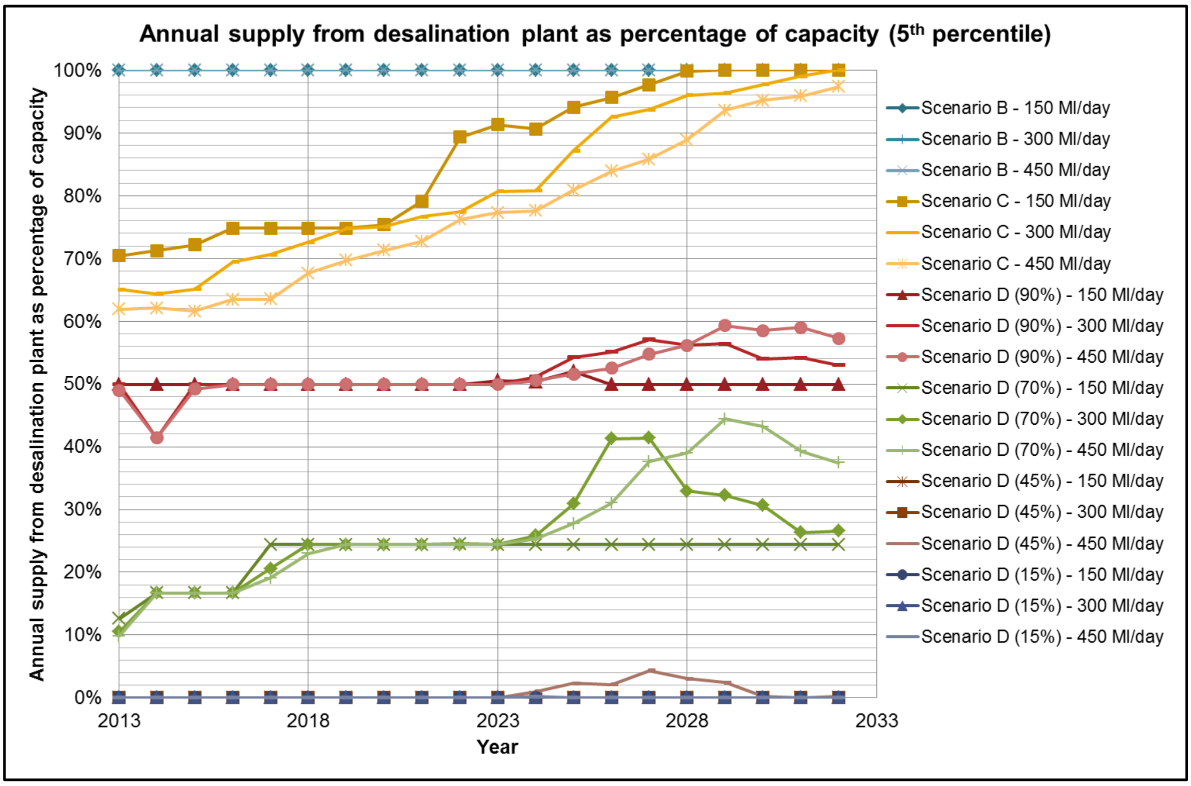

Figure C3: 5th percentile annual supply from desalination plant as a percentage of the desalination plant capacity

APPENDIX D: SYSTEM CURTAILMENT

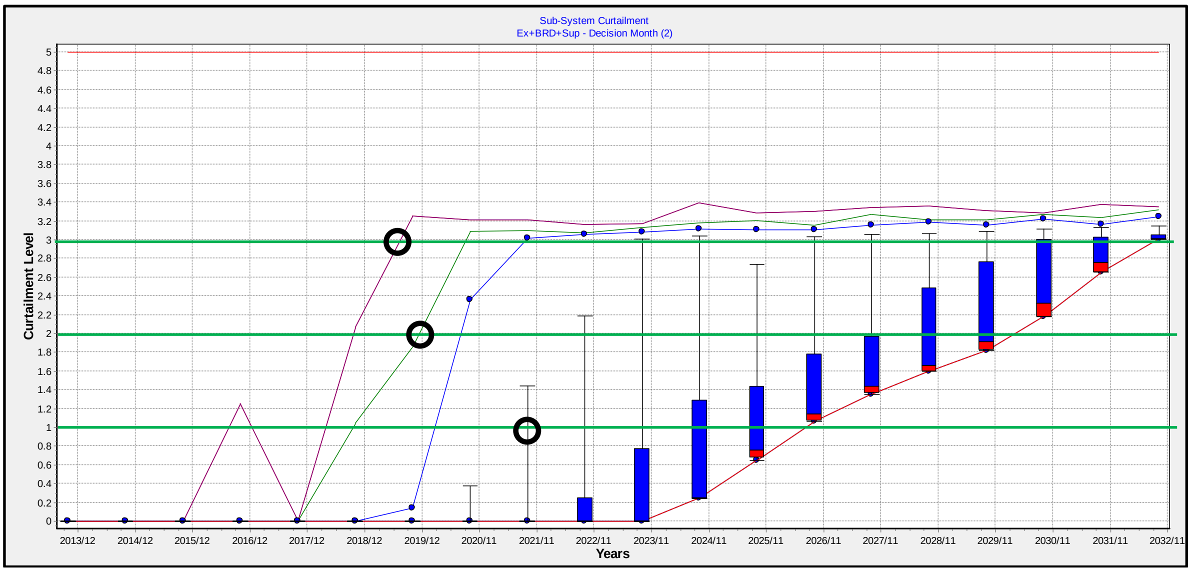

Figure D1: System curtailment with 150 Mℓ/day desalination plant for Scenario B

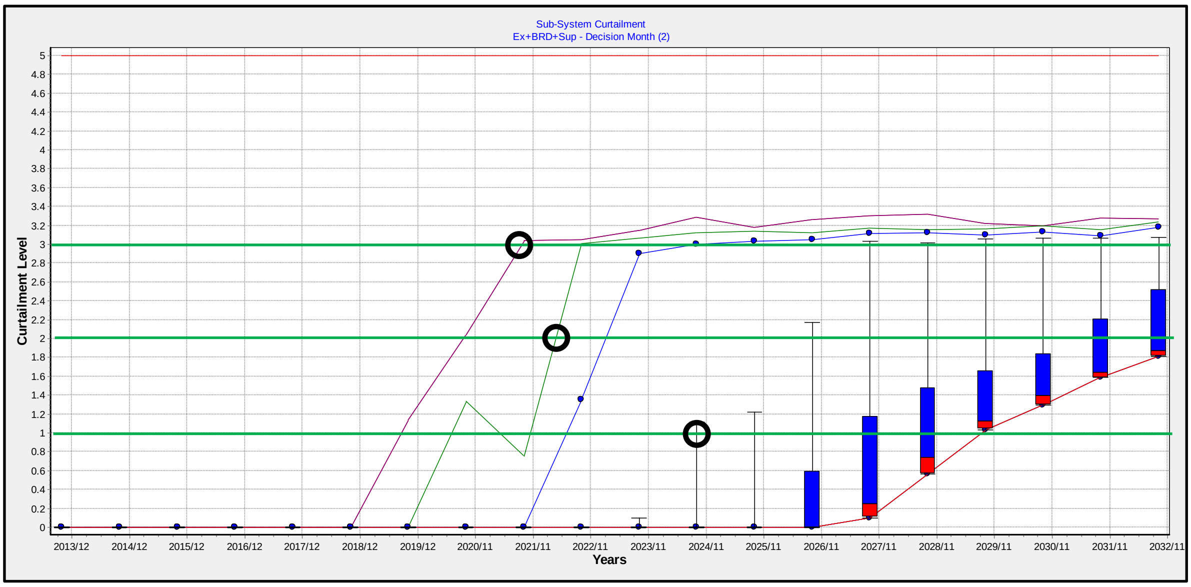

Figure D2: System curtailment with 300 Mℓ/day desalination plant for Scenario B

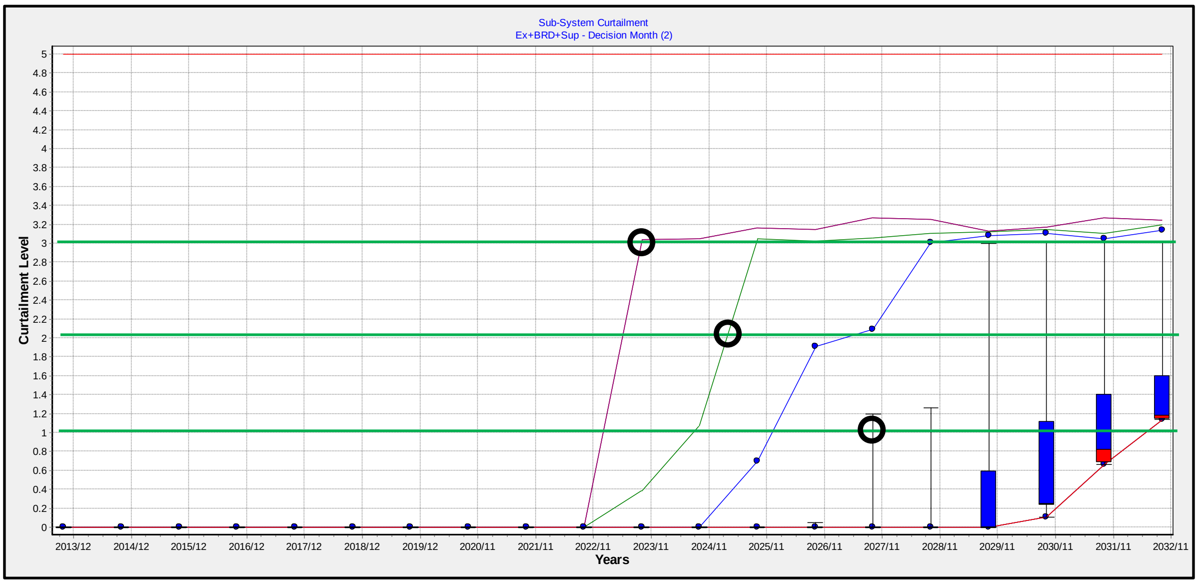

Figure D3: System curtailment with 450 Mℓ/day desalination plant for Scenario B

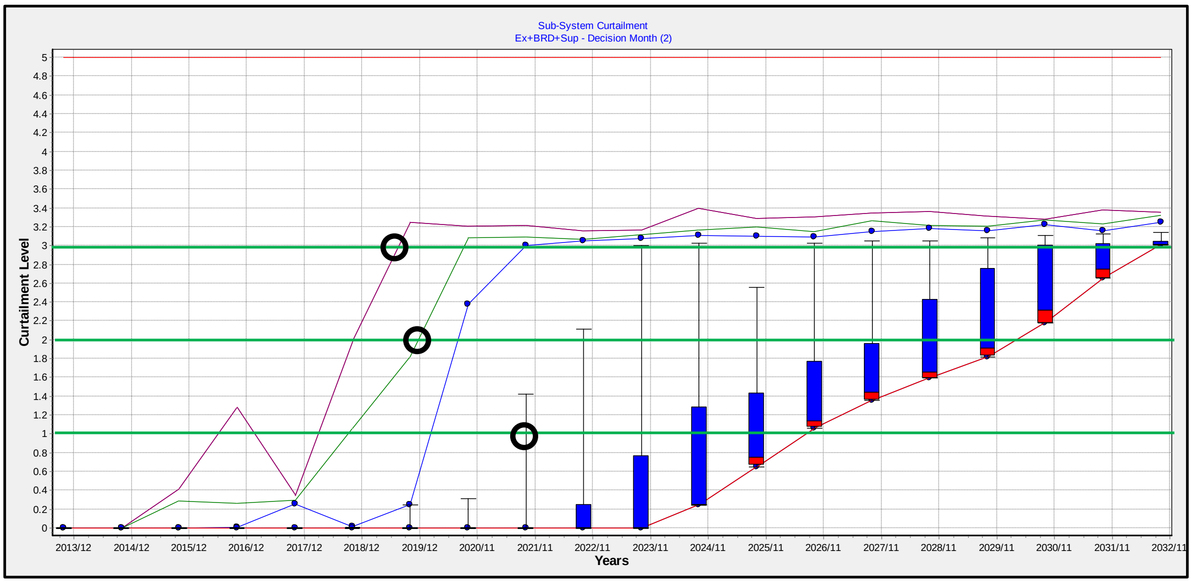

Figure D4: System curtailment with 150 Mℓ/day desalination plant only operational when dams are not spilling (Scenario C)

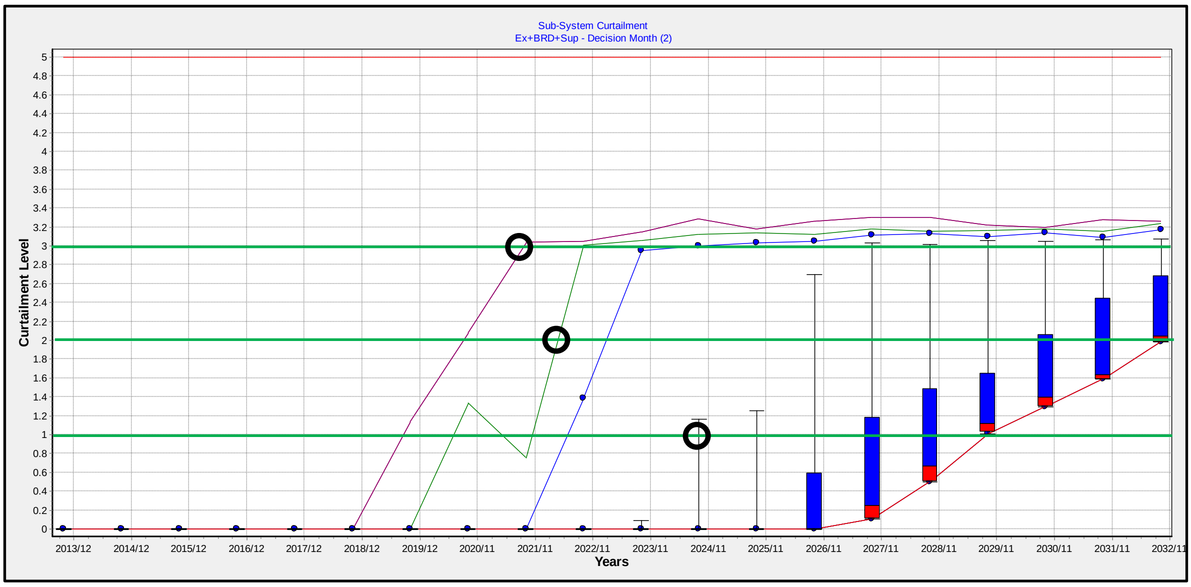

Figure D5: System curtailment with 300 Mℓ/day desalination plant only operational when dams are not spilling (Scenario C)

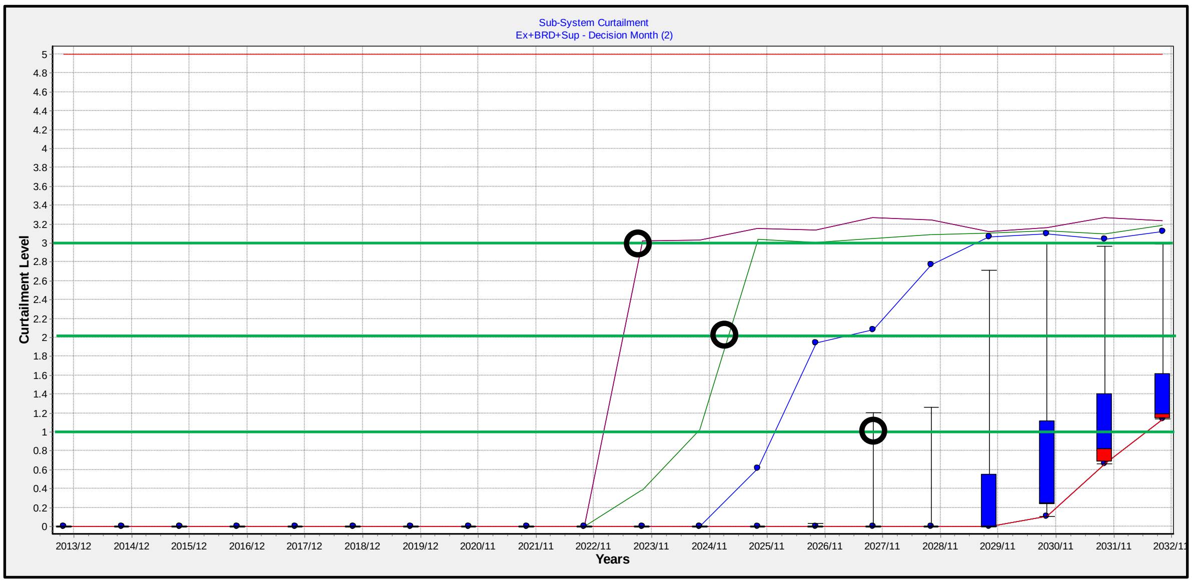

Figure D6: System curtailment with 450 Mℓ/day desalination plant only operational when dams are not spilling (Scenario C)

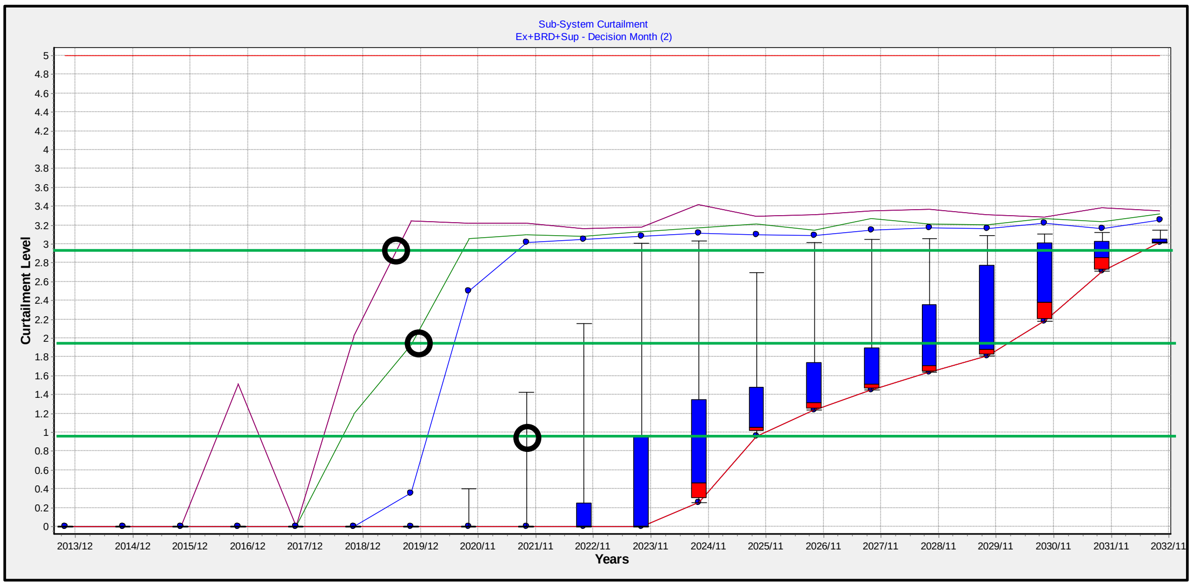

Figure D7: System curtailment with 150 Mℓ/day desalination plant only operational when system storage drops below 91 percent (Scenario D)

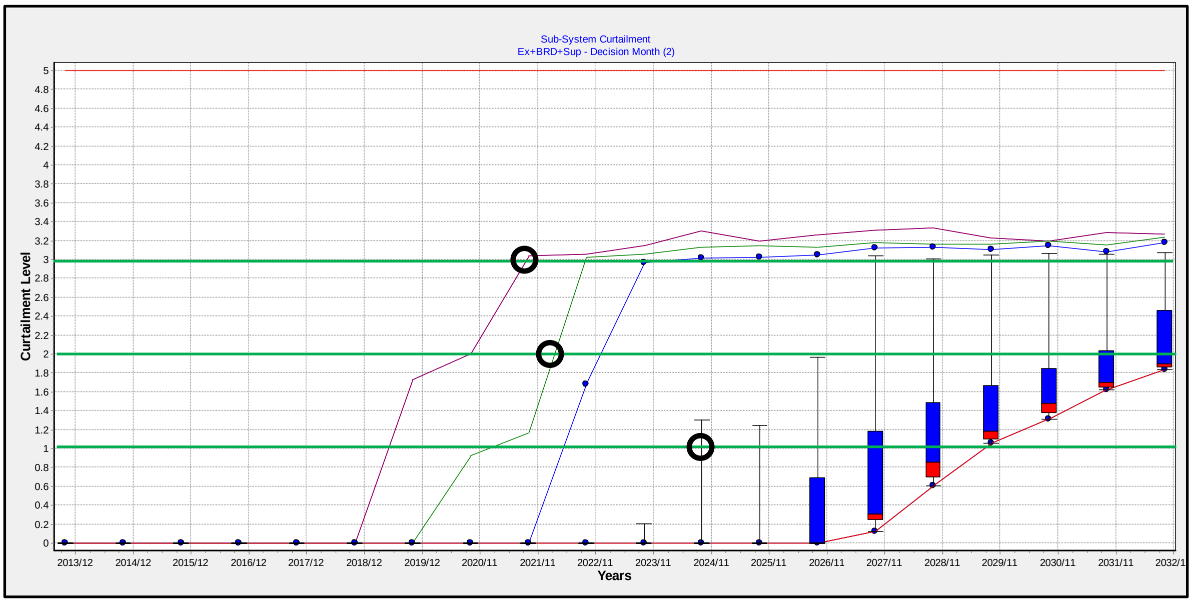

Figure D8: System curtailment with 300 Mℓ/day desalination plant only operational when system storage drops below 91 percent (Scenario D)

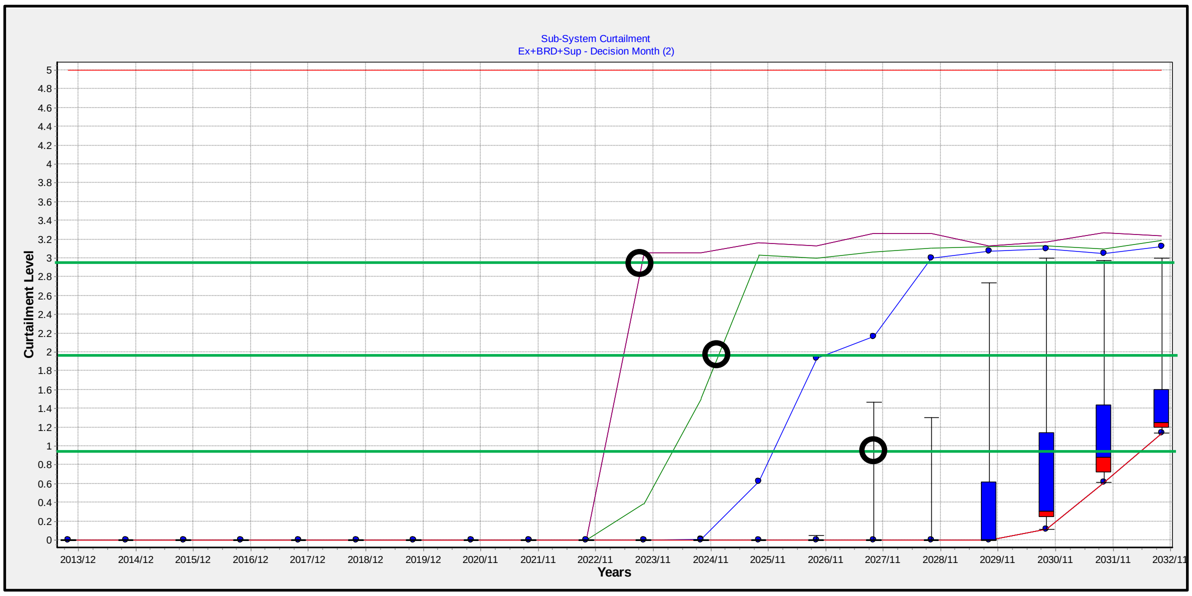

Figure D9: System curtailment with 450 Mℓ/day desalination plant only operational when system storage drops below 91 percent (Scenario D)

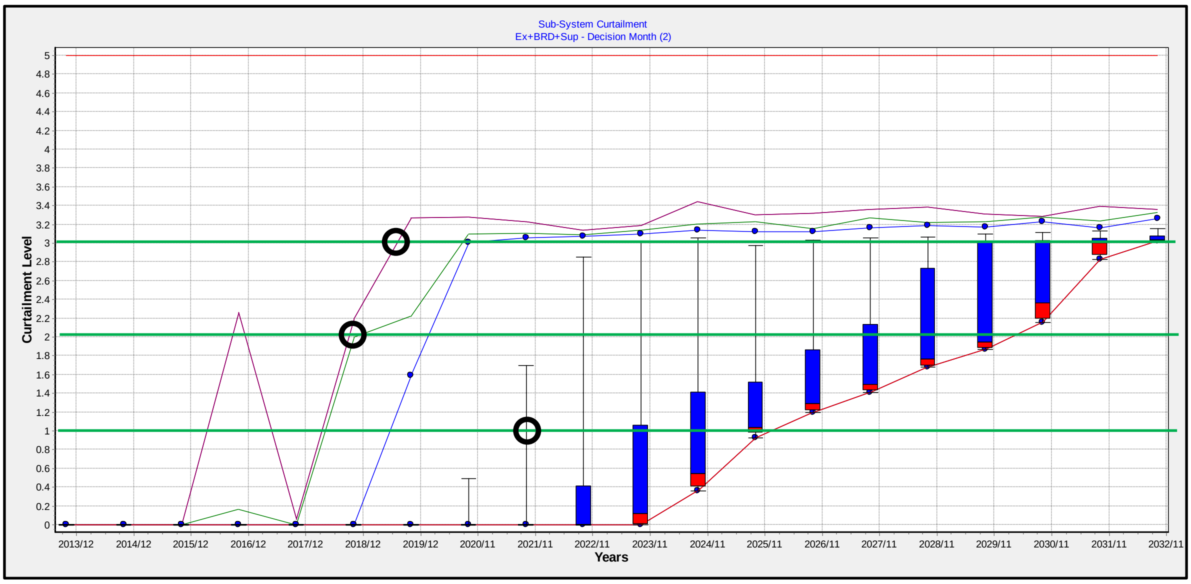

Figure D10: System curtailment with 150 Mℓ/day desalination plant only operational when system storage drops below 70 percent (Scenario D)

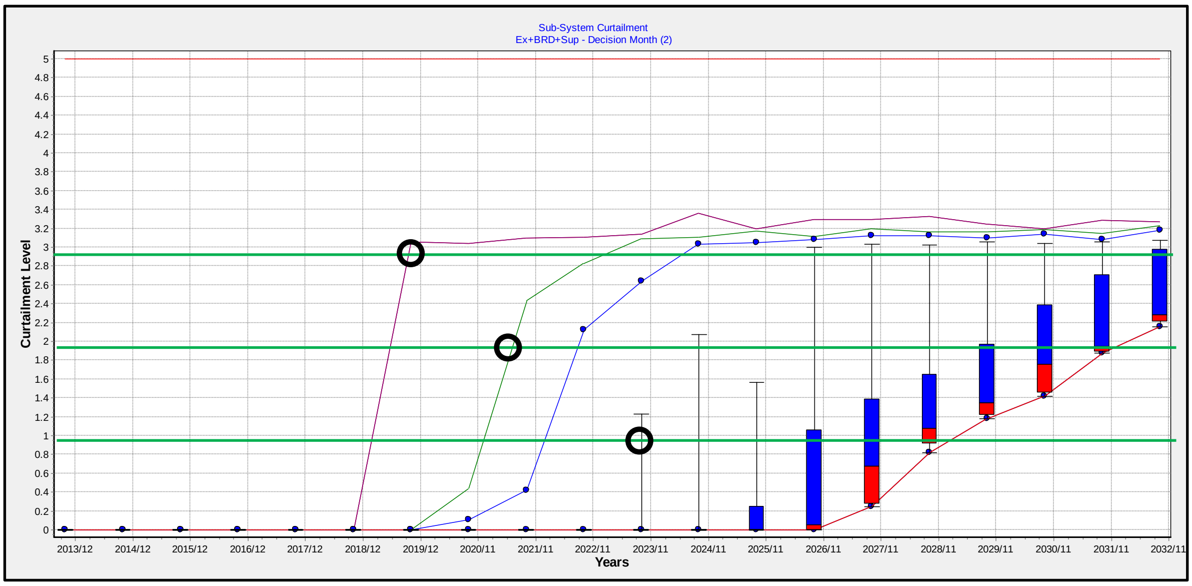

Figure D11: System curtailment with 300 Mℓ/day desalination plant only operational when system storage drops below 70 percent (Scenario D)

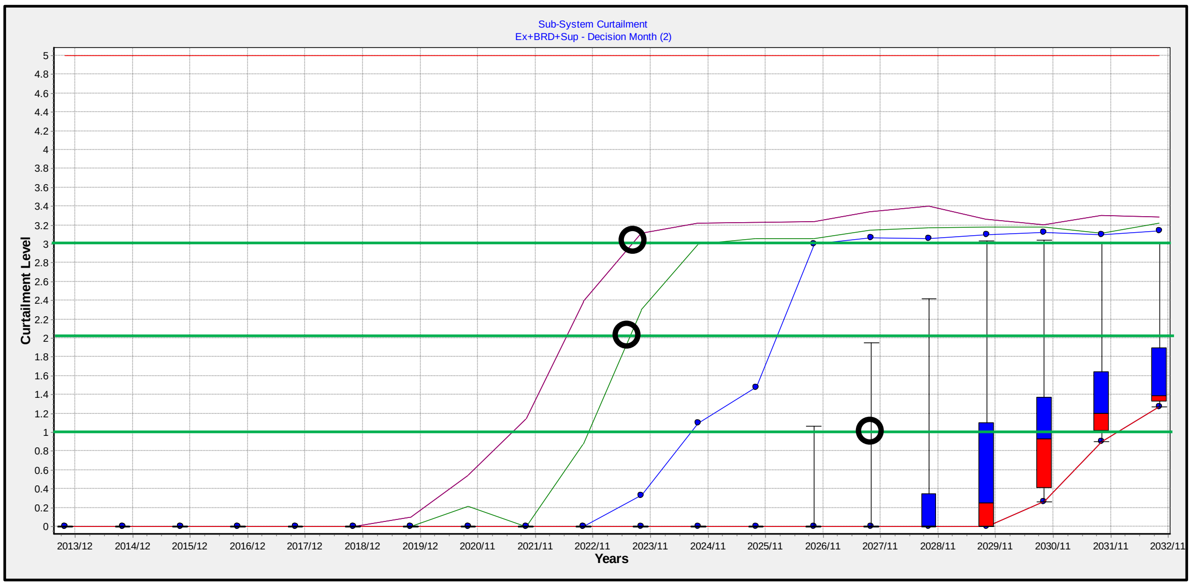

Figure D12: System curtailment with 450 Mℓ/day desalination plant only operational when system storage drops below 70 percent (Scenario D)

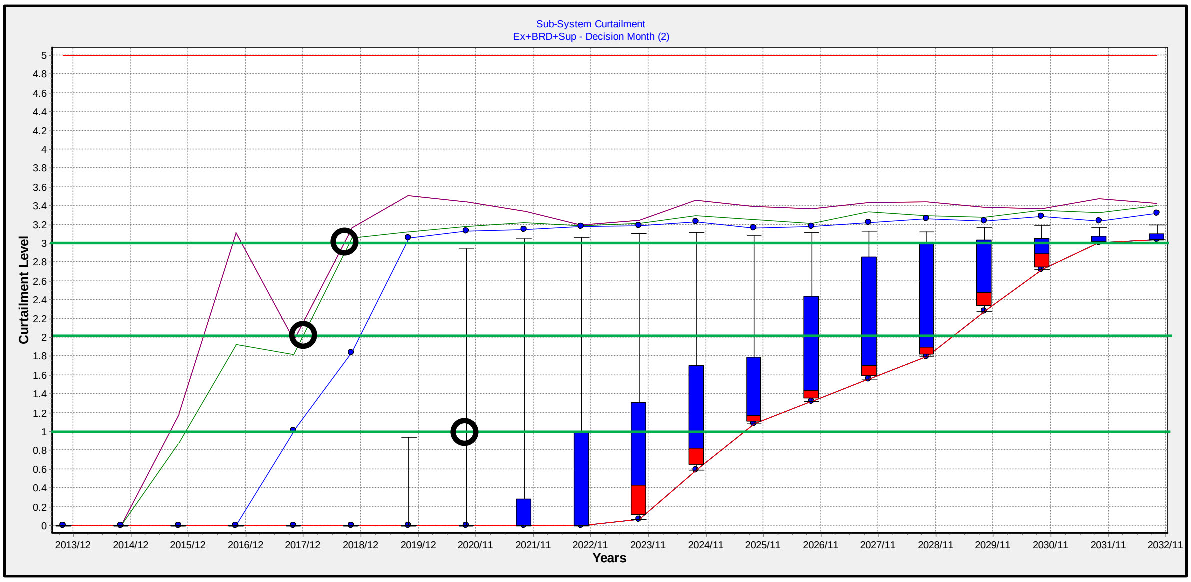

Figure D13: System curtailment with 150 Mℓ/day desalination plant only operational when system storage drops below 46 percent (Scenario D)

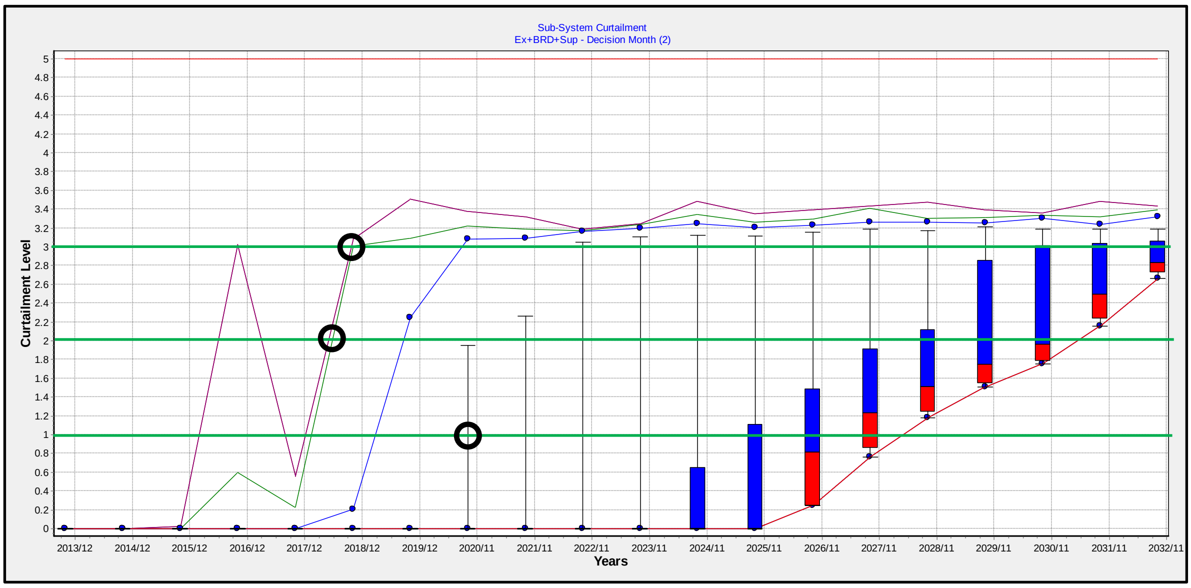

Figure D14: System curtailment with 300 Mℓ/day desalination plant only operational when system storage drops below 46 percent (Scenario D)

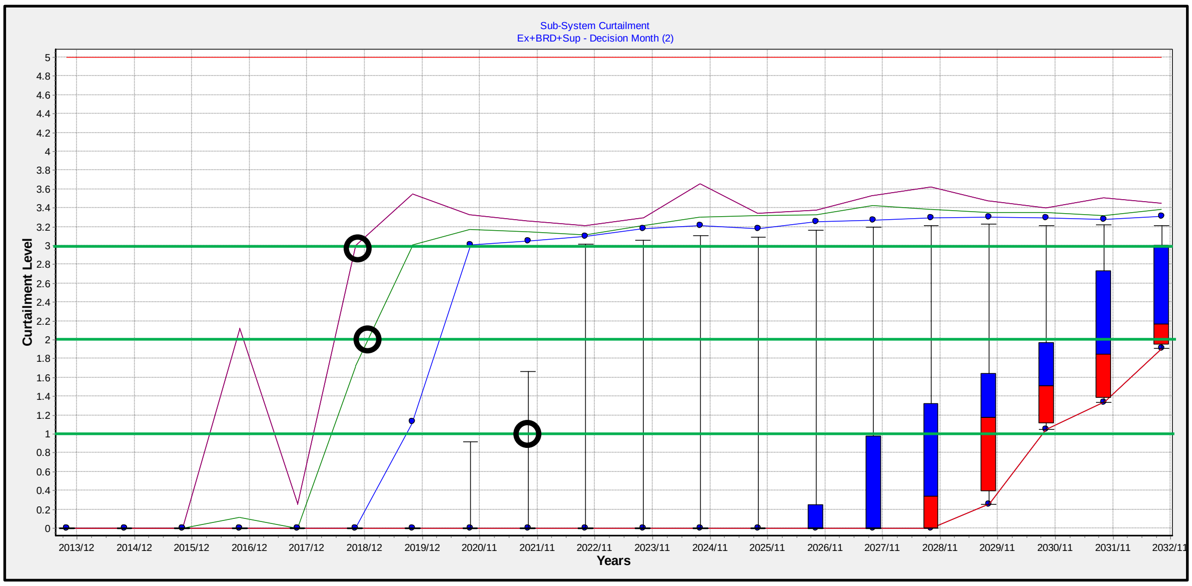

Figure D15: System curtailment with 450 Mℓ/day desalination plant only operational when system storage drops below 46 percent (Scenario D)

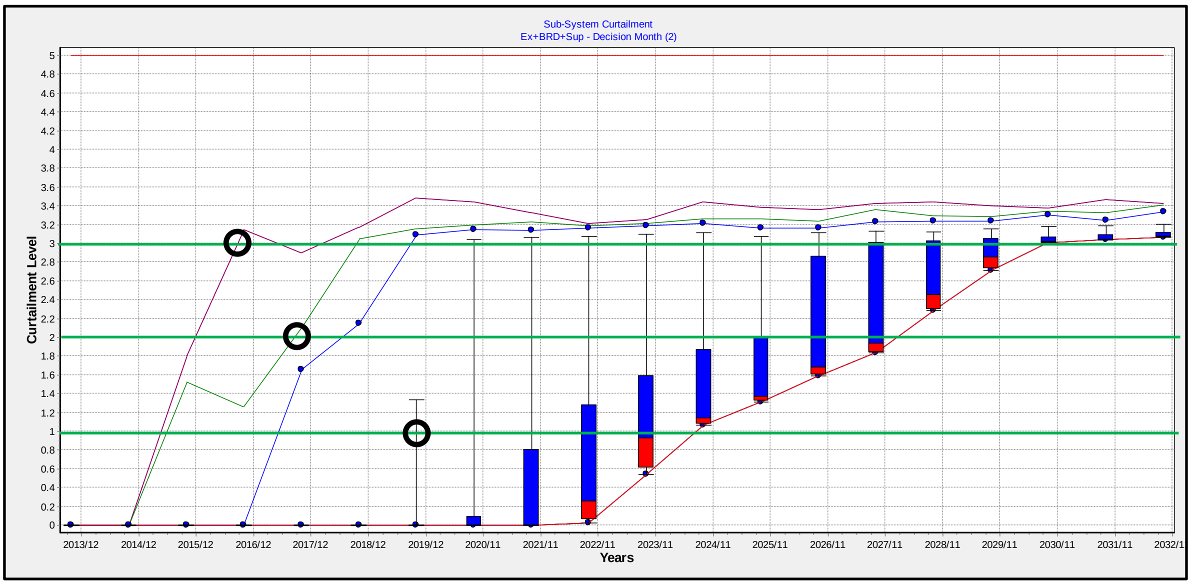

Figure D16: System curtailment with 150 Mℓ/day desalination plant only operational when system storage drops below 14 percent (Scenario D)

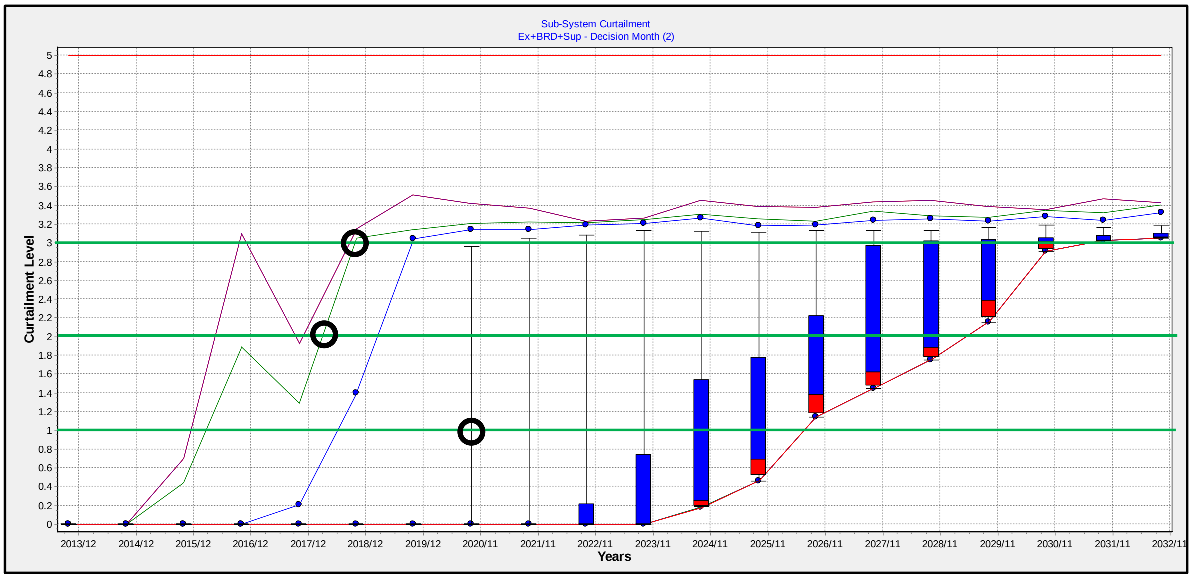

Figure D17: System curtailment with 300 Mℓ/day desalination plant only operational when system storage drops below 14 percent (Scenario D)

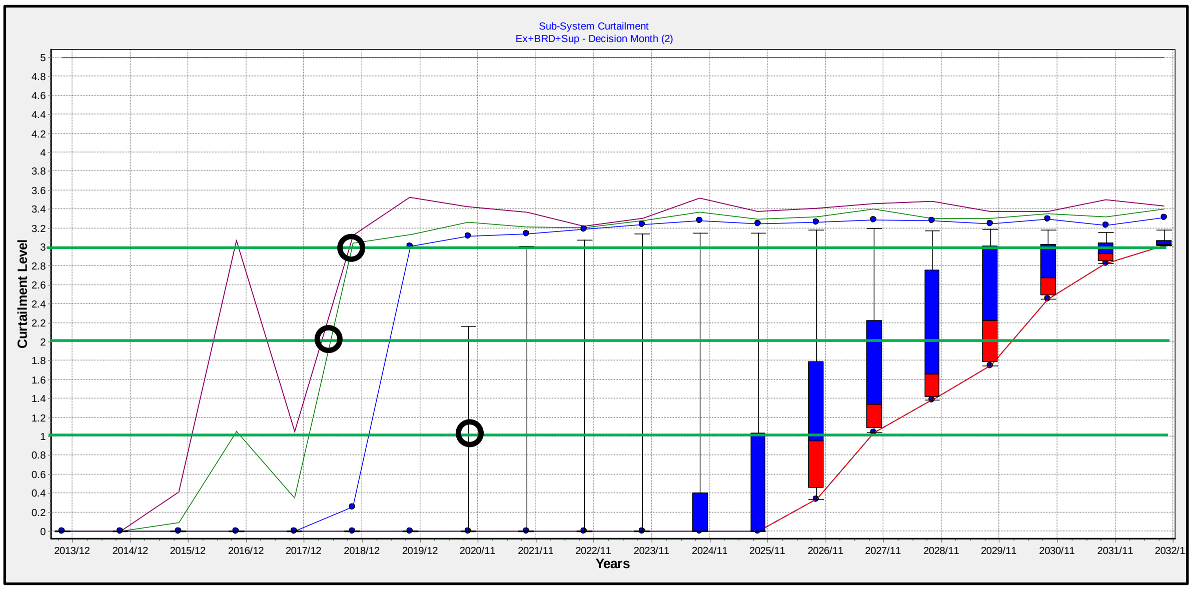

Figure D18: System curtailment with 450 Mℓ/day desalination plant only operational when system storage drops below 14 percent (Scenario D)

APPENDIX E: UNIT REFERENCE VALUES

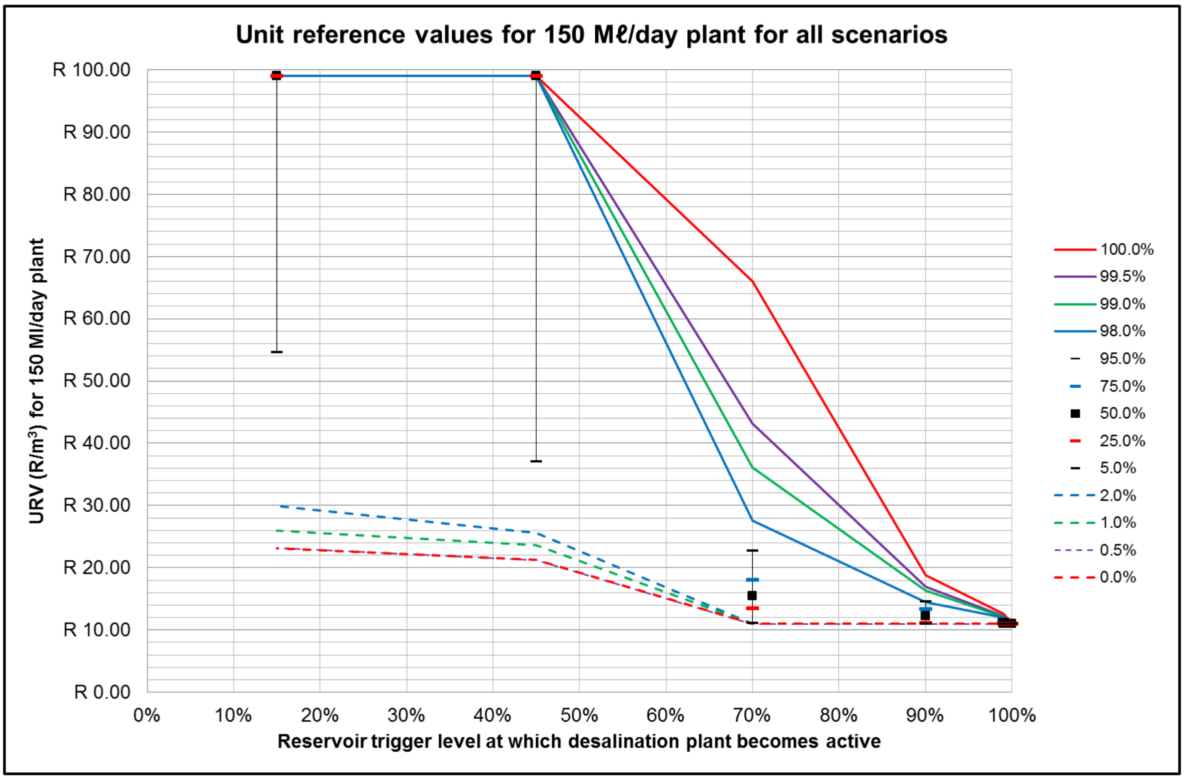

Figure E1: Box-and-whisker plot of unit reference values for a 150 Mℓ/day desalination plant

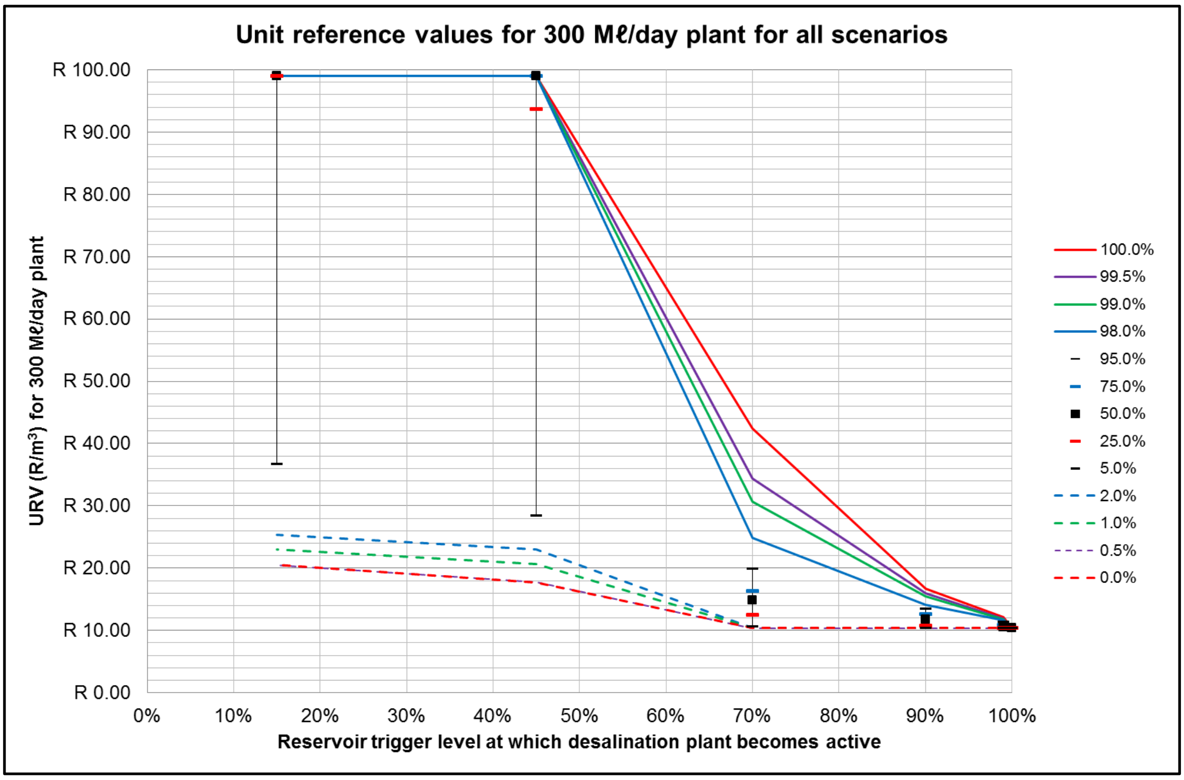

Figure E2: Box-and-whisker plot of unit reference values for a 300 Mℓ/day desalination plant

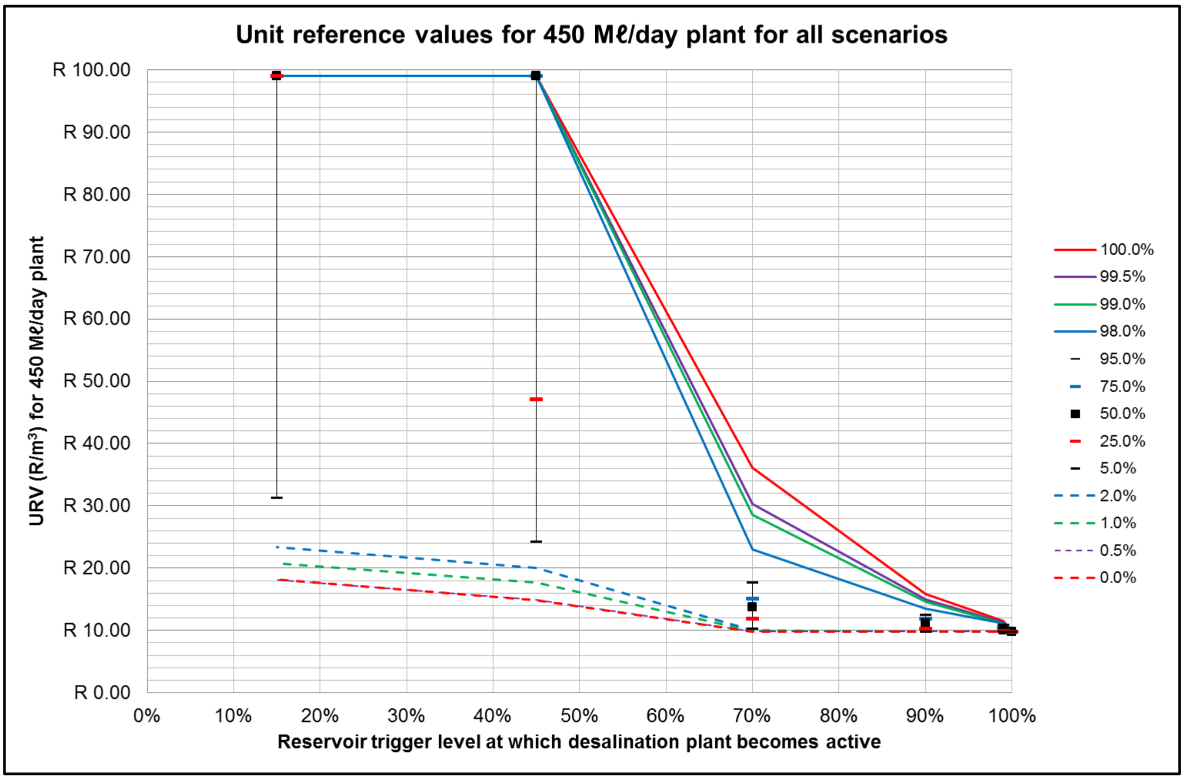

Figure E3: Box-and-whisker plot of unit reference values for a 450 Mℓ/day desalination plant