A Decision Support Framework for identifying optimal water supply portfolios: Metropolitan Adelaide Case Study

Volume 1: Main Report

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Executive summary

The Urban Water Blueprint for Metropolitan Adelaide is an initiative to demonstrate the South Australian Government's commitment to adopt integrated urban water management (IUWM). It aims to provide a plan that incorporates IUWM principles (i.e. an IUWM plan) to guide policy reforms and infrastructure investment priorities for the diversification of water supplies for Metropolitan Adelaide in a cost-effective, socially acceptable and environmentally sustainable manner.

This study has been initiated by the South Australian Government through the Goyder Institute for Water Research, as part of the Optimal Water Resources Mix project (October 2012–May 2014), to inform policy questions related to the Urban Water Blueprint. The objectives of this study are to:

- provide a method to identify the most cost-effective and environmentally sustainable mix of water sources to meet current and future potable and non-potable water demands in a given town/city, in a manner that was acceptable to the community living in that town/city
- demonstrate the applicability of the method to Metropolitan Adelaide by considering current (2013), 2025 and 2050 potable and non-potable water demands, and by identifying the most cost-effective and environmentally sustainable portfolios of water sources to meet these demands, in a manner that was acceptable to the community living in Metropolitan Adelaide.

The study has used a systems analysis approach, in particular a combined simulation-optimisation approach, to develop a multi-objective decision making framework for the evaluation and selection of optimal water sources for cities and towns, in terms of a defined set of objectives. It is called the Integrated Urban Water Management Decision Support Framework (IUWM DSF). The Framework takes into account technical, economic, environmental and social factors in assessing combinations of traditional and alternative water sources. The utility of the Framework has been demonstrated by applying it to a case study of planning future water resources for Metropolitan Adelaide for the period up to 2050. The specific years considered for planning are 2013 (i.e. current), 2025 and 2050.

The IUWM DSF uses the National Hydrologic Modelling Platform (NHMP) in Australia, i.e. the Source model developed as part of the eWater Cooperative Research Centre, and a number of other models to implement the combined simulation and optimisation approach. The optimisation component has been implemented using the Insight module, which provides a multi-objective genetic algorithm approach to search for the optimal solutions. The simulation component has been implemented in the Schematic and Catchment modules. The simulation component is supported by a monthly multiple regression model for predicting monthly water demands, CSIRO's Water Partition Balance Model (WAPABA) for predicting monthly inflows to surface water reservoirs in Mount Lofty Ranges, a stochastic rainwater tank model for quantifying the expected yield and runoff capture aspects from rainwater tanks and a stochastic model for predicting wastewater inflows to wastewater treatment plants, over the time period of simulation. All supporting models have climatic factors such as rainfall, evaporation and/or temperature as independent variables to account for the variability of climate and its impact on supply and discharge dynamics. This is the first study in Australia in which the NHMP has been applied to inform policy questions related to an IUWM plan being developed at a major city scale.

The water sources considered in the case study are: surface reservoirs in the Mount Lofty Ranges, pumping water from the River Murray, desalination, harvested stormwater (for non-potable use), reclaimed wastewater (for non-potable use), groundwater, rainwater (from household tanks for non-potable use and hot water use) and demand management. The monthly per capita water demand of Metropolitan Adelaide has been modelled using SA Water's Cooling-Degree-Day-12 monthly demand model. The modelled monthly water demands have been multiplied by the population in 2013, 2025 and 2050, to determine the water demand corresponding to the planning years. The total water demand has then been split into residential and non-residential, potable and non-potable. Of the total demand, 42% is for potable use and 58% is for non-potable use. The computed average annual total demand by considering the variability of climate over a 30-year period (July 1983 to June 2013) for 2013, 2025 and 2050 planning years are 173 GL, 183 GL and 213 GL, respectively.

The simulation model incorporated in the IUWM DSF includes all the sources and demands mentioned above, except the groundwater source. This is because of the unavailability of sufficient data on both spatial and temporal extents of the current use of groundwater in Metropolitan Adelaide. The simulation model also includes the capital and operational cost of infrastructure required to supply fit-for-purpose water from sources to demands, the embodied and operational energy consumed when supplying water and the methods to account for supply, demand and discharge interactions associated with supplying water from the sources to the demands. The level of decentralisation considered in the simulation model includes, disaggregation of the water demand of Metropolitan Adelaide to three demand zones, recycled water supply from 3 wastewater treatment plants (i.e. Bolivar, Glenelg and Christies Beach), 70 stormwater harvesting schemes spread across Metropolitan Adelaide and household scale rainwater tanks and demand management measures. The stormwater schemes are lumped into 25 Aquifer Storage and Recovery (ASR) schemes. Statistical up scaling methods have been employed to quantify water quantity related impacts of household scale sources. The simulation model also incorporates the preferences of consumers and stakeholders (collated as part of the Optimal Water Resources Mix project) when setting priorities on supply from the various sources. However, it should be noted that the dataset on preferences may not be fully representative to metropolitan Adelaide because data collection is limited to a defined number of focus groups.

Linking of this simulation model with the multi-objective optimisation capabilities available in the NHMP has allowed identifying a range of efficient solutions and trade-offs between the objectives, for 2013, 2025 and 2050, in terms of four objectives. These objectives, subject to a number of constraints, are:

- the present value of capital and operational costs of infrastructure needed to supply water and treat the wastewater
- total energy usage which is the sum of embodied energy and operational energy
- the volumetric reliability of the non-potable network
- the total discharge of wastewater and stormwater to Gulf St Vincent.

The constraints ensure that:

- at least 99.5% time-based reliability for the potable network is met
- the environmental flow requirements are fully met
- the amount of water pumped from the River Murray does not exceed 650 GL/year over any consecutive 5-year period.

Note that costs assessed in this study are relative costs that include the capital cost and maintenance cost of new infrastructure and the operating cost of existing and new infrastructure. The capital cost of existing infrastructure at the time of the study (2013) has not been included as it is considered to be a sunk cost. Similarly the maintenance cost of existing infrastructure has not been included as it will be required regardless of the combination of new options chosen.

Cost and energy have been based on a 25-year period of analysis, whereas the evaluation of system reliability has been based on monthly simulation over 30 years. However, the simulation model contains datasets to enable the simulation to be carried out for 50 years. Of the seven supply options considered, only five have been included in the multi-objective optimisation (the River Murray, Mount Lofty Ranges, desalination plant, recycled water and stormwater). Rainwater tanks and demand management options have not been included in the multi-objective due to limited time availability to undertake optimisation. However, the potential impact of rainwater tanks and demand management options on the above mentioned objectives has been quantified as part of the simulation.

Specific conclusions from the Case Study include:

- The Mount Lofty Ranges catchments are generally the preferred source for potable water supply due to their low cost and energy. Water from the River Murray is generally the second choice for potable use. The desalination plant is used primarily as a backup supply in dry years.
- Apart from the use of water from the Mount Lofty Ranges catchments, lower cost solutions favour the use of more River Murray water for non-potable use while solutions with lower discharge to Gulf St Vincent favour the use of treated wastewater and harvested stormwater. Also, treated wastewater is a more cost-effective option for reducing discharge to the Gulf than harvested stormwater.
- Rainwater tanks have the potential to reduce the demand for water from other sources by about 12% (if 100% uptake is assumed), which is a significant contribution to Metropolitan Adelaide's water supply. However, rainwater tanks are not a cost-effective solution as they are expensive and energy intensive compared to the other sources. In addition, it should

be noted that 100% uptake rate may not be practical. Nevertheless, this assumption allows estimating the maximum water saving potential from rainwater tanks. Further, it should be noted that the availability of representative data to quantify yield and discharge implications of rainwater tanks was limited at the time of conducting the study, in particular datasets on household demands at the end use scale. Therefore, conclusions regarding rainwater tanks should be used as indicative and proof-of-concept purposes only.

- The use of demand management options for in-house appliances such as low flow shower heads, front loading washing machines and dual flush toilets have the potential to reduce total water consumption by about 5%.
- The trade-offs developed between cost and energy show that the minimum cost solution is not the minimum energy solution and vice versa. There is a marked trade-off between cost and discharge of wastewater and stormwater to the Gulf, with reduced discharges requiring significant investment in capital and operational costs.

The case study has been undertaken as a 'proof-of-concept'. Overall, the case study has demonstrated the ability of an IUWM DSF based on a combined simulation and optimisation approach to identify efficient portfolios of supply sources and the trade-offs associated with them, by taking into account a large number of objectives, constraints and options, when planning water resources for a diversified urban water system.

The case study suffers from a number of limitations. Some are due to the unavailability of representative datasets and some are due to limitations present in the version of NHMP used for this study. Therefore, the following recommendations are made with regard to further research:

- Expand the objective related to financial cost to include the benefits of supplying additional water to such end users as industry, agriculture, residential users and for green space. Consider including an ecosystem services approach that takes into account emerging datasets on willingness to pay for improving water quality in the Gulf St Vincent.
- Include the emerging new datasets on the future climate, residential end uses, potential non-potable users and groundwater for identifying the optimal supply portfolios.
- Evaluate the effects of lumping of the storages in MLR and the existing and new stormwater schemes to identify the extent to which the lumping affects the objectives considered in the study.
- Assess the technical feasibility, cost-effectiveness, public health and environmental effects of using treated stormwater and treated wastewater for potable uses. Any such use would need to be compatible with the SA Water drinking water guidelines.
- Improve the methods used for estimating the environmental impacts on Gulf St Vincent either through modelling water quality directly and/or modelling the ecological impact.
- Include rainwater tanks and demand management as options in the multi-objective optimisation. Improve the results related to rainwater tanks and demand management by using representative datasets for Metropolitan Adelaide, in particular utilise the household

demand data collated as part of the Optimal Water Resource Mix: Task 4, to provide improved estimates for tank yield and demand reductions due to demand management.

- Develop an improved version of the simulation-optimisation model by using the latest version of NHMP. This will allow the priorities of the individual sources to be optimised, instead of setting the priorities manually. The latter approach has been used in the current study and it has introduced a bias when supplying water from a multiple set of sources.

The further work described above will enhance the outputs of the Metropolitan Adelaide case study. This will result in more robust outcomes than the current study, which has been undertaken as a demonstration or a 'proof-of-concept' of the IUWM DSF.

1 Introduction

1.1 Background

South Australia is the driest state in Australia. Ninety percent of the state has an average rainfall of less than 300 mm per year and it can be highly variable (Cullen, 2004). The average annual rainfall in Adelaide is 544 mm. The public water supply side of the urban water cycle in Metropolitan Adelaide draws on a diverse network of sources including surface water from ten reservoirs spread throughout the Mount Lofty Ranges (MLR) with supplementary water from the River Murray (RM), and more recently (since late 2011) desalinated seawater from the Adelaide Desalination Plant (ADP). Adelaide's urban water cycle also features utilisation of 26% of reclaimed wastewater for non-potable uses, which is the highest amongst Australian utilities for recycled water use (National Water Commission, 2013), and an approximate annual volume of 5 GL of harvested stormwater (Personal communication with Steve Gatti, AMLR NRM Board, April 2014). Both recycled water and harvested stormwater is currently utilised for non-potable uses such as open space irrigation and peri-urban irrigation. Adelaide also features 44% of households with rainwater tanks, which is the second highest amongst Australian major cities (Brisbane has the highest number of rainwater tanks, i.e. 47%) (Australian Bureau of Statistics, 2013). Some households use rainwater for garden use only, whereas some households have internally plumbed rainwater tanks for such uses as toilet flushing, clothes washing, hot water and garden watering.

The traditional practice of urban water management in Metropolitan Adelaide has been challenged in recent years. This is due to a number of emerging needs for ensuring:

- water security for a growing urban population in a changing climate, which is predicted to be drier and warmer than the historical climate (Government of South Australia, 2007)
- healthy inland waterways and coastal waters (Government of South Australia, 2013)
- liveable and productive urban environments (Government of South Australia, 2010a).

The South Australian Government has identified integrated urban water management (IUWM), as a possible solution to address these needs. The government's commitment to implement integrated urban water management principles has been demonstrated through the actions identified in the Water for Good Strategy (The Government of South Australia, 2010a) and the 30-Year Plan for Greater Adelaide (The Government of South Australia, 2010b), as well as an initiative to integrate water and recycled wastewater planning (called the 'Urban Water Blueprint') for Metropolitan Adelaide.

The Urban Water Blueprint is aimed at providing a comprehensive overview of the urban water environment of Adelaide, and setting a broad and agreed vision for urban water management for Adelaide. It also aims to set infrastructure investment priorities and policy reforms, especially in relation to the investments in stormwater harvesting and wastewater recycling. This is to ensure that the urban water management can contribute to achieving significant social, economic and environmental benefits.

A number of projects were initiated by the South Australian Government through the Goyder Water Research Institute, a partnership between the three universities based in South Australia and the CSIRO Australia, to inform the development of the Urban Water Blueprint. One such project was the Optimal Water Resources Mix (OWRM) project. The objective of the OWRM was to provide some fundamental foundation knowledge required to develop the Urban Water Blueprint, which included an improved knowledge on water use, sustainable use of available water sources and the current infrastructure base, externalities associated with various water sources and the community preferences in relation to the use of available sources. To fulfil this objective, the OWRM project was formulated with seven key tasks. These tasks were aimed at:

- Engaging with stakeholders to ensure methods and outputs were acceptable to the stakeholders (Task 1)
- Developing a modelling capability to predict both supply security and stormwater and treated wastewater discharges to the Gulf, when utilising different mix of fit-for-purpose water sources (Task 2)
- Understanding how a multi-objective optimisation based approach can be used to identify efficient and sustainable solutions (Task 3)
- Understanding and predicting household water use for individual households (Task 4)
- Understanding governance structures being adopted by other cities around the world to support integrated urban water management (Task 5)
- Understanding how the community in Metropolitan Adelaide value healthy coastal waters and green space (Task 6)
- Understanding the community preferences for supplying water from a diversified portfolio of water sources (Task 7).

This report describes the technical work undertaken as part of Tasks 2 and 3.

1.2 Objectives

Task 2 and Task 3 were integrated seamlessly to achieve the following objectives:

- to provide a method to identify the most cost-effective and environmentally sustainable mix of water sources to meet potable and non-potable water demands in a given town/city, in a manner that was acceptable to the community living in that town/city
- to demonstrate the applicability of the method to Metropolitan Adelaide by considering current (i.e. 2013), 2025 and 2050 potable and non-potable water demands and by identifying the most cost-effective and environmentally sustainable portfolios of water sources to meet these demands in a manner that was acceptable to the community in Metropolitan Adelaide.

The sources to be considered were:

- River Murray
- surface water from Mount Lofty Ranges (MLR)
- desalinated sea water from the Adelaide Desalination Plant (ADP)
- recycled water from Bolivar, Glenelg and Christies Beach wastewater treatment plants
- harvested stormwater
- rainwater collected in tanks
- groundwater.

In addition to the above sources, water savings from various demand management options were to be considered.

1.3 Report structure

Chapter 1 describes the context, background and objectives of the work described in this report.

Chapter 2 describes the methodology adopted to develop the Integrated Urban Water Management (IUWM) Decision Support Framework (DSF).

Chapter 3 describes the study area (Metropolitan Adelaide) in terms of key parameters such as climate, land uses, population and households.

Chapter 4 describes a demonstration of how to apply IUWM DSF to Metropolitan Adelaide. This includes formulating the water supply issue in Metropolitan Adelaide and demonstration of combined simulation-optimisation approach to identify optimal solutions to this water supply issue, in terms of a defined set of objectives.

Chapter 5 describes results of the study and provides a discussion of the results.

Chapter 6 provides conclusions and describes recommendations on further work that could be undertaken to improve the quality of the outputs.

Appendices are contained in a separate Volume 2.

2 Methodology

The systems analysis method is a widely used approach to inform water resources planning (Biswas, 1976; Loucks et al., 1981; Loucks and van Beek, 2005). It can be used to inform planning studies on both traditional urban water management and integrated urban water management (IUWM) (Maheepala et al., 2010). When the systems analysis method is used to inform IUWM planning studies, it incorporates IUWM principles (Burn et al., 2012), which include minimising usage of resources to provide urban water services (e.g. fresh water, energy, materials); minimising wastes generated from the urban water system through recovering resources from wastes; enhancing liveability by providing acceptable levels of service; and improving the wellbeing of ecosystems.

Several studies have incorporated IUWM principles into systems analysis to assist in identifying the most efficient water management options. The spatial scale of these studies varies from development scale (i.e. a suburb or may be a couple of suburbs) to local government scale. The development scale studies include the studies by Diaper and Maheepala (2003), Grant et al., (2006), Maheepala et al., (2006), Sharma et al., (2009) and Harnett et al., (2009). The local government scale/regional town studies include the studies by Kirono et al., (2013), WBM BMT(2012), and Paton et al. (2009).

In contrast to the past studies, this study has used the systems analysis method to develop a generic methodology (or a framework) to inform the identification of the most cost-effective, environmentally sustainable and socially acceptable mix of water sources to meet water demands for a major city. Hence the spatial scale of this study is a city. The purpose of this framework is to inform the policy questions related to development of an IUWM plan for a city. Hence this framework is called 'IUWM Decision Support Framework (DSF)'.

The IUWM DSF required measurable objectives to assess the cost effectiveness, environmental sustainability and social acceptability, which were expected to be defined as part of adopting the IUWM DSF. The particular systems analysis techniques used in the IUWM DSF were simulation and multi-objective optimisation. These two techniques were used both as standalone approaches and as a combined approach in the past, for water supply systems with multiple reservoirs to inform both long-term and operational planning (Labadie, 2004; Rani and Moreira, 2010). In the IUWM DSF, a combined simulation-optimisation approach was adopted to search for the solutions that could best meet a defined set of objectives.

The IUWM DSF comprised eight components (Figure 1), and they were:

1. **Identify overall goals**, i.e. identifying the purpose of applying the IUWM DSF. For example, for this study, the purpose of applying the IUWM DSF was to inform policy questions related to the development of Urban Water Blueprint or IUWM Plan for Metropolitan Adelaide, e.g. what are the most cost effective, environmentally

sustainable and socially acceptable water supply sources to meet current and future demands of Metropolitan Adelaide?;

2. **Formulate the problem**, i.e. formulating a problem to achieve the overall goal. For example, for this study, the problem could be defined as, how could different portfolios of water supply be evaluated to identify the optimal portfolio in terms of a set of defined objectives?;
3. **Identify objectives, decision variables and constraints**, i.e. defining the objectives to measure the achievability of goals, identifying influencing variables of the objectives, and identifying the limits that defined the scope of problem;
4. **Translate objectives and constraints into measurable criteria**, i.e. defining a metric (or set of metrics) to facilitate quantification of each objective (e.g. metrics related to the objective on environmental sustainability could be energy consumption and discharge of stormwater and wastewater to receiving waters)
5. **Identify alternative options**, i.e. identifying the opportunities or options to improve the system in terms of the goals
6. **Evaluate alternative options in terms of the measurable criteria**, i.e. use of appropriate techniques to quantify the metric of each objective (e.g. hydrological modelling to quantify stormwater discharging to receiving waters);
7. **Identify the efficient options using multi-objective optimisation**, i.e. use of an appropriate optimisation technique (e.g. genetic algorithm and linear programming) to identify solutions that best meet the objectives; and
8. **Select preferred options**, i.e. selecting preferred solutions from a large number of optimal solutions (i.e. output of #7), considering preferences and values of stakeholders and using an appropriate technique such as multi-criteria analysis

Application of the IUWM DSF to Metropolitan Adelaide is described in Chapter 4. The application study demonstrated components #1 to #7 only. Component #8 was not included in the current study due to limitations in funding. The key characteristics of the study area are described in the next chapter, before describing the application of the IUWM DSF.

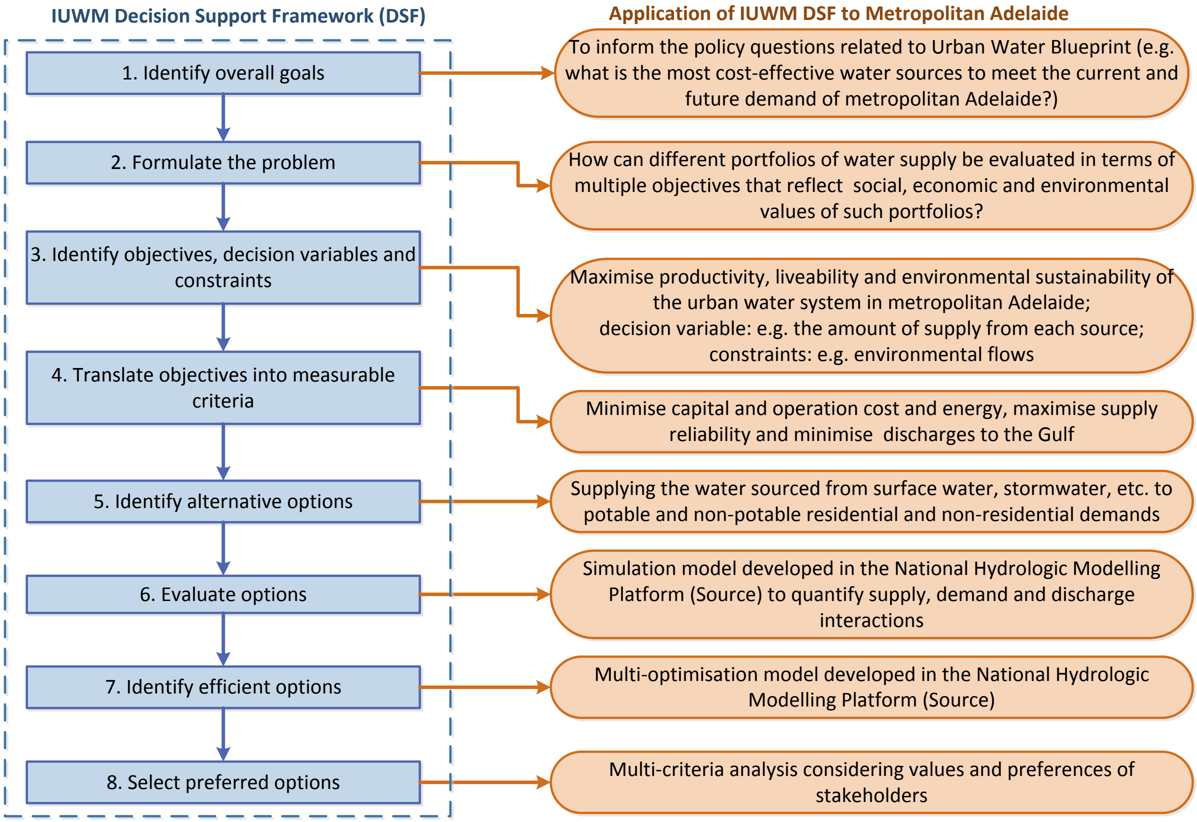


Figure 1: Key components of the IUWM Decision Support Framework and a high level description of its application to Metropolitan Adelaide

3 Study area

This chapter describes the study area of the IUWM DSF application, in terms of the key variables used to describe the study area (i.e. population, land uses and climate); the demand for water generating from the study area; and the water supply sources currently used to meet the water demand of the study area.

3.1 Geographic extent

The area covered by Metropolitan Adelaide generally extends from the north of the town of Gawler in the north to Sellicks beach in the City of Onkaparinga in the south, and from the east of the towns of Bridgewater and One Tree Hill in the east, to the east coast of the Gulf St. Vincent. The study area included a majority of the area covered by Metropolitan Adelaide and the major growth areas located outside the Gawler local government area, i.e. Concordia and Roseworthy growth areas, and excluded a portion area governed by Adelaide Hills Local Government, between Kangaroo Creek and Mount Bold reservoirs (Figure 2). In this report, the study area is referred as 'Metropolitan Adelaide'.

3.2 Climate

Metropolitan Adelaide has a hot Mediterranean climate, which is characterised by mild winters with moderate rainfall and hot, dry summers. The mean maximum temperature during summer months (December to February) is 29 ^{0}C, but there is a considerable variation in temperature and generally, there is at least one day in each year with the daily temperature of 40 ^{0}C, or above (Suppiah et al., 2006). The mean minimum temperature during winter months (June to August) is 15 ^{0}C (Figure 3). The mean annual rainfall is 544 mm (from 1977 to 2014). The monthly rainfall varies from 15 mm in February to 79 mm in June (Figure 4).

The application of the IUWM DSF to the Metropolitan Adelaide case study required consideration of the current climate (defined as 2013) and the expected climate in 2025 and 2050. The variability of the current climate was represented by considering the climate over the period of 50 years, from 1963 to 2013. The Goyder Research Institute funded project on climate change was consulted to obtain the variability of climate expected for 2025 and 2050. However, climate projections for 2025 and 2050 were not available at the time of undertaking the case study. Hence the potential changes to required climate parameters were obtained from CSIRO's OzClim Climate Scenario Generator, available at: http://www.csiro.au/ozclim/home.do, which provided approximate values for the expected changes to climatic parameters in a defined year, compared to the year 1990 (Table 1). The values were obtained in the form of average annual and average seasonal values (for winter, summer, autumn and spring).

Figure 2: Geographic extent of the study area

Table 1: Metropolitan Adelaide annual rainfall and temperature in 2025 and 2050 (source: CSIRO OzClim Climate Scenario Generator)

Year	2025	2050
Reduction in rainfall compared to 1990	19.9%	38%
Reduction in rainfall compared to 2013 (computed by using linear interpolation)	6.82%	23.43%
Change in temperature compared to 1990 (^{0}C)	0.52	1.06
Change in temperature compared to 1990 (^{0}C) (computed by using linear interpolation)	0.18	0.65

The expected changes in annual rainfall and the average annual temperature in year 2025 were a 19.9% reduction and a 0.52 ^{0}C increase, respectively (Table 1), compared to 1990. In the year 2050, the expected changes rainfall and the average annual temperature were a 38% reduction and a 1.06 ^{0}C increase, respectively (Table 1), compared to 1990. Linear interpolation was used to estimate the changes in rainfall and temperature, compared to the current climate, i.e. 2013. Changes compared to 2013 were used to develop datasets corresponding to 2025 and 2050.

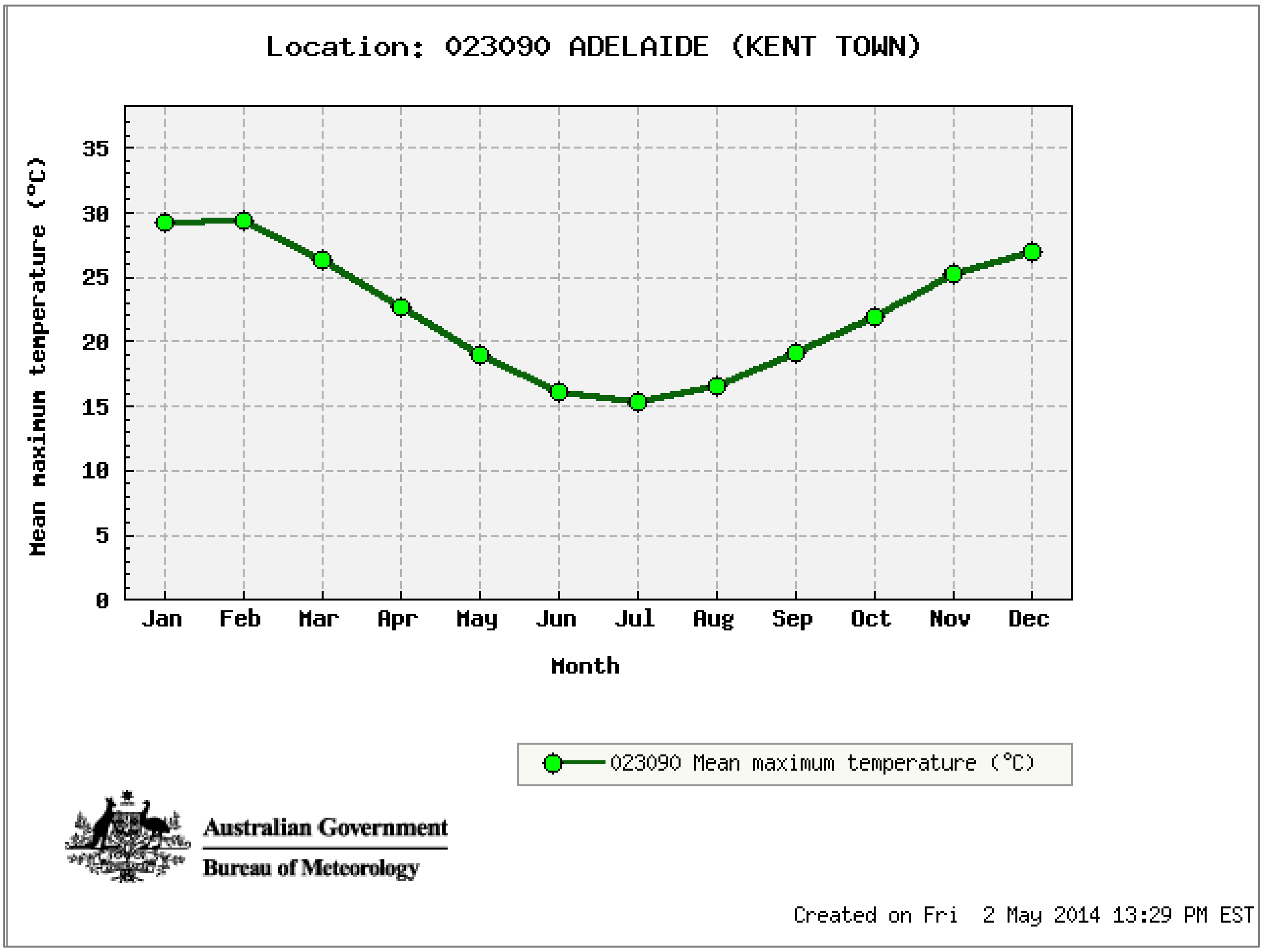


Figure 3: Mean temperature distribution in Adelaide (source: Bureau of Meteorology; station 023090 Kent Town, using the observed records from 1977 January to 2014 May)

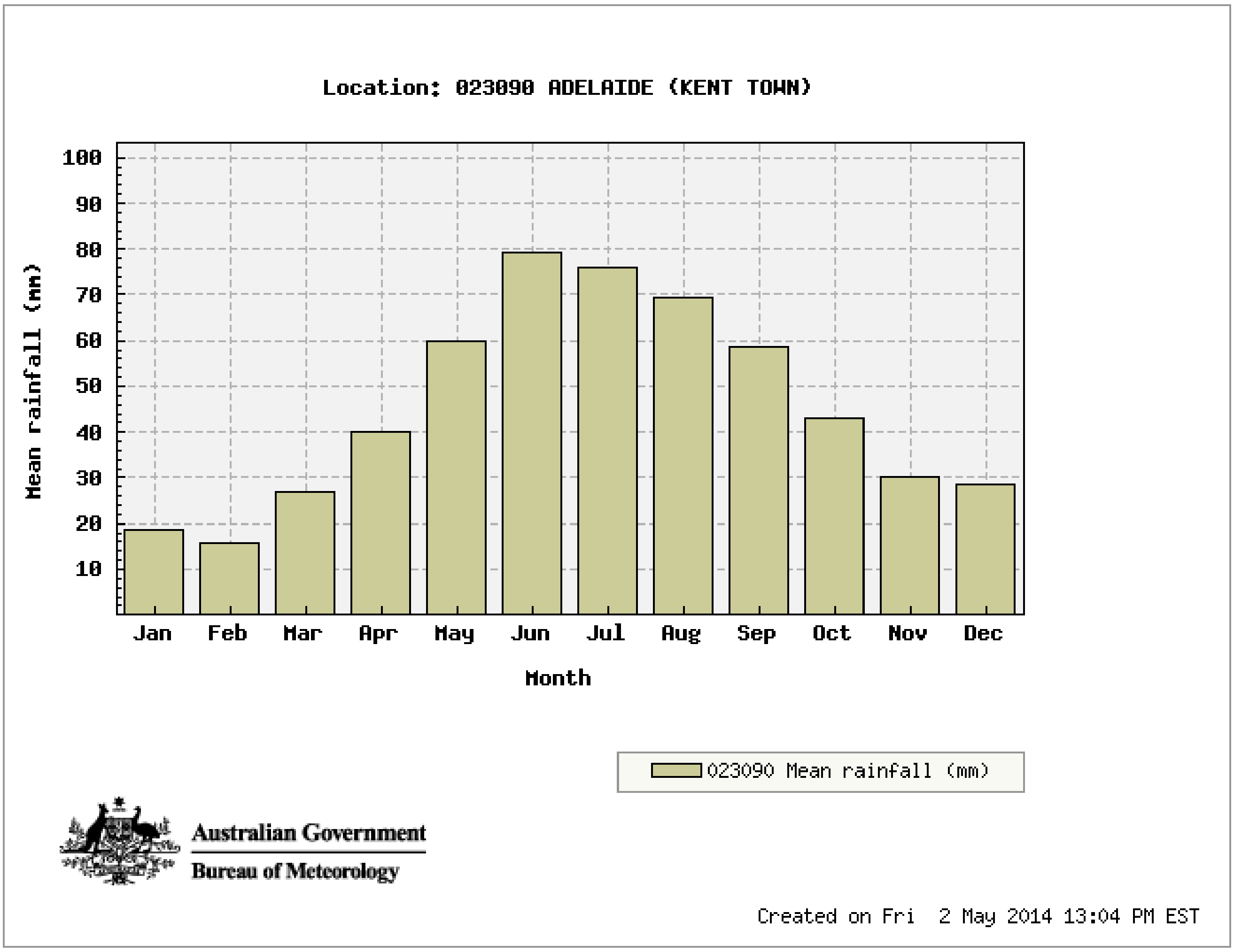


Figure 4: Average monthly rainfall distribution in Adelaide (source: Bureau of Meteorology; station 023090 Kent Town, using the observed records from 1977 January to 2014 May)

3.3 Water sources

Historically, Metropolitan Adelaide has relied heavily on surface water from the Mount Lofty Ranges (MLR) supplemented with water extracted from the River Murray (RM). Since late 2011, these water sources have been supplemented with desalinated seawater from the Adelaide Desalination Plant (ADP).

Water from the MLR, the RM and the ADP is drawn from a diverse network of infrastructure, consisting of ten storages in MLR, pumping stations extracting water from the RM, pipelines transferring water from the RM to the storages in MLR, pumping stations and pipelines to move water from north to south and south to north (i.e. north-south pipe interconnection) and pipe connections to link the ADP into the existing pipe network. The major supply zones, storages in MLR and water treatment plants are shown in Figure 5.

Figure 5: Major supply areas and storages in MLR (source: Spies and Dandy, 2012)

The ten storages in the MLR spread across five major catchments:

- South Para catchment, which contains South Para, Warren and Barossa Reservoirs
- Little Para catchment, which contains Little Para Reservoir
- River Torrens catchment, which contains Millbrook, Kangaroo Creek and Hope Valley Reservoirs
- Onkaparinga catchment, which contains Mount Bold and Happy Valley Reservoirs
- Myponga catchment, which contains Myponga Reservoir.

The total runoff from the MLR catchments into the ten storages mentioned above is about 170 GL/year in an average year, of which 15 GL/year is lost by evaporation and 34 GL/year spills (Sustainable Focus and Clark, 2008). The MLR provide approximately 60% of water supply to Metropolitan Adelaide in an average year (EPA South Australia, 2005).

Of the ten storages mentioned above, nine storages were included in the study. Myponga reservoir was not included. This was because majority of the supply area of Myponga reservoir was located outside of the study area.

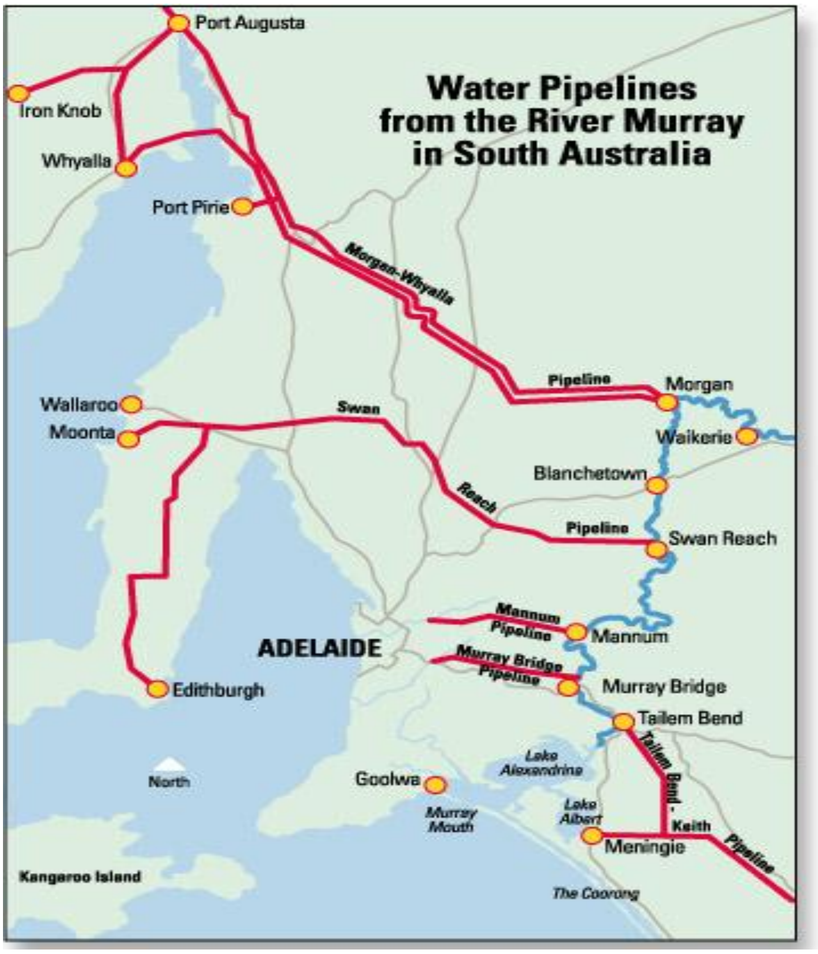


Figure 6: Pipelines to draw water from the River Murray in South Australia: water drawn from Murray Bridge, Mannum and Swan Reach is used to supply water to Metropolitan Adelaide (source: http://www.murrayriver.com.au/about-the-murray/water-use-and-consumption/)

Table 2: The stormwater harvesting schemes that are currently operational in Metropolitan Adelaide

	Stormwater harvesting scheme	Catchment[1]	Potential yield[1] in ML/year
1	Evanston South	Smiths Creek	185
2	Blakeview	Smiths Creek	308
3	Munno Para West	Smiths Creek	1241
4	Andrews Farm	Smiths Creek	400
5	Andrews Farm South	Smiths Creek	500
6	Edinburgh Parks North	Adams Creek	630
7	Edinburgh Parks South	Adams Creek	760
8	Kaurna Park	Adams Creek	551
9	Springbank Park	Adams Creek	398
10	Burton West	Adams Creek	308

	Stormwater harvesting scheme	Catchment[1]	Potential yield[1] in ML/year
11	Greater Edinburgh	Greater Edinburgh	1990
12	Glenelg Golf Course	Brownhill/Kewswick Creek	460
13	Pooraka Upgrade	Dry Creek	1360
14	Madeira	Christie Creek	153
15	Brodie Road	Christie Creek	655
16	Morrow Road	Christie Creek	509
17	Wynn Vale Dam	Dry Creek	346
18	Montaque Road	Dry Creek	549
19	Paddocks	Dry Creek	584
20	Parafield	Dry Creek	862
21	Bennet Road Drain	Dry Creek	480
22	Greenfields 1&2	Dry Creek	3269
23	Reynella East	Field River	351
24	Royal Adelaide Golf Course	Port Road	200
25	Grange Golf Course	Port Road	300
26	Old Morphettville Racecourse	Sturt River	325

Note 1: Wallbridge and Gilbert (2009)

The pipelines that draw water from the RM are shown in Figure 6. Of the five pipelines, the following three pipelines supply water to Metropolitan Adelaide:

- Mannum-Adelaide pipeline
- Murray Bridge-Onkaparinga pipeline
- Swan Reach-Stockwell (SRS) pipeline.

On average, the River Murray provides about 40% of Adelaide's mains water. However, in a drought year this can be as high as 90% (Water Proofing Adelaide, 2005).

Adelaide's urban water cycle also features utilisation of 26% of treated wastewater for non-potable purposes, which is the highest amongst Australian utilities (National Water Commission, 2013). There are three major wastewater treatment plants (WWTP) in Metropolitan Adelaide. These are located at Bolivar, Glenelg and Christies Beach. Bolivar WWTP now processes almost 70% of Metropolitan Adelaide's wastewater. Recycled water is used to supplement non-potable water consumption in a few residential developments and some major industries including peri-urban agriculture. At present, there are about 26 stormwater harvesting schemes currently operational (Table 2), which provides water to open space irrigation, golf courses and sporting grounds. The potential yield of the operational schemes in 2013 is about 17.6 GL/year (Table 2), which may be slightly conservative because the expected harvesting amount by 2013 is about 20 GL/year (Government of South Australia 2010a and 2010c). However, as mentioned in Chapter 1, the amount of harvested stormwater currently being used is about 5 GL.

3.4 Population and households

Adelaide is the fifth largest city in Australia. The population in the Greater Adelaide Region, which is slightly larger than the study area, is 1.225 million and the average household size is 2.4 (Australian Bureau of Statistics, 2011). The population of the Greater Adelaide Region was taken as the population of the study area. Based on Australian Bureau of Statistics (2011) projection of the median scenario, the population of Adelaide is likely to increase to 1.56 million in 2050. Using a linear interpolation, the population of Adelaide in 2013 and 2025 can be estimated as 1.23 and 1.35 million, respectively (Paton et al., 2013). The population of the study area in 2013, 2025 and 2050 are summarised in Table 3.

The household size in South Australia is projected to decline to between 2.0 and 2.2 per household by 2026 due to the increase in the number of single person households (Trewin, 2004). Using linear interpolation, the estimated household size in Adelaide is expected to be 2.39, 2.21 and 1.88 people per household in 2013, 2025 and 2050, respectively. The number of households in the study area was computed by dividing the population, by the average household size. The number of households computed for 2013, 2025 and 2050 are also shown in Table 3.

Table 3: Population, number of households and average household size in the study area for 2013, 2025 and 2050

Year	2011	2013	2025	2050
Population (million)	1.225	1.23	1.35	1.56
Average household size (people/household)	2.4	2.39	2.21	1.88
Number of households	510,417	514,644	610,860	829,787

3.5 Land uses

For the purpose of defining land uses, hydrological catchments located within the study area (Figure 7) were considered. This was because the key purpose of defining land uses was to compute runoff generating from the study area through hydrologic modelling (this will be described in Chapter 4).

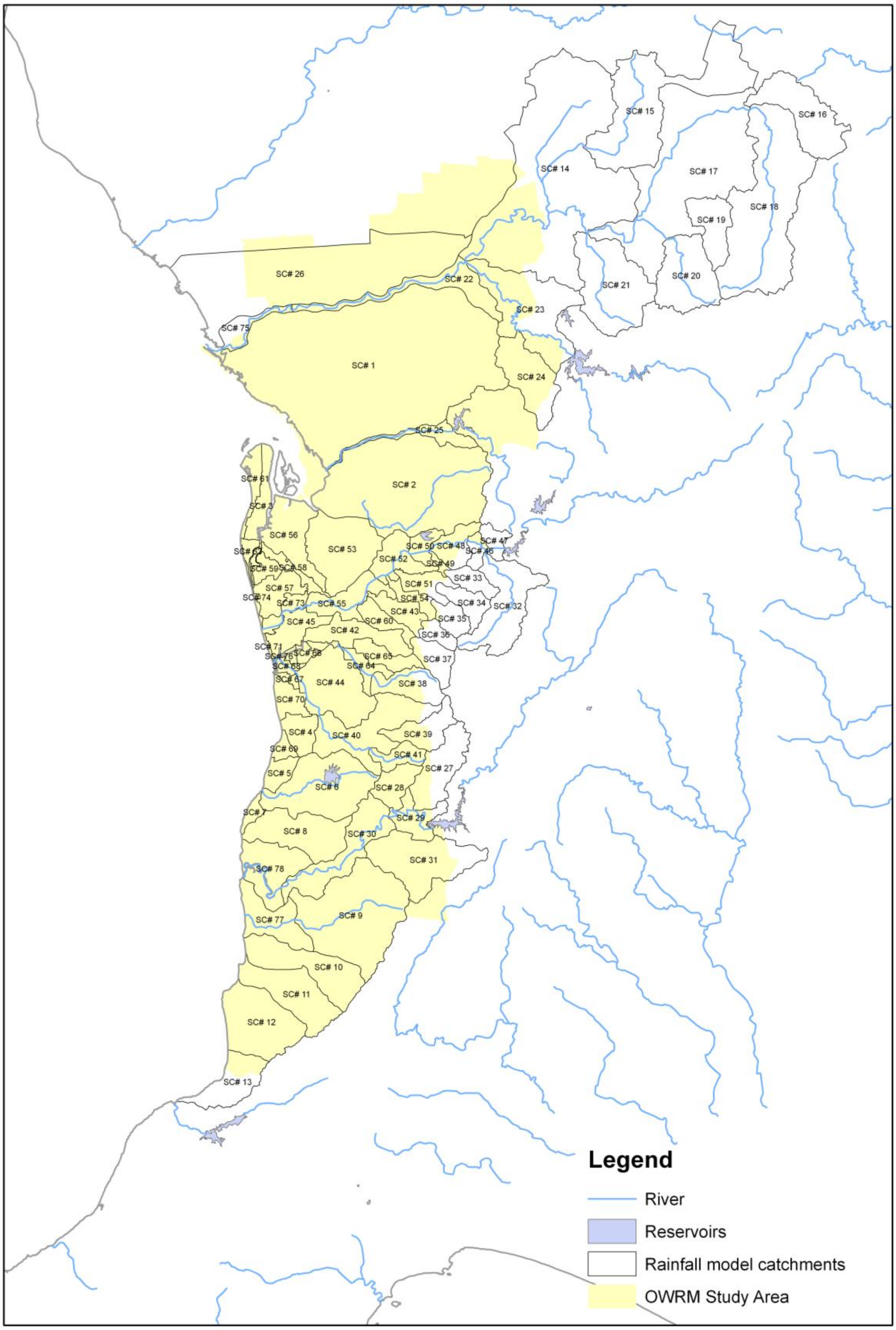


Figure 7: The extent of hydrological catchments included in the study area (note: SC# xx is a number given to sub-catchments for identification purposes)

Table 4: Classification of land uses in the study area[1]

Classification as per 2012 land use layer[2]	Classification used in OWRM study	Area (ha)	% of Land use
COMMERCIAL	Commercial	2,699	1.01%
EDUCATION	Commercial	2,209	0.82%
PUB_INSTITUTION	Commercial	2,703	1.01%
RET_COMMERCIAL	Commercial	1,581	0.59%
SERVICES	Commercial	59	0.02%
FORESTRY	Forestry	28,274	10.54%
RESERVE	Forestry	5,970	2.23%
AGRICULTURE	Horticulture/Ag	26,574	9.91%
HORTICULTURE	Horticulture/Ag	30,244	11.28%
FOOD_INDUSTRY	Industry	849	0.32%
INDUSTRIAL	Industry	17	0.01%
UTIL_INDUSTRY	Industry	5,804	2.16%
LIVESTOCK	Livestock	62,128	23.17%
MINE_QUARRY	Mining	2,443	0.91%
GOLF	Open space	1,206	0.45%
RECREATION	Open space	3,299	1.23%
VACANT	Open space	3,147	1.17%
RESIDENTIAL NATIVE COVER	Open space	50	0.02%
ROAD	Road	21,679	8.08%
RURAL_RESID	Rural living	24,860	9.27%
NONPRIVATE_RESID	Urban	564	0.21%
RESIDENTIAL	Urban	30,984	11.55%
VACANT_RESID	Urban	3,719	1.39%
WWTP	WWTP	1,178	0.44%
BEACH	Water	10	0.00%
RESERVIORS	Water	241	0.09%
WATER	Water	5,657	2.11%
Total		**268,151**	**100.00%**

Note 1: Study area in the context of land uses includes the extent of area covered by the hydrological catchments in the study area (see Figure 7); Note 2: From the Department of Planning, Transport and Infrastructure

Land uses in the study area (Table 4) were determined by using the 2012 GIS land use layer generated by Department of Planning, Transport and Infrastructure (DPTI). The 2012 land use layer represented the most up to date land uses created based on the valuation information and the valuation parcel boundaries. Hence a parcel could have multiple valuations over it. It meant one polygon could have multiple land use classes associated with it, which could result in a duplicated area calculation. To fix this issue, the most recent aerial photo obtained from Nearmap (http://nearmap.com/au) was used to identify land uses. The extra land use classes were manually deleted. In addition, the 2012 land use layer had unmapped areas. The 2008 land use data, obtained from DEWNR was applied to fill the gaps. According to the data shown in Table 4, the proportion of land uses used for urban purposes (i.e. land uses classified as urban, commercial, industrial, open space and roads) is about 30% of the total area.

The future land uses related to 2025 and 2050 were created by adding the growth areas obtained from the Greater Adelaide 30-years Plan (Figure 8) to the 2012 land use layer obtained from the Department of Planning, Transport and Infrastructure. Changes to urban areas in 2025 and 2050 relative to 2012 land uses are shown in Table 5.

Table 5: Future urban growth areas

Location[1]	2012 Urban (ha)	2025 Urban (ha)	2050 Urban (ha)
SC #1	3596	6327	7540
SC #2	4952	6001	6001
SC #75	0	63	63
SC #14	478	492	1079
SC #26	107	218	518
SC #13	73	117	117
SC #22	307	346	346
SC #12	385	419	419
SC #25	216	222	222
SC #17	639	684	684
SC #21	108	118	118
SC #53	1832	1836	1836
SC #56	1045	1046	1046

Note 1: the location is defined in terms of the sub-catchment

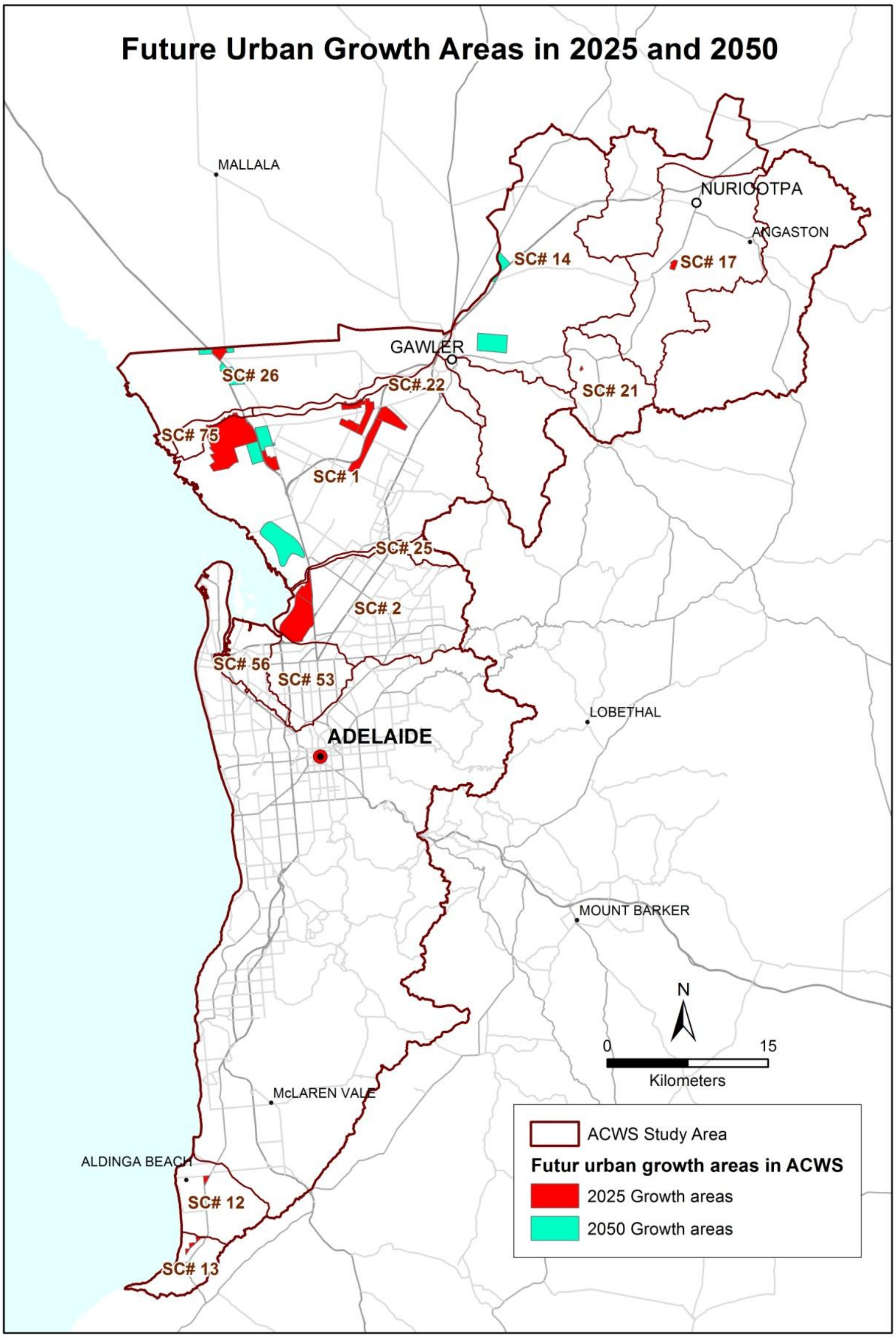


Figure 8: Future urban growth areas included in the study (source: 30-year Plan for the Greater Adelaide Region); Note: ACWS (Adelaide Coastal Water Study) study area encompasses the hydrologic catchments considered for the study

3.6 Water demand

Water demand generated from the study area was computed by considering the current demand, additional demand due to new developments, climate variability, climate change and potential reductions in demand due to demand management options currently in place. The modelling method used was SA Water's Cooling-Degree-Day-12 (or CDD12) monthly demand model. The CDD12 model was a multiple regression model of the following form:

$$D = 241.408 + 16.5N$$

Equation 1

Where, D was the monthly demand in litres per person per day (L/p/day) and N was the total number of days with daily average temperature greater than 12 ^{0}C, during the month being modelled. The daily average temperature is determined by dividing the sum of daily maximum and daily minimum temperatures by 2.

The daily maximum and minimum temperatures of BOM station 023090: Kent Town were used to predict the current (2013) monthly demand, over a 50-year period from July 1963 to June 2013, in L/p/day, which was then converted to ML/month by using the population figures shown in Table 3. The modelled monthly demand for 2013 in ML, is shown in Figure 9. The modelled annual demand over calendar years during 1964 to 2012 is shown in Figure 10.

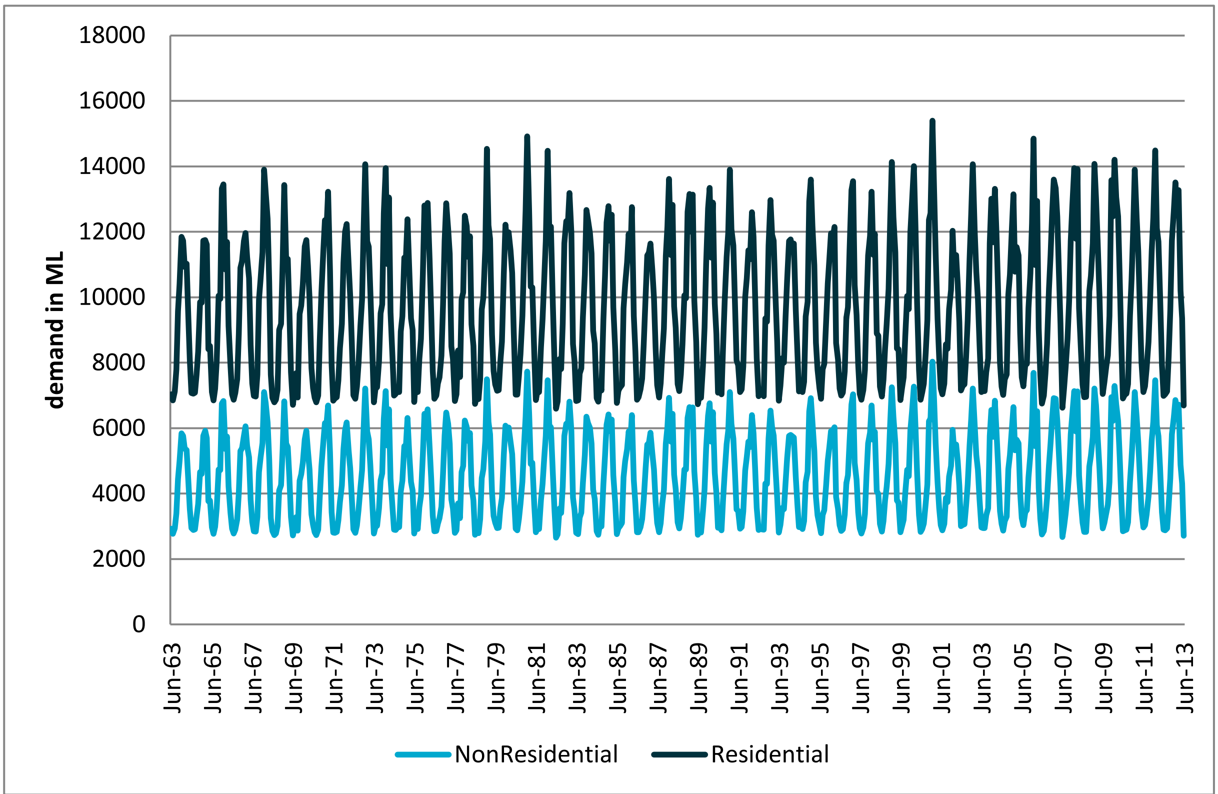


Figure 9: Monthly demand over the 50-year period (July 1963–June 2013) computed by using SA Water's CDD12 monthly demand model, for current climate (i.e. 2013 scenario)

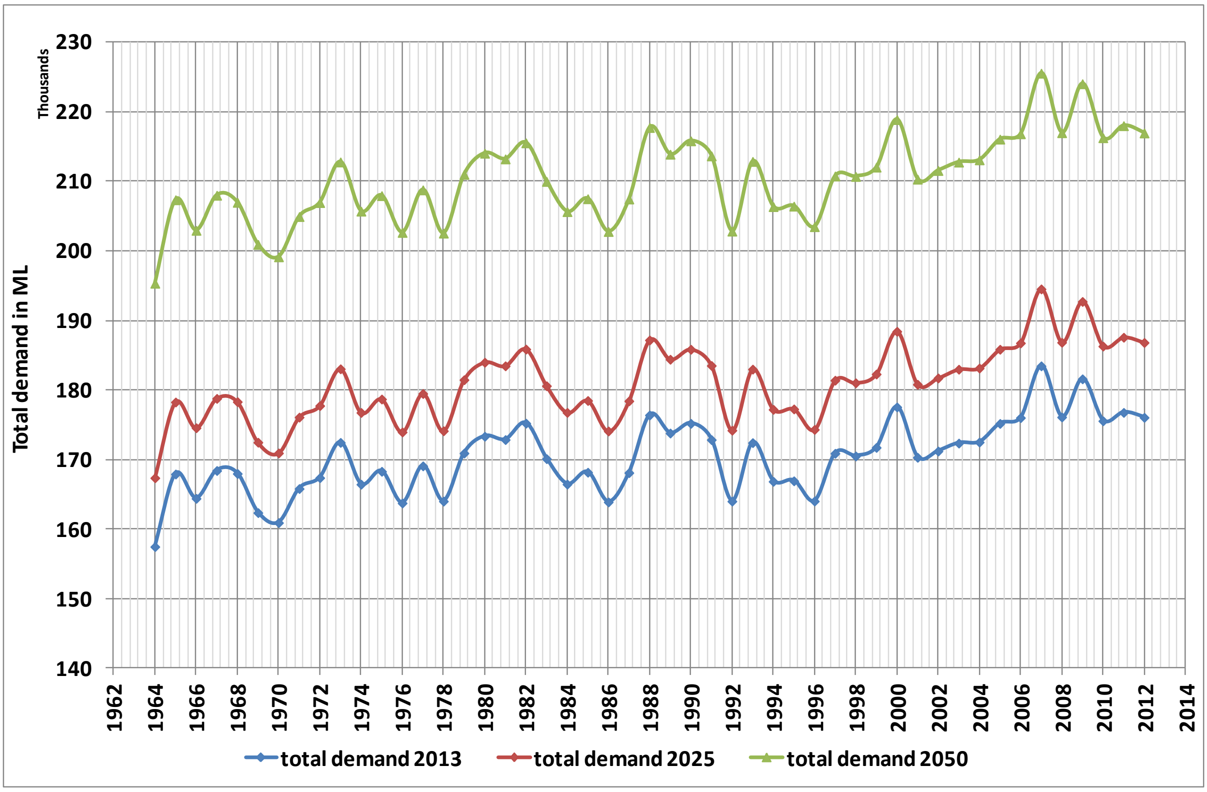


Figure 10: The annual variability of the modelled demand over the 50-year period (July 1963–June 2013) for 2013, 2025 and 2050

The same approach was used to determine 2025 and 2050 demand, but the temperature data were modified by using the expected changes to temperature shown in Table 1. Table 6 shows the average annual demand in 2013, 2025 and 2050, i.e. 170,456 ML, 180,944 ML and 210,413 ML respectively. This demand was considered to be generated from the study area shown in Figure 2.

Table 6: Average water demand of the study area in 2013, 2025 and 2050 (computed by using SA Water's CDD12 monthly demand model)

Year	Average water demand generated from the study area in ML/year	
	Average over 50 years (July 1963 – June 2013)	Average over 30 years (July 1983 – June 2013)
2013	170,456	172,518
2025	180,944	183,108
2050	210,413	212,880

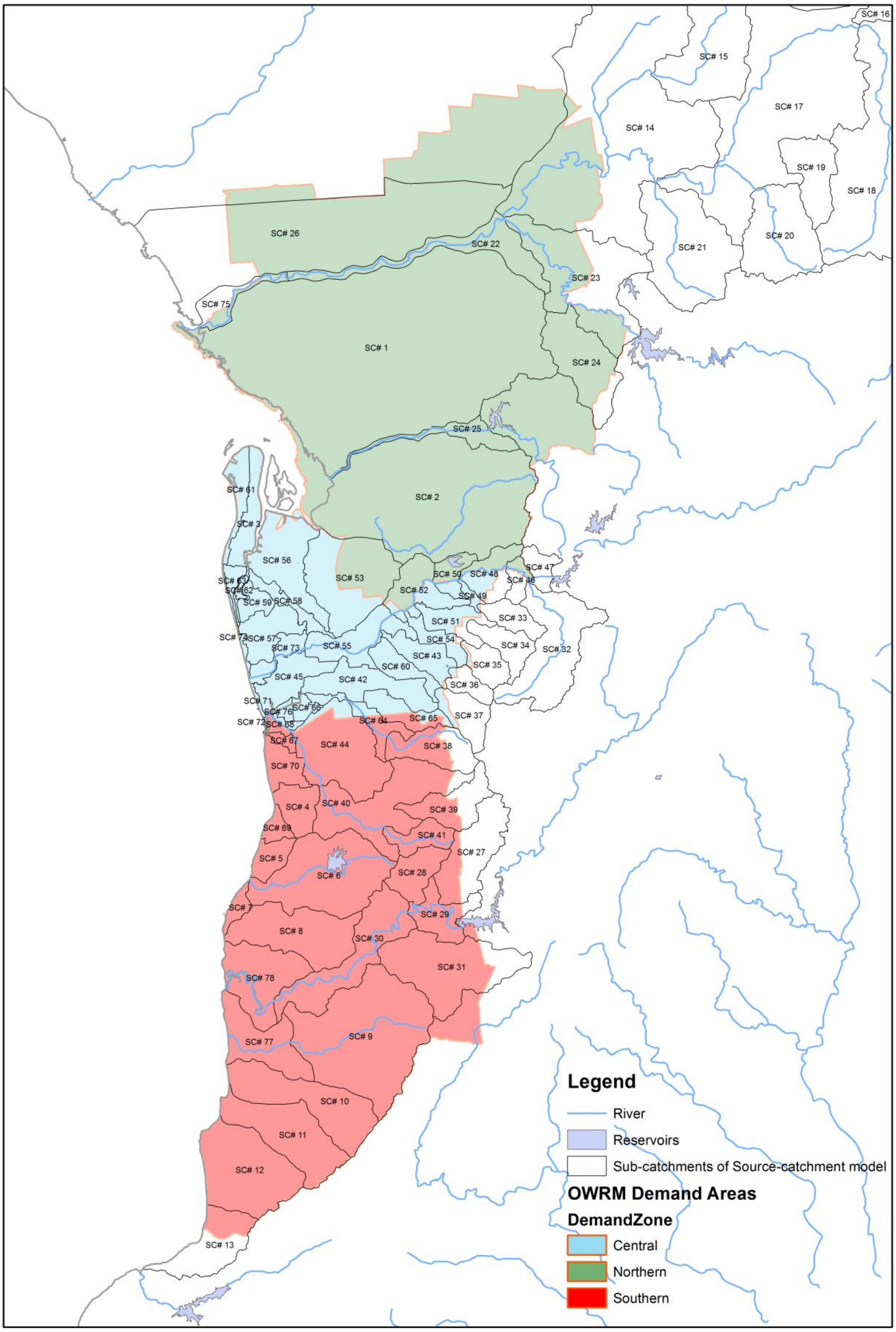


Figure 11: Demand zones, illustrated in green (northern demand zone), orange (central demand zone) and blue (southern demand zone)

The demand generated from the study area was disaggregated into three zones to examine how to supply the demand using different decentralised supply sources and centralised sources (Figure 11). The demand zones were called: north, central and south demand zones. The area covered by north, central and south demand zones represented the northern, combined eastern and western and southern local government regions, respectively, of the 30-Year Plan for Greater Adelaide.

It should be noted that 170,466 ML current average annual demand included 28,500 ML of average annual demand to account for the water users that would currently use alternative water sources, i.e. those users currently not connected to SA Water supply system. Thus the average of the modelled water demand currently being supplied by SA Water was 141,966 ML/year. The 28,500 ML/year additional demand assumed 10,000 ML/year of supply from stormwater, 1,000 ML/year of supply from rainwater tanks and 17,500 ML/year supply from wastewater recycling. Further, it was assumed that ground water available in the study area could meet on average 10,000 ML of annual demand, in addition to this 28,500 ML average annual demand. Thus the average water demand of the study in 2013 was 180,466 ML/year, of which 141,966 (i.e. about 79%) was assumed to be supplied by MLR, RM and the ADP. The 28,500 GL of average annual demand was scaled up by considering the potential change cooling-degree-days in 2025 and 2050, in order to predict monthly demand in 2025 and 2050. It should also be noted that all the average demands figures mentioned above represent the average demands expected to occur, under the climatic condition that is similar to the climatic condition occurred during July 1963 to June 2013.

Table 7: Reduction in demand in 2025 and 2050 due to current trend in adopting demand management options (installation of efficient toilets, 3-star showerheads and front loading washing machines), applied to the demand computed by using CDD12 model

Month	Reduction in demand in 2025 and 2050
January	2%
February	2%
March	3%
April	4%
May	5%
June	7%
July	7%
August	6%
Sept	6%
October	4%
November	3%
December	2%

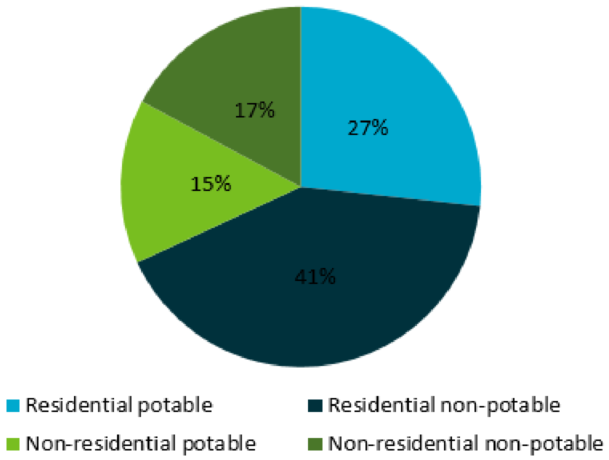


Figure 12: Breakdown of total demand to residential, non-residential, potable and non-potable demands

The 2025 and 2050 demands shown above included not only the potential impact of climate, but also the potential impact of the current trend in adopting demand management options. That is, it was assumed that by 2025 all residential households would move to efficient toilets, as these were the only options available for purchase and had been mandated as the only option that could be installed. In addition, 84% maximum uptake rate was assumed for 3-star showerheads and front loaders by 2025, considering the diffusion of innovation theory (Rogers, 2003), which assumed approximately 16% of people were ‘laggards’ who would only adopt innovation when forced. These assumptions led to a reduction of monthly demand values computed by using CDD12 model by the amounts shown in Table 7.

The total water demand was disaggregated to residential and non-residential sectors. Each sector consisted of potable and non-potable components, which disaggregated the demand into four categories (Table 8):

- Residential potable
- Residential non-potable
- Non-residential potable
- Non-residential non-potable.

Table 8: A summary of average annual demand (ML) over July 1963 to Jun 2013

Demand component	2013	2025	2050
Residential potable	45,510	47,963	55,460
Residential non-potable	70,577	75,010	89,553
Non-residential potable	25,137	27,594	31,913
Non-residential non-potable	29,232	30,377	33,486
Total residential	116,086	122,973	145,014
Total non-residential	54,369	57,971	65,399
Total demand	170,456	180,944	210,413

Table 9: Composition of the residential demand

Demand component	End use	% of total residential demand (OWRM project)	% of total residential demand (Water for Good Plan)
Potable	Kitchen	13%	11%
	Bath and shower	24%	20%
	Other indoor	2%	2%
	Total potable	**39%**	**33%**
Non-potable	Laundry	16%	16%
	Toilet	13%	11%
	Outdoor	32%	40%
	Total non-potable	**61%**	**67%**

Table 10: Number of households in North, Central and South demand zones

Demand Zone	2013	2025	2050
North	172,413	204,647	277,989
Central	204,036	242,182	328,980
South	138,195	164,031	222,818
Total	**514,644**	**610,860**	**829,787**

Table 11: Distribution of demands (in ML) in the three demand zones in 2013, 2025 and 2050

Demand component	2013			2025			2050		
	North	Central	South	North	Central	South	North	Central	South
Residential potable	14,926	17,387	13,197	15,733	18,325	13,905	18,189	21,188	16,084
Residential non-potable	23,148	26,964	20,465	24,602	28,657	21,751	29,372	34,214	25,968
Non-residential potable	9,040	11,235	4,861	9,924	12,334	5,337	11,477	14,264	6,172
Non-residential non-potable	10,513	13,066	5,653	10,925	13,577	5,875	12,043	14,967	6,476
Total residential	38,074	44,351	33,662	40,335	46,982	35,655	47,561	55,402	42,052
Total non-residential	19,554	24,301	10,515	20,849	25,911	11,211	23,520	29,231	12,648
Total demand	**57,628**	**68,651**	**44,176**	**61,184**	**72,893**	**46,867**	**71,081**	**84,632**	**54,699**

The residential demand to non-residential demand split was 68 to 32, i.e. 68% of the total demand was considered to be residential and 32% of the total demand was considered to be non-residential.

Of the total demand, 27% was residential-potable, 41% was residential non-potable, 15% was non-residential potable and 17% was non-residential non-potable (Figure 12). That is, of the total demand, 42% was potable and 58% was non-potable.

The potable to non-potable split of the residential demand was considered as 39 to 61. The potable to non-potable split of the non-residential demand was considered as 54 to 46.

The composition of the residential water use assumed for this study is shown in Table 9. As shown in this table, the residential potable demand was assumed to consist of kitchen (13% of residential use), bathroom (24% of residential use) and other indoor uses (2% of residential use). The residential non-potable component was assumed to be consisting of laundry (16% of residential use), toilet (13% of residential use) and outdoor uses (32% of residential use). There were differences in the composition of the residential water use, used for this study and that used for Water for Good Plan (Table 9). This was because this study reflected 2013 behaviour of residential water use, whereas Water for Good Plan reflected the water use behaviour in 2009. For residential demand, the cold water to hot water split was considered as 60 to 40 (George Wilkenfeld and Associates, 2003).

The distribution of the total demand computed using SA Water's CDD12 monthly demand model was based on the distribution of households in the study area, which was assumed to be 34%, 40% and 27%, for north, central and south demand zones, respectively (Table 10). The population in each demand zone was then determined by multiplying the number of households in each demand zone (Table 10) with the household size given in Table 3.

4 Application of IUWM DSF to study area

This Chapter describes how to apply the IUWM DSF described in Chapter 2, to Metropolitan Adelaide, i.e. the study area described in Chapter 3. This Chapter has 7 sections representing the first 7 components of the IUWM DSF. Each section describes an application of a particular component, to Metropolitan Adelaide.

4.1 Identify overall goals

The purpose of applying the IUWM DSF was identified as part of formulating the study, in collaboration with key stakeholders. The key stakeholders of the study were South Australia Water Corporation (SA Water), South Australia Department of Environment, Water and Natural Resources (DEWNR), South Australia Environmental Protection Agency (SA EPA) and Adelaide and Mount Lofty Ranges Natural Resource Management Board (AMLR NRM Board).

The goal of this study was to inform the following policy questions related to the Urban Water Blueprint (or IUWM Plan) being developed for the Metropolitan Adelaide:

- How the use of the various water resources available could be optimised for both potable and non-potable use in Metropolitan Adelaide, by balancing the cost-effective use of the existing infrastructure, efficient consumption of energy, improved long-term supply security and improved coastal water quality?
- How could the decentralised (or localised approaches) to water management best be used to provide a greater benefit to the community?

4.2 Formulate the problem

Since the policy questions to be informed were related to water supply, and the available water in some water sources were climate dependent, a problem was formulated with the following objectives:

- To develop a generic framework for integrated urban water management and to demonstrate its application to Metropolitan Adelaide
- To inform the development of future water plans for Metropolitan Adelaide through identifying and evaluating an integrated set of water supply and demand management options that reflect social, economic and environmental values.

4.3 Identify objectives, decision variables and constraints

4.3.1 Objectives and constraints

In total four objectives were considered to address the above-mentioned goals.

The objectives were to:

1. minimise the present value of the life cycle cost of infrastructure over 25 years with a discount rate of 6% (TC)
2. minimise thepresent value of energy consumption, including embodied energy over 25 years (TE)
3. maximise the volumetric reliability of the non-potable component of the system supply (VR_{ND})
4. minimise total stormwater and wastewater discharge to the Gulf (D_{SW-WW}),

subject to the following constraints:

1. time supply reliability of the potable component of the system demand (TR_P) must be 99.5%
2. environmental flow releases from reservoirs must be met
3. monthly target volumes of the storages in MLR catchments must be met
4. the maximum amount of water extracted from the River Murray must be limited to 650 GL over any consecutive five year period.

Note that costs assessed in this study are relative costs that include the capital cost and maintenance cost of new infrastructure and the operating cost of existing and new infrastructure. The capital cost of existing infrastructure at the time of the study (2013) has not been included as it is considered to be a sunk cost. Similarly the maintenance cost of existing infrastructure has not been included as it will be required regardless of the combination of new options chosen.

4.3.2 Decision variables

The decision variables are the variables that have a significant influence on the above mentioned objectives. Their values can be chosen by decision-makers for 'what-if' type evaluation. Alternatively, the best values for the decision variables can be chosen through multi-objective optimisation.

Since the goal of this study was to identify the optimal supply mix, the decision variables were chosen to control the amount of water that could be supplied from the available sources. Accordingly, the following decision variables were utilised:

- the amount of water that can be drawn from MLR, subject to the constraints on monthly target volumes and environmental flow releases;
- the amount of water that can be drawn from the River Murray from each pipeline, subject to extraction and pipe capacities;
- the amount of water that can be drawn from ADP, subject to its maximum capacity;
- the stormwater schemes to be implemented (i.e. on/off for 25 schemes), subject to a maximum seasonal injection volume, which is defined as the amount of non-potable water demand of the previous season; and

- the amount of recycling capacity of WWTPs to be increased, subject to their maximum treatment capacity.

It should be noted that rainwater harvesting, groundwater use and demand management options were not considered part of the optimisation due to time limitations of the project. Hence decision variables were not defined for rainwater harvesting, groundwater use and demand management options. The suggested decision variables for these options included:

- the uptake rate of rainwater harvesting
- the uptake rate of different demand management options
- the maximum amount of groundwater use subject to the sustainable yield of groundwater.

4.4 Translate objectives and constraints into measurable criteria

Four measurable criteria were defined to address the objectives defined in section 4.3:

1. present value of total life cycle cost
2. total energy
3. volumetric reliability of non-potable water supply
4. stormwater and wastewater discharges to the Gulf

In addition, the time-based reliability of water supply was used as a constraint in the optimisation.

The total life cycle cost, TC ($), was computed as follows (based on Marchi et al., 2014):

$$TC = CC + PV(OC)$$

Equation 2

where, CC ($) is the capital cost and, $PV(OC)$ ($) is the present value of the operational cost OC ($/year).

Similarly, the total energy TE (MWh) was calculated as follows:

$$TE = CE + PV(OE)$$

Equation 3

where, CE (MWh) is the capital (or embodied) energy and $PV(OE)$ (MWh) is the present value of the operational energy OE (MWh/year).

The capital and operational costs, the capital and operational energy, the volumetric supply reliability of non-potable water demands and the discharge related impacts calculation methods are described below.

4.4.1 Capital cost

The capital costs were related to:

- the construction of selected stormwater systems, including the cost of upgrading stormwater distribution systems for non-residential non-potable purposes and residential non-potable purposes (CC_{SW})
- the upgrade of the recycling capability of current wastewater treatment facilities, which includes the additional cost of upgrading wastewater distribution systems for residential users (CC_{WW}).

It was assumed that there were no upgrades to the reservoirs in the MLR or the pipelines delivering water from the River Murray to Adelaide. In addition, the ADP was considered to be operational without an upgrade being required up to the year 2050. Furthermore, the capital cost of upgrading the recycling capability of current wastewater treatment facilities (CC_{WW}) was assumed to include the cost of the distribution system to supply recycled wastewater to the non-potable component of the non-residential demand.

Therefore, the capital cost CC (\$) was computed as follows:

$$CC = CC_{SW} + CC_{WW}$$

Equation 4

Where CC_{SW} (\$) is the capital cost associated with the stormwater option and CC_{WW} (\$) is the capital costs associated with the recycled wastewater option.

The total capital cost of stormwater harvesting, CC_{SW}, was the sum of the capital cost for all schemes implemented. A scheme comprised infrastructure to collect, treat, store and distribute stormwater for non-potable purposes. The method of harvesting, treatment and storage assumed was a basin and wetland followed by aquifer storage and recovery (ASR) (see Section 4.5 for details). A scheme consisted of a number of smaller individual harvesting schemes that were hydrologically connected. The number of lumped schemes considered was 25 which were based on an aggregation of 70 individual schemes (see Section 4.5 for details). The capital costs included the cost of the basin, wetland and ASR system as well as a distribution system required to deliver the water to consumers.

$$CC_{SW} = \sum_{i=1}^{25} C_i . D_i$$

Equation 5

Where, C_i (\$) is the capital cost of lumped scheme i, and D_i is the decision variable (0 or 1) for the 25 aggregated schemes (see Table 29, for details of 25 lumped schemes).

The schemes that were fully operational in 2013 (Table 2) were assigned a zero capital cost for the scheme and its associated distribution network (C_i = 0). These schemes were assumed to be implemented (D_i = 1).

The capital cost C_i (\$) was computed as:

$$C_i = C_{lumpedScheme,i} + 0.8 * Yield_{harvest,i} * \left(C_{dist,res} * P_{NPres} + C_{dist,non-res} * P_{NPnon-res}\right)$$

Equation 6

where,

- $C_{lumpedScheme,i}$ is the capital cost of building the stormwater harvesting scheme (wetland, wells, etc..) ($)
- $Yield_{harvest,i}$ is the potential yield (note that this quantity was multiplied by 0.8 to account for the fact that not all water injected could be extracted, i.e. 20% injected water was considered to be lost through deep percolation) (ML/year)
- $C_{dist,res}$ is the unit capital cost ($38,920 per ML/year of yield) associated with the construction of the distribution system to residential users
- $C_{dist,non-res}$ is the unit capital cost ($8,182/(ML/year)) for the construction of the distribution system for non-residential users
- P_{NPres} is the proportion of the non-potable demand of residential users compared to the total non-potable demand of 2013
- $P_{NPnon-res}$ is the proportion of non-potable demand of non-residential users compared to the total non-potable demand of 2013.

It should be noted that the percentage of residential and non-residential supply from stormwater was assumed to be equal to the proportion of residential and non-residential non-potable demand in each of the three regions (northern, southern and central). The capital cost computed using Equation 6 is shown in Table 12.

For wastewater treatment plants, the cost of upgrading wastewater reuse capacity of each treatment plant, was assumed to be related to the increase in maximum volumetric capacity,

$$CC_{WW} = \sum_{WWTP=Bolivar,Glenelg,Christies\ Beach} \left(C_{upgrade} + C_{dist}.RS_{WWTP}\right).\Delta Q_{WWTP}$$

Equation 7

Where

- CC_{WW} is the capital cost of upgrading wastewater facilities ($)
- ΔQ is the increase in capacity to recycle water (ML/year)
- $C_{upgrade}$ is the capital cost for additional reuse capacity ($/(ML/year))
- C_{dist} is capital cost for a residential distribution system ($/(ML/year))
- RS_{WWTP} is the supply going to residential customers as fraction of total reuse from each treatment plant. This variable is the variable set by the optimiser for each WWTP, in the range [0–1]
- $C_{upgrade}$= 40,684 $/(ML/$year$)$ and C_{dist} = 10,000 $/(ML/year).

Table 12: Capital cost of lumped stormwater schemes

No	Scheme ID	Scheme Name	C_i($M)
1	SWAC1Schm	Adams creek	47.51
2	SWBC42Schm	Brown Hill creek #1	2.69
3	SWBC64Schm	Brown Hill creek #2	6.81
4	SWBC65Schm	Brown Hill creek #3	4.92
5	SWBC76Schm	Brown Hill creek #4	142.50
6	SWBI53Schm	Barker Inlet	148.09
7	SWCC8Schm	Christie Creek	0
8	SWDC2Schm	Dry Creek	0
9	SWFR6Schm	Field River	87.49
10	SWGA57Schm	Grange area	46.98
11	SWGR1Schm	Greater Edinburgh	34.65
12	SWGR22Schm	Gawler River	179.49
13	SWLP1Schm	Little Para #1	49.089
14	SWLP25Schm	Little Para #2	30.70
15	SWMC56Schm	Magazine Creek (Range Wetlands)	77.60
16	SWME45Schm	Mile End Drain	99.41
17	SWOR78Schm	Onkaparinga River	81.05
18	SWPC77Schm	Pedler Creek	43.42
19	SWPR58Schm	Port Road #1	18.35
20	SWPR59Schm	Port Road #2	14.46
21	SWRT60Schm	River Torrens #1	12.88
22	SWRT73Schm	River Torrens #2	144.96
23	SWSC1Schm	Smith creek	0
24	SWSR40Schm	Sturt creek #1	31.34
25	SWSR44Schm	Sturt creek #2	163.39

4.4.2 Operational costs

Operational costs occur throughout the whole design life of a facility and are usually associated with the consumption of energy or materials (e.g. to operate a pump) and expenses associated with personnel. The operational costs can vary from one time period to another, e.g. a pump can be operated or switched off depending on needs.

In this study, a distinction is drawn between existing and new facilities as indicated in Table 13.

Table 13: Operational costs included in this study

Type of Facility	Operational Costs
Existing facilities	Operating costs only
New facilities	Operating and maintenance costs

Operational costs have been included for both existing and new facilities. For existing infrastructure, the operational costs consist only of operating costs. These operating costs depend on the volume of water that is supplied from these sources in the future. Ongoing maintenance costs for existing infrastructure will be incurred regardless of the new options chosen and so they has not been including in the cost analysis.

The new facilities that are considered in the study include new stormwater harvesting schemes, upgrades of wastewater treatment facilities and distribution networks for the treated stormwater and wastewater. Operational costs for new facilities include both operating and maintenance costs. Maintenance costs have been estimated as an average cost per kL produced rather than as a fixed cost per year. Hence they will be zero in any year that a facility has zero output. As it is unlikely that any facility will have zero output in future years, this error in estimation is acceptable given the other uncertainties in the cost estimates.

The exception is the Adelaide Desalination Plant that could be operated at low levels of output (after the initial proving period) in most years and will only be run at high levels of output during drought years. The assumed operating cost of the Adelaide desalination plant is \$30m per year plus \$1 /kL produced. Thus there is a fixed cost of \$30m per year regardless of output. This is a constant that is included in the cost estimates of all options and so does not make any difference to the choice between options.

The total present value of the operational cost, *PV(OC)* (\$)**,** assuming the operational cost was the same for each year of the planning period, was calculated as follows:

$$PV(OC) = OC\left[\frac{1-(1+i)^{-n}}{i}\right]$$

Equation 8

where, i is the discount rate used (i.e. 6%); n is the length of the planning period (i.e. 25 years); and OC is the annual operational cost of the system (\$/year), which was calculated as follows:

$$OC = OC_{ML} + OC_{RM} + OC_{ADP} + OC_{SW} + OC_{WW}$$

Equation 9

Where, OC is the annual operational cost of the system (\$/year), OC_{ML}, OC_{RM}, OC_{ADP}, OC_{SW} and OC_{WW} (\$/year) are the annual operational cost of supplying water from the Mount Lofty storages, the River Murray, the ADP, harvested stormwater and reclaimed wastewater, respectively. These operational costs were calculated as follows:

$$OC_{ML} = S_{ML} \times UC_{ML}$$

Equation 10

$$OC_{RM} = S_{RM} \times UC_{RM}$$

Equation 11

$$OC_{ADP} = S_{ADP} \times UC_{ADP} + FC_{ADP}$$

Equation 12

$$OC_{SW} = I_{SW-NRNP} \times UC_{SW-NRNP} + I_{SW-RNP} \times UC_{SW-RNP}$$

Equation 13

$$OC_{WW} = Vol_{WW} \times UC_{Tr} + S_{WW} \times UC_{Re}$$

Equation 14

where,

- S_{ML}, S_{RM}, S_{ADP} and S_{WW} are average annual demand supplied from Mount Lofty storages, the River Murray, the ADP and wastewater recycling, respectively (kL/year)
- $I_{SW-NRNP}$ and I_{SW-RNP} are the NRNP and RNP volume of stormwater injected in the aquifer, respectively (kL/year)
- UC_{ML}, UC_{RM} and UC_{ADP} are the unit cost of supplying water from Mount Lofty storages, the River Murray and the ADP, respectively ($/kL)
- $UC_{SW-NRNP}$ and UC_{SW-RNP} are the unit cost to supply NRNP and RNP demand using stormwater, respectively ($/kL)
- FC_{ADP} ($/year) is the fixed running cost of the ADP regardless demand was supplied from the ADP or not, which was 30 M$/year
- Vol_{WW} is the total volume of treated wastewater (kL/year)
- UC_{Tr} is the unit cost of secondary treatment ($/kL)
- UC_{Re} is the additional unit cost of tertiary treatment compared to secondary treatment ($/kL).

The unit costs are summarised in Table 14.

Table 14: Summary of unit costs

Variables	Unit cost ($/kL)
UC_{ML}	0.23
UC_{RM}	0.44
UC_{ADP}	1
$UC_{SW-NRNP}$	0.42
UC_{SW-RNP}	0.69
UC_{Tr}	1.1
UC_{Re}	0.9

It should be noted that $UC_{SW-NRND}$ and UC_{SW-RND} included both the injection and the extraction costs, which resulted in consideration of extraction costs even if stormwater was not extracted. Such a situation could arise if supplying from other sources were cheaper than supplying from a particular stormwater harvesting scheme, even though there was some costs involved in implementing the scheme. Even though this type of situations was acceptable from a computational point of view, it is not a desirable situation from a practical point of view. This is because a scheme would not be implemented if water is not extracted. Hence we considered that implementation of schemes with a significant low volumes of stormwater extracted for non-potable use (i.e. allowing to store stormwater underground) was an undesirable behaviour, which would need further work to examine possible solutions.

It should also be noted that the percentage of residential and non-residential supply from stormwater was assumed to be equal to the proportion of residential and non-residential non-potable demand in the three regions (northern, southern and central) (Table 15).

Table 15: Percentage of residential and non-residential demand for non-potable uses for 2013

Region	Percentage non-residential demand	Percentage residential demand
Northern	31.23%	68.77%
Central	32.64%	67.36%
Southern	21.64%	78.36%

4.4.3 Capital energy

The capital energy CE (also called embodied energy) accounted for the energy associated with the construction of stormwater schemes, CE_{SW}, which included the energy of constructing the scheme itself plus the energy associated with the distribution systems for non-residential non-potable purposes and residential non-potable purposes, and the energy of constructing wastewater distribution systems for non-residential non-potable purposes and residential non-potable purposes (i.e. $CC_{WW-NRNP}$ and CC_{WW-RNP}). This was because no upgrades to the reservoirs in Mount Lofty ranges, the pipelines delivering water from the River Murray to Adelaide and the ADP were considered by the year 2050 and the energy related to the upgrade of the recycling capability of current wastewater treatment facilities (CE_{WW}) could not be estimated due to insufficient data. Therefore, the capital energy CE was calculated as follows:

$$CE = CE_{SW} + CE_{WW-NRNP} + CE_{WW-RNP}$$

Equation 15

where CE is the capital energy (MWh), CE_{SW} is the capital energy associated with the construction of stormwater schemes (MWh), $CC_{WW-NRNP}$ is the capital energy of constructing wastewater distribution systems for non-residential non-potable purposes (MWh) and CC_{WW-RNP} is the capital energy of constructing wastewater distribution systems for residential non-potable purposes (MWh). It should be noted that capital energy is implicitly included in the capital costs of new schemes as it is a component of the cost of production of all materials used.

The capital energy of the construction of each of the 25 aggregated stormwater schemes, including the capital energy for distribution, was calculated based on their injection yield (Marchi et al., 2014) and the proportion of residential and non-residential non-potable demand (Table 15).

As for capital cost, the schemes that were fully operational in 2013 (Table 2) were assigned a zero capital energy for the scheme and its associated distribution network ($CE_i = 0$). These schemes were assumed to be implemented ($D_i = 1$).

$$CE_{SW} = \sum_{i=1}^{25} CE_i . D_i$$

Equation 16

Where, CE_{SW} is the capital energy associated with the construction of stormwater schemes (MWh) and CE_i is the capital energy of the aggregated scheme (MWh), and D_i is the decision variable (0 or 1) for the 25 aggregated schemes.

The capital cost CE_i was computed as:

$$CE_i = CE_{lumpedScheme,i} + 0.8 * Yield_{harvest,i} * \left(CE_{dist,res} * P_{NPres} + CE_{dist,non-res} * P_{NPnon-res}\right)$$

Equation 17

where,

- $CE_{lumpedScheme,i}$ is the capital energy of building the ASR scheme (wetland, wells, etc..) in MWh, assumed to be an average value of 5.131 MWh per ML/year of yield
- $Yield_{harvest,i}$ (ML/year) is the yield that can be harvested and injected in the aquifer (note that this quantity is multiply by 0.8 to account for the fact that not all water injected can be extracted)
- $CE_{dist,res}$ is the unit capital energy (25.981 MWh per ML/year of yield) associated with the construction of the distribution system to residential users
- $CE_{dist,non-res}$ is the unit capital energy (3.257 MWh/(ML/year)) for the construction of the distribution system for non-residential users
- P_{NPres} is the proportion of the non-potable demand of residential users compared to the total non-potable demand of 2013
- $P_{NPnon-res}$ is the proportion of non-residential demand.

Table 16: Capital energy of stormwater schemes

No	Scheme ID	Scheme Name	Capital Energy (MWh)
1	SWAC1Schm	Adams creek	31910.92
2	SWBC42Schm	Brown Hill creek #1	1573.333
3	SWBC64Schm	Brown Hill creek #2	3980.721
4	SWBC65Schm	Brown Hill creek #3	2933.683

No	Scheme ID	Scheme Name	Capital Energy (MWh)
5	SWBC76Schm	Brown Hill creek #4	79766.07
6	SWBI53Schm	Barker Inlet	77377.64
7	SWCC8Schm	Christie Creek	0
8	SWDC2Schm	Dry Creek	0
9	SWFR6Schm	Field River	47462.8
10	SWGA57Schm	Grange area	23694.77
11	SWGR1Schm	Greater Edinburgh	18712.37
12	SWGR22Schm	Gawler River	96943.12
13	SWLP1Schm	Little Para #1	26416.33
14	SWLP25Schm	Little Para #2	16522.21
15	SWMC56Schm	Magazine Creek (Range Wetlands)	33949.86
16	SWME45Schm	Mile End Drain	54327.36
17	SWOR78Schm	Onkaparinga River	42685.09
18	SWPC77Schm	Pedler Creek	25921.19
19	SWPR58Schm	Port Road #1	10823.77
20	SWPR59Schm	Port Road #2	8530.117
21	SWRT60Schm	River Torrens #1	7222.165
22	SWRT73Schm	River Torrens #2	77434.50
23	SWSC1Schm	Smith creek	0
24	SWSR40Schm	Sturt creek #1	16135.26
25	SWSR44Schm	Sturt creek #2	84133.84

The capital energy computed using Equation 17 is shown in Table 16.

For wastewater treatment plant, the capital energy of upgrading the distribution system for delivering recycled wastewater to residential users was:

$$CE_{WW-RNP} = \sum_{WWTP=Bolivar,Glenelg,Christies\ Beach} UCE_{WW-RNP} * RS_{WWTP} * \Delta Q_{WWTP}$$

Equation 18

$$CE_{WW-NRNP} = \sum_{WWTP=Bolivar,Glenelg,Christies\ Beach} UCE_{WW-NRNP} * (1 - RS_{WWTP}) * \Delta Q_{WWTP}$$

Equation 19

Where

- $CC_{WW-NRNP}$ is the capital energy of constructing wastewater distribution systems for non-residential non-potable purposes (MWh)
- CC_{WW-RNP} is the capital energy of constructing wastewater distribution systems for residential non-potable purposes (MWh)
- ΔQ_{WWTP} is the increase in capacity to recycle water (ML/year)

- UCE_{WW-RNP} is the unit capital energy (25.981 MWh/ML) for additional reuse capacity for residential use
- $UCE_{WW-NRNP}$ is the unit capital energy (3.257 MWh/ML) for additional reuse capacity for non-residential use
- RS_{WWTP} is the supply going to residential customers as fraction of total reuse from each treatment plant.

4.4.4 Operating energy

The total present value of the operating energy**,** assuming that it was the same for each year of the planning period, was calculated as follows:

$$PV(OE) = OE \times 25$$

Equation 20

where, $PV(OE)$ is the present value of the operating energy (MWh), 25 is the planning period (i.e. 25 years); and OE (MWh/year) is the annual operating energy of the system, which was calculated as follows:

$$OE = OE_{ML} + OE_{RM} + OE_{ADP} + OE_{SW} + OE_{WW}$$

Equation 21

Where, OE_{ML}, OE_{RM}, OE_{ADP}, OE_{SW} and OE_{WW} (MWh/year) are the annual operating energy of pumping from the Mount Lofty storages, pumping from River Murray, supplying water from the ADP, recycling stormwater and wastewater, respectively. The operating energy values were calculated as follows:

$$OE_{ML} = S_{ML} \times UE_{ML}$$

Equation 22

$$OE_{RM} = S_{RM} \times UE_{RM}$$

Equation 23

$$OE_{ADP} = S_{ADP} \times UE_{ADP}$$

Equation 24

$$OE_{SW} = I_{SW-NRNP} \times UE_{SW-NRNP} + I_{SW-RNP} \times UE_{SW-RNP}$$

Equation 25

$$OE_{WW} = Vol_{WW} \times UE_{Tr} + S_{WW} \times UE_{Re}$$

Equation 26

where,

- S_{ML}, S_{RM}, S_{ADP} and S_{WW} are the average annual demand supplied from Mount Lofty storages, the River Murray, the ADP and wastewater recycling, respectively (ML/year)

- $I_{SW-NRND}$ and I_{SW-RND} are the non-residential and residential non-potable stormwater volumes injected in the aquifer, respectively (ML/year)
- UE_{ML}, UE_{RM} and UE_{ADP} are the unit energy of supplying water from Mount Lofty storages, the River Murray and the ADP, respectively (MWh/ML)
- $UE_{SW-NRNP}$ and UE_{SW-RNP} are the unit energy to supply non-residential and residential non-potable demands using stormwater (including injection, extraction and distribution), respectively (MWh/ML)
- Vol_{WW} is the total volume of treated wastewater (ML/year)
- UE_{Tr} was the unit energy of secondary treatment (MWh/ML)
- UE_{Re} is the additional unit energy of tertiary treatment compared to secondary treatment (MWh/ML).

Table 17: Summary of unit energies

Variables	Unit energy (MWh/ML)
UE_{ML}	0.3
UE_{RM}	1.9
UE_{ADP}	5.0
$UE_{SW-NRNP}$	0.63
UE_{SW-RNP}	0.97
UE_{Tr}	0.40
UE_{Re}	0.29

Note that the energy use for extraction and distribution differ depending on the final use for the water, with residential reuse being more energy intensive. Since the actual end use was not known at the time of injection the extraction costs were estimated based on the proportion of the demand from residential and non-residential users. Values for the unit energies are summarised in Table 17.

4.4.5 Volumetric reliability of non-potable demand

Volumetric reliability was considered as an important measure of supply security. It was defined as the total non-potable supply divided by the total non-potable demand over the simulation period (expressed as a percentage). Volumetric reliability was considered to be important for non-potable demands, as it indicated the potentially amount of non-potable supply that would need to be supplemented using potable sources. In addition, it was also related to the possibility of imposing water restrictions. The volumetric reliability of non-potable demands (VR_{NP}) was calculated using the following equation:

$$VR_{NP} = S_{NP}/D_{NP} \times 100\%$$

Equation 27

where, VR_{NP} is the volumetric reliability, S_{NP} is the total volume of NP water supplied over the simulation period (kL) and D_{NP} (kL) is the total demand of NP water over the simulation period.

4.4.6 Stormwater and wastewater discharges into the Gulf

This metric was used to assess the impact of stormwater and wastewater discharge on the health of the ecosystem in Gulf St Vincent. The environmental impact included the loss of seagrass and loss of fisheries. Turbidity and nutrients were considered to be the key contributors to seagrass loss with nutrients responsible for about 90% of the loss (Fox et al., 2007). Both stormwater and treated wastewater discharge contributed to increased turbidity and nutrients in the Gulf.

The average concentration of suspended sediment and nitrogen in stormwater were calculated using data provided in the Adelaide Coastal Waters Study (ACWS) (Fox et al., 2007). This is summarised in Table 18. More recent water quality studies (McDowell and Pfennig, 2013) indicated that these values had not changed significantly over the last few years. SA Water provided data on the concentrations of suspended solids and nitrogen in wastewater discharges in 2012/13 (Table 18).

Considering a relative importance (RI) of nutrients ($\mathrm{RI_N}$) and turbidity ($\mathrm{RI_T}$) to seagrass loss of 75% to 25%, the RI of reducing stormwater and wastewater $\mathrm{W_{SW}}:\mathrm{W_{WW}}$ could be calculated as follows:

$$\mathrm{W_{SW}}:\mathrm{W_{WW}} = \frac{\mathrm{RI_N} \times \mathrm{C_{N-SW}} + \mathrm{RI_T} \times \mathrm{C_{S-SW}}}{\mathrm{RI_N} \times \mathrm{C_{N-WW}} + \mathrm{RI_T} \times \mathrm{C_{S-WW}}} = 0.85:1.0$$

Equation 28

where, $\mathrm{C_{N-SW}}$ and $\mathrm{C_{S-SW}}$ are the concentration of nitrogen and suspended solids in stormwater (Kg/ML), respectively; and $\mathrm{C_{N-WW}}$ and $\mathrm{C_{S-WW}}$ are the concentrations of nitrogen and suspended solids in wastewater (kg/ML), respectively.

Table 18: Summary of nitrogen and turbidity concentration discharges from stormwater and wastewater

Source	Average Nitrogen Concentration (kg/ML)	Average Suspended Solids Concentration (kg/ML)
Stormwater 2003	1.33	60.08
Wastewater 2003	13.28	35.21

The total weighted discharge of stormwater and wastewater into the Gulf ($\mathrm{D_{SW-WW}}$) was calculated as follows:

$$\mathrm{D_{SW-WW}} = \mathrm{W_{SW}} \times \mathrm{D_{SW}} + \mathrm{W_{WW}} \times \mathrm{D_{WW}}$$

Equation 29

where, $\mathrm{D_{SW-WW}}$ (kL) is the total discharge from stormwater and wastewater into the Gulf, $\mathrm{D_{SW}}$ and $\mathrm{D_{WW}}$ (kL) are the total volume of stormwater and treated wastewater discharged into the Gulf during the simulation period, respectively.

However, following a series of discussions with the representatives of EPA South Australia, AMLR NRM Board and SA Water, it was decided to use a W_{SW} to W_{WW} ratio as 1:1, as a better indicator for the potential impact on the water of Gulf St Vincent. For this particular set of weights, the objective D_{SW-WW} represents the total volumetric wastewater and stormwater discharges to the Gulf.

4.4.7 Time-based reliability for potable supply

As discussed earlier, the reliability of potable demand was calculated on a time basis and was constrained to be greater than or equal to 99.5%.

4.5 Identify alternative options

The following supply options were considered:

- The RM, the MLR catchments and the ADP could supply potable water to both residential and non-residential potable and non-potable demands
- Harvested stormwater could be supplied to both residential and non-residential non-potable demands only
- Recycled water could be supplied to both residential and non-residential non-potable demands only
- Rainwater stored and tanks could supply residential non-potable demands (i.e. toilet, laundry and outdoor water) and hot water demands.

4.6 Evaluate alternative options

To evaluate the alternative options described in Section 4.5, in terms of the objectives described in Section 4.3, a monthly water balance model, capable of simulating supply and demand dynamics, as well as supply and discharge dynamics of the alternative options described in Section 4.5, was developed.

As part of developing a simulation model, an appropriate simulation method was sought by considering the purpose and the spatial extent of the simulation. An emphasis was given to a simulation method that could be easily coupled with an appropriate optimisation method, and could represent both runoff generation and transportation from urban catchments, along with supply from multiple sources, particularly from such sources as stormwater and recycled wastewater. In addition, consideration was given to a modelling method that could be used by the stakeholders as a decision support tool to inform policy questions related to urban water management, upon completion of the project. Furthermore, key stakeholders (SA Water, EPA South Australia and DEWNR) were consulted as part of formulating the study, to understand preferences for modelling platforms.

The simulation methods that best fit these criteria were WaterCress model (Clark et al. 2002) and Source model (eWater, 2012). Both models had the technical capacity to meet the needs of simulation mentioned above. However, when considering future modelling needs and preferences of key stakeholders, the Source model was clearly the best suited modelling platform because, of its following attributes:

- It has a flexible structure to select a level of model complexity appropriate to the problem at hand and within any constraints imposed by the available data and knowledge (eWater, 2012);
- It has been accepted as the National Hydrologic Modelling Platform by the Australian Government as part of the National Water Initiative. Hence it gives a consistent national approach to take advantage of a more mobile workforce;
- It has the potential to integrate models from differing jurisdictions;
- It is being supported by the South Australian Government as part of their commitment to the National Water Initiative through utilising of it where appropriate and investing in further development of the tool with an aim of creating a community of practitioners; and
- It has a formal software development structure and is maintained and further developed as a future modelling capability by an organisation supported by the Australian Government (www.ewater.com.au).

Hence the Source model was chosen as the preferred method for the simulation. Details of the Source model can be found in eWater (2012).

It should be noted that the other commonly used simulation tools for water supply planning in Australia include REALM (Perera et al., 2005), Wathnet (Kuczera, 1992). However, most of the simulation capabilities of both these models were improved and incorporated in the Source model.

The Source model was developed as a river basin model to inform water resource planning in river basins. Hence the applicability of the Source to urban water systems was unknown. The ability of the Source model to simulate both supply and demand, as well as supply and discharge dynamics of urban water systems, in particular, systems with both centralised and decentralised water supply options was unknown. This is the first study in Australia which explored the use of Source model to inform integrated water management planning, at a city scale.

In this section, we describe the monthly water balance simulation model developed using the Source model including the modelling methodology developed for each supply source to examine supply, demand and discharge interactions.

4.6.1 Modelling supply, demand and discharge interactions using Source

The Source model is equipped with the following key modules:

- Source-schematic: to examine supply and demand dynamics and water allocation processes, to inform water allocation management in river basins and water supply management in urban areas

- Source-catchment: to examine runoff and contaminant generation and transportation processes and hydrologic routing, to inform catchment hydrologic and water quality management
- Source-insight: to identify optimal water allocation options in river basins and supply options in urban areas.

The IUWM DSF utilised all three modules (Figure 13). The Source-schematic and Source-catchment modules were used in the simulation component to simulate supply, demand and discharge interactions (i.e. component 6 of the IUWM DSF). The Source-insight module was utilised in the optimisation component (i.e. component 7 of the IUWM DSF).

The Source model is developed and maintained by eWater.[1]. New versions are released as new features are added to the software. The version of the Source model used in the IUWM DSF is version 3.3.0.236. This was the latest version available when the study commenced in October 2012. The users of the IUWM DSF should use version 3.3.0.236, which is available from the eWater.

In Source version 3.3.0.236, Source-insight module can be dynamically linked with Source-schematic module only. An implication of this feature was that the optimisation model implemented in Source-insight module could communicate only with the simulation model developed in Source-schematic module, but not with the catchment simulation model developed in the Source-catchment module.

Accordingly, the IUWM DSF utilised the Source-schematic module to simulate supply, demand and discharge interactions, and compute the value of the objective function for different values of decision variables (see Section 4.3.2 for decisions variables). Hence the methods used to simulate supply, demand and discharge interactions (described in Section 4.6.1 to 4.6.9) and the methods used to translate the objective function into measurable criteria (described in Section 4.4) were incorporated in the simulation model developed in the Source-schematic module. The key features of the simulation model developed in the Source-schematic module are illustrated in Figure 13.

[1] http://www.ewater.com.au/)

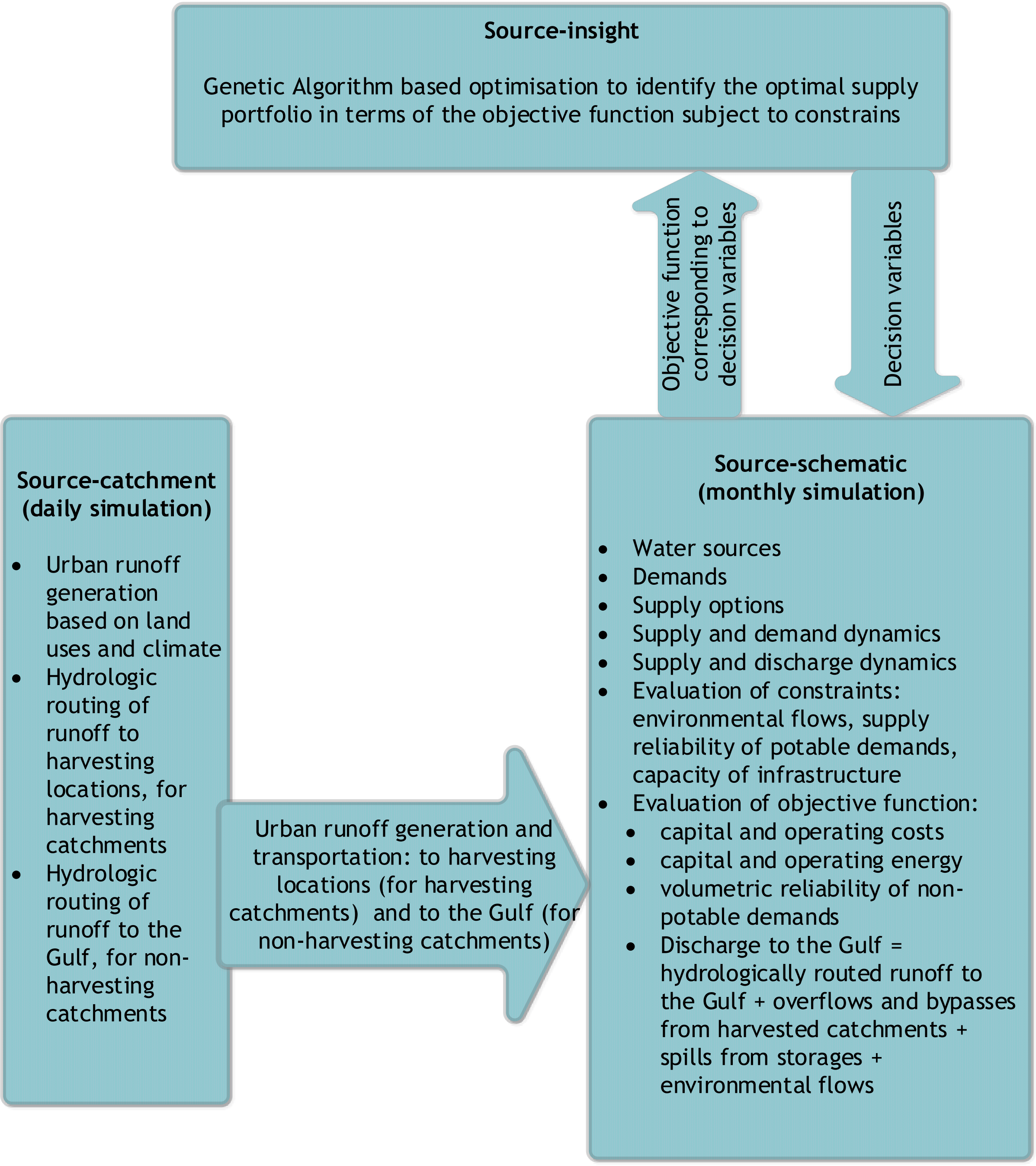


Figure 13: Interactions between Source modules and how the Source model was used in the IUWM DSF

The Source-catchment module was utilised only to examine runoff generation and transportation processes associated with urban catchments. In particular, the Source-catchment module examined runoff transportation until runoff was captured for possible consumption. If urban runoff was not captured, the Source-catchment module routed the uncaptured urban runoff hydrologically to receiving waters, i.e. Gulf St Vincent. On the other hand, if the runoff was captured for possible consumption, supply and discharge dynamics from the point of capture was examined by using the Source-schematic module. An implication of this approach was that any overflows and by-pass flows from the infrastructure provided for runoff capture, were not hydrologically routed, instead such flows were added to the hydrologically routed flows at the point of discharge, to determine the total discharge volume. All of the water not diverted by schemes was assumed to flow to the receiving waters. Figure 13 shows how the modules of Source model were used in the IUWM DSF to identify optimal supply portfolios in terms of the objectives described in Section 4.3 subject to the constraints (described in Section 4.3).

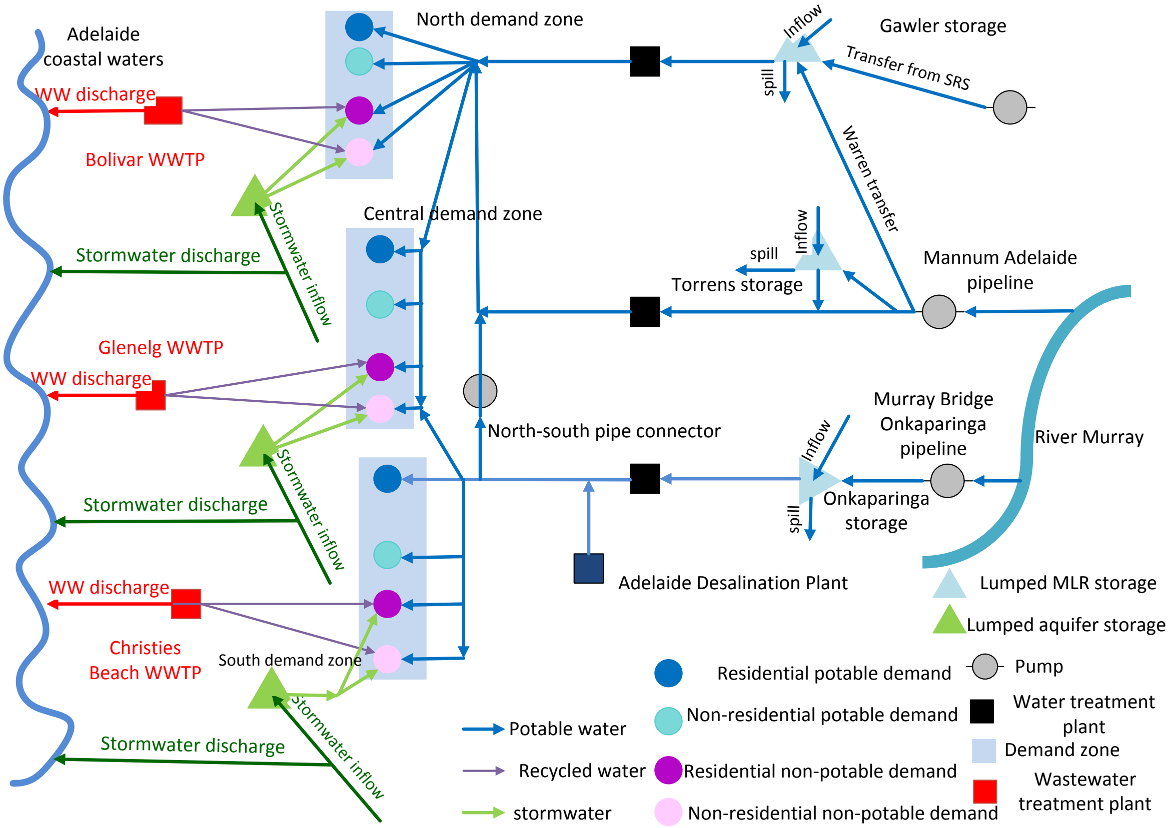


Figure 14: A schematic diagram showing supply from River Murray, MLR catchments, ADP, wastewater and stormwater sources to residential and non-residential, potable and non-potable demands

The time-step of the simulation was decided by considering the datasets available for examining supply and demand dynamics. These included inflows to MLR storages, demands and the existing supply system operating rules. Since such data were available on a monthly basis, and the availability of time and resources was limited to generate new datasets of a different temporal resolution, the time-step of simulation for examining supply, demand and discharge dynamics, was considered to be a month. However, for examining runoff generation and hydrological routing, a monthly time-step was considered to be not appropriate. Hence the Source-catchment module was set up on a daily basis to provide data on stormwater inflows or urban catchment runoff (Figure 13). Daily runoff values were aggregated to monthly, at the harvesting locations and fed into the Source-schematic module, to undertake supply, demand and discharge interactions.

Spatially, the simulation model represented three demand zones in the study area (Figure 11). A few supply sources were located outside the study area (i.e. the RM and the storages in MLR) and the rest was located within the study area (i.e. the ADP, stormwater harvesting schemes, recycled water plants, groundwater and households with rainwater tanks and demand management options).

The simulation model developed in the Source-schematic module represented water sources as nodes, water demand of zones as nodes, supply from sources to demands as links, and discharges to receiving water as links. The source nodes received water from inflow nodes. Inflows were defined by using either datasets generated outside of the Source model or datasets generated by the Source-catchment and Source-schematic modules. The modelling method of each source, including the inflow generation method of each source is described in the rest of this section.

The demand nodes represented potable and non-potable, residential and non-residential water demands of the north, central and south demand zones (Figure 11). That is, each demand zone comprised four demand nodes to represent: residential potable, residential non-potable, non-residential potable and non-residential non-potable demands. A schematic diagram of the simulation model with five sources, i.e. River Murray, MLR catchments, ADP, wastewater and stormwater and 12 demand nodes is shown in Figure 14. The simulation model was equipped with datasets to cover a period of 50 years, from July 1963 to June 2013.

As described above, the simulation model contained the methods to perform monthly water balance of the study area and compute the objective function consisting of capital and operational infrastructure cost, capital and operation energy, volumetric reliability of non-potable demands and wastewater and stormwater discharges to the Gulf subject to the constraints described in section 4.3.1. Therefore, if required, the simulation model could be used as a standalone tool, to examine the consequences of 'what-if' scenarios in terms of the criteria used in the objective function, as well as the components of the water balance (inflows to sources, storage behaviour of sources, etc.).

4.6.2 Modelling extractions from the River Murray

Extraction of water from the River Murray was modelled as an infinite capacity storage node with no rainfall on or evaporation from the surface, and a defined capacity valve on the release link downstream of the storage. The valve capacity represented the capacity of pumps at the extraction location.

The extractions from the River Murray were provided to the supply system via three pipelines:

- Mannum-Adelaide pipeline (capacity 364 ML/d)
- Murray Bridge-Onkaparinga pipeline (capacity 510 ML/d)
- Swan Reach-Stockwell pipeline (capacity 79 ML/d).

A 'Maximum Order Constraint' node was provided on the release link of the each storage, downstream of the valve, to control the amount of water extracted from the River Murray, either manually or through optimisation (i.e. *Optimisation RM to MLR* in Figure 15).

The model maintained a running total volume of water pumped from RM over a five year cycle. If the limit was breached, a flag was triggered that set the capacity of three Maximum

Order Constraint nodes to 0, effectively shutting off supply from the RM until the cycles restarted (i.e. *Limit RM pumping 5-year cycle* in Figure 15).

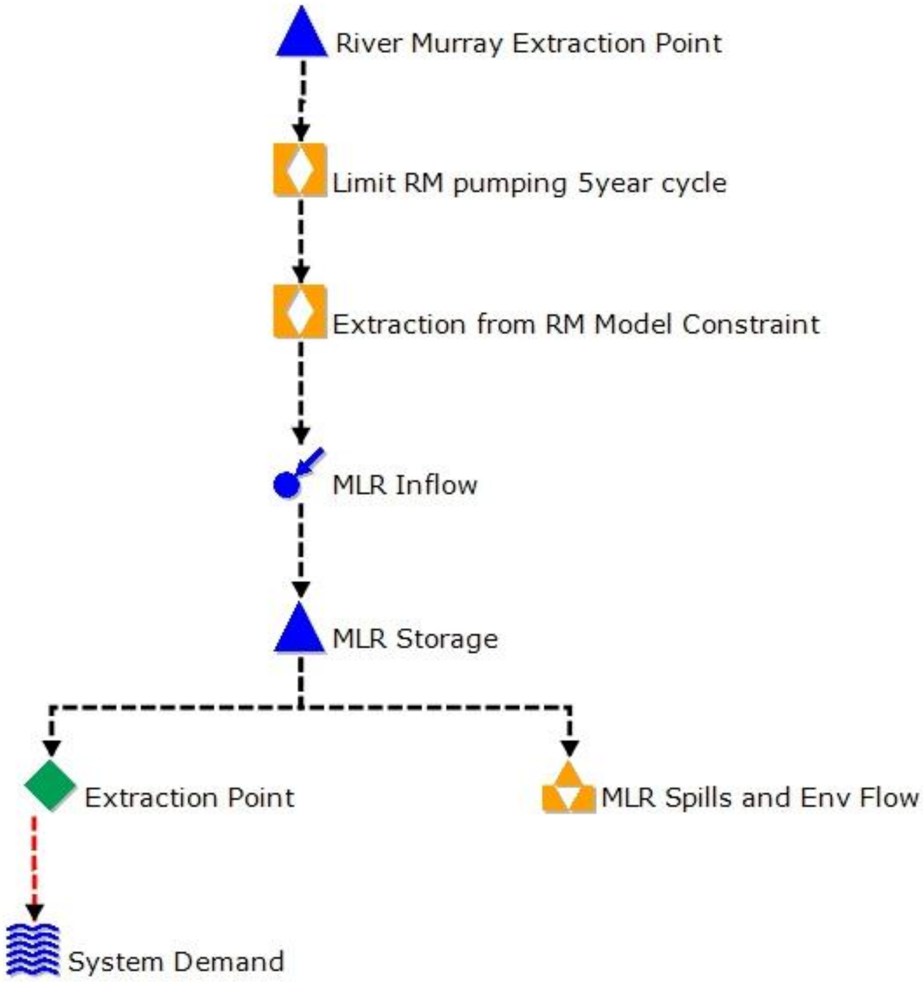


Figure 15: A schematic diagram illustrating the modelling method adopted for extracting water from the River Murray

Another limit imposed on extractions from the RM was a Maximum Order Constraint node which allowed for setting a maximum volume per time-step to be pumped (i.e. *Extraction from RM Model Constraint* in Figure 15). The value of this node was set by a 'global variable', which denoted a variable that could be manipulated by Source-insight module.

Extracted water from the River Murray was utilised only for human consumption and maintaining minimum storage targets in the storages in MLR, i.e. water was not extracted from the River Murray for meeting such demands as environmental flow and evaporation from a dam's surface. Owing to the fact that releases were made from the MLR storages (*MLR Storage* in Figure 15) to meet both human demands (*System Demand* in Figure 15) and environmental requirements (*MLR Spills and Env Flow* in Figure 15), the model distinguished between releases when reporting the volume of demand supplied by each water source. In this case, an expression (details on 'expressions' are given in eWater, 2012) used to calculate the volume of demand supplied by the RM was assumed to be as much as possible of the flow released by the MLR storages for meeting demands, up to the maximum of the volume pumped from the RM. Any balance of orders was therefore assumed to be met by releases from the MLR catchments. In instances where the inflow into the MLR storages (*MLR Inflow* in Figure 15) was too low to maintain storage levels above the targets after demand orders were met, further extractions were made from the RM and stored in the MLR storages to maintain required target storages.

The Source-schematic module uses a Network Linear Programming (NLP) approach to supply water from multiple sources to demands. The NLP approach optimises water supply from multiple sources each month based on penalty costs assigned to each supply source. The penalty costs set preferences for sources.

For the RM source, the penalty costs were manipulated manually to ensure the RM extractions were utilised to meet potable demands prior to utilising as a backup supply for non-potable demands.

It should be noted however that the current modelling methodology adopted by SA Water optimises water supply from multiple sources across multiple months to ensure that pumping is spread out so that water security can be maintained while maximising the efficiency of the system and to minimise the use of high energy costs periods. Optimisation from month to month (such as in NLP) can leave the system vulnerable to extreme weather events or supply interruptions (eg black water or major algal bloom in RM). However, the Source model can overcome this limitation if NLP is used in conjunction with the Insight module, and the insight module is allowed to use penalty costs as decision variables. We could not use penalty costs as decision variables due to some software development limitations in the version of Source model used for this study.

4.6.3 Modelling storages in MLR

The surface water storages and weirs in the MLR catchments were aggregated to three storages, in order to reduce the complexity of pipe connections. The combined capacity of the three surface water storages was 171 GL.

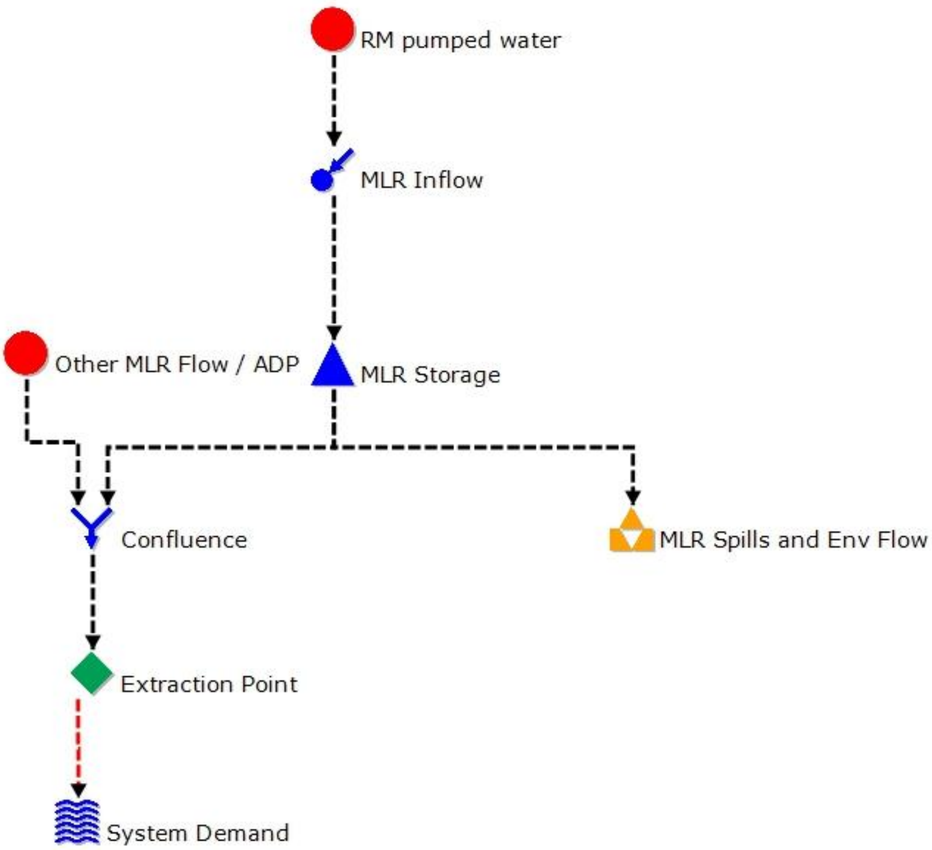


Figure 16: A schematic diagram illustrating sourcing water from a storage in the MLR catchment

Table 19: Environmental flow requirements of aggregated storages (source: http://www.naturalresources.sa.gov.au/adelaidemtloftyranges/water/managing-water/water-courses/environmental-flows)

Month	Environmental flow requirement of aggregated storages (ML)		
	Gawler	Torrens	Onkaparinga
Jan	0	7.75	0
Feb	0	7	0
Mar	120	7.75	500
Apr	0	7.5	900
May	120	7.75	930
Jun	413.5	7.5	1850
Jul	226.3	7.75	930
Aug	226.3	7.75	900
Sep	413.5	7.5	1850
Oct	226.3	7.75	930
Nov	269.1	802.5	300
Dec	0	7.75	310
Total	2015	886.25	9400

Table 20: Monthly targets storage levels (in GL) for the aggregated storages in MLR catchments

	Gawler	Torrens	Onkaparinga
Max Capacity GL	54.1	59.1	57.8
Dead Storage GL	6.2	9.1	5.7
Jan	26.0	31.0	29.0
Feb	24.5	26.0	25.0
Mar	24.5	22.0	20.0
Apr	23.3	20.0	17.0
May	23.3	18.0	15.0
Jun	23.3	13.4	12.2
Jul	24.5	22.0	23.0
Aug	28.0	29.0	29.0
Sep	30.4	36.5	34.5
Oct	30.4	42.0	37.0
Nov	30.4	39.0	37.0
Dec	28.0	36.5	33.0

The three aggregated storages were:

- Gawler, which represented the lumped storage of Warren, Barossa and South Para reservoirs
- Torrens, which represented the lumped storage of Millbrook, Kangaroo Creek and Hope Valley reservoirs and Gumeracha and Torrens Gorge weirs
- Onkaparinga, which represents the lumped storage of Mount Bold and Happy Valley reservoirs and Clarendon weir.

An aggregated storage was modelled as a finite capacity storage, reflecting the aggregation of the storages that it represented (Figure 16). A storage node received inflow from an inflow node (e.g. *MLR Inflow* in Figure 16) and made releases to meet human consumption and environmental flow requirement. A 'Minimum Flow' node defined the environmental flow requirement (e.g. *MLR Spills and Env Flow* node in Figure 16). Spills from a storage contributed to meeting the environmental flow as well, i.e. if spills at a particular time-step was greater than the environmental flow requirement at that time-step, water was not released from the storage.

The environmental flow requirements of the aggregated storages are given in Table 19. Releases from the storages for human consumption were governed by the monthly target storage levels. The target storage level for an aggregated storage was derived from aggregating the targets of individual storages. The targets could vary from month to month. This was modelled by configuring Ordering Network Costs in the Source-schematic module. The configuration ensured that the portion of the water stored above a given target level was of a lower cost (encouraging its use) compared to the portion of water stored below that given target (discouraging its use). By making the portion of the storage below the target level to be less favourable than the other sources in the model, the MLR storages would meet target storages at the end of each time-step in most cases. Table 20 lists the target storage for each month in GL.

Modelling inflows to MLR storages

Monthly inflows for the three storages were sourced from SA Water. However, the inflow records were available only for July 1998 to June 2013 (15 years). The average annual inflows over the last 15 years, for Barossa, Torrens and Onkaparinga aggregated storages, computed from this observed data were: 19,897 ML, 35,436 ML and 48,576 ML, respectively.

WAPABA (Water Partition and Balance) model (Wang et al., 2011) was used extend the inflows from July 1963 to June 2013. This was to allow a 50-year the simulation, under the current climate, i.e. 2013 scenario. WAPABA was a monthly water balance model, for predicting monthly streamflows based on monthly rainfall and potential evapotranspiration (PET) (Figure 17). It partitioned the monthly rainfall into catchment water consumption and catchment water yield. The actual water available for evapotranspiration was partitioned to actual evapotranspiration and water remaining in the soil water store. The catchment water yield was partitioned into surface runoff and groundwater store, which contributed to base flow. The monthly runoff was the sum of surface water runoff and base flow. This illustrated in Figure 17. The model consisted of four parameters, which were determined through calibration.

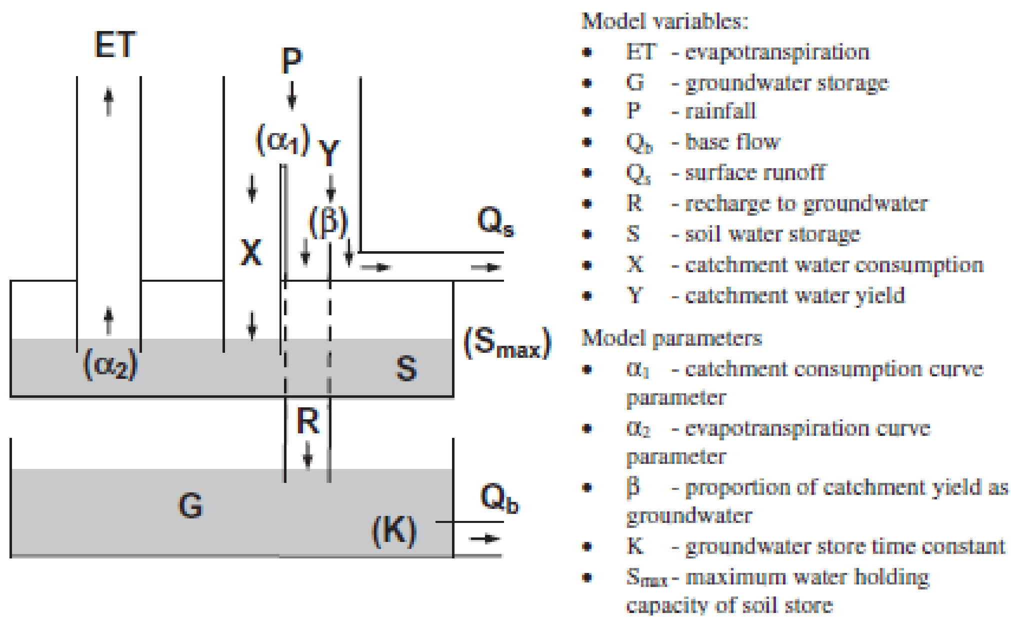


Figure 17: WAPABA model (source: Wang *et al.*, 2011)

Table 21: Extending MLR catchment inflows to 50 years using WAPABA Model (Wang et al., 2011)

Input data	Observed inflow 1998-2013 Observed rainfall 1952-2013 Observed evaporation 1952-2013 (Patch Point data from Silo)
Calibration	Warm-up period: 1952-1998 Calibration period: 1999-2012 Model efficiency (Nash and Sutcliff, 1970) Barossa: 0.77 Torrens: 0.63 Onkaparinga: 0.62
Simulation	Warm-up period: 1952-1962 Simulation period: 1962-2013

Rainfall and potential evapotranspiration (PET) data from Williamstown (BOM 23752), Millbrook (BOM 23731) and Mount Bold Reservoir (BOM 23734) were used for modelling the inflows to Barrossa, Torrens and Onkaparinga storages, respectively. The method of extending inflow records for the simulation period involved calibrating and validating the WAPABA model using the observed data, and generating monthly inflows over the period of simulation using the calibrated WAPABA model (Table 21). The modelled inflows for 2013 scenario, for Barossa, Torrens and Onkaparinga aggregated storages are shown in Figure 18, Figure 19 and Figure 20, respectively.

Table 22 Comparison of key statistics of observed monthly flows (Jul 1998 to May 2013) and modelled monthly flows (July 1962 to June 2013)

Statistic	Barossa		Torrens		Onkaparinga	
	observed	Modelled	observed	Modelled	observed	Modelled
Mean	1671.8	1813.4	2971.4	4483.9	4066.6	3940.2
Standard deviation	3436.6	3723.2	5124.9	9382.3	6508.3	5886.6
Minimum	0.0	0.0	0.0	0.0	0.0	0.0
Maximum	25401.0	34423.3	32712.0	71525.0	37724.0	38619.3
Coefficient of variation	2.1	2.1	1.7	2.1	1.6	1.5

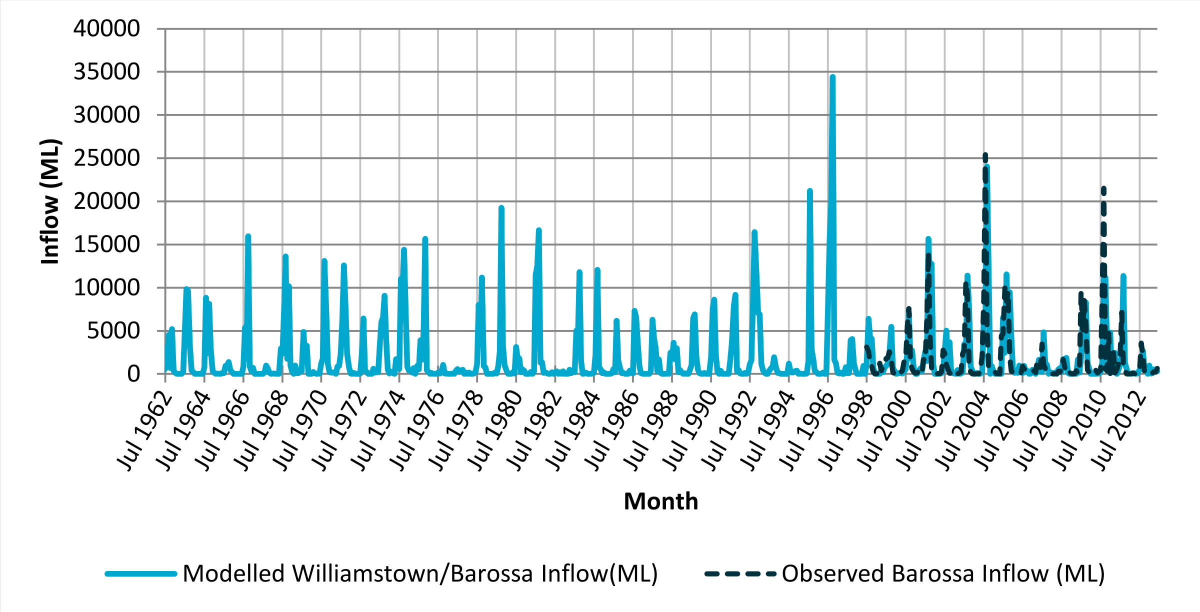


Figure 18: Monthly modelled inflows to Barossa aggregated storage (July 1962 – June 2013)

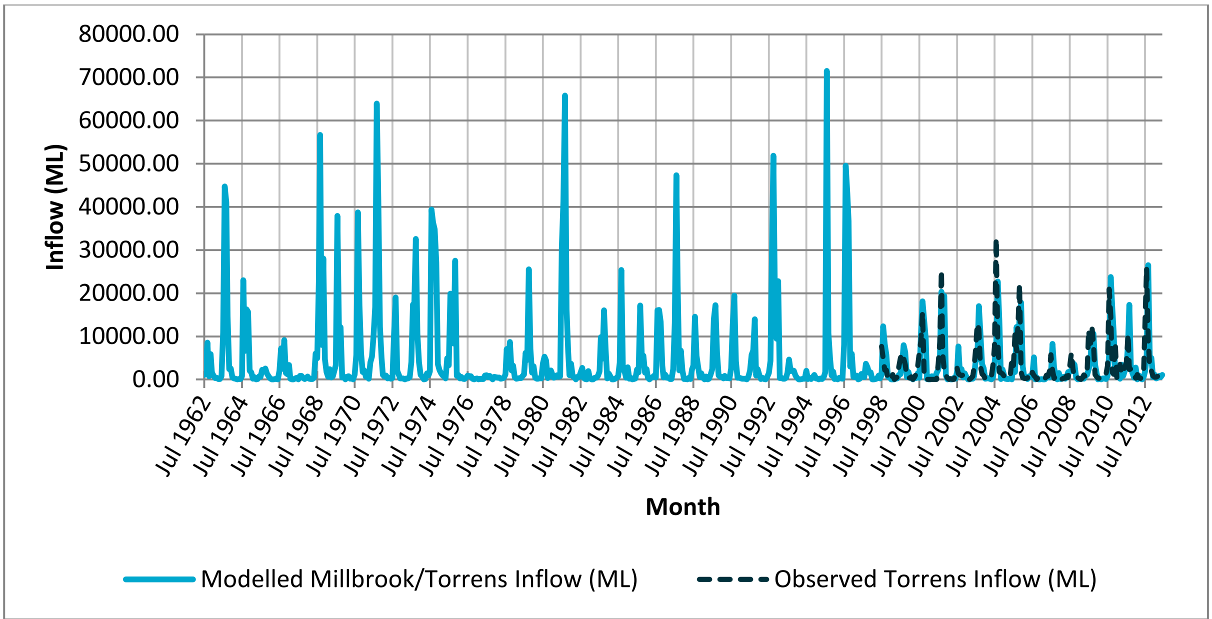


Figure 19: Monthly modelled inflows to Torrens aggregated storage (July 1962 – June 2013)

As mentioned in Chapter 3, this study used the average seasonal (i.e. summer, autumn, winter and spring) changes obtained from CSIRO OzClim climate change generator to generate climate data corresponding to 2025 and 2050 scenarios, in the absence of representative climate projections for metropolitan Adelaide. Spending time and resources to determine monthly changes to inflows under future climate, using these average seasonal changes to climate, was considered as not sensible and useful. Hence, this study used the report by Heneker and Cresswell (2010), which examined the potential impact of climate change on water resource availability in the MLR, to estimate the potential changes to inflows under the future climate. Heneker and Cresswell (2010) used A2 (higher greenhouse gas emissions) and B2 (lower greenhouse gas emissions) scenarios, and examined inflows to all the storages in MLR, lower Torrens area and the urban areas in MLR, from 2035 to 2064.

Using linear interpolation, the percent reduction in inflow in 2025 and 2050, compared to current (i.e. 2013), was estimated as 9% and 26%, respectively for the rural catchments, and 6% and 17% respectively for urban catchments. These reductions represent the average of reductions corresponding to A2 and B2 emission scenarios.

The reductions corresponding to rural catchments were used to reduce the monthly inflows determined by using the WAPABA model, to determine the inflows to Barossa, Torrens and Onkaparinga aggregated storages, under 2025 and 2050 scenarios. Similarly, the reductions corresponding urban catchments were applied to monthly urban runoff under the current climate (i.e. 2013), to determine urban runoff under 2025 and 2050. It is strongly recommended that 2025 and 2050 MLR inflows and urban runoff be replaced in any further work based on this study, using the climate projections developed by the Goyder Water Research Institute funded project on climate change.

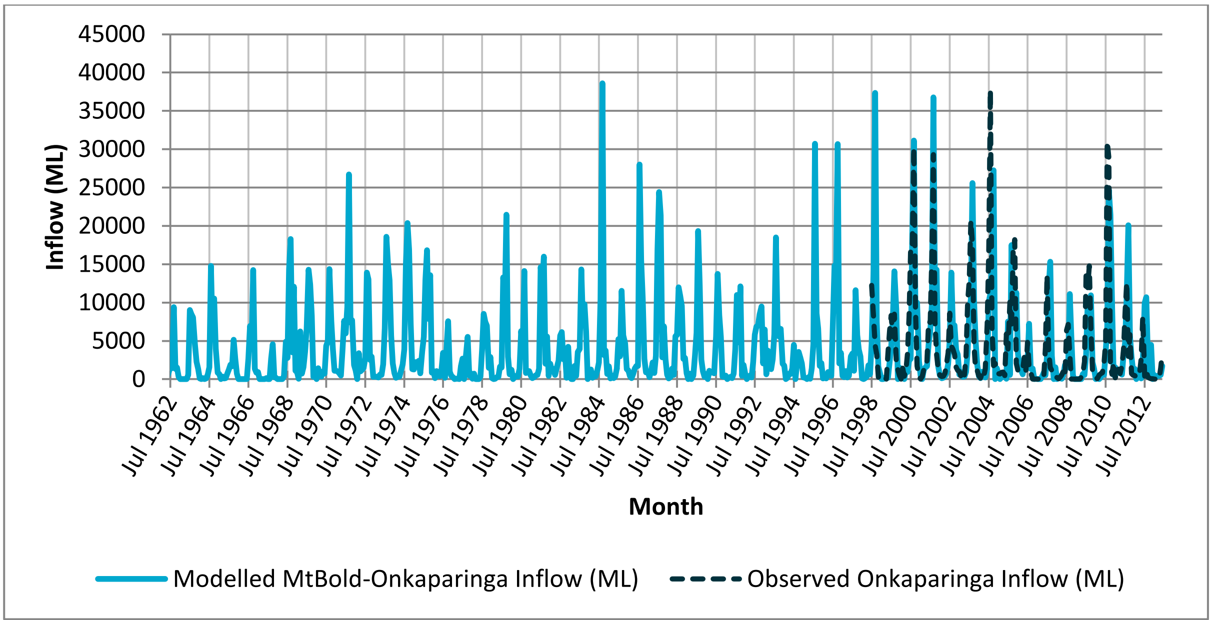


Figure 20: Monthly modelled inflows to Onkaparinga aggregated storage (July 1962 – June 2013)

4.6.4 Modelling supply from Adelaide Desalination Plant

The Adelaide Desalination Plant (ADP) was modelled as an infinite capacity storage with no rainfall on or evaporation from the surface, and a defined capacity valve on the release link downstream of the storage. The valve capacity represented the capacity of the ADP, i.e. 300 ML/d.

Another limit on the use of the ADP within the model was a Maximum Order Constraint node which allowed for setting a maximum volume per time-step of supply from the ADP (*Extraction from ADP Model Constraint* in Fig. 16). The value of this node was set by a global variable, thus allowing direct manipulation and assignment by Source-insight module.

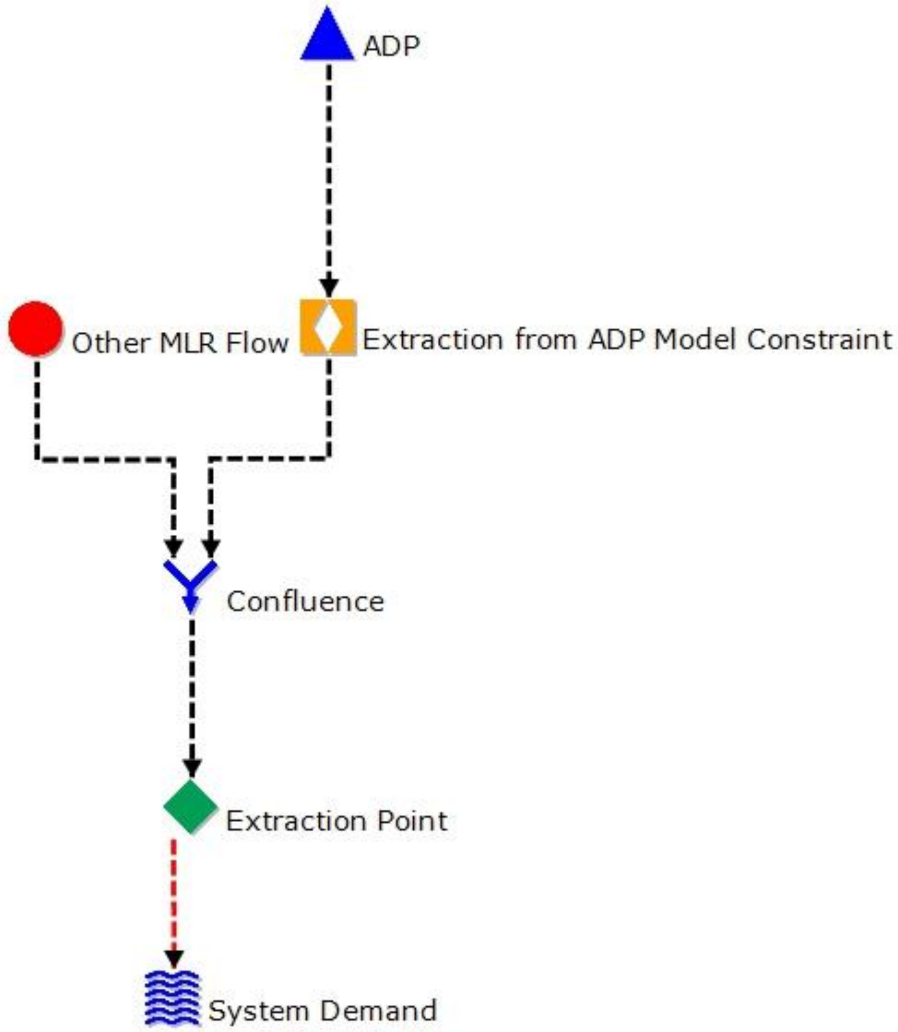


Figure 21: A schematic diagram showing the modelling method adopted for sourcing water from the ADP

4.6.5 Modelling recycled water and treated wastewater discharge

Supplying recycled water from three major wastewater treatment plants (WWTPs): Bolivar, Glenelg and Christies Beach, to non-potable demand of both residential and non-residential users were included in the simulation model. All three plants currently produce recycled water and distribute it for non-potable demand of non-residential use. In addition to these three major plants, the plant at Bolivar has a high-salinity treatment plant. However, treated effluent from this plant was not considered suitable for recycling.

Each WWTP was assumed to be supplying only a single demand zone. The northern demand zone was assumed to be supplied by Bolivar WWTP. Similarly, the central zone was supplied by Glenelg WWTP and the southern demand zone by Christies WWTP. Any treated wastewater not reused, was assumed to be discharged to the ocean, on a month by month basis i.e. no long term storage capacity provided at each WWTP (Figure 22). If required, further treatment was provided to treated wastewater, before supplying it to non-potable demands as ‘recycled water’ (Figure 22).

The following variables were used to control the use of recycled water from each WWTP (i.e. decision variables of the optimisation):

- capacity to produce recycled water
- the proportion of recycled water provided to residential customers.

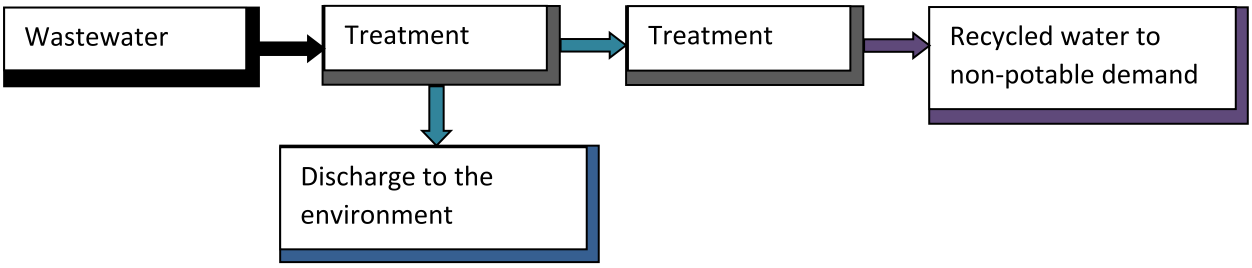


Figure 22: Conceptualisation of recycled water generation process

Each plant assumed to have a 'base capacity', which defined the existing capacity to produce recycled water (Table 23), and a maximum capacity to produce treated wastewater. It was assumed that this maximum capacity defined the maximum allowable upgrade to a plant, to produce recycled water (assuming all of the treated wastewater could be recycled).

Table 23: Existing capacity to produce recycled water at Bolivar, Glenelg and Christies Beach WWTPs

WWTP	Current Plant Capacity (ML/year)	Current Recycling Capacity (ML/year)
Bolivar	60,225	38,325
Glenelg	21,900	3,800
Christies beach	16,425	16,425

The capacity to produce recycled water for each WWTP was defined, if the recycled water was to be used as a source of supply. It could be any continuous value between 0 to the maximum capacity. Any increase above the existing recycling capacity was considered as an upgrade, and a cost would be added, to reflect the capital cost of the upgrade.

Treated wastewater not recycled, was assumed to be discharged to the coastal waters, which might contribute to pollution effects in the coastal waters. The proportion of recycled water provided to residential customers was assumed to be the same as the proportion of the total volume of recycled water supplied to residential non-potable purposes. The remaining portion of the recycled water produced was supplied to non-residential non-potable purposes.

When representing recycled water in the simulation model (Figure 23), a storage node was used to represent a wastewater treatment plant (WWTP). The link downstream of the storage node represented the outflow from a WWTP (i.e. treated wastewater). This link was connected to a splitter node which split the treated wastewater stream to a recycled water stream and a treated wastewater stream (Figure 23). The link representing recycled water stream was connected to a second splitter node, which split recycled water into residential non-potable supply stream and non-residential non-potable supply stream. The link representing the treated wastewater stream (i.e. portion of treated wastewater not recycled), was connected to a gauge node, to represent the discharge of treated wastewater stream to the environment, which occurred at the end of each simulation time-step. This representation assumed that:

- inflow to a WWTP at a particular simulation time-step (say, T) was treated fully during that time-step
- the treated wastewater produced in time-step T was available to use as recycled water, if there was a demand for recycled water in time-step T
- the portion of treated wastewater not recycled in time-step T, was discharged to the environment, at the end of time-step T (i.e. no storage provided for treated wastewater).

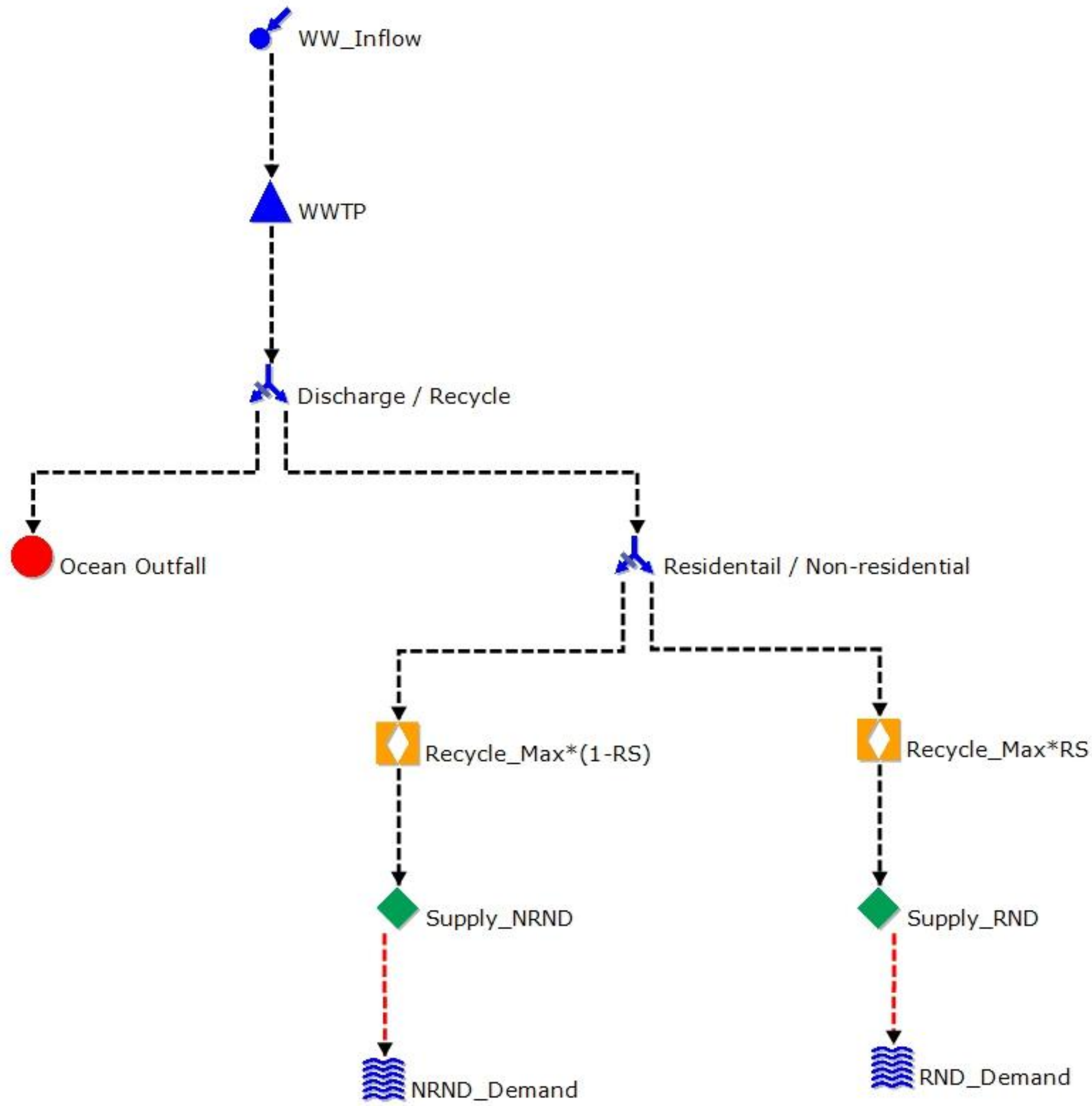


Figure 23: Schematic diagram illustrating wastewater treatment and reuse modelling method

The two links associated with the second splitter were provided with maximum flow nodes, one for each link. The capacity of the each maximum flow node was computed from the following two variables, which could be defined either by the users of the IUWM DSF or as part of the optimisation (as decision variables):

- the recycled water production capacity of the plant, which could vary from 0 to the maximum capacity to produce recycled water
- the proportion of recycled water supplied to residential non-potable purposes.

The sum of the capacities of two links associated with the second splitter represented the recycled water production capacity of the plant. If the sum of the capacities of two links associated with the second splitter was greater than the base capacity of the plant, an upgrade would be provided, otherwise no upgrade would be provided. As mentioned above, the capital cost would reflect the cost of plant upgrade.

Modelling inflows to WWTPs

Daily wastewater inflow data for each major WWTP was available for the period 1991 to 2013 (23 years). As mentioned above, the simulation model was set up to examine supply, demand and discharge interactions over a 50-year period, on a monthly basis. Hence the historical wastewater inflows were examined, with a view to developing a model to extend the observed wastewater inflows over a 50-year period, as well as to examine the potential changes to wastewater inflow under the future climate. The model was intended to capture the seasonality in wastewater flows brought about by groundwater infiltration and other seasonal processes. In addition, the natural variability in the observed results was included, to make it more realistic.

Table 24: Fitted seasonality model parameters for WWTPs in Metropolitan Adelaide

Model Parameters	Bolivar	Glenelg	Christies Beach
Q_{offset} (ML/month)	2.1	0.9	0.7
Q_{VarMax} (ML/month)	234.3	104.5	62.3
ϕ (days)	222	195	236
Model Efficiency (Nash and Sutcliff, 1970)	0.40	0.37	0.59

While there was some degree of long-term variation in wastewater inflows, no obvious long term trends in the flow were observed. The historical data was smoothed using a symmetric moving average with an averaging window of 1-year. The seasonal variation in flows was isolated by subtracting the smoothed data from the original values. Consolidating seasonal variation by month showed a trend for higher flows during the cooler months of the year while in summer the flows were reduced (Figure 24), which indicated that groundwater infiltration was significant.

Fitting a 3rd order polynomial suggested the variation followed a trend similar to a sine curve. The fitted function was of the form:

$$Q_{seasonality} = Q_{offset} + Q_{VarMax} \mathrm{SIN}(T+\phi)$$

Equation 30

Where, $Q_{seasonality}$ is the flow (ML/month) during a specific day of the year, Q_{offset} (ML/month) was a constant, Q_{VarMax} was the magnitude of the seasonal variability in flows (ML/month), T was the year fraction (day/365) and ϕ is the phase offset to align the peak in the curve. The coefficients of the fitted function are given in Table 24.

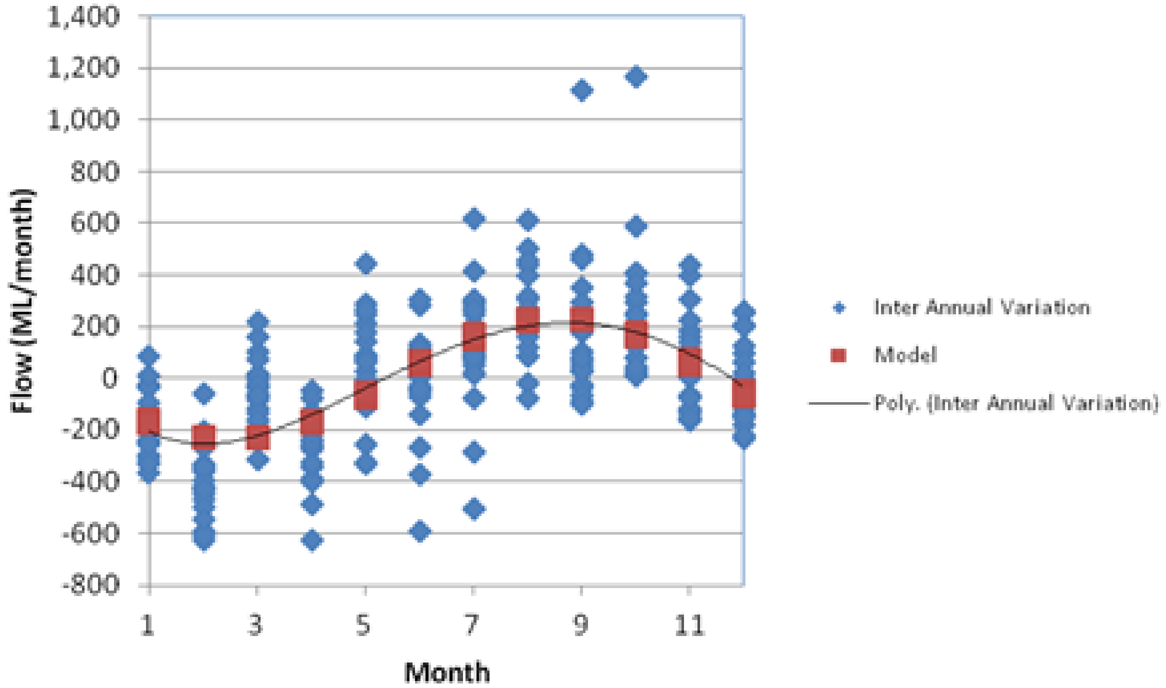


Figure 24: Observed seasonal variation in wastewater inflows for the Bolivar WWTP 1991–2013 (note: month 1 represents January and month 12 represents December)

The model appeared to reproduce the seasonal trend in flow in an acceptable manner (i.e. Nash and Sutcliff (1970) model efficiency values for Bolivar, Glenelg and Christies Beach inflows were 0.40, 0.37 and 0.59, respectively). However, the actual flow data appeared to be much more variable in the short term with peaks either higher or lower than the modelled values. For example, modelled and observed wastewater inflows to Bolivar WWTP are shown in Figure 25.

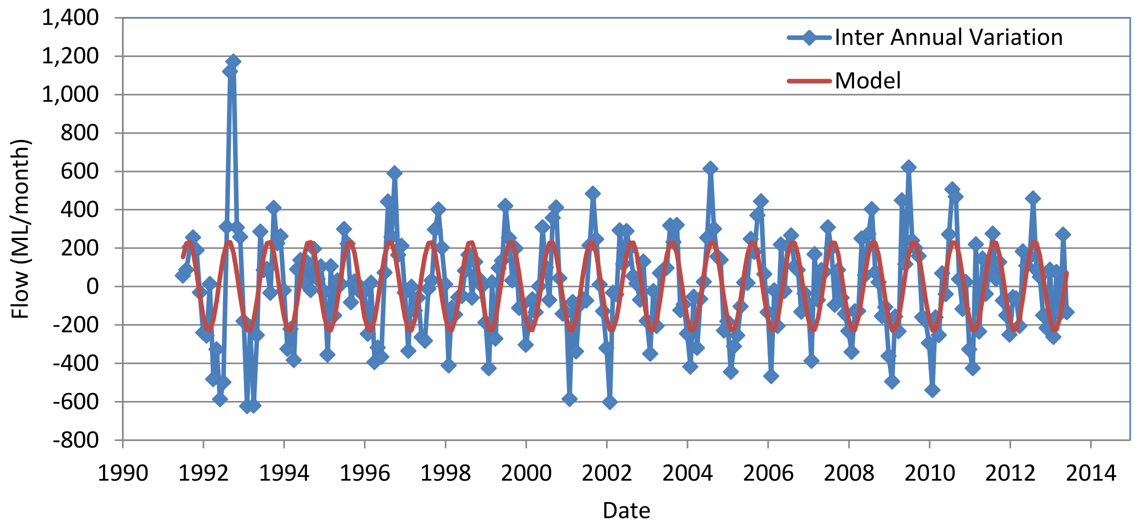


Figure 25: Comparison of modelled (red line) and observed (blue line) seasonal variation in wastewater inflows for the Bolivar WWTP 1991–2013

The difference ($Q_{variability}$) between the observed seasonal variation and that predicted by the model (Figure 25) were grouped by month and the distribution of values investigated (Figure 24). In addition, data for $Q_{variability}$ were compared for each WWTP. There was no correlation between pairs of data suggesting that there was no underlying process that modified the flow from the simple seasonal variation model. For example, if high monthly rainfall would result in $Q_{variability}$ values higher than predicted using the model, it would be expected that values from all WWTP would be higher (correlation). This was not the case (Figure 26) suggesting that the variation was random.

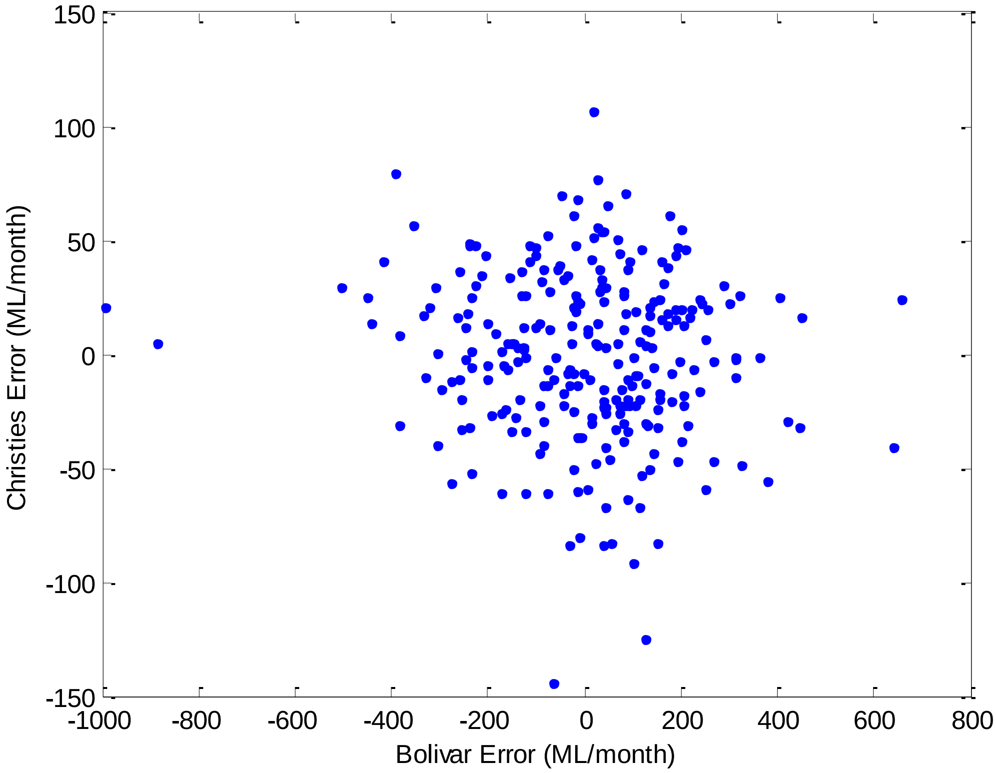


Figure 26: Scatter plot comparing $Q_{variability}$ values for Bolivar and Christies Beach WWTPs 1991–2013

Table 25: Probability Distributions used to model 'variable' component of the wastewater inflow

Month	Bolivar			Glenelg			Christies Beach		
	Distribution	K-S	p	Distribution	K-S	p	Distribution	K-S	p
January	normal	0.13	0.81	normal	0.11	0.91	logistic	0.10	0.96
February	logistic	0.08	0.99	normal	0.10	0.96	normal	0.13	0.78
March	Hyper secant	0.08	0.99	Gumbel Max	0.08	0.99	normal	0.09	0.99
April	gamma	0.10	0.97	Gumbel Max	0.15	0.67	normal	0.11	0.89
May	normal	0.08	0.99	normal	0.10	0.96	3P-weibull	0.11	0.91
June	Laplace	0.16	0.56	Laplace	0.11	0.92	3P-weibull	0.11	0.94
July	Laplace	0.14	0.77	Laplace	0.13	0.83	normal	0.10	0.95
August	Laplace	0.10	0.96	normal	0.10	0.96	3P-weibull	0.15	0.68
September	beta	0.16	0.61	3P-weibull	0.12	0.90	uniform	0.13	0.80

Month	Bolivar			Glenelg			Christies Beach		
	Distribution	K-S	p	Distribution	K-S	p	Distribution	K-S	p
October	beta	0.14	0.74	3P-weibull	0.10	0.97	3P-weibull	0.08	0.99
November	beta	0.11	0.94	3P-weibull	0.13	0.80	normal	0.15	0.66
December	beta	0.13	0.79	3P-weibull	0.10	0.95	normal	0.09	0.99

Possible causes for this observed randomness were not known. It could be due to groundwater infiltration due to factors other than climate (e.g. leaked water from underground pipes). Probability distributions were employed to describe this observed randomness in wastewater inflows. Accordingly, for each WWTP, twelve probability distributions were identified, one for each month (Table 25). Easy Fit 5.5 Professional (MathWave Technologies, 2010) software was used to identify statistically best-fit probability distributions for each month. Kolmogorov-Smirnov (K-S) Goodness of Fit test was used evaluate statistically best-fit distribution. The p-values implied the probability of acceptance of the probability distributions and as seen from Table 25, the best fit distributions are acceptable at more than 50% rate for all the months. Random values from probability distributions were generated using the Matlab script 'Randraw' (Alex Bar-Guy, 2005).

The above-described analysis led to describing of monthly wastewater inflows of each WWTP, using the following relationship:

$$Q_{ww}=Q_{base} + Q_{seasonality} + Q_{variability}$$

Equation 31

where, Q_{ww} is the wastewater inflow of each WWTP (ML/month), Q_{base} (ML/month) is the long-term annual average wastewater flow, computed from the observed wastewater inflow data, $Q_{seasonality}$ is the monthly variability in flows due to groundwater and stormwater infiltration and other seasonal factors (ML/month), $Q_{variability}$ is the random variation observed in recorded wastewater inflow data (ML/month).

Table 26: Average annual modelled wastewater inflow for2013, 2025 and 2050

Year	Average modelled wastewater inflow (ML) (July 1963 to June 2013)		
	Bolivar	Glenelg	Christies Beach
2013	4156	1452	877
2025	4564	1595	963
2050	5277	1846	1113

The long-term annual average wastewater flow generally linked to population. Hence, for 2013 scenario, which represented the current climate, the observed daily wastewater inflows were used to compute Q_{base} were: 4177 ML/month, 1469 ML/month and 878 ML/month for Bolivar, Glenelg and Christies Beach WWTPs. For 2025 and 2050 scenarios, these values were scaled up by using the projected population in Table 3, i.e. Q_{base} in 2025 = Q_{base} in 2013 X (population in 2025/population in 2013).

The seasonality and variability were assumed to be unchanged by the increase in population. However, these could be affected by the change in climate. Nevertheless, it was assumed that the seasonality and variability in 2013 was applicable to 2025 and 2050. This assumption was made in the absence of climate data under the influence of climate change. Matlab was also used to generate monthly wastewater inflows over a 50 year time span, using Equation 31, which were then used as input to the simulation model developed in Source-schematic module. The average annual and average monthly modelled wastewater inflow using Equation 31 are shown in Table 26, Table 27 and Figure 27.

Table 27: Average monthly wastewater inflow

Month	Average modelled wastewater inflow (ML) (July 1963 to June 2013)		
	Bolivar	Glenelg	Christies Beach
January	4056	1425	814.5
February	4102	1449	845.6
March	3753	1317	792.6
April	4054	1399	867.2
May	3984	1377	862.6
June	4228	1436	904.4
July	4328	1485	916.8
August	4397	1493	931.1
September	4413	1524	953.0
October	4234	1531	900.4
November	4245	1512	898.9
December	4081	1477	839.0

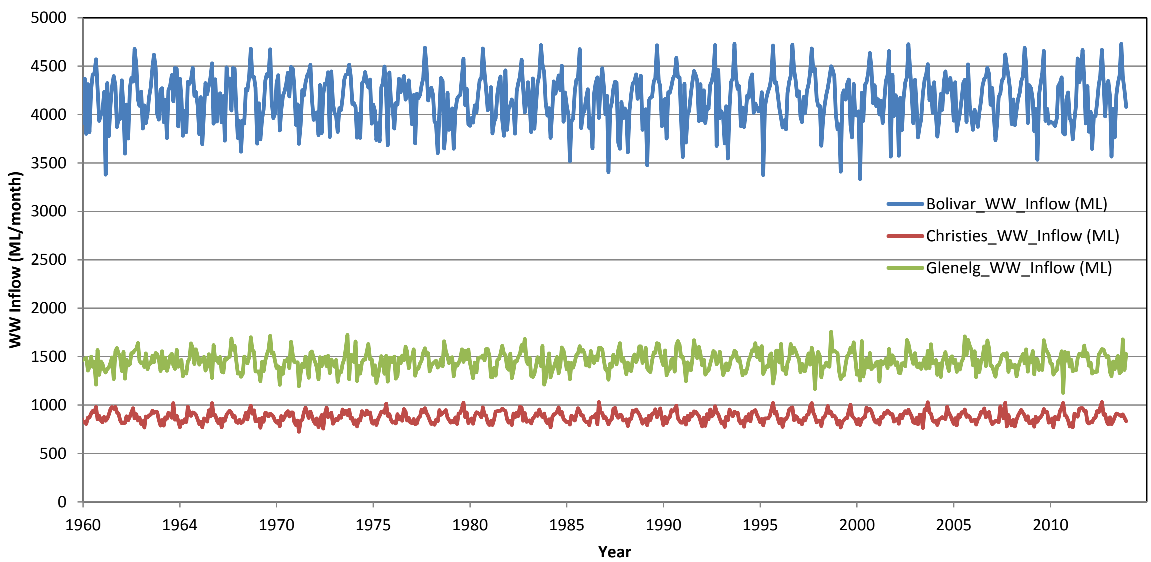


Figure 27: Modelled monthly wastewater inflow for 2013 scenario by considering the climatic variability during 1960 to 2013

4.6.6 Modelling stormwater harvesting and discharge

The harvesting method of stormwater was assumed to be Aquifer Storage and Recovery (ASR). An ASR is a common injection-extraction system, which uses the same well for both injection of treated stormwater into an aquifer and recovery of treated stormwater from that aquifer (Figure 28).

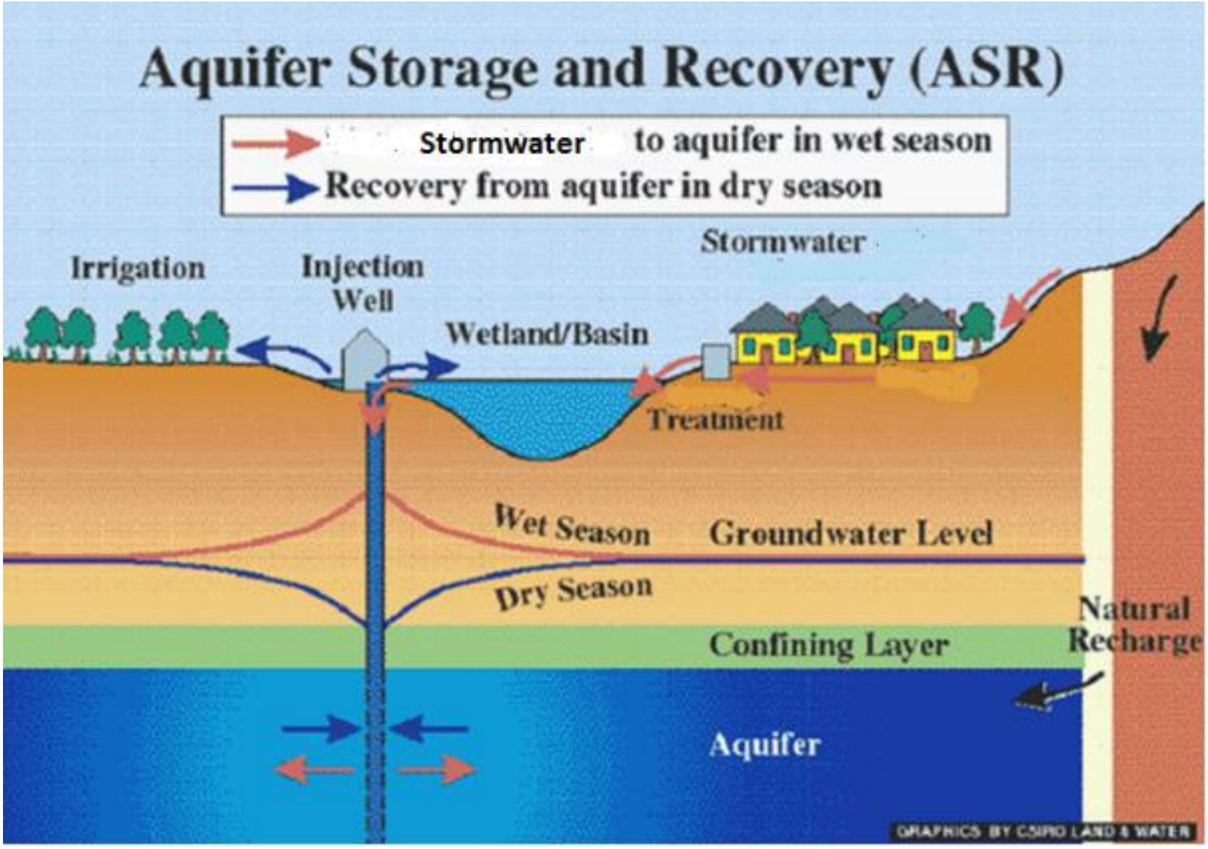


Figure 28: Stormwater harvesting method – Aquifer Storage and Recovery (Image sourced from: CSIRO)

An ASR scheme was assumed to be comprised a diversion of stormwater from a drainage system to a holding pond, from which captured stormwater discharges to a wetland, which provides biological treatment to stormwater, prior to injection to the aquifer. The injection was considered to be carried out during winter season, from May to September (5 months). The recovery was considered to be carried out during summer season, November to April (6 months). A minimum retention period of one month was assumed between injection and recovery of treated stormwater. The amount of injection and recovery was assumed to be controlled by the optimisation.

Based on Wallbridge and Gilbert (2009), 70 existing and proposed harvesting sites across Metropolitan Adelaide were identified (Table 28). Of the 70 schemes, 27 schemes were operational in 2013 (Table 28). Whether to utilise or not to utilise a particular scheme was considered as a decision variable of the multi-objective optimisation. However, 70 schemes could result in a large decision space due to a many number of possible combinations. Hence the 70 schemes were lumped into 25 schemes by considering the hydrologic connectivity of schemes (Table 28). Some lumped schemes consisted of both existing and proposed schemes whereas some lumped schemes consisted of only proposed schemes (Table 28). It was assumed that the existing schemes in a lumped scheme would be decommissioned if a lumped scheme was not utilised for sourcing water. It should be noted that this assumption could lead to less use of stormwater, and hence it is recommended that for future studies, the 25 lumped schemes be disaggregated by considering both hydrologic connectivity and the presence of existing schemes.

Table 28: Stormwater harvesting schemes considered (70 schemes lumped to 25 schemes)

No [1]	No [2]	Lumped Scheme ID	Lumped scheme name	W&G[3] scheme name	W&G catchment	W&G Potential yield (ML/year)	Operational in 2013?
1	1	SWAC1Schm	Adams creek	Olive Grove	Adams Creek	303	no
	2	SWAC1Schm	Adams creek	Edinburgh Parks North	Adams Creek	630	yes
	3	SWAC1Schm	Adams creek	Edinburgh Parks South	Adams Creek	760	yes
	4	SWAC1Schm	Adams creek	Kaurna Park	Adams Creek	551	yes
	5	SWAC1Schm	Adams creek	Springbank Park	Adams Creek	398	yes
	6	SWAC1Schm	Adams creek	Burton West	Adams Creek	308	yes
	7	SWAC1Schm	Adams creek	Summer Road	Adams Creek	575	no
	8	SWAC1Schm	Adams creek	Cheetham saltworks	Dry creek	783	no
2	9	SWBI53Schm	Barker Inlet	Pooraka Upgrade	Dry Creek	1360	yes
	10	SWBI53Schm	Barker Inlet	Islington Railyards	Barker Inlet	2052	no
	11	SWBI53Schm	Barker Inlet	North Arm East	Barker Inlet	1240	no
	12	SWBI53Schm	Barker Inlet	Hindmarsh Enfield Prospect	Barker Inlet	790	no
3	13	SWBC42Schm	Brown Hill creek #1	South Parklands (Peacock)	Brownhill/Kewswick Creek	83	no
4	14	SWBC64Schm	Brown Hill creek #2	Orphange	Brownhill/Kewswick Creek	210	no
5	15	SWBC65Schm	Brown Hill creek #3	Urrbrae	Brownhill/Kewswick Creek	140	no
6	16	SWBC76Schm	Brown Hill creek #4	Glenelg Golf Course	Brownhill/Kewswick Creek	460	yes
	17	SWBC76Schm	Brown Hill creek #4	Browhill Creek Airport	Brownhill/Kewswick Creek	3130	no
	18	SWBC76Schm	Brown Hill creek #4	Adelaide Airport	Sturt River	1078	no
7	19	SWCC8Schm	Cristie Creek	Madeira	Christie Creek	153	yes
	20	SWCC8Schm	Cristie Creek	Brodie Road	Christie Creek	655	yes
	21	SWCC8Schm	Cristie Creek	Morrow Road	Christie Creek	509	yes
8	22	SWDC2Schm	Dry Creek	Wynn Vale Dam	Dry Creek	346	yes
	23	SWDC2Schm	Dry Creek	Montaque Road	Dry Creek	549	yes

No [1]	No [2]	Lumped Scheme ID	Lumped scheme name	W&G[3] scheme name	W&G catchment	W&G Potential yield (ML/year)	Operational in 2013?
	24	SWDC2Schm	Dry Creek	Paddocks	Dry Creek	584	yes
	25	SWDC2Schm	Dry Creek	Parafield	Dry Creek	862	yes
	26	SWDC2Schm	Dry Creek	Bennet Road Drain	Dry Creek	480	yes
	27	SWDC2Schm	Dry Creek	Greenfields 1&2	Dry Creek	3269	yes
9	28	SWFR6Schm	Field River	Happy Valley Reservoir	Field River	890	no
	29	SWFR6Schm	Field River	Reynella East	Field River	351	yes
	30	SWFR6Schm	Field River	Young St	Field River	430	no
	31	SWFR6Schm	Field River	Elizabeth Cresent	Field River	945	no
10	32	SWGR22Schm	Gawler River	Gawler River	Gawler River	4740	no
	33	SWGR22Schm	Gawler River	Gawler Racecourse	Gawler River	306	no
11	34	SWGA57Schm	Grange area	Royal Adelaide Golf Course	Port Road	200	yes
	35	SWGA57Schm	Grange area	Grange Lakes	Grange Area	350	no
	36	SWGA57Schm	Grange area	pump from torreens to reserves	Grange area	900	no
12	37	SWGR1Schm	Greater Edinburag	Dawson Rd retarding basin	Gawler River	118	no
	38	SWGR1Schm	Greater Edinburag	Buckland Park	Gawler River	856	no
	39	SWGR1Schm	Greater Edinburag	Greater Edinburgh	Greater Edinburgh	1990	yes
13	40	SWLP1Schm	Little Para #1	Whites Road	Little Para	1045	no
	41	SWLP1Schm	Little Para #1	Bolivar	Little Para	330	no
14	42	SWLP25Schm	Little Para #2	Moss Road	Little Para	700	no
	43	SWLP25Schm	Little Para #2	Pioneer Park	Little Para	160	no
15	44	SWMC56Schm	Magazine Creek (Range Wetlands)	Cheltenham Racecourse	Magazine Creek	1180	no
	45	SWMC56Schm	Magazine Creek (Range Wetlands)	Range Wetlands	Magazine Creek	611	no
16	46	SWME45Schm	Mild End Drain	University Fields	River Torrens	2016	no
	47	SWME45Schm	Mild End Drain	Adelaide Shores	Mile End	850	no
17	48	SWOR78Schm	Onkaparinga River	Hackam South	Onkaparinga River	447	no
	49	SWOR78Schm	Onkaparinga River	Garland Reserve	Onkaparinga River	330	no
	50	SWOR78Schm	Onkaparinga River	Rural pumped flows	Onkaparinga River	1260	no

No [1]	No [2]	Lumped Scheme ID	Lumped scheme name	W&G[3] scheme name	W&G catchment	W&G Potential yield (ML/year)	Operational in 2013?
18	51	SWPC77Schm	Pedler Creek	Pedler Creek	Peddler Creek	756	no
	52	SWPC77Schm	Pedler Creek	reserve B	Peddler Creek	481	no
19	53	SWPR58Schm	Port Road #1	Port Road Median	Port Road	571	no
20	54	SWPR59Schm	Port Road #2	Riverside Golf Course	Port Road	450	no
	55	SWPR59Schm	Port Road #2	Grange Golf Course	Port Road	300	yes
21	56	SWRT60Schm	River Torrens #1	Botanic Gardens	River Torrens	170	no
	57	SWRT60Schm	River Torrens #1	Victoria Park	Brownhill/Kewswick Creek	211	no
22	58	SWRT73Schm	River Torrens #2	Bonython Park	River Torrens	4085	no
23	59	SWSC1Schm	Smith creek	Evanston South	Smiths Creek	185	yes
	60	SWSC1Schm	Smith creek	Blakeview	Smiths Creek	308	yes
	61	SWSC1Schm	Smith creek	Munno Para West	Smiths Creek	1241	yes
	62	SWSC1Schm	Smith creek	Andrews Farm	Smiths Creek	400	yes
	63	SWSC1Schm	Smith creek	Andrews Farm South	Smiths Creek	500	yes
	64	SWSC1Schm	Smith creek	NEXY retardin basin	Smiths Creek	854	yes
24	65	SWSR40Schm	Sturt creek #1	Science Park	Sturt River	770	no
25	66	SWSR44Schm	Sturt creek #2	Oaklands Park North	Sturt River	290	no
	67	SWSR44Schm	Sturt creek #2	Oaklands Park South	Sturt River	414	no
	68	SWSR44Schm	Sturt creek #2	Disused train from Brownhill	Sturt River	1511	no
	69	SWSR44Schm	Sturt creek #2	New Morphettvile Racecourse	Sturt River	1800	no
	70	SWSR44Schm	Sturt creek #2	Old Morphettville Racecourse	Sturt River	325	yes

Note 1: lumped scheme no.; 2: individual scheme no.; 3: Wallbridge and Gilbert (2009)

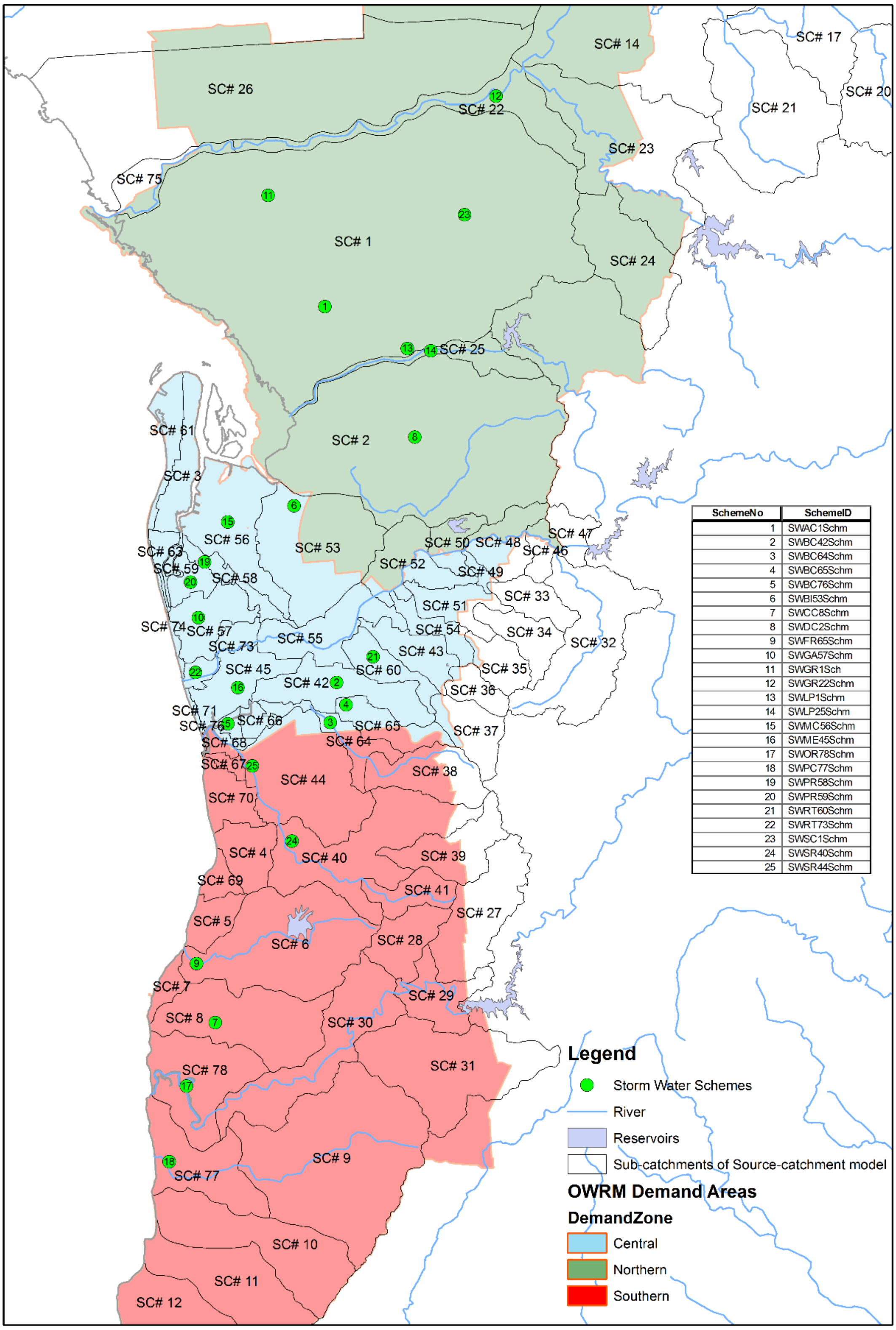


Figure 29: Locations of Lumped stormwater harvesting schemes (note: approximate locations only, i.e. scheme ID is shown at the outlet of the sub-catchment within which the scheme is located)

Table 29: Details of lumped stormwater harvesting schemes

No	Scheme ID	Scheme location[1]	Scheme name	Sub-catchment area (ha)	Areal coefficient	Scheme catchment area (ha)	Scheme efficiency	Ccatchments contributing to scheme inflow
1	SWAC1Schm	1	Adams creek	32934.75	0.203	6683.90	0.70	
2	SWBC42Schm	42	Brown Hill creek #1	2211.65	0.440	973.53	0.65	
3	SWBC64Schm	64	Brown Hill creek #2	1002.06	1.000	1002.06	0.65	
4	SWBC65Schm	65	Brown Hill creek #3	1163.36	0.719	836.00	0.65	
5	SWBC76Schm	76	Brown Hill creek #4	198.87	1.000	198.87	0.65	38, 64, 65, 42, 66
6	SWBI53Schm	53	Barker Inlet	4586.00	0.661	3031.00	0.74	
7	SWCC8Schm	8	Cristie Creek	3779.25	0.969	3662.00	0.43	
8	SWDC2Schm	2	Dry Creek	14222.25	0.950	13511.00	0.72	
9	SWFR6Schm	6	Field River	5529.25	0.919	5082.45	0.55	
10	SWGA57Schm	57	Grange area	1498.75	0.435	651.40	0.63	
11	SWGR1Schm	1	Greater Edinburag	32934.75	0.109	3580.00	0.53	
12	SWGR22Schm	22	Gawler River	2820.00	0.357	1005.90	0.55	
13	SWLP1Schm	1	Little Para #1	32934.75	0.010	320.00	0.61	
14	SWLP25Schm	25	Little Para #2	1230.32	1.000	1230.32	0.61	
15	SWMC56Schm	56	Magazine Creek (Range Wetlands)	2972.44	0.436	1297.00	0.74	
16	SWME45Schm	45	Mild End Drain	1576.75	0.681	1073.00	0.58	
17	SWOR78Schm	78	Onkaparinga River	3108.86	0.634	1972.00	0.28	
18	SWPC77Schm	77	Pedler Creek	2593.56	0.054	140.10	0.25	
19	SWPR58Schm	58	Port Road #1	665.56	1.000	665.56	0.56	
20	SWPR59Schm	59	Port Road #2	593.25	0.767	455.10	0.56	
21	SWRT60Schm	60	River Torrens #1	1008.75	1.000	1008.75	0.50	
22	SWRT73Schm	73	River Torrens #2	741.62	1.000	741.62	0.50	32, 33, 34, 35, 36, 43, 54, 51, 46, 47, 48, 49, 50, 52, 37, 60, 55
23	SWSC1Schm	1	Smith creek	32934.75	0.244	8022.00	0.69	
24	SWSR40Schm	40	Sturt creek #1	4586.25	1.000	4586.25	0.64	39, 41
25	SWSR44Schm	44	Sturt creek #2	4309.00	1.000	4309.00	0.64	

Note 1: in terms of the sub-catchment ID

Stormwater inflow to each lumped scheme was computed through hydrologic modelling. The Source-catchment module (see Appendix 2, for details) was used for hydrologic modelling, which considered runoff generation and transportation processes from 78 sub-catchments (Figure 29). The approximate location of the 25 lumped stormwater harvesting schemes (in terms of the sub-catchment within which a lumped scheme is located) are also shown in Figure 29.

The catchments of lumped schemes did not fully encompass any particular sub-catchment. Hence to compute the inflow to a lumped scheme, portion of the sub-catchment area contributing to the scheme, called 'areal coefficient', was defined. The areal coefficient of each lumped scheme is given in Table 29. These were estimated by overlaying the catchments of lumped schemes on a GIS map of the sub-catchments.

The amount of stormwater captured by a lumped scheme from its catchment generally dependent on runoff generation characteristics of its catchment during winter months, and the capacities of diversion channel/pipe, holding pond and wetland, as well as the capacity of pump used to inject stormwater into the aquifer. Since stormwater schemes were generally small-scale systems, they did not generally have the capacity to store an inflow occurring over a month, which was the temporal scale of Source-schematic model. Also, injection of stormwater into the aquifer generally occurred continuously over winter months. Hence to determine the amount of stormwater that could be injected by a lumped scheme on a monthly basis required up-scaling of injection volumes computed through an appropriate time-step, which might be either sub-daily or daily, depending on the capacity of a lumped scheme to divert, hold, treat and inject stormwater. Since each scheme had differing capacities of holding ponds, wetlands, pumps and wells, up-scaling had to be carried out either for each scheme individually or for a generic scheme with probability distributions for input variables to represent the variability across all the schemes.

However, given the limitations in time and resources, up-scaling method was not followed. Instead, the catchment scale potential yield expressed as percent of median catchment runoff, given in Wallbridge and Gilbert (2009), was used to provide guidance on the proportion of stormwater (generated from scheme's catchment) available to inject. We called this variable 'scheme efficiency' because it represented individual scheme's ability capture, store, treat and inject stormwater. For example, for the lumped scheme located in Adams Creek, the scheme efficiency derived was 0.7, which was based on Wallbridge and Gilbert (2009)'s estimate for the potential annual yield for Adams Creek, i.e. 70%. The scheme efficiency computed in this manner for each lumped scheme is given in Table 29.

Thus the monthly injection volume of a lumped scheme was computed as follows:

$$I_T^i = \alpha^i \beta^i \emptyset_T$$

Equation 32

Where, I_T^i is the inflow of lumped scheme i in month T (ML/month), α^i is the areal coefficient of lumped scheme i, β^i is the scheme efficiency of lumped scheme i and $\emptyset_T$ is the inflow of the sub-catchment where the lumped scheme i is located (ML/month), computed from Source-catchment Module (computed on a daily basis and aggregated to monthly), described in Appendix 2.

Twenty percent of the injected stormwater was considered to be lost through deep percolation. The amount of stormwater available to recover in time-step T for lump scheme i, S_T^i , is given by:

$$S_T^i = S_{T-1}^i + I_T^i - 0.2I_T^i$$

Equation 33

Where S_T^i (ML) is the amount of stormwater available to recover in time-step T for lump scheme i, S_{T-1}^i (ML) is the amount of stormwater available at the time step T-1 and I_T^i (ML) is the volume injected at the time T. The injected stormwater accumulated in the aquifer over the winter months (i.e. May to September). The accumulated stormwater was recovered during summer months (i.e. November to April), for non-potable uses. The amount of recovery was based on the demand for stormwater. Any unused stormwater in the aquifer at the end of summer months remained in the aquifer and mixed with the stormwater injected into the aquifer in the next winter months. This could result in an accumulation of stormwater in the aquifer and might lead to undesirable groundwater impacts due to building up of water pressure. To avoid such undesirable impacts, a limit had been defined to the seasonal injection volume, by considering the demand for non-potable use in the previous season, i.e. maximum allowable seasonal injection volume was considered as the demand for non-potable use in the previous summer season.

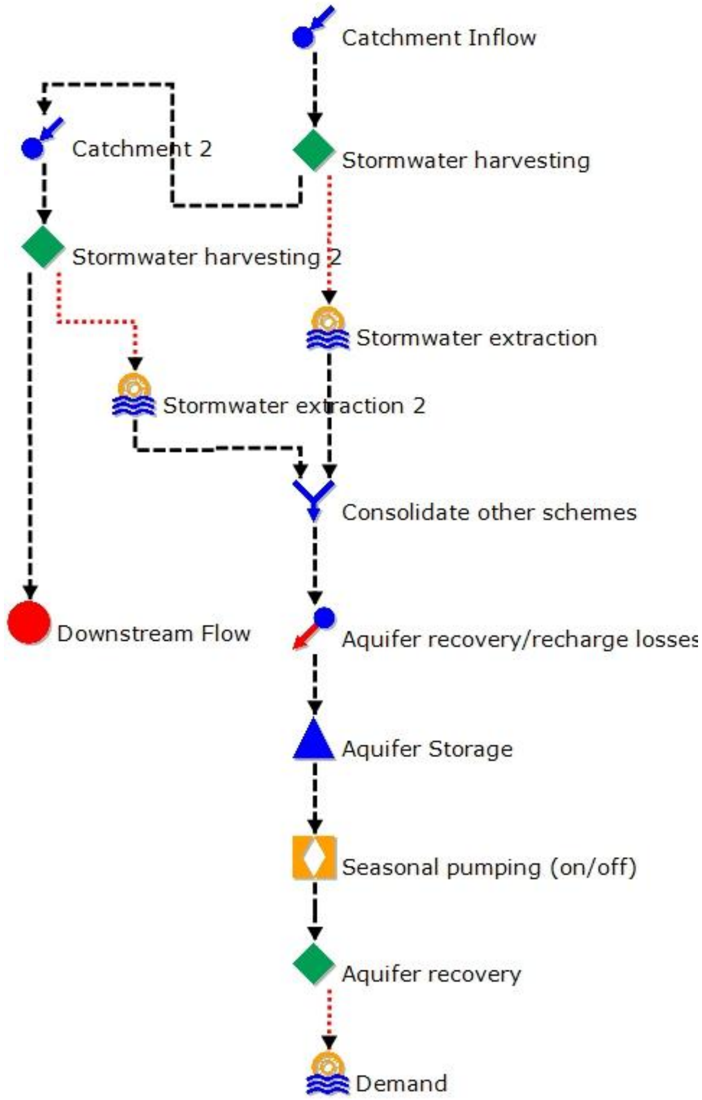
The amount of stormwater not captured by a scheme (i.e. $\left(1 - \alpha^i \beta^i\right) \emptyset_T\right)$, flows to the next downstream sub- catchment, where it might be captured by another scheme or simply flows through all the downstream sub-catchments and discharges to the sea.

In the simulation model, an ASR scheme was represented as follows:

- an inflow node represented inflow from the sub-catchment within which the scheme was located
- an extraction node connected to the inflow node and a demand node connected to the extraction node represented the portion of the infow captured and injected into the aquifer (i.e. $\alpha^i \beta^i \emptyset_T$)
- a gauge node represented the portion of inflow not captured, if there was another scheme not located downstream of the scheme
- a return flow link from the demand node represented the flow path of injected stormwater into the aquifer
- a storage node represented the aquifer storage
- a link downstream of the storage node represented recovered stormwater from the aquifer, which then connected to a splitter node to supply to non-potable portion of residential and non-residential demands.

For simplicity of node-link structure of the Source-schematic model, all the aquifer storages located in a particular demand zone were lumped, which resulted in three aquifer storages, each located in north, central and south demand zones. There were called: north aquifer storage, central aquifer storage and south aquifer storage. It should be noted that lumping of aquifer storages of each lumped scheme had no impact on the amount of stormwater available for recovery.

Representation of two ASR schemes in series, in the Source-schematic model is shown in Figure 30. When two ASR schemes in series, the uncaptured runoff from the ASR scheme located upstream, flows into the ASR scheme located downstream. Extractions from both schemes are considered to be injected into one of the three aquifers mentioned above. Any uncaptured runoff of the scheme located downstream flows into a gauge node. Figure 30 (right) shows two ASR schemes in parallel. Uncaptured runoff of each scheme flows into a gauge node. The flows at each gauge are aggregated to compute the total uncaptured flows by ASR schemes. The aggregated flows at each gauge are added to the hydrologically routed runoff (by the Source-catchment model) at discharge locations.



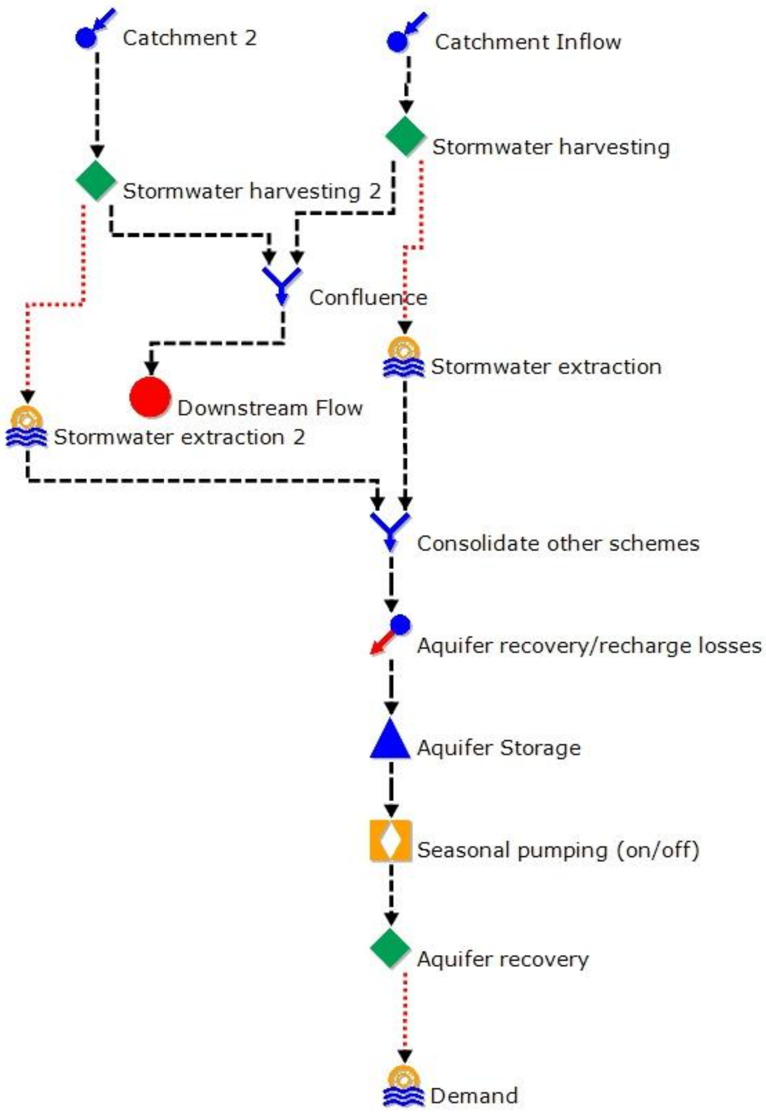


Figure 30: Two schematic diagrams, illustrating the modelling method of two harvesting schemes in series (top diagram) and in parallel (bottom diagram), in the Source model

Modelling urban runoff and stormwater discharge to the Gulf

The catchment model developed as part of the Adelaide Coastal Water Quality Improvement Plan (WBM BMT, 2008), was chosen to compute stormwater runoff from urban areas. This catchment model was developed in the E2 modelling platform, which was the same platform used in the Source model. However, the following limitations were identified with regard to the catchment model of WBM BMT (2008):

- sub-catchment boundary delineation was wrong in some areas, in terms of hydrologic connectivity
- node-link networks were not correct in the Sturt River and Brown Hill Creek catchments;
- hydrology was calibrated using a single gauging station with factors applied for other regions which led to large areas with a poor hydrological calibration
- land use data needed to be updated to 2012 to better reflect the base scenario and current hydrological conditions

- given the simulation model was setup to for a period of 50 years, climate data needed to be extended to 50 years.

Table 30: Calibration and validation periods and selected optimisation functions and respective daily Nash and Sutcliff (1970) efficiency (NSE) and total volume differences for calibration and validation periods

Gauge	Location	Optimization function	Calibration			Validation		
			Daily NSE	Total volume difference (%)	Period	Daily NSE	Total volume difference (%)	Period
A5030547	Christie Creek	NSE daily and bias penalty	0.637	3.60%	30/11/2000-31/12/2007	0.68	-4%	1/1/2008-31/12/2012
A5030503	Onka River	NSE daily and bias penalty	0.565	5.10%	13/04/1967-23/02/1989	0.67	1.20%	7/01/2000–1/02/2003
A5040529	River Torrens	NSE daily and flow duration	0.911	-1.10%	1/01/1980–31/12/1999	0.88	-3.70%	1/01/2000–31/12/2012
A5040576	Sturt River	NSE daily and flow duration	0.651	3.50%	2/09/1994-31/12/2003	0.610	8.60%	1/01/2004-1/06/2009
A5040583	Brown Hill Creek	NSE daily and bias penalty	0.740	-5.10%	1/01/1994-31/12/2005	0.72	7.60%	1/1/2006-31/12/2012
A5050505	Gawler River	NSE daily	0.789	8.30%	1/01/1970 - 31/12/1994	0.16	68.60%	1/1/1996-31/12/2003
A5030543	Pedler Creek	NSE daily and bias penalty	0.230	7.30%	4/07/2000 - 06/03/2013	Not enough data		

Given the above limitations, a rebuild of the catchment model of WBM BMT (2008) was considered necessary. Details of the rebuild are given in Appendix 1. A brief summary of the rebuild is given below.

The catchment model of WBM BMT (2008) consisted of 57 sub-catchments. The new catchment model delineated the study area into 78 sub-catchments (see Figure 7). The model was then updated with 2013 land uses described in section 3.5 and the gridded daily rainfall and potential evapotranspiration surfaces obtained from the Bureau of Meteorology.

The catchment model was calibrated and validated for seven gauges as shown in Figure 31 and Table 30. The gauges were selected based on their spatial distribution and the availability of observed data that the calibration was based on. The calibration for each catchment was done using Calibration Wizard available in the Source model, which provided a series of objective functions and search algorithms for model calibration. The wizard also allowed for the use of multiple gauges with different weights placed on the importance of different gauges for the overall calibration. Instead of choosing one objective function for the model calibration, four different objective functions were used and the final choice of parameters was based on the daily NSE (Nash- Sutcliffe Efficiency) (Nash and Sutcliffe, 1970) and total volumes for each parameter set obtained by considering NSE (daily), NSE (daily) and bias penalty, NSE (daily) and flow duration (NSE weight = 0.7) and NSE (daily) and log flow duration (NSE weight = 0.7). The record length for the different gauges used in the calibration varied from catchment to catchment, and therefore the calibration and validation periods vary for different gauges. In all cases, a 1-year warming period before calibration was adopted. The calibration and validation periods are given in Table 30.

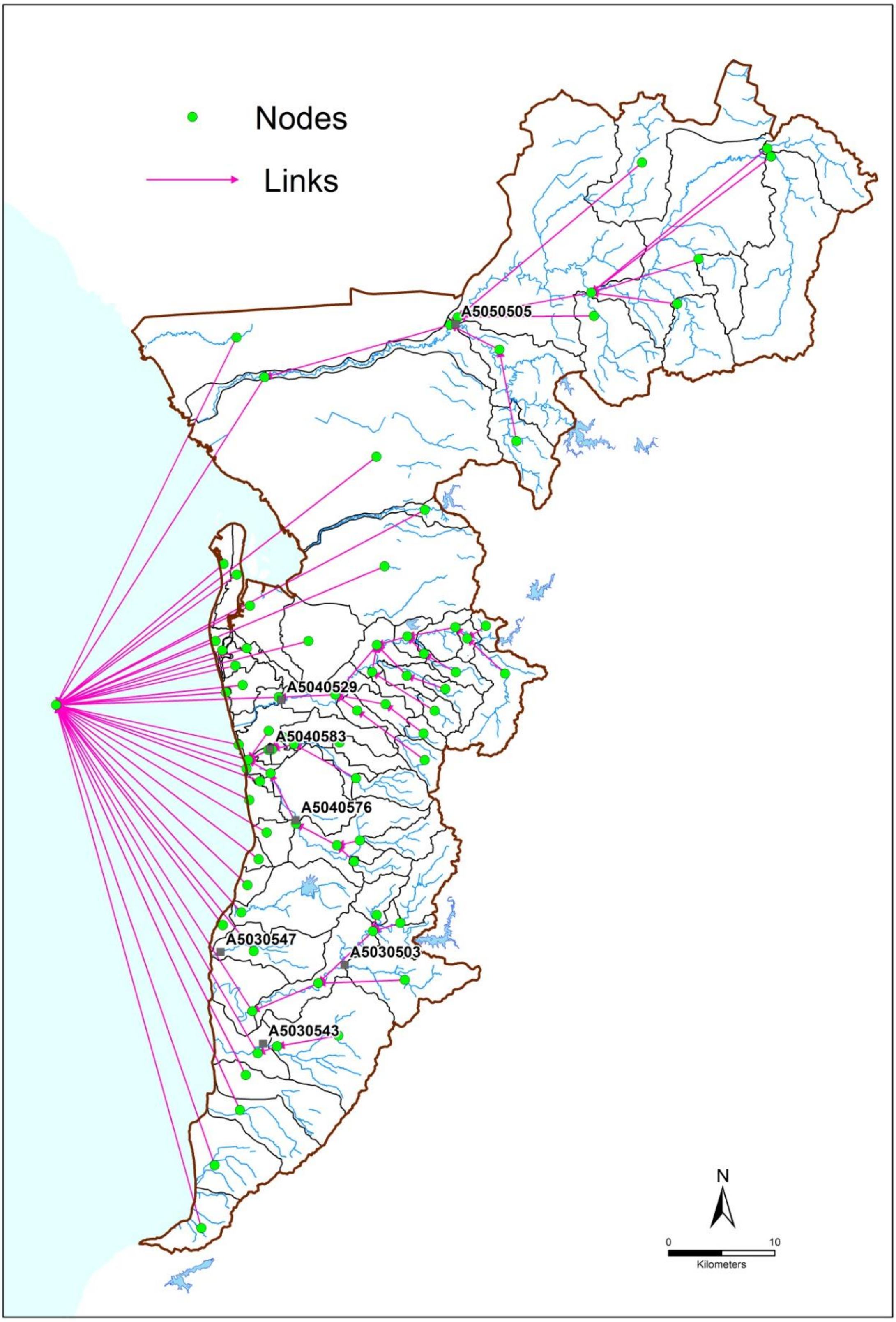


Figure 31: Gauging stations used for calibration and the node-link network of the catchment model

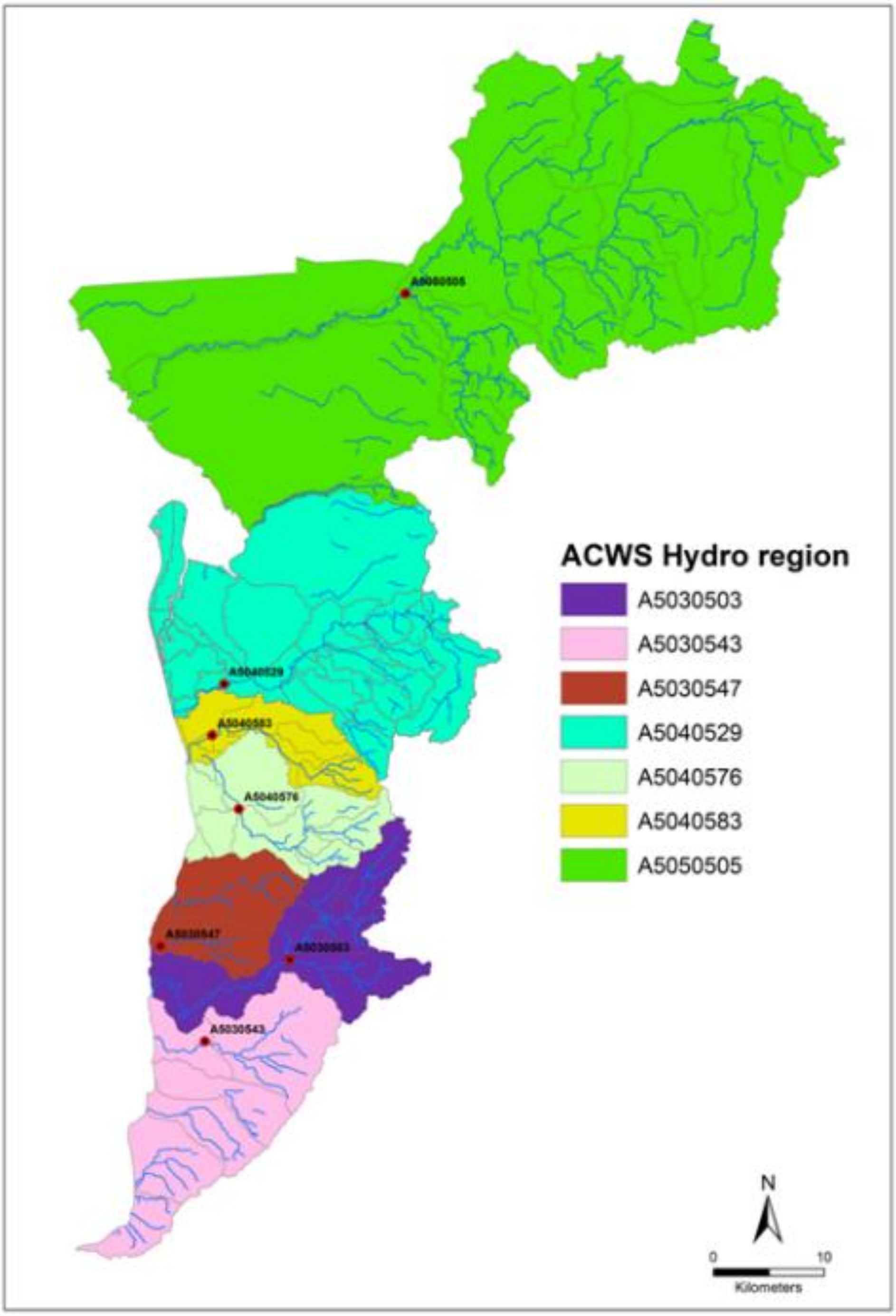


Figure 32: Hydrological parameterisation

The hydrological model was carefully constructed to contain sub-catchment outlets at gauge stations used for model calibration. The model parameters for any sub-catchments located upstream of a gauge were obtained through calibration. Hydrological parameterisation of the remainder of the model involved the adoption of parameter sets from nearby calibrated catchments having simular land use and soil types. Accordingly, the hydrological model included 7 hydrological regions as shown in Figure 32.

The calibrated catchment model was then executed for 50 years from July 1963 to June 2013 on a daily basis to obtain daily runoff for all the 78 sub-catchments. These were then fed to the Source-schematic model as input data to compute inflows for stormwater harvesting schemes.

Estimating constituent load discharging to the Gulf

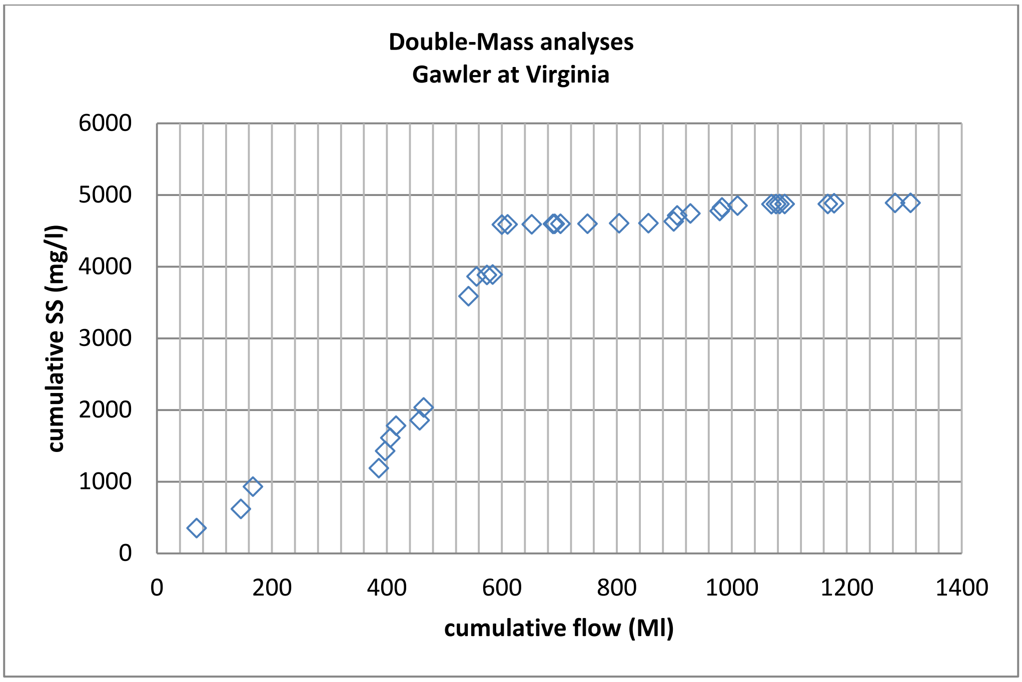
Quantifying constituent loads discharging to the Gulf St Vincent was not part of the current study. However, an attempt was taken to estimate the annual N (nitrogen), P (phosphorous) and SS (suspended solids) loads discharging to the Gulf because these were the key water quality parameters considered in the coastal water quality improvement plan of Metropolitan Adelaide (McDowell and Pfennig, 2013). The aim of this work was to provide a better interpretation to the multi-objective optimisation related objective aimed at minimising wastewater and stormwater discharges to the Gulf St Vincent, in terms of constituent loads. Due to the limited time and funding availability, however, the focus of this work was on estimating N, P and SS associated with stormwater discharges to the Gulf only.

Table 31: Sites used to examine relationship between stormwater flows and TP, TN and TSS

Station Number	Site Name	River	Location	Catchment area (km^2)	Flow data used
A5050510	Virginia	Gawler	34:38:22.6 S, 138:32:27.6 E	1170	1972–2013
A5041014	Seaview road Bridge	Torrens	34:56:05.8 S, 138:29:58.9 E		2010–2013
A5031010	South Road (u/s)	Field River	35:05:16.4S, 138:29:43.1 E	26.16	2000–2009
A5030547	Galloway road (d/s)	Christies Creek	35:07:33.3S, 138:28:50.1E	35.9	2000–2013
A5041009	Barker wetland outlet	Port River	34:49:45.8S, 138:34:14.9 E	N/A	2004–2013

The specific aim was to provide a relationship (or relationships) between the constituents mentioned above and the stormwater flow and, to use these relationships to estimate the amount of constituents discharging to the Gulf. It was not expected that these relationships be used as part of the optimisation, rather the expectation was, if required, these relationships be used in the optimal solutions being identified through multi-objective optimisation, to obtain an indication on the amount of N, P and SS loads discharging to the Gulf. However, since the focus was only on stormwater, these relationships would be of limited use for interpreting total N, P and SS loads to the Gulf because discharges included both stormwater and wastewater. Hence we will not use these relationships in the results discussed in Chapter 5. Nevertheless, the methodology followed and the relationships derived are described below, to assist with any future studies in this regard.

The method involved selecting sites with sufficient data on stormwater flow and the constituents mentioned above, processing and cleaning the data as required, double-mass analysis to investigate homogeneity of the constituent data in a full range of flow regime expected at the selected sites, flow duration analysis to understand probability of occurrence of specific flow values, as well as constituent values, and develop relationships to quantify loading of SS, N and P in the stormwater for the selected sites.



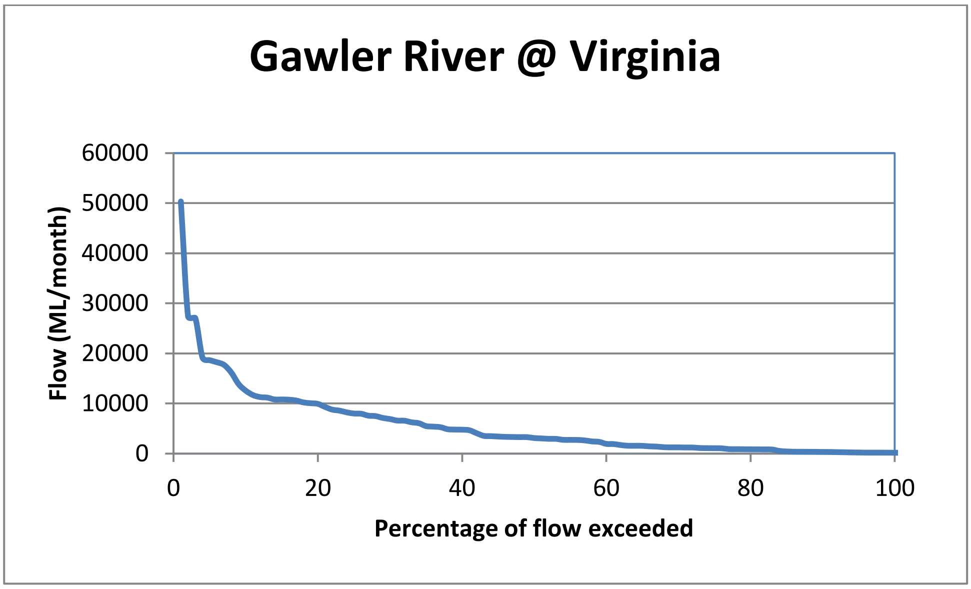
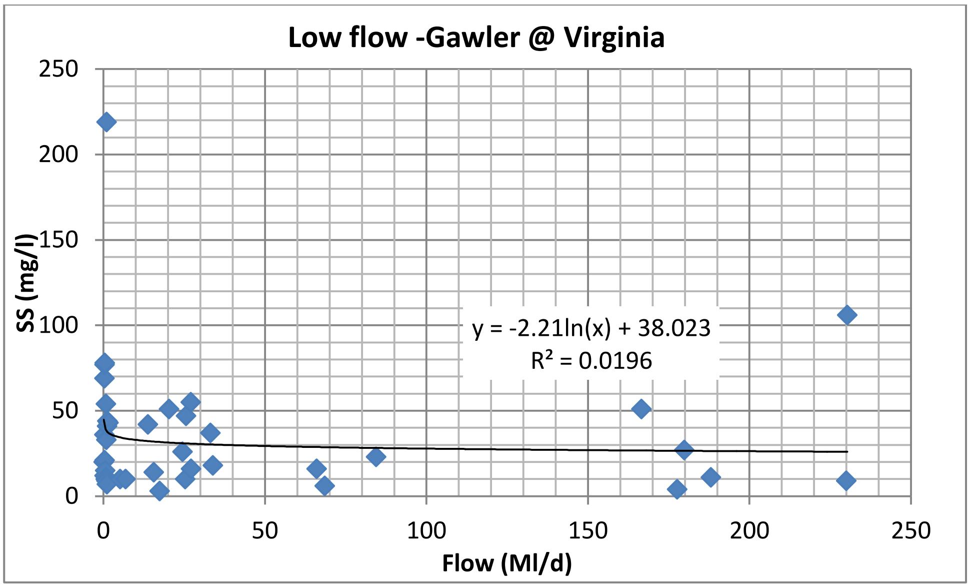


Figure 33: Double-mass analysis of suspended solids (SS) versus mean flow (top chart) and monthly flow duration curve (bottom chart) of Gawler River data at Virginia

Five sites were identified through consultation with EPA South Australia and SA Water Corporation (Table 31). The recorded data for each constituent represented average concentration since the last sampling date. Hence, the average flow between sampling dates were computed. Time interval between water quality sampling dates varied from one week to few months. Hence, the appropriate time period within which mean flow was estimated, was decided subjectively, by considering the magnitude and the sequence of the flow data within the two sampling dates. The derived average flow data and the measured water data of N, P and SS were then analysed by using double-mass and flow duration methods to identify meaningful relationships. For an example, the analysis conducted for Gawler River at Virginia for SS is described below. The same analysis was followed for N and P for Gawler River at Virginia, as well as for other sites shown in Table 31.



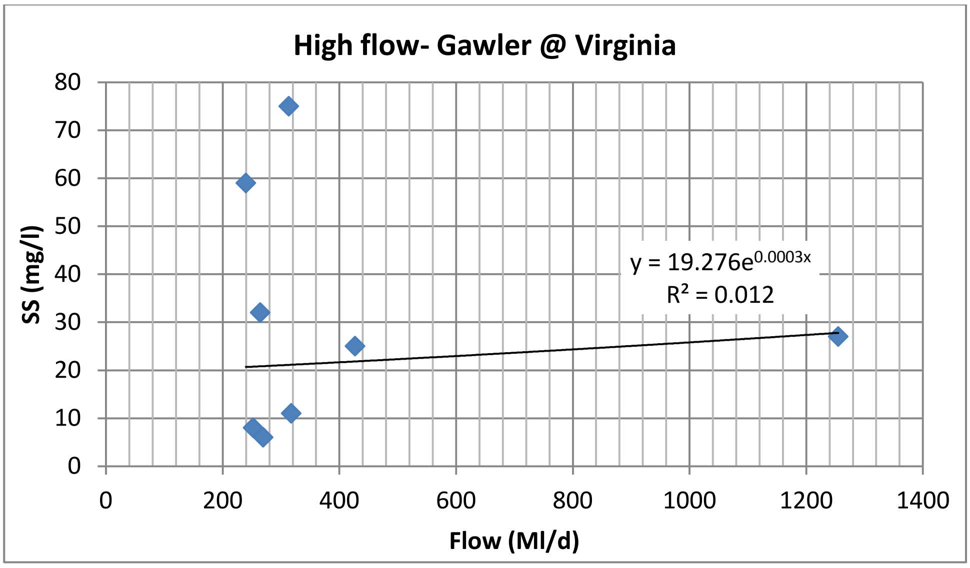


Figure 34: Flow duration analysis SS versus flow for low flow (top chart) and high flow (bottom chart) regimes of Gawler River data at Virginia

The double-mass analysis (Figure 33) for SS and mean flow at Gawler River at Virginia indicated two possible trends, one for low flows and another for high flows. These trends were examined in detail (Figure 34). However, no clear relationship between TSS and flow was evident for both high flow and low flow regimes. Consequently, it was decided to express mean values of SS for different flow bands (or regimes). The flow bands were identified by using flow duration analysis. Since the optimisation was supported by monthly simulation of flows, monthly flow duration analysis was performed (Figure 33). It was evident from the monthly flow duration curve (Figure 33) that the flow could be divided into two groups: 0–880 ML/month occurring at least 80% of the time (i.e. the percentage of time exceeded was greater than 20%) and a flow greater than 880 ML/month occurring at least 20% of the time (i.e. the percentage of time exceeded was less than 20%). Hence the monthly flow corresponding to 20% time exceeded was chosen as a threshold to develop a relationship between SS and the flow. Following the same process for N and P, as well for N, P and SS for other stations, the relationship shown in Table 32 was developed.

Table 32: Relationships developed for estimating SS, P and N loads discharging to the Gulf at selected locations, based on the monthly flow

Station	% of time flow exceeded in monthly FDC	Mean flow (ML/month)	TSS (mg/l)	TN (mg/l)	TP (mg/l)
Gawler River	< 20%	>880	36.4	1.72	0.19
	>20%	<880	30	2.83	0.18
Torrens River	<20%	>4650	76	1.49	0.1
	>20%	<4650	25	1.2	0.07
Field River	<20%	>385	25	1.26	0.07
	>20%	<385	17	1.45	0.07
Christies Creek	< 20%	>300	142	1.81	0.19
	20–40%	300–170	105	1.29	0.10
	>40%	<170	81	1.57	0.10
Barker inlet	<20%	>180	55	1.0	0.16
	>20%	<180	30	0.97	0.13

For example, if the flow in the Gawler River was 700 ML in a particular month, the estimated amount of TSS discharging to the Gulf, corresponding to that month would be = 36.4 Kg. It should be noted that this method requires a calibration, for which better quality data on water quality parameters are essential. At present such data do not exist. Hence the above method should be used cautiously, noting that the values given in Table 32 provide indicative estimates only.

4.6.7 Modelling supply and discharge implications of rainwater tanks

In South Australia, the current policy on rainwater tanks recommends the use of rainwater tanks for residential use, in particular for toilets, cold water taps in laundry and all outdoor uses. Accordingly, the study assumed that rainwater tanks were used only for residential use and the tanks were internally plumbed to provide water to non-potable uses (i.e. toilets, cold water taps in laundry and all outdoor uses), as well as for hot water use. Further, it was assumed that the households with an internally plumbed rainwater tanks used a front loading washing machine, which used cold water and heated it up internally, i.e. hot-water heater did not supply water to the washing machine and the laundry use mostly consisted of the water usage by the washing machine. These assumptions allowed rainwater tanks to supply 100% of non-potable residential demand and 40% of potable residential demand (considering 60 to 40 split between cold water and hot water, George Wilkenfield and Associates (2003)). When tanks run-out of water, these residential end uses were considered to be supplied from fit-for-purpose sources, which might include stormwater, recycled water, groundwater, the ADP, the RM and the MLR catchments.

The presence of rainwater tanks, in particular internally plumbed rainwater tanks, reduces the demand for mains water, which can result in an increase in supply reliability at the system scale. Similarly, the presence of rainwater tanks reduces the amount of runoff discharging to receiving waters by capturing rainwater that would otherwise discharges to the drainage system as stormwater.

The supply from rainwater tanks depend on such factors as prevailing climate, tank volume, area of the roof connected to the tank and household water use (Fewkes and Butler, 2000; Coombes and Barry, 2007; Mitchell, 2007; Basinger et al., 2010; Khastagir and Jayasuriya, 2010; Palla et al., 2011 and Maheepala et al., 2011). Through field measurement of physical characteristics of rainwater tanks in south-east Queensland, Biermann et al. (2012) have showed that the tanks sizes and connected roof areas vary spatially despite the fact that there is a recommended tank size of 5 kL and a minimum roof area connected to the tank of 100 m^2 (Queensland Development Code Mandatory Part (MP) 4.2, 2008). Using Biermann et al. (2012) data and the observed household water use data of Beal and Stewart (2011), Maheepala et al. (2013) have shown that supply from rainwater tanks in south east Queensland can vary between 6 and 121 kL/household/year.

Given residential water use generally varies from household to household and there is a minimum size of 1 kL recommended by the SA Government for rainwater tanks, it is expected that supply from rainwater tanks in Adelaide varies spatially. Ignoring the spatial variability generally leads to an overestimation of tank supply and an underestimation of tank overflows, which will result in an overestimation of supply reliability at the system scale and an underestimation of runoff from urban catchments with rainwater tanks. The amount of overestimation in tank supply per household, due to ignoring the spatial variability of tank supplies is, in the order of 22% for Gold Coast and 16% in Brisbane (Maheepala et al., 2012), 14% for Melbourne (Mitchell et al., 2008; Xu et al., 2010) and 18% for Canberra (Maheepala et al., 2011).

To put this into Adelaide context, assume an actual tank yield of 40 kL/household/year. If 20% overestimation is assumed, the estimated tank yield will be 48 kL/household/year. In 2013, Adelaide's population is 514,644. The current adoption rate is 44.4%, i.e. 228,502 households currently have rainwater tanks. Hence the overestimated amount is: 228,502 households X 8 kL/household/year= 1,828,016 kL/year or 1.83 GL/year. SA Water's currently supply about 140 GL/year for Metropolitan Adelaide. The amount of desalinated water use in 2011/12 in Adelaide was 4.229 GL (National performance report 2011–12: urban water utilities, March 2013). Thus, the overestimation of tank yield resulting from ignoring the spatial variability of factors affecting rainwater tank yield is equivalent to about 43% of the desalinated water use in Adelaide 2011/12 and 1.3% of the current average annual supply. Hence it can be said that ignoring the spatial variability of tank supplies may not be significant from a point of view of the total supply to Metropolitan Adelaide, but it is certainly significant from a point of view of the use of alternative sources such as desalinated water and stormwater (current usage is about 5 GL/year).

Stochastic simulation of tank storage behaviour is the recommended method for quantifying both yield and runoff implications of rainwater tanks spread across a large urban area such as a city or a town (Mitchell et al., 2008; Xu et al., 2010; Maheepala et al., 2011 and 2013). Accordingly, we used this method for this study, and used the tank model described in

Mitchell et al. (2008) for the stochastic simulation of tank storage. A schematic diagram of this model is shown in Figure 35.

The Mitchell et al. (2008) rainwater tank (RWT) model was a water balance model, capable of simulating the processes involved in translating rainfall into roof runoff, and the tank storage by considering the demand drawn from the tank. It consisted of two modules: a rainfall-runoff module, which computed the amount of roof runoff into the tank, and a storage module, which computed the amount of water stored in the tank, using a 'yield-after-spill' operating rule. The yield-after-spill operating rule allowed the tank to supply water to satisfy the demand placed on the tank, after allowing water to spill from the tank if the inflow to the tank in a particular simulation time-step was greater than amount of water that could be held in the tank in that particular time-step. The yield-after-spill rule provided an accurate estimate for yield calculation compared to the approach that allowed supply from the tank to occur before the spillage, i.e. yield-before-spill rule (Fewkes and Butler, 2000; Mitchell, 2007).

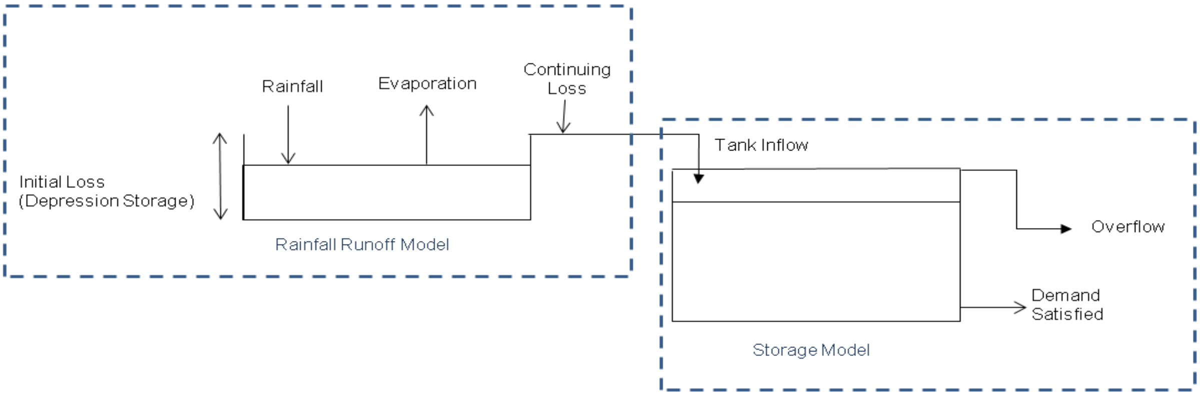
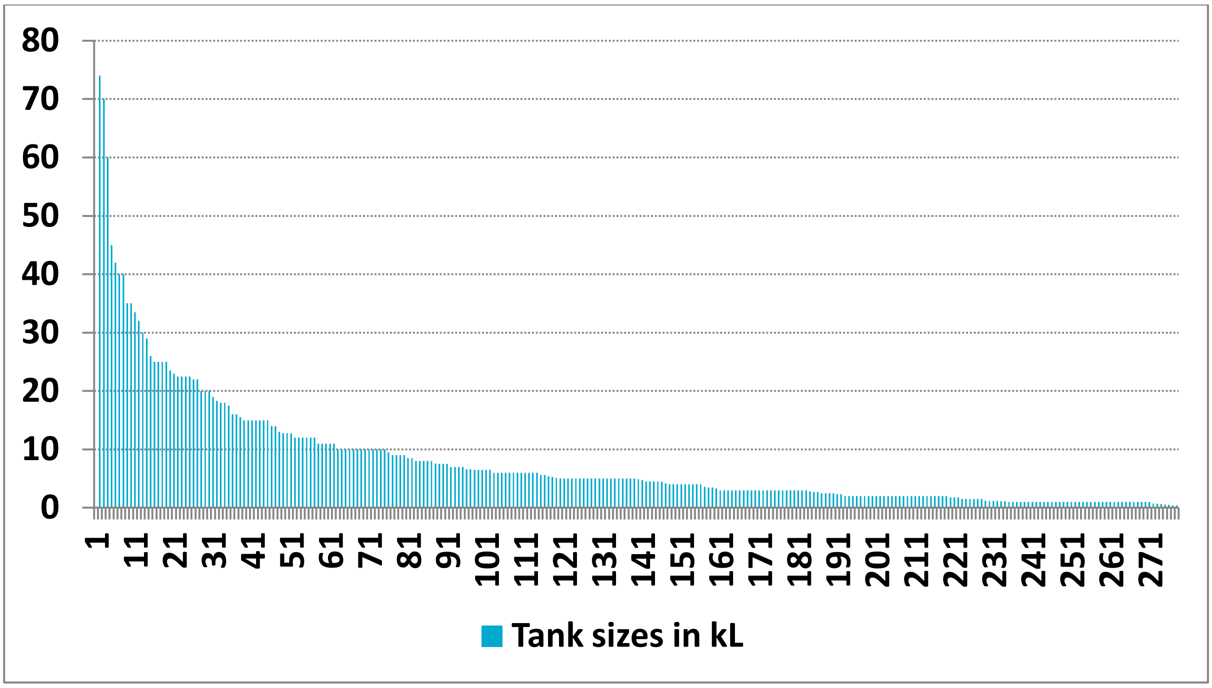


Figure 35: Schematic representation of the rainwater tank model (source: Neumann 2011, adapted from Mitchell *et al.*, 2008)

The input data required for the rainwater tank model included rainfall and potential evaporation over the simulation period, connected roof areas, initial and continuing losses from roofs, tank sizes and the household demands at the end use scale. The connected roof areas, initial and continuing losses from roofs and tank sizes could be specified either as an average value or as a probability distribution function, with a minimum and a maximum value. The model allowed the use of only normal and log-normal distributions for the input variables. The demand was to be specified as a time series containing either an average value or a set of probable values, to account for the temporal variability of the water use over the time period of simulation.



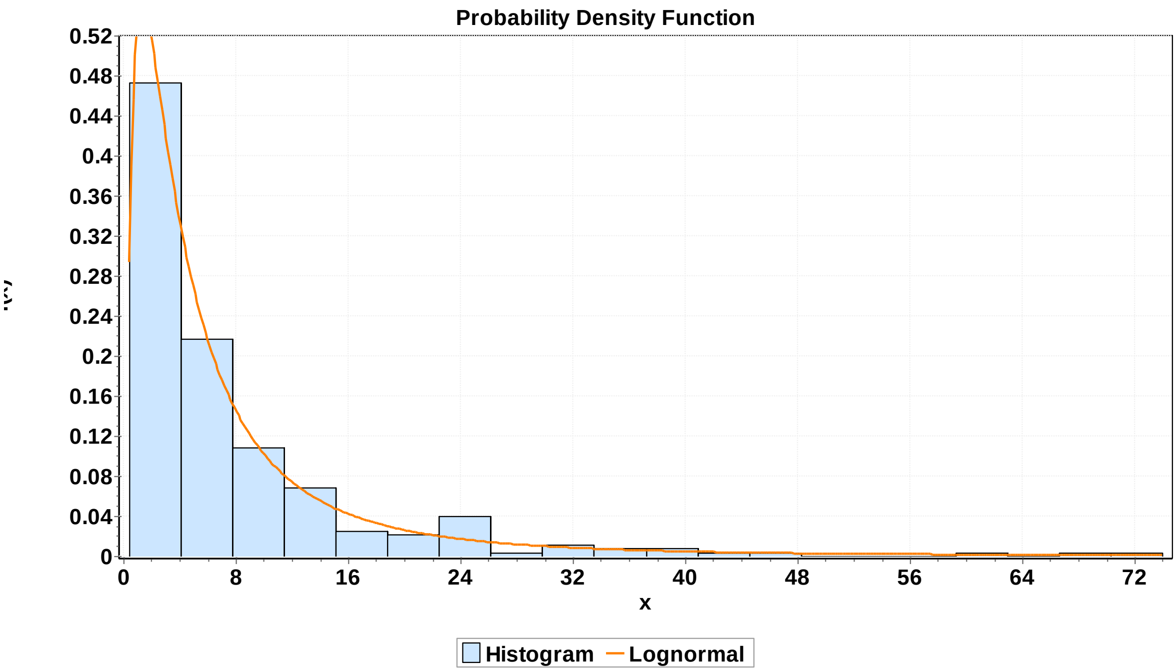


Figure 36: Observed rainwater tank sizes from 277 households in Metropolitan Adelaide and fitting of the observed tank sizes to log-normal distribution (Kolmogorov-Smirnov test statistic = 0.077, accepted at 5% significant level; p-value = 0.0816)

The following data were used to derive input data for the stochastic rainwater tank simulation (summarised in **Error! Reference source not found.**):

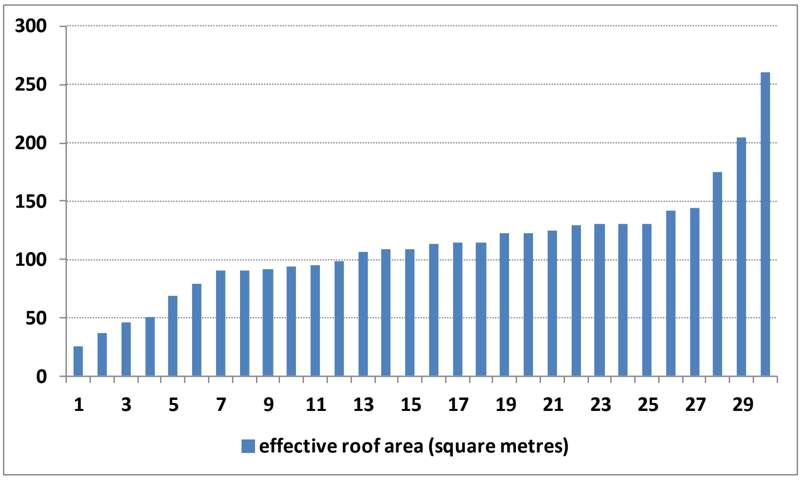
- Daily rainfall and evaporation data of North Adelaide (BOM station 023011).
- The tank sizes collated as part of a survey conducted by Task 4 of the OWRM project (Figure 36). This dataset contained tank sizes from 277 households. The sizes varied from 0.4 kL to 74 kL, with a mean of 7.98 kL, a median of 4.9 kL and a standard deviation of 10.32 kL. The data were fitted to both normal and log-normal distributions. The log-normal

distribution provided a better fit, in terms of Kolmogorov-Smirnov Goodness of Fit test, i.e. Kolmogorov-Smirnov test statistic was 0.077, which could be accepted at 5% significant level because the p-value corresponding 5% significant level was 0.0816 (see Figure 36).

- Data for connected roof areas were not available for in Metropolitan Adelaide. Hence the dataset reported in Biermann *et al*., (2012) was used to demonstrate the methodology. It should be noted that this dataset should be replaced with a representative dataset for any future studies aim to compute yield and overflow implications of rainwater tanks in Adelaide. The dataset comprised connected roof areas of 30 households with internally plumbed rainwater tanks in Redlands Local Government Area in South East Queensland (Figure 37). Roof areas in this data sample varied from 25 m^2 to 260 m^2, with a mean of 111.6 m^2 and a standard deviation of 47.14 m^2. The normal distribution provided a good fit, i.e. Kolmogorov-Smirnov test statistic was 0.182, which could be accepted at 20% significant level because the p-value corresponding 20% significant level was 0.244 (see Figure 37).
- The data on roof losses were sourced from Xu et al., (2010). Again, it should be noted that this dataset was used to demonstrate the methodology and it should be replaced with a representative dataset for any future studies aim to compute yield and overflow implications of rainwater tanks in Adelaide.
- As mentioned above, household demand data at the end use scale for a representative sample of houses were required to represent the spatial variability of demand placed on the rainwater tanks. However, such data being collated at the time of undertaking this study. Hence we used a single time series for each residential end use, developed by splitting monthly demand described in Section 3.6, using the factors given in Table 9.

Table 33: Rainwater tank parameters

	Tank size (kL)	Effective roof area (m^2)	Initial loss (mm)	Continuing loss (%)
Observed minimum	0.40	25.00	0	0
Observed Mean	7.98	111.63	0.5	15
Observed maximum	74.00	260.00	1.75	30
Standard Deviation	10.32	47.14	0.5	5
Sample size	277	30	N/A	N/A
Probability distribution	Log normal	Normal	Normal	Normal



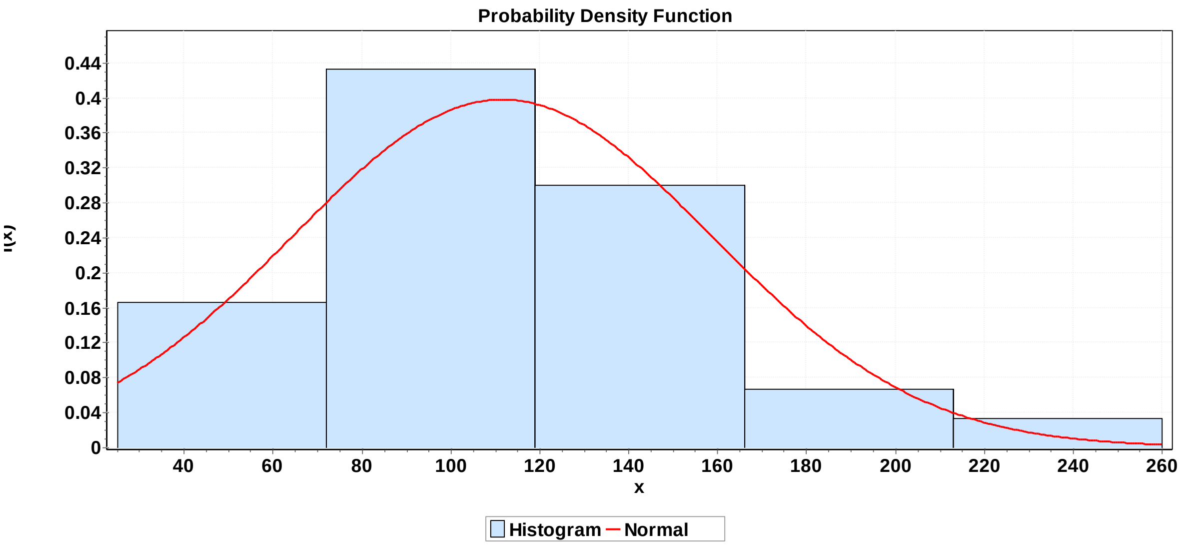


Figure 37: Connected roof area of households with internally plumbed rainwater tanks (source: Biermann *et al.*, 2012) (Kolmogorov-Smirnov test statistic = 0.182, accepted at 20% significant level; p-value = 0.244)

The stochastic simulation was performed for 50-years from 1 July 1963 to 30 June 2013, on a daily basis. The outputs of the stochastic simulation included time series of values for tank supply (i.e. tank yield) in kL/household/day, tank inflow in kL/household/day and tank overflow (i.e. spill) in kL/household/day. The expected values computed from these outputs represented most probable (or the expected) supply, inflow and overflow, which could be up scaled to Metropolitan Adelaide, by simply multiplying the number of households in Metropolitan Adelaide. The expected supply, inflow and overflow from a typical rainwater tank in Metropolitan Adelaide computed through the stochastic simulation method described above are given in Table 34.

The tank yield figures given in Table 34 implied 43 kilolitres per household as the expected annual yield (or 3.6 kilolitres per household as the expected monthly yield) from an internally plumbed rainwater tank in Metropolitan Adelaide. It should be noted that the expected annual yield computed for the study area was of similar order of magnitude to the observed average annual yield for Brisbane, i.e. 40 kilolitres per household (Umapathi et al., 2012) and the computed average annual yield for Brisbane by using the same stochastic simulation approach, i.e. 42 kilolitres per household (Maheepala et al., 2013). However, the long term average rainfall of Brisbane and Adelaide are 1144 mm and 548 mm, respectively. Therefore, the expected annual yield of rainwater tanks computed for Adelaide, by using the available data appears to at higher end of the expected yield. This could be due to the following reasons:

- The demand placed on rainwater tanks is assumed to be comprised of the total water demand of laundry, toilet, outdoor and hotwater, which is about 80% of the total household demand. As the demand placed on the tank increases, the tank supply tends to increase, i.e. yield tends to increase;
- In the absence of household end use water demand data, the demand placed on tanks has been computed by disaggregating monthly household demands to daily end uses by using the factors given in Table 9 and the number of days per month (i.e. 30 days), which do not allow the variability exhibited by household end use water demands to be represented accurately. This can result in an overestimation of the demand placed on tanks. Consequently, tank yield can be higher than expected;
- Tank sizes obtained from the household survey have indicated 8 kL as the average tank size, which is considerably larger tank size, which tends to give higher yield; and
- The data on roof sizes and roof losses are literature based values (which have been used due to the unavailability of such data) and their applicability to Adelaide is not known.

Therefore, it can be said that while the method used to compute the expected yield of rainwater tanks is a valid and robust method, the data used by this study has limitations. In addition, the assumption made with regard to possible uses of rainwater may not be appropriate for Adelaide. Therefore, the expected yield computed by this study should be used cautiously and the applicability of the data and the assumptions made with regard to rainwater use, should be reviewed for future studies.

The method described above with regard to rainwater tanks, has described how water quantity related implications of a household scale source can be up scaled to the study area scale. The method described below, shows how the expected tank supply and tank overflow computed through the stochastic simulation can be used to compute the reduction in demand for water supplied by the other sources and the reduction in urban catchment runoff, respectively, in the presence of rainwater tanks.

Table 34: Expected tank yield, overflow and inflow, computed over 50 year period from July 1963 to June 2013

Expected values	Average annual (kL/hh)	Average monthly (kL/hh)	Range for monthly values (kL/hh)	Coefficient of variation for monthly values
Tank yield	43	3.6	0-9.2	0.65
Tank overflow	5	0.4	0-4.8	1.51
Tank inflow	50	4.1	0–17.3	0.75

Assume$\dot{D}_t$, $\dot{I}_t$ and $\dot{O}_t$ were the expected tank supply, tank inflow and tank overflow respectively, at time t, computed through the stochastic simulation of storage behaviour of a rainwater tank (described above). It should be noted that since the stochastic simulation was performed on a daily basis, t represented a day. Since supply reliability assessment was performed on a monthly basis, $\dot{D}_t$ should be converted to a monthly time series by aggregating the daily values. If $\dot{D}_T$ is the corresponding monthly value of $\dot{D}_t$ and T represent a month, the amount of water supplied by tanks in demand zone i at time T (i.e. $\overline{D}_T^i$) could be computed as follows:

$$\overline{D}_T^i = \alpha N_i \dot{D}_T$$

Equation 34

Where, $\overline{D}_T^i$ the amount of water supplied by tanks in demand zone i at time T (kL/month), α is the adoption rate of rainwater tanks, N_i is the number of households in demand zone *i* (Table 10) and $\dot{D}_T$ (kL/month) is the corresponding monthly value of $\dot{D}_t$, which is the expected tank supply at time t.

As per the assumptions mentioned above, residential non-potable demand was supplied first. If there was water remaining in the tank, after meeting non-potable demands, the hot water component of the residential potable demand was supplied. Thus, in the presence of rainwater tanks, residential non-potable demand in zone i in time T (say, RNP_T^i) and residential potable demand zone i in time T (say, RP_T^i) could be computed as follows:

if $\overline{D}_T^i \leq RNP_T^i$, $RNP_T^i = RNP_T^i - \overline{D}_T^i$ and $RP_T^i = 0$

Equation 35

if $\overline{D}_T^i > RNP_T^i$, $RNP_T^i = 0$ and $RP_T^i = RP_T^i - \left(\overline{D}_T^i - RNP_T^i\right)$

Equation 36

where $\overline{D}_T^i$ is the amount of water supplied by tanks in demand zone i at time T (kL/month), RNP_T^i is the residential non-potable demand in zone i in time T (kL/month) and RP_T^i is the residential potable demand zone i in time T (kL/month).

The presence of rainwater tanks, captured runoff which would otherwise discharge to the receiving waters. The following method was used to quantify the amount of runoff reduction at the sub-catchment scale, in the presence of rainwater tanks.

Assume there were j = 1 to J sub-catchments that contributed to runoff discharging from the study area to coastal waters. The runoff from jth sub-catchment at time t (say, R_t^j) could be computed as follows:

$$R_t^j = R_t^j - \propto N_j \dot{I}_t + \propto N_j \dot{O}_t$$

Equation 37

Where, R_t^j is the runoff from jth sub-catchment at time t (kL/month), N_j is the number of households in sub-catchment j, $\dot{I}_t$ is the expected tank inflow at time t (kL/month) and $\dot{O}_t$ is the expected tank overflow at time t (kL/month), computed through stochastic simulation of storage behaviour of a rainwater tank, respectively.

The amount of water captured by the rainwater tanks in sub-catchment j at time t (C_t^j) could be computed as follows:

$$C_t^j = \propto N_j \dot{I}_t - \propto N_j \dot{O}_t$$

Equation 38

where C_t^j is the amount of water captured by the rainwater tanks in sub-catchment j at time t (kL/month) and N_j, $\dot{I}_t$ and $\dot{O}_t$ have the same meaning as above.

The number of household in each sub-catchment (i.e. N_j) was determined by assuming that the housing density was homogeneous across Metropolitan Adelaide. This assumption was made in the absence of data on housing densities at the sub-catchments scale.

Firstly, the housing density at the study area scale was computed, which was then multiplied by the area of each sub-catchment, to determine the number of houses in each sub-catchment. For example, for 2013 scenario, the housing density was computed by dividing the total households in 2013 (i.e. 514,644) by the total urban area (i.e. 34,229.66 ha). This gave 15 houses/ha or a block size of 667 m^2 as the housing density at the study area scale, which was then multiplied by the area of urban land uses in each sub-catchment. The number of households in each sub-catchment computed in this manner for scenario 2013 is shown in Table 35. The same method was adopted to determine the housing distribution at the sub-catchment scale for 2025 and 2050.

Table 35: Number of households in sub-catchments where sub-catchments are specified as per the Source Model

Sub-catchment ID	Number of households		
	2013	2025	2050
78	14445	17145	23290
77	7536	8945	12151
76	590	700	951
75	0	0	0
74	1503	1785	2424
73	5723	6793	9227
72	58	69	94
71	184	219	297
70	8680	10303	13995
69	2204	2616	3553
68	2619	3108	4222
67	1925	2285	3104
66	2262	2685	3647
65	7549	8961	12172
64	8954	10628	14437
63	2654	3150	4279

Sub-catchment ID	Number of households		
	2013	2025	2050
62	843	1000	1359
61	5336	6334	8604
60	6529	7750	10527
59	3394	4029	5473
58	4230	5020	6819
57	12825	15223	20678
56	15714	18652	25337
55	11299	13411	18218
53	27537	32685	44399
45	7679	9115	12381
44	34641	41117	55853
42	12496	14833	20149
40	18211	21616	29363
38	3665	4350	5909
37	1154	1370	1861
26	1605	1905	2588
25	3251	3859	5242
24	0	0	0
23	3093	3672	4988
22	4616	5479	7442
13	1098	1303	1770
12	5792	6875	9339
11	2270	2695	3661
10	1090	1294	1758
9	1714	2034	2764
8	15836	18796	25533
7	635	754	1024
6	27465	32600	44284
5	4545	5395	7328
4	9819	11654	15831
3	8324	9880	13421
2	74446	88364	120033
1	54066	64174	87173
27,28,29,30,31	876	1040	1412
41,39	2601	3087	4194

Sub-catchment ID	Number of households		
	2013	2025	2050
32,33,34,35,36,43,54,51,46,47,48,49,50,52	51879	61578	83648
14, 15, 16, 17, 18, 19, 20, 21	7183	8526	11581
Total	**514644**	**610860**	**829787**

Capital and operational costs of rainwater tanks

As described above, the study considered the spatial variability of tank sizes. Hence an appropriate approach to determine the cost of a tank was to compute either:

1. Average or median value of the distribution of costs, depending on the skewness of the distribution of costs; or
2. The cost of either median or average value of the distribution of tanks, depending on the skewness of the distribution of tanks.

For this study, approach #1 was adopted. Since the distribution of tank sizes was a skewed (Figure 36), the median tank size was used, i.e. 4.9 kL (rounded of to 5 kL). Thus, the cost (and also energy) computation methods described below represented those of a 5kL rainwater tank.

Table 36: capital cost for a 5 kL rainwater tank (source Paton et al., 2013)

Item	Cost in 2013 dollars
Purchasing of a 5 kL tank	1088
Delivery and installation	702
Dolomite base	270
Pump	864
Plumbing cost –for an existing house	1080
Plumbing cost –for a new house	324
Total cost –for an existing house	**4004**
Total cost – for a new house	**3248**

The capital cost of a rainwater tank included the cost of purchasing the tank, delivery and installation of the tank, the cost of dolomite base and pump, and the cost of plumbing. Except for cost of plumbing, all the other costs were considered the same for both an existing and a new household (see Table 36).

The current adoption rate of rainwater tanks in Greater Metropolitan Adelaide is 44.4% and this adoption rate appears to be fairly constant over the last six years (ABS Environmental Issues: Water use and Conservation, Mar 2013). Hence it is assumed that there is no capital (or installation) cost for the values of decision variable up to 44.4%.

Table 37: Number of new households in North, Central and South demand zones

Demand Zone	2013	2025	2050	2013	2025	2050
	Total number of new households			New households as a percent of total households		
North	7,758	32,234	73,342	4.5%	15.8%	26.4%
Central	9,180	38,146	86,798	4.5%	15.8%	26.4%
South	6,218	25,836	58,787	4.5%	15.8%	26.4%
Total	**23,156**	**96,216**	**218,927**	**4.5%**	**15.8%**	**26.4%**

The capital cost of rainwater tanks for 2013, 2025 and 2050 were computed as follows:

Capital cost (\$)= 0, when α ≤ 44.4%

Equation 39

Capital cost (\$)= $\propto N\{4004\beta + 3248(1-\beta)\}$, when α > 44.4%

Equation 40

Where, α was the adoption rate of rainwater tanks, N was the total number of households and β was the percent of existing households.

The proportion of new households in each demand zone in 2013 was assumed to be 4.5%. This will give the proportion of new households in 2025 and 2050 for each zone as 15.8% and 26.4% respectively (Table 37). Hence the proportion of existing households in 2013, 2025 and 2050 are: 95.5%, 84.2% and 73.6% respectively.

The average operational cost of a rainwater tank was assumed to be \$22 per year for maintenance (in 2013\$) and \$0.36 per KL for electricity (in 2013\$). Hence the average operation cost for a household ($\bar{C}$) in \$/year/household, was computed as:

$$\bar{C} = 22 + 0.36\frac{\sum_{T=1}^{n} \dot{D}_T}{\acute{N}}$$

Equation 41

Where, $\bar{C}$ is the average operation cost for a household (\$/year/household), $\dot{D}_T$ was rainwater tank yield in month T computed through stochastic simulation, n was number of simulation periods and, Ń is the number of simulation years.

Therefore, the total operation cost over 25 years is given by the following equation:

Total operation cost (in \$) = α X N X 12.8 X $\bar{C}$

Equation 42

Where, 12.8 is the present worth factor for a 25 year life and 6% discount rate, α is the adoption rate of rainwater tanks, N is the total number of households.

Energy consumption of rainwater tanks

The energy consumption of a rainwater tank comprised embodied energy of the tank and installation material, and the energy consumed by the pump (i.e. operational energy). Based on Paton (2013), the embodied energy for a 5kL tank was 2024 kwh, and the energy consumed by the pump for a 5 kL tank that supplies garden, toilet, laundry and hotwater service was 1.41 kwh/kL.

The average annual operational energy consumption per household ($\bar{E}$), in kwh/year/household, was computed as follows:

$$\bar{E} = 1.41 \frac{\sum_{T=1}^{n} \dot{D}_T}{\acute{N}}$$

Equation 43

Where $\bar{E}$ is the average annual operational energy consumption per household (kWh/year/household), $\dot{D}_T$ was rainwater tank yield in month T computed through stochastic simulation, n was number of simulation periods and, Ń is the number of simulation years.

The total energy consumption over 25 years in kwh =

$$\alpha N(2024 + 25\bar{E})$$

Equation 44

Where α is the adoption rate of rainwater tanks, N is the total number of households, 2024 is the embodied energy for a 5kL tank (kWh) and 25 is the number of years considered.

4.6.8 Modelling demand management options

The following demand management (DM) options were considered:

- dual 6/3 litre toilets
- 3-star showerheads
- front loading washing machines (or clothes washers).

The current adoption rate of 6/3 litre toilets, 3-star showerheads and front loading washing machines were assumed to be 46%, 55% and 54% respectively. These current adoption rates were based on the survey conducted as part of Task 4 of the Optimal Water Resource Mix Project. If DM was imposed, 100%, 84% and 84% of adoption rates were assumed for 6/3 litre toilets, 3-star showerheads and front loading washing machines, respectively (Table 38). The 84% maximum adoption rate was assumed based on the diffusion of innovation theory (Rogers, 2003) that assumed approximately 16% of people were 'laggards' who only adopt innovation when forced to do so.

Table 38: Adoption rates of demand management options

DM option	Current adoption rate without demand management	Assumed adoption rate for 2013 scenario with demand management
Dual 6/3 litre toilets	46%	100%
3-star showerheads	55%	84%
Front loading washing machines	54%	84%

The reduction in potable and non-potable residential demand due to the adoption of DM as per the rates given in Table 38 was computed by using the Behavioural End-use Stochastic simulator (BESS) (Thyer et al., 2009), which was a model for predicting water demand of household end uses (Table 39). The assumptions made and the data used to compute the reduction in demand due to above-mentioned DM options are described in Appendix 2.

Table 39: Percent reduction in residential demands due to demand management, compared to the demand without demand management for 2013 scenario

Month	Percent reduction in Residential Potable demand (i.e. β_t)	Percent reduction in Residential Non-potable demand (i.e. θ_t)
January	4%	5%
February	4%	6%
March	4%	7%
April	4%	10%
May	4%	13%
June	4%	18%
July	4%	18%
August	4%	17%
September	4%	15%
October	4%	11%
November	4%	8%
December	4%	6%

When DM was imposed, the reduction rates given in Table 39 were used to compute the residential potable and non-potable demands as follows:

$$D_t^i = D_t^i(1 - \beta_t/100)$$

Equation 45

$$ND_t^i = ND_t^i(1 - \theta_t/100)$$

Equation 46

Where, D_t^i was the residential potable demand in zone *i* in time *t* (kL/month), ND_t^i was the residential non-potable demand in zone *i* in time *t* (kL/month), and β_t and θ_t were the percent reduction in residential potable and non-potable demands due to demand management, respectively, compared to the demand without demand management. β_t and θ_t are given in Table 39.

The average reduction in demand over the simulation period, say S_{DM}, was found by adding the reduction in potable and non-potable demands over all the simulation time-steps (i.e. N) and all the demand zones, and dividing it by the number of simulation years, (say, Ń), as given in Equation 47:

$$S_{DM}=\frac{1}{N}\sum_{i=1}^{3}\sum_{t=1}^{N}(\beta_t\, D_t^i + \theta_t N D_t^i)$$

Equation 47

Capital and operational costs of demand management

The average cost of a top loading washing machine (installed) was assumed to be $794. Considering that front loading washing machines were generally replacements to top loading washing machines, the differential cost of front loaders compared to top loaders was used to compute the capital cost of front loading washing machines. The differential cost between front loading machines compared to top loading machine was assumed to be $200. A life time of 8 years was also assumed.

Thus, the PV of unit cost of a washing machine was: 200 (1 + 1.06^{-8} + 1.06^{-16} + 1.06^{-24}) = $454. This unit cost was applied to houses that did not already have a front loading washing machine, which were assumed to be 46% of existing households and a 46% of new households.

The cost of 3-star showerhead (installed) was assumed to be $50. This unit cost was applied to existing houses, which was assumed to be 45% of the total housing stock. The new households were considered to be installed with 3-star showerheads. Hence this unit cost was not applied to new houses. Assuming that the current stock of showerheads would be replaced over a 10 year period with 3-star showerheads, the PV of 3-star showerhead cost was computed by taking the difference between the PV of the cost of replacement now and the replacement in 5 years' time (on average).

Hence, the PV of replacement cost was: 50 - 50*$(1.06)^{-5}$ = $13.

For the 2025 and 2050 scenarios, it was assumed that all showerheads were of 3-star rating.

Table 40: Unit costs of DM options in 2013, 2025 and 2050

Demand Management Option	2013		2025		2050	
	PV Unit Cost	% of houses to be applied	PV Unit Cost	% of houses to be applied	PV Unit Cost	% of houses to be applied
Top loading washing machine	454	30%	454	30%	454	30%
Low flow showerhead	13	29%	0	-	0	-
6/3 dual flush toilet	190	54%	0	-	0	-

A similar approach was adopted for costing of 6/3 dual flush toilet. The cost of a 6/3 dual flush toilet (installed) was considered to be $753. This unit cost was applied to the existing houses, which was assumed to be 54% of the total housing stock. All new houses were installed with 6/3 dual flush toilets. Assuming that the current stock of toilets was replaced over a 10 year period with 6/3 dual flush toilets, the PV of cost was the difference between the PV of the cost of replacement now or replacement in 5 years' time (on average).

Hence, PV of replacement cost was: 753 - 753*$(1.06)^{-5}$ = $190.

For the 2025 and 2050 scenarios, all toilets were assumed to be dual flush toilets.

The unit costs computed by using the above-mentioned approaches are summarised in Table 40.

The above unit costs were then used in the following manner to compute the total costs with DM:

- for 2013 with DM: PV cost = (454 X 0.3 + 13 X 0.29 + 190 X 0.54) X total no of houses
- for 2025 with DM: PV cost = (454 X 0.3) X total no of houses
- for 2050 with DM: PV cost = (454 X 0.3) X total no of houses.

Energy savings of demand management options

Both front loading washing machines and top loading washing machines were considered to be using the same amount of cold water, which implied that there was no impact on the energy consumption, when switching to front loading washing machines. However, this assumption may not be valid because in general front loading washing machines use less water compared to the top loaders. Therefore, the estimated energy savings should be treated as conservative. Also, switching to 6/3 litre dual flush toilets did not have any impact on the energy consumption. However, the use of 3-star showerheads could reduce energy consumption due to lower water use if the duration of a shower event remained the same with and without installing a 3-star showerhead.

Assuming that the shower duration remained the same, the estimated water saving resulting from a 3-star showerhead was 5.5 L/person per day (or 2 kL /person/year). Assuming an average 29.1 kwh of energy saving for a kilolitre of hot water, the average annual energy saving for a household in 2013 was computed as:

Energy saved in kwh/household/year = (5.5 X 365 X 2.4/1000)X29.1 = 140. 2

Equation 48

Energy saved over 25 years in kwh/household = 140.2 X 25= 3505.1

Equation 49

Hence energy savings (or negative energy consumption) with and without DM, over 25 years was computed as follows:

For 2013 with DM: energy consumption (in kwh) = *-3505.1* X total no of houses

Equation 50

For 2025 with DM: energy consumption = 0.0

Equation 51

for 2050 with DM: energy consumption = 0.0

Equation 52

Impact of demand management options on wastewater

Adoption of DM options has the potential to reduce wastewater flows due to lower water usage in shower, toilet and laundry. A relationship between residential demand and wastewater inflow should be examined to quantify the impact of DM options on wastewater inflow. This required identification of the wastewater generating from residential uses. However, sufficient data were not available to characterise wastewater inflows in terms of their sources. Hence we were unable to develop a relationship between residential water demand and wastewater inflow. The potential reductions in wastewater inflow (and recycled water) due to DM options were therefore, not accounted for in the simulation model.

4.6.9 Modelling groundwater supply

Historically, groundwater has been extracted from the deep tertiary aquifer systems beneath the Metropolitan Adelaide, for industrial users and for large irrigation users such as market gardens, schools and golf courses (Zulfic et al., 2008). In addition, some households use groundwater as a supplementary source of water for outdoor uses.

At present, there are about 25,700 wells in the study area (Figure 38). A majority of the wells, in particular the wells used for stock and domestic uses are not metered, and also, not licensed (i.e. not monitored or metered). At present licensed volumes are known for about 700 wells, i.e. about 3% of the total wells (Figure 39). Therefore, it can be said that the current usage of groundwater in Metropolitan Adelaide is largely unknown. On several occasions up until the 1970s, SA Water extracted up to 10 GL/year from a number of bores in the western suburbs of Adelaide to supplement the Mount Lofty Ranges reservoir water supply during drought years. The most recent estimate of groundwater extraction in the Metropolitan Area is about 10 –12 GL/year, with most extractions coming from the T1 aquifer, which is defined as the shallowest Tertiary aquifer system. The Tertiary aquifers are made of Tertiary sediments and groundwater occurs mainly in four mostly confined Tertiary aquifers, designated T1 to T4 in order of increasing depth. The thickness of the T1 aquifer in the study area varies approximately from 25 m to 120 m (Zulfic et al., 2008). Salinity distribution in the T1 aquifer varies from less than 500 mg/litre to about 3500 mg/litre (details of hydrogeology in the study area are given in Zulfic et al., 2008).

To consider groundwater as an alternative source, information on the amount of water available, and the existing and potential users of this source were required. Since licensed allocation amounts were known only for a small proportion of wells (i.e. approximately 3% of the total wells), it was difficult to estimate the extent of current groundwater use in terms of the volume, users and the pattern of use. Discussions with the South Australian DEWNR indicated that at present, available groundwater in the study area was considered as fully allocated, which indicated that groundwater could not be considered as an alternative supply source for the future years.

Therefore, a decision was made not to include groundwater as a supply source in the simulation model. To balance supply and demand, the amount of demand currently supplied by the groundwater was excluded from the water demand being considered by the study. Based on the estimates described above, the excluded amount of the demand, by considering the supply from groundwater was, approximately 10 GL/year. It should be noted that the demand figures described in Section 3.6 have accounted for the demand being currently supplied by groundwater, i.e. 10 GL/year should be added to the total demand figures given in Table 6, if it is required to report the total demand being considered by the study, noting the assumption that 10 GL/year is supplied by groundwater for 2013, 2025 and 2050 scenarios.

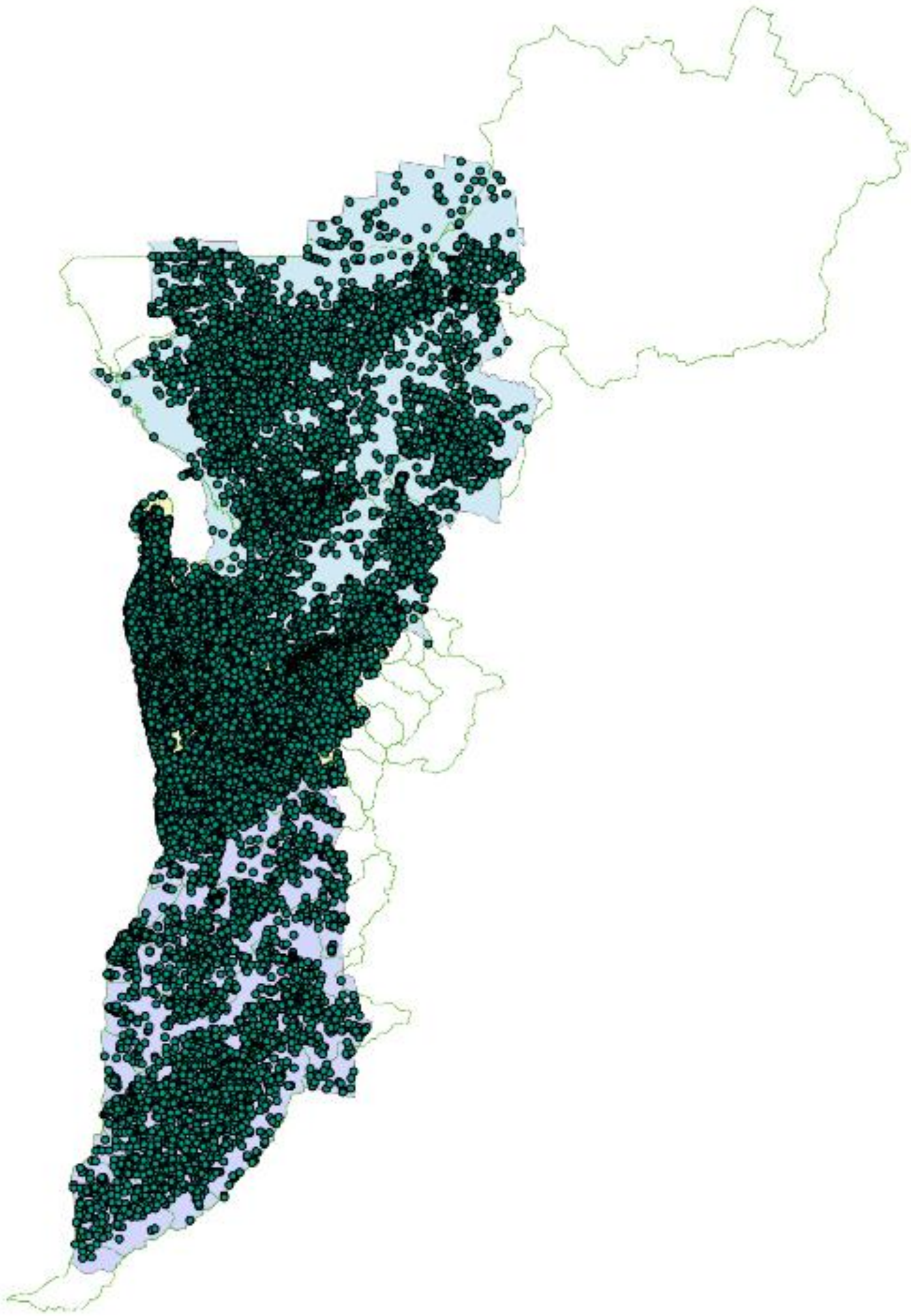


Figure 38: Groundwater wells in the study area: both licensed and unlicensed (data sourced from: DEWNR, South Australia)

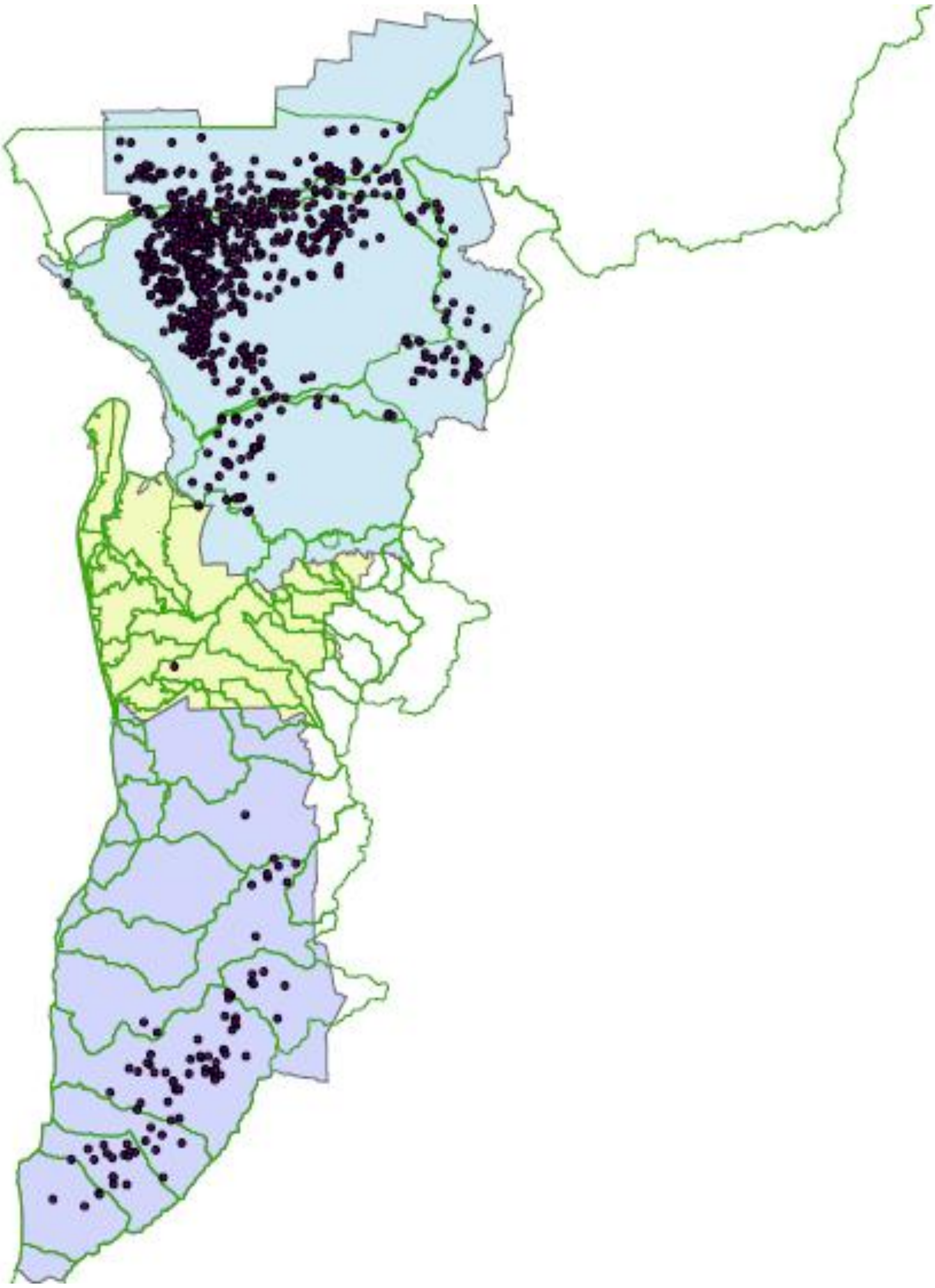


Figure 39: Groundwater wells with licenses to extract water (data sourced from: DEWNR, South Australia)

4.7 Identify efficient options

An optimisation method based on Genetic Algorithm (GA) approach was used to identify efficient solutions. The optimisation method was seamlessly integrated with the simulation method (described in Section 4.6) to evaluate a large number of options in terms of the objectives defined in Section 4.3, with an aim of identifying the most efficient options and the associated trade-offs.

The Insight module of the Source model (eWater, 2012) was employed to implement the optimisation component of the simulation-optimisation approach adopted in the IUWM DSF. The Insight module included the Non-dominated Sorting Genetic Algorithm II (NSGA-II) (Deb, 2002). In this section, an overview of multi-objective optimisation problem is introduced and the details of multi-objective GAs are discussed, prior to presenting results in Chapter 5.

4.7.1 Overview of multi-objective optimisation

A multi-objective optimisation problem uses the concept of domination introduced by Fonseca and Fleming (1993) to deal with the tradeoffs between or among conflicting objectives (Deb, 2002). Solution x is said to dominate solution y, if both of the following conditions are true:

- Solution x is no worse than solution y in all objectives
- Solution x is strictly better than solution y in at least one objective.

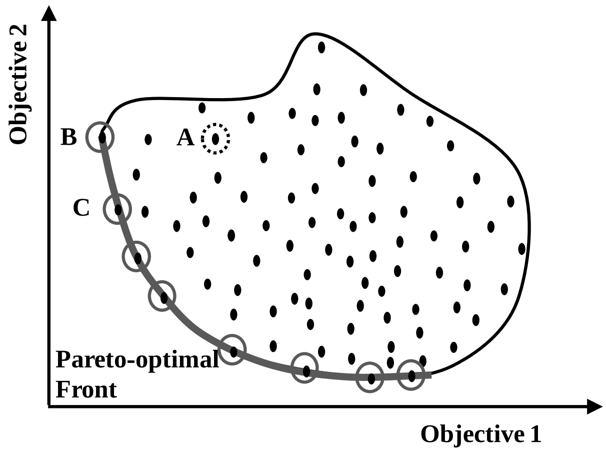


Figure 40: Objective space and Pareto-optimal front of a two-objective minimization problem

An example of the trade-offs between two conflicting objectives of a minimization problem is shown in Figure 40. Each black dot in this figure represents a solution point in the objective space. It can be seen that Solution B is better than Solution A in terms of objective 1, but they both have the same value for objective 2. As a result, Solution B is said to dominate Solution A. Comparing Solutions B and C, B is better in terms of objective 1 but worse in terms of objective 2. Therefore, Solutions B and C are called non-dominated solutions. The solutions within the solid circles dominate all other solutions in the objective space. However, they are non-dominated solutions to each other. These non-dominated solutions are called Pareto-optimal solutions. They form a front (the darker line), referred to as the Pareto-optimal front.

If there is higher level information available for decision making, a biased search can be used to find desired solutions among the Pareto-optimal solutions. However, in most cases such information is not available. Therefore, the Pareto-optimal solutions are equally important. The ultimate goal of multi-objective optimization is to find all of these Pareto-optimal solutions. However, multi-objective optimization problems are complex and non-linear. Consequently, it is often impossible to find all Pareto-optimal solutions within the desired/available computational time using current technologies. Seen in this way, there are two goals in multi-objective optimization (Deb, 2002):

- to find a set of non-dominated solutions as close to the Pareto-optimal front as possible
- to find a set of non-dominated solutions as diverse within the optimality region as possible.

The quality of a multi-objective optimisation algorithm can be judged by using these two goals.

4.7.2 Multi-objective optimisation using Genetic Algorithms approach

Genetic algorithms (GAs) are a global optimisation method developed by John Holland and his students at the University of Michigan (Goldberg, 1989). As the name suggests, the concept of GAs is inspired by the natural phenomenon of heredity, in which the principle of 'survival of the fittest' is used to select more suitable trial solutions. In each generation of a GA, a population of alternative solutions, each represented by a vector of decision variables called a chromosome or string, is evaluated and selected based on the objectives of the optimization problem and varied (e.g. crossover and mutation) to create offspring. This process is repeated and it is expected that after some generations, the GA will produce offspring that are superior to their parent counterparts.

The general framework of a GA is shown in Figure 41. A GA first requires a genetic representation of the solution domain (chromosomes or strings) and a mathematical representation of the objective domain (objective functions). An encoding scheme is required to link the genetic representation of each string in a GA to its corresponding physical solution in the real world, which enables the objective functions of the string to be evaluated. A GA relies on three genetic operators – selection, crossover (or mating) and mutation – to produce offspring. The objective function value is used directly in the selection process as an indicator of the quality of the string and to decide whether or not a string will participate in the mating process (crossover). Once a string is selected into the mating pool, it will be paired up with another selected string and parts of each string will be exchanged (or crossed over) to produce child solutions. Mutation of each individual offspring may then occur to introduce diversity and prevent premature convergence to local optima, which is defined as the best solution(s) in a small local region of the search space. By applying the three genetic operators repeatedly, GAs maintain good solutions in the current generation and explore the searching space for better solutions in the next generation. This searching process will stop when certain stopping criteria are met.

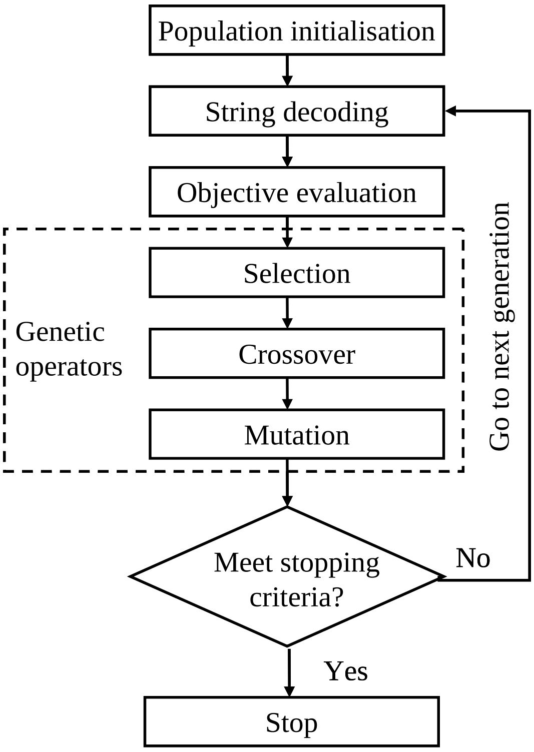


Figure 41: Framework of a typical GA

A multi-objective GA differs from a traditional single objective GA in that in the selection process non-dominated sorting is used to compare solutions regarding all objectives.

4.7.3 Non-dominated Sorting Genetic Algorithm II (NSGA-II)

Non-dominated Sorting Genetic Algorithm II or NSGA-II was developed by Deb et al. (2002) and is classified as a second generation multi-objective GA. In addition to the conventional steps of GAs described above, NSGA-II has four special features, which address the concerns over traditional (or first generation) multi-objective GAs, including the high computational complexity, the lack of elitism and the need of specifying an additional parameter.

First of all, a special book-keeping strategy is used in the non-dominated sorting process of NSGA-II. Instead of repeatedly ranking the dominated solutions in the population for each rank, every solution in the population is checked with a partially filled dominating population until the partially dominating population grows to include all non-dominated solutions. In this approach, the maximum computation required for the non-dominated sorting of the entire population is of O(MN2) instead of O(MN3), which reduces computational complexity significantly. Secondly, instead of ranking the parent population only, as in traditional multi-objective GAs, a global population, which combines both the parent and child populations, is ranked in NSGA-II. This global population guarantees that good solutions in the parent population will not be lost due to crossover or mutation, thus elitism is introduced into the algorithm. Thirdly, a crowding distance comparison is used to compare solutions within the same rank to maintain the diversity of non-dominated solutions; hence, a sharing parameter is not required.

Furthermore, the traditional constraint handling method used by most GAs is not very effective and requires specification of the value of an additional parameter (Vairavamoorthy, 2000). An efficient constraint handling method (Deb, 2000) based on tournament selection and referred to as constrained tournament selection (Deb, 2000) is used in NSGA-II. In this tournament selection, the feasibility and constraint violation of each solution are first checked against all constraints. A solution x is said to dominate a solution y, if any of the following is true (Deb, 2002):

- Solution x is feasible and solution y is infeasible
- Solutions x and y are both feasible, but solution x has a smaller fitness function value (minimizing fitness function value is assumed)
- Solutions x and y are both infeasible, but solution x has a smaller constraint violation.

In this way, a penalty coefficient is not required and feasible solutions always have priority over infeasible solutions.

4.7.4 Multi-objective Optimisation Parameters

There are a number of parameters that need to be determined when undertaking optimisation using a multi-objective genetic algorithm (MOGA). These parameters include the size of the population, the number of generations, the probability of crossover, the probability of mutation, the simulated binary crossover distribution index (if real number decision variables are used), the polynomial mutation distribution index (if real number decision variables are used) and the random seed (i.e. the random starting point). However, when using Insight, only the size of the population, the number of generations and the random seed can be specified by the user with all the other parameters being fixed within the source code of Insight.

For this project, a range of population sizes and numbers of generations were tested during the development of the model. These are given in Table 41. The best results were obtained with a population of 100 and a number of generations equal to 200.

Table 41: Optimisation parameters used

Parameter	Values tested	Final value selected
Population size	24, 48 and 100	100
Number of generations	30, 100, 200 and 500	200
Random seed used		0.123, 0.147

In order to test convergence of the MOGA, plots of the hypervolume versus the number of generations were obtained. The hypervolume was a measure of the convergence of any multi-objective algorithm (Zitzler, 1999). A typical plot obtained in this study was shown in Figure 42. It demonstrated a little change in hypervolume over the last 40 generations, thus indicating that reasonable convergence was achieved.

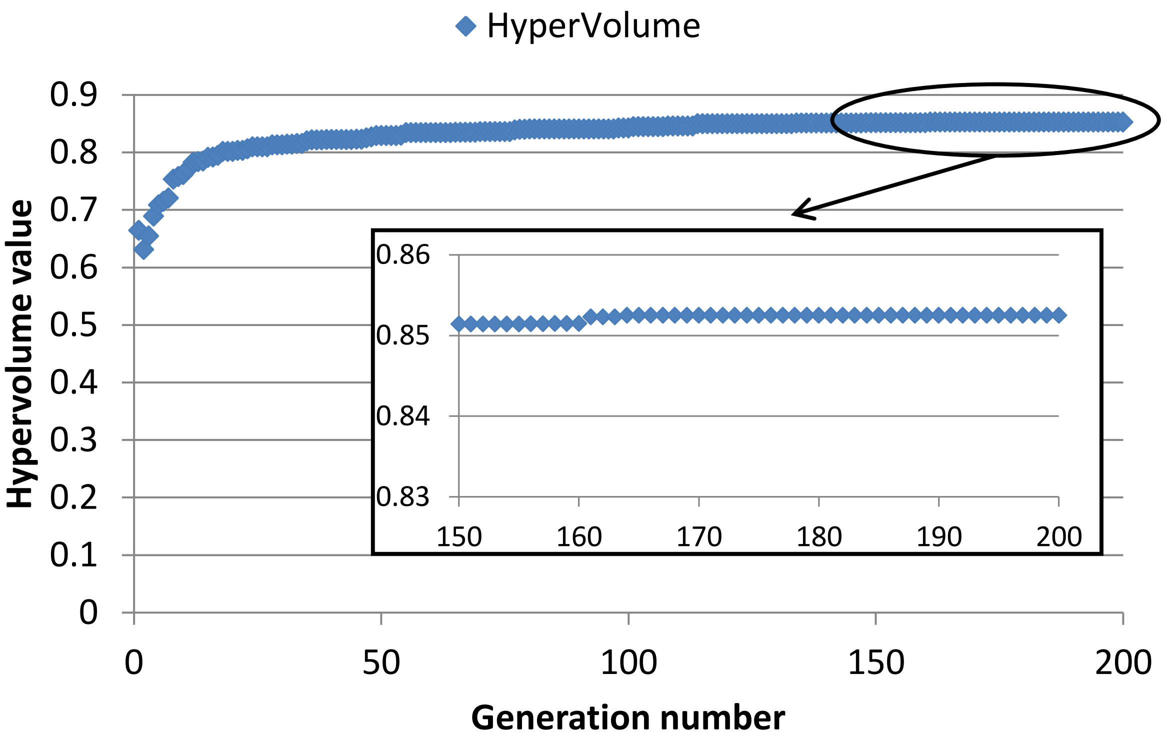


Figure 42: Example of hypervolume variation with the number of generations (for model with Priority Set #1 and seed 0.123 for the 2013 scenario)

As GAs contained some stochastic operators they were sensitive to the random starting point (i.e. random seed) used to initialise the first population of solutions. In this study, two different random seeds were used to check if near-global optimal solutions were found. Hence, the solutions presented in this report comprised the combined Pareto-optimal solutions from two different runs. These random seeds were given in Table 41. It should be noted that, ideally, a larger number of random seeds should be trialled (e.g. 10). However, this was not possible due to time limitations of the study.

4.7.5 Priority of water supply sources

The Source-schematic module uses network linear programming (NetLP) to allocate water to demand nodes in the presence of multiple sources. The NetLP approach requires the order of priority be specified for sources, using 'penalty costs'.

The version of the Source model used in this study does not allow these penalty costs to interact with the Genetic Algorithm optimisation approach being used in the Source-insight module. This has the disadvantage that the optimal priority order cannot be determined through the Genetic Algorithm optimisation approach. However, it has the advantage of being able to take into account the stakeholders' preferences directly. The priorities for the various water sources used in this study are summarised in Table 42.

Priority Set #1 was based on the order of priorities expressed by participants in a survey carried out as part of the Goyder Institute's managed aquifer recharge stormwater use options project (Mankad et al, 2013). It reflected the community's desire to use harvested stormwater and reclaimed wastewater for non-potable purposes, if possible. The same survey indicated that the majority of participants were not willing to pay more for harvested stormwater than the (then) current price of water.

Table 42: Priority sets of water sources for potable and non-potable use

Priority Set	Basis	Priority order for Potable Use[1]	Priority order for Non-Potable Use[1]
#1	Survey carried out as part of the Managed aquifer recharge stormwater use options project (Mankad et al. 2013)	Mt Lofty Ranges River Murray Desalinated water	Harvested stormwater Reclaimed wastewater Mt Lofty Ranges River Murray Desalinated water
#2	Minimisation of assumed operational costs	Mt Lofty Ranges River Murray Desalinated Water	Mt Lofty Ranges River Murray Harvested stormwater Reclaimed wastewater Desalinated water
#3	Preferences of focus groups interviewed as part of the Optimal Water Resources Mix for Metropolitan Adelaide Study	Mt Lofty Ranges Desalinated Water River Murray	Harvested stormwater Reclaimed wastewater Mt Lofty Ranges Desalinated water River Murray

Note 1: The lowest number has the highest priority

Priority Set #2 was based primarily on the operational cost of each source assumed in this study. It reflected a desire to keep operational costs to a minimum.

Priority Set #3 was based on the preferences expressed by the focus groups as part of this study (Optimal Water Resource Mix Project Task 6). The priorities inferred from the focus groups actually had desalinated water ranked ahead of water from the Mt Lofty Ranges, but this was considered to be unreasonable because of the high cost and energy associated with desalination and the fact that it was government policy to use the desalination plant as an emergency source in droughts. However, the community’s desire to reduce the extraction of water from the River Murray was captured by giving this source the lowest priority in Priority Set #3.

5 Results and discussion

5.1 Optimal solutions

The IUWM DSF was applied to Metropolitan Adelaide to examine efficient (or near optimal) supply options for three scenarios, which represented the current (2013) and two future (2025 and 2050) climate and population conditions.

As stated in Section 4.3.1 the four objectives considered are as follows:

1. minimise the present value of the life cycle cost of infrastructure over 25 years with a discount rate of 6% (TC)
2. minimise thepresent value of energy consumption, including embodied energy over 25 years (TE)
3. maximise the volumetric reliability of the non-potable component of the system supply
4. minimise total stormwater and wastewater discharge to the Gulf

The optimal solutions for 2013, 2025 and 2050 are described below.

Each scenario has been optimised separately, so the values of the objectives and the infrastructure associated with each scenario is not incremental to the preceding scenarios.

5.1.1 Optimal solutions for 2013 scenario

For each scenario, specific priority set (listed in Table 42) and random number generator, 100 non-dominated solutions were obtained. For example, Figure 43, Figure 44 and Figure 45 show the trade-offs obtained between pairs of the four objectives for the specific case of the 2013 scenario with Priority Set #1 and seed 0.123.

Note that all the solutions found comply with the constraint on the time-based reliability of potable water supply (>99.5%). Note that some of the solutions presented in Figure 43 may have a larger cost and energy consumption than others, but they are still non-dominated because they may have a lower discharge of wastewater and stormwater into the Gulf and a larger non-potable volumetric reliability.

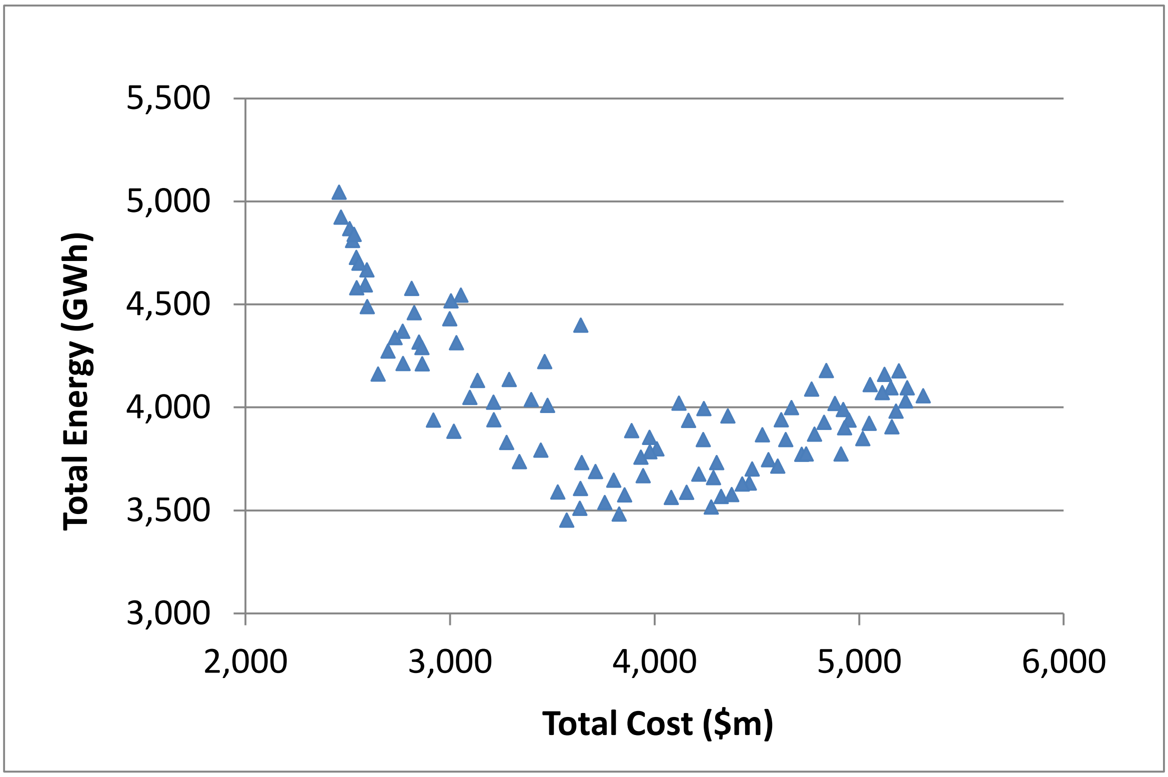


Figure 43: Results of costs and energies for the 2013 scenario with Priority Set #1 and seed 0.123

It can be seen in Figure 43 that there is a trade-off between cost and energy consumption; cheaper solutions have significantly high energy consumption as they save money on the capital costs of new infrastructure. As the capital cost increases, the energy consumption decreases, but, after a certain point the energy consumption increases again. This is probably due to increasing use of harvested stormwater and treated wastewater that have higher energy values (partly due to embodied energy in their distribution systems) as shown in Figure 45, in which it is clearly shown that lowering the discharges to the Gulf increases the total cost of the solutions.

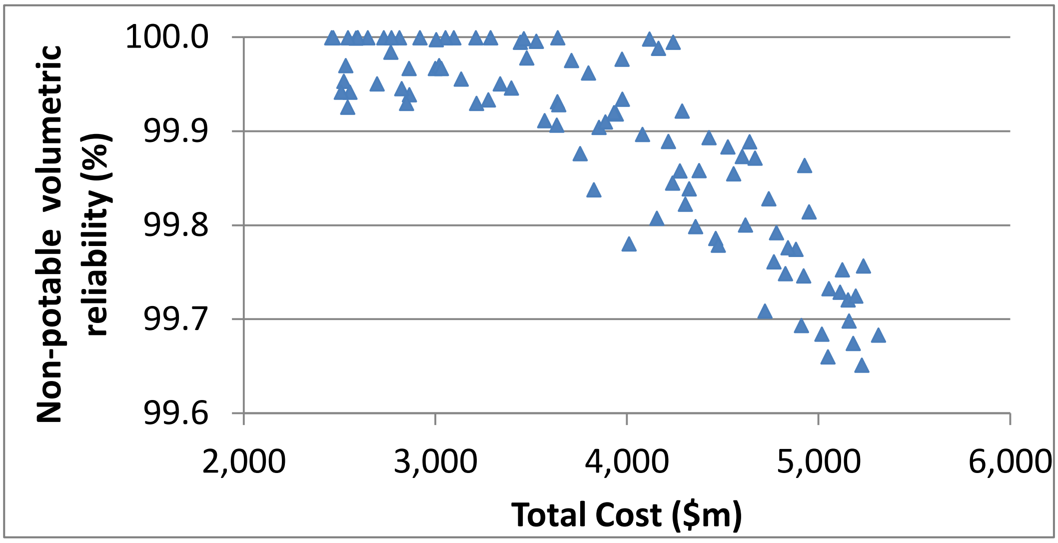


Figure 44: Results of costs and non-potable volumetric reliability for the 2013 scenario with Priority Set #1 and seed 0.123

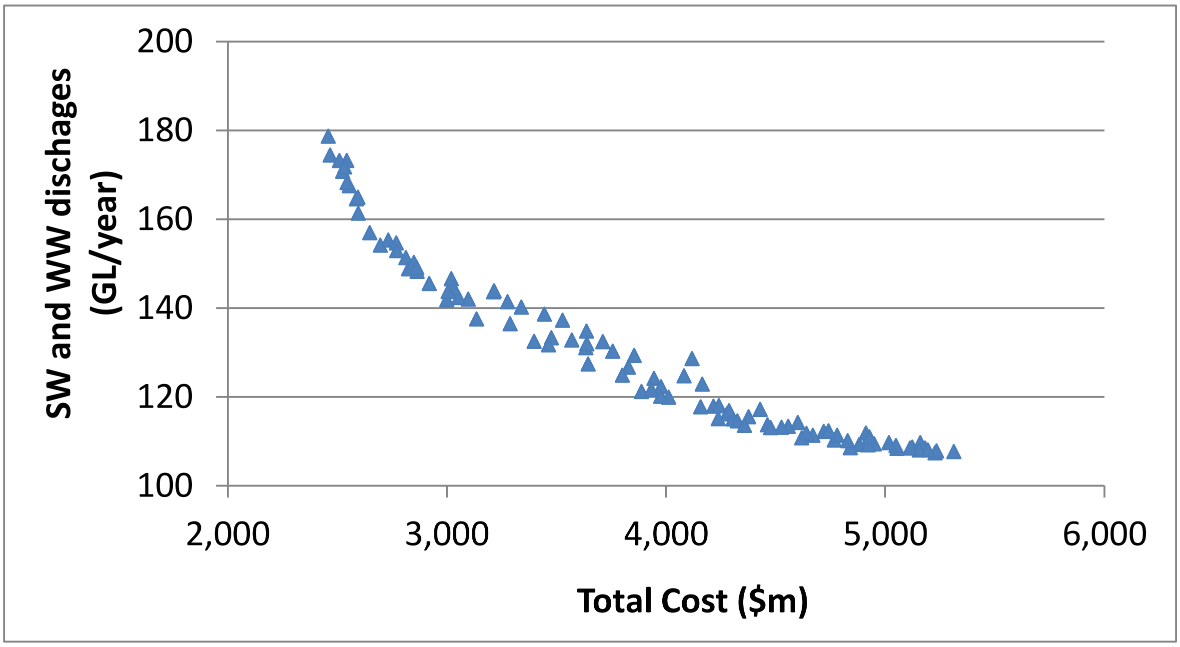


Figure 45: Results of costs and total stormwater and wastewater discharges for the 2013 scenario with Priority Set #1 and seed 0.123

Figure 44 indicates that the volumetric reliability of non-potable water is consistently high (between 99.65% - 100%). The total cost of the solutions with high volumetric reliability for non-potable water vary, approximately from 2.5 $m to 5.5 $m. This is because the higher cost solutions use more stormwater which is slightly less reliable. However, such solutions tend to provide higher reduction in discharges to the Gulf, compared to the lower cost solutions.

Optimisation runs have been carried out for Priority Sets #1 and #2 with two different random seeds. Figure 46 shows the effect of using different random seeds (i.e. 0.123 and 0.147) and different priorities on the final results for the 2013 scenario: it can be seen that, with all other parameters equal, using a different seed leads to different final solutions, as the optimisation algorithm may not have fully converged. However, for Priority Set #1 there is reasonable agreement between the points on the two Pareto fronts. Likewise for Priority Set #2 there is reasonable agreement between the two sets of results, although there appears to be more of a scatter of solutions in this case. It can also be seen that the priorities chosen for the different sources pushes the algorithm search towards specific regions of the search space. In Figure 46, many solutions found using Priority Set #1 have lower energy consumption than the solutions with Priority Set #2.

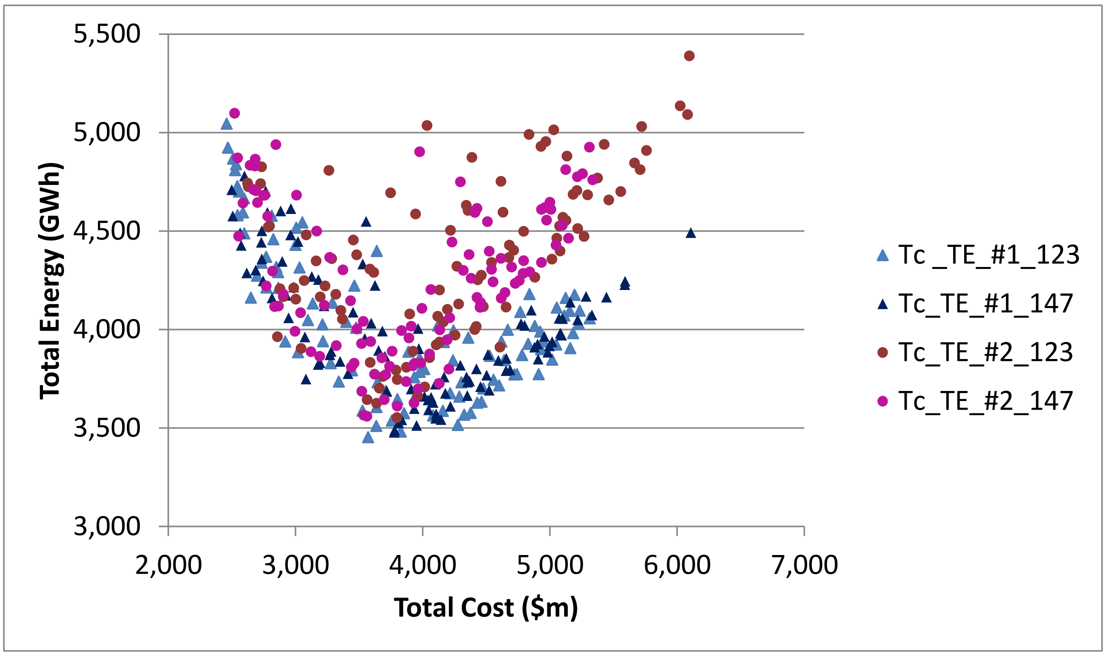


Figure 46: Results of costs and energies of 2 different seeds (0.123 and 0.147) for the 2013 scenario with Priority Sets #1 and #2, where Tc = total cost, and TE = total energy

Note that all the solutions found comply with the constraint on the time based reliability of potable demand (>99.5%) in this case and also for the other scenarios. In Figure 46 all of the 400 solutions are represented. However, some of them are dominated (e.g. there are solutions that are worse in all four objectives). As dominated solutions are not optimal solutions, they have been excluded from further analysis. Note that, as the mix of water resources of the solutions may be important for the decision makers, Appendix 3 shows the results of the non-dominated solutions within each model priority.

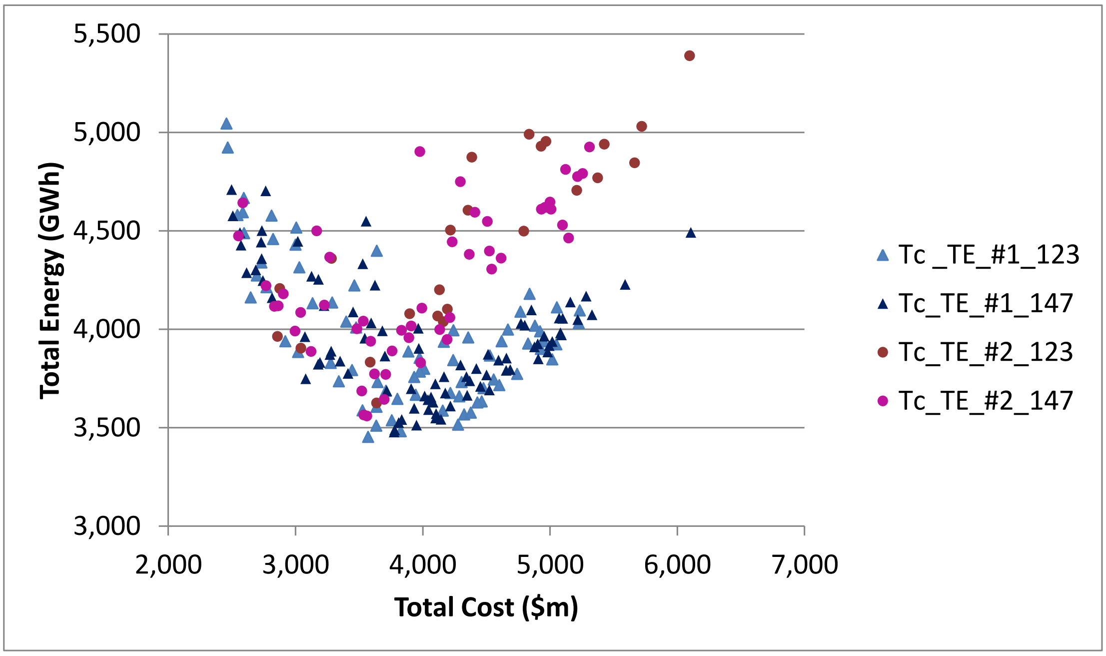


Figure 47: Costs and energies of non-dominated solutions using 2 different seeds (0.123 and 0.147) for the 2013 scenario with Priority Sets #1 and #2, where Tc = total cost and TE = total energy

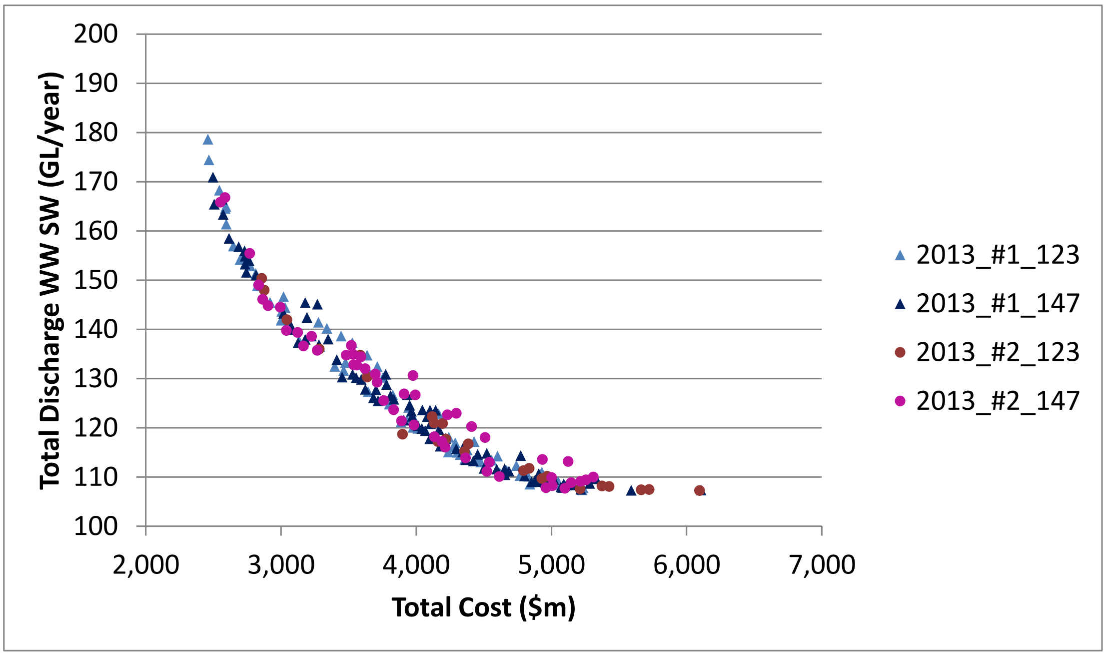


Figure 48: Costs and discharges of non-dominated solutions using 2 different seeds (0.123 and 0.147) for the 2013 scenario with Priority Sets #1 and #2

The non-dominated solutions are presented in Figure 47 to Figure 59. Figure 47 shows the 233 non-dominated solutions among the 400 solutions obtained in terms of total cost and energy. Note that solutions presented in this figure may have a larger cost and energy consumption than others, but they are still non-dominated because they may have a lower discharge of wastewater and stormwater into the Gulf and/or a larger non-potable volumetric reliability.

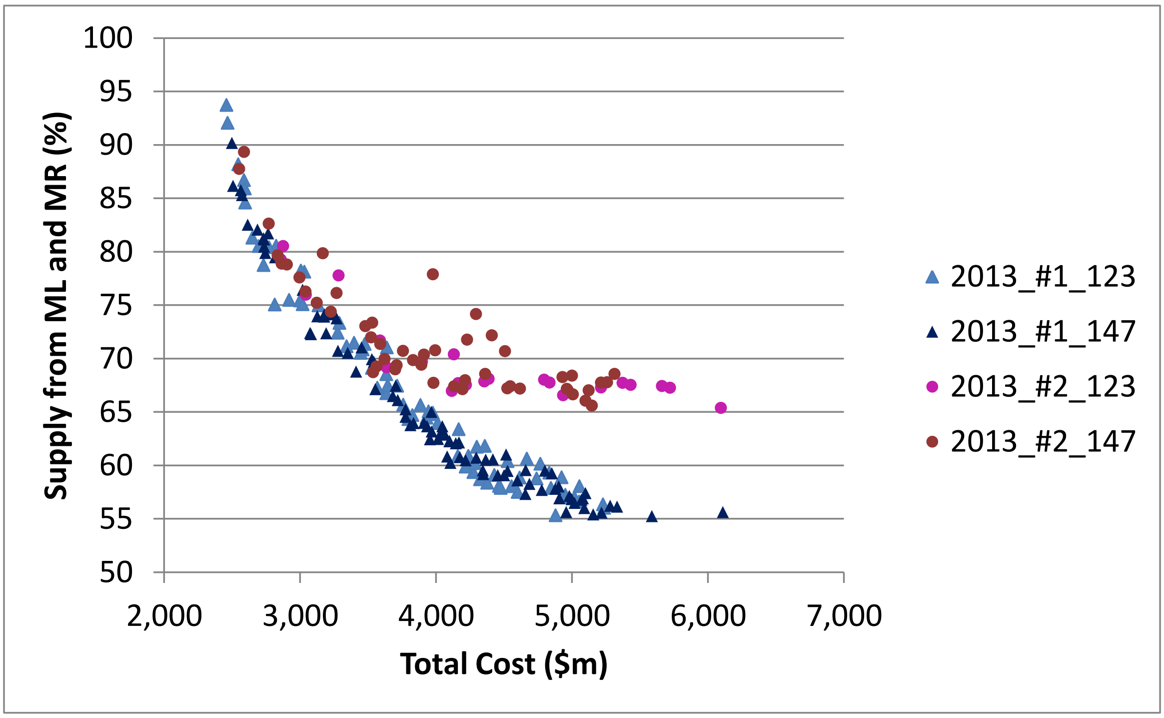


Figure 49: Costs and supply from Mount Lofty (ML) and Murray River (MR) of non-dominated solutions using 2 different seeds (0.123 and 0.147) for the 2013 scenario with Priority Sets #1 and #2

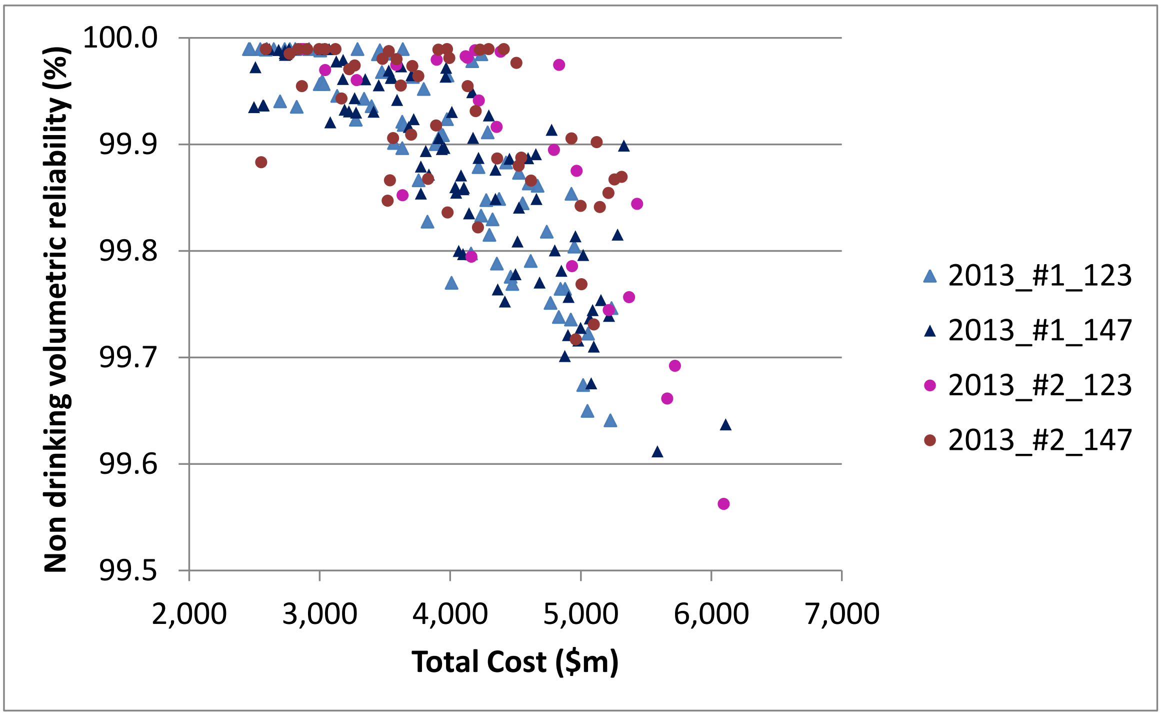


Figure 50: Costs and non-potable volumetric reliability of non-dominated solutions using 2 different seeds (0.123 and 0.147) for the 2013 scenario with Priority Sets #1 and #2

As expected, there is a trade-off between the total cost of the solutions and the discharge of wastewater and stormwater into the Gulf (Figure 48), as less expensive solutions are characterised by larger discharges. In general, as withdrawing water from Mount Lofty Catchments and from the River Murray is less expensive than recycling stormwater and wastewater, the cheapest solutions exploit the first two sources (Figure 49). Note that Figure 49 clearly shows the impact of the priorities during the optimisation, as with the Priority Set #2, solutions use more water from the Mount Lofty Ranges and the River Murray than with the Priority Set #1, particularly on the right hand side of the graph, where the most expensive solutions are plotted.

The non-potable volumetric reliability of the solutions is usually high (Figure 50).

The solutions found using Priority Set #3 are similar to the solutions found with the other two priority sets in terms of objectives (total cost, energy consumption, non-potable volumetric reliability and stormwater and wastewater discharges) for the three scenarios considered. Hence further analysis of the solutions will be carried out using Priority Sets #1 and #2 only. Results for the 2013 scenario using the Priority Set #3 can be found in Appendix 3.

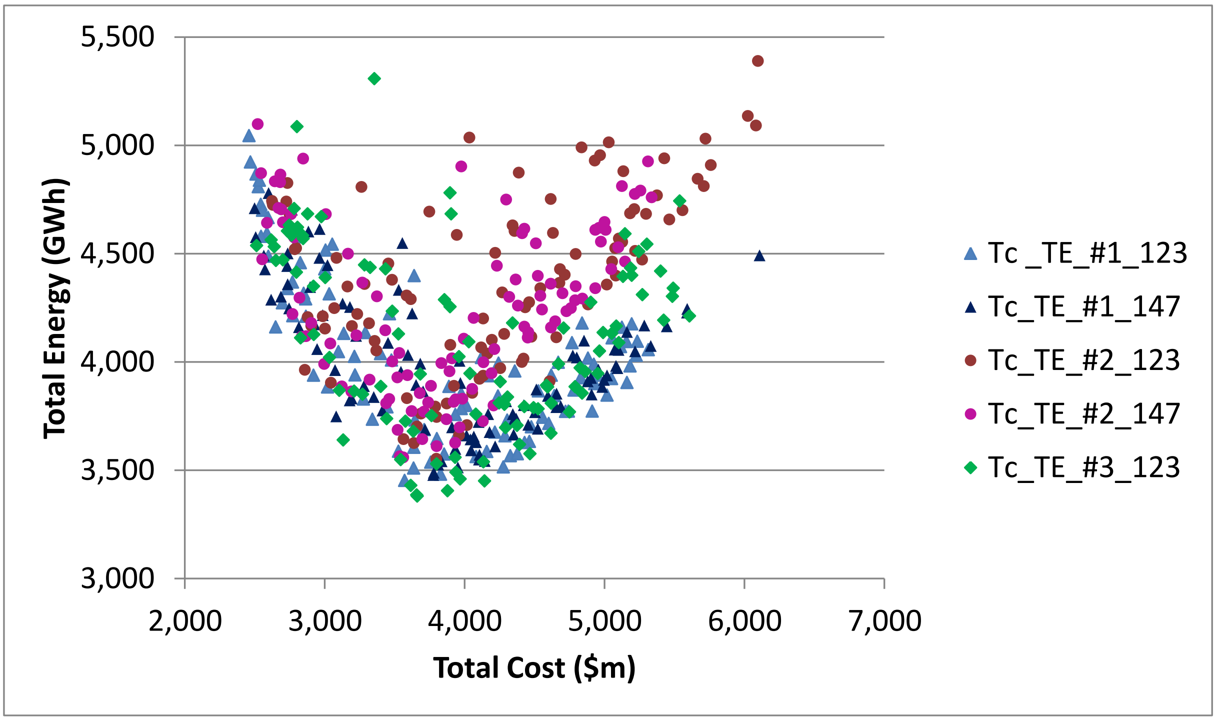


Figure 51: Total costs and total energy of the solutions found for the 2013 scenario with Priority Sets #1, #2 and #3, where Tc = total cost and TE = total energy

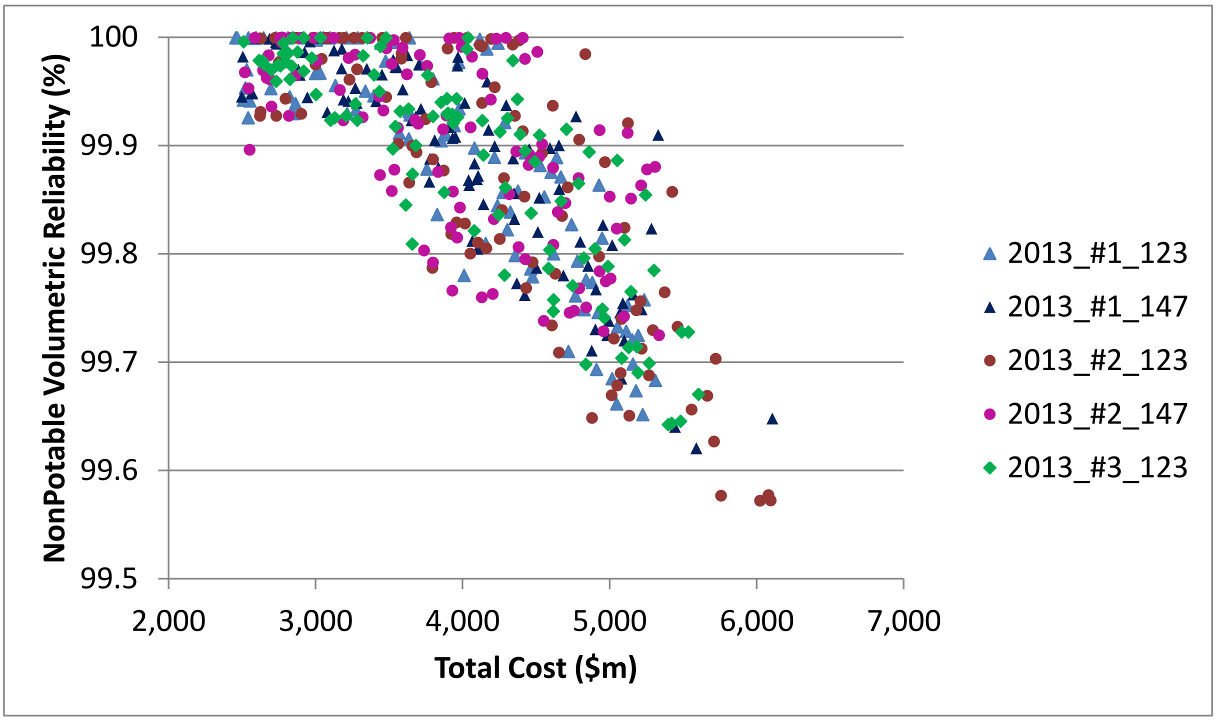


Figure 52: Total costs and volumetric reliability of the solutions found for the 2013 scenario with Priority Sets #1, #2 and #3

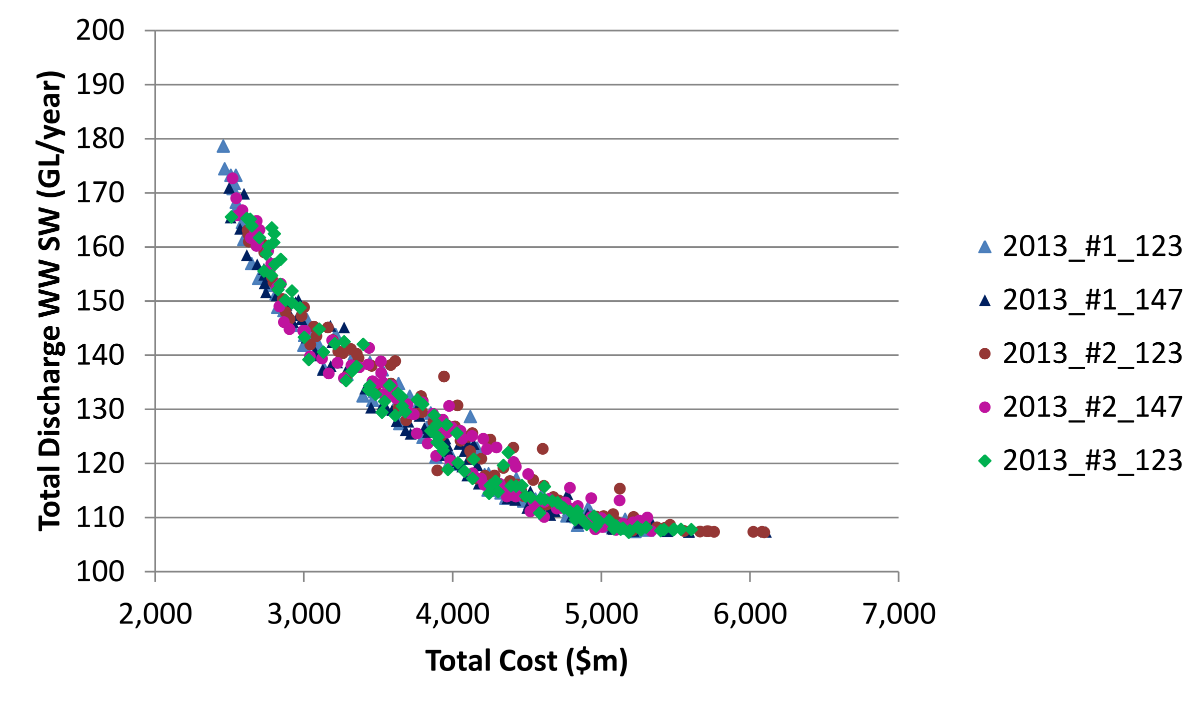


Figure 53: Total costs and total discharges of the solutions found for the 2013 scenario with Priority Sets #1, #2 and #3

Figure 54 shows the total capital and operational costs for the non-dominated solutions for Priority Sets #1 and #2. The solution numbers are set so that solution number 1 has the lowest present value of total cost and solution number 233 has the highest present value of total cost. It can be seen that the total operational costs for Priority Sets #1 and #2 are relatively constant (Figure 54), while total capital costs can vary from about zero to about $3500m for the 2013 scenario and therefore have a large influence on the total cost.

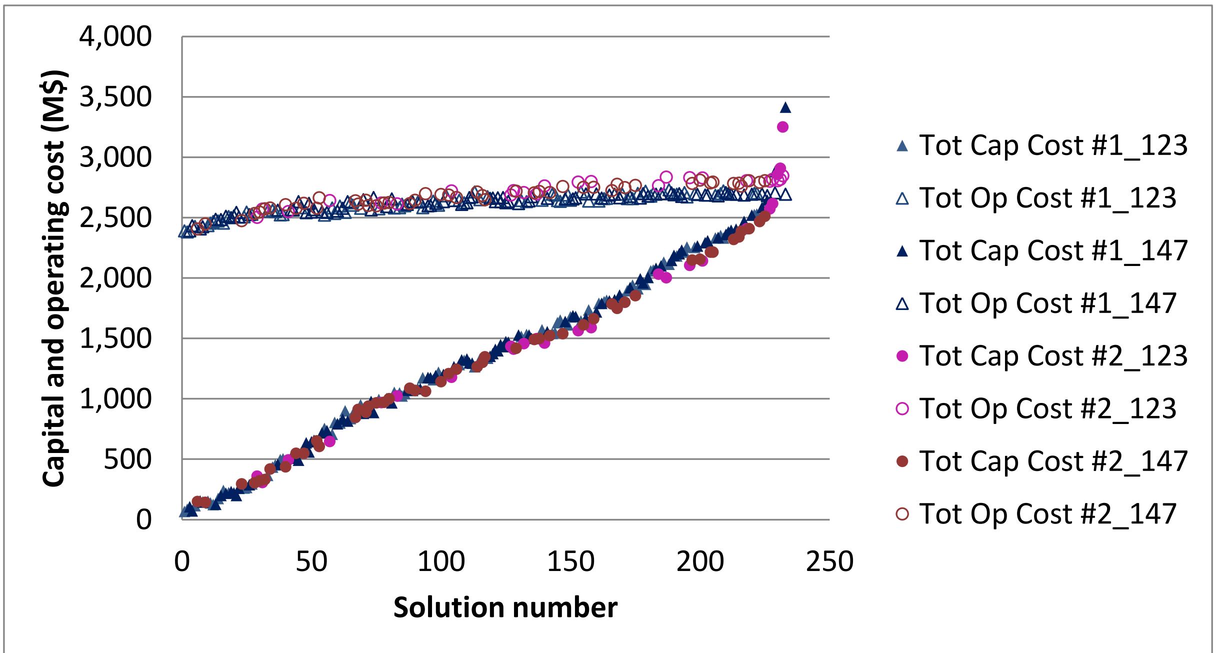


Figure 54: Capital and operational costs of non-dominated solutions using 2 different seeds (0.123 and 0.147) for the 2013 scenario with Priority Sets #1 and #2

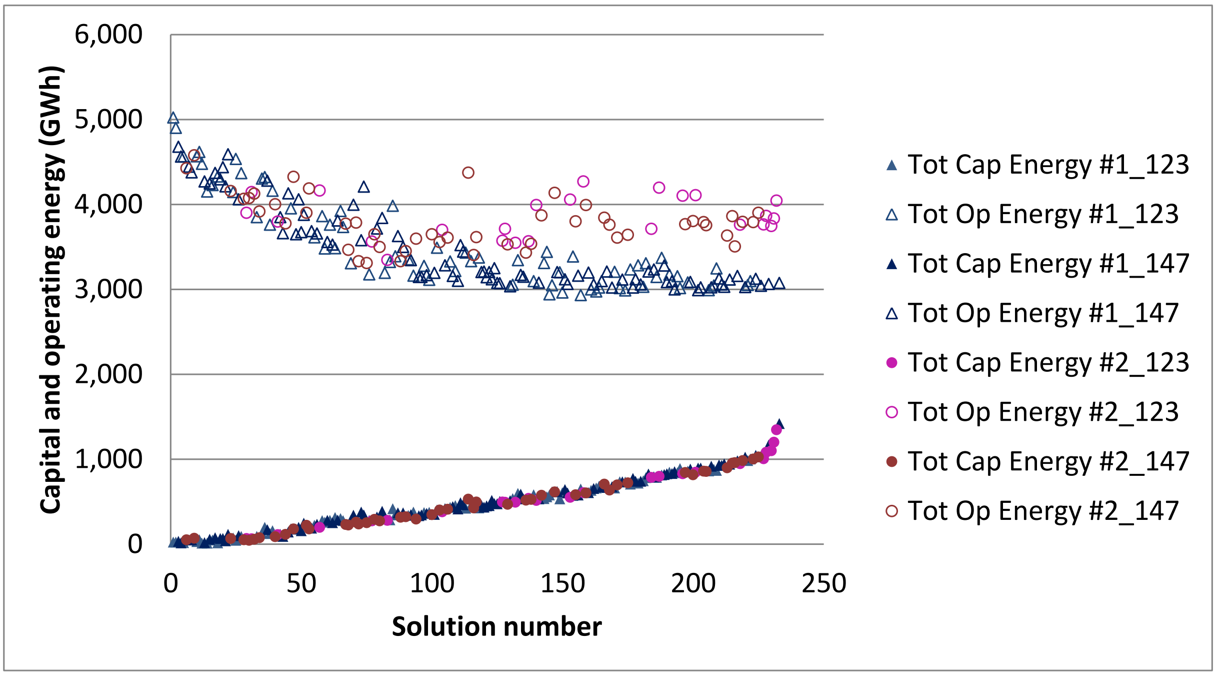


Figure 55: Capital and operational energy of non-dominated solutions using 2 different seeds (0.123 and 0.147) for the 2013 scenario with Priority Sets #1 and #2

Figure 55 shows the capital and operational energy for the non-dominated solutions with Priority Sets #1 and #2. As expected, the total energy is mostly operating energy: the reason is that most sources do not require an upgrade (e.g. Mount Lofty Ranges infrastructure), but also because of the limitations in computing the embodied energy (for example, the embodied energy for pump replacement has not been considered).

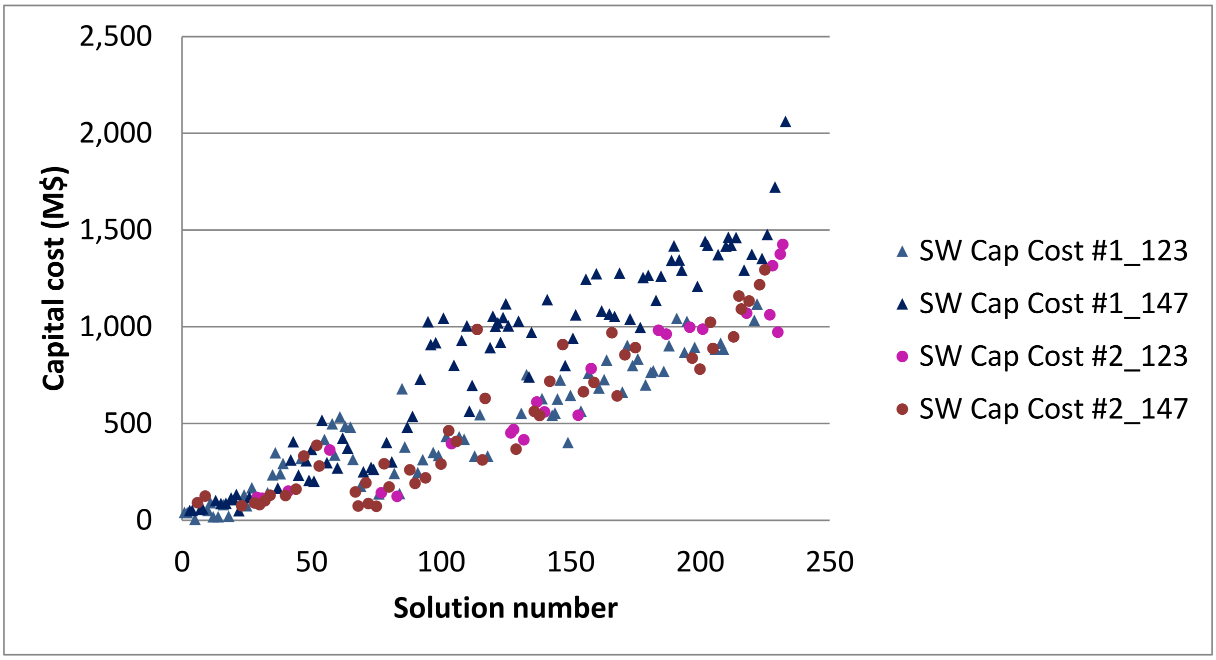


Figure 56: Capital cost of stormwater (SW) of non-dominated solutions using 2 different seeds (0.123 and 0.147) for the 2013 scenario with Priority Sets #1 and #2

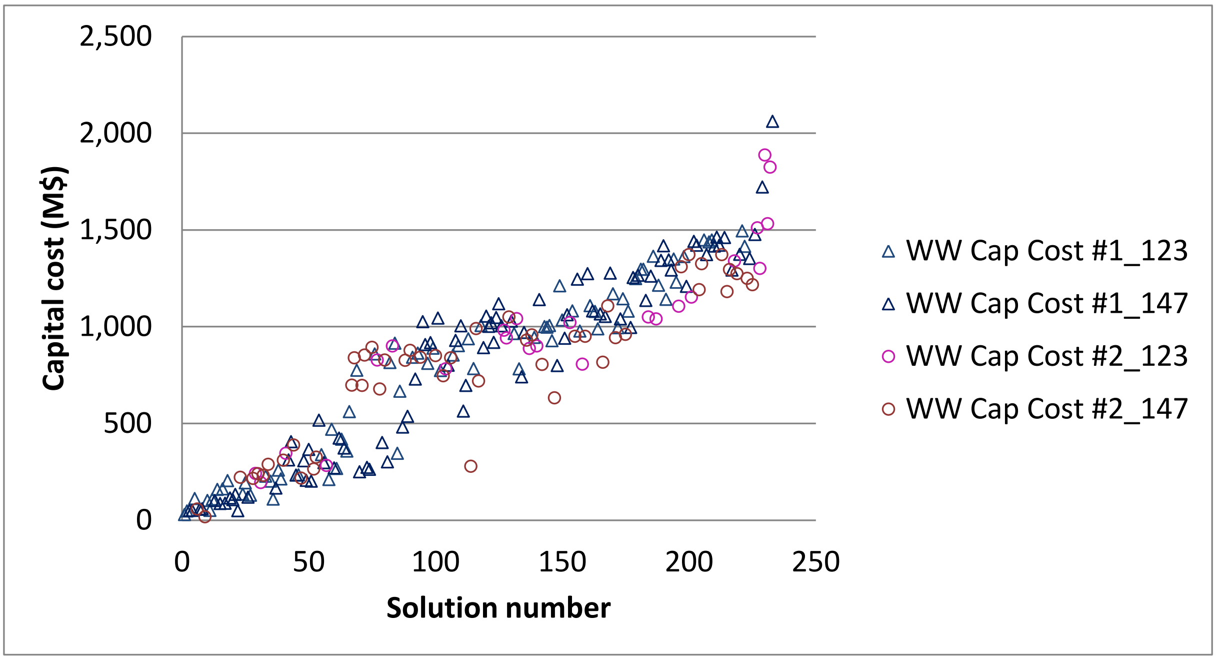


Figure 57: Capital cost of wastewater (WW) of non-dominated solutions using 2 different seeds (0.123 and 0.147) for the 2013 scenario with Priority Sets #1 and #2

Figure 56 and Figure 57 show the capital costs of stormwater (SW) and wastewater (WW) for the non-dominated solutions with Priority Sets #1 and #2.The capital cost of wastewater (WW) and stormwater (SW) are comparable, with the costs associated to the wastewater recycling plant being slightly larger than the capital costs of stormwater (Figure 56 and Figure 57). The fact that the algorithm favours the use of recycled wastewater is caused by the different operational costs and by the different seasonal availability of the two sources.

Figure 58 shows the operational costs of the various sources for the non-dominated solutions with Priority Sets #1 and #2. The solutions highlight that that the largest part of the operational costs is due to the wastewater treatment. This includes the normal treatment before the discharge and some recycling. This is followed by the operational costs of the Adelaide Desalination plant (ADP) (note that it has been assumed that the ADP maintenance and operational costs are $30m/year even when the plant is out of operation), followed by Mount Lofty Catchments, Murray River and stormwater operational costs.

Note that the solutions with the lowest total costs (the ones presented on the left hand side of the graph) use more River Murray water. The most expensive solutions replace part of the River Murray water with stormwater and wastewater to improve the environmental objective.

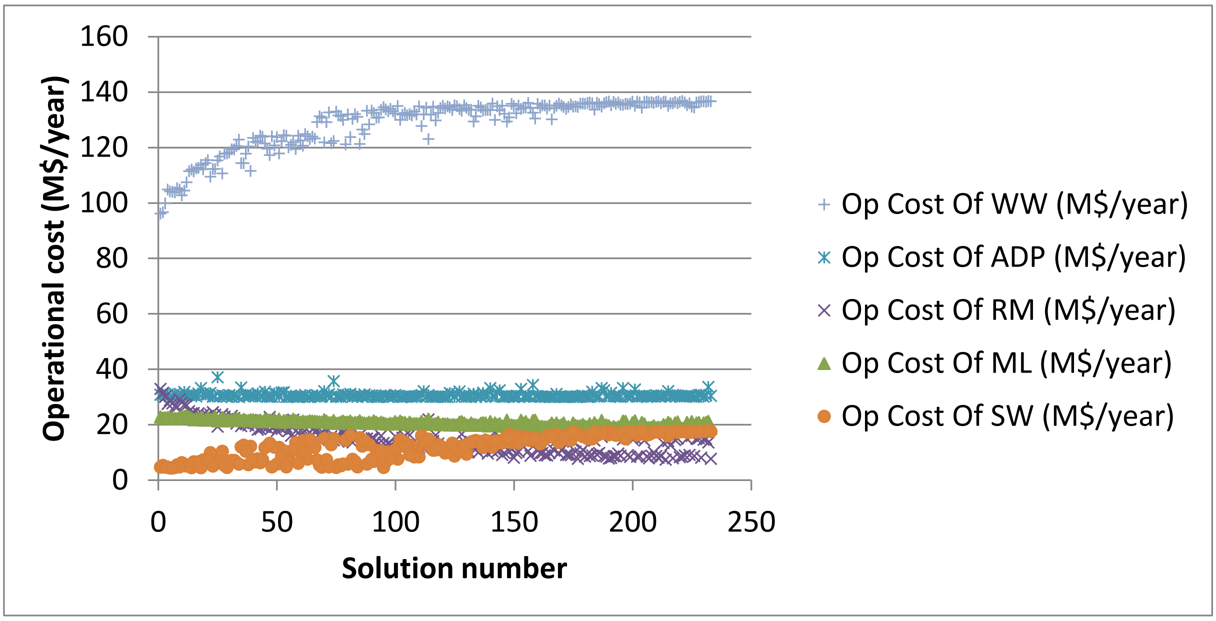


Figure 58: Operational costs of the various sources: Mount Lofty (ML), Murray River (MR), Adelaide Desalination plant (ADP), stormwater (SW) and wastewater (WW) of non-dominated solutions using 2 different seeds (0.123 and 0.147) for the 2013 scenario with Priority Sets #1 and #2

As can be seen in Figure 59, the Pareto optimal solutions supply about 50% of the 2013 demand using water from the Mount Lofty Ranges, as this is the cheapest source, then about 10–40% is supplied using the River Murray (the second cheapest source) and recycled wastewater. The maximum use of stormwater is about 10% while the desalination plant is used on only a few occasions to make up for the potable demand in drought conditions. It should be noted that many solutions have a large use of wastewater and stormwater instead of the cheaper River Murray water, because using wastewater and stormwater reduces the discharge to the Gulf.

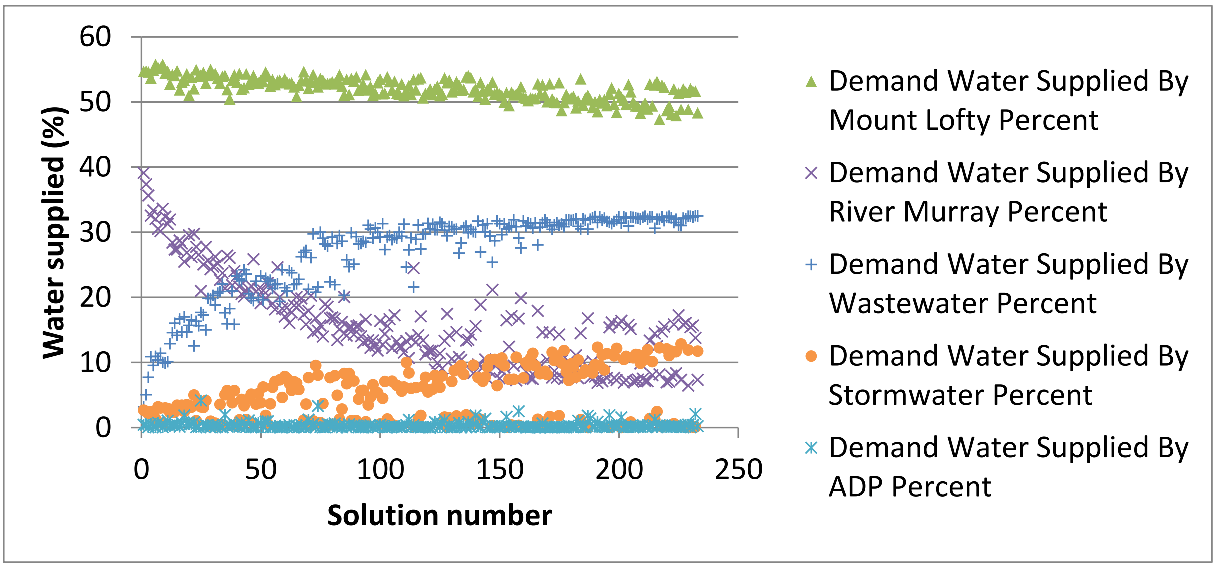


Figure 59: Water supplied by the various sources: Mount Lofty (ML), Murray River (MR), Adelaide Desalination plant (ADP), stormwater (SW) and wastewater (WW) of non-dominated solutions using 2 different seeds (0.123 and 0.147) for the 2013 scenario with Priority Sets #1 and #2

5.1.2 Optimal solutions for 2050 scenario

Trends similar to the ones presented above were found also for the 2025 and 2050 scenarios. The results for the 2025 scenario can be found in Appendix 9 while Figure 60 to Figure 72 shows the Pareto front for 2050 scenario.

It should be noted that the 2013, 2025 and 2050 scenarios were optimised separately. Hence the costs and energy values presented for the 2025 and 2050 scenarios are not incremental to the 2013 scenario or to each other but are for the total infrastructure required to meet the 2025 and 2050 demands (respectively).

Figure 60 shows that the energy consumption and cost for the 2050 scenario will be larger than the 2013 scenario, because of the larger demand. While for the 2013 case the solutions had a total cost in the range \$2500m–\$6500m and a total energy in the range 3500–5500 GWh, for the 2050 scenario solutions have a total cost in the range \$3000m–\$7000m and a total energy in the range 5500–8000 GWh. As shown in Figure 61 and Figure 62, respectively, the volumetric reliability of non-potable water is still high and the discharges of wastewater and stormwater are almost in the same range as in the 2013 scenario.

Figure 63 to Figure 65 include Priority Sets #1, #2 and #3 and cover the same range of total cost, total energy, non-potable volumetric reliability and total discharge to the Gulf as when only Priority Sets #1 and #2 were considered. For the total cost and energy analysis only Priority Sets #1 and #2 will be used. Additional results for the 2050 scenario using the Priority Set #3 can be found in Appendix 6.

The capital costs of the non-dominated solutions are similar to the 2013 scenario (about \$3500m) (Figure 66), although a larger capital cost may be expected for the 2050 scenario. Note that this result is probably due to the fact that it is better to increase the operational costs (now the present value of the operational cost is about \$500m larger than the 2013 scenario) than building new infrastructure.

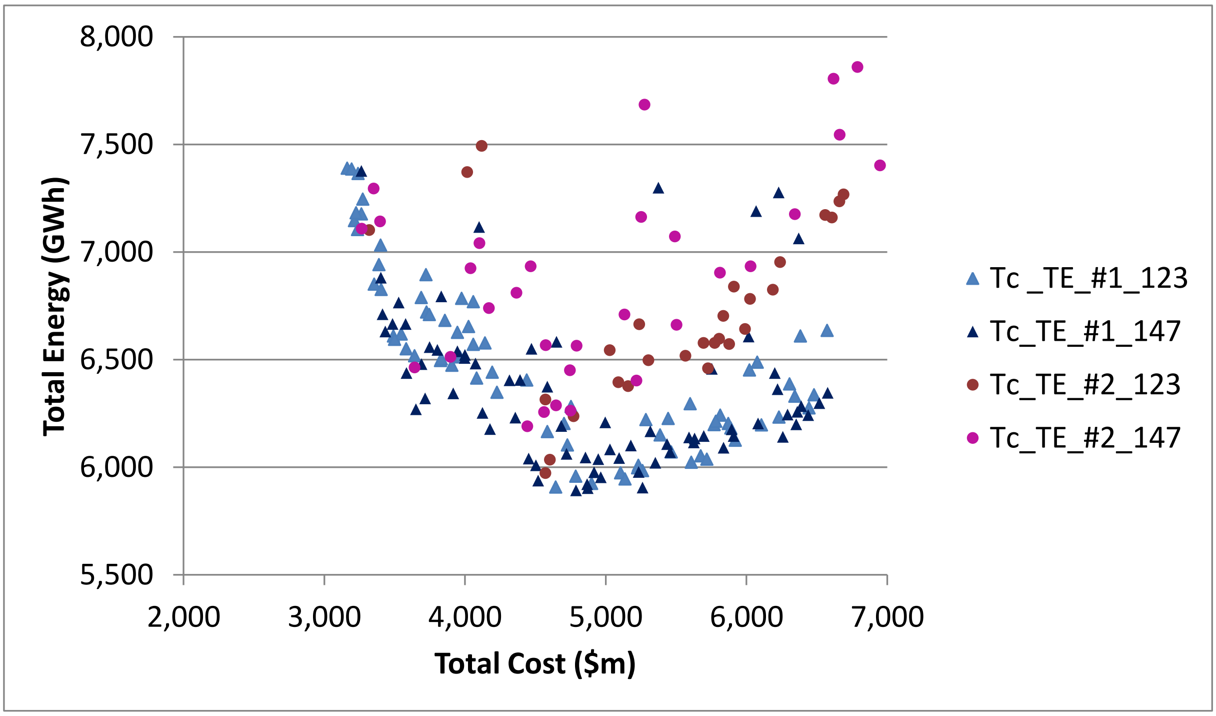


Figure 60: Costs and energies of non-dominated solutions using 2 different seeds (0.123 and 0.147) for the 2050 scenario with Priority Sets #1 and #2

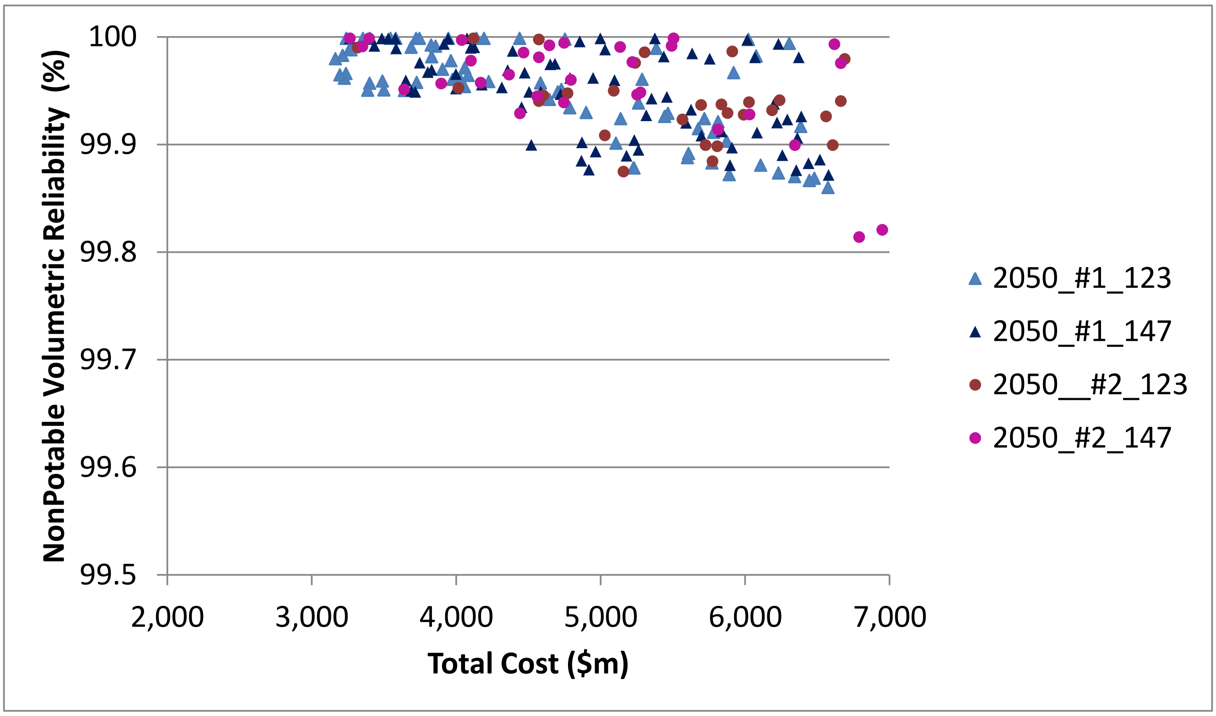


Figure 61: Costs and non-potable volumetric reliability of non-dominated solutions using 2 different seeds (0.123 and 0.147) for the 2050 scenario with Priority Sets #1 and #2

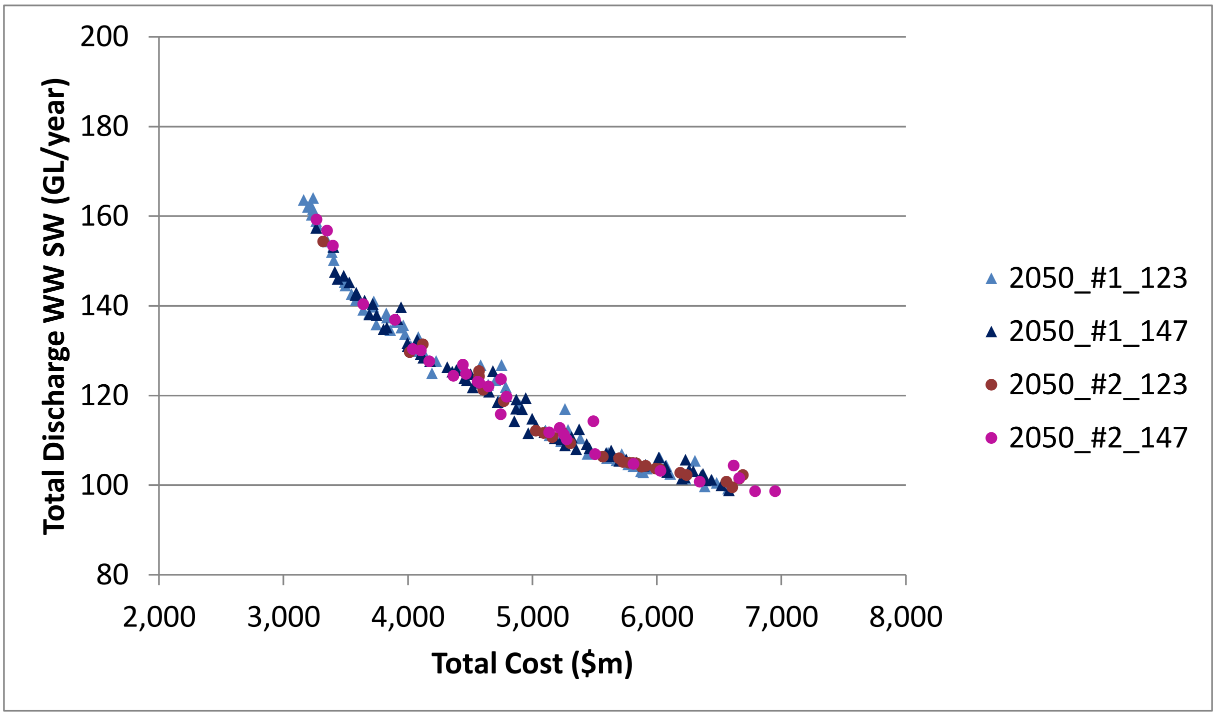


Figure 62: Costs and discharges of non-dominated solutions using 2 different seeds (0.123 and 0.147) for the 2050 scenario with Priority Sets #1 and #2

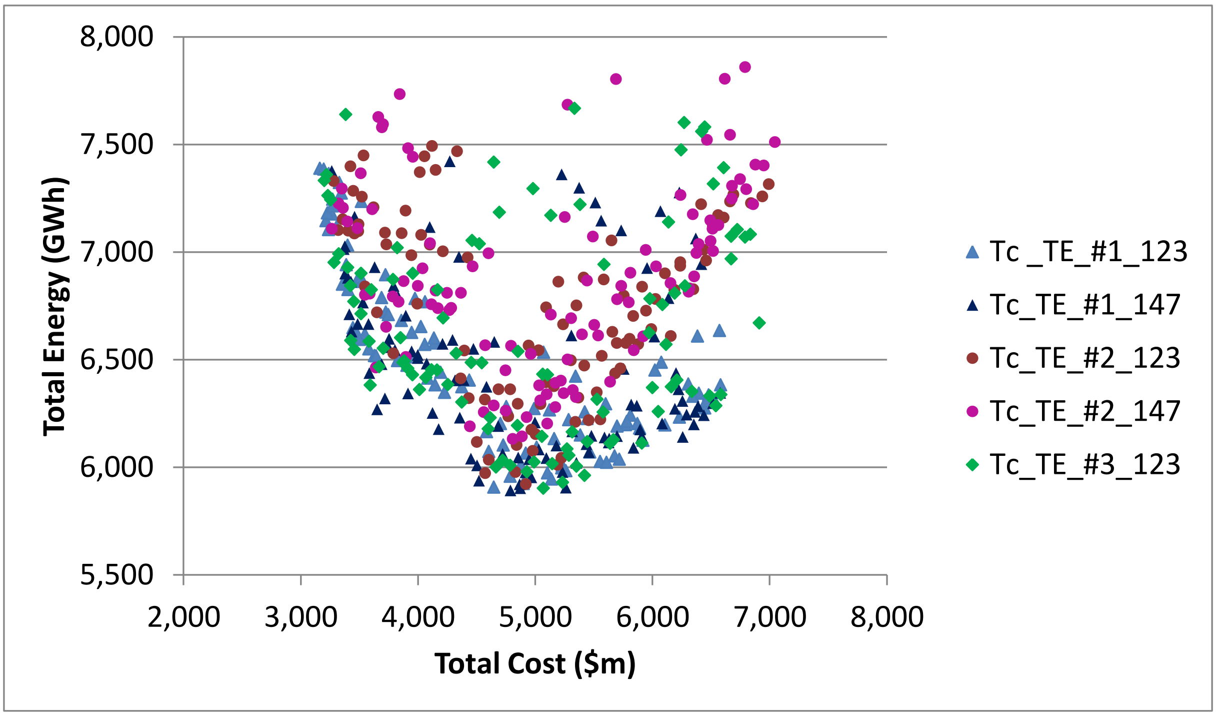


Figure 63: Total costs and total energy of the solutions found for the 2050 scenario with Priority Sets #1, #2 and #3

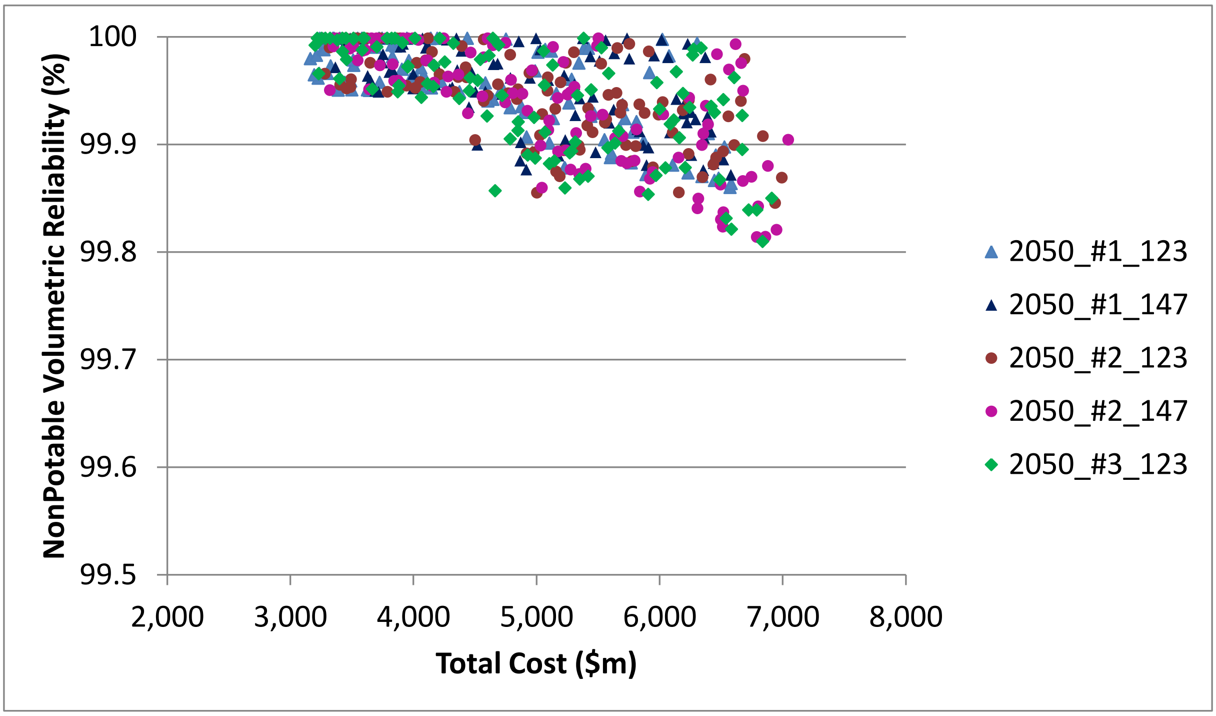


Figure 64: Total costs and volumetric reliability of the solutions found for the 2050 scenario with Priority Sets #1, #2 and #3

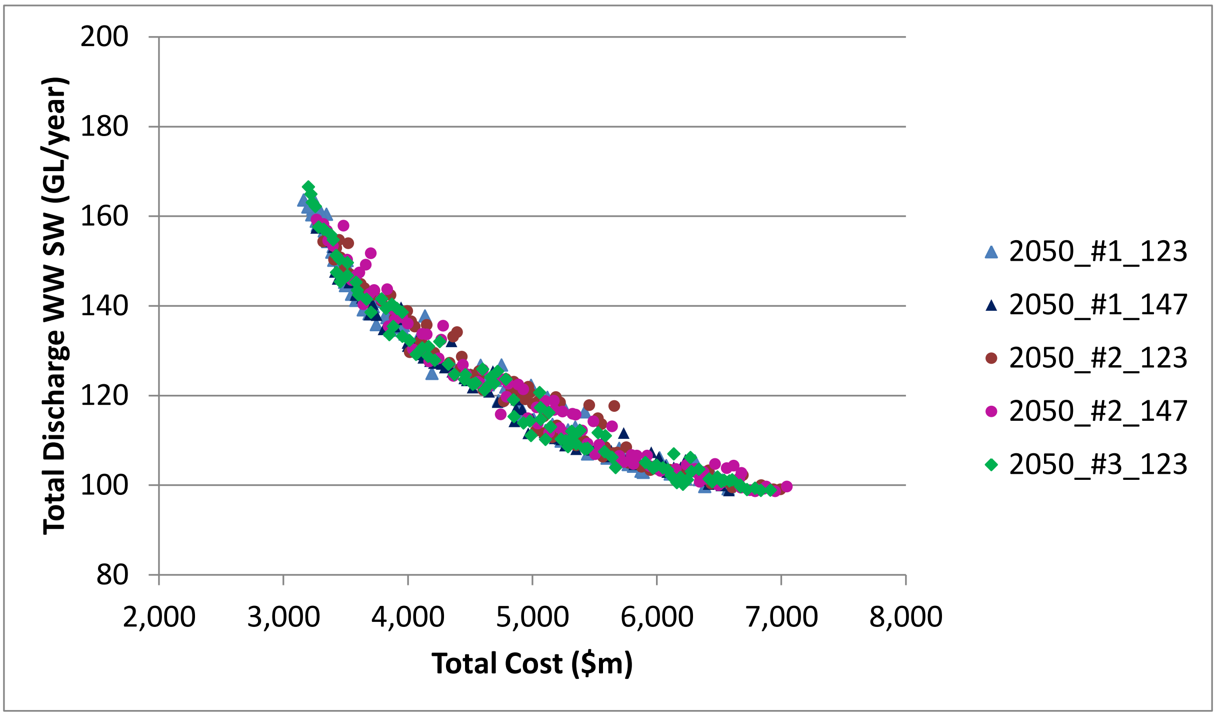


Figure 65: Total costs and total discharges of the solutions found for the 2050 scenario with Priority Sets #1, #2 and #3

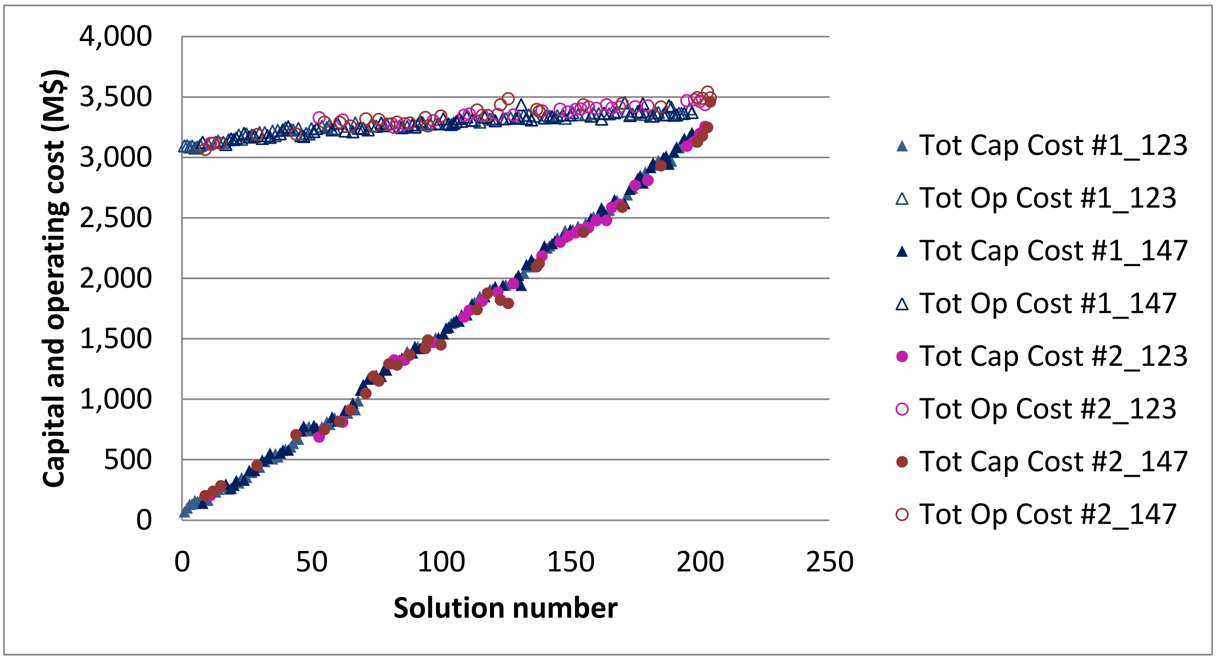


Figure 66: Capital and operational costs of non-dominated solutions using 2 different seeds (0.123 and 0.147) for the 2050 scenario with Priority Sets #1 and #2

The capital/embodied energy for the 2050 scenario is similar to 2013 case. However, the operational energy is almost doubled, moving from a range of 3000–5000 GWh to 5000–7000 GWh (Figure 67).

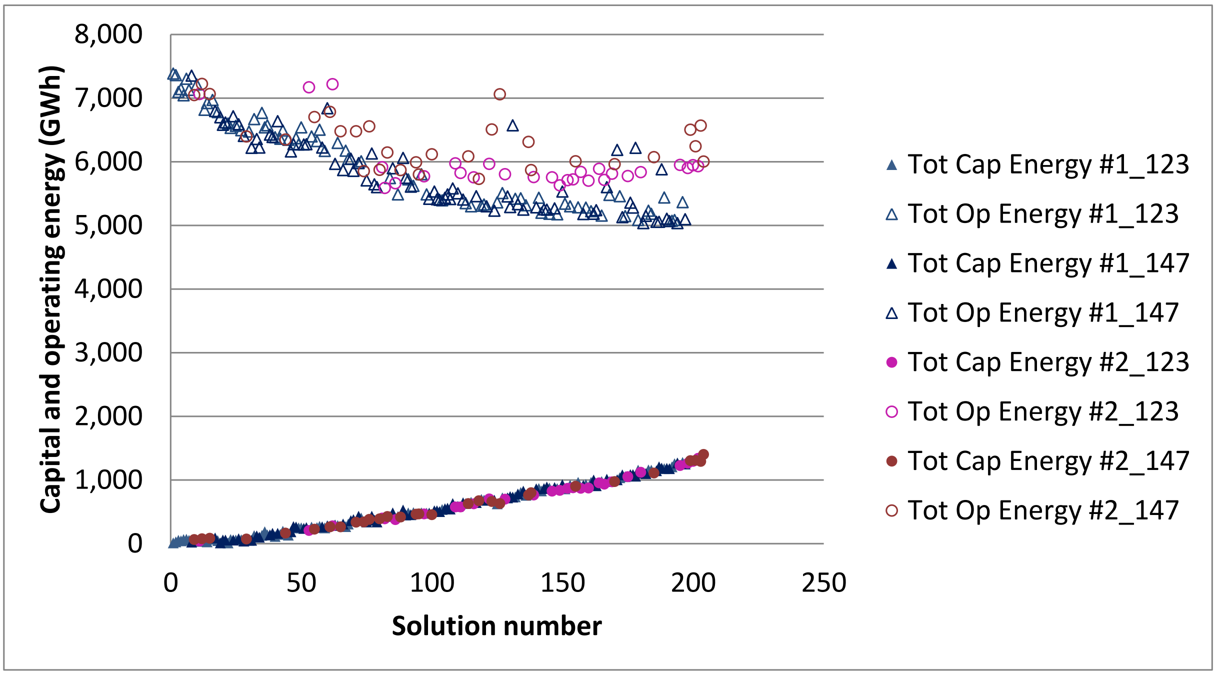


Figure 67: Capital and operational energy of non-dominated solutions using 2 different seeds (0.123 and 0.147) for the 2050 scenario with Priority Sets #1 and #2

The capital investments in stormwater and wastewater reuse for the 2050 scenario are similar to the 2013 case, but, as shown in Figure 68 and Figure 69, more solutions have a clear preference for recycled wastewater. This can be due to the limitations of the model, but also to the fact that the stormwater is available only for part of the year, as all stormwater schemes are assumed to involve ASR. It is likely that if other types of stormwater reuse schemes are considered, a different result would be obtained.

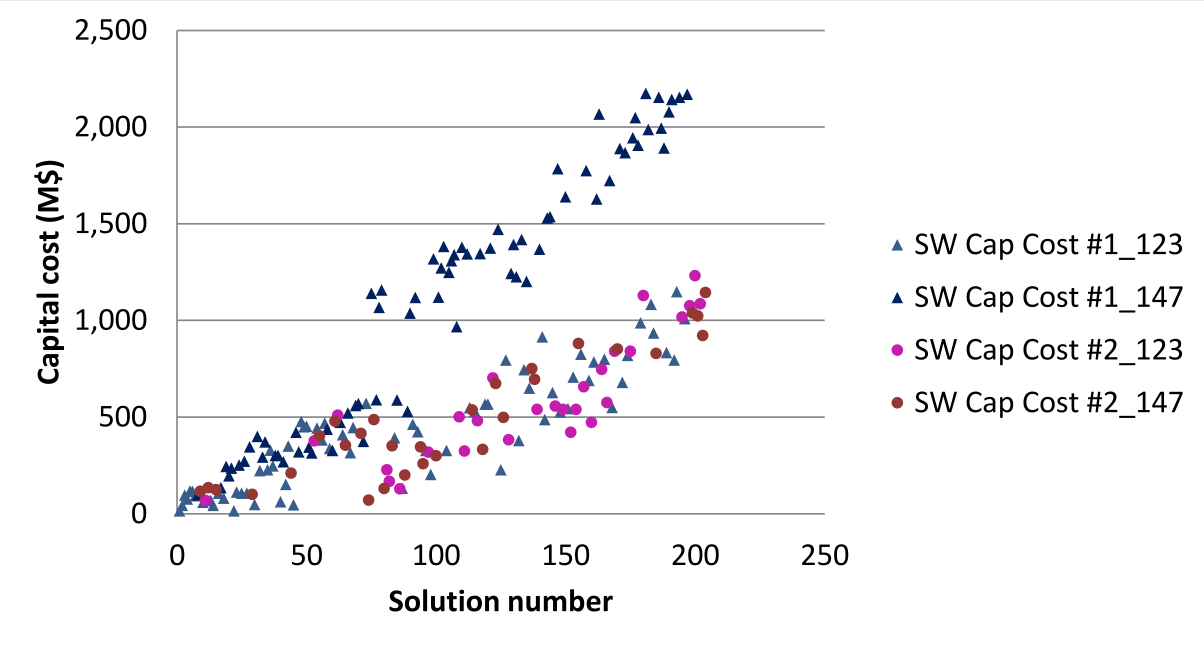


Figure 68: Capital cost of stormwater (SW) of non-dominated solutions using 2 different seeds (0.123 and 0.147) for the 2050 scenario with Priority Sets #1 and #2

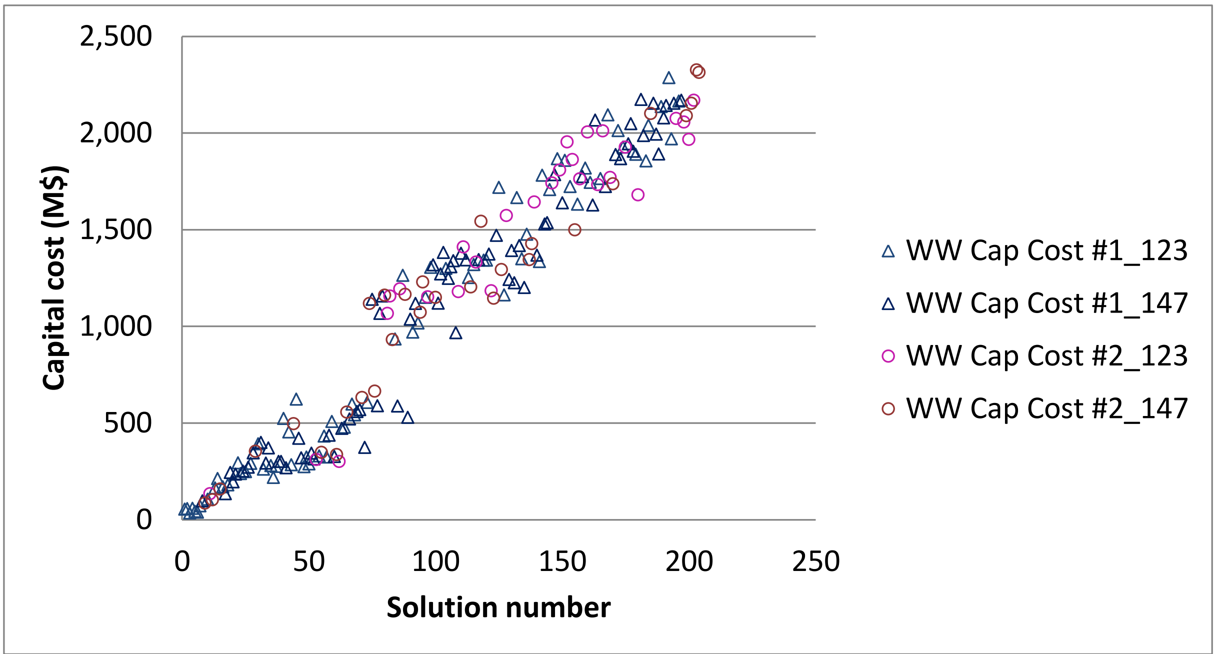


Figure 69: Capital cost of wastewater (WW) of non-dominated solutions using 2 different seeds (0.123 and 0.147) for the 2050 scenario with Priority Sets #1 and #2

Figure 70 shows the operational costs divided by source for 2050. As expected, all costs are larger, but the cheapest solutions still prefer the use of Mount Lofty and Rive Murray water. It should also be noted that the Adelaide desalination plant is used more often than in the 2013 scenario as shown also in Figure 71.

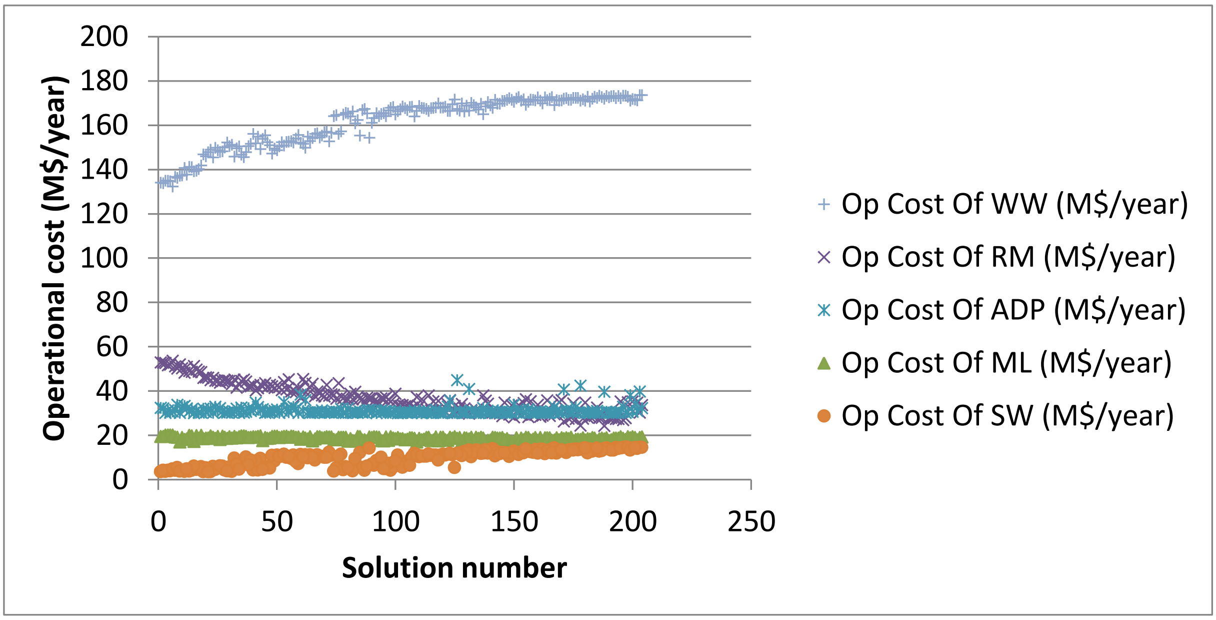


Figure 70: Operational costs of the various sources: Mount Lofty (ML), Murray River (MR), Adelaide Desalination plant (ADP), stormwater (SW) and wastewater (WW) of non-dominated solutions using 2 different seeds (0.123 and 0.147) for the 2050 scenario with Priority Sets #1 and #2

Figure 71 shows that the cheapest solutions made a larger use of Murray River water (now increased up to 50% compared to the 40% of 2013 scenario) and that the percentage of supply from Mount Lofty is almost constant at about 35%. In this case, it is likely that this percentage is the proportion of demand that can be covered by this supply source: because of the increased demand, the maximum supply percentage from Mount Lofty decreases from 50% to about 35%.

Figure 72 shows that also for this scenario the choice of the priorities influences the exploration of the optimisation algorithm, especially in the region where solutions are more expensive: in this case, the model with the highest priority to Mount Lofty and River Murray water will use these sources more.

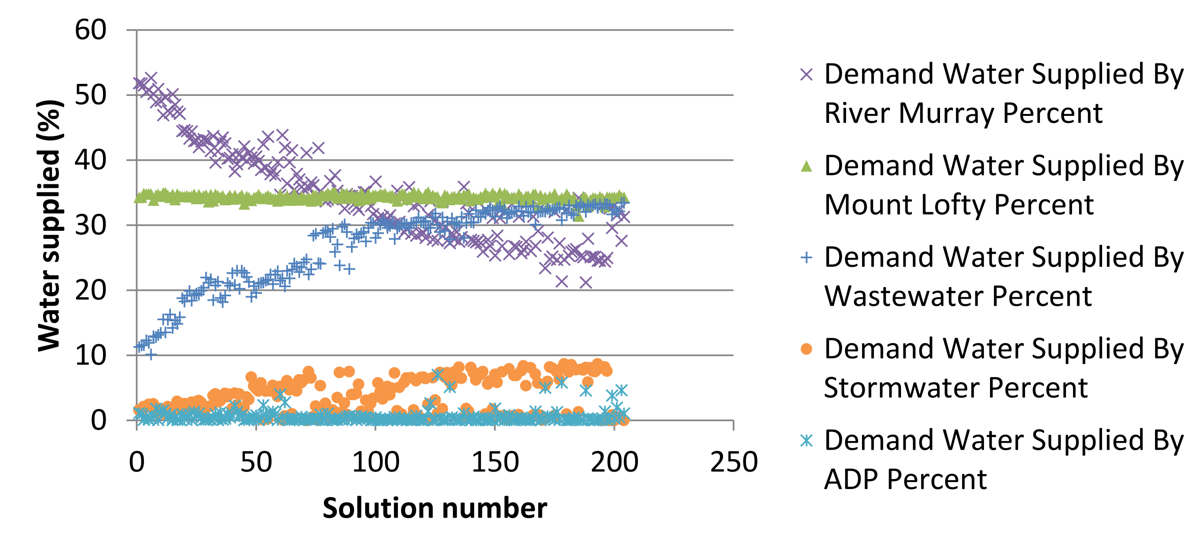


Figure 71: Water supplied by the various sources: Mount Lofty (ML), Murray River (MR), Adelaide Desalination plant (ADP), stormwater (SW) and wastewater (WW) of non-dominated solutions using 2 different seeds (0.123 and 0.147) for the 2050 scenario with Priority Sets #1 and #2

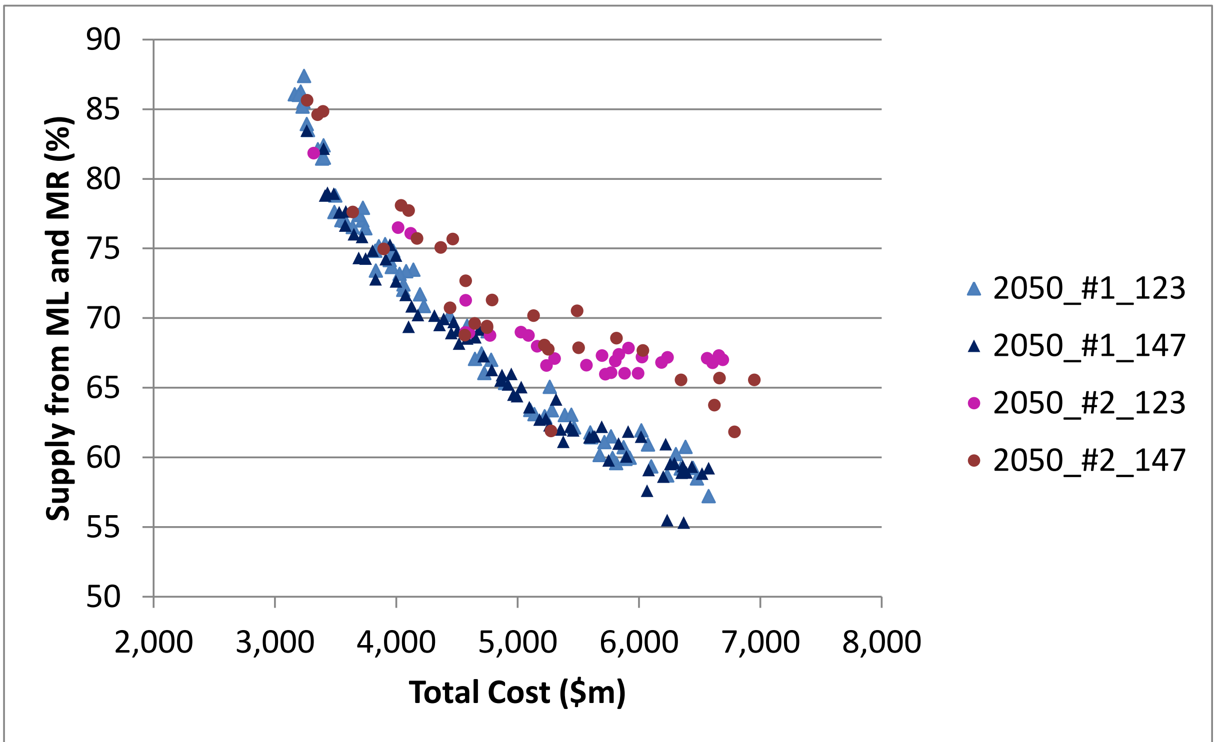


Figure 72: Costs and supply from Mount Lofty (ML) and Murray River (MR) of non-dominated solutions using 2 different seeds (0.123 and 0.147) for the 2050 scenario with Priority Sets #1 and #2

5.2 Preferred solutions

As all the solutions obtained are Pareto optimal, from a technical point of view additional information is needed in order to decide which solution represents the best compromise among the objectives. This is usually undertaken through a multi-criteria decision analysis

(MCA) process. As this study does not include a complete MCA, we used the technique of compromise programming and stakeholder preferences to identify preferred solutions.

5.2.1 Compromise programming

Compromise programming is an approach used to identify the best compromise among different objectives and has the advantage that it requires less information from the stakeholders. CP seeks the solution that has the minimum distance from an ideal solution (Ballestero, 2007): in our case, the ideal solution has simultaneously the minimum total cost, minimum operating energy, minimum discharge to the Gulf from stormwater and wastewater and the maximum non potable volumetric reliability. As this ideal solution cannot be achieved, a utility function that takes into account the distance of each objective from its optimum value is implemented. Note that these distances are normalised to avoid scaling problems, and that, in order to favour balance between the objectives, the value of *s* = 2 has been chosen. In the following equation it is assumed that all objectives are to be minimised.

$$Min\ D_i = \sum_k \alpha_k{}^2\left[\left(Z_{k,i} - Z_{k,min}\right)/\left(Z_{k,max} - Z_{k,min}\right)\right]^2$$

Equation 53

Where D_i is the distance of the solution *i* from the ideal solution, α_k is the relative weight of objective *k* (assumed to all equal to 1 in this work), $Z_{k,i}$ is the value of the objective *k* for solution *i*, and $Z_{k,min}$ and $Z_{k,max}$ are the minimum and maximum value of that objective.

Considering that energy consumption has the less immediate impact for the stakeholders, other solutions that compromise (i) only cost, non-potable volumetric reliability and stormwater and wastewater discharges to the Gulf; (ii) cost and non-potable volumetric reliability and (iii) cost and discharges have been evaluated. Cost has been maintained in all evaluation, as all the stakeholders (water industry, government and users), have a direct interest in it.

5.2.2 Preferred solutions for 2013 scenario

Among the 233 non-dominated solutions found for the 2013 scenarios using Priority Sets #1 and #2, six solutions have been selected as shown in Table 43. Note that, in the last column, 'Min Tc' refers to the minimum cost solution, 'Min TE' refers to the solutions with the minimum total energy, 'Min Discharge' refers to the solution that has the minimum volume of stormwater and wastewater discharged to the Gulf, 'Max NP Vol Rel' refers to the solution with the maximum non potable volumetric reliability. The solution labelled with 'CP1234' is the best solution according to compromise programming when all the four objectives are taken into account. The other compromise solutions are 'CP134', where only cost, reliability and discharges are taken into account; 'CP13' where only costs and reliability are taken into account and 'CP14' where costs and discharges are considered.

The values of the objectives of the selected solutions are given in Table 43. It can be seen that solutions with the lowest discharges to the Gulf usually have larger total cost. It can also be seen that, for the most expensive solutions, the operational costs of Mount Lofty and River Murray sources decrease as they supply a smaller volume of water (Table 44). The *Total Supply* values given in Table 44 are the average annual supply for the 30 years simulated period. It can be seen that the three major sources used to supply the demand are the Mount Lofty Catchments (ML), the River Murray (RM) and recycled wastewater (WW). Note that WW is used for non-potable purposes and that the discharges to the Gulf are reduced by increasing the supply from wastewater (WW) and stormwater (SW).

Table 43: Objective function value for the selected non-dominated solutions of 2013 scenario

No	Total Cost (M$)	Cost/kL ($/kL)	Total Energy (GWh)	Energy/kL (kWh/kL)	System Demand NP Volumetric Rel (%)	Total System Discharges SW and WW (GL/year)	Notes
1	2459	0.57	5045	1.17	100.00	179	Min Tc, Max Discharge, Max TE, CP13
44	3123	0.73	3887	0.90	100.00	139	Max NP Vol Rel
64	3453	0.80	4088	0.95	99.97	130	CP134, CP14
76	3570	0.83	3453	0.80	99.91	133	Min TE
97	3798	0.88	3646	0.85	99.96	125	CP1234
233	6111	1.42	4492	1.04	99.65	107	Max Tc, Min Discharge, Min NP Vol Rel

Table 44: Supply from each source for the selected non-dominated solutions of 2013 scenario

No	Total supply (GL/yr)					Water supplied by (percent)				
No	ML	RM	ADP	SW	WW	ML	RM	ADP	SW	WW
1	94.1	67.3	0.7	4.6	5.5	54.7%	39.1%	0.4%	2.7%	3.2%
44	93.5	36.0	0.1	2.1	40.5	54.3%	20.9%	0.1%	1.2%	23.5%
64	90.1	32.3	0.0	12.1	37.7	52.3%	18.8%	0.0%	7.0%	21.9%
76	91.8	24.0	0.0	6.3	50.0	53.3%	13.9%	0.0%	3.7%	29.0%
97	87.8	23.0	0.0	11.3	50.1	51.0%	13.4%	0.0%	6.5%	29.1%
233	83.1	12.5	0.3	20.1	55.8	48.3%	7.3%	0.2%	11.7%	32.5%

Table 45 shows the values of some of the decision variables, while Table 46 shows the capital and operational costs of the selected solutions: it can be seen that the most expensive solutions have larger capital and operational costs associated with stormwater harvesting and recycled wastewater. From Table 45 it can be seen that the number of new stormwater schemes implemented increases from 3 for the cheapest solution to 18 for the most expensive solution. However, on average 0.91 to 7.46 GL/year is stored in the aquifer without being used to supply the user demands. For some solutions, such as solution 44, the stormwater stored in the aquifer is more than three times the water supplied to the users; for other solutions, such as solution 233, the average volume stored in the aquifer is small compared to the supply and the stormwater schemes implemented are effectively used to supply the non-potable demand. Storing stormwater in the aquifer reduces the discharges to the Gulf, so the optimisation algorithm may decide to implement stormwater schemes to improve this objective. However, there is also the possibility that the model is not able to match injection with extraction and these results in stormwater being accumulated in the aquifer.

Table 45: Decision variables values and final aquifer storage for the selected non-dominated solutions of 2013 scenario

No	MN ADP to Onka Conf (ML/mth)	MN MA to Torrens (ML/mth)	MN MBO to Onka (ML/mth)	MNSRS to Gawler (ML/mth)	WW DistCap SimLimit Bolivar (ML/year)	WW DistCap SimLimit Christies (ML/year)	WW DistCap SimLimit Glenelg (ML/year)	WW DistCap SimLimit Bolivar Fraction RND	WW DistCap SimLimit Christies Fraction RND	WW DistCap SimLimit Glenelg Fraction RND	No. of new SW schemes	Average annual SW aquifer storage (GL/year)	Notes
1	2709	2170	13300	593	1050	4200	328	0.60	0.49	0.51	3	1.56	Min Tc, Max TE, Max Discharge, CP13
44	41	1564	5716	263	33200	8460	5378	0.73	0.79	0.24	3	7.46	Max NP Vol Rel
64	12	2780	11789	331	41292	15158	2304	0.53	0.09	0.73	9	6.96	CP134, CP14
76	17	1080	6410	26	32200	14000	15000	0.60	0.60	0.84	7	0.91	Min TE
97	17	1190	13900	509	39900	9970	13600	0.61	0.38	0.49	5	2.21	CP1234
233	1934	550	11572	179	58478	15655	16978	0.74	0.88	0.78	18	2.80	Max Tc, Min Discharge, Min NP Vol Rel

Note: MN stands for Maximum Node, which represents maximum capacity of the pipeline; ADP stands for Adelaide Desalination Plant; MA stands for Murray-Adelaide pipeline; MBO stands for Murray Bridge-Onkaparinga pipeline and SRS stands for Swan Reach Stockwell pipeline; and WW stands for wastewater

Table 46: Capital and operational costs for each source for the selected non-dominated solutions of 2013 scenario

Cost Index	PV of Capital Cost (M$)	PV of Op. Cost (M$)	Capital Cost Of SW (M$)	Capital Cost Of WW (M$)	Op Cost Of ML (M$/year)	Op Cost Of RM (M$/year)	Op Cost Of ADP (M$/year)	Op Cost Of SW (M$/year)	Op Cost Of WW (M$/year)
1	68	2391	40	29	23	33	31	5	96
44	547	2576	159	388	21	19	30	7	124
64	818	2635	445	373	21	17	30	15	123
76	995	2575	135	859	21	14	30	5	131
97	1160	2638	350	810	20	13	30	10	133
233	3416	2695	1354	2062	19	8	30	18	137

5.2.3 Preferred solutions for 2050 scenario

Note that solutions of 2025 scenarios are in Appendix 9 as the results are intermediate between the 2013 and 2050 scenarios.

Table 47: Objective function value for the selected non-dominated solutions of 2050 scenario

No	Total Cost (M$)	Cost/kL ($/kL)	Total Energy (GWh)	Energy/kL (kWh/kL)	System Demand NP Volumetric Rel (%)	Total System Discharges SW and WW (GL/year)	Notes
1	3165	0.60	7390	1.39	99.98	164	Min Tc, Max TE, CP13
6	3240	0.61	7365	1.39	100.00	164	Max Discharge
67	4196	0.79	6440	1.21	100.00	125	CP134, CP14
99	4788	0.90	5893	1.11	99.96	120	Min TE
101	4859	0.91	6046	1.14	100.00	114	CP1234
108	4996	0.94	6208	1.17	100.00	115	Max NP Vol Rel
196	6575	1.24	6637	1.25	99.86	99.2	Min NP Vol Rel
197	6576	1.24	6345	1.19	99.87	98.8	Min Discharge

As shown in Table 47, the total costs and total energy have increased compared to the 2013. However, the reliability of non-potable water is almost the same, while the discharges to the Gulf have slightly decreased: this can be due to the increased wastewater recycling and to the fact that often more water is stored in the aquifer without being used. For the selected solutions, the number of new stormwater schemes implemented is slightly smaller than in the 2013 scenario, probably due to the convergence of the algorithm.

The results for the 2050 scenario shows that the volume of water withdrawn from all the sources will be in general larger than for the 2013 scenario to make up for the increased demand (Table 48). Note also that despite the increased demand, stormwater keeps accumulating in the aquifer, as it has not been possible to force the NetLP to use the whole volume of stored water.

As shown by Table 48, all sources (except Mount Lofty Catchments) are exploited more. In terms of percentage of demand supplied, Mount Lofty source decreases in 2050 and its supply is replaced by River Murray, recycled wastewater, stormwater reuse and desalinated water.

Table 48: Supply divided from each source for the selected non-dominated solutions of 2050 scenario

No	Total supply (GL/yr)					Water supplied by (percent)				
No	ML	RM	ADP	SW	WW	ML	RM	ADP	SW	WW
1	72.8	110.1	2.3	3.4	23.9	34.3%	51.8%	1.1%	1.6%	11.2%
6	73.9	111.8	0.7	4.6	21.5	34.8%	52.6%	0.3%	2.2%	10.1%
67	71.9	80.5	0.3	9.6	50.2	33.9%	37.9%	0.2%	4.5%	23.6%
99	73.5	67.3	0.0	7.2	64.4	34.6%	31.7%	0.0%	3.4%	30.3%
101	73.2	66.1	0.0	12.2	61.1	34.4%	31.1%	0.0%	5.7%	28.7%
108	72.6	64.3	1.0	15.4	59.2	34.2%	30.2%	0.5%	7.3%	27.8%
196	69.7	51.8	2.9	17.4	70.6	32.8%	24.4%	1.4%	8.2%	33.2%
197	72.8	53.0	0.0	16.0	70.5	34.3%	24.9%	0.0%	7.5%	33.2%

Table 49 shows the values of some of the decision variables, while Table 50 shows the capital and operational costs of the selected solutions. Compared to the 2013 scenario, the selected solutions increased the wastewater recycling capacity more and exploit more the River Murray Source. The use of Mount Lofty water is nearly the same as in 2013 as probably some of the solutions already exploited this source at its maximum potential. Compared to 2013, also ADP costs slightly increased.

Table 49: Decision variables values and final aquifer storage for the selected non-dominated solutions of 2050 scenario.

No	MN ADP to Onka Conf (ML/month)	MNMA to Torrens (ML/month)	MN MBO to Onka (ML/month)	MNSRS to Gawler (ML/month)	WW DistCap SimLimit Bolivar (ML/year)	WW DistCap SimLimit Christies (ML/year)	WW DistCap SimLimit Glenelg (ML/year)	WWDist Cap Sim Limit Bolivar Fraction RND	WW DistCap Sim Limit Christies Fraction RND	WW Dist Cap SimLimit Glenelg Fraction RND	No. of new SW schemes	Average annual SW aquifer storage (GL/year)	Notes
1	5846	5710	13800	654	18300	14200	2050	0.12	0.20	0.21	1	1.34	Min Tc, CP13
6	774	8805	13969	68	32596	9678	671	0.01	0.35	0.15	4	1.65	Max Discharge
67	1279	10476	13632	499	40457	11269	7216	0.59	0.56	0.95	8	5.06	CP134, CP14
99	24	4927	10183	38	41107	11949	20812	0.71	0.72	0.65	6	0.04	Min TE
101	3	6269	11374	57	40452	12236	18387	0.71	0.44	0.53	8	2.78	CP1234
108	1168	5134	11447	354	38920	11270	16437	0.69	0.70	0.52	11	0.05	Max NP Vol Rel
196	7861	7600	10300	1060	56700	14300	21800	0.66	0.89	0.85	15	1.49	0
197	11	4237	9730	274	59039	14777	20837	0.66	0.63	0.73	18	3.15	Min Discharge

Table 50: Capital and operational costs from each source for the selected non-dominated solutions of 2050 scenario

No	PV of Capital Cost (M$)	PV of Operational Cost (M$)	Capital Cost Of SW (M$)	Capital Cost Of WW (M$)	Op Cost Of ML (M$/year)	Op Cost Of RM (M$/year)	Op Cost Of ADP (M$/year)	Op Cost Of SW (M$/year)	Op Cost Of WW (M$/year)
1	69	3095	14	55	19	53	32	4	134
6	154	3087	115	38	20	54	31	5	132
67	913	3283	315	598	19	40	30	11	156
99	1510	3279	192	1318	19	34	30	6	168
101	1540	3320	420	1120	19	33	30	11	166
108	1697	3298	730	968	19	33	31	12	164
196	3174	3400	1009	2166	19	27	33	14	173
197	3204	3372	1034	2169	18	28	30	15	173

Results presented in this Section are affected by a number of limitations. In addition to the limitations on the algorithm parameters, the results are also affected by approximations in the modelling of all of the sources including lumping of demand zones, reservoirs and stormwater schemes as well as approximations in the distribution networks for harvested stormwater and treated wastewater.

In particular, the stormwater schemes have been grouped into 25 equivalent schemes. In setting up the capital costs for the construction of the scheme and associated distribution network, the costs and yields reported in Wallbridge and Gilbert (2009) have been considered. Note that capital costs of the schemes already in operation have been considered to be zero. However, it has been assumed that all of the schemes are considered to have aquifer storage and recovery (ASR) and that stormwater can be injected for only 5 months of the year (May to September) and extracted in the remaining months. This assumption does not take into account that some schemes in the Wallbridge and Gilbert (2009) may not use ASR or may be able to use water during the whole year.

Moreover, the total harvestable yield for each scheme estimated in Wallbridge and Gilbert (2009) does not match exactly with the modelled value and, more importantly, the extraction in the model is usually lower than the injection, resulting in stormwater accumulating in the aquifer. This has an effect on the cost estimation, as all the costs associated with stormwater use (injection and extraction) have been associated with the harvesting and collection facilities so as to encourage the algorithm to use stormwater once a scheme is implemented. In general, the costs presented in this Section may be overestimated as the costs for distribution of stormwater are included even if the water is not supplied and this missing supply is provided by other sources.

A remaining final issue with the NetLP is that it has not been possible to completely control the solutions obtained. For example, in the model stormwater might be extracted from the aquifer for less than five months of the year even when the stormwater has the highest priority of use.

5.3 Impact of rainwater harvesting and demand management

The impact of rainwater tanks (RWTs) (100% uptake rate) and DM options were examined for the solution ID #1 of 2013 scenario (given in Table 43, i.e. the Pareto optimal solution for lowest cost and highest discharge) and 2050 scenarios (given in Table 47, i.e. the Pareto optimal solution for lowest cost and highest discharge). Results for 2013 scenario are shown in Table 51, Table 52 and Table 53. Results for 2050 scenario are shown in Table 54, Table 55 and Table 56.

5.3.1 Rainwater tanks

The following observations can be made with regard to the use of rainwater, for 2013 scenario:

- Use of RWTs in solution ID #1 (which exhibited the lowest cost, but the highest discharge to the Gulf without RWTs) increased the cost by 88%, increased the total energy by 17%, reduced the discharge to the Gulf by 5% and reduced the demand for water from other sources by about 12%, of this solution
- Comparison of solution ID #1 with RWTs with solution ID #233 (which exhibited the highest cost and energy consumption, but the lower discharge to the Gulf compared to other solutions listed in Table 51), indicated that the use of RWTs reduced the cost by 24%, increased the total energy by 32%, increased the discharge to the Gulf by 59% and reduced the demand for water from other sources by about 12%, compared to those of solution ID #233
- Comparison of solution ID #1 with solution ID #44 (which exhibited the highest volumetric reliability to non-potable demands) indicated that the use of RWTs increased the cost by 48%, increased the total energy by 52%, increased the discharge to the Gulf by 22% and reduced the demand for water from other sources by about 12%, compared to those of the solution ID #44.

The following observations can be made with regard to the use of rainwater, for 2050 scenario:

- Use of RWTs in solution ID #1 (which exhibited the lowest cost, but the highest discharge to the Gulf without RWTs) increased the cost by 106%, increased the total energy by 17%, reduced the discharge to the Gulf by 6% and reduced the demand for water from other sources by about 16%, of this solution
- Comparison of solution ID #1 with RWTs with solution ID #196 (which exhibited the highest cost and energy consumption, but the lower discharge to the Gulf compared to other solutions listed in Table 54), indicated that the use of RWTs reduced the cost by 1%, increased the total energy by 30%, increased the discharge to the Gulf by 55% and reduced the demand for water from other sources by about 16%, compared to those of solution ID #196
- Comparison of solution ID #1 with solution ID #6 (which exhibited the highest volumetric reliability to non-potable demands) indicated that the use of RWTs increased the cost by 101%, increased the total energy by 18%, increased the discharge to the Gulf by 22% and

reduced the demand for water from other sources by about 16%, compared to those of the solution ID #44.

Table 51: Impact of having rainwater tanks (RWTs) and demand management (DM) for solutions #1of 2013 scenario

Solution ID	Total Cost (M$)	Total Energy (GWh)	Non-potable Volumetric reliability (%)	Total discharge (GL/year)	Comments
1	2459	5045	100.0%	179	Minimum cost and maximum discharge
1/RWT	4623	5927	100.0%	170	21.5 GL/year supplied by RWTs
1/DM	2449	2780	100%	173	8.7 GL/year water savings with DMs
1/RWT & DM	4616	3711	100%	165	30.2 GL/year supplied by RWTs and DM
44	3123	3887	100.00%	139	Maximum non-potable volumetric reliability
64	3453	4088	99.97%	130	Compromised between cost, reliability and discharge objectives
76	3570	3453	99.91%	133	Minimum energy
97	3798	3646	99.96%	125	Compromise between all four objectives
224	5282	4167	99.83%	109	Minimum discharge
233	6111	4492	99.65%	107	Minimum discharge

Therefore, it could be said that the use of RWTs would not be a preferred supply option for both 2013 and 2050 scenarios, if the preference was to minimise the total cost. Similarly, the use of RWTs would not be a preferred supply option for both 2013 and 2050 scenarios, if the preference was to minimise the total energy consumption. Similar to the cost and energy consumption mentioned above, the use of RWTs would not be a preferred supply option, if the preference was to minimise the total discharge to the Gulf, because higher reductions in discharges (about 25% compared to about 5% with RWTs) to the Gulf could be obtained with about 30% increase in cost in both 2013 and 2050 scenarios (compared to 88% increase in cost with RWTs in 2013 and 106% increase in cost with RWTs in 2050), by utilising harvested stormwater and wastewater, compared to utilising RWTs.

In summary, it could be said that while RWTs could support the diversification of supply options by shifting demand on the RM, ADP and MLR, by reducing the total demand by about 12% in 2013 and 16% in 2050, RWTs alone could reduce discharge to the Gulf only by about 5%. The results indicated conjunctive use of harvested stormwater, recycled water, ADP and the RM and MLR in an appropriate quantities could provide higher reductions in discharges to the Gulf without increasing the cost no more than about 30%, compared to the minimum cost/maximum discharge solution.

It should however, be noted that the datasets used to compute supply and discharge implications of rainwater tanks may not be representative for Metropolitan Adelaide because at the time of conducting the study there was lack of representative data on tank size, household demands and connected roof areas to tanks. Therefore, the above mentioned results should be used as indicative and proof-of-concept purposes only.

Table 52: Supply by source for rainwater tanks (RWT) and demand management (DM) for solutions #1of 2013 scenario

No	Total supply (GL/yr)						
	ML	RM	ADP	SW	WW	RWT	DM
1	94.1	67.3	0.7	4.6	5.5	0	0
1/RWT	91.5	50	0.14	4.1	5.2	21.5	0
1/DM	93.5	59.5	0.52	4.6	5.5		8.7
1/RWT & DM	90.3	42.8	0.11	4.1	4.9	21.5	8.7
44	93.5	36	0.1	2.1	40.5	0	0
64	90.1	32.3	0	12.1	37.7	0	0
76	91.8	24	0	6.3	50	0	0
97	87.8	23	0	11.3	50.1	0	0
224	82.4	14.3	0.2	20	55	0	0
233	83.1	12.5	0.3	20.1	55.8	0	0

Table 53: Supply as a percent of total demand for RWTs and DM for solutions #1of 2013 scenario

No	Total supply as a percent of total demand met						
	ML	RM	ADP	SW	WW	RWT	DM
1	54.7%	39.1%	0.4%	2.7%	3.2%	0.0%	0.0%
1/RWT	53.1%	29.0%	0.1%	2.4%	3.0%	12.5%	0.0%
1/DM	54.3%	34.5%	0.3%	2.7%	3.2%	0.0%	5.0%
1/RWT & DM	52.4%	24.8%	0.1%	2.4%	2.8%	12.5%	5.0%
44	54.3%	20.9%	0.1%	1.2%	23.5%	0.0%	0.0%
64	52.3%	18.8%	0.0%	7.0%	21.9%	0.0%	0.0%
76	53.3%	13.9%	0.0%	3.7%	29.0%	0.0%	0.0%
97	51.0%	13.4%	0.0%	6.5%	29.1%	0.0%	0.0%
224	47.9%	8.3%	0.1%	11.6%	32.0%	0.0%	0.0%
233	48.3%	7.3%	0.2%	11.7%	32.5%	0.0%	0.0%

5.3.2 Demand management options

The following observations can be made with regard to the use of the DM option:

- Use of DM options reduced the energy consumption by about 45% in 2013 and by about 8% in 2050. The lower reduction in 2050 was due to the fact that by 2050, most households already comprised DM options such as front loading washing machines (84%), 3-star rated showerheads (84%) and 6/3 L dual flush toilets (100%). Hence, only a small percentage of households would be remaining to adopt DM options, compared to 2013 scenario
- The impact on the total cost due to the adoption of DM was negligible in both 2013 and 2050
- The reduction in total demand due to adoption of DM in both 2013 and 2050 was about 5% of the total demand of the respective years.

Table 54: Impact of having rainwater tanks (RWTs) and demand management (DM) for solutions #1of 2050 scenario

Solution ID	Total Cost (M$)	Total Energy (GWh)	Non-potable Volumetric reliability (%)	Total discharge (GL/year)	Comments
1	3165	7390	99.98%	164	Minimum cost and maximum discharge
1/RWT	6507	8660	99.97%	154	33.2 GL/year supplied by RWTs
1/DM	3107	6774	99.97%	157	10.9 GL/year water savings with DMs
1/RWT & DM	6448	8092	99.96%	148	44.1 GL/year supplied by RWTs and DM
6	3240	7365	100.00%	164	Max discharge, but maximum non-potable volumetric reliability
67	4196	6440	100.00%	125	Compromised between cost, reliability and discharge objectives
99	4788	5893	99.96%	120	Minimum energy
101	4859	6046	100.00%	114	Compromise between all four objectives
108	4996	6208	99.86%	115	Compromise between all four objectives
196	6575	6637	99.87%	99.2	Minimum discharge
197	6576	6345	99.87%	99.8	Minimum discharge

In summary, it can be said that DM did not have a much impact on the total cost, but it reduced the energy consumption by about 45% for 2013 scenario. Therefore, reducing the demand through DM while supplying water from a mix of sources seemed appropriate and it would help reduce the reliance on the current potable water sources (i.e. the RM, the MLR and ADP), by about 5%.

It should however, be noted that the costs of demand management campaigns were not included in the cost considered for demand management options. Such data were not available to the study, at the time of conducting the study. Therefore, the above mentioned results should be used as indicative and proof-of-concept purposes only.

Table 55: Supply by source with RWTs and DM for solutions #1of 2050 scenario

No	Total supply (GL/yr)						
	ML	RM	ADP	SW	WW	RWT	DM
1	72.8	110.1	2.3	3.4	23.9	0	0
1/RWT	71.7	81.8	0.48	2.6	22.8	33.2	0
1/DM	72.6	100.3	1.7	3.4	23.6		10.9
1/RWT & DM	71.4	72.2	0.23	2.6	22.2	33.2	10.9
6	73.9	111.8	0.7	4.6	21.5	0	0
67	71.9	80.5	0.3	9.6	50.2	0	0
99	73.5	67.3	0.0	7.2	64.4	0	0
101	73.2	66.1	0.0	12.2	61.1	0	0
108	72.6	64.3	1.0	15.4	59.2	0	0
196	69.7	51.8	2.9	17.4	70.6	0	0

Table 56: Supply as a percent of total demand with RWTs and DM for solutions #1of 2050 scenario

No	Total supply as a percent of total demand met						
	ML	RM	ADP	SW	WW	RWT	DM
1	34.3%	51.8%	1.1%	1.6%	11.2%	0.0%	0.0%
1/RWT	33.7%	38.5%	0.2%	1.2%	10.7%	15.6%	0.0%
1/DM	34.2%	47.2%	0.8%	1.6%	11.1%	0.0%	5.1%
1/RWT & DM	33.6%	33.9%	0.1%	1.2%	10.4%	15.6%	5.1%
44	34.8%	52.6%	0.3%	2.2%	10.1%	0.0%	0.0%
64	33.8%	37.9%	0.1%	4.5%	23.6%	0.0%	0.0%
76	34.6%	31.7%	0.0%	3.4%	30.3%	0.0%	0.0%
97	34.4%	31.1%	0.0%	5.7%	28.7%	0.0%	0.0%
224	34.2%	30.3%	0.5%	7.2%	27.9%	0.0%	0.0%

6 Summary, conclusions and recommendations

6.1 Summary

The study can be summarised as follows:

- A multi-objective decision support framework for evaluating and selecting water sources for major cities has been developed. It is called the Integrated Urban Water Management Decision Support Framework (IUWM DSF).
- The IUWM DSF takes into account technical, economic, environmental and social factors in assessing combinations of alternative water sources.
- The IUWM DSF has been implemented by using National Hydrologic Modelling Platform (NHMP). This is the first time that the NHMP has been used to inform urban water resources planning at a major city scale.
- The utility of the framework has been demonstrated by applying it to a case study of planning future water resources for Metropolitan Adelaide for the period up to 2050. Hence the case study should be treated as a 'proof-of-concept' only.
- A simulation model has been developed for Metropolitan Adelaide by using the NHMP's catchment and schematic modules. It considers the following water sources: surface reservoirs in the Mount Lofty Ranges, pumping from the River Murray, desalination, harvested stormwater (for non-potable use), reclaimed wastewater (for non-potable use), groundwater, rainwater (from household tanks for non-potable use) and demand management.
- The water demand of Metropolitan Adelaide has been split into residential and non-residential, potable and non-potable. Residential non-potable demand included toilet flushing, laundry use and all outdoor use.
- The simulation model has been seamlessly linked with a multi-objective optimisation model to identify efficient water sources for the planning years 2013, 2025 and 2050. The optimisation model includes the following five sources: surface reservoirs in the Mount Lofty Ranges, pumping from the River Murray, desalination, harvested stormwater (for non-potable use) and reclaimed wastewater (for non-potable use).
- The objectives considered are the present value of cost, total energy usage, the volumetric reliability of the non-potable network and the total discharge of wastewater and stormwater to Gulf St Vincent.
- Preferences of consumers and stakeholders have been taken into account by setting priorities on supply from the various sources.
- Multi-objective optimisation has been used to produce a range of efficient solutions (relative to the four objectives considered) and trade-offs between the various objectives.

- This case study involving simulation modelling and optimisation has demonstrated the ability of the framework to identify efficient portfolios of supply sources and the trade-offs associated with them, by taking into account a large number of objectives, constraints and options when planning water resources for a diversified urban water system. It also has the potential to evaluate the influence of factors such as climate variability and change and population growth when planning these systems.

6.2 Conclusions

Specific conclusions from the Case Study include:

- The Mount Lofty Ranges catchments were the preferred source for potable water supply due to their low cost and energy.
- Water from the River Murray was generally the second choice for potable use.
- In all scenarios only a small volume of water was drawn from the desalination plant due to its high cost and energy. It was used primarily as a backup supply in dry years.
- Depending on the priorities set, various combinations of water from the River Murray, treated wastewater and harvested stormwater were used to meet non-potable demand.
- Lower cost solutions tended to use more River Murray water for non-potable use while solutions with low discharge to Gulf St Vincent used more treated wastewater and harvested stormwater.
- Treated wastewater is a more cost effective option for reducing discharge to the Gulf than harvested stormwater.
- Rainwater tanks have the potential to reduce the demand for water from other sources by about 12% (if 100% uptake is assumed), but they are not a cost effective solution for Adelaide's water supply as they are expensive and energy intensive compared to the other sources. It should however, be noted that the availability of representative data to quantify yield and discharge implications of rainwater tanks was limited at the time of conducting the study, in particular datasets on household demands at the end use scale. Therefore, conclusions regarding rainwater tanks should be used as indicative and proof-of-concept purposes only.
- The use of demand management options for in-house appliances such as low flow shower heads, front loading washing machines and dual flush toilets have the potential to reduce total water consumption by about 5%.
- The trade-offs developed between cost and energy show that the minimum cost solution was not the minimum energy solution and vice versa.
- There is a marked trade-off between cost and discharge of wastewater and stormwater to the Gulf, with reduced discharges requiring significant investment in capital and operational costs.
- The Adelaide system has a very high security of supply due to the capacity to pump water from the River Murray and the desalination plant. All solutions evaluated in this study maintained a very high reliability of supply. This assessment was based on an evaluation of water quantity but not water quality as the latter was outside the scope of the study.

6.3 Key assumptions

It is important to note that the above-mentioned conclusions arising from the case study are dependent on the data and assumptions used. The primary assumptions are described in Chapters 2-5. The key limitations of the case study are listed below:

- While the optimisation model includes the economic costs associated with supplying water from the various sources. It has not considered the economic benefits to industry, agriculture, residential users and for green space.

All cost and energy values used in this study are based on literature values and hence have an associated level of uncertainty. A sensitivity analysis to these assumed values should be undertaken in subsequent research.

- All of the modelling and optimisation was carried out for a single set of population projections and a single climate change scenario. The sensitivity of these assumed values need to be assessed in subsequent research.
- Demand in the Metropolitan Adelaide area has been divided into three demand zones. In reality, demand varies over the entire area and the ability to distribute water to this distributed demand is simplified in the model.
- The nine reservoirs in the Mt Lofty Ranges have been lumped into three equivalent reservoirs. This overestimates the capacity of these reservoirs to store water as spill in the model will not occur from a lumped reservoir until all of the component reservoirs are full. In really, a component reservoir can spill when other component reservoirs are only partially full, although the ability to transfer water between reservoirs (within limitations) and the grouping of reservoirs by catchment will limit this modelling error.
- Only non-potable use was considered for harvested stormwater and treated wastewater. This is consistent with the current policy of the South Australian Government.
- The 70 existing and potential stormwater harvesting schemes outlined in Wallbridge and Gilbert (2009) have been lumped into 25 equivalent schemes. This simplifies the hydrology and operation of the schemes.
- It has been assumed that all stormwater harvesting schemes involve aquifer storage and recovery. As some of the sub-schemes involve biofilters or direct supply, there is some simplification of the costs and supply capacities of the schemes.
- It is assumed that all stormwater harvesting schemes inject harvested stormwater into the aquifer for five months of the year from May to September and extract water for the remaining seven months of the year. This underestimates the potential yield from stormwater as it may be possible for it to be injected in the aquifer at other times of the year.
- Each of the 25 stormwater harvesting schemes in the model may be either implemented or not implemented in full. 27 of the 70 sub-schemes are existing sub-schemes. The capital costs of these sub-schemes are ignored in the model. The schemes that consist entirely of existing sub-schemes are automatically implemented. However, if a scheme that contains some existing sub-schemes is not is implemented, the harvested supply of those sub-

schemes is not included in the model. This means that the total yield from stormwater is underestimated in the model results.

- The cost and energy of the distribution systems associated with new harvested stormwater and wastewater systems are handled in a simplified manner as an equivalent cost or energy per mega litre or per household.
- The impacts of discharging stormwater and wastewater to Gulf St Vincent are handled by considering the single objective of minimising the total volume of stormwater and wastewater discharged to the Gulf. This is a simplified treatment of a complex environmental objective.
- The optimisation runs did not include rainwater tanks or demand management. The effects of these options were assessed using simulation only.
- Groundwater use in the Adelaide Metropolitan system was greatly simplified by assuming that all available groundwater was fully allocated and would continue to be used at current levels.
- Because of the current configuration of the Source and Insight models, a set of priorities for the various water sources had to be specified for potable and non-potable supplies. This gave the opportunity to bias the results towards particular water sources. However, it would be good if, in addition, Source could be set up to optimise the priority of each water source so that a comparison can be made with the results obtained using pre-specified priorities.
- The following four objectives were considered in the study: present value of total cost, total energy, volumetric reliability of non-potable supply and the total discharge of stormwater and wastewater to the Gulf St Vincent. These objectives could be expanded in subsequent research by taking an ecosystem services approach to include all relevant economic, environmental and social objectives.

6.4 Recommendations for further research

- Expand the economic objective to include the benefits of supplying additional water to the following end users: industry, agriculture, residential users and for green space.
- Undertake a sensitivity analysis on the values assumed for the cost and energy of the various sources.
- Expand the objectives to include other environmental and social objectives. This could include using an ecosystem services approach.
- Model and optimise the options considering a range of population and climate change scenarios.
- Include the latest results from the end use survey to improve the estimates of water demand used in the model.
- Evaluate the effects of lumping the reservoirs on estimates on system spill, total cost and energy requirements.
- Consider the use of blended stormwater and wastewater for non-potable uses.

- Assess the technical feasibility, cost-effectiveness, public health and environmental effects of using treated stormwater and treated wastewater for potable uses. Any such use would need to be compatible with the SA Water Drinking Water Guidelines.
- Review the grouping and modelling of existing and new stormwater schemes to identify the extent to which this affects the accuracy of the yields, costs and energy requirements of using harvested stormwater.
- Improve the modelling and costing of pipe systems to distribute harvested stormwater and wastewater.
- Improve the methods used for estimating the environmental impacts on Gulf St Vincent either through modelling water quality directly and/or modelling the ecological impact.
- Include the results of the Goyder-funded study on the community willingness-to-pay to improve the seagrasses in Gulf St Vincent.
- Include rainwater tanks and demand management as options in the optimisation studies.
- Integrate the results of the Goyder-funded study on the groundwater resources of Metropolitan Adelaide to better integrate options for use of groundwater sources and storage in the optimisation.
- Develop improved versions of Source and Insight that can allow the priorities of the individual sources to be optimised.

7 Appendices

This report has nine appendices. They are described in Volume 2 of this report. The appendices are listed below, for easy reference:

A 1. Rebuild of Metropolitan Adelaide Source Catchment Model
A 2. Reduction in water demand due to demand management
A 3. Non-dominated solutions for 2013 scenario (Priority Sets #1 and #2)
A 4. Optimal solutions for 2013 scenario (Priority Set #3)
A 5. Optimal solutions for 2025 scenario (Priority Sets #1, #2 and #3)
A 6. Optimal solutions for 2050 scenario (Priority Set #3)
A 7. Command line instructions for Insight module
A 8. Key global variables included in Source simulation modules
A 9. Estimating stormwater related constituent loads discharging to the Gulf.

8 Terms and abbreviations

Term	Description
ADP	Adelaide Desalination Plant
AMLR NRM Board	and Adelaide and Mount Lofty Ranges Natural Resource Management Board
ASR	Aquifer Storage and Recovery
CDD12	Cooling-Degree-Day-12
CP	Compromise Programming
CP13	Solution of the compromise programming taking into account total costs and volumetric reliability of the non-potable supply
CP14	Solution of the compromise programming taking into account total costs and total stormwater and wastewater discharges to the Gulf
CP134	Solution of the compromise programming taking into account total costs, volumetric reliability of the non-potable supply and total stormwater and wastewater discharges to the Gulf
CP1234	Solution of the compromise programming taking into account total costs, total energy, volumetric reliability of the non-potable supply and total stormwater and wastewater discharges to the Gulf
D_{SW-WW}	Total stormwater and wastewater discharge into the Gulf
DSF	Decision Support Framework
DEWNR	South Australia Department of Environment, Water and Natural Resources
DPTI	Department of Planning, Transport and Infrastructure
GA	Genetic Algorithm
IUWM	Integrated Urban Water Management
Link	Link allows flow in 1 direction between 2 nodes.
Link models	Component models associated with links
MCA	multi-criteria decision analysis
ML	Mount Lofty
MLC	Mount Lofty Catchments
MLR	Mount Lofty Ranges
MOGA	Multi-objective Genetic Algorithm
NetLP	Network Linear Programming
Node	Point where flow is combined and/or separated and must mass balance each time step
Node models	Component models associated with nodes
NP Vol Rel	Non-potable Volumetric Reliability

Term	Description
NP	Non-Potable
NHMP	National Hydrologic Modelling Platform
NRNP	Non-Residential Non-Potable
NSE	Nash-Sutcliff Efficiency
NSGA-II	Non-dominated Sorting Genetic Algorithm II
OWRM	Optimal Water Resource Mix project
P	Potable
PET	Potential Evapotranspitation
RM	River Murray
RNP	Residential Non-Potable
RR model	Rainfall-runoff model
RRL	Rainfall-runoff Library
RWT	Rainwater Tank
SA EPA	South Australia Environmental Protection Agency
SA Water	South Australia Water Corporation
SRS	Swan Reach-Stockwell pipeline.
SW	Stormwater
TC	Total Cost
TE	Total Energy
TR_{P}	Time supply reliability of potable component of the system demand
TWCM	Total Water Cycle Management
$\mathrm{VR}_{\mathrm{NP}}$	Volumetric supply reliability of non-potable component of the system demand
WSUD	Water Sensitive Urban Design includes urban design features designed to attenuate peak stormwater flows and improve stormwater quality
WW	Wastewater
WWTP	wastewater treatment plants

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